City of Ann Arbor
Stormwater Model Calibration and Analysis Project

Final Report

June 1, 2015

CDM Smith

Contents

1. Executive Summary 1
  A. Purpose: 1
  B. Model Configuration: 1
  C. Major Project Outcomes: 2
2. Project Structure 4
  A. Task 1 – Phase I Public Engagement 4
  B. Task 2 – Preliminary Model Calibration and Validation 5
  C. Task 3 – Data Collection 6
  D. Task 4 – Final Model Calibration and Validation 12
  E. Task 5 – Phase I Documentation 13
  F. Task 6 – Procedures 14
  G. Task 7 – Training 14
  H. Task 8 – Phase II Public Engagement 14
  I. Task 9 – Model Analysis and Recommendations 14
  J. Task 10 – Verify FEMA Mapping 15
  K. Task 11 – Documentation 15
3. Stormwater Modeling 16
  A. Model background and calibration 16
    i. Storm Events for Calibration 16
    ii. Calibration Methods 17
    iii. Calibrated Model Parameters 17
    iv. Calibration Results 20
  B. Model Validation 24
    i. Validation Events 24
    ii. Validation Results 24
  C. Existing conditions modeling 26
4. Stormwater System Improvements 29
  A. Study Area Selection 29
  B. Improvements modeling 31

C. Site descriptions and recommendations 32
i. Lower Allen Creek 33
ii. Edgewood/Snyder 37
iii. Park Place Apartments 40
iv. Churchill Downs 42
v. East University/South University 50
vi. Mulholland Avenue 55
vii. Scio Church / S. Seventh Street 58
viii. Glendale/Charlton 62
ix. Glen Leven 65
x. Church Street / Cambridge Road 66
xi. Village Oaks / Chaucer Court 67
xii. Parkwood/Pittsfield Village 68
xiii. Signature Drive 71
xiv. South Industrial/Packard Road Area 72
xv. Traver/Barton 75
xvi. Glendale Circle at Virginia Park 77
xvii. Westgate and Maple Village Redevelopment 81
xviii. Plymouth and Green Road Redevelopment 83
D. Stormwater Improvement Conclusions 83
5. Stormwater Management Scenarios 85
A. Citywide Stormwater Management Scenarios 85
B. Future Conditions 89
6. FEMA Floodplain Comparison 91
7. Project Conclusions 95

Appendices

Appendix A - Existing Conditions System Capacity Analysis
Appendix B - Future Conditions Stormwater Management Analysis
Appendix C - Floodplain Comparison Maps

Figures

Figure 1-1 – Stormwater System Components 2
Figure 2-1 – 2013 Flow Monitor Locations and Tributary Areas 6
Figure 2-2 – Conceptual Description of Manning's Equation 8
Figure 2-3 – Rain Gauges for Final Model Calibration 9
Figure 2-4 – Large Event Data Gathering Sites 10
Figure 2-5 – Model Overland Flow Channels and 2D Surface Locations 13
Figure 3-1 – Hydrologic Soil Groups (HSG) for stormwater model areas 18
Figure 3-2a-d – Flow Hydrographs for Major Monitors for 6/27/13 Event 23
Figure 3-3a-d – Flow Hydrographs for Major Monitors for 8/12/13 Event 23
Figure 3-4a-d – Flow Hydrographs for Major Monitors for 6/13/13 Event 24
Figure 3-5 – Design Storm Events 26
Figure 3-6 – Cumulative Rainfall Distributions 27
Figure 3-7 – Example HGL Condition Map 28
Figure 4-1 – Allen Creek Stormwater System Overview 34
Figure 4-2 – Stormwater Improvement Comparison for Allen Creek at Madison Avenue 35
Figure 4-3 – Stormwater Improvement Comparison for Allen Creek at Hill Street 36
Figure 4-4 – Existing conditions results for Edgewood/Snyder (10% AEP, 12-hour storm) 37
Figure 4-5 – Conceptual Layout of Green Streets Alternative for Edgewood/Snyder 38
Figure 4-6 – Conceptual Layout of Storage Alternative for Edgewood/Snyder 38
Figure 4-7 – Conceptual Layout of Conveyance Alternative for Edgewood/Snyder 39
Figure 4-8 – Flow Hydrograph Comparison for Conveyance Alternative at Edgewood/Snyder 39
Figure 4-9 – Existing conditions results for Park Place Apartments (10% AEP, 12-hour storm) 41
Figure 4-10 – Conceptual Layout for Storage Alternative at Park Place Apartments 41
Figure 4-11 - Conceptual Layout for Conveyance Alternative at Park Place Apartments 42
Figure 4-12 – Flow Hydrograph Comparison for Conveyance Alternative at Park Place Apartments 42
Figure 4-13 – Existing conditions results for Churchill Downs (10% AEP, 12-hour storm) 43
Figure 4-14 – Conceptual Layout for Green Streets Alternative for Churchill Downs 44
Figure 4-15 – Water Surface Elevation comparison for Green Streets Alternative for Churchill Downs 45
Figure 4-16 – Conceptual Layout for Storage Alternative for Churchill Downs 47
Figure 4-17 – Conceptual Layout for Conveyance Alternative for Churchill Downs 48
Figure 4-18 – Water Surface Elevation Comparison for Conveyance Alternative at Churchill Downs 49
Figure 4-19 – Existing conditions results for East University (10% AEP, 12-hour storm) 50
Figure 4-20 – Conceptual Layout for Green Streets Alternative for East University 51
Figure 4-21 – Flow Hydrograph Comparison for Green Streets Alternative for East University 52
Figure 4-22 – Conceptual Layout for Green Streets/UM Detention Alternative for East University 52
Figure 4-23 – Flow Hydrograph Comparison for UM Detention Alternative for East University 53
Figure 4-24 – Conceptual Layout for Conveyance Alternative for East University 54
Figure 4-25 – Flow Hydrograph for Conveyance Alternative for East University 54
Figure 4-26 – Existing conditions results for Murray-Washington Drain at Mulholland Avenue 56
Figure 4-27 – Conceptual Layout for Surface Storage Alternative for Mulholland Ave 56
Figure 4-28 – Flow Hydrograph for Surface Storage Alternative for Mulholland Avenue 57

Figure 4-29 – Conceptual Layout for Conveyance Alternative for Mulholland Ave 57
Figure 4-30 – Existing conditions results for Scio Church / S. Seventh Street 59
Figure 4-31 – Conceptual Layout for Storage Alternative for Scio Church / S. Seventh Street 60
Figure 4-32 – Flow Hydrograph for Storage Alternative for Scio Church / S. Seventh Street 60
Figure 4-33 – Conceptual Layout for Conveyance Alternative for Scio Church / S. Seventh Street 61
Figure 4-34 – Flow Hydrograph for Conveyance Alternative for Scio Church / S. Seventh Street 62
Figure 4-35 – Existing conditions results for Glendale/Charlton 63
Figure 4-36 – Conceptual Layout for Upstream Detention Alternative for Glendale/Charlton 63
Figure 4-37 – Flow Hydrograph for Upstream Detention Alternative for Glendale/Charlton 64
Figure 4-38 – Conceptual Layout for Conveyance Alternative for Glendale/Charlton 64
Figure 4-39 – Existing conditions results for Glen Leven 66
Figure 4-40 – Existing conditions results for Church Street / Cambridge Road 66
Figure 4-41 – Conceptual Stormwater Improvements Layout for Church Street / Cambridge Road 67
Figure 4-42 – Existing conditions results for Village Oaks / Chaucer Court 67
Figure 4-43 – Conceptual Layout for Detention Alternative at Village Oaks/Chaucer Court 68
Figure 4-44 – Existing conditions results for Parkwood / Pittsfield 69
Figure 4-45 – Conceptual Layout for Pittsfield/Parkwood Storage/Conveyance 69
Figure 4-46 – Flow Hydrograph for Conveyance/Storage Alternative at Pittsfield/Parkwood 70
Figure 4-47 – Flow Hydrograph for Conveyance Alternative at Pittsfield/Parkwood 70
Figure 4-48 – Existing conditions results for Signature Drive 71
Figure 4-49 – Signature Drive Alternative Configuration 72
Figure 4-50 – Existing conditions results for South Industrial Area 73
Figure 4-51 – Conceptual Layout for Green Streets Alternative for S. Industrial Area 74
Figure 4-52 – Green Streets Alternative Pipe Capacity Results for S. Industrial Area 75
Figure 4-53 – Existing conditions results for Traver/Barton 76
Figure 4-54 – Conceptual Layout for Conveyance Improvement Alternative for Traver/Barton 76
Figure 4-55 – Ponding at Wooded Area behind Glendale Circle 77
Figure 4-56 – Location of Underground Storage at Virginia Park 78
Figure 4-57 – Location of Upstream Surface Storage for Glendale Circle / Virginia Park 79
Figure 4-58 – Commercial and Multi-Family Residential Parcels with Redevelopment Potential 80
Figure 4-59 – Existing conditions results for Westgate/Maple Village 82
Figure 4-60 – Model results for Redevelopment Scenario for Westgate/Maple Village 82
Figure 4-61 – Existing conditions results for Plymouth and Green Road 83
Figure 5-1 – Potential Infiltration for Green Street Application 86
Figure 5-2 – Residential Rain Gardens in the City of Ann Arbor 87
Figure 6-1 – Comparison of FEMA FIRM Effective and InfoSWMM Model Results 91

Tables

Table 2-1 – Flow Monitor Tributary Area Characteristics 7
Table 2-2 – List of Large Event Data Gathering Sites 11
Table 3-1 – Summary of Calibration and Validation Events 17
Table 3-2 – Green-Ampt Infiltration Parameters 19
Table 3-3 – Summary of % Runoff Routed to Pervious Surface Based on Land Use/Land Cover 19
Table 3-4 – % Difference for Model-Predicted Volume to Monitor-Observed Volume 21
Table 3-5 – % Difference for Model-Predicted Flow Rate to Monitor-Observed Flow Rate 22
Table 3-6 – Summary of Validation Events 24
Table 3-7 – % Difference for Model-Predicted vs. Monitor-Observed Volume/Peak Flow 25
Table 4-1 – Preliminary Study Area Prioritization 30
Table 4-2 – SWM project alignment with CIP scoring criteria 33
Table 4-3 – Recommended Edgewood/Snyder Option 40
Table 4-4 – Recommended Churchill Downs Solution 49
Table 4-5 – Recommended East University/South University Solution 55
Table 4-6 – Recommended Mulholland Drive Solution 58
Table 4-7 – Recommended Glendale/Charlton Solution 65
Table 4-8 – Recommended Parkwood/Pittsfield Village Solution 71
Table 4-9 – Recommended Signature and Waymarket Solution 72
Table 4-10 – Recommended Traver/Barton Solution 77
Table 4-11 – Recommended Glendale Circle at Virginia Park Solution 81
Table 4-12 – Summary of Recommended Stormwater Management Alternatives 84
Table 5-1 – Infiltration Standard Excerpted from Green Streets Policy 85
Table 5-2 – Future Scenarios Assumptions for Stormwater Management Strategies 89
Table 6-1 – Floodplain Delineation Data Sources 92
Table 6-2 – Comparison of FEMA FIRM to Model-based Floodplain Data 92
Table 6-3 – Floodplain Comparison Using LiDAR Contour Data Only 93

1. Executive Summary

The stormwater model calibration and analysis project (SWM project) began in July 2012 with an expected 2.5 - 3 year timeline. Preliminary model calibration was performed in 2012 using available data sources, additional calibration data was collected in 2013, and final model calibration and analysis using the collected information was completed in 2014. Project documentation, including this report, was finalized in early 2015.

A. Purpose:

The overall goal of the SWM project was to develop the computer model as a stormwater analysis tool for the entire City of Ann Arbor drainage system and to provide answers to the City's current stormwater system management questions. Specifically, the project developed to address the following objectives:

- Provide an accurate stormwater model of the entire City of Ann Arbor conveyance system, calibrated and validated using collected flow and rainfall data
- Involve stakeholders and interested citizens in the project to build awareness of the stormwater collection system and assist with the collection of stormwater system information for large rainfall events.
- Analyze existing stormwater system performance to determine the current level of service provided to the residents of the City of Ann Arbor and to recommend improvements to the stormwater system.
- Evaluate the effectiveness of potential stormwater management strategies to determine the return on these investments.
- Utilize the results of the updated model to provide a comparison point for the existing FEMA Flood Insurance Rate Map (FIRM) 100-year floodplain delineation.
- Implement a modeling strategy that will allow for flexibility to address climate change and other future changes with the stormwater system or with stormwater management policies.

B. Model Configuration:

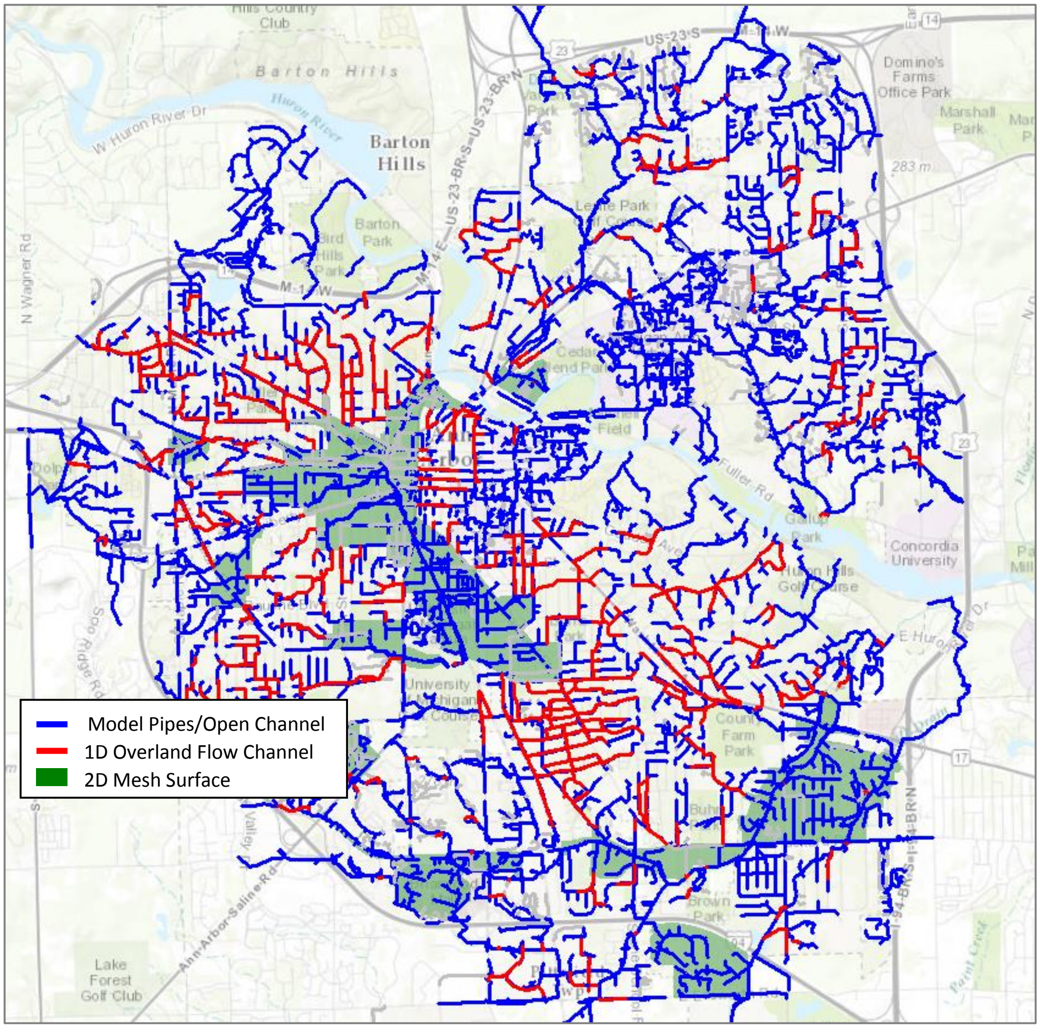
To accomplish these objectives, the stormwater model needed to include stormwater conveyance items beyond just stormwater pipes and open channels. The elements included in the analysis are presented in **Figure 1-1** on the next page, and described below:

- Catchment Areas – A detailed analysis of the areas tributary to the stormwater system inlets was performed in a previous phase of stormwater model development. These catchment areas and inlet locations were updated based on the stormwater data collection and analysis activities.
- Conveyance System – The stormwater computer model was developed using the available information collected in a previous phase of stormwater model development for stormwater inlets, pipes, manholes, open channels, 300 existing stormwater basins, and outfalls. The engineering characteristics of these elements including sizes, slopes, and material of construction were incorporated into the model setup to allow the stormwater conveyance through this network.
- Street Conveyance – Since the stormwater model was intended for simulation work for large events, it explicitly incorporated the street system as conveyance elements where this takes

place in the system. This provided an accurate representation of the movement of water throughout the City of Ann Arbor.

- Surface Storage/Conveyance – For more detailed simulations of the movement and extent of stored water, the surface storage and conveyance system in areas where stormwater was known to accumulate was explicitly incorporated into the stormwater model.

Figure 1-1 – Stormwater System Components



C. Major Project Outcomes:

The primary outcome of the SWM project is the delivery of the calibrated stormwater model itself. The City's investment in this project has allowed for the development of a tool for municipal stormwater management that is highly complex and refined. The model is capable of providing valuable information for various applications, from green infrastructure planning and stormwater system design, to floodplain analysis and emergency management. Output from the model for each of these applications can be relied upon confidently as the best information available. Most critically, the model can continue to be utilized easily and efficiently by the City to help optimize the allocation of stormwater utility funding.

Following are the major findings that developed from the stormwater analysis work:

- Majority of City Meets the Design Standard Level of Service – The analysis work has determined that the stormwater conveyance system is, in general, performing at a consistent design level of service for most areas of the City. The current stormwater system design standard for the City of Ann Arbor is the 10% annual exceedance probability (AEP), 12-hour storm. This storm is 2.9" of rainfall using NOAA Atlas 14 rainfall volumes. However, in the Allen Creek watershed and in the Malletts Creek watershed, there are areas where surface flooding is predicted during the 10% AEP storm and in some cases during the 20% AEP storm. It is important to note that design storm standards have increased periodically so that much of the City's stormwater system was designed and built to handle a smaller storm as compared to the current 10% AEP storm.
- Recommended Improvements Developed to Address Level of Service Concerns – To address these limitations in the level of service in these locations, a total of 16 study areas were evaluated for potential stormwater system improvements and these improvements were presented in a series of public meetings in November, 2014. The recommended improvements will be considered as part of the City's CIP Programming process. The total estimated capital cost of the recommended stormwater improvements was determined to be approximately $34 million in year 2017 dollars. These recommended improvements do not include the cost of long-term stormwater management strategies that were recommended specifically for the Allen Creek watershed, which are estimated to be another $80 million to $120 million.
- Green Streets and Rain Garden Policies Yield Expected Stormwater Benefits – The evaluation of stormwater management strategies under future implementation timelines indicated that the City should continue with incorporating the Green Streets Policy with street redesign projects and promoting the residential rain garden programs. There should also be significant efforts put into encouraging compliance with new development standards during redevelopment of commercial, multi-family, and school or University properties.
- FEMA Floodplain Comparison Developed – A floodplain delineation was performed using flow and water level data generated by the new InfoSWMM model for the 1% annual exceedance probability (AEP) storm. Using NOAA Atlas 14 rainfall volumes, this storm is a 5.11" rain event over 24 hours. The 1% AEP floodplain delineation generated using the newer data was compared with the existing FEMA Flood Insurance Rate Map (FIRM) floodplain contours.
- Project Documentation will Allow Continued Stormwater Analysis – Project documentation is being provided to the City, including archives of project files and model files. Training sessions and written procedures for model updates and storm scenario updates have been prepared that will allow City staff to continue to utilize the stormwater model as a system management tool.

2. Project Structure

This project is the second element of the stormwater system management program which the City of Ann Arbor (City) has implemented as follows:

- Stormwater GIS and Model Project (SGM); 2006-2009: This project included review of as-built drawings for stormwater system facilities, creation of a provisional geographic information system (GIS), collection of flow and rainfall data for large tributary areas, and conversion of the GIS to an InfoSWMM base hydraulic model. InfoSWMM is the hydraulic modeling software that was selected by the City of Ann Arbor to integrate modeling activities with the ArcGIS software which is used to manage the utility information. InfoSWMM software is constructed around the Environmental Protection Agency Stormwater Management Model (EPA SWMM) dynamic rainfall-runoff model.

- Stormwater Model Calibration and Analysis Project (SWM); 2012-2015: This project included two phases, with the first focused on calibration, and the second focused on analysis.

    - Phase I – Preliminary calibration, data collection, final calibration of the stormwater model

    - Phase II – Use of the calibrated model to perform an analysis of the level of services, review of the stormwater improvements needed to meet the level of service desired, and modeling to allow a comparison of the floodplain defined by the separate FEMA model analysis

This purpose of this report is to serve as a single source of project information, with a primary focus on the Phase II analysis, results, and recommendations.

Individual task summaries developed for the SWM project are provided as a reference, and directions to obtain more detailed versions of project documentation and output are included.

A. Task 1 – Phase I Public Engagement

The objective of this task was to understand the community issues and concerns with the management of stormwater that should be addressed throughout the project. It was also intended to gain an understanding of the specific stormwater-related questions and concerns in different sections of the city to help focus the modeling in these areas.

Work on Task 1 included development of a public engagement strategy, management of the City's project website, and the development of a stormwater advisory group (SWAG), which helped to plan and implement the public engagement strategy. The primary public engagement work item in Phase I was a series of seven public meetings held throughout the City during 2013 to gather information about experiences of the residents in these different areas with stormwater and their expectations for the City's stormwater management programs. This information was also obtained via a community-wide online stormwater survey that ran in parallel with the public outreach work. The Phase I public engagement effort was summarized in a Phase I Technical Memorandum, which can be found as part of the project file archive.

Another aspect of the Task 1 work was initial engagement with the City's Technical Oversight and Advisory Group (TOAG) for wet-weather projects. At interim steps during the project, City staff and/or CDM Smith staff presented project updates. Formal project presentations were made to the TOAG on March 20, 2014 at the end of final model calibration and on December 11, 2014, following the public meeting presentations. The TOAG group will also be assisting with review of the final project report in spring 2015.

B. Task 2 – Preliminary Model Calibration and Validation

The objective of this task was to utilize the stormwater model assembled under the prior project and utilize previously collected rainfall, flow, and level data to perform a preliminary calibration of the stormwater model. This version of the model was also validated using independent storm events to evaluate the stormwater model performance and to generate recommendations for model improvements.

During this task, model updates were made to incorporate recent changes in infrastructure or hydrology. A field verification task was utilized to perform additional field investigation to verify key topographic or hydraulic elevations. The model was also updated to account for physical inlet restrictions and for sump pump flows generated by the Footing Drain Disconnect (FDD) Program.

A preliminary model calibration effort was performed using stormwater flow and level data collected during the 2007 Stormwater GIS and Model (SGM) project. The 2007 data set was supplemented with records from long term USGS gauges located at the outlets of Allen Creek and Malletts Creek. Model simulation output was compared to the flow data, and the model parameters were iteratively adjusted to align model performance to be reflective of the measured data. Validation storms were used to evaluate model performance after calibration, which helped to understand locations where additional flow and rainfall data would be helpful to prepare a better model.

The preliminary calibration task was summarized in a preliminary calibration technical memorandum, which was provided to the City of Ann Arbor in 2013. The preliminary calibration report concluded that additional data collection and calibration should be performed for the following reasons:

- The dormant season model calibration was limited due by the lack of dormant season calibration events. Additional soil parameter calibration was needed to improve dormant season calibration.
- Provide additional support for upstream boundary conditions for locations where stormwater flows enter the City. The City's stormwater system does not extend into these areas but the stormwater behavior in these areas directly affects the City system and must be included in the model. These selected locations included Traver Creek at M-14 and Malletts Creek at I-94.
- Collect data for better model refinement in selected study locations. Since the 2007 data collection effort, large storms had highlighted collection system performance and level of service concerns in Malletts Creek and along lower Allen Creek. Additional monitoring of major branches of these creeksheds was recommended.
- Improve calibration and validation to meet percent difference goals of 15% on volume and 20% on peak flow, when comparing model-predicted values to monitored values.

C. Task 3 – Data Collection

The objectives of Task 3 were to develop a monitoring plan to collect additional flow and rainfall data and implement the monitoring plan at the selected locations.

Along with the three USGS stream gauges at the Allen Creek mouth, Doyle Park and the Malletts Creek mouth, a total of 15 temporary flow monitors were installed throughout the City and used to monitor system performance between March and November 2013 to support final model calibration efforts. **Figure 2-1** and **Table 2-1** show the location of these monitors and their tributary areas.

Figure 2-1 – 2013 Flow Monitor Locations and Tributary Areas

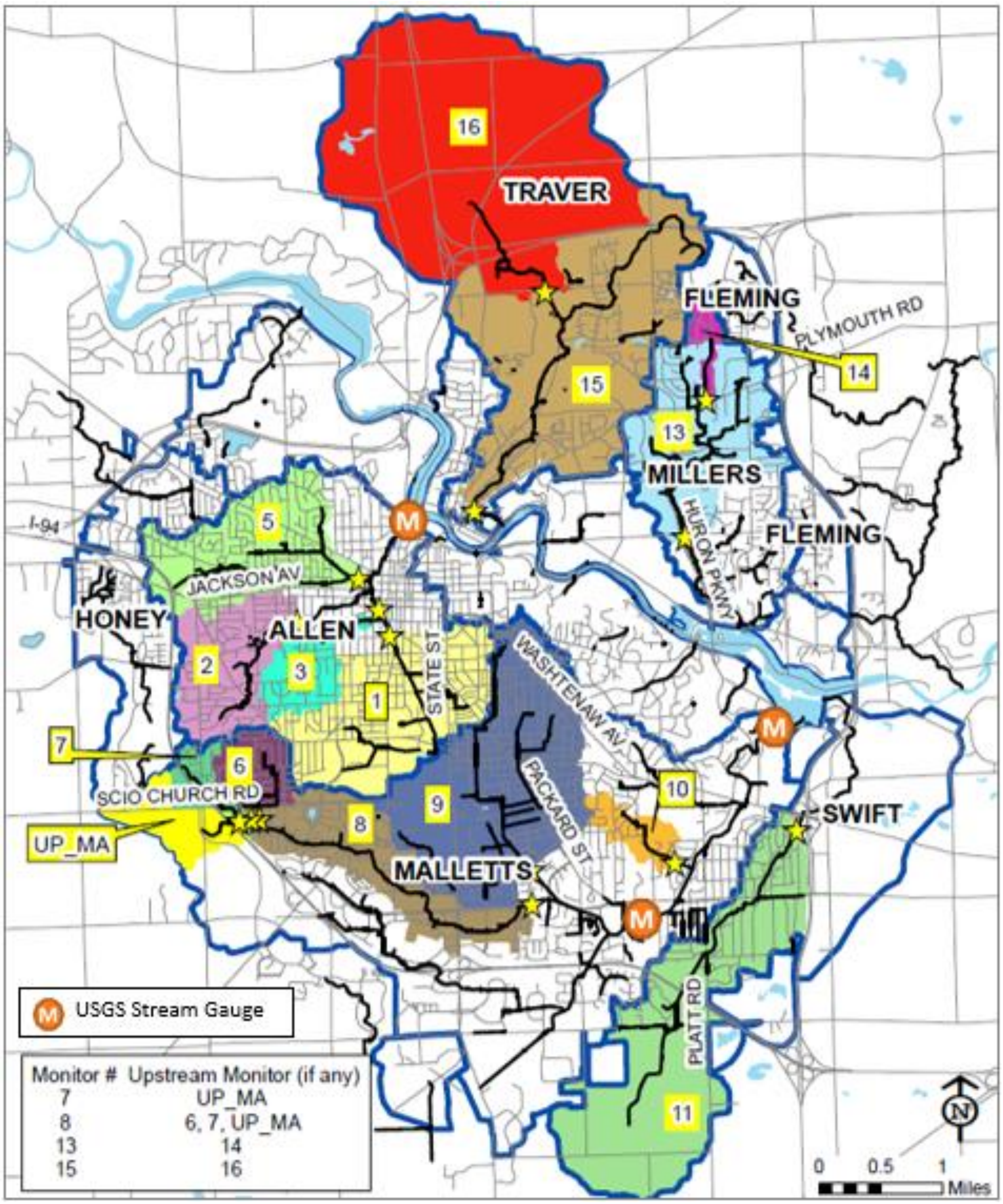


Table 2-1 – Flow Monitor Tributary Area Characteristics

Watershed	Site #	Area (acre)	Impervious %	Structure ID	Notes	Dates Installed (all 2013)
Allen	1*	1,003	55	92-61836	Upstream 30% of Allen Creek tributary area (south) In FEMA floodplain	3/29 – 11/26
	2*	557	48	92-60256	Murray-Washington Drain	
	3	244	41	92-60016	Eberwhite Drain 2007 Monitor Site #3	
	4	Number not used				
	5*	812	42	92-063256	Immediate downstream of 2007 Monitor Site #2. Monitor both branches west of West Park	3/29 – 6/22, 7/10 – 11/26
Malletts	6*	222	45	92-52016	2007 Monitor Site #6 Upstream of Lansdowne area	4/10 – 11/26
	7*	392	33	92-52033	Upstream of Lansdowne area (west of I-94)	
	8*	1,283	42	92-51565	Lansdowne + Eisenhower	3/29 – 11/26
	9*	1,459	45	92-50565	Portion of Malletts Creek tributary area with no in-line detention ponds	4/12 – 11/26
	10*	152	31	92-50865	2007 Monitor Site #10	4/10 – 11/26
	UP_MA*	228	30	92-52034	For Upper Malletts Creek project Township area bounded by I-94, Oak Valley Blvd, AA District Library and Scio Church Rd	5/10 – 11/26
Swift Run	11*	1,631	18	91-51339	Swift Run before exiting Ann Arbor (level-only gauge)	4/30 – 6/23, 7/25 – 11/26
	12	Number not used				
Millers	13	969	38	91-51591	Downstream monitoring	4/12 – 11/26

Watershed	Site #	Area (acre)	Impervious %	Structure ID	Notes	Dates Installed (all 2013)
Traver	14	90	40	92-54857	Georgetown area	3/29 – 11/26
	15*	4,466	13	91-50318	Flow meter at the box culvert immediate downstream of HRWC level gauge	4/10 – 11/26
	16*	2,648	5	91-50193	Monitor runoff response from rural areas outside Ann Arbor	

***** Located in County Drain

For each location the area, imperviousness, structure identification number, and various comments are provided in the table. Except for the Swift Run site where only a level probe was installed, Teledyne ISCO 2150 area-velocity flow modules were deployed to measure level, velocity and flows at each site. Data were downloaded on-site and reviewed on a monthly basis. The collected information was corrected when data quality was deemed poor. Typically this was due to velocity sensor errors, but level data were generally available and consistent. Calculations based on the Manning's equation (see below) and stage-discharge relationships were developed for most of the sites to allow for correction of flow data using level only.

Figure 2-2 – Conceptual Description of Manning's Equation

$$Q = \frac{Cn}{n} AR^{2/3} S_f^{1/2}$$

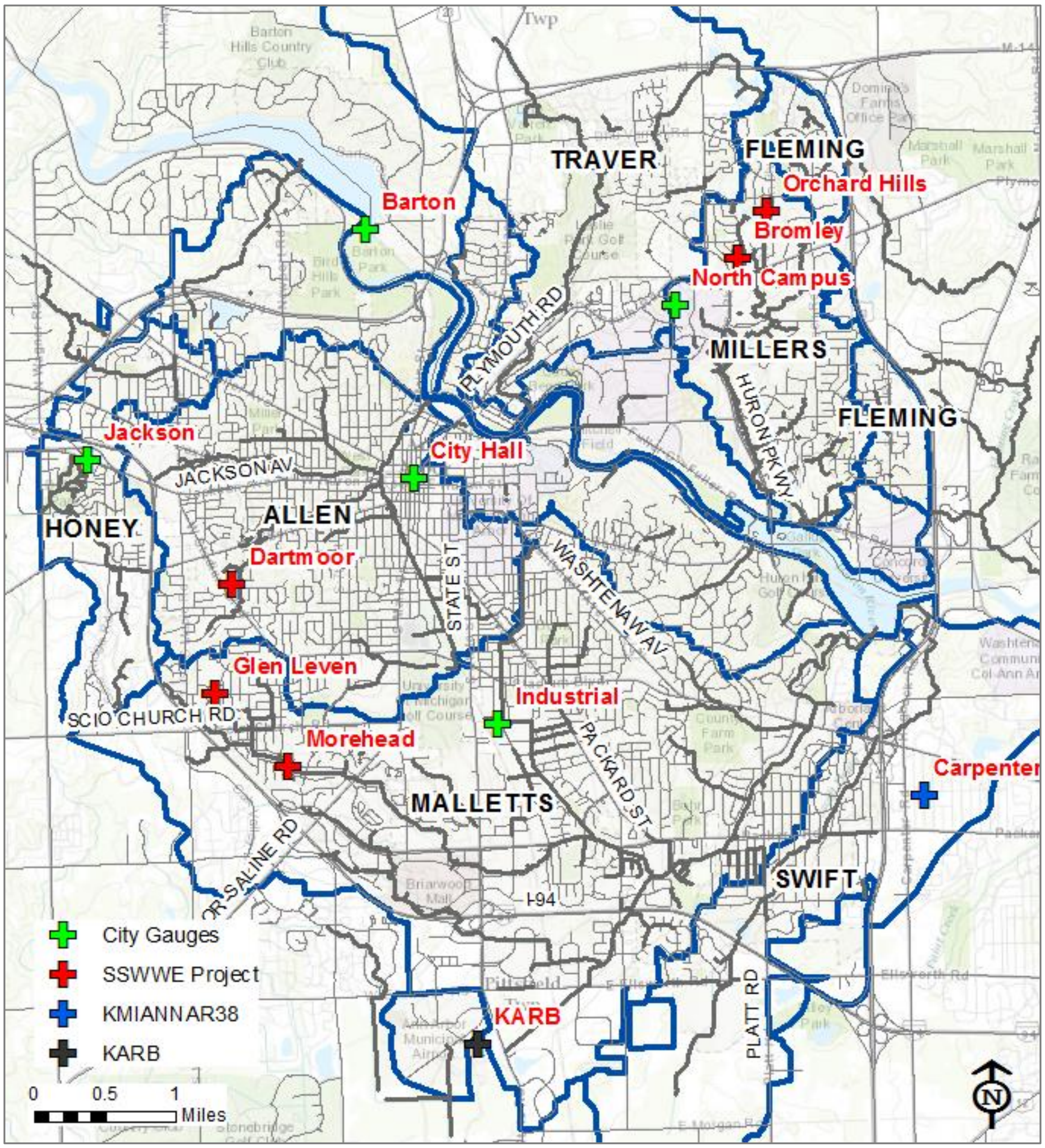
Q = Flowrate
$AR^{2/3}$ = Conveyance (depth, channel shape)
S_f = Energy Slope

Data from 12 ground-based rain gauges from different sources were collected to support the model calibration efforts. New rain gauges were installed at North Campus and at City Hall as part of this project. The gauges used during calibration included the following locations:

- Permanent City-maintained rain gauges: Barton Dam, Jackson Road, South Industrial, North Campus, City Hall
- Temporary rain gauges installed for the Sanitary Sewer Wet Weather Evaluation Project: Glen Leven, Morehead, Bromley, Dartmoor, Orchard Hills
- Rain gauges from National Oceanic and Atmospheric Administration (NOAA) / National Weather Service (NWS): KARB (located at Ann Arbor Airport)
- Carpenter Elementary School gauge (KMIANNAR38) available on Weather Underground

Figure 2-2 shows the location of rain gauges. These gauges were used to calibrate radar rainfall data and compute rainfall volume for each model subcatchment. Issues with the power supply and with gauge operation were frequently noted for the South Industrial gauge during the data collection period. As a result, this site was not used for analysis for some of the calibration and validation events. The South Industrial gauge was later relocated as part of this project and the power supply issues have also been resolved.

Figure 2-3 – Rain Gauges for Final Model Calibration



To supplement the flow and rainfall data, a program was established to gather observational data of surface flooding and other stormwater behavior at targeted sites throughout the City. With input from City staff and neighborhood groups, a total of 42 locations were identified for Large Event Data

Gathering (LEDG). The data collection plan for these sites (shown in **Figure 2-3**) consisted of two components:

- Storm corps observations – Citizen volunteers worked with established observation sites to document the extent of flooding during large rain events. Photographs and visual observations were also collected.
- Crest-stage gauges - Crest-stage gauges were installed at locations around the City of Ann Arbor watersheds to understand the runoff response and extent of flooding during intense storm events. These gauges recorded maximum water levels for large events.

