DRAINAGE MASTER PLAN FOR ALLEN BASIN STUDY AREA

VOLUME I of II

Prepared for:
CITY OF MEMPHIS
Division of Public Works / Engineering

MICHAEL R. PHILLIPS
REGISTERED ENGINEER
AGRICULTURE
02/28/22
COMMERCE
NO. 109471
STATE OF TENNESSEE

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ALLWORLD
Project Management, LLC

Prepared by:
Neel-Schaffer, Inc. (Prime Consultant)
210 25th Avenue North, Suite 800
Nashville, TN 37203
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EXECUTIVE SUMMARY

The City of Memphis (City) commissioned the Neel-Schaffer, Inc. Team (NSI Team) to perform a drainage master plan study of the watersheds within the Allen Basin Study Area. The purpose of this study is to identify flooding problems and recommend solutions within the watershed, including conceptual design and estimated costs. This report consists of two volumes – Volume I includes background information for the study, a summary of known drainage/flooding issues, and descriptions and costs of conceptual solutions; and Volume II consists of study results and supporting technical information. Refer to Section 3.1.8 of this report (Volume I) for a description of the watershed outfalls, as well as watershed overview maps in Section 5 of Volume II (Exhibits 5.1 thru 5.6). The recommended drainage improvement projects will be identified and prioritized based on results of a hydrologic and hydraulic model developed for this study. The modeling and mapping performed for this study were based on field survey topographical information supplemented with LIDAR topography as necessary. The subject model was developed with InfoSWMM software and will be available to the City for use beyond this project.

The study team consists of three consulting firms. Neel-Schaffer, Inc. is the prime consultant responsible for project management, hydrologic/hydraulic modeling, topographic surveying, and report preparation. Powers Hill Design, LLC is responsible for public involvement and geographic information systems (GIS). Allworld Project Management, LLC is responsible for field reconnaissance and GIS support.

Major project components include a public outreach meeting, field reconnaissance, field surveying of storm infrastructure, channel cross sections and road profiles, hydrologic/hydraulic model development, conceptual drainage improvement recommendations, and final report.

Results of the enclosed study indicate that a total of eight proposed improvement projects are recommended for implementation. The total estimated planning-level construction cost of all recommended improvements for the entire study area is $20,690,249, which includes surveying, engineering, and contingent costs. Refer to Section 3.5 of this report for a detailed breakdown of estimated costs for each project.

Refer to Volume II for the following supporting documentation:

- Watershed overview map and location of the proposed project areas;
- Conceptual layouts for each proposed project area;
- InfoSWMM model results for Existing and Improved Conditions; and
- Floodplain Inundation Map Exhibits for Existing and Improved Conditions.

The proposed improvement projects were prioritized according to criteria discussed in Section 4 of this report. It is recommended to perform the proposed improvement projects in the order shown in the table on the following page:

Order of Implementation – Drainage Improvement Projects for Allen Basin Study Area

Priority/Order	Project ID	Project Location/Description
1	D-17	Pippin Street to Trudy Street
2	D-29	Frayser Raleigh Road
3	D-19	Pippin Street to Trudy Street
4	D-12	Ridgemont Road and Trudy Street
5	A-1	Beacon Hills Drive and Sunny Hill Drive
6	D-23	Dorado Avenue and Windermere Drive
7	D-21	Battlefield Drive, Kerwin Drive and Twinmont Street
8	D-9	Gruber Drive

1 PROJECT INTRODUCTION AND BACKGROUND

1.1 Purpose

The City of Memphis (City) commissioned the Neel-Schaffer, Inc. Team (NSI Team) to perform a drainage master plan study of the watershed in the Allen Basin Study Area. The purpose of this study is to identify flooding problems and recommend solutions within this watershed, including conceptual design and estimated costs. This report consists of two volumes – Volume I includes background information for the study, a summary of known drainage/flooding issues, and descriptions and costs of conceptual solutions; and Volume II consists of study results and supporting technical information. Refer to Section 3.1.8 of this report (Volume I) for a description of the watershed outfalls and model boundary conditions, as well as watershed overview maps in Section 5 of Volume II. The recommended drainage improvement projects will be identified and prioritized based on results of a hydrologic and hydraulic model developed for this study. The modeling and mapping performed for this study were based on field survey topographical information supplemented with LIDAR topography as necessary. The subject model was developed with InfoSWMM software (Innovyze, Version 14.7, Update #2) and will be available to the City for use beyond this project. The location of the Allen Basin Study Area is shown on Exhibit 5.1 in Volume II

1.2 Scope

A. Public Outreach and Meetings
   The public outreach effort focused on (a) informing the public of the drainage study, (b) providing a platform for citizens to inform the City of existing drainage related issues, and (c) presentation of the study results. One public outreach meeting took place for this study and will be discussed in Section 2.2 of this report.

B. Data Collection and Survey
   1. GIS-based topographic information was used as the main source of topographic information beyond the top of channel banks. All channel and pipe sections were surveyed, if possible, including discrete points to define the top and invert elevations of each drainage structure (inlet, headwall, manhole, etc.). All channel sections were surveyed from the top of the left overbank to the top of the right overbank, at a minimum. Where necessary, channel cross section surveys were extended into the floodplain to provide adequate floodplain definition for the modeling effort.
   2. The land surveying efforts met or exceeded the requirements defined below:
      a. The horizontal datum for survey work was NAD83, TN Zone 4100, as derived from the NGS National Spatial Reference System (NSRS). Horizontal survey data collection complied with “SECOND ORDER” standard, as defined in Table A-4 of the current TDOT Survey Manual.
      b. The vertical datum for all survey work was based on the City of Memphis Benchmark Network. Vertical survey data collection was compliant with “THIRD ORDER” standard, as defined in table A-5 of the current TDOT Survey Manual.
      c. The distance between channel cross sections was variable, depending on channel geography and transitions. Generally, the spacing between cross-sections is 500 feet, or less. However, in prismatic stretches with consistent slope and no known issues, further spacing was permissible.

d. Surveyed cross-sections were obtained at each significant change in channel geometry, at all points of concentrated storm water discharge, and immediately up and downstream of structure crossings.
e. All survey data collected for structures was consistent with standards provided by the City.
f. The following features were included in the land surveying effort, at a minimum:
    i. All open channels;
    ii. All pipes 24" and larger in the tributary drainage network;
    iii. All pipes downstream from an identified flooding concern;
    iv. All structures (headwalls, bridges, inlets, etc.) along drainage features meeting the above criteria.

3. All surveyed road crossings and outfalls were photographed. All such photographs were georeferenced. Structures crossing an open channel were photographed from the upstream and downstream vantage points. Structures discharging into the channel were photographed from the channel. Photographs were provided as a GIS layer. Georeferenced photographs were located relatively close to the subject matter but are not expected to have survey-grade accuracy.
4. The survey data was coordinated with existing information, including plans and GIS data, to develop a more accurate depiction of the drainage basin and network to be studied.
5. Critical buildings requiring a surveyed finished floor elevation (FFE) were identified during the study.

C. Hydrologic and Hydraulic Modeling

1. All hydrologic and hydraulic modeling is consistent with Volume 2 of the City of Memphis/ Shelby County Stormwater Management Manual. Modeling was based on Green-Ampt infiltration methods, a SCS 24-hour storm, and the SWMM runoff model with statistical recurrence intervals of 2, 5, 10, 25, 50, and 100 years.
2. The hydrologic and hydraulic model was validated, to the extent practicable, based on available information and/or observations. Stream depth and rainfall data collected by The University of Memphis and provided by the City, was used in the above validation.
3. All hydrologic and hydraulic modeling efforts were completed in the InfoSWMM modeling software package using sound engineering judgment and modeling practices.
4. The project was modeled using a combination 1D storm sewer network model with 1D surface flow hydrologic and hydraulic model; which relies on traditional 1D model mechanics to characterize flow throughout the underground drainage network, channels, and overland flow areas.
5. Based on input from City staff, the NSI Team studied the impacts of potential capital improvement projects on the modeled flooded areas. Final recommendations for capital improvement projects were developed to be pursued in subsequent years. Recommendations include planning-level cost estimates and conceptual layouts.

1.3 Team Members

Neel-Schaffer, Inc. (Prime Consultant) – Project Management, Modeling, and Topographic Surveying
210 25th Avenue North, Suite 800
Nashville, TN 37203
Phone: 615-383-8420

Mike Phillips, P.E., CFM – Mike.phillips@neel-schaffer.com, 615-383-8420, Senior Project Manager
Mike Cooper, P.E. – Michael.cooper@neel-schaffer.com, 615-383-8420, Project Manager
Richard Taylor, CFM – Richard.taylor@neel-schaffer.com, 615-383-8420, Hydraulic Modeling Technician
Richard Sullivan, P.E. – Richard.sullivan@neel-schaffer.com, 615-383-8420, Corporate Officer

Powers Hill Design, LLC – Public Involvement / Geographic Information Systems (GIS)
80 Monroe Avenue, Suite 420
Memphis, TN 38103
Phone: 901-543-8000
Steve Hill, P.E. – Shill@phdmemphis.com, 901-543-8000, Project Manager/Co-Owner
Nisha Powers – Npowers@phdmemphis.com, 901-543-8000, President/Co-Owner

Allworld Project Management, LLC – Geographic Information Systems Support
60 N. B.B. King Boulevard
Memphis, TN 38103
Phone: 901-881-2985
Jacob Wiley, PE – Engineering and GIS Support – jacob@allworldmail.com, 901-362-3300, Project Manager
Michael A. Hooks, Jr. – Chief Executive Officer – mahjr@allworldmail.com

1.4 Existing Flooding Problems

Areas of existing flooding or potential flooding were identified from City GIS data, interviews with local residents, information gathered at public meetings and the InfoSWMM model of the study area. Locations of existing flooding problems within the study area are shown on Exhibit 5.2 within Section 5 of Volume II.

1.4.1 Information from Interviews and Public Meetings

NSI Team obtained no historic complaint information from the City Drainage Engineer or the City GIS database prior to starting work on the project. One Public Outreach Meeting was held with general advertisements posted at several public locations within the subject study area as well as on the project website; additionally, individual meeting notices were sent directly to potentially flooded properties. The public outreach efforts generated no emails from citizens received via the project website, nor feedback regarding flooding problems via the public meeting.

1.4.2 Field Investigation

Allworld Project Management staff conducted a field investigation of the study area in November and December 2017. The field investigation included walking all major drainage networks of the study area, taking measurements as needed and photographing infrastructure. Field notes were used to verify drainage sub-basin boundaries, overland flow paths and existing infrastructure. Areas of potential flooding were also noted.

1.4.3 Identification of Existing Flooding Problems from InfoSWMM

As will be described in Section 3 of this report, a hydrologic and hydraulic model of existing drainage networks within the study area was compiled using InfoSWMM modeling software (Innovyze InfoSWMM 14.7, Update #2). The results of the InfoSWMM model were used to identify areas of existing flooding.

2 REVIEW OF PREVIOUS STUDIES AND AVAILABLE DATA

2.1 Discussion with City Engineer's Office

Prior to beginning work on the project, the NSI Team met with the City of Memphis Engineering Department staff to discuss historical flooding issues within the study area.

Throughout the NSI Team's study, several conversations took place with other City staff, including George Cox and the Public Works department, who provided ancillary information related to minor drainage and flooding issues. This information was used to help determine focus areas for proposed improvements that will be discussed later in this report.

2.2 Initial Public Outreach Meeting Summary

A public meeting was held at the Breath of Life Christian Center (3795 Frayser Raleigh Road) on Thursday, March 1, 2018. The meeting was advertised through the project website; individual email notices were sent to local community leaders, and many mail-outs were sent directly to homeowners within the study area. Additionally, a number of flyers advertising the public meeting were posted at various City and commercial locations. The public meeting was attended by 6 residents and included a brief discussion of the project goals, followed by individual meetings with all attendants to obtain information about their specific drainage problems.

No complaints from the residents living within the Allen Basin Study Area were received during the public meetings or via the City's website. A copy of the flyer advertising the public meeting, as well as a list of locations where advertisement flyers were posted and an attendance sign-in sheet from the meeting, are presented in Volume II, Section 1, Exhibits 1.1 through 1.3.

3 RESULTS AND DETAILS FOR MAJOR SUB-BASINS

3.1 Hydrologic and Hydraulic Modeling Approach

3.1.1 Hydrologic Model Parameters

A preliminary GIS shapefile was obtained from the City of Memphis/Shelby County (City), which contained the approximate watershed boundaries for the subject study area. The watershed boundaries were compared to existing LiDAR data, which was also obtained from the City. The outer boundaries of the watershed were adjusted based on LiDAR data and/or field observations. The subject watershed boundaries include all areas that contribute runoff to the Allen Basin Study Area. Due to the size of and complexity of the drainage network within the study area, the study area was divided into two areas; the West Study area and the East Study area. The West Study area encompasses approximately 1146 acres (1.79 square miles) and is roughly bounded by the Loosahatchie River Drainage Canal on the north, Spring Valley Drive and Old Allen Road on the east, Saint Elmo Avenue on the south and Range Line Road and Sunny View Drive on the west. The East Study area covers approximately 3562 acres (5.57 square miles) and is roughly bounded by the Loosahatchie River Drainage Canal on the north, Raleigh Millington Road to the east, McDuff and Arwine Roads to the south and Spring Valley Drive and Old Allen Road on the west. The total area of the Allen Basin study Area is approximately 4,708 acres, or 7.36 square miles. Exhibit 5.1 (Volume II) presents the entire study area and the division of the West and East Study Areas. Exhibit 5.3 (Volume II) shows the drainage network within the study area, 5-foot contours and areas selected for recommended drainage improvements, which will be discussed in later section of this report.

A GIS shapefile was obtained from the City, which contained an inventory of existing storm water infrastructure (inlets, culverts, manholes, etc.) within the subject study area (developed by University of Memphis). This information, coupled with field-run survey data, was utilized to determine the location and specifications of storm water inlets within the study area. Once the inlet locations were established and field-verified, the watersheds were sub-divided into multiple sub-basins, each of which contributes flow into a single receiving drainage inlet. Sub-basin boundaries were established by utilizing the contour data and field observations were used to verify the sub-basin delineations. In total, 430 sub-basins were delineated within the study area, ranging in area from approximately 0.02 acres to approximately 347.4 acres. A table is included in Volume II of this report (Table 2.1), which includes hydrologic parameters computed within these 430 sub-basins. The following sections describe how these parameters were computed. A map showing the sub-basins is included in Volume II as Exhibits 5.4~~.~~

3.1.2 Watershed Land Use Characterization

A Land Cover Classification file system raster image was obtained from the University of Memphis for use in this study. This raster image encompasses Shelby County, Tennessee, and categorizes multiple land use classifications into one of the following grid codes:

- Water (Grid Code 1)
- Tree Canopy (Grid Code 2)
- Other Vegetation (Grid Code 3)
- Impervious (Grid Code 5)

In order to establish a land use shapefile for the subject watershed, the raster image was clipped to the study area boundary and exported as a separate shape file. The land cover classifications were inspected and determined to be outdated when compared to recent aerial imagery within ArcGIS. Therefore, new development (i.e., parking lots and commercial buildings) were digitized and classified. Then, fields were added to the shapefile to represent the Manning's N-value for overland flow for each land use classification. In addition, fields were added to represent the depth of depression storage for each classification. These fields were then populated with the following initial values, assigned by land use classification per the InfoSWMM User's Guide. The following Table 1 shows initial calculated Manning's "N" values and Depression Storage Depth values. Please refer to Section 3.2.1 for a discussion of calibration procedures and Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

Table 1. Manning's "N" Values and Depression Storage Depths - Initial Values*

Land Use/Cover	Impervious N-value	Pervious N-value	Impervious Depression Storage Depth (inches) *	Pervious Depression Storage Depth (inches) *
Water / Wetlands	N/A	0.24	N/A	0.2
Tree Canopy	N/A	0.4	N/A	0.3
Other Vegetation	N/A	0.15	N/A	0.2
Bare Soil	N/A	0.05	N/A	0.1
Impervious	0.012	N/A	0.07	N/A

* *These initial values were slightly adjusted during the model calibration process (see Section 3.2.1, Model Calibration Efforts).*

A map of Land Use categories of the study watershed is included in Volume II as Exhibit 5.6. A table of percentages of Land Use categories is included on that map.

3.1.3 Watershed Soil Characterization

The USDA Natural Resources Conservation Service (NRCS) Web Soil Survey website (http://websoilsurvey.sc.egov.usda.gov) was utilized to obtain a detailed soil map and related information for the subject watershed. The watershed boundaries were imported into the Web Soil Survey as the area of interest, and the resulting soil map was downloaded as a shapefile. In addition, the associated Soil Surface Texture Summary Table was downloaded from the website, which provides the soil texture classification (loam, silt loam, etc.) for each soil type present within the area of interest. The data on this table shows that the soils within the study area predominantly consist of soils with a silt loam texture. A map of soil types found within the study area is included in Volume II as Exhibits 5.5. A table of soil types with percentages of each type within the study watershed is included on this map.

The following fields were added to this soil shapefile to represent the Green-Ampt Infiltration Method parameters that were required for hydrologic modeling:

- Capillary suction head (inches)
- Saturated Hydraulic Conductivity (inches/hour)
- Initial Soil Moisture Deficiency (ratio – feet/feet). Equivalent to soil porosity minus soil field capacity.

These fields were then populated with initial values, assigned by soil texture classification according to the InfoSWMM User's Guide. The following Table 2 shows initial calculated soil parameter values. Please refer to Section 3.2.1 for a discussion of calibration procedures and Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated soil parameters.

Table 2. Hydrologic Soil Parameters (Green-Ampt) - Initial Values*

Soil Type	Capillary Suction Head (inches) *	Saturated Hydraulic Conductivity (inches/hour) *	Initial Soil Moisture Deficit (ratio) *
silt loam	6.69	0.26	0.217
silty clay loam	10.63	0.04	0.129

* *These initial values were slightly adjusted during the model calibration process (see Section 3.2.1, Model Calibration Efforts).*

3.1.4 Sub-basin Parameterization

Utilizing the shapefiles and associated information mentioned above, the following sub-basin parameters were computed for use in hydrologic modeling:

3.1.4.1 Green-Ampt Infiltration Method Parameters

In order to compute Green-Ampt Infiltration Method Parameters within each sub-basin, the soil shapefile was intersected with the sub-basin shapefile (see Section 3.1.3, Watershed Soil Characterization). The resultant shapefile was exported to Excel, and weighted values (by area) for the saturated hydraulic conductivity, capillary suction head, and initial soil moisture deficit were computed for each sub-basin. The initial calculated values for capillary suction head, saturated hydraulic conductivity, and initial soil moisture deficit were adjusted during the model calibration process (see Section 3.1.3, Watershed Soil Characterization and Section 3.2.1, Model Calibration Efforts).

