Green Infrastructure Effectiveness Analysis

v11/16/2021

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Executive Summary

The City of Madison wanted to understand if traditionally-sized green infrastructure (also known as volume control infrastructure (VCI)) can be used for flood control. For purposes of this study, traditionally-sized green infrastructure is stormwater management sized for the small storm events – typically the 1-year or 100% chance storm event.

The computer models created for the watershed studies were utilized for this study. These computer models were used because they were constructed for purposes of flood control and they reflect the current conditions of the watershed study areas.

The majority of the analysis described in this document is for the Pheasant Branch watershed, a 3,300-acre, mixed land use, watershed located on the far west side of the City of Madison.

The computer model hydrology (the component that generates the stormwater runoff) is separated into three parts: 1) directly connected impervious area (DCIA), non-directly connected impervious area (NDCIA), and 3) pervious area. DCIA is impervious that drains to a hard surface (like a street or driveway) and then into the stormwater conveyance system. Non-directly connected impervious area is impervious area than drains to a pervious area (like a lawn) before it reaches the stormwater conveyance system. This separation was initially done to aid in calibration of the models, but, was then taken advantage of for this study.

The analysis was conducted in two phases, with a peer review conducted after phase one.

The first phase evaluated the peak flow reductions for traditionally-sized infrastructure installed at typical levels of DCIA treatment. The levels were 10, 15, 20, and 25% of the DCIA. The results peak flow reduction was between 4 and 10.5%.

The second phase evaluated how much traditionally-sized green infrastructure it would take to achieve 1) 20% reduction in the 100-year (1% chance) event peak flow and 2) the watershed study flood targets. The results found that 75% of the DCIA would need to be treated to achieve a 20% reduction in the 1% chance (100-year) peak flow. The results also found that even if all DCIA was treated (100% DCIA), the watershed study targets would still not be met.

Cost estimates were completed for the various scenarios. Constructing VCI to treat 25% of the DCIA is estimated to cost approximately $78 million. Constructing VCI to treat 75% of the DCIA is estimated to cost $147 million.

The final step in the analysis was to understand how much the grey infrastructure (infrastructure typically installed for peak flow control such as larger pipes, detention basins, etc) could be downsized if the 75% DCIA VCI was also constructed. The results indicated that the grey infrastructure could be reduced by one size if VCI treating 75% of the DCIA was installed. The grey infrastructure cost savings is approximately $7 million.

Section 1. Introduction

The City of Madison, Wisconsin Engineering Division would like to understand the effectiveness of green infrastructure to help meet the City's flood mitigation goals. The City is evaluating traditional flood control solutions (increase pipe capacity, detention basins, etc.), so for simplicity, the City is referring to green infrastructure as "volume control infrastructure". Volume control infrastructure will be infrastructure meant to infiltrate or evapotranspire stormwater. Types of volume control infrastructure includes rain gardens, pervious pavement, green roofs, biofilters, and infiltration basins.

City Engineering conducted a pilot planning level analysis for the Pheasant Branch Watershed known as Phase I. The purpose of the analysis was to understand the data available and how it could efficiently be used to evaluate the effectiveness of volume control infrastructure. Engineering staff conducted the iterative analysis with check-ins and discussion.

Following the Phase I analysis, the City asked for a peer review from a consultant with extensive experience in Green Infrastructure Modeling and Design. Based on feedback, the City revised some components of the analysis. The City conducted the revised analysis known as Phase II.

In parallel, the City contracted with two consultants conducting flood mitigation watershed studies for different watersheds in the city. The consultants were asked to repeat Phase I of the analysis to understand if the results from the Pheasant Branch Watershed Study were repeatable.

Once the City's Phase II and the consultant's Phase I analysis were complete, the City used the results from both to create the analysis all consultants would complete for all watershed studies.

The analysis and results are described in this document.

Pheasant Branch Watershed

Pheasant Branch Watershed is located on the far west side of Madison and comprises approximately 3,300 acres of urban and agricultural land use.

Development in the watershed began in the 1970s and is still occurring today. Areas of the watershed developed after 1983 were subject to the City's stormwater management requirements. From 1983 to present data, stormwater management requirements became more robust, and today's requirements involve some volume control infrastructure.

The hydrologic and hydraulic model for the watershed was built in XP-SWMM 1D/2D. The hydrology and stormwater infrastructure are modeled in 1D while the large conveyance channels and overland flow is modeled in 2D.

The watershed is broken up into 452 subcatchments. The hydrology for the watershed utilizes SWMM hydrology and Horton Infiltration. Each subcatchment is broken into three components: 1) Directly Connected Impervious Area (DCIA), 2) Non-Directly Connected Impervious Area (NDCIA), and 3) Pervious Area.

The percent of DCIA and NDCIA is taken from the WinSLAMM model's standard land uses. The standard land uses representative the average source areas (roofs, driveways, sidewalks, etc) and connectedness in a typical land use. The standard land uses were developed by aerial photograph and field investigations of over 500 sites. A spreadsheet showing the breakdown of the standard land uses can be found in Appendix A. More information on the WinSLAMM model can be found here: http://www.winslamm.com/.

In each subcatchment, the NDCIA is routed to the pervious area through subcatchment routing. The flow path and slope for all components is the same – that calculated for the overall subcatchment.

City of Madison Flooding Level of Service Goals

The City of Madison has the following level of service goals for flood mitigation:

1. 10-year design storm event:
    a. No surcharging onto the street for up to the 10-year design storm; water shall becontained within the pipes and structures.
    b. There are locations within the City where low points exist that pond water; theselow points are excluded from this goal and will be addressed as streets are redesigned.
    c. For locations limited by known inlet capacity, allow no more than 0.5 feet of waterabove storm sewer inlet rim.
2. 25-year design storm event:
    a. Centerline of street to remain passable during 25-year design storm with no morethan 0.2 feet of water at the centerline.
    b. Note that the Watershed Study modeling approach will not explicitly account for cross flow conditions where more gutter flow on one side of the street can overtopthe crown.
3. 100-year design storm event:
    a. No home or business will be flooded during the 100-year design storm.
    b. Enclosed depressions to be served to the 100-year design storm (which can includesafe overland flow within street, easements, greenways or other public lands).
    c. Greenway crossings at streets to be served to the 100-year design storm.
    d. No more than 0.5 feet of water on the centerline of arterial roads for passage ofemergency vehicles
4. 500-year design storm event:
    a. Safely convey stormwater; i.e. limited impact on private property
5. Provide flooding solutions that do not negatively impact downstream properties.

Table 1-1 provided the Design Storm Depths and Distribution used for evaluating the City's Flood Mitigation goals.

Table 1-1. Design Storm Depths and Distribution

Recurrence Storm	Rainfall Depth (in)	Rainfall Distribution
1-year (100% Chance)	2.49	MSE4 – 24 hour
2-year (50% Chance)	2.84	MSE4 – 24 hour
5-year (20% Chance)	3.45	MSE4 – 24 hour
10-year (10% Chance)	4.09	MSE4 – 24 hour
25-year (4% Chance)	5.02	MSE4 – 24 hour
100-year (1% Chance)	6.66	MSE4 – 24 hour
500-year (0.2% Chance)	8.94	MSE4 – 24 hour

Volume Control Modeling Goals

The purpose of this analysis was to understand the following:

1. Determine the volume of water that would need to be managed if a certain percentage of DCIA were to be treated,
2. Determine if there is a break point/ target for volume control treatment,
3. Assess the watershed to see how much area is available for volume control,
4. Assess the impact of infiltration size for the 1- year event on larger storms,
5. Assess the sensitivity of the existing drainage system to the implementation of volume controlstormwater devices to treat directly connected impervious areas, and
6. Determine if the combination of the peak flow control efforts and volume control efforts canprovide a quantifiable increase in the level of service.

Section 2. Phase I Analysis

The Phase I Analysis was conducted in three parts. First, the volume of water that could be treated for each of the impervious area treatment goals was calculated. Second, the land available for treatment was calculated and compared to the land needed. Finally, a watershed model with the treatment was developed and executed. The results were summarized and an initial conceptual cost estimate was prepared.

A Microsoft Excel spreadsheet (called “GI Analysis Phase I 2020_07_03.xlsx”) accompanies this document and is referred to in numerous locations.

Part 1 – Estimate Potential Volume Control Effects per Subcatchment

Initially, the City wanted to understand if treating a certain percentage of the DCIA would generate a significant reduction in runoff volume or peak flows. 10%, 15%, 20%, and 25% of DCIA in each subcatchment was routed out of the drainage network. To do this, the associated impervious area from the DCIA for each subcatchment was simply removed from the model. The assumption in this step was that each DCIA percentage was fully treated by volume control infrastructure. “Step 1 Calc DCIA Removed” and “Step 1 Areas Table” in the accompanying spreadsheet were used to calculate the areas for each percentage.

The Runoff Block of XP-SWMM was run for 10%, 15%, 20%, and 25% DCIA removed scenarios. The peak flows and runoff volumes for each subcatchment were summarized in the “Step 1 Flow cfs Results” and “Step 1 Volume ac_ft Results” tabs. The results are graphed in the “Flow Results Chart Per”, “Volume Results Chart ac-ft”, and “Volume Results Chart Per” tabs. Figure 2-1 shows a graph of the peak flow results.

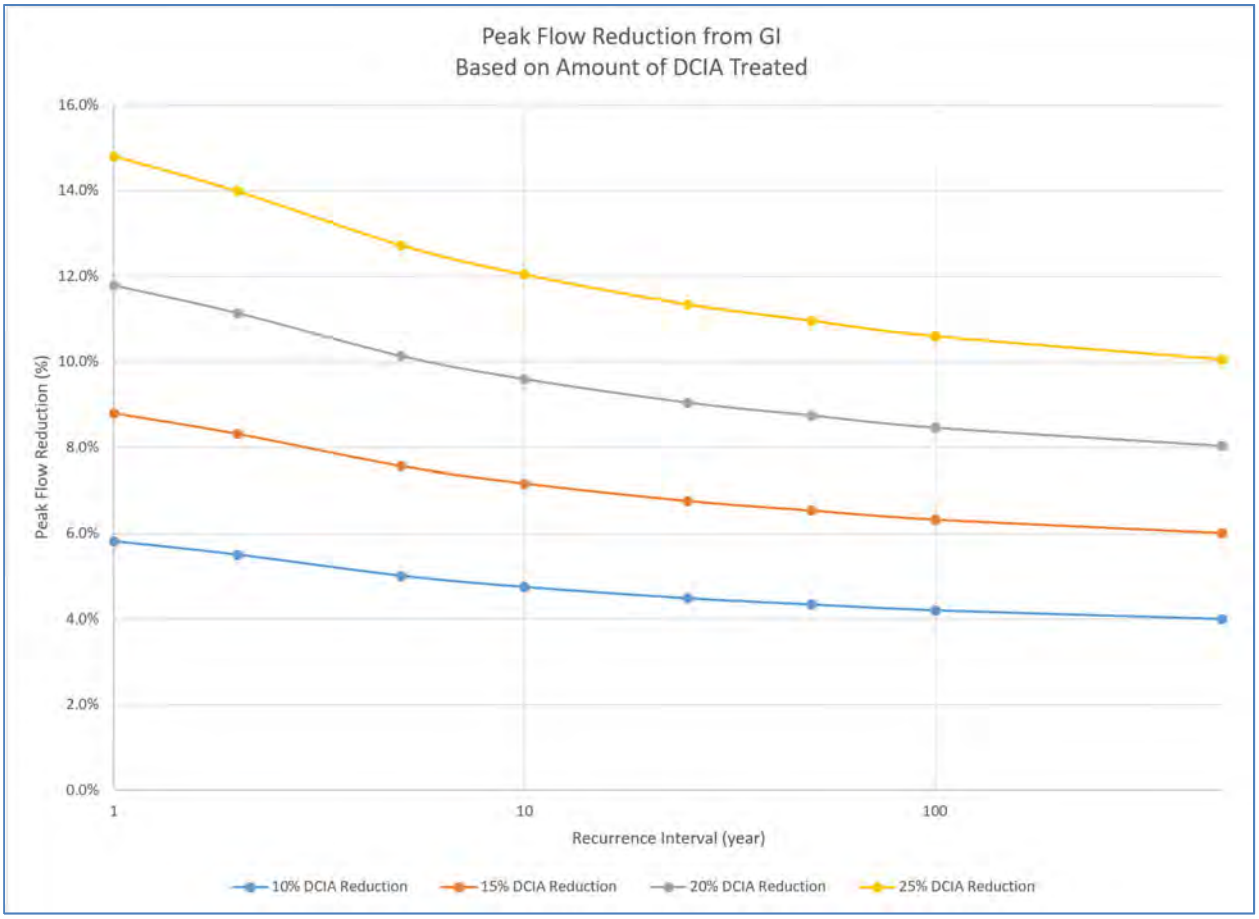

Figure 2-1. Peak flow reduction for each DCIA Scenario

Part 2 – Initial Opportunities Analysis

Volume Control Infrastructure Layer

The City conducted a desktop analysis utilizing available Geographic Information Systems (GIS) data to understand where there are opportunities to install volume control infrastructure. This analysis was conducted over the span of several months and in conjunction with various City agencies including Forestry, Historic & Cultural Resources, Parks, and Water Utility.

Following the inter-agency meetings and review, the following areas were removed from the available area:

- Airports
- Primary building footprints with a 10 foot buffer
- Accessory building footprints
- Buffer of 3 feet along parcel boundaries
- Cemeteries
- Depth to bedrock < 5’
- Depth to groundwater < 5’
- Hydrologic soil group D

- Open water
- Landfills
- Railroads
- Springs
- Wellhead protection zone
- Wetlands
- Salt routes
- Arterial and collector streets (0 < function class < 5)
- Slope > 12 %

The remaining pervious and impervious areas were considered available for volume control infrastructure.

Initial Volume Control Infrastructure Scenarios

The City decided upon three scenarios for the size of volume control infrastructure that could be installed. They are described below:

1. Realistic (sf): The area for green infrastructure assuming realistic size green infrastructure isinstalled
2. Maximize (sf): The area for green infrastructure assuming the realistic sized area is maximized
3. Available GI Space (sf): The area for green infrastructure if all the available space is utilized.

The Table 2-1 below shows the parameters used for each scenario. All scenarios include a 3-foot buffer around the volume control infrastructure as a transition to the existing land cover.

Table 2-1. Scenario Land Use and VCI Area Assumptions

Parcel Land Use	All Available GI Space (sf)	Maximize (sf)	Realistic (sf)
Residential	if >3000 SF	240	120
Residential	If <3000 SF and >1000 SF	120	60
Residential	If <1000 SF and >60 SF	60	30
Commercial/Industrial/ Institutional	Any size	All available	All available
Parks	Any size	10% of available	5% of available
Open space (not parks)	Any size	20% of available	10% of available
Streets	Any size	20% of available	10% of available

This layer was intersected with each subcatchment and the areas for each scenario were calculated for each subcatchment. Tabs"PB_GI_Areas_More_Restrictive" and "PB_GI_Less_Restrictive" show the results of the calculations.

Note, this analysis was done with both a "More Restrictive" layer and a "Less Restrictive" layer. The More Restrictive layer excluded additional areas where volume control infrastructure could be implemented such as areas within the wellhead protection zone, areas with shallow bedrock or groundwater, and areas with high slope.

The area of available space for volume control infrastructure in the Pheasant Branch Watershed for the more restrictive layer for this initial analysis was calculated to be 1,736 acres and the less restrictive layer's available area was calculated to be 2,197 acres.

After internal discussions, it was decided the "Less Restrictive" layer would be the layer to use in determining the available space in each subbasin for VCI.

Step 1: Calculate the Area needed for Volume Control Infrastructure for each Subcatchment

Using the potential volume that could be infiltrated from Part 1, the area needed to capture that volume was calculated for each subcatchment, for each of the three scenarios. An assumed depth of 1-foot (open area storage) was assumed for each VCI (the potential storage from subgrade porosity was ignored).

Tab "Step 1b Area Needed for GI" shows the results of the calculations.

Please note, this analysis was done for both the 1-year and 10-year storm event volumes. Following internal discussions, it was decided that future calculations would only be done for the 1-year storm event volume.

Step 2: Compare the Potential VCI Volume to the Volume Reduction Calculated in Part 1 for each Subcatchment

The area needed to treat the volume was compared to the area available for each scenario.

Tab "2 Step 5 Compare Areas" shows the results of the comparison. Columns DV-EN summarize the results.

The Figure 2-2 shows the maximum percent of DCIA that can be treated for each subcatchment for the 1- year, 24-event with the land available for the More Restrictive, Realistic Scenario.

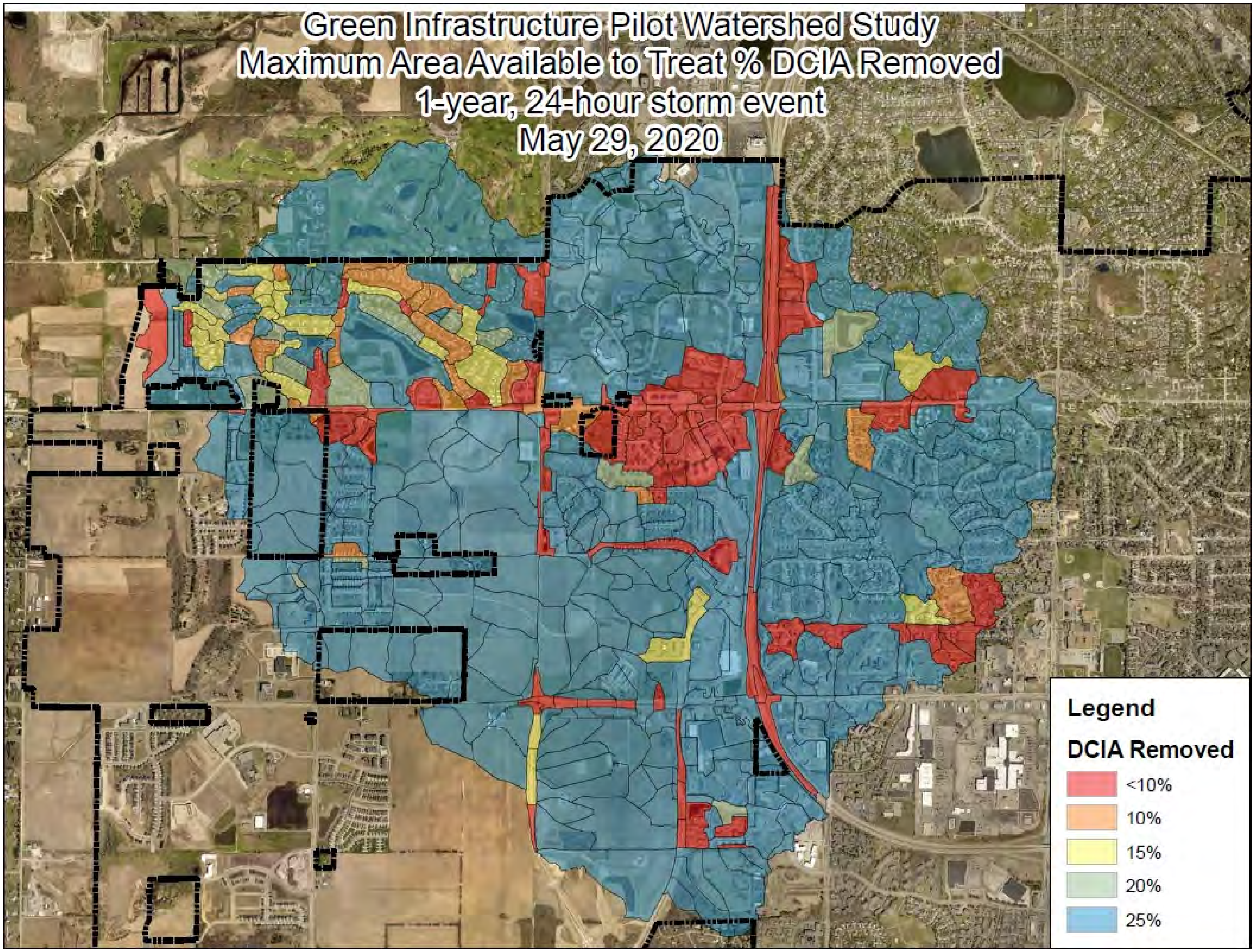

Figure 2-2. Maximum percent of DCIA that can be treated for each subcatchment for the 1-year, 24-event for the More Restrictive, Realistic Scenario

Figure 2-2 indicates that the majority of the watershed has land available to treat 25% of the DCIA.

Additional calculations were conducted to understand if at least ½ inch of runoff volume was being treated from all impervious surfaces. The results of the analysis show that treating 25% of the DCIA equates to treating ½ inch of runoff from all impervious surfaces.

Columns DA – DF of tab "Step 1 Volume ac-ft Results" summarize the results of the impervious surfaces runoff volume calculations.

As a result of this Phase I analysis, the City chose a treatment goal of 25% of the DCIA for Pheasant Branch Watershed. This goal was selected for two reasons:

1. Overall, land is available to treat 25% of the DCIA in the watershed
2. Treating 25% of the DCIA also treats ½ inch of runoff from all impervious surfaces (using a weighted average). ½ inch of runoff is the treatment goal for the City of Milwaukee greeninfrastructure plan and represents the "first flush" of stormwater.

Step 3: Create Max Available Scenario

Step 1 above assumes that land in some subcatchments can be over-treated to account for the subcatchments where land is not available to treat 25% of the DCIA. The Max Available Scenario was created to understand what the impacts of runoff volume and peak flow reduction would be if overtreatment did not occur. This scenario was set up using the maximum DCIA that could be treated given the available area (up to 25%).

Tab "2 Step 5 CA Figure" shows the maximum % DCIA that can be treated (up to 25%). The runoff volume and peak flow calculations from this step are not summarized in the spreadsheet.

Part 3 – Develop Final VCI Model

The final part to the Phase I Analysis evaluates the impact volume control infrastructure has on the conveyance system of the watershed. The volume control infrastructure for this analysis will be sized for the 1-year storm event. For storms in excess of the 1-year event, the volume control infrastructure will overflow. Therefore, a model representing the volume control from the 1-yr volume control infrastructure is needed to understand the net impact from the volume control infrastructure.

Running the full model (hydrology, 1D hydraulics, and 2D) for the work done in Part 1 would overestimate the reductions for storm events above the 1-year storm event. This is because the representative area of DCIA was removed and, as a result, the runoff volume for that sub-area for each storm event was also removed.

To develop the final VCI model, 25% of the DCIA was added to the NDCIA for each subcatchment. Then, the pervious depression storage was adjusted until the 1-year storm event results from Part 1 matched the 1-year storm events from this Part for the subcatchments.

The same methodology was utilized for the Max Available Scenario.

Tabs “Step 2a Move DCIA to NDCIA” and “Step 2a Move DCIA to NDCIA Avai” show the calculations to move the DCIA to the NDCIA. Tabs “Step 2c Pervious Dep Stor Calc” and “Step 2c Pervious Dep Stor C Av” show the calculations to modify the pervious depression storage. Tabs “Step 3 Flow cfs Results” and “Step 3 Volume ac_ft Results” show the results of the analysis for each subcatchment.

The model was then run for both scenarios for the 1-, 10-, and 100-year, 24-hour events. Figures 2-3 and 2-4 show the resulting peak flows at two locations within the watershed.

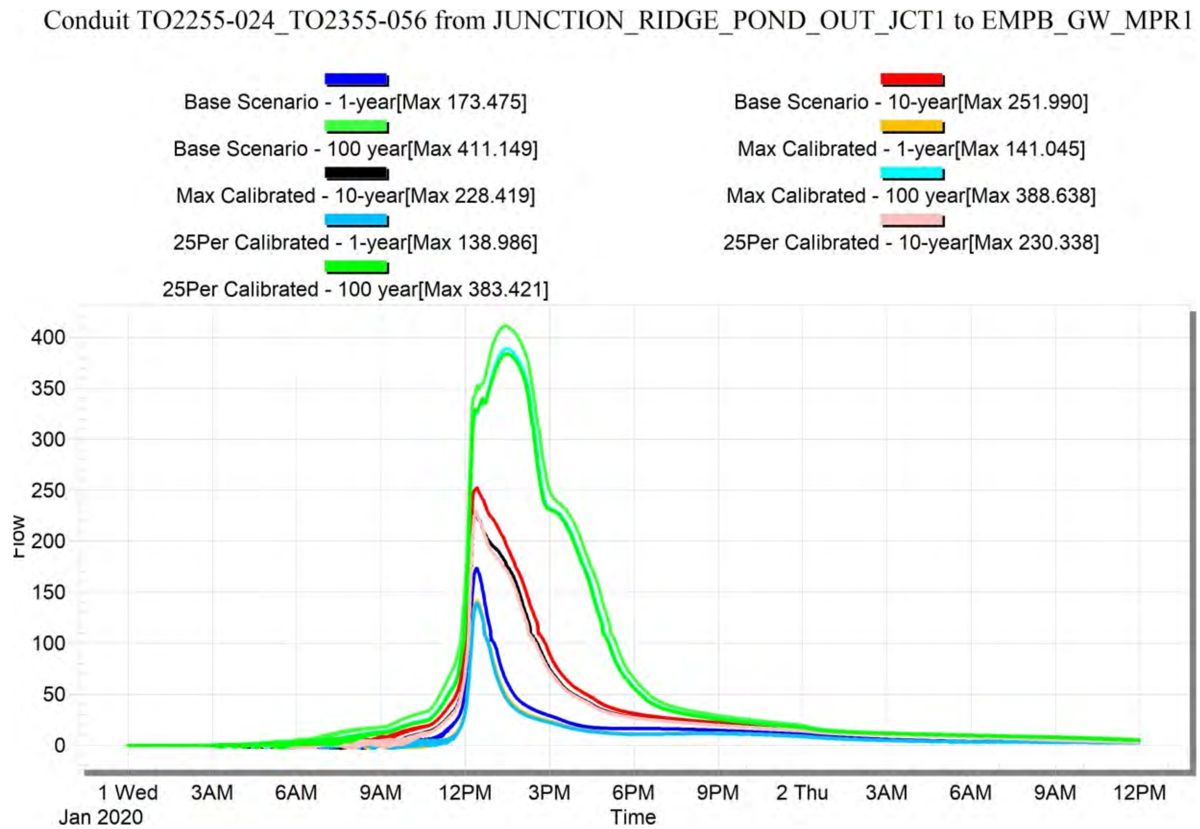

Figure 2-3. XP-SWMM Output at culvert from Target Pond under Beltline to Sauk Creek Greenway

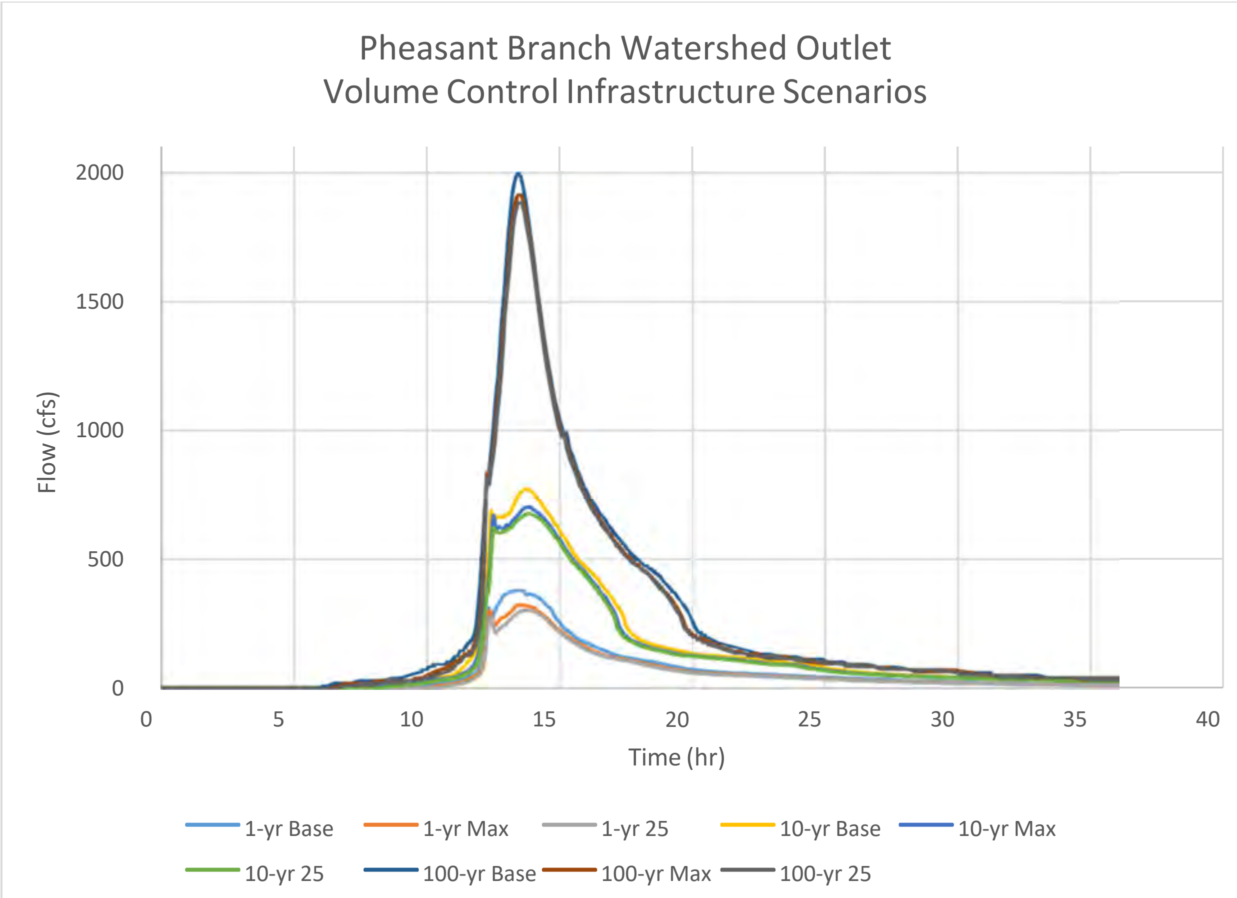

Figure 2-4. Outlet of Pheasant Branch Watershed (Upstream of Greenway Boulevard)

Table 2-2 shows a summary of the peak flows at the outlet of the Pheasant Branch Watershed and the reductions.

