Iowa Flood Resilient Communities Cohort: Potential Flood Mitigation Solutions for Hagerman and Amy Drive Neighborhoods Muscatine, IA

by

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IOWA FLOOD RESILIENT COMMUNITIES COHORT: POTENTIAL FLOOD MITIGATION SOLUTIONS FOR HAGERMAN AND AMY DRIVE NEIGHBORHOODS MUSCATINE, IOWA

In December 2023, the American Flood Coalition (AFC) launched the Iowa Flood Resilient Communities Cohort, which provides support in accessing federal funding for flood projects. The cohort includes local officials and community partners from four Iowa communities: Columbus Junction, Dubuque, Manchester and Muscatine. AFC partnered with the Iowa Flood Center (IFC) to provide technical support in exploring flood mitigation alternatives and to assist in conceptualizing project designs that could be further pursued.

IFC provided technical assistance to the City of Muscatine in its exploration of potential detention structures and their benefits to a downstream flood-prone neighborhood. The modeling study and findings are summarized in following sections.

1. Background

Homeowners along Amy Drive in Muscatine, Iowa deal with frequent flooding. Runoff originating from three drainage areas shown in Figure 1-1 are the major contributors. The two largest contributing catchments with drainage areas of 109 and 87 acres are situated to the west of the flood prone area. Runoff generated in these catchments typically flows east along Hagerman Drive before entering Amy Drive. Local runoff from the 82-acre catchment overlapping Amy Drive, Hagerman Drive, and Roscoe Avenue also contributes to flooding. As upstream runoff is conveyed along Roscoe Avenue, it is funneled to the west end of Amy Drive. The runoff flows east through Amy Drive and easily enters driveways and accumulates in the lower levels of homes, as shown in Figure 1-2. A typical home on Amy Drive is shown in Figure 1-3. Runoff continues flowing downstream through Amy Drive and into Iowa Field, shown in Figure 1-4, before continuing to the east.

Like many older municipalities, much of Muscatine's sewer system began as combined sewer, wherein stormwater and sanitary sewage are collected together and conveyed to the city's wastewater treatment plant (WWTP). This type of combined system is problematic because it is frequently overwhelmed by severe storms, allowing sanitary waste to overflow into streets and properties. Additionally, the WWTP can be often overloaded, and stormwater debris accumulates within the system and damages pumping equipment. While the city has completed several sewer separation projects, the study area still has a combined system.

In discussions with representatives from the City of Muscatine, construction of detention storage within the study area was identified as a possible feasible flood mitigation solution. Proposed locations for these detention pond projects are shown in Figure 1-5. Additionally, city representatives identified these projects as being a potential solution to reduce sandy debris entering their facilities and reduce costly repairs or removal efforts. IFC agreed to develop conceptual detention structure designs and supporting hydrologic and hydraulic models and simulations.

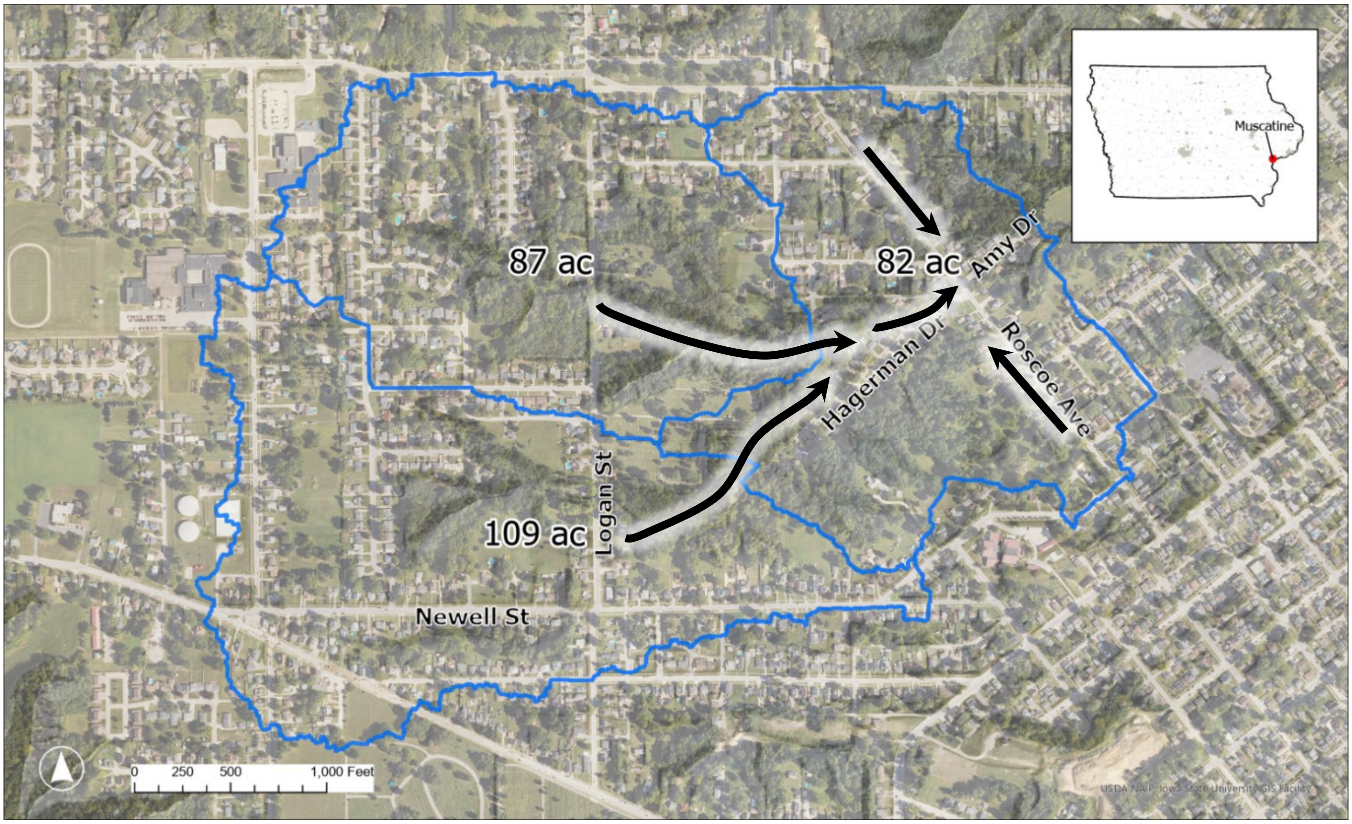

Figure 1-1. Drainage areas near flood prone Amy Drive in Muscatine, Iowa.

Figure 1-2. Runoff enters along the west end of Amy Drive and then enters driveways, flooding lower levels of homes.

Figure 1-3. Typical home along Amy Drive in Muscatine, Iowa. Garage access is at basement level, often below street grade.

Figure 1-4. A close-up view of flood-prone Amy Drive.