3.1.4.2 Sub-basin Area

The surface area of each sub-basin was calculated using ArcGIS and was populated in the appropriate attribute field. As stated previously, the overall study area is approximately 4,707 acres (7.35 square miles) and 430 sub-basins were delineated, ranging in area from approximately 0.02 acres to approximately 347.45 acres.

3.1.4.3 Sub-basin Length

The sub-basin length represents the average overland flow length within the sub-basin. The sub-basin length was estimated from the existing LiDAR topography, recent aerial photography, survey data and information gathering during the site investigation. Maximum lengths of various possible flow paths were averaged with focus placed on the portion of the flow path with the slower response. In most cases that is the length of overland flow before runoff would enter gutters and closed drainage systems. Not every flow length was measured; a value of 100 feet was found to be representative for most catchments. This assumption was then spot-checked and revised as necessary for outlier values at extremely small basins, odd basin shapes, largely impervious basins, etc. The sub-basin length values were not adjusted during the model calibration process.

3.1.4.4 Sub-basin Width

The sub-basin width is a parameter that is associated with the response time of the basin. When the width is small the response time will be slow. When the width is large, the response time will be short. Initial estimates of sub-basin width were determined by dividing sub-basin area by the sub-basin length. The sub-basin widths were later refined study area-wide during model calibration. Please refer to Section 3.2.1 for a discussion of calibration procedures and Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

3.1.4.5 Sub-basin Slope

The slope of each sub-basin was calculated from the LiDAR surface, by converting each sub-basin to a multi-patch polygon and calculating the average slope of the polygons. These values were adjusted during the model calibration process since favorable model results were not obtained. Please refer to Section 3.2.1 for a discussion of calibration procedures and Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

3.1.4.6 Manning’s N-value for Overland Flow over Impervious Portions

The Manning’s N-value for overland flow over impervious portions of the watershed was computed by intersecting the land use shapefile with the sub-basin shapefile (see Section 3.1.2, Watershed Land Use Characterization). The resultant shapefile was then exported to Excel, and weighted values (by area) of this parameter were computed for each sub-basin. These values were not adjusted during the model calibration process.

3.1.4.7 Manning’s N-value for Overland Flow over Pervious Portions

The Manning’s N-value for overland flow over pervious portions of the watershed was computed by intersecting the land use shapefile with the sub-basin shapefile (see Section 3.1.2, Watershed Land Use Characterization). The resultant shapefile was then exported to Excel, and weighted values (by area) of this parameter were computed for each sub-basin. These values were adjusted during the model calibration process since favorable model results were not obtained. Please refer to Section 3.2.1 for a discussion of calibration procedures and Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

3.1.4.8 Depth of Depression Storage on Impervious Portions

The depth of depression storage on impervious portions of the sub-basins was computed by intersecting the land use shapefile with the sub-basin shapefile (see Section 3.1.2, Watershed Land Use Characterization). The resultant shapefile was then exported to Excel, and weighted values (by area) of this parameter were computed for each sub-basin. These values were not adjusted during the model calibration process. Refer to Section 3.2.1 (Model Calibration Efforts) and Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

3.1.4.9 Depth of Depression Storage on Pervious Portions

The depth of depression storage on pervious portions of the sub-basins was computed by intersecting the land use shapefile with the sub-basin shapefile (see Section 3.1.2, Watershed Land Use Characterization). The resultant shapefile was then exported to Excel, and weighted values (by area) of this parameter were computed for each sub-basin. The initial calculated values for Pervious Depression Storage were adjusted during the model calibration process. Refer to Sections 3.1.2 (Watershed Land Use Characterization) and Section 3.2.1 (Model Calibration Efforts), as well as Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

3.1.4.10 Sub-basin Percent Impervious

The percent impervious of each sub-basin was calculated by intersecting the land use shapefile with the sub-basin shapefile. The resultant shapefile was exported to Excel, and the total impervious area within each sub-basin was determined. This total impervious area within each sub-basin was then divided by the total sub-basin area to determine the percent impervious of each sub-basin. The initial calculated values for Pervious Depression Storage were adjusted during the model calibration process. Refer to Section 3.1.2 (Watershed Land Use Characterization) and Section 3.2.1 (Model Calibration Efforts), as well as Tables 2.2.1 and 2.2.2 (Volume II) for the final calibrated parameters.

3.1.4.11 Sub-basin Percent Zero Impervious

This parameter represents the percent of the impervious area with no depression storage; for the purposes of this study, this was assumed to correspond to building footprint areas. A shapefile containing building footprints was contained within the LiDAR dataset. In order to determine the percent zero impervious for each sub-basin, the building footprint shapefile was intersected with the sub-basin shapefile. The resultant shapefile was exported to Excel, and the total building footprint area within each sub-basin was determined. This total building footprint area within each sub-basin was then divided by the total sub-basin impervious area to determine the percent zero impervious of each sub-basin.

3.1.5 Hydraulic Model Parameters

The surveying firms Harris and Associates Land Surveyors, LLC (subconsultant to Neel-Schaffer, Inc.) and Geodesy Professional Services (subconsultant to Powers Hill Design, LLC) were retained to provide storm water infrastructure survey services for this study. Prior to commencement of surveying activities, an extensive review of the University of Memphis GIS data was performed to establish surveying needs for the watershed. Shapefiles were created and distributed to both firms, identifying locations of required drainage structures, channel cross sections, and major road crossings that needed to be surveyed. In addition, a detailed field investigation was performed by Allworld Project Management, LLC to identify additional features to be surveyed.

The required surveying across the entire watershed was divided into multiple phases (Basin A – west of New Allen Road; and Basin B – east of New Allen Road). Once all phases of surveying work were complete, the watershed's comprehensive hydraulic feature inventory was assembled. Below is a brief description of the major components of the watershed's hydraulic feature inventory.

3.1.5.1 Junctions

Approximately 778 junctions were identified within the watershed. Junction types included roadway drainage inlets (i.e. - No. 10, 11, & 12 inlets, 6-72 inlets, etc.), yard inlets (i.e. – 3x3 and 4x4 inlets), and drainage manholes. In addition, survey personnel obtained culvert locations, dimensions, and invert elevations where closed drainage systems discharge into the drainage network.

The survey personnel also provided surveyed junctions within the channels of the study area at the upstream and downstream ends of major road crossings, and at major confluences and other locations required to sufficiently describe the hydraulic properties of the stream network.

As part of their raw survey data, survey personnel provided text files of the junction data. Information provided within this raw survey data included:

- Point/Node ID
- Northing and Easting
- Grate/Structure Type
- Outlet Type (headwall, pipe, etc.)
- Invert Elevations
- Grate Elevations
- Top of Curb Elevations

3.1.5.2 Closed Conduits

While performing the surveying activities throughout the watershed, the survey personnel made note of the dimensions, type, and invert elevation of each conduit entering and existing each junction. This information was provided in an Excel spreadsheet, which defined each conduit by its upstream and downstream node ID. Additional conduit information provided within this spreadsheet included:

- Upstream Grate/Structure Type
- Outlet Type (if conduit is discharging into an open water body)
- Upstream Node ID
- Upstream Grate Elevation
- Downstream Node ID
- Downstream Grate Elevation
- Conduit Type (RCP, CMP, etc.)
- Conduit Dimensions
- Conduit Length
- Conduit Upstream Invert Elevation
- Conduit Downstream Invert Elevation

For ease of modeling and tracking purposes, each conduit was given a distinct identifier, which coincided with its upstream node ID and downstream node ID. For instance, the conduit which connects Junction D06_144 to Junction D06_100 was assigned the Conduit ID D06_144_100. Additionally, some conduits were given a prefix to specify what type of drainage feature is represented by the data. The prefix “S” was used for links representing streets or other paved surfaces and the prefix "Ov" was used for links representing or overland flow paths. These conduit IDs were added to the conduit database for the watershed.

Using the junction shapefile (described above) as a starting point, a new shapefile was created in ArcGIS for the conduits. A command was executed in ArcGIS, which connected each of the junctions with a line, and named that line according to its origination and destination junctions. This naming convention resulted in conduit lines being given the same ID in the shapefile to match the conduit spreadsheet. The spreadsheet was joined to the conduit shapefile (by conduit name), which resulted in a comprehensive watershed conduit shapefile. In total, there are approximately 919 conduit links within the watershed with 531 of them representing closed conduits.

3.1.5.3 Box Culverts

Of the approximately 531 closed conduit links within the watershed, there are 32 box culverts which correspond to major road crossings within the study area. While performing the surveying activities throughout the watershed, survey personnel made note of the following box culvert information in an Excel spreadsheet:

- Road Name (at channel crossing)
- Structure Type (reinforced concrete box culvert)
- Number of Structure Barrels
- Width, Height, and Sump of Structure
- Structure Length
- Structure Upstream Invert Elevation
- Structure Downstream Invert Elevation

The conduit shapefile was updated to ensure that the links associated with these eleven box culverts included the correct structural information.

3.1.5.4 Channel Cross Sections

Surveyor personnel collected ground data for 211 channel cross sections within the Allen Basin study area. Information included survey point numbers, elevations, and descriptions (ground, channel toe of slope, channel top of bank, center of channel flow, roadway centerline, etc.). Cross sections were located immediately upstream and downstream of road crossings, at breaks in channel shape/slope, and near major inflow locations.

The surveyed channel cross sections were merged with expanded ground data derived from LiDAR surface data in Excel, in order to encompass the anticipated floodplain boundaries at each location. Then, cross section lines were intersected with the land use shapefile to obtain left and right overbank Manning's N-values for each cross section. The channel N-value was derived from field notes, site photos, and/or aerial imagery. Each combined cross section was associated with its respective conduit (i.e., channel) link within InfoSWMM. Cross section plots are included in Volume II of this report (see Exhibit 7.1).

3.1.6 Historical Gage Data

Historical rain gage and stage gage data was supplied by the University of Memphis for use in calibrating the hydraulic model for this study. Rainfall depths, recorded at 5-minute increments, were provided for the period from November 30, 2017, to April 12, 2018 at the following rain gage:

- Rain gage identification number EM28449, located at Fire Station Number 49 (4351 New Allen Road)

In addition, water stage/depth values recorded at 5-minute increments were provided for the same two time periods at the following two pressure transducers/crest stage gages:

- Stage gage A-1, serial number 2032152 (New Allen Road; located in concrete median channel approximately 446 feet southwest of the intersection of New Allen Road and Egypt Central Road; and
- Stage gage A-2, serial number 2034468 (Tessland Road; located in concrete channel approximately 30 feet downstream of the Tessland Road cross drain outlet and approximately 150 feet west of Trudy Street)

The location of the gages is shown in Volume II. Exhibit 3.1 shows the gage locations within the city and Exhibit 5.1 shows the location of the one rain gage and two stream gages with respect to the study's watershed boundary limits. Upon analysis of rainfall and corresponding channel stage data, two storm events were isolated and selected as the calibration events:

- Calibration Event 1 occurred on December 22, 2017. A total of 3.08 inches of rainfall was recorded between 5:50 A.M. and 5:55 P.M. (a duration of 12 hours and 5 minutes) on that date. This rainfall event roughly corresponds to the 1-year 12-hour rainfall event taken from NOAA Atlas 14 Frequency/Duration data. Calibration Event 1 has the largest recorded peak in the stage records.
- Calibration Event 2 occurred from 8:15 P.M. on February 27, 2018, to 8:50 A.M. on March 1, 2018. A total of 4.81 inches of rainfall was recorded in 36 hours and 35 minutes. This event roughly corresponds to the 3-year 36-hour rainfall event taken from NOAA Atlas 14 Frequency/Duration data. Calibration Event 2 is a multiple peak event chosen to evaluate the timing of the InfoSWMM model.

Once the InfoSWMM model was calibrated (see Section 3.2.1, Model Calibration Efforts), a third rainfall event was utilized as the model verification event. This storm event produced about the same amount of rainfall volume as Calibration Event 1 but was slightly less intense.

- Verification Event occurred from 3:05 A.M. on February 21, 2018 to 3:40 A.M. on February 22, 2018. A total of 3.41 inches of rainfall was recorded in 24 hours and 35 minutes. This storm roughly corresponds to the 1-year 24-hour rainfall event taken from NOAA Atlas 14 Frequency/Duration data.

All rainfall gages and pressure transducers were operational during the three storm events. Since none of the rainfall events were large enough to result in flooding, no high-water marks (water stain or debris lines) were observed and no corresponding flood depths were measured in the field.

The location of the three gages, photographic documentation of two stream gage locations and a summary of the analysis of the calibration and verification rainfall events is presented in Volume II, Exhibit 3.1.

3.1.7 SCS Design Storm Data

Once the InfoSWMM model was calibrated/verified to the maximum extent practicable, frequency rainfall data was entered into the model. The following table summarizes storm frequency, duration, and rainfall depths found in Table 2-2 of the City of Memphis/Shelby County Storm Water Management Manual:

Table 3. Table of NRCS (SCS) 24-Hour Rainfall Depths (From Table 2-2 of Memphis/Shelby County Storm Water Management Manual)

Frequency	Duration	Rainfall Depth (in)
2 year	24 hour	4.01
5 year	24 hour	4.89
10 year	24 hour	5.58
25 year	24 hour	6.52
50 year	24 hour	7.27
100 year	24 hour	8.02

The NRCS (SCS) Type II 24-hour storm precipitation distribution was utilized to develop frequency rainfall data for use in the InfoSWMM model. The frequency rainfall depths shown in the table above were input to the InfoSWMM model and were utilized to simulate the various frequency storms for existing condition and proposed alternative conditions.

3.1.8 InfoSWMM Boundary Conditions

The Allen Basin Study Area contains four watersheds draining to three outfalls along the Loosahatchie River Drainage Canal (refer to Exhibits 5.1, 5.2 and 5.3 in Volume II). The outfall for sub-watershed A is located approximately 2,200 feet (0.4 miles) downstream (west) of the Canadian National Railroad (Railroad) bridge across the Loosahatchie River Drainage Canal (northeast of Sunny Hill Drive and Sunnybrook Drive). Watersheds B and C share an outfall, which is located immediately upstream of the Railroad bridge. The outfall for Watershed D is located approximately 5,900 feet (1.1 miles) upstream (east) of the Railroad bridge and approximately 6,675 feet (1.26 miles) downstream (west) of the Raleigh Millington Road bridge.

The three outfalls included in the study area are all subject to tailwater influence from Loosahatchie River Drainage Canal. In order to determine the boundary conditions to be utilized in the InfoSWMM model, the Flood Insurance Study (FIS) for Shelby County, Tennessee and Incorporated Areas (revised February 6, 2013) was reviewed. The water surface elevations from the Loosahatchie River Drainage Canal Flood Profile were plotted using a logarithmic scale against corresponding annual chance percentages. From this logarithmic plot, the 2-, 5-, and 25-year water surface elevations were extrapolated. These values were used as initial estimates of outfall tailwater boundary conditions.

According to data found within the FEMA FIS, the backwater from Loosahatchie River Drainage Canal extends significantly upstream in each of the four watersheds; this results in an unrealistic condition by which to evaluate local drainage conditions. It was noted from the FIS that the drainage area of the Loosahatchie River Drainage Canal downstream of the Railroad bridge is approximately 570 square miles, while the drainage area to each of the three Allen Basin outfalls range from 0.35 square miles to 5.56 square miles. A coincident peak analysis of the watersheds is beyond the scope of this study. However, it is unlikely that frequency event discharges (i.e., 2-year through 100-year) would occur on the Loosahatchie River Drainage Canal coincident with peaks from the local watershed. Additionally, no high-water marks were available for the Allen Basin study area to aid in calibration of boundary conditions. Based on the analysis of high backwater from the Loosahatchie River Drainage Canal, it was determined that using normal depth settings for the outfalls provided a more realistic representation of the hydraulic and hydrologic response of the four watersheds to local storm events.

3.2 Existing Condition Analysis and Review

3.2.1 Model Calibration Efforts

As described in Section 3.1.6, Historical Gage Data, two rainfall events were selected for calibration of the InfoSWMM model. Calibration of modeling parameters within the InfoSWMM model was performed to attempt to reproduce the maximum recorded pressure transducer reading at New Allen Road and Tessland Road nodes within the model for both rainfall events. Green-Ampt parameters were varied by significant amounts. However, the model proved to be more sensitive to other modeling parameters. Therefore, the Green-Ampt parameters were reset to the original values. The optimal combination of adjustments included globally increasing parameters as shown below (refer to Tables 2.2.1 and 2.2.2, Volume II):

- Sub-catchment Width, -30%
- Sub-catchment Slope, -20%
- Manning's N for Pervious Portion, +30%
- Depression Storage for Pervious Portion, +30%
- Sub-catchment Imperviousness, -10%

It is noted that even though the calibration events were the largest recorded rainfall events, they were relatively minor storms. The total rainfall produced the December 22, 2017 storm was 3.08 inches in 12 hours and 5 minutes. The total rainfall produced by the February 27, 2018 to March 1, 2018 storm was 4.81 inches in 36 hours and 35 minutes. When compared to point precipitation frequency estimates from NOAA Atlas 14, these storms roughly compare to the 1-year 12-hour and 3-year 36-hour storms, respectively. The large adjustments to the modeling parameters shown above are a result of calibrating the InfoSWMM model to such a small rainfall event.

Through multiple calibration iterations, calibration model results obtained were within ± 0.5 foot of all gage/measured elevations for the New Allen Road and Tessland Road gages. Refer to Table 3.2, Calibration and Verification Summary Table, within Volume II for more information. A comparison between the originally calculated hydrologic parameters and the final calibrated hydrologic parameters is included in Tables 2.2.1 and 2.2.2, Comparison of Final Calibrated Hydrologic Model Parameters with Initial Calculated Values (Volume II).

