Todd Creek Drainage Master Plan

Final Report Volume 2





TETRA TECH

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1.0 INTRODUCTION

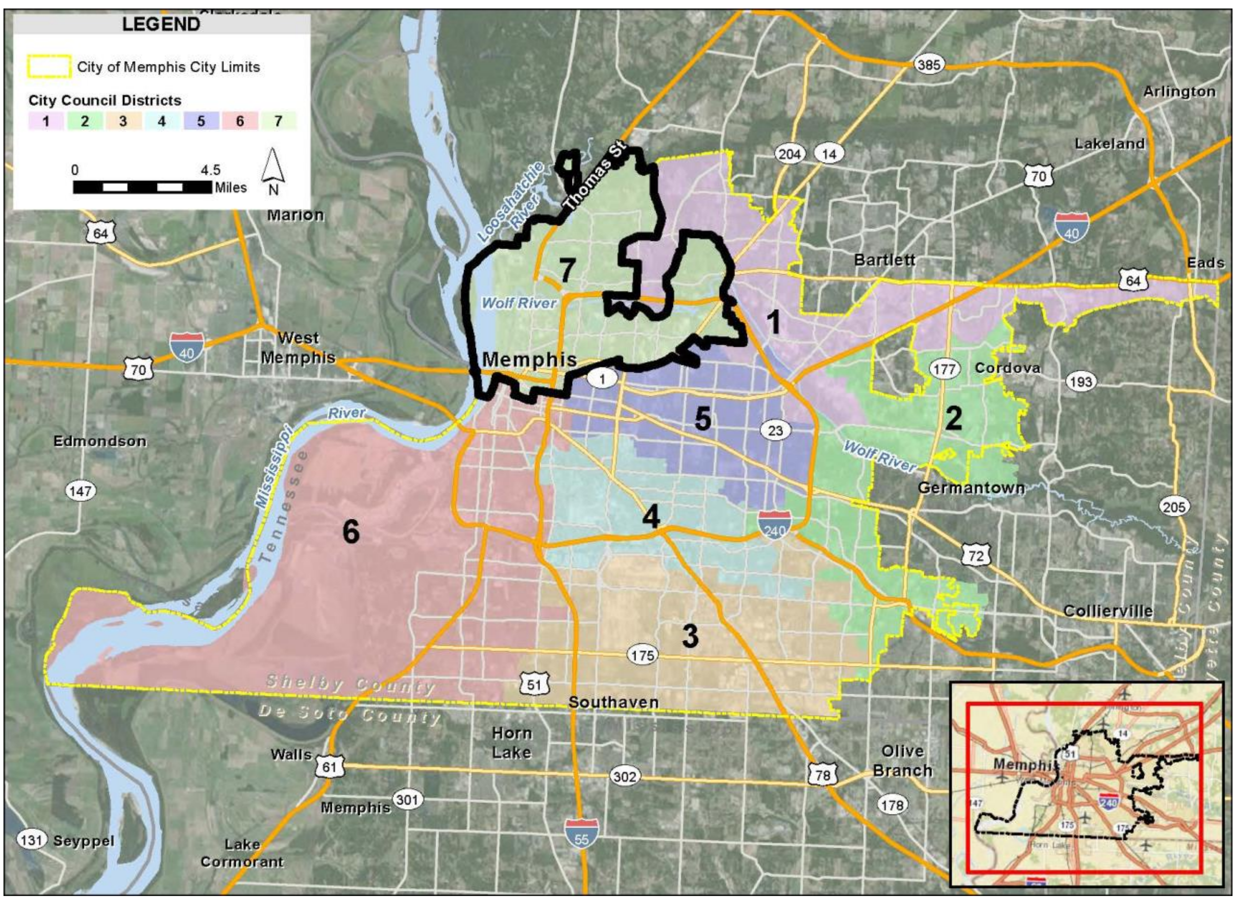
Beginning in 2014 the City of Memphis (City) initiated a multi-year citywide Stormwater Master Planning Program to evaluate the entire stormwater drainage system and identify improvement opportunities to help alleviate hydraulic contractions and flooding. To achieve this end, the City identified three specific goals and determined a geographical implementation schedule. The goals are to:

- Collect data on the existing stormwater drainage systems and develop a comprehensive geographic information system (GIS) dataset for future City planning and maintenance efforts.
- Identify areas that lack hydraulic conveyance capacity and the corresponding extents of flooding.
- Evaluate and recommend improvements to alleviate flooding in the stormwater drainage system.

The geographical implementation schedule was determined by subdividing the stormwater drainage system into seven Stormwater Districts that roughly correspond to the seven City Council Districts. Each of the Stormwater Districts were then sub-divided into smaller, more manageable sizes for implementing a complete master planning study for each sub-divided area.

The City intends to complete a master planning study of one sub-divided area in each of the seven districts until all areas have been evaluated. The sub-divided areas have been prioritized based on a “heat map” analysis of flooding complaints received by the City of Memphis Engineering and Public Works Departments. Within District 7, the highest priority area was determined to be the Todd Creek Study Area. Figure 1.1 identifies the seven council districts, highlighting District 7 where the Todd Creek Study Area is located.

Figure 1.1: City of Memphis Stormwater Districts

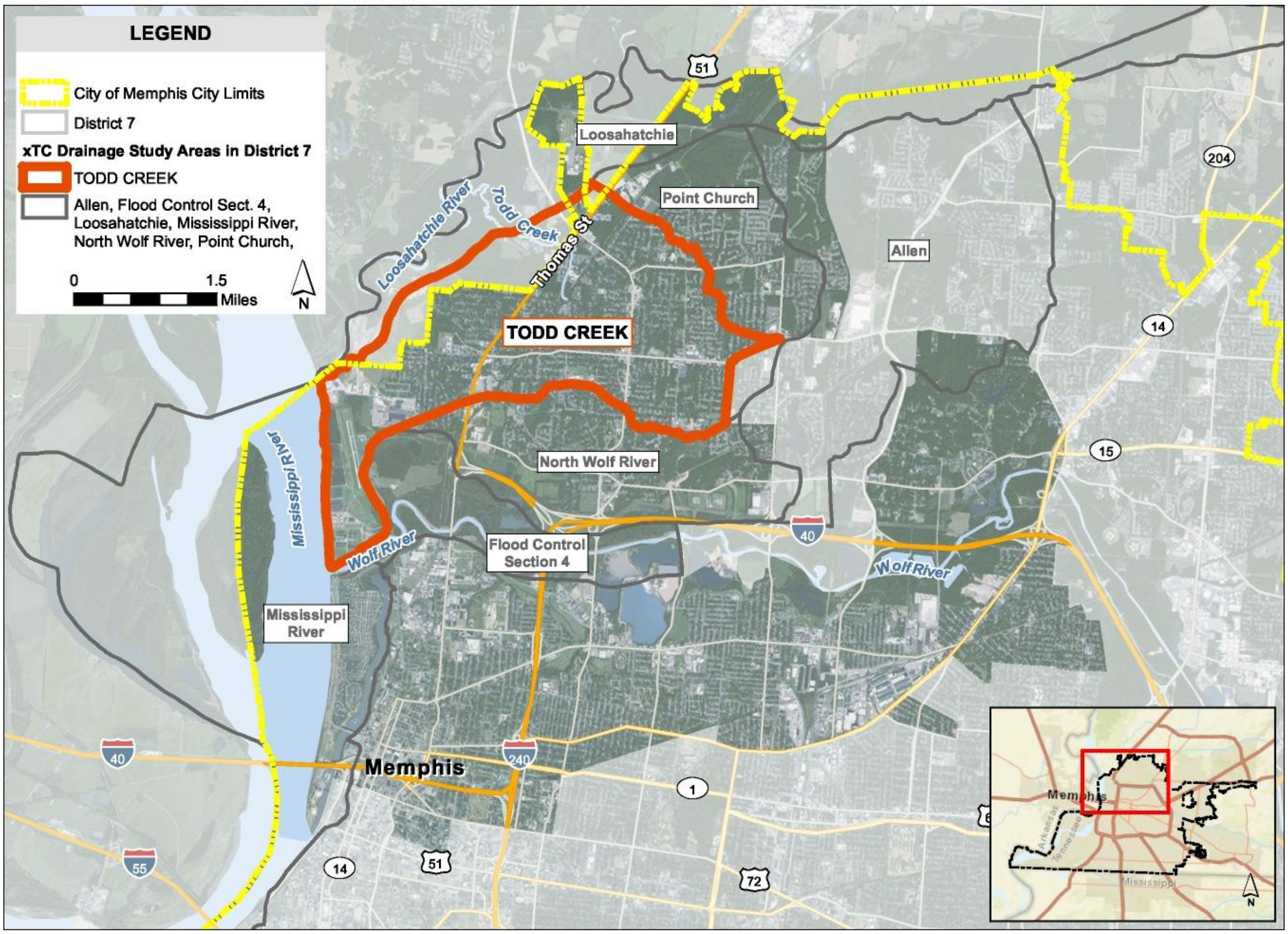


1.1 TODD CREEK STUDY AREA

The Todd Creek Study Area (Study Area) is located in the northwest portion of the City within Stormwater District 7. The Study Area consists of approximately 5,136 acres situated between the Mississippi River to the west, Loosahatchie River to the west/north, Coventry Drive/Hillside Avenue to the north, Range Line Road to the east and Whitney Avenue/Dellwood Avenue/Clifton Road to the south. Figure 1.2 presents the Study Areas included in District 7, highlighting the Todd Creek Study Area.

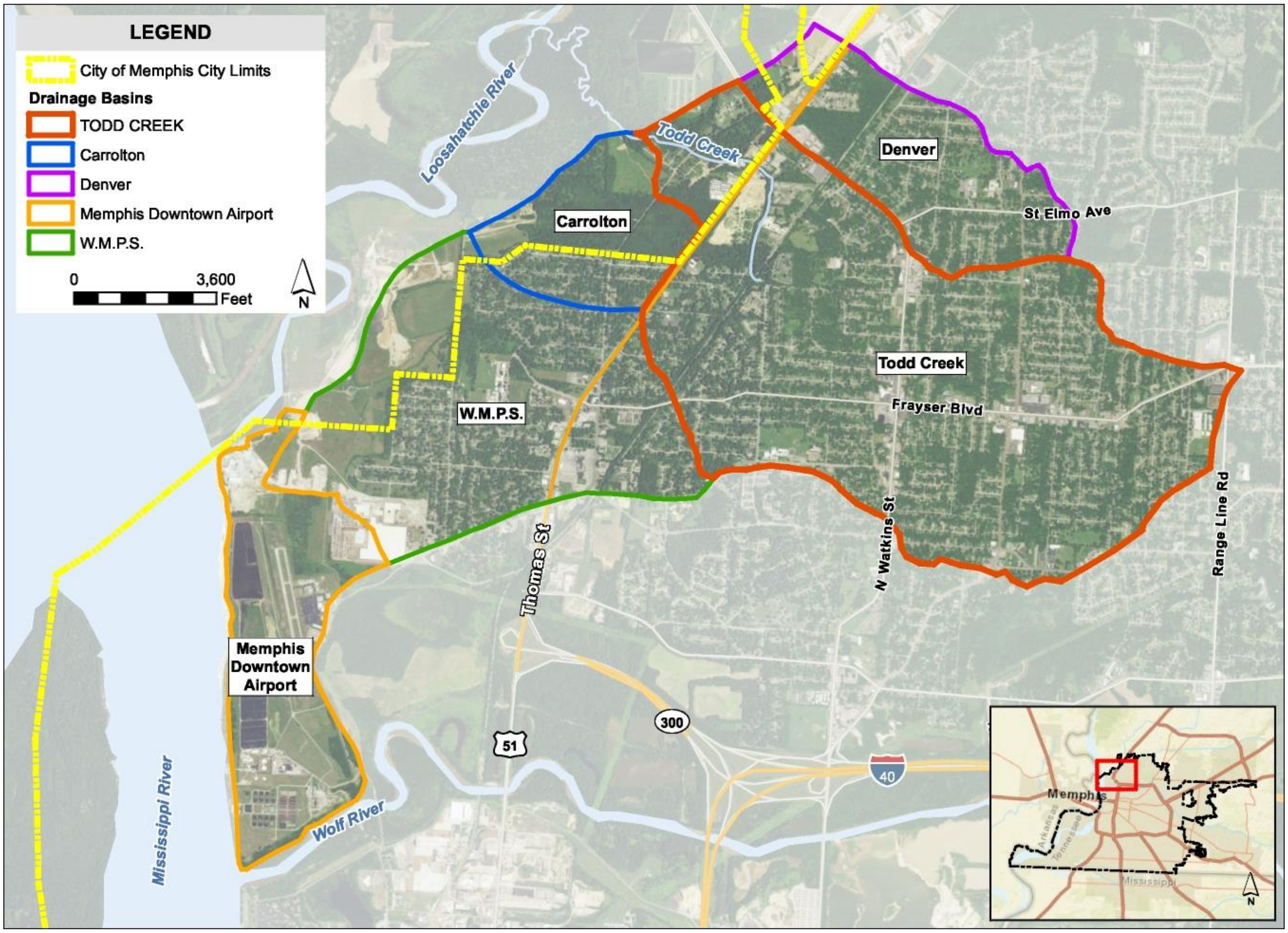
The Todd Creek Study Area is located within the Frayser community which has a long history dating back to the early 1820's. In the mid 1900's, Frayser developed into a working-class suburb with growth attributed to nearby industries. Frayser was annexed into the City in 1958, the same year that International Harvester Company opened the largest farm-equipment manufacturing plant in the South, on 260 acres of land just northeast of General DeWitt Spain Airport. In the 1970s and 1980s, the area began an economic and population decline as factories and other industries closed for various reasons. The International Harvester plant, which once had about two thousand local employees, closed in May 1983. Currently the area is still economically challenged with few major employers and many properties in a state of disrepair, or simply abandoned.

Figure 1.2: Todd Creek Study Area Location Map



The Todd Creek Study Area consists of five watersheds: Todd Creek, Denver Branch, WMPS, Carrolton, and the Memphis Downtown Airport. Todd Creek, Denver Branch, WMPS and Carrolton drain to the Loosahatchie River. The Memphis Downtown Airport watershed discharges into the Mississippi River. Figure 1.3 presents the five watersheds that comprise the Todd Creek Study Area.

Figure 1.3: Todd Creek Study Area Drainage Basins



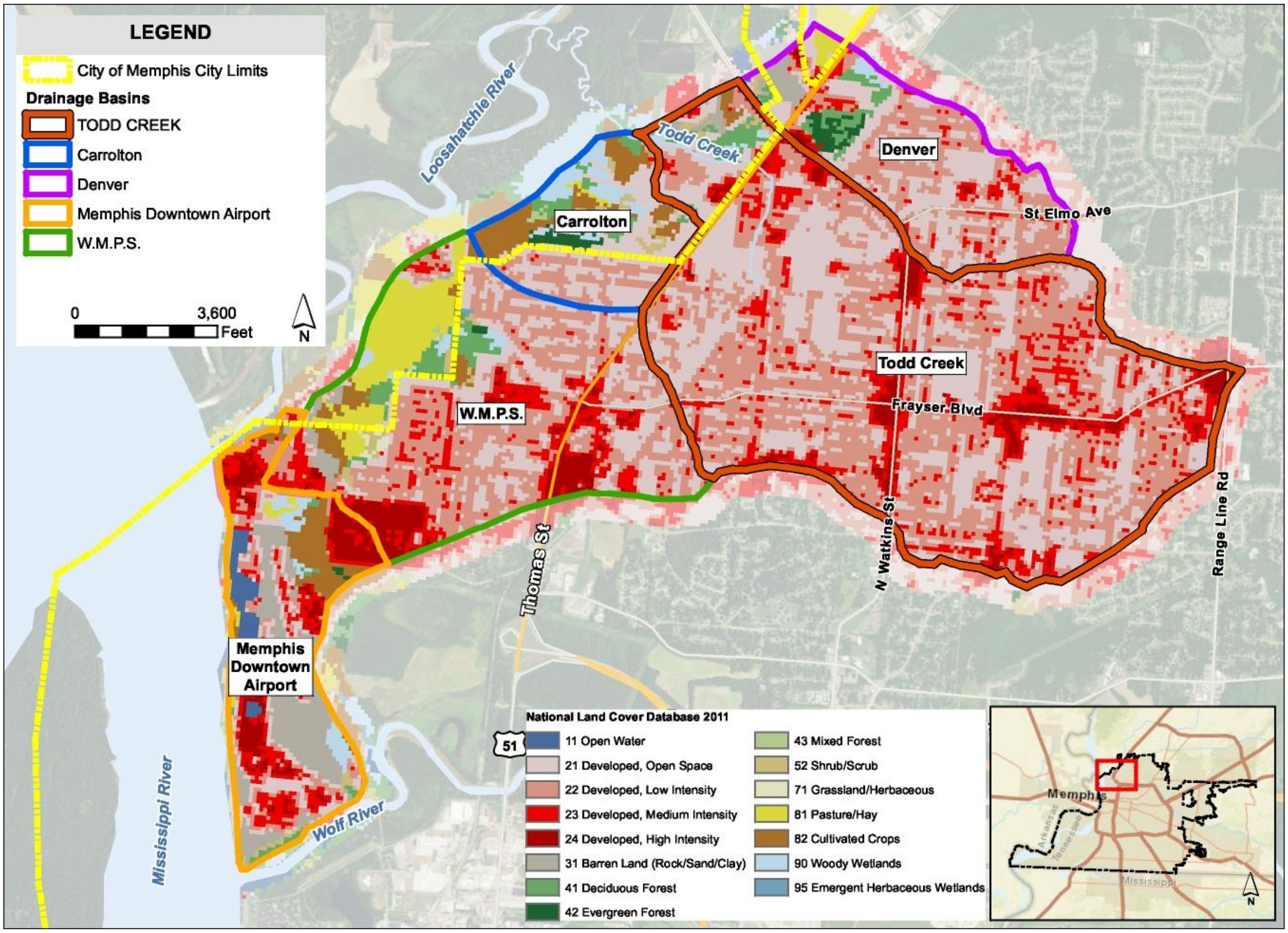
The Todd Creek Study Area has a population of approximately 20,900. Land use for the Todd Creek Study Area consists of 55 percent residential, 12 percent commercial, 11 percent open space and 21 percent “other” land use (NLCD 2010). The Todd Creek watershed is the largest and most densely developed portion of the Study Area, followed by WMPS and Denver. Table 1.1 provides a summary of the population and land use characteristics for each of the five watersheds in the Study Area. Figure 1.4 provides a plan view schematic of the land uses across the Todd Creek Study Area.

Table 1.1: Todd Creek Study Area Land Use Statistics						
Drainage Basin	Population [1]	Area, ac	Residential	Commercial	Open Space	Other
Todd Creek	12,970	2,408	1,668	204	46	490
Denver	2,211	609	310	47	20	232
WMPS	4,940	1,149	660	65	5	419
Carrolton	715	404	155	0	188	61
Downtown Airport	24	566	0	284	282	0
Total	**20,860**	5,136	2,793	600	541	1,202

1 – Population data based on 2010 US Census Bureau data.

2 – Land use data from the Shelby County GIS parcel data, see Section 3.3 for additional details.

Figure 1.4: Todd Creek Study Area Land Use



Using NRCS soil data, an analysis was performed to characterize the underlying soils for the Todd Creek Study area. The NRCS soil data was obtained in GIS format and geo-processed based on the five sub-watersheds. Table 1.2 presents a summary of each soil type and Hydrologic Soil Group by watershed. The dominate soil type in the Study Area is silty loam and is mainly classified as NRCS Hydrologic Soil Group B (75%), with the remainder in Hydrologic Soil Groups D (18%) and C (7%).

Figure 1.5 presents the geographic extents of the NRCS soil data for the Todd Creek Study Area. The NRCS soil data is color-coded based on soil type and Hydrologic Soil Group: orange/tan shaded areas are Hydrologic Soil Group B, green shaded areas are Hydrologic Soil Group C, and blue shaded areas are Hydrologic Soil Group D. The eastern limit to the center of the study area is dominated by Hydrologic Soil Group B. From the center of the study area to the western limit, there are intrusions of Hydrologic Soil Group C and D.

Table 1.2: NRCS Soil Classification and Hydrologic Soil Group Data

Soil Classification	Todd Creek	Denver	WMPS	Carrolton	Airport	Total
Hydrologic Soil Group B	**2,039**	**438**	**927**	**179**	**237**	**3,820**
Filled land silty	272	69	53	1	118	513
Graded land	837	170	257	16	15	1,296
Memphis silt loam	930	199	617	161	28	1,934
Robinsonville fine sandy loam	0	0	0	0	22	22
Robinsonville silt loam	0	0	0	0	55	55
Hydrologic Soil Group C	**99**	**50**	**71**	**69**	**71**	**361**
Adler silt loam	0	0	0	0	4	4
Bowdre silty clay	0	0	4	0	0	4
Calloway silt loam	28	14	5	11	0	57
Collins silt loam	0	0	7	28	5	40
Commerce silt loam	0	0	0	0	62	62
Grenada silt loam	69	37	23	4	0	133
Loring silt loam	2	0	33	26	0	61
Hydrologic Soil Group D	**269**	**121**	**151**	**156**	**257**	**954**
Falaya silt loam	266	91	97	93	3	550
Henry silt loam	3	26	16	0	0	46
Sharkey clay	0	0	29	0	121	150
Tunica silty clay	0	0	9	0	133	142
Waverly silt loam	0	3	0	63	0	66
Water	**1**	**0**	**0**	**0**	**0**	**1**
Totals	**2,408**	**609**	**1,149**	**404**	**566**	**5,136**

Figure 1.6 presents a topographic relief map for the Todd Creek Study Area. The Study Area experiences approximately 140 feet of fall from the highpoints along the boundaries with the adjacent stormwater study areas and the outlet to the Mississippi and Loosahatchie Rivers. The watershed has significant slopes from the highpoints to low lying and flatter areas that contain larger conveyance networks. The flatter areas with larger conveyance systems are those most likely to experience flooding when sufficient capacity is not available.

The low-lying areas along the western boundary of the study area are located within the Loosahatchie 100-year floodplain. The Loosahatchie River floodplain is dominated by backwater from the Mississippi River during periods of high river levels.

Figure 1.5: NRCS Soil Classification and Hydrologic Soil Group Data

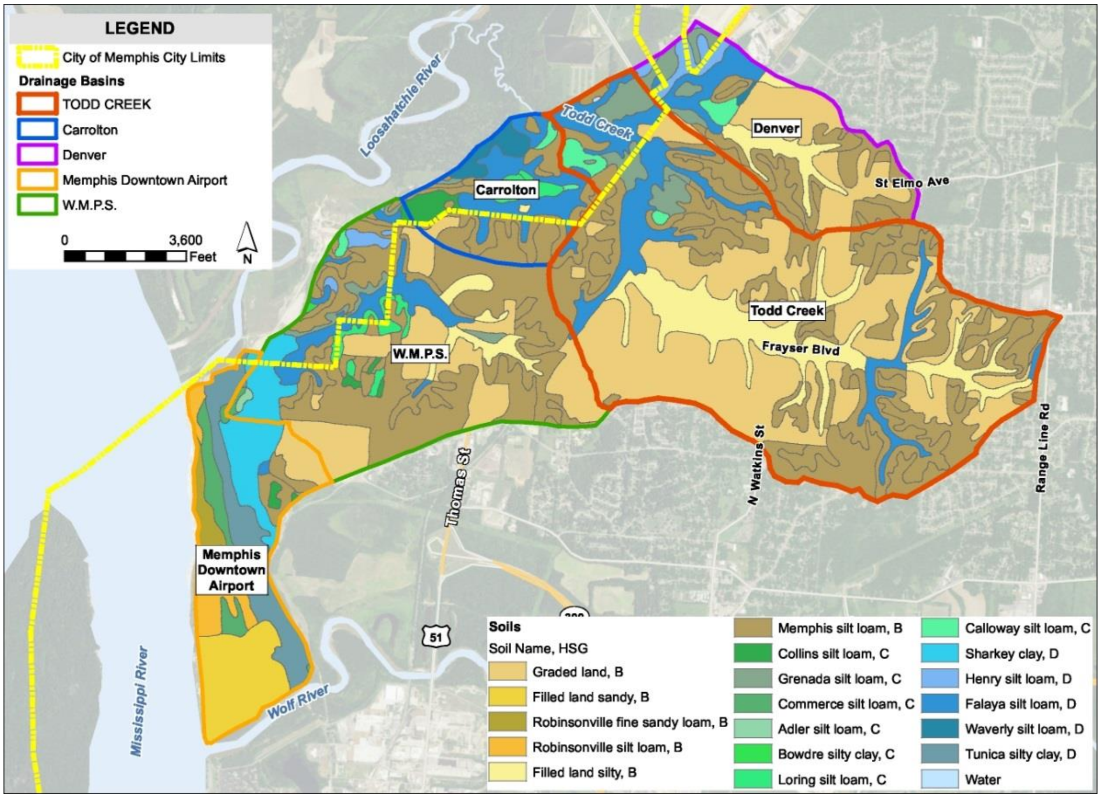
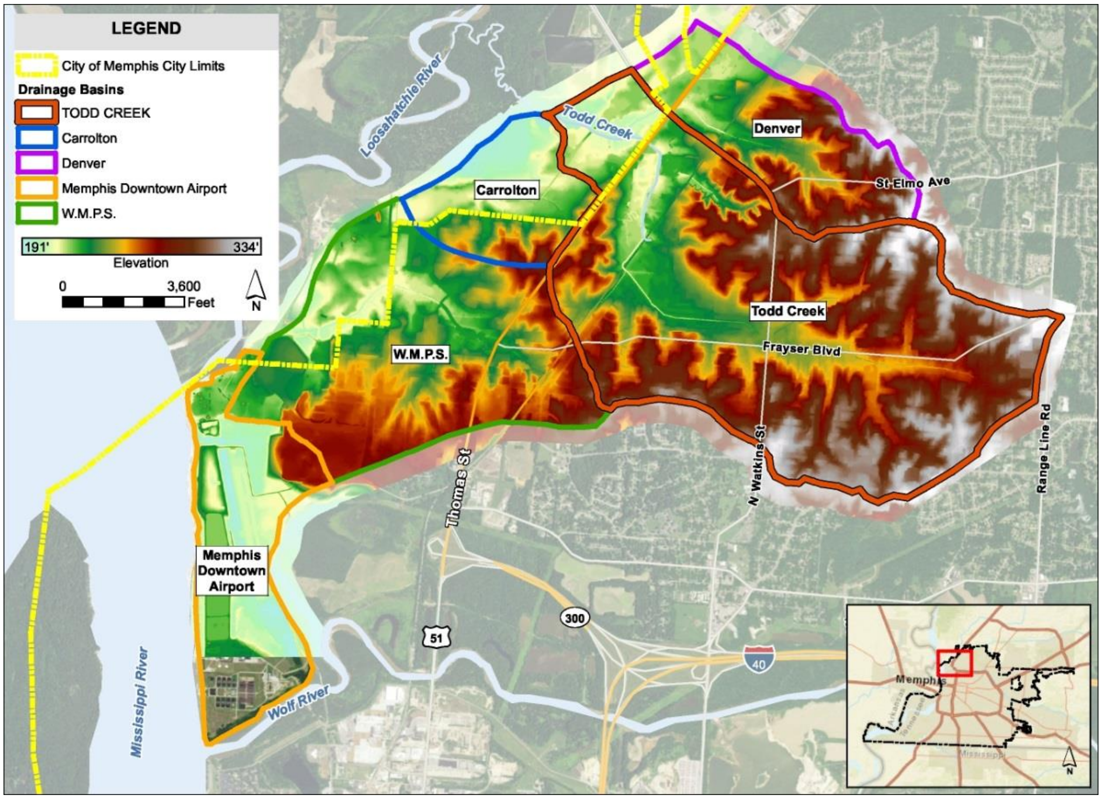


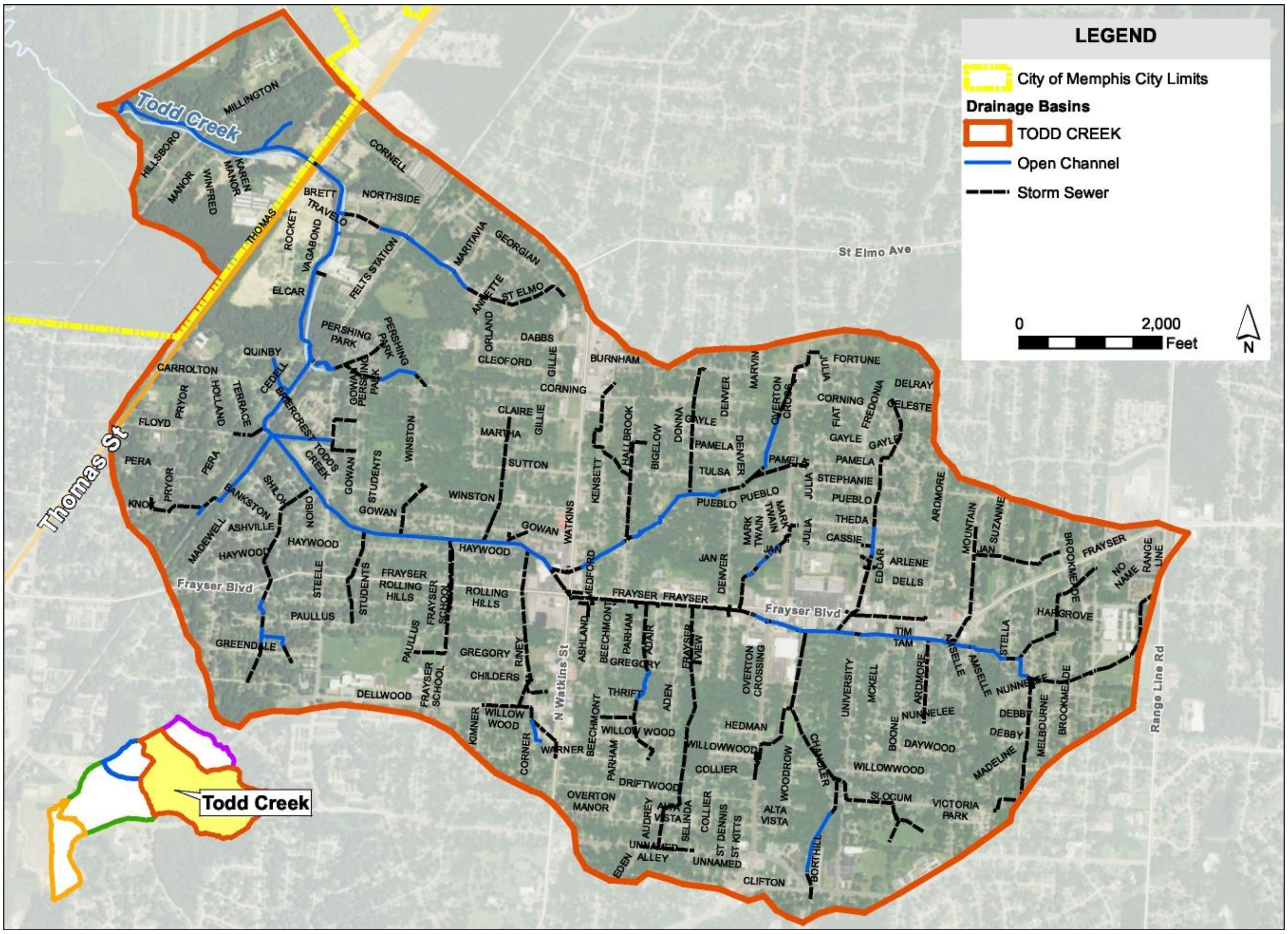
Figure 1.6: Todd Creek Study Area Topographic Relief



1.1.1 Todd Creek Watershed

The Todd Creek Watershed, as shown in Figure 1.7, is the largest of the five watersheds that comprise the Todd Creek Study Area (2,408 acres or 47 percent). The watershed is drained by Todd Creek and flows to the Loosahatchie River. Located downstream and west of Millington Road, Todd Creek is a natural open channel surrounded by undeveloped open space within the floodplain of the Loosahatchie River. Between Millington Road and Watkins Street, Todd Creek is a concrete-lined channel (30 feet to 42 feet bottom width) with several road and rail crossings. Between Watkins Street and Denver Street, Todd Creek flows primarily through a closed box culvert (7 feet high by 22 feet wide) within the Frayser Boulevard right-of-way. Todd Creek then transitions to an open channel between Denver and Derby Streets, and back to a closed pipe system between Derby and Trezevant Streets. The main branch of Todd Creek terminates at Trezevant Street with minor local drainage systems discharging to the upstream terminus of the system.