Table 2-2. Peak Flow Comparison at Outlet of Pheasant Branch

Storm Event	Base Scenario	25% DCIA Rerouted		Max DCIA Rerouted	
	Peak Flow (cfs)	Peak Flow (cfs)	Percent Reduction (%)	Peak Flow (cfs)	Percent Reduction (%)
1-yr	377	302	20%	322	15%
10-yr	772	677	12%	702	9%
100-yr	1,998	1,889	5%	1,920	4%

As expected the more frequent storm events have a higher peak flow reduction than the less frequentstorm events.

Phase I Conceptual Cost Estimate

The City recently completed construction of volume control infrastructure in a pilot study area in the City. Utilizing the bid tab from that project, construction of the volume control infrastructure is approximately $40 per square foot. The volume control installed in the pilot study area is a mix of residential rain gardens, pervious pavement, and underground storage trenches. This combination of practices is representative of the combination that would be constructed in the Pheasant Branch Watershed.

The City has conducted research regarding the average maintenance cost for volume control infrastructure. The average cost is approximately $3 per square foot of volume control infrastructureper year.

The costs from the pilot study area and maintenance research were applied to the potential treatment area for Pheasant Branch Watershed. Table 2-3 shows the results of the conceptual cost estimate.

Table 2-3. Conceptual Cost Estimate

Scenario	Total Impervious Area Treated (acres)	Total TreatmentArea (ac)	Estimated VCI Construction Cost ($)	Estimated VCI Maintenance Cost ($/yr)
25% DCIA Rerouted	221.2	45.0	$78,359,000	$5,877,000
Max % DCIA Rerouted	164.5	35.0	$60,954,000	$4,572,000

Phase I Total Phosphorus Load Reduction Estimate

Volume control infrastructure can be constructed for several reasons. A common reason is stormwater pollution reduction. The City of Madison has a WPDES Individual MS4 Permit and is part of the Rock River Total Maximum Daily Loads (TMDL). The Rock River TMDL requires reductions in total phosphorus.

A simplified analysis of the total phosphorus that would be removed from the treated area was conducted in WinSLAMM. WinSLAMM is a continuous simulation, water quality model using small storm hydrology. It has the ability to simulate many types of source areas, included directly connected and disconnected impervious areas. It can evalute many types of traditional and volume control infrastructure.

In the model, the treated area was assumed to be half roofs and half streets (streets with an assumed width of 40 feet and intermediate texture) in a residential land use. The roofs and streets were routed to a biofilter with a 1 foot of depth and a native soil seepage rate of 0.3 inch/hr. The, the City estimated that the total phosphorus reduction from the 25% DCIA Rerouted scenario is approximately 382 pounds per year. The total phosphorus reduction from theMaximum Rerouted scenario is approximately 284 pounds per year.

Section 3. Peer Review

The City contracted with Birchline Planning, LLC to revise the City of Madison's ordinances and policies with respect the distributed green infrastructure. Tetra Tech, Inc. is on the Birchline team for the project. Tetra Tech is a leader of green infrastructure analysis and design in the stormwater industry. The City asked Tetra Tech to review the City's Phase I analysis for technical soundness as well as areas where the analysis could be revised/modified to further meet the City's goals. The review can be found in Appendix A.

In general, the peer reviewer found the Phase I Analysis to be technically sound.

Tetra Tech offered ideas to enhance the analysis. After internal discussions, the following were incorporated into the Phase II Analysis discussed in the next section:

- Expand the Opportunities Analysis to include flat roofs that could have green roofs installed
- Because the purpose of the analysis is for flood control, expand the targeted amount of area to be treated. The City thinks that about 20% reduction in peak flows for the 100-year event could result in a decrease in infrastructure size. As such, an analysis was conducted to determine the percentage of DCIA to treat to achieve 20% reduction in peak flow for the 100-year event at the outlet of the watershed.
- Expand the benefits to more than just flood control and water quality improvement.   Green infrastructure has many other benefits besides the direct effect on stormwater, including improved aesthetics, reduction in heat island effect, climate resiliency, etc.   These additional benefits will be summarized.

Section 4. Phase II Analysis

The Phase II Analysis was conducted in a similar manner as Phase I. First, the reductions in peak flows that could be achieved by routing 25%, 75%, and 100% of the DCIA to appropriately sized green infrastructure was determined. Second, the opportunities analysis was expanded to include sidewalks, flat roofs, and residential streets that are not salt results. Finally, the land needed to treat both the 25%, 75%, and 100% DCIA was overlaid onto the watershed map for Pheasant Branch Watershed. The results were summarized, benefits were documented, and a conceptual cost estimate was prepared.

A Microsoft Excel spreadsheet accompanies this document (called "GI Analysis Phase 2 2021_02_08.xlx") and is referred to in numerous locations.

Part 1 – Estimate Peak Flow Reductions from 25%, 75%, and 100% DCIA

Part 1: Determine 25%, 75%, and 100% DCIA

25%, 75%, and 100% of DCIA in each subcatchment was routed out of the drainage network. To do this, the associated impervious area from the DCIA for each subcatchment was simply removed from the model. The assumption in this step was that each DCIA percentage was fully treated by volume control infrastructure. "Step 1a Calc DCIA Removed" and "Step 1 Areas Table" in the accompanying spreadsheet were used to calculate the areas for each percentage.

The Runoff Block of XP-SWMM was run for 25%, 75%, and 100% DCIA removed scenarios. The peak flows and runoff volumes for each subcatchment were recorded.

Part 2: Develop full VCI model for 25% DCIA, 75%, and 100% DCIA

To develop the full VCI model, 25% of the DCIA was added to the NDCIA for each subcatchment for the 25% DCIA scenario, 75% of the DCIA was added to the NDCIA for each subcatchment for the 75% DCIA scenario, and 100% of the DCIA was added to the NDCIA for each subcatchment for the 100% DCIA scenario. Then, the pervious depression storage was adjusted until the 1-year storm event results from Part 1 matched the 1-year storm events from this Part for the subcatchments.

Tab "Step 2a Move DCIA to NDCIA" shows the calculations to move the DCIA to the NDCIA. Tab "Step 2c Pervious Dep Stor Calc" shows the calculations to modify the pervious depression storage. Tabs "Step 3 Flow cfs Results" and "Step 3 Volume ac_ft Results" show the results of the analysis for each subcatchment.

The model was then run for all scenarios for the 1-, 10-, and 100-year, 24-hour events. Tables 4-1a through 4-1c below show the results for the scenarios for the 1-, 10-, and 100-year, 24-hour events.

Table 4-1a. 25% DCIA Scenario Peak Flow Results and Comparison

		Existing Conditions			25% DCIA Treated					
					1-yr		10-yr (cfs)		100-yr (cfs)	
Recording Location	Location	1-yr (cfs)	10-yr (cfs)	100-yr (cfs)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)
1	Outlet of Swallowtail Pond crossing Pleasant View Road	47	68	359	48	-2%	64	5%	337	6%
2	Target Pond Outlet (3 culverts) West to East Under Beltline	166	292	468	131	21%	245	16%	445	5%
3	Culverts East to West under Beltline just downstream of Wexford Pond	260	646	1,435	214	18%	561	13%	1,342	6%
4	Watershed Outlet	228	744	2,114	220	3%	649	13%	1,998	5%

Table 4-1b. 75% DCIA Scenario Peak Flow Results and Comparison

		Existing Conditions			75% DCIA Treated					
					1-yr		10-yr (cfs)		100-yr (cfs)	
Recording Location	Location	1-yr (cfs)	10-yr (cfs)	100-yr (cfs)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)
1	Outlet of Swallowtail Pond crossing Pleasant View Road	47	68	359	36	24%	48	29%	294	18%
2	Target Pond Outlet (3 culverts) West to East Under Beltline	166	292	468	63	62%	164	44%	396	15%
3	Culverts East to West under Beltline just downstream of Wexford Pond	260	646	1,435	110	58%	419	35%	1,109	23%
4	Watershed Outlet	228	744	2,114	175	23%	515	31%	1,699	20%

Table 4-1c. 100% DCIA Scenario Peak Flow Results and Comparison

		Existing Conditions			100% DCIA Treated					
					1-yr		10-yr (cfs)		100-yr (cfs)	
Recording Location	Location	1-yr (cfs)	10-yr (cfs)	100-yr (cfs)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)
1	Outlet of Swallowtail Pond crossing Pleasant View Road	47	68	359	19	59%	47	31%	211	41%
2	Target Pond Outlet (3 culverts) West to East Under Beltline	166	292	468	34	80%	123	58%	348	26%
3	Culverts East to West under Beltline just downstream of Wexford Pond	260	646	1,435	32	88%	319	51%	862	40%
4	Watershed Outlet	228	744	2,114	112	51%	425	43%	1,346	36%

As an additional scenario, all of the surface cover in each subcatchment was revised to be pervious. This was done as a comparison of the effectiveness of green infrastructure to something that can be visualized. The built conveyance system remained in the model. This scenario was run in the full VCI model. The results provided in Table 4-1d show that treating 100% of the DCIA is very close to a scenario where all the surface cover in a watershed is pervious.

Table 4-1d. 100% Land in Watershed is Pervious Scenario Peak Flow Results and Comparison

		Existing Conditions			Entire Watershed 100% Pervious					
					1-yr		10-yr (cfs)		100-yr (cfs)	
Recording Location	Location	1-yr (cfs)	10-yr (cfs)	100-yr (cfs)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)
1	Outlet of Swallowtail Pond crossing Pleasant View Road	47	68	359	22	54%	46	31%	223	38%
2	Target Pond Outlet (3 culverts) West to East Under Beltline	166	292	468	17	90%	134	54%	384	18%
3	Culverts East to West under Beltline just downstream of Wexford Pond	260	646	1,435	26	90%	286	56%	917	36%
4	Watershed Outlet	228	744	2,114	95	58%	265	64%	1,332	37%

The all-pervious analysis shown in Table 4-1d still resulted in street and structure flooding. This indicates that even if 100% of all impervious surfaces were returned to pervious, the built stormwater conveyance system is not sized large enough.

Progress towards City's Flood Mitigation Targets

The beginning of this document describes the City's Flood Mitigation Level of Service goals. The modeling results were reviewed to understand how the 25% DCIA, 75% DCIA, and 100% DCIA scenarios helped the City meet the goals.

Figures 4-1 and 4-2 below show the impact of the 25% DCIA scenario on the 10-year and 100-year goals. Figures 4-3 and 4-4 below show the impact of the 75% DCIA scenario on the 10-year and 100-year goals. Figures 4-5 and 4-6 below show the impact of the 100% DCIA scenario on the 10-year and 100-year goals.

For purposes of this analysis, streets are considered removed from flooding if there is no longer water along the centerline of the road and structures are considered removed from flooding if there is no longer water touching the 5-foot buffer placed around the building's outline.

Table 4-2 shows the existing streets and structures impacted and how they are reduced for each scenario.

Table 4-2. Street Segments and Structures Removed From Flooding

	Street Segments (miles) 10-year Storm Event			**Structures (#) 100-year Storm Event**		
Total	52.1			2,920		
Scenario	**Flooded**	**Percent**	**Removed**	**Flooded**	**Percent**	**Removed**
Existing Conditions	9.9	19%	N/A	118	4%	N/A
Grey Infrastructure						
Proposed Conditions	1.0	2%	8.9	57	2%	61
Green Infrastructure						
25% DCIA	8.7	17%	1.1	109	4%	9
75% DCIA	5.8	11%	4.1	79	3%	39
100% DCIA	2.3	4%	7.5	51	2%	67
Both Green and Grey	0.5	1%	9.3	15	1%	103