Figure 2-4 – Large Event Data Gathering Sites

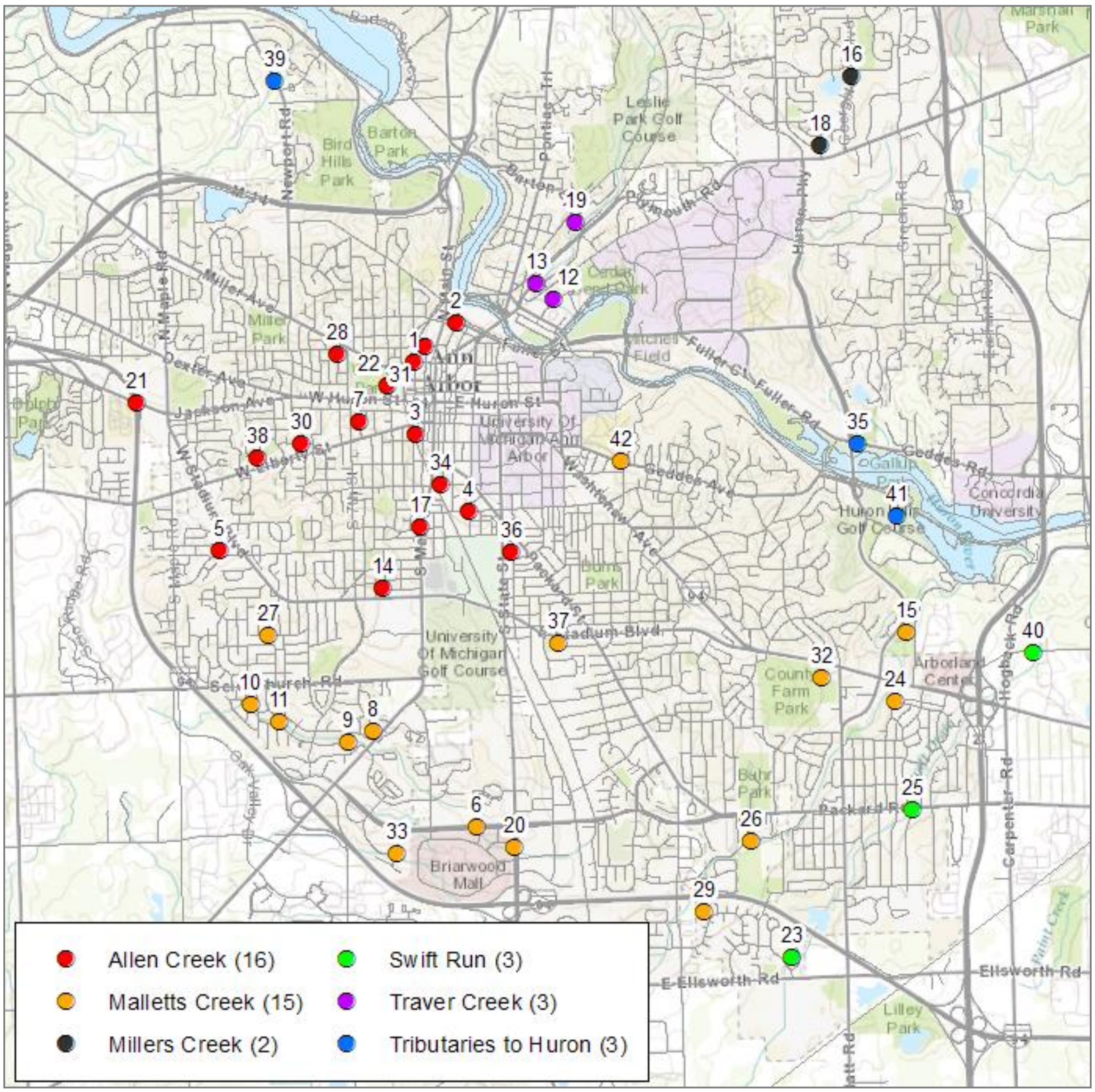


Table 2-2 shows a list of site IDs and locations for the LEDG sites. The majority of the sites were located in the Allen and Malletts Creek watersheds (16 and 15 respectively). Frequent street flooding was

reported at these sites during intense storm events in the past and the locations were refined using citizen reports gathered at neighborhood stormwater meetings in January through March of 2013.

Table 2-2 – List of Large Event Data Gathering Sites

Allen Creek		**Malletts Creek**	
ID	Location	ID	Location
1	1st and Kingsley	6	Eisenhower and Plaza Dr
2	Depot/4th/Summit	8	2295 Chaucer Ct
3	First and William	9	1115 Morehead Ct
4	Hill and Division	10	Churchill/Wiltshire Intersection
5	Park Place Apartments	11	2279 Mershon
7	306 Mulholland	15	Brentwood Sq.
14	Edgewood and Snyder	20	State and Mall Dr
17	Davis and S Main	24	Parkwood and Fernwood
21	I-94 and Jackson	26	Doyle Park dam
22	West Park	27	Avondale and Catalina
28	504 Maple Ridge (south of Arborview)	29	Englewood and Manitou
30	Bemidji and Montgomery	32	Meri Lou Murray Recreation Center
31	Felch/N. Ashley intersection	33	Signature and Waymarket
34	Madison and 4th (Fingerle)	37	Iroquois south of Stadium
36	1128 White St	42	Geddes and Linden
38	Behind Glendale Circle (west of Virginia Park)		
Millers Creek		**Swift Run**	
ID	Location	ID	Location
16	2369 Georgetown (south of Bluett)	23	University Townhouses
18	Prairie and Briarcliff	25	Packard and Pittsfield
		40	Swift Run at Clark Rd
Traver Creek		**Tributary to Huron River**	
ID	Location	ID	Location
12	Traver Creek at Nielsen Ct	35	Geddes/Fuller/Huron Pkwy
13	Plymouth Park adjacent to Manna Market	39	Newport Creek at Newport Rd
19	Traver Creek at Barton Dr	41	Huron Hills Golf Course

LEDG data was used during calibration to validate flooding predictions. It was also used during the existing conditions modeling to assist in the delineation of localized flooding areas.

Data collected from rainfall and flow monitoring, as well as from the LEDG program, has been provided to the City as part of the final data files for the project.

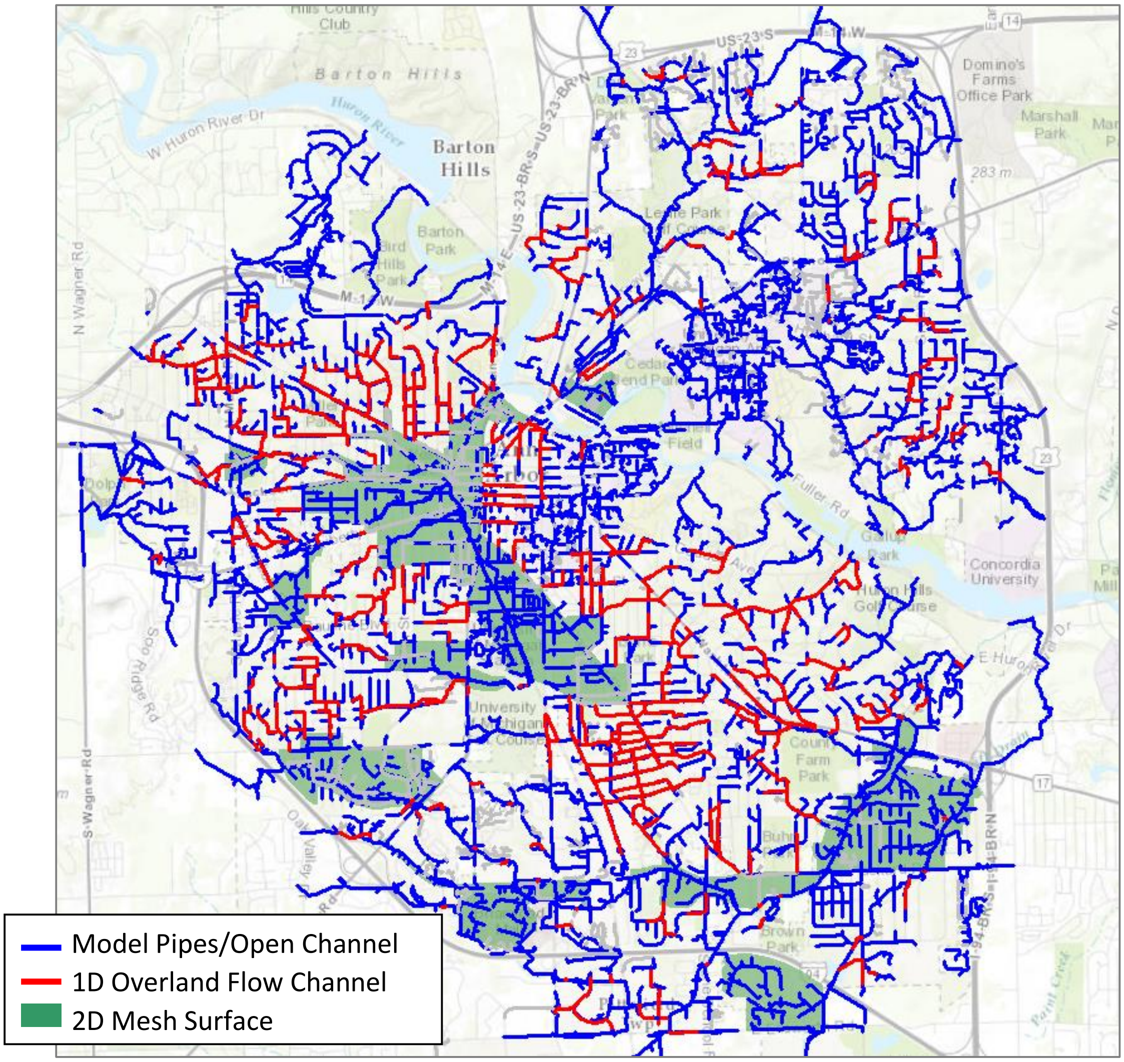
D. Task 4 – Final Model Calibration and Validation

The objective of Task 4 was to utilize the preliminary calibrated model and newly collected flow and rainfall data to provide final model calibration and validation.

Prior to final calibration, the model hydrology and hydraulics were updated to 2013 conditions. Significant changes are described as follows:

- New or modified stormwater facilities were included for West Park, County Farm Park, and for the Traver Creek improvements in Leslie Park Golf Course.
- FDD flows were added to the model and represented as Rainfall Dependent Inflow/Infiltration (RDII) hydrographs. This allowed for analysis of different FDD Program scenarios without having to manually adjust hydrologic parameters. The FDD scenario evaluations were presented in an FDD Flows Technical Memorandum, dated November 20, 2013. This tech memo can be found in the final project documentation.
- 1D and 2D overland flow channels were also incorporated into the model for calibration. 1D refers to one-dimensional modeling, where overland flow is represented by a secondary model link between the two manholes. In 2D, or two-dimensional modeling, overland flows is represented by surface polygons that are based on elevation contour data. **Figure 2-4** shows the areas with 1D and 2D overland flow surfaces. The 2D surface occupies more than 10% of the model area, mostly located within Federal Emergency Management Agency (FEMA) 100-year floodplain and flood-prone areas.

Figure 2-5 – Model Overland Flow Channels and 2D Surface Locations



In general, the model updates were made to align the model framework with the actual system conditions present during the 2013 monitoring period.

Final calibration was performed to refine and improve the model parameters established in preliminary calibration. The detailed process and results of calibration are presented in Section 3 of this report, and in the Final Calibration Report.

E. Task 5 – Phase I Documentation

The objective of Task 5 was to provide comprehensive documentation of the model update and calibration processes for future reference. This was accomplished primarily in the delivery of the project model, which includes all calibration scenarios as part of the InfoSWMM scenario manager.

This task also included delivery of an archive of project data files and documentation, including the flow and rainfall data, GIS data files generated throughout the project, and other administrative documentation.

Phase I work was summarized in the final calibration report, which can be found in the project data file archive.

F. Task 6 – Procedures

The objective of Task 6 was to provide written support to City of Ann Arbor staff that will routinely use or update the model with new stormwater management features, infrastructure changes, or with new design storm information.

The model procedures were developed in conjunction with the model training sessions described in Task 7. These written procedure documents cover the steps needed to incorporate new BMPs or other stormwater improvements into the model. A separate procedure document was created to explain storm update procedures, which could be used to modify design storm information or to create a new storm scenario altogether.

G. Task 7 – Training

The objective of Task 7 was to develop training materials and provide both general and detailed training for the newly developed modeling tools. Detailed training sessions were held on March 2-3, 2015 with City staff who will be the primary model users. General training to explain the model development and model applications was held on March 24, 2015. The training presentations were included as handouts in each session and copies are also included in the project file archives.

H. Task 8 – Phase II Public Engagement

The objective of Task 8 was to continue the information sharing and public education processes that were established in Phase I, while adding new activities to disseminate project results and recommendations.

Three public meetings were held in November 2014, with dates and times selected to enable maximum community participation:

- Wednesday, November 5 – 6:30 p.m. Ann Arbor District Library – Downtown
- Thursday, November 6 – 10:00 a.m. Ann Arbor District Library – Downtown
- Sunday, November 9 – 2:30 p.m. Ann Arbor District Library – Malletts Creek

The purpose of these meetings was to share the project's findings, including proposed recommendations and the rationale behind each. Meeting attendees were invited to indicate their level of interest among all the geographic areas in which recommended system improvements were proposed, in order to properly prioritize the contents of the presentation.

The other new public engagement activity in Phase II was the development of a stormwater video that would help to draw attention to the project and to stormwater management issues facing the City of Ann Arbor. The stormwater video entered production in March 2015 and will be released near the end of the project schedule.

I. Task 9 – Model Analysis and Recommendations

The objective of Task 9 was to utilize the final calibrated model to evaluate the performance of the stormwater drainage system throughout the City of Ann Arbor and to identify and analyze proposed improvements.

The basis for evaluating the existing conditions performance of the stormwater system performance was a series of design storm scenarios, that include different volumes and rainfall distributions based on the annual exceedance probability (AEP) standards established in NOAA Atlas 14. A range of storms was analyzed from 100% AEP to 0.2% AEP. In general, the 10% AEP, 12-hour duration storm and the 20% AEP, 1-hour duration storm were used to evaluate the level of service being provided by the stormwater system. The 10% AEP storm is the current stormwater design standard, but most areas of the City were constructed to a smaller storm recurrence standard and at a time when the storm volumes associated with the standards were smaller. Analysis of the 20% storm allowed for identification of areas that would first begin to have capacity problems as the storm size increases.

Locations were identified where the current pipe capacity cannot convey the flows generated by these storms, and where surface flooding occurs as a result of the capacity shortfall. A list of study locations was developed and potential stormwater improvement alternatives were considered for each location. These included alternatives for stormwater Best Management Practices (BMPs), local and regional stormwater storage, and conveyance improvements.

The calibrated model was also used to analyze stormwater management impacts. For future condition scenarios, the model was used to predict the impacts of broad stormwater management initiatives, such as residential rain gardens, commercial property redevelopment, and the City's Green Streets program for stormwater management in right-of-way (ROW) areas.

Details on the model analysis work and stormwater improvement recommendations are included in Sections 4 and 5 of this report.

J. Task 10 – Verify FEMA Mapping

The objective of Task 10 was to compare the calibrated model results to existing FEMA Flood Insurance Rate Map (FIRM) flood mapping to provide the City with an additional source of flood level data that could be used for future floodplain analysis and management.

The InfoSWMM model was used with a 1% AEP, 24-hour storm, and peak flows and peak water surface elevation (WSEL) data were generated. The water surface elevations from the model were then used to delineate floodplain contours using the latest Light Detection and Ranging (LiDAR)-based topographic data and differences between the model-based contours and the FEMA FIRM floodplain contours were compiled. The comparison data was provided to the City of Ann Arbor to support future floodplain management decisions.

K. Task 11 – Documentation

Final documentation for the project includes this final report, along with project model files and data files generated during Phase II activities.

3. Stormwater Modeling

A. Model background and calibration

Preliminary calibration of the stormwater model was performed using available flow monitoring data collected by CDM Smith as part of the Stormwater GIS and Model development (SGM) project. The SGM flow data from 2007 was supplemented with United States Geologic Service (USGS) flow data from long term flow gauges. In total, nine (9) storm events from May 2007 to March 2012 were selected for the preliminary calibration effort. It was found that during the growth-season events, model results were generally within 15% of volumes and 20% of peak flows observed at the monitors and USGS gauges.

A percent difference of 15% for volume and of 20% for peak flows were the initial targets used by CDM Smith to evaluate the effectiveness of calibration, based on experience with other stormwater models of similar size and level detail. The model was validated using three (3) storm events from 2007 and was generally within 20% of volumes for monitored flows. The peak flow comparison was also within 20% for most meter areas, but there were some areas with wider variability (in the range of 50% difference) between model-predicted and monitor-observed flows.

The preliminary calibration report concluded that additional data collection and calibration should be performed for the following reasons:

- The dormant season model calibration was limited due by the lack of dormant season calibration events. Additional soil parameter calibration will be needed to improve dormant season calibration.
- Provide additional support for upstream boundary conditions for locations where stormwater flows enter the City. These include Traver Creek at M-14 and Malletts Creek at I-94
- Collect data for better model refinement in expected study locations. Since the 2007 data collection effort, large storms have highlighted collection system performance and level of service concerns in Malletts Creek and along lower Allen Creek. Additional monitoring of major branches of these creeksheds was recommended.
- Improve calibration and validation to meet percent difference goals of 15% on volume and 20% on peak flow, when comparing model-predicted values to monitored values.

The preliminary calibration report, submitted in 2013, included the conclusions above and recommended additional flow and rainfall monitoring in 2013 to be used for a final model calibration. Final calibration and validation were performed in early 2014, using the flow and rainfall data collected during 2013.

i. Storm Events for Calibration

Unlike 2007, the monitoring period between March and November 2013 yielded a few large events that significantly tested the performance of the storm drainage system. That includes the June 27th 2013 event that caused surface flooding in parts of the Allen Creek and Malletts Creek watersheds. A total of seven 2013 storm events of various volumes were selected for calibration (**Table 3-1**).

Table 3-1 – Summary of Calibration and Validation Events

#	Date	Precip Total (in)	Sources	Season
1	6/27/2013	1.1 – 3.0	calibrated radar rain data	growth
2	8/12/2013	1.7 – 2.9	calibrated radar rain data	growth
3	10/31/2013	1.5 – 1.9	calibrated radar rain data	growth*
4	6/13/2013	1.3 – 1.8	calibrated radar rain data	growth
5	4/17/2013	1.3 – 1.6	ground gauges	dormant/growth transition
6	7/9/2013	0.1 – 1.2	ground gauges	growth
7	8/27/2013	0.3 – 0.6	ground gauges	growth

* This low-intensity long-duration event was observed to behave like growth season event after calibration

The total precipitation (measured at individual gauges) of these events ranged from 0.1 inches to 3.0 inches. 5-minute calibrated radar rainfall data in 1km x 1km resolution were purchased from Vieux Inc. for the four largest events. For the other events, precipitation at each subcatchment was computed with ground gauge records with inverse-distance-weighted interpolation, which assigns precipitation to each subcatchment using a weighted calculation based on the nearest ground gauges.

ii. Calibration Methods

The model calibration was performed using an iterative approach by refining the following model parameters to match model-simulated hydrographs with flow monitoring data:

- Green-Ampt infiltration parameters
- Percent of runoff routed from impervious to pervious surface (related to % of directly-connected impervious surface)
- Subcatchment width (overland flow length)
- Manning's n (roughness coefficient) for impervious and pervious surface
- Depression storage for impervious and pervious surface (negligible on larger storms)

Due to the model's large scale, the calibration first started by matching flow hydrographs at downstream gauges (USGS stream gauges at Allen Creek, Doyle Park and Malletts Creek mouth, Swift Run (#11), Millers Creek (#13) and Traver Creek (#15)). This first calibration step was then followed by matching the flow hydrographs for the upstream temporary monitors. In addition, there was an emphasis placed on matching flow hydrographs for the larger storms rather than the smaller storms.

iii. Calibrated Model Parameters

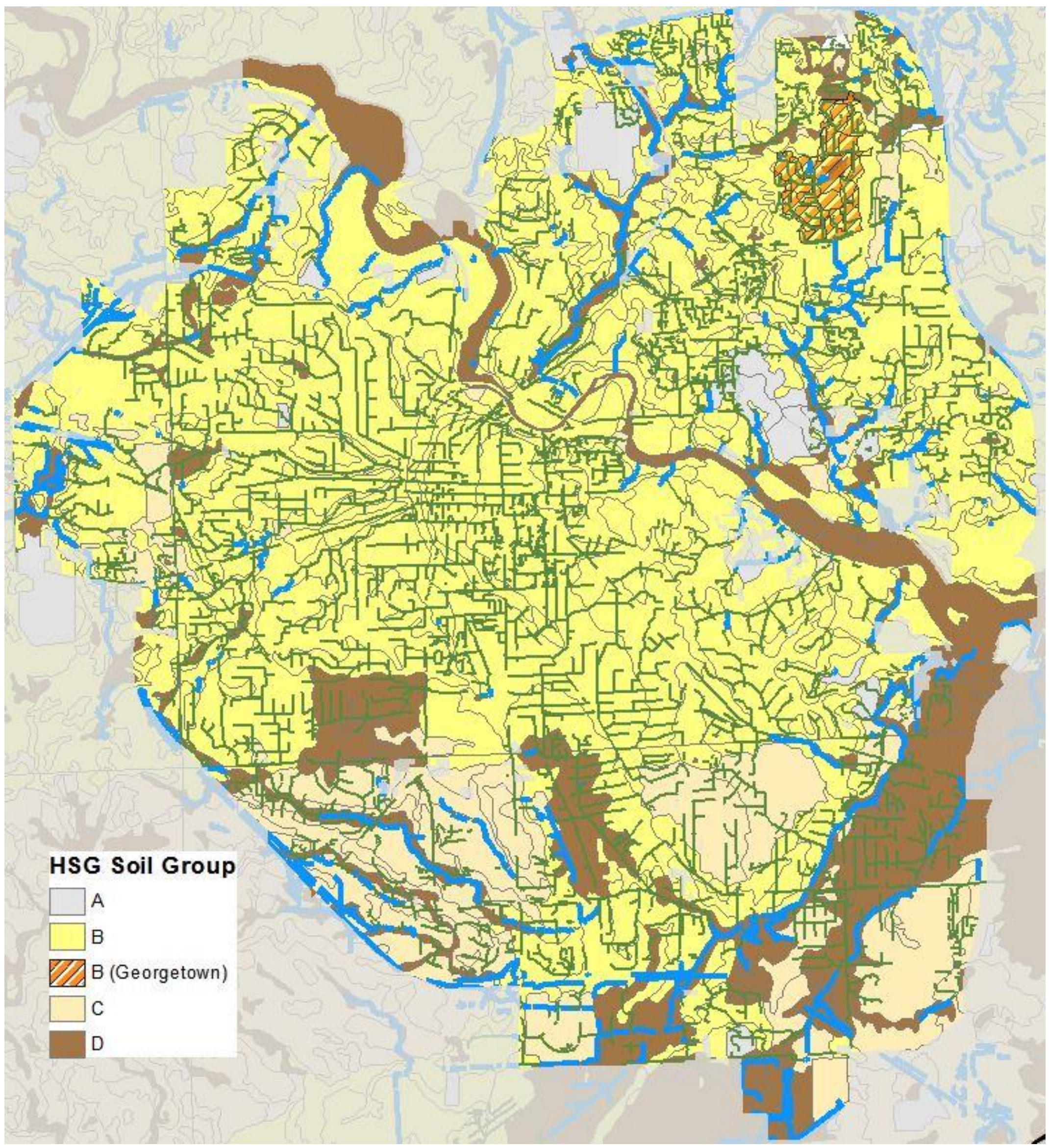
During the final round of model calibration, model parameters were fine-tuned to reflect the new hydrologic conditions as discussed below:

<u>Soil Parameters</u>

The soil parameters in the model affect the amount of rainfall that is predicted to infiltrate into the ground. Originally, four different soil types were set up in model setting based on the Hydrologic Soil Group (HSG) Soil Group (A, B, C and D). After going through the iterative calibration process and upon further review of the United States Department of Agriculture (USDA) Soil Map and potential soil infiltration rates, an additional soil parameter group (B1) was added. The monitor #14 (Georgetown) area has primarily type B soil according to the USDA Soils Map, but the model continued to over-estimate runoff peak and volume. A better match was obtained when the B1 soil parameter was used, which included increasing the soil infiltration rate from 1 in/hr to 1.8 in/hr.

The Malletts Creek area upstream of the Mary Beth Doyle Park pond is primarily of type C soil, but the USDA Soil Map showed that the soil infiltration rates of the first foot of soil more closely resemble type B soil. To better match the storm sewer hydrographs for these storm events, these areas were assigned to have type B soil. Soil classification data is shown in **Figure 3-1**.

Figure 3-1 – Hydrologic Soil Groups (HSG) for stormwater model areas



The dormant season soil parameters were not adjusted from the 2007 parameters because there were no large storm events during the dormant season in 2013.

Table 3-2 summarizes the Green-Ampt soil parameters assigned for each model soil type:

Table 3-2 – Green-Ampt Infiltration Parameters

Model Soil Type	Suction (in), Conductivity (in/hr), Initial Deficit (fraction)		
	Growth Season	Dormant/Growth Transition	Dormant Season (May 2010, Nov 2011)
A	2.41, 2.35, 0.312	3.94, 0.7, 0.221	3.75, 0.5, 0.191
B	4.15, 1.0, 0.252	4.97, 0.4, 0.182	8.19, 0.15, 0.149
B1	4.15, 1.8, 0.252		
C	6.2, 0.3, 0.174	9.86, 0.06, 0.096	12.45, 0.02, 0.079
D	7.52, 0.2, 0.158	12.45, 0.02, 0.079	12.93, 0.01, 0.073

Percent Runoff Routing from Impervious to Pervious Surface

This parameter is related to the directly-connected impervious surface, and has important impacts on runoff volume. It was assigned to a range of values primarily based on land use and land cover. Compared to preliminary calibration, the percentages were increased slightly as shown in **Table 3-3**.

Subcatchment Width (Overland Flow Length)

Overland flow length was simplified to either 100 or 150 feet in urban areas and 500 feet for rural areas. During preliminary calibration, the subcatchment width parameter was assigned as one of 12 different flow lengths ranging from 50 to 200 feet, based on subcatchment slope and imperviousness. These were within the range of typical values as suggested from the EPA SWMM Help Manual.

Table 3-3 – Summary of % Runoff Routed to Pervious Surface Based on Land Use/Land Cover

Land Use/ Land Cover	% Routed	
	Preliminary	Final
Commercial	25 – 40 (mostly 25)	25 – 40 (mostly 40)
Downtown (Imp >85%)	5	
Residential	56 – 68	60 – 72
Road/parking lot	10	10 – 20
Water body	0	
Wooded/non-developed area	90 – 100	

Manning's n for Impervious and Pervious Surface (Overland Flow)

The overland flow Manning's n is a model parameter that relates to overland flow velocities, affecting both runoff and infiltration. Typical values were used for this parameter. It is set at 0.05 for impervious surfaces, and 0.2 for pervious surfaces. This parameter was found to have slight impact in shaping peak flows and volume in calibration.

Depression Storage for Impervious and Pervious Surface

Depression storage parameters represent the initial surface storage volume that is filled during a precipitation event prior to the start of any runoff. Assignment of these parameters was simplified compared to preliminary calibration. Typical values were used: 0.08 inch for impervious surfaces, 0.16 inch for grass areas, and 0.2 inch for wooded areas.

iv. Calibration Results

In general, the model was able to replicate the hydrographs at the USGS stream gauges and temporary flow monitors. The results were generally within 15% for volumes and 20% for peak flows, which match with calibration goals for a stormwater model of this size and complexity.

Tables 3-4 and **3-5** show the event-specific percent difference in volume and peak flow for each monitor location. The percent difference in each case is calculated using the following formula:

$$\%\ \boldsymbol{Difference} = \frac{(\boldsymbol{Model\ predicted\ value} - \boldsymbol{Monitor\ observed\ value})}{(\boldsymbol{Monitor\ observed\ value})}\ \boldsymbol{x}\ \mathbf{100}\%$$

Figures 3-3 to **3-4** show hydrographs for some of the major monitors while the full version of the final calibration report includes all the hydrographs at these monitors and gauges for reference.

Discussions of some of the outliers are as follows:

- Monitor Sites #1 and #2 were surcharged/flooded during 6/27/13 storm. Flows were likely under-reported by the monitors at these sites during peak flow.
- The model under-predicted flows for Traver Creek sites for 4/17 (#15 and #16), and 6/13 (#15) storms by at least 30%. There seemed to be an unaccounted flow source from outside the city limits after those storm events. Review of nearby rain gauge data did not reveal additional precipitation in the vicinity. Additional field investigation work in Ann Arbor Township would be required to determine why the response for these storms varied from other storms for which the calibration was better matched.
- Monitor #10 in the Malletts Creek watershed had good agreement on hydrologic response pattern but poor volume agreement for the 10/31/2013 event. Because other events for this monitor had more consistent agreement, this was likely due to monitor error, possibly from fall leaf debris. This was also the smallest monitored tributary area, with the lowest flows, making it more subject to this type of problem.
- For the Swift Run monitoring site, the culvert configuration did not allow for installation of an ultrasonic flow meter. Instead, a continuous level monitor was installed, and a rating curve that had been developed in 2007 was used to calculate flow. The rating curve provides a correlation between the level monitor reading and a predicted flow rate. However, the measured flow rate values were much lower than model predictions, suggesting that the rating curve may not have been representative in 2013 (potentially due to changes in sediment levels in the culverts or changes in streambank characteristics). As a result, the model parameters were refined to match model-predicted levels with recorded levels for this site (#11)

Table 3-4 – % Difference for Model-Predicted Volume to Monitor-Observed Volume

Flow Monitor		Calibration						
		6/27/13	8/12/13	10/31/13	6/13/13	4/17/13	7/9/13	8/27/13
Allen	1	**66%**	**43%**	18%		15%	1%	9%
Allen	2	**269%**	9%					4%
Allen	3	23%	16%	-12%	12%	**-41%**	22%	21%
Allen	5		9%	-3%	5%	-1%		6%
Allen	USGS	3%	-1%	10%	4%	-8%	-7%	-4%
Malletts Creek	6	-7%	6%	12%	3%	-13%	11%	13%
Malletts Creek	7	-15%	-5%	-6%	-13%	-21%	-13%	-2%
Malletts Creek	8	8%	9%	-7%	-3%	-11%	4%	-27%
Malletts Creek	9	19%	4%	12%	-5%	-15%	17%	**33%**
Malletts Creek	10	4%	10%	**107%**	1%	-15%	5%	11%
Malletts Creek	UP_MA	-10%	7%	14%	-6%		7%	7%
Malletts Creek	USGS Doyle	1%	2%	-7%	-3%	-4%	2%	3%
Malletts Creek	USGS Malletts	1%	-9%	-8%	4%	-12%	-7%	6%
SR	11 (level)		2%	0%				-3%
Millers	13		8%		-13%	-25%	14%	17%
Millers	14	-16%	9%	30%	-2%	-6%	19%	**113%**
Traver	15	-4%	-5%	-3%	**-30%**	**-32%**	-6%	1%
Traver	16	-4%	-5%	4%	-4%	**-71%**	-2%	4%

- For areas with open channels, there seemed to be a prolonged runoff response not effectively represented by the Green-Ampt infiltration model. This was apparent when monitored flows dropped off more slowly than the model prediction, lasting for many hours after the 4/17 event. This prolonged runoff response was represented by adding response hydrographs based on the Rainfall Dependent Inflow/Infiltration RTK method (RDII RTK) along the open channel reaches in Malletts Creek, Swift Run and Traver Creek.
- The distance-weighted average of ground rain gauge data did not seem to be representative enough for the 8/27 event. Although the runoff volumes were matched within 15% for most of the sites, the model missed the first runoff peak as recorded by the flow monitors.