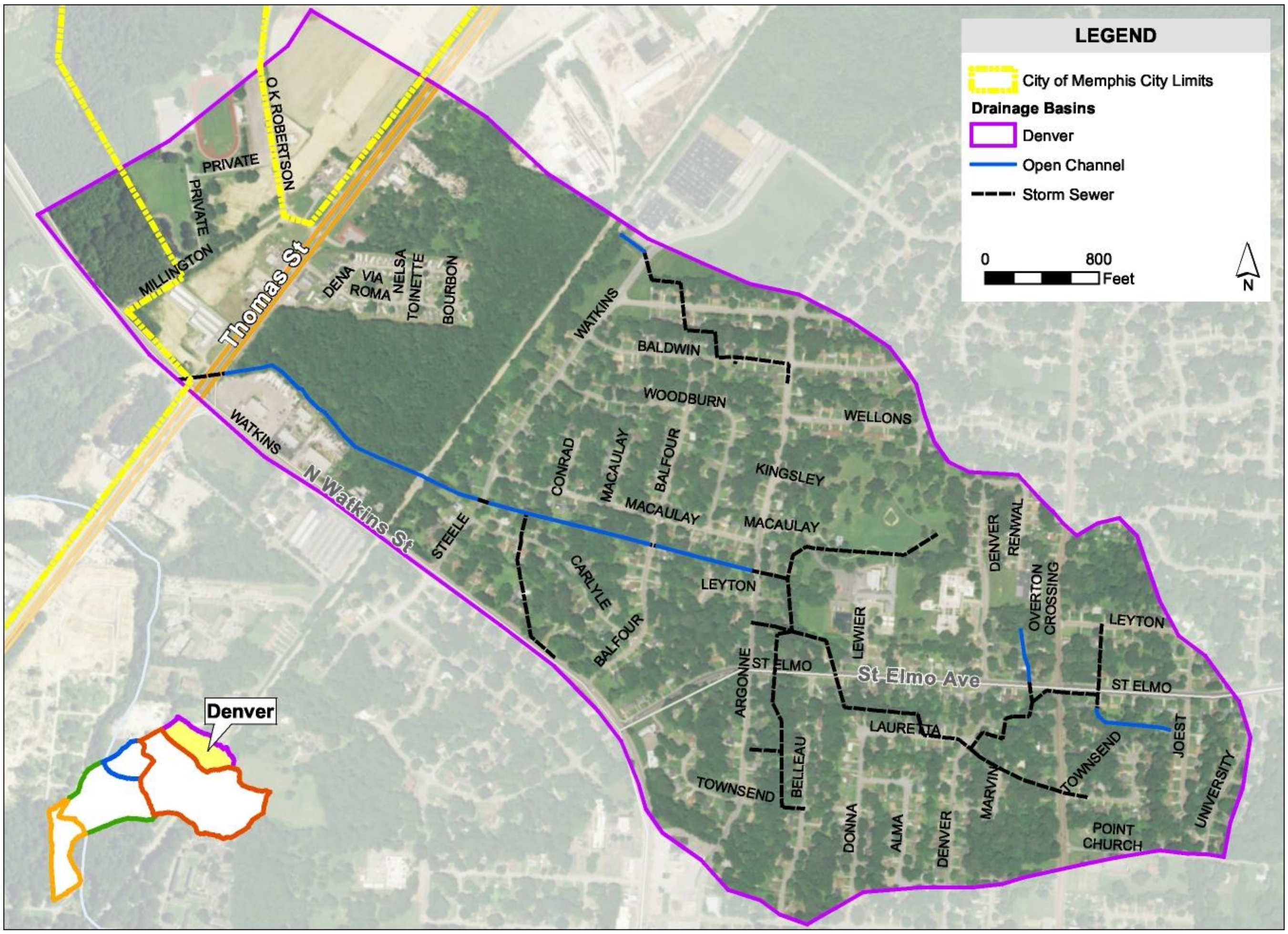
Figure 1.7: Todd Creek Watershed



1.1.2 Denver Watershed

The Denver Branch of Todd Creek, as shown in Figure 1.8, is located in the northern most portion of the Todd Creek Study Area, including 609 acres or approximately 12 percent of the total Study Area. Denver Branch discharges to Todd Creek just upstream of Millington Road in an undeveloped floodplain area. Denver Branch is an open channel from Todd Creek to the Canadian National Railroad (formerly Illinois Central Railroad) located west of Steele Street. Upstream of the Canadian National Railroad to Argonne Street, Denver Branch is a concrete-lined channel (approximately 9 feet bottom width). Upstream of Argonne Street, Denver Branch is primarily a closed pipe storm sewer system.

Figure 1.8: Denver Watershed

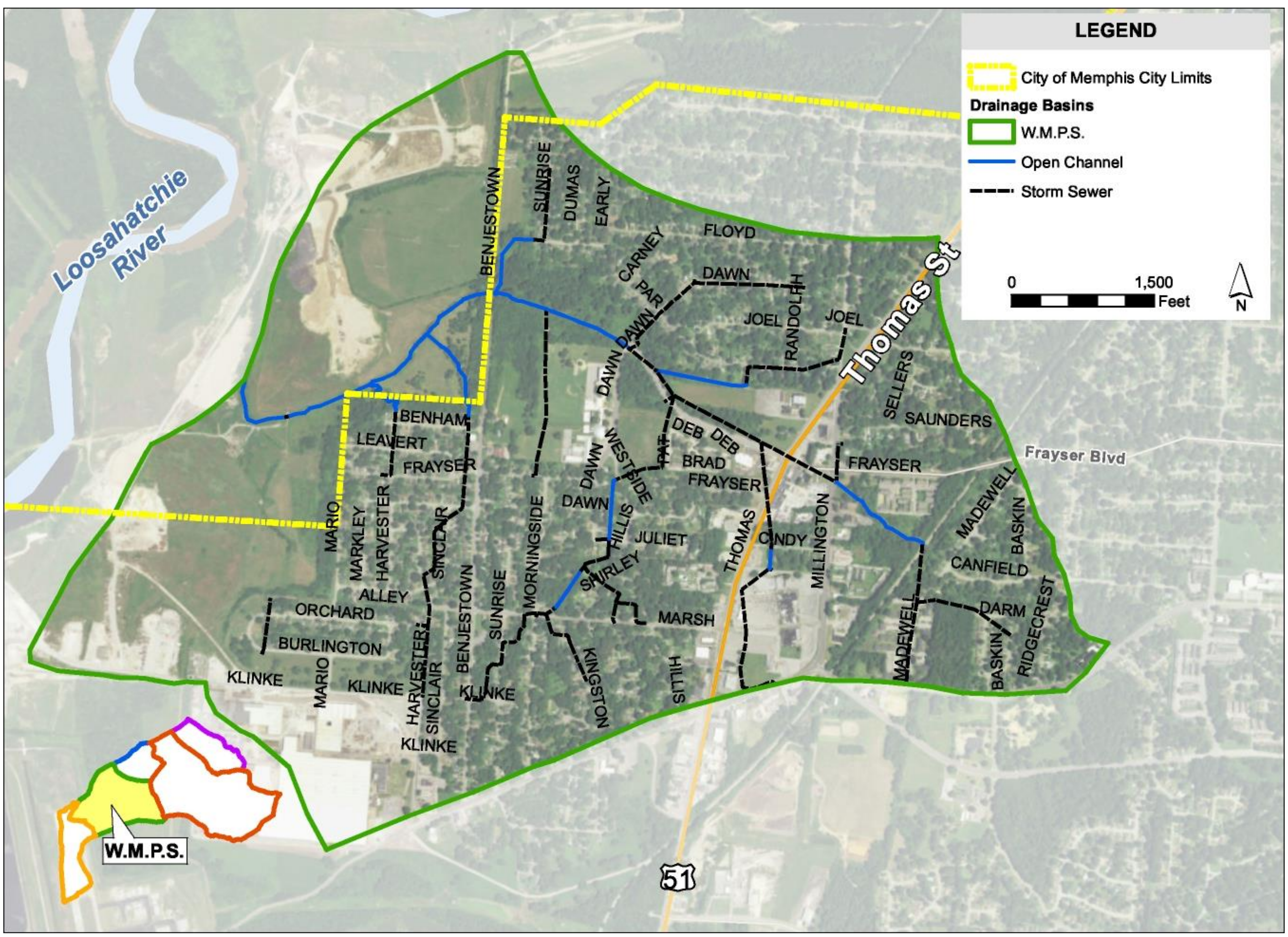


1.1.3 WMPS Watershed

The WMPS Watershed, as shown in Figure 1.9, is located in the central-western part of the Todd Creek Study Area, including 1,149 acres or approximately 22 percent of the total Study Area. The name "WMPS" refers to the call letters for the local AM radio station, originally named after the now-defunct Memphis Press-Scimitar newspaper. The initial "W" is the call letter normally issued by the federal government to broadcast stations east of the Mississippi River. The WMPS broadcast towers were previously located on the west side of Benjestown Road, between Par Avenue and Carrolton Avenue. The towers have since been relocated over 12 miles east in the City of Bartlett. The Memphis Light, Gas and Water (MLGW) currently has telemetry antennae at the former WMPS broadcast tower location.

The WMPS system meets the Loosahatchie River approximately one mile upstream of the confluence of the Loosahatchie River and Mississippi River. The lower WMPS drainage system is an open channel from the Loosahatchie River to Dawn Drive. Between Dawn Drive and Millington Road, the WMPS system has a box culvert ranging in size from seven to eighteen feet bottom width and five to seven feet height. Between Millington Road and the Canadian National Railroad tracks, the WMPS system consists of approximately 1,200 feet of open channel. Upstream of the tracks are local drainage systems including a mix of closed storm sewer and open channel.

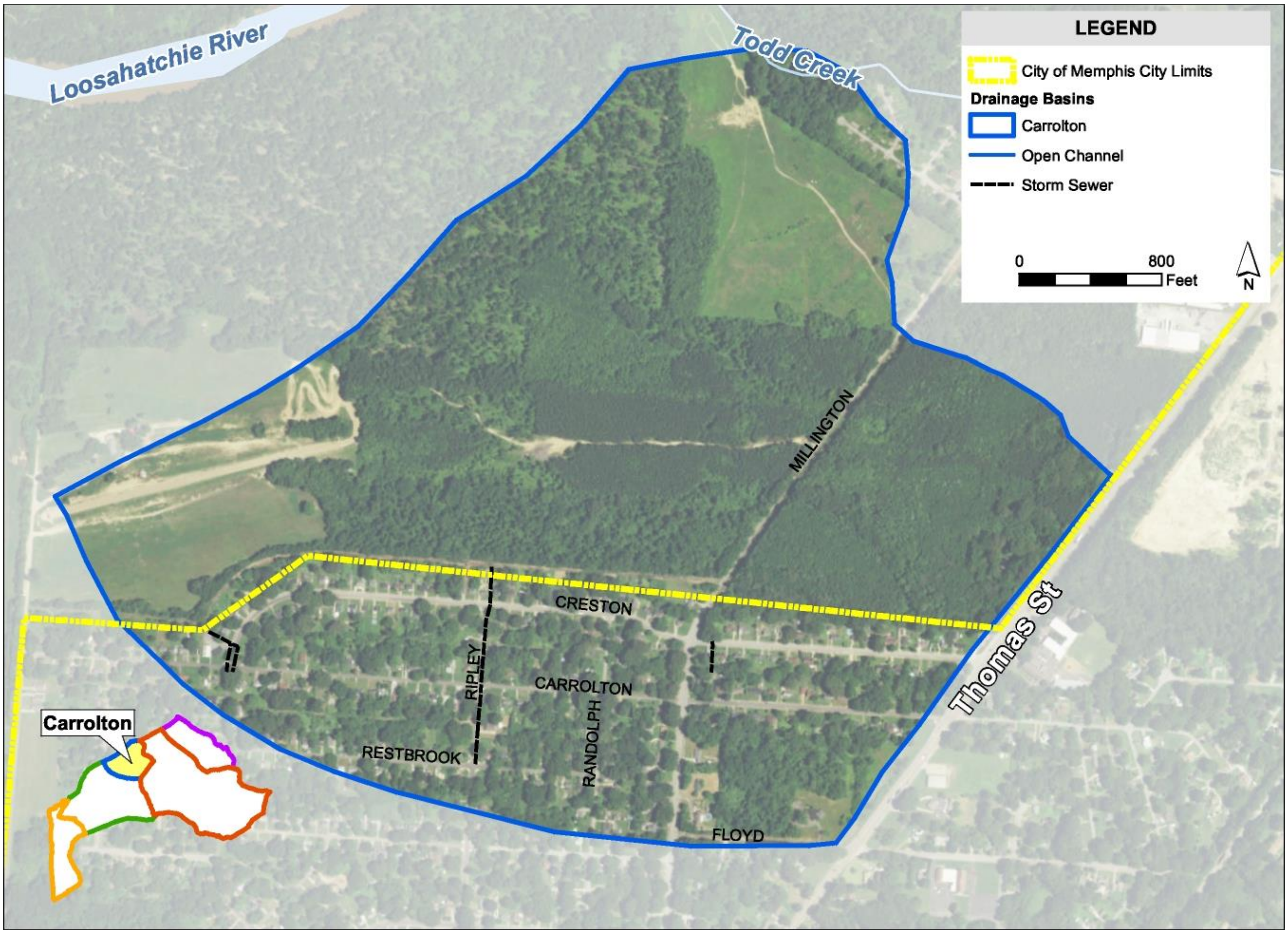
Figure 1.9: WMPS Watershed



1.1.4 Carrolton Watershed

The Carrolton Watershed, as shown in Figure 1.10, is the smallest and least developed of the five watersheds in the Todd Creek Study Area, including 404 acres or approximately 8 percent of the total Study Area. The Carrolton Watershed drain portions of Creston Street, Carrolton Street, Restbrook Street, Early Street, Ripley Street, Randolph Street and Millington Road. Approximately 62 percent of the Carrolton Watershed is undeveloped. The stormwater drainage system in the Carrolton Watershed discharges to the Loosahatchie River via overland flow and/or small drainage ditches.

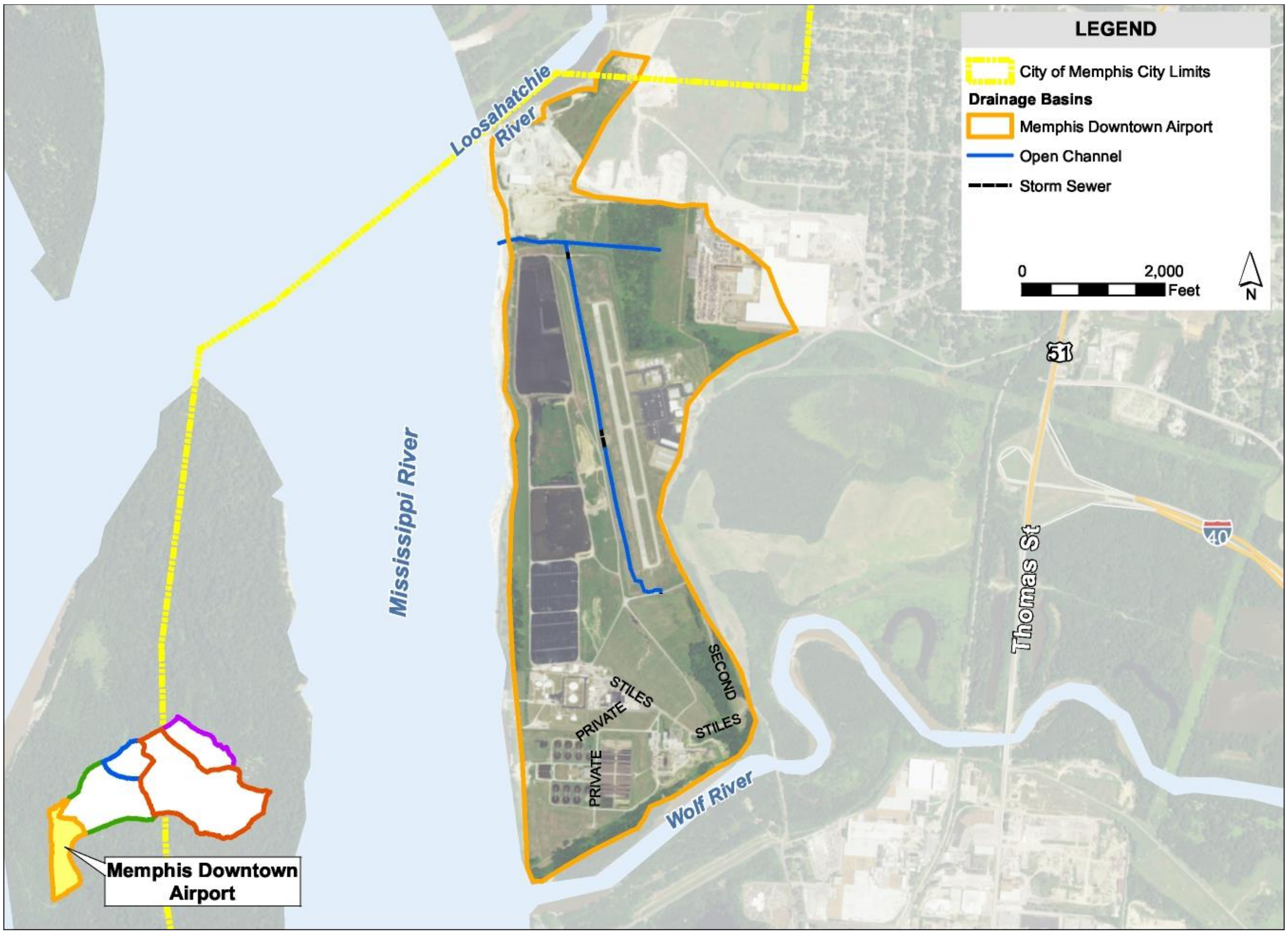
Figure 1.10: Carrolton Watershed



1.1.5 Memphis Downtown Airport Watershed

The Memphis Downtown Airport Watershed, as shown in Figure 1.11, is located in the southwestern most corner of the Todd Creek Study Area, including 566 acres or approximately 11 percent of the total Study Area. The watershed consists primarily of the General DeWitt Spain Airport (FAA ID "M01"), MC Stiles Wastewater Treatment Plant and some commercial/industrial properties along its northern border. There are no residential structures within this watershed. The primary drainage network in this watershed consist of an open channel ditch system that serves the airport, paralleling the runways on the west side of the runways.

Figure 1.11: Memphis Downtown Airport Watershed



2.0 PUBLIC OUTREACH

At the project outset, Tetra Tech and All World Project Management identified key stakeholders such as the Frayser Neighborhood Council, school and church leaders, and other community groups to encourage attendance at public meetings. A stakeholder coordination meeting was held on June 12, 2014 at Excline's Pizza on North Watkins Street. The Tetra Tech team then conducted two public meetings within the Todd Creek Study Area: on July 22, 2014 at Innovation Church; and July 24, 2014 at Martin Luther King Preparatory Academy (former Frayser High School).

Information presented at the public meetings consisted of background information about the City of Memphis Stormwater Master Planning Program, the Todd Creek Study Area extents and scope of work, project goals, schedule update, and general examples of potential improvement projects. Especially important to the public meetings was giving citizens a forum to discuss observed drainage problems with the project team, and an opportunity to record input for use during the study. Tetra Tech provided a method of public comment via hard copy forms, email address, and US mail. Public comments received during the public outreach phase are included in Appendix A. Feedback from local citizens improved the evaluation, identification, and prioritization of project opportunities during the study.

Upon review of the comments received, the following areas or locations were identified as having some type of flooding concern (note: this does not include prior complaints submitted to the City):

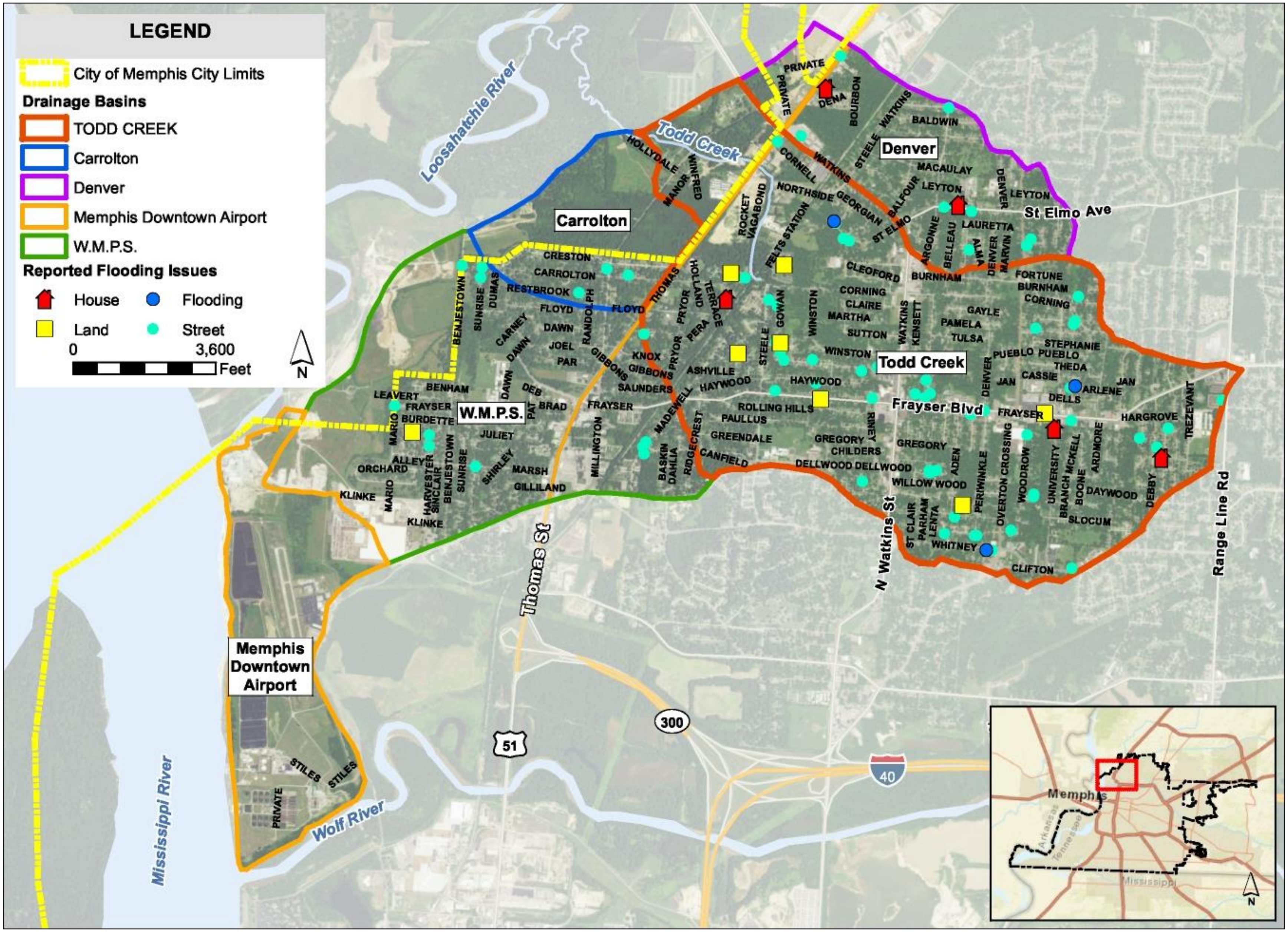
- Along U.S. Highway 51
- Frayser Boulevard at Watkins Street
- 1647 Dellwood Avenue at Steel Street
- Residential flooding at Gowin Street
- 1538 Rolling Hills Drive at Student Street
- Residential flooding at Tillens at Wellons Avenue
- 2360 Clear Park Drive
- 1739 Haywood Avenue

In addition to the flooding complaints received during the public outreach process, the City provided a GIS data file with prior flooding complaints that the received from November 2003 through December 2014, and summarized in Table 2.1 below. The results of the complaint data were used to develop a "heat map" for the Study Area highlighting the density of complaints received, as shown in Figure 2.1 below.

Table 2.2.1: Todd Creek Study Area Flooding Complaints

Drainage Basin	House	Land	Street	Unidentified	Total
Todd Creek	3	7	52	3	65
Denver	2	0	11	0	13
WMPS	0	1	10	0	11
Carrolton	0	0	5	0	5
Downtown Airport	0	0	0	0	0
Total	**5**	**8**	**78**	**3**	**94**

Figure 2.1: Flooding Complaint "Heat Map"



3.0 MODEL DEVELOPMENT

The Stormwater Master Planning Program determined that InfoSWMM® would be the modeling platform used in the study. InfoSWMM is primarily a graphical user interface (GUI) for the US-EPA SWMM model and is well-suited for large scale stormwater planning activities. The US-EPA SWMM model includes an explicit solution of the St Venant equations of motion. US-EPA SWMM is widely accepted and is capable of accurately modeling a wide range of hydraulic conditions including: gravity closed pipe or open channel flow, surcharged systems, branched and looped networks, pump stations and force mains, diversion structures, pond/basin storage and flow reversals.

The hydraulic model was developed to include storm sewer pipes and other closed conduits that are 24-inch diameter and larger. The City provided data for model calibration including stream level (depth of flow) and precipitation values. The model is intended to provide a basis for analyzing the existing storm sewer system, identify hydraulic contraction locations and capacity limitations, and identify improvement opportunities to reduce flooding at or near homes and businesses in the Todd Creek Study Area.

3.1 FIELD DATA COLLECTION

Prior to the implementing the Stormwater Master Planning Program, the City had already begun GIS mapping of the existing public stormwater system. However this was limited to digitization from record drawings, without any field verification; and the digitization had only been completed for some areas of the City. In the Todd Creek Study Area, the digitized GIS mapping (per record drawings) was available for many but not all components of the system. The City also provided a GIS database for flooding-related service requests received from November 2003 through December 2014, including location and type of flooding (e.g. home, street, yard or general flooding). Historical stormwater flow data was not available for model calibration purposes.

To increase model accuracy and focus capital expenses to those areas most in need, the City included a significant field data collection effort in each of the master planning projects. The Tetra Tech team performed a field survey to accurately locate each storm drainage structure connecting to a 24-inch diameter or larger pipe; and to record size/connectivity of storm sewers and cross-sectional data at open channels and culverts/bridges. In conjunction with the University of Memphis, the City separately collected stream level and precipitation data to provide information to support the model calibration.

3.1.1 Survey Data

The Todd Creek Study Area is approximately 5,136 acres and required a major surveying effort to accurately map the main tributary drainage network. For the purposes of this study, the main tributary drainage network is defined as the portion of the existing drainage system from the basin outfalls up to and including the most upstream pipes 24-inches in diameter or greater. Generally, the survey scope included the following key items:

- 600 drainage structures (manholes, inlets, and non-culvert headwalls) connecting to storm sewer pipes that are 24-inches in diameter or larger, and other closed conduits of equivalent or larger size. To avoid system gaps, structures connecting to pipes smaller than 24-inch diameter were also surveyed when downstream of other surveyed features.
- 35 road crossings (bridges and pipe or box culverts), each including the bridge/culvert geometry, road centerline profile, and channel cross-sections immediately upstream and downstream of the crossing.
- 284 approved cross-sections along 12 miles of open channel at approximately 500-foot maximum intervals, and at significant changes in horizontal/vertical alignment or cross-sectional geometry.

- Geo-referenced photos at open-to-closed flow transitions including road crossings, storm sewer outfalls and other features of interest. Common drainage structures such as manholes and inlets were typically not photographed, in accordance with City-provided criteria.
- 95 finished floor elevations at strategic locations along the study reaches.

In order to begin the GIS mapping and stormwater modeling effort as soon as possible, the Study Area was sub-divided into two areas which were surveyed separately by THY, Inc. and Geodesy Professional Services, LLC. The field surveying began in May 2014 and was substantially completed in September 2014. After the existing conditions modeling was substantially complete, the finished floor elevations were selected for surveying in April 2015 and the surveying was completed in May 2015.

3.1.2 Level & Precipitation Metering

The City of Memphis implemented the 2014 Metering Program within the Todd Creek Study Area to collect data suitable for model calibration purposes. The Metering Program included two level meters and two rain gauges which were installed from June 2014 through mid-October, 2014. The City engaged the University of Memphis Ground Water Institute (GWI) to implement the Metering Program. The University was contracted to install and maintain level meters and rain gauges throughout the duration of the program, download data at regular intervals, provide quality assurance/quality control (QA/QC) activities and transmit the data to the Project Team. Table 3.1 presents a summary of the level meter and rain gauge locations, and period of record at each location.

The two level meters, which metered water level exclusively, were installed at the following locations (see Figure 3.1):

- The Steele Street level meter was installed along Denver Branch, where it is a concrete-lined channel, at the upstream end of a culvert crossing at Steel Street . The concrete-lined channel at the meter has a bottom width of nine feet and vertical walls. The channel bottom has a slight slope towards the center forming a shallow triangular section, approximately six inches below the toe of the vertical walls. The Steele Street meter represents approximately 323 tributary acres of the total 565 acres in the Denver drainage system.
- The Frayser School Drive level meter was installed along Todd Creek, where it is a concrete-lined channel, at the upstream end of a bridge crossing at Frayser Drive. The concrete-lined channel at the meter has a bottom width of 31 feet and vertical walls. The channel bottom has a slight slope towards the center forming a shallow triangular section, approximately six inches below the toe of the vertical walls. The Frayser School Drive meter represents approximately 1,396 tributary acres of the total 2,408 acres in the Todd Creek drainage system.

Figures 3.2 and 3.3 show photographs of the level meters installed at Steel Street and Frayser School Drive, respectively.

The two rain gauges were installed at the following locations (see Figure 3.1):

- The MC Stiles WWTP rain gauge was installed at MC Stiles Wastewater Treatment Plant (WWTP) located at 2303 N. Second Street. Rainfall was collected from June 23, 2014 through October 9, 2014. Twelve rainfall events were recorded throughout the monitoring period. An event was identified as recorded rainfall totalling 0.1 inches or greater with an inter-event duration criteria of six hours. The largest event recorded at this rain gauge was 6.897 inches in 27.5 hours on June 28, 2014.
- The Police Academy rain gauge was installed at the Police Training Academy located at 4371 OK Robertson Road. Twelve rainfall events were recorded throughout the monitoring period. An event was identified as recorded rainfall equalling 0.1 inches or greater with an inter-event duration criteria of 6 hours. The largest event recorded at this rain gauge was 6.133 inches in 9 hours on September 11, 2014.