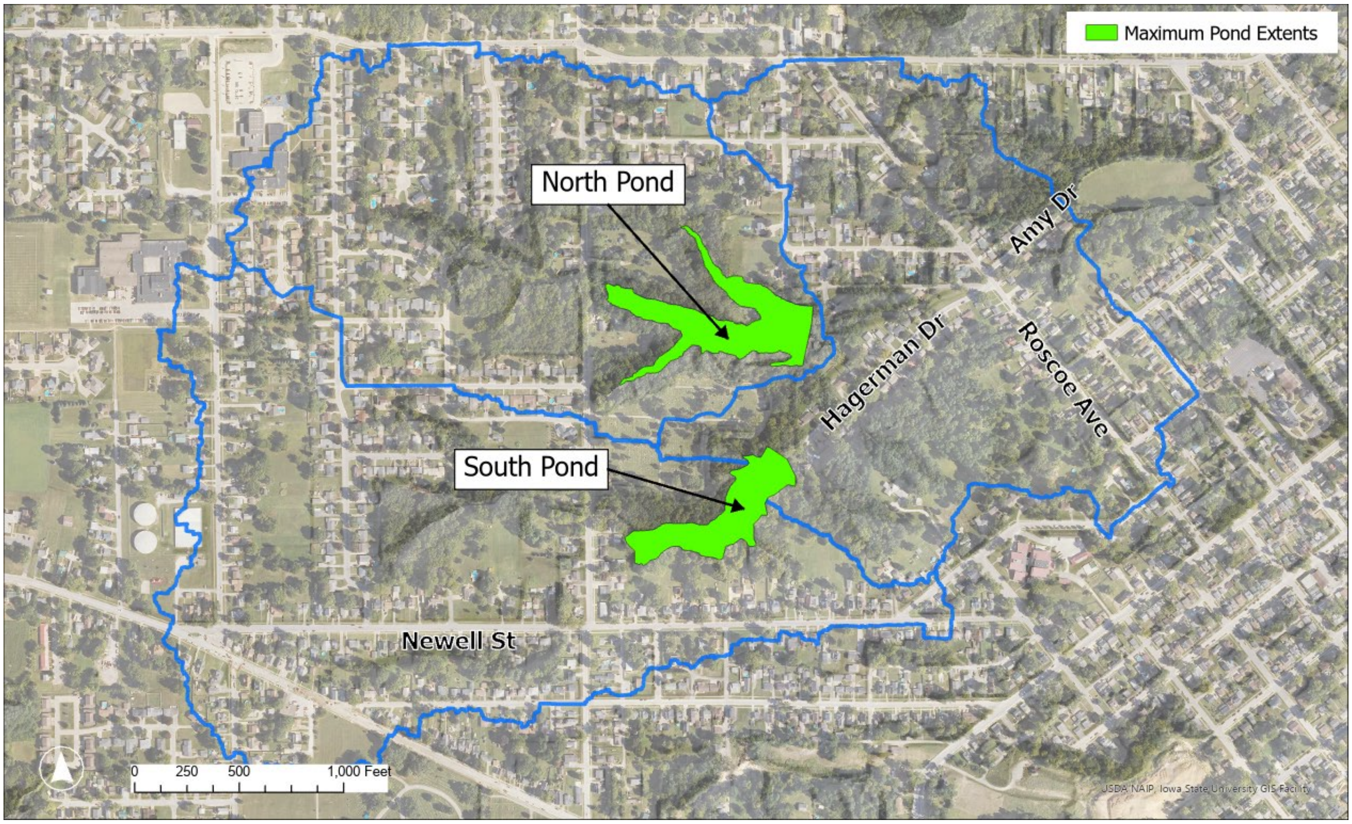

Figure 1-5. Approximate locations of detention ponds and the maximum pond extent at the embankment height.

2. Model Development

IFC utilized Computational Hydraulics Incorporated (CHI) PCSWMM software to develop a coupled one-dimensional (1D) / two-dimensional (2D) PCSWMM model to simulate rainfall-runoff, stormwater infrastructure, and overland flow processes simultaneously. Using the modeled flow from design storms, the outlet structures for each detention pond were designed.

Hydrologic Modeling

Rainfall-runoff was simulated on a 2D surface grid shown in Figure 2-1, at an average resolution of 800 square feet, a minimum of 200 square feet, and a maximum of 1400 square feet. Smaller grid elements of 200 square feet were generated along street centerlines to capture street flow, and grid elements of 800 square feet were generated through steeper sloped areas. Each subcatchment polygon shape was parameterized with an SCS Curve Number, as shown in Figure 2-2, based on underlying landcover, impervious surface, and soil types.

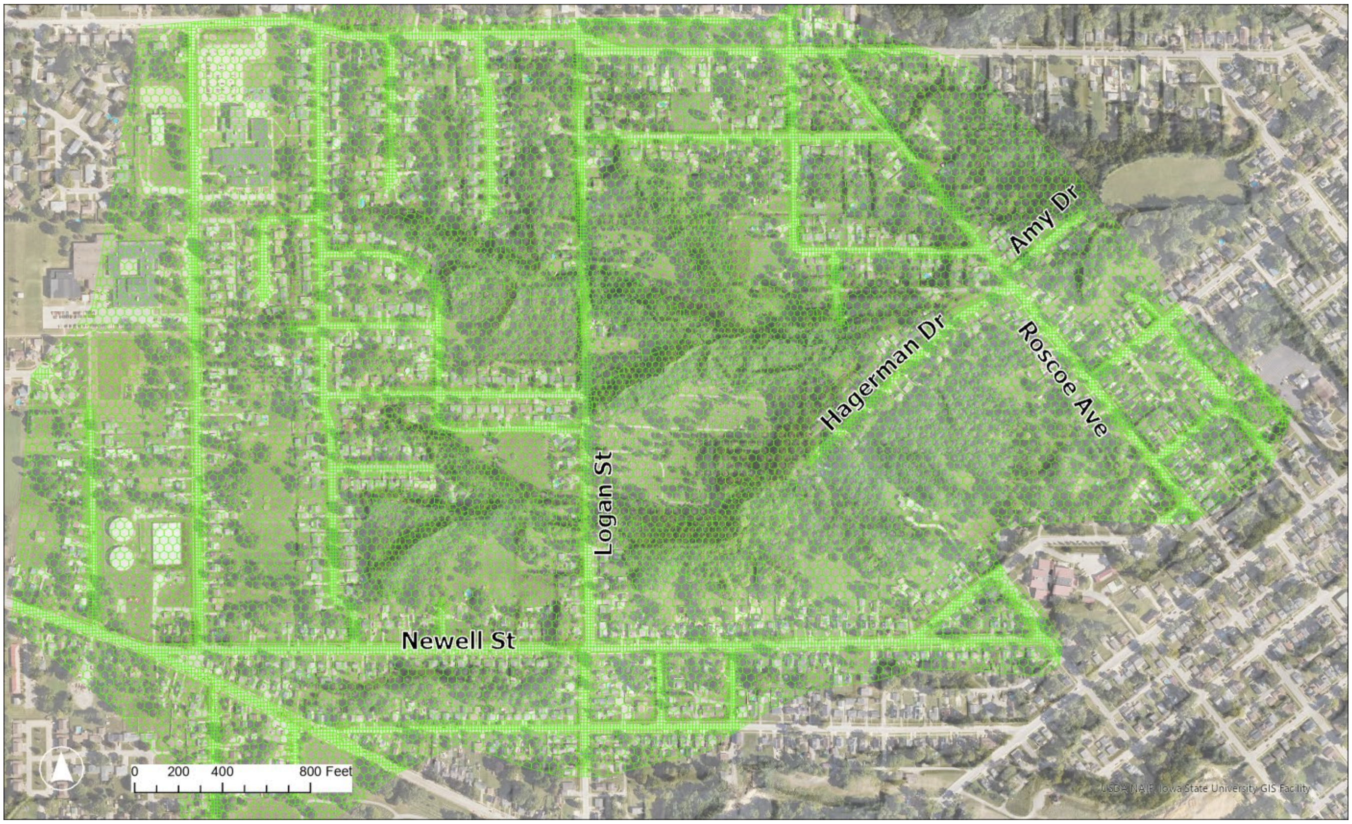

Figure 2-1. 2D grid used in simulating rainfall-runoff, smaller grid elements used to capture street flow and steep slopes.

