TETRA TECH

# Memo

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| ***To:*** | Chuck Hauber, Brian Olson, Dave Koontz, Alan Brubaker, Summit County Engineer (SCE) |
| ***From:*** | Cole Blasko, Dai Thomas, Kevin Kratt, Bill Carlson, Tetra Tech Inc. |
| ***Date:*** | April 11, 2025 |
| ***Subject:*** | Aurora Shores 2-D HEC-RAS Model |

## EXECUTIVE SUMMARY

This memorandum describes an analysis of the flooding conditions in the Aurora Shores neighborhood and surrounding areas in Reminderville, Ohio. The Aurora Shores neighborhood is identified as a problem area in the Cuyahoga River Watershed Study being performed by Tetra Tech for the Summit County Engineer. The neighborhood has experienced repeated flooding in the vicinity of the Nautilus Trail and Anchorage Cove intersection and in the backyards of homes on Windjammer Trail and Sea Ray Cove. This memorandum serves as a supporting document to the larger study and the model results will be used, in part, to develop recommendations for the Aurora Shores problem area.

Tetra Tech first developed a comprehensive understanding of the history of flooding in Aurora Shores, the stream restoration of Pond Brook, and the construction of the wetland cells through a review of all known existing studies, plans, and analyses to date. To assess these flooding issues, a two-dimensional hydraulic model was developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS). Five scenarios were set up to answer the following questions:

- *Scenario 1*: Does the 100-year storm flow in the headwaters of the Channel Brook watershed (north of Glenwood Boulevard) result in flooding of the boating canal levees?
- *Scenario 2*: Does the increase in Aurora Lake's water level from the 100-year storm result in overtopping of the boating canal levee and localized flooding?
- *Scenario 3*: Would large flows through the Aurora Lake spillway cause flooding in the Aurora Shores neighborhood via a backwater effect?
- *Scenario 4*: What is the root cause of the flooding in the vicinity of Windjammer Trail and Sea Ray Cove?
- *Scenario 5*: Agri-Drains are designed to control flow between the ditches and constructed wetlands adjacent to Pond Brook. These Agri-Drains are located on Summit County Metroparks land at the northeast corner of Wetland Cell 1 along Ditch 1 and at Ditch 3's confluence with Pond Brook (**Figure E - 1**). Can the operational elevation of these Agri-Drains be modified to better control the flooding in the vicinity of Windjammer Trail and Sea Ray Cove?

The results of *Scenarios 1* through *3* indicate that flooding does not originate from the Channel Brook watershed, the boating canal itself, or Aurora Lake. Some of the historic flooding was likely due water backing up at the Clipper Clove culvert, which conveys flows from Pond Brook underneath the boating canal. The culvert was upsized in Spring 2024 per specifications from the 2021 OHM study that showed a reduction in flood levels ranging from 0.1- to 1.1-feet for the 1-year event to the 2020 Labor Day event. The 2021 OHM study provided another scenario that included storage detention in addition to the Clipper Cove culvert replacement. This combined scenario resulted in reduction of flood levels ranging

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from 1.3- to 2.4-feet for the 1-year event to the 2020 Labor Day event. These flood level reductions are presumed to solve the flooding issue at the Nautilus Trail and Anchorage Cove intersection.

The results of *Scenarios 4* and *5* indicate that precipitation naturally pools in the backyards of homes along Windjammer Trail and Sea Ray Cove because the backyards are 6-inches lower than the ground elevation of the wetland and only 6-inches higher than the bottom of the ditch (**Figure E - 2**). As such, water naturally flows towards the backyards. Poor infiltration capacity and relatively flat existing topography worsen the issue and result in ponding on the properties for extended periods of time. These locations of flooding are in the 1990 and 2016 FEMA 100-Year Flood Zone (**Figure E - 3**).

A review of historic maps circa 1963 and 1906 (**Figure E - 4**) reinforces this conclusion because it shows that roads and buildings in the Aurora Shores neighborhood, specifically where the flooding of recent years has been observed, were built in low areas historically dominated by natural wetlands and swamps. Additionally, the increase in impervious area from the development of Aurora Shores increases runoff which is not fully offset by the limited number of detention basins in the neighborhood. The footprint of Aurora Lake has also approximately doubled in response to a four-foot increase in water surface elevation between 1906 and 1963 (**Figure E - 4**).

In addition to the results of *Scenarios 4* and *5*, the historic extent of USGS designated swamp area, and FEMA 100-year flood zones, the conclusion that this location in Aurora Shores has historically flooded and will continue to flood is substantiated by the natural underlying soils. As shown in the Soil Survey of Summit County the Willette Muck occupies the flooded backyards on Windjammer Trail and Sea Ray Cove. The soil is described as consisting of “very poorly drained organic soils that formed in muck deposits 16 to 42 inches thick”, having “slow permeability” with a “high water table for long periods unless they are drained”. These soil characteristics are indicative of flooding and must be taken into account when developing recommendations to mitigate the flooding.

Frequency of flooding has likely increased in recent years because there were more large storms (i.e., 100-year event or greater) between 2020 and 2024 than during the previous 60 years combined (Global Historical Climatology Network; **Figure E - 5**).

Tetra Tech also utilized a 1-D HEC-RAS model developed by Stantec in 2017 to test whether increasing the hydraulic capacity of the S.R. 82 and Railroad bridges downstream of the Aurora Shores neighborhood would results in lowered water surface elevations where the localized flooding is observed. Results of these scenarios showed negligible changes in water surface elevations even with the bridges removed in their entirety.