Once the InfoSWMM model was considered to be calibrated, the February 21-22, 2018 storm event was entered into the model and executed to verify results against maximum pressure transducer readings taken at the New Allen Road and Tessland Road gages (see Section 3.1.6, Historical Gage Data). The total rainfall produced by this storm was 3.41 inches in 24 hours and 35 minutes. Verification results at the both gage locations were within ± 0.5 foot of all gage/measured elevations .

Given the small rainfall events recorded at the gages, and the extent of adjustment to the modeling parameters, the model was considered to be "calibrated" and “verified” and was therefore used for frequency storm analysis.

3.2.2 General Discussion of Modeling Approach

The InfoSWMM model generated as part of this study produces detailed results within the subject study area, which consists of older urban development with smaller, undersized stormwater conveyance features that experience ponding of runoff in higher storm events. During the model setup, care was taken to identify all the storage areas where water would pond due to inadequate pipe system and inlet capacity and downstream grades. However, one area where the program did not respond well is calculating the water split (intercepted vs bypassed) on interceptor inlets. The percentage of water intercepted by an interceptor inlet is variable dependent on the flow quantity flowing to the inlet, but the program uses a fixed interception rate. This will tend to over-predict the bypass amount in the smaller storms and under-predict it in the larger ones. In the Recommended Improvement section of the report, additional inlet bypass studies were recommended to make sure adequate inlets would be made available in the sump areas when the improvement designs were performed.

3.2.3 General Discussion of Modeling Calibration

All the storms used in the model calibration and verification were low intensity storms. The December 22, 2017 storm (Calibration Event 1) had 3.08 inches of rainfall in the 12-hour 5-minute period between 5:50 am and 5:55 pm, per the gage at Fire Station No. 49 (refer to Figure 1 below). Per NOAA Atlas 14 Depth Duration Frequency Curves this storm roughly corresponds to the 1-year 12-hour event.

The February 27 – March 1, 2018 storm (Calibration Event 2) had 4.81 inches of rainfall in the 36-hour 35-minute period between 8:15 pm on February 27 to 8:P50 am on March 1 (refer to Figure 2 below). Per NOAA Atlas 14 Depth Duration Frequency Curves this storm roughly corresponds to the 3-year 36-hour event.

The February 21-22 storm (Verification Event) had 3.41 inches of rainfall in the 24-hour and 35-minute period between 3:05 am on February 21 and 3:40 am on February 22 (refer to Figure 3 below). Per

NOAA Atlas 14 Depth Duration Frequency Curves this storm roughly corresponds to the 1-year 24-hour event.

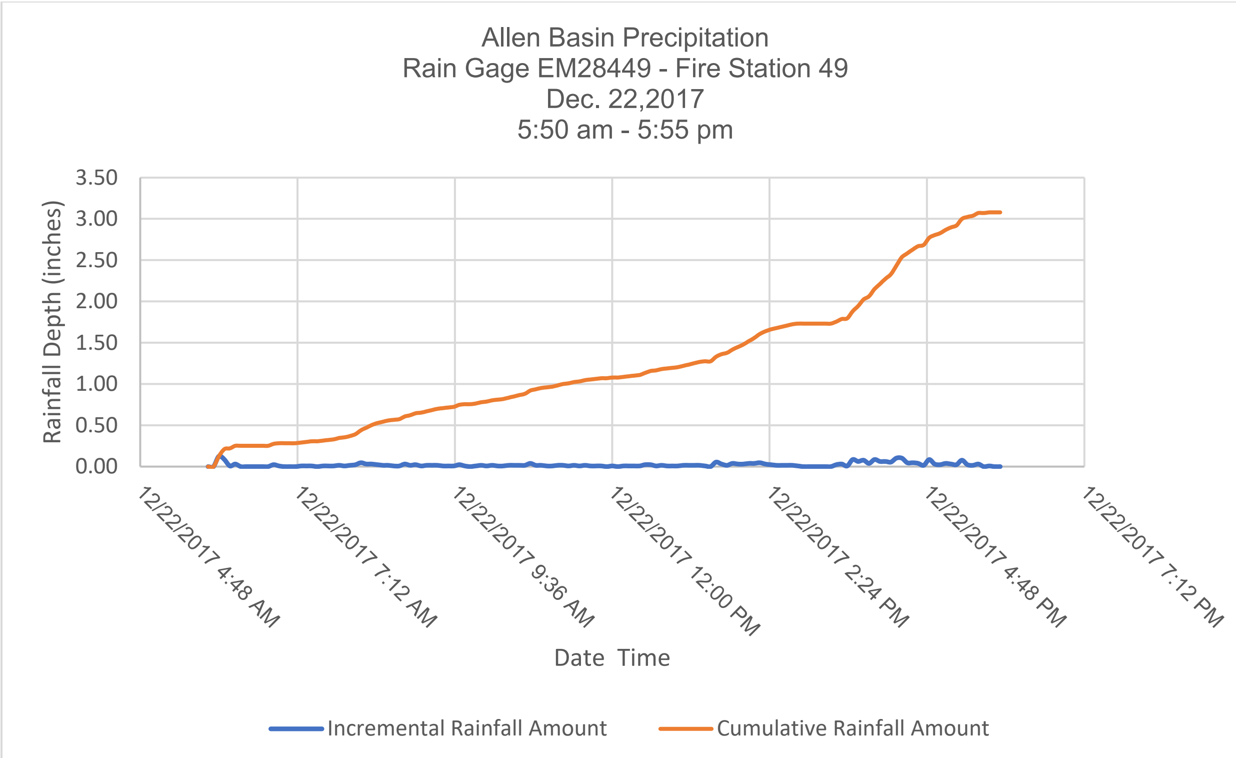

Figure 1. Calibration Rainfall Event 1 - December 22,2017

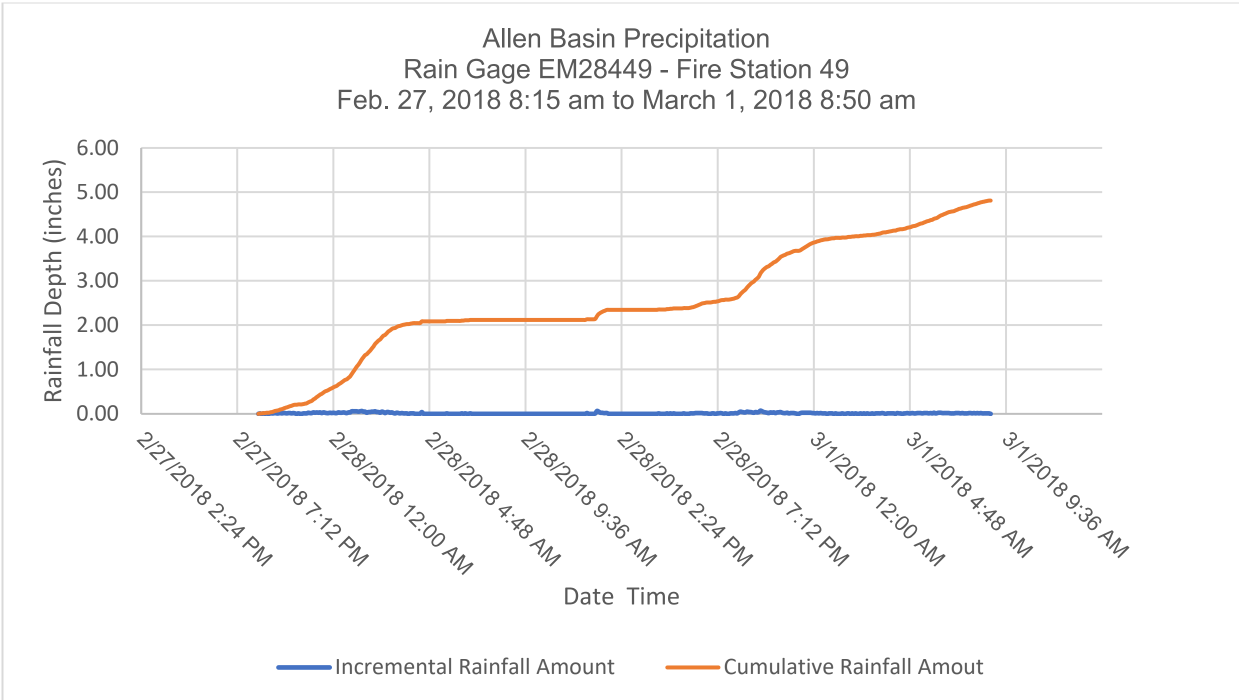

Figure 2. Calibration Rainfall Event 2 - February 27 to March 1, 2018

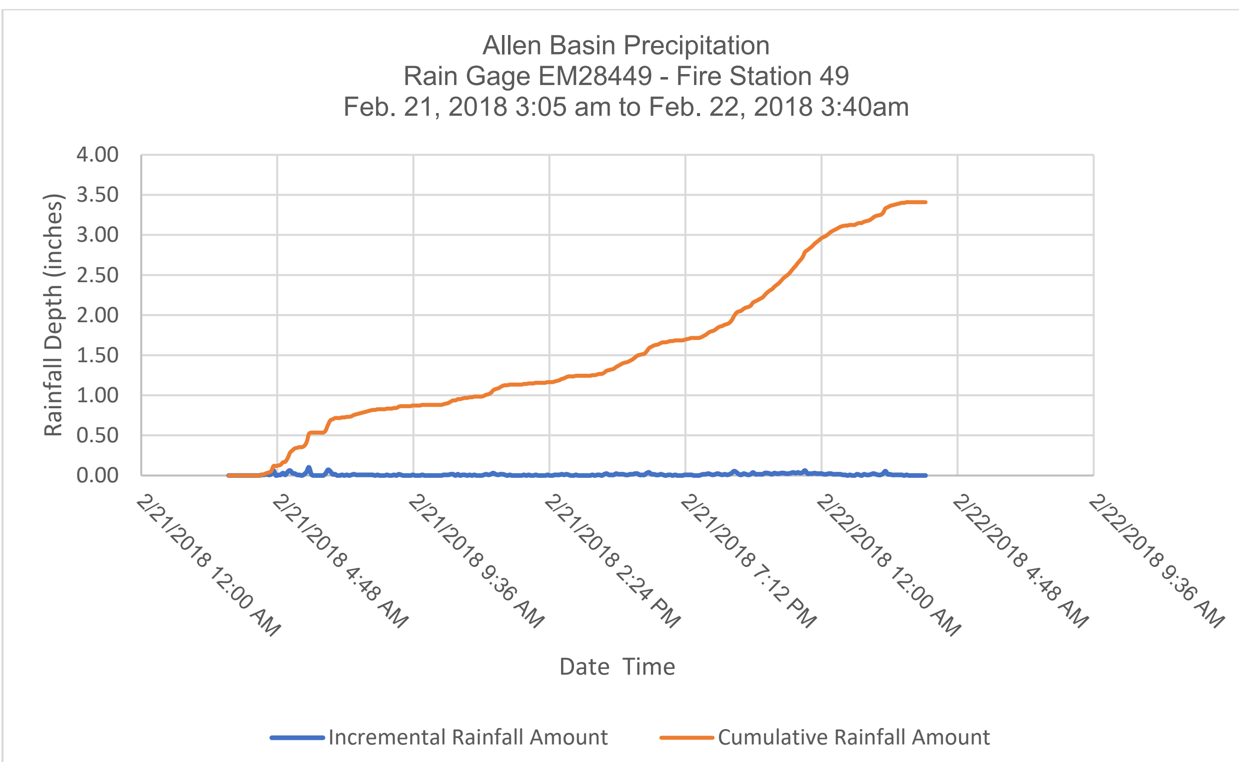

Figure 3. Verification Rainfall Event - February 21 to February 22, 2018

3.3 Development and Modeling of Potential Solutions

3.3.1 Modeling Scenarios using InfoSWMM Scenario Manager

One of the advantages of using the InfoSWMM software for this watershed analysis is its ability to model unlimited drainage/flooding improvement scenarios within the same model. Once the Existing Condition analysis was completed, various areas that experience frequent and severe flooding were evaluated for the potential for future drainage improvements. Each area was assigned a number with a prefix to designate the watershed in which the area is located. For example, area A-1 is the first area evaluated in Watershed A. A total of 28 areas were evaluated (refer to Exhibit 5.2 with Volume II).

InfoSWMM's Scenario Manager was used to model improvements for 8 of these areas. Multiple variations and combinations of improvements were analyzed to determine the optimal recommendations. A total of 10 different scenarios were analyzed within the Scenario Manager to evaluate potential drainage improvements, as described below. Each scenario was associated with the six frequency rainfall events (i.e., 2- through 100-year rainfall) to evaluate the impact of the improvements on various levels of flooding. Table 10.1 in Volume II provides a list of the InfoSWMM scenarios used to evaluate drainage improvement alternatives in the Allen Basin Study Area.

3.3.2 Selection of Drainage Improvement Project Areas

The InfoSWMM model of existing condition and the existing condition floodplains were reviewed to determine problem areas and potential drainage improvement areas. A total of 28 areas of existing flooding were reviewed but not all areas selected for improvement evaluation. In order to prioritize flooding areas, in terms of potential improvements, the following criteria were used (listed in order of importance):

- Finished floor elevation (FFE) flooding of residential structures;
- Street and roadway level of service (LOS); and
- The potential of floodwaters impeding the ingress/egress of emergency vehicles and the disruption of traffic flow were considered.

Based on the above criteria, 8 areas were selected for the evaluation of potential drainage improvements. The locations of the projects areas are shown on Exhibit 5.3 (Volume II). The following sections discuss each project area.

3.3.3 Allen Basin Watershed Recommended Drainage Improvements

Eight areas were identified within the Allen Basin Study Area that have the potential for significant mitigation of flooding impacts through infrastructure improvement. Exhibit 5.3 (Volume II) shows the location of each project area. The following sections provide a detailed discussion of each project area and the recommended drainage improvements.

3.3.3.1 Area A-1 – Sunny Hill Drive

Existing Condition

Area A-1 extends from Sunny Hill Drive southeast to Beacon Hill Drive and is shown on Exhibits A-1.1 and A-1.2 (Volume II, Section 8). Two existing closed drainage systems convey drainage from the south and form a junction behind 2573 Sunny Hill Drive. The western system begins near the intersection of Davey Drive and Davey Circle. The storm sewer is comprised of various size pipes and conveys drainage north between Suncrest Drive and Beacon Hill Drive to the previously mentioned junction. The eastern storm sewer system begins on Range Line Road and extends to the north to the junction behind 2573 Sunny Hill Drive. The total drainage area at the junction is approximately 71 acres. A 54" RCP trunk line runs from the junction to the system outfall on the north side of Sunny Hill Drive. The total drainage area at the outfall is approximately 87 acres. Additionally, an 18" RCP with a 6-72 curb inlet is located on Sunny Hill Drive and serves to drain the road and adjacent areas. Land use within the drainage area is primarily residential.

The InfoSWMM Model of existing condition shows that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of both Sunny Hill Drive and Beacon Hill Drive is less than the 2-year storm event. This low LOS shows the potential of flood water to impede the ingress/egress of emergency vehicles during relatively minor flooding events. The existing condition model also indicates that at least one house within the project area, 4527 Beacon Hill Drive, may experience flooding of the finished floor elevation (FFE) during the 5-year storm event. The existing condition model also shows that 29 parcels in the project area experience nuisance flooding during the 2-year event, and 46 parcels experience nuisance flooding during the 100-year event.

The site was selected for potential drainage improvements because of the low roadway LOS. Frequent flooding of Sunny Hill Drive and Beacon Hill Drive has the potential to impede ingress/egress of vehicles in the event of an emergency. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing the pipe sizes.

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. The existing system running from Beacon Hill Drive to the junction behind 2573 Sunny Hill Drive is a 30" RCP and would need to be replaced by a 48" RCP to reduce the flooding on Beacon Hill Drive. The existing LOS of Beacon Hill drive is less than the 2-year storm event. With the installation of the 48" RCP, the improved LOS would be the 10-year storm event. The 18" RCP lateral pipe on Sunny Hill Drive would also be replaced with a 36" RCP. This would increase the LOS of Sunny Hill Drive from less than the 2-year storm event level to the 100-year storm event.

The existing pipe runs between 4527 and 4531 Beacon Hill Drive. The installation of a larger diameter pipe at this location presents several constructability and easement issues due to the proximity of the residential structures. The construction and design challenges of this option, together with the relatively low LOS improvements at Beacon Hill Drive, makes this option questionable from a construction cost viewpoint. Therefore, a second alternative was considered.

RECOMMENDED ALTERNATIVE

Alternative 2 – Infrastructure Improvements

During a meeting with the City, it was noted that the parcels located at 4566 Sunny Brook Drive and 4543 Beacon Hill Drive are currently vacant. It was decided to explore the possibility of using the vacant lots to re-route the Beacon Hill Drive storm sewer system and avoid the constructability issues of installing a larger diameter pipe between houses. Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system.

The recommended improvements, as shown on Exhibits A-1.1 and A-1.2 within Volume II and as summarized below, start at the 6-72 curb inlets located near 4527 and 4531 Beacon Hill Drive. The exiting 30” RCP that runs north between the houses will be plugged and abandoned in-place. The two 6-72 curb inlets and lateral pipes will be replaced and will drain into a new drainage manhole (DMH) in the middle of Beacon Hill Drive. The new drainage system then extends east to the cul-de-sac at Beacon Hill Drive and Mirror Avenue, then turns north toward Sunny Hill Drive and eventually discharges near the outfall location.

A summary of the recommended improvements is shown below and can be found on Exhibits A-1.1 and A-1.2:

- Plug existing 30” RCP and abandon in place;
- On the north side of Beacon Hill Drive, replace 16 linear feet 30” RCP with 24” RCP draining to a new DMH in the middle of Beacon Hill Drive. Remove and replace existing 6-72 inlet;
- On the south side of Beacon Hill Drive, replace 16 liner feet of 30” RCP with 36” RCP draining to a new DMH in the middle of Beacon Hill Drive. Remove and replace existing 6-72 inlet;
- Install 190 linear feet of 48” RCP from the new DMH in the middle of Beacon Hill Drive east to the new DMH at the intersection of Beacon Hill Drive and Mirror Avenue;
- Install new DMH at the intersection of Beacon Hill Drive and Mirror Avenue;
- Install 280 linear feet 48” RCP north to new DMH at Sunnybrook Drive;
- Install new DMH in Sunnybrook Drive;
- Install 74 linear feet 48” RCP east to a new DMH;
- Install 144 linear feet 48” RCP north to new system outfall;
- Install 102 linear feet of grass channel to from outfall to main stream;
- Remove and replace two existing 6-72 curb inlets at Sunny Hill Drive;
- Replace existing 33 linear feet of 18” RCP with 36” RCP;
- Remove and replace existing 4 x 4 inlet behind 4524 and 4528 Suncrest Drive;
- Replace 228 linear feet of 36” RCP with 42” RCP from 4 x 4 inlet to junction behind 2573 Sunny Hill Drive; and
- Install headwall and wingwalls with permanent erosion control measures at new storm sewer outfall north of Sunnybrook Drive.