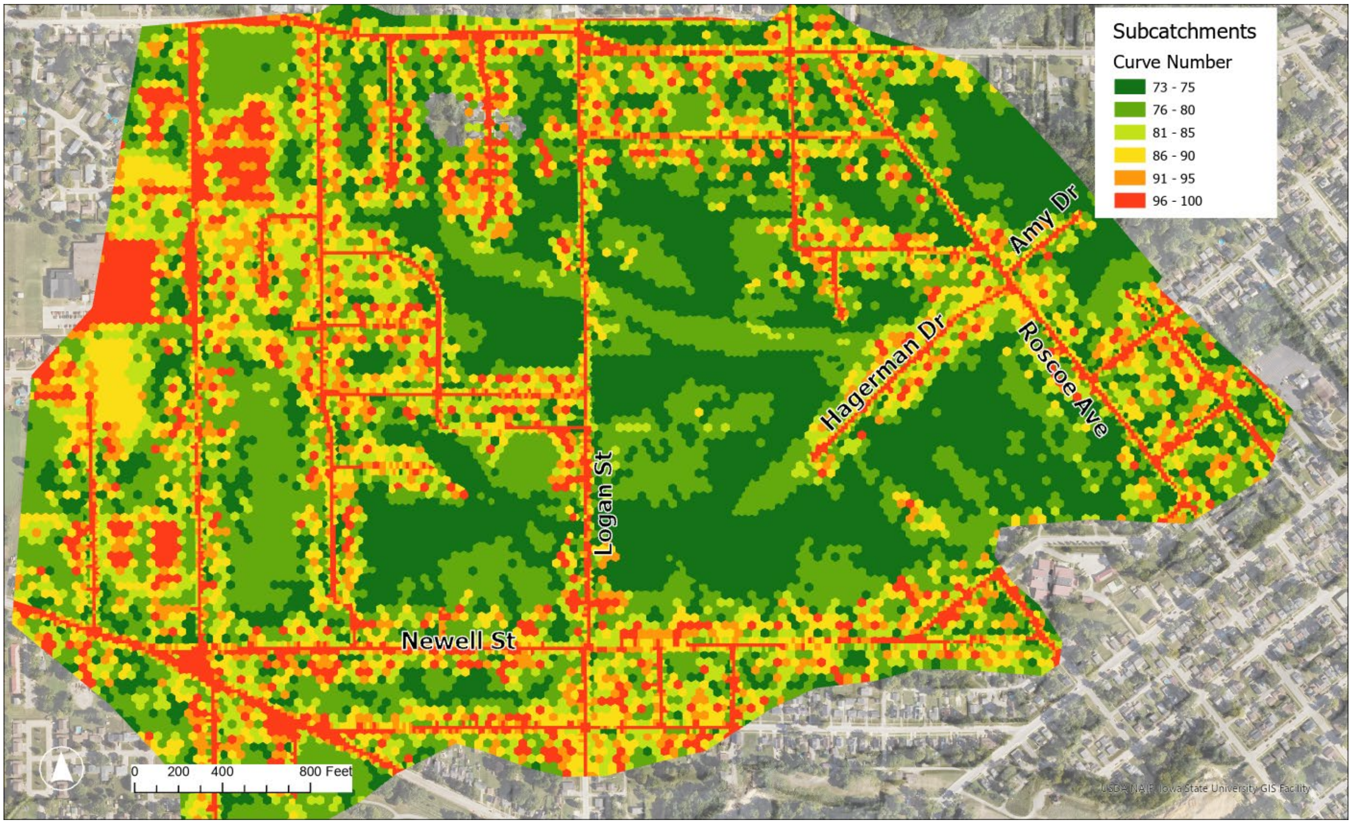

Figure 2-2. Subcatchment grid SCS curve numbers based on underlying landcover, impervious surface, and soil type.

Hydraulic Modeling

As rainfall-runoff is generated at each subcatchment, runoff can flow within the 2D grid cells based on terrain elevations. Generally, runoff flows downslope, and accumulates as it goes downstream through the watershed. Terrain elevations derived from LiDAR data collected in 2020 are shown in Figure 2-3.

An additional modeling consideration is the underlying storm water infrastructure that captures and conveys surface runoff through storm sewers. The city of Muscatine provided a geographic information system (GIS) database of their sanitary and storm water infrastructure, shown in Figure 2-4, which was incorporated into the PCSWMM model as 1D elements. Manholes, catch basins, and intake nodes, also shown in Figure 2-4, were linked to the 2D surface grid to allow interaction between the surface and subsurface sewer system.

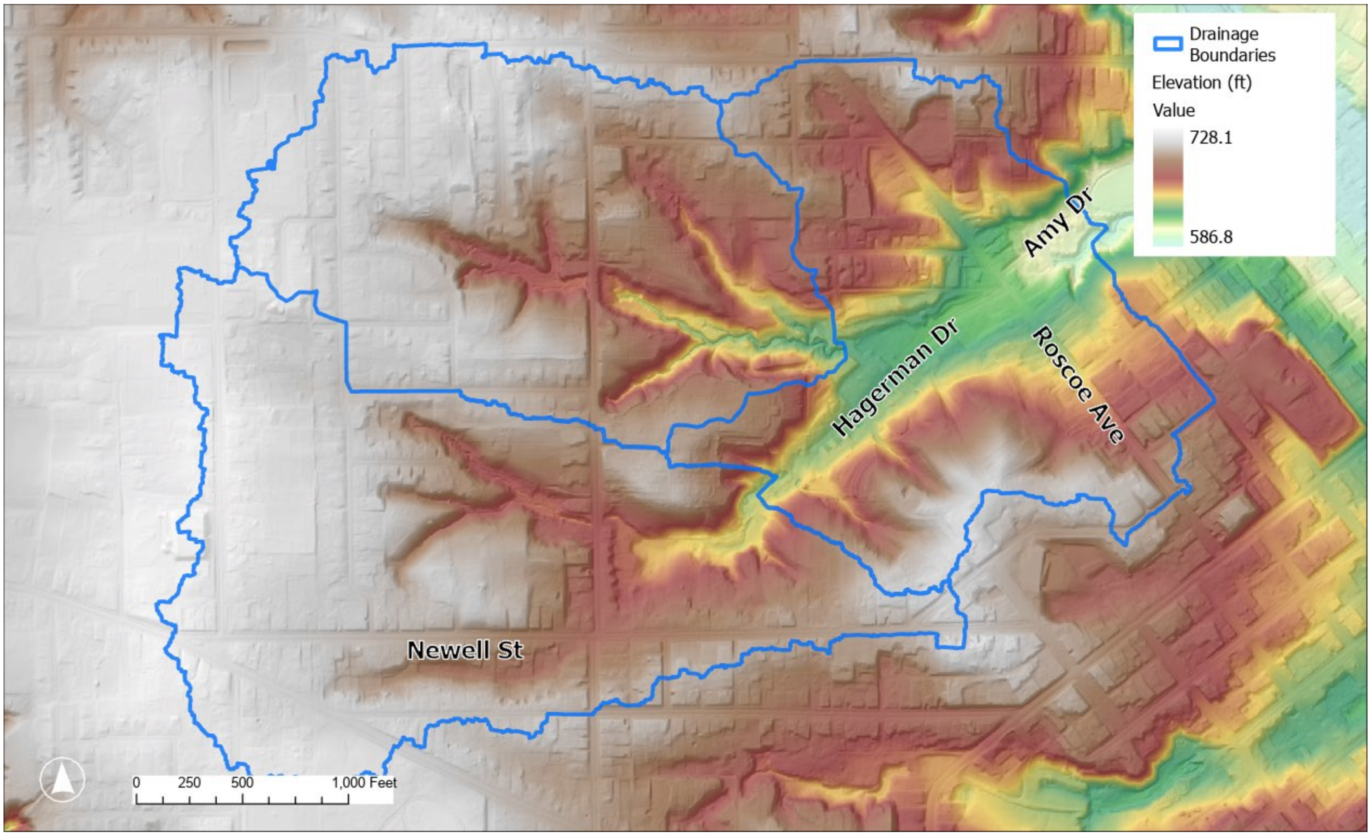

Figure 2-3. Lidar elevations used to develop the 2D surface grid.