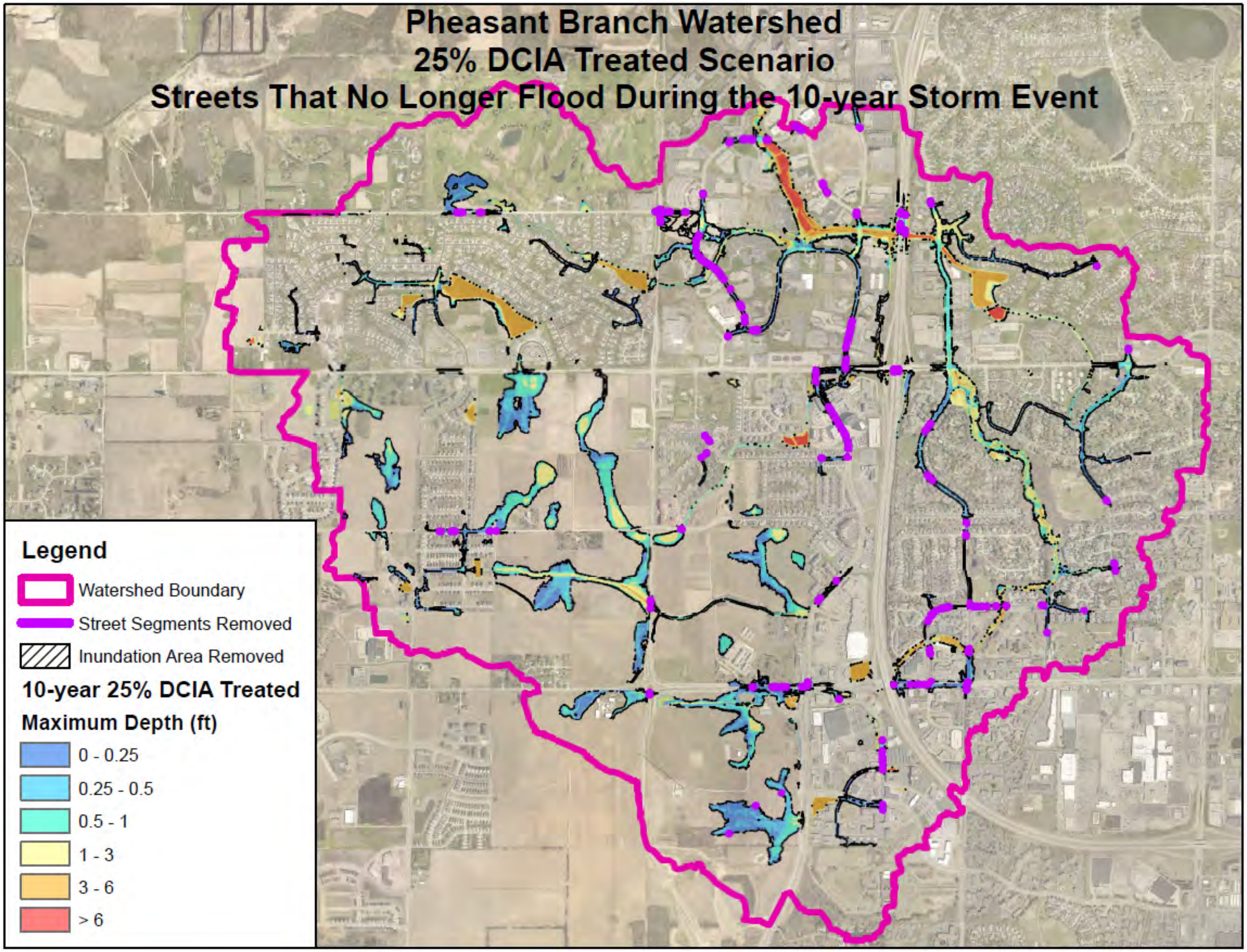

Figure 4-1. 25% DCIA Treated Scenario, 10-year Storm Event

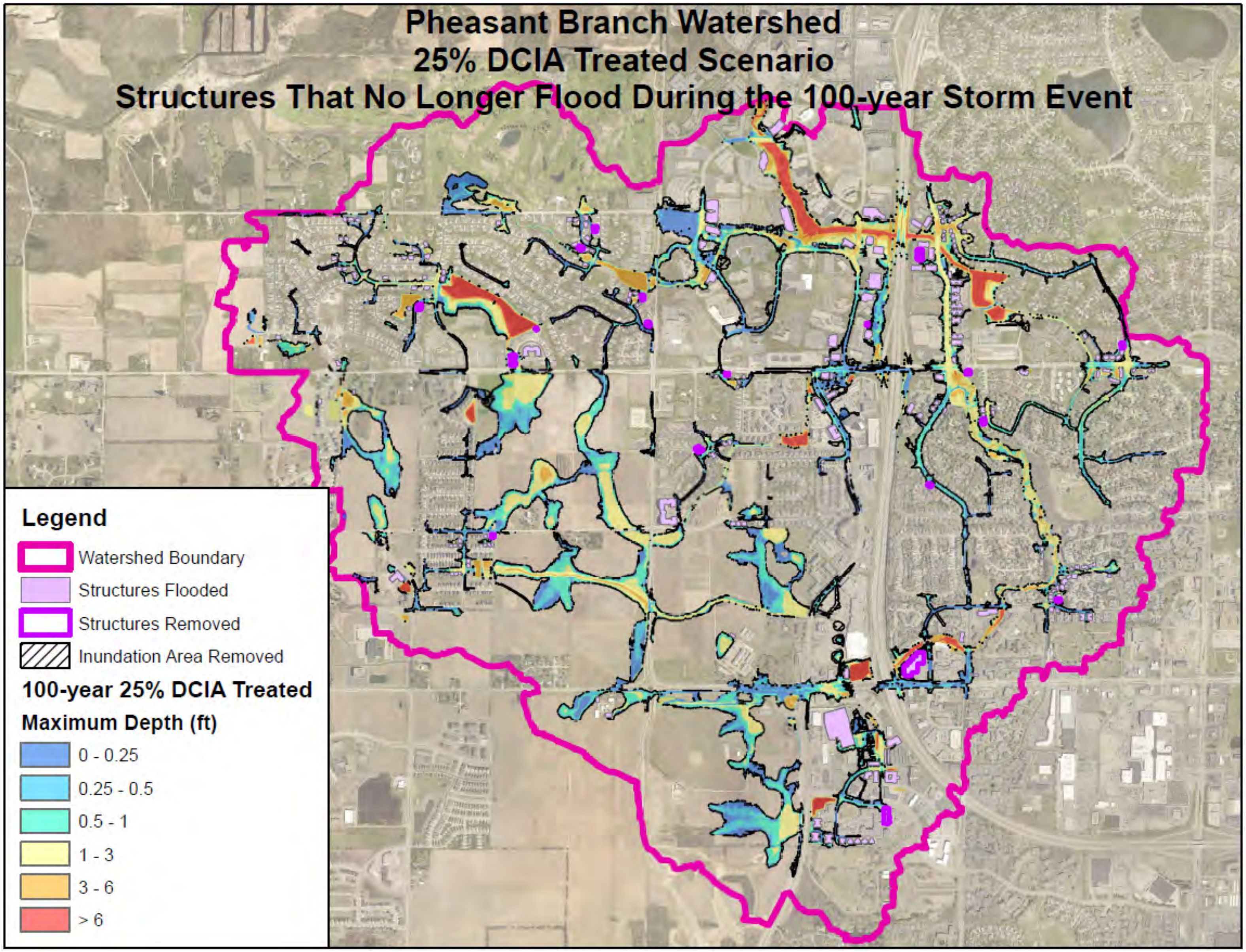

Figure 4-2. 25% DCIA Treated Scenario, 100-year Storm Event

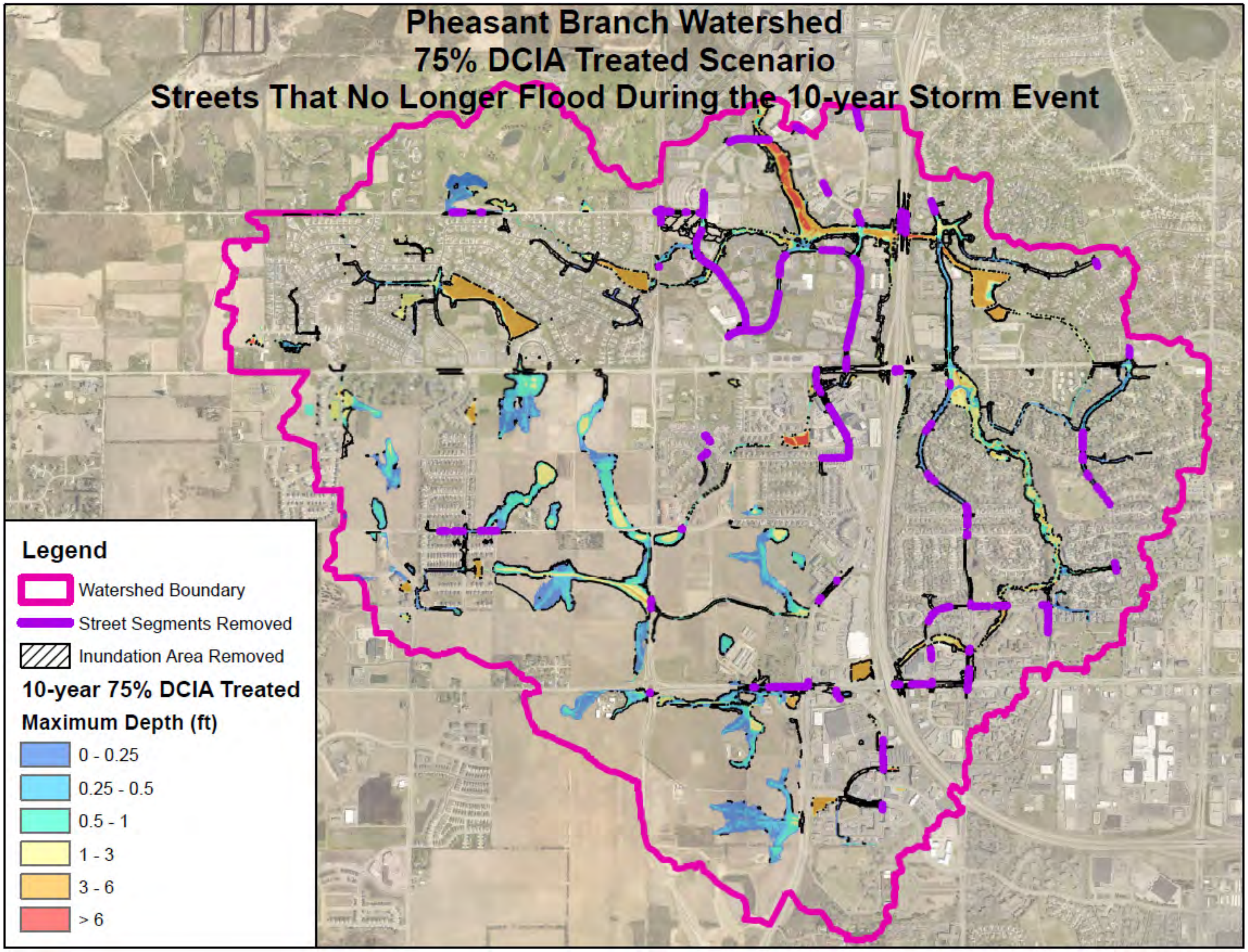

Figure 4-3. 75% DCIA Treated Scenario, 10-year Storm Event

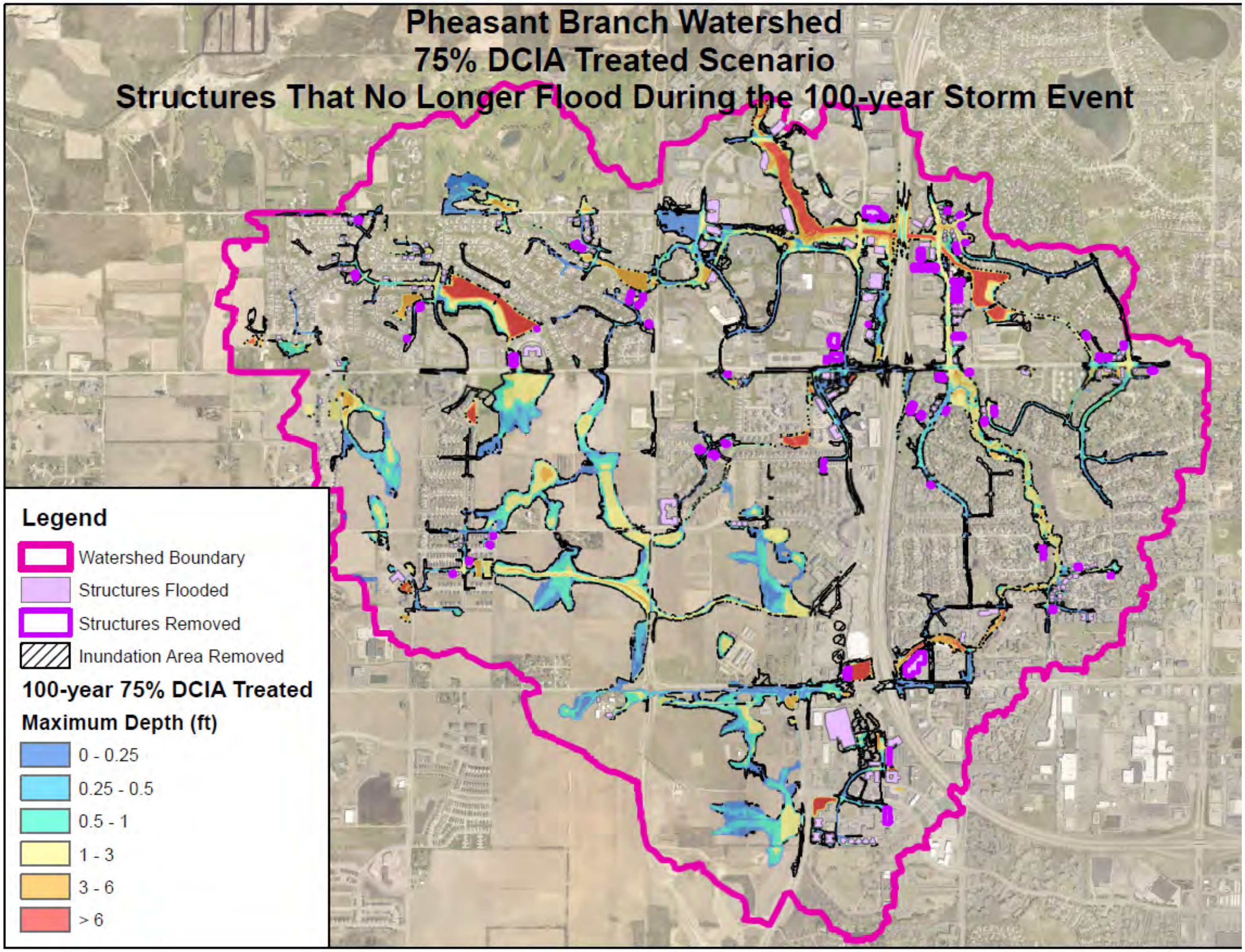

Figure 4-4. 75% DCIA Treated Scenario, 100-year Storm Event

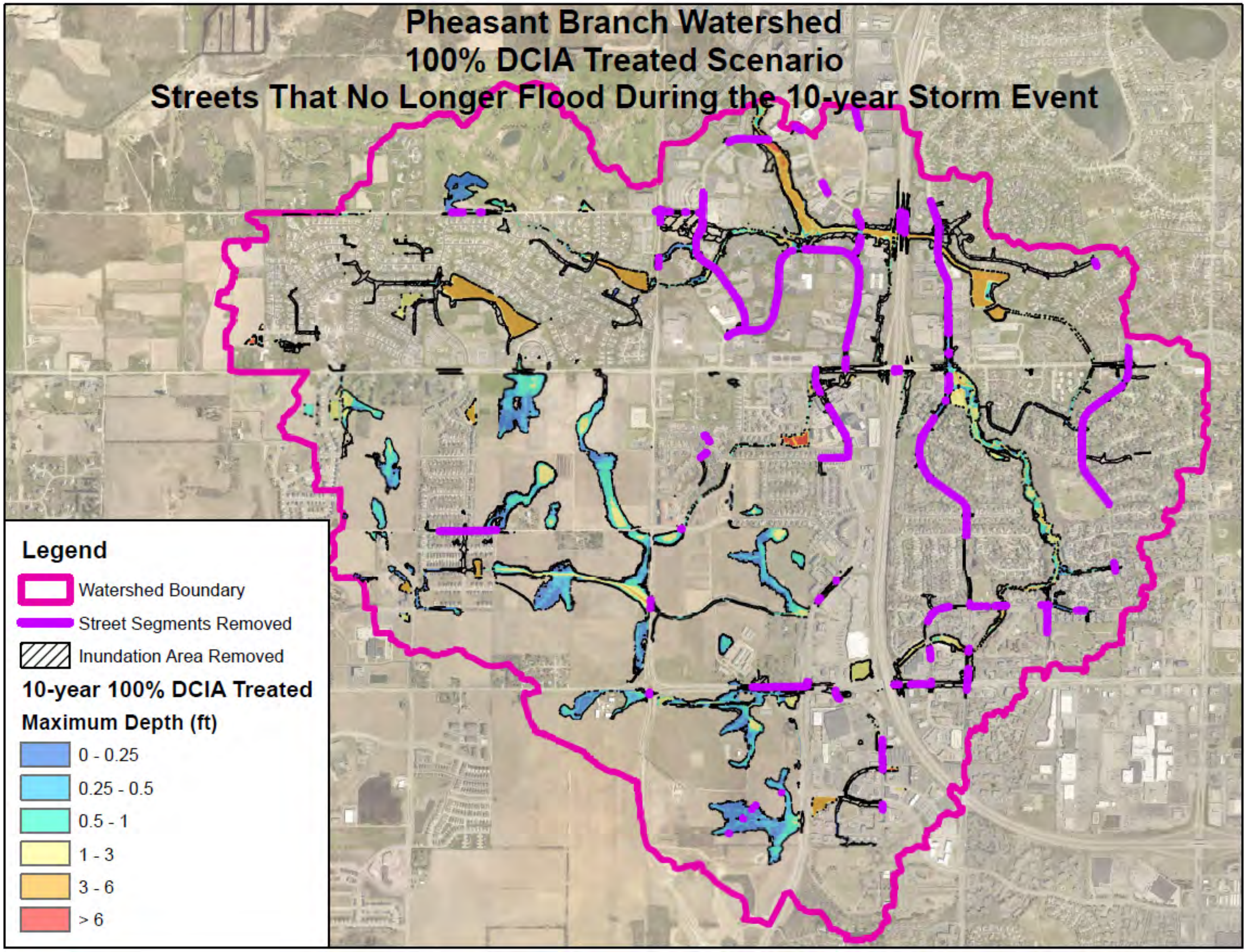

Figure 4-5. 100% DCIA Treated Scenario, 10-year Storm Event

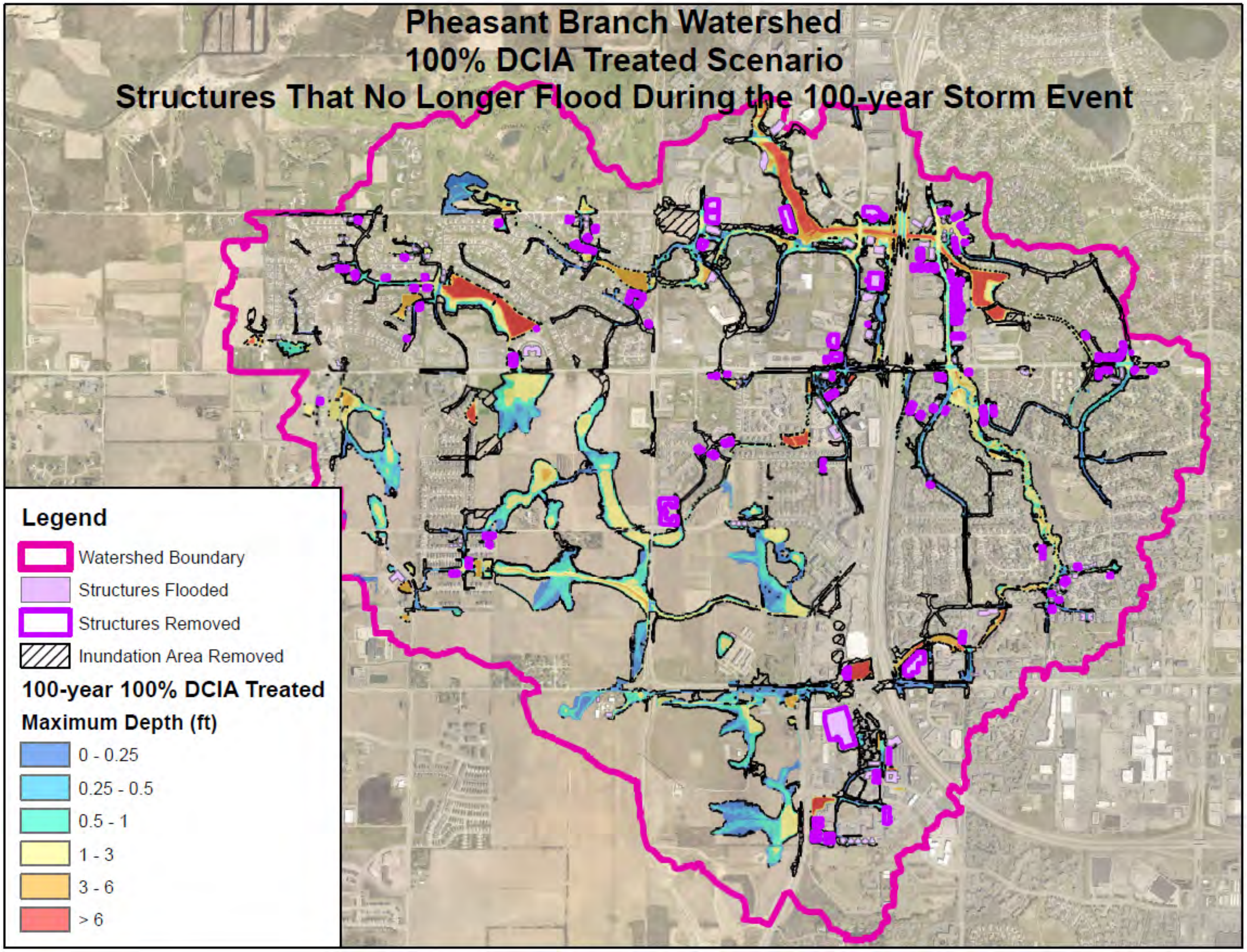

Figure 4-6. 100% DCIA Treated Scenario, 100-year Storm Event

Part 2 – Expand the Opportunities Analysis

The opportunities analysis conducted in Phase I was expanded to include non-residential flat roofs. The analysis was also updated to separately consider pervious areas, streets, and sidewalks available for volume control infrastructure.

The analysis was mostly the same as before except that the three foot buffer was only excluded along parcel boundaries that have residential land use on both sides (instead of along all parcel boundaries). Also, the universal three foot buffer was changed so that there is a three foot buffer around driveways only, and a one foot buffer around the other types of impervious. The potential pervious areas and sidewalks were then calculated separately, and slopes greater than 12 percent were not excluded for pervious areas nor sidewalks.

To find the streets that could potentially be pervious, all of the areas that were excluded as described above were removed from the street polygons along with large areas (greater than 1000 ft^2) of street that had a slope greater than 12 percent.

The area of available space for volume control infrastructure in the Pheasant Branch Watershed for this revised analysis was calculated to be: 2,009 acres.

Figure 4-7 shows the area available for volume control infrastructure.

Note, this is a planning level, desktop analysis. Each site needs a detailed investigation before volume control infrastructure could be installed.

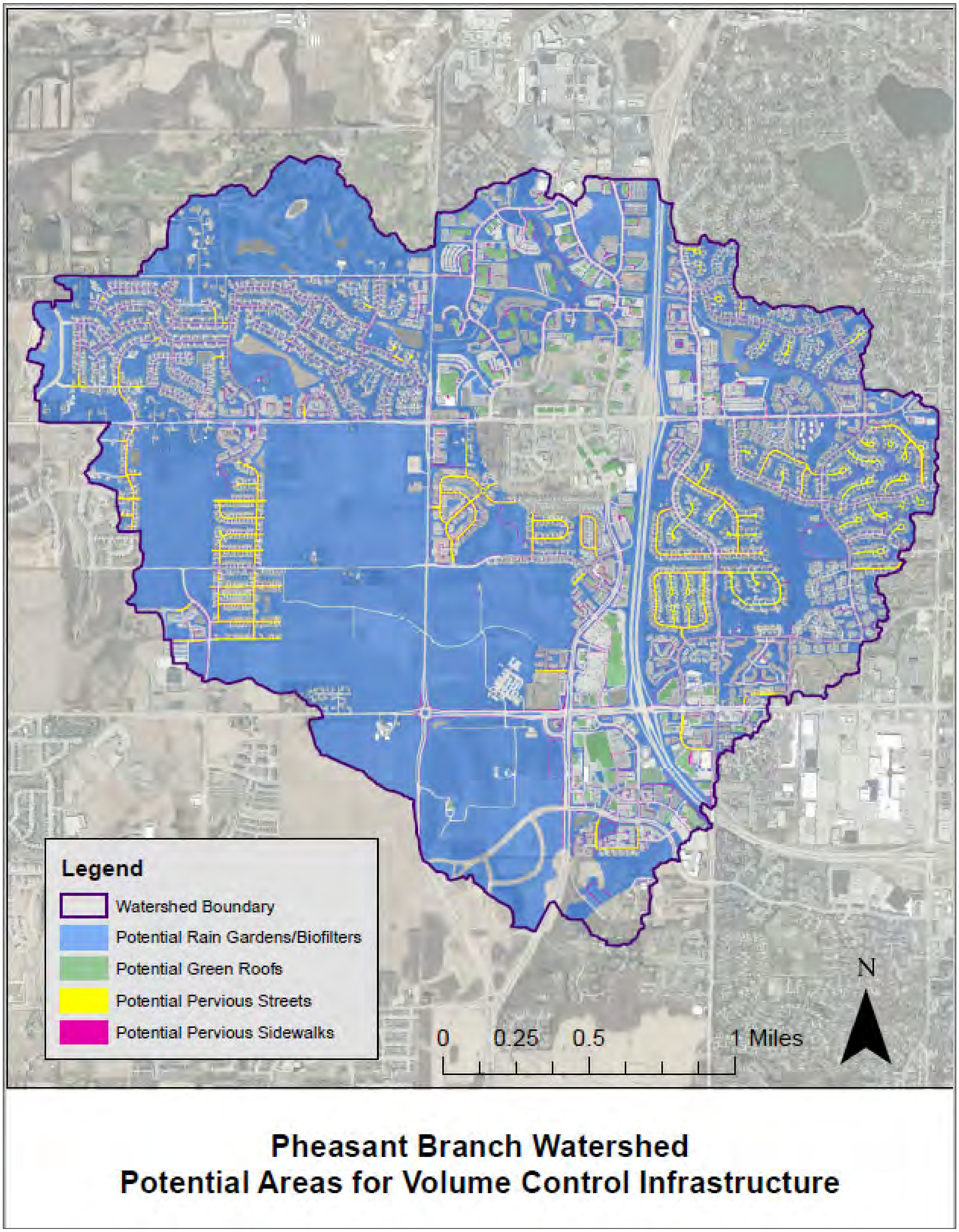

Figure 4-7. Potential Areas for Volume Control Infrastructure

Part 3 –Amount of Land Needed for VCI

Using the potential volume that could be infiltrated from Part 1, the area needed to capture that volume was calculated for each subcatchment, for each of the three scenarios. An assumed depth of 1-foot (open area storage) with an additional foot of rock-filled depth with a porosity of 0.30 was assumed for each VCI. Tab “Step 3 Area Needed for GI” includes the calculation to determine the area for each subcatchment.

Because numbers in a table can be tough to visualize, the area needed was rendered. Figures 4-8, 4-9, and 4-10 show the area needed to treat 25% of the DCIA, 75% of the DCIA, or 100% of the DCIA for each subcatchment.

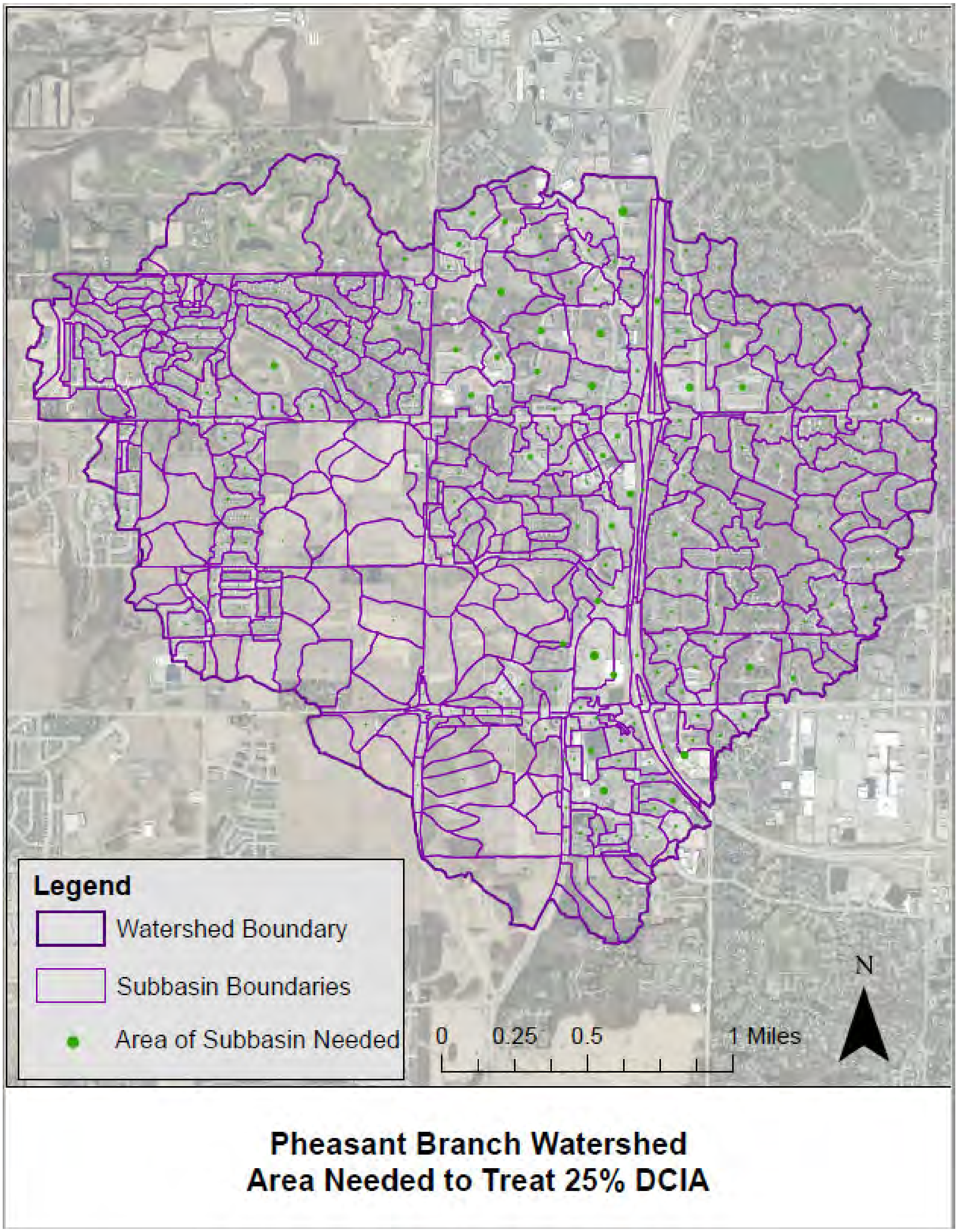

Figure 4-8. Area of Each Subbasin Needed to Treat 25% DCIA

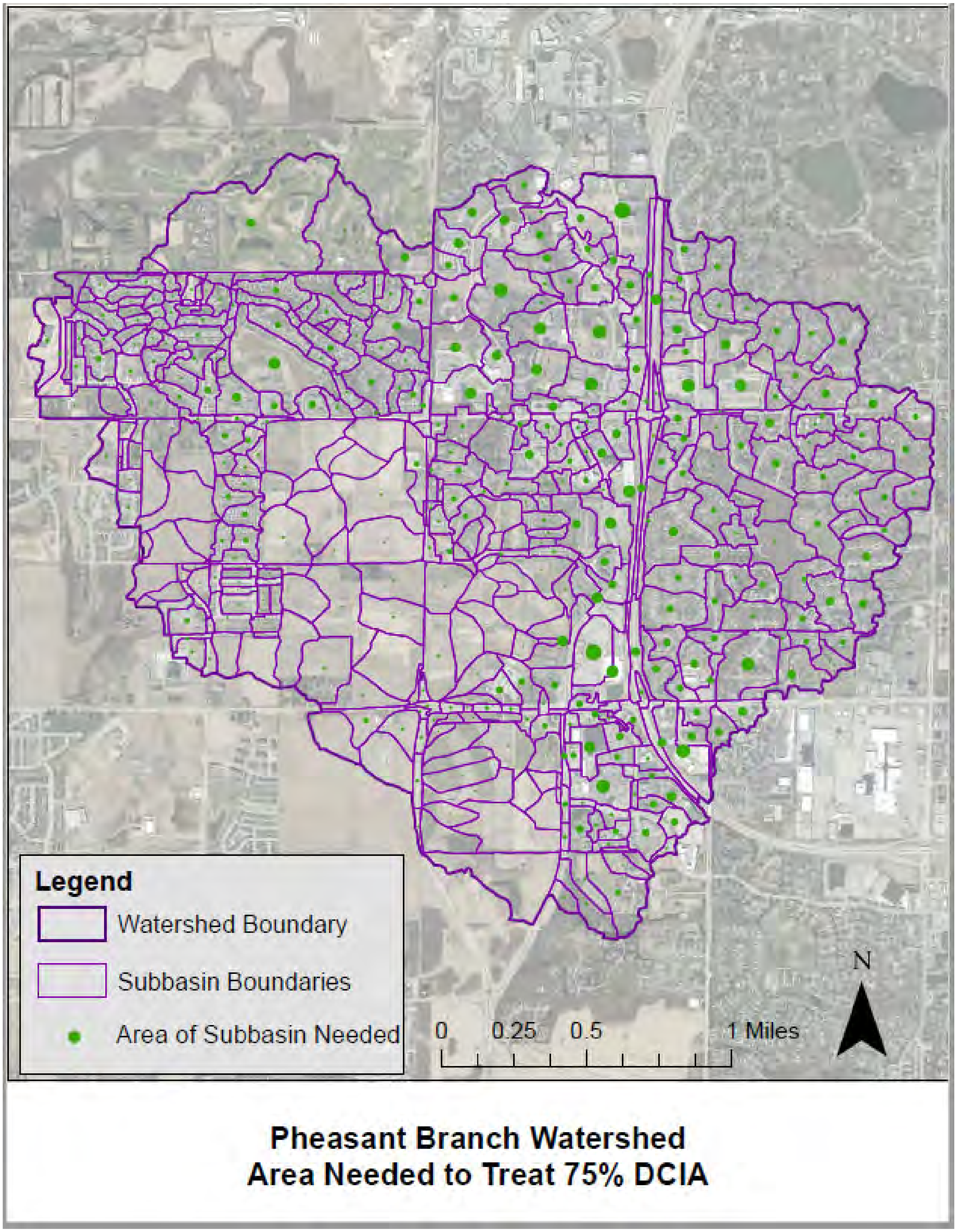

Figure 4-9. Area of Each Subbasin Needed to Treat 75% DCIA

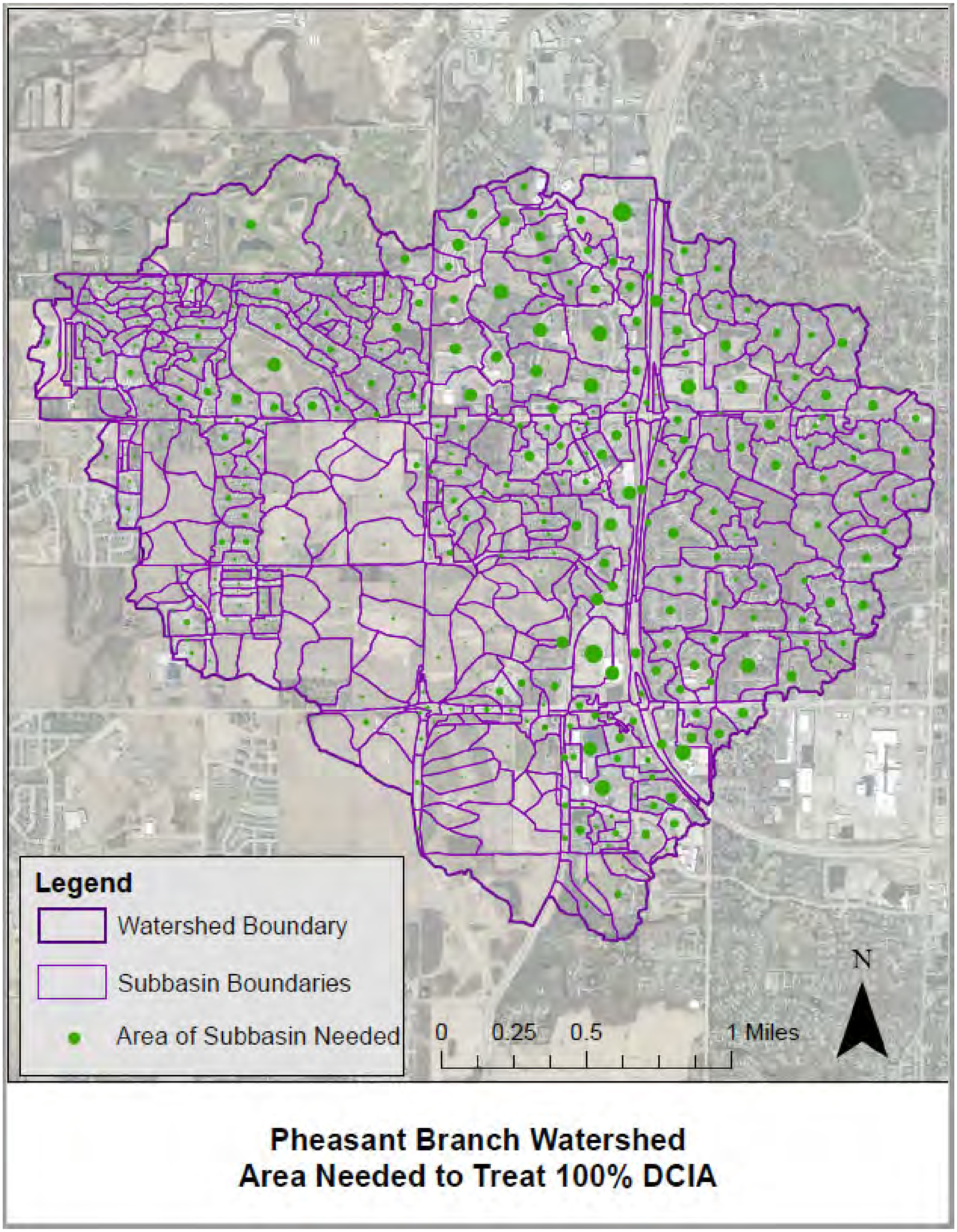

Figure 4-10. Area of Each Subbasin Needed to Treat 100% DCIA

Phase II Conceptual Cost Estimate

Utilizing the unit costs from Phase I, the construction and maintenance costs were estimated for all scenarios. A unit cost for construction of $40 per square foot was used. A unit cost of $3 per square foot was used for maintenance costs.

Table 4-3 shows the results of the conceptual cost estimate.

Table 4-3. Phase II Conceptual Cost Estimates

Scenario	Total Impervious Area Treated (acres)	Total Treatment Area (ac)	Estimated VCI Construction Cost ($)	Estimated VCI Maintenance Cost ($/yr)
25% DCIA Treated	221.4	28.1	$49,025,000	$3,677,000
75% DCIA Treated	664.2	84.4	$147,076,000	$11,031,000
100% DCIA Treated	885.6	112.5	$196,101,000	$14,708,000

Phase II Total Phosphorus Load Reduction Estimate

As noted in Phase I, volume control infrastructure can be constructed for several reasons, including stormwater pollution reduction. Using WinSLAMM, the City estimated that the total phosphorus reduction from the 25% DCIA Treated scenario is approximately 382 pounds per year, the total phosphorus reduction from the 75% Treated scenario is 1,141, and the total phosphorus from the 100% DCIA Treated scenario is 1,528.