The greater capacity of the improvements described above would improve the flooding issues at Sunny Hill Drive and Beacon Hill Drive. The existing LOS of both Sunny Hill Drive and Beacon Hill Drive is less than the 2-year storm event. With the proposed drainage system, the LOS of both streets would be improved to the 100-year storm event. The FFE of the house at 4527 Beacon Hill Drive is below the existing 5-year hydraulic grade line (HGL). With the proposed system installed, the FFE at 4527 Beacon Hills Dr. would be above the proposed 100-year HGL. Additionally, nuisance flooding would be reduced from 39 parcels to 8 parcels during the 2-year storm event and from 46 to 31 parcels during the 100-year storm event. The proposed improvements would increase the flooding depth downstream of Sunny Hill Drive by a maximum of 0.42 feet. No structures are present in the floodplain area immediately downstream of Sunny Hill Drive.

The stormwater drainage system improvements described about will require new easements to be obtained on the vacant lots at 4543 Beacon Hill Drive and 4566 Sunny Brook Drive (refer to Exhibit A-1.1, Volume II, Section 8).

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 4. Project Area A-1 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Sunny Hill Drive	< 2-year Storm Event	100-year Storm Event
Beacon Hill Drive	< 2-year Storm Event	100-year Storm Event

Table 5. Project Area A-1 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
4527 Beacon Hills Drive	FFE Below 5-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- Final storm sewer alignment and easement restrictions will need to be evaluated since the storm sewer will be re-routed through the currently vacant lots;
- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities will be a factor in the final storm sewer design;

- The utility survey should include a comprehensive survey of sanitary sewer locations and invert elevations on Beacon Hill Drive, Sunny Hill Drive and Sunnybrook Drive;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access; and
- Permanent erosion control measures may be required at the outfall of the proposed storm sewer system north of Sunnybrook Drive.

3.3.3.2 Area D-9 – Gruber Drive

Existing Condition

Area D-9 extends from Canadian National Railroad (Railroad) west to the intersection of Gruber Drive and Ferdie Cove and is shown on Exhibits D-9.1 and D-9.2 (Volume II, Section 8). An existing closed drainage system conveys drainage from Ferdie Cove east under Gruber Drive, then northeast through a residential development to the Gruber Cove cul-de-sac. From there, the storm sewer runs northeast to the system outfall just west of the Canadian National Railroad. An open channel carries stormwater discharges from the system outfall to the cross drain under the Railroad. Pipe sizes of the storm sewer system range from 18” RCP at Ferdie Cove to 42” RCP at the system outfall. The cross drain under the Railroad is a 48” CMP.

When the existing storm sewer system becomes surcharged, overflow discharges to the surface resulting in flow along streets and overland swales between houses before reaching the system outfall. The surface discharges begin when the Ferdie Cove inlets become surcharged. Overland flow paths generally follow the storm sewer alignment (see Exhibits D-9.1 and D-9.2, Volume II, Section 8) and contribute to the roadway and FFE flooding issues within the project area.

The drainage area of the Ferdie Cove inlets is approximately 5 acres, and the drainage area at the storm sewer outfall is approximately 36 acres. The total drainage area at the Railroad cross drain is approximately 42 acres. Land use within the drainage area is primarily residential with open space adjacent to the railroad embankment.

The InfoSWMM Model of existing condition shows that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area range from less than the 2-year storm event to the 5-year storm event. Three houses within the project area may experience flooding of the finished floor elevations (FFEs) during the 2-year through 10-year storm events. Additionally, nuisance flooding potential impacts 38 parcels in the project area during the 2-year storm event and 46 parcels during the 100-year event.

The site was selected for potential drainage improvements due to the low roadway LOS and FFE flooding. Frequent flooding of Gruber Drive and Ajanders Drive has the potential to impede ingress/egress of vehicles in the event of an emergency. Flooding of Gruber Cove has the potential to isolate six residential properties from emergency services. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing the pipe sizes.

RECOMMENDED ALTERNATIVE

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer system thereby reducing surcharge depths and surface flow.

As stated previously, a significant portion of the existing storm sewer system runs from Gruber Drive northeast to Gruber Cove, then continues northeast to Ajanders Drive. This portion of the existing system passes between numerous residential structures. Replacing the existing storm sewer in this area with larger diameter pipes would present significant constructability problems and utility conflicts. Therefore, it is recommended that a new storm sewer line be installed along Gruber Drive east to Ajanders Drive, then north to intersect the existing system (see Volume II, Section 8, Exhibits D-9.1 and D-9.2). This alignment not only avoids the constructability issues of the residential properties but utilizes public right-of-way as well.

The recommended improvements, as shown on Exhibits D-9 .1 and D-9.2 and summarized below, start at Ferdie Cove and extend to the Railroad.

- Replace 552 linear feet of 18” RCP with 24” RCP;
- Replace 60 linear feet of 18” RCP with 36” RCP;
- Replace 157 linear feet of 27” RCP with 36” RCP;
- Replace 30 linear feet 21” RCP with 24” RCP;
- Plug existing 36” RCP and abandon in place;
- Install 600 linear feet 48” RCP and 2 DMH’s from existing 36” RCP east to the intersection of Gruber Drive and Ajanders Drive;
- Install 555 linear feet of 48” RCP and 2 DMH’s from the intersection of Gruber Drive and Ajanders Drive north to the existing storm sewer system;
- Replace 137 linear feet 36” RCP with 48” RCP;
- Replace 80 linear feet 42” RCP with 54” RCP;
- Install headwall and wingwalls at system outfall;
- Replace 130 linear feet 21” RCP with 24” RCP from Ajanders Cove to storm sewer trunk line;
- Replace 50 linear feet 48” CMP with 2 – 54” RCP (Canadian National Railroad);
- Replace inlets and DMH as required; and
- Install headwall and wingwalls with permanent erosion control measures as required at the storm sewer outfall and outlet of Railroad culvert.

The greater capacity of the improvements shown above would improve the flooding issues at for all streets within the project area. Gruber Drive, Gruber Cove and Ajanders Drive. The existing LOS of both Gruber Drive and Ajanders Drive is less than the 2-year storm event. With the proposed drainage system, these two streets would be improved to the 10-year storm event. The existing LOS of Gruber Cove is less than the 2-year storm event and with the proposed improvements would be improve to the 50-year storm event. As shown on the tables below, adjacent streets within the project area would also see a significantly increased LOS. FFE flooding within the project area would also be improved. Nuisance flooding in the project area would be reduced from 38 parcels to 3 parcels during the 2-year storm event, and from 46 parcels to 42 parcels during the 100-year storm event.

The proposed improvements would increase the flooding depth downstream of the railroad cross drain by a maximum of 0.13 feet. No structures are present in the floodplain area immediately downstream of the Railroad.

The proposed alignment re-routes the exiting storm sewer east along Gruber Drive then north along Ajanders Drive. This proposed alignment avoids significant constructability issued that would be encountered with enlarging the existing storm sewer system in the residential areas.

The importance of the culvert under the railroad should be noted. As with the entire drainage system in the project area, the existing 42" CMP culvert under the railroad has insufficient capacity to adequately drain such a large area. The surcharging of the Railroad culvert causes backwater upstream to the storm sewer system outfall, thereby further decreasing the storm sewer capacity. Therefore, the replacement of the existing 42" CMP railroad culvert with 2 – 54" RCP's is a critical design component. Coordination with Canadian National Railroad will be required during the design and construction of the railroad cross drain.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 6. Project Area D-9 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Ajanders Cove	2-year Storm Event	25-year Storm Event
Ajanders Drive	< 2-year Storm Event	10-year Storm Event
Gruber Cove	< 2-year Storm Event	50-year Storm Event
Gruber Drive	< 2-year Storm Event	10-year Storm Event
Point Church Road	< 2-year Storm Event	25-year Storm Event
Ferdie Cove	5-year Storm Event	100-year Storm Event

Table 7. Project Area D-9 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3921 Gruber Cove	FFE Below 25-year HGL	FFE Above 50-year HGL
3888 Point Church Road	FFE Below 5-year HGL	FFE Above 10-year HGL
3896 Point Church Road	FFE Below 5-year HGL	FFE Above 25-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access;
- Permanent erosion control measures may be required at the outfall of the storm sewer system and the outlet of the Railroad culvert; and
- Coordination with Canadian National Railroad will be necessary for the design and construction of the Railroad Cross drain.

3.3.3.3 Area D-12 – Ridgemont Road and Trudy Street

Existing Condition

Area D-12 includes Trudy Cove, Ridgemont Avenue, and Trudy Street and is shown on Exhibit D-12 (Volume II, Section 8). An existing closed drainage system conveys drainage from Ridgemont Avenue west to the cross drain under Ridgemont Avenue between Windermere Road and Hobson Cove. The stormwater discharges are conveyed north from Ridgemont Avenue in an open channel to the Trudy Cove cross drain. The Ridgemont Avenue Storm sewer is a 27” RCP. The Ridgemont Avenue cross drain is an 11-foot wide by 5.83-foot high Reinforced Concrete Box Culvert (RCBC) and the Trudy Cove cross drain is a 15-foot wide by 5.83-foot high RCBC.

When the existing storm sewer system becomes surcharged overflow discharges to the surface resulting in flow west along Ridgemont Avenue and southwest through overland swales between houses to Trudy Street. The overland discharges then travel south on Trudy Street to the inlets at Emerson Avenue (Refer to Exhibit D-12). The overland discharges contribute to the roadway and FFE flooding issues on Trudy Street.

The drainage area of the Ridgemont Avenue inlets is approximately 15 acres, and the drainage area at the storm sewer outfall (at the Ridgemont Avenue RCBC) is approximately 26 acres. The total drainage area at the Ridgemont Avenue cross drain is approximately 517 acres, and the drainage area of the Trudy Cove cross drain is 522 acres. Land use within the drainage area is primarily residential with areas of open space in the upper reaches of the watershed.

The InfoSWMM Model of existing condition indicates that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area range from less than the 2-year storm event to the 2-year storm event. Seven houses within the project area may experience flooding of the finished floor elevation (FFE) during the 2-year through 10-year storm events. In addition, 68 properties within the project area experience nuisance flooding during the 2-year storm event, and 81 properties experience nuisance flooding during the 100-year storm event.

The site was selected for potential drainage improvements because of the low roadway LOS and FFE flooding. Frequent flooding of Trudy Cove, Ridgemont Avenue, and Trudy Street has the potential to impede ingress/egress of vehicles in the event of an emergency. Additionally, flooding of these roadways has the potential to isolate more than a dozen residential properties from emergency services. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing pipe sizes.

RECOMMENDED ALTERNATIVE

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer systems, thereby reducing surcharge depths and surface flow.

The recommended improvements, as shown on Exhibit D-12 (Volume II, Section 8) and summarized below, start at Trudy Cove. The backwater from Trudy Cove RCBC reduces the capacity of the Ridgemont Avenue RCBC, which in turn reduces the capacity of the Ridgemont Avenue storm sewer. Even with the improvement of the Trudy Cove RCBC, the Ridgemont Avenue RCBC must be improved to reduce the flooding at Ridgemont Avenue. The Ridgemont Avenue storm sewer improvements utilize larger diameter pipes along the same storm sewer alignment.

Enlargement of the Ridgemont Avenue storm sewer reduces the surcharge at the intersection of Ridgemont Avenue and Ridgemont Cove, thereby reducing the surface discharges to Trudy Street. However, the reduction of the overland flow is not enough to mitigate the street and FFE flooding on Trudy Street. Therefore, the drainage pipe that runs between 4241 and 4247 Trudy Street must also be improved.

A summary of the recommended improvements is shown below and can be found on Exhibit D-12:

- Replace 53 linear feet of 15-foot wide by 5.83-foot high RCBC with two 14-foot wide by 8-foot high RCBC (Trudy Cove);
- Replace 82 linear feet of 11-foot wide by 5.83-foot high RCBC with two 10-foot wide by 7-foot wide RCBC (Ridgemont Avenue);
- Replace 272 linear feet of 27” RCP with 48” RCP (Ridgemont Av);
- Replace 587 linear feet of 27” RCP with 42” RCP (Ridgemont Av);
- Replace 138 linear feet of 30” RCP with a single 6-foot wide by 4-foot high RCBC (Trudy St);
- Install 138 linear feet of 6-foot wide by 4-foot high RCBC (Trudy St);
- Replace inlets and DMH as required; and
- Install permanent erosion control measures at all outfalls and outlets as required.

The existing drainage system between 4241 and 4247 Trudy Street is a 30” RCP. Modeling of the system shows that this system requires much more stormwater conveyance capacity than the existing system provides. However, due to the close spacing of the homes in this area construction of the required drainage pipes would be problematic. Therefore, this analysis assumes that two separate 6-foot wide by 4-foot high RCBC’s would be constructed passing between two separate homes (refer to Figure D-12). This assumption also presents constructability issues such as utility conflicts and building foundation stabilization. Such issues will need to be evaluated and addressed during the design phase should this project be selected for construction.

The greater capacity of the improvements shown above would improve the flooding issues on Trudy Cove, Ridgemont Avenue, and Trudy Street. The existing LOS of Trudy Cove is the 2-year storm event and the LOS with the recommended drainage improvements is the 100-year storm event. The existing LOS of Ridgemont Avenue at the RCBC is less than the 2-year storm event and the improved LOS is the 25-year storm event. The existing LOS of Ridgemont Avenue east of the RCBC, at Hobson Cove and Ridgemont Cove, is less than the 2-year storm event. The improved LOS at these locations are the 50-year and 100-year storm events, respectively. The existing LOS of Trudy Street is the 2-year storm event, and the improved LOS is the 100-year storm event.

As stated previously, houses in the project area experience FFE flooding during large rainfall events with the existing drainage system. With the recommended improvements, the FFE of all seven houses evaluated would be above the improved 100-year flood elevation. Reduction of nuisance flooding with the proposed drainage improvement is minimal. During the 2-year storm event, the number of properties experiencing nuisance flooding is reduced from 68 to 56. During the 100-year storm event 73 properties experience nuisance flooding with the proposed improvements, while 81 properties experience nuisance flooding with the existing system.

A cross drain is located at New Allen Road approximately 1,340-feet downstream of Trudy Cove. The proposed improvements would increase the flooding depth at New Allen Road by 0.18 feet during the 2-year storm event. The flooding depth at New Allen Road during the 5-year through 100-year storm events ranges from 0.0’ to 0.07’. These slight increases in depth are not significant enough to affect the LOS of New Allen Road. Additionally, there are no residential structures or other buildings within the 100-year floodplain between Trudy Cove and New Allen Road.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 8. Project Area D-12 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Trudy Cove	2-year Storm Event	100-year Storm Event
Ridgemont Road	< 2-year Storm Event	25-year Storm Event
Ridgemont Rd @ Hobson Cove	< 2-year Storm Event	50-year Storm Event
Ridgemont Rd @ Ridgemont Cove	< 2-year Storm Event	100-year Storm Event
Trudy Street	2-year Storm Event	100-year Storm Event

Table 9. Project Area D-12 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3635 Trudy Cove	FFE Below 25-year HGL	FFE Above 100-year HGL
3645 Trudy Cove	FFE Below 25-year HGL	FFE Above 100-year HGL
4229 Trudy Street	FFE Below 2-year HGL	FFE Above 100-year HGL
4235 Trudy Street	FFE Below 2-year HGL	FFE Above 100-year HGL
4241 Trudy Street	FFE Below 5-year HGL	FFE Above 100-year HGL
4247 Trudy Street	FFE Below 5-year HGL	FFE Above 100-year HGL
4253 Trudy Street	FFE Below 2-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Constructability issues associated with the two 6-foot wide by 4-foot high RCBC's on Trudy Street will need to be evaluated and addressed;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access; and
- Permanent erosion control measures may be required at any or all outfalls.

3.3.3.4 D-17 – Kerwin Drive and Pippin Street

Existing Condition

Area D-17 extends from the system outfall west of Trudy Street east to Pippin Street and is shown on Exhibit D-17 (Volume II, Section 8). An existing closed drainage system conveys drainage from Pippin Street west to Kerwin Drive, then north to Dante Avenue. The system then runs west along Dante Avenue to the system outfall west of Trudy Street. The existing system is comprised of pipes ranging in diameter from 27-inches to 36-inches.

When the existing storm sewer system becomes surcharged overflow discharges to the surface. The surface discharges begin at Pippin Street where the overflow spills into grassed swales. The swales convey the discharges northwest, between residential structures, to the intersection of Kerwin Drive and Dante Avenue. At that point, the surface flow can reenter the closed system; however, if that system is also surcharged, overflow will continue on the surface of Dante Avenue toward Trudy Street. At Trudy Street, the overland (surcharged) flow conveys west between homes to either the system outfall or north along Trudy Street. Refer to Exhibit D-17. The overland discharges contribute to the roadway and FFE flooding in the project area.

The drainage area of the Pippin Street inlets is approximately 16 acres, and the drainage area at the storm sewer outfall is approximately 29 acres. Land use within the drainage area is primarily residential with areas of open space in the upper reaches of the watershed.

The InfoSWMM Model of existing condition indicates that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area range from less than the 2-year storm event to the 10-year storm event. Six houses within the project area may experience flooding of the finished floor elevation (FFE) during the 2-year through 25-year storm events. Additionally, the model shows that 12 properties within the project area experience nuisance flooding during the 2-year storm event, and 28 properties experience nuisance flooding during the 100-year storm event.