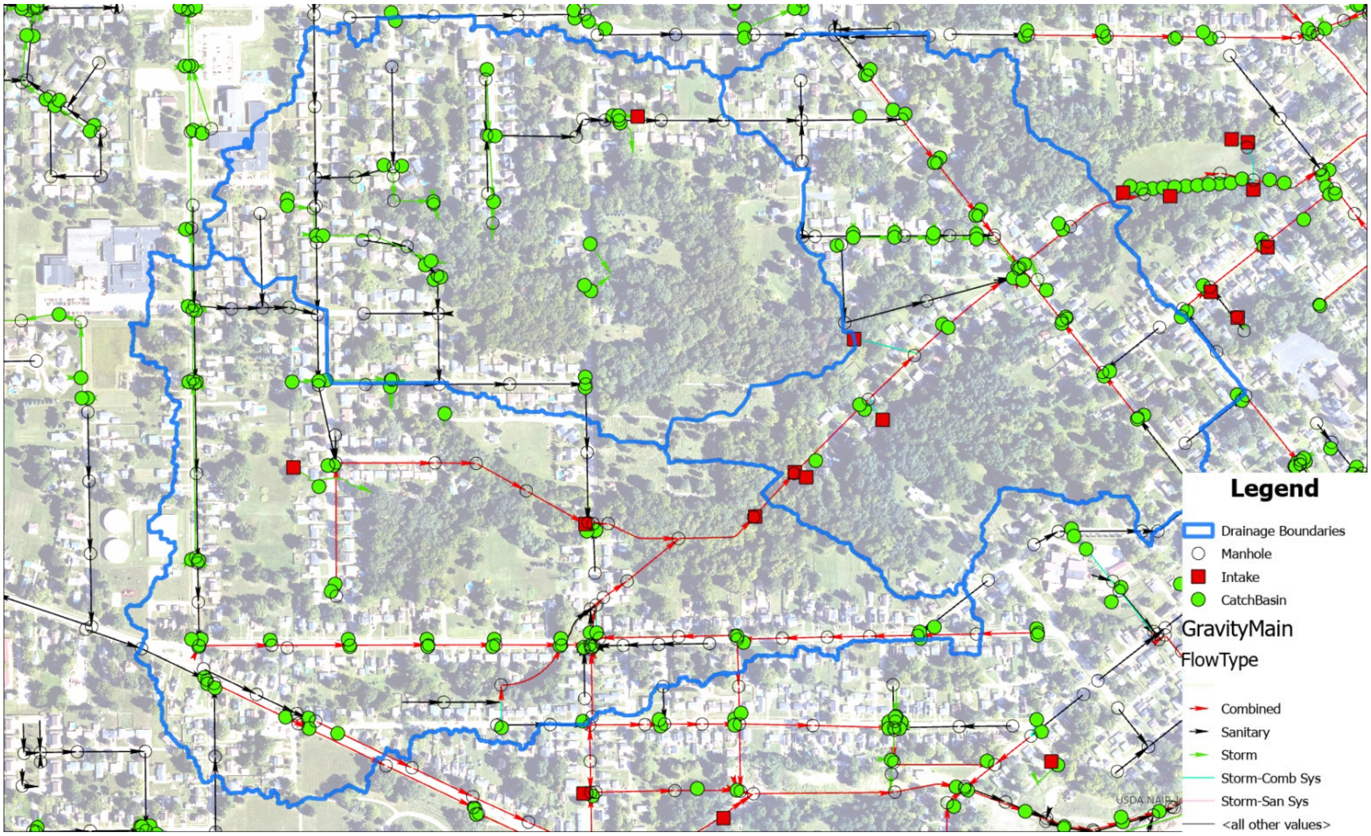

Figure 2-4. Stormwater and sanitary sewer infrastructure provided by the City of Muscatine that was incorporated in the 1D/2D PCSWMM model.

3. Design Storms

Precipitation frequency estimates over a 24-hour storm duration were provided by NOAA Atlas 14 and are shown in Table 3-1. These precipitation depths were distributed on a 24-hour Soil Conservation Service (SCS) Type II rainfall hyetograph to develop a rainfall time series for each return period, as shown in Figure 3-1. These design storm events distribute much of the rainfall over a short period near the 12th hour.

Table 3-1. NOAA Atlas 14 precipitation frequency estimates for 24-hour duration.

Return Period	1-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Annual Exceedance Probability, AEP	100	50	20	10	4	2	1
Total Depth (in)	2.63	3.07	3.86	4.57	5.63	6.52	7.47

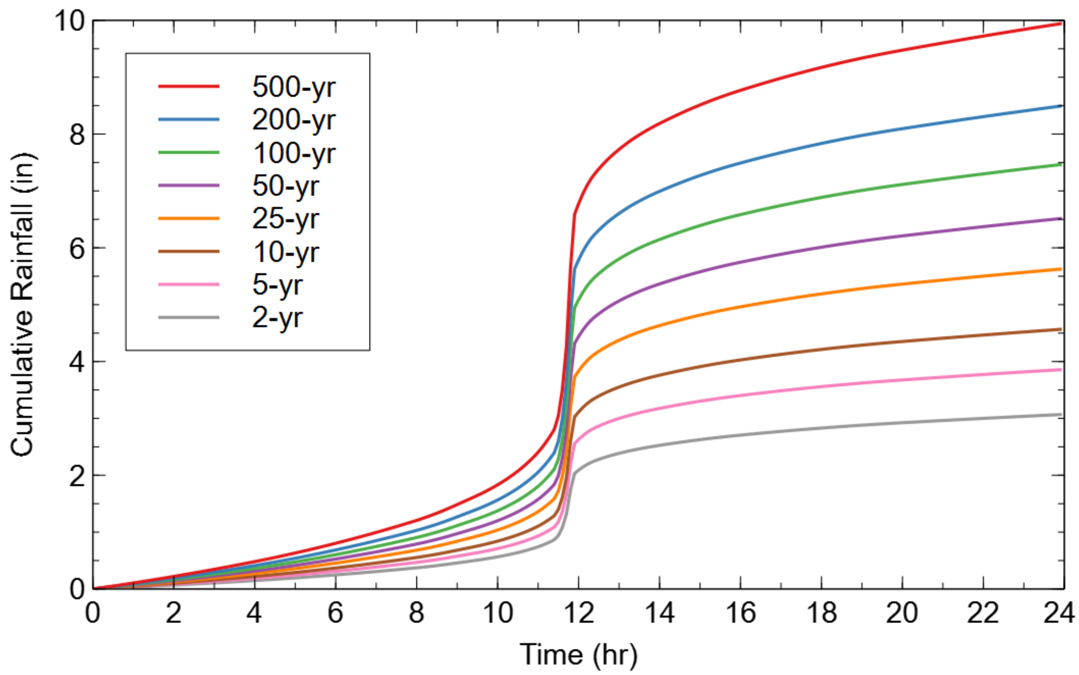

Figure 3-1. SCS 24-hr, Type II, design storm cumulative rainfall

4. Detention Projects

The proposed locations for the detention ponds and their maximum ponded extents are shown in Figure 1-5. These locations are within steep, timbered ravine areas, and would likely not be visible from surrounding properties. Their surface area and storage relationships relative to elevation were developed using GIS tools and are shown in Figure 4-1. The invert elevation of the North Pond storage is lower than the South Pond. Directly comparing surface area and storage is done by converting elevations to pond depth, as shown in Figure 4-2. The South Pond has nearly twice as much storage as the North Pond at the same depth. However, the North Pond has a smaller drainage area than the South Pond.

Using the simulated inflows (discussed in Section 5) and pond storage curves, the outlets of the pond projects were designed using HydroCAD software. Principal outlets were designed for each pond project with an emphasis on maximizing peak flow reductions across all return periods and preventing activation of the emergency spillway. The multi-stage outlets have low-flow inlets with a rock infiltration bed and a smaller diameter pipe. This type of low-flow inlet will prevent debris from entering the storm sewer system and allow for dry detention storage much of the year. At higher pond pool elevations, overflow into a larger diameter riser pipe begins and continues attenuating inflows until the emergency spillway is activated. The emergency spillways are only activated at return periods greater than the 100-year rainfall event.

Outlet pipe diameters and spillway elevations were iteratively adjusted until the performance and safety of the structures were well balanced. Conceptual designs of pond outlet structures for the North Pond and South Pond are shown in Figure 4-3 and Figure 4-4, respectively. The North Pond assumes the existing 12-inch storm sewer connecting to a surface drain near the proposed project is upgraded to a 24-inch storm sewer pipe.