Table 3-5 – % Difference for Model-Predicted Flow Rate to Monitor-Observed Flow Rate

Flow Monitor		Calibration						
		6/27/13	8/12/13	10/31/13	6/13/13	4/17/13	7/9/13	8/27/13
Allen	1	**33%**	**41%**	15%		**46%**	-8%	-10%
	2	**184%**	0%					-8%
	3	**36%**	-6%	2%	-5%	-26%	-23%	-23%
	5		-3%	19%	-4%	25%		-4%
	USGS	-4%	2%	-11%	-11%	-9%	-4%	**-45%**
Malletts Creek	6	31%	14%	-16%	5%	-4%	-8%	-26%
	7	29%	-2%	-6%	-11%	**-38%**	-11%	-18%
	8	20%	9%	-19%	-7%	-16%	8%	-25%
	9	-7%	-1%	23%	10%	-3%	2%	-24%
	10	**46%**	-4%	**32%**	-7%	4%	16%	-4%
	UP_MA	4%	-10%	12%	-6%		-10%	**-36%**
	USGS Doyle	-11%	17%	15%	-9%	0%	10%	-1%
	USGS Malletts	-17%	-24%	-9%	-11%	-26%	13%	**-49%**
SR	11 (level)		-6%	-3%				**-31%**
Millers	13		15%		6%	-9%	-12%	-29%
	14	4%	11%	18%	16%	-7%	-10%	**83%**
Traver	15	-11%	-3%	-13%	-19%	-19%	8%	**-45%**
	16	-29%	9%	-1%	2%	**-82%**	4%	19%

Figure 3-2a-d – Flow Hydrographs for Major Monitors for 6/27/13 Event

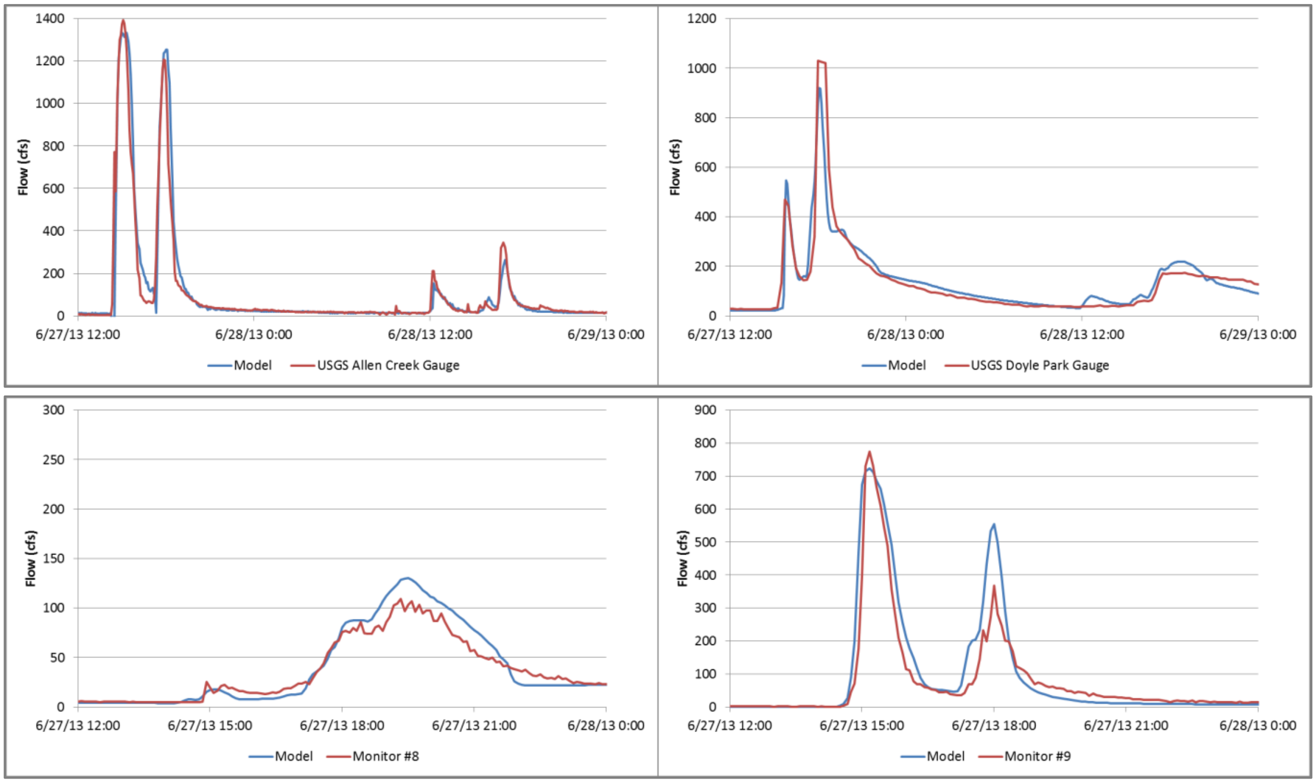


Figure 3-3a-d – Flow Hydrographs for Major Monitors for 8/12/13 Event

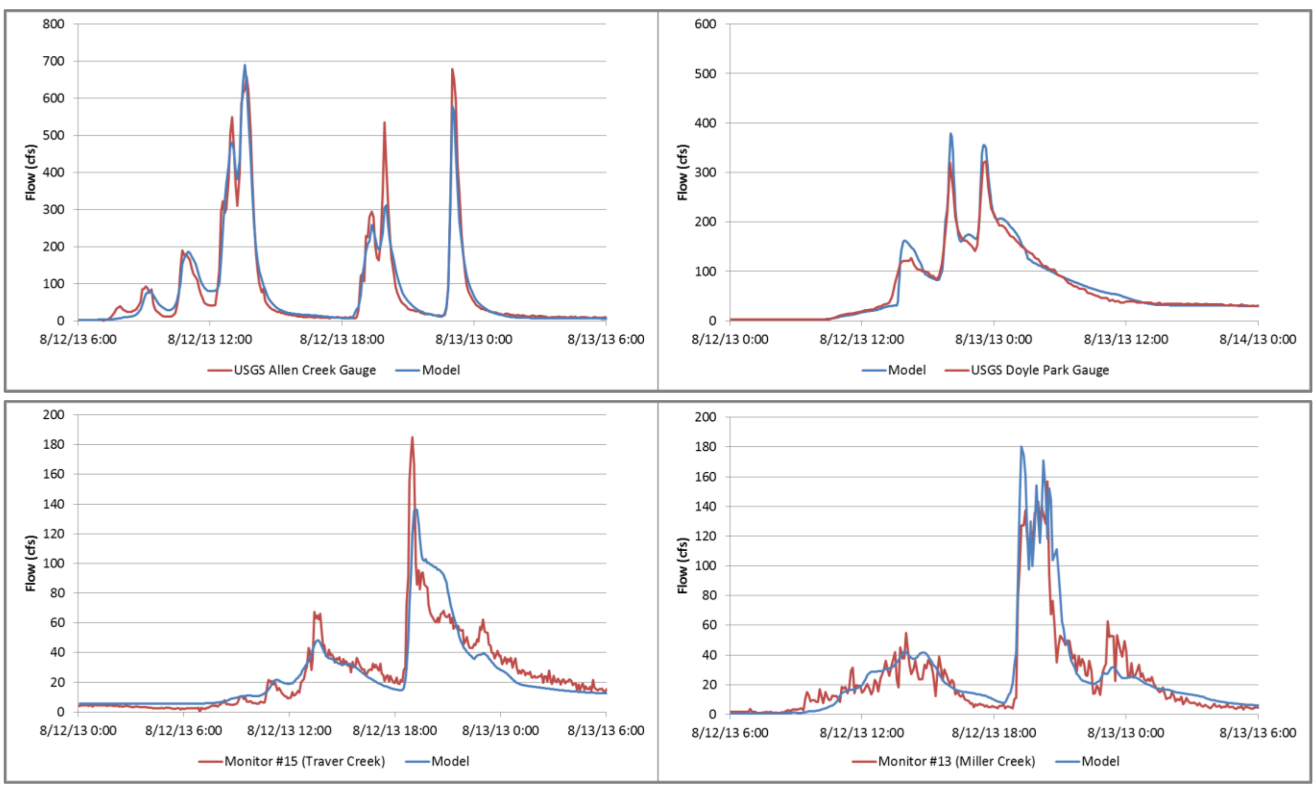
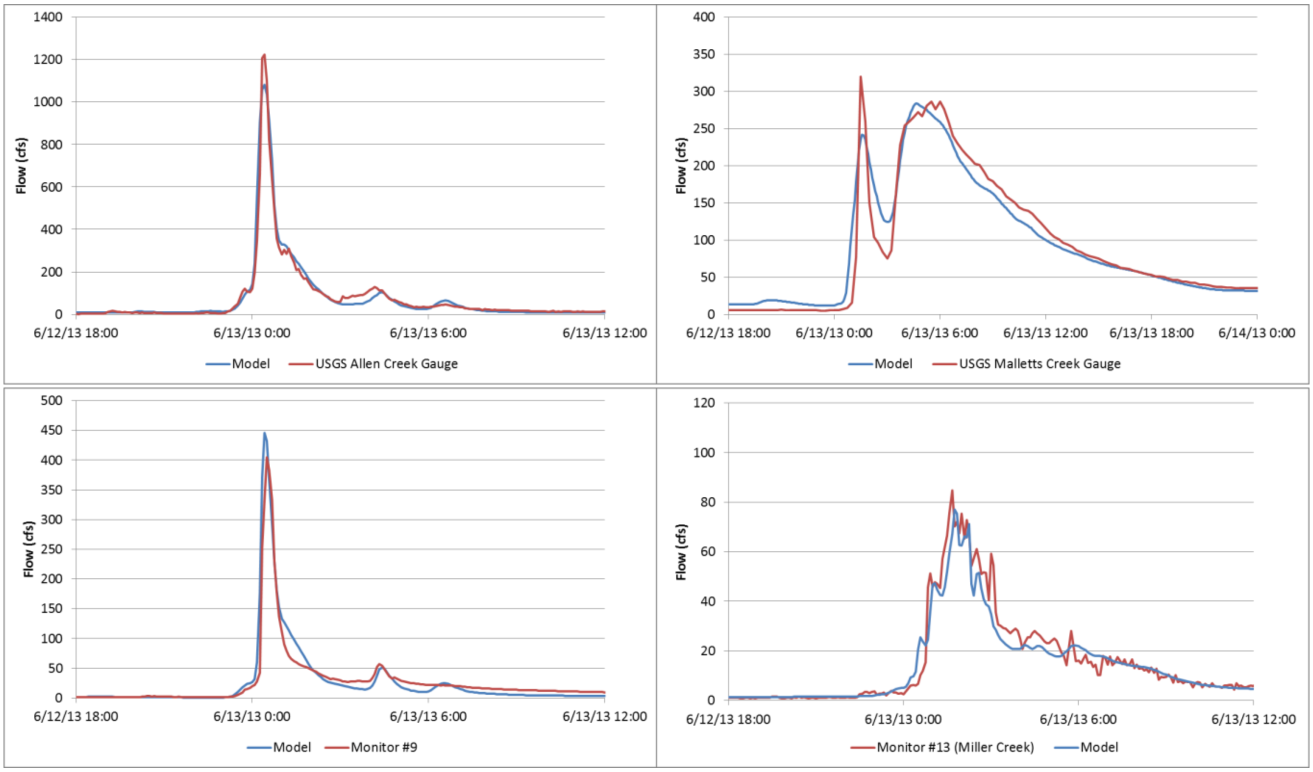


Figure 3-4a-d – Flow Hydrographs for Major Monitors for 6/13/13 Event



B. Model Validation

i. Validation Events

Three (3) storm events in 2013 were selected for model validation. The total precipitation for these events ranged from 0.1 inches for the 7/27 event to 1.6 inches for the 10/5/2013 event. **Table 3-6** summarizes the range of precipitation computed for the monitoring districts for each of the validation events.

Table 3-6 – Summary of Validation Events

#	Date	Precip Total (in)	Sources	Season
1	10/5/2013	1.3 – 1.6	ground gauges	growth
2	11/17/2013	0.6 – 0.8	ground gauges	dormant
3	7/27/2013	0.1 – 0.5	ground gauges	growth

ii. Validation Results

Table 3-7 summarizes the comparison of runoff volume and peak flow values between the model-predicted and monitor-observed data. As with the calibration comparison, the validation results are presented in terms of a % difference. The comparison was made at all gauges with available data, including the USGS gauges.

Table 3-7 – % Difference for Model-Predicted vs. Monitor-Observed Volume/Peak Flow

Flow Monitor		Volume			Peak		
		7/27/13	10/5/13	11/17/13	7/27/13	10/5/13	11/17/13
Allen	1	21%	13%	11%	-8%	0%	7%
	2	-4%			0%	-11%	
	3	13%	18%	-14%	-25%	-6%	-27%
	5		4%	6%		17%	26%
	USGS	6%	9%	13%	-8%	-11%	-13%
Malletts Creek	6	12%	11%	-10%	-16%	-4%	9%
	7	3%	1%	-16%	18%	-6%	-13%
	8	-14%	-3%	4%	7%	7%	-7%
	9	23%	72%	5%	-16%	-6%	10%
	10	-3%	-1%	7%	0%	7%	13%
	UP_MA	39%	10%	2%	-23%	16%	34%
	USGS Doyle	0%	10%	-13%	-3%	-3%	-26%
	USGS Malletts	-4%	13%	-14%	-48%	-27%	-27%
SR	11 (level)	8%	0%	-7%	7%	-2%	10%
Millers	13	16%	14%		-10%	-12%	
	14	6%	19%	18%	-33%	-2%	-11%
Traver	15	0%	-3%	-5%	20%	-30%	-12%
	16	3%	2%	-3%	21%	4%	-12%

More calibration information is available in the final calibration report, which shows all of the calibration hydrographs at each monitor for each event. In general, the model-predicted flows and volumes were within 15% of recorded data. As noted earlier, this falls within the expected range of agreement for stormwater models of this size and level of detail.

The calibration and validation work performed with 2013 data had good agreement between model-predicted values and monitor-observed values for volume and flow rate. Adjustments were made to the preliminary model parameters to improve the model performance, including:

- Establishment of a B1 soil classification
- Runoff parameter refinement for more sensitive parameters, specifically with % routing
- Simplification of parameter assignments for subcatchment width and depression storage, which have less impact on model results

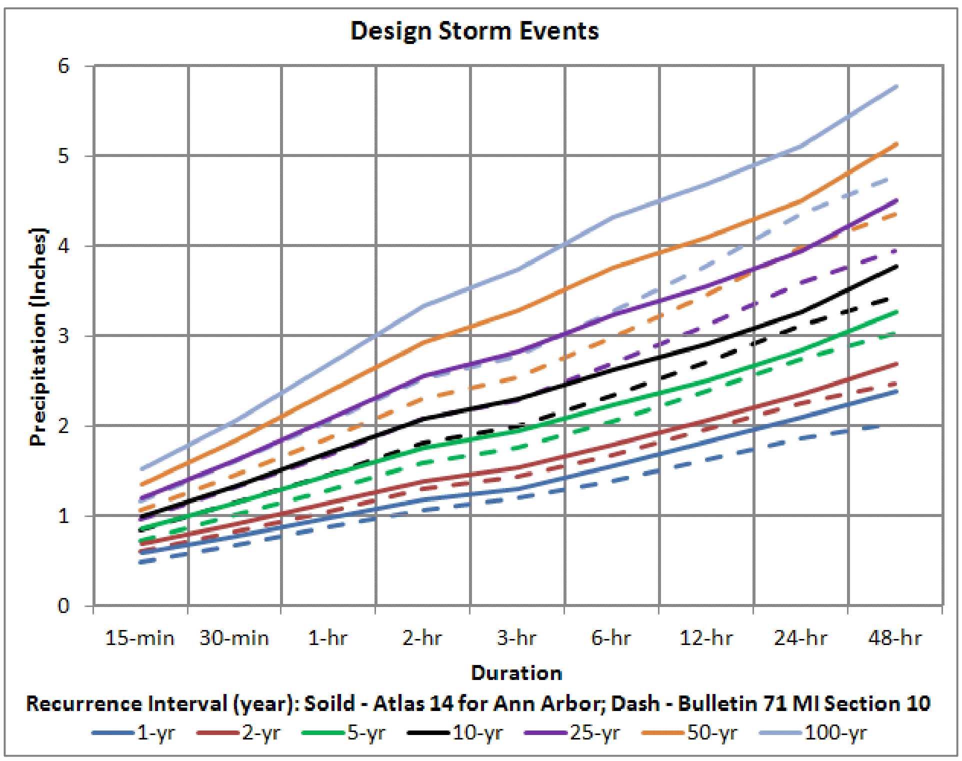
C. Existing conditions modeling

The final calibrated model was used to determine the level of service provided by the existing storm drainage system and to help identify priority areas for improvements. Eight (8) design storm simulations, as shown in **Figure 3-5**, were prepared to identify capacity constraints and flooding locations in the system. The range of design storms include:

- 100% annual exceedance probability (AEP) 1-Hour: 0.97" (could serve as baseline for BMP evaluation)
- 50% AEP 24-Hour: 2.35" (could serve as baseline for BMP evaluation)
- 20% AEP 1-Hour: 1.44" (Older part of the system were designed for old 20% storm volume)
- 10% AEP 12-Hour: 2.90" (Represents current design standard)
- 4% AEP 24-Hour: 3.93"
- 2% AEP 24-Hour: 4.5"
- 1% AEP 24-Hour: 5.11" (Design standard for detention storage, used for FEMA map comparison)
- 0.2% AEP 24-Hour: 6.74" (new probability from Atlas 14, also used in FEMA flood analysis)

Rainfall volumes were obtained from NOAA Rainfall Atlas 14 Volume 8 (version 2). They were 8% to 28% higher compared to Bulletin 71 (Please refer to Design Storm Tech Memo for detailed discussion).

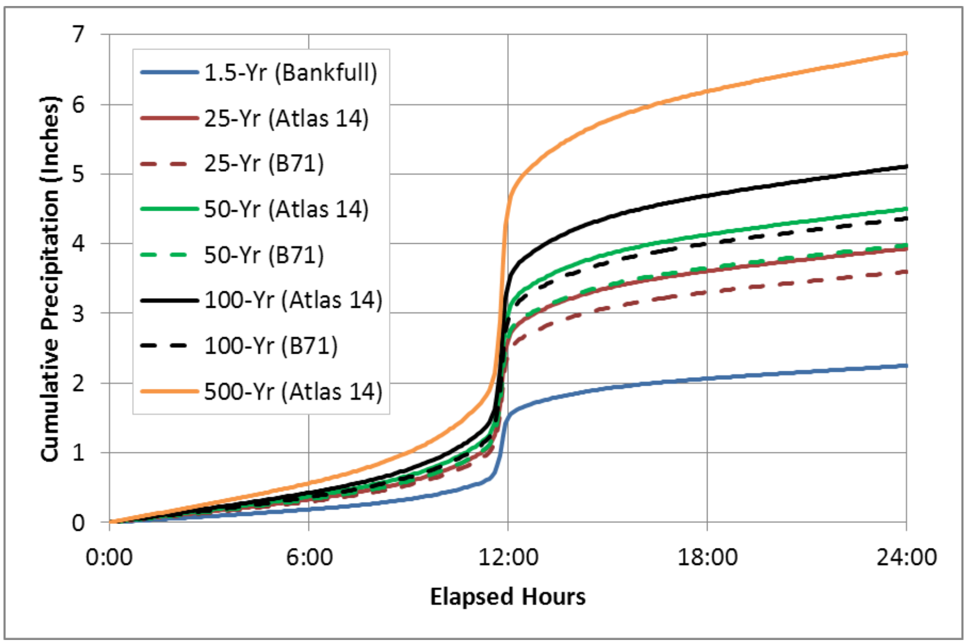
Figure 3-5 – Design Storm Events



The Huff 1st quartile, alternating block, and SCS Type II distributions were used for 1-hour, 12-hour and 24-hour duration storms, respectively. An alternating block distribution is similar to SCS Type II except it is not limited to 24-hour duration storms. Both of these distributions represent an intense rainfall pattern that is commonly associated with thunderstorm activity likely to occur during summer. These rainfall distributions are shown in **Figure 3-6**.

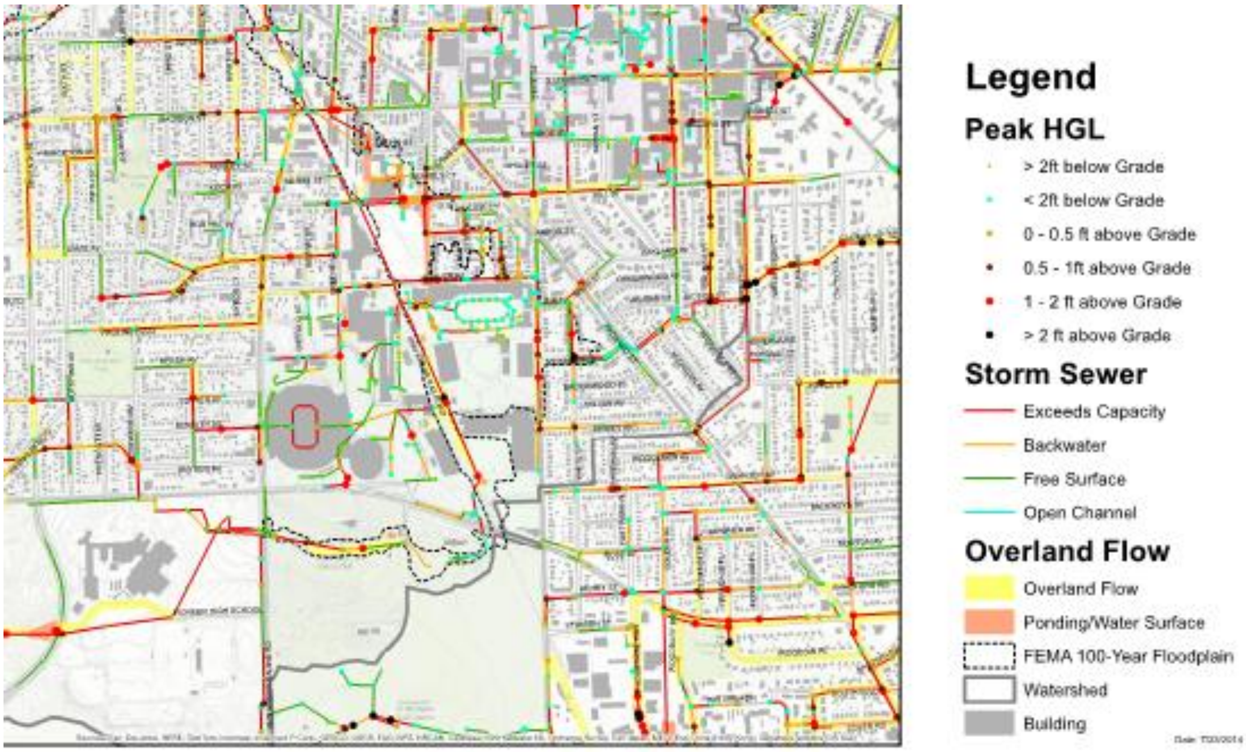
Climate change was a frequent point of discussion during the project. The use of newer rainfall volume standards from NOAA Rainfall Atlas 14 for design storms was one consideration. As noted in the previous paragraph, use of the SCS Type II distribution was another decision made so that the project was considering not only the most intense type of storm event, but potentially accounting for more frequent storms of this type in the future.

Figure 3-6 – Cumulative Rainfall Distributions



Appendix A contains two series of sewer system maps showing the level of service provided by the existing storm drainage system in different parts of the City: Capacity Exceedance maps and Peak flow condition maps. For the capacity exceedance map, pipes were color-coded based on the smallest design storms that pipe capacity was exceeded. For the peak flow condition maps (one map per design storm), pipes were shown in green if capacity is not exceeded, yellow if backwater condition occurred, and red if capacity is exceeded during storms. **Figure 3-7** below shows an example peak flow condition map.

Figure 3-7 – Example HGL Condition Map



In addition to the pipe capacity, the maps also show locations where flooding would occur during different design storm events. Surface flooding locations were categorized into either street overland flow (usually with less than 6 inch of water) or ponding (more than 6 inches of water), and their boundaries were delineated using LiDAR data provided by Washtenaw County.

With higher precipitation estimates from Atlas 14, most of the current drainage system had pipe capacities that were more in line with the 20% AEP storm instead of the 10% AEP storm, which is the current standard. While it was not unexpected that newer parts of the system and open channels can usually handle larger storm events better than older parts of the system, most areas of the stormwater system are still able to convey the 10% AEP, 12-hour storm without significant flooding. This includes almost the entire creekshed areas for Traver Creek, Millers Creek, Swift Run, Newport Creek, and areas that drain directly to the Huron River, where only a few isolated surface flooding areas were identified for additional study during review of existing conditions model data.

The Allen Creek and the Malletts Creek watersheds include more impervious surface area and in general have older stormwater infrastructure. Therefore, most of the capacity issues and surface flooding areas are located in these two creeksheds. Further information on the process used to identify priority areas for improvement and the associated recommendations are discussed in Section 4.

4. Stormwater System Improvements

A. Study Area Selection

Existing conditions modeling results were reviewed in a series of progress meetings and workshops with City Staff in the spring and summer of 2014. Sewer system maps were generated showing the pipe segments that were within design capacity for flow and those that had model-predicted flows that would exceed the design capacity. The maps also showed model nodes where surcharging to ground was predicted (where the water surface elevation would exceed the manhole rim elevation).

Existing conditions results are included in the maps in Appendix A. For the initial review, the current stormwater system design standard storm was used. This design storm has a 10% Annual Exceedance Probability (AEP), and a duration of 12 hours, with a rainfall volume of 2.9 inches. The initial review of the system performance under this storm event showed that the many areas of the system were unable to convey this storm. This has primarily been due to recent changes in the design storm standard, so that the current 10% AEP storm is larger than it was when these pipes were designed and constructed. As a result, a smaller design storm was also evaluated to identify potential locations for stormwater improvements. When the 20% AEP, 1-hour duration storm, with a volume of 1.44", was reviewed with the model, more distinct areas with performance issues were revealed.

For both the 10% AEP, 12-hour storm and the 20% AEP, 1-hour storm, preliminary screening locations were identified by comparing model-predicted flow to design capacity and by identifying locations with predicted surface flooding. The preliminary screening list was also compared with LEDG sites and with public input about flooding locations that was gathered in Phase I public meetings and surveys.

Once preliminary screening was complete, the sites were prioritized using two risk metrics:

- The **probability metric** considered the frequency of flooding occurrence, with the following ratings of 1, 2, or 3:
    1. Model predicts flooding in 10% AEP storm, but no reports
    2. Model predicts flooding in 20% AEP storm and/or frequent public reports
    3. Model predicts flooding in 50% AEP storm and/or frequent public reports

- The **impact metric** considered the extent or severity of flooding with the following ratings:
    1. Flooding limited to streets and parking areas with a depth of 6" or less
    2. Flooding affects private properties, typically with predicted depths of 6" - 12"
    3. Flooding affects structures, typically with predicted depths greater than 12"

These two metrics were multiplied together to generate an overall flooding risk rating, with a higher value indicating a higher risk of flood damage. The assigned values and prioritization are shown in **Table 4-1**:

Table 4-1 – Preliminary Study Area Prioritization

Site	Watershed	P	I	R (P x I)
1. Lower Allen Creek – Main Branch	Allen	3	3	9
2. Edgewood / Snyder	Allen	3	2	6
3. Park Place Apartments	Allen	2	3	6
4. Churchill Downs / Lansdowne	Malletts	2	3	6
5. South University / East University	Allen	3	2	6
6. Mulholland Drive	Allen	2	2	4
7. Scio Church / S. Seventh Street	Malletts	2	2	4
8. Glendale / Charlton	Allen	2	1	2
9. Glen Leven	Allen	2	1	2
10. Church St / Cambridge	Malletts	2	1	2
11. Village Oaks / Chaucer Ct	Malletts	1	2	2
12. Parkwood / Pittsfield Village	Malletts	2	1	2
13. Signature Drive	Malletts	2	1	2
14. S Industrial / Packard Rd area	Malletts	2	1	2
15. Traver / Barton	Traver	2	1	2

Additional sites were identified during the public meeting series that either had not been selected for study or had been eliminated during the preliminary screening process. To address the questions about these sites, they have been included in the comments below:

- Glendale Circle / Virginia Park – This location is predicted to have flooding affecting private properties during the 10% AEP, 12-hour storm, so it should have been included in the original screening, with a probability metric of 1 and an impact metric of 2. A full evaluation of stormwater improvements for this site is included in section 4C.
- Geddes Road at Huron Parkway – This reported flooding may have been related to a culvert problem that was repaired in the past couple years. The model does not predict flooding that would impact any roadways or private properties for the 10% AEP, 12-hour storm.
- Newport Road at Westport – The model predicts some surface flooding in the 10% AEP, 12-hour storm, but overland conveyance allows flow into the wooded area to the east along an existing drainage easement. This site would have probability and impact metrics of 1, so it was not included in screening for evaluation of stormwater improvements.
- Washtenaw Avenue at South University Avenue – Attendees at the public meetings mentioned some surface drainage issues affecting properties on Washtenaw Avenue. The model predicts

some overland flow in the areas of Wilmot Street and South University, but no extensive surface flooding to the south along Washtenaw Avenue. It is likely that the affected property, which sits lower than the roadway, receives roadway runoff during intense rainfall events due to catch basin limitations and/or curb, gutter, and roadway grading issues. Washtenaw Avenue is an MDOT business route so any improvements to the stormwater system would most likely be initiated as part of an MDOT roadway improvement project.

B. Improvements modeling

Three conceptual approaches were considered for stormwater improvement alternatives. These approaches were constructed in the model to represent how these stormwater improvements would function at each study location and how they would impact the stormwater system performance. While the screening process used surface flooding and property impacts as screening criteria, the improvements modeling used the current stormwater design standard (handling the 10% AEP, 12-hour storm with water surface elevations at least 2' below the ground surface) as a design performance goal.

1. *Green Streets / Localized BMPs:*

   The Green Streets improvement concept aims to minimize runoff volume through localized storage and infiltration within the City right-of-way (ROW).

   The City's Green Streets policy includes on-site infiltration standards for public roadway and right-of-way (ROW) construction and reconstruction projects. The policy calls for infiltration of 1 inch (1^{st} flush), 2.35 inches (50% annual chance 24-hour storm) or 3.26 inches (10% annual chance 24-hour storm) of total precipitation volume that falls on the ROW, depending on site soil conditions, slope and proximity to floodplain. It was assumed that on-site infiltration is not practical in areas that have historically had groundwater levels within 5 feet of the ground surface.

   To represent the Green Streets BMPs, the "depression storage" parameter for the relevant sub-catchments was increased accordingly to represent additional storage of runoff and the subsequent infiltration within ROW. The additional depression storage volume was calculated as the area-weighted average between storage in the ROW area (1 to 3.26 in) and non-ROW area (0.08 in for impervious area and 0.16 inch for pervious area).

2. *Engineered Storage:*

   This concept aims to reduce peak flow rates by detaining runoff flows with designated underground or surface storage locations.

   Large underground or surface detention facilities were considered based on availability of large open space. It is assumed that the facilities would be drained by gravity so their depths would be limited by the invert elevations of the adjacent storm drainage system. Some realignment of existing storm sewers would usually be involved to re-route runoff to the desired engineered storage location. Siting involves initial assessment of utility conflicts based on GIS data, but further evaluation would be required upon moving to the preliminary design phase for any of these locations.

When evaluating the storage elements in the stormwater model, these facilities were either represented as a rectangular storage node or as a large conduit link. The storage volume for each location was determined by storing enough 10% AEP storm runoff to minimize flooding at the study location, while limiting outflow from the storage feature(s) to the pre-development release rate standard of 0.15 cfs/acre.

3. *Conveyance Improvement:*

   This conceptual improvement approach is intended to move runoff offsite from the study location by providing additional capacity in the pipe system.

   This concept looks at increasing the capacity of the existing drainage system to convey more runoff downstream from the study area and reduce the peak hydraulic grade line (HGL) to be at least 2 feet below ground during the 10% AEP, 12-hour storm. This is an iterative approach that could include increasing the size of existing storm pipes or installing new storm relief pipes.

Improvements were all evaluated using the 10% AEP, 12-hour design storm (2.9 inch). Improvement scenarios for each site were based on one of the concepts or a combination, if improvements could not be achieved by one concept alone. Not all of the conceptual approaches were considered for each site, since their application at some sites would not be feasible or practical.

It is noted that the scope of this project was focused on using the model to evaluate stormwater system changes but other approaches should also be considered for addressing the study areas. Alternative approaches could include the purchase and/or modification of affected properties so that predicted surface flooding does not affect private property. This approach would not improve the system to the current stormwater design standard, but it may be significantly less costly. Model output showing the number of parcels and structures affected by predicted surface flooding could be used for further consideration of this approach.

C. Site descriptions and recommendations

The stormwater improvements evaluations are presented in this section following a similar format to the public meeting presentations. For each study area, the following items are described:

- Problem Definition
- Alternatives analysis
- Evaluation summary and recommendation

The evaluation summary was developed to support the prioritization of each recommended project as part of the City's Capital Improvements Programming (CIP). The stormwater model and improvements evaluation were used to generate output that would align with City's established scoring criteria, as shown in **Table 4-2**:

Table 4-2 – SWM project alignment with CIP scoring criteria

CIP Criteria	Weighting	SWM Output for City Scoring
System Influence/Capacity	100	# of parcels benefitted; # of structures benefitted
Water Quality	100	% reduction in peak flow and volume
Safety/Compliance/Emergency Preparedness	75	Notes on potential safety issues during construction or O&M activities
Coordination with Other Projects	75	
Funding	70	Cost estimate
O&M (Operations & Maintenance)	70	O&M cost estimates
User Experience (Level of Service)	65	Net improvement in LOS
Partnerships	65	Notes on impacts/benefits for WCWRC, UM or Townships
Sustainability/Environmental Goals	50	Notes on alignment with Sustainability goals
Innovation	40	Notes on inclusion of BMPs
Master Plan Objectives	25	Notes on alignment with Master Plan goals

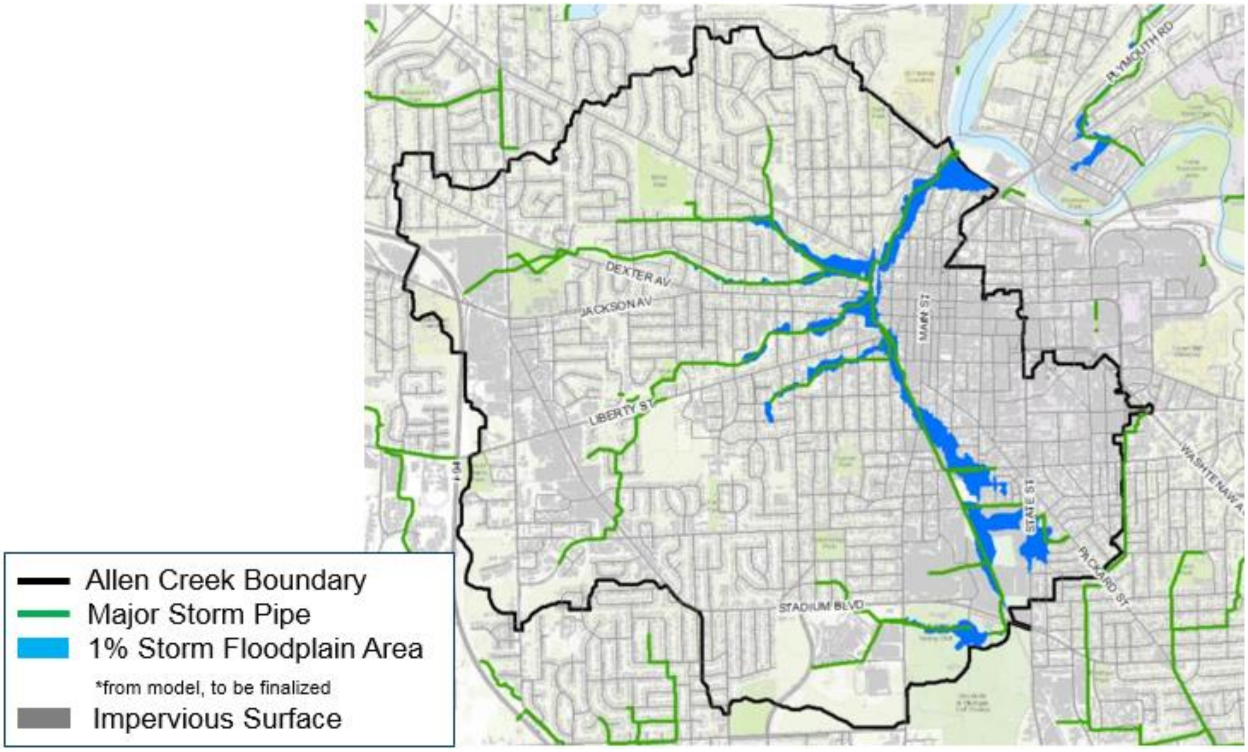
Where cost estimates are presented, these have been developed using unit costs from current City construction projects with cost escalation to year 2017. The Springwater Subdivision Improvements Project was used for direct unit costs for storm sewer pipe, and multipliers were added to account for design/engineering, other structures and utilities, and construction contingencies to develop the overall project costs presented. A similar approach was taken for project cost estimates for infiltration BMPs, underground storage, and surface storage. Upper end cost estimates from more complex projects were used to estimate costs for areas where construction would be more difficult.

i. Lower Allen Creek

The Allen Creek tributary area has a much higher proportion of impervious surfaces compared to other areas of Ann Arbor. The Allen Creek watershed includes downtown Ann Arbor, as well as the majority of the University of Michigan Central Campus and South Campus areas. Major branches of Allen Creek extend to the west, collecting drainage from the west side of Ann Arbor.

Almost the entire length of the creek has been enclosed in storm sewers that are owned by either the City of Ann Arbor or the Washtenaw County Water Resources Commissioner. The lower sections of the enclosed creek were built in the early 1900's and only have capacity to convey the 50% AEP storm. Surface flooding occurs frequently in lower areas and extensive surface flooding is predicted in the 1% AEP storm, as shown in **Figure 4-1**.

Figure 4-1 – Allen Creek Stormwater System Overview



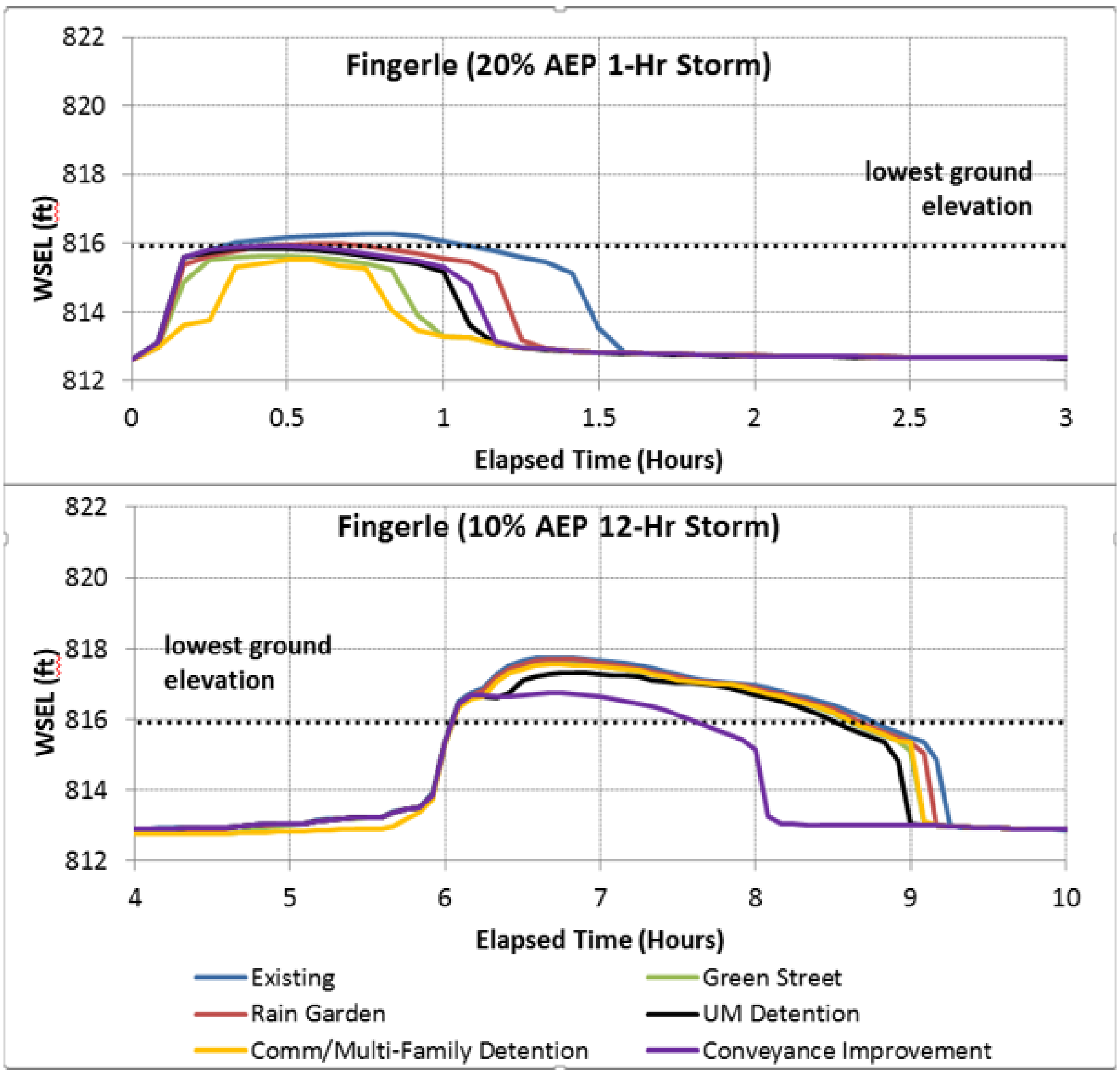
The 1997 stormwater master plan for the City of Ann Arbor evaluated conveyance improvements, and it was estimated that increasing the pipe size to accommodate the 10% AEP design storm at that time would cost around $40 million. A similar evaluation was prepared as part of this project and an overall estimate range of $150M - $200M was established for conveyance of the 10% AEP design storm. This cost estimate includes land acquisition of properties that would be substantially impacted by the expanded pipe footprint, but a complete engineering analysis to evaluate the feasibility of construction and land acquisition was beyond the scope of this project.