The site was selected for potential drainage improvements because of the low roadway LOS and FFE flooding. Frequent flooding of Kerwin Drive and Trudy Street has the potential to impede ingress/egress of vehicles in the event of an emergency. Coincidental flooding of Trudy Street and Kerwin Drive has the potential to isolate 24 residential properties from emergency services on Dante Avenue. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing the pipe sizes.

RECOMMENDED ALTERNATIVE

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer system, thereby reducing surcharge depths and surface flow.

The recommended improvements start at Pippin Street and continue along the storm sewer to the system outfall. Exhibit D-17 (Volume II, Section 8) shows the extent of the recommended improvements summarized below. Enlargement of the existing storm sewer reduces the surcharges throughout the project area, thereby reducing the surface discharges.

A summary of the recommended improvements is shown below and can be found on Exhibit D-17:

- Replace 163 linear feet of 36” RCP with two 4-foot wide by 4-foot high RCBC;
- Replace 441 linear feet of 30” RCP with 48” RCP;
- Replace 819 linear feet of 27” RCP with 48” RCP; and
- Replace inlets and DMH as required.

The greater capacity of the improvements shown above would improve the flooding issues on Trudy Street, Dante Avenue, Kerwin Drive, and Pippin Street. The existing LOS at the intersection of Trudy Street and Dante Avenue is less than the 2-year storm event. The LOS with the improved storm sewer is the 25-year storm event. It should be noted that the storm sewer at this intersection is subject to high tailwater from the downstream open channel. The LOS off all other roads within the project area are increased to the 100-year storm event with the proposed improvements.

As stated previously, six houses in the project area may experience FFE flooding during the 2-year through 25-year storm events. With the recommended improvements, the FFEs of all six houses analyzed would be above the improved 100-year flood elevations. The recommended improvements would also reduce the properties experiencing nuisance flooding. During the 2-year event, the model shows 12 properties experiencing spell flooding with the existing drainage system. Only 6 properties would experience such nuisance flooding during the 2-year event with the recommended improvements. During the 100-year storm event, nuisance flooding would be reduced from 28 properties to 12 properties.

The proposed improvements would increase the flooding depth immediately downstream of the system outfall by a maximum of 0.47-feet. The increase in depths is not significant enough to induce any additional roadway or FFE flooding.

The recommended improvements, as shown on Exhibit D-17, utilize the existing storm sewer alignment and easements. Using larger diameter pipes may present significant constructability issues in two areas. The upstream reach of the proposed storm sewer, between Pippin Street and Kerwin Drive, lies in between (and in proximity to) residential structures. During the design phase of this project, an alternate alignment for this section of storm sewer should be investigated. Likewise, the downstream segment of proposed storm sewer, between Trudy Street and the system outfall, passes between closely spaced residential structures. During the design phase of this project, the possibility of splitting the proposed system into multiple, small pipes passing between multiple houses should be explored.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 10. Project Area D-17 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Trudy Street	< 2-year Storm Event	25-year Storm Event
Dante Avenue	2-year Storm Event	100-year Storm Event
Kerwin Drive at Dante Avenue	2-year Storm Event	100-year Storm Event
Pippin Street	< 2-year Storm Event	100-year Storm Event
Saint Elmo Avenue	10-year Storm Event	100-year Storm Event

Table 11. Project Area D-17 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3919 Dante Avenue	FFE Below 5-year HGL	FFE Above 100-year HGL
4014 Kerwin Drive	FFE Below 50-year HGL	FFE Above 100-year HGL
4008 Kerwin Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
4002 Kerwin Drive	FFE Below 50-year HGL	FFE Above 100-year HGL
4001 Pippin Street	FFE Below 5-year HGL	FFE Above 100-year HGL
4007 Pippin Street	FFE Below 25-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Alternate alignments of the upstream and downstream segments of the proposed storm sewer should be evaluated to avoid constructability issues associated with closely spaced residential structures;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access; and
- Permanent erosion control measures may be required at the system outfall west of Trudy Street.

3.3.3.5 Area D-19 – Pippin Street

Existing Condition

Area D-19 extends from the system outfall west of Kerwin Drive east to Twinmont Street and is shown on Exhibit D-19 (Volume II, Section 8). An existing closed drainage system conveys drainage from Twinmont Street west to through several residential areas to the system outfall located west of Kerwin Drive. The existing system is comprised of pipes ranging in diameter from 24-inches to 36-inches. Most of the storm sewer is on an alignment which places the storm sewer between residential structures.

When the existing storm sewer system becomes surcharged overflow discharges to the surface. The surface overflow begins at Twinmont Street where discharges spill into a grassed swale between 3909 and 3917 Twinmont Street. The swale conveys the discharge west, between residential structures, to Bowmar Cove where the discharge can re-enter the closed system if it is not surcharged. From that point the surface flow paths generally follow the storm sewer alignment to Pippin Street. From there the surface flow path diverges from the storm sewer alignment, passing between 3918 and 3926 Kerwin Drive (Refer to Exhibit D-19). The overland discharges contribute to roadway and FFE flooding in the project area.

The drainage area of the Twinmont Street inlets is approximately 16 acres. The total drainage area at Pippin Street is approximately 35 acres, and the total drainage area at the system outfall is 41 acres. Land use within the drainage area is primarily residential.

The InfoSWMM Model of existing condition indicates that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area is less than the 2-year storm event. Seven houses within the project area may experience flooding of the finished floor elevation (FFE), with flooding frequencies ranging from the less than the 2-year storm event to the 25-year storm event. Additionally, the model shows that 41 properties experience nuisance flooding during the 2-year storm event, and 58 properties experience nuisance flooding during the 100-year storm event.

The site was selected for potential drainage improvements because of the low roadway LOS and FFE flooding. Frequent flooding of the five roads in the project area has the potential to impede ingress/egress of vehicles in the event of an emergency. Longsneck Cove and Bowmar Cove are two cul-de-sacs in the project area (see Exhibit D-19); flooding of these cul-de-sacs has the potential to isolate ten or more homes from emergency services. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing pipe sizes.

RECOMMENDED ALTERNATIVE

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer system, thereby reducing surcharge depths and surface flow.

The recommended improvements as shown on Exhibit D-19 start at Twinmont Street and continue west to the system outfall. Enlargement of the existing storm sewer reduces the surcharges throughout the project area, thereby reducing the surface discharges.

As stated previously, the existing storm sewer runs between residential structures for most of its length. Placing larger diameter pipes along the existing alignment will present constructability issues such as foundation and trench stabilization. Alternative alignments should be explored during the design process to mitigate the potential constructability issues. One such alternative alignment is shown on Exhibit D-19.

The existing outlet pipe of the closed system is a 36” RCP which runs between 3911 and 3917 Kerwin Drive. The analysis of recommended improvements indicates that the existing pipe should be replaced with two 4-foot wide by 4-foot high RCBC. Installation of such large structures between the two homes will not be possible. Therefore, one of the RCBC's has been routed to the south, to pass between 3905 and 3911 Kerwin Drive (refer to Exhibit D-19).

A summary of the recommended improvements is shown below and can be found on Exhibit D-19:

- Replace 136 linear feet of 36" RCP with 4-foot wide by 4-foot high RCBC and headwall;
- Install 230 linear feet of 4-foot wide by 4-foot high RCBC with DMH and headwall;
- Replace 341 linear feet of 36" RCP with two 4-foot wide by 4-foot high RCBC;
- Replace 68 linear feet of 24" RCP with 30" RCP;
- Replace 519 linear feet of 30" RCP with 42" RCP;
- Replace 356 linear feet of 27" RCP with 42" RCP;
- Replace 164 linear feet of 24" RCP with 36" RCP;
- Replace inlets and DMH as required; and
- Install permanent erosion control measures at two drainage outfalls as required.

The greater capacity of the improvements shown above would improve the flooding throughout the project area. The existing LOS of all streets within the project area is less than the 2-year storm event. The LOS of all streets is increased to the 100-year storm event with the recommended improvements.

As stated previously, seven houses in the project area may experience FFE flooding during the 2-year through 25-year storm events. With the recommended improvements, the FFE of all seven houses analyzed would be above the improved 100-year flood elevations. Nuisance flooding would also be significantly reduced with the recommended improvements. During the 2-year storm event the number of properties experiencing nuisance flooding would be reduced from 41 properties to 12 properties. During the 100-year storm event nuisance flooding would be reduced from 58 properties to 19 properties.

The proposed improvements would increase the flooding depth immediately downstream of the system outfall by a maximum of 0.05-feet. The increase in depths is not significant enough to induce any additional roadway or FFE flooding.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 12. Project Area D-19 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Kerwin Drive	< 2-year Storm Event	100-year Storm Event
Pippin Street	< 2-year Storm Event	100-year Storm Event
Longsneck Cove	< 2-year Storm Event	100-year Storm Event
Bowmar Cove	< 2-year Storm Event	100-year Storm Event
Twinmont Street	< 2-year Storm Event	100-year Storm Event

Table 13. Project Area D-19 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3911 Kerwin Drive	FFE Below 2-year HGL	FFE Above 100-year HGL
3917 Kerwin Drive	FFE Below 2-year HGL	FFE Above 100-year HGL
3926 Kerwin Drive	FFE Below 10-year HGL	FFE Above 100-year HGL
3918 Kerwin Drive	FFE Below 10-year HGL	FFE Above 100-year HGL
3937 Pippin Street	FFE Below 2-year HGL	FFE Above 100-year HGL
3929 Pippin Street	FFE Below 25-year HGL	FFE Above 100-year HGL
3914 Pippin Street	FFE Below 50-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- New drainage easement will be required for the re-routed storm sewer at 3911 Kerwin Drive;
- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Alternate alignments of the upstream and downstream segments of the proposed storm sewer should be evaluated to avoid constructability issues associated with closely spaced residential structures;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access; and
- Permanent erosion control measures may be required at the two storm sewer outfalls west of Kerwin Drive.

3.3.3.6 Area D-21 – Battlefield Drive, Kerwin Drive and Twinmont Street

Existing Condition

Area D-21 extends from the system outfall north of Lucky Trail Drive eastward along Battlefield Drive, Twinmont Street, and Longhollow Drive towards Longhollow Cove, and from Lucky Trail Drive south towards Timberwood Drive. The project area can be seen on Exhibits D-21.1 thru D-21.3 (Volume II, Section 8). The existing Lucky Trail Drive cross drain is included in the project area because it influences the tailwater of the Battlefield Drive closed drainage system. The Battlefield Drive storm sewer system begins at the system outfall, just west of the intersection of Battlefield Drive and Kerwin

Drive and runs southeast to the intersection of Battlefield Drive and Twinmont Street. The system extends north along Twinmont Street, then follows Longhollow Drive east to Longhollow Cove. The southern portion of the drainage system begins at the existing open channel west of and near 3719 Kerwin Drive and extends east to Barberry Street south of Twin Lakes Drive. The Lucky Trail cross drain is an 8-foot wide by 6-foot high RCBC. The Battlefield Drive storm sewer system is comprised of pipes ranging in diameter from 24-inches to 42-inches. The storm sewer is located primarily under paved roadways, except for the beginning of the segment near Longhollow Cove, the last segment leading to the outfall, and most of the southern portion of the system. The last segment of the pipe, a 42-inch RCP, passes between 3777 Kerwin Drive and 3785 Kerwin Drive. The Kerwin Drive/Barberry Drive storm sewer is comprised of pipes ranging in diameter from 24-inches to 48-inches on an alignment which causes the pipes to pass between residential structures.

When the existing storm sewer system becomes surcharged overflow discharges to the surface. The surcharge from the Battlefield Drive system begins at Longhollow Cove where the overflow discharges spill onto the street. The surface flow continues along the paved roads to the intersection of Battlefield Drive and Kerwin Drive. The surface discharge then ponds until it is sufficiently deep to spill over the curb and flow west, between homes, to the open channel west of Kerwin Drive.

The surface flow of the southern drainage system begins at the Barberry Street inlets. When the surcharged water is deep enough, the discharge spills over into a grassed swale and flows to the west between the homes at 3709 and 3715 Barberry Street. From that point the surface flow follows a grassed swale north and then east to Kerwin Drive, then along Kerwin Drive north to Battlefield Drive. An 18-inch RCP is located between 3741 and 3747 Kerwin Drive; however, the inlet of that pipe is significantly lower that the hydraulic grade line of the downstream open channel. The existing condition analysis of this system indicates that the 18-inch pipe receives negative flow (i.e., backflow) from the open channel onto Kerwin Drive during large rainfall events. The overland discharges, in addition to this backflow from the open channel, contribute to the roadway and FFE flooding in the project area.

The total drainage area at Lucky Trail Drive is approximately 169 acres. The following table summarizes the drainage area at other key points within the project area. Land use within the drainage area is primarily residential, with open and wooded space predominate south of Timberwood Drive.

Table 14. Project Area D-21 - Summary of Drainage Areas

System	Location	Approximate Drainage Area (Acres)
Lucky Trail Drive	Lucky Trail Drive	169
Battlefield Drive	Longhollow Cove	9
	Longhollow Drive & Twinmont Street	16
	Twinmont Street & Battlefield Drive	39
	Battlefield Drive & Barberry Street	52
	Battlefield Drive & Kerwin Drive	59
Kerwin Drive/Barberry Street	Barberry Street	9
	Kerwin Drive	66

The InfoSWMM Model of existing condition indicates that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area range from the less than 2-year storm event to the 25-year storm event. Frequent and coincident flooding of Kerwin Drive, Battlefield Drive, Twinmont Street, Longhollow Drive and Barberry Street has the potential to seriously impede the ingress/egress of emergency vehicles. Eight houses within the project area may experience flooding of the finished floor elevation (FFE), with flooding frequencies ranging from the 2-year storm event to the 10-year storm event. Additionally, the model shows that 113 properties in the project area experience nuisance flooding during the 2-year storm event, and 147 properties experience nuisance flooding during the 100-year storm event.

The site was selected for potential drainage improvements because of the low roadway LOS and FFE flooding. Frequent and coincident flooding of Kerwin Drive, Battlefield Drive, Twinmont Street, Longhollow Drive and Barberry Street has the potential to seriously impede the ingress/egress of emergency vehicles.

Two alternatives were considered for this project area. The first alternative, which is the recommended alternative, replaces existing drainage pipes with larger size pipes. For the second alternative two areas of potential expanded stormwater detention were identified and evaluated.

RECOMMENDED ALTERNATIVE

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer system, thereby reducing surcharge depths and surface flow. The recommended improvements as shown on Exhibits D-21.1 thru D-21.3 (Volume II, Section 8) extend throughout the project area.

As stated previously, the existing Lucky Trail Drive cross drain influences the tailwater of the Battlefield Drive closed drainage system. Unless the cross drain is improved, potential mitigation of the flooding at the intersection of Battlefield Drive and Kerwin Drive is limited. The recommended improvement at Lucky Trail Drive is to replace the existing 8-foot high by 6-foot wide RCBC cross drain with a 12-foot wide by 6-foot high RCBC.

The Battlefield Drive storm sewer system capacity is increased using a variety of pipe shapes and sizes. Round pipes are used on the upper reaches of the storm sewer, while elliptical pipes are used under a large portion of Battlefield Drive to insure adequate cover over the pipes.

It is necessary to use a RCBC on the lower reach of the storm sewer system to mitigate the flooding issues at the intersection of Battlefield Drive and Kerwin Drive. As stated previously, the last segment of the pipe passes between 3777 Kerwin Drive and 3785 Kerwin Drive. The InfoSWMM analysis shows that it is necessary to replace the existing 42-inch RCP with three 6-foot wide by 4-foot high RCBC. Construction of three adjacent RCBC's between the houses would not be practical. Therefore, two of the new RCBC's have been routed to the north and south as shown on Exhibit D-21.1. The feasibility of constructing such a large structure between houses will need to be investigated during the design process. It may be desirable to install several smaller structures on alternate alignments to mitigate the flooding issues.

It will be necessary to plug and abandon the exiting 18-inch pipe at 3741/3747 Kerwin Drive. As described above, this pipe receives negative flow from the open channel west of Kerwin Drive. This will require installation of a new storm sewer pipe, with appropriately sized inlets, that extends from the existing pipe north to the intersection of Battlefield Drive and Kerwin Drive. Refer to Exhibit D-21.1

The Kerwin Drive/Barberry Street storm sewer requires the existing pipes be replaced by a 7-foot wide by 4-foot high RCBC (with DMH and headwall) to mitigate the flooding issues at Barberry Street. The feasibility of constructing such a large structure between houses will need to be investigated during the design process. It may be desirable to install several smaller structures on alternate alignments to mitigate the flooding issues.

A summary of the recommended improvements is shown below. The recommended improvements for Lucky Trail Drive, Battlefield Drive, Twinmont Street, and Longhollow Drive are shown on Exhibit D-21.2 and D-21.2 (Volume II, Section 8). The recommended improvements for the Kerwin Drive/Barberry Street storm sewer are shown on Exhibit D-21.3 (Volume II, Section 8).

Lucky Trail Drive (Exhibit D-21.1):

- Replace 69 linear feet of 8-foot wide by 6-foot high RCBC with 12-foot wide by 6-foot high RCBC (Lucky Trail Drive) with headwalls; and
- Install permanent erosion control measures at the RCBC outlet as required.

System Outfall West of Kerwin Drive to Longhollow Drive (Exhibits D-21.1 and D-21.2):

- Install 180 linear feet of 6-foot wide by 4-foot high RCBC with DMH and headwall;
- Install 170 linear feet of 6-foot wide by 4-foot high RCBC with DMH and headwall;
- Replace 128 linear feet of 42" RCP with 6-foot wide by 4-foot high RCBC with headwall;

- Replace 146 linear feet of 42-inch RCP with 12-foot-wide x 4-foot high RCBC;
- Replace 785 linear feet of 42-inch RCP with 76” wide by 48” high elliptical RCP (60” round equivalent);
- Replace 173 linear feet of 36” RCP with 76” wide by 48” high elliptical RCP (60” round equivalent);
- Replace 324 linear feet of 27” RCP with 36” RCP;
- Replace 427 linear feet of 27” RCP with 30” RCP;
- Replace 443 linear feet of 24” RCP with 30” RCP;
- Install 276 linear feet (2) 60” wide by 38” high elliptical RCP (48-inch round equivalent) with appropriately sized inlets (along Kerwin Drive);
- Replace inlets and DMH’s as required; and
- Install permanent erosion control measures at three storm sewer system outfalls west of Kerwin Drive as required.