Conclusions and recommendations informed by the five model scenarios include the following:

- <u>*Channel Brook 100-Year Event*</u>
    - Model Parameters – Discharge from the Channel Brook watershed was simulated at the 100-year event flow volumes while keeping all other parameters at baseflow conditions.
    - Conclusion - The 100-year discharge from the headwaters of the Channel Brook watershed does not overtop the Channel Brook levees.
    - Recommendation – Maintain the dimensions of the Glenwood Boulevard crossing as this constriction attenuates the flood wave downstream.
- <u>*Aurora Lake Stage Increase*</u>
    - Model Parameters - The Aurora Lake spillway gate is operable between a range of 996.5-feet and 1002.1-feet with a normal operating elevation of 1001.0-feet which sets the surface area of the lake at 377-acres. The stage of Aurora Lake was linearly increased to achieve overtopping of boating canal levees.

- Conclusion – The Channel Brook boating canal first overtops at an elevation of 1001.5 feet near its downstream end 400 feet away from its confluence with Aurora Lake.
- Recommendation – Control the stage in Aurora Lake such that storage is available prior to precipitation events to prevent Aurora Lake stage from reaching 1001.5 feet. Lowering the stage of Aurora Lake by 1-foot provides 471 acre-feet of storage which is greater than the 100-year flow volumes coming into the lake (352.2 acre-feet).

- *Aurora Lake Spillway Backwater*
  - Model Parameters – Informed by HEC-1 modeling by ODNR, 779 cfs (349,639 gallons per minute) were released from the Aurora Lake spillway. These flows are approximately 10-times greater than Pond Brook baseflow and equal to 12% of the probable maximum flood in the HEC-1 model. This resulted in a released volume of 822 acre-feet and lowered the stage of the lake by 1.7-feet which is 2.3 times greater than the volume needed to store the 100-year event volumes.
  - Conclusion – Reservoir releases of this magnitude represent a worst-case scenario where Aurora Lake is close to overtopping its emergency spillway and is not expected under normal dam operation. These flows result in a backwater effect approximately equal to the inundated extent of the FEMA 100-year floodplain.
  - Recommendation – Control releases from the Aurora Lake spillway such that discharge of this magnitude is not reached under normal operations.
- *Design Agri-Drain Elevation = 992.5 feet* and *Raised Agri-Drain Elevation = 994.5 feet*
  - Model Parameters – Agri-Drains were set at two different elevations to evaluate their potential to reduce localized flooding. The 100-year precipitation event was applied to the model and all other unsteady flow hydrograph boundary conditions were set to steady baseflow conditions.
  - Conclusion – Agri-Drains at design elevations provide discharge at a controlled rate to pass through the embankment during low flows but water surface elevations during the 100-year precipitation event flood backyards. Cumulative flows downstream of the embankment via Ditch 1 are reduced when the Agri-Drain invert elevation is raised but water surface elevations are not improved because the embankment is overtopped regardless. Equilibrated water surface elevations across the wetland, ditch, and backyards indicate removal of the Agri-Drain and the embankment in their entirety will not resolve inundation of the 100-year event.
  - Recommendation –As part of the Cuyahoga River Watershed Study, Tetra Tech has provided three concept plans that outline the proposed recommended alternatives to control the flooding in the backyards of homes on Windjammer Trail and Sea Ray Cove. These concept plans are provided as an appendix to the Cuyahoga River Watershed Study and are summarized below.
    1. Alternative 1 proposes maintaining the existing conditions and allow the backyards to flood within the FEMA 100-year floodplain.
    2. Alternative 2 proposes the installation of a system of field drains in an east-west direction to convey water to the adjacent ditch. This alternative is not expected to solve the flooding issue but rather, will decrease the time in which the yards are flooded. Feasibility depends on a detailed elevation survey to ensure adequate grade exists between the yards and the ditch. As such, the exact design of the drains has not yet been established.
    3. Alternative 3 proposes removal of water from the area using the following three components: (1) Install field drains in a north-south direction to convey water out of the backyards; (2) Construct two retention

basins with impermeable lined bottoms; and (3) Install a pump station to remove water from the retention basins into the adjacent ditch system. This alternative will require the acquisition of three properties where elevations are most suitable (Parcel IDs 6600902, 6600903 and 6600904). Feasibility and design specifications for the proposed alternative have not yet been established.

- *Increased Hydraulic Capacity Downstream*
  - Model Parameters – The S.R. 82 and Railroad bridges were adjusted in the Stantec (2017) 1-D model under four scenarios: (1) As-Built; (2) 20-ft Abutment Expansion; (3) Floodplain Abutment Expansion; and (4) Bridges Removed.
  - Conclusion – Modifications to the hydraulic capacity of the bridges via changes in the abutment opening widths resulted in negligible changes in the water surface elevations upstream, even when bridges were removed in their entirety.
  - Recommendation – Increasing the hydraulic capacity of the S.R. 82 or Railroad bridges will not mitigate the flooding of homes along Windjammer Trail and Sea Ray Cove.