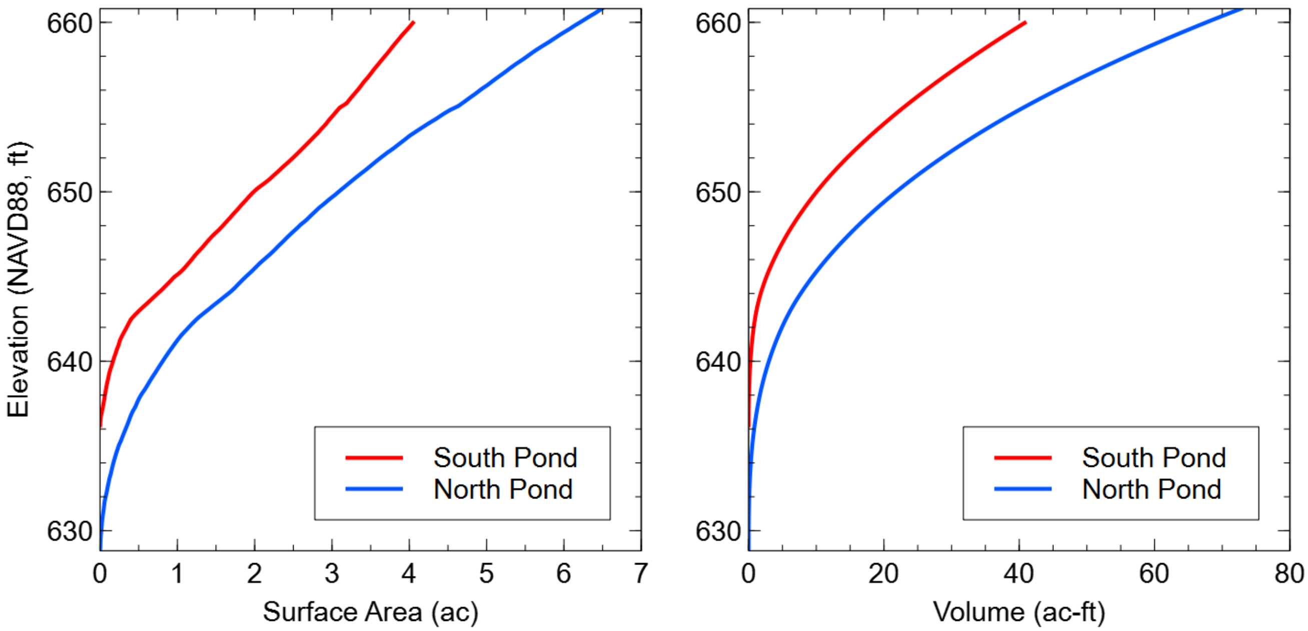

Figure 4-1. Elevation-surface area and elevation-storage relationships for the North and South Pond projects.

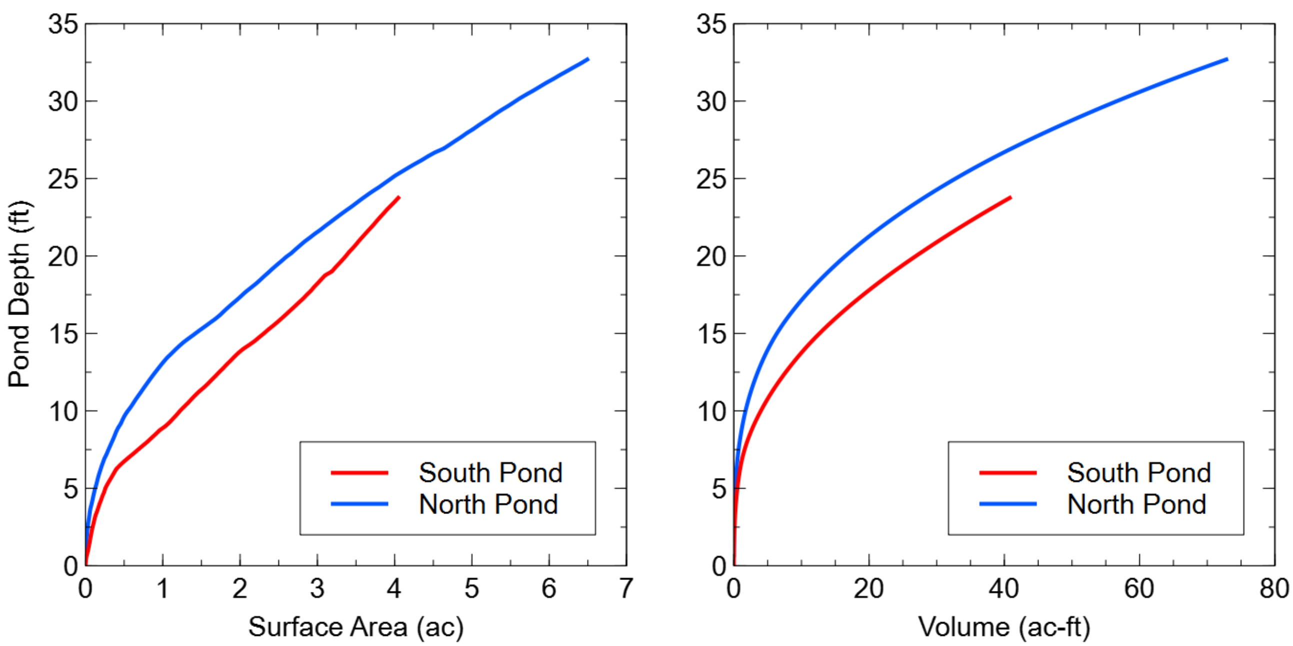

Figure 4-2. Depth-surface area and depth-storage relationships for the North and South pond projects.

Conceptual Design

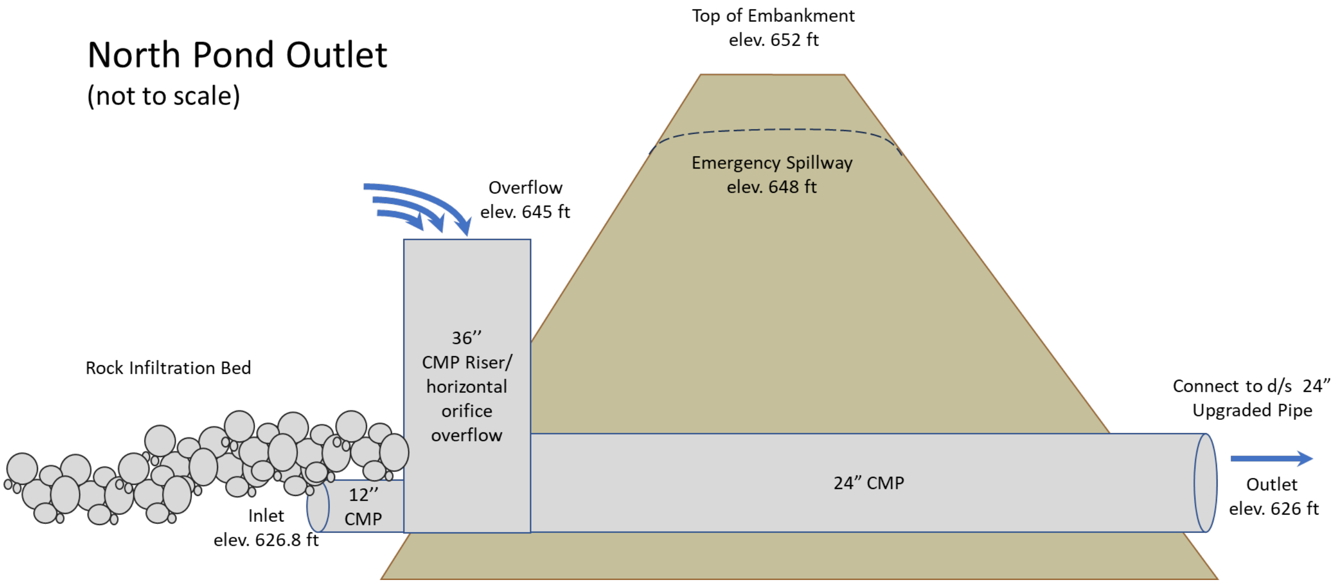

Figure 4-3. Conceptual design of the North Pond outlet structure (not to scale).

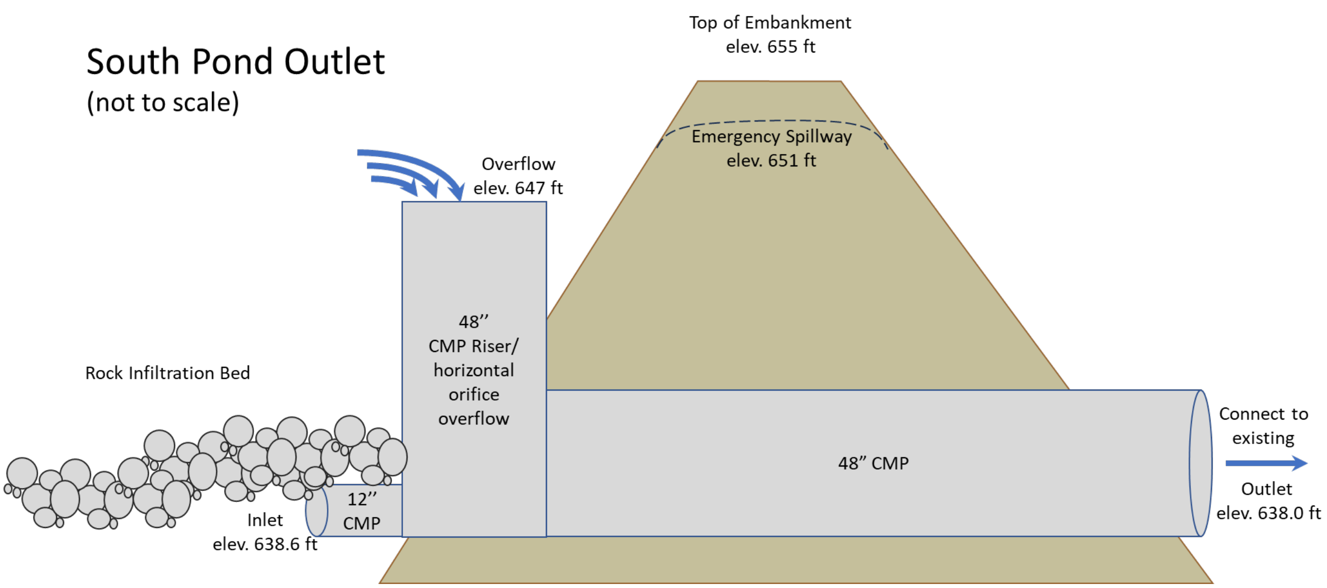

Figure 4-4. Conceptual design of the South Pond outlet structure (not to scale).