Kerwin Drive to Barberry Street (Exhibit D-21.3):

- Replace 222 linear feet of 48” RCP with 243 linear feet of 7-foot wide by 4-high RCBC with DMH and headwall;
- Replace 180 linear feet of 24” RCP with 7-foot wide by 4-foot high RCBC;
- Replace inlets and DMH as required; and
- Install permanent erosion control measures storm sewer system outfall as required.

The greater capacity of the improvements shown above would improve the flooding throughout the project area. The existing LOS of the Lucky Trail cross drain is the 5-year storm event. The LOS would be increased to the 10-year storm event with the recommended improvements. The existing LOS of less than the 2-year storm event at the intersection of Battlefield Drive and Kerwin Drive would be improved to the 10-year storm event. At 3741/3747 Kerwin Drive (the location of the 18-inch RCP to be plugged and abandoned), the existing LOS of the street is less than the 2-year storm event and would be increased to the 100-year storm event with the improvements. The recommended improvement would increase the LOS of all other streets within the project area to the 100-year storm event.

As stated previously, eight houses in the project area may experience FFE flooding during large rainfall events with the existing drainage system. With the recommended improvements, four of the eight houses would have the FFE above the 100-year hydraulic grade line, and two of the houses would have the FFE above the 50-year hydraulic grade line. The house at 3747 Kerwin Drive would have the FFE flooding improved from the 2-syear storm event to the 5-year storm event. The house at 3741 Kerwin Drive would have the FFE flooding improved from the 2-year storm event to the 10-year storm event. Additionally, nuisance flooding would be significantly improved throughout the project area. Of the 113 properties experiencing nuisance flooding during the 2-year storm event with the existing drainage systems, only 57 would experience nuisance flooding during the same event with the recommended improvements. During the 100-year storm event 147 properties experience nuisance flooding with the exiting drainage systems. With the recommended improvements, 102 properties would experience nuisance flooding with the recommended improvements.

The proposed improvements would increase the flooding depths immediately downstream of Lucky Trail Drive by a maximum of 0.37-feet. There are no structures within the floodplain in that area.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 15. Project Area D-21 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Lucky Trail Drive	5-year Storm Event	10-year Storm Event
Kerwin Dr @ Battlefield Dr	< 2-year Storm Event	10-year Storm Event
Battlefield Dr @ Barberry St	2-year Storm Event	100-year Storm Event
Battlefield Dr @ Dagmar St	< 2-year Storm Event	100-year Storm Event
Battlefield Dr @ Twinmont St	< 2-year Storm Event	100-year Storm Event
Twinmont St	25-year Storm Event	100-year Storm Event
Twinmont St @ Longhollow Dr	10-year Storm Event	100-year Storm Event
Longhollow Dr @ Longhollow Cv	5-year Storm Event	100-year Storm Event
3741/3747 Kerwin Dr	< 2-year Storm Event	100-year Storm Event
3719 Kerwin Dr	5-year Storm Event	100-year Storm Event
Barberry St	< 2-year Storm Event	100-year Storm Event

Table 16. Project Area D-21 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3785 Kerwin Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
3777 Kerwin Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
3769 Kerwin Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
3763 Kerwin Drive	FFE Below 25-year HGL	FFE Below 100-year HGL
3755 Kerwin Drive	FFE Below 25-year HGL	FFE Below 100-year HGL
3747 Kerwin Drive	FFE Below 5-year HGL	FFE Below 10-year HGL
3741 Kerwin Drive	FFE Below 5-year HGL	FFE Below 25-year HGL
3733 Kerwin Drive	FFE Below 10-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- New drainage easements will be required for the re-routed storm sewer from Kerwin Drive to the storm sewer outfalls;
- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Alternate alignments of the proposed storm sewer should be evaluated to avoid constructability issues associated with closely spaced residential structures;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access; and
- Permanent erosion control measures may be required at the three storm sewer outfalls west of Kerwin Drive and at the outlet of the lucky Trail Drive RCBC.

Alternative 2 – Infrastructure Improvements and Expanded Stormwater Detention

Two areas were identified as having the potential for expanded stormwater detention.

The first area is located west of Kerwin Drive near the outfall of the existing 18” RCP at 3741/3747 Kerwin Drive (refer to Figure 4 below). The area’s current land use is open space and wooded. It would be possible to grade the area to provide an additional 1.5 acre-feet of stormwater storage. The discharges from the Kerwin Drive/Barberry storm sewer system as well as runoff from the areas to the west would be diverted into the new detention area. The expanded detention has the potential of reducing the discharges at the Battlefield Drive storm sewer outfall and the Lucky Trail Drive cross drain. Construction of this area would require property or easement acquisition from private individuals.

The parcel located immediately west of the Kerwin Drive/Barberry outfall is currently owned by Shelby County. However, the maximum elevation of the parcel is more than 20-feet above the open channel. The amount of grading necessary to achieve additional storage volume makes the use of this site impractical.

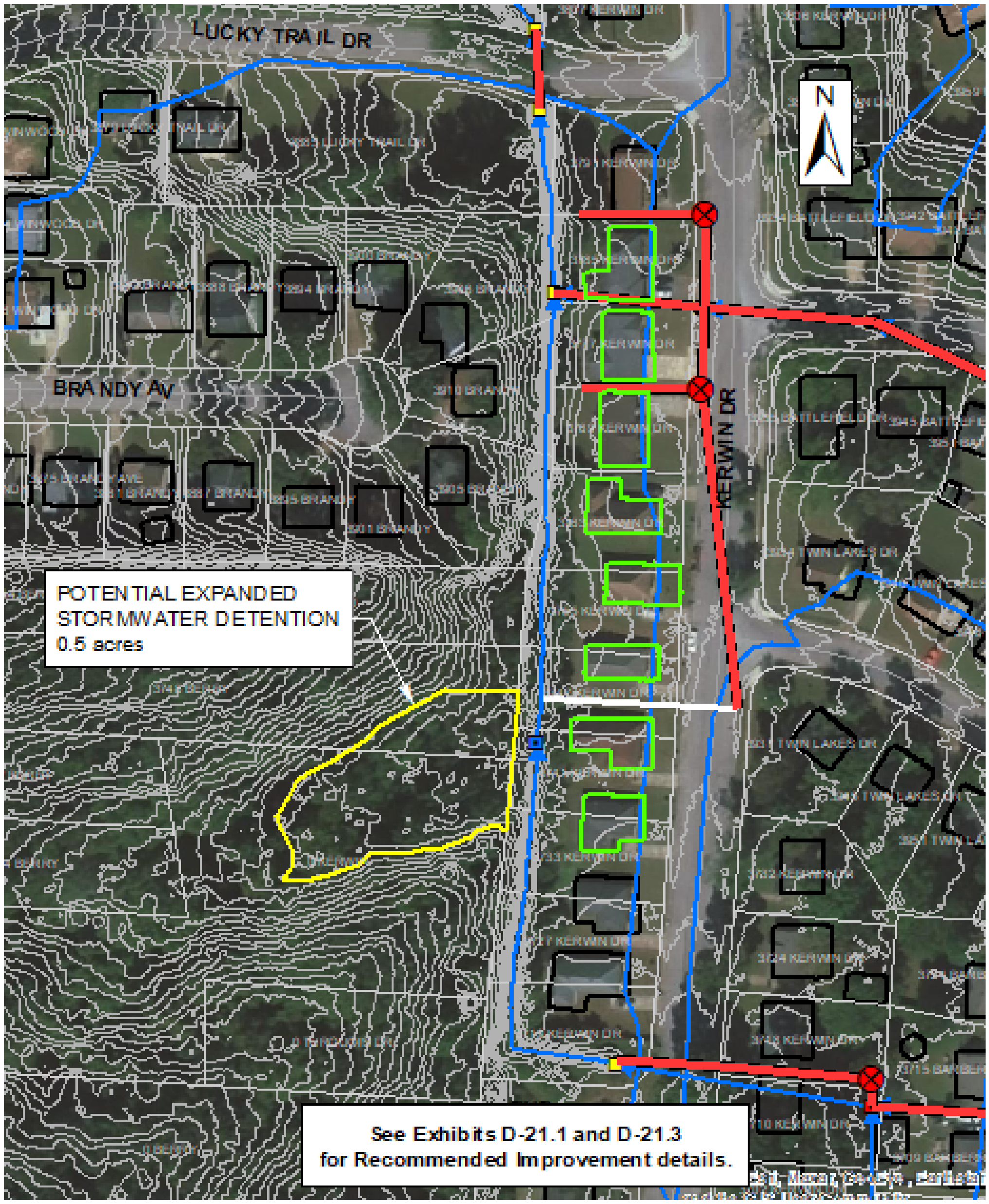

Figure 4. Potential Expanded Detention Adjacent to Battlefield Drive Outfall

The second area of potential detention is located south of Timberwood Drive (refer to Figure 5 below). The existing land use in this area is open space and wooded. It would be possible to grade the area to provide an additional 3.5 acre-feet of stormwater storage, which has the potential of reducing the discharges at the Barberry Street inlets. Construction of this area would require property or easement acquisition from four private individuals.

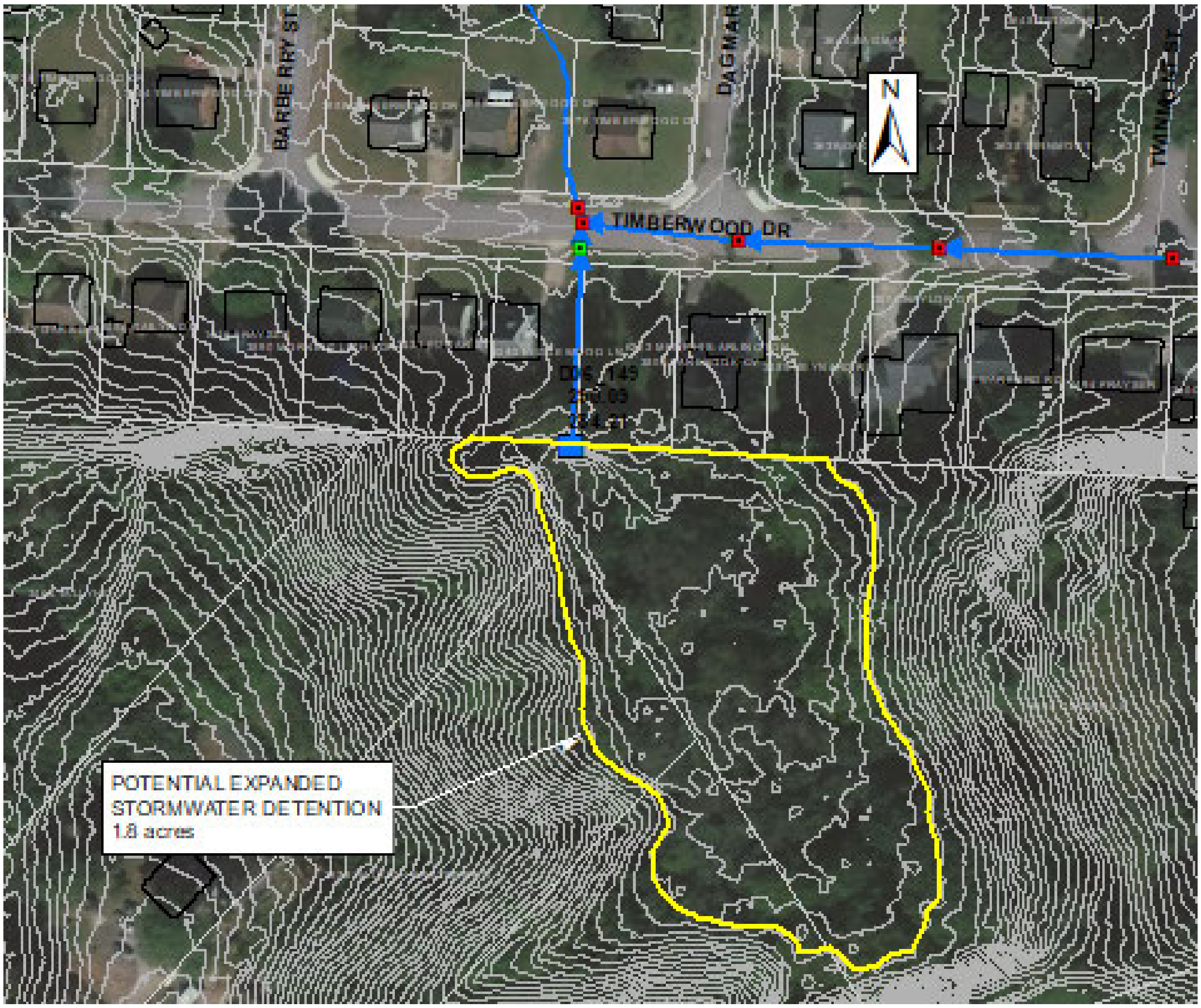

Figure 5. Potential Expanded Detention Upstream of Timberwood Drive

The appropriate data was input into the InfoSWMM model to evaluate the effectiveness of the two detention areas in mitigating the flooding issues within the project area. The results show that the additional stormwater storage is insufficient to reduce flooding in the project area without the same storm sewer improvements shown in Alternative 1 (Recommended). With little or no additional benefit compared to Alternative 1, and with additional cost, this alternative was judged to be non-feasible.

3.3.3.7 Area D-23 – Dorado Avenue

<u>Existing Condition</u>

Area D-23 extends from the system outfall near Russelwood Drive east to Longmont Drive and is shown on Exhibits D-23.1 and D-23.2 (Volume II, Section 8). An existing closed drainage system conveys drainage from Longmont Drive west to Windermere Drive, then south to Dorado Avenue. The system continues west along Dorado Avenue to Wolf Trail Drive, then north to an existing DMH at 3707 Wolf Trail Drive. The final segment of the storm sewer runs northwest from the DMH to the system outfall. The existing system is comprised of pipes ranging in diameter from 27-inches to 42-inches.

Most of the storm sewer is located under paved roadways except for the last two segments. University of Memphis CAESER GIS data show the alignment of the last two segments of existing storm sewer starting at the DMH in front of 3707 then continuing northwest between the homes at 3707 and 3711 Wolf Trail Drive to a DMH located near the northwestern parcel line of the two homes. The field survey shows the storm sewer alignment passing under the home at 3711 Wolf Trail Drive to a surveyed DMH near the northeastern corner of the 3711 Wolf Train Drive parcel. The last two segments of the existing storm sewer have been modeled as per the best available data, which is the field survey.

When the existing storm sewer system becomes surcharged, overflow discharges to the surface. The surface discharges begin at Longmont Drive where the overflow spills onto the roadway. The surface flow continues along the roadways for the entire length of the system until reaching the DMH at 3707 Wolf Trail Drive. The surface flow then ponds up to a depth sufficient to spill over into the grassed areas between homes before flowing northwest to the system outfall. The surface discharges on the roadways contribute to the roadway and FFE flooding in the project area.

The drainage area of the at Longmont Drive is approximately 10 acres and the total drainage area at the system outfall is 65 acres. Land use within the drainage area is primarily residential. The following table summarizes the drainage areas at key points within the project area (refer to Exhibits D-23.1 and D-23.2).

Table 17. Project Area D-23 - Summary of Drainage Areas

Location	Approximate Drainage Area (acres)
Longmont Drive	10
Dorado Avenue & Windermere Drive	33
Dorado Avenue & Wolf Trail Drive	44
3707 Wolf Trail Drive	53
System Outfall	65

The InfoSWMM Model of existing condition shows that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area range from less than the 2-year storm event to the 5-year storm event. Six houses within the project area may experience flooding of the finished floor elevation (FFE), with flooding frequencies ranging from the 2-year storm event to the 50-year storm event. 60 properties within the project area experience nuisance flooding during the 2-year storm event, and 97 properties experience nuisance flooding during the 100-year storm event.

The site was selected for potential drainage improvements because of the low roadway LOS and FFE flooding. Frequent flooding of the streets in the project area has the potential to impede ingress/egress of vehicles in the event of an emergency. Coincidental flooding of roads and cul-de-sacs in the project area has the potential to isolate over 30 homes from emergency services. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing pipe sizes.

RECOMMENDED ALTERNATIVE

Alternative 1 – Infrastructure Improvements

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer system, thereby reducing surcharge depths and surface flow. The recommended improvements as shown on Exhibits D-23.1 and D-23.2 (Volume II, Section 8) start at Longmont Drive and continue west to the system outfall.

As stated previously, the existing storm sewer runs under paved roads for most of its length. The last segment of existing storm sewer, which is a 42-inch RCP, passes between 3707 and 3711 Wolf Trail Drive. Analysis of the storm sewer shows that the existing pipe needs to be replaced with two 4-foot wide by 4-foot High RCBC's to mitigate flooding issues on Wolf Trail Drive. Installation of such large adjacent boxes between the two homes is not feasible. Therefore, the RCBC's have been routed to the north and south, to pass between 3707/3711 and 3711/3715 Wolf Trail Drive (refer to Exhibit D-23.1).

It should be noted that the field survey performed for this project area disagrees with the University of Memphis CAESER GIS data records regarding the alignment of the last segment of existing storm sewer. For the purpose of this study, it is assumed that the field survey is the most-accurate source of data and that the existing 42" RCP under 3711 Wolf Trail Drive will need to be plugged and abandoned in place. The actual alignment of this existing pipe and the decision to plug/abandon it will need to be evaluated further during the design phase.

A summary of the recommended improvements is shown below and can be found on Exhibits D-23.1 and D-23.2:

- Replace 279 linear feet of 42" RCP with 4-foot wide by 4-foot high RCBC with headwall;
- Install 272 linear feet of 4-foot wide by 4-foot high RCBC with DMH;
- Install 546 linear feet 4-foot wide by 4-foot high RCBC with DMH and headwall;
- Plug 42" RCP and abandon in place;

- Replace 262 linear feet of 42” RCP with 6-foot wide by 4-foot high RCBC (Wolf Trail Drive);
- Replace 579 linear feet of 42” RCP with 6-foot wide by 4-foot high RCBC (Dorado Avenue);
- Replace 301 linear feet of 36” RCP with 6-foot wide by 4-foot high RCBC (Dorado Avenue);
- Replace 314 linear feet of 36“ RCP with 48” RCP (Windermere Drive);
- Replace 83 linear feet of 30” RCP with 36” RCP (Windermere Drive);
- Replace 327 linear feet of 27” RCP with 36” RCP (Windermere Drive and Longmont Drive);
- Replace inlets and DMH’s as required; and
- Install permanent erosion control measures at storm sewer system outfall northeast of Wolf Trail Drive as required.