Section 5. Reduce Size of Traditional Infrastructure

Following the evaluation in Section 4, Engineering wanted to understand if the traditional flood control infrastructure (storm sewer, detention ponds, etc) could be downsized and still meet flood control goals if the VCI was installed.

The 75% DCIA Scenario was selected for this exercise. This scenario was chosen because it results in a 20% reduction of peak flows at the watershed outlet and it was theorized that a 20% reduction in peak flow could equate to downsizing infrastructure by one size.

The hydrologic parameters from the 75% DCIA Scenario were entered into the computer model with the Ultimate Proposed Conditions solutions. The Ultimate Proposed Conditions solutions are all the infrastructure improvements that need to be made to meet the City's flood control goals, including regional detention, greenway improvements, and local sewer upgrades. The computer model with the traditional infrastructure and VCI was run and the results were summarized.

One-Size Infrastructure Reduction

Next the infrastructure in the Ultimate Proposed Solutions model was downsized by one size. For purposes of this exercise, one size is:

- Circular Pipe = 6 inches from the diameter
- Rectangular Box Culvert = 1 foot from the span, 1 foot from the rise
- Elliptical = 1 size in the Elliptical sizing chart, i.e. 53"x34" went to 49"x32"

The computer model with the downsized infrastructure was run and the results were summarized. This was then compared to the results from the Ultimate Proposed Conditions model. The comparison showed that the downsized infrastructure with the 75% DCIA scenario hydrology could be downsized and still meet results.

Two-Size Infrastructure Reduction

To understand how much the traditional infrastructure could be downsized, the traditional infrastructure was downsized by one more size. The computer model with the VCI and traditional infrastructure that was downsized by two sizes was run and the results were summarized. This time, the results showed that the flood reduction targets could not be met with the two-size down traditional infrastructure and the VCI.

Infrastructure Cost Reduction

The cost estimate for the Ultimate Proposed Conditions traditional infrastructure was compared to the cost of the one-size reduced infrastructure. The one-size reduced infrastructure is approximately $7 million dollars less than the Ultimate Proposed Conditions traditional infrastructure cost. The cost estimate for the 75% DCIA VCI scenario is approximately $147 million. Therefore, the reduction in cost from downsizing the traditional infrastructure in the Ultimate Proposed Conditions solution will not offset the cost of installing VCI.

Section 6. Additional Benefits of Volume Control Infrastructure

Volume control infrastructure has more benefits than traditional grey infrastructure (peak flow control) solutions. The Dane County Office of Energy and Climate Change published a paper in 2020 entitled "Green Infrastructure for Dane County: Benefits Add Up." This paper describes the type of volume control infrastructure, its benefits and challenges, and provides cost information. References from the paper have been included at the end of this document for ease of reference.

Page 3 of the report describes some major findings in the study, including:

1. A green roof installation cost is about $100 per month less than that of the conventional roof considering their lifespans of 50 years and 20 years respectively (GSA, 2011).
2. Employee's health and productivity are strongly linked to the offices' green environment with Green Infrastructure (National Geographic, 2017).
3. A green roof increases solar panels' efficiency to generate electricity by cooling solar panels' micro-climate, ranging from 3 percent to 16 percent (BuildingGreen 2013).
4. Pervious pavements will decrease about 77 percent of annual salt usage during winter maintenance (County Materials Corporation, 2016).

Section IV includes a long list of benefits including:

- Environmental
  - Increases climate resiliency
  - Improves Water Quality
  - Improves Air Quality
  - Reduced Urban Heat Island Effect
  - Less Salt Use Reduce Water Pollution
  - Enhanced Biodiversity/ Habitat Connectivity
- Societal Benefits
  - Improved Health
  - Health Equity in Community Engagement
  - Reduced Crime Rate
  - Improved Aesthetics
  - Increased Public Education with More Exposure to Green Infrastructure and Nature
  - Social Cohesion
  - More Green Jobs
- Economic Benefits
  - Increased Property Value
  - Increased Employee Productivity
  - Increased Green Jobs
  - Energy Savings from GI with Efficiency Measures

Details regarding each benefit can be found in the Dane County Office of Energy and Climate Change. Many other similar publications are available that document the non-stormwater benefits of green infrastructure.

Section 7. City of Madison Initial Agency Comments

As part of the watershed study efforts, Engineering staff met with various agency staff to discuss the proposed peak flow control solutions. During these meetings, green infrastructure was touched on by some agencies. Below are the initial comments. This section will be expanded as more meetings occur specifically regarding implementation of volume control infrastructure citywide.

Traffic Engineering

Some streets in the City of Madison do not lend themselves to volume control infrastructure. Examples include densely urbanized, space-limited, traffic corridors with large traffic counts, such as Atwood Avenue and Winnebago Street.

Water Utility

Water utility staff are supportive of volume control infrastructure in pervious areas and sidewalks. Pervious pavement in streets can cause challenges for water utility infrastructure. Water mains are generally older and have separation requirements from stormwater systems. Installation of pervious pavement could cause watermain replacement and realignment where it would not have been required with typical pavement.

Additionally, water utility wells have a mandatory 400 foot buffer around any well that exempts construction of a stormwater control measures. Beyond this buffer are the wellhead protection zones where infiltration is prohibited for purposes of protecting groundwater quality. Stormwater control measure in the wellhead protection zones require a clay liner, thus allowing no infiltration.

These areas were removed from the opportunities analysis early on in this study.

Parks

In the recent past, City of Madison Parks has installed green infrastructure where possible on Parks projects. Parks is generally supportive of green infrastructure installation.

Fire Department

Engineering discussed use of volume control infrastructure, specifically pervious pavement, in the Fire Lanes with the Madison Fire Department. Some equipment the Fire Department uses, including the ladder trucks, require a strong base to set the equipment supports on. If pervious pavement is to be used in these areas, the Fire Department requests that the load capacity of the pervious pavement is reviewed to ensure Fire Department equipment can be used safely.

Section 8. Results of Parallel Watershed Phase I Analysis

The City of Madison chose two watersheds currently under study to conduct a modified Phase I Analysis. The purpose of the analyses was to understand if watersheds with different development characteristics found the same results as was found for the Pheasant Branch Watershed Phase I Analysis. The scope for the analyses conducted by the watershed study consultants can be found in Appendix C.

Wingra West Watershed

Wingra West Watershed is a 1,763-acre, fully urbanized watershed with a large detention area in approximately the center of the watershed's stormwater conveyance system. Development occurred in this watershed from the 1930's to the 1970's, before many of the City's stormwater management requirements were put in place. This watershed was selected due to the large detention area as opposed to the distributed detention in the Pheasant Branch Watershed. The intent is to understand if that has an effect on the Phase I results.

The analysis found similar results to those found for Pheasant Branch. Some areas had larger peak flow reductions depending upon the storm event and some had smaller peak flow reductions. The outlet of the Wingra West Watershed found a 6.8% reduction in the 100-year peak flow for the 25% DCIA scenario. This is very similar to the 5% found for the Pheasant Branch Watershed.

Details of the Phase I Analysis for the Wingra West Watershed can be found in Appendix D.

Willow Creek Watershed

Willow Creek Watershed is a 1,953-acre, fully urbanized watershed with very few detention areas. Development occurred in this watershed from the 1930's to the 1970's, before many of the City's stormwater management requirements were put in place. This watershed was selected due to the lack of detention areas as opposed to the distributed detention in the Pheasant Branch Watershed.

Additionally, the hydrologic modeling uses for this watershed uses a rain-on-grid method as opposed to the traditional hydrology method of delineating subcatchments and flow paths of the Pheasant Branch and Wingra West Watershed Studies.

The intent is to understand if the differences have an effect on the Phase I results.

The analysis found similar results to those found for Pheasant Branch with a few exceptions. Some areas had larger peak flow reductions depending upon the storm event and some had smaller peak flow reductions. Additionally, some areas saw significance increases in peak flow (the Pheasant Branch and Wingra West Watersheds saw minor increases a few locations, but none were of the magnitude found in the Willow Creek Watershed.) The consultant conducting this analysis indicated the results for these locations are due to the instability in the model calculations in these areas. Therefore, these specific results can be ignored for purposes of this analysis.

Details of the Phase I Analysis for the Willow Creek Watershed can be found in Appendix E.

Section 9. Additional Analysis – Pervious Area Infiltration Rates

Engineering met to discuss the results from Section 4 and 5. Following the discussion, Engineering conducted further analysis. The first analysis involved evaluating the significance of pervious infiltration rates on peak flow reductions. The infiltration rate parameters for the watershed studies generally used the parameters for turf areas in urban environments. Many of the pervious volume control infrastructure will be planted with deep rooted vegetation. Studies have found that deep routed vegetation significantly increase the infiltration rates of the infrastructure (Selbig, W.R., and Balster, Nicholas, 2010).

Two alternatives were evaluated for the 75% DCIA Scenario. The first alternative assumed all pervious areas were planted with deep rooted vegetation, regardless of if it is volume control infrastructure. The second alternative assumed only the pervious volume control infrastructure was planted with deep rooted vegetation. To simulate the deep routed vegetation, the Maximum and Minimum Infiltration Rates for the Horton Infiltration Parameters were doubled.

Tables 9-1 and 9-2 shows the results of the alternatives evaluation.

Table 9-1. 75% DCIA Scenario with Deep Routed Vegetation for All Pervious Area

		Existing Conditions			**75% DCIA Treated**						**75% Modified DCIA - All Pervious Area has Roots that Promote Infiltration**					
					1-yr		**10-yr (cfs)**		**100-yr (cfs)**		**1-yr**		**10-yr (cfs)**		**100-yr (cfs)**	
Recording Location	**Location**	**1-yr (cfs)**	**10-yr (cfs)**	**100-yr (cfs)**	**Peak Flow (cfs)**	**Difference (%)**	**Peak Flow (cfs)**	**Difference (%)**	**Peak Flow (cfs)**	**Difference (%)**	**Peak Flow (cfs)**	**Difference (%)**	**Peak Flow (cfs)**	**Difference (%)**	**Peak Flow (cfs)**	**Difference (%)**
1	Outlet of Swallowtail Pond crossing Pleasant View Road	47	68	359	36	24%	48	29%	294	18%	9	80%	50	26%	185	48%
2	Target Pond Outlet (3 culverts) West to East Under Beltline	166	292	468	63	62%	164	44%	396	15%	38	77%	129	56%	299	36%
3	Culverts East to West under Beltline just downstream of Wexford Pond	260	646	1,435	110	58%	419	35%	1,109	23%	27	90%	255	61%	808	44%
4	Watershed Outlet	228	744	2,114	175	23%	515	31%	1,699	20%	103	55%	234	68%	1,122	47%

Table 9-2. 75% DCIA Scenario with Deep Routed Vegetation for Pervious Volume Control Infrastructure Only

		Existing Conditions			75% DCIA Treated						75% Modified DCIA - Pervious GI has Roots that Promote Infiltration					
					1-yr		10-yr (cfs)		100-yr (cfs)		1-yr		10-yr (cfs)		100-yr (cfs)	
Recording Location	Location	1-yr (cfs)	10-yr (cfs)	100-yr (cfs)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)	Peak Flow (cfs)	Difference (%)
1	Outlet of Swallowtail Pond crossing Pleasant View Road	47	68	359	36	24%	48	29%	294	18%	39	17%	53	22%	291	19%
2	Target Pond Outlet (3 culverts) West to East Under Beltline	166	292	468	63	62%	164	44%	396	15%	60	64%	157	46%	392	16%
3	Culverts East to West under Beltline just downstream of Wexford Pond	260	646	1,435	110	58%	419	35%	1,109	23%	99	62%	412	36%	1,091	24%
4	Watershed Outlet	228	744	2,114	175	23%	515	31%	1,699	20%	160	30%	507	32%	1,664	21%

The results of the analysis show a minor increase in the reduction of peak flow rates for the alternative where deep rooted vegetation is only evaluated in the pervious volume control infrastructure. The alternative with all pervious vegetation assumed to have deep roots provided a significant reduction in peak flow rates compared to the 75% DCIA Base Scenario, in some instances, almost doubling the reduction.

Section 10. Green Infrastructure Pilot Study with USGS

The City of Madison, in conjunction with the United States Geological Survey (USGS), started a Green Infrastructure Pilot Study in late 2019. Volume control infrastructure was installed with the Toepfer, Holly, Euclid, and St. Clair Street Reconstruction Project completed in late 2020. The following volume control infrastructure was installed:

- Pervious Pavement: installed on the south side of Euclid Ave and St Clair St
- Stormwater Terraces: There are different options for stormwater terraces, and the treatments are rated as a bronze, silver or gold treatment, depending on the anticipated impact of the treatment. All Stormwater Terraces will help improve water quality in the lakes, as well as reduce flooding.
- Rock Cribs: Sited in suitable terraces to infiltrate driveway and sidewalk stormwater runoff. These include pervious pavement and a buried section of rocks with grass over the top.
- Pervious sidewalk panels: Installed on the sidewalk between the driveway and street where rock cribs were installed.

The above is treating 15% of the DCIA in one of the Pilot Study subcatchments.

Figure 10-1 shows the location of the project and the volume control infrastructure.

The USGS began monitoring the pilot study area in late 2019. Monitoring equipment was set up in four locations; 3 locations to measure the volume reduction from the volume control infrastructure, and one as a control site. Fully automated, continuously recording, rain gages and flow monitoring equipment was installed at the four sites.

The initial monitoring of the Pilot Study area shows no measurable reduction in runoff volume at the outlet. Monitoring will continue for the next 4 years to monitoring changes of runoff volume in the subcatchments.

More information on the Pilot Study can be found here:
https://www.cityofmadison.com/engineering/projects/green-infrastructure-study

Figure 10-1. Green Infrastructure Pilot Study Infrastructure Types and Locations

Section 11. Conclusions and Next Steps

Traditionally-sized green infrastructure can be used for flood control, however, it takes a significate amount of it to be successful. Additionally, even though it can be used to help with flood control, traditional infrastructure is also needed to meet the City's watershed study flood targets.

The amount of directly connected impervious area treated to make a measurable difference in peak flows for large storm events is about 75%. This means that 75% of the DCIA in a watershed must be treated with green infrastructure. DCIA makes up a mix of roofs, driveways, sidewalks, and streets.

- In the Medium Density Residential No Alleys (MDRNA) standard land use, roofs are approximately 19 percent, streets make up approximately 53 percent, sidewalks make up approximately 5 percent, and driveways make up approximately 23 percent.
- In the Strip Mall Commercial (STRIPCOM) standard land use, roofs are approximately 22 percent, streets make up approximately 22 percent, sidewalks make up approximately 5 percent, driveways make up approximately 2 percent, and parking makes up approximately 45 percent.

Need to Treat More than Private Property

Table 11-1 below show the average make up for each major land use. The detailed breakdown for each standard land use can be found in Appendix A.

Table 11-1 DCIA Source Area Breakdown per Major Land Use

Source Area	Residential	Institutional	Commercial	Industrial	Other Urban
Roofs	23%	32%	27%	31%	6%
Parking	9%	30%	48%	51%	20%
Driveways	17%	6%	3%	3%	25%
Sidewalks	6%	4%	4%	1%	6%
Streets	45%	17%	17%	14%	40%

This means that for residential and other urban land uses (parks) land uses, the streets will need to be treated if the City wants to achieve the goal of treating 75% DCIA. For commercial, institutional, and industrial areas, if all other DCIA is treated, it is possible to reach the goal without treating the streets.

Next Steps

The next step in this study is to take these results to the various City agencies and the Mayor.

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Appendix A. WinSLAMM Standard Land Use Source Area Breakdown

WinSLAMM Standard Land Use Source Area Breakdown

	Residential										Institutional			Commercial				Industrial		Other Urban		
StandardLandUseCode	SUBR	HDRWA	HDRNA	MOBH	DUPLEX	MDRNA	MDRWA	MFR	LDR	HRR	INST	SCH	HOSP	OFFPARK	STRIPCOI	DOWNTO\	SHOPCEN	LI	MI	CEM	OPEN	PARK
StandardLandUseDescription	Suburban I	High Densi	High Densi	Mobile Hor	Duplex	Medium D(	Medium D(	Multi Famil	Low Densi	High Rise (	Misc. Instit	Schools	Hospital	Office Park	Strip Comr	Downtown	Shopping (	Light Indus	Medium In	Cemetary	Open Spac	Parks
Roof_FlatDirectlyConnected	0	0	0	16.9	0	0	0	3.4	0	19	5.39	15	31.8	13.17	19.7	40.73	21.61	20.51	16.8	0.55	0.55	0.1
Roof_FlatDisconnected	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.27	4.46	0	0	0
Roof_PitchedDirectlyConnected	0	13.6	10.3	0	4.48	4.5	3.2	14.1	1.9	0	8.44	0	3.92	0	3.7	0	0	2.57	1.85	0	0	0.11
Roof_PitchedDisconnected	2.6	10.6	11.1	0	12.06	10.5	13.6	3.2	6.1	0	0.58	0	0	0	0	0	0	0	0	0.55	0	0.25
PavedParking_DirectlyConnected	0	0.4	0	13.4	0	0.2	0.7	10.8	0.1	21.8	27.21	10.65	20.78	42.87	40.9	23.01	60.68	32.94	34.09	2.24	0	4.19
PavedParking_Disconnected	0.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
UnpavedParking_DirectlyConnected	0	0	0	0	0	0	0	0.5	0	0	0	0	0	0	0	0	0.34	0	0	0	0	0
UnpavedParking_Disconnected	0	0	0	0.6	0	0	0	0	0	0	0	0	0.59	0	1.4	0	0	6.34	14.61	0.07	0	0.22
Driveways_DirectlyConnected	1.6	0.7	14.1	12.3	5.31	5.6	1.6	1.9	3.2	0.9	3	1.98	6.12	4.62	2	1.48	1.81	2.56	1.8	7.67	0	1.21
Driveways_Disconnected	1.2	0	0	0	0	1.9	0.4	0.9	1.3	0	0	0	0	0	0	0	0	0	1	0	0	0
Sidewalks_DirectlyConnected	0	3.2	2	0.5	3.96	1.1	3	2.1	0.35	3.4	2.2	2.91	2.8	1.36	4.3	8.35	0.54	1.28	0.45	0.06	0.58	0.49
Sidewalks_Disconnected	0.1	3.2	2	0.5	0	1.1	2.9	2.1	0.35	1.2	0	0	0	0	0	0	0	0	0.45	0	0	0
Streets1_Area	0.7	5.4	4.5	0.6	0	3.7	0	6.1	2.2	9.1	4	2.68	4.24	3.11	11.5	19.96	4.22	1.84	1.13	0.72	1.72	1
Streets2_Area	3.3	13.2	9	3	13.31	7.6	13.5	8.5	4.2	4.5	6.76	4.33	6.64	8	8.6	2.21	2.17	8.68	6.2	0.72	2.02	2.27
Streets3_Area	0	1.6	0	0	0	1.5	2.2	0	0.6	4.5	1.32	1.64	0	0	0	0	0	0.32	0.22	0	0	0
LargeLandscapedArea	0	0	0	0	0	0.2	0	1.4	0	0	5.34	22.09	5.32	5.82	0	0	0	3.51	2.81	86.4	0.59	77.95
SmallLandscapedArea	84.8	41.5	41	44.7	60.88	57.5	52.5	38	74.8	22.9	26.55	17.43	14.05	18.19	5.8	3.56	4.53	9.86	4	0.23	0	0.85
UndevelopedArea	5.5	0.3	0	4.5	0	0.4	0	3	4.4	9.5	1.83	0.42	0.19	0	0.2	0	2.93	4.34	5.37	0.48	94.54	0
Playgrounds_DirectlyConnected	0	0	0	0	0	0	0	0	0	0	1.7	17.33	0	0	0	0	0	0	0	0	0	0.9
Playgrounds_Disconnected	0	0	0	0	0	0	0	0.1	0	0	1.7	0	0	0	0	0	0	0	0	0	0	0.9
IsolatedAreas	0.1	0	0.1	1	0	0.2	0.1	0.1	0.2	0	0	0	1.05	0	0	0	0	0	0	0.28	0	7.08
WaterBodyAreas	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
OtherPerviousAreas	0	6.3	5.9	0	0	4	6.3	3.8	0.2	3.2	2.65	2.19	2.5	2.29	1.9	0.62	0.82	2.77	4.53	0	0	0
OtherDirectlyConnectedImperviousAreas	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0.35	0	0	0	0	0
OtherPartiallyConnectedImperviousAreas	0	0	0	0	0	0	0	0	0.1	0	1.33	1.35	0	0.57	0	0.08	0	0.21	0.23	0.03	0	2.48
Summary																						
Directly Connected Impervious Area	5.6	38.1	39.9	48.7	27.06	24.2	24.2	47.4	12.55	63.2	60.02	56.52	76.3	73.13	90.7	95.74	91.72	70.7	62.54	11.96	4.87	10.27
Disconnected Impervious Area	4	13.8	13.1	1.1	12.06	13.5	16.9	6.3	7.85	1.2	3.61	1.35	0.59	0.57	1.4	0.08	0	8.82	20.75	0.65	0	3.85
Total Impervious Area	9.6	51.9	53	49.8	39.12	37.7	41.1	53.7	20.4	64.4	63.63	57.87	76.89	73.7	92.1	95.82	91.72	79.52	83.29	12.61	4.87	14.12
Pervious Area	90.3	48.1	46.9	49.2	60.88	62.1	58.8	46.2	79.4	35.6	36.37	42.13	22.06	26.3	7.9	4.18	8.28	20.48	16.71	87.11	95.13	78.8
Isolated Area	0.1	0	0.1	1	0	0.2	0.1	0.1	0.2	0	0	0	1.05	0	0	0	0	0	0	0.28	0	7.08
Total	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
Source Area DCIA Breakdown																						
Roofs	0%	36%	26%	35%	17%	19%	13%	37%	15%	30%	23%	27%	47%	18%	26%	43%	24%	33%	30%	5%	11%	2%
Parking	0%	1%	0%	28%	0%	1%	3%	23%	1%	34%	45%	19%	27%	59%	45%	24%	66%	47%	55%	19%	0%	41%
Driveways	29%	2%	35%	25%	20%	23%	7%	4%	25%	1%	5%	4%	8%	6%	2%	2%	2%	4%	3%	64%	0%	12%
Sidewalks	0%	8%	5%	1%	15%	5%	12%	4%	3%	5%	4%	5%	4%	2%	5%	9%	1%	2%	1%	1%	12%	5%
Streets	71%	53%	34%	7%	49%	53%	65%	31%	56%	29%	20%	15%	14%	15%	22%	23%	7%	15%	12%	12%	77%	32%

Appendix B. Phase I Peer Review

MEMO

To:	Caroline Burger, Greg Fries, and Janet Schmidt
Cc:	Juli Beth Hinds
From:	Dan Christian and Jim Bachhuber
Date:	October 8, 2020
Subject:	Distributed Green Infrastructure Analysis Review

1.0 INTRODUCTION

1.1 BACKGROUND INFORMATION

Watershed studies for purposes of flood control are being conducted across the City of Madison. Although these studies evaluate a variety of flood control approaches for their performance, the City is particularly interested in the role green infrastructure (GI) can play in stormwater flood control. There is no universal definition of green infrastructure, however the term is generally characterized as non-regional stormwater management measures that are located close to high runoff sources, and promote storage, infiltration and/or evapotranspiration of stormwater. The City conducted a modeling analysis of the impacts of GI utilizing the Pheasant Branch Watershed as a test area. The analysis showed that GI reduced the peak flow from the 1 percent AEP by 4% to 5% at the outlet of the watershed.

The City plans on providing a GI analysis approach for the consultants conducting the first three watershed studies (Spring Harbor, Stricker's/Mendota, and Wingra West) to utilize in their analyses. After studying results from the first three watersheds, the City will determine if this analysis should be conducted in other watersheds being modeled for flood control purposes

1.2 REVIEW OBJECTIVE

City Engineering conducted a pilot planning level analysis of GI performance for the Pheasant Branch Watershed. The purpose of the analysis was to understand the data available and how it could efficiently be used to evaluate the effectiveness of GI to reduce the peak flow during large storm events.

2.0 SCOPE

2.1 REVIEW THE GI ANALYSIS FOR PHEASANT BRANCH WATERSHED

Through an amendment to an existing contract, The City requested Birchline Planning LLC (and the subconsultants Tetra Tech and Jim Bachhuber) to review the modeling analysis information provided by the City (7/14/20) for the Pheasant Branch Watershed. Specifically, the scope of work required a review and comment on the following files:

- GI Analysis 2020_07_03.xls
- GreenInfrastructureEffectivenessAnalysis_PheasantBranchWatershed_v2020_07_14.pdf
- Proposed VCI Scope v2020_07_14_for_Review.pdf

The review addressed the following questions:

1) Is the basis of the analysis technically sound based on current professional standards/methodologies to evaluate the effectiveness of green infrastructure on a large scale?
2) Is the analysis missing something? Should the City look at something additional during this analysis?
3) Is there something in the analysis that is not needed?

4) Does the proposed scope describe the analysis the City is asking for?
5) Is there a better way to do this type of analysis understanding the tools the City has? Tools include a green infrastructure suitability layer, XP-SWMM models using SWMM Hydrology and Horton Infiltration.

2.2 DISCUSS ALTERNATIVE DISTRIBUTED GI MODELING APPROACHES

Based on best professional judgement provide opinions on the additional issues listed below:

6) Within the watersheds being modeled in Madison, is there justification for analyzing multiple locations within a watershed for DGI impacts on large storm peak flow rates as opposed to only evaluating the peak flow rate at the outlet of the watershed? (NOTE: The City uses the term "distributed green infrastructure" – DGI – to describe the concept of using GI at multiple locations throughout a watershed.)
7) What are the benefits and limitations of modeling the "extreme DGI" scenario of converting all non-right-of-way (ROW) impervious surfaces to pervious? The purpose of the analysis is to determine if the City's existing stormwater detention and conveyance system is able to safely pass a target design storm peak flow even under the "extreme DGI" scenario.
8) Should the City evaluate stormwater management scenario(s) that looks at the amount of TSS and TP reduction that could be achieved in the areas where there is no current treatment?

3.0 TECHNICAL REVIEW

The focus of the review is the *Green Infrastructure Planning Level Analysis for Pheasant Branch Watershed* document (v07/14/2020) prepared by the City. The *GI Analysis 2020_07_03.xls* file was used as supplemental information to understand some of the technical details. The City refers to green infrastructure as "volume control infrastructure", which is meant to promote stormwater infiltration. The stated intent of the modeling analysis was to:

1) Determine the volume of water that would need to be managed if a certain percent of the directly connected impervious area (DCIA) were to be treated,
2) Determine if there is a break point or target for volume control treatment
3) Assess the watershed to see how much area is available for volume control,
4) Assess the impact of infiltration (from DGI) sized for the 1-year event on larger storms,
5) Assess the sensitivity of the existing drainage system to the implementation of volume control stormwater devices to treat directly connected impervious areas, and
6) Determine if the combination of the peak flow control efforts and volume control efforts can provide a quantifiable improvement in the level of service.

3.1 CITY'S APPROACH FOR ANALYZING DGI

The City's analysis was conducted in three parts.

Part 1. Determine the resulting flows after assuming complete retention of a certain fraction of DCIA. In this step selected percentages of DCIA (10%, 15%, 20% and 25%) were removed from each subcatchment in the model. For example, if a subcatchment had 1-acre of DCIA, the area would be decreased to 0.90, 0.85, 0.80, and 0.75-acres respectively.