5. Simulation Results – Existing Conditions

The PCSWMM model was used to simulate the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year rainfall events. Inflow hydrographs at the potential pond project locations are shown in Figure 5-1. These flows and runoff volumes were used to design the pond outlet structures. The flow hydrographs include any storm sewer flows in addition to the surface flows. This is particularly important at the South Pond because stormwater collected by a combined sewer (shown in Figure 2-4) will likely be daylighted and will also enter the South Pond. The total storm sewer pipe flow into the South Pond for each return period is shown in Figure 5-2. The peak flow through this pipe is 88 cubic feet per second (cfs) and is reached at the 10-year event. The percentage of total inflow to the South Pond carried by the storm sewer pipe is also shown in Figure 5-2. The storm sewer system upstream of the South Pond captures approximately 40% of the total flow for the 2- through 10-year event and decreases to approximately 10% for the 500-year event.

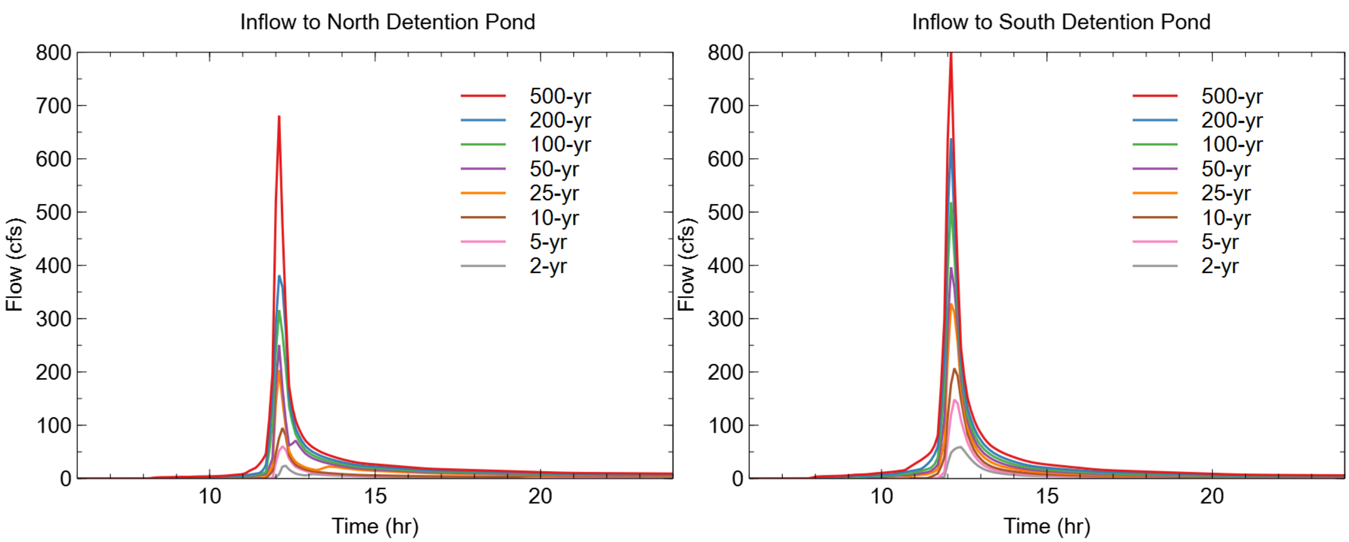

Figure 5-1. Simulated inflows into the North Pond (left) and the South Pond (right). The South Pond flows include storm sewer flows.

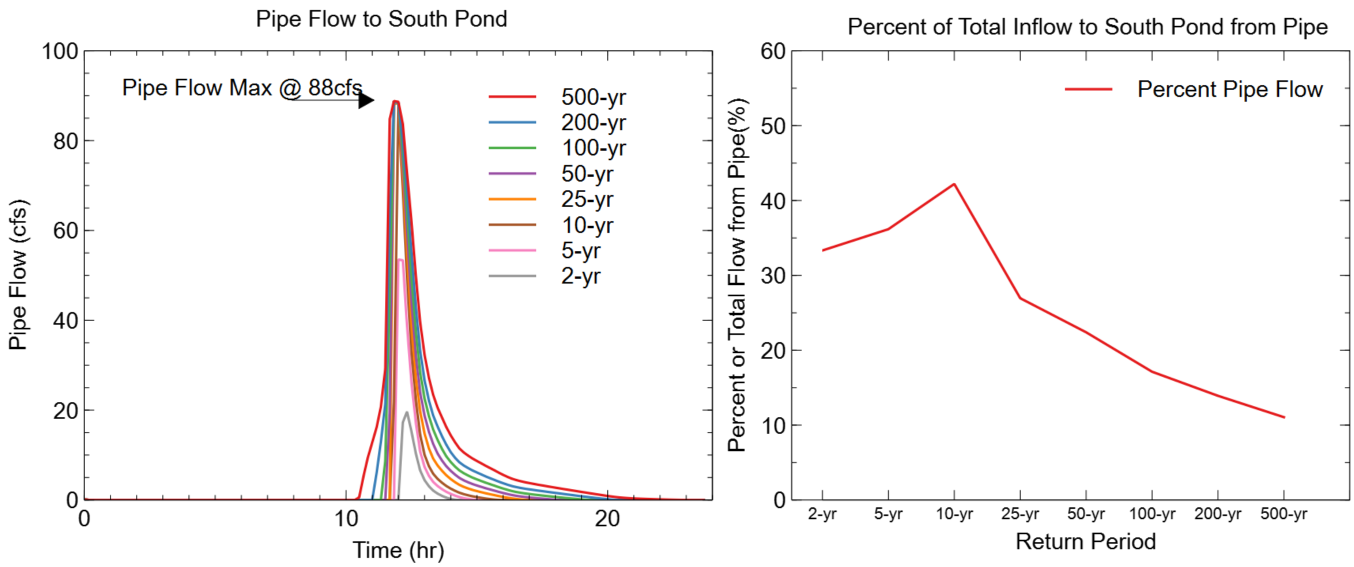

Figure 5-2. Storm sewer flow hydrographs (left) into the South Pond, percent of total flow into South Pond from storm sewer flow (right).

6. Simulation Results – Detention Projects

After optimizing the outlet structure designs, final simulations of inflow and outflow hydrographs for the North Pond and South Pond were extracted and are shown in Figure 6-1 and Figure 6-2, respectively. Significant attenuation of flood peaks can be seen by comparing the inflow and outflow hydrographs at each pond structure. A 1:1 plot of the inflow and outflow peaks for each pond is shown in Figure 6-5. Average peak flow reductions for the North and South ponds were 90% and 77%, respectively. Tabulated peak flows, reductions, pool elevations, depth, and storage for the North and South ponds are shown in Table 6-1 and Table 6-2, respectively.

The pond projects can potentially reduce flood depths in addition to reducing storm water peak flows. Maximum simulated flood depths for each return period were extracted from the middle of Amy Drive and are shown in Figure 6-6. These results indicate the pond projects could significantly reduce maximum flood depths along Amy Drive, especially impactful for larger, less frequent events.