The greater capacity of the proposed improvements shown above would increase the roadway LOS to the 100-year storm event throughout the project area. As stated previously, houses in the project area may experience FFE flooding during large rainfall events with the existing drainage system. With the recommended improvements, the FFE of all six houses analyzed would be above the improved 100-year flood elevations. Nuisance flooding will be significantly improved with the recommended improvements described above. During the 2-year storm event, 23 properties may experience nuisance flooding with the recommended improvements, while 6 properties see such flooding with the existing drainage system. During the 100-year storm event only 43 properties would experience nuisance flooding rather than 97. The proposed improvements would not increase the flooding depth downstream of the system outfall.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 18. Project Area D-23 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Wolf Trail Drive	< 2-year Storm Event	100-year Storm Event
Dorado Avenue	< 2-year Storm Event	100-year Storm Event
Windermere Drive	< 2-year Storm Event	100-year Storm Event
Longmont Drive	5-year Storm Event	100-year Storm Event

Table 19. Project area D-23 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3698 Wolf Trail Drive	FFE Below 100-year HGL	FFE Above 100-year HGL
3706 Wolf Trail Drive	FFE Below 50-year HGL	FFE Above 100-year HGL
3711 Wolf Trail Drive	FFE Below 100-year HGL	FFE Above 100-year HGL
3634 Dorado Avenue	FFE Below 5-year HGL	FFE Above 100-year HGL
3719 Longmont Drive	FFE Below 10-year HGL	FFE Above 100-year HGL
3725 Longmont Drive	FFE Below 10-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- New drainage easements will be required for the re-routed storm sewer from Wolf Trail Drive to the system outfall;
- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improvements to maintain residential access;
- During the design of the recommended improvements, the alignment of the last two segments of existing storm sewer will need to be determined;
- Alternate alignments of the downstream segments of the proposed storm sewer should be evaluated to avoid constructability issues associated with closely spaced residential structures; and
- Permanent erosion control measures may be required at the outfall of the proposed storm sewer system.

3.3.3.8 Area D-29 – Frayser Raleigh Road

Existing Condition

Area D-29 extends from the system outfall on the north side of Frayser Raleigh road northeast to South Ridge Avenue and is shown on Exhibit D-29 (Volume II, Section 8). An existing closed drainage system conveys drainage from South Ridge Avenue southwest, through residential properties, to Edgefield Drive then south to the intersection of Edgefield Drive and Teresa Cove. The storm sewer then runs west, through a residential development, to Windermere Drive and south to Frayser Raleigh Road. The final segments of the storm sewer system run west along Frayser Raleigh Road to the system outfall. The existing system is comprised of pipes ranging in diameter from 18-inches to 42-inches.

When the existing storm sewer system becomes surcharged, overflow discharges to the surface. The surface discharges begin at South Ridge Avenue where the overflow spills over the curb and into the grassed areas between houses flowing to the southwest onto Edgefield Drive. The surface discharge then flows south along Edgefield Drive to approximately 3593 Edgefield Drive. At that point the flow again spills over into the grassed areas between homes and flows southwest to storm sewer inlets on Windermere Street. Should those inlets also be surcharged, the excess flow travels west on Frayser Raleigh Road to the system outfall. The surface discharge on the roadways contribute to the roadway and FFE flooding in the project area.

The drainage area of the at South Ridge Avenue is approximately 18 acres and the total drainage area at the system outfall is 35 acres. Land use within the drainage area is primarily residential with open areas in the upper portions of the drainage area. The following table summarizes the drainage areas at key points within the project area (refer to Exhibit D-29).

Table 20. Project Area D-29 - Summary of Drainage Areas

Location	Approximate Drainage Area (acres)
South Ridge Avenue	18
Edgefield Drive & Teresa Cove	27
Frayser Raleigh Dr & Windermere Drive	34
System Outfall	35

The InfoSWMM Model of existing condition indicates that the system has insufficient capacity to adequately drain such a large area. The existing level of service (LOS) of the streets within the project area range from less than the 2-year storm event to the 2-year storm event. Five houses within the project area may experience flooding of the finished floor elevations (FFEs), with flooding frequencies ranging from the 2-year storm event to the 25-year storm event. Additionally, 35 properties in the project experience nuisance flooding during the 2-year storm event, and 43 properties experience nuisance flooding during the 100-year storm event.

The site was selected for potential drainage improvements because of the low roadway LOS and FFE flooding. Frequent flooding of the roads in the project area has the potential to impede ingress/egress of vehicles in the event of an emergency. Several homes on Teresa Cove have the potential to be isolated from emergency services during large rainfall events. It is noted that there is no opportunity to add stormwater storage or detention in the upland portions of the drainage area. Therefore, the only alternative considered was to improve the drainage infrastructure by increasing the pipe sizes.

<u>RECOMMENDED ALTERNATIVE</u>

<u>Alternative 1 – Infrastructure Improvements</u>

Several combinations of pipe types and diameters were evaluated to provide additional capacity to the system. To mitigate the flooding issues within the project area, it is necessary to enlarge the diameter of the storm sewer system, thereby reducing surcharge depths and surface flow. It may prove difficult to construct the larger diameter along the existing alignments that pass between houses. Therefore, the recommended improvements include new drainage pipes that use different alignments than the existing system (refer to Exhibit D-29). Where the new pipe alignment differs from the exiting alignment the existing pipes are to be left in service, thereby resulting in smaller diameter pipes and a cost saving for the new system. It should be noted that the new storm sewer from Teresa Cove south will tie into an existing 18” diameter storm sewer system under Frayser Raleigh Road; from there, a new DMH will be installed and the existing culvert conveying westward under Frayser Raleigh Road will be upsized.

A summary of the recommended improvements is shown below and can be found on Exhibit D-29 (Volume II, Section 8):

- Replace 481 linear feet of 42” RCP with 6-foot wide by 5-foot high RCBC (Frayser Raleigh Road), with headwall at system outfall;
- Replace 83 linear feet of 36” RCP with 42” RCP (Windermere Drive);
- Replace 62 linear feet of 24” RCP with 42” RCP (Frayser Raleigh Road);
- Replace 246 linear feet of 18” RCP with 42” RCP with DMH (Frayser Raleigh Road);
- Install 227 linear feet of new 42” RCP (Edgefield Drive);
- Install junction box at Teresa Cove leaving existing 36-inch RCP in-service;
- Replace 222 linear feet of 30” RCP with 48” RCP (Edgefield Drive);
- Install 261 linear feet of new 36” RCP with DMH (South Ridge Avenue);
- Replace inlets and DMH as required; and
- Install permanent erosion control measures at storm sewer system outfall as required.

The greater capacity of the improvements shown above would improve the flooding throughout the project area. The LOS of the roadways located within the existing floodplain would be increased to the 100-year storm event with the recommended improvements. As stated previously, five houses in the project area may experience FFE flooding with the existing drainage system. With the recommended improvements, the FFE of all five houses analyzed would be above the improved 100-year flood elevations. Nuisance flooding would be slightly improved by the recommended improvements. Of the 35 properties that experience nuisance flooding during the 2-year storm event with the existing drainage system, 23 would have nuisance flooding with the proposed improvements. During the 100-year storm event, the number of properties experiencing nuisance flooding would decrease from 43 to 31. The proposed improvements would increase the flooding depth immediately downstream of the system outfall by a maximum of 0.1 feet. This increase is not enough to create additional flooding risk to structures downstream.

A summary of the flooding improvements to be realized by the above recommendations is shown below.

Table 21. Project Area D-29 - Change in Roadway Level of Service (LOS)

Location	Existing Condition LOS	Proposed Condition LOS
Windermere Drive	2-year Storm Event	100-year Storm Event
Edgefield Drive	2-year Storm Event	100-year Storm Event
Teresa Cove	2-year Storm Event	100-year Storm Event
South Ridge Avenue	< 2-year Storm Event	100-year Storm Event

Table 22. Project Area D-29 - Change in Finished Floor Elevation (FFE) Flooding

Location	Existing Condition FFE Flooding	Proposed Condition FFE Flooding
3587 Edgefield Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
3593 Edgefield Drive	FFE Below 10-year HGL	FFE Above 100-year HGL
3605 Edgefield Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
3612 Edgefield Drive	FFE Below 5-year HGL	FFE Above 100-year HGL
3707 Southridge Avenue	FFE Below 50-year HGL	FFE Above 100-year HGL

Inlet Analysis

University of Memphis CAESER GIS data show numerous inlets of various types associated with the existing storm sewer system within the project area. A detailed inlet analysis should be performed as a part of the design of the above improvements to ensure that the roadway drainage system has enough capacity to drain the roadway with no spread width issues or street flooding.

Design Considerations

Design of the infrastructure improvements shown above is beyond the scope of this study. However, several items have been noted that will need to be addressed in the design of the improvements:

- The recommended improvements require the re-routed storm sewer to tie into an existing 18" storm sewer trunk line under Frayser Raleigh Road. Further modeling to evaluate the impacts on the existing trunk line to the east will be required;
- A complete survey of utilities in the area will need to be conducted. No utility survey was performed as a part of this study. It is probable that underground utilities and possibly sanitary sewers will be a factor in the final storm sewer design;
- Construction phasing of the recommended improvements will need to be evaluated during the design of the recommended improve to maintain residential access; and
- Permanent erosion control measures may be required at the outfall of the proposed storm sewer system.

3.4 Existing and Improved Conditions Results

Comparison tables included in Section 4 (Frequency Flood Events) of Volume II of this report show InfoSWMM model results of the Existing Condition frequency events and recommended improvements described in the preceding sections. Results are reported for the 2-year through 100-year frequency storm events at all points of interest, including all surveyed structure floor elevations (homes and commercial buildings) and road crossings. Red highlighting indicates that structure floors are inundated for a given storm event, and blue highlighting indicates that structure floors or other critical elevations are close to being flooded (within one-half foot). The Roadway Level of Service (LOS) Tables in Section 4 show the Level of Service ratings for all major road crossings in the watershed for Existing and Improved Conditions. Also included within Section 4 are tables which depict the downstream impacts of the recommended improvements.

Floodplain inundation maps for Existing Condition (with Improved Condition overlay) are provided within Section 6 (Floodplain Map Exhibits) of Volume II of this report. These exhibits include the proposed floodplains resulting from implementation of the recommended improvements as described in the sections above.

3.5 Cost Estimates

Planning-level construction cost estimates were developed for all proposed recommended projects within the Allen Basin Study Area. A line-item breakdown of all costs (which includes surveying, engineering, and construction administration costs along with contingencies, but does not include property acquisition, utility relocation, or easement costs) for each project area is provided below. Additionally, a summary table is provided below which includes the total estimated construction cost per proposed recommended project and the total overall study area costs.

It is recommended to refer to Section 5 (study area overview map) and Section 8 (conceptual layouts) within Volume II along with the cost estimate for each proposed project area below.

ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT A-1
BEACON HILLS DRIVE AND SUNNY HILLS DRIVE

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE		AMOUNT	
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	1900	$	2.50	$	4,750.00
CLEARING AND GRUBBING	L.S.	1	$	5,000.00	$	5,000.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	2169	$	50.00	$	108,425.54
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	968	$	40.00	$	38,738.28
SIDEWALK REPLACEMENT	S.Y.	9	$	45.00	$	405.00
FENCE REMOVAL AND REPLACEMENT	L.F.	150	$	60.00	$	9,000.00
REPLACE EXISTING 6-72 INLET	EACH	4	$	5,000.00	$	20,000.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	1	$	5,000.00	$	5,000.00
CONSTRUCT PROPOSED DRAINAGE MANHOLE	EACH	3	$	5,000.00	$	15,000.00
REPLACE EXISTING 4x4 INLET	EACH	1	$	10,000.00	$	10,000.00
PLUG EXISTING 30" RCP	EACH	1	$	3,500.00	$	3,500.00
REMOVAL OF PIPE (18" RCP)	L.F.	33	$	20.00	$	660.00
REMOVAL OF PIPE (30" RCP)	L.F.	32	$	30.00	$	960.00
REMOVAL OF PIPE (36" RCP)	L.F.	228	$	35.00	$	7,980.00
24" RCP	L.F.	16	$	117.00	$	1,872.00
36" RCP	L.F.	49	$	125.00	$	6,125.00
42" RCP	L.F.	228	$	130.00	$	29,640.00
48" RCP	L.F.	688	$	135.00	$	92,880.00
INSTALL HEADWALL	EACH	1	$	5,000.00	$	5,000.00
TRENCH BACKFILL	C.Y.	4411	$	10.00	$	44,106.20
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	459	$	30.00	$	13,767.59
EXCAVATE DRAINAGE CHANNEL	C.Y.	933	$	30.00	$	27,988.37
MACHINED RIP-RAP (CLASS B)	TON	54	$	40.00	$	2,177.78
GEOTEXTILE FABRIC (TYPE IV)	S.Y.	37	$	3.00	$	112.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$	67,963	$	67,963.16
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$	46,000.00	$	46,000.00
TRAFFIC CONTROL	L.S.	1	$	10,000	$	10,000.00
			Sub-total		$	577,050.92
			20% Contingency		$	116,000.00
			Surveying, Engineering, & Constr. Admin. (15%)		$	87,000.00
			TOTAL		$	780,050.92

***PLEASE NOTE - the cost estimate does not include property acquisition, utility relocation, or easement costs.**

December 2021

ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-9
GRUBER DRIVE

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE		AMOUNT	
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	4490	$	2.50	$	11,225.00
CLEARING AND GRUBBING	L.S.	1	$	2,500.00	$	2,500.00
FENCE REMOVAL AND REPLACEMENT	L.F.	254	$	60.00	$	15,240.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	3641	$	50.00	$	182,066.66
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	1626	$	40.00	$	65,048.77
SIDEWALK REPLACEMENT	S.Y.	25	$	45.00	$	1,120.00
REPLACE CONCRETE DRIVEWAYS	S.Y.	133	$	45.00	$	6,000.00
CONSTRUCT PROPOSED DRAINAGE MANHOLE	EACH	4	$	10,000.00	$	40,000.00
PLUG EXISTING 36" RCP	EACH	1	$	4,000.00	$	4,000.00
REMOVAL OF PIPE (18" RCP)	L.F.	612	$	20.00	$	12,240.00
REMOVAL OF PIPE (21" RCP)	L.F.	160	$	23.00	$	3,680.00
REMOVAL OF PIPE (27" RCP)	L.F.	157	$	28.00	$	4,396.00
REMOVAL OF PIPE (36" RCP)	L.F.	137	$	35.00	$	4,795.00
REMOVAL OF PIPE (42" RCP)	L.F.	80	$	40.00	$	3,200.00
REMOVAL OF PIPE (48" CMP)	L.F.	50	$	45.00	$	2,250.00
24" RCP	L.F.	712	$	117.00	$	83,304.00
36" RCP	L.F.	217	$	125.00	$	27,125.00
48" RCP	L.F.	1117	$	135.00	$	150,795.00
54" RCP	L.F.	180	$	140.00	$	25,200.00
JACK AND BORE OF 54" RCP UNDER RAILROAD	L.F.	100	$	1,500.00	$	150,000.00
REPLACE EXISTING HEADWALL	EACH	1	$	5,000.00	$	5,000.00
CONSTRUCT PROPOSED HEADWALL	EACH	3	$	5,000.00	$	15,000.00
TRENCH BACKFILL	C.Y.	5171	$	10.00	$	51,705.55
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	994	$	30.00	$	29,825.32
EXCAVATE DRAINAGE CHANNEL	C.Y.	211	$	30.00	$	6,322.06
MACHINED RIP-RAP (CLASS C)	TON	96	$	40.00	$	3,858.75
GEOTEXTILE FABRIC (TYPE IV)	SY	47	$	3.00	$	141.75
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$	135,906	$	135,905.83
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$	91,000.00	$	91,000.00
TRAFFIC CONTROL	L.S.	1	$	10,000	$	10,000.00
			Sub-total		$	1,142,944.70
			20% Contingency		$	229,000.00
			Surveying, Engineering, & Constr. Admin. (15%)		$	172,000.00
			TOTAL		$	1,543,944.70

***PLEASE NOTE** - the cost estimate does not include property acquisition, utility relocation, or easement costs.

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ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-12
RIDGEMONT ROAD & TRUDY STREET

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE		AMOUNT	
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	2260	$	2.50	$	5,650.00
CLEARING AND GRUBBING	L.S.	1	$	2,500.00	$	2,500.00
FENCE REMOVAL AND REPLACEMENT	L.F.	150	$	60.00	$	9,000.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	2270	$	50.00	$	113,516.89
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	1014	$	40.00	$	40,557.31
SIDEWALK REPLACEMENT	S.Y.	11	$	45.00	$	495.00
REPLACE EXISTING 6-72 INLET	EACH	2	$	5,000.00	$	10,000.00
CONSTRUCT PROPOSED 672 INLET	EACH	1	$	5,000.00	$	5,000.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	4	$	10,000.00	$	40,000.00
REMOVAL OF PIPE (27" RCP)	L.F.	859	$	28.00	$	24,052.00
REMOVAL OF PIPE (30" RCP)	L.F.	138	$	30.00	$	4,140.00
REMOVAL OF 11' X 5.83' RCBC	L.F.	82	$	150.00	$	12,300.00
REMOVAL OF 15' X 5.83' RCBC	L.F.	53	$	225.00	$	11,925.00
42" RCP	L.F.	587	$	130.00	$	76,310.00
48" RCP	L.F.	272	$	135.00	$	36,720.00
6' x 4' RCBC	L.F.	276	$	1,200.00	$	331,200.00
DOUBLE BARREL 10' x 7' RCBC	L.F.	82	$	3,000.00	$	246,000.00
DOUBLE BARREL 14' x 8' RCBC	L.F.	53	$	3,500.00	$	185,500.00
REPLACE EXISTING HEADWALL	EACH	5	$	5,000.00	$	25,000.00
INSTALL HEADWALL	EACH	1	$	5,000.00	$	5,000.00
TRENCH BACKFILL	C.Y.	2637	$	10.00	$	26,369.40
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	577	$	30.00	$	17,321.44
MACHINED RIP-RAP (CLASS B)	TON	385	$	40.00	$	15,380.56
MACHINED RIP-RAP (CLASS C)	TON	152	$	40.00	$	6,097.78
GEOTEXTILE FABRIC (TYPE IV)	SY	338	$	3.00	$	1,015.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$	187,658	$	187,657.56
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$	126,000.00	$	126,000.00
TRAFFIC CONTROL	L.S.	1	$	10,000	$	10,000.00
			Sub-total		$	1,574,707.94
			20% Contingency		$	315,000.00
			Surveying, Engineering, & Constr. Admin. (15%)		$	237,000.00
			TOTAL		$	2,126,707.94

*PLEASE NOTE - the cost estimate does not include property acquisition, utility relocation, or easement costs.