Table 3.1: Level and Precipitation Metering				
Meter/Gauge ID	**Location**	**Latitude / Longitude**	**Tributary Area, ac**	**Metering Dates**
Steele Street Level Meter	Upstream end of Steele Street crossing of Denver Branch. Two blocks north of Watkins Street.	35.219489° N 90.008908° W	323	6/24/2014 to 10/09/2014
Frayser School Dr Level Meter	Upstream end of the Frayser School Drive crossing of Todd Creek. Two blocks north of Frayser Boulevard.	35.234351° N 90.007135° W	1,396	6/24/2014 to 10/09/2014
MC Stiles WWTP Rain Gauge	MC Stiles WWTP at 2303 N. Second Street.	35.186194° N 90.054736° W	N/A	6/23/2014 to 10/09/2014
Police Academy Rain Gauge	Police Training Academy at 4371 OK Robertson Road.	35.245676° N 90.013431° W	N/A	6/23/2014 to 10/09/2014

Figure 3.1: Todd Creek Study Area Metering Locations

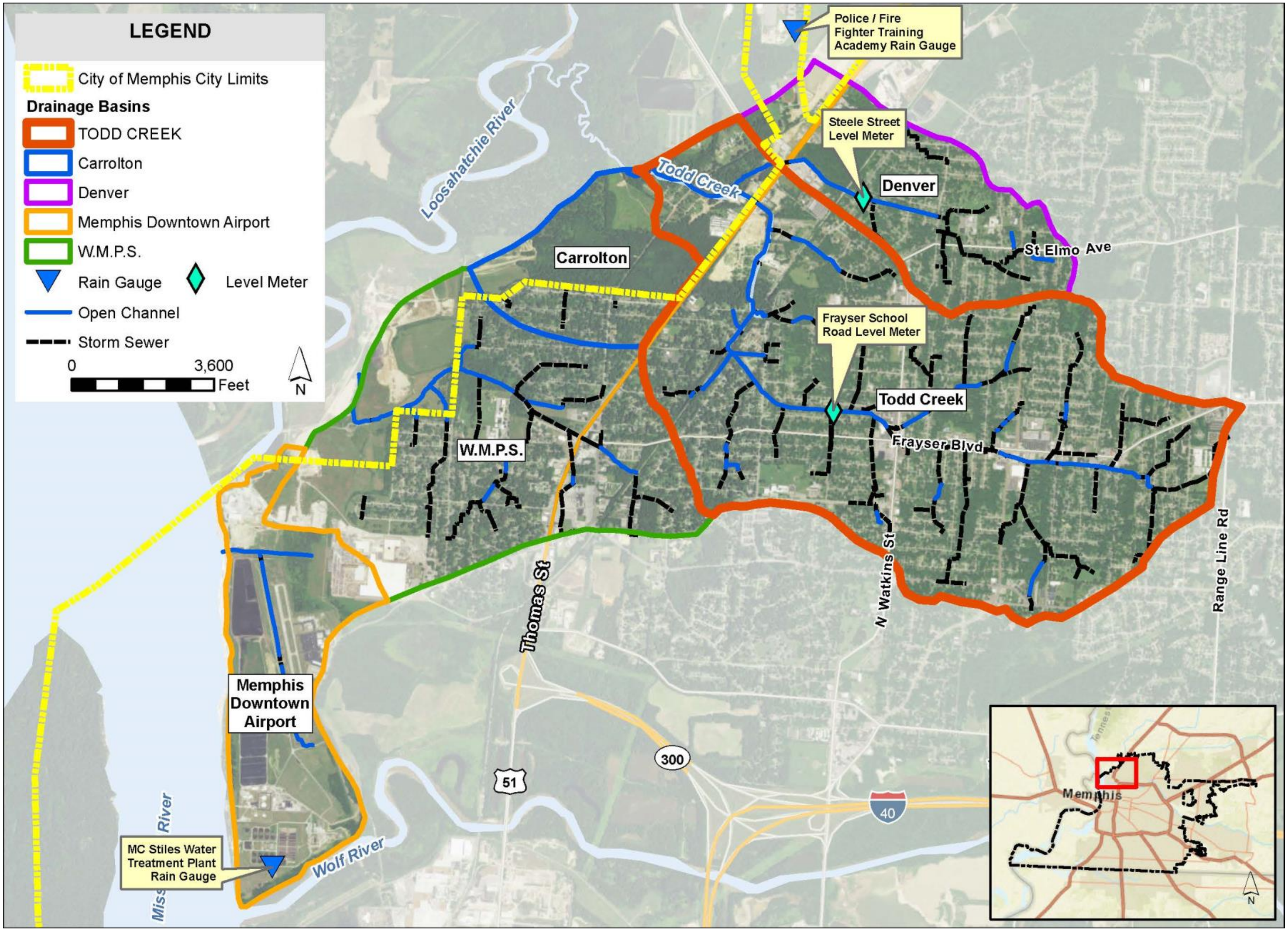


Figure 3.2: Location for Level Meter at Steele Street

Figure 3.3: Location for Level Meter at Frayser School Drive

3.1.3 GIS Development

A significant deliverable for this project is the development of a highly accurate GIS database of the storm drainage system. The field survey data was used as the basis to develop the GIS database. Field survey data was collected in both CAD and GIS formats. Data received in CAD format was brought into the GIS environment for quality checks. If any questions or omissions were found during the initial review, the surveyor was contacted to update or collect the missing information. Once the data was confirmed; the line, point and attribute data was imported in to a GIS database schema created to support the project needs. For data that was received in tabular format, the data was loaded directly to a temporary GIS point feature class in the proper projection.

The points were then classified for the different feature types (i.e. manholes, catch basins, culverts, open channels or closed pipes). This feature class was used to isolate any omissions or problems with the data. After any issues were resolved the data and attributes were loaded to the GIS point feature classes with the corresponding attribute information. The line work was then digitized and populated with attributes using the features found in the point data. Using this method, the complete storm conveyance framework was able to be established and located with a high degree of accuracy.

3.2 HYDRAULIC MODEL DEVELOPMENT

The GIS database was used as the starting point for the hydraulic model network. The GIS data was directly imported into the InfoSWMM modeling platform, including points for manholes or transitions in conveyance network and lines for pipes, open channels, culverts and bridges. The survey efforts obtained the majority of required data for each model element. Missing attributes were developed from as-built records or subsequent field investigations.

The Todd Creek hydraulic model includes 52,174 lineal feet of open channel and 108,170 lineal feet of storm sewer drainage components. The model also includes 80,056 lineal feet of overland flow paths to convey flooded water to downstream locations where adequate capacity exists. The model includes 11 outfalls representing the downstream boundary or terminal node of each drainage system. Normal flow depth of the connecting conduit is used as the boundary condition to determine the outfall stage for each drainage system. Table 3.2 provides summary quantities of conveyance components included in the hydraulic model. Refer to Figure 3.1 in Section 3.1.2 for a plan view schematic of the modeled stormwater conveyance network for the Todd Creek Study Area.

	Table 3.2: Summary of Hydraulic Conveyance Network			
Drainage Basin	**Storm Sewer, Feet**	**Open Channel, Feet**	**Overland Flow, Feet**	**# of Culverts or Bridges**
Todd Creek	67,638	27,607	495	21
Denver	12,015	5,988	1,573	5
WMPS	26,239	11,466	11,441	2
Carrolton	1,784	--	47,460	--
Downtown Airport	494	7,113	19,088	3
Total	108,170	52,174	80,056	31

The Todd Creek watershed is the largest portion of the overall study area with over 59 percent of the stormwater conveyance network. The Todd Creek watershed hydraulic network is comprised of 95,245 lineal feet of open channel and storm sewer with 21 culverts or bridges. The WMPS is the second largest system with over 23 percent of the modeled drainage system and two bridges/culverts. The Denver, Memphis Downtown Airport and Carrolton areas comprise approximately 11 percent, five percent and one percent of the total modeled drainage system, respectively. The Memphis Downtown Airport system primarily includes the open channel drainage system that parallels the airport runways.

3.2.1 Representation of Open Channels

Typical of many urban stormwater drainage systems, the Todd Creek Study Area includes a combination of open channel and closed pipe systems. The open channel portions of the system were represented with cross-sectional data obtained though the field survey effort and existing County topographic datasets.

The field survey effort focused on collecting data within the channels banks. The Shelby County Light Detection and Ranging (LIDAR) data was used to characterize out-of-bank and floodplain areas. The field surveyed cross-sections were merged with the County LIDAR data to produce a single three-dimensional (3D) representation of the entire watershed including channel and floodplain areas. Model cross-sections were cut from the 3D surface at each open channel section. Development of a single 3D surface of the entire watershed increased the efficiency of model development, allowing for simple recreation or extension of cross-section limits at any phase of the project. This 3D surface was also used to delineate floodplain limits for design storm simulations of existing and proposed conditions.

Photographs were used to characterize the channel and floodplain Manning "n" values for each reach. Typical Manning "n" values were obtained from the City of Memphis/Shelby County Stormwater Management Manual.

3.2.2 Representation of Bridges and Culverts

The Todd Creek system includes many culverts at road crossings of the open channel system. InfoSWMM includes routines that mimic culvert hydraulic calculations as presented in the United States Federal Highway Administration (FHWA) Hydraulic Design Series Number 5: Hydraulic Design of Highway Culverts (HDS5). Each culvert in the Todd Creek system was evaluated for size, material and headwall configuration to select a FHWA Culvert Codes to best approximate culvert hydraulics. Culvert specific entrance and exit minor loss coefficients were applied based on the inlet/outlet configuration.

The Todd Creek system includes eight bridge spans at road or railroad crossings of the open channel system. InfoSWMM uses a relatively new curve type to construct a custom shape for closed conduits that cannot be reasonably defined using the standard shapes typically used for culverts. The curve represents depth to full depth ratio on the Y-axis, and width to full depth ratio on the X-axis. This specifies how the width of the cross section varies with height, where both width and height are scaled relative to the sections maximum depth. This allows the roadway bridge deck low chord to represent the top of a closed conduit over a natural or concrete channel. Bridge spans are then modeled as a closed conduit culvert as described above.

At each culvert and bridge span location in the model, parallel conduits were provided to represent the culvert and the overland flow in the event that the culvert capacity was exceeded. The overland flow path was visually determined from the topographic data. A new conduit was created for the overland flow and a cross-section was cut to represent the shape of the flow path.

3.3 HYDROLOGIC MODEL DEVELOPMENT

The City of Memphis determined that the USEPA SWMM Runoff Method would be the hydrologic model for use in the Stormwater Program. The SWMM Runoff Method is well-suited for use in urban and rural stormwater runoff modeling and drainage system master planning.

The SWMM Runoff Method uses the Kinematic Wave Method to calculate hydrologic response to precipitation. The following hydrologic parameters determine the magnitude and shape of the runoff hydrograph produced by the SWMM Runoff Method for a given storm event:

- Subcatchment Area, Width, and Ground Slope
- Directly Connected Impervious Area (DCIA)
- Percent of Impervious Area with Zero Depression Storage
- Soil Infiltration
- Evaporation

The surface runoff rate for a subcatchment is calculated by using Manning's equation solved for a rectangular plane with negligibly small side heights:

$$Q = \frac{1.486}{n} * W * (d - d_s)^{\frac{5}{3}} * S^{\frac{1}{2}}$$

Where,

Q = surface runoff rate (cfs),

n = Manning's roughness coefficient,

W = width of the plane (ft),

d = depth (ft),

d_s = depression storage (ft), and

S = average slope of the plane (ft/ft)

Subcatchments for the Todd Creek Study Area were manually delineated using the GIS shapefiles of the storm sewer system provided by the City and a digital elevation model (DEM) created from Shelby County LIDAR data. The DEM was processed to create a stream network file to assist in making manual delineation decisions. The delineations were performed based on the following general criteria:

- Provide an input to each of the upstream terminal ends of the system
- Subdivide the tributary areas at locations where open channel transitions to a closed pipe system
- Subdivide large catchments to prevent "overloading" of a single receiving node.

The final delineation resulted in 304 subcatchments with an average size of 16.9 acres. The subcatchments range in size from 0.2 to 112 acres. Subcatchments are generally larger in less urban areas that area not drained by the storm sewer system and smaller in the urban and dense residential areas which have an increased number of storm sewer inlets.

The overland flow paths were calculated for each subcatchment. The flow paths were then used to estimate catchment width and slope. Catchment widths were calculated by dividing the subcatchment area by the length of the overland flow path. Slopes were calculated as a difference between upstream and downstream elevation along the overland flow path divided by its length. The width and slope parameter are most sensitive to peak flow rate and timing. These parameters were used to provide necessary adjustments to the resulting runoff during the calibration process. The preliminary slope and width values were adjusted uniformly within an area to achieve the desired calibration results.

3.3.1 Directly Connected Impervious Area

The Multi-Resolution Land Characteristics (MRLC) National Land Cover Database 2011 (NLCD) imperviousness coverage was used to calculate the initial imperviousness value for each subcatchment. The catchment delineations and NLCD coverages were overlaid to calculate the area weighted average percent imperviousness for each subcatchment. The percent impervious value (i.e. DCIA) is most sensitive to the total volume of runoff for a catchment. The percent impervious value was used to adjust the total volume of the predicted runoff to best match measured values during the calibration process. The percent impervious values were adjusted uniformly within an area until the desired calibration results were obtained. Figure 3.4 presents the relative density of impervious areas in the Todd Creek Study Area. Table 3.3 summarizes the final calibrated and validated average percent impervious values used from the model input data file.

Figure 3.4: Todd Creek Study – Impervious Coverage

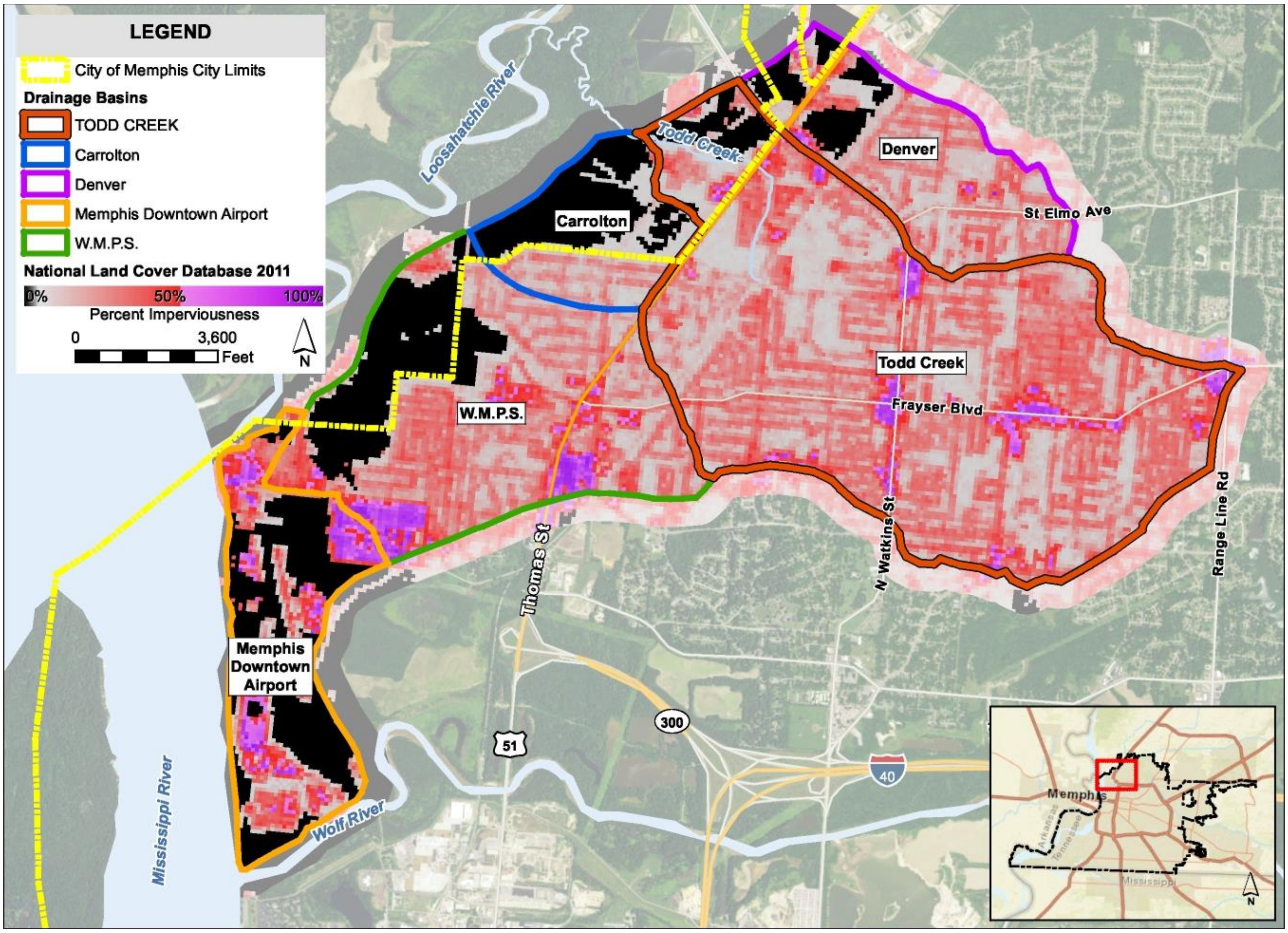


Table 3.3: Todd Creek Study Area – Map Based Percent Impervious		
Drainage Basin	**Area, ac[1]**	**Percent Impervious[2]**
Todd Creek	2,408	33.94
Denver	609	26.43
WMPS	1,149	29.25
Carrolton	404	7.24
Memphis Downtown Airport	566	26.13
Total	5,136	29.25

1: Some of the areas within the Todd Creek Study Area drain via sheet flow directly to the Loosahatchie River and are not included in the hydraulic model results.
2: Impervious data from NLCD 2011 GIS overlay.

3.3.2 Soil Infiltration Parameters

Infiltration is the movement of water through the soil surface and into the soil and groundwater table. It is related to the moisture conditions in the surface soil zone. The infiltration capacity of a soil at any given time is the maximum rate at which water can enter the soil. Typically, on well-drained porous soils, recovery of infiltration capacity is quite rapid. However, for heavier clay soils the recovery rate is likely to be slower.

The City of Memphis standardized on the USEPA SWMM Green-Ampt method. The Green-Ampt equation is a physically based model that can provide a good description of the infiltration process using readily available soil classification data.

The formulation of the Green-Ampt equation is a two-stage model. The first step predicts the volume of water that will infiltrate before the surface becomes saturated. From this point onward, infiltration capacity is then computed directly by the Green-Ampt equation.

$$I = \frac{K_a * S_w * (\theta_a - \theta_i)}{i - K_a}$$

Where,

I = infiltration volume (in^2)

K_a = saturated hydraulic conductivity (in/hr)

S_w = capillary suction head (in)

θ_a= saturated water contents (in)

θ_i = initial water contents (in)

i = rainfall (in/hr)

Three parameters need to be specified for each subcatchment. These parameters are capillary suction head, saturated hydraulic conductivity, and the initial moisture deficit of the soil. These parameters were selected from textbook values based on the percentages of soil types in each catchment. Table 3.4 presents the range of soil parameters used in the Todd Creek model.

Table 3.4: Infiltration Parameters			
Parameter	**Minimum Value**	**Maximum Value**	**Median Value**
Saturated Hydraulic Conductivity	0.028	2.977	0.995
Suction Head	2.625	9.159	5.063
Initial Moisture Deficit	0.076	0.19	0.161

The United State Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) Soil Survey Geographic Database (SSURGO) was used as the basis for classifying soil characteristics of the Todd Creek Study Area. Sub-catchments were overlaid with SSURGO data to determine the underlying soil texture, hydrologic soil group (HSG) and hydraulic conductivity for each delineation. The resulting values represent area weighted averages for each parameter based on the percentage of a particular soil type within each sub-catchment.

Suction head and initial moisture deficit (IMD) values were selected from typical textbook values (Rawls, W.J. et al., 1983, J. Hyd. Engr., 109:1316) based on the percentage of particular soil textures in each sub-catchment. The Manning's roughness coefficient was set to 0.10 for pervious areas and to 0.015 for impervious areas. The depression storage values ranged from 0.15 inches to 0.415 inches.

3.4 MODEL CALIBRATION AND VALIDATION

Model validation is the process of adjusting both hydrologic (flow development) and hydraulic (flow routing) variables to best match actual measured flow, depth, and rainfall data. The result is a hydrologic and hydraulic model of an existing collection system that best represents flow responses to wet weather conditions and hydraulic grade lines within the drainage system. A properly validated hydrologic and hydraulic model provides a valuable tool for many types of analyses including simple capacity analyses and complex master planning evaluations.

Model validation is a multi-step process that involves calibration of hydrologic parameters and the subsequent verification that the parameters produce the best fit results for wet weather flows. Model calibration is the iterative process to identify the runoff and hydraulic routing parameters that most closely match the flow/rainfall data. An initial set of hydrologic parameters (catchment slope, width and percent impervious) are input to the model and used to develop flow results. The results are compared to flow monitoring data and adjustments are made to the runoff variables (catchment slope, width and percent impervious) to provide a closer fit to the measured data. This process is performed until model predictions best fit the monitoring data.

The verification process requires that some events are set aside during the model calibration process and used at the end to confirm the parameter optimization efforts. The verification process includes two components: (1) presentation of model predictions for the verification events set aside during calibration efforts and (2) for all events, a statistical analysis of modeling and flow monitoring data to confirm the model calibration results provide the best fit to measured flows and rainfall data. Refer to Section 3.4.2 for a discussion of model parameters adjusted during the calibration process and overall ability of the model to predicted measured conditions.

3.4.1 Model Validation Data

The Todd Creek Study Area model was validated using data collected at two level meter locations: Todd Creek at Frayser School Drive; and Denver Branch at Steele Street. The in-stream data included level measurements from June 2014 to October 2014. The City also collected precipitation data at the MC Stiles Wastewater Treatment Plant and at the Memphis Police Training Academy.

The level data was directly used to validate peak level predictions to measured data. The time-series level data was converted to flow rate with a depth vs flow rating curve.

The flow and level data were evaluated for responses to wet weather conditions. A total of 14 wet weather events were identified. Table 3.5 provides a summary of the events including precipitation, level and flow statistics. The two largest precipitation events occurred on June 28, 2014 (5.72-6.90 inches of rainfall) and September 11, 2014 (5.08-6.13 inches of rainfall). These events resulted in significantly greater peak level, flow and event volumes as compared to other events, predominantly less than one inch of total rainfall.

Of the 14 rainfall events that occurred during the monitoring period, two events were kept aside for the model verification process (e.g. not included in the calibration analyses). The July 8, 2014 and October 2, 2014 events were used for the verification process. The other twelve events were used in the model calibration process. Figure 3.5 and Figure 3.6 present the rating curves developed from the hydraulic models for the Frayser School Drive and Steele Street monitoring locations, respectively. The time-series flow data was used to validate the peak flow and event volumes for rainfall events in the calibration period.

The modeled rating curves result in less capacity than the typical nominal flow rate as calculated using the Manning Equation. Field survey data was used for the Manning curve calculations at each site. Field survey data indicates channel bottom widths of 31 feet and nine feet and slopes of 0.4-percent and 0.32-percent at Frayser School Drive and Steel Street sites, respectively. The drainage system includes many culverts/bridges and lateral flow inputs to the stream that cause the rating curve to diverge from the "normal" flow calculated from the Manning Equation. The Frayser School Drive rating curve shows a break or deflection in the curve at approximately 5 feet of depth. This depth coincides with the top of the culvert at the meter location indicating the culvert is submerged. At this point, the culvert is more restrictive to the flow and the corresponding flow rate increases less with the depth of flow.

Figure 3.5: Frayser School Drive Depth vs Flow Rate Rating Curve

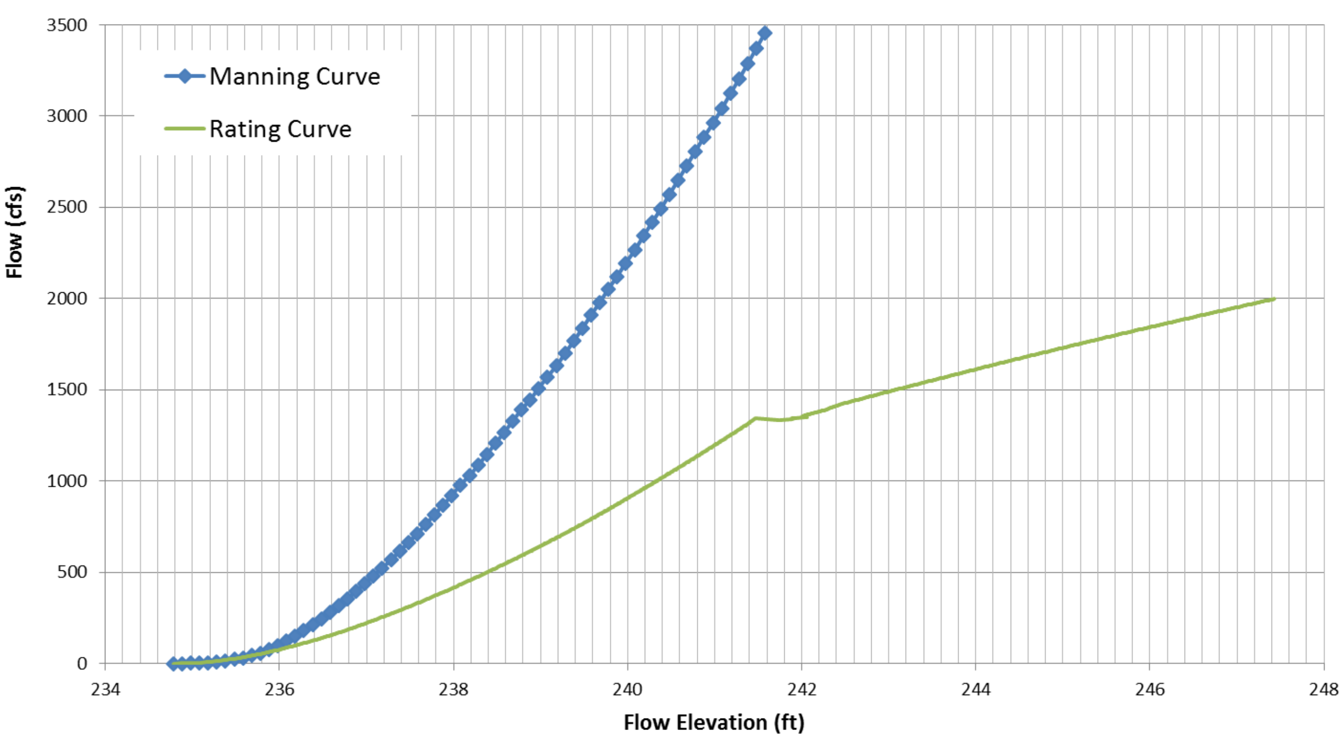


Figure 3.6: Steele Street Depth vs Flow Rate Rating Curve

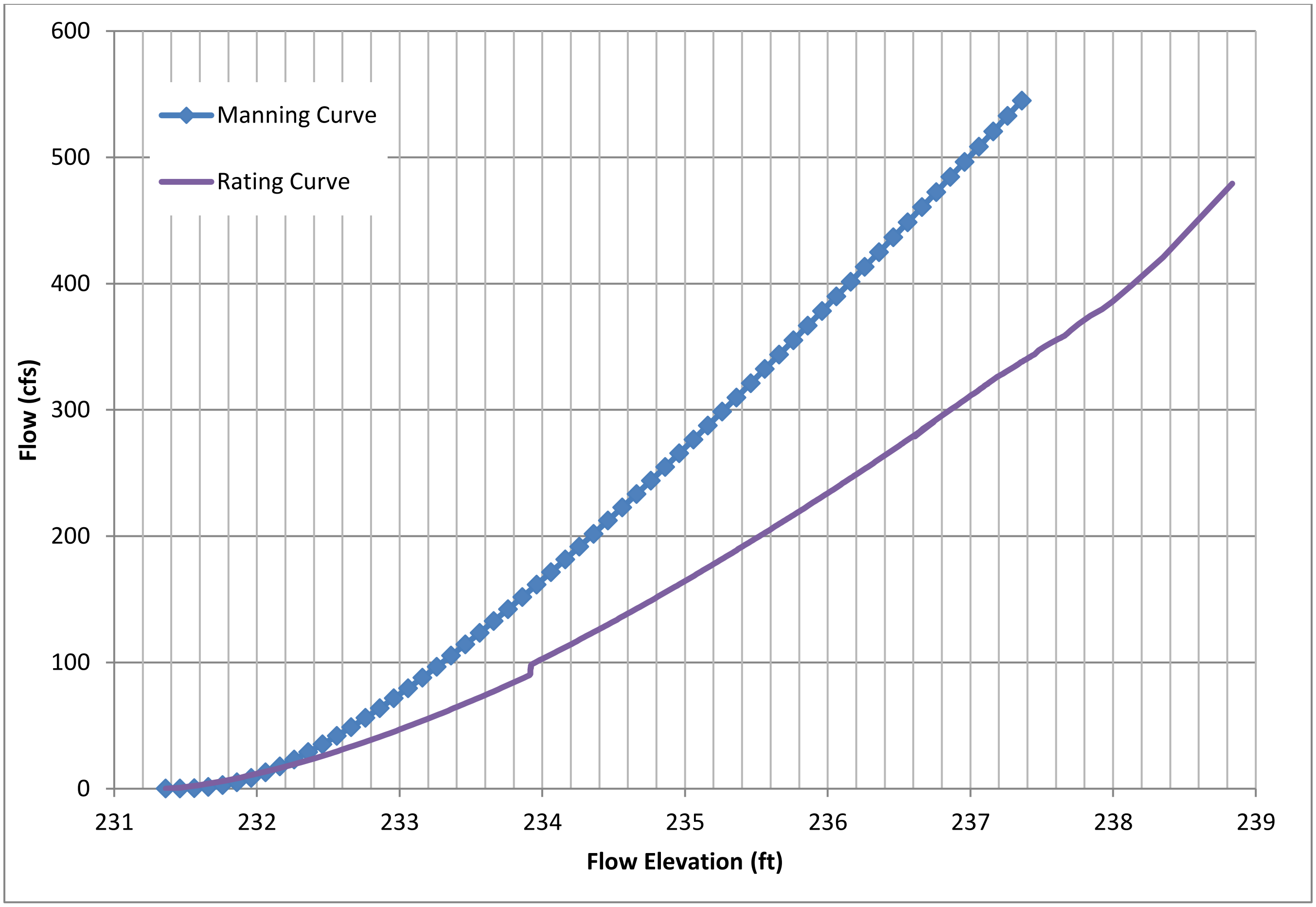


	Table 3.5: Calibration and Verification Events				
Event No.	**Date**	**MC Stiles WWTP**	**Police Academy**	**Steele Street**	**Frayser School Drive**
1	06/24/14[1]	N/A	0.87 IN, 0.35 IN/HR	1.4 FT, 36.8 CFS, 2.4 MG	2.3 FT, 229 CFS, 7.5 MG
2	06/25/14[1]	0.10 IN, 0.09 IN/HR	0.10 IN, 0.06 IN/HR	0.1 FT, N/A CFS, N/A MG	0.3 FT, 5.6 CFS, 0.4 MG
3	06/28/14[1]	6.90 IN, 1.07 IN/HR	5.72 IN, 1.51 IN/HR	4.1 FT, 198 CFS, 21.8 MG	5.2 FT, 898 CFS, 126 MG
4	07/01/14[1]	0.28 IN, 0.09 IN/HR	0.43 IN, 0.30 IN/HR	0.6 FT, 11 CFS, 0.7 MG	0.3 FT, 6.3 CFS, 0.6 MG
5	**07/08/14[2]**	**0.46 IN, 0.24 IN/HR**	**1.69 IN, 1.29 IN/HR**	**2.7 FT, 100 CFS, 4.0 MG**	**2.5 FT, 272 CFS, 12.9 MG**
6	07/14/14[1]	0.26 IN, 0.17 IN/HR	0.37 IN, 0.28 IN/HR	0.7 FT, 14.8 CFS, 0.3 MG	1.3 FT, 85.1 CFS, 2.4 MG
7	07/17/14[1]	0.25 IN, 0.09 IN/HR	0.18 IN, 0.06 IN/HR	0.1 FT, N/A CFS, N/A MG	0.2 FT, 4.2 CFS, 0.6 MG
8	08/07/14[1]	0.22 IN, 0.22 IN/HR	0.10 IN, 0.10 IN/HR	0.1 FT, 1 CFS, 0.13 MG	0.5 FT, 15.7 CFS, 0.6 MG
9	08/08/14[1]	0.09 IN, 0.05 IN/HR	0.14 IN, 0.10 IN/HR	0.1 FT, N/A CFS, N/A MG	0.1 FT, 2.5 CFS, 0.09 MG
10	08/30/14[1]	0.77 IN, 0.19 IN/HR	0.68 IN, 0.14 IN/HR	0.1 FT, 1.8 CFS, 1.2 MG	0.8 FT, 34.1 CFS, 2.3 MG
11	09/11/14[1]	5.08 IN, 2.32 IN/HR	6.13 IN, 2.80 IN/HR	6.4 FT, 378 CFS, 23.3 MG	8.1 FT, 1,480 CFS, 123 MG
12	**10/02/14[2]**	**1.02 IN, 0.72 IN/HR**	**0.91 IN, 0.61 IN/HR**	**1.6 FT, 45.2 CFS, 0.95 MG**	**2.2 FT, 224 CFS, 6.1 MG**
13	10/06/14[1]	0.27 IN, 0.18 IN/HR	0.28 IN, 0.21 IN/HR	0.2 FT, 2.7 CFS, 0.09 MG	0.5 FT, 12.2 CFS, 0.6 MG
14	10/07/14[1]	N/A	0.41 IN, 0.37 IN/HR	0.3 FT, 5.5 CFS, 0.2 MG	0.4 FT, 9.1 CFS, 0.4 MG

1: Model Calibration Event.
2: Model Verification Event.

The Todd Creek Study Area model was validated against four criteria to best match the shape and magnitude of the measured data:

- Peak flow rate
- Total event volume
- Peak event depth of flow
- Visual comparison of model predicted and measured time-series hydrograph data.

The validation process includes a graphical and statistical comparison of the model predictions against metering data. For each event, the values predicted by the model (y-axis) were plotted against the measured data (x-axis). A theoretical 1:1 line (i.e. a 45 degree line) is plotted to divide the chart into two zones. Ideally, the modeled vs measured data points would fall exactly on the 1:1 line indicating a perfect match. Complete datasets rarely fall on the 1:1 line due to uncertainties in rainfall and flow measurement equipment and varying meteorological and antecedent moisture conditions. Data points that fall below the 1:1 line indicate the model is under-predicting the measured data. Conversely, if the data point lies above the 1:1 line, the model is over-predicting the measured data. In practice, the validation process is meant to develop a model that best represents the average conditions for which the model will be applied. A visual comparison was also performed for each validation event to confirm the model predicted hydrograph includes similar shape, timing and responsiveness as measured in the field. An example of the graphical validation results and visual hydrograph comparison are presented in Figure 3.7 and Figure 3.8, respectively. The event numbers are identified in Table 3.5.

Figure 3.7: Frayser School Drive Example Validation Event Volume Plot

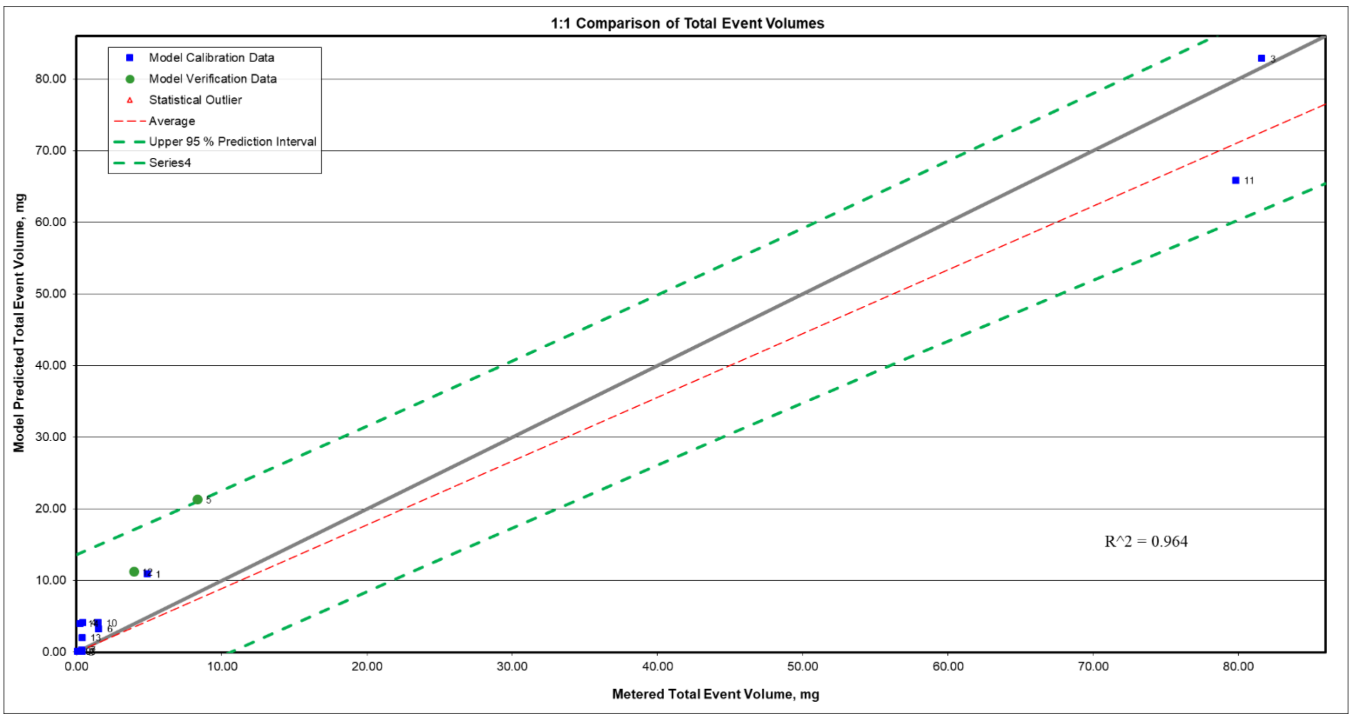
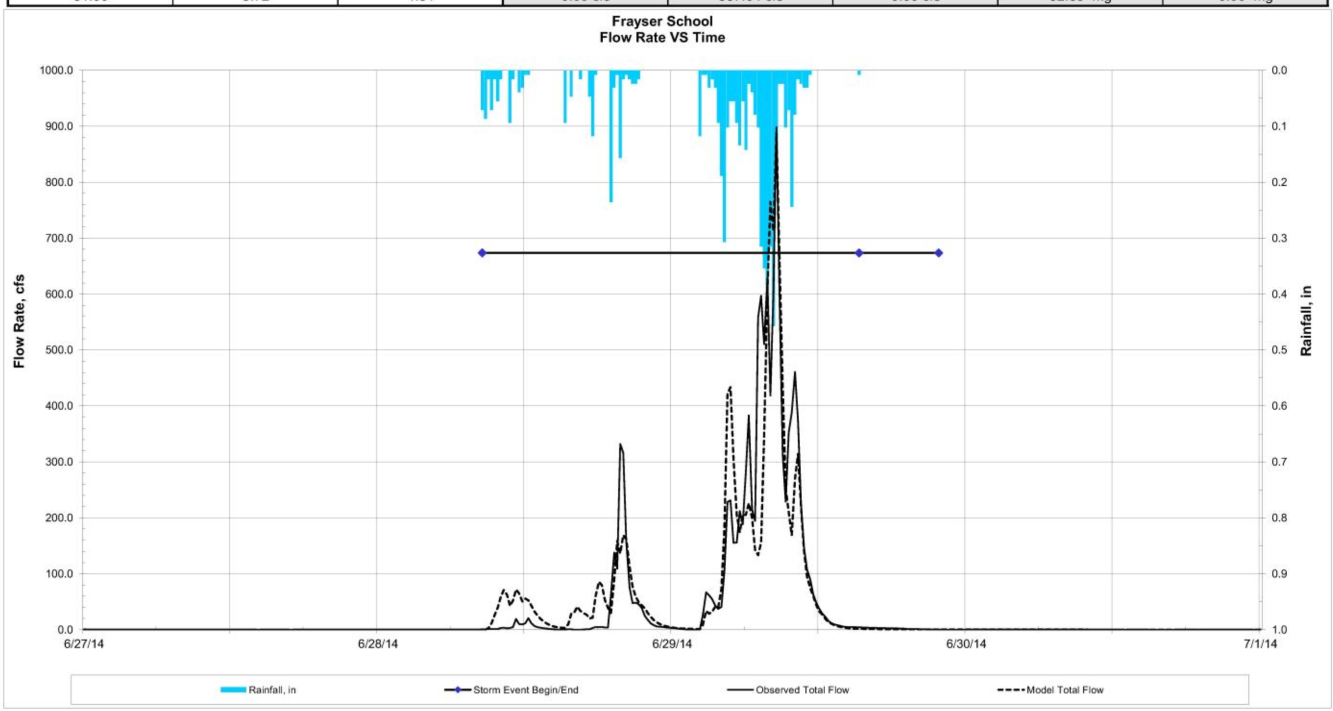


Figure 3.8: Frayser School Drive Example Hydrograph Comparison

Calibration/Meter Location	Area (ac)	Population	Parcels	Storm Begin	Storm End	RDII End
Frayser School	600	35,000	3,500	6/28/14 8:30	6/29/14 15:15	6/29/14 21:45

Observed Average DWF	Observed Peak Flow	Observed Max 24-hour Volume	Observed Peak Wet Weather Flow	Observed Wet Weather Volume	Observed Percent Capture (%)	Observed Peaking Factors	Event Identification Number
0.00 cfs	898.01 cfs	80.58 mg	910.76 cfs	Total Event	Total Event	Peak Flow / Avg DWF	3
0 gal/cap/day	16,583 gal/cap/day	2,302 gal/cap/day	16,818 gal/cap/day	84.09 mg	90.18%	#DIV/0!	Kept Event
0 gal/prop/day	165,828 gal/prop/day	23,024 gal/prop/day	168,182 gal/prop/day	Max 24-hour	Max 24-hour	Peak DWF / Avg DWF	Validation Weight
0 gal/ac/day	967,331 gal/ac/day	134,304 gal/ac/day	981,061 gal/ac/day	83.10 mg	89.12%	#DIV/0!	1

Storm Duration (hr)	Total Rainfall (in)	Peak Rainfall Intensity (in/hr)	Model Predicted Average DWF	Model Predicted Total Peak Flow	Model Predicted Wet Weather Peak Flow	Model Predicted Total Volume	Model Predicted Wet Weather Volume
31.00	5.72	1.51	0.00 cfs	887.04 cfs	0.00 cfs	82.89 mg	0.00 mg



3.4.2 Validation Results

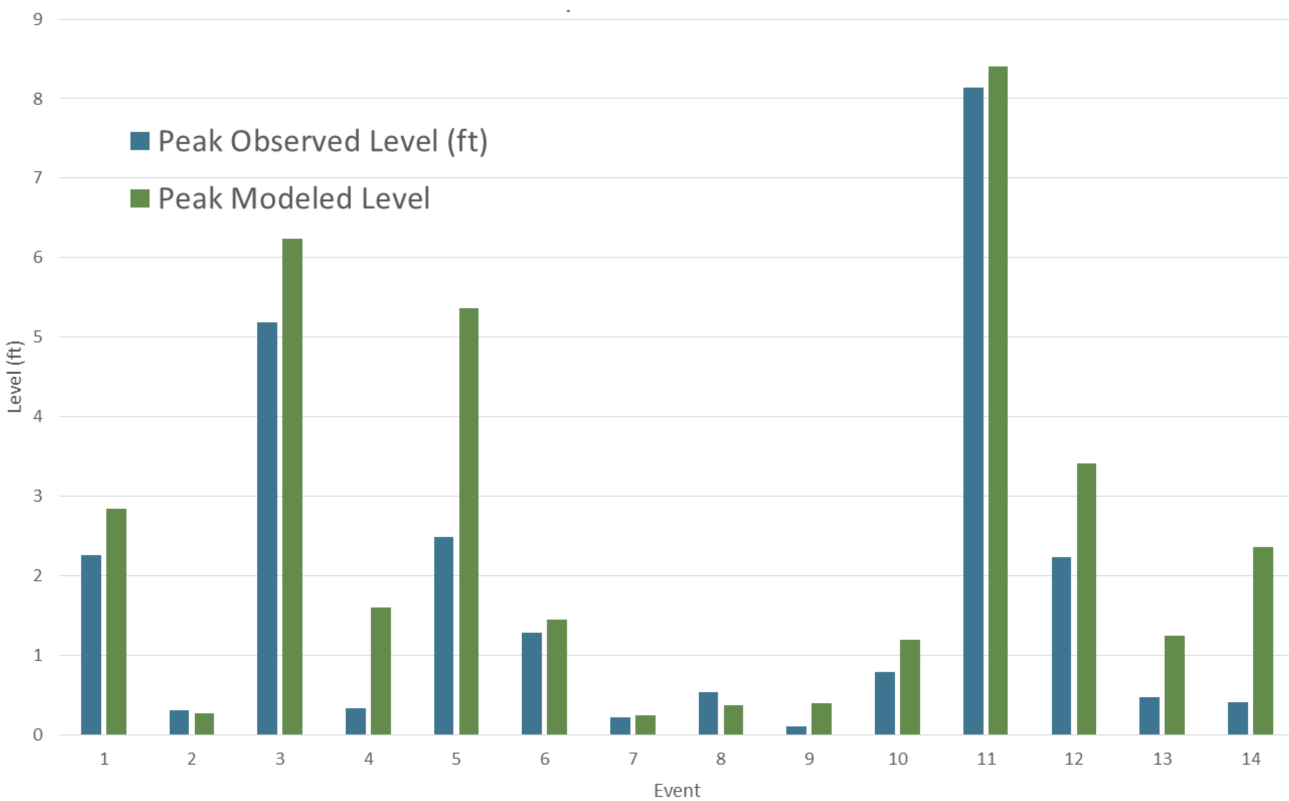
Current industry standards (i.e. WaPUG Modeling guidelines) suggest that model calibration is acceptable when a trendline drawn through the modeled versus metered data has a slope of -10% to +15% for event volume and -15% to +25% for peak flow. In general, the model developed for the Todd Creek Study Area meets these criteria and is considered well calibrated. A summary of the calibration results is presented in Appendix B.

The Frayser School Drive level meter includes approximately 1,396 acres (58%) of the Todd Creek drainage area including the majority of highly developed residential and commercial areas. In general, the model either predicts or slightly over-predicts most events with the one exception; the September 11, 2014 event. The September 11th event is the largest event in the monitoring period. For this event, the model under predicts peak flow rate and volume by approximately 20 percent and 23 percent, respectively. The model slightly over predicts the level for the September 11th event. However, the model accurately predicts the peak flow rate and volume for the June 28th, 2014 event, the second largest event that occurred. The model over-predicts the level for this event.

The Steele Street level meter includes approximately 323 acres (54%) of the Denver Branch drainage area including the majority of developed residential areas. Similar to the Todd Creek calibration, the model either predicts or slightly over-predicts most events. For the September 11th event, the model under predicts peak flow rate by approximately 33 percent and over predicts event volume by 13 percent. For the June 28th event, the model accurately predicts peak flow rate and event volume. The model over-predicts the level for this event.

The subcatchment percent impervious, width and slope are the model parameters adjusted during the calibration process. Generally the subcatchments associated with the Frayser level meter showed an increase of 13% in percent impervious area (as compared to NLCD 2011 based starting point) and varying increases in catchment width and slope. The sub catchments associated with the Steele level meter generally showed a decrease of 5% in percent impervious area (as compared to NLCD 2011 based starting point) and varying adjustments to the catchment width and slope. In general, the model over-predicts for smaller events and is in line with larger events that would be more similar to a design storm event condition. Since the model is intended to be simulated for larger design storm event conditions, the model is considered to be well calibrated.

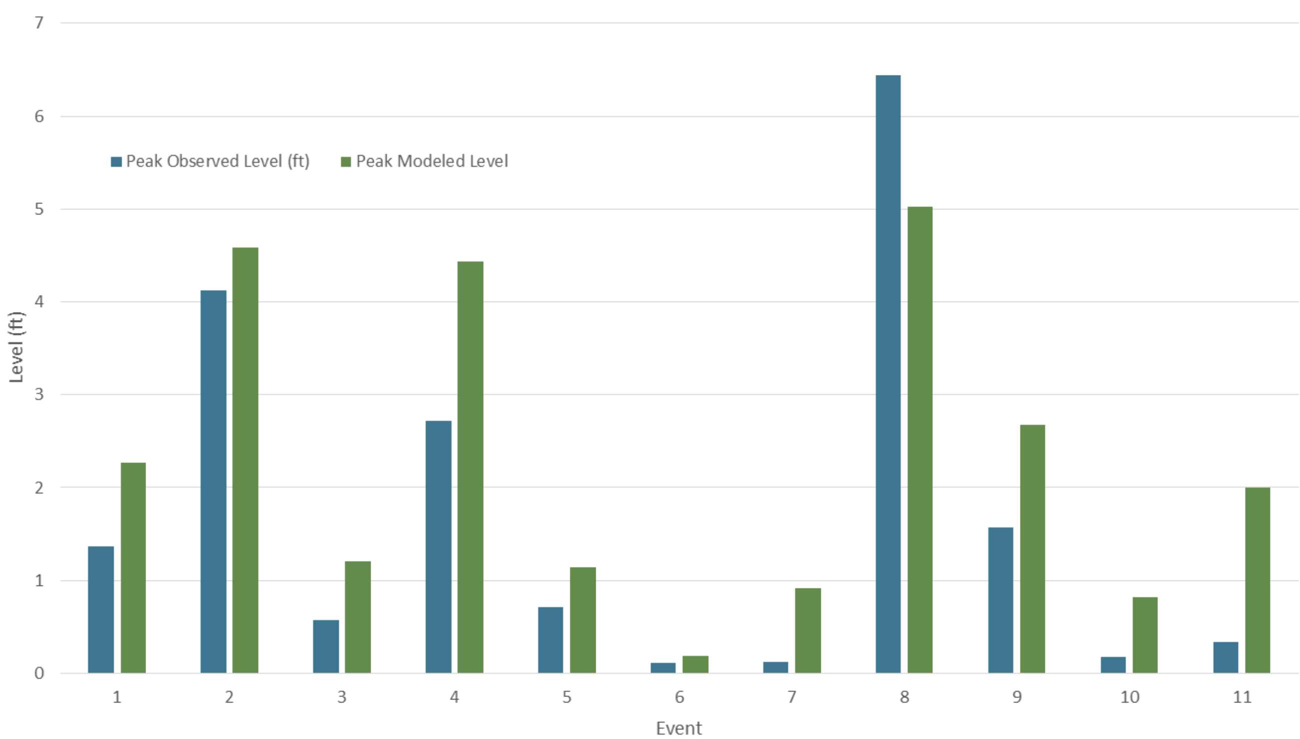
Figure 3.9: Frayser School Drive Calibration – Peak Level Results



Note: Events #5 and #12 were used for model validation. All others used for model calibration purposes.

	Table 3.6: Frayser School Drive Calibration – Peak Level Results				
Event ID	Event Date	Rainfall (in)	Duration (hrs)	Peak Observed Level (ft)	Peak Modeled Level (ft)
1	6/24/14	0.87	27.5	2.3	2.8
2	6/25/14	0.08	4.5	0.3	0.3
3	6/28/14	5.65	37.25	5.2	6.2
4	7/1/14	0.23	9.25	0.3	1.6
5	7/8/14	1.62	9.75	2.5	5.4
6	7/14/14	0.23	5.0	1.3	1.4
7	7/17/14	0.17	14.5	0.2	0.2
8	8/7/14	0.10	2.5	0.5	0.4
9	8/8/14	0.03	2.0	0.1	0.4
10	8/30/14	0.61	28.0	0.8	1.2
11	9/11/14	6.11	12.75	8.1	8.4
12	10/2/14	0.89	11.25	2.2	3.4
13	10/6/14	0.24	3.25	0.5	1.3
14	10/7/14	0.40	8.25	0.4	2.4

Figure 3.10: Steele Street Calibration – Peak Level Results



Note: Events #4 and #9 were used for model validation. All others used for model calibration purposes.

	Table 3.7: Steele Street Calibration – Peak Level Results				
Event ID	Event Date	Rainfall (in)	Duration (hrs)	Peak Observed Level (ft)	Peak Modeled Level (ft)
1	6/24/14	0.87	28.0	1.4	2.3
2	6/28/14	5.65	32.5	4.1	4.6
3	7/1/14	0.23	12.5	0.6	1.2
4	7/8/14	1.62	7.5	2.7	4.4
5	7/14/14	0.17	1.5	0.7	1.1
6	8/7/14	0.10	3.25	0.1	0.2
7	8/30/14	0.61	29.5	0.1	0.9
8	9/11/14	6.06	7.75	6.4	5.0
9	10/2/14	0.88	9.0	1.6	2.7
10	10/6/14	0.24	1.75	0.2	0.8
11	10/7/14	0.40	7.75	0.3	2.0

4.0 EXISTING CONDITIONS

4.1 APPROACH

The Todd Creek Study Area model was calibrated and validated to the recorded flow monitoring data. Design storm events were used to evaluate the system's capacity. The events that were chosen included the 2-, 5-, 10-, 25-, 50-, and 100-year, 24-hour design storms. These storm events were developed using the published Intensity-Duration-Frequency (IDF) charts specifically assigned for the City and Shelby County to create design storm hyetographs. The IDF curves for the City are based on data published in the "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 2 (G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M. Yekta, and D. Riley). Hyetographs for the City and Shelby County for a 24-hour storm duration were developed using the National Resource Conservation Service (NRCS) Type II 24 Hour Storm Distribution. For example, a dimensionless unit hyetograph for a 24-hour storm event is shown in Figure 4.1. Table 4.1 indicates the total depth for each design event. The design storm event cumulative distribution curve I is pretested in Figure 4.1. Figure 4.2 and Figure 4.3 present the design storm event hyetographs applied for this project.

Table 4.1: Design Storm Rainfall Depth	
Design Event	**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

Figure 4.1: NRCS Type II, 24-Hour Design Storm Cumulative Distribution Curve

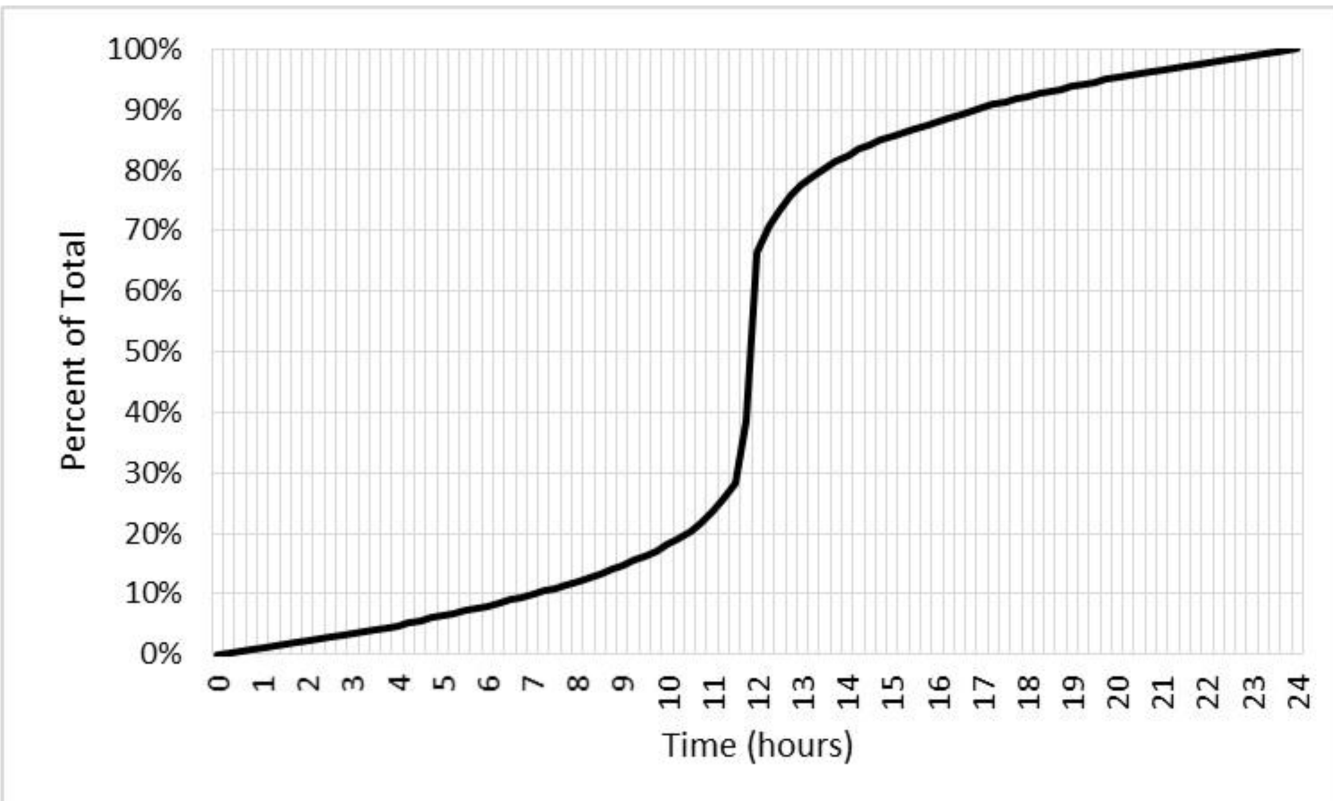


Figure 4.2: NRCS Type II Design Storm Event Hyetograph – Hours 10-14

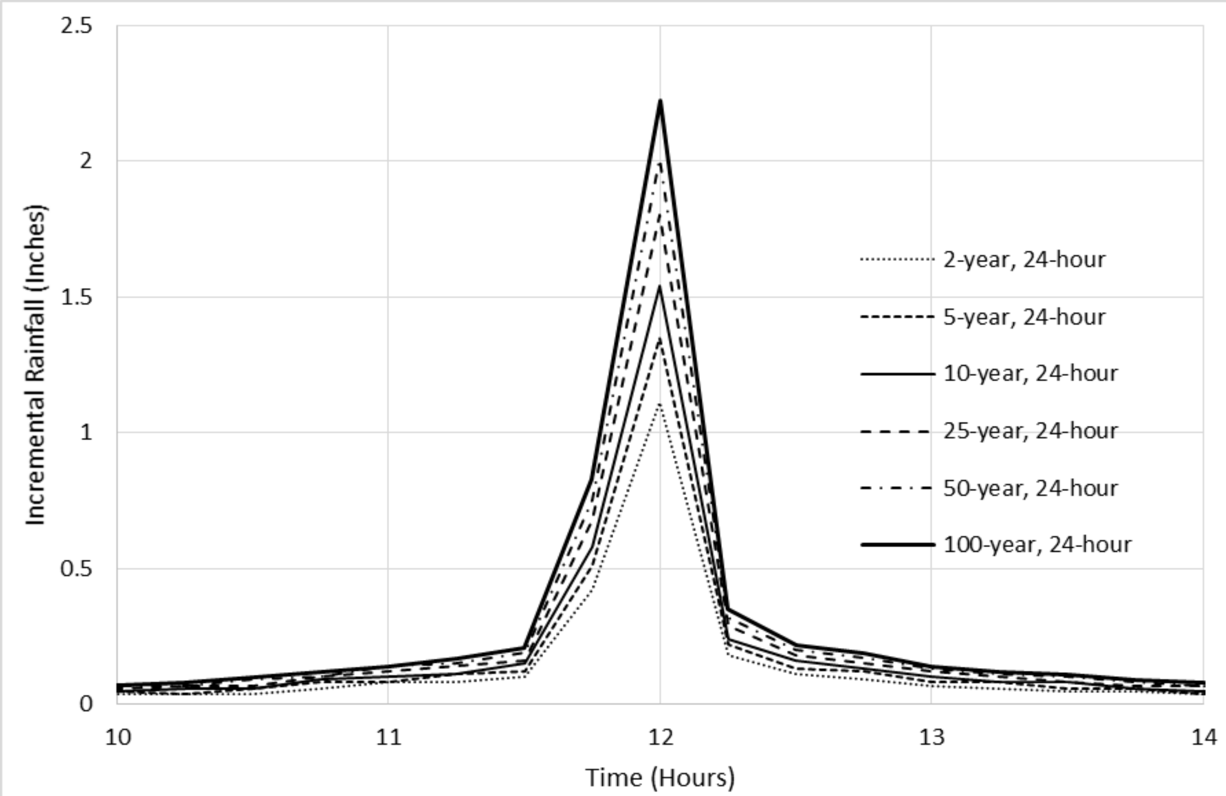
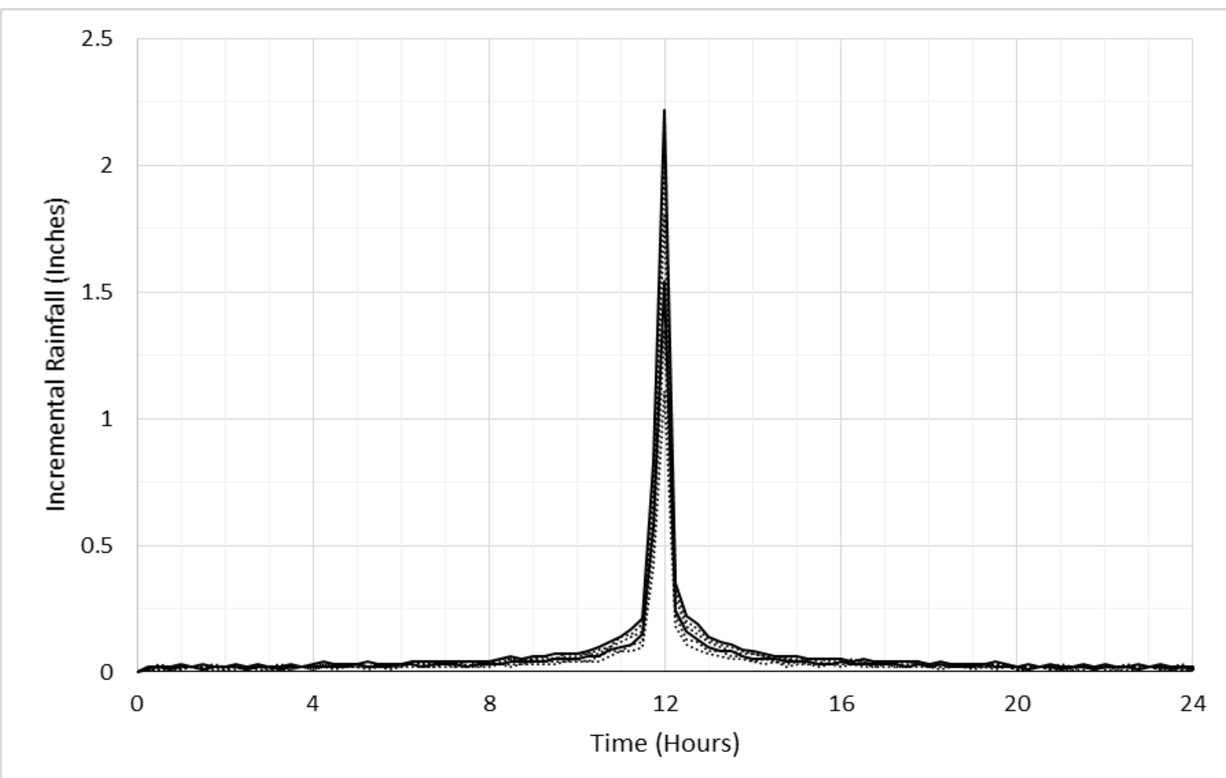


Figure 4.3: NRCS Type II Design Storm Event Hyetograph – Hours 0-24



The existing conditions model was run with the design storm events over a 2-day simulation period and then evaluated for capacity and flooding. The “capacity” of the system can be described in many ways. In simple terms, capacity is based strictly on the cross sectional area, slope, and material surface roughness. This is referred to as full flow capacity, which is the maximum flow conveyed without becoming surcharged, or operating under pressure. When a pipe is surcharged, the water level at one end of the pipe is higher than the pipe itself. The higher water level, referred to as head, puts the pipe under pressure and forces more flow through the pipe. Greater surcharge depths will result in more flow through the pipe. Therefore, “capacity” is a relative term that corresponds with the amount of head on the pipe.

Often, a term such as “10-year capacity” is used to describe a storm sewer. A more specific description would be “10-year full capacity without surcharging”, or “10-year capacity with the hydraulic gradeline (HGL) below the rim”. The HGL is synonymous with head; when drawing a profile of a storm sewer line, the HGL connects the water level at each manhole on the sewer line.

The City’s storm sewer system is extensive and complex, with multiple interconnected systems consisting of storm sewer pipes, culverts, bridges, and open natural and concrete channels. It is not sufficient to solely assign an “X-year capacity” description to a reach of pipe or channel; the purpose of this study was to develop a more detailed understanding of the system’s function and identify existing capacity limitations that cause localized and system wide flooding. With this understanding in mind, there are several figures and exhibits in this report that are key to describing the capacity of the system. They are described below:

Hydraulic Grade Line Profiles: A series of hydraulic profiles have been developed including the storm water conduit type, invert, rim, ground elevation, 10-year, 24-hour design storm event hydraulic grade line, nominal capacity and flow rate. These exhibits, presented in Appendix C, show each significant branch of the drainage network per watershed.

Floodplain Maps: The floodplain inundation maps were produced using InfoSWMM Risk Assessment Manager to create flood inundation Digital Elevation Maps (DEMs). These exhibits show a plan view of the modeled network and the extent of the 10-year, 24-hour and 100-year, 24-hour design storm event flooding events. An existing conditions surface model DEM (five foot by five foot pixels) was created by merging the topographic survey data collected via this project with the Shelby County LIDAR elevation data. The floodplain delineations are based on interpolating the HGL between model nodes and comparing the water surface elevations to the surface model. The floodplains are color-coded by depth of flooding: Red is greater than two feet in depth; Yellow between one and two feet in depth; and Green between zero and one foot in depth.

These exhibits are presented in Appendices D and E (existing conditions: 10-year and 100-year floodplains, respectively) and Appendix F (proposed conditions: 10-year and 100-year floodplains, respectively).

Overland Flow Locations: Typical with many stormwater conveyance networks, the Todd Creek Study Area system includes areas where the local storm sewer does not have sufficient inlet and/or conveyance capacity that results in surface flooding or overland flow to locations that have sufficient capacity.

Finished Floor Elevations: The existing conditions model (2-year, 24-hour design event results) were used to identify structures that potentially flood more frequently than others. Ninety-five (95) residential structures were identified to be field surveyed to obtain a finished floor elevation (FFE). The structure’s address and finished floor elevation are shown on the hydraulic profiles (Appendix C) and floodplain delineation exhibits (Appendices D, E, G, and H).

The surveyed FFEs were compared to the five-foot pixel DEM surface elevations at each structure’s location. It was determined that 84% of the surveyed FFEs were equal to or greater than one foot above the DEM surface elevation. Therefore, when analyzing the extent of surface flooding, the study did not include solutions to eliminate structures within the floodplain if the depth of flooding was less than one foot. This approach was presented to the City during a preliminary-findings progress meeting on March 2, 2015; and the City agreed that the approach was reasonable in consideration of the limited resolution of existing digital data.

4.2 SYSTEM CAPACITY ANALYSIS RESULTS

The following results are based on the 10-year, 24-hour design storm event model run for existing conditions. The hydraulic profiles for each branch of the model are presented in Appendix C. The following sections include Profile Identifications that reference the appropriate sheet in Appendix C. For example, the model branch T-1 is associated with Sheet T-1 of the profiles.

Please note that all of the branch ***subtitles*** are described ***from downstream location to upstream location***. For example, branch T-2 is located starting at the Todd Creek Main Branch, and extends eastward to Georgian Drive.

Appendices D and E provide enlarged maps for the existing system as well as floodplain delineations for the 10-year, 24-hour and 100-year, 24-hour design storm events. Following each individual branch analysis, the corresponding Appendix D/E (10-year/100-year) detail map pages are provided for cross-reference purposes. The Appendix pages are listed from downstream to upstream, similarly to the branch subtitle descriptions.

4.2.1 Todd Creek System

The Todd Creek System is the largest of the five drainage areas studied containing over 20 sub-drainage areas contributing to Todd Creek. The total drainage area is 2,408 acres. The main branch of Todd Creek outfalls to the Loosahatchie River west of Thomas Street. The Todd Creek System consists of open natural channel, open and closed concrete channel, bridges, culverts and storm sewers.

T-1 Todd Creek Main Branch: from Loosahatchie River eastward to Range Line Road

T-1 originates as a collection of storm sewers in residential neighborhoods and is piped west from Range Line Road to an open concrete channel just west of Debby Street. The open concrete channel continues west beyond Stella Street, Mountain Terrace Street, Ardmore Street, University Street, Woodrow Street, and Overton Street under bridges or through culverts until reaching Frayser Boulevard. At Frayser Boulevard the open concrete channel becomes a closed concrete channel along Frayser Boulevard between Denver Street and Watkins Street. Just west of Watkins Street, the closed concrete channel becomes an open concrete channel again transporting flows northwest under Frayser School Drive, Steele Street and the Canadian National Railroad under bridges/culverts. The open concrete channel then transitions into an open natural channel that flows under Thomas Street (US Highway 51) and Old Millington Road before the confluence of Todd Creek with the Loosahatchie River. The portion of Todd Creek main branch west of Thomas Street is outside the corporate limits of the City of Memphis.

The 10-year, 24-hour HGL for the majority of the pipes and open and closed channels is above full-pipe capacity and the ground elevation which causes localized flooding along most of the Todd Creek main branch. Overall, there are 57 structures that are within the zero-to-one-foot-depth floodplain, 21 structures within the one to two feet depth floodplain and nine (9) structures within the greater than two feet depth floodplain. Twenty (20) FFE's were surveyed along T-1 and six (6) have elevations below the 10-year, 24-hour HGL. The City has received 20 flooding complaints from locations along T-1.

(Appendix D/E detail map pages: 1-5, 7, 8, 12-16, 21)

T-2 Branch: from T-1 Main Branch eastward to Georgian Drive

T-2 consists of storm sewers and open channels through a residential neighborhood that convey flow west from St. Elmo Avenue to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above full-pipe capacity, and in places rises above the ground surface elevation, which causes localized flooding at Pershing Park Drive. One (1) structure is within the zero-to-one-foot-depth floodplain. The City has received four flooding complaints from locations along T-2.

(Appendix D/E detail map pages: 3, 5, 6)

T-3 Branch: from T-1 Main Branch eastward to Corning Avenue

T-3 consists of storm sewers and open channels through a residential neighborhood transporting flow west from Corning Avenue and Pershing Park Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for portions of these sewers is above full-pipe capacity, leading to minor surface flooding at Pershing Park Drive. One (1) structure is within the zero-to-one-foot-depth floodplain. The City has received one flooding complaint from a location along T-3.

(Appendix D/E detail map pages: 4, 5, 8)

T-4 Branch: from T-1 Main Branch northward to Quinby Drive

T-4 consists of an open channel and two minor culverts through a residential neighborhood transporting flow southeast from Quinby Drive to the main branch of Todd Creek. The two culverts are undersized causing localized flooding along Cedell Drive. Six (6) structures are within the zero-to-one-foot-depth floodplain. The City has received one flooding complaint from a location along T-4.

(Appendix D/E detail map pages: 7, 4)

T-5 Branch: from T-1 Main Branch westward to Knox Avenue

T-5 consists of storm sewers and open channel through a residential neighborhood transporting flow east from Knox Avenue to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above full-pipe capacity, and in placed rises above the ground surface, which causes localized flooding along Pryor Street. Three (3) structures are within the zero-to-one-foot-depth floodplain. The City has received one flooding complaint from a location along T-5.

(Appendix D/E detail map page: 7)

T-6 Branch: from T-1 Main Branch eastward to Gowan Avenue

T-6 consists of storm sewers and open channel through a residential neighborhood transporting flow west from Gowan Avenue to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevation in some locations causing localized flooding at Gowan Avenue. Two (2) structures are within the zero-to-one-foot-depth floodplain. The City has received two flooding complaints from locations along T-6.

(Appendix D/E detail map pages: 7, 8)

T-7 Branch: from T-1 Main Branch southward to Canfield Avenue

T-7 consists of storm sewers through a residential neighborhood transporting flow north from Canfield Avenue to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation causing localized flooding at Frayser Boulevard and Haywood Avenue. Three (3) structures are within the zero-to-one-foot-depth floodplain. The City has received one flooding complaint from a location along T-7.

(Appendix D/E detail map pages: 7, 11, 17)

T-8 Branch: from T-1 Main Branch southward to Paulus Drive

T-8 consists of storm sewers through a residential neighborhood transporting flow north from Paulus Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevation which causes localized flooding at Paulus Drive, Frayser Boulevard and Haywood Avenue. One (1) structure is within the zero-to-one-foot-depth floodplain. The City has received one flooding complaint from a location along T-8.

(Appendix D/E detail map page: 12)

T-9 Branch: from T-1 Main Branch northward to Winston Drive

T-9 consists of storm sewers through a residential neighborhood transporting flow south from Winston Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevation which causes localized flooding at Winston Drive and Gowan Avenue. Two (2) structures are within the zero-to-one-foot-depth floodplain. The City has received one flooding complaint from a location along T-9.

(Appendix D/E detail map pages: 12, 8)

T-10 Branch: from T-1 Main Branch southward along Frayser School Drive

T-10 consists of storm sewers through a residential neighborhood transporting flow north from Paulus along Frayser School Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Paulus Drive and Frayser School Drive. Eight (8) structures are within the zero-to-one-foot-depth floodplain and one (1) structure is within the one to two feet depth floodplain. Two (2) FFE's were surveyed along T-10 and one (1) is below the 10-year, 24-hour HGL. The City has received two flooding complaints from locations along T-10.

(Appendix D/E detail map page: 12)

T-11 Branch: from T-1 Main Branch northward to Claire Drive

T-11 consists of storm sewers through a residential neighborhood transporting flow south from Claire Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Claire Drive, Winston Drive and Gowan Avenue. Two (2) structures are within the zero-to-one-foot-depth floodplain. One (1) FFE was surveyed along T-11 and none are below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 12, 8)

T-12 Branch: from T-1 Main Branch northward to Gowan Avenue

T-12 consists of storm sewers through a residential neighborhood transporting flow southwest from Gowan Avenue to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Gowan Avenue. Flooding in this area is most likely due to insufficient capacity of the main branch of Todd Creek. Structures within the floodplain at this location were identified in the T-1 existing capacity section. One (1) FFE was surveyed along T-12 and none are below the 10-year, 24-hour HGL. The City has received two flooding complaints from locations along T-12.

(Appendix D/E detail map pages: 12, 13)

T-13 Branch: from T-1 Main Branch southward to Warner Drive

T-13 consists of storm sewers through a residential neighborhood transporting flow north from Warner Drive along Riney Street to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Corner Drive, Willowwood Avenue and Riney Street. Four (4) structures are within the zero-to-one-foot-depth floodplain and two (2) structures are within the one to two feet depth floodplain. Two (2) FFE's were surveyed along T-13 and none are below the 10-year, 24-hour HGL. The City has received two flooding complaints from locations along T-13.

(Appendix D/E detail map pages: 13, 12, 18, 19)

T-14 Branch: from T-1 Main Branch northward to Pamela Drive

T-14 consists of a collection of storm sewers through a residential neighborhood transporting flow southwest from Julia Street, Overton Crossing Street, Burnham and Argonne to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Overton Crossing Street, Julia Street, Pamela Drive, Medferd Street, Kensett Drive and Hallbrook Street. Twenty two (22) structures are within the zero-to-one-foot-depth floodplain, eight (8) structures are within the one to two feet depth floodplain and one (1) structure is within the greater than two feet depth floodplain. Eight (8) FFE's were surveyed along T-14 and none are below the 10-year, 24-hour HGL. The City has received two flooding complaints from locations along T-14.

(Appendix D/E detail map pages: 13, 9, 10)

T-15 Branch: from T-1 Main Branch southward along Ashland Street

T-15 consists of storm sewers through a commercial development transporting flow north from Ashland Street to a small storage basin and ultimately to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is below the crown of the pipe. No structures are affected by flooding in this reach.

(Appendix D/E detail map page: 13)

T-16 Branch: from T-1 Main Branch southward to Warner Drive

T-16 consists of storm sewers through a residential neighborhood transporting flow north from Warner Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for the upper reach of the sewers is below the crown of the pipe. The 10-year, 24-hour HGL for the lower reach of the sewers is above full-pipe capacity and below the ground elevation. No structures are affected by flooding in this reach. The City has received six flooding complaints from locations along T-16.

(Appendix D/E detail map pages: 13, 19)

T-17 Branch: from T-1 Main Branch southward to Whitney Avenue

T-17 consists of storm sewers through a residential neighborhood transporting flow north from Whitney Avenue, along the east side of Aden Street and west side of Frayser View Drive, to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Whitney Avenue, Aden Street and Frayser View Drive. Twelve (12) structures are within the zero-to-one-foot-depth floodplain, three (3) structures are within the one to two feet depth floodplain and two (2) structures are within the greater than two feet depth floodplain. Five (5) FFE's were surveyed along T-17 and two (2) are below the 10-year, 24-hour HGL. The City has received seven flooding complaints from locations along T-17.

(Appendix D/E detail map pages: 13, 19, 22)

T-18 Branch: from T-1 Main Branch northward to Overton Crossing Street

T-18 consists of storm sewers and open channels through a residential neighborhood transporting flow southwest from east of Overton Crossing Street to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding near the intersection of Cassie Avenue and Overton Crossing Street. However, no structures are within the resulting floodplain.

(Appendix D/E detail map page: 14)

T-19 Branch: from T-1 Main Branch southward to Collier Drive

T-19 consists of a collection of storm sewers through a residential neighborhood transporting flow north from Slocum Avenue, Whitney Avenue and Overton Crossing Street; and eventually along Woodrow Street to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Boone Street, University Street, Willowwood Avenue, Chandler Street, Slocum Avenue, Overton Crossing Street and Woodrow Street. Five (5) structures are within the zero-to-one-foot-depth floodplain, two (2) structures are within the one to two feet depth floodplain and one (1) structure is within the greater than two feet depth floodplain. Three (3) FFE's were surveyed along T-19 and one (1) is below the 10-year, 24-hour HGL. The City has received five flooding complaints from locations along T-19.

(Appendix D/E detail map pages: 14, 20)

T-20 Branch: from T-1 Main Branch northward to Felipe Street

T-20 consists of storm sewers through a residential neighborhood transporting flow south from Felipe Street along east side of University Street to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at Corning Avenue, University Street, Cassie Avenue, Felipe Street and Celeste Drive. Five (5) structures are within the zero-to-one-foot-depth floodplain, five (5) structures are within the one to two feet depth floodplain and one (1) structure is within the greater than two feet depth floodplain. Two (2) FFE's were surveyed along T-20 and none are below the 10-year, 24-hour HGL. The City has received six flooding complaints from locations along T-20.

(Appendix D/E detail map pages: 14, 10)

T-21 Branch: from T-1 Main Branch southward to Nunnelee Avenue

T-21 consists of storm sewers through a residential neighborhood transporting flow north from Nunnelee Avenue along Ardmore Street to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding along Ardmore Street. However, no structures are impacted by flooding in the street.

(Appendix D/E detail map pages: 15, 21)

T-22 Branch: from T-1 Main Branch northeastward to Brookmeade Street

T-22 consists of storm sewers through a residential neighborhood transporting flow west from Brookmeade Street to Mountain Terrace Street, and south along Mountain Terrace to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding at the intersection of Mountain Terrace Street and Jan Drive. Three (3) homes are within the zero-to-one-foot-depth floodplain and five (5) homes are within the one to two feet depth floodplain. One (1) FFE was surveyed along T-22 and none are below the 10-year, 24-hour HGL.

(Appendix D/E detail map page: 15)

T-23 Branch: from T-1 Main Branch northward to Trezevant Street

T-23 consists of storm sewers through a residential neighborhood transporting flow southwest from Trezevant Street to the main branch of Todd Creek. The HGL for the majority of the sewers is above the crown of the pipe and above ground elevations which causes localized flooding along Brookmeade Street and Hargrove Avenue. Two (2) homes are within the zero-to-one-foot-depth floodplain, one (1) structure is within the one to two feet depth floodplain and one (1) structure is within the greater than two feet depth floodplain. Two (2) FFE's were surveyed along T-23 and none are below the 10-year, 24-hour HGL. The City has received one flooding complaint from a location along T-23.

(Appendix D/E detail map page: 15)

T-24 Branch: from T-1 Main Branch southward to Madeline Circle

T-24 consists of storm sewers through a residential neighborhood transporting flow north along Debby Street to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is below the crown of the pipe and below ground elevations. However, some localized flooding occurs at the upper reach which is most likely caused by an offset in the pipe inverts resulting in a 1.65' drop inlet and flat gradient of upstream pipe, based on survey data. This causes one (1) structure to be within the zero-to-one-foot-depth floodplain and one (1) structure to be within the one to two feet depth floodplain. One (1) FFE was surveyed along T-24 and none are below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 15, 21)

T-25 Branch: from T-1 Main Branch westward to Pera Drive

T-25 consists of storm sewers through a residential neighborhood transporting flow east along Floyd Avenue from Pera Drive to the main branch of Todd Creek. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above ground elevations. Some localized flooding occurs near Floyd Avenue. However, no structures are impacted by flooding in the street.

(Appendix D/E detail map page: 7)

4.2.2 Denver Trunk System

The Denver Trunk System consists of four branches contributing to the main branch of the Denver watershed and one branch to the north of the watershed. The main branch of the Denver watershed flows into Todd Creek west of Thomas Street (US Highway 51). The total Denver drainage area is 609 acres. The drainage system consists of open channel (concrete and natural), bridges, culverts, and storm sewer pipes. The majority of the collection system is piped and the main branch transitions from an open concrete channel to an open natural channel near the confluence with Todd Creek.

D-1 Denver Main Branch: from Todd Creek eastward to Townsend Avenue

D-1 originates in a residential neighborhood near Townsend Avenue and is piped west to an open concrete channel near Argonne Street. There are three road crossings: at Balfour Street, Steele Street and the Canadian National Railroad before the concrete channel transitions to a natural channel, which flows into Todd Creek (T-1 Main Branch). The majority of the upper reach storm sewers are above the crown of the pipe and the 10-year, 24-hour HGL fluctuates above and below ground elevations which causes flooding near the intersection of Denver Street and Lauretta Avenue. Most of the open concrete channel contains the flow within the channel. There is a slight increase in HGL at the Steele Street culvert, but flow does not overtop the roadway at this location or any of the other crossings. There are three (3) structures within the zero-to-one-foot-depth floodplain and three (3) structures within the one to two feet depth floodplain between Marvin Street and Denver Street. Two (2) FFE's were surveyed along D-1 and one (1) is below the 10-year, 24-hour HGL. The City has received nine flooding complaints from locations along D-1.

(Appendix D/E detail map pages: 2, 3, 23, 6, 25)

D-2 Branch: from D-1 Main Branch southward to Watkins Street

D-2 consists of storm sewers through a residential neighborhood transporting flow north from Watkins Street to Denver's main open channel. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe and above the ground surface which causes localized flooding along Watkins Street and Carlyle Avenue. Three (3) homes are within the zero-to-one-foot-depth floodplain and one (1) structure is within the one to two feet depth floodplain. One (1) FFE was surveyed along D-2 and none are below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 23, 6)

D-3 Branch: from D-1 Main Branch eastward to Georgian Hills Junior High School

D-3 consists of storm sewers through Georgian Hills Park transporting flow west to Denver's main open channel. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe and below the ground surface. No homes are within the resulting floodplain.

(Appendix D/E detail map pages: 23, 24)

D-4 Branch: from D-1 Main Branch southward to Belleau Street

D-4 consists of storm sewers through a residential neighborhood transporting flow north from Townsend Avenue to Denver's main open channel. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe and above the ground surface which causes localized flooding in backyards between Argonne Street and Belleau Street, and along St. Elmo Avenue. Four (4) homes are within the zero-to-one-foot-depth floodplain and one (1) home is within the greater than two feet depth floodplain. Two (2) FFE's were surveyed along D-5 and one (1) is below the 10-year, 24-hour HGL. The City has received three flooding complaints from locations along D-4.

(Appendix D/E detail map pages: 23, 6)

D-5 Branch: from D-1 Main Branch northward to Leyton Avenue

D-5 consists of storm sewers through a residential neighborhood transporting flow southwest from Leyton Avenue and St. Elmo Avenue to Denver's main open channel. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe and above the ground surface causing localized flooding in backyards between Overton Crossing Street and Denver Street. According to the survey data there is an existing 42-inch to 36-inch hydraulic contraction located at the intersection of Denver Street and Lauretta Avenue. Nine (8) homes are within the zero-to-one-foot-depth floodplain. Four (4) FFE's were surveyed along D-5 and one (1) is below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 25, 24)

D-6 Branch: from D-6 outfall to Argonne Street

D-6 consists of a separate storm sewer network through a residential neighborhood, transporting flow northwest from Argonne Street to an open channel on the northwest side of Steele Street. The open channel eventually discharges into an undeveloped area on the north side of Canadian National Railroad. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe and above the ground surface which causes localized flooding along Coventry Drive. Three (3) homes are within the zero-to-one-foot-depth floodplain, one (1) home is within the one to two feet depth floodplain and one (1) home is within the greater than two feet depth floodplain. One (1) FFE was surveyed along D-6 and none are below the 10-year, 24-hour HGL. The City has received one flooding complaint from a location along D-6.

(Appendix D/E detail map page: 23, 40)

4.2.3 WMPS System

The WMPS System consists of eleven separate systems contributing to the main branch of the WMPS watershed. The total WMPS drainage area is 1,149 acres. The drainage system consists of open channels, bridges, culverts, and storm sewer pipes. The majority of the developed outer reaches of the watershed have a sewer system that transports storm water runoff to the main branch which is predominately open channel.

W-1 WMPS Main Branch: from Loosahatchie River to Dahlia Street

W-1 originates in a residential neighborhood and is piped west under the Canadian National Railroad to an open channel. According to the survey data there is an existing 42-inch to 24-inch hydraulic contraction located at the railroad crossing. This causes localized flooding in the neighborhood southeast of the railroad. As a result, four (4) structures are within the zero-to-one-foot-depth floodplain and one (1) structure is within the one to two feet depth floodplain. The open channel that is between the railroad and Millington Street does not contain flow within the banks of the channel which results in one (1) structure within the zero-to-one-foot-depth floodplain. The open channel flow then enters a 4' x 9' box culvert along Frayser Boulevard that transitions to a 5' x 20' culvert near Dawn Drive. This section of closed concrete channel has insufficient capacity which results in localized flooding at seven (7) structures within the zero-to-one-foot-depth floodplain and one (1) structure within the one to two feet depth floodplain. W-1 then becomes a natural open channel from the intersection of Dawn Drive and Frayser Boulevard, to the confluence with the Loosahatchie River. The portion of WMPS main branch west of Benjestown Street is outside the corporate limits of the City of Memphis.

There are two road crossings along this reach of natural channel: a 6'x8' double box culvert at Benjestown Street, and a 5'x7' double box culvert at an unnamed private road. Both of these culverts are undersized and cause an increase in the 10-year, 24-hour HGL upstream of the culvert. However, flows do not overtop the roadway. The open channel does not completely contain the flow within the banks of the channel, but the majority of the adjacent land is undeveloped floodplain. The hydraulic contraction created at the Benjestown Street culvert causes the floodplain extents to reach nearby residential areas resulting in three (3) structures within the zero-to-one-foot-depth floodplain and four (4) structures within the greater than two feet depth floodplain. One (1) FFE was surveyed along W-1 and it is below the 10-year, 24-hour HGL. The City has received three flooding complaints from locations along W-1.

(Appendix D/E detail map pages: 30, 27, 28, 31, 32, 35, 17)

W-2 Branch: from W-1 Main Branch southward along Harvester Lane

W-2 originates at the intersection of Frayser Drive and Harvester Lane, and transports stormwater north along Harvester Lane to a headwall. Natural open channel then carries flow to the confluence with the WMPS main branch. The portion of W-2 branch north of Benham Avenue and an existing subdivision is outside the corporate limits of the City of Memphis. The upper reach of the storm sewer has insufficient capacity which causes localized flooding at the Frayser Drive and Harvester Lane intersection. However, no structures are within the extents of the floodplain. The City has received one flooding complaint from a location along W-2.

(Appendix D/E detail map page: 30)

W-3 Branch: from W-1 Main Branch southward to Klinke Avenue

W-3 consists of storm sewers through a residential neighborhood transporting flow north towards a headwall north of Benham Avenue. An open channel then carries flow to the confluence with the WMPS main branch. The portion of W-3 branch north of Benham Avenue and an existing subdivision is outside the corporate limits of the City of Memphis. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe but only the upper reach has an HGL above the ground surface causing localized flooding along Harvester Lane. Six (6) homes are within the zero-to-one-foot-depth floodplain and one (1) home is within the one to two feet depth floodplain. One (1) FFE was surveyed along W-1 and it is below the 10-year, 24-hour HGL. The City has received three flooding complaints from locations along W-3.

(Appendix D/E detail map pages: 27, 28, 31, 30, 33)

W-4 Branch: from W-1 Main Branch northward along Sunrise Street

W-4 consists of storm sewers through a residential neighborhood transporting flow south towards a headwall west of Sunrise Street. An open channel then carries flow to the confluence with the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewer pipes is above the crown of the pipe, however the 10-year, 24-hour HGL is below the ground elevation. According to the survey data there is an existing 30-inch to 24-inch hydraulic contraction located at the intersection of Par Avenue and Sunrise Street. Two (2) structures are within the zero-to-one-foot-depth floodplain and four (4) structures are within the greater than two feet depth floodplain. Two (2) FFEs were surveyed along W-1 and none are below the 10-year, 24-hour HGL. The City has received three flooding complaints from locations along W-4.

(Appendix D/E detail map pages: 28, 26)

W-5 Branch: from W-1 Main Branch southward to Frayser Drive

W-5 consists of storm sewers transporting flow north from Frayser Drive to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and below the ground elevation. The sewer runs through the back side of the City owned Westside Middle School property and no structures are within the resulting floodplain.

(Appendix D/E detail map pages: 28, 31)

W-6 Branch: from W-1 Main Branch northeastward to Randolph Street

W-6 consists of storm sewers transporting flow west along Dawn Drive to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation. Most of the flooding occurs near the intersection of Dawn Drive and Ripley Street, causing 13 structures to be within the zero-to-one-foot-depth floodplain. One (1) FFE was surveyed along W-6 and one (1) is below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 28, 29)

W-7 Branch: from W-1 Main Branch northeastward to Millington Street

W-7 consists of storm sewers transporting flow south along Millington Street and west along Par Avenue to an open channel, and eventually to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation, causing localized flooding at the intersection of Millington Street and Joel Avenue. The open channel does not contain the flow within the banks of the channel. Seven (7) residential homes are within the zero-to-one-foot-depth floodplain and two (2) are within the one to two feet depth floodplain. Six (6) FFE’s were surveyed along W-7 and three (3) are below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 31, 32, 29)

W-8 Branch: from W-1 Main Branch southward to Klinke Avenue

W-8 is made up of multiple stormwater collection systems within the Westside Neighborhood that flow north from Klinke Avenue, Kingston Street and Marsh Avenue. The systems combine into a single trunkline south of Juliet Avenue, flowing northeast to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation, causing flooding at Klinke Avenue, Sunrise Street, Marsh Avenue, Kingston Avenue, Morningside Drive, Shirley Circle, Juliet Avenue, Dawn Cove, Dawn Drive, Westside Drive, Greendale Circle, Frayser Circle, Brad Drive and Par Drive. The open channel segment north of Marsh Avenue does not contain the flow within the banks of the channel. There are 28 structures within the zero-to-one-foot-depth floodplain and 11 structures within the one to two feet depth floodplain. Eighteen (18) FFE's were surveyed along W-8 and nine (9) are below the 10-year, 24-hour HGL. The City has received one flooding complaint from a location along W-8.

(Appendix D/E detail map pages: 31, 34)

W-9 Branch: from W-1 Main Branch southward to Whitney Avenue

W-9 consists of storm sewers transporting flow north along the east side of Thomas Street (US Highway 51) to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation, causing flooding at Cindy Lane. Two (2) structures are within the zero-to-one-foot-depth floodplain and two (2) are within the one to two feet depth floodplain. Two (2) FFE's were surveyed along W-9 and one (1) is below the 10-year, 24-hour HGL.

(Appendix D/E detail map pages: 32, 35)

W-10 Branch: from W-1 Main Branch northward along Millington Street

W-10 consists of storm sewers transporting flow south along Millington Street to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation near the confluence with the main branch. However no structures are within the resulting floodplain.

(Appendix D/E detail map page: 32)

W-11 Branch: from W-1 Main Branch southward to Dellwood Avenue

W-11 consists of storm sewers transporting flow north along Madewell Drive to the WMPS main branch. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation, causing flooding along Madewell Drive. However no structures are within the resulting floodplain.

(Appendix D/E detail map page: 35)

W-12 Branch: from W-12 outfall southward to Burlington Circle

W-12 is a smaller separate storm collection system located at the southwest corner of the Westside Neighborhood. The 10-year, 24-hour HGL for the majority of the sewers is below the crown of the pipe and therefore no localized flooding occurs in this area.

(Appendix D/E detail map page: 33)

4.2.4 Carrolton System

The Carrolton System consists of three separate storm sewer networks collecting stormwater runoff from a residential neighborhood transporting flow north to an undeveloped area. The area of drainage for C-1, C-2 and C-3 is 21, 29 and 24 acres respectively.

C-1 Branch: from C-1 outfall southeastward to Carrolton Avenue

C-1 consists of storm sewers and is located near the intersection of Early Street and Carrolton Avenue. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and the ground elevation which causes localized flooding at the intersection of Early Street and Carrolton Avenue. Three (3) structures are within the zero-to-one-foot-depth floodplain.

(Appendix D/E detail map page: 26)

C-2 Branch: from C-2 outfall southward to Restbrook Avenue

C-2 consists of storm sewers transporting flow north along Ripley Street. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation which causes flooding at the intersection of Ripley Street and Carrolton Avenue. However no structures are within the 10-year, 24-hour floodplain. The City has received one flooding complaint from a location along C-2. The outfall of C-2 branch appears to be outside the corporate limits of the City of Memphis.

(Appendix D/E detail map page: 39, 29)

C-3 Branch: from C-3 outfall southward to Creston Avenue

C-3 consists of storm sewers transporting flow north from Carrolton Avenue to Creston Avenue. The 10-year, 24-hour HGL for the majority of the sewers is above the crown of the pipe and above the ground elevation which causes flooding in backyards south of Creston Avenue. Two (2) structures are within the zero-to-one-foot-depth 10-year, 24-hour floodplain. The City has received four flooding complaints from locations along C-3.

(Appendix D/E detail map page: 39)

4.2.5 Memphis Downtown Airport System

The Memphis Downtown Airport System consist of two (2) main branches. The majority of the system is natural open channel with a few box culverts. During the 10-year, 24-hour design storm event, the flow stays within the top of bank of the open channels and passes freely through the culverts. The City has not received specific flooding complaints from residents in this basin. However it is generally known that the airport area has been subject to historic Mississippi River flooding – which is beyond the scope of this study.

(Appendix D/E detail map pages: 36-38)

4.3 FLOODPLAIN DELINEATIONS

Overall basin flood inundation maps for the 10-year, 24-hour storm are shown on Figures 4.3 to 4.7. Appendices D and E include enlarged floodplain delineations for the 10-year, 24-hour and 100-year, 24-hour design storm events, respectively. The floodplains are presented with three color-coded depth classifications to help identify the severity of the flooding. The three classifications are:

- Green - zero to one foot depth
- Yellow - one to two feet in depth
- Red - greater than two feet in depth.

The following tables (Table 4.2 to Table 4.5) provide the number of structures within each depth classification in each watershed for the existing 10-year and 100-year, 24-hour design events.

Table 4.2: Carrolton Flooded Structures

Flooding Depth (feet)	Existing 10-year, 24-hour Floodplain	Existing 100-year, 24-hour Floodplain
0 – 1	5	10
1 - 2	--	2
> 2	--	1
Total	5	13
Total > 1	--	3

Table 4.3: Denver Flooded Structures		
Flooding Depth (feet)	**Existing 10-year, 24-hour Floodplain**	**Existing 100-year, 24-hour Floodplain**
0 – 1	22	34
1 - 2	5	29
> 2	2	9
Total	29	72
Total > 1	7	38

Table 4.4: Todd Creek Flooded Structures		
Flooding Depth (feet)	**Existing 10-year, 24-hour Floodplain**	**Existing 100-year, 24-hour Floodplain**
0 – 1	140	229
1 - 2	49	157
> 2	14	72
Total	203	458
Total > 1	63	229

Table 4.5: WMPS Flooded Structures		
Flooding Depth (feet)	**Existing 10-year, 24-hour Floodplain**	**Existing 100-year, 24-hour Floodplain**
0 – 1	70	98
1 - 2	18	48
> 2	7	21
Total	95	167
Total > 1	25	69

Figure 4.4: 10-Year, 24-Hour Design Storm Event Flood Inundation Map - Carrolton Area

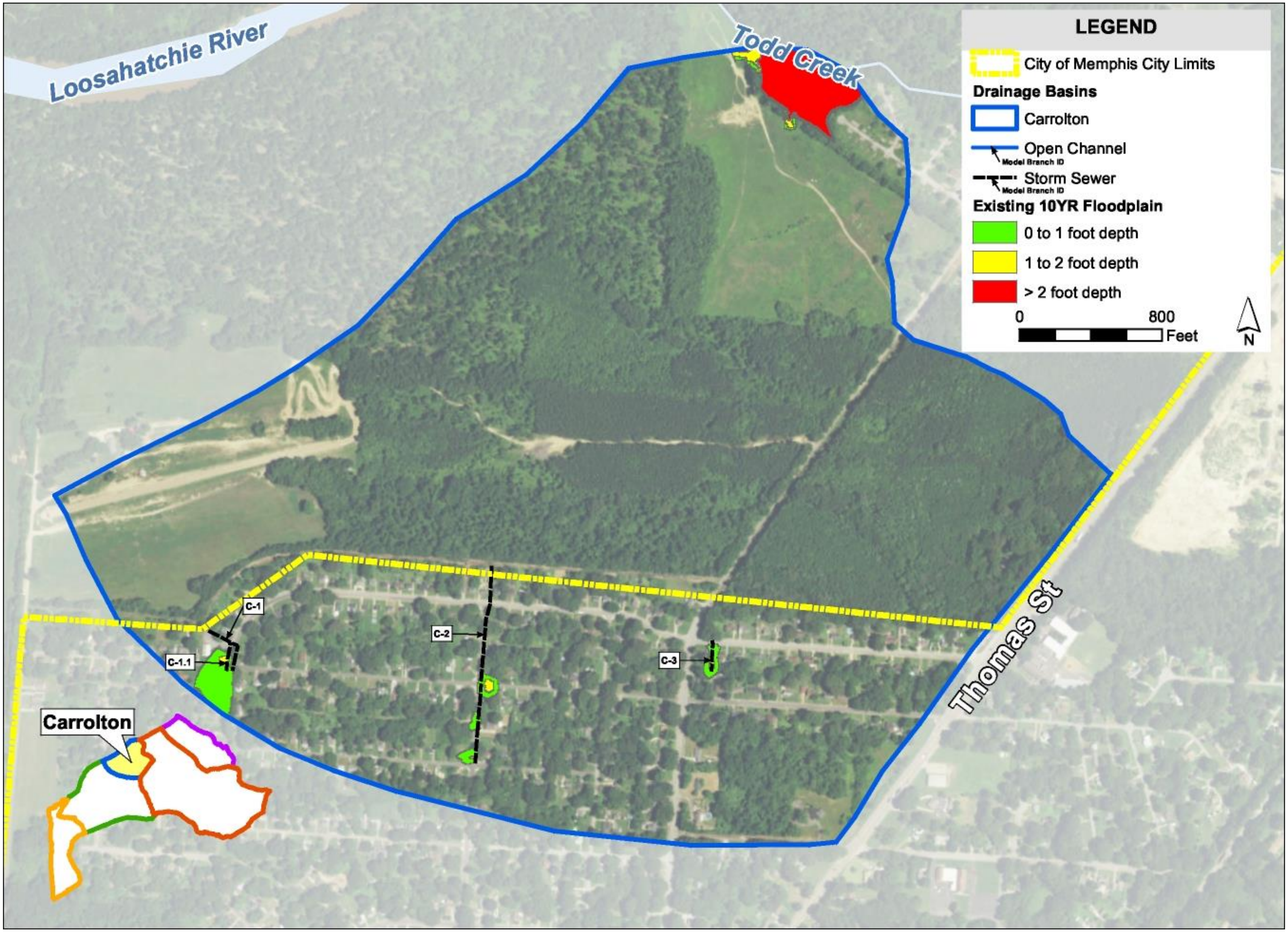


Figure 4.5: 10-Year, 24-Hour Design Storm Event Flood Inundation Map – Denver Branch

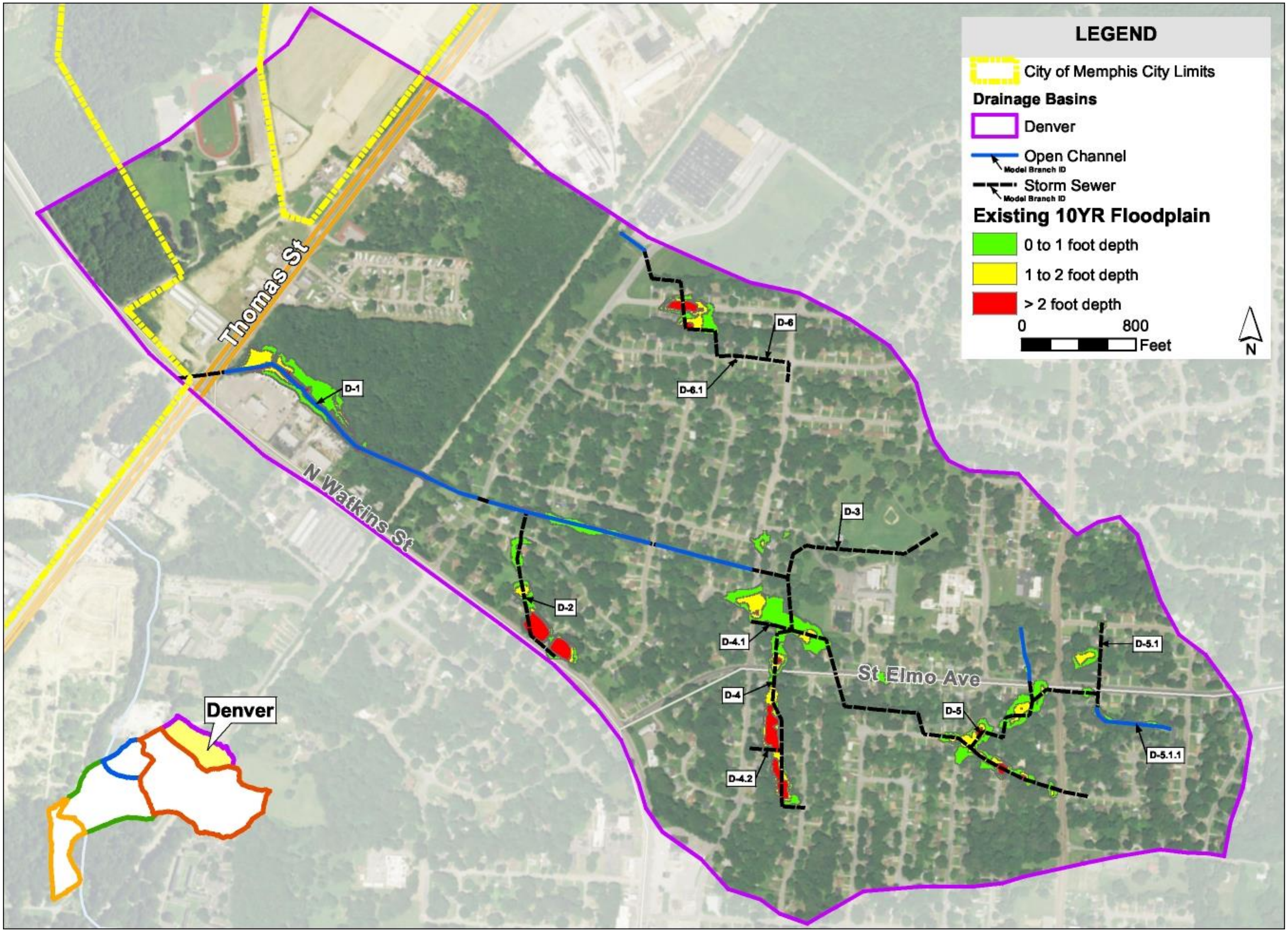


Figure 4.6: 10-Year, 24-Hour Design Storm Event Flood Inundation Map – Memphis Downtown Airport

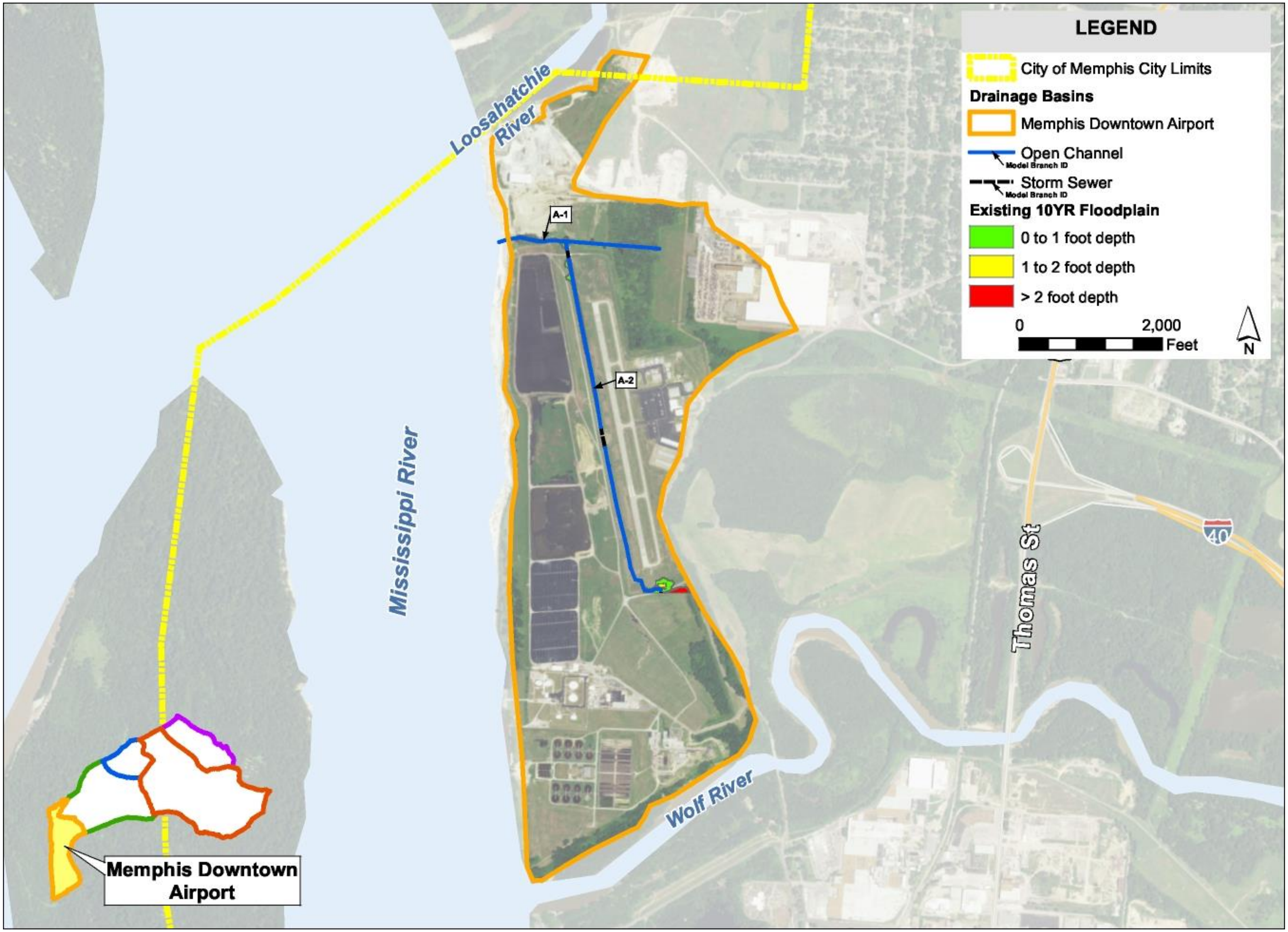


Figure 4.7: 10-Year, 24-Hour Design Storm Event Flood Inundation Map – Todd Creek

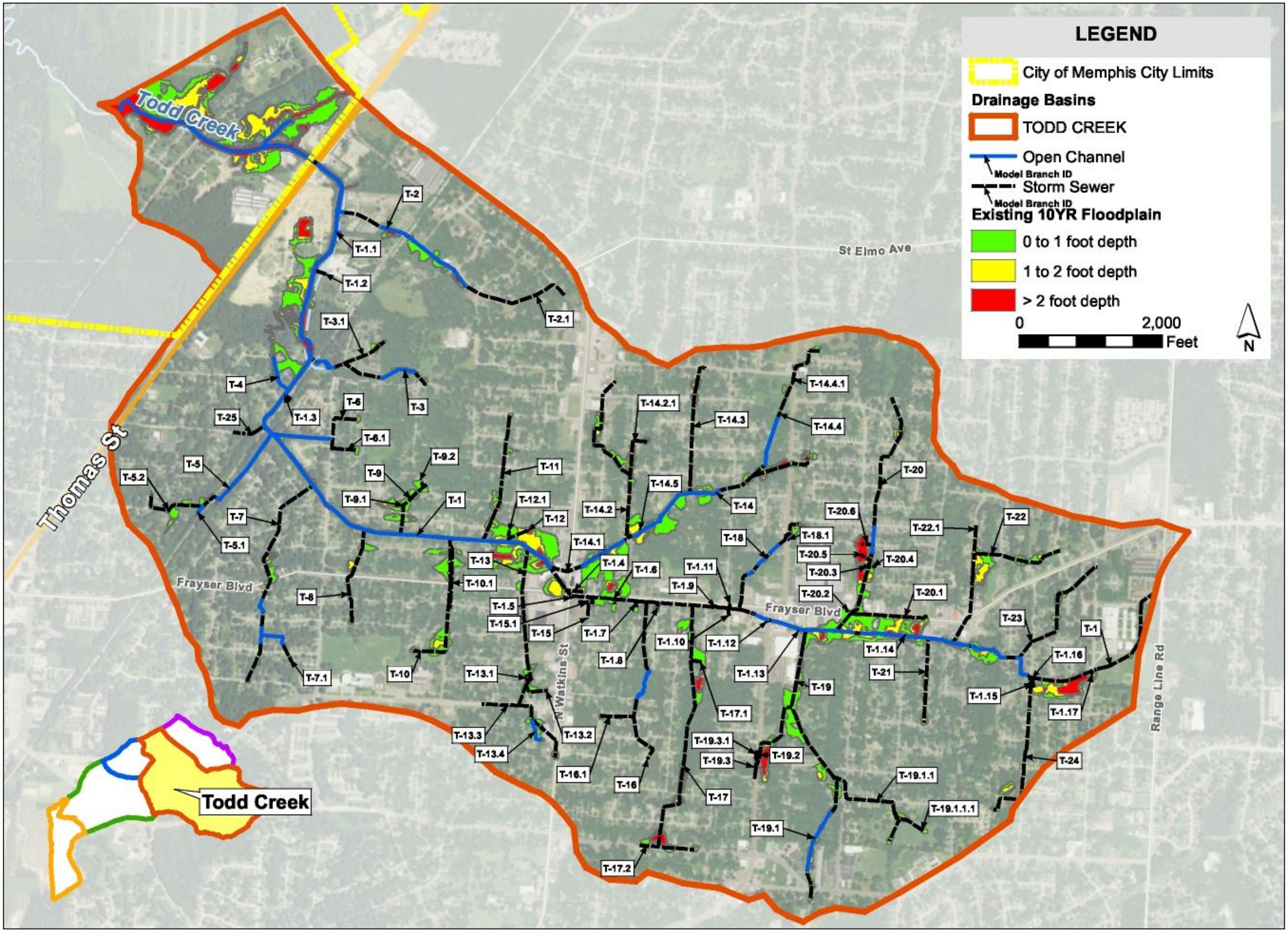
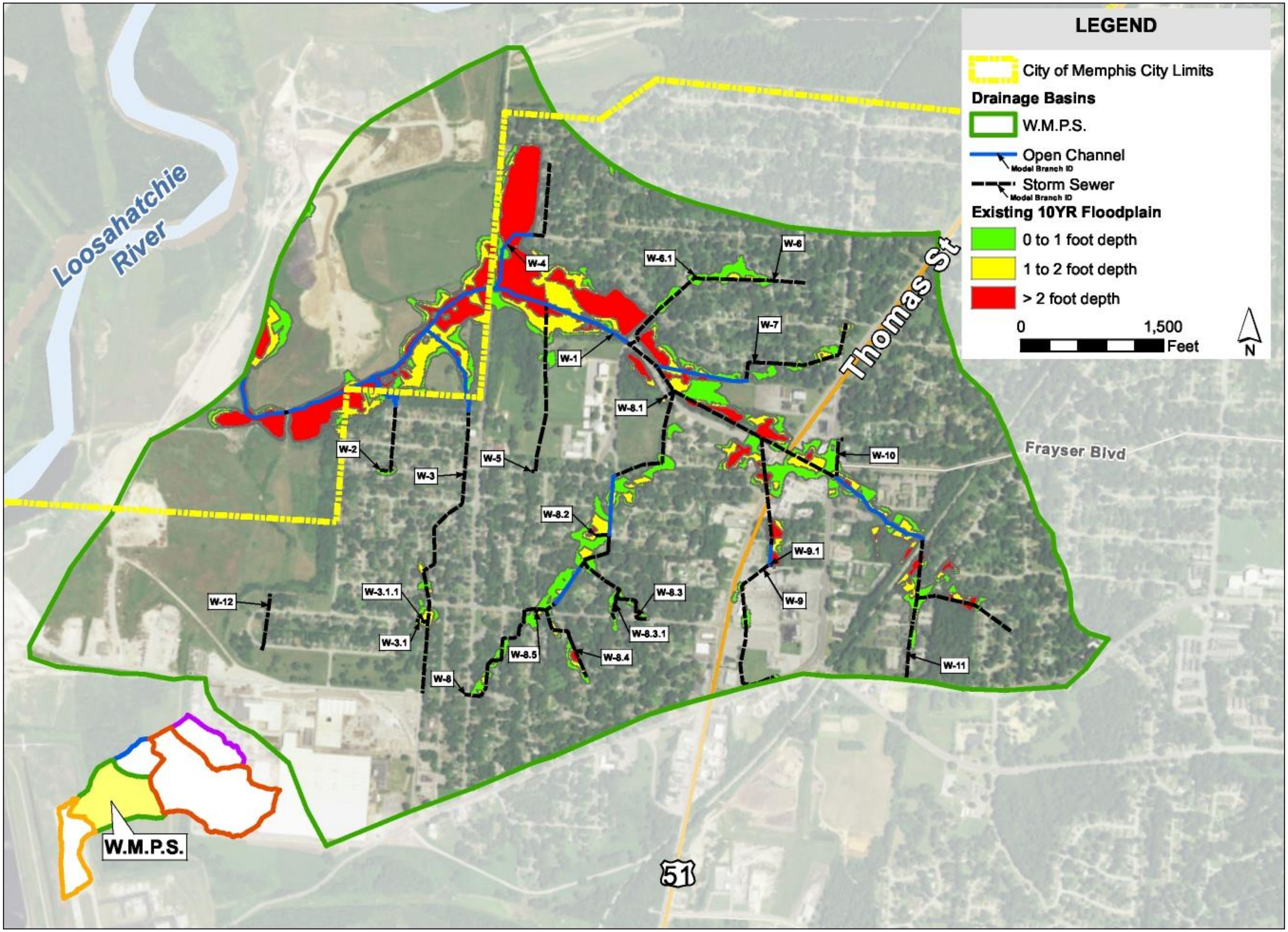


Figure 4.8: 10-Year, 24-Hour Design Storm Event Flood Inundation Map – WMPS Area



5.0 ALTERNATIVES EVALUATION

5.1 APPROACH TO EVALUATING ALTERNATIVES

An evaluation of alternatives was performed to identify and quantify opportunities to improve system performance and reduce the frequency or severity of flooding. The two primary alternatives evaluated were increased detention storage to attenuate peak flows, and increased conveyance to remove isolated hydraulic restrictions (“bottlenecks”) and reduce system HGLs. The overarching goal was to determine if the system could` be retrofitted with these types of measures to increase capacity enough to handle the 10-year, 24-hour design event. However, after completing the assessment of the existing system, it became apparent that this level of protection could not be accomplished due to the complexity of the system and the many hydraulic constraints.

The first step during the evaluation of alternatives was to identify significant hydraulic contractions and areas of significant flooding. Localized hydraulic contractions were removed and sites were identified for potential storage facilities. The storage capacities of each site were then maximized based on allowable footprints and site constraints. Even with these measures in place, several locations continued to experience significant flooding. Therefore, additional system conveyance improvements (upsizing channels/pipes) were identified to help alleviate flooding. The objective was to eliminate as many structures as possible from the 10-year, 24-hour design event floodplain. For this study a structure is considered to be the main dwelling of each parcel. Secondary structures like garages, sheds and trailers were not included in the analysis. Appendix F includes project location maps that show the recommended conceptual improvements in each project area.

Once the capacity improvement alternatives were determined a secondary storage analysis was performed to determine the amount of acre-feet of storage required at key upstream locations to eliminate or reduce the recommended capacity improvements. Most of the secondary storage locations were not identified as potential storage locations based on the existing land use, therefore these storage alternatives would require land acquisition and razing of multiple homes to accommodate the storage volume required to reduce the number of structures flooded during the 10-Year, 24-Hour design storm event.

Planning-level opinions of probable cost were also developed for the recommended drainage improvements. The cost opinions are based on contemporary costs from similar projects, Tennessee Department of Transportation Average Unit Prices data (2014 awarded contracts), and engineering judgement. In general, a 35% markup was included in the adjusted unit costs for mobilization/demobilization, maintenance of traffic, bonds and insurance, contingency, engineering and permitting. In addition, a standard 25% contingency was included for planning purposes. The planning-level cost opinions do not include the secondary storage alternatives.

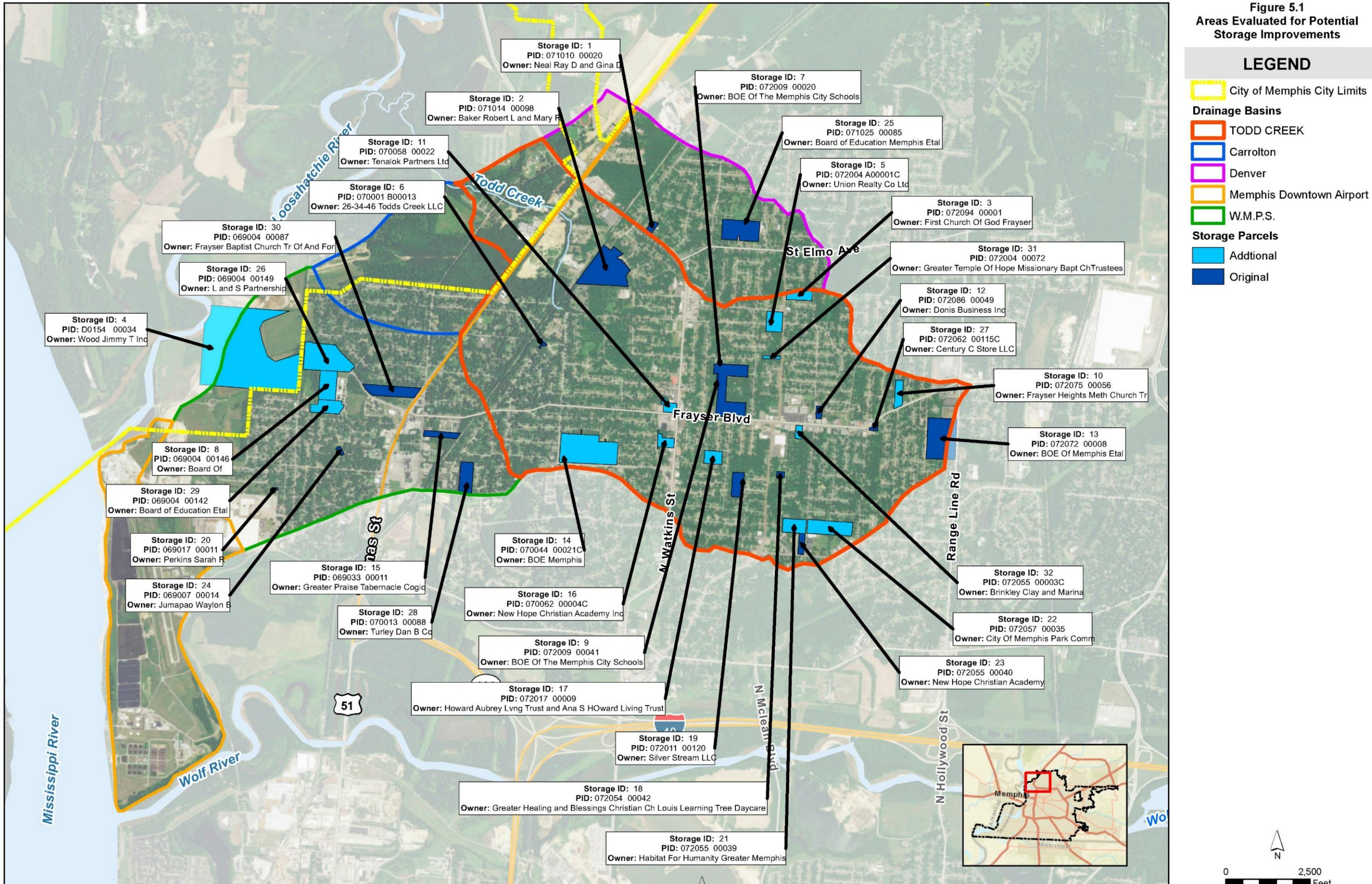
The planning-level costs opinions are summarized in Table 6.2. Appendix I includes more detailed planning-level opinions of probable construction costs for each alternative project.

5.2 STORAGE ALTERNATIVES

A meeting was held on March 2, 2015 with representatives from the City to review the status of model development activities and potential alternatives that could be evaluated to reduce flooding. A total of 18 potential storage locations were presented to the City for consideration (see Figure 5.1).

The sites that were presented are larger open areas adjacent to major components of the storm drainage system in areas of significant flooding. Of the 18 sites reviewed, three were chosen to be viable options based on the location and current property ownership. These three locations were then analyzed to determine the maximum storage capacity at the site. The storage basins were modeled with the 100-year, 24-hour design storm event to ensure that the available volume could contain a large event without causing additional adverse flooding conditions. Storage basin outlets were adjusted to maximize the available storage capacity. A 15-inch diameter outlet pipe is the smallest allowable size to prevent maintenance issues, in accordance with the Memphis and Shelby County Storm Water Management Manual. The following is a description of the three storage basin locations chosen for analysis.

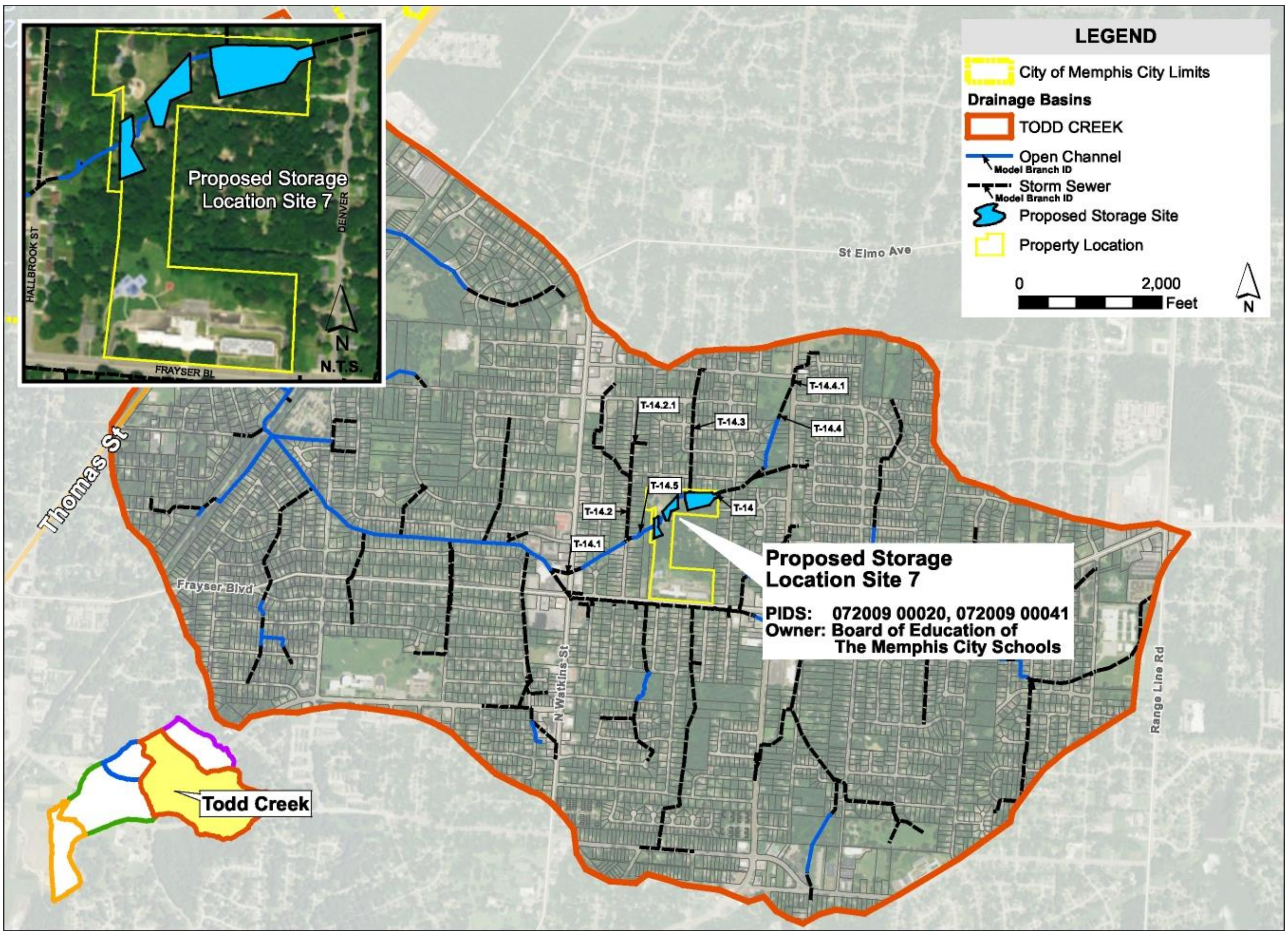
Figure 5.1: Areas Evaluated for Potential Storage Sites



5.2.1 Site 7 – Denver Park Storage Alternative

Site 7 - Denver Park: This site is located within Denver Park near the intersection of Pueblo and Denver in the Todd Creek System drainage area (T-14). Storage at this location was provided by enhancing the open channel cross sections throughout this reach of the model to increase the cross sectional flow area. Three different areas were modified within the park and positioned to avoid the ongoing development of Denver Park which includes paved pedestrian pathways and two pedestrian bridge spans, which are being built outside the scope of this study. The proposed bridge spans were incorporated into the model based on design plans provided by the City. The three open storage basins at this location consist of a 4-foot deep basin with a maximum footprint of 16,500 square feet; a 7-foot deep basin with a maximum footprint of 18,200 square feet; and a 6-foot deep basin with a maximum footprint of 73,900 square feet.   The total potential storage capacity for all three basins is 636,700 cubic feet. The planning-level cost opinion for Site 7 is $1,233,569. Figure 5.2 presents the conceptual planning improvements proposed at Site 7: Denver Park.

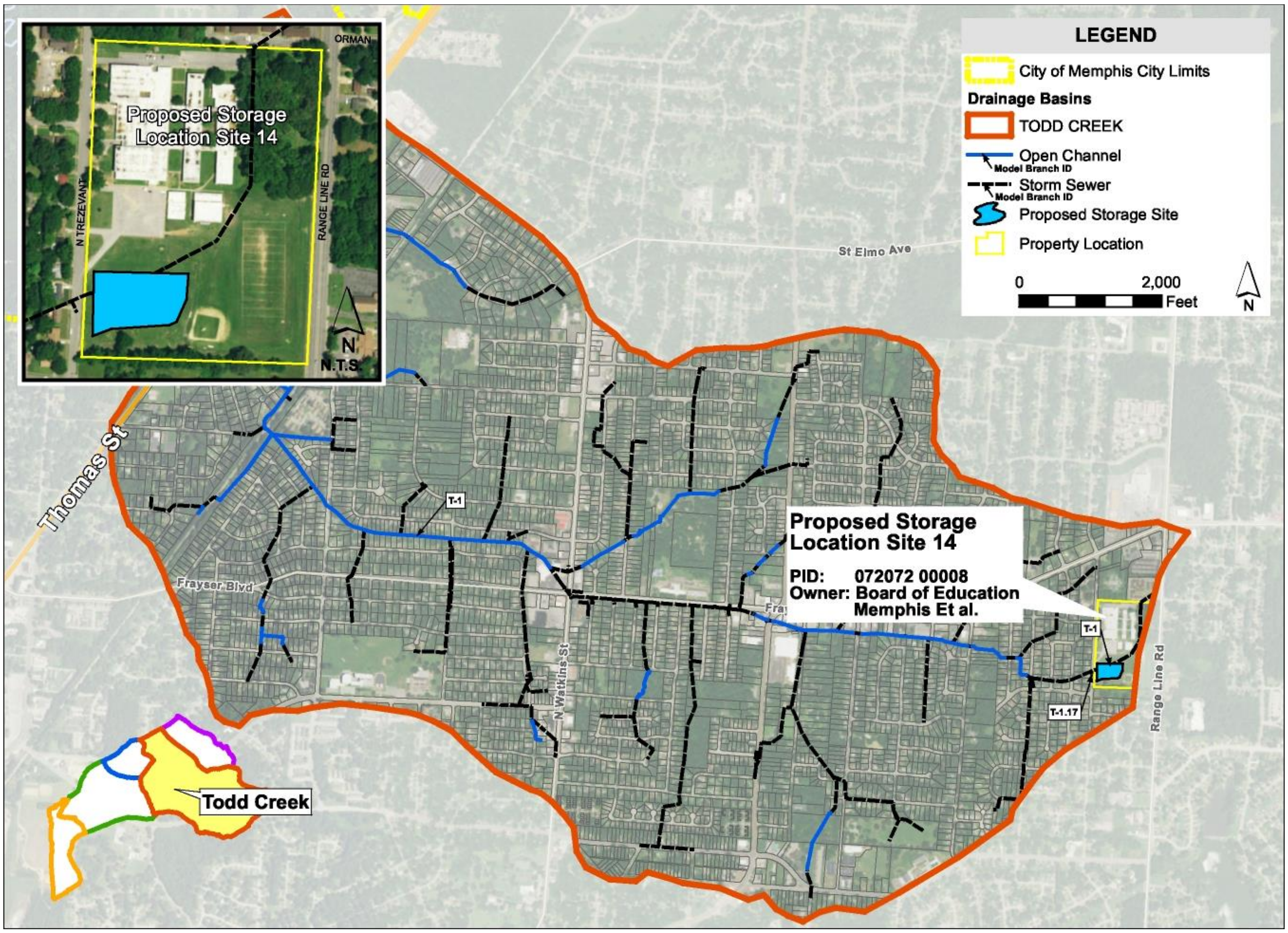
Figure 5.2: Conceptual Denver Park Storage Alternative



5.2.2 Site 14 – Trezevant High School Storage Alternative

Site 14 – Trezevant High School: This site is located at Trezevant High School near the intersection of Nunnelee and Trezevant in the Todd Creek System drainage area (T-1). The site is a softball field, but various satellite images show the field to be in a state of disrepair. It is unknown if the school is currently using this portion of the school's recreational areas. The storage basin at this location consists of a 7.5 foot deep open basin with a maximum footprint of 74,000 square feet and total potential capacity of 464,000 cubic feet. The basin is fed by a 48-inch pipe with a 50 acre contributing drainage area. Using a 24-inch outlet pipe, the maximum depths obtained during the 10-year and 100-year design events are 5.2 feet and 7.4 feet, respectively. The planning-level cost opinion for Site 14 is $959,451. Figure 5.3 presents the conceptual planning improvements proposed at Site 14: Trezevant High School.

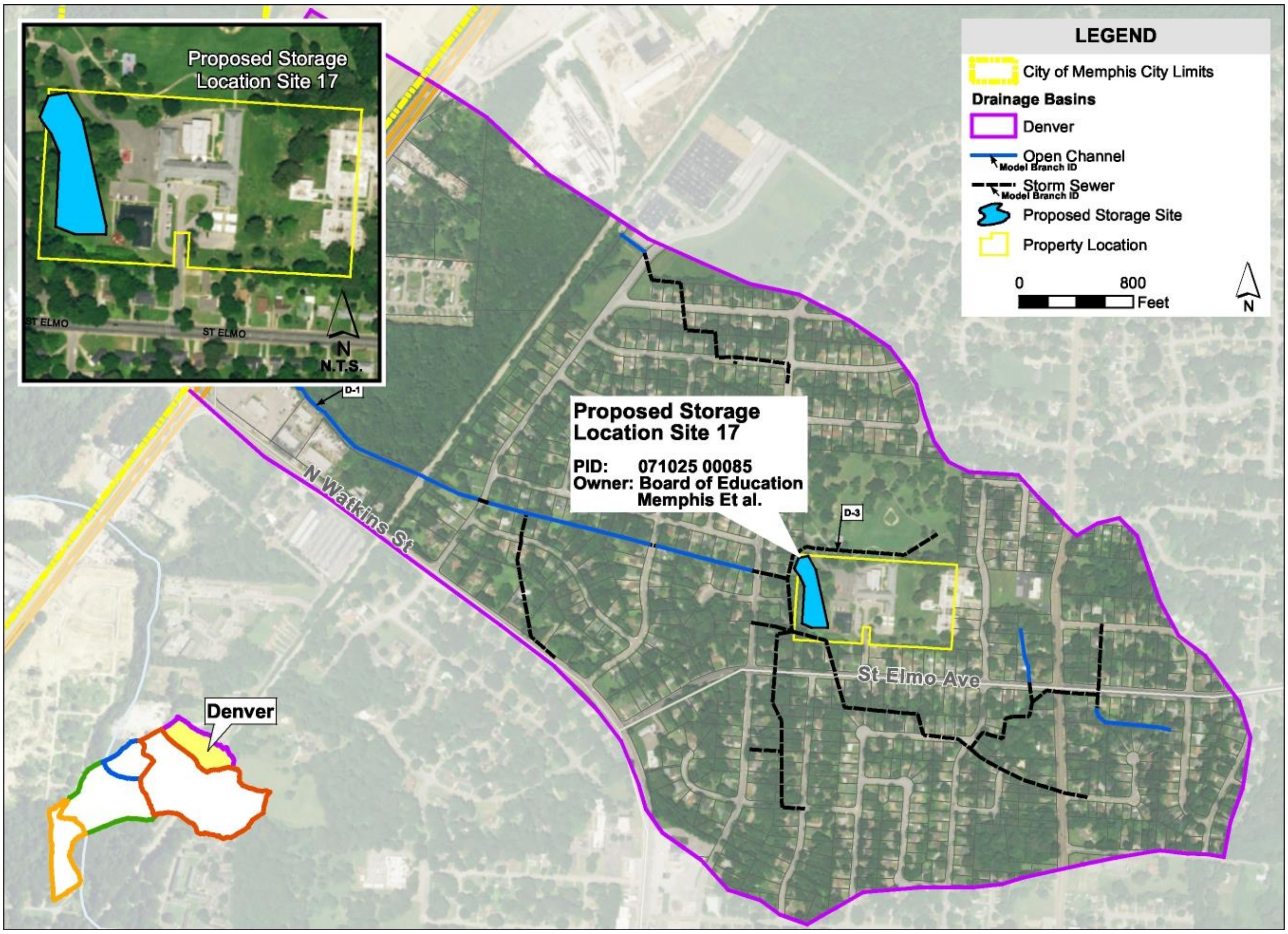
Figure 5.3: Conceptual Trezevant High School Storage Alternative



5.2.3 Site 17 – Georgian Hills Park Storage Alternative

Site 17 – Georgian Hills Park: This site is located at Georgian Hills Park near the intersection of Argonne and Kinglsey in the Denver System drainage area (D-3). The potential storage basin at this location consists of an 8 foot deep open basin with a maximum footprint of 59,000 square feet and total capacity of 360,000 cubic feet. The basin is fed by a 30- and 36-inch pipe with a 63 acre contributing drainage area. Using a 18-inch outlet pipe, the maximum depths obtained during the 10-year and 100-year design events are 5.3 feet and 7.8 feet, respectively. The planning-level cost opinion for Site 17 is $1,052,327. Figure 5.3 presents the conceptual planning improvements proposed at Site 17: Georgian Hills Park.

Figure 5.4: Conceptual Georgian Hills Park Storage Alternative



The proposed storage basins decrease the HGL and reduce the extents of the floodplain, but storage alone is not enough to eliminate the majority of the homes from the 10-year design event floodplain. The total planning-level cost opinion for the three recommended storage improvements is $2,597,000.

5.3 SYSTEM CAPACITY ALTERNATIVES

This section provides details on each of the evaluated alternatives that are recommended for the Todd Creek Study Area. The recommended improvements are summarized by watershed and branch ID which is also tied to the hydraulic profiles. Appendix C includes hydraulic profiles of the main branches in each watershed and all branched with a recommended improvement. Therefore not every minor branch from the model will have a hydraulic profile in Appendix C. Appendices G and H includes detailed floodplain delineations of the 10-year and 100-year proposed improvements, respectively. Appendix F provides enlarged plan-view figures of the proposed improvements including location and size. The planning-level costs opinions are summarized in Table 6.2. Appendix I includes a more detailed breakdown of planning-level cost opinions for each project.

5.3.1 Todd Creek System Capacity Alternatives

T-1 Alternative

The T-1 network has flooding that begins at the Frayser School Boulevard crossing and continues upstream to Trezevant Street. Severe flooding areas include properties along Haywood Avenue, Gowan Avenue, Riney Street, University Street, Frayser Boulevard, Mountain Terrace Street, Nunnelee Avenue, Brookmeade Street and Trezevant Street.

The T-1 alternative consists of the following improvements (listed upstream to downstream):

a. Storage Site 14 at Trezevent High School, east of Trezevant St. (refer to Section 5.2.2).
b. Upsizing 384 LF of 48-inch pipe to 4’ x 8’ box culvert from Trezevant St. to Brookmeade St.
c. Upsizing 580 LF of 48-inch to 5’ x 10’ box culvert from Brookmeade St. to Debby St.
d. Increase width of 428 LF of open concrete channel to 10’ from Debby St. to Stella St.
e. Replace 6’ x 6.5’ culvert with a 6’ x 13’ culvert at Stella St.
f. Increase width of 970 LF of open concrete channel to 13’ from Stella St. to Ardmore St. and widen the following road crossings:
    - Replace 5’ x 6.5’ culvert with a 5’ x 20’ culvert at Mountain Terrace St.
    - Replace 5.5’ x 10.5’ culvert with a 6’ x 20’ culvert at Ardmore St.
g. Increase width of 2,388 LF of open concrete channel to 30’ from Ardmore St. to east of Frayser Blvd. and widen the following road crossings:
    - Replace 6’x11’ bridge span with a 6’ x 30’ culvert at University St.
    - Replace 6.5’ x 13’ culvert with a 6.5’ x 30’ culvert at Woodrow St.
    - Replace 5’ x 15.5’ culvert with an 8’ x 30’ culvert at Overton Crossing St.
h. Increase width of 1,411 LF of open concrete channel to 40’ from northwest of Frayser Blvd/Watkins St. intersection to Frayser School Dr.
i. Replace 6.75’x40’ bridge span with a 6.75’ x 45’ culvert at Frayser School Dr.

By increasing the conveyance capacity at these locations, 29 structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Forty three (43) structures are removed from the zero-to-one-foot-depth floodplain and fourteen (14) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-1 alternative is $11,001,318.

The T-1 secondary storage alternative analysis indicates that approximately 43 acre-feet of storage along T-1 would be required in addition to Storage Site 14 to eliminate the recommended capacity improvements west of Stella Street. The secondary storage sites included Site 32 with 38 acre-feet of storage, generally located between University Street and Mountain Terrace Street along Frayser Boulevard; and Site 11 with 15 acre-feet of storage near Watkins Street and Frayser Boulevard. Since there did not appear to be viable secondary storage locations not needing significant property acquisition and demolition, the secondary storage alternatives were not further investigated.

(Appendix F map pages: 1-5; Appendix G/H map pages: 1-5, 7, 8, 12-16, 21)

T-10 Alternative

The T-10 network is flooding along Frayser School Drive. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-10 alternative consists of upsizing 383 LF of 30-inch to 36-inch pipe and 422 LF of 36-inch to 42-inch pipe. By increasing the conveyance capacity at these locations, one (1) structure will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. One (1) structure is removed from the zero-to-one-foot-depth floodplain and seven (7) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-10 alternative is $556,728.

(Appendix F map page: 6; Appendix G/H map page: 12)

T-11 Alternative

The T-11 network is flooding along Gowan Avenue. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-11 alternative consists of upsizing 291 LF of 42-inch to 48-inch pipe and 291 LF of 42-inch to 54-inch pipe. By increasing the conveyance capacity at these locations, one (1) structure is removed from the zero-to-one-foot-depth floodplain and one (1) will remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-11 alternative is $445,740.

The T-11 secondary storage alternative analysis indicates that approximately one acre-foot of storage along T-11 would be required to eliminate the recommended capacity improvements. No secondary storage alternative sites were identified along T-11 that would not need significant property acquisition and demolition, therefore secondary storage alternative was not further investigated.

(Appendix F map page: 7; Appendix G/H map pages: 12, 13)

T-13 Alternative

The T-13 network areas of concern include flooding along Corner Drive and Riney Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-13 alternative consists of upsizing 873 LF of 24-inch to 42-inch pipe, 177 LF of 36-inch to 42-inch pipe, 183 LF of 48-inch to 72-inch pipe at a new gradient, 445 LF of 54-inch to 72-inch pipe and 1,119 LF of 60-inch to 72-inch pipe. By increasing the conveyance capacity at these locations, two (2) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Two (2) structures are removed from the zero-to-one-foot-depth floodplain and five (5) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-13 alternative is $2,512,940.

The T-13 secondary storage alternative analysis indicates that approximately seven acre-feet of storage along T-13 would be required to eliminate the recommended capacity improvements. Storage Alternate Site 16 (4 acres) was identified as a storage area along T-13. However, this location would requiresignificant property acquisition and demolition, therefore the secondary storage alternative was not further investigated.

(Appendix F map pages: 8, 9; Appendix G/H map pages: 13, 12, 18, 19)

T-14 Alternative

The T-14 network areas of concern include flooding near the intersection of Julia Street and Pamela Drive and along Hallbrook Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-14 alternative consists of upsizing 98 LF of 27-inch to 36-inch pipe, 582 LF of 30-inch to 36-inch pipe near Pamela and 205 LF of 3' x 4' elliptical to 3' x 8' elliptical pipe, 336 LF of 3' x 5' elliptical to 3' x 8' elliptical pipe from Overton Crossing Street to Denver, Site 7 storage (refer to storage alternative in Section 5.2.1) through Denver Park, upsizing 174 LF of 4' x 5' elliptical pipe to 5' x 10' box culvert near Hallbrook Street, an increased 661 LF of open channel width by 10' from Hallbrook Street to Watkins Street and upsizing 474 LF of 72-inch pipe to 7' x 10' box culvert from Watkins Street to the confluence of the Todd Creek main branch.

The T-14.2 alternative consist of upsizing 83 LF of 27-inch to 36-inch pipe, upsizing 243 LF of 27-inch to 60-inch pipe, 501 LF of 30-inch to 60-inch pipe, and removing 234 LF of 27-inch hydraulic contraction to 60-inch pipe.

By increasing the conveyance capacity at these locations, nine (9) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Twenty (20) structures are removed from the zero-to-one-foot-depth floodplain and nine (9) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-14 alternative is $3,636,834.

The T-14 secondary storage alternative analysis indicates that approximately thirteen acre-feet of storage along T-14, and two acre-feet along T-14.2 would be required to eliminate the recommended capacity improvements. Two Alternate Storage Sites were identified along T-14; Site 7 – 21 acres and Site 31 – 1 acre. However, both sites would require significant property acquisition and demolition, and no sites were identified along T-14.2, therefore the secondary storage alternative was not further investigated.

(Appendix F map pages: 10-12; Appendix G/H map pages: 13, 9, 10, 25)

T-17 Alternative

The T-17 network areas of concern include flooding at the upper reach along Whitney Avenue and the lower reach along Frayser View Drive and Aden Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-17 alternative consists of upsizing 505 LF of 24-inch to 48-inch pipe along Whitney Avenue, upsizing 216 LF of 36-inch to 60-inch pipe, 588 LF of 42-inch to 60-inch pipe, 971 LF of 48-inch to 60-inch pipe, 425 LF of 4' x 6.3' elliptical pipe to 5' x 6.3' elliptical pipe, 648 LF of 4' x 6.3' elliptical pipe to 6' x 6.3' elliptical pipe south of Whitney to the dead end of Frayser View Drive, and upsizing 777 LF of 60-inch to 84-inch pipe along Frayser View Drive.

By increasing the conveyance capacity at these locations, five (5) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Nine (9) structures are removed from the zero-to-one-foot-depth floodplain and five (5) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-17 alternative is $3,675,232.

The T-17 secondary storage alternative analysis indicates that approximately six acre-feet of storage along T-17 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 19 (6 acres) was identified as a potential storage site along T-17. However, since significant property acquisition and demolition would be required, the secondary storage alternative was not further investigated.

(Appendix F map pages: 13, 14; Appendix G/H map pages: 13, 19, 22)

T-19 Alternative

The T-19 network areas of concern include flooding along Overton Crossing Street and near the intersection of University Street and Slocum Avenue. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-19 alternative consists of upsizing 275 LF of 24-inch to 42-inch pipe, 388 LF of 36-inch to 60-inch pipe and upsizing 672 LF of 42-inch to 60-inch pipe between Overton Crossing Street and Woodrow Street.

The T-19.1 alternate consists of upsizing 207 LF of 42-inch to 60-inch pipe at the intersection of University Street and Slocum Avenue.

By increasing the conveyance capacity at these locations, three (3) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. No structures are removed from the zero-to-one-foot-depth floodplain and eight (8) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-19 alternative is $826,110.

The T-19 secondary storage alternative analysis indicates that approximately two acre-feet of storage along T-19 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 18 (1 acre) was identified as a potential storage site along T-19. However, significant property acquisition and demolition would be required, therefore the secondary storage alternative was not further investigated.

(Appendix F map page: 15; Appendix G/H map pages: 14, 20)

T-20 Alternative

The T-20 network main areas of concern include flooding at the intersection of Corning Avenue and Gayle Drive, and along University south of Cassie. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-20 alternative consists of upsizing 609 LF of 36-inch to 42-inch pipe between Corning Avenue and Pamela Drive, increase the width of 222 LF of open concrete channel between Theda Avenue and Cassie Avenue including upsizing the 4’ x 6.33’ horizontal elliptical culvert to 5’ x 15’ box culvert at Cassie Avenue, upsizing 933 LF of 4’ x 6.33’ horizontal elliptical pipe to 5’ x 10’ box culvert from south of Cassie Avenue to Frayser Boulevard and upsizing 322 LF 5’ x 6’ culvert to 5’ x 10’ box culvert from south of Frayser Boulevard to the confluence with the main branch of Todd Creek.

By increasing the conveyance capacity at these locations, six (6) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Three (3) structures are removed from the zero-to-one-foot-depth floodplain and six (6) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-20 alternative is $2,825,090.

The T-20 secondary storage alternative analysis indicates that approximately 11 acre-feet of storage along T-20 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 12 (2 acres) was identified as a potential storage site along T-20. However, significant property acquisition and demolition would be required, therefore the secondary storage alternative was not further investigated.

(Appendix F map pages: 16, 17; Appendix G/H map pages: 14, 10)

T-22 Alternative

The T-22 network has flooding at the intersection of Jan Drive and Mountain Terrace Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-22 alternative consists of upsizing 573 LF of 36-inch to 60-inch pipe, upsizing 365 LF of 48-inch to 60-inch pipe, and upsizing 386 LF of 54-inch to 60-inch pipe. By increasing the conveyance capacity at these locations, five (5) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Three (3) structures are removed from the zero-to-one-foot-depth floodplain and two (2) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-22 alternative is $1,313,208.

The T-22 secondary storage alternative analysis indicates that approximately two acre-feet of storage along T-22 would be required to eliminate the recommended capacity improvements. A four acre site, Alternate Storage Site 10, was identified as a possible storage site at the upper reach of T-22. However, since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 18; Appendix G/H map page: 15)

T-23 Alternative

The T-23 network has flooding in the backyards along Brookmeade Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-23 alternative consists of upsizing 663 LF of 30-inch to 42-inch pipe and upsizing 654 LF of 36-inch to 42-inch pipe. By increasing the conveyance capacity at these locations, two (2) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. No structures are removed from the zero-to-one-foot-depth floodplain and four (4) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-23 alternative is $580,572.

The T-23 secondary storage alternative analysis indicates that approximately one acre-feet of storage along T-23 would be required to eliminate the recommended capacity improvements. No alternate storage sites were identified along T-23. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 19; Appendix G/H map page: 15)

T-24 Alternative

The T-24 network is flooding at the upper reach along Madeline Circle. The flooding is due to limited capacity caused by an offset pipe within the existing system. There are no existing sewers in the vicinity with excess capacity or hydraulically accessible storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The T-24 alternative consists of replacing 28 LF of 27-inch pipe with 27-inch pipe and removing the existing offset. By removing the offset at this location, one (1) structure will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. No structures are removed from the zero-to-one-foot-depth floodplain and two (2) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for T-24 alternative is $40,170.

(Appendix F map page: 20; Appendix G/H map pages: 15, 21)

5.3.2 Denver Trunk System Capacity Alternatives

D-1 Alternative

The D-1 network areas of concern include flooding at the upper reach between Overton Crossing Street and Denver Street, and near the intersection of Argonne Street and Leyton Cove. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or hydraulically accessible storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The D-1 alternative consists of upsizing 170 LF of 30-inch to 42-inch pipe, upsizing 90 LF of 30-inch to 72-inch pipe, upsizing 109 LF of 36-inch to 72-inch pipe, upsizing 250 LF of 42-inch to 72-inch pipe, upsizing 1,152 LF of 5' x 9' culvert to 5' x 10' culvert, upsizing 621 LF of 60-inch to 96-inch pipe and upsizing 268 LF of 84-inch to 96-inch pipe.

By increasing the conveyance capacity at these locations, three (3) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. One (1) structures are removed from the zero-to-one-foot-depth floodplain and three (3) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for D-1 alternative is $3,449,169.

The D-1 secondary storage alternative analysis indicates that approximately 2 acre-feet of storage, generally located between Belleau Street and Denver Street along St. Elmo (Alternate Storage Site 25 – 15 acres), would be required to eliminate the recommended capacity improvements between Argonne Street and Denver Street. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 21; Appendix G/H map pages: 2, 3, 23, 6, 25)

D-2 Alternative

The D-2 network areas of concern include flooding along Carlyle Avenue and Watkins Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The D-2 alternative consists of upsizing 497 LF of 36-inch to 54-inch pipe, upsizing 405 LF of 36-inch to 60-inch pipe, upsizing 40 LF of 4' x 5' elliptical to 4' x 6' elliptical pipe and upsizing 165 LF of 42-inch to 48-inch pipe. By increasing the conveyance capacity at these locations, one (1) structure is removed from the zero-to-one-foot-depth floodplain and two (2) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for D-2 alternative is $754,021.

The D-2 secondary storage alternative analysis indicates that approximately one acre-feet of storage along D-2 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 1 (2 acres) was identified as a potential storage site along D-2. However, since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 22; Appendix G/H map pages: 3, 23, 6)

D-3 Alternative

D-3 refers to storage Site 17 which is presented in Section 5.2.1.

(Appendix F map page: 23; Appendix G/H map pages: 23, 24)

D-4 Alternative

The D-4 network areas of concern include flooding between Argonne Street and Belleau Street, from Townsend Avenue to St. Elmo Avenue. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The D-4 alternative consists of upsizing 350 LF of 30-inch to 48-inch pipe. By increasing the conveyance capacity at these locations, one (1) structure will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Three (3) structures are removed from the zero-to-one-foot-depth floodplain and two (2) will remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost for D-4 alternative is $165,662.

The D-4 secondary storage alternative analysis indicates that approximately five acre-feet of storage along D-4 would be required to eliminate the recommended capacity improvements. No alternate storage sites were identified along D-4. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 24; Appendix G/H map pages: 23, 6)

D-5 Alternative

The D-5 network areas of concern include flooding in streets and yards between Leyton Avenue and St. Elmo Avenue, and along the storm sewer system from St. Elmo Avenue to Denver Street. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The D-5 alternative consists of upsizing 498 LF of 24-inch to 36-inch pipe. By increasing the capacity at this location, two (2) structures are removed from the zero-to-one-foot-depth floodplain and seven (7) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for D-5 alternative is $217,047.

The D-5 secondary storage alternative analysis indicates that approximately two acre-feet of storage along D-5 would be required to eliminate the recommended capacity improvements. No alternate storage sites were identified along D-5. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 25; Appendix G/H map pages: 25, 24)

D-6 Alternative

The D-6 network areas of concern include flooding along Coventry Drive. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The D-6 alternative consists of upsizing 179 LF of 30-inch to 48-inch pipe, 207 LF of 30-inch to 54-inch pipe, 336 LF of 36-inch to 54-inch pipe and 203 LF of 42-inch to 54-inch pipe. By increasing the capacity at these locations, three (3) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Two (2) structures are removed from the zero-to-one-foot-depth floodplain and two (2) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for D-6 alternative is $530,188.

The D-6 secondary storage alternative analysis indicates that approximately four acre-feet of storage would be required along D-6 to eliminate the recommended capacity improvements. No alternate storage sites were identified along D-6. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 26; Appendix G/H map pages: 23, 40)

5.3.3 WMPS System Capacity Alternatives

W-1 Alternative

The W-1 network has flooding located at the Canadian National Railroad, along Frayser Boulevard between Thomas Street (US Highway 51) and Dawn Drive, and at the Benjestown and unnamed road crossings. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-1 alternative consists of upsizing 636 LF of 24-inch to 42-inch pipe to eliminate the existing hydraulic contraction at the upstream railro ad crossing, upsizing 1,044 LF of the 4’ x 9’ to 5’ x 10’ box culvert along Frayser Boulevard, replacing the double 6’ x 8’ culvert with a 8’ x 32’ span culvert at Benjestown Street and replacing the double 5’ x 7’ box culvert with a 10’ x 40’ span culvert at the unnamed road.

By increasing the conveyance capacity at these locations, two (2) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Three (3) structures are removed from the zero-to-one-foot-depth floodplain and 12 remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-1 alternative is $2,588,077.

(Appendix F map pages: 27-29; Appendix G/H map pages: 30, 27, 28, 31, 32, 35, 17)

W-3 Alternative

The W-3 network has flooding located along Harvester Lane. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-3 alternative consists of upsizing 576 LF of 24-inch to 36-inch pipe. By increasing the conveyance capacity at this location, one (1) structure is removed from the greater-than-one-foot-depth floodplain and eight (8) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-3 alternative is $190,911.

The W-3 secondary storage alternative analysis indicates that approximately one acre-feet of storage along W-3 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 20 (< 1 acre) was identified as a potential storage site in the upper reach of W-3. However, since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 30; Appendix G/H map pages: 27, 28, 31, 30, 33)

W-4 Alternative

The W-4 network has flooding located along Sunrise Street. The flooding is caused by limited capacity due to a 30-inch to a 24-inch hydraulic contraction within the existing system along Par Avenue. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-4 alternative consists of upsizing 135 LF of 24-inch to 30-inch pipe to eliminate the existing hydraulic contraction along Par Avenue. By increasing the conveyance capacity at this location, four (4) structures remain within the greater-than-one-foot-depth floodplain and two (2) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-4 alternative is $101,734.

The W-4 secondary storage alternative analysis indicates that approximately one acre-feet of storage would be required along W-4 to eliminate the recommended capacity improvements. No alternate storage sites were identified along W-4. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 31; Appendix G/H map pages: 28, 26)

W-6 Alternative

The W-6 network has flooding located along Dawn Drive. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-6 alternative consists of upsizing 435 LF of 42-inch to 54-inch pipe and upsizing 1,275 LF of 2.17' x 6.5' elliptical pipe to 54-inch pipe to reduce the flooding along Dawn Drive. By increasing the conveyance capacity at this location, six (6) structures will be removed from the zero-to-one-foot-depth floodplain and nine (9) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-6 alternative is $1,342,839.

The W-6 secondary storage alternative analysis indicates that approximately three acre-feet of storage along W-6 would be required to eliminate the recommended capacity improvements. No alternate storage sites were identified along W-6. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 32; Appendix G/H map pages: 28, 29)

W-7 Alternative

The W-7 network has flooding located along Millington Street and Par Avenue. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-7 alternative consists of upsizing 393 LF of 30-inch to 36-inch pipe and upsizing 783 LF of 36-inch to 48-inch pipe to reduce the flooding along Par Avenue. By increasing the conveyance capacity at this location, two (2) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Three (3) structures are removed from the zero-to-one-foot-depth floodplain and six (6) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-7 alternative is $820,626.

The W-7 secondary storage alternative analysis indicates that approximately three acre-feet of storage along W-7 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 30 (11 acres) was identified as a potential storage site along W-7. However, since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 33; Appendix G/H map pages: 31, 32, 29)

W-8 Alternative

The W-8 network areas of concern include flooding along Kingston Street, between Klinke Avenue and Morningside Drive, near the intersection of Marsh Avenue and Northgate Street, and along Dawn Drive near Juliet Avenue. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-8 alternative consists of upsizing 44 LF of 24-inch to 48-inch pipe, upsizing 1,246 LF of 36-inch, 48 LF of 42-inch, 97 LF of 48-inch and 134 LF of 52-inch to 54-inch pipe between Klinke Avenue and Morningside Street, upsizing 89 LF of 54-inch to 5' x 10' box culvert, and 467 LF of 4' x 9' culvert to a 5' x 15' culvert near Juliet Avenue, increase 627 LF of open channel by 14' along Dawn Drive and increase 1,237 LF of the 4.5' x 7' culvert to a 5' x 20' culvert between Westside Drive and Deb Drive to reduce flooding depths in this area.

The W-8.3 alternative consists of upsizing 177 LF of 24-inch to 36-inch pipe near the intersection of Marsh Avenue and Northgate Street.

The W-8.4 alternative consists of upsizing 453 LF of 24-inch to 36-inch pipe from Kingston Street to Marsh Avenue.

By increasing the conveyance capacity at this location, eleven (11) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year, 24-hour design storm event. Twenty one (21) structures are removed from the zero-to-one-foot-depth floodplain and 13 remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-8 alternative is $6,749,513.

The W-8 secondary storage alternative analysis indicates that approximately ten acre-feet of storage along W-8 would be required to eliminate the recommended capacity improvements. Alternate Storage Site 24 (1 acre) was identified as a potential storage site along W-8. However, since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map pages: 34, 35; Appendix G/H map pages: 31, 34)

W-9 Alternative

The W-9 network has flooding located along Thomas Street (US Highway 51) from Cindy Lane to Frayser Boulevard. The flooding is due to limited capacity within the existing system. There are no existing sewers in the vicinity with excess capacity or viable storage locations that can provide relief to the area. Therefore, the available option is to provide a new adequately sized sewer system.

The W-9 alternative consists of upsizing 771 LF of 60-inch to 84-inch pipe, 259 LF of 60-inch to 4' x 9' pipe to reduce flooding depths in this area. By increasing the conveyance capacity at this location, five (5) structures will be removed from the greater-than-one-foot-depth floodplain for the 10-year design event. Four (4) structures are removed from the zero-to-one-foot-depth floodplain and four (4) remain within the zero-to-one-foot-depth floodplain for the 10-year, 24-hour design storm event. The planning-level cost opinion for W-9 alternative is $1,149,370.

The W-9 secondary storage alternative analysis indicates that approximately seven acre-feet of storage along W-9 would be required to eliminate the recommended capacity improvements. No alternate storage sites were identified along W-9. Since there did not appear to be viable storage locations not needing significant property acquisition and demolition, the secondary storage alternative was not further investigated.

(Appendix F map page: 36; Appendix G/H map pages: 32, 35)

5.3.4 Carrolton System Capacity Alternatives

The Carrolton System is at or above full-pipe capacity causing localized flooding, however there are no structures within the greater than one foot depth floodplain for the 10-year, 24-hour design storm event. Therefore, no storage or capacity alternatives were evaluated for this system.

5.3.5 Memphis Downtown Airport System Capacity Alternatives

The Memphis Downtown Airport System does not appear to experience severe flooding and the modeled system appears to convey the 10-year, 24-hour design storm event flows. Therefore, no storage or capacity improvements were identified for this system.

5.4 FLOODPLAIN DELINEATIONS

The existing condition 10-year and 100-year floodplain delineations are shown in Appendices D and E , respectively. The proposed alternatives 10-year and 100-year floodplain delineations are shown Appendices G and H, respectively. The floodplains are presented with three depth classifications to help identify the severity of the flooding. The three classifications are:

- Green - zero to one foot depth
- Yellow - one to two feet in depth
- Red - greater than two feet in depth.

Figures 5.5 to 5.9 show the existing versus proposed 10-Year, 24-Hour floodplain results per drainage area.

Figure 5.5: 10-year, 24-Hour Design Storm Event Floodplain Delineations

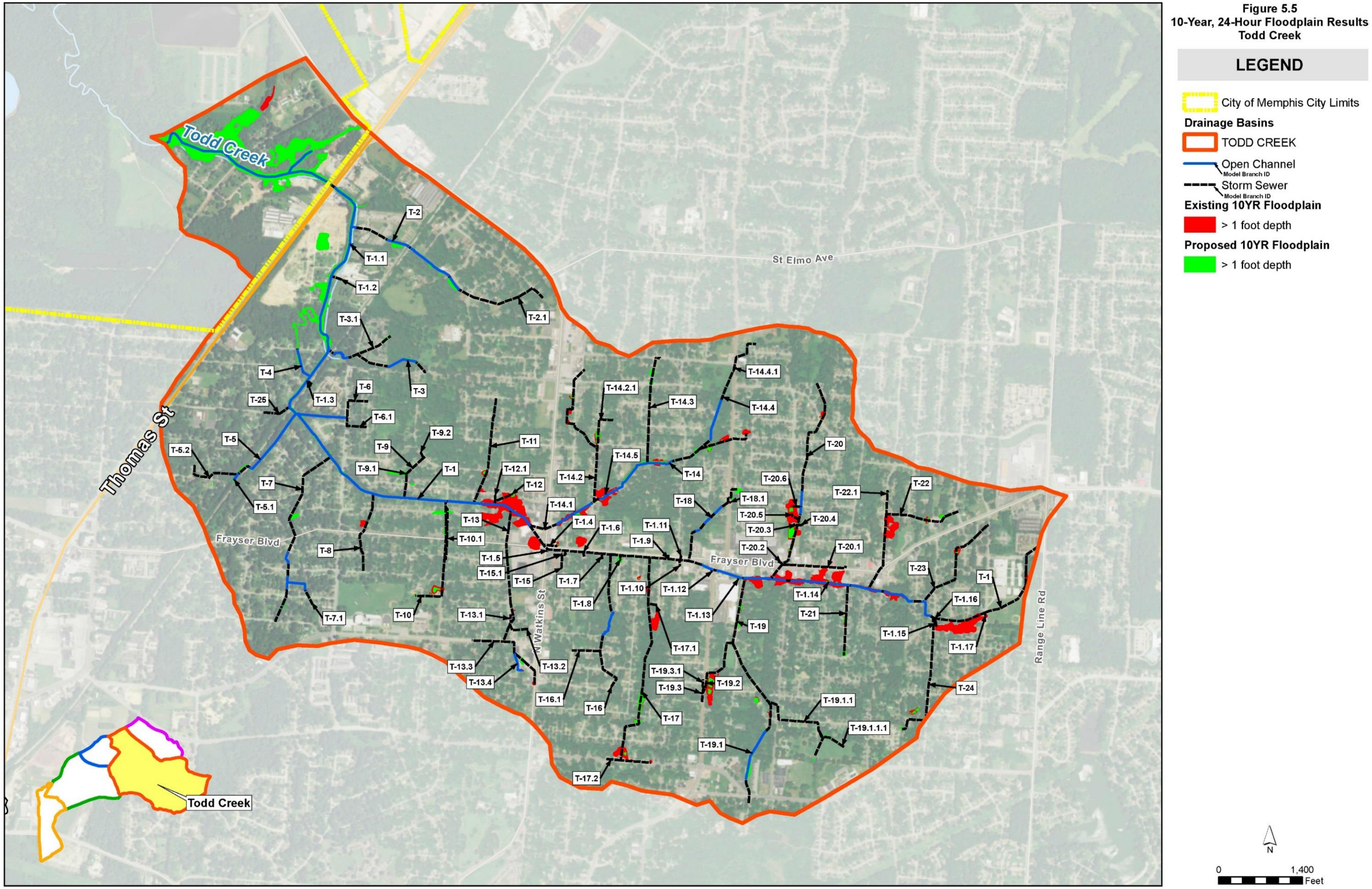


Figure 5.6: 10-Year, 24-Hour Floodplain Results – Denver Area

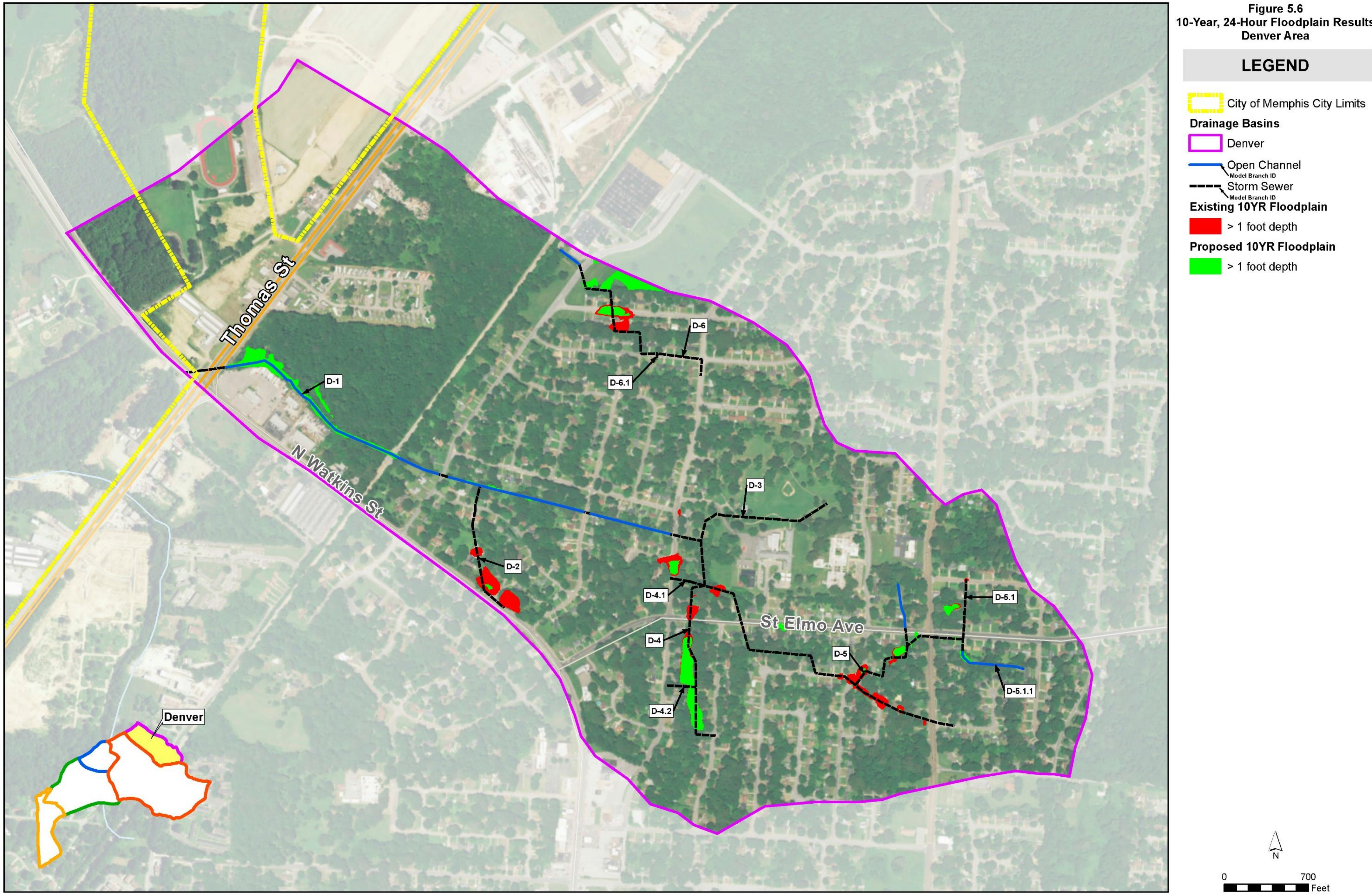


Figure 5.7: 10-Year, 24-Hour Floodplain Results – WMPS Area

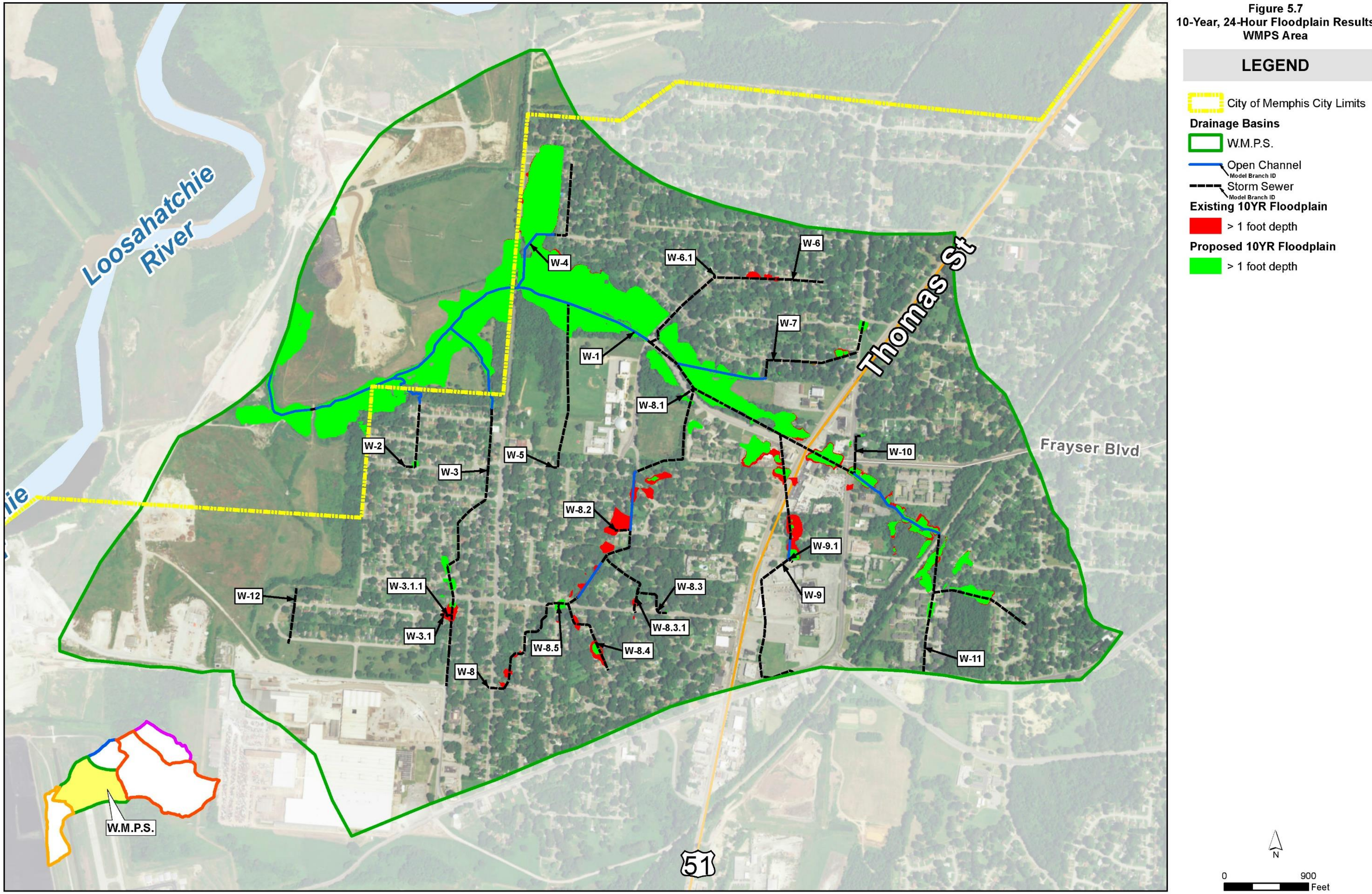


Figure 5.8: 10-Year, 24-Hour Floodplain Results – Carrolton Area

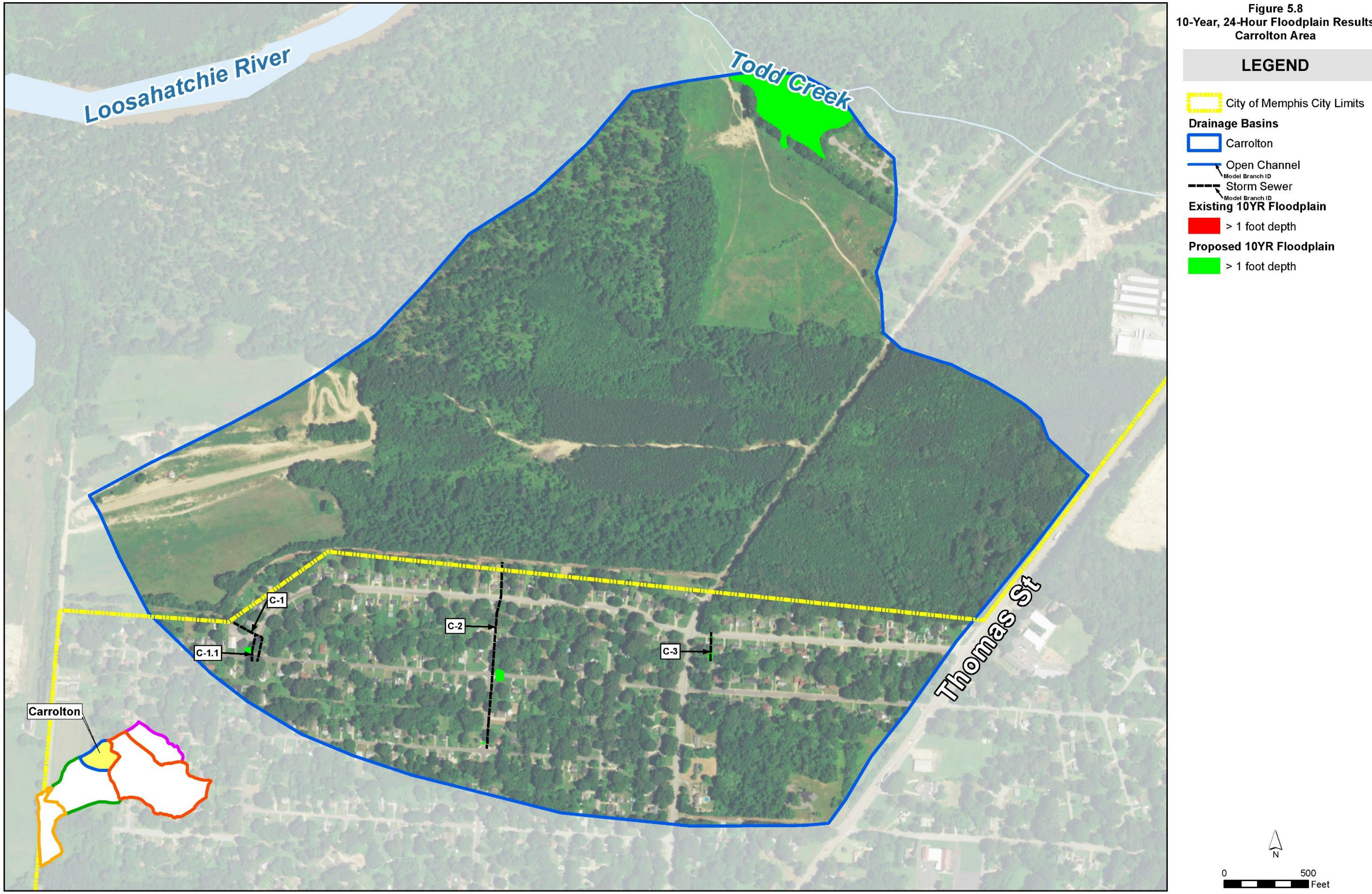
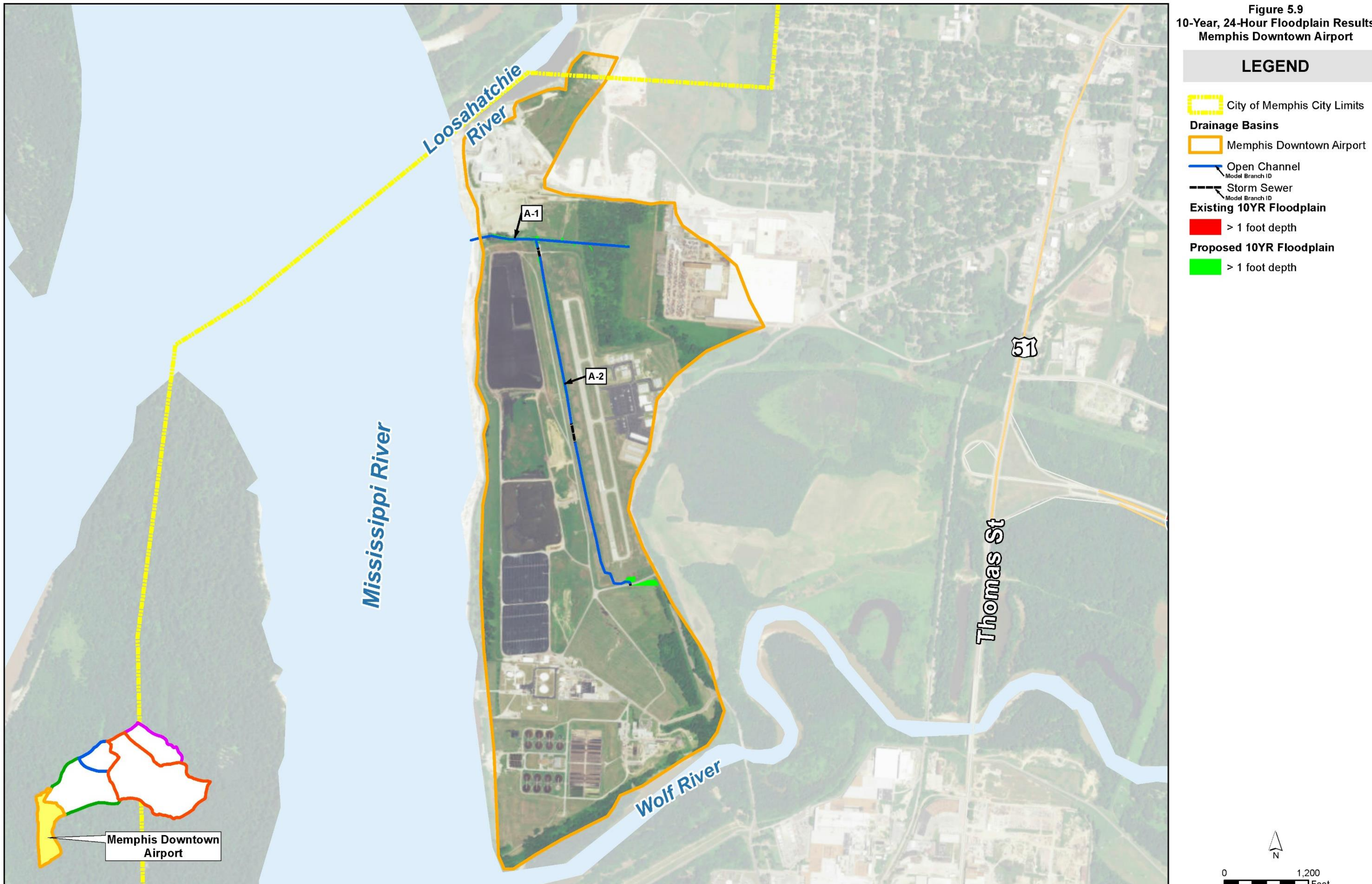


Figure 5.9: 10-Year, 24-Hour Floodplain Results – Memphis Downtown Airport



The following tables indicate the number of structures within each depth classification for the existing and proposed 10-year and 100-year design storm events per drainage area.

Table 5.1: Carrolton Flooded Structures with Improvements		
Flooding Depth (feet)	Proposed 10-year, 24-hour Floodplain	Proposed 100-year, 24-hour Floodplain
0 – 1	5	10
1 - 2	--	2
> 2	--	1
Total	5	13
Total > 1	--	3

Table 5.2: Denver Flooded Structures with Improvements		
Flooding Depth (feet)	Proposed 10-year, 24-hour Floodplain	Proposed 100-year, 24-hour Floodplain
0 – 1	16	35
1 - 2	--	24
> 2	--	2
Total	16	61
Total > 1	--	26

Table 5.3: Todd Creek Flooded Structures with Improvements		
Flooding Depth (feet)	Proposed 10-year, 24-hour Floodplain	Proposed 100-year, 24-hour Floodplain
0 – 1	82	160
1 - 2	--	50
> 2	--	10
Total	82	220
Total > 1	--	60

Table 5.4: WMPS Flooded Structures with Improvements		
Flooding Depth (feet)	Proposed 10-year, 24-hour Floodplain	Proposed 100-year, 24-hour Floodplain
0 – 1	54	104
1 - 2	2	35
> 2	2	10
Total	58	149
Total > 1	4	45

6.0 RECOMMENDATIONS

A series of alternatives have been developed and evaluated for the purpose of providing options to help alleviate flooding within the Todd Creek Study Area. These alternatives were based on the hydrologic and hydraulic model developed using survey, as-built data, and calibrated to water level and precipitation data. The alternatives provide the City with planning level cost opinions for Capital Improvement Plan budgeting. The City could realize cost savings if additional flow and precipitation data is collected to further refine the hydrologic and hydraulic model for sizing during final design efforts. The sizing and effectiveness of improvements assumes that all of the projects are implemented. For example, if an upstream project is completed prior to the constructing recommended downstream improvements, the downstream system may not have sufficient capacity which could lead to adverse impacts in those areas.

The recommended improvements have been prioritized based on constructability and the effectiveness of reducing residential flooding. Projects were prioritized as follows: storage improvements first, followed by conveyance improvements in a downstream to upstream manner. Projects located upstream of the main branches were prioritized based on the effectiveness of removing homes from the predicted floodplain. The number of homes within and removed from the 10-year and 100-year design storm events floodplains are independent of each other. Therefore number of homes removed for the 100-year event could be less than the 10-year event due to the location of specific residential structures within each floodplain and the floodplain reductions from each improvement. Table 6.1 summarizes the total approximate costs by priority level, and the anticipated total number of homes removed from the 10-year and 100-year floodplains, if all of the projects in each priority grouping are implemented.

Table 6.2 summarizes the recommended improvements for each project including type of improvement, priority level, approximate project cost, number of homes in the existing 10-year and 100-year floodplain, and anticipated number of homes removed from the 10-year and 100-year floodplains if the project is implemented. The Priority 1 improvements provide the most benefit and would remove approximately 233 residential structures from the 100-year floodplain at a cost of approximately $28.8M (or $124k per home removed). The Priority 2 and 3 improvements would remove a total of 23 residential structures from the 100-year floodplain at a cost of approximately $17.5M (or $763k per home removed). The Priority 2 and 3 projects have a significantly higher cost benefit ratio and is the main reason for their lower priority ranking.

The cumulative anticipated project cost for Priority 1, 2 and 3 improvements is $46.3M. To optimize the City's capital expenditure for the greatest positive impact, we recommend a phased implementation approach. We recommend implementing the Priority 1 improvements and then evaluating the remaining system to determine if isolated flooding continues to persist. If isolated flooding does continue to occur, the City can implement specific Priority 2 and/or 3 projects as needed to address specific issues.

Table 6.1: Summary of Improvement by Priority

Priority Level	Project ID's	Total Cost Opinion, $	Homes Removed from 10-Year Floodplain	Homes Removed from 100-Year Floodplain
1	T-1, T-14, T-24, D-1, D-4, W-1, W-8, W-9	$28,780,113	131	233
2	T-17, T-19, T-22, T-23, D-3, D-5, D-6	$8,194,000	23	14
3	T-10, T-11, T-13, T-20, D-2, W-4, W-6, W-7	$9,359,718	16	9

6.1 ADDITIONAL ACTION ITEM RECOMMENDATIONS

In addition to the priority recommendations listed in Table 6.2, the following recommendations are included:

1. Field verify existing hydraulic contractions located at the following locations:
    a. Intersection of Denver Street and Lauretta Avenue (D-5 branch, 42-inch to 36-inch)
    b. Canadian National Railroad crossing (W-1 branch, 42-inch to 24-inch)
    c. Intersection of Par Avenue and Sunrise Street (W-4 branch, 30-inch to 24-inch).
2. Field verify existing offset pipe located at the intersection of Madeline Circle and Debby Street (T-24 branch).
3. Cross-reference City recorded flooding complaints with the flooding areas identified by the model. This data could be used to alter the preliminary priority assigned to each storage and capacity alternative discussed in Section 5.

Table 6.2: Prioritization of the Recommended Improvements

Watershed	Priority Level	Project ID	Description	# of Homes in Existing 10-Year Floodplain	Homes Removed from 10-Year Floodplain[2]	Cost: Benefit Ratio ($/Home Removed 10-Year Floodplain)	# of Homes in Existing 100-Year Floodplain	Homes Removed from 100-Year Floodplain[2]	Cost: Benefit Ratio ($/Home Removed 100-Year Floodplain)	Planning-Level Project Cost Opinion, $[1]
Todd Creek	1	T-1	Capacity Improvements for T-1 Main Branch and Storage Site 14	86	72	$152,796	230	173	$63,591	$11,001,318
Todd Creek	1	T-14	Capacity Improvements for T-14 Branch and Storage Site 7	31	22	$165,311	55	31	$117,317	$3,636,834
Todd Creek	1	T-24	Capacity Improvements for T-24 Branch	2	0	N/A	4	1	$40,170	$40,170
Todd Creek	2	T-17	Capacity Improvements for T-17 Branch	17	12	$306,269	22	8	$459,404	$3,675,232
Todd Creek	2	T-19	Capacity Improvements for T-19 Branch	8	0	N/A	22	2	$413,055	$826,110
Todd Creek	2	T-22	Capacity Improvements for T-22 Branch	8	6	$218,868	11	2	$656,604	$1,313,208
Todd Creek	2	T-23	Capacity Improvements for T-23 Branch	4	0	N/A	6	1	$580,572	$580,572
Todd Creek	3	T-10	Capacity Improvements for T-10 Branch	9	2	$278,364	13	2	$278,364	$556,728
Todd Creek	3	T-11	Capacity Improvements for T-11 Branch	2	1	$445,740	6	3	$148,580	$445,740
Todd Creek	3	T-13	Capacity Improvements for T-13 Branch	6	1	$2,512,940	12	0	N/A	$2,512,940
Todd Creek	3	T-20	Capacity Improvements for T-20 Branch	11	5	$565,018	14	2	$1,412,545	$2,825,090
Denver	1	D-1	Capacity Improvements for D-1 Main Branch	6	3	$1,149,723	42	8	$431,146	$3,449,169
Denver	1	D-4	Capacity Improvements for T-4 Branch	5	3	$55,221	8	2	$82,831	$165,662
Denver	2	D-3	School Storage Site 17 at D-3 Branch	0	0	N/A	0	0	N/A	$1,052,327
Denver	2	D-5	Capacity Improvements for D-5 Branch	9	2	$108,524	9	0	N/A	$217,047
Denver	2	D-6	Capacity Improvements for D-6 Branch	5	3	$176,729	8	0	N/A	$530,188
Denver	3	D-2	Capacity Improvements for D-2 Branch	4	2	$377,011	5	1	$754,021	$754,021
WMPS	1	W-1	Capacity Improvements for W-1 Main Branch	14	2	$1,294,039	23	2	$1,294,039	$2,588,077
WMPS	1	W-8	Capacity Improvements for W-8 Branch	39	26	$259,597	66	12	$562,459	$6,749,513
WMPS	1	W-9	Capacity Improvements for W-9 Branch	7	3	$383,123	14	4	$287,343	$1,149,370
WMPS	3	W-4	Capacity Improvements for W-4 Branch	6	0	N/A	7	1	$101,734	$101,734
WMPS	3	W-6	Capacity Improvements for W-6 Branch	13	4	$335,710	26	1	$1,342,839	$1,342,839
WMPS	3	W-7	Capacity Improvements for W-7 Branch	9	3	$273,542	11	0	N/A	$820,626
Carrolton			No Recommended Improvement.							
Airport			No Recommended Improvement.							

Footnotes:

1 – See Appendix I for breakdown cost opinions for each project, with explanatory notes.

2 – Number of residential structures removed from the floodplain (not including out-buildings, garages or sheds).

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