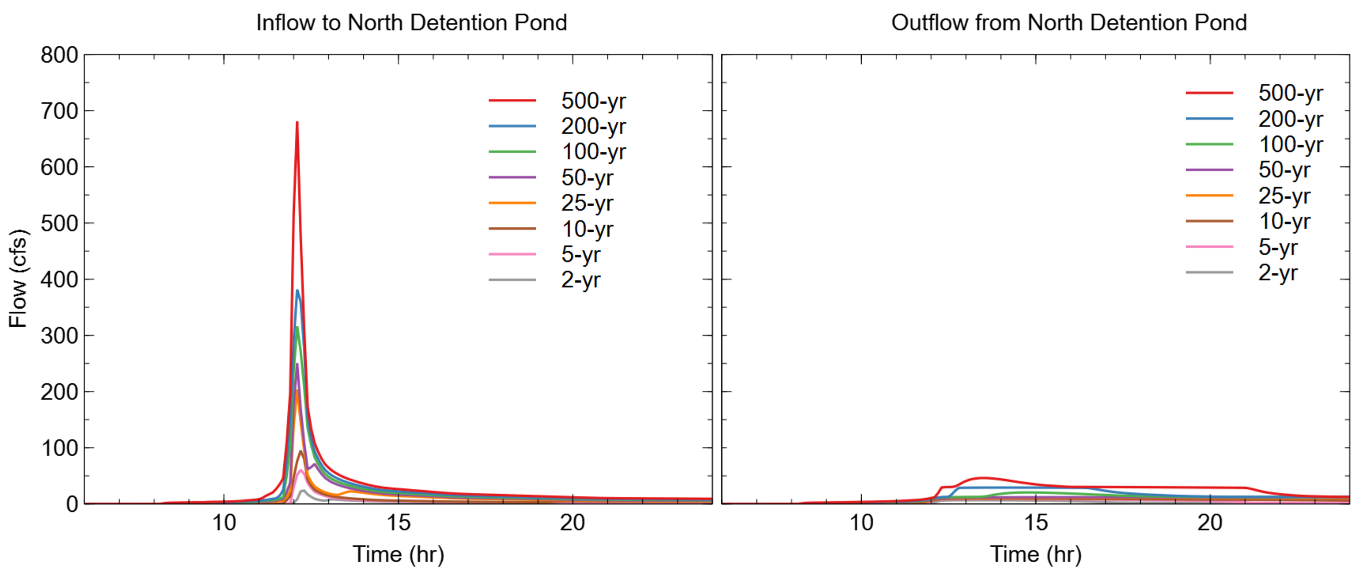

Figure 6-1. North Pond inflow and outflow hydrographs.

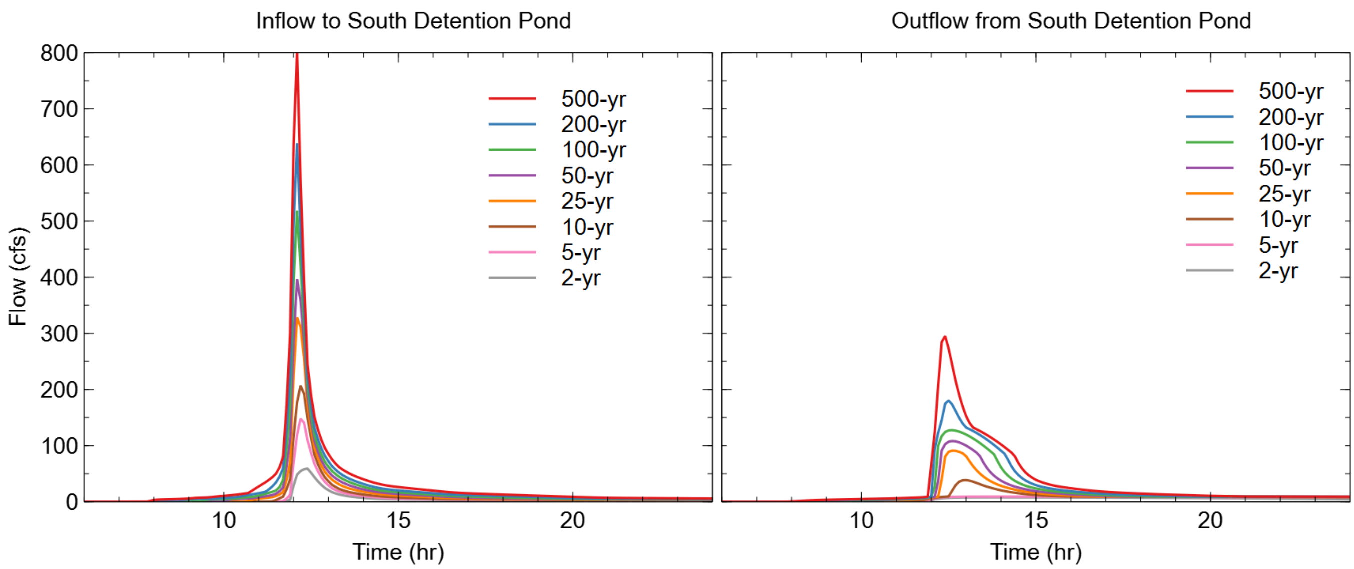

Figure 6-2. South Pond inflow and outflow hydrographs.

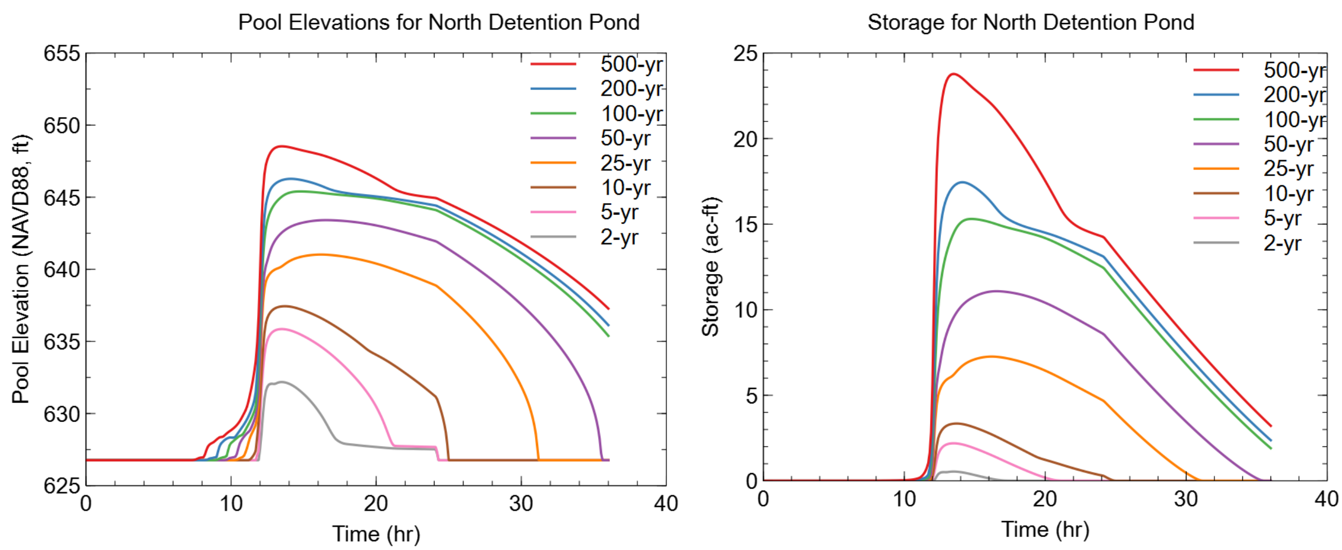

Figure 6-3. Pond pool elevation and storage hydrographs for the North Pond.