Part 2. This part of the analysis identified potential locations for GI to be conceptually placed. The analysis utilized the City's Geographic Information System (GIS) and was conducted in in conjunction with various City agencies. For each land use type, three scenarios were developed (1) the total available space for GI (meaning 100% placement of GI in all areas. This scenario is not meant to reflect reality, but it represents an idealistic maximum level of control.), (2) the maximum practical space for GI, and (3) a realistic space for GI. Additionally, restricted areas were identified including wellhead protection zones, shallow bedrock, high groundwater and steep slopes. However, the restricted areas were not factored into the final analysis. The runoff reduction volume from Part 1 for a 1-year storm event was determined and converted to an equivalent total area needed (assuming 1-ft storage depth). The area needed was then compared to the available space determined through the GIS analysis.

Part 3. Using the hydrologic / hydraulic model (SWMM) , 25% of the DCIA for each subcatchment was moved to the non-directly connected impervious area (NDCIA) and depression storage was increased in the NDCIA portion of the subcatchment to offset the increased flow from the 1-year storm event. Hence, this simulates GI sized for the 1-year storm event for 25% of the DCIA. The model was then solved for a range of storm events, up to the 100-year event. An opinion of probable construction and maintenance costs were estimated based on a fixed unit cost. A total phosphorus load reduction was estimated using WinSLAMM.

3.2 MANAGING LARGE STORM PEAK FLOW WITH GI

Results of the City's analysis show a 5% peak flow reduction when rerouting 25% of the DCIA area to NDCIA, and a 4% reduction for the "Max DCIA Rerouted" (to NDCIA) scenario at the outlet of the Pheasant Branch Watershed for a 1% AEP (100-year) event. The principle reason for the relatively low percent removal during a large storm event is that the GI practice (represented by depression storage) is filled by the initial rainfall-runoff so that during the peak intensity of the rainfall the primary mechanism for reducing runoff is infiltration. The selected rainfall distribution plays a significant role in this. It should also be noted that the percent change in peak flow reported under the different scenarios are likely within the error range of the model itself. Hence this means the resulting values from the various scenarios may be considered as "no change" or very little change.

3.3 REVIEW COMMENTS ON THE APPROACH

The following comments and questions are offered as we worked through the analysis approach and results.

Removing Areas in the Model. Part 1 took an overly simplistic approach to determining the impact GI would have on the resulting flows. Removing the area from the model assumes that no runoff, regardless of the storm size, contributes flow to the collection system which is not realistic. However, this does provide a quick and simple first approximation and the approach was improved in Part 3. In the future, if the Part 1 approach is used, we recommend maintaining the total area of the subcatchment and instead shifting the DCIA to NDCIA or pervious area.

GI for DCIA Only. GI was only considered for DCIAs. Implementation of GI for NDCIA should also be considered. NDCIAs can still have a significant runoff contribution, particularly for large storm events. A common example of NDCIA would be a rooftop with disconnected downspouts discharging onto a lawn. A common GI practice to use in this scenario would be to covert a portion of the lawn to bioretention thereby increasing the storage and infiltration of the stormwater.

Figure 1 Example High Density Area

Types of GI Assumed. The memo did not state what types of GI practices were assumed to be applicable in the analysis. Based on the information presented, it appears that only a common bioretention was conceptually placed on existing vegetated areas. Hence no loss of impervious surface was anticipated. For example, a bioretention system in a bump-out along a road or in the corner of a parking lot was not considered which would reduce the impervious area and provide a level of management. Additionally, it does not appear that any type of porous pavement, tree boxes or green roofs were considered. Water harvesting practices could also be used in place of some of the bioretention areas. Leaching systems are another type of practice which might be considered. These may include discrete systems such as a leaching catch basin or subsurface storage vaults but may also include long linear piped systems designed to exfiltrate water. Leaching systems can be placed under impervious surfaces with no net loss or negative impact to the impervious surface. All these systems could be represented in the model.

The impact of only considering open green space for GI implementation is that dense commercial areas (e.g. southwest corner of Old Sauk Rd & W Beltline Hwy, shown in Figure 1) are determined to only have a relatively small amount of GI possible (<10% DCIA removed) and yet these types of development are exactly the area's most in need of improved stormwater management. We suggest documenting the analysis to explain the types of GI considered and under what circumstances. This documentation will also provide a consistent approach for subsequent watershed analysis with generic DGI application.

<u>25% DCIA Max Scenario</u>. Part 1 of the analysis limited the maximum amount DCIA removed to 25%. We were unsure why this limit was placed. Part 2 shows that even with the modest 10-25% DCIA reduction more space is needed than available and would suggest that a 25% reduction in DCIA may be physically not feasible. On the other hand, the reason not enough space is shown may be due to the limited types of GI assumed and if a wider range of GI practices was considered (i.e. the previously mentioned green roofs, impervious pavement, etc.) a larger amount of DCIA removal may be possible.

<u>25% DCIA Reduction Target for Pheasant Branch</u>. Part 2 Step 2 of the analysis (bottom of page 5 of the City's memo) states that the City chose a treatment goal of 25% of the DCIA for Pheasant Branch Watershed for two reasons, one of which was that overall, "land is available to treat 25% of the DCIA in the watershed." According the MS Excel worksheet *2 Step 5 Compare Areas* the cumulative area needed for the 1-year event with a 25% DCIA reduction is 1,958,970 square feet (column EK) and the cumulative area still needed (column EL) is shown in Table 1. Unless we're interpreting the information incorrectly, since the cumulative area still needed is greater than zero, this suggests that land is not available to treat the 25% DCIA reduction.

Table 1 25% DCIA Reduction, 1-year Event

Restrictions	GI Area	Cumulative Area Still Needed (sf)
More Restrictive	Realistic	435,108
Less Restrictive	Realistic	275,598
More Restrictive	Maximized	334,975
Less Restrictive	Maximized	208,655
More Restrictive	All Available	203,834
Less Restrictive	All Available	56,161

<u>Size GI for 1-year Event Volume</u>. Part 2 of the analysis initially considered the 1- and 10-year event volumes but then stated that "*future calculations would only be done for the 10-year storm*". If the objective is to manage the peak flow from severe storms, sizing stormwater management practices (including GI) for larger storm events would be beneficial. There is no assessment regarding the feasibility of "upsizing" the standard GI design to capture and hold a larger amount of runoff.

3.4 SCOPE QUESTIONS

The following specific questions were identified in the scope of work to be addressed (refer to Section 2.1 and 2.2). The questions and answers are provided below.

Question 1: Is the basis of the analysis technically sound?

Based on our opinion and professional judgement the methodology followed to evaluate DGI on a large watershed scale is technically sound. Specifically, the approach used in Part 3 of the analysis whereby depression storage on pervious surface was increased to represent an equivalent amount of DGI is a reasonable approach to use in a SWMM model.

Question 2: Is the analysis missing something?

As mentioned in our review comments, we suggest: 1) broadening the type of GI practices considered and 2) including GI for NDCIA. These two changes should result in GI practices possible on more land which then will lead to a reconsideration of the upper "treatable" limit (currently set at 25% of the DCIA).

The current approach focuses only on DGI practices. It is our understanding that the City has already assessed larger scale management measures (i.e. enhancing existing and/or adding new regional stormwater basins) and "grey infrastructure (i.e. larger storm sewers, re-routing, pumps, etc.). If this is the case, the City may wish to combine the "regional" approach, the "DGI approach", and grey infrastructure to assess flood benefits from a comprehensive management scheme. It is very likely that no one solution will achieve the flood control goals.

Green (or living) roofs do not appear to be considered in the analysis and would require a slightly different approach than relying on modeling them as pervious area depression storage. If green roofs are considered in the evaluation, a similar modeling approach would be to increase the impervious area depression storage by the equivalent storage amount provided by a typical green roof design. Although green roofs are designed to drain, during a discrete design storm the use of impervious area depression storage is a reasonable approximation.

We suggest expanding the way in which the benefits of GI are documented. Currently the primary measure of success is the expected peak flow reduction from the 1 percent AEP (100-year) discrete design storm with a MSE 4 distribution at the downstream end of the watershed. This is a very high standard to meet with DGI practices. The peak flow reduction at an intermediate point (for example conduit TO2255-024_TO2355-056 in the memo) was also quantified along with total phosphorus (TP) pollution reduction. We suggest processing the model with a continuous rainfall record and quantifying the reduced magnitude, frequency and duration of flooding as a result of the GI practices to document the improved resiliency of the system. Other additional benefits of GI (such as heat island benefits, pollution reduction, etc.) could be quantified as well for a holistic approach.

Question 3: Is there something in the analysis that is not needed?

Part 1 of the analysis was a good first step to begin to quantify the impact of distributed GI but is not needed moving forward. Quantifying the available space (Part 2) and modeling the impact of GI (Part 3) are still needed.

Question 4: Does the proposed scope describe the analysis the City is asking for?

Our review found the draft scope of work was technically sound to address the analyses. We suggest changing the scope approach to be consistent with the recommendations in this memo such as:

- Part 1 - remove
- Part 2 - expand the opportunity analysis to include other types of DGI and addressing GI management on NDCIA
- Part 3 - solve the model for fractions of the total the total DGI opportunities up to 100%.

We did not review the scope of work for its clarity to carry out the tasks, or legal aspects.

Question 5: Is there a better way to do this type of analysis understanding the tools the City has?

Based on our current review and our understanding of the City's objectives, we do not have a preferred alternative approach to recommended beyond the suggested changes discussed in this memo.

Question 6: Within the watersheds being modeled in Madison, is there justification for analyzing multiple locations within a watershed for DGI impacts on large storm peak flow rates as opposed to only evaluating the peak flow rate at the outlet of the watershed?

We suggest quantifying the benefits at multiple points within the watersheds. Portions of the watershed may be more amenable to GI practices than others and the results of this may not be apparent unless results at intermediate points in the watershed are considered. Key locations for discharge analysis may be identified based upon the drainage areas' density of land use and potential for DGI, important locations from a safety and/or emergency access perspective, or other factors.

Question 7: What are the benefits and limitations of modeling the "extreme DGI" scenario of converting all not right-of-way (ROW) impervious surfaces to pervious.

Modeling any extreme condition provides a boundary for the expected results. Understanding if the municipal collection system can handle just the road rights-of-way (ROWs) may be useful in understanding the scope of the problem. If the existing collection system cannot handle the runoff from the roadway, regardless of what is done on private property, it would then suggest that improvements within the ROW are required regardless of what is done outside of the ROW.

Question 8: Should the City evaluate stormwater management scenario(s) that looks at the amount of TSS and TP reduction that could be achieved get in the areas where there is no current treatment?

When selecting a preferred alternative, we believe it is important to equitably compare the options. This means quantifying a range of benefits, including water quality improvements, for all the options. However, since the focus of the analysis, and primary goal of the program is flood management (i.e. large events), a weighting system or prioritization value should be given to the flood control benefits over water quality.

4.0 IDEAS TO CONSIDER

4.1 MODIFIED GI DESIGN APPROACHES

Traditionally GI practices have been designed and constructed for water quality treatment purposes. For water quality treatment purposes, capturing and treating the initial runoff, and the more frequent storms are the most beneficial. If a primary objective is to manage the peak flow from large storm events, criteria for design and construction of GI practices should be revisited. Presumably the City still wants the water quality improvement benefits of the GI in addition to peak flow control. Some ideas to achieve both a water quality and peak flow control objectives include:

- Increase the size the GI practice to capture more water.
- A multistage outlet control structure whereby small storm events are fully captured, and large storm events receive peak flow control. This approach will require additional storage volume.
- A smart outlet control structure whereby discharge from the practice is automatically regulated by looking at forecasted rainfall and minimizes discharge during times of peak rainfall. Multiple smart outlets may be coordinated to stagger discharge in order to minimize the combined peak discharge.

4.2 ROAD CORRIDORS

Determining where GI practices can fit within roadway corridors is challenging. The City has conducted this approach over several years. One approach to consider is to group roadways into common classes and based on representative block(s) determine the amount of GI that could be constructed. Refer to Figure 2 for of an example GI quantity along a commercial street. The amount of GI can then be presented on a unit basis, e.g. square feet of GI per linear foot of roadway. Refer to Table 2 for an example of GI by typical street (note this is an example for a specific watershed study, not a generic case). The quantity of GI to include for modeling purposes can then be calculated within a GIS once the quality of different roadway typologies is known.

Figure 2 Example GI on a Commercial Street Section

Table 2 Characteristics of Typical Street Sections with Area of Practical Maximum GI

	Practice Area (sft)			ROW Width (ft)	Ex. Sidewalk width (ft)	Ex. Street Width (ft)	Street Length (ft)
Street Classification	Bioretention	Pervious Pavement	Planter Box (Lined)				
Alley		8,100		16			585
Commercial	4,900	11,100		66	5	46	956
Industrial			9,500	66	12	42	1218
Residential	4,400	13,000		66	5	30	722

4.3 ECOSYSTEMS SERVICES ANALYSIS

Another option to consider involves a more holistic approach by incorporating city goals beyond flood control. For example, a multifunctional benefits analysis involving issues such as infrastructure resilience, heat island, air quality, groundwater recharge, erosion, plant biodiversity, tree canopy, habitat, access to greenspace, landscape aesthetics, and social vulnerability. This type of analysis can demonstrate the true benefits of GI over constructing larger storm sewers to only increase conveyance. An analysis of this type often identifies the demand within the study area for various services (based on city goals) and pairs that with potential benefits provided by each type of stormwater management practice.

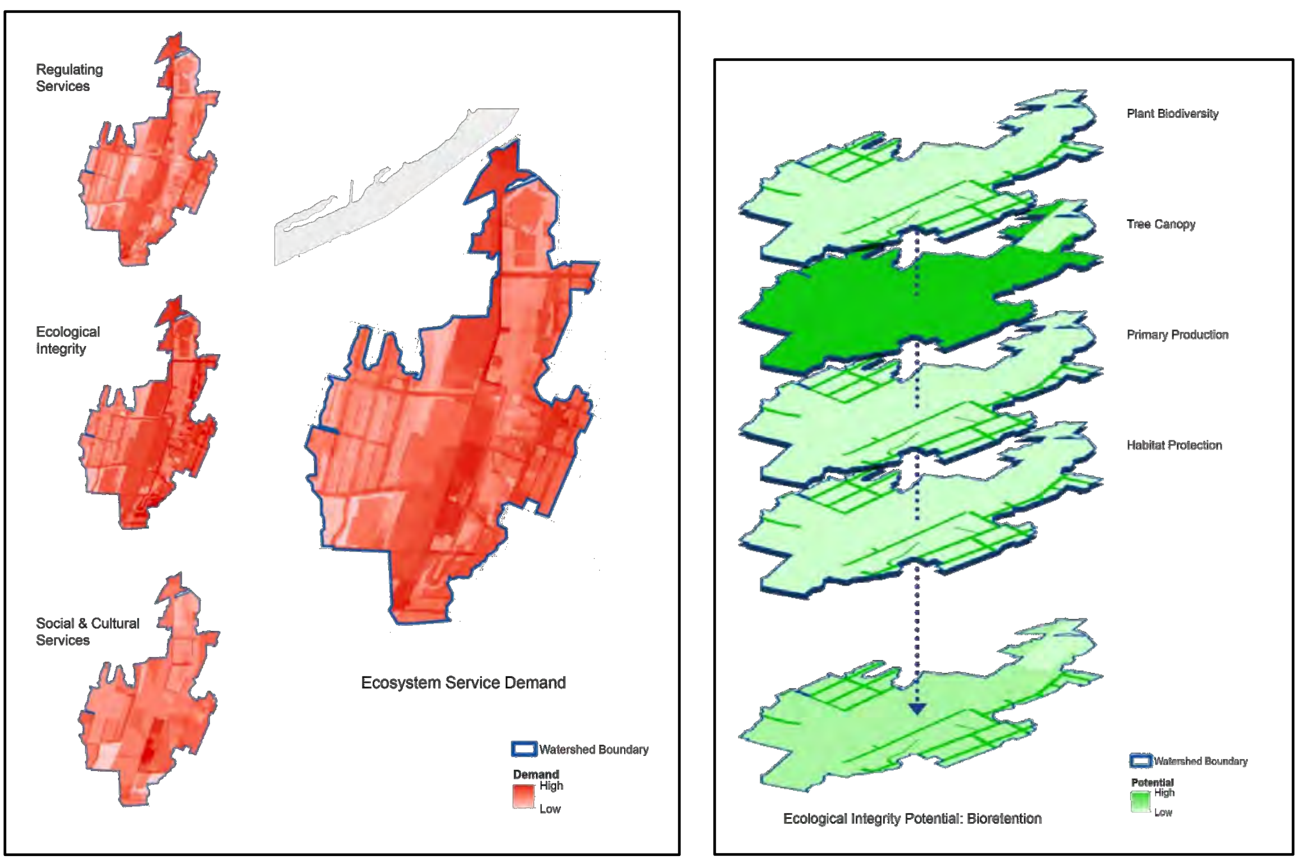


Figure 3 Example Ecosystem Services Analysis

Appendix C. Phase I Analysis Scope for Consultants

Watershed Studies – Phase I Volume Control Proposed Tasks
v10/08/2020

The City has conducted an initial analysis of the effect of distributed green infrastructure (volume control practices) in the Pheasant Branch Watershed. The intent of the analysis is to understand if sizing distributed green infrastructure in the traditional manner (generally the smaller, more frequent events) has an effect on the peak flows for the large storm events (> 10-yr recurrence interval).

The City requests this analysis be repeated in two more watersheds in the City to understand if the results are similar. The two watersheds chosen are Wingra West and Willow Creek.

a. Wingra West was chosen because it's fully urbanized (unlike Pheasant Branch) and it has a large storage area (Odana Hills Golf Course Ponds) in the middle.
b. Willow Creek was chosen because it's a fully urbanized watershed with little to no storage and is being done with rain-on-grid. If The City goes forward with more rain-on-grid, we want to see how the distributed GI analysis would be done with that approach.

The scope below describes Phase 1 of the distributed green infrastructure (volume control) analysis. There may be additional Phases depending upon the results of this analysis and the extended analysis the City is conducting in-house.

The City has also created a working spreadsheet that the Consultant may utilize for this effort if they deem appropriate.

Task 6.1 – Phase I Volume Control Analysis (VC)

<u>Step 1 – Create the models</u>

Determine the effect on peak flows for the 10-year and 100-year, 24-hour events if 1) 10% of the DCIA (Directly Connected Impervious Area) is routed to green infrastructure and 2) 25% of the DCIA is routed to green infrastructure.

To do this, move the %DCIA removed to NDCIA (Non-Directly Connected Impervious Area) for each subcatchment. Then, adjust the pervious area storage based on the assumed sizing of the distributed green infrastructure (DGI). For purposes of this exercise, when adjusting the pervious area storage assume: 1) the DGI has a depth of 1 foot of open air storage, 2) no engineered media, and 3) the porosity of the underlying native soil is not taken into account.

Create one model for the 10% DCIA removed and a second model for the 25% DCIA removed.

Run both models for the 1-yr, 10-yr, and 100-yr events and tabulate results.

Initial deliverable includes:

1) Table noting the flooding depth (ft) and peak flow (cfs) for the 25 locations identified during Task 4.

Step 2 – Meet with the City to go over the results.

Following Step 1, the Consultant will meet with the City to go over the results.

Deliverable includes:

1. Meeting minutes from meeting

Step 3 – Finalize analysis based on meeting with City

Revise the analysis based on feedback from the City.

Deliverables for this step shall include:

1) Color figures showing the maximum extent of flooding during each storm event. The figures shall be color coded to show depth of flooding (typical ranges utilized are: 0.01'-0.25', 0.25'-0.5', 0.5'-1.0', and greater than 1').
2) Table noting the flooding depth (ft) and peak flow (cfs) for the 25 locations identified during Task 4.
3) Model files and documentation.
4) GIS files generated for model development. All files shall be delivered to the City on an external hard drive.

Task 6.1 – Phase II Volume Control Analysis (VC) - TBD

Appendix D. Wingra West Phase I Analysis

Memorandum

Brown and Caldwell
8383 Greenway Blvd. # 600
Middleton, WI 53562

Date: April 21, 2021

Prepared for: City of Madison: Caroline Burger, PE, Env SP
Lauren Striegl, PE
Phil Gaebler, PE

Subject: Wingra West Watershed – Green Infrastructure Evaluation
Project No.: City of Madison: 8412, BC: 153394

From: Mike Wegner, PE & Doug Joachim, PE
Copy:

Background

In response to recent flooding events, including an extreme flood event resulting from 10.5 inches of rain over a 12-hour period in August 2018, the City of Madison (City) has begun conducting comprehensive stormwater management studies for each major watershed within the City. The studies are intended to help the City better understand the causes of flooding in each watershed and identify potential improvements to their stormwater management system to mitigate future flooding issues.

The stormwater management studies are conducted in two phases: The first phase includes the development of a hydrologic/hydraulic (H&H) stormwater runoff model that represents the physical and drainage properties of the watershed under existing conditions. The second phase utilizes the model developed in the first phase to evaluate alternative methods and infrastructure improvements that can eliminate or reduce flooding impacts from large rain events.

As part of this program, Brown and Caldwell (BC) is currently completing the Wingra West Watershed Study in western Madison. An existing conditions model was developed and calibrated using XPSWMM™ 2D modeling software. The results of the existing conditions analysis are detailed in a report dated April 17, 2020 entitled "Wingra West Watershed: Existing Conditions Report."

As part of the second phase of the Wingra West Watershed Study, an evaluation of the potential benefits that could be achieved using distributed green infrastructure (GI) for volume control (VC) was conducted. Conceptually, GI will facilitate the infiltration of stormwater runoff, which will reduce peak stormwater flows and reduce downstream flood impacts while also limiting the size and cost of future infrastructure improvements designed for flood control. This tech memo details the modeling process and results obtained from this analysis.

Model Assumptions

A detailed evaluation of GI, and its potential impacts on flooding during larger storm events, was completed for the Pheasant Branch Watershed by the City of Madison. The analysis of GI for the

Wingra West Watershed was completed following the methodology developed by the City for the Pheasant Branch Watershed.

To simplify the GI analysis, the following model assumptions were used:

- GI will be applied uniformly across the watershed.
- Each subcatchment in the watershed has adequate space available to implement GI.
- The GI will be sized to fully contain the 100% chance storm event from the GI contributing area. Infiltration within the GI practices is ignored.
- The effects of GI will be approximated in the model by moving a portion of each subcatchment's directly connected impervious area (DCIA) to non-DCIA and increasing the depth of storage in the subcatchment's pervious area.

Analysis Scenarios

For this analysis, two GI scenarios were analyzed in the Wingra West watershed for the 100%, 10% and 1% chance rainfall events. The scenarios considered the impacts to the watershed if different amount of impervious area within the watershed are directed to GI.

The first scenario assumed 10% of DCIA will be re-directed to GI. The second scenario assumed 25% of the DCIA will be re-directed to GI.

Model Modifications

The Wingra West existing conditions model was modified to incorporate GI through the following steps:

1. A model was created and run for the 100% chance storm event (2.49 inches of rainfall over 24 hours) with 10% and 25% DCIA removed entirely for Scenarios 1 and 2 respectively for each modeled subcatchment. The results of this model run were then compared with the model results from Step 2.
2. A model was created and run for the 100% chance storm event with 10% and 25% DCIA moved to non-DCIA for Scenarios 1 and 2 respectively for each subcatchment. The storage available in each sub-catchment's pervious area was "scaled up" to allow additional storage equal to the 100% chance storm event (2.49 inches) multiplied by the DCIA area moved.
3. Because infiltration occurs as part of the hydrologic calculations within the model in the pervious area, the models created in Step 2 were shown to be reducing peak flows more than the models in Step 1. To rectify this situation, the models in Step 2 were run iteratively by reducing the amount of storage available in the pervious area until the peak flows generated in these models were approximately equal to the peak flows generated in Step 1.

Evaluation of GI Impacts

The final models identified in Step 3 above were executed for the 100%, 10% and 1% chance rainfall events for Scenarios 1 and 2 to determine the resulting impact of GI in the Wingra West watershed. Peak flows and peak water surface elevations at twenty-five locations (key locations identified in the existing conditions report) and at the watershed outlet were compared to existing conditions results. The complete results are summarized in Tables VC-1, VC-2, and VC-3 following this memo. The locations that correspond to the comparison points on the table are shown on Figure 4-6 (excerpted from the Wingra West Watershed: Existing Conditions Report).

The analysis showed larger reductions (measured by percent change) in peak water surface elevations and peak flow rates during the smallest event (100% chance rainfall event) considered.

The impact of GI was more limited during the 10% and 1% chance events. The largest reduction in flow rate observed for the 1% chance event was 8% with reductions of 2% to 3% more typical. At the outfall from the watershed (the outlet from Secret Pond); the reduction in peak flow rate was 6.8%, 5.6%, and 2.4% for the 100%, 10%, and 1% chance storm events, respectively.