*Disclaimer: This report was prepared without consultation or input from the Aurora Shores Homeowners Association or its representatives.*

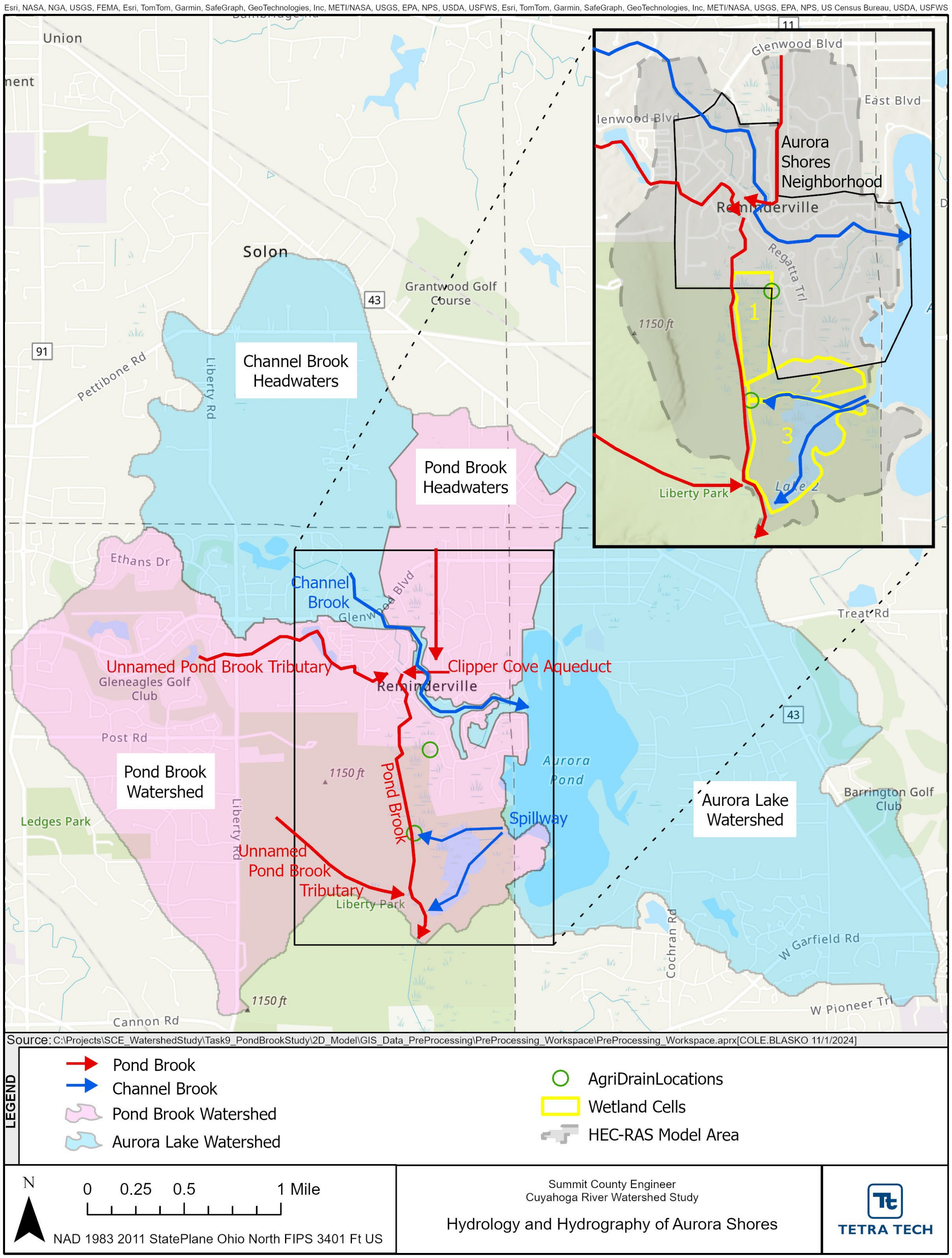


Figure E - 1. Schematic of the hydrology and hydrography surrounding the Aurora Shores neighborhood

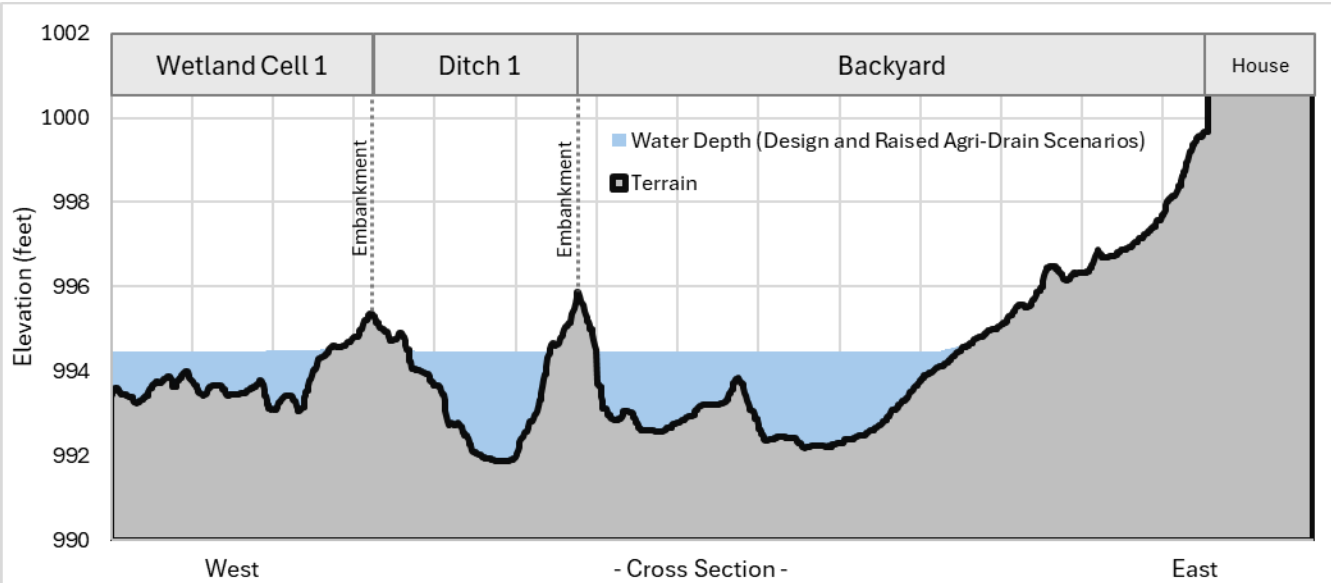


Figure E - 2. Cross section of terrain and water surface elevations downstream of the Agri-Drain and embankment structure