December 2021

ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-17
PIPPIN STREET TO TRUDY STREET

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE		AMOUNT	
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	2846	$	2.50	$	7,115.00
CLEARING AND GRUBBING	L.S.	1	$	5,000.00	$	5,000.00
FENCE REMOVAL AND REPLACEMENT	L.F.	255	$	60.00	$	15,300.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	1893	$	50.00	$	94,653.61
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	117	$	40.00	$	4,697.62
SIDEWALK REPLACEMENT	S.Y.	8	$	45.00	$	375.00
REPLACE EXISTING 6-72 INLET	EACH	1	$	5,000.00	$	5,000.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	6	$	10,000.00	$	60,000.00
REMOVAL OF PIPE (27" RCP)	L.F.	819	$	28.00	$	22,932.00
REMOVAL OF PIPE (30" RCP)	L.F.	441	$	30.00	$	13,230.00
REMOVAL OF PIPE (36" RCP)	L.F.	163	$	35.00	$	5,705.00
48" RCP	L.F.	1260	$	135.00	$	170,100.00
4' x 4' RCB	L.F.	163	$	1,000.00	$	163,000.00
REPLACE EXISTING HEADWALL	EACH	1	$	5,000.00	$	5,000.00
TRENCH BACKFILL	C.Y.	4747	$	10.00	$	47,468.13
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	749	$	30.00	$	22,474.56
MACHINED RIP-RAP (CLASS B)	TON	54	$	40.00	$	2,177.78
GEOTEXTILE FABRIC (TYPE IV)	SY	37	$	3.00	$	112.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$	96,651	$	96,651.10
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$	65,000.00	$	65,000.00
TRAFFIC CONTROL	L.S.	1	$	10,000	$	10,000.00
			Sub-total		$	815,991.80
			20% Contingency		$	164,000.00
			Surveying, Engineering, & Constr. Admin. (15%)		$	123,000.00
			TOTAL		$	1,102,991.80

***PLEASE NOTE** - the cost estimate does not include property acquisition, utility relocation, or easement costs.*

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ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-19
PIPPIN STREET

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	AMOUNT
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	2924	$ 2.50	$ 7,310.00
CLEARING AND GRUBBING	L.S.	1	$ 10,000.00	$ 10,000.00
FENCE REMOVAL AND REPLACEMENT	L.F.	336	$ 60.00	$ 20,160.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	480	$ 50.00	$ 24,017.44
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	215	$ 40.00	$ 8,580.95
SIDEWALK REPLACEMENT	S.Y.	54	$ 45.00	$ 2,445.00
REPLACE CONCRETE DRIVEWAYS	S.Y.	206	$ 45.00	$ 9,290.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	1	$ 10,000.00	$ 10,000.00
CONSTRUCT PROPOSED DRAINAGE MANHOLE	EACH	6	$ 10,000.00	$ 60,000.00
REMOVAL OF PIPE (24" RCP)	L.F.	232	$ 25.00	$ 5,800.00
REMOVAL OF PIPE (27" RCP)	L.F.	356	$ 28.00	$ 9,968.00
REMOVAL OF PIPE (30" RCP)	L.F.	519	$ 30.00	$ 15,570.00
REMOVAL OF PIPE (36" RCP)	L.F.	477	$ 35.00	$ 16,695.00
30" RCP	L.F.	68	$ 120.00	$ 8,160.00
36" RCP	L.F.	164	$ 125.00	$ 20,500.00
42" RCP	L.F.	875	$ 130.00	$ 113,750.00
4' x 4' RCB	L.F.	366	$ 1,000.00	$ 366,000.00
DOUBLE BARREL 4' x 4' RCBC	L.F.	341	$ 2,000.00	$ 682,000.00
EXISTING HEADWALL REPLACEMENT	EACH	1	$ 5,000.00	$ 5,000.00
CONSTRUCT PROPOSED HEADWALL	EACH	1	$ 5,000.00	$ 5,000.00
TRENCH BACKFILL	C.Y.	1168	$ 10.00	$ 11,675.21
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	1961	$ 30.00	$ 58,815.11
MACHINED RIP-RAP (CLASS B)	TON	109	$ 40.00	$ 4,355.56
GEOTEXTILE FABRIC (TYPE IV)	SY	75	$ 3.00	$ 224.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$ 221,297	$ 221,297.44
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$ 148,000.00	$ 148,000.00
TRAFFIC CONTROL	L.S.	1	$ 10,000	$ 10,000.00
			Sub-total	$ 1,854,613.71
			20% Contingency	$ 371,000.00
			Surveying, Engineering, & Constr. Admin. (15%)	$ 279,000.00
			TOTAL	$ 2,504,613.71

***PLEASE NOTE** - the cost estimate does not include property acquisition, utility relocation, or easement costs.

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ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-21
BATTLEFIELD DRIVE, KERWIN DRIVE AND TWINMONT STREET

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	AMOUNT
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	7190	$ 2.50	$ 17,975.00
CLEARING AND GRUBBING	L.S.	1	$ 10,000.00	$ 10,000.00
FENCE REMOVAL AND REPLACEMENT	L.F.	325	$ 60.00	$ 19,500.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	5828	$ 50.00	$ 291,422.06
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	2603	$ 40.00	$ 104,119.27
SIDEWALK REPLACEMENT	S.Y.	69	$ 45.00	$ 3,120.00
REPLACE EXISTING 6-72 INLET	EACH	4	$ 5,000.00	$ 20,000.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	9	$ 10,000.00	$ 90,000.00
CONSTRUCT PROPOSED DRAINAGE MANHOLE	EACH	3	$ 10,000.00	$ 30,000.00
PLUG EXISTING 18" RCP	EACH	1	$ 3,000.00	$ 3,000.00
REMOVAL OF PIPE (24" RCP)	L.F.	675	$ 25.00	$ 16,875.00
REMOVAL OF PIPE (27" RCP)	L.F.	751	$ 28.00	$ 21,028.00
REMOVAL OF PIPE (36" RCP)	L.F.	173	$ 35.00	$ 6,055.00
REMOVAL OF PIPE (42" RCP)	L.F.	1059	$ 40.00	$ 42,360.00
REMOVAL OF PIPE (48" RCP)	L.F.	222	$ 45.00	$ 9,990.00
REMOVAL OF 8' x 6' RCB	L.F.	69	$ 200.00	$ 13,800.00
30" RCP	L.F.	870	$ 120.00	$ 104,400.00
36" RCP	L.F.	376	$ 125.00	$ 47,000.00
60" x 38" ERCP	L.F.	552	$ 220.00	$ 121,440.00
78" x 48" ERCP	L.F.	958	$ 250.00	$ 239,500.00
6' x 4' RCB	L.F.	478	$ 1,200.00	$ 573,600.00
7' x 4' RCB	L.F.	423	$ 1,300.00	$ 549,900.00
12' x 4' RCB	L.F.	146	$ 1,400.00	$ 204,400.00
12' x 6' RCB	L.F.	69	$ 1,500.00	$ 103,500.00
EXISTING HEADWALL REPLACEMENT	EACH	2	$ 5,000.00	$ 10,000.00
CONSTRUCT PROPOSED HEADWALL	EACH	3	$ 5,000.00	$ 15,000.00
TRENCH BACKFILL	C.Y.	8282	$ 10.00	$ 82,821.78
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	1808	$ 30.00	$ 54,253.76
MACHINED RIP-RAP (CLASS B)	TON	340	$ 40.00	$ 13,611.11
GEOTEXTILE FABRIC (TYPE IV)	SY	233	$ 3.00	$ 700.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$ 422,906	$ 422,905.65
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$ 282,000.00	$ 282,000.00
TRAFFIC CONTROL	L.S.	1	$ 10,000	$ 10,000.00
			Sub-total	$ 3,534,276.62
			20% Contingency	$ 707,000.00
			Surveying, Engineering, & Constr. Admin. (15%)	$ 531,000.00
			TOTAL	$ 4,772,276.62

***PLEASE NOTE - the cost estimate does not include property acquisition, utility relocation, or easement costs.**

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ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-23
DORADO AVENUE AND WINDERMERE DRIVE

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	AMOUNT
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	5926	$ 2.50	$ 14,815.00
CLEARING AND GRUBBING	L.S.	1	$ 10,000.00	$ 10,000.00
FENCE REMOVAL AND REPLACEMENT	L.F.	100	$ 60.00	$ 6,000.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	3592	$ 50.00	$ 179,595.44
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	1604	$ 40.00	$ 64,165.86
SIDEWALK REPLACEMENT	S.Y.	13	$ 45.00	$ 600.00
REPLACE CONCRETE DRIVEWAYS	S.Y.	35	$ 45.00	$ 1,575.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	10	$ 10,000.00	$ 100,000.00
CONSTRUCT PROPOSED DRAINAGE MANHOLE	EACH	2	$ 10,000.00	$ 20,000.00
PLUG EXISTING 42" RCP	EACH	1	$ 4,500.00	$ 4,500.00
REMOVAL OF PIPE (27" RCP)	L.F.	222	$ 28.00	$ 6,216.00
REMOVAL OF PIPE (30" RCP)	L.F.	83	$ 30.00	$ 2,490.00
REMOVAL OF PIPE (36" RCP)	L.F.	615	$ 35.00	$ 21,525.00
REMOVAL OF PIPE (42" RCP)	L.F.	1358	$ 40.00	$ 54,320.00
36" RCP	L.F.	410	$ 125.00	$ 51,250.00
48" RCP	L.F.	314	$ 135.00	$ 42,390.00
4' x 4' RCB	L.F.	1429	$ 1,000.00	$ 1,429,000.00
6' x 4' RCB	L.F.	1142	$ 1,200.00	$ 1,370,400.00
EXISTING HEADWALL REPLACEMENT	EACH	1	$ 5,000.00	$ 5,000.00
CONSTRUCT PROPOSED HEADWALL	EACH	1	$ 5,000.00	$ 5,000.00
TRENCH BACKFILL	C.Y.	6801	$ 10.00	$ 68,014.32
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	1347	$ 30.00	$ 40,414.19
MACHINED RIP-RAP (CLASS B)	TON	54	$ 40.00	$ 2,177.78
MACHINED RIP-RAP (CLASS C)	TON	76	$ 40.00	$ 3,048.89
GEOTEXTILE FABRIC (TYPE IV)	SY	75	$ 3.00	$ 224.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$ 525,408	$ 525,408.22
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$ 351,000.00	$ 351,000.00
TRAFFIC CONTROL	L.S.	1	$ 10,000	$ 10,000.00
			Sub-total	$ 4,389,129.70
			20% Contingency	$ 878,000.00
			Surveying, Engineering, & Constr. Admin. (15%)	$ 659,000.00
			TOTAL	$ 5,926,129.70

*PLEASE NOTE - the cost estimate does not include property acquisition, utility relocation, or easement costs.

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ENGINEER'S OPINION OF CONSTRUCTION COST (CONCEPT-LEVEL)
PROPOSED PROJECT D-29
FRAYSER RALEIGH ROAD

ITEM DESCRIPTION	UNIT	QUANTITY	UNIT PRICE		AMOUNT	
TEMPORARY HIGH VISIBILITY CONSTRUCTION FENCE	L.F.	3164	$	2.50	$	7,910.00
CLEARING AND GRUBBING	L.S.	1	$	5,000.00	$	5,000.00
ASPHALT REMOVAL AND REPLACEMENT	S.Y.	2458	$	50.00	$	122,901.61
MINERAL AGGREGATE, TYPE A BASE, GRADING D	TON	1098	$	40.00	$	43,910.29
SIDEWALK REPLACEMENT	S.Y.	70	$	45.00	$	3,150.00
REPLACE EXISTING DRAINAGE MANHOLE	EACH	7	$	10,000.00	$	70,000.00
CONSTRUCT PROPOSED DRAINAGE MANHOLE	EACH	2	$	10,000.00	$	20,000.00
REMOVAL OF PIPE (18" RCP)	L.F.	246	$	20.00	$	4,920.00
REMOVAL OF PIPE (24" RCP)	L.F.	62	$	25.00	$	1,550.00
REMOVAL OF PIPE (30" RCP)	L.F.	222	$	30.00	$	6,660.00
REMOVAL OF PIPE (36" RCP)	L.F.	83	$	35.00	$	2,905.00
REMOVAL OF PIPE (42" RCP)	L.F.	481	$	40.00	$	19,240.00
36" RCP	L.F.	261	$	125.00	$	32,625.00
42" RCP	L.F.	618	$	130.00	$	80,340.00
48" RCP	L.F.	222	$	135.00	$	29,970.00
6' x 5' RCB	L.F.	481	$	1,250.00	$	601,250.00
EXISTING HEADWALL REPLACEMENT	EACH	1	$	5,000.00	$	5,000.00
TRENCH BACKFILL	C.Y.	5470	$	10.00	$	54,704.56
BEDDING MATERIAL (PIPE) CLASS B	C.Y.	755	$	30.00	$	22,659.89
MACHINED RIP-RAP (CLASS B)	TON	54	$	40.00	$	2,177.78
GEOTEXTILE FABRIC (TYPE IV)	SY	37	$	3.00	$	112.00
EROSION CONTROL (15% CONSTRUCTION COST)	L.S.	1	$	170,548	$	170,547.92
MOBILIZATION (10% CONSTRUCTION COST, ROUND UP)	L.S.	1	$	114,000.00	$	114,000.00
TRAFFIC CONTROL	L.S.	1	$	10,000	$	10,000.00
			Sub-total		$	1,431,534.05
			20% Contingency		$	287,000.00
			Surveying, Engineering, & Constr. Admin. (15%)		$	215,000.00
			TOTAL		$	1,933,534.05

*PLEASE NOTE - the cost estimate does not include property acquisition, utility relocation, or easement costs.

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Table 23. Overall Study Area Cost Summary

Project ID	Project Short Description	Estimated Project Cost (Including Surveying, Engineering, Construction Administration & Contingencies)
A-1	Beacon Hills Drive and Sunny Hill Drive	$ 780,051
D-9	Gruber Drive	$ 1,543,945
D-12	Ridgemont Road and Trudy Street	$ 2,126,708
D-17	Pippin Street to Trudy Street	$ 1,102,992
D-19	Pippin Street to Trudy Street	$ 2,504,614
D-21	Battlefield Drive, Kerwin Drive and Twinmont Street	$ 4,772,277
D-23	Dorado Avenue and Windermere Drive	$ 5,926,130
D-29	Frayser Raleigh Road	$ 1,933,534
Total Estimated Cost of Improvements for Allen Basin Study Area		**$ 20,690,249**

4 CONCLUSIONS AND FINAL RECOMMENDATIONS

A total of 8 proposed project areas are recommended for implementation within the Allen Basin Study Area. The total estimated planning-level construction cost of all recommended improvements for the entire study area is **$20,690,249,** which includes surveying, engineering, construction administration, and contingent costs.

Refer to Section 5 of Volume II for an overview map of the Allen Basin Study Area; Exhibit 5.3 depicts the location of the recommended improvement projects.

Refer to Section 8 of Volume II for conceptual layouts for each proposed project area.

Refer to Section 4 of Volume II for InfoSWMM model results for Existing and Improved Conditions.

Refer to Section 6 of Volume II for Floodplain Inundation Map Exhibits for Existing and Improved Conditions.

Order of Improvements

Typically, it is recommended to implement drainage improvements in a downstream-to-upstream order, particularly when conveyance capacity is being added. However, top priority should be given to areas where structural flooding occurs in frequent storm events. High priority should also be given to areas where roadway flooding occurs at major arterial roadways. Areas where both structural and residential roadway flooding occurs frequently should also be given priority, particularly for areas where loss of access for emergency vehicles occurs. Secondary priority should be given to areas where local or residential roadway flooding occurs in frequent storm events with no loss of access. Nuisance flooding (i.e., yard flooding) was considered but given lower priority than structural and roadway flooding.

As described in Section 3.3.2 above, a total of 28 areas of existing flooding were initially evaluated to determine the magnitude and extent of inundation and associated impacts to surrounding structures and roadways. Using the criteria described above, as well as consideration of cost-effectiveness and general constructability, 8 of these areas were selected for further evaluation and implementation. Once conceptual improvements were developed at each of the 8 project locations, the resulting improvements to flooding could be determined at relevant points of interest (i.e., specific property addresses and roadway crossings). Given the LOS improvement associated with each improvement project, and the tabulated planning-level construction costs for each project as presented in Section 3.5 above, an evaluation of the general benefit of each project could be made.

In order to determine the recommended order of project implementation, the total project cost was divided by the number of structures removed from flooding for each project. Using this tabulated unit cost, the 8 projects were sorted in ascending order from least-expensive per structure removed from flooding (D-17; $183,832 per structure) to most-expensive per structure removed from flooding (D-9; $1,543,945 per structure). For projects D-12, D-19, and D-29, where the project cost per structure removed from flooding was within approximately $80,000 of one another, the three projects were sorted in ascending order based on the cost per roadway crossing removed from flooding.

Based on the above criteria and computational methodology, it is recommended to perform the proposed improvement projects in the order shown in the table below:

Table 24. Order of Implementation - Drainage Improvement Project for Allen Basin Study Area

Priority/Order	Project ID	Project Location/Description
1	D-17	Pippin Street to Trudy Street
2	D-29	Frayser Raleigh Road
3	D-19	Pippin Street to Trudy Street
4	D-12	Ridgemont Road and Trudy Street
5	A-1	Beacon Hills Drive and Sunny Hill Drive
6	D-23	Dorado Avenue and Windermere Drive
7	D-21	Battlefield Drive, Kerwin Drive and Twinmont Street
8	D-9	Gruber Drive