Because of the scale of the Allen Creek flooding problems, the project team recognized that a single improvement strategy, such as the conveyance improvements noted above, would be very difficult to implement and would have a high construction cost. Therefore, the model evaluation process for the Lower Allen Creek was designed to provide comparative information on different improvement strategies so that long term programs could be put in place to reduce or manage stormwater flows as effectively and efficiently as possible. The major sources of stormwater runoff are from impervious surfaces and management of these sources was considered in the following strategies:

- Right-of-Way areas - Green Streets Policy - Infiltration criteria based on Green Street Policy
- Residential properties - Rain gardens for single family homes- Capture the runoff from first 1” of precipitation
- University of Michigan properties - 1% AEP storm detention for all UM properties
- Commercial/Multi-family Residential properties - Storm detention for all commercial / multi-family properties per current development standards

As a reference, the results for these different strategies are shown in comparison to the 1997 master plan conveyance improvement strategy. **Figure 4-2** below shows the predicted water surface elevation for baseline conditions (blue) and for the other stormwater improvement strategies for Allen Creek at Madison near the Fingerle Lumber property.

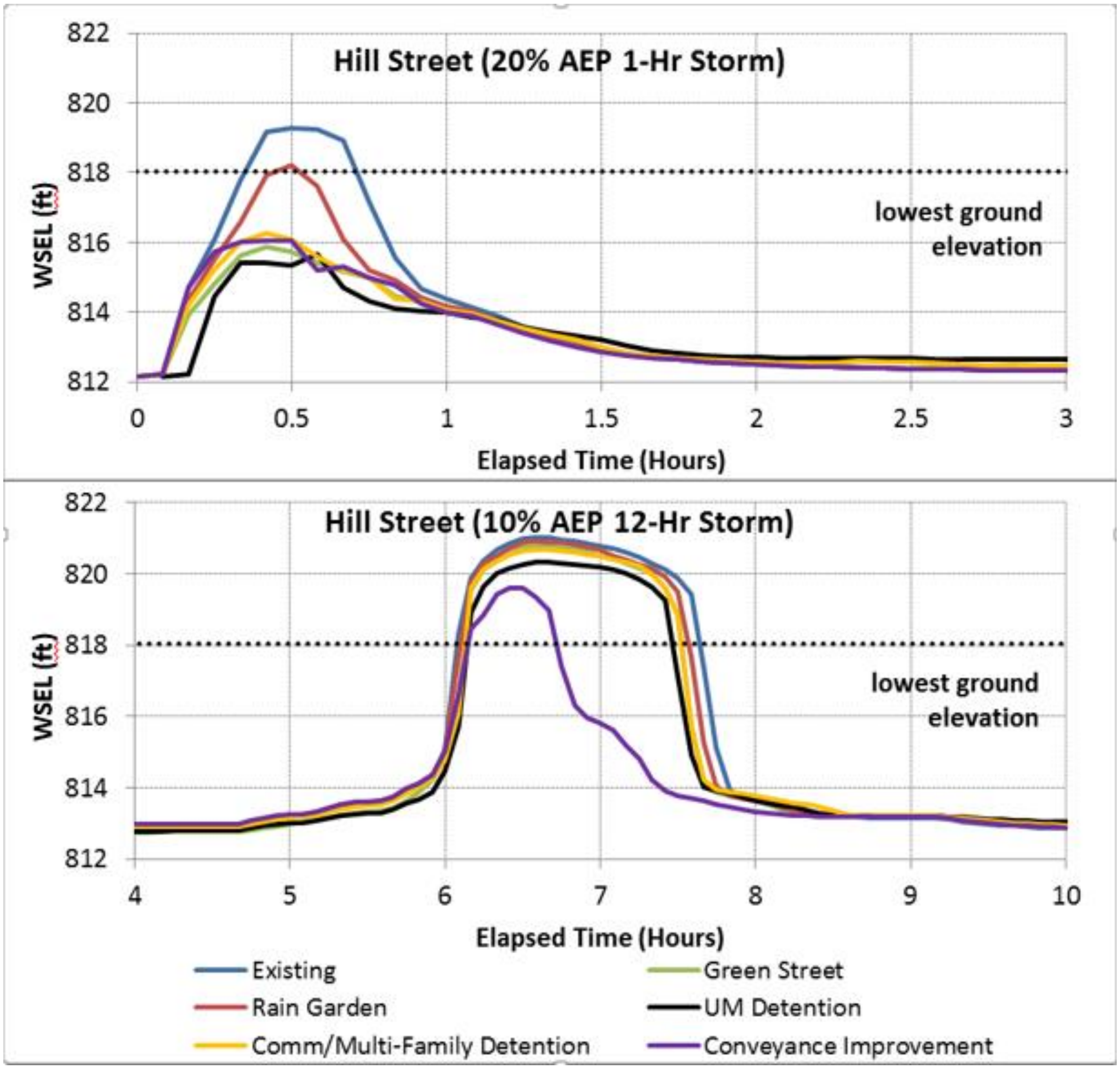
Figure 4-2 – Stormwater Improvement Comparison for Allen Creek at Madison Avenue



The top graphic shows the water surface elevation (WSEL) for the 20% AEP, 1-hour storm. While the model predicts surface flooding under baseline conditions, each of the individual improvement strategies would bring water levels below the ground surface at this location. For the 10% AEP, 12-hour storm, however, the individual stormwater management strategies have minimal impacts on peak water levels.

Similar results are seen at Hill Street in **Figure 4-3**, although it is notable that the impacts of University of Michigan properties are more significant since they make up a larger portion of the tributary area to this location.

Figure 4-3 – Stormwater Improvement Comparison for Allen Creek at Hill Street



Recommendation

The individual stormwater management strategies are not sufficient to eliminate flooding in the 10% AEP, 12-hour design storm as the pipe capacity along most of the lower sections of Allen Creek would still be exceeded. However, each strategy can be effective at reducing the frequency of flooding, and are especially effective during smaller storm events. University of Michigan properties are significant for the local stormwater system and for Allen Creek in the Hoover to Hill Street area. Our recommendation is to continue work on all of the studied stormwater management strategies to achieve incremental improvements in reducing peak stormwater flows over time.

Application of the Green Streets policy throughout the Allen Creek watershed, would require an investment of $80 million to $120 million (in 2017 dollars to match other project cost information). Other stormwater management alternatives would generally be funded by private property owners as part of redevelopment or as part of future stormwater management policies, so these costs have not been included. The cost of Green Streets implementation, which would be spread over many decades as roadways are reconstructed, compares favorably to a conveyance improvement for Allen Creek, which was estimated to cost up $150 million to $200 million, and which would require significant property acquisition in areas that would impacted by installation of a large pipeline.

For stormwater management on private property, the City should be proactive in creating and enacting policies that require property owners to manage stormwater on site. Requiring stormwater management during redevelopment would be a good next step, but incentivizing the implementation of stormwater management should also be considered. This approach could be similar to the current residential stormwater credit programs for becoming a RiverSafe Home partner, or building a rain garden or installing rain barrels.

Additional information about model analysis of stormwater management options for both Allen Creek and other creeksheds is included in section 5 of this report. Section 5 presents the options in different levels of combination in terms of the projected level of completion under future scenarios.

ii. Edgewood/Snyder

This location is characterized by street flooding in the low area at the intersection of Edgewood and Snyder. While the stormwater drainage system travels south across W. Stadium, the surface grade of W. Stadium is higher than the Edgewood/Snyder intersection, preventing a surface outflow pathway as shown in **Figure 4-4**. The upstream pipe system along Martha Avenue and Snyder does not have sufficient capacity to convey the 10% AEP design storm, so overland street flow is predicted.

Figure 4-4 – Existing conditions results for Edgewood/Snyder (10% AEP, 12-hour storm)

Alternative 1: Green Streets and Storage

Soil conditions in this area are expected to be suitable for infiltration so a significant infiltration capacity was assumed for the right of way (ROW) areas. The modeling assumed 3.26" of infiltration for the full extent of the upstream ROW, as shown in **Figure 4-5**. This would provide a total infiltration volume of 2.22 million gallons (MG). Even with this level of infiltration, pipe upsizing would be required along Edgewood and 0.22 MG of underground storage would still be required.

Figure 4-5 – Conceptual Layout of Green Streets Alternative for Edgewood/Snyder

Alternative 2: Conveyance Improvement and Storage

The model was used to evaluate a storage improvement alternative, as shown in **Figure 4-6**. Pipe upsizing would be provided along Martha Avenue, Snyder, and Edgewood to address the street flow, and 0.64 MG of storage volume would be required. Siting for a specific storage location was beyond the scope of this evaluation, but the open area between Stadium Blvd. and the existing Pioneer High School retention basin is shown as the general location assumed for the modeling analysis.

Figure 4-6 – Conceptual Layout of Storage Alternative for Edgewood/Snyder

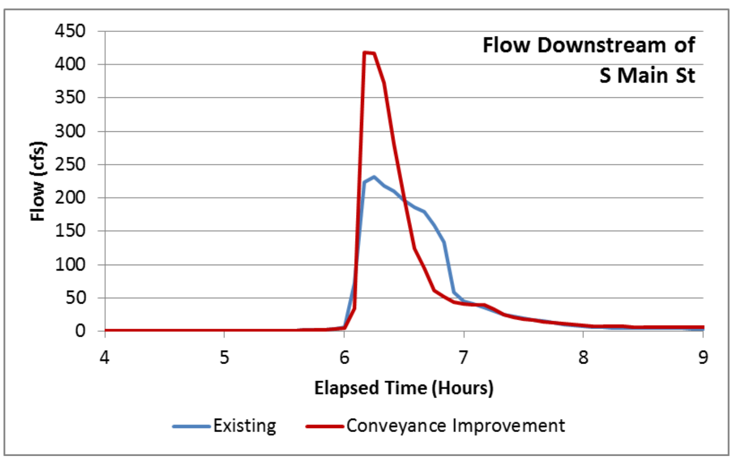
Alternative 3: Conveyance Improvement and Relief

In this alternative, the conveyance improvements are made in the neighborhood and the increased flows are bypassed around the Pioneer High retention basin, since this facility is already at its capacity. This option is shown in **Figure 4-7**.

Figure 4-7 – Conceptual Layout of Conveyance Alternative for Edgewood/Snyder

During the 10% AEP storm, the conveyance improvements would primarily move street overland flow into the expanded pipe system. Moving these flows downstream more quickly would nearly double the peak flows and would impact the performance of the stormwater ponds on the University of Michigan golf course, as shown in **Figure 4-8**. A 54" diameter relief pipe would be needed for this option and the total length of pipe upsizing would be 6,900 LF.

Figure 4-8 – Flow Hydrograph Comparison for Conveyance Alternative at Edgewood/Snyder



Recommendation

The recommended solution for Edgewood/Snyder is the local conveyance and storage alternative as shown in **Table 4-3**. This approach would reduce properties affected by flooding in the 10% AEP storm by 15 properties and would reduce the risk of structure impacts by 6.

Table 4-3 – Recommended Edgewood/Snyder Option

Alternative	Probable Cost
Green Streets + Engineered Storage	$7.0m - $7.9m
Conveyance Improvement + Engineered Storage	$3.5m - $4.1m
Conveyance Improvement + Relief Pipe	$2.5m - $2.9m

Evaluation Matrix Criteria	
System Influence/Capacity	15 parcels w/ improved drainage; 6 structures at reduced risk of flooding
Water Quality	2% reduction in peak flow; no change in volume
Funding	$4.1m capital cost; $10K annual O&M cost
Level of Service (LOS)	LOS improves from 50% AEP storm to 10% AEP storm
Other Criteria	Opportunities for upstream BMPs in future; improved vehicle access; partnership potential with AAPS, UM

Other considerations for this recommended alternative include coordination with the upcoming West Stadium improvements project and potential local storage at the Edgewood/Snyder intersection, especially if the church parking lot at the southeast corner could be utilized.

The City should also consider a long term phasing approach where the local flooding issue at Edgewood/Snyder is addressed first, with other neighborhood improvements addressed in the future. While it would not immediately address the 10% AEP storm, this approach may be the most feasible and cost-efficient. This approach would likely include the following steps:

1. Upsize pipe across W. Stadium at Edgewood to provide outlet capacity
2. Provide local storage at Edgewood/Snyder intersection or south of Stadium Blvd. to reduce peak flows through storage and infiltration.
3. Evaluate street flooding impacts versus Green Streets impacts as road reconstruction projects are completed in the future.

iii. Park Place Apartments

The stormwater system problem at this location is caused by both the pipe size and the surface grading, which prevents an overland flow pathway. Under existing conditions, the pipe capacity is reached during the 50% AEP storm, and surface flooding begins to appear at the 20% AEP storm or larger. Surface flooding affects the lower level units of the apartment building located at the eastern edge of the property, as shown in **Figure 4-9**.

Figure 4-9 – Existing conditions results for Park Place Apartments (10% AEP, 12-hour storm)

Alternative 1: Infiltration BMPs

Because this is a private property, a Green Streets approach was not considered. Instead, the infiltration volume needed to allow the existing system to convey the 10% AEP storm was calculated. 0.93 MG of infiltration would be required, which would be difficult to achieve in this area, due to limited space and unknown soil infiltration capacity. This alternative would require significant property owner cooperation, as most of the infiltration area is located outside of the City's drainage easement.

Alternative 2: Detention Storage

Surface flooding can be controlled in the 10% storm with some pipe upsizing at the bottom of the parking lot area, and underground detention in the open area at the eastern edge of the property. This alternative is feasible but would require work outside of the City's existing drainage easement. This option is shown in **Figure 4-10**.

Figure 4-10 – Conceptual Layout for Storage Alternative at Park Place Apartments

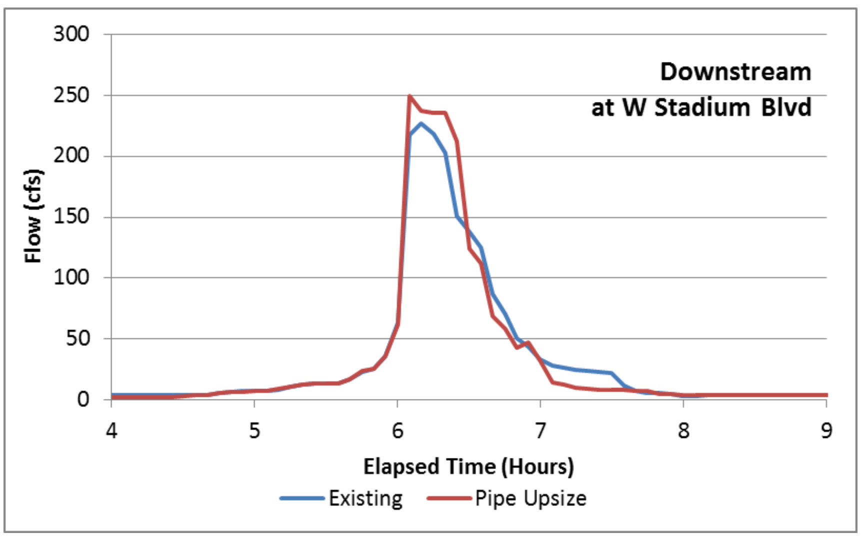
Alternative 3: Conveyance Improvement

Pipe upsizing can be provided to convey peak flows for the 10% AEP storm with only minimal impacts on downstream peak flows. This alternative would require upsizing of storm pipes from Pennsylvania Ave to W. Stadium Blvd, as shown in **Figure 4-11**. By conveying the larger storm, the basement apartment units would be protected up to the 4% AEP 24-hour storm.

Figure 4-11 - Conceptual Layout for Conveyance Alternative at Park Place Apartments

Downstream peak flows at W. Stadium would increase by approximately 10% as shown in **Figure 4-12**. This increase could be mitigated using local storage or BMPs at available locations on the property.

Figure 4-12 – Flow Hydrograph Comparison for Conveyance Alternative at Park Place Apartments



Recommendation

The recommended approach for improvements for the Park Place Apartments would be to provide conveyance improvements, which can be provided within the City's existing drainage easement at a reasonable cost and without any major property impacts. To mitigate peak flow increases downstream, the City should seek a cooperative solution with the property owners to provide infiltration within the property.

iv. Churchill Downs

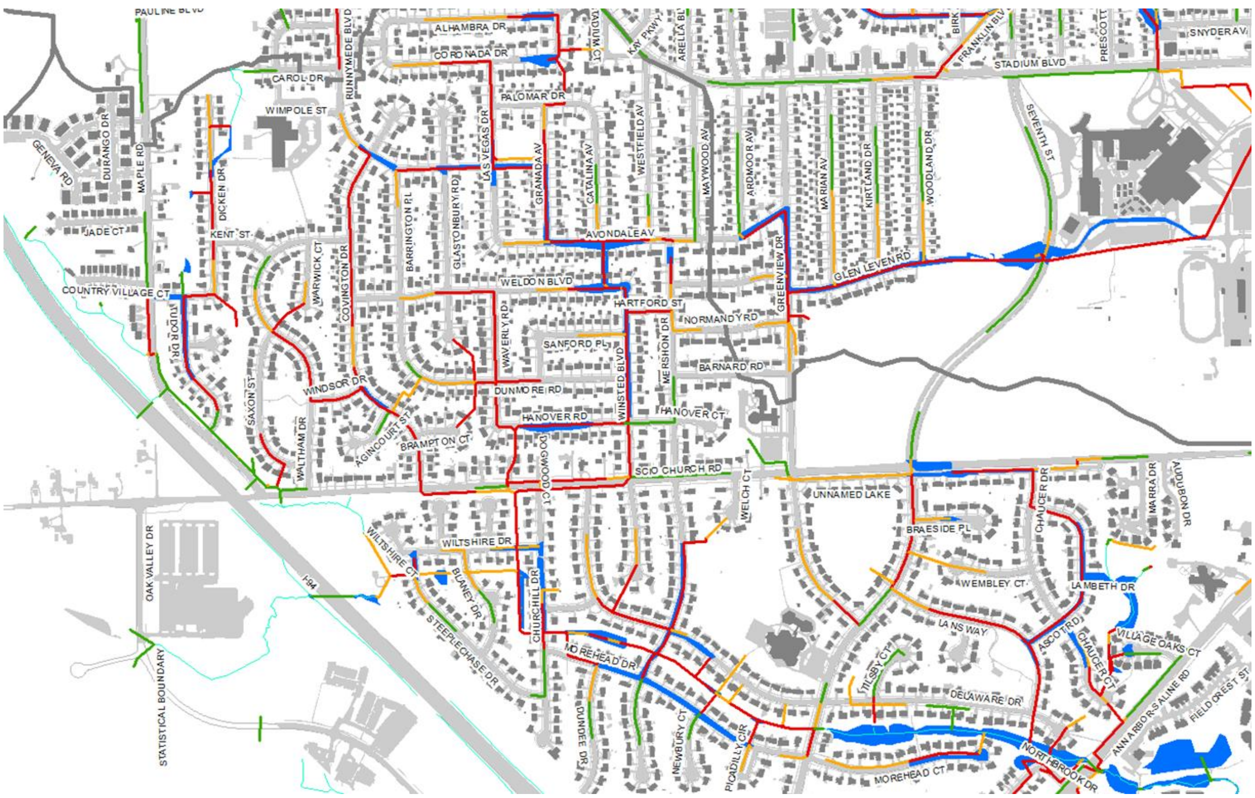
The Churchill Downs subdivision is located in the upper portion of the Malletts Creek watershed. The creek itself is a County Drain from Ann Arbor-Saline Road up to I-94. Local Ann Arbor stormwater pipes collect stormwater flows from the local streets, as well as the Glen Leven neighborhood, which is located north of Scio Church Road. Portions of Pittsfield Township, located west of I-94, also drain into this area.

The County Drain sections of the stormwater system, along with other local pipes, reach their capacity during the 50% AEP storm and surface flooding is predicted in the 10% AEP storm. Stormwater drainage issues in this area were highlighted during the March 15, 2012 event, when surface flooding affected numerous properties and streets.

The Upper Malletts Stormwater Conveyance Study, completed in early 2014, considered potential stormwater improvements to control flooding under a storm equivalent to the March 15, 2012 event. It should be noted that the 10%, 12-hour design standard has a much greater volume than the March 15, 2012 event, which was a shorter duration event, with a peak rainfall duration of only 75 minutes and a total storm duration of less than 3 hours.

Figure 4-13 below shows the existing conditions modeling results for the 10% AEP storm for the Churchill Downs and Lansdowne neighborhoods. Pipe capacity is exceeded for most of the stormwater system and surface flooding is predicted in many locations.

Figure 4-13 – Existing conditions results for Churchill Downs (10% AEP, 12-hour storm)

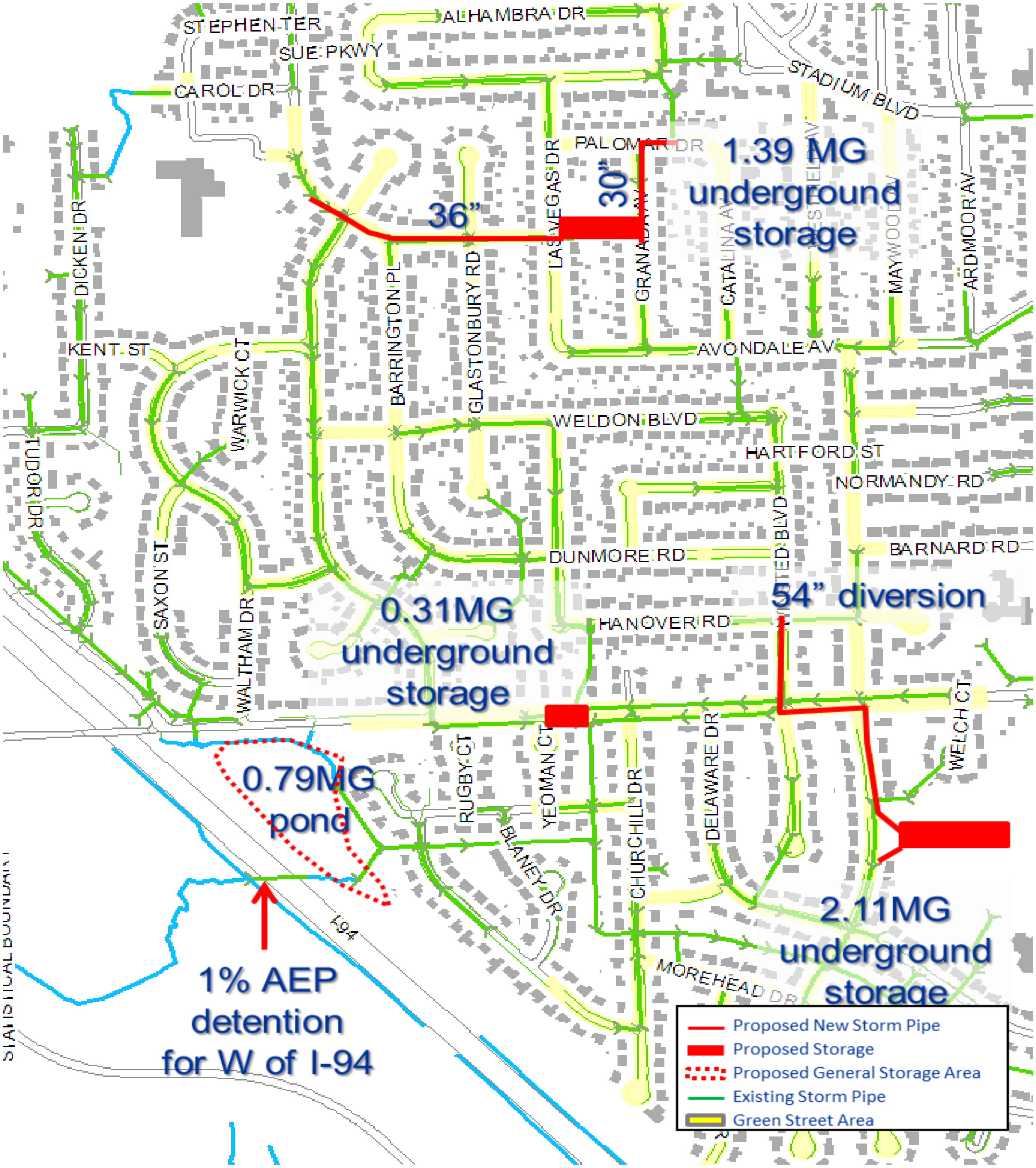


Alternative 1: Green Streets Improvements

Alternative 1 was built around the City's Green Streets policy for runoff control in right of way (ROW) areas. Because of poor soils for infiltration, BMPs were assumed to provide capture and storage of the first flush 1" of ROW runoff. These measures alone were not sufficient to achieve current stormwater design standards, so some conveyance and storage improvements are also included in this alternative.

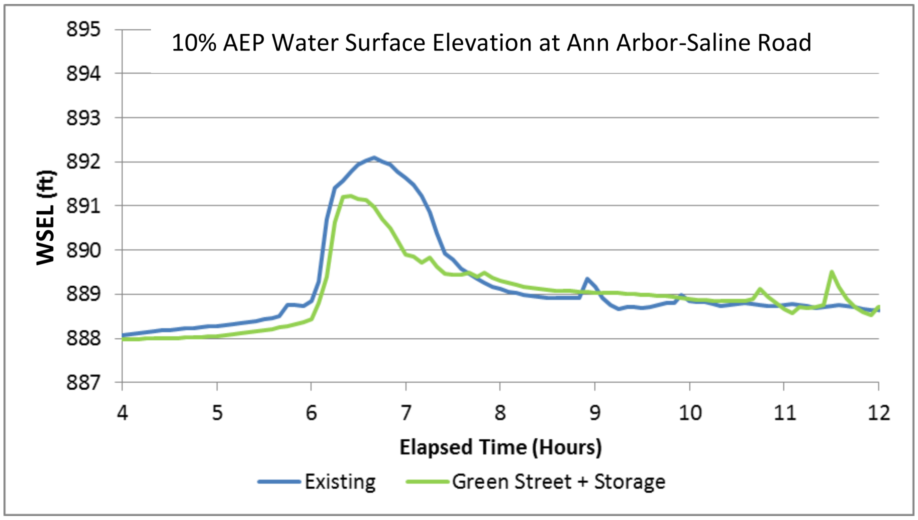
The alternative 1 conceptual layout is shown in **Figure 4-14**. More details on the individual stormwater improvement features are included in alternative 2, which was developed with a focus on stormwater storage.

Figure 4-14 – Conceptual Layout for Green Streets Alternative for Churchill Downs



Using this alternative, the model predicts that the stormwater system would be within capacity during the 10% AEP storm, and the neighborhood outlet pipe at Ann Arbor – Saline Road would have a lower peak water surface elevation, as shown in **Figure 4-15**.

Figure 4-15 – Water Surface Elevation comparison for Green Streets Alternative for Churchill Downs



Alternative 2: Local and Regional Storage

Because of the limited infiltration soils, some conveyance improvements and storage would be required to supplement a BMP-focused alternative, as described in Alternative 1. Taking away the BMPs for ROW runoff, more stormwater flows would need to be conveyed and stored but the overall nature of the pipes and storage facilities would not need to change. As shown in **Figure 4-16**, the same locations are utilized for conveyance and storage improvements, although the sizing does increase.

Notable features of this alternative are as follows:

- Underground storage at Las Vegas Park – Storm drain pipes along Runnymede and Granada would be upsized to convey 10% AEP design flows. These increased flows would be mitigated at Las Vegas Park, where underground storage could be provided without significant impacts on trees or park uses.
- Winsted Blvd. diversion and Lawton Park underground storage – The current drainage pathway for the tributary area north of Winsted Blvd. (including Weldon Blvd., Avondale Ave, and connecting streets to the north) is west along Scio Church Road to the County Drain behind properties on the west side of Churchill Drive. This alternative would divert flows from Winsted Blvd. into a new storm drain pipe that would convey flows to a new underground storage basin at Lawton Park.
- Surface storage pond at Eisenhower Park – Stormwater flows from Maple Road, Tudor Drive, and Dicken Drive are conveyed across Scio Church Road through an open channel pathway in Eisenhower Park and then into the County Drain at Churchill Downs Park. Storage of these flows is recommended in Eisenhower Park in a surface storage pond. Other options for storage could be explored to the north along Maple Road or the I-94 corridor, but Eisenhower Park was

assumed for the purposes of evaluating flow impacts in this study. The small storage area under Scio Church Road west of Churchill could also be eliminated if stormwater flows from Covington were diverted to Eisenhower Park. Under this scenario, the size of the Eisenhower basin would need to be expanded to accommodate additional volume.

- Upstream detention for areas west of I-94 – Currently, a 54" diameter pipe brings flow from I-94 and Oak Valley Drive under the freeway and into the Churchill Downs neighborhood at Churchill Downs Park. While some properties in Pittsfield Township have stormwater controls, a control basin at the freeway culvert would reduce peak flows into the county drain. This area is outside of the City of Ann Arbor so any infrastructure improvements would have to be designed and constructed in cooperation with the Washtenaw County Water Resources Commissioner, the Michigan Department of Transportation (MDOT), and Pittsfield Township.

Figure 4-16 – Conceptual Layout for Storage Alternative for Churchill Downs

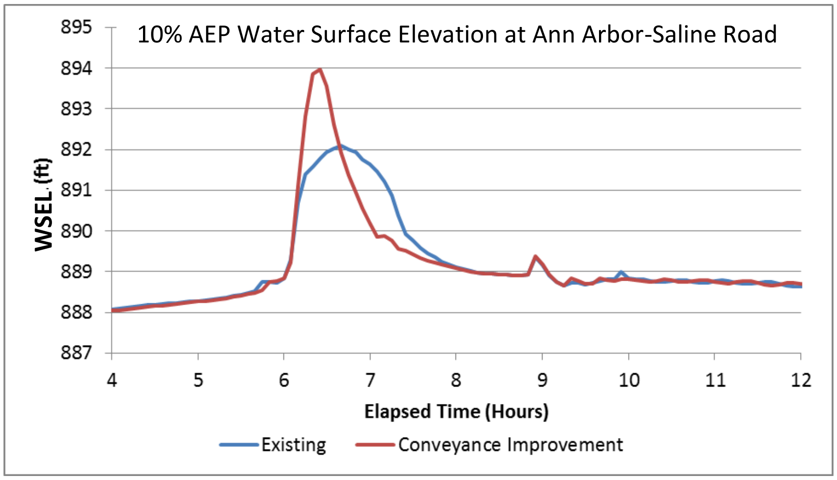
Alternative 3 – Conveyance Improvement

The stormwater model was used to evaluate an alternative focused around increased conveyance capacity. Starting with Runnymede Blvd., Palomar Drive, and Granada Avenue, larger pipes would be installed to convey the flows predicted for the 10% AEP storm, as shown in **Figure 4-1**7. Following the main flow pathway along Avondale, Weldon, Winsted, and Scio Church, the pipe size would be increased to 54" and then 72" diameter. Once the County Drain is reached, the predicted flows would require a parallel relief storm pipe of 72" to 84" in diameter. With limited space in the backyard areas, the relief pipe would likely need to be installed along Churchill Drive, Delaware Drive or Morehead Drive.

Figure 4-17 – Conceptual Layout for Conveyance Alternative for Churchill Downs

With the increased conveyance capacity along the primary drainage pathway, peak flows during the 10% AEP storm would be increased by nearly 100% and the peak water surface elevation at the neighborhood outlet at Ann Arbor – Saline road would increase by 2 feet, as shown in **Figure 4-18** below.

Figure 4-18 – Water Surface Elevation Comparison for Conveyance Alternative at Churchill Downs



Recommendation

Because of the soil characteristics in this area, a BMP-focused alternative cannot achieve 10% AEP stormwater management without some conveyance and storage facilities. The incremental cost of increasing the sizes of these facilities to handle the stormwater makes the storage-focused alternative the best solution for this study area, as shown in **Table 4-4**.

Table 4-4 – Recommended Churchill Downs Solution

Alternative	Probable Cost
Conveyance Improvement + Regional Storage + Green Street	$27m - $31m
Conveyance Improvement + Regional Storage	$14m - $16m
Conveyance Improvement	Not feasible; Increases flooding

Evaluation Matrix Criteria	
System Influence/Capacity	54 parcels with improved drainage; 7 structures at reduced risk of flooding
Water Quality	28% decrease in peak flow, no change in volume
Funding	$16m capital cost; $25k annual O&M cost increase
Level of Service	LOS improves from 20% AEP storm to 10% AEP storm
Other Criteria	Improved vehicle access; Low-to-medium soil infiltration; Possible partnership with Pittsfield Twp., MDOT;

While the total cost of the improvements is high, the different features can be implemented selectively to achieve improved stormwater system performance. The recommended improvements should be prioritized as follows to provide the greatest impacts on flows and on locations with predicted surface flooding:

1. Winsted Blvd. diversion and Lawton Park underground storage - This storage basin and the associated flow diversions and conveyance upgrades would have the greatest impact on flooding locations south of Scio Church Road. However, it would also be the most costly ($7M - $8M) due to the size of the underground storage required.

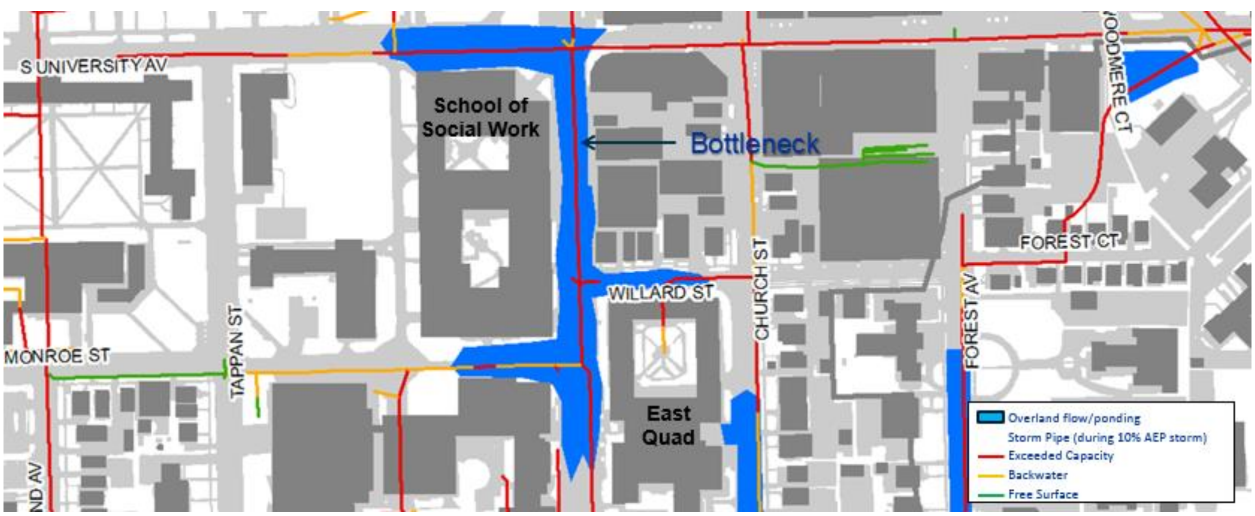
2. Surface storage pond at Eisenhower Park – Taken by itself, this storage feature has a less significant impact on stormwater system performance and flooding because of its smaller volume, but it would be much less costly, and would be necessary to eliminate flooding in Churchill Downs. Depending on how flows from Covington Drive and from west of I-94 are handled, this feature is estimated to cost $1.5M - $2M.