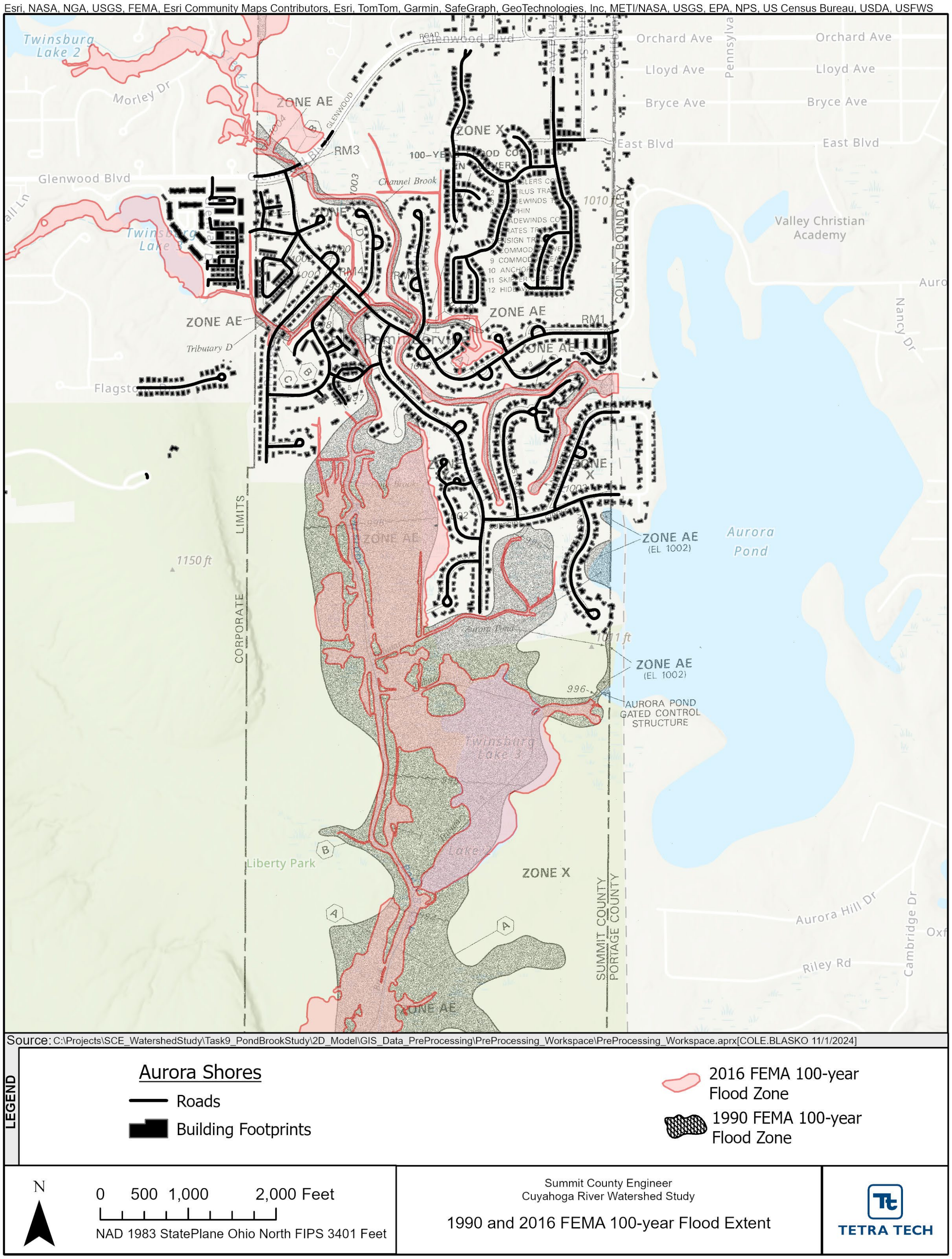


Figure E - 3. Present day building footprints and roads within the Aurora Shores neighborhood overlapped FEMA 100-year flood zones from 1990 and 2016