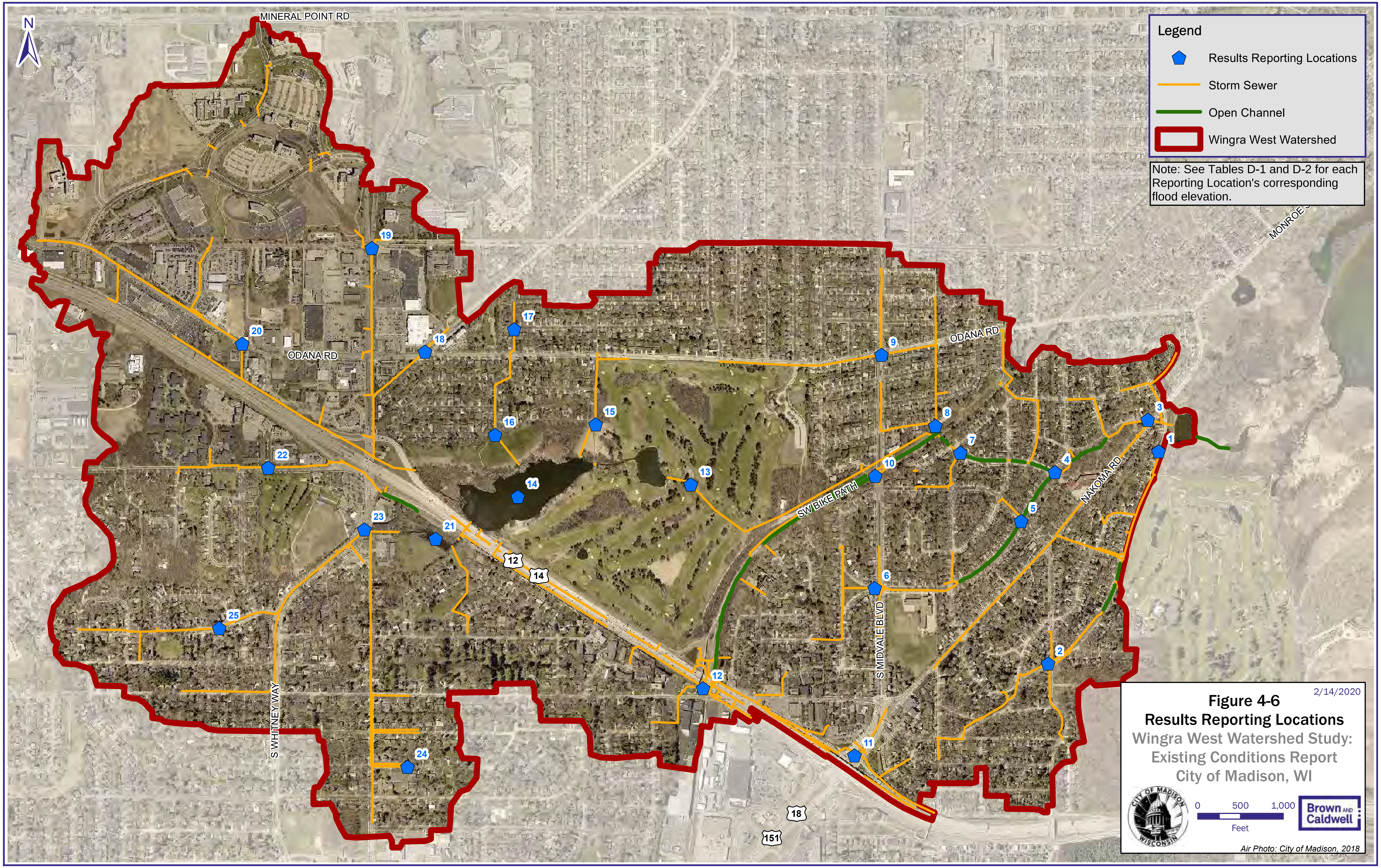


Figure 4-6
Results Reporting Locations
Wingra West Watershed Study:
Existing Conditions Report
City of Madison, WI

Path: P:\Madison, City of\153394 West Wingra Watershed Study\GIS\MXDs\ExCondReportFigures\Fig4-6-ReportingLocations.mxd

Table VC-1 - 100% Chance Storm Event
Volume Control Analysis Results
Wingra West Watershed Study, City of Madison, WI

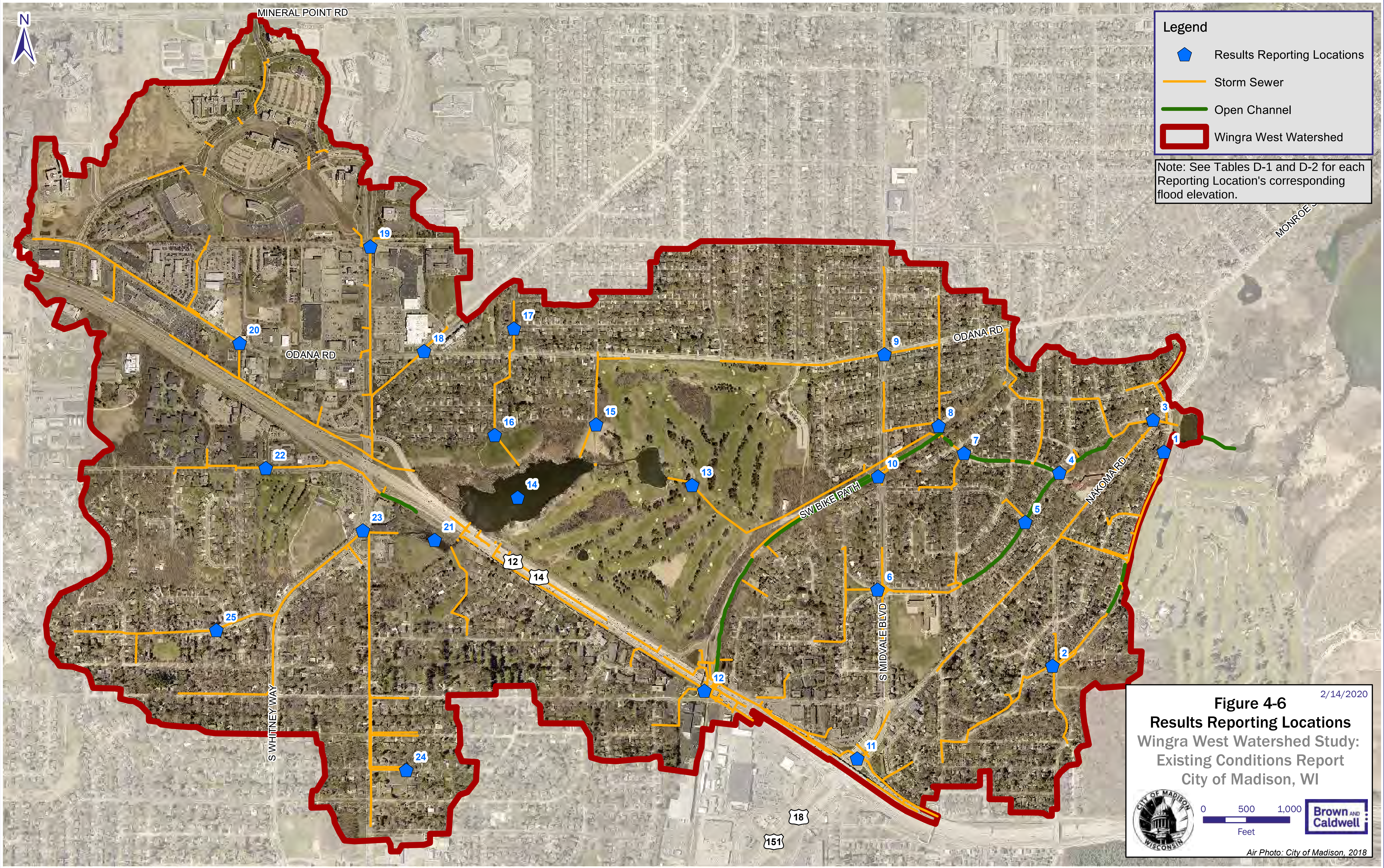
					Existing Conditions Results			10% DCIA Rerouted to Green Infrastructure						25% DCIA Rerouted to Green Infrastructure					
					100% Chance Storm			100% Chance Storm						100% Chance Storm					
Point	XP-SWMM Node	XPSWMM Link	Location	Flood Elevation	Peak WSE	Flood Depth (ft)	Peak Flow (cfs)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)
	PD_4061-006	MMY_1_WW_OUTFALL_1	Secret Pond Outfall	867.5	860.4	-7.1	229.1	860.4	-7.1	0.0	223.8	-5.4	-2.3%	860.3	-7.2	-0.1	213.5	-15.6	-6.8%
1	WI02-D-0193-H-001	3961-075_3961-076	Manitou Way Low-Point - Approximately 400-ft south of Nakoma Rd	862.6	863.1	0.4	41.9	863.0	0.4	-0.1	33.0	-9.0	-21.4%	862.9	0.2	-0.2	19.9	-22.0	-52.4%
2	WI02-D-0193-H-009	3964-009_3964-008	Intersection of Seminole Hwy & Manitou Way	929.3	925.5	-3.8	4.3	925.4	-3.9	-0.1	4.1	-0.2	-4.9%	925.2	-4.1	-0.3	3.7	-0.5	-12.7%
3	WI02-C-0190-H-000	BC_JCT_51_BC_JCT_52	Intersection of Chippewa Dr & Nakoma Rd	873.0	874.7	1.7	284.9	874.5	1.5	-0.2	268.2	-16.7	-5.9%	874.1	1.1	-0.6	239.2	-45.7	-16.0%
4	WI02-C-0188-H-000	3961-050_BC_JCT_50	Intersection of Chippewa Dr & Cherokee Dr	892.8	894.2	1.3	271.8	894.1	1.3	-0.1	256.5	-15.3	-5.6%	894.0	1.2	-0.2	234.0	-37.8	-13.9%
5	AS3862-003	3862-018_3862-003	Intersection of Yuma Dr & Cherokee Dr	912.8	910.8	-2.0	75.5	910.7	-2.1	-0.2	70.7	-4.7	-6.3%	910.4	-2.4	-0.4	61.6	-13.8	-18.3%
6	WI02-C-0188-H-011	3763-008_3763-016	Intersection of Midvale Blvd & Cherokee Dr	967.5	968.0	0.5	28.5	968.0	0.5	0.0	26.5	-2.1	-7.2%	967.9	0.4	-0.1	23.1	-5.4	-19.0%
7	WI02-C-0188-H-017	BC_JCT_140_BC_JCT_142	Intersection of Chippewa Dr & Council Crest	937.9	938.1	0.1	184.8	938.0	0.1	0.0	174.7	-10.1	-5.5%	938.0	0.1	0.0	156.4	-28.4	-15.4%
8	IN3761-056	WAITECIRCLE_3861-045	West end of Waite Circle	956.8	958.4	1.6	174.3	958.2	1.4	-0.1	163.8	-10.4	-6.0%	957.9	1.1	-0.4	146.3	-27.9	-16.0%
9	AS3760-034	3760-034_3760-015	Intersection of Midvale Blvd & Odana Rd	986.9	986.4	-0.5	37.5	986.3	-0.6	-0.2	37.2	-0.3	-0.8%	985.9	-1.0	-0.6	36.7	-0.9	-2.3%
10	AS3762-026	3762-035_3762-026	Intersection of Midvale Blvd & SW Bike Path	972.7	969.1	-3.6	4.4	969.1	-3.6	0.0	4.3	-0.1	-2.4%	969.1	-3.6	-0.1	4.1	-0.3	-6.3%
11	WI02-C-0324-H-000	3765-017_BC_JCT_129	Verona Road, Immediately North of Beltline Highway	1002.7	1002.7	0.0	29.2	1002.7	0.0	0.0	26.0	-3.2	-10.9%	1002.7	0.0	0.0	20.4	-8.8	-30.0%
12	WI02-C-0324-H-005	7645-023_10_BC_JCT_106	Intersection of Hammersley Rd & Beltline Hwy - Southwest of Hwy	997.2	998.6	1.5	9.7	998.6	1.4	-0.1	8.8	-0.9	-9.4%	998.5	1.3	-0.2	7.5	-2.2	-22.3%
13	STOR_12	3562-001_3662-002	Odana Golf Course - Entrance of 42" Storm Sewer	978.0	979.5	1.5	21.7	979.4	1.4	-0.1	19.8	-1.9	-8.6%	979.2	1.2	-0.3	16.8	-4.9	-22.5%
14	PD_3462-002	PD_3462-002_WEIR_1 PD_3462-002_WEIR_2 Link636	Odana Pond	980.0	979.5	-0.5	111.0	979.4	-0.6	-0.1	87.3	-23.7	-21.4%	979.2	-0.8	-0.3	60.4	-50.6	-45.6%
15	WI01-A-0179-H-001	3460-019_3461-001	Intersection of Milward Dr & Wedgewood Way	982.3	983.5	1.1	13.5	983.3	1.0	-0.1	12.5	-0.9	-6.9%	983.1	0.7	-0.4	10.9	-2.6	-19.2%
16	WI01-A-0179-H-003	3461-007_3461-008	Intersection of Milward Dr & Dearholt Rd	981.3	982.4	1.1	20.2	982.4	1.1	0.0	18.8	-1.5	-7.3%	982.3	1.0	-0.1	16.6	-3.7	-18.2%
17	WI01-A-0179-H-006	3460-002_3460-003	Intersection of Sherwood Rd & Dearholt Rd	989.6	990.5	1.0	8.7	990.5	0.9	-0.1	8.6	-0.1	-0.9%	990.4	0.8	-0.1	8.5	-0.2	-2.3%
18	WI01-A-0180-H-002	3360-020_3360-019	Intersection of Odana Rd & Segoe Rd	1005.9	1006.6	0.7	7.7	1006.5	0.6	-0.1	6.9	-0.8	-10.6%	1006.5	0.6	-0.1	5.3	-2.4	-31.2%
19	WI01-A-0187-H-006	3359-006_3359-008	Intersection of Tokay Blvd & Whitney Way	1018.7	1018.8	0.1	24.6	1018.8	0.1	0.0	21.9	-2.7	-11.1%	1018.8	0.1	0.0	17.7	-6.9	-28.1%
20	WI01-A-0183-H-001	3160-002_BC_JCT_21	Odana Rd, approximately 325-ft west of Medical Circle	1028.6	1029.5	0.9	134.9	1029.4	0.8	-0.1	121.1	-13.8	-10.2%	1029.3	0.7	-0.2	102.9	-32.0	-23.7%
21	PD_3362-020	WW_GW_07_WW_GW_09	Orchard Ridge Valley Park Pond	979.0	979.8	0.8	119.3	979.6	0.6	-0.1	116.0	-3.3	-2.8%	979.4	0.4	-0.4	110.3	-9.0	-7.5%
22	WI01-B-0184-H-010	3261-016_3261-017	Intersection of Shroeder Rd & Shroeder Ct	1002.0	1002.1	0.1	52.2	1002.1	0.1	0.0	48.3	-3.9	-7.6%	1002.1	0.1	0.0	40.7	-11.5	-22.0%
23	WI01-B-0184-H-000	3362-012_3362-015	Intersection of Whitney Way & Gilbert Rd	981.7	982.2	0.5	150.5	982.1	0.5	0.0	141.3	-9.2	-6.1%	982.0	0.4	-0.1	125.4	-25.1	-16.7%
24	WI01-B-0181-H-001	3365-013_3365-024	Tolman Ter low-point, approximately 425-ft east of Gilbert Rd	1038.6	1039.8	1.2	4.5	1039.6	1.1	-0.1	4.3	-0.1	-3.0%	1039.4	0.9	-0.3	4.1	-0.3	-6.9%
25	WI01-B-0182-H-006	3164-028_3164-008	Intersection of Piping Rock Rd & Manhasset Pl	1004.9	1005.1	0.2	23.7	1005.1	0.2	0.0	22.3	-1.4	-6.1%	1005.1	0.2	0.0	20.0	-3.7	-15.4%

Table VC-2 - 10% Chance Storm Event
Volume Control Analysis Results
Wingra West Watershed Study, City of Madison, WI

					Existing Conditions Results			10% DCIA Rerouted to Green Infrastructure						25% DCIA Rerouted to Green Infrastructure					
					10% Chance Storm			10% Chance Storm						10% Chance Storm					
Point	XP-SWMM Node	XPSWMM Link	Location	Flood Elevation	Peak WSE	Flood Depth (ft)	Peak Flow (cfs)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)
	PD_4061-006	MMY_1_WW_OUTFALL_1	Secret Pond Outfall	867.5	860.9	-6.6	302.2	860.9	-6.6	0.0	299.1	-3.1	-1.0%	860.8	-6.7	-0.1	285.4	-16.8	-5.6%
1	WI02-D-0193-H-001	3961-075_3961-076	Manitou Way Low-Point - Approximately 400-ft south of Nakoma Rd	862.6	866.2	3.5	138.4	866.1	3.5	-0.1	137.6	-0.8	-0.6%	866.1	3.5	-0.1	138.4	-0.1	-0.1%
2	WI02-D-0193-H-009	3964-009_3964-008	Intersection of Seminole Hwy & Manitou Way	929.3	930.4	1.2	11.5	930.4	1.1	-0.1	11.2	-0.3	-2.7%	930.3	1.0	-0.2	10.6	-1.0	-8.4%
3	WI02-C-0190-H-000	BC_JCT_51_BC_JCT_52	Intersection of Chippewa Dr & Nakoma Rd	873.0	875.9	2.9	489.6	875.9	2.9	0.0	482.8	-6.9	-1.4%	875.8	2.8	-0.1	468.5	-21.1	-4.3%
4	WI02-C-0188-H-000	3961-050_BC_JCT_50	Intersection of Chippewa Dr & Cherokee Dr	892.8	894.9	2.1	503.1	894.9	2.1	0.0	491.6	-11.5	-2.3%	894.8	2.0	-0.1	475.6	-27.5	-5.5%
5	AS3862-003	3862-018_3862-003	Intersection of Yuma Dr & Cherokee Dr	912.8	912.6	-0.2	145.3	912.5	-0.3	-0.1	139.5	-5.7	-4.0%	912.3	-0.5	-0.3	132.4	-12.9	-8.9%
6	WI02-C-0188-H-011	3763-008_3763-016	Intersection of Midvale Blvd & Cherokee Dr	967.5	968.8	1.3	62.3	968.8	1.3	0.0	61.2	-1.1	-1.7%	968.7	1.2	-0.1	59.3	-3.0	-4.8%
7	WI02-C-0188-H-017	BC_JCT_140_BC_JCT_142	Intersection of Chippewa Dr & Council Crest	937.9	938.2	0.3	308.7	938.2	0.3	0.0	302.7	-6.1	-2.0%	938.2	0.2	0.0	294.7	-14.0	-4.5%
8	IN3761-056	WAITECIRCLE_3861-045	West end of Waite Circle	956.8	958.8	2.0	302.6	958.8	2.0	0.0	295.2	-7.4	-2.4%	958.8	2.0	0.0	285.6	-17.1	-5.6%
9	AS3760-034	3760-034_3760-015	Intersection of Midvale Blvd & Odana Rd	986.9	986.9	0.0	44.0	986.9	0.0	-0.1	43.2	-0.8	-1.9%	986.8	-0.1	-0.1	41.9	-2.1	-4.9%
10	AS3762-026	3762-035_3762-026	Intersection of Midvale Blvd & SW Bike Path	972.7	972.3	-0.4	48.1	972.2	-0.5	-0.1	44.5	-3.6	-7.5%	972.0	-0.7	-0.3	40.0	-8.1	-16.9%
11	WI02-C-0324-H-000	3765-017_BC_JCT_129	Verona Road, Immediately North of Beltline Highway	1002.7	1002.8	0.1	45.0	1002.8	0.1	0.0	44.6	-0.4	-1.0%	1002.8	0.1	0.0	43.4	-1.6	-3.6%
12	WI02-C-0324-H-005	7645-023_10_BC_JCT_106	Intersection of Hammersley Rd & Beltline Hwy - Southwest of Hwy	997.2	999.1	1.9	21.8	999.1	1.9	0.0	21.7	-0.2	-0.7%	999.0	1.9	0.0	21.4	-0.4	-1.8%
13	STOR_12	3562-001_3662-002	Odana Golf Course - Entrance of 42" Storm Sewer	978.0	981.0	3.0	52.2	980.9	2.9	-0.1	50.1	-2.1	-4.0%	980.8	2.8	-0.2	47.2	-5.0	-9.5%
14	PD_3462-002	PD_3462-002_WEIR_1 PD_3462-002_WEIR_2 Link636	Odana Pond	980.0	981.0	1.0	355.6	980.9	0.9	-0.1	338.9	-16.7	-4.7%	980.8	0.8	-0.2	317.7	-37.9	-10.7%
15	WI01-A-0179-H-001	3460-019_3461-001	Intersection of Milward Dr & Wedgewood Way	982.3	984.1	1.8	23.5	984.1	1.8	0.0	23.5	-0.1	-0.3%	984.1	1.7	-0.1	23.4	-0.1	-0.6%
16	WI01-A-0179-H-003	3461-007_3461-008	Intersection of Milward Dr & Dearholt Rd	981.3	983.1	1.8	42.8	983.1	1.8	-0.1	38.4	-4.4	-10.3%	983.0	1.7	-0.1	33.4	-9.5	-22.2%
17	WI01-A-0179-H-006	3460-002_3460-003	Intersection of Sherwood Rd & Dearholt Rd	989.6	991.2	1.6	12.6	991.2	1.6	0.0	12.3	-0.3	-2.7%	991.2	1.6	0.0	11.9	-0.8	-6.1%
18	WI01-A-0180-H-002	3360-020_3360-019	Intersection of Odana Rd & Segoe Rd	1005.9	1007.1	1.2	18.5	1007.0	1.1	-0.1	15.0	-3.5	-18.8%	1006.9	1.0	-0.2	15.0	-3.6	-19.2%
19	WI01-A-0187-H-006	3359-006_3359-008	Intersection of Tokay Blvd & Whitney Way	1018.7	1018.9	0.2	43.3	1018.9	0.2	0.0	39.6	-3.7	-8.5%	1018.9	0.2	0.0	37.2	-6.1	-14.1%
20	WI01-A-0183-H-001	3160-002_BC_JCT_21	Odana Rd, approximately 325-ft west of Medical Circle	1028.6	1029.9	1.4	172.9	1029.9	1.3	-0.1	172.9	0.0	0.0%	1029.7	1.2	-0.2	171.6	-1.2	-0.7%
21	PD_3362-020	WW_GW_07_WW_GW_09	Orchard Ridge Valley Park Pond	979.0	981.8	2.8	181.0	981.7	2.7	-0.1	181.5	0.4	0.2%	981.6	2.6	-0.2	181.8	0.8	0.4%
22	WI01-B-0184-H-010	3261-016_3261-017	Intersection of Shroeder Rd & Shroeder Ct	1002.0	1003.1	1.1	107.4	1003.0	1.0	-0.1	102.3	-5.1	-4.8%	1002.9	0.9	-0.2	93.8	-13.7	-12.7%
23	WI01-B-0184-H-000	3362-012_3362-015	Intersection of Whitney Way & Gilbert Rd	981.7	983.5	1.8	245.2	983.4	1.7	-0.1	245.8	0.6	0.3%	983.3	1.6	-0.2	247.2	2.0	0.8%
24	WI01-B-0181-H-001	3365-013_3365-024	Tolman Ter low-point, approximately 425-ft east of Gilbert Rd	1038.6	1040.9	2.4	6.8	1040.9	2.3	0.0	6.7	-0.1	-1.2%	1040.9	2.3	0.0	6.6	-0.1	-1.9%
25	WI01-B-0182-H-006	3164-028_3164-008	Intersection of Piping Rock Rd & Manhasset Pl	1004.9	1005.3	0.4	59.4	1005.3	0.4	0.0	58.3	-1.1	-1.8%	1005.3	0.4	0.0	56.0	-3.3	-5.6%

Table VC-3 - 1% Chance Storm Event
Volume Control Analysis Results
Wingra West Watershed Study, City of Madison, WI

Point	XP-SWMM Node	XPSWMM Link	Location	Flood Elevation	Existing Conditions Results 1% Chance Storm			10% DCIA Rerouted to Green Infrastructure 1% Chance Storm						25% DCIA Rerouted to Green Infrastructure 1% Chance Storm					
					Peak WSE	Flood Depth (ft)	Peak Flow (cfs)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)
	PD_4061-006	MMY_1_WW_OUTFALL_1	Secret Pond Outfall	867.5	861.5	-6.0	447.5	861.5	-6.0	0.0	443.7	-3.7	-0.8%	861.5	-6.0	0.0	436.7	-10.7	-2.4%
1	WI02-D-0193-H-001	3961-075_3961-076	Manitou Way Low-Point - Approximately 400-ft south of Nakoma Rd	862.6	866.5	3.9	168.9	866.5	3.9	0.0	169.5	0.6	0.3%	866.5	3.9	0.1	167.4	-1.5	-0.9%
2	WI02-D-0193-H-009	3964-009_3964-008	Intersection of Seminole Hwy & Manitou Way	929.3	931.2	1.9	25.1	931.2	1.9	0.0	24.9	-0.2	-0.7%	931.1	1.9	0.0	24.6	-0.5	-1.9%
3	WI02-C-0190-H-000	BC_JCT_51_BC_JCT_52	Intersection of Chippewa Dr & Nakoma Rd	873.0	876.7	3.7	757.3	876.7	3.7	0.0	748.5	-8.8	-1.2%	876.6	3.7	0.0	747.8	-9.5	-1.2%
4	WI02-C-0188-H-000	3961-050_BC_JCT_50	Intersection of Chippewa Dr & Cherokee Dr	892.8	895.6	2.8	811.7	895.6	2.8	0.0	805.9	-5.8	-0.7%	895.6	2.7	0.0	795.4	-16.4	-2.0%
5	AS3862-003	3862-018_3862-003	Intersection of Yuma Dr & Cherokee Dr	912.8	913.9	1.1	283.8	913.8	1.0	0.0	280.6	-3.1	-1.1%	913.8	1.0	-0.1	276.0	-7.7	-2.7%
6	WI02-C-0188-H-011	3763-008_3763-016	Intersection of Midvale Blvd & Cherokee Dr	967.5	969.4	1.9	109.4	969.4	1.9	0.0	108.9	-0.4	-0.4%	969.4	1.9	0.0	108.3	-1.1	-1.0%
7	WI02-C-0188-H-017	BC_JCT_140_BC_JCT_142	Intersection of Chippewa Dr & Council Crest	937.9	939.4	1.5	423.8	939.4	1.4	0.0	418.5	-5.3	-1.3%	939.3	1.4	-0.1	411.4	-12.3	-2.9%
8	IN3761-056	WAITECIRCLE_3861-045	West end of Waite Circle	956.8	963.1	6.3	400.5	963.0	6.2	-0.1	399.8	-0.6	-0.2%	963.0	6.2	-0.1	398.8	-1.6	-0.4%
9	AS3760-034	3760-034_3760-015	Intersection of Midvale Blvd & Odana Rd	986.9	987.4	0.5	69.3	987.4	0.5	0.0	68.7	-0.6	-0.9%	987.4	0.5	0.0	67.5	-1.8	-2.6%
10	AS3762-026	3762-035_3762-026	Intersection of Midvale Blvd & SW Bike Path	972.7	973.6	0.9	149.2	973.6	0.9	0.0	147.1	-2.1	-1.4%	973.6	0.9	0.0	144.7	-4.5	-3.0%
11	WI02-C-0324-H-000	3765-017_BC_JCT_129	Verona Road, Immediately North of Beltline Highway	1002.7	1003.8	1.1	52.5	1003.8	1.1	0.0	52.4	0.0	-0.1%	1003.7	1.0	-0.1	52.4	0.0	-0.1%
12	WI02-C-0324-H-005	7645-023_10_BC_JCT_106	Intersection of Hammersley Rd & Beltline Hwy - Southwest of Hwy	997.2	999.5	2.4	22.1	999.5	2.3	0.0	22.2	0.1	0.4%	999.5	2.3	0.0	22.2	0.1	0.5%
13	STOR_12	3562-001_3662-002	Odana Golf Course - Entrance of 42" Storm Sewer	978.0	983.0	5.0	71.4	982.9	4.9	-0.1	71.0	-0.3	-0.5%	982.8	4.8	-0.2	70.6	-0.8	-1.1%
14	PD_3462-002	PD_3462-002_WEIR_1 PD_3462-002_WEIR_2 Link636	Odana Pond	980.0	983.0	3.0	480.3	982.9	2.9	-0.1	468.4	-11.9	-2.5%	982.8	2.8	-0.2	449.2	-31.1	-6.5%
15	WI01-A-0179-H-001	3460-019_3461-001	Intersection of Milward Dr & Wedgewood Way	982.3	984.5	2.2	25.6	984.5	2.2	0.0	25.6	0.0	0.0%	984.5	2.1	0.0	25.6	0.0	0.0%
16	WI01-A-0179-H-003	3461-007_3461-008	Intersection of Milward Dr & Dearholt Rd	981.3	984.0	2.7	110.2	984.0	2.7	0.0	108.3	-2.0	-1.8%	983.9	2.7	0.0	104.7	-5.5	-5.0%
17	WI01-A-0179-H-006	3460-002_3460-003	Intersection of Sherwood Rd & Dearholt Rd	989.6	991.7	2.1	30.9	991.7	2.1	0.0	30.6	-0.4	-1.1%	991.7	2.1	0.0	30.0	-0.9	-2.9%
18	WI01-A-0180-H-002	3360-020_3360-019	Intersection of Odana Rd & Segoe Rd	1005.9	1007.8	1.9	34.6	1007.8	1.9	0.0	33.3	-1.3	-3.9%	1007.7	1.8	-0.1	31.9	-2.8	-8.0%
19	WI01-A-0187-H-006	3359-006_3359-008	Intersection of Tokay Blvd & Whitney Way	1018.7	1019.5	0.7	93.5	1019.4	0.7	0.0	92.8	-0.7	-0.8%	1019.4	0.6	-0.1	90.8	-2.7	-2.9%
20	WI01-A-0183-H-001	3160-002_BC_JCT_21	Odana Rd, approximately 325-ft west of Medical Circle	1028.6	1030.5	2.0	230.0	1030.5	1.9	0.0	228.5	-1.5	-0.6%	1030.4	1.9	-0.1	225.9	-4.1	-1.8%
21	PD_3362-020	WW_GW_07_WW_GW_09	Orchard Ridge Valley Park Pond	979.0	983.9	4.9	201.5	983.9	4.9	-0.1	202.0	0.5	0.3%	983.8	4.8	-0.2	202.8	1.3	0.7%
22	WI01-B-0184-H-010	3261-016_3261-017	Intersection of Shroeder Rd & Shroeder Ct	1002.0	1003.8	1.8	188.0	1003.8	1.8	0.0	185.7	-2.3	-1.2%	1003.7	1.7	-0.1	181.9	-6.1	-3.3%
23	WI01-B-0184-H-000	3362-012_3362-015	Intersection of Whitney Way & Gilbert Rd	981.7	985.2	3.5	268.3	985.1	3.4	0.0	268.9	0.6	0.2%	985.1	3.4	-0.1	269.4	1.1	0.4%
24	WI01-B-0181-H-001	3365-013_3365-024	Tolman Ter low-point, approximately 425-ft east of Gilbert Rd	1038.6	1041.9	3.4	22.5	1041.9	3.4	0.0	22.3	-0.2	-0.7%	1041.9	3.3	0.0	22.0	-0.5	-2.1%
25	WI01-B-0182-H-006	3164-028_3164-008	Intersection of Piping Rock Rd & Manhasset Pl	1004.9	1005.8	0.9	113.0	1005.8	0.9	0.0	112.6	-0.5	-0.4%	1005.8	0.8	-0.1	112.0	-1.1	-1.0%