3. Underground storage at Las Vegas Park – This feature would primarily reduce street flooding and overland conveyance along Runnymede and Avondale and would not significantly reduce flooding in the Churchill Downs area. With an estimated cost of $5.5M - $6M, this element of the storage alternative is only recommended in order to bring the entire study area to a consistent design standard.

v. East University/South University

Street flooding is predicted along East University Avenue and South University Avenue during the 10% AEP storm as shown in **Figure 4-19** below. This surface flooding was verified during the June 2013 storm.

Figure 4-19 – Existing conditions results for East University (10% AEP, 12-hour storm)

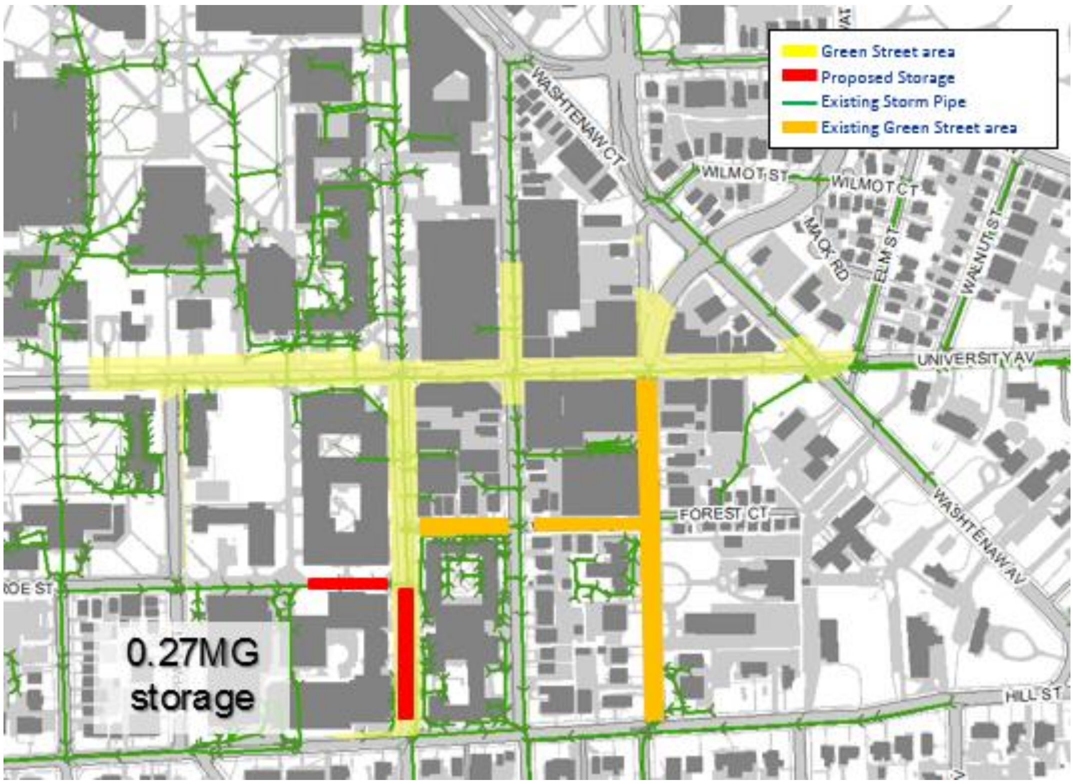


The stormwater pipe size along East University Avenue between South University and Willard is particularly undersized, causing a bottleneck that reaches its capacity during the 50% AEP storm. In addition to the predicted street flooding, below-grade loading docks and building entrances at the University of Michigan's School of Social Work Building are affected.

Alternative 1A – Engineered Storage and Green Streets

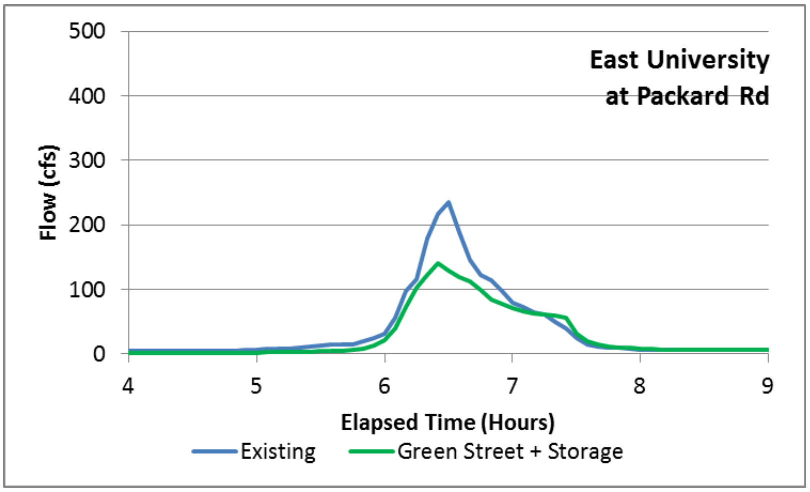
To complement streets that have already been reconstructed according to the Green Streets policy, this alternative considered implementation of the policy along similar streets in the tributary area to this study location. Washtenaw Avenue was not included because it is an MDOT roadway. Streets east of Washtenaw were not included because they are not likely to be on the same reconstruction schedule as the streets west of Washtenaw. With these assumptions for BMP implementation, some localized stormwater storage along the Monroe Pedestrian Mall and under East University north of Hill would be required to meet the 10% AEP design standard. This conceptual layout is shown in **Figure 4-20**.

Figure 4-20 – Conceptual Layout for Green Streets Alternative for East University



The model evaluation of this alternative indicates that flows would be reduced significantly at the neighborhood outlet where East University meets Packard Road, as shown in **Figure 4-21**.

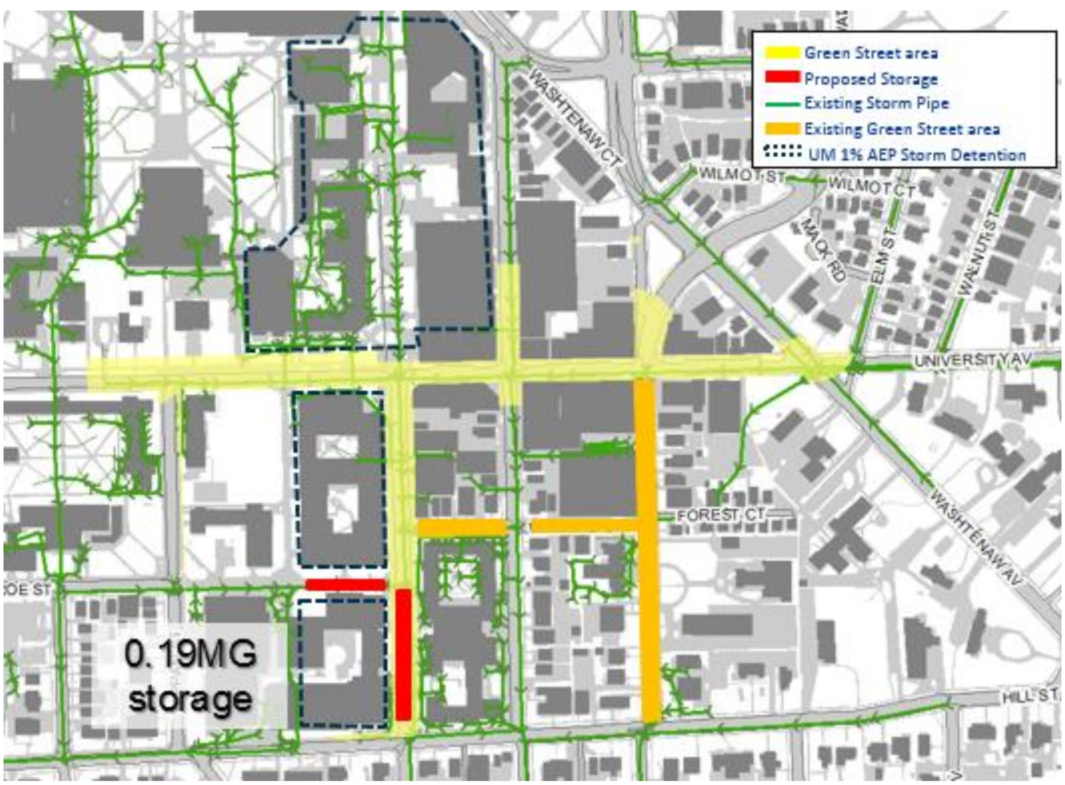
Figure 4-21 – Flow Hydrograph Comparison for Green Streets Alternative for East University



Alternative 1B – Engineered Storage and Green Streets with UM 1% AEP Detention

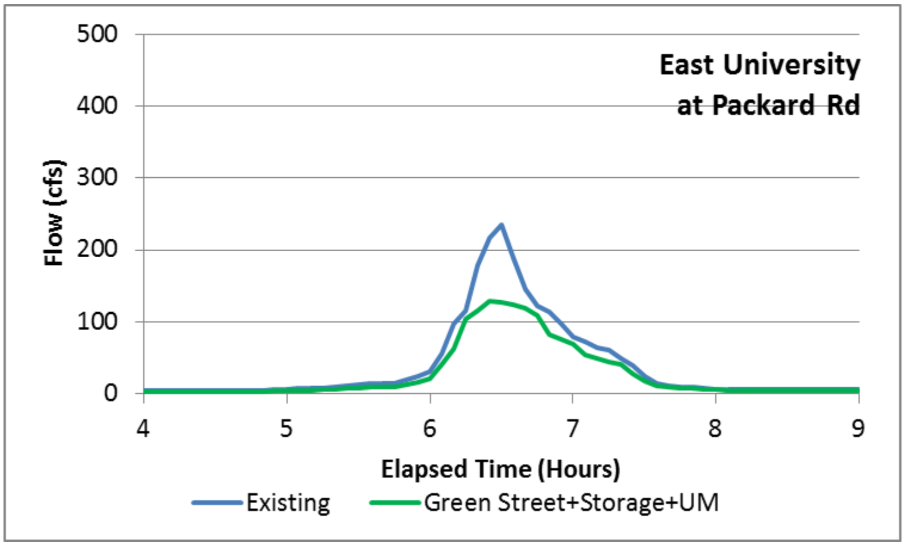
As a point of comparison for the relative impacts of ROW stormwater runoff and University property runoff, this alternative includes the same ROW improvements as Alternative 1A, but also includes 1% storm detention for University of Michigan properties located in the tributary area to this study location, as shown in **Figure 4-22**. This detention requirement would be consistent with the requirements for a new development in Washtenaw County.

Figure 4-22 – Conceptual Layout for Green Streets/UM Detention Alternative for East University



As shown in **Figure 4-23**, there would be some slight reductions in flows and volumes (when compared to alternative 1A) and the storage volume required at Monroe Mall and under East University would be reduced by 30% to 0.19 MG.

Figure 4-23 – Flow Hydrograph Comparison for UM Detention Alternative for East University

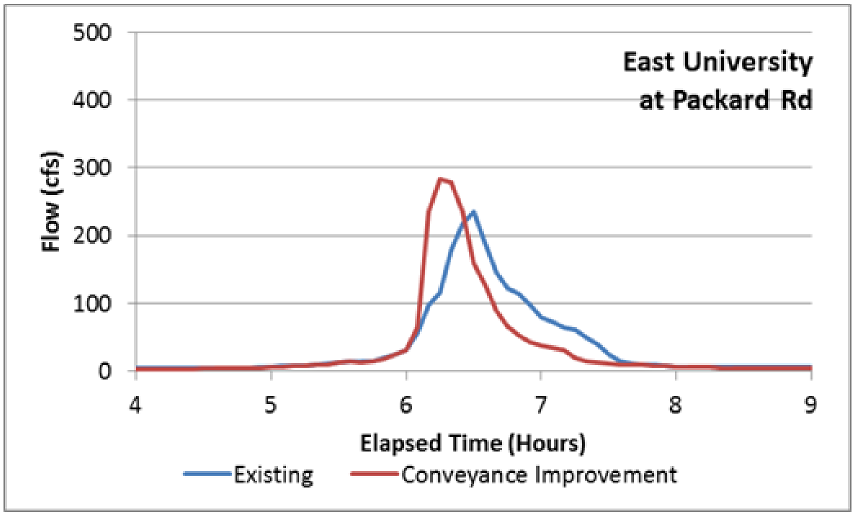


Alternative 2 – Conveyance Improvement

The model was used to evaluate a conveyance improvement for the East University study area, but with no local storage location to mitigate the increased flow, this option is not feasible. **Figures 4-24** and **4-25** show the conceptual layout and resulting flow hydrograph for this alternative.

Figure 4-24 – Conceptual Layout for Conveyance Alternative for East University

Figure 4-25 – Flow Hydrograph for Conveyance Alternative for East University



Recommendation

The Green Streets improvements in combination with local storage are recommended for this study area. Partnering with the University of Michigan to further reduce flows through local stormwater management initiatives would reduce the storage volume requirements and should be pursued.

Table 4-5 – Recommended East University/South University Solution

Alternative	Probable Cost
Green Streets + Engineered Storage	$3.4m - $3.8m
Green Streets + Engineered Storage + UM 1% AEP Storm Detention	$3.2m - $3.6m
Conveyance Improvement	$1.8m - $2.9m

Evaluation Matrix Criteria	
System Influence/Capacity	2 structures with below grade loading docks at reduced risk of flooding
Water Quality	50% decrease in peak flow; some reduction in volume
Funding	$3.6m capital cost; $17k annual O&M cost increase
Level of Service	LOS improves from 50% AEP storm to 10% AEP storm
Other Criteria	Meets sustainability goals; improved vehicle access; Possible partnership with UM

vi. Mulholland Avenue

This study location reviewed the Murray-Washington branch of Allen Creek between S. Seventh Street and W. Washington. Surface flooding has been reported historically at Mulholland Avenue and at Murray Avenue, with surcharging through the manhole on Mulholland reported most frequently. The model analysis of existing conditions showed that the pipe capacity in this area is reached during the 50% AEP storm, with a flat pipe between Murray and Washington causing the worst bottleneck. Once surface flooding begins at either Mulholland or Murray, overland flow is predicted between houses and in backyards. This is shown in **Figure 4-26**.

Figure 4-26 – Existing conditions results for Murray-Washington Drain at Mulholland Avenue

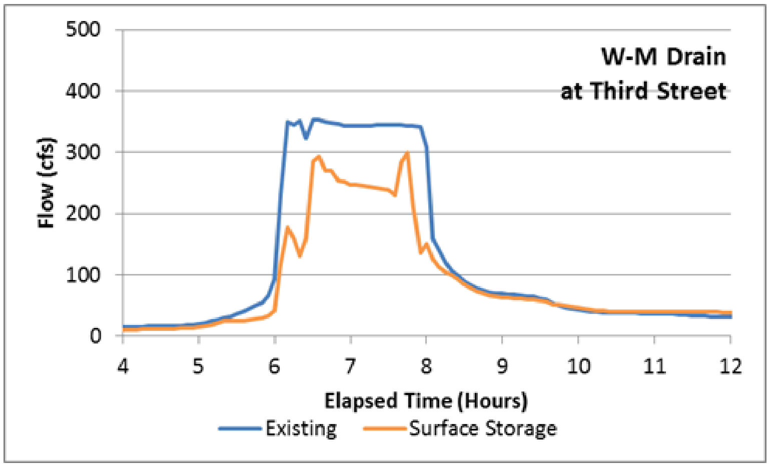
Alternative 1A – Surface Storage at Slauson Field

Reduction of peak flows with upstream storage was evaluated for this location. In alternative 1A, a shallow surface storage basin would be constructed in the open field adjacent to Slauson Middle School, between Eighth Street and Crest Avenue, as shown in **Figure 4-27**. A control structure would be required to restrict flows at this location and direct flow into the surface storage, and a low berm would be required along Eighth Street to retain the flows in the field area. Other considerations to limit the duration of flooding and to allow for proper post-event drainage would also be needed.

Figure 4-27 – Conceptual Layout for Surface Storage Alternative for Mulholland Ave

The location would allow for up to 2.2 MG of storage with an average depth of 2 feet. This volume would delay the downstream peak by approximately 2 hours, reducing peak flows by 15%, as shown in **Figure 4-28**.

Figure 4-28 – Flow Hydrograph for Surface Storage Alternative for Mulholland Avenue



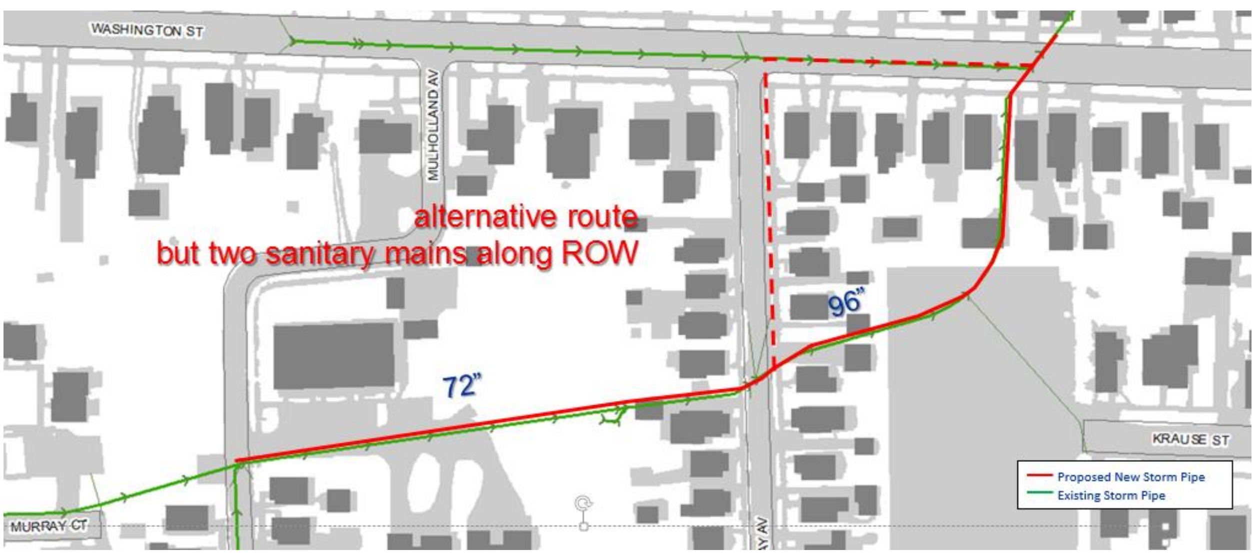
Alternative 1B – Above Grade Storage Tank

This alternative would be similar to Alternative 1A, but it would put the storage volume into an above grade storage tank near Crest Avenue. This would avoid issues with open surface storage but would take up space that is currently used for soccer, sledding, and other recreational activities. Impacts on flows would be similar to what is shown for Alternative 1A.

Alternative 2 – Conveyance Improvements

To address the localized flow restrictions, pipe upsizing could be performed between Mulholland and Washington to meet the 10% design storm flow rates. As shown in **Figure 4-29**, this would require construction in an older neighborhood, without much room to work, and large-diameter pipes. An alternative routing along Murray to Washington could be considered but would also likely have conflicts with other existing utilities, including sanitary sewer mains.

Figure 4-29 – Conceptual Layout for Conveyance Alternative for Mulholland Ave



The conveyance alternative would provide a significant improvement in reducing the frequency of surface flooding, from the 20% storm to the 2% AEP storm. However, peak flows would increase downstream in the Allen Creek watershed so mitigation of the peak flows would be recommended. This could potentially be accomplished with a storage basin at the University of Michigan Parking lot at the end of Krause Street but the proximity to the 100-year floodplain, and potentially high groundwater levels, could limit the capabilities of this site. A storage volume of 1.6 MG would be needed for the 10% AEP storm, which would be difficult to achieve.

Recommendation

Despite the potential difficulties of establishing an agreement to utilize an Ann Arbor Public Schools property, the location characteristics and available space at Slauson Middle School make the surface storage alternative the recommended solution. The probable cost for this location is potentially lower than what is shown in **Table 4-6** below since the engineering work and construciton required would be minimal, but there would also be significant unknowns with requirements for safely and sustainably storing stormwater at the site and for providing operations and maintenance support.

Table 4-6 – Recommended Mulholland Drive Solution

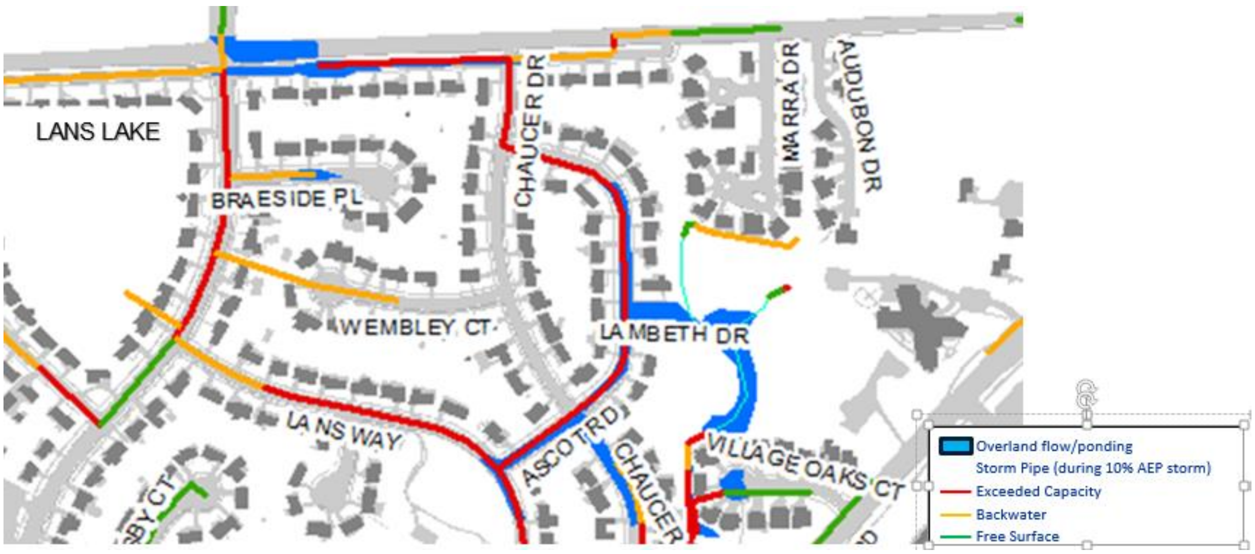
Alternative	Probable Cost
Surface Storage (AAPS property)	\$1.7m - \$1.9m
Above-Grade Engineered Storage	\$7.8m - \$9.3m
Conveyance Improvement	\$3.0m - \$4.1m

Evaluation Matrix Criteria	
System Influence/Capacity	12 parcels w/ improved drainage; 7 structures at reduced risk of flooding
Water Quality	15% decrease in peak flow; possible reduction in volume if infiltration BMPs are included
Funding	\$1.9m capital cost; potential increase in annual O&M cost
Level of Service	LOS improves from 20% AEP storm to 10% AEP storm
Other Criteria	Safety concerns with stormwater in a recreational space; Partnership with AAPS; Supports sustainability and master plan goals for Allen Creekshed

vii. Scio Church / S. Seventh Street

Although this study location is also part of the Upper Malletts Creek area (along with the Churchill Downs area described in section 4-C.4), the stormwater system is impacted by a separate tributary area so it was analyzed separately. The existing stormwater conveyance system reaches capacity during the 50% AEP storm and surface flooding is predicted for the 10% AEP storm, for which overland flow is predicted along Scio Church Road, Ascot Road, and Chaucer Court. These model findings were validated during storms in 2010 and 2012, when surface flooding was experienced along Scio Church Road, Ascot, and Chaucer, as shown in **Figure 4-30**. Some of these issues are also inter-related with overland flow in the Village Oaks-Chaucer drain that can be affected by overland flow down Lambeth Drive.

Figure 4-30 – Existing conditions results for Scio Church / S. Seventh Street



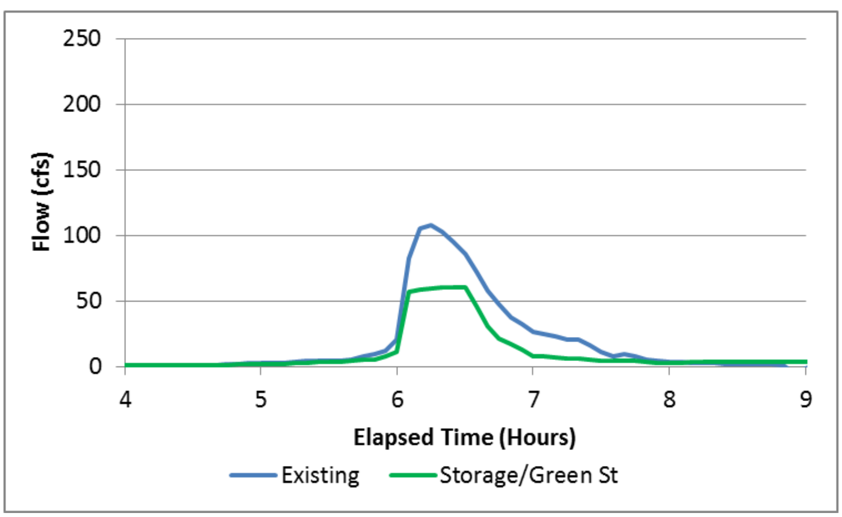
Alternative 1 – BMPs / Engineered Storage

Because the soils in this area are not expected to be favorable for infiltration, any ROW stormwater BMPs would function like local storage features. Specific locations were not identified for this study as the impacts on the stormwater conveyance system would be similar and the most efficient locations could be determined based on soil investigations and with input from the public. Potential storage locations are shown in **Figure 4-31**, and these could be located under the pavement, in the ROW, or in adjacent properties depending on all design considerations. Portions of the storage volume could also be moved to other portions of the tributary area as roadway reconstruction projects are implemented.

Figure 4-31 – Conceptual Layout for Storage Alternative for Scio Church / S. Seventh Street

The impacts of this alternative on flow rates were evaluated at the outlet of the Lans Way storm sewer into Malletts Creek. As shown in **Figure 4-32**, the peak flow is reduced by almost 50% and the volume is released much more slowly over time.

Figure 4-32 – Flow Hydrograph for Storage Alternative for Scio Church / S. Seventh Street



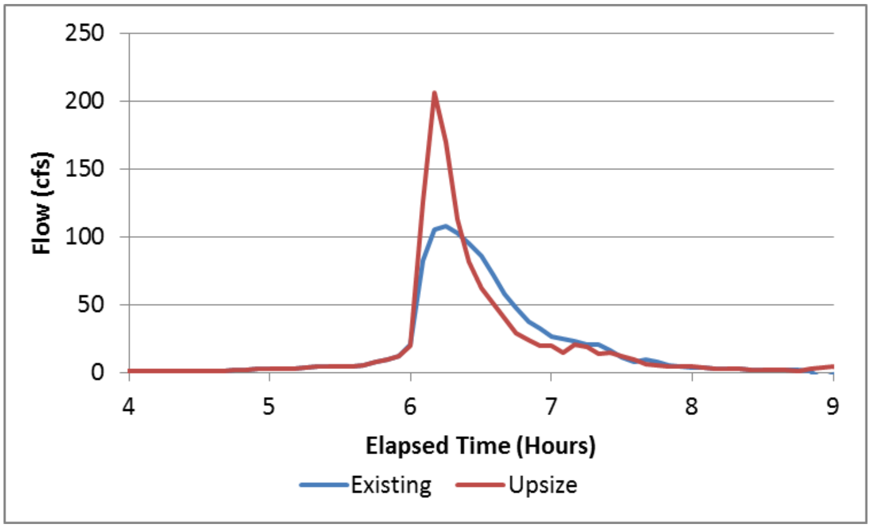
Alternative 2 – Conveyance Improvements

For comparison with the local storage option presented in alternative 1, pipe upsizing would be required along South Seventh, and all of Lans Way and all of Ascot Road to meet 10% AEP storm design standards, as shown in **Figure 4-33**.

Figure 4-33 – Conceptual Layout for Conveyance Alternative for Scio Church / S. Seventh Street

While the cost of this alternative would be lower, it would increase peak flows to Malletts Creek by nearly 100%, as shown in **Figure 4-34**.

Figure 4-34 – Flow Hydrograph for Conveyance Alternative for Scio Church / S. Seventh Street



Recommendation

To bring this study area to current stormwater design standards, a combination of engineered localized storage and BMPs could be provided. While this approach is more costly than a pipe upsizing approach, it would have the advantages of reducing peak flows to Malletts Creek, which better aligns with the watershed's Total Maximum Daily Load (TMDL) requirements and with the City's goals for sustainability.

Alternative	Probable Cost
Engineered Storage/BMP	\$2.1m - \$2.4m
Conveyance Improvement	\$1.3m - \$1.7m

Evaluation Matrix Criteria	
System Influence/Capacity	10 parcels at reduced risk of flooding
Water Quality	45% decrease in peak flow
Funding	\$2.4m capital cost; \$12k annual O&M cost increase
Level of Service	LOS improves from 20% AEP storm to 10% AEP storm
Other Criteria	Improved vehicle access; low infiltration potential

viii. Glendale/Charlton

This study area was identified by local residents during the Phase I public meeting series, where it was noted that street flooding and other stormwater and sanitary sewer issues have been experienced during large storms. The existing conditions modeling for the area shows that the stormwater pipes are at capacity during the 50% to 100% AEP storms, but surface flooding is generally limited to street overland flow along Charlton Avenue, where there is no storm sewer currently, and street ponding at low spots on Orchard Street and Glendale Drive. This is shown in **Figure 4-35**.

Figure 4-35 – Existing conditions results for Glendale/Charlton

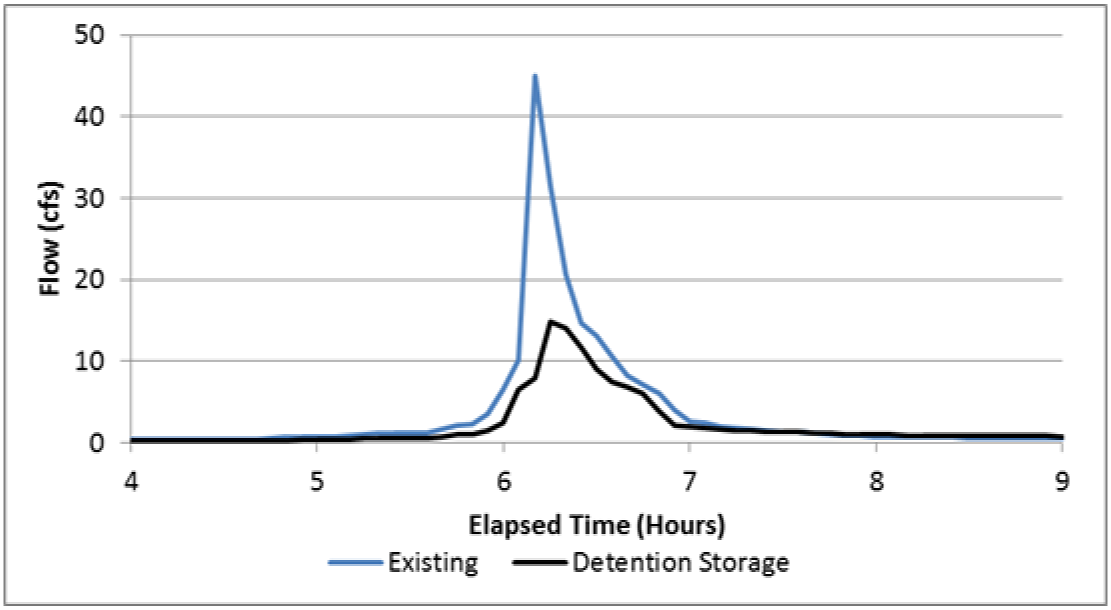
Alternative 1 – Detention for upstream multi-family properties

Because the upstream area has a very small ROW area, when compared to the size of multi-family properties, a ROW BMP option was not considered for this study area. Instead, a redevelopment scenario was considered for the Charlton Apartments and Hillside Terrace properties. This alternative assumes that 1% AEP storm detention would be provided for these two properties, which would align with new development requirements. For the total area of approximately 8 acres as shown in **Figure 4-36**, a storage volume of 0.44 MG would be required.

Figure 4-36 – Conceptual Layout for Upstream Detention Alternative for Glendale/Charlton

The impacts on flows in the downstream stormwater system would be dramatic for this alternative. As shown in **Figure 4-37** below, the detention storage reduces peak flows from 45 cubic feet per second (cfs) to 15 cfs at Glendale Drive. This decrease in peak flows would eliminate street flooding for the study area for the 10% AEP storm.

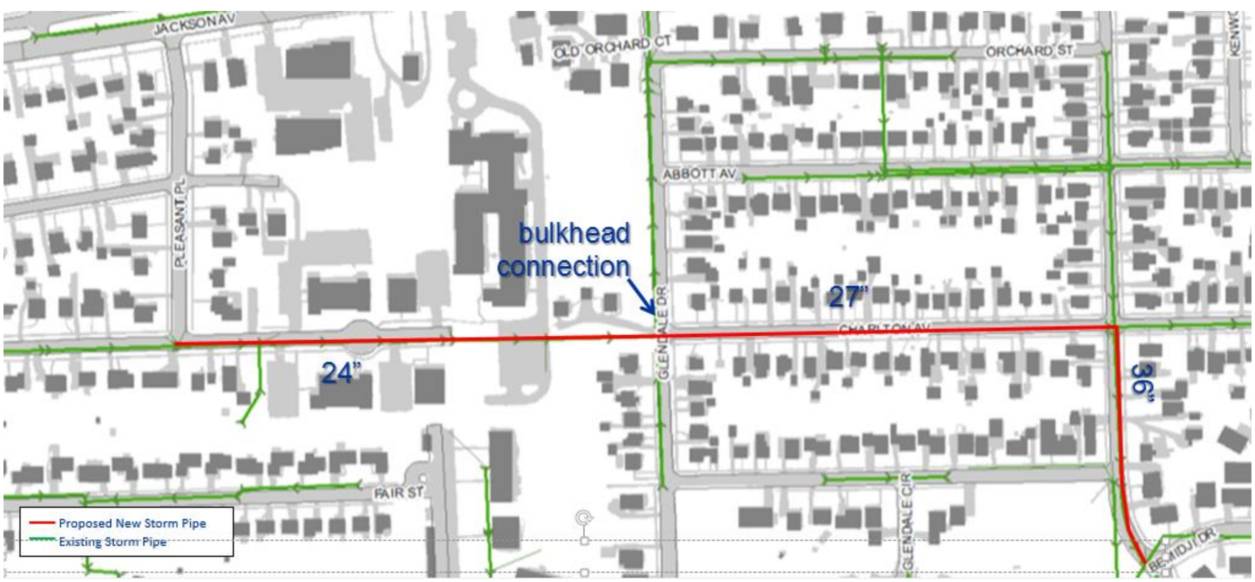
Figure 4-37 – Flow Hydrograph for Upstream Detention Alternative for Glendale/Charlton



Alternative 2 – Conveyance Improvement

This alternative considered an increase in system conveyance capacity by upsizing the existing storm sewer from Pleasant Place to Glendale Drive, bulkheading the current connection to the Glendale Drive storm sewer, and constructing a new storm pipe along Charlton to Virginia Avenue. Pipe upsizing would also be needed along Virginia to Bemidji Drive. This conceptual layout is shown in **Figure 4-38**.

Figure 4-38 – Conceptual Layout for Conveyance Alternative for Glendale/Charlton



The conveyance improvement would generally be re-routing overland flow into a storm pipe so there is no significant change in peak flow in the Murray-Washington Drain.