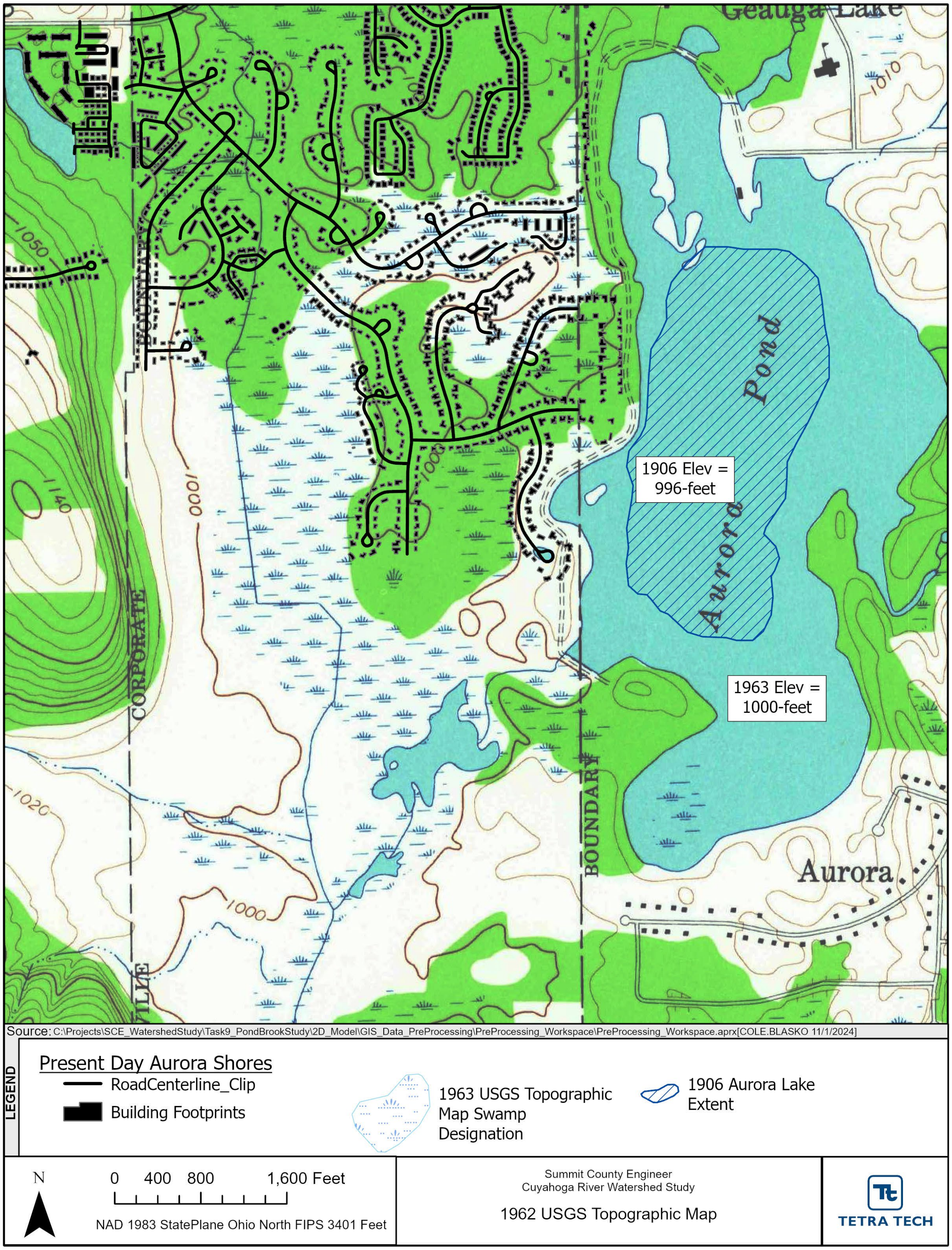


Figure E - 4. Present day building footprints and roads within the Aurora Shores neighborhood overlapped with a 1963 USGS topographic map

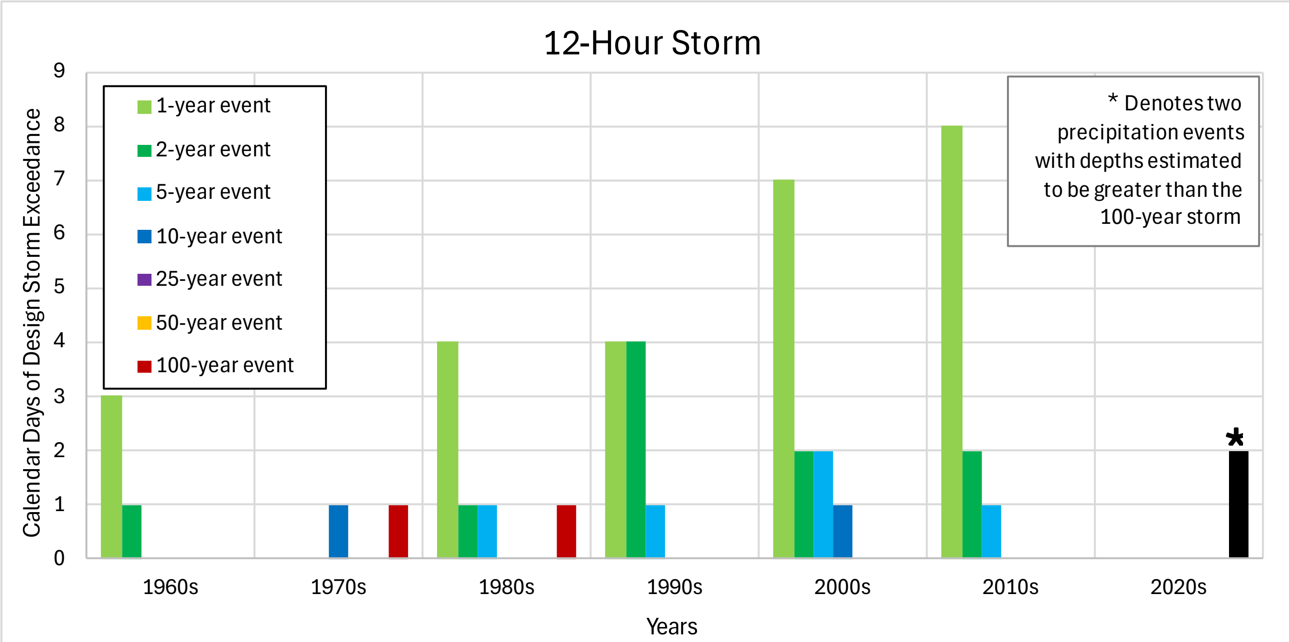


Figure E - 5. Calendar days in which design storm frequencies were exceed by the 12-hour duration precipitation event

# 1.0 HYDROLOGY OVERVIEW

The hydrology and hydrography in the Aurora Shores neighborhood are complex due to two overlapping stream networks. The Channel Brook stream network conveys flows from the northwest to the southeast toward Aurora Lake[1], while the Pond Brook stream network conveys flows from the north-central area to the south, west of Aurora Lake. The intersection of these networks is the Clipper Cove aqueduct near the center of the Aurora Shores neighborhood. These two watersheds and associated stream networks are described in detail below and shown as a simplified schematic in **Figure 1**.