2/14/2020

Figure 4-6
Results Reporting Locations
Wingra West Watershed Study:
Existing Conditions Report
City of Madison, WI

CITY OF MADISON WISCONSIN

Brown AND Caldwell

Air Photo: City of Madison, 2018

Path: P:\Madison, City of\153394 West Wingra Watershed Study\GIS\MXDs\ExCondReportFigures\Fig4-6-ReportingLocations.mxd

Table VC-3 - 1% Chance Storm Event
Volume Control Analysis Results
Wingra West Watershed Study, City of Madison, WI

					Existing Conditions Results			10% DCIA Rerouted to Green Infrastructure						25% DCIA Rerouted to Green Infrastructure					
					1% Chance Storm			1% Chance Storm						1% Chance Storm					
Point	XP-SWMM Node	XPSWMM Link	Location	Flood Elevation	Peak WSE	Flood Depth (ft)	Peak Flow (cfs)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)
	PD_4061-006	MMY_1_WW_OUTFALL_1	Secret Pond Outfall	867.5	861.5	-6.0	447.5	861.5	-6.0	0.0	443.7	-3.7	-0.8%	861.5	-6.0	0.0	436.7	-10.7	-2.4%
1	WI02-D-0193-H-001	3961-075_3961-076	Manitou Way Low-Point - Approximately 400-ft south of Nakoma Rd	862.6	866.5	3.9	168.9	866.5	3.9	0.0	169.5	0.6	0.3%	866.5	3.9	0.1	167.4	-1.5	-0.9%
2	WI02-D-0193-H-009	3964-009_3964-008	Intersection of Seminole Hwy & Manitou Way	929.3	931.2	1.9	25.1	931.2	1.9	0.0	24.9	-0.2	-0.7%	931.1	1.9	0.0	24.6	-0.5	-1.9%
3	WI02-C-0190-H-000	BC_JCT_51_BC_JCT_52	Intersection of Chippewa Dr & Nakoma Rd	873.0	876.7	3.7	757.3	876.7	3.7	0.0	748.5	-8.8	-1.2%	876.6	3.7	0.0	747.8	-9.5	-1.2%
4	WI02-C-0188-H-000	3961-050_BC_JCT_50	Intersection of Chippewa Dr & Cherokee Dr	892.8	895.6	2.8	811.7	895.6	2.8	0.0	805.9	-5.8	-0.7%	895.6	2.7	0.0	795.4	-16.4	-2.0%
5	AS3862-003	3862-018_3862-003	Intersection of Yuma Dr & Cherokee Dr	912.8	913.9	1.1	283.8	913.8	1.0	0.0	280.6	-3.1	-1.1%	913.8	1.0	-0.1	276.0	-7.7	-2.7%
6	WI02-C-0188-H-011	3763-008_3763-016	Intersection of Midvale Blvd & Cherokee Dr	967.5	969.4	1.9	109.4	969.4	1.9	0.0	108.9	-0.4	-0.4%	969.4	1.9	0.0	108.3	-1.1	-1.0%
7	WI02-C-0188-H-017	BC_JCT_140_BC_JCT_142	Intersection of Chippewa Dr & Council Crest	937.9	939.4	1.5	423.8	939.4	1.4	0.0	418.5	-5.3	-1.3%	939.3	1.4	-0.1	411.4	-12.3	-2.9%
8	IN3761-056	WAITECIRCLE_3861-045	West end of Waite Circle	956.8	963.1	6.3	400.5	963.0	6.2	-0.1	399.8	-0.6	-0.2%	963.0	6.2	-0.1	398.8	-1.6	-0.4%
9	AS3760-034	3760-034_3760-015	Intersection of Midvale Blvd & Odana Rd	986.9	987.4	0.5	69.3	987.4	0.5	0.0	68.7	-0.6	-0.9%	987.4	0.5	0.0	67.5	-1.8	-2.6%
10	AS3762-026	3762-035_3762-026	Intersection of Midvale Blvd & SW Bike Path	972.7	973.6	0.9	149.2	973.6	0.9	0.0	147.1	-2.1	-1.4%	973.6	0.9	0.0	144.7	-4.5	-3.0%
11	WI02-C-0324-H-000	3765-017_BC_JCT_129	Verona Road, Immediately North of Beltline Highway	1002.7	1003.8	1.1	52.5	1003.8	1.1	0.0	52.4	0.0	-0.1%	1003.7	1.0	-0.1	52.4	0.0	-0.1%
12	WI02-C-0324-H-005	7645-023_10_BC_JCT_106	Intersection of Hammersley Rd & Beltline Hwy - Southwest of Hwy	997.2	999.5	2.4	22.1	999.5	2.3	0.0	22.2	0.1	0.4%	999.5	2.3	0.0	22.2	0.1	0.5%
13	STOR_12	3562-001_3662-002	Odana Golf Course - Entrance of 42" Storm Sewer	978.0	983.0	5.0	71.4	982.9	4.9	-0.1	71.0	-0.3	-0.5%	982.8	4.8	-0.2	70.6	-0.8	-1.1%
14	PD_3462-002	PD_3462-002_WEIR_1 PD_3462-002_WEIR_2 Link636	Odana Pond	980.0	983.0	3.0	480.3	982.9	2.9	-0.1	468.4	-11.9	-2.5%	982.8	2.8	-0.2	449.2	-31.1	-6.5%
15	WI01-A-0179-H-001	3460-019_3461-001	Intersection of Milward Dr & Wedgewood Way	982.3	984.5	2.2	25.6	984.5	2.2	0.0	25.6	0.0	0.0%	984.5	2.1	0.0	25.6	0.0	0.0%
16	WI01-A-0179-H-003	3461-007_3461-008	Intersection of Milward Dr & Dearholt Rd	981.3	984.0	2.7	110.2	984.0	2.7	0.0	108.3	-2.0	-1.8%	983.9	2.7	0.0	104.7	-5.5	-5.0%
17	WI01-A-0179-H-006	3460-002_3460-003	Intersection of Sherwood Rd & Dearholt Rd	989.6	991.7	2.1	30.9	991.7	2.1	0.0	30.6	-0.4	-1.1%	991.7	2.1	0.0	30.0	-0.9	-2.9%
18	WI01-A-0180-H-002	3360-020_3360-019	Intersection of Odana Rd & Segoe Rd	1005.9	1007.8	1.9	34.6	1007.8	1.9	0.0	33.3	-1.3	-3.9%	1007.7	1.8	-0.1	31.9	-2.8	-8.0%
19	WI01-A-0187-H-006	3359-006_3359-008	Intersection of Tokay Blvd & Whitney Way	1018.7	1019.5	0.7	93.5	1019.4	0.7	0.0	92.8	-0.7	-0.8%	1019.4	0.6	-0.1	90.8	-2.7	-2.9%
20	WI01-A-0183-H-001	3160-002_BC_JCT_21	Odana Rd, approximately 325-ft west of Medical Circle	1028.6	1030.5	2.0	230.0	1030.5	1.9	0.0	228.5	-1.5	-0.6%	1030.4	1.9	-0.1	225.9	-4.1	-1.8%
21	PD_3362-020	WW_GW_07_WW_GW_09	Orchard Ridge Valley Park Pond	979.0	983.9	4.9	201.5	983.9	4.9	-0.1	202.0	0.5	0.3%	983.8	4.8	-0.2	202.8	1.3	0.7%
22	WI01-B-0184-H-010	3261-016_3261-017	Intersection of Shroeder Rd & Shroeder Ct	1002.0	1003.8	1.8	188.0	1003.8	1.8	0.0	185.7	-2.3	-1.2%	1003.7	1.7	-0.1	181.9	-6.1	-3.3%
23	WI01-B-0184-H-000	3362-012_3362-015	Intersection of Whitney Way & Gilbert Rd	981.7	985.2	3.5	268.3	985.1	3.4	0.0	268.9	0.6	0.2%	985.1	3.4	-0.1	269.4	1.1	0.4%
24	WI01-B-0181-H-001	3365-013_3365-024	Tolman Ter low-point, approximately 425-ft east of Gilbert Rd	1038.6	1041.9	3.4	22.5	1041.9	3.4	0.0	22.3	-0.2	-0.7%	1041.9	3.3	0.0	22.0	-0.5	-2.1%
25	WI01-B-0182-H-006	3164-028_3164-008	Intersection of Piping Rock Rd & Manhasset Pl	1004.9	1005.8	0.9	113.0	1005.8	0.9	0.0	112.6	-0.5	-0.4%	1005.8	0.8	-0.1	112.0	-1.1	-1.0%

Table VC-2 - 10% Chance Storm Event
Volume Control Analysis Results
Wingra West Watershed Study, City of Madison, WI

					Existing Conditions Results			10% DCIA Rerouted to Green Infrastructure						25% DCIA Rerouted to Green Infrastructure					
					10% Chance Storm			10% Chance Storm						10% Chance Storm					
Point	XP-SWMM Node	XPSWMM Link	Location	Flood Elevation	Peak WSE	Flood Depth (ft)	Peak Flow (cfs)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)
	PD_4061-006	MMY_1_WW_OUTFALL_1	Secret Pond Outfall	867.5	860.9	-6.6	302.2	860.9	-6.6	0.0	299.1	-3.1	-1.0%	860.8	-6.7	-0.1	285.4	-16.8	-5.6%
1	WI02-D-0193-H-001	3961-075_3961-076	Manitou Way Low-Point - Approximately 400-ft south of Nakoma Rd	862.6	866.2	3.5	138.4	866.1	3.5	-0.1	137.6	-0.8	-0.6%	866.1	3.5	-0.1	138.4	-0.1	-0.1%
2	WI02-D-0193-H-009	3964-009_3964-008	Intersection of Seminole Hwy & Manitou Way	929.3	930.4	1.2	11.5	930.4	1.1	-0.1	11.2	-0.3	-2.7%	930.3	1.0	-0.2	10.6	-1.0	-8.4%
3	WI02-C-0190-H-000	BC_JCT_51_BC_JCT_52	Intersection of Chippewa Dr & Nakoma Rd	873.0	875.9	2.9	489.6	875.9	2.9	0.0	482.8	-6.9	-1.4%	875.8	2.8	-0.1	468.5	-21.1	-4.3%
4	WI02-C-0188-H-000	3961-050_BC_JCT_50	Intersection of Chippewa Dr & Cherokee Dr	892.8	894.9	2.1	503.1	894.9	2.1	0.0	491.6	-11.5	-2.3%	894.8	2.0	-0.1	475.6	-27.5	-5.5%
5	AS3862-003	3862-018_3862-003	Intersection of Yuma Dr & Cherokee Dr	912.8	912.6	-0.2	145.3	912.5	-0.3	-0.1	139.5	-5.7	-4.0%	912.3	-0.5	-0.3	132.4	-12.9	-8.9%
6	WI02-C-0188-H-011	3763-008_3763-016	Intersection of Midvale Blvd & Cherokee Dr	967.5	968.8	1.3	62.3	968.8	1.3	0.0	61.2	-1.1	-1.7%	968.7	1.2	-0.1	59.3	-3.0	-4.8%
7	WI02-C-0188-H-017	BC_JCT_140_BC_JCT_142	Intersection of Chippewa Dr & Council Crest	937.9	938.2	0.3	308.7	938.2	0.3	0.0	302.7	-6.1	-2.0%	938.2	0.2	0.0	294.7	-14.0	-4.5%
8	IN3761-056	WAITECIRCLE_3861-045	West end of Waite Circle	956.8	958.8	2.0	302.6	958.8	2.0	0.0	295.2	-7.4	-2.4%	958.8	2.0	0.0	285.6	-17.1	-5.6%
9	AS3760-034	3760-034_3760-015	Intersection of Midvale Blvd & Odana Rd	986.9	986.9	0.0	44.0	986.9	0.0	-0.1	43.2	-0.8	-1.9%	986.8	-0.1	-0.1	41.9	-2.1	-4.9%
10	AS3762-026	3762-035_3762-026	Intersection of Midvale Blvd & SW Bike Path	972.7	972.3	-0.4	48.1	972.2	-0.5	-0.1	44.5	-3.6	-7.5%	972.0	-0.7	-0.3	40.0	-8.1	-16.9%
11	WI02-C-0324-H-000	3765-017_BC_JCT_129	Verona Road, Immediately North of Beltline Highway	1002.7	1002.8	0.1	45.0	1002.8	0.1	0.0	44.6	-0.4	-1.0%	1002.8	0.1	0.0	43.4	-1.6	-3.6%
12	WI02-C-0324-H-005	7645-023_10_BC_JCT_106	Intersection of Hammersley Rd & Beltline Hwy - Southwest of Hwy	997.2	999.1	1.9	21.8	999.1	1.9	0.0	21.7	-0.2	-0.7%	999.0	1.9	0.0	21.4	-0.4	-1.8%
13	STOR_12	3562-001_3662-002	Odana Golf Course - Entrance of 42" Storm Sewer	978.0	981.0	3.0	52.2	980.9	2.9	-0.1	50.1	-2.1	-4.0%	980.8	2.8	-0.2	47.2	-5.0	-9.5%
14	PD_3462-002	PD_3462-002_WEIR_1 PD_3462-002_WEIR_2 Link636	Odana Pond	980.0	981.0	1.0	355.6	980.9	0.9	-0.1	338.9	-16.7	-4.7%	980.8	0.8	-0.2	317.7	-37.9	-10.7%
15	WI01-A-0179-H-001	3460-019_3461-001	Intersection of Milward Dr & Wedgewood Way	982.3	984.1	1.8	23.5	984.1	1.8	0.0	23.5	-0.1	-0.3%	984.1	1.7	-0.1	23.4	-0.1	-0.6%
16	WI01-A-0179-H-003	3461-007_3461-008	Intersection of Milward Dr & Dearholt Rd	981.3	983.1	1.8	42.8	983.1	1.8	-0.1	38.4	-4.4	-10.3%	983.0	1.7	-0.1	33.4	-9.5	-22.2%
17	WI01-A-0179-H-006	3460-002_3460-003	Intersection of Sherwood Rd & Dearholt Rd	989.6	991.2	1.6	12.6	991.2	1.6	0.0	12.3	-0.3	-2.7%	991.2	1.6	0.0	11.9	-0.8	-6.1%
18	WI01-A-0180-H-002	3360-020_3360-019	Intersection of Odana Rd & Segoe Rd	1005.9	1007.1	1.2	18.5	1007.0	1.1	-0.1	15.0	-3.5	-18.8%	1006.9	1.0	-0.2	15.0	-3.6	-19.2%
19	WI01-A-0187-H-006	3359-006_3359-008	Intersection of Tokay Blvd & Whitney Way	1018.7	1018.9	0.2	43.3	1018.9	0.2	0.0	39.6	-3.7	-8.5%	1018.9	0.2	0.0	37.2	-6.1	-14.1%
20	WI01-A-0183-H-001	3160-002_BC_JCT_21	Odana Rd, approximately 325-ft west of Medical Circle	1028.6	1029.9	1.4	172.9	1029.9	1.3	-0.1	172.9	0.0	0.0%	1029.7	1.2	-0.2	171.6	-1.2	-0.7%
21	PD_3362-020	WW_GW_07_WW_GW_09	Orchard Ridge Valley Park Pond	979.0	981.8	2.8	181.0	981.7	2.7	-0.1	181.5	0.4	0.2%	981.6	2.6	-0.2	181.8	0.8	0.4%
22	WI01-B-0184-H-010	3261-016_3261-017	Intersection of Shroeder Rd & Shroeder Ct	1002.0	1003.1	1.1	107.4	1003.0	1.0	-0.1	102.3	-5.1	-4.8%	1002.9	0.9	-0.2	93.8	-13.7	-12.7%
23	WI01-B-0184-H-000	3362-012_3362-015	Intersection of Whitney Way & Gilbert Rd	981.7	983.5	1.8	245.2	983.4	1.7	-0.1	245.8	0.6	0.3%	983.3	1.6	-0.2	247.2	2.0	0.8%
24	WI01-B-0181-H-001	3365-013_3365-024	Tolman Ter low-point, approximately 425-ft east of Gilbert Rd	1038.6	1040.9	2.4	6.8	1040.9	2.3	0.0	6.7	-0.1	-1.2%	1040.9	2.3	0.0	6.6	-0.1	-1.9%
25	WI01-B-0182-H-006	3164-028_3164-008	Intersection of Piping Rock Rd & Manhasset Pl	1004.9	1005.3	0.4	59.4	1005.3	0.4	0.0	58.3	-1.1	-1.8%	1005.3	0.4	0.0	56.0	-3.3	-5.6%

Table VC-1 - 100% Chance Storm Event
Volume Control Analysis Results
Wingra West Watershed Study, City of Madison, WI

					Existing Conditions Results			10% DCIA Rerouted to Green Infrastructure						25% DCIA Rerouted to Green Infrastructure					
					100% Chance Storm			100% Chance Storm						100% Chance Storm					
Point	XP-SWMM Node	XPSWMM Link	Location	Flood Elevation	Peak WSE	Flood Depth (ft)	Peak Flow (cfs)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)	Peak WSE	Flood Depth (ft)	Change in Flood Depth (ft)	Peak Flow (cfs)	Change in Peak Flow (cfs)	Change in Peak Flow (%)
	PD_4061-006	MMY_1_WW_OUTFALL_1	Secret Pond Outfall	867.5	860.4	-7.1	229.1	860.4	-7.1	0.0	223.8	-5.4	-2.3%	860.3	-7.2	-0.1	213.5	-15.6	-6.8%
1	WI02-D-0193-H-001	3961-075_3961-076	Manitou Way Low-Point - Approximately 400-ft south of Nakoma Rd	862.6	863.1	0.4	41.9	863.0	0.4	-0.1	33.0	-9.0	-21.4%	862.9	0.2	-0.2	19.9	-22.0	-52.4%
2	WI02-D-0193-H-009	3964-009_3964-008	Intersection of Seminole Hwy & Manitou Way	929.3	925.5	-3.8	4.3	925.4	-3.9	-0.1	4.1	-0.2	-4.9%	925.2	-4.1	-0.3	3.7	-0.5	-12.7%
3	WI02-C-0190-H-000	BC_JCT_51_BC_JCT_52	Intersection of Chippewa Dr & Nakoma Rd	873.0	874.7	1.7	284.9	874.5	1.5	-0.2	268.2	-16.7	-5.9%	874.1	1.1	-0.6	239.2	-45.7	-16.0%
4	WI02-C-0188-H-000	3961-050_BC_JCT_50	Intersection of Chippewa Dr & Cherokee Dr	892.8	894.2	1.3	271.8	894.1	1.3	-0.1	256.5	-15.3	-5.6%	894.0	1.2	-0.2	234.0	-37.8	-13.9%
5	AS3862-003	3862-018_3862-003	Intersection of Yuma Dr & Cherokee Dr	912.8	910.8	-2.0	75.5	910.7	-2.1	-0.2	70.7	-4.7	-6.3%	910.4	-2.4	-0.4	61.6	-13.8	-18.3%
6	WI02-C-0188-H-011	3763-008_3763-016	Intersection of Midvale Blvd & Cherokee Dr	967.5	968.0	0.5	28.5	968.0	0.5	0.0	26.5	-2.1	-7.2%	967.9	0.4	-0.1	23.1	-5.4	-19.0%
7	WI02-C-0188-H-017	BC_JCT_140_BC_JCT_142	Intersection of Chippewa Dr & Council Crest	937.9	938.1	0.1	184.8	938.0	0.1	0.0	174.7	-10.1	-5.5%	938.0	0.1	0.0	156.4	-28.4	-15.4%
8	IN3761-056	WAITECIRCLE_3861-045	West end of Waite Circle	956.8	958.4	1.6	174.3	958.2	1.4	-0.1	163.8	-10.4	-6.0%	957.9	1.1	-0.4	146.3	-27.9	-16.0%
9	AS3760-034	3760-034_3760-015	Intersection of Midvale Blvd & Odana Rd	986.9	986.4	-0.5	37.5	986.3	-0.6	-0.2	37.2	-0.3	-0.8%	985.9	-1.0	-0.6	36.7	-0.9	-2.3%
10	AS3762-026	3762-035_3762-026	Intersection of Midvale Blvd & SW Bike Path	972.7	969.1	-3.6	4.4	969.1	-3.6	0.0	4.3	-0.1	-2.4%	969.1	-3.6	-0.1	4.1	-0.3	-6.3%
11	WI02-C-0324-H-000	3765-017_BC_JCT_129	Verona Road, Immediately North of Beltline Highway	1002.7	1002.7	0.0	29.2	1002.7	0.0	0.0	26.0	-3.2	-10.9%	1002.7	0.0	0.0	20.4	-8.8	-30.0%
12	WI02-C-0324-H-005	7645-023_10_BC_JCT_106	Intersection of Hammersley Rd & Beltline Hwy - Southwest of Hwy	997.2	998.6	1.5	9.7	998.6	1.4	-0.1	8.8	-0.9	-9.4%	998.5	1.3	-0.2	7.5	-2.2	-22.3%
13	STOR_12	3562-001_3662-002	Odana Golf Course - Entrance of 42" Storm Sewer	978.0	979.5	1.5	21.7	979.4	1.4	-0.1	19.8	-1.9	-8.6%	979.2	1.2	-0.3	16.8	-4.9	-22.5%
14	PD_3462-002	PD_3462-002_WEIR_1 PD_3462-002_WEIR_2 Link636	Odana Pond	980.0	979.5	-0.5	111.0	979.4	-0.6	-0.1	87.3	-23.7	-21.4%	979.2	-0.8	-0.3	60.4	-50.6	-45.6%
15	WI01-A-0179-H-001	3460-019_3461-001	Intersection of Milward Dr & Wedgewood Way	982.3	983.5	1.1	13.5	983.3	1.0	-0.1	12.5	-0.9	-6.9%	983.1	0.7	-0.4	10.9	-2.6	-19.2%
16	WI01-A-0179-H-003	3461-007_3461-008	Intersection of Milward Dr & Dearholt Rd	981.3	982.4	1.1	20.2	982.4	1.1	0.0	18.8	-1.5	-7.3%	982.3	1.0	-0.1	16.6	-3.7	-18.2%
17	WI01-A-0179-H-006	3460-002_3460-003	Intersection of Sherwood Rd & Dearholt Rd	989.6	990.5	1.0	8.7	990.5	0.9	-0.1	8.6	-0.1	-0.9%	990.4	0.8	-0.1	8.5	-0.2	-2.3%
18	WI01-A-0180-H-002	3360-020_3360-019	Intersection of Odana Rd & Segoe Rd	1005.9	1006.6	0.7	7.7	1006.5	0.6	-0.1	6.9	-0.8	-10.6%	1006.5	0.6	-0.1	5.3	-2.4	-31.2%
19	WI01-A-0187-H-006	3359-006_3359-008	Intersection of Tokay Blvd & Whitney Way	1018.7	1018.8	0.1	24.6	1018.8	0.1	0.0	21.9	-2.7	-11.1%	1018.8	0.1	0.0	17.7	-6.9	-28.1%
20	WI01-A-0183-H-001	3160-002_BC_JCT_21	Odana Rd, approximately 325-ft west of Medical Circle	1028.6	1029.5	0.9	134.9	1029.4	0.8	-0.1	121.1	-13.8	-10.2%	1029.3	0.7	-0.2	102.9	-32.0	-23.7%
21	PD_3362-020	WW_GW_07_WW_GW_09	Orchard Ridge Valley Park Pond	979.0	979.8	0.8	119.3	979.6	0.6	-0.1	116.0	-3.3	-2.8%	979.4	0.4	-0.4	110.3	-9.0	-7.5%
22	WI01-B-0184-H-010	3261-016_3261-017	Intersection of Shroeder Rd & Shroeder Ct	1002.0	1002.1	0.1	52.2	1002.1	0.1	0.0	48.3	-3.9	-7.6%	1002.1	0.1	0.0	40.7	-11.5	-22.0%
23	WI01-B-0184-H-000	3362-012_3362-015	Intersection of Whitney Way & Gilbert Rd	981.7	982.2	0.5	150.5	982.1	0.5	0.0	141.3	-9.2	-6.1%	982.0	0.4	-0.1	125.4	-25.1	-16.7%
24	WI01-B-0181-H-001	3365-013_3365-024	Tolman Ter low-point, approximately 425-ft east of Gilbert Rd	1038.6	1039.8	1.2	4.5	1039.6	1.1	-0.1	4.3	-0.1	-3.0%	1039.4	0.9	-0.3	4.1	-0.3	-6.9%
25	WI01-B-0182-H-006	3164-028_3164-008	Intersection of Piping Rock Rd & Manhasset Pl	1004.9	1005.1	0.2	23.7	1005.1	0.2	0.0	22.3	-1.4	-6.1%	1005.1	0.2	0.0	20.0	-3.7	-15.4%

Appendix E. Willow Creek Phase I Analysis

Memo

To: Caroline Burger, PE – City of Madison

From: Alistair Hancox, PE and Eric Thompson, PE – MSA Professional Services

Subject: Willow Creek – Green Infrastructure Assessment

Date: September 21, 2021

PURPOSE

This memorandum summarizes results of modeling simulations of the installation of Distributed Green Infrastructure (DGI) uniformly throughout the Willow Creek watershed for purposes of reducing the frequency and/or severity of flooding. Specific to this evaluation, DGI means installation of practices designed to reduce runoff volumes during rainfall events. While this implies infiltration of runoff, for this event-based study, the target volumes were simply retained in the upper watershed areas and not allowed to pass downstream.

METHODOLOGY

The City of Madison, in conjunction with other consultants, had conducted green infrastructure assessments in studies predating MSA's work in the Willow Creek Watershed. These prior studies all utilized a traditional hydrology model which divided each sub-watershed up into Directly Connected Impervious Area (DCIA), Unconnected Impervious Area (UCIA), and Pervious Area. To estimate the benefits of green infrastructure, and simulate the interception of flow from impervious areas and the subsequent diversion of these flows to pervious areas where they were allowed a chance to infiltrate, a fraction of the estimated existing DCIA was converted to UCIA (10% and 25%, for the respective simulations).