Recommendation

Either alternative would be feasible and effective at improving the stormwater system performance for the Glendale/Charlton study area. The upstream detention storage would be consistent with the City's sustainability goals and the cost would be the responsibility of the property owners if the improvements can be required as part of property redevelopment. However, to allow comparison with other alternatives and study areas, the overall project cost is shown in **Table 4-7**.

Table 4-7 – Recommended Glendale/Charlton Solution

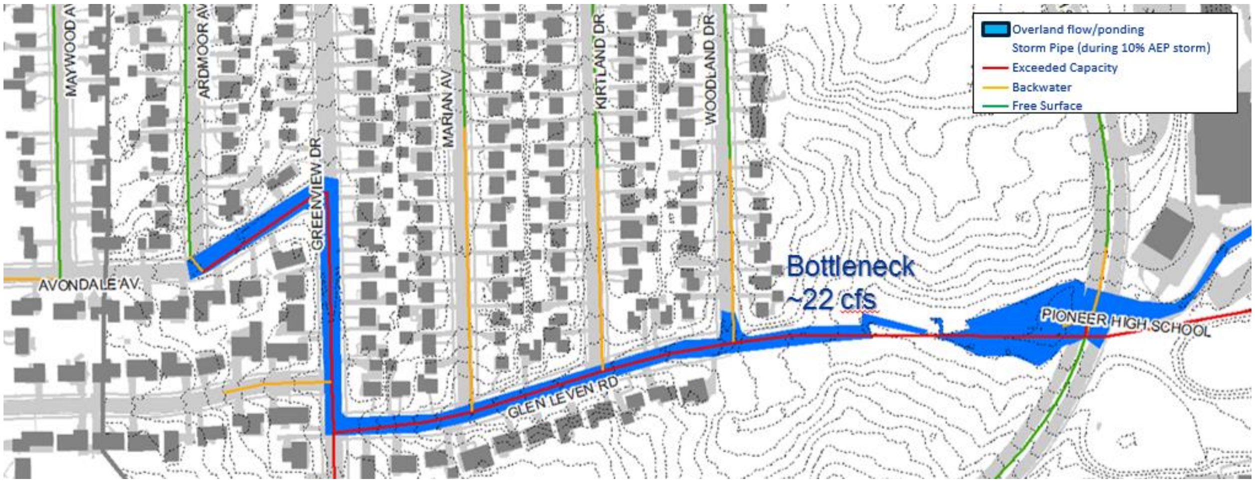
Alternative	Probable Cost
Upstream Detention Storage	\$1m - \$1.2m
Conveyance Improvement	\$0.6m - \$0.7m

Evaluation Matrix Criteria	
System Influence/Capacity	Reduces street flooding only
Water Quality	65% decrease in peak flow
Funding	\$1.2m capital cost; \$6k annual O&M cost increase
Level of Service	LOS improves from 50% AEP storm to 10% AEP storm
Other Criteria	Improved vehicle access; May be addressed by redevelopment requirements

ix. Glen Leven

Existing conditions modeling for the Glen Leven area predicts storm pipe capacity issues for the 50% AEP storm and greater. Surface flooding is predicted for the 10% AEP storm, although the flows are generally confined to the streets and Pioneer Woods as shown in **Figure 4-39**.

Figure 4-39 – Existing conditions results for Glen Leven



A conveyance improvement with surface storage in Pioneer Woods was considered for this area but further consideration is needed to better understand why local observations do not match with the model predictions. It has been noted that sanitary sewer modeling for this area has found more flows than expected so the hydrology for this area, including runoff and inflow/infiltration mechanisms, needs to be better understood before any stormwater improvements are recommended.

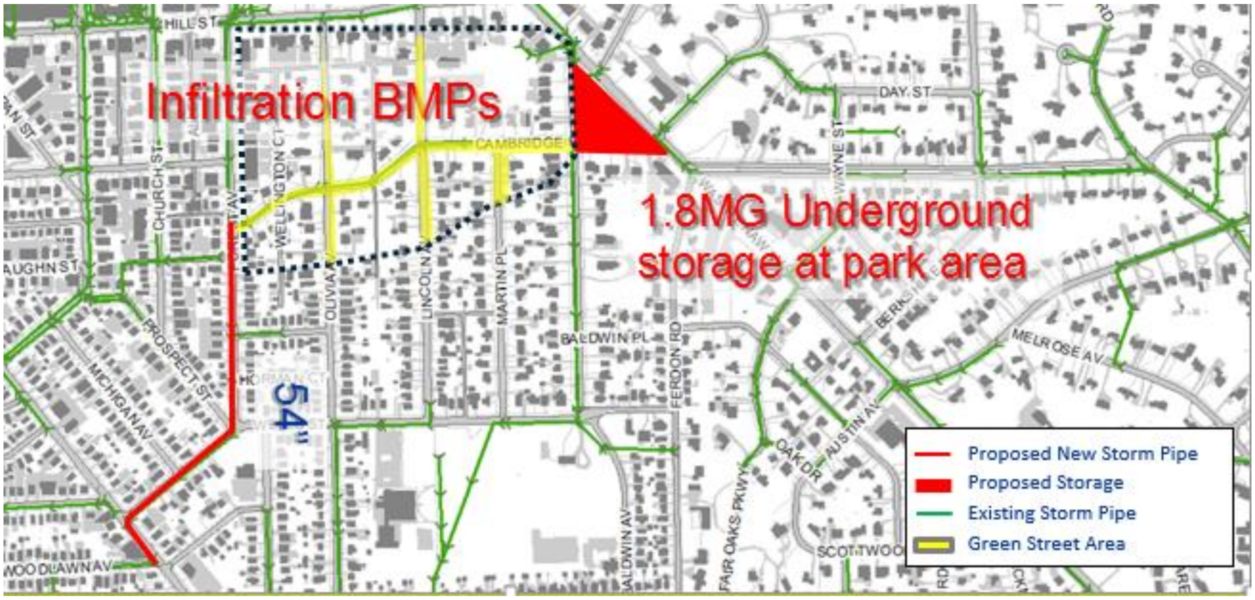
x. Church Street / Cambridge Road

This study area was identified from the existing conditions modeling because the pipe capacity is predicted to be reached during the 50% AEP storm. Street flooding and overland flow is predicted for the 10% AEP storm along Baldwin Avenue, Cambridge Road, S. Forest Avenue, and Church Street, as shown in **Figure 4-40**.

Figure 4-40 – Existing conditions results for Church Street / Cambridge Road

As with the Glen Leven area previously, the street flooding predicted by the model has not been validated by observations. Alternatives are available for both conveyance and storage/BMP improvements (see **Figure 4-41** below) but they would require a significant capital cost and would be addressing a problem that has not been shown to significantly impact properties. No stormwater improvements are recommended for this study area.

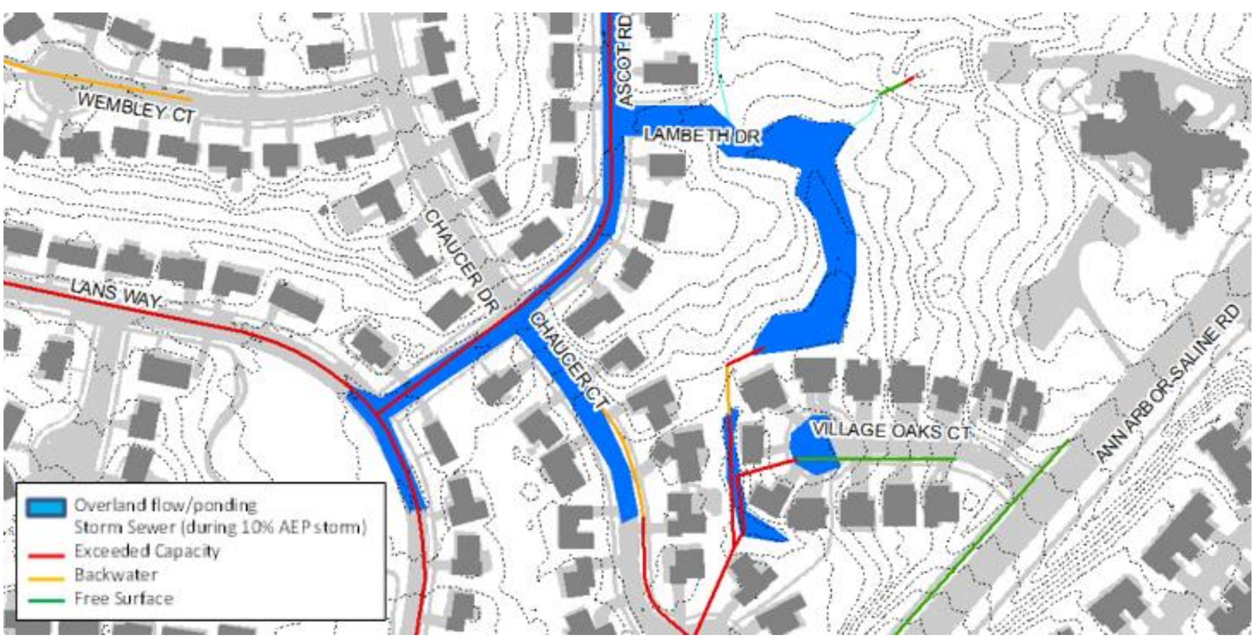
Figure 4-41 – Conceptual Stormwater Improvements Layout for Church Street / Cambridge Road



xi. Village Oaks / Chaucer Court

This location was identified from existing conditions modeling because the pipe capacity is reached during the 50% AEP storm. Backyard flooding between Village Oaks Court and Chaucer Court is predicted during the 10% AEP storm, along with street flooding in the cul-de-sac of Village Oaks Court, as shown in **Figure 4-42**.

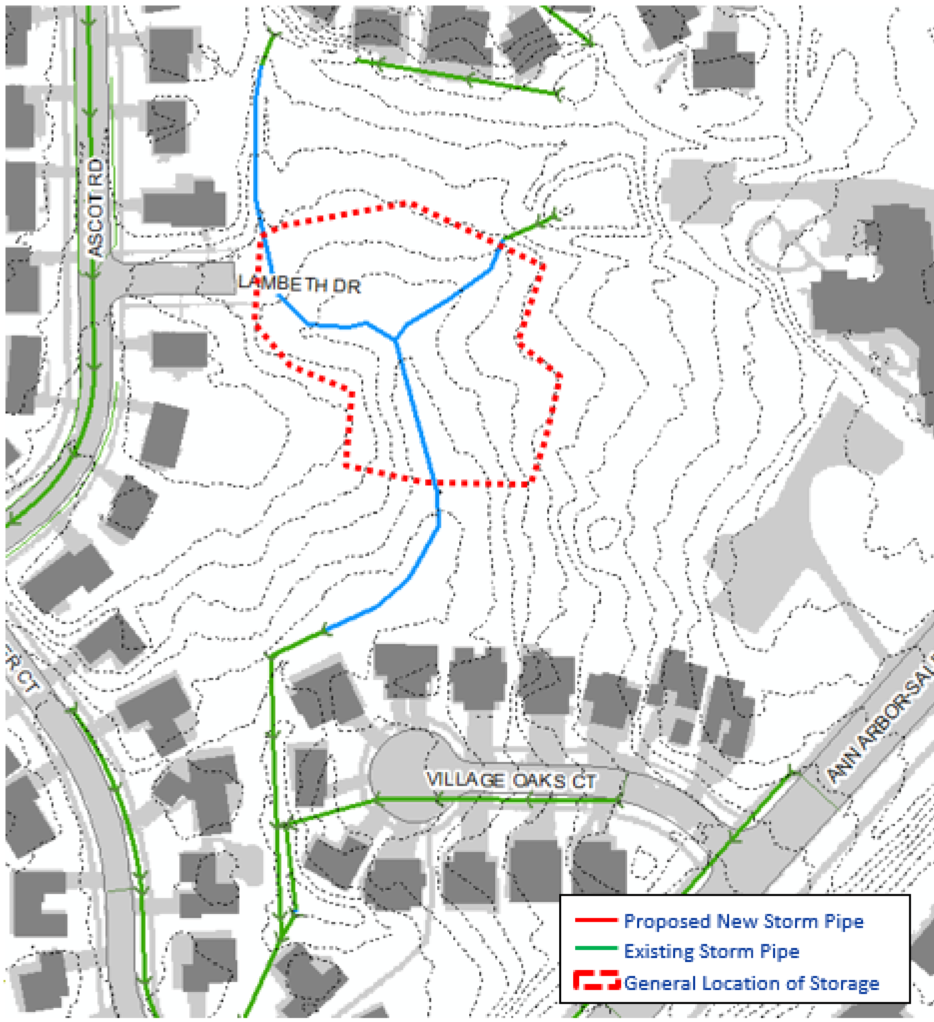
Figure 4-42 – Existing conditions results for Village Oaks / Chaucer Court



A detailed study of this area was performed in 2013 and a regional detention basin was recommended for the area north of Village Oaks Court. The alternatives analysis for this area consisted of verifying the performance of the proposed basin using the current version of the stormwater mode, as shown in **Figure 4-43.**

Under the proposed alternative, the peak flow coming from the basin would be reduced from 40 cfs to 1 cfs. The flows from Village Oaks Court would not be affected but the backyard flooding would be reduced in frequency from the 10% AEP storm to the 2% AEP storm.

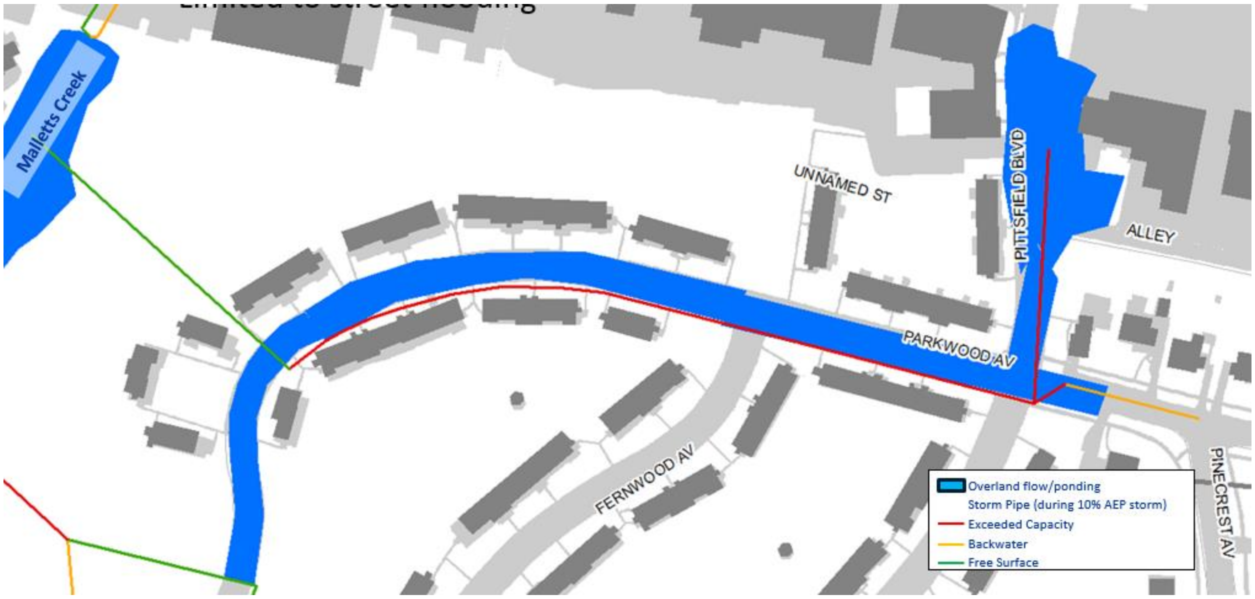
Figure 4-43 – Conceptual Layout for Detention Alternative at Village Oaks/Chaucer Court



xii. Parkwood/Pittsfield Village

This study area was identified during the public meetings in Phase I of the project. Residents reported street flooding during large storms and overland flow into the open space between buildings between Fernwood and Parkwood. The existing conditions modeling showed a pipe along Parkwood with a capacity of less than 3 cfs, which is not sufficient to convey the 100% AEP storm. The model predicts that flooding would be confined to the street area as shown in **Figure 4-44**, but other factors such as inlet blockages could lead to more extensive surface flooding.

Figure 4-44 – Existing conditions results for Parkwood / Pittsfield



Alternative 1 – Conveyance and Storage

Because of the relatively small tributary area, and the capacity issue with the existing pipe, some conveyance improvements are recommended along Pittsfield and Parkwood. Alternative 1 includes the recommended pipe upsizing as shown in **Figure 4-45**, but it also includes a new connection to the surface depression area off of Parkwood Avenue to store excess runoff so flows are not increased to Malletts Creek. The predicted outflow hydrograph is shown in **Figure 4-46**.

Figure 4-45 – Conceptual Layout for Pittsfield/Parkwood Storage/Conveyance

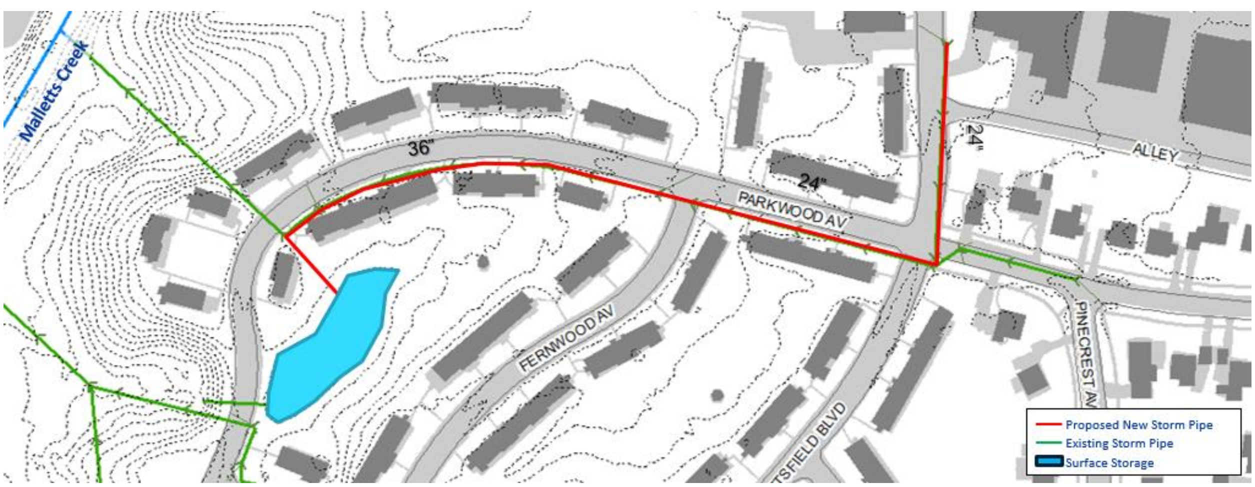
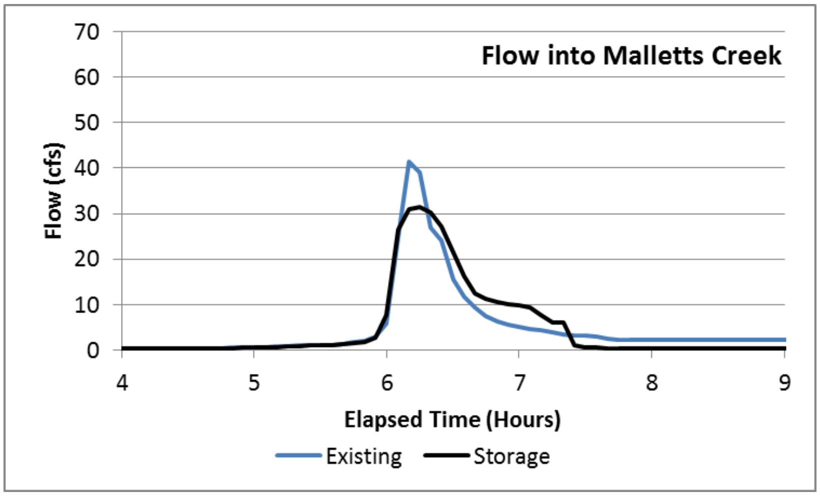


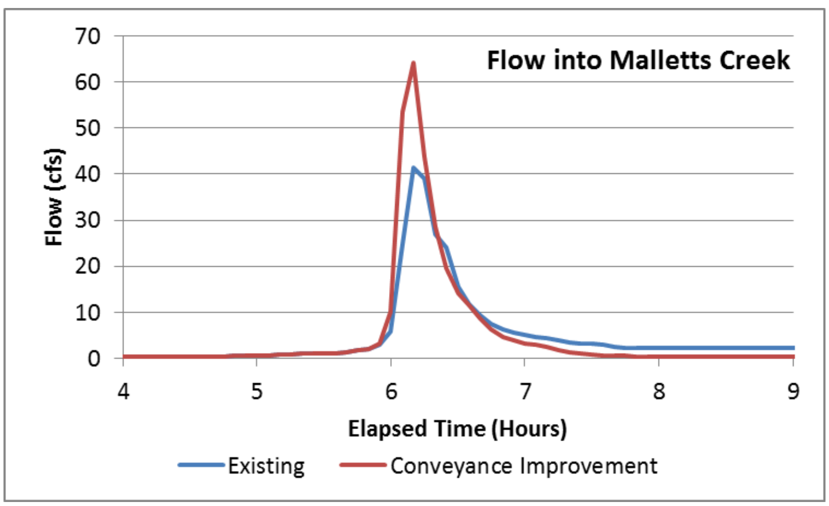
Figure 4-46 – Flow Hydrograph for Conveyance/Storage Alternative at Pittsfield/Parkwood



Alternative 2 – Conveyance Improvement

Alternative 2 would include the pipe upsizing only. This would result in a 50% increase in peak flows to Malletts Creek, although there would be no change in the predicted water surface elevation. This result is shown in **Figure 4-47**.

Figure 4-47 – Flow Hydrograph for Conveyance Alternative at Pittsfield/Parkwood



Recommendation

The property on Washtenaw Avenue between Pittsfield Blvd. and Yost Blvd. contributes approximately 25% of the runoff to this study area so redevelopment of that property with stormwater controls should be a priority. Even with detention at that site, however, pipe upsizing would be necessary along Pittsfield and Parkwood to convey the 10% AEP storm. Either of the proposed alternatives would be effective at addressing the stormwater system performance issues and selection should be made based on the willingness of Pittsfield Village property management to allow surface storage. The surface storage solution in the lawn areas between units could be adapted to other portions of the property to address other stormwater issues. This is presented in **Table 4-8**.

Table 4-8 – Recommended Parkwood/Pittsfield Village Solution

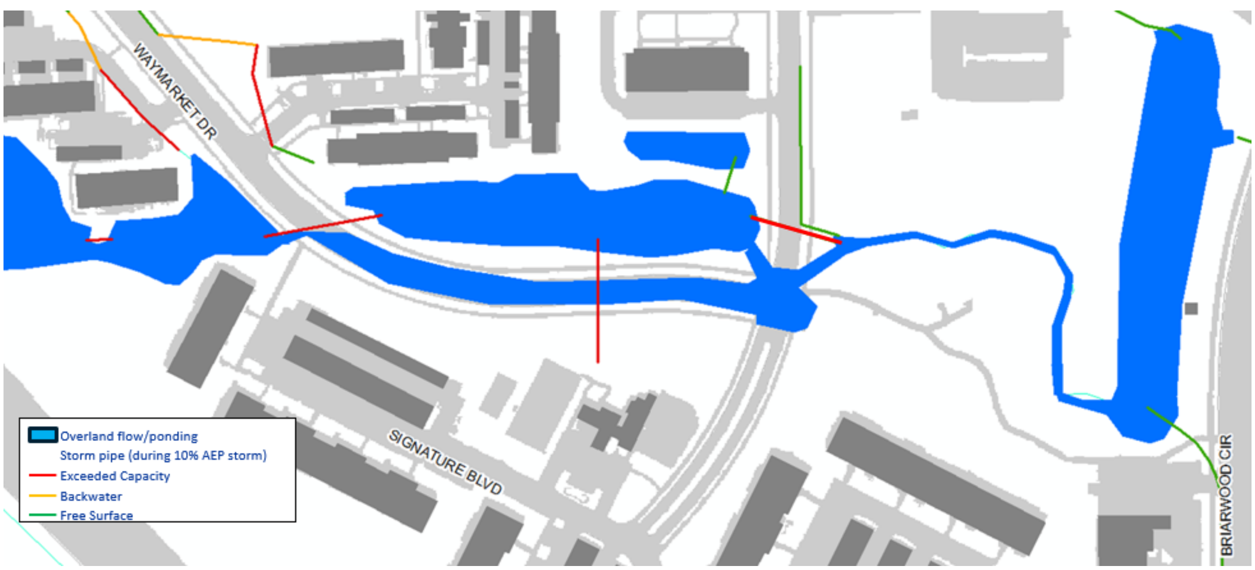
Alternative	Probable Cost
Conveyance Improvement + Surface Storage	\$0.4m - \$0.5m
Conveyance Improvement	\$0.4m - \$0.5m

Evaluation Matrix Criteria	
System Influence/Capacity	Primarily street flooding
Water Quality	25% reduction in peak flow
Funding	\$0.5m capital cost; \$1k annual O&M cost increase
Level of Service	LOS improves from 100% AEP storm to 10% AEP storm
Other Criteria	Potential partnership with Village Co-op; scalable solution

xiii. Signature Drive

This study location was identified from the existing conditions model results screening. The culvert under Signature Drive just north of Waymarket is undersized, causing surface ponding in the intersection and in the detention area to the north of Waymarket Drive during the 10% AEP storm. The surface flooding also affects Waymarket Drive to the west of Signature Drive and other connecting detention basins at nearby properties, as shown in **Figure 4-48**.

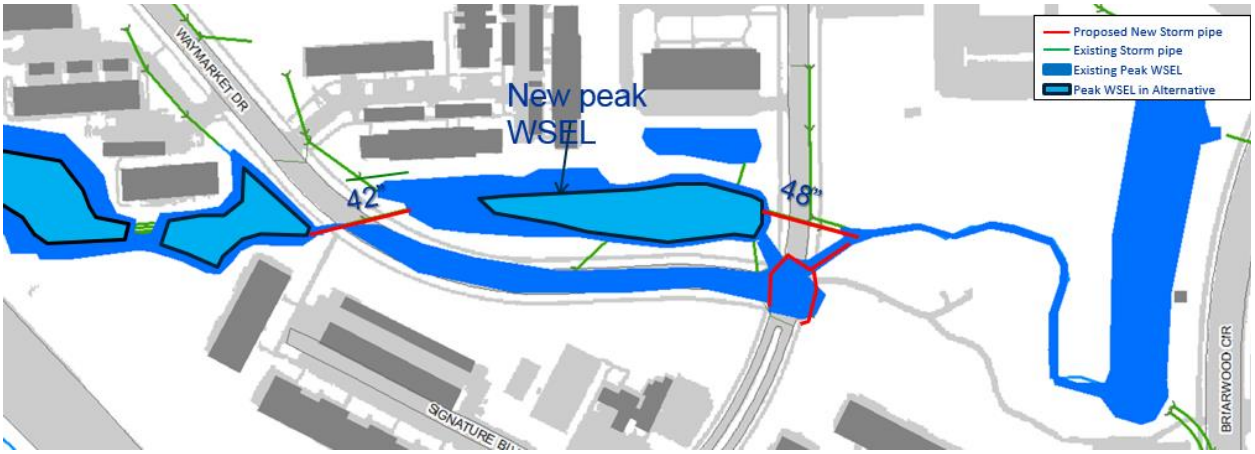
Figure 4-48 – Existing conditions results for Signature Drive



Recommendation

Because the existing detention basins are functioning as designed and the flow restrictions are limited to short pipe sections, a conveyance improvement alternative was the only approach considered for this location. As shown below in **Figure 4-49**, the culverts under Signature Drive and Waymarket Drive should be upsized and new catch basins should be installed at the intersection to convey flows downstream.

Figure 4-49 – Signature Drive Alternative Configuration



The increased flows will be handled by the existing detention pond at Briarwood Circle with a resulting increase in water surface elevation (WSEL) of only 0.1 feet. The street flooding will be eliminated along Signature and Waymarket and the peak WSEL in the existing detention basins will be reduced.

Table 4-9 – Recommended Signature and Waymarket Solution

Alternative	Probable Cost
Conveyance Improvement	\$127K - \$153K

Evaluation Matrix Criteria	
System Influence/Capacity	Reduces intersection flooding
Water Quality	30% increase in peak flow; no change in volume
Funding	\$0.2m capital cost; no increase in annual O&M cost
Level of Service	LOS improves from 20% AEP storm to 10% AEP storm
Other Criteria	

xiv. South Industrial/Packard Road Area

This neighborhood was identified during the existing conditions model results screening, showing up as one of the few areas of the City where the sewer system is at capacity during the 20% AEP, 1-hour storm. While overland flow is predicted starting with the 50% storm in some locations, and during the 10% storm for almost the entire area, these flows are generally confined to the streets. There were not any notable reports of flooding from the residents of this area during the public engagement process, although some City staff noted the area of Harpst/Rosewood/Tremel as a known street flooding location, as shown in **Figure 4-50**.

Figure 4-50 – Existing conditions results for South Industrial Area

Alternative 1 – Green Streets Implementation

Although the soils in this area have low infiltration potential due to clay soil and high groundwater, there is a large upstream tributary area with residential ROW areas that would be suitable for localized storage BMPs. These areas are shown in **Figure 4-51**.

Figure 4-51 – Conceptual Layout for Green Streets Alternative for S. Industrial Area

The reduced runoff resulting from these improvements would minimize street flooding and overland flow for the 20% AEP storm. The pipe capacity would still be exceeded in the 10% AEP storm in most locations. Model results for the Green Streets alternative under the 20% AEP storm are shown in **Figure 4-52**.

Figure 4-52 – Green Streets Alternative Pipe Capacity Results for S. Industrial Area

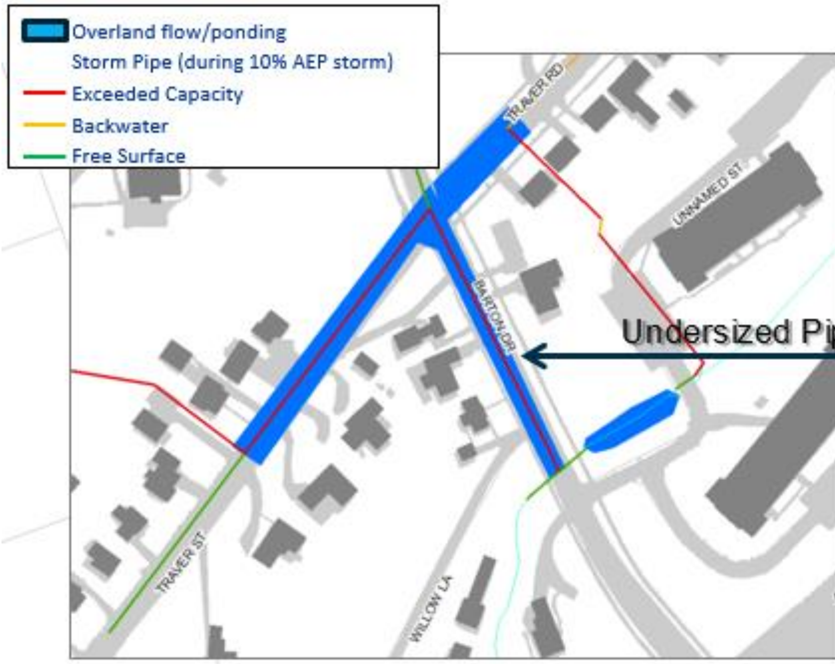
Recommendation

Because of the minimal impacts and the extensive scope of work, this area is not recommended as a priority for stormwater improvements. As conditions allow for Green Streets implementation as part of other neighborhood improvements, however, these efforts should be made to help reduce runoff flows and minimize the frequency of flooding in downstream areas.

xv. Traver/Barton

This study location has one pipe segment along Barton Drive south of Traver Road that was identified as undersized during existing conditions modeling. Currently, the pipe capacity is reached during the 100% AEP storm, and the collection system can be overwhelmed by overland flow coming downhill along Traver. The curbs along Barton and the current placement of catch basins also prevent street flow from leaving the roadway in some locations as shown in **Figure 4-53**.

Figure 4-53 – Existing conditions results for Traver/Barton



Recommendation

Due to surface grades, and a low potential for runoff infiltration, a conveyance alternative is recommended for this location. The existing pipes along Traver and Barton should be substantially upsized from 12” diameter to 30” and 36”, respectively, as shown in **Figure 4-54**. In addition, curb cuts at the Traver Creek crossing should be built to allow for overland drainage into Traver Creek during intense rainfall events. These improvements would have a negligible increase in WSEL and peak flows in Traver Creek. This recommendation is shown in **Table 4-10**.

Figure 4-54 – Conceptual Layout for Conveyance Improvement Alternative for Traver/Barton

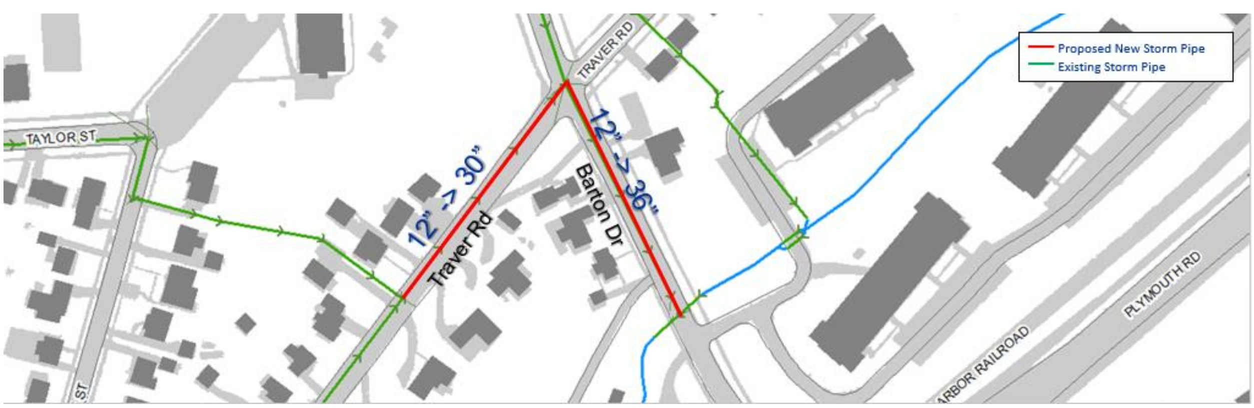


Table 4-10 – Recommended Traver/Barton Solution

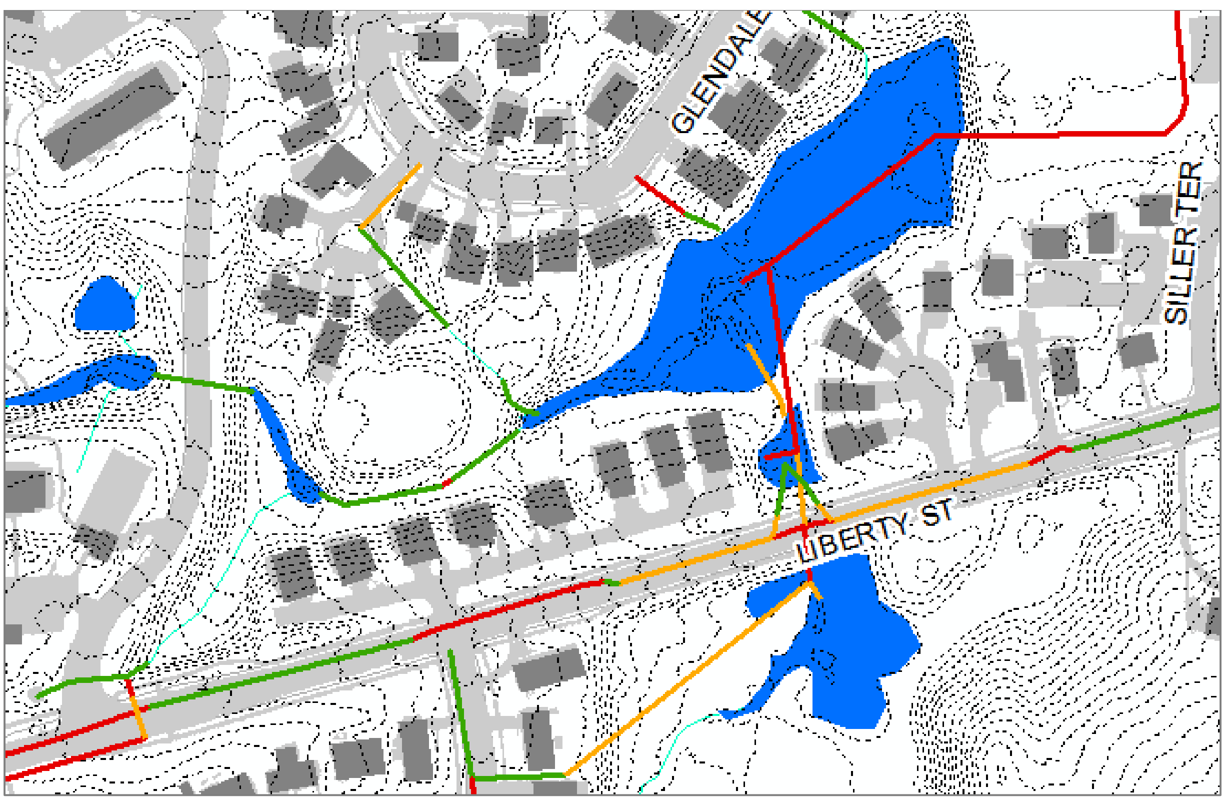
Alternative	Probable Cost
Conveyance Improvement	$200K - $250K

Evaluation Matrix Criteria	
System Influence/Capacity	Reduces roadway flooding
Water Quality	Negligible increase in peak flow
Funding	$0.2m capital cost; no increase in annual O&M cost
Level of Service	LOS improves from 20% AEP storm to 10% AEP storm
Other Criteria	

xvi. Glendale Circle at Virginia Park

Noted in section 4-A, this site was not originally included as part of the preliminary screening since the flooding area is part of an open channel drainage that offers natural detention storage, and structures have not historically been affected. Also, this site is only 3,500 feet upstream of the Mulholland site (**Section 4-C.6**). However, the 54" storm pipe that passes beneath Virginia Park did not have sufficient capacity to handle peak flow during the 20% AEP storm, and some property owners along Glendale Circle have noted that flooding encroaches onto their properties, as shown in the **Figure 4-55**.