- The Aurora Lake watershed starts in the Channel Brook headwaters northwest of Glenwood Boulevard. The Channel Brook boating canal conveys flow in a south-easterly direction to its confluence with Aurora Lake. The rest of the watershed is located east of Aurora Lake, where other tributaries contribute flow to the lake. While these tributaries are outside of the HEC-RAS model area, their contributing flows do impact the stage, storage, and outflow of Aurora Lake. The spillway of Aurora Lake is located at the southwest end of the lake and delivers flows back to the Pond Brook stream network via Wetland Cell 3 of Liberty Park.
- Pond Brook and its headwaters originate upstream of the Clipper Cove Aqueduct which passes flow underneath the Channel Brook boating canal. Note that this aqueduct is the intersection between the two stream networks and no interaction between the flows occur here; they are entirely separate systems. An unnamed tributary from the west has its confluence with Pond Brook just downstream of the aqueduct. Pond Brook then flows south through reaches where restoration projects occurred between 2004 and 2009. A second unnamed tributary from the west and outflow from the Aurora Lake spillway confluence with Pond Brook near the downstream end of the hydraulic model.

[1] Aurora Lake is also referred to as Aurora Pond on publicly available mapping.

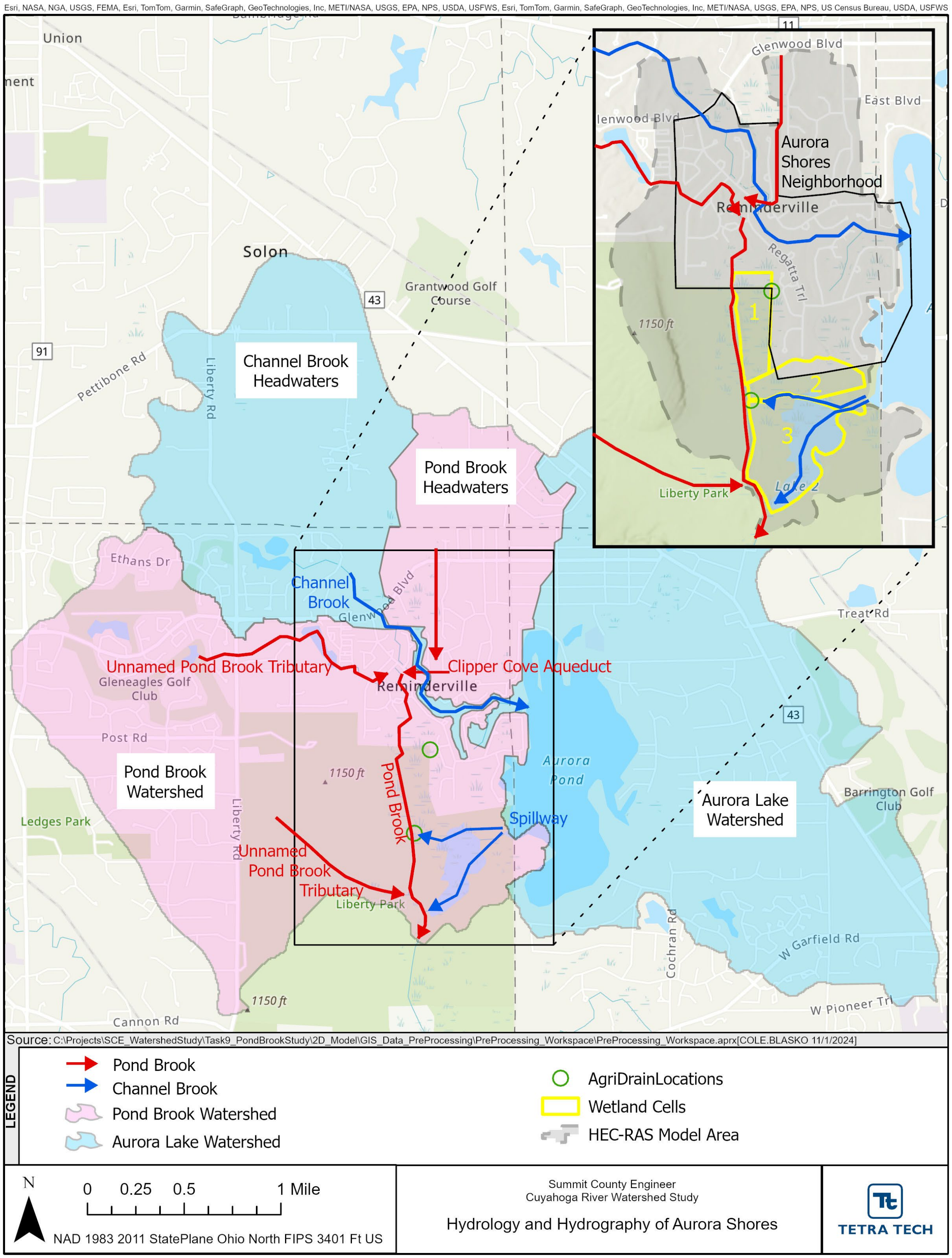


Figure 1. Schematic of the hydrology and hydrography surrounding the Aurora Shores neighborhood