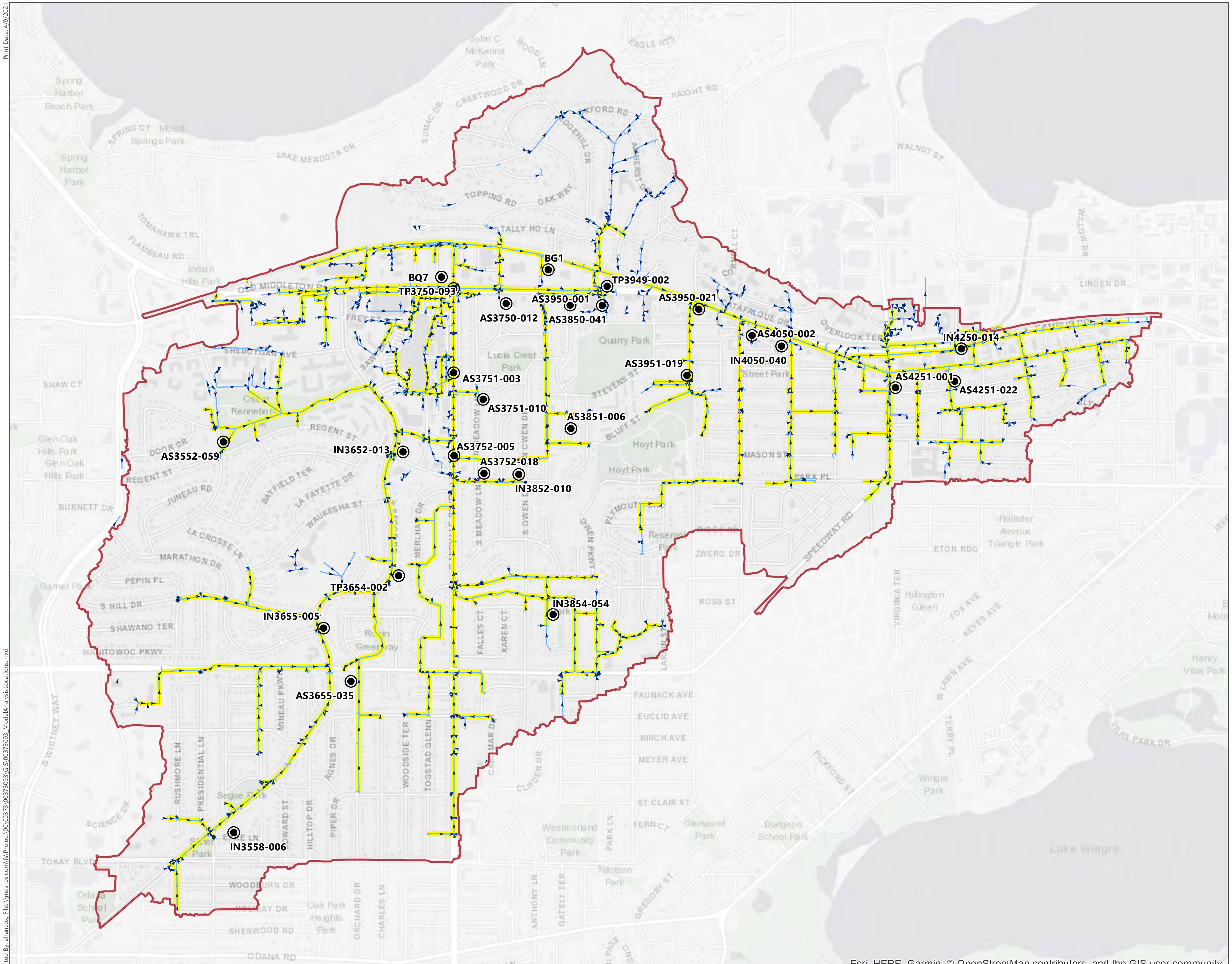
The Willow Creek Watershed Study model was built using a Rain-on-Grid hydrologic method which generates runoff everywhere throughout the catchment simultaneously, depending on the unique ground cover and infiltration parameters of each individual grid cell. Impervious areas cannot be designated by the user as either DCIA or UCIA and so allocations could not be reapportioned as they were with the prior studies. To estimate the effects of a 10% and 25% reduction in DCIA, MSA built a greatly simplified version of the watershed model, reducing all input to a single traditional-hydrology 1-D node representing the entire 1,913-acre watershed. This model was calibrated by adjusting the subcatchment width parameter until runoff volumes matched, within reason, those of the full detail calibrated rain-on-grid Existing Conditions model. The single 1-D node model was then adjusted to convert 10% and 25% of DCIA to UCIA and solved to determine the resulting reduction in total watershed runoff volume for the 50% AEP, 10% AEP, and 1% AEP (a total of six scenarios). The single 1-D node model was then set aside from this study and the full detail rain-on-grid model was modified to include distributed 1-D hydraulic storage nodes distributed proportionately throughout the study area such that their cumulative volume equaled the volume reduction reported for each scenario evaluated using the single 1-D node model.

RESULTS

Attached are detailed tabulated results for peak elevation and peak flow rates at 28 selected locations within the rain-on-grid Willow Creek model. The locations were chosen to provide an even distribution throughout the watershed, as well as at key locations where known flooding occurs.

The Willow Creek XP-SWMM model results did show some instabilities in 1D results at some of the nodes and links analyzed during this assessment. The result of such instabilities was for some nodes to show greater flood depths during the greater DCIA disconnected scenarios. These instances were reviewed, and it was determined that these conditions result from specific issues associated with flood conditions occurring within the model during selected flood event resulting from specific physical conditions that the model is predicting to occur and are independent of the DGI analysis. An unfortunate effect of these instabilities is that without interpretation they provide misleading results. For purposed of this study, those nodes should be ignored.

In general, the Flood Elevation Results show minor improvements throughout the watershed for 10% DCIA disconnection and slightly greater improvements for 25% DCIA disconnection. Improvements were most evident during the 50% AEP (smaller) event and were somewhat negligible for the 1% AEP (large) event.



Model Analysis Locations

WIllow Creek Watershed Study

City of Madison

Willow Creek Watershed

Existing Stormwater Pipe

Modeled Pipes in XPSWMM

Proposed Analysis Location (27)

Data Sources:
Storm System: City of Madison
Proposed 96" Pipe: KL Engineering

MSA

0 0.125 0.25 Miles

Print Date: 4/9/2021

Green Infrastructure Analysis - Willow Creek Watershed Study

Flood Elevation Results

Node	Ground Elevation	Existing						10% DCIA Disconnected						25% DCIA Disconnected					
		2-Year		10-Year		100-Year		2-Year		10-Year		100-Year		2-Year		10-Year		100-Year	
		Elevation	Depth	Elevation	Depth	Elevation	Depth	Elevation	Change	Elevation	Change	Elevation	Change	Elevation	Change	Elevation	Change	Elevation	Change
IN3854-054	986.74	984.43	0.00	986.75	0.01	988.09	1.35	984.21	0.0	986.18	0.0	988.10	0.0	983.83	0.0	984.66	0.0	988.02	-0.1
IN3558-006	983.90	983.16	0.00	985.48	1.58	985.98	2.08	983.08	0.0	985.43	-0.1	985.98	0.0	983.01	0.0	985.40	-0.1	985.94	0.0
AS3655-035	938.30	937.13	0.00	939.80	1.50	940.66	2.36	937.14	0.0	939.80	0.0	940.65	0.0	937.16	0.0	939.63	-0.2	940.62	0.0
AS3552-059	931.16	932.51	1.35	933.52	2.36	934.17	3.01	932.37	-0.1	933.42	-0.1	934.13	0.0	932.23	-0.3	933.16	-0.4	934.07	-0.1
IN3655-005	926.50	925.52	0.00	927.34	0.84	928.03	1.53	925.31	0.0	927.31	0.0	928.04	0.0	925.05	0.0	927.31	0.0	927.85	-0.2
TP3654-002	911.21	911.66	0.45	912.25	1.04	912.94	1.73	911.54	-0.1	912.26	0.0	912.94	0.0	911.33	-0.3	912.01	-0.2	912.81	-0.1
IN3652-013	901.54	903.23	1.69	904.40	2.86	904.92	3.38	903.06	-0.2	904.24	-0.2	904.92	0.0	902.65	-0.6	903.62	-0.8	904.76	-0.2
AS3851-006	897.67	899.55	1.88	899.98	2.31	900.71	3.04	899.54	0.0	899.98	0.0	900.70	0.0	899.50	0.0	899.94	0.0	900.67	0.0
AS3752-018	893.84	894.77	0.93	896.15	2.31	897.14	3.30	894.51	-0.3	896.04	-0.1	897.14	0.0	891.85	-0.9	893.57	-2.3	896.76	-0.4
IN3852-010	893.63	894.72	1.09	896.07	2.44	897.14	3.51	894.54	-0.2	895.87	-0.2	897.15	0.0	893.77	-1.0	894.85	-1.2	896.76	-0.4
AS3752-005	893.09	894.76	1.67	895.73	2.64	896.76	3.67	894.58	-0.2	895.69	0.0	896.74	0.0	888.79	-1.7	889.12	-2.6	896.29	-0.5
AS3751-003	890.10	891.16	1.06	891.90	1.80	892.65	2.55	891.01	-0.2	891.86	0.0	892.64	0.0	884.90	-1.1	887.61	-1.8	892.29	-0.4
AS3751-010	889.77	891.69	1.92	892.48	2.71	893.34	3.57	891.49	-0.2	892.43	0.0	893.32	0.0	890.69	-1.0	891.30	-1.2	892.93	-0.4
AS3951-019	889.35	890.69	1.34	890.87	1.51	891.09	1.74	890.70	0.0	890.86	0.0	891.09	0.0	890.68	0.0	890.87	0.0	891.09	0.0
TP3750-093	884.99	889.25	4.26	890.61	5.62	889.31	4.32	890.71	1.5	888.23	-2.4	891.63	2.3	879.34	-4.3	887.50	-3.1	889.15	-0.2
AS3750-012	884.20	886.62	2.42	887.21	3.01	887.92	3.72	886.46	-0.2	887.16	0.0	887.92	0.0	880.80	-2.4	882.23	-3.0	887.62	-0.3
BQ7	883.35	886.61	3.26	887.48	4.13	888.56	5.21	885.89	-0.7	887.45	0.0	888.51	0.0	881.10	-3.3	884.34	-3.1	888.10	-0.5
BG1	881.57	881.88	0.31	883.36	1.79	883.83	2.26	881.67	-0.2	883.05	-0.3	883.83	0.0	876.46	-0.3	881.69	-1.7	883.56	-0.3
TP3949-002	880.18	880.45	0.27	882.35	2.17	882.77	2.59	880.80	0.4	882.31	0.0	882.77	0.0	874.69	-0.3	880.49	-1.9	882.55	-0.2
AS3950-021	879.78	880.94	1.16	882.46	2.68	881.86	2.08	881.04	0.1	882.22	-0.2	882.10	0.2	873.43	-1.2	880.92	-1.5	882.53	0.7
AS3950-001	879.30	881.07	1.77	882.71	3.41	883.32	4.02	880.22	-0.8	882.64	-0.1	883.29	0.0	876.56	-1.8	880.12	-2.6	882.91	-0.4
AS3850-041	879.03	881.07	2.04	882.76	3.73	883.34	4.31	880.24	-0.8	882.69	-0.1	883.37	0.0	876.53	-2.0	880.14	-2.6	882.98	-0.4
AS4050-002	875.54	876.40	0.86	878.10	2.56	880.08	4.54	874.45	-0.9	877.93	-0.2	879.59	-0.5	873.00	-0.9	876.43	-1.7	878.78	-1.3
IN4050-040	874.68	873.98	0.00	878.04	3.36	880.02	5.34	873.84	0.0	877.29	-0.7	879.48	-0.5	872.66	0.0	873.44	-3.4	878.69	-1.3
AS4251-001	871.38	868.24	0.00	870.93	0.00	874.45	3.07	867.81	0.0	870.44	0.0	873.67	-0.8	866.85	0.0	868.23	0.0	872.80	-1.7
AS4251-022	870.26	866.59	0.00	869.74	0.00	871.00	0.74	866.26	0.0	868.32	0.0	870.28	-0.7	865.09	0.0	866.72	0.0	870.26	-0.7
IN4250-014	857.08	859.28	2.20	860.14	3.06	865.60	8.52	859.19	-0.1	859.83	-0.3	864.48	-1.1	858.75	-0.5	859.59	-0.5	861.36	-4.2

* Denotes a node were minor instabilities have occurred in the 1D model results.
* Flood Depths Reduced due to DCIA Reduction.
* Flood Depths Increased due to DCIA Reduction.

Green Infrastructure Analysis - Willow Creek Watershed Study

Pipe Flow Results

	Existing			10% DCIA Disconnected						25% DCIA Disconnected					
	2-Year	10-Year	100-Year	2-Year		10-Year		100-Year		2-Year		10-Year		100-Year	
Pipe	Flow (cfs)	Flow (cfs)	Flow (cfs)	Flow (cfs)	Change	Flow (cfs)	Change	Flow (cfs)	Change	Flow (cfs)	Change	Flow (cfs)	Change	Flow (cfs)	Change
IN3558-006_IN3558-001	100.9	132.2	132.0	89.3	-11%	129.4	-2%	128.7	-3%	86.7	-14%	129.0	-2%	130.1	-1%
IN3854-054_DT3854-006	60.2	90.4	102.3	56.3	-6%	83.8	-7%	102.3	0%	49.9	-17%	63.2	-30%	101.7	-1%
AS3655-035_DT3655-036	9.3	20.0	20.4	9.3	0%	20.0	0%	20.3	-1%	9.4	1%	19.9	0%	20.2	-1%
AS3552-059_AS3552-057	15.6	15.9	16.0	14.8	-5%	15.9	0%	15.7	-2%	12.3	-21%	15.5	-3%	16.1	1%
IN3655-005_IN3654-041	317.4	350.2	349.6	308.7	-3%	351.5	0%	351.3	0%	296.0	-7%	349.4	0%	353.4	1%
TP3654-002_IN3653-012	267.7	266.7	264.5	266.7	0%	265.9	0%	260.8	-1%	271.7	2%	264.7	-1%	261.1	-1%
IN3652-013_TEE5	312.1	324.8	288.7	303.5	-3%	313.8	-3%	279.5	-3%	290.1	-7%	332.8	2%	362.2	25%
AS3851-006_AS3851-001	9.8	10.3	10.6	9.9	1%	10.3	0%	10.6	0%	11.5	17%	12.1	18%	10.7	0%
AS3752-018_AS3752-056	29.6	49.9	51.9	30.2	2%	48.9	-2%	51.1	-1%	31.0	5%	38.3	-23%	51.1	-1%
IN3852-010_AS3752-022	18.2	22.0	22.5	18.7	3%	21.9	-1%	22.8	1%	18.9	4%	22.1	0%	22.8	1%
AS3752-005_TP3752-004	387.6	368.4	363.9	338.8	-13%	380.3	3%	399.0	10%	106.1	-73%	69.0	-81%	264.8	-27%
AS3751-003_AS3751-002	555.7	539.7	552.7	539.7	-3%	540.9	0%	552.7	0%	183.9	-67%	316.7	-41%	550.3	0%
AS3751-010_AS3751-017	9.5	9.2	8.5	9.4	-1%	9.3	1%	8.6	1%	8.5	-11%	9.1	-1%	9.0	5%
AS3951-019_AS3951-018	3.5	2.8	2.9	3.4	-2%	2.8	0%	3.0	1%	3.4	-3%	3.4	19%	3.0	0%
TP3750-093_TP3750-058	483.2	472.6	423.9	482.1	0%	457.3	-3%	464.0	9%	233.6	-52%	400.6	-15%	284.4	-33%
AS3750-012_AS3849-026	69.7	65.8	41.6	75.4	8%	50.0	-24%	54.7	32%	15.7	-78%	31.5	-52%	53.3	28%
BQ6_BQ5	39.6	38.5	37.7	38.0	-4%	38.5	0%	37.9	0%	6.3	-84%	20.4	-47%	37.9	1%
BG1_TP3849-005	440.9	431.5	428.6	489.5	11%	434.8	1%	430.8	1%	331.0	-25%	442.1	2%	441.8	3%
TP3949-002_TP3949-001	540.5	498.2	517.0	515.0	-5%	501.9	1%	528.0	2%	354.0	-35%	475.3	-5%	502.1	-3%
AS3950-021_TP4050-008	586.2	659.1	646.4	547.8	-7%	640.8	-3%	674.1	4%	370.7	-37%	484.2	-27%	658.4	2%
AS3950-001_IN3949-010	30.8	30.8	31.0	25.4	-18%	30.1	-2%	30.1	-3%	20.6	-33%	30.4	-1%	30.4	-2%
AS3850-041_IN3850-036	102.0	123.0	84.9	102.3	0%	117.9	-4%	89.4	5%	21.2	-79%	106.8	-13%	135.6	60%
AS4050-002_TP4150-004	585.3	560.7	537.5	525.3	-10%	667.1	19%	589.6	10%	404.7	-31%	571.9	2%	540.2	1%
AS4050-002_TP4150-004.1	542.8	769.9	717.4	536.8	-1%	764.7	-1%	721.5	1%	424.2	-22%	512.4	-33%	713.3	-1%
AS4251-001_AS4251-033	597.1	746.8	817.6	570.6	-4%	680.8	-9%	820.1	0%	451.8	-24%	571.5	-23%	743.2	-9%
AS4251-022_AS4251-016	643.9	731.2	846.2	612.0	-5%	715.6	-2%	837.4	-1%	479.9	-25%	641.3	-12%	796.9	-6%
IN4250-014_IN4250-013	4.8	4.1	6.3	6.1	27%	4.6	11%	6.4	2%	4.8	-1%	5.4	31%	6.3	0%

* Denotes a pipe were minor instabilities have occurred in the 1D model results.
* Flows Reduced due to DCIA Reduction.
* Flows Increased due to DCIA Reduction.

Memo

To:	Lauren Striegl P.E., City of Madison
From:	Alistair Hancox P.E. & Eric Thompson P.E., MSA Professional Services
Subject:	Willow Creek Watershed Study - Hydraulic Model Instabilities
Date:	September 24, 2021

The purpose of this memo is to discuss the variability in modeling results across Existing and Proposed condition scenarios for the Willow Creek Watershed Study XP SWMM model. It has been known to MSA, and we have shared with the City, that 1D modeling results recorded at some locations within the model have identified rapid changes in pipe flow and water elevation. The model calibration process and the August 2018 simulation results give us confidence that the model is producing reliable outputs, irrespective of the 1D instabilities. The model results were also discussed with multiple Innovyze technical experts who also saw no reason for concern.

However, with the hope of producing a better product for the City, we have strived to reduce the occurrences of the instabilities as much as possible. Variables modified to increase stability were:

- Reducing the 1D Hydraulic timestep from 60 seconds to 1 second. Additional timesteps were tested (5 seconds and 0.5 seconds), but the best balance of model accuracy and simulation time was achieved with the 1 second timestep.
- Checking “Additional mass balance iteration” within the 2D Job Controls.

The main changes to previous model results can be observed in the smaller storm events. On the following page, images of both the Existing Conditions 2-year and 100-year simulation change in peak depth are shown. Raster depth changes of less than 0.1 ft are not shown on these maps. It should be reiterated that these images are not inundation maps, but the difference between the two versions of the peak inundation depth raster. We can see in Image 1 that the 2-year event produces less inundation along Midvale Blvd and University Ave, when incorporating the above modeling changes. The changes had little to no influence on the 100-year results. As the storm frequency reduced, the difference became less significant.

The 1D elevation results from the University Ave/Shorewood intersection give greater insight into why the peak inundation depths may have changed. See Figures 1 and 2 below. These show that with the smaller 1D timestep there is less oscillation between the spill elevation of the node (Ground Elevation) and the surcharged water level. There is a significant reduction in peak elevation during the 2-year event, but not during the 100-year event. The oscillations which occur when the node begins to surcharge are smoothed slightly in the revised model which is then reflected on the inundation depth result file.

As mentioned above, decreasing the 1D timestep does come at a cost to model solve time, which is why this was initially set high, at 60 seconds. Reducing this to 1 second, seems to add approximately 20 – 30% to model solve time.

Image 1 – Existing Condition 2-year Peak Depth Change

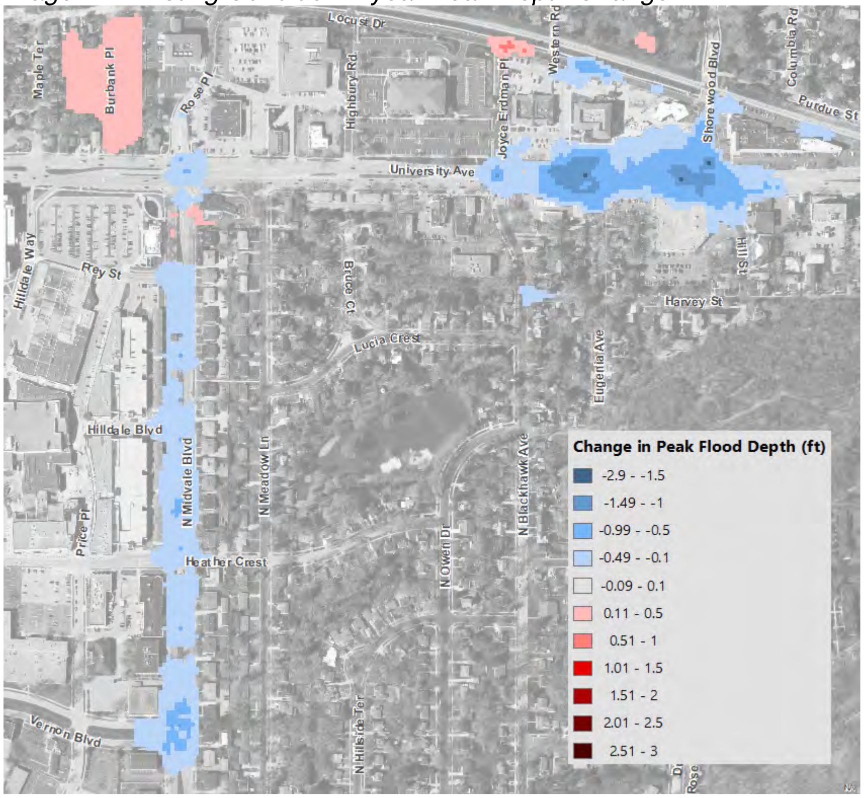


Image 2 – Existing Condition 100-year Peak Depth Change

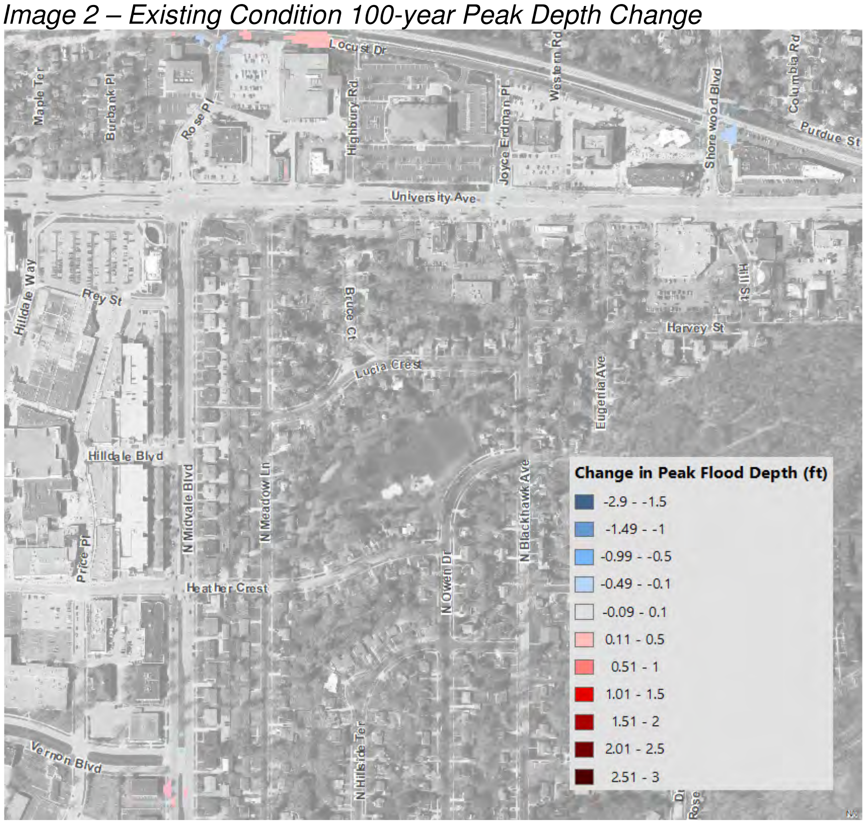


Figure 1 - Water Elevation Result at Node P900E (University Ave/Shorewood Blvd) during the 2-year Event

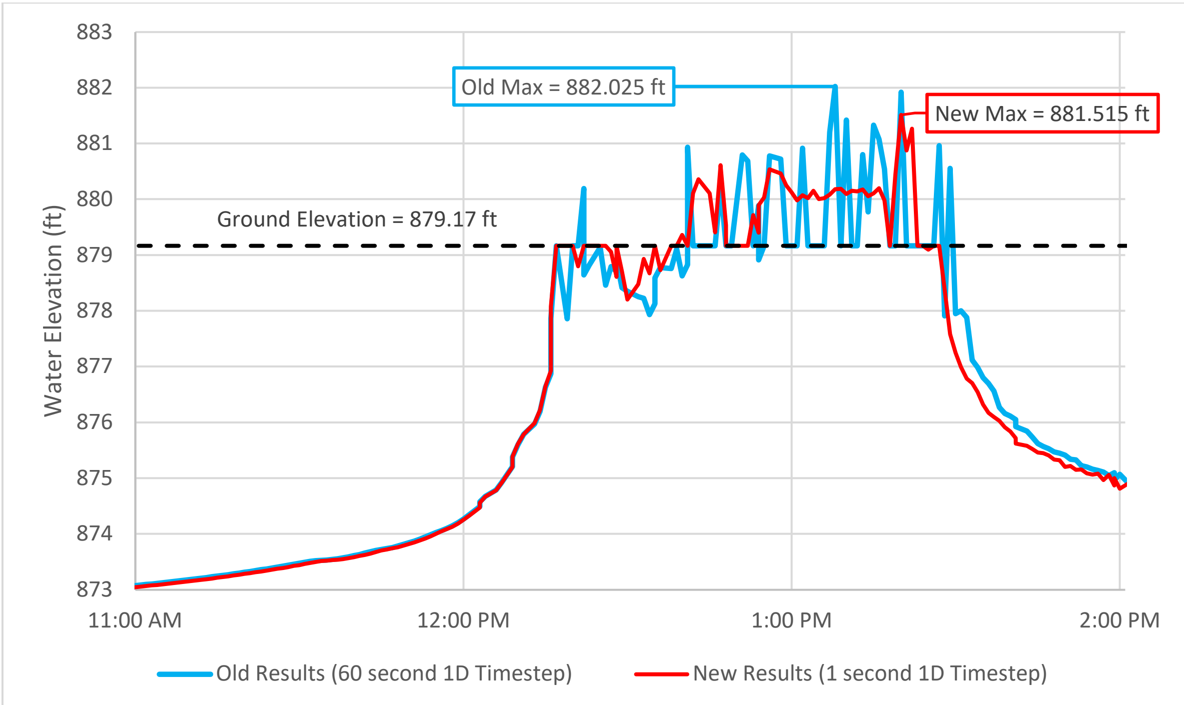


Figure 2 - Water Elevation Result at Node P900E (University Ave/Shorewood Blvd) during the 100-year Event