Figure 4-55 – Ponding at Wooded Area behind Glendale Circle



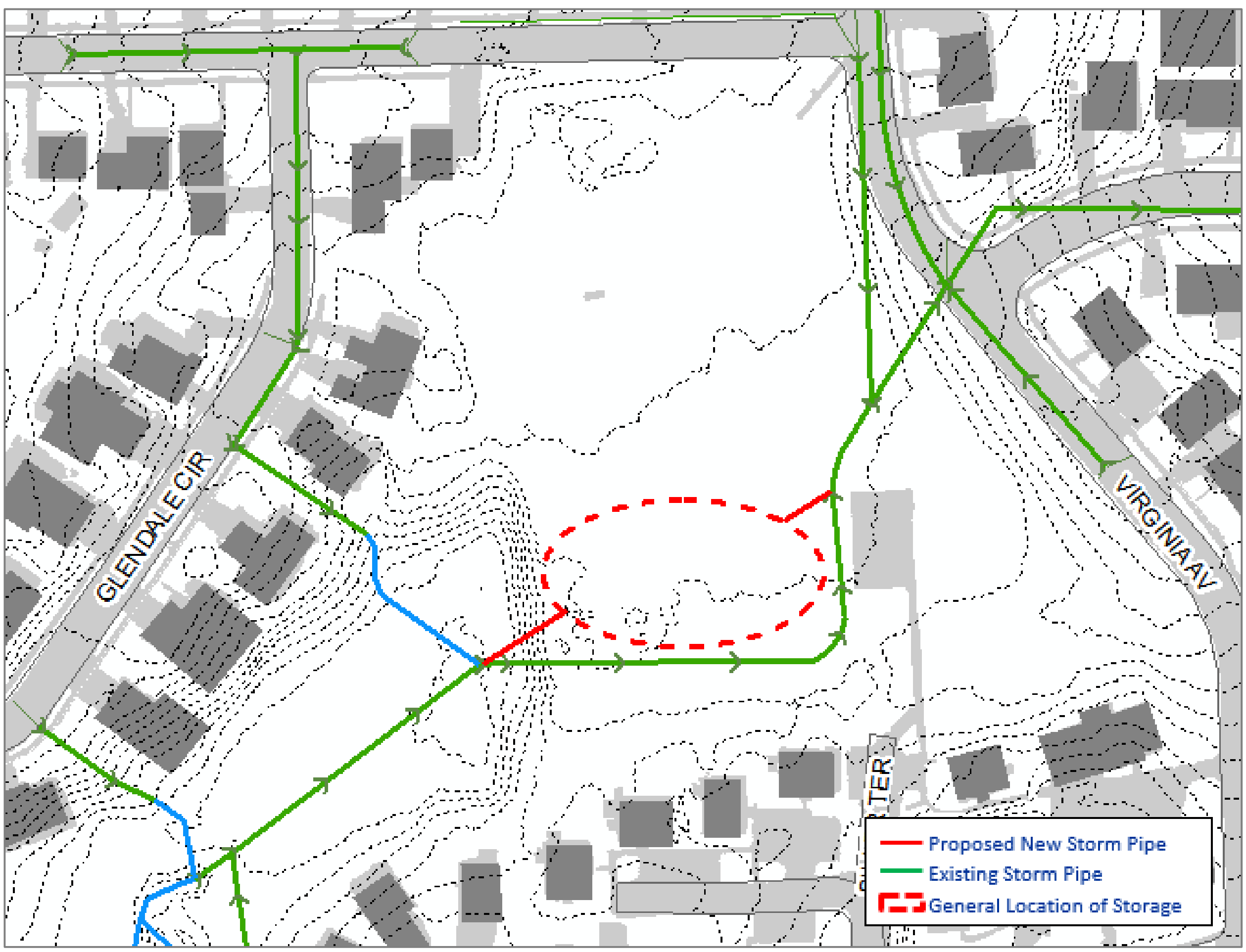
Two storage alternatives were considered for this site. A conveyance improvement alternative is prohibitive because of existing flooding issues at Mulholland Drive downstream. Similar to the analysis for that site, implementation of the Green Streets policy alone would not eliminate flooding issues for the 10% AEP storm. Ponding at the wooded area behind Glendale Circle would drop by 3 inches at most. The current peak flood depth in existing conditions for the 10% storm is predicted to be 4'.

The impacts of other stormwater management activities in tandem with the Green Streets policy are evaluated in Alternative 3.

Alternative 1 – Deep Underground Storage at Virginia Park

This alternative includes moving existing surface storage volume to an underground storage tank at Virginia Park. Due to the significant difference in elevation between the wooded area and Virginia Park, the tank would have to be installed nearly 30 feet below grade. The size of the tank would be 2.7 MG to reduce ponding at the wooded area to below 1 foot in depth. The storage would include a pipe connecting to inlet (88-64592) at the wooded area and a restricted outlet control structure connecting to the adjacent storm sewer. Runoff would be diverted to the storage once the 54" storm sewer downstream is surcharged. **Figure 4-56** shows the general location and configuration of the underground storage tank.

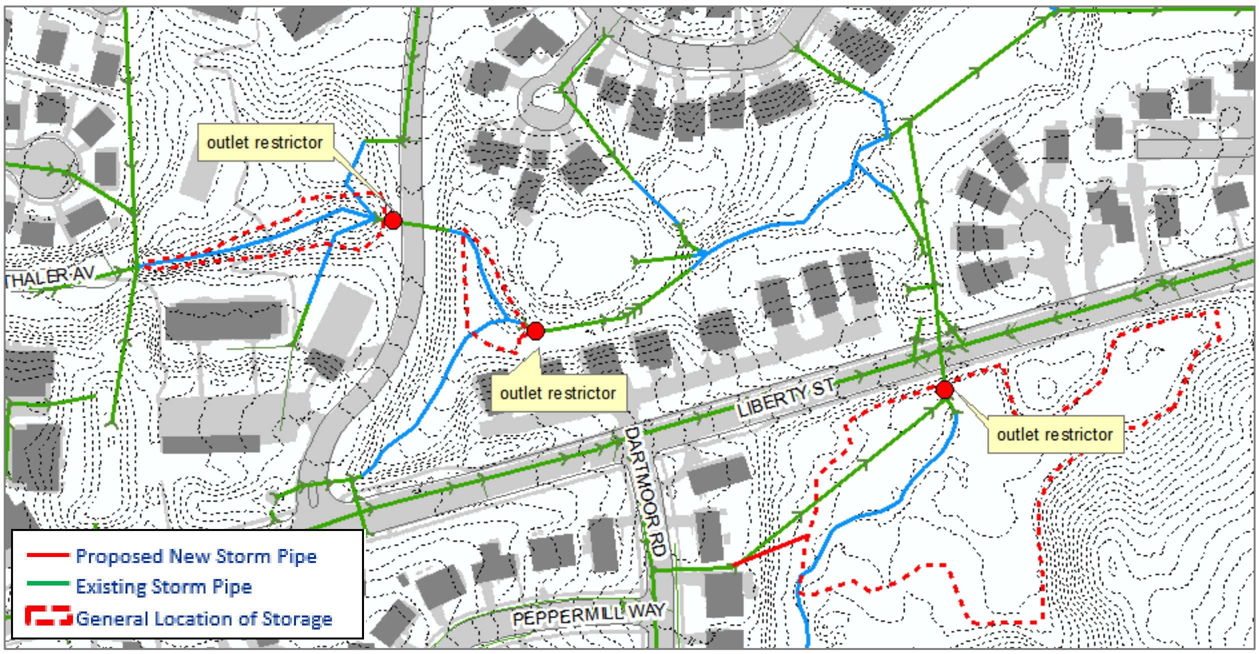
Figure 4-56 – Location of Underground Storage at Virginia Park



Alternative 2 – Surface Storage Upstream

This alternative aims at reducing peak flows entering the Glendale Circle backyard area by detaining additional volumes in open channel storage at Westwood Apartments to the west and in localized depression storage in Eberwhite Woods. Outlet restrictors would be installed at these locations to reduce the overall peak flow to below 270 cfs. **Figure 4-57** shows the locations of the additional upstream storage areas and outlet restriction devices. While this alternative would reduce potential flooding risk for properties on Glendale Circle, it would effectively move surface flooding to other areas. Eberwhite Woods is a sensitive nature area and increasing the frequency and extent of surface flooding could be problematic.

Figure 4-57 – Location of Upstream Surface Storage for Glendale Circle / Virginia Park



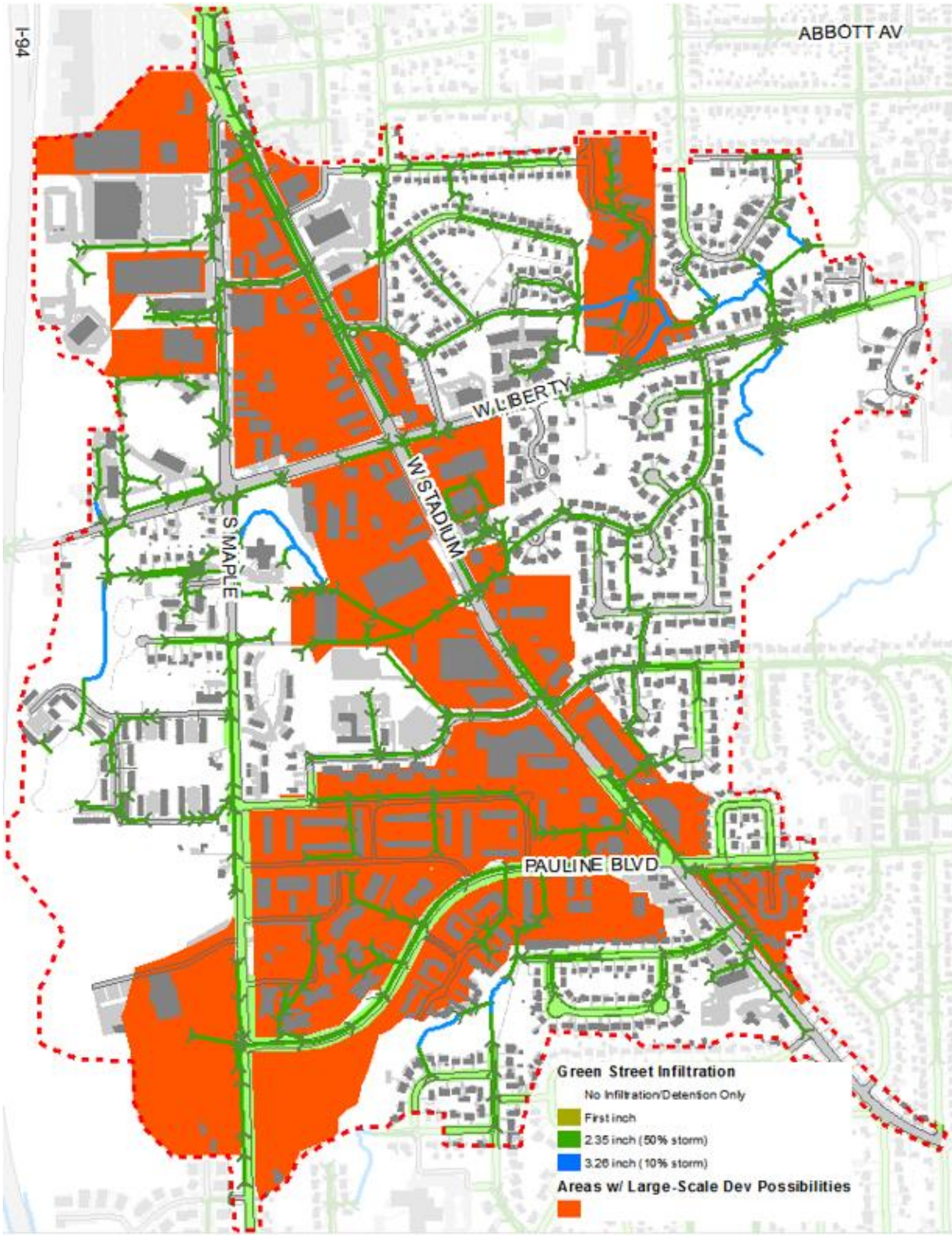
Alternative 3 – Stormwater Management

Ponding at the wooded area could be reduced to less than 6 inches in the 10% AEP storm if the following stormwater management activities were implemented altogether in upstream areas.

- 1% storm on-site detention for all redevelopment of commercial properties on W Stadium Blvd and S Maple Road
- Storage of 1-inch runoff from impervious surface of residential properties
- Green Streets with on-site infiltration for City ROW areas upstream

The most effective of these activities would be the on-site detention for commercial and multi-family residential properties. As shown in **Figure 4-58** below, the W. Stadium and S. Maple/Pauline areas have some large properties that were built without stormwater controls.

Figure 4-58 – Commercial and Multi-Family Residential Parcels with Redevelopment Potential



Recommendation

Each of the storage alternatives would effectively be moving the volume that is currently in the Glendale Circle backyard area to other locations where the storage may have reduced impacts on property owners. Since these other impacts have not been evaluated in detail, a long term stormwater management strategy is the recommended approach to incrementally reduce flooding at this location. These improvements would spread the cost impacts out over time and would benefit both this location and the Allen Creek watershed overall. Where a portion of the surface storage features in alternative 2 are shown to be feasible, these could be implemented to provide additional surface flooding mitigation. The recommended solution is shown in **Table 4-11**.

The stormwater system improvement alternatives presented for this location assumed ponding in the Glendale Circle backyard area would be reduced to less than 1 foot. Further studies should determine the acceptable level of ponding at the backyard to utilize the already-available natural surface storage. The proposed alternatives could all be scaled back accordingly.

Table 4-11 – Recommended Glendale Circle at Virginia Park Solution

Alternative	Probable Cost
Underground Storage	\$10 - \$11m
Surface Storage	\$1.7 - \$1.8m
Long-term Stormwater Management	\$5.1 - \$5.8m + private funding

Evaluation Matrix Criteria	
System Influence/Capacity	Reduces surface flooding that impacts private properties
Water Quality	20% reduction in peak flow
Funding	\$6M capital cost for ROW areas; Additional cost for redevelopment and residential rain gardens
Level of Service	Improves from 20% AEP to 10% AEP storm
Other Criteria	Stormwater management meets sustainability goals; partnership opportunities with private property owners

xvii. Westgate and Maple Village Redevelopment

During review of existing conditions model results, it was suggested that the impacts of detention for properties with large areas of impervious surface should be considered. In the Allen Creek watershed, the Westgate and Maple Village shopping centers were built prior to stormwater detention requirements, and as a result have large roof areas and parking lots that discharge to the stormwater system without any runoff controls. In total, the impervious area of these two parcels is greater than 50 acres in size.

The existing conditions model results for the stormwater network in the Westgate and Maple Village shopping centers are shown in **Figure 4-59**.

Figure 4-59 – Existing conditions results for Westgate/Maple Village

For this evaluation, the model was adjusted to include 1% AEP storm detention for these parcels. The northern portion of Westgate (which drains to the north) would require a detention volume of 0.91 MG. Maple Village would require a detention volume of 2.82 MG.

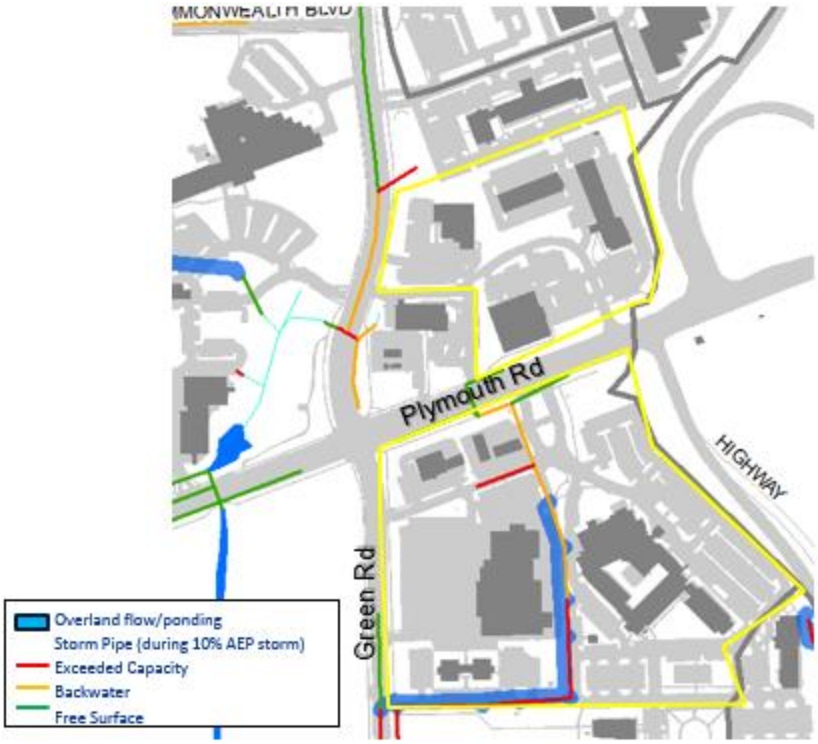
Under the 10% AEP storm, most of the impacts of the redevelopment would be seen immediately downstream of the new detention at Vets Park. Under existing conditions, the 10% storm causes surface flooding through much of the park area, and this flooding would be substantially reduced by the upstream detention, as shown in **Figure 4-60** below. However, because Vets Park is currently providing this storage, the impacts of new detention farther downstream are minimal. Surface flooding depths at depression areas along the West Park-Miller drain would be reduced by less than 0.5 feet and there would be negligible changes in water levels and flow rates at Revena Blvd. and at locations downstream. These impacts are shown in **Figure 4-60**. There would also be negligible impacts on FEMA floodplain elevations under 1% AEP storm simulations.

Figure 4-60 – Model results for Redevelopment Scenario for Westgate/Maple Village

xviii. Plymouth and Green Road Redevelopment

Similar to the evaluation in the previous section, a redevelopment scenario was considered for the commercial properties at Plymouth Road and Green Road. This includes the Red Roof Inn property and the office complexes on the northeast corner, and the Holiday Inn, shopping center, and office complex located on the southeast corner, as outlined in yellow in the figure below. The configuration in this area is shown in **Figure 4-61**.

Figure 4-61 – Existing conditions results for Plymouth and Green Road



1% AEP storm detention for these properties, which total around 27 acres in area, would require a 2 MG detention volume. Because of the nature of the Millers Creek watershed, this area is not generally prone to flooding issues, but the properties themselves would have improved drainage and street flooding would be minimized on Green Road and at the Green Road commuter parking lot. There would be negligible changes in WSEL at and downstream of Baxter Road. The reduction in peak flows would be beneficial in reducing channel erosion issues.

Additional analysis of the impacts of applying new detention requirements during redevelopment is described in Section 5, when it is included with broader stormwater management activities in future condition analysis.

D. Stormwater Improvement Conclusions

The stormwater improvements evaluation generated a list of recommended improvements to address study areas where stormwater system performance is not meeting the current design standards. It has been noted that some of the study locations have not been validated by actual observations, but it is important to recognize that the 10% AEP, 12-hour storm is a large rain event, and that some portions of the City may not have experienced a storm of this size under current development conditions.

A summary of the study areas and the recommended stormwater management alternatives is shown in the following **Table 4-12**.

Table 4-12 – Summary of Recommended Stormwater Management Alternatives

Site	Watershed	Recommendation	Cost Estimate
1. Lower Allen Creek – Main Branch	Allen	BMP-Combination	$80m - $120m*
2. Edgewood/Snyder	Allen	Conveyance-Storage	$4.1m
3. Park Place Apartments	Allen	Conveyance	$1.0m
4. Churchill Downs/Lansdowne	Malletts	Conveyance-Storage	$16m
5. S. University/E. University	Allen	BMP-Storage	$3.6m
6. Mulholland Drive	Allen	Storage	$1.9m
7. Scio Church/S. Seventh	Malletts	BMP-Storage	$2.4m
8. Glendale/Charlton	Allen	Storage	$1.2m
9. Glen Leven	Allen	Further Study	--
10. Church St./Cambridge	Malletts	None	--
11. Village Oaks/Chaucer Ct.	Malletts	Storage	$1.2m
12. Parkwood/Pittsfield Village	Malletts	Storage	$0.5m
13. Signature Drive	Malletts	Conveyance	$0.2m
14. S. Industrial/Packard Rd.	Malletts	None	--
15. Traver/Barton	Traver	Conveyance	$0.2m
16. Glendale Circle / Virginia Park	Allen	BMP-Storage	$5.1m*

**Cost estimates for these sites are based on Green Streets policy implementation only. Other portions of the recommended stormwater management improvements would take place on private property and would not be funded by the City.*

In total, the recommended improvements are projected to cost approximately $34 million in year 2017 dollars. This does not include long term stormwater management improvements which have been recommended for the Lower Allen Creek and for the Glendale Circle/Virginia Park study areas.

Prioritization of the recommended improvements will be considered as part of the City's Capital Improvements Programming process.

5. Stormwater Management Scenarios

A. Citywide Stormwater Management Scenarios

The stormwater model was utilized to evaluate the potential impacts of expanding low-impact development (LID) and green infrastructure (GI) concepts citywide to the stormwater system. LID and GI are decentralized stormwater best management practices (BMPs) that infiltrate and/or detain runoff close to its source. By reducing site runoff and peak flow rates, these features can improve the level of service provided by the existing stormwater system. In this study, the following stormwater strategies were considered:

- *Green Streets:* The City's Green Streets policy includes on-site infiltration standards for public roadway and right-of-way (ROW) construction and reconstruction projects. The policy calls for infiltration of 1 inch (1st flush), 2.35 inches (50% annual chance 24-hour storm) or 3.26 inches (10% annual chance 24-hour storm) of total precipitation volume that falls on the ROW, depending on site soil conditions, slope and proximity to floodplain (**Table 5-1**).

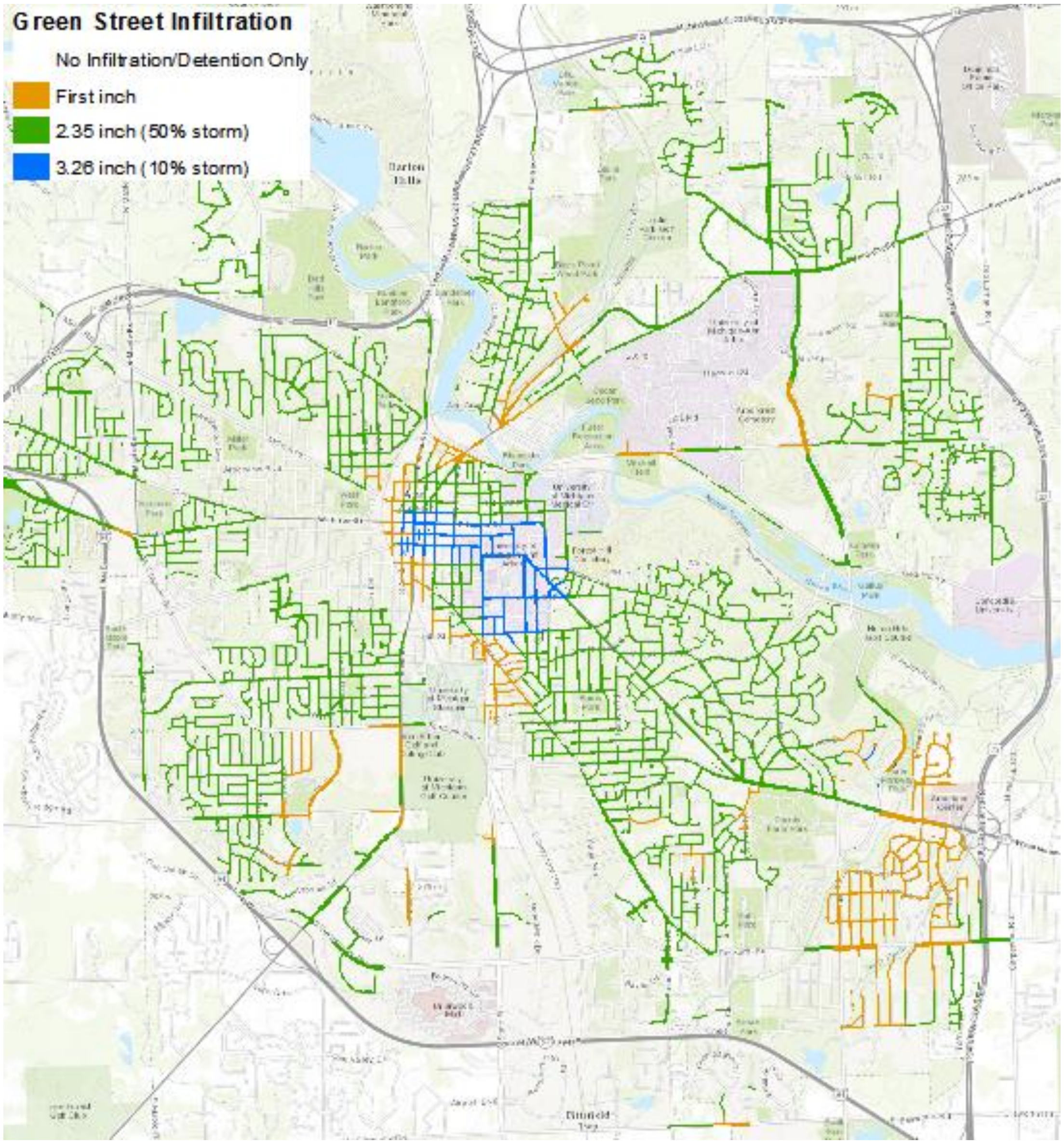
Table 5-1 – Infiltration Standard Excerpted from Green Streets Policy

Site Conditions	Infiltration Standard
• Within the floodplain, or • Slopes > than 20%, or • Soil infiltration rate < 0.6 in/hr	First 1 inch
• Not in the floodplain, and • Slopes < than 20%, and • Soil infiltration rate between 0.6 in/hr – 2.0 in/hr	50% annual chance - 24 hour event (2.35")
• Not in the floodplain, and • Slopes < than 20%, and • Soil infiltration rate >2.0 in/hr	10% annual chance – 24 hour event (3.26")

Notes: Soil Infiltrations Rates are based on A and B soil classifications in the Soil Survey of Washtenaw County, Michigan (1977).
Rainfall frequency estimates are derived from NOAA Atlas 14 Volume 8 (2013).

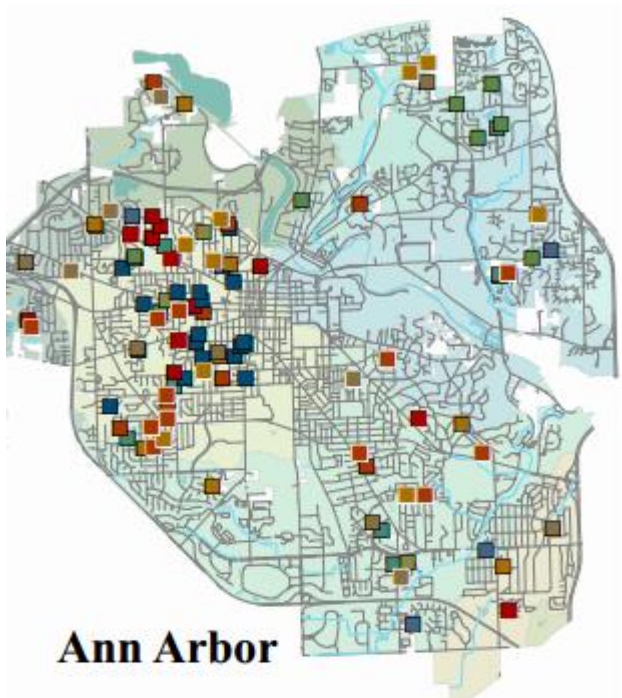
Figure 5-1 shows the applicable infiltration standard with streets color-coded based on soil map information. It is assumed that on-site infiltration is not available in areas with groundwater levels within 5 feet of the ground surface. Streets already reconstructed with Green Street concepts were not included in the mapping and the model analysis of this approach.

Figure 5-1 – Potential Infiltration for Green Street Application



- *Rain Gardens for Single-/Two-Family Homes:* Currently the City requires storage of 1st flush (1 inch) of runoff for new impervious area on an individual single- or two-family parcel if the net increase in impervious area exceeds 200 sf. There are residential stormwater credits available for customers that become RiverSafe Home Partners, install rain barrels, or create a rain garden, cistern, or drywell. Support for rain garden design and construction is available through Washtenaw County's Rain Garden Assistance Program, and rain gardens have already been installed through many areas of the City, as shown in **Figure 5-2**. This scenario assumes that these rain garden initiatives were applied broadly to allow for storage of first flush for all impervious surface areas for all single- and two-family homes citywide. For a typical parcel, this would require a rain garden with a capacity of approximately 1500 gallons. This would add up to 67MG of rain garden storage if applied citywide.

Figure 5-2 – Residential Rain Gardens in the City of Ann Arbor

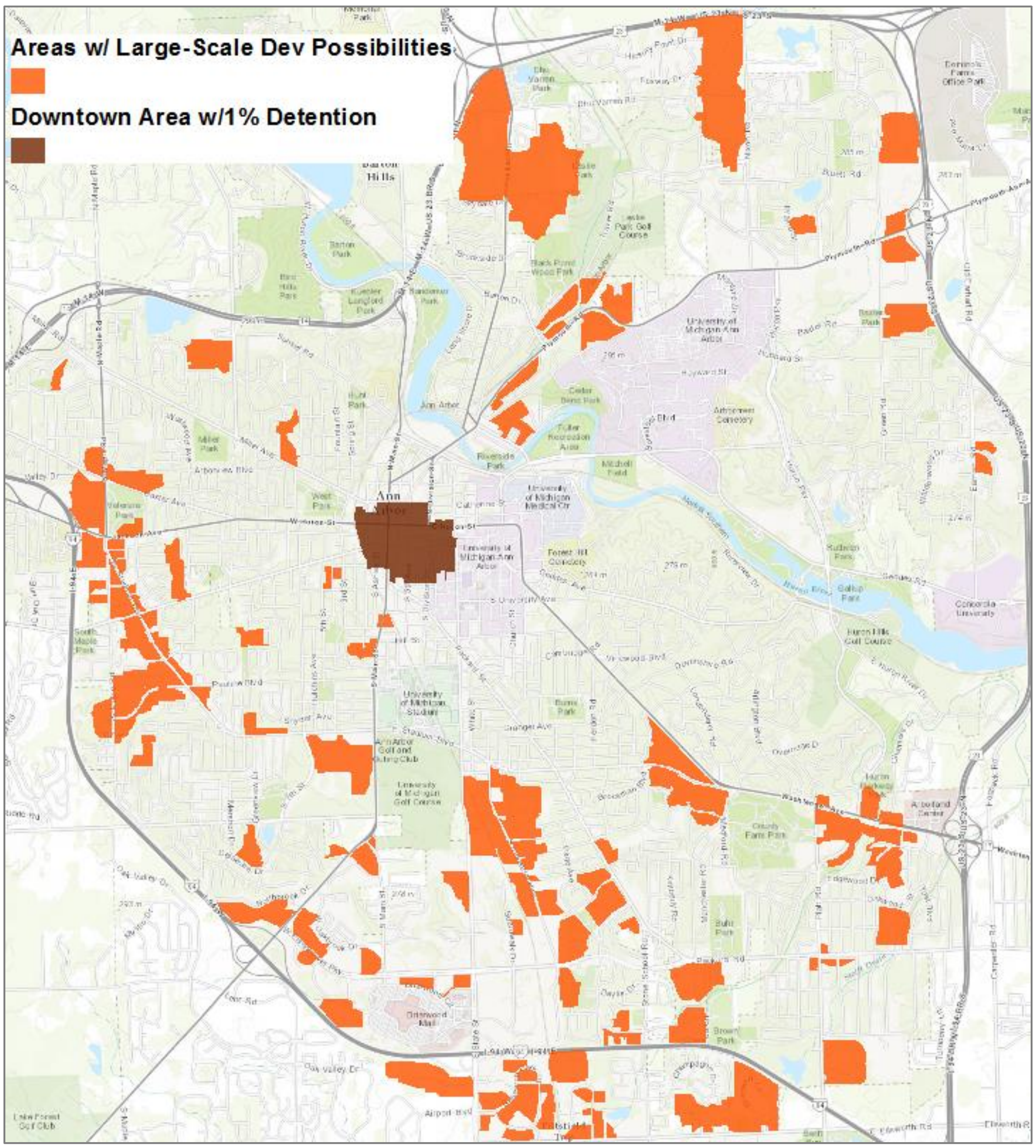


(Source: Washtenaw County Rain Garden Assistance Program, colors indicate different years of rain garden installations)

- *University of Michigan Redevelopment*: This scenario assumes that the University of Michigan's stormwater management strategy would align with new development requirements of the City and Washtenaw County Water Resources Commissioner's Office (WCWRC). This would include infiltrating at least the 1st inch of runoff (1st flush) and detaining runoff from 1% AEP 24-hour storm events for all University properties discharging into County drains or the City's storm sewer system. Most of Central and Athletic Campus areas drain to Allen Creek while the eastern part of North Campus drains to Millers Creek.
- *Downtown Stormwater Management*: On top of Green Streets in the downtown area, this scenario assumes 1% AEP storm detention would be provided for the entire tributary area between Catherine Street to the north, State Street to the east, Jefferson Street to the south and railroad to the west. This strategy is based on recent experience with stormwater management work on South Fourth Avenue, and other soil testing in downtown areas, which indicated that 1% AEP storm detention and infiltration can be achieved. These areas are all tributary to Allen Creek and are shown in brown in **Figure 5-3**.
- *New Development and Redevelopment of Commercial and Multi-Family Parcels*: This stormwater management approach accounts for redevelopment of commercial, multi-family

and public properties larger than 1 acre that are currently without any existing on-site stormwater control. Following the latest WCWRC's stormwater design standards, 1% storm detention would be provided along with storage/infiltration of at least the first flush. **Figure 5-3** maps the locations of these properties in orange. These properties are concentrated around W. Stadium Blvd in the upper tributary area of Allen Creek, S. Industrial, Research Park, and Washtenaw/Huron Parkway areas in Malletts Creek. This also includes undeveloped areas at Dhu Varren/Pontiac Trail and Dhu Varren/Nixon Road that are expected to have future large-scale residential development.

Figure 5-3 – Potential Infiltration and 1% Storm Detention Areas



These different stormwater management strategies were evaluated in the modeling for Lower Allen Creek, which was presented in Section 4C of this report. That analysis compared the relative impacts of the different strategies at different locations along Allen Creek and under different design storm scenarios. The next section presents our analysis of city-wide application of combined strategies under future condition scenarios.