## 2.0 PREVIOUS STUDIES

Multiple studies, restoration projects, and modeling efforts have been performed in the Aurora Shores neighborhood in recent years. In brief, the history of the area can be summarized by the following distinct events and studies:

- 1960s - 1970s: Pond Brook was channelized to quickly drain the low-lying area for development of the Aurora Shores neighborhood.
- 2004 – 2009: The initial study, design, and subsequent restoration of Pond Brook takes place (Wetlands Resource Center et al., 2006). Note that the reach immediately south of Aurora Shores within the HEC-RAS model area was restored in this timeframe but the larger restoration further downstream continued thereafter.
- 2017: Stantec (2017) performed a peer review study to assess the multiple phases of the Pond Brook restoration. The study used a 1-dimensional (1-D) HEC-RAS model that was developed circa-2015 with a modeled extent covering only the restored reaches of Pond Brook. The 1-D model showed that the restoration results in a decrease of the water surface elevation at all locations downstream of Nautilus Trail.
- 2021: Buckeye Engineering (2021) performed a hydrology study of Channel Brook to predict the 24-hour 100-year flow in Channel Brook at Glenwood Boulevard crossing.
- 2021: OHM Advisors (2021) developed a SWMM model for the replacement of the Clipper Cove Aqueduct in the City of Reminderville (OHM Advisors, 2021). The model extends from Clipper Cove upstream to the headwaters of the watershed but does not include Pond Brook reaches downstream of the Aurora Shores neighborhood or the tributaries from the west. This study and model informed the replacement of the Clipper Cove aqueduct replacement, which began construction in Spring 2024.

These studies analyzed individual aspects of the Pond Brook watershed, however, none of them holistically evaluated the watershed from its headwaters north of Glenwood Boulevard to a location far enough downstream to assess the flooding occurring in the Aurora Shores neighborhood.

## 3.0 MODEL DEVELOPMENT

Tetra Tech developed a new 2-dimensional (2-D) hydraulic model of the watershed to assess the flooding in the Aurora Shores neighborhood for the following reasons. Firstly, none of the previous studies accounted for all components of the stream network. Secondly, a full 2-D model provides better prediction of the extent and depth of inundation in areas with complex channel and overbank conditions compared to 1-D models. And thirdly, the model provides a good basis for future studies and restoration designs.

The hydraulic model includes the following components:

- Channel Brook boating canal
- Pond Brook and its restored reach
- Aurora Lake and its spillway
- Upstream contributing areas north of Glenwood Boulevard
- Two unnamed tributaries from the west that contribute flows to Pond Brook
- The constructed wetlands and their water surface elevation control structures
- The system of ditches constructed to drain the neighborhood and convey flows around the wetland cells

The modeling was performed using the U.S. Army Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 6.4.1 software, a standard among the engineering community for analyzing one- and two-dimensional hydraulic conditions in rivers and open channels. HEC-RAS is used to simulate movement of a flood hydrograph over a system of 2-D grid elements. The software can accommodate larger grid cell sizes than many other 2-D models using pre-processed hydraulic property tables. These tables represent the details of the underlying terrain for each grid cell, and thus retain the needed detail while using larger grid cells than most other 2-D hydraulic models (USACE, 2024).

Input for the model consists of a series of topographic data of the channel and overbank, hydraulic roughness parameters (typically specified by the semi-empirical Manning roughness coefficient), and boundary conditions for each discharge.

Due to the lack of measured flows and water-surface elevations, the hydraulic model was not calibrated.

## 3.1 TERRAIN

The HEC-RAS model requires an input "terrain" that represents the existing channel and overbank topography. Base terrain data was downloaded from the HEC RAS "Download Terrain Data from USGS" window. The "USGS Original Project Resolution OH_Statewide_Phase1_2019_B19" data was used because it had the finest spatial resolution for the study area. To get comprehensive coverage of the study area, 216 data tiles were mosaiced together. The terrain is a 1.25-foot pixel resolution Digital Terrain Model (DTM) that consists of square pixels for an area with a representative elevation. The model computes the hydraulic properties for each element in the terrain and uses the terrain to develop the depth, velocity, and water-surface elevation mapping.

The following terrain modifications were performed manually to achieve a better representation of true terrain conditions:

- Building footprints were downloaded from the Summit County GIS Open Data portal. These data were collected in 2000 and therefore required the manual digitization of additional buildings that were constructed in the study area after 2000. Satellite imagery circa 2023 was used to identify and digitize these additional buildings. The final building shapefile was used as a modification layer in the terrain where all buildings were artificially raised by 20 feet.
- The water surface of Channel Brook was represented in the downloaded terrain data and therefore, the subaqueous portion of the channel was manually burned into the terrain based on data from the FEMA FIRM study and Flood Profile Panel 025P (FEMA, 2016).
- Like Channel Brook, the primary ditch at the outlet of the Aurora Lake Dam spillway, otherwise known as Ditch 3, also required its bathymetry to be manually burned into the terrain in select locations. FEMA data was not available for this ditch. Therefore, the ditch bathymetry was estimated based on linear interpolation of ditch reaches where the bathymetry was well represented.

Elevations of selected reaches in Channel Brook and Pond Brook were verified against the Hydrology Study, Channel Brook (Buckeye Engineering, 2021) and OHM PCSWMM cross section data (OHM Advisors, 2021). However, elevations of the terrain and strucutres (Section 3.4) were not field surveyed by Tetra Tech.  Final terrain data is shown in **Figure 2**.

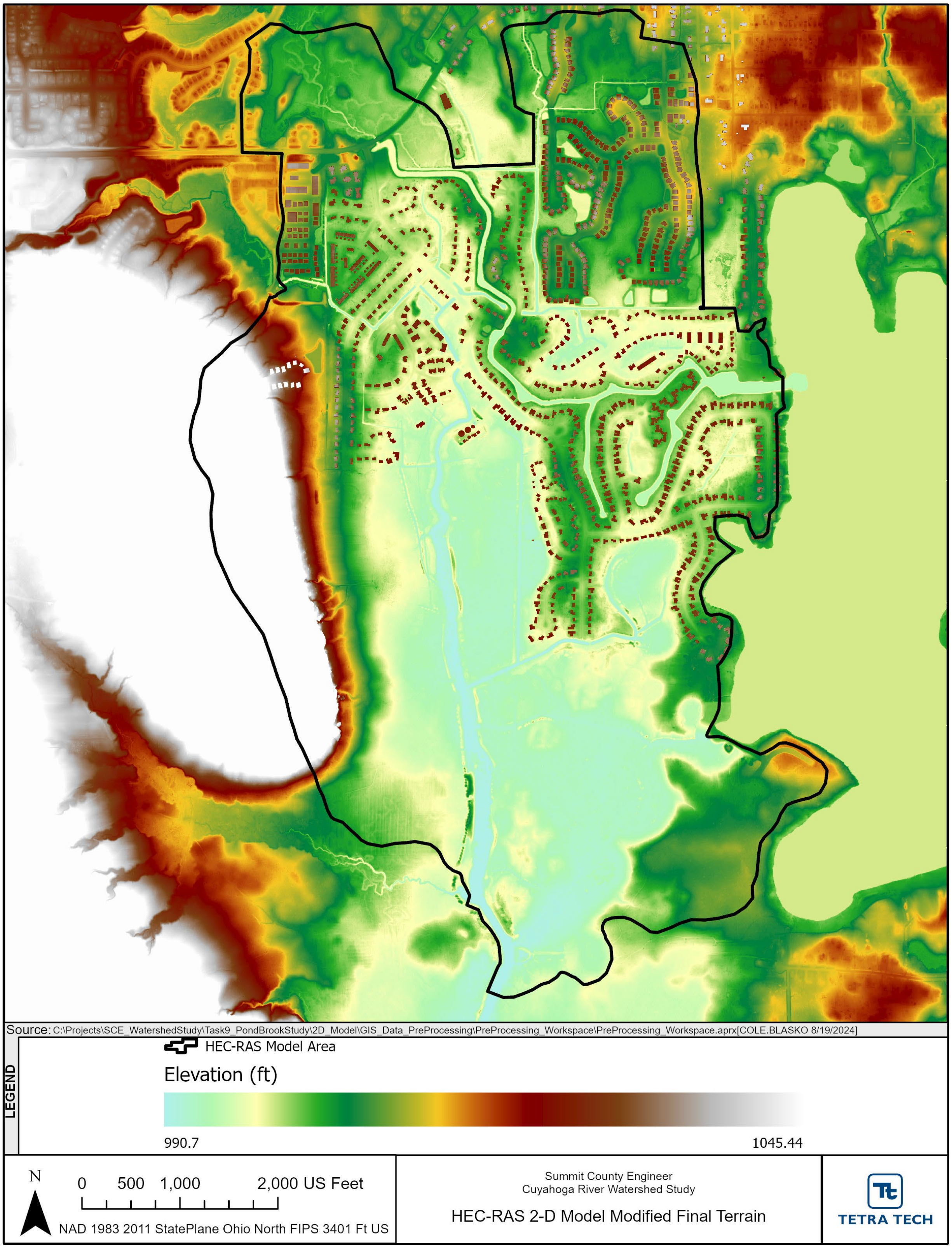


Figure 2. HEC-RAS 2-D model modified terrain

## 3.2 MODEL MESH

The 2-D flow area is the region of a model in which the flow through that region will be computed with the HEC-RAS 2-D flow computation algorithms. The perimeter and computational mesh (or grid) define the model domain and spatial resolution of the model, respectively. Breaklines and refinement regions are used to improve the computational mesh to better match changes in terrain.

Efforts were made to ensure the model area perimeter matched watershed divides or was digitized at a location where a known boundary condition was available to estimate flows incoming to the model. The model perimeter was also developed so that the two primary areas of documented flooding and all components of the stream network that would have impact on the water surface elevations in these areas were encompassed by the model.

More specifically, the following data was used to define the model perimeter (numbers in the list correspond to those shown in **Figure 3**):

1. West tributary flow hydrograph boundary condition
2. Pond Brook watershed divide
3. West tributary flow hydrograph boundary condition
4. Channel Brook flow hydrograph boundary condition
5. SWMM basin 6 flow hydrograph boundary condition
6. SWMM basin 7 flow hydrograph boundary condition
7. SWMM basin 10 flow hydrograph boundary condition
8. SWMM basin 14 flow hydrograph boundary condition
9. SWMM basin 32 flow hydrograph boundary condition
10. Channel Brook confluence with Aurora Lake stage hydrograph boundary condition
11. Watershed divide between Pond Brook and direct drainage to Aurora Lake
12. Aurora Lake spillway flow hydrograph boundary condition
13. Modeled area outflow simulated as normal depth boundary condition