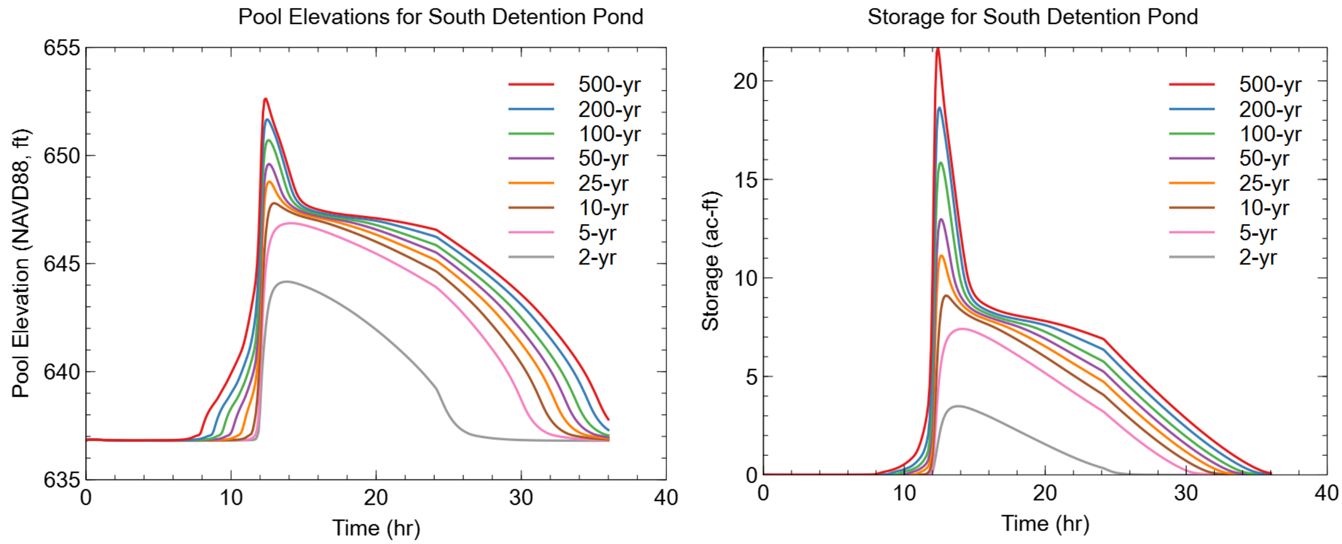

Figure 6-4. Pond pool elevation and storage hydrographs for the South Pond.

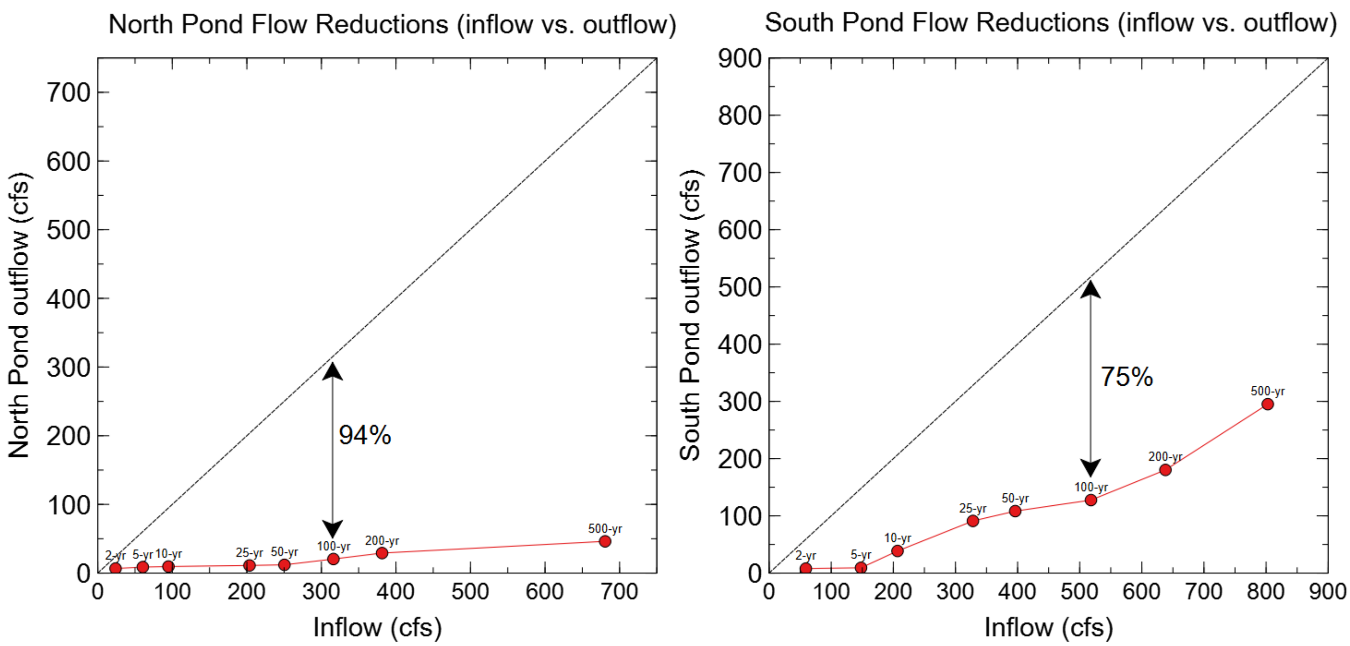

Figure 6-5. Pond peak inflow vs. pond peak for the North Pond (left) and South Pond (right). The average peak flow reduction provided by the North Pond was 90%, while the average peak flow reduction provided by the South Pond was 77%

Table 6-1. Tabulated simulation results for the North Pond.

Return Period	Peak Inflow (cfs)	Peak Outflow (cfs)	Reduction (%)	Peak Pool Elev. (NAVD88, ft)	Peak Pool Depth (ft)	Peak Storage (ac-ft)
2-yr	24	7	72%	632.2	5.4	0.5
5-yr	60	9	86%	635.9	9.1	2.2
10-yr	95	10	90%	637.4	10.7	3.3
25-yr	204	11	95%	641.0	14.3	7.3
50-yr	250	12	95%	643.4	16.7	11.1
100-yr	316	20	94%	645.4	18.6	15.3
200-yr	381	29	92%	646.3	19.5	17.4
500-yr	681	46	93%	648.5	21.8	23.8

Table 6-2. Tabulated simulation results for the South Pond.

Return Period	Peak Inflow (cfs)	Peak Outflow (cfs)	Reduction (%)	Peak Pool Elev. (NAVD88, ft)	Peak Pool Depth (ft)	Peak Storage (ac-ft)
2-yr	59	8	87%	644.2	7.3	3.5
5-yr	148	9	94%	646.9	10.0	7.4
10-yr	207	39	81%	647.8	11.0	9.1
25-yr	328	91	72%	648.8	12.0	11.1
50-yr	396	108	73%	649.6	12.8	13.0
100-yr	518	128	75%	650.7	13.9	15.9
200-yr	638	180	72%	651.7	14.9	18.7
500-yr	803	295	63%	652.6	15.8	21.7

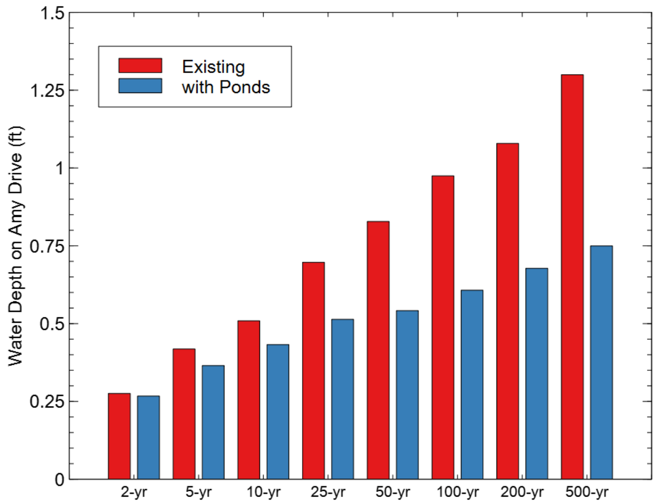

Figure 6-6. Maximum simulated depths on Amy Drive for each return period for existing conditions and with pond projects (left). The location of the reported depths is shown in the aerial photo (right).

7. Summary and Recommendations

The proposed detention ponds provide significant peak flow reductions and will reduce loading of downstream stormwater infrastructure. This will benefit all downstream homeowners, especially those on Amy Drive. An additional benefit is reduction of debris entering downstream sewer facilities. Actual flow reductions will depend on the capacities of downstream stormwater infrastructure during an event. For this analysis, it was assumed that pond outflows can be accommodated by the downstream stormwater conduits. As potential future sewer separation projects are developed, connections to the principal spillways of the ponds will need to be considered.

Detailed design and specifications of the pond embankments and outlet structures will be required by a professional engineer. Permits will be required from Iowa Department of Natural Resources (Iowa DNR) Dam Safety Program. Feasibility of these projects depends on securing funding and landowner participation.

8. References

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