B. Future Conditions

The stormwater management strategies described in the previous section are to be broadly applied and should be considered as long-term stormwater management initiatives. Three (3) future scenarios were included: 2040, 2065 and 2115 to show potential progress over time. It was assumed that all of these strategies would be completed citywide by 2115 (in 100 years), and the levels of completion were determined based roughly on the redevelopment/reconstruction interval for each type of property. The commercial and multi-family percentages were weighted between the downtown properties and those outside of the downtown area. The actual implementation schedule for each scenario would vary depending on feasibility, funding availability, and changes in stormwater management policies. For the purposes of this evaluation, **Table 5-2** shows the assumption of percent completion for each of the future conditions scenarios.

Table 5-2 – Future Scenarios Assumptions for Stormwater Management Strategies

Future Scenario	2040	2065	2115
Green Streets	25%	50%	100%
Residential Rain Gardens	50%	100%	100%
University Redevelopment	50%	100%	100%
Downtown Storage and Infiltration	25%	50%	100%
Commercial and Multi-Family Redevelopment	45%	85%	100%

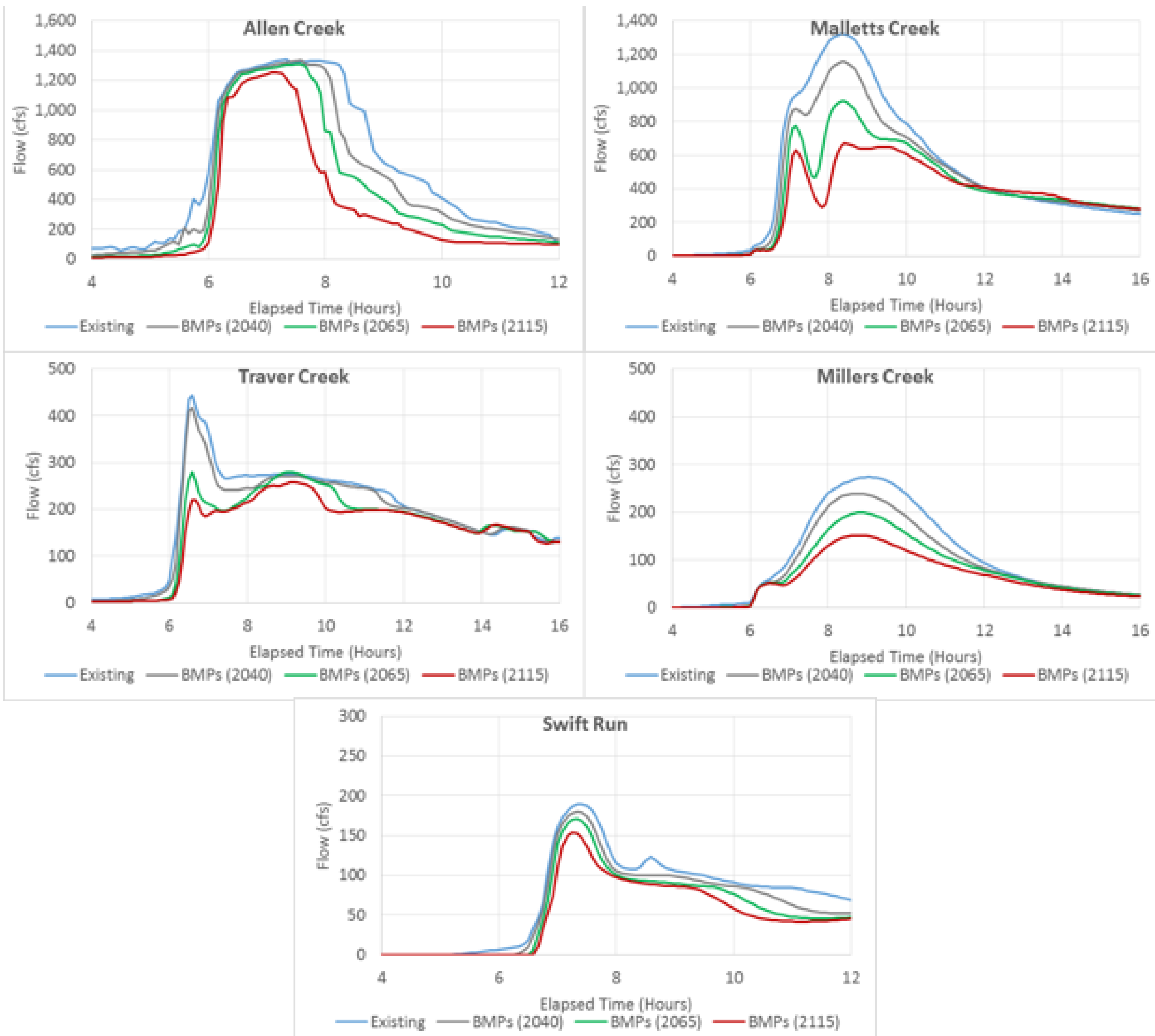
Appendix B contains a series of maps showing the combined impact of all stormwater strategies in 2040, 2065 and 20115 scenarios under current 20% AEP 1-hour and 10% AEP 12-hour design storms for each watershed.

Figures 5-4 to **5-8 below** show hydrographs at the downstream end of each major creekshed under the 10% AEP storm for the different future condition scenarios, and are compared to the current conditions. These strategies could reduce both runoff volume and peak flow and improve the level of service in large portions of the drainage system. For example, peak flow exiting Malletts Creek could be dropped by more than 50% by 2115 because the Mary Beth Doyle Park regional detention basin would no longer be full and overflow during the 10% AEP storm.

However, as shown in the maps in **Appendix B,** all of these strategies combined could not completely eliminate flooding in the 1% AEP floodplain and in other frequent flooding areas. For example, ponding at Edgewood/Snyder would be reduced by almost 3 feet but not eliminated during the 10% AEP storm. BMPs like residential rain gardens, as well as those employed as part the Green Streets policy, are designed to be most effective in more frequent storms that are much smaller in size and less intense than the 10% and 20% AEP design storm events evaluated here.

Results of the stormwater management modeling indicate that the greatest impact of the combined strategies in terms of peak flow reduction would be seen in Malletts Creek, along with Traver Creek and Millers Creek. The peak flow impacts are less pronounced for Allen Creek and for Swift Run. The results for Allen Creek are noticeably unstable at lower flow rates. This instability in the model predictions is due to the location of the observation point at the mouth of Allen Creek, where it is affected by the assumed level of the Huron River.

Figures 5-4 to 5-8 – Flow Hydrographs for Current and Future Conditions (10% AEP, 12-Hr Storm)



The results shown in Figures 5-4 to 5-8 once again indicate that significant improvements in stormwater system performance can be achieved through stormwater management policies and programs. Section 4.C.i provides a comparison of the individual stormwater management strategies, and includes recommendations for future stormwater management policies in the Allen Creek watershed.

6. FEMA Floodplain Comparison

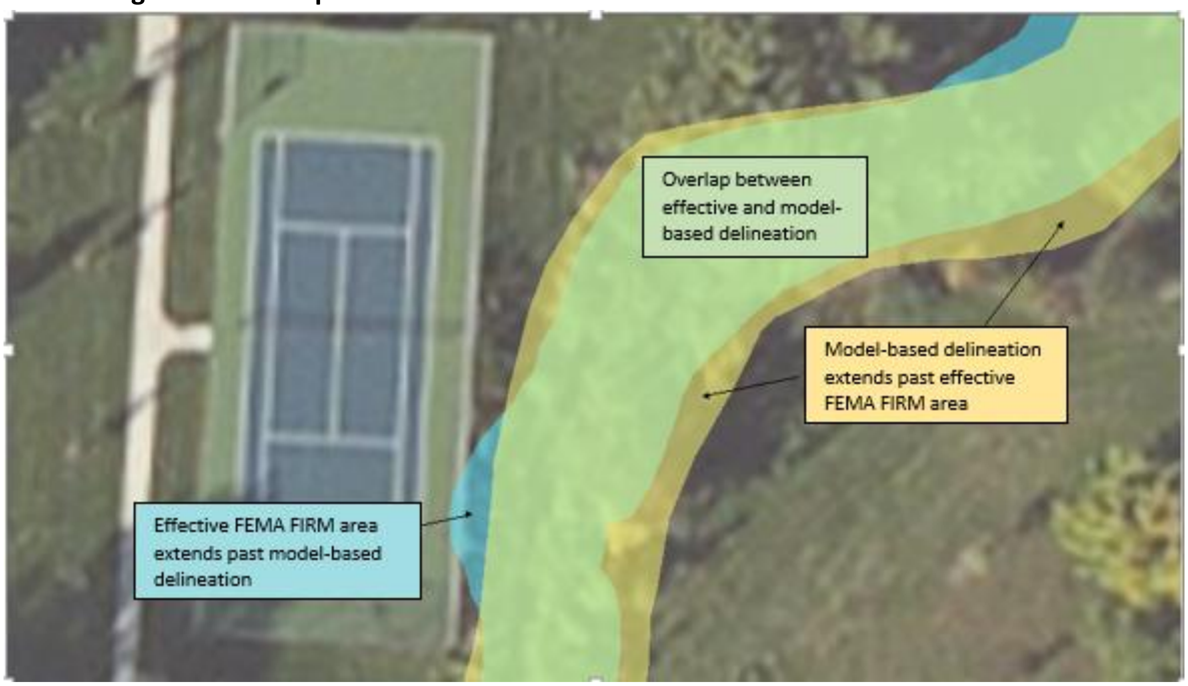
The objective of the FEMA floodplain comparison was to compare the calibrated InfoSWMM model results to existing FEMA Flood Insurance Rate map (FIRM) floodplain maps. The delineation based on the InfoSWMM model data would provide the City with an additional source of flood level data that could be used for future floodplain analysis and management.

The existing FEMA FIRM floodplain areas were delineated as part of a FEMA study in 2013, using HEC-RAS stormwater model results. Separate HEC-RAS models were developed for Allen Creek, Malletts Creek, Traver Creek, Millers Creek, and Swift Run. The calculation methods for each model varied between steady state and non-steady state models, and they each had different approaches to estimate runoff.

As part of this project, the InfoSWMM model was used to simulate a 1% AEP, 24-hour storm, and peak flows and peak water surface elevation (WSEL) data were generated. The water surface elevations from the model were then used to delineate floodplain contours using the latest LIDAR-based topographic data, and differences between the model-based contours and the FEMA floodplain contours were compiled.

An example of the comparison is shown in **Figure 6-1** below for the Swift Run Drain.

Figure 6-1 – Comparison of FEMA FIRM Effective and InfoSWMM Model Results



Complete maps showing the floodplain comparison by Creekshed are shown in Appendix C.

Table 6-1 provides a comparison of the different models and data sets used in the two delineations.

Table 6-1 – Floodplain Delineation Data Sources

	FEMA FIRM Maps	Model-Based
Model Software	HEC-RAS	InfoSWMM
Steady/Un-steady Flow	Steady (except for portions of Traver Creek study)	Unsteady
Storm Standard Source	TP-40/ISWS Bulletin 71	NOAA Atlas 14
Storm Volume	4.36”/4.75”	5.11”
Hydrologic Analysis / Response Representation	Various (Rainfall-Runoff Unit Hydrograph method, Brater’s Unit Hydrograph method, MDEQ SCS, SCS unit-hydrograph)	SCS Type II / Green-Ampt infiltration
Elevation Contour Data Source	DEM (1997), field survey	LiDAR (2009)

The comparison of the InfoSWMM model-based 1% floodplain area to the existing FEMA FIRM floodplain area was made using ArcGIS software. For each creekshed, tabulations were made for the modeled floodplain surface area (acres), and the number of parcels and buildings affected by the modeled floodplain area, in each case compared to the effective FEMA FIRM map area. These results are shown in **Table 6-2**.

Table 6-2 – Comparison of FEMA FIRM to Model-based Floodplain Data

		Total	Allen	Malletts	Millers	Swift	Traver
FEMA FIRM Effective (within City Limit)	**Acres**	462	123	151	51	76	62
	Buildings	499	390	55	4	28	22
	Parcels	887	483	219	24	101	60
Model Delineated (within City Limit)	**Acres**	514	145	173	55	79	62
	Buildings	565	404	88	6	57	10
	Parcels	1205	635	352	25	120	73
Model Delineated (within Effective Limit of Study)	**Acres**	425	98	143	55	79	50
	Buildings	427	307	47	6	57	10
	Parcels	841	404	233	25	119	60
Model Delineated (beyond Effective Limit of Study)	**Acres**	89	47	29	0	0	12
	Buildings	138	97	41	0	0	0
	Parcels	238	121	111	0	0	6

		Total	Allen	Malletts	Millers	Swift	Traver
Added during comparison	**Acres**	176	56	77	11	14	19
	Buildings	242	130	76	2	33	1
	Parcels	318	152	133	1	19	13
Removed during comparison	**Acres**	125	34	55	7	10	18
	Buildings	176	116	43	0	4	13
	Parcels	0	0	0	0	0	0

There were two notable areas of the delineation and comparison where the FEMA FIRM mapping study limits led to significant differences

- Allen Creek south of Hill Street – In the current FEMA floodplain, the area of Allen Creek located south of Hill Street is not included. Using the InfoSWMM model data, the floodplain delineation would extend south through Hoover and S. State Street, covering an additional 47 acres. The area outside of the FEMA FIRM effective area would include 97 buildings and 121 parcels.

- Upper Malletts Creek – The scope of the existing FEMA floodplain delineation did not extend west of South Seventh Street because of tributary area size limitations in the mapping procedure. Using the citywide stormwater model for stormwater data would not have this restriction so the Upper Malletts Creek area was included in the delineation. The model-based floodplain area beyond the FEMA FIRM Effective study area would include an additional 14 acres, with 41 additional buildings and 98 additional parcels.

During the floodplain delineation and comparison, it was noted that many of the differences were a result of using newer LiDAR based contour data. To better understand the source of differences in the predicted floodplain areas, the City asked for a delineation using the existing FEMA FIRM flood delineated areas, while adjusting to utilize updated LiDAR elevation contours.

Table 6-3 – Floodplain Comparison Using LiDAR Contour Data Only

		Total	Allen	Malletts	Millers	Swift	Traver
FEMA Effective	**Acres**	462	123	151	51	76	62
	Buildings	499	390	55	4	28	22
	Parcels	887	483	219	24	101	60
LiDAR Contour	**Acres**	519	131	191	40	77	80
	Buildings	604	440	79	4	55	26
	Parcels	946	521	223	20	118	64
Net Change	**Acres**	57	8	40	-10	1	18
	Buildings	105	50	24	0	27	4
	Parcels	59	38	4	-4	17	4

The same delineation process was used and the results of the comparison are shown below in **Table 6-3**. A portion of the overall net change in acreage, buildings, and parcels included in the floodplain areas can be attributed to the updated LiDAR elevation contours. However, the updated rainfall volume and resulting flow data, and the addition of previously excluded areas in Allen Creek south of Hill Street and in the Upper Malletts Creek area west of South Seventh Street were the major factors in the differences shown in the floodplain area comparison.

7. Project Conclusions

The overall goals of the City of Ann Arbor Stormwater Model Calibration and Analysis project were to develop the model as a stormwater analysis tool and to provide answers to the City's current stormwater system management questions. Upon completion of the project, the following outcomes and conclusions are reported.

- **The citywide stormwater model has been updated to reflect the current system configuration and it has been calibrated based on collected flow and rainfall data.**

  Model updates were made prior to preliminary calibration to add model functionality, including representation of overland flows. Preliminary calibration with 2007 data provided improvements in model performance but was limited by a lack of large storm data. Additional data collection was recommended to improve dormant season parameters, boundary condition information, and calibration accuracy overall.

  Additional model updates were made to reflect 2013 stormwater system configuration and to allow for 2D modeling as part of final calibration. The calibration and validation work performed with 2013 data had good agreement between model-predicted values and monitor-observed values for volume and flow rate. Adjustments were made to the preliminary model parameters to improve the model performance. In general, the model-predicted flows and volumes were within 15% of recorded data, which fall within the expected range of agreement for stormwater models of this size and level of detail.

- **The project was able to involve stakeholders and interested citizens in the project.**
  A number of public engagement initiatives were utilized during the project and the following items were noted:
  - A high level of public participation was observed in Phase I public meetings and in the online stormwater survey, especially from areas that have been affected by recent flooding.
  - Areas that had not been affected by recent flooding were not well represented in Phase I public meetings.
  - The large event data gathering (LEDG) program was a successful public engagement activity, attracting a "Citizen Storm Corps", made up of interested residents who were able to participate directly in stormwater management observations.
  - The Stormwater Advisory Group (SWAG) was formed primarily to provide review and guidance of public interactions, but ended up providing valuable technical input and feedback throughout the entire project. The SWAG was made up primarily of stormwater professionals, representatives from local watershed groups, and interested citizens.
  - Phase II public meetings were reasonably well-attended, reflecting an overall interest in stormwater management issues by Ann Arbor residents.
  - A stormwater video was developed as part of the project that will highlight the importance and relevance of stormwater management in the City of Ann Arbor.

- The project had input from the over-arching wet-weather projects Technical Oversight and Advisory Group (TOAG) at key technical milestones, including after final calibration and during the stormwater improvements evaluations.

- **The existing stormwater system performance was evaluated for a range of design storms, leading to a set of potential stormwater system improvements.**

  The stormwater system is performing at a consistent design level of service for most areas of the City. The 10% annual exceedance probability (AEP), 12-hour storm is the current design standard, which is a 2.9" storm using NOAA Atlas 14 rainfall volumes. In the Allen Creek watershed and in the Malletts Creek watershed, there are areas where surface flooding is predicted during the 10% AEP storm and in some cases during the 20% AEP storm. Sixteen study areas were evaluated for potential stormwater system improvements and these were presented in a series of public meetings in November 2014. The recommended improvements total over $34 million and will be considered as part of the City's CIP Programming. Implementation of longer term stormwater management strategies are recommended for the Allen Creek watershed. The Green Streets portion of these improvement strategies was estimated at $80 million to $120 million.

- **The model was used to evaluate the effectiveness of stormwater management strategies.**

  The evaluation of future stormwater management strategies indicated that the City should continue runoff reduction programs, including the Green Streets Policy and Residential Rain Garden Programs. There should also be significant efforts put into encouraging compliance with new development standards during redevelopment of commercial, multi-family, and school or University properties. Future condition modeling scenarios show the potential for significant improvements in stormwater system performance, especially during more frequent storm events.

- **New model data was produced, allowing for comparison with existing FEMA FIRM Map 100-year floodplain delineation.**

  A FEMA FIRM floodplain comparison was performed using updated LiDAR elevation contours and also using flow and water level data generated by the new InfoSWMM model for the 1% annual exceedance probability (AEP) storm. The 1% AEP floodplain was delineated using these two data sets for comparison with the existing FEMA FIRM floodplain contours. The improved refinement of 1% AEP floodplain data will be available for future FEMA floodplain mapping and will support better decision-making on floodplain management issues.

- **Supporting documentation was produced, which will allow the City to utilize the stormwater model as a system management tool.**

  Project documentation being provided to the City includes archives of project data files and model files. Training sessions and written procedures for model updates and storm scenario updates have been prepared that will enable a smooth transition of stormwater modeling responsibilities and capabilities to City Staff. The model will be capable of providing output for various applications, from green infrastructure planning and stormwater system design, to floodplain analysis and emergency management. In addition, the City can build in procedures for model adaptation so that adjustments can be made to reflect future stormwater system performance monitoring or to respond to new storms or storm standards.

Appendix A

Existing Conditions System Capacity Analysis

(This section contains sensitive utility information and has been removed for security reasons. Please contact the City of Ann Arbor Public Services Area Administrator to schedule an appointment to view this information in person.)

Appendix B

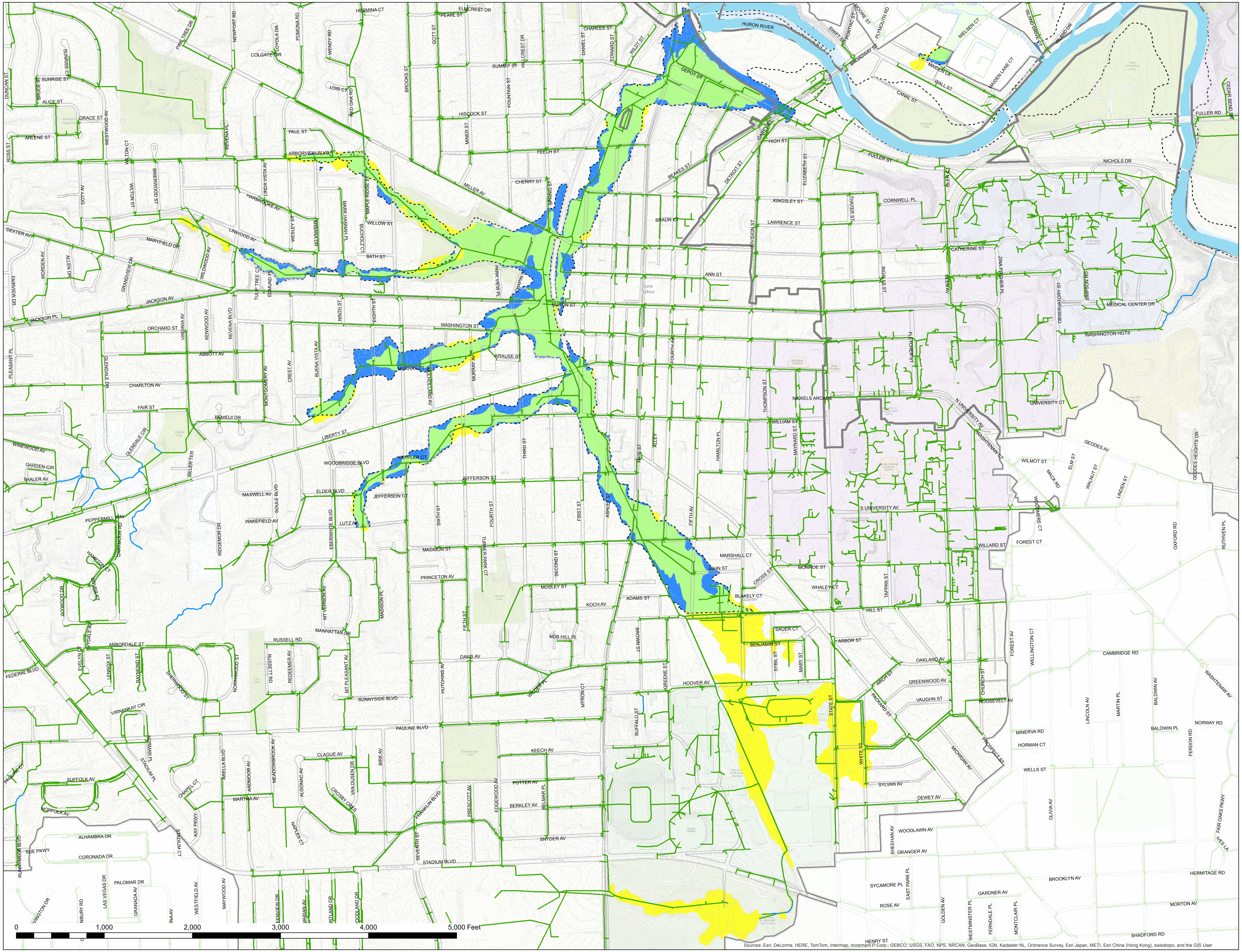
Future Conditions Stormwater Management Analysis

(This section contains sensitive utility information and has been removed for security reasons. Please contact the City of Ann Arbor Public Services Area Administrator to schedule an appointment to view this information in person.)

Appendix C

Floodplain Comparison Maps

Ann Arbor Storm Drainage System FEMA Floodplain Comparison Effective FEMA FIRM Area vs Model Delineation (Allen Creek)



Storm Drainage System

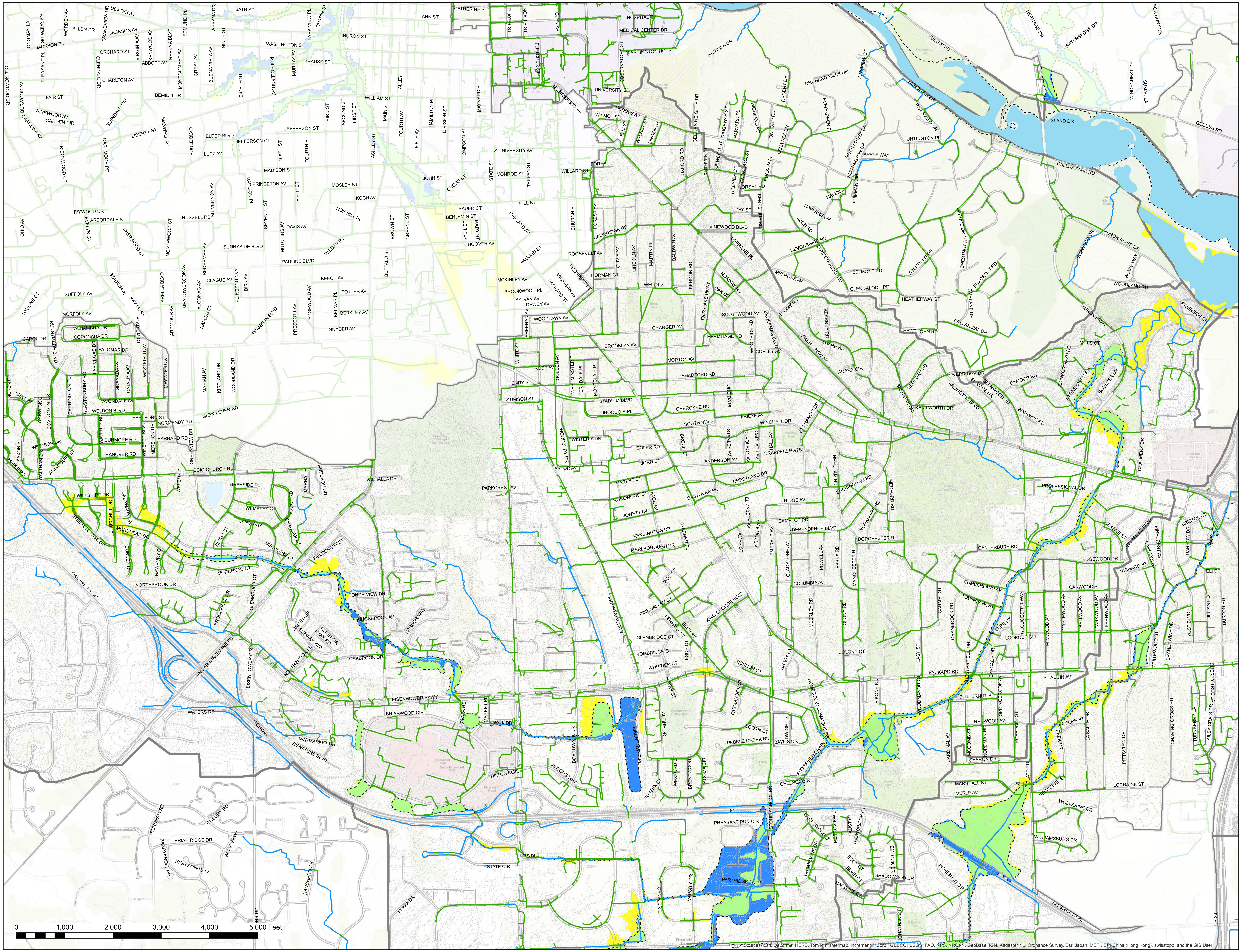
- Storm Sewer
- Open Channel

Floodplain

- Overlap between Effective and Model-Based Delineation
- Model-Based Delineation Extends Beyond Effective FEMA FIRM Area
- Effective FEMA FIRM Area Extends Beyond Model-Based Delineation
- Current Effective FEMA Zone AE

Date: 3/30/2015

Ann Arbor Storm Drainage System FEMA Floodplain Comparison Effective FEMA FIRM Area vs Model Delineation (Malletts Creek/ Swift Run)



Storm Drainage System

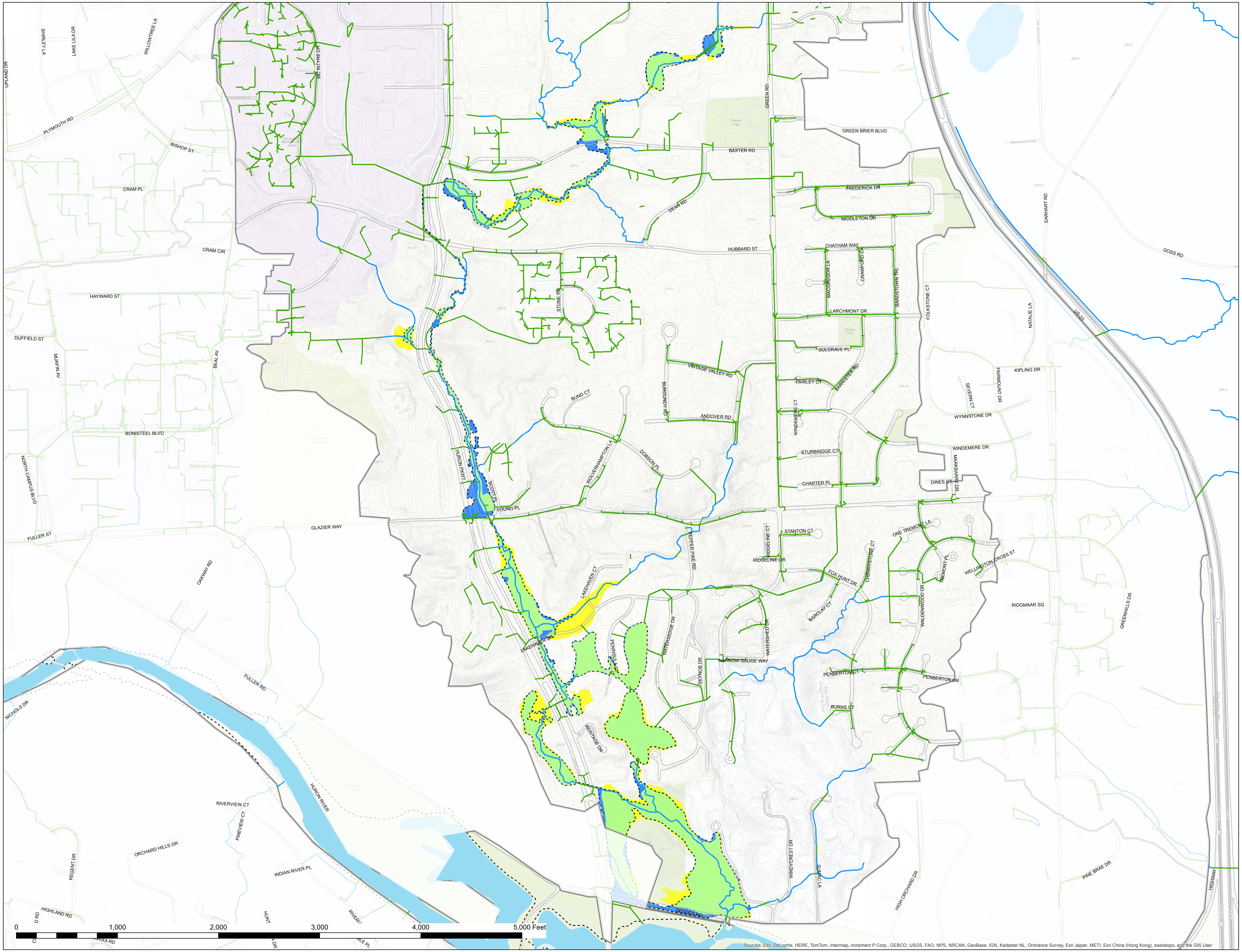
- Storm Sewer
- Open Channel

Floodplain

- Overlap between Effective and Model-Based Delineation
- Model-Based Delineation Extends Beyond Effective FEMA FIRM Area
- Effective FEMA FIRM Area Extends Beyond Model-Based Delineation
- Current Effective FEMA Zone AE

Date: 3/30/2015

Ann Arbor Storm Drainage System FEMA Floodplain Comparison Effective FEMA FIRM Area vs Model Delineation (Millers Creek)



Storm Drainage System

MILLERS

Storm Sewer

Open Channel

Floodplain

Overlap between Effective and Model-Based Delineation

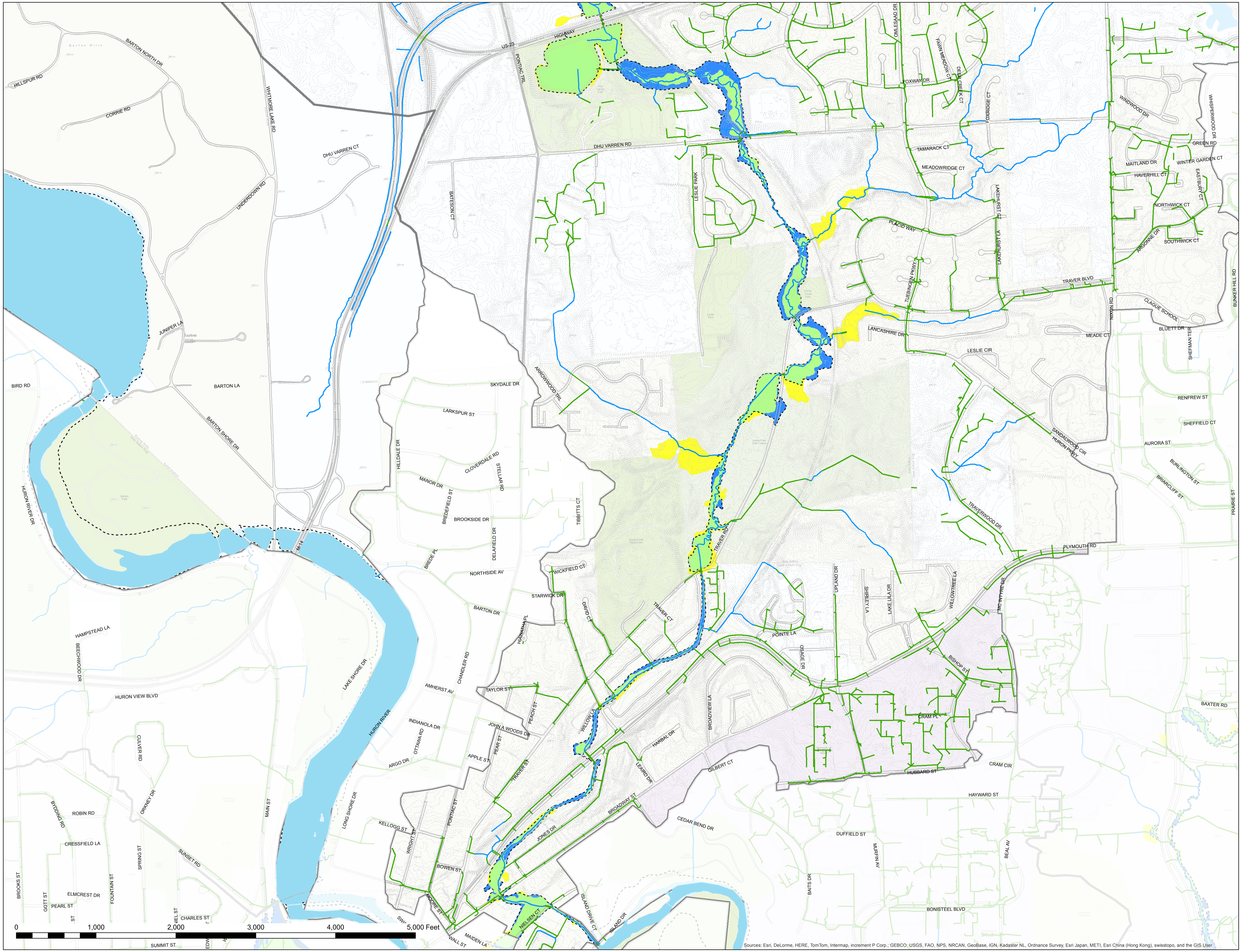
Model-Based Delineation Extends Beyond Effective FEMA FIRM Area

Effective FEMA FIRM Area Extends Beyond Model-Based Delineation

Current Effective FEMA Zone

Date: 3/30/2015

Ann Arbor Storm Drainage System FEMA Floodplain Comparison Effective FEMA FIRM Area vs Model Delineation (Traver Creek)



Storm Drainage System

TRAVER

Storm Sewer

Open Channel

Floodplain

Overlap between Effective and Model-Based Delineation

Model-Based Delineation Extends Beyond Effective FEMA FIRM Area

Effective FEMA FIRM Area Extends Beyond Model-Based Delineation

Current Effective FEMA Zone

Date: 3/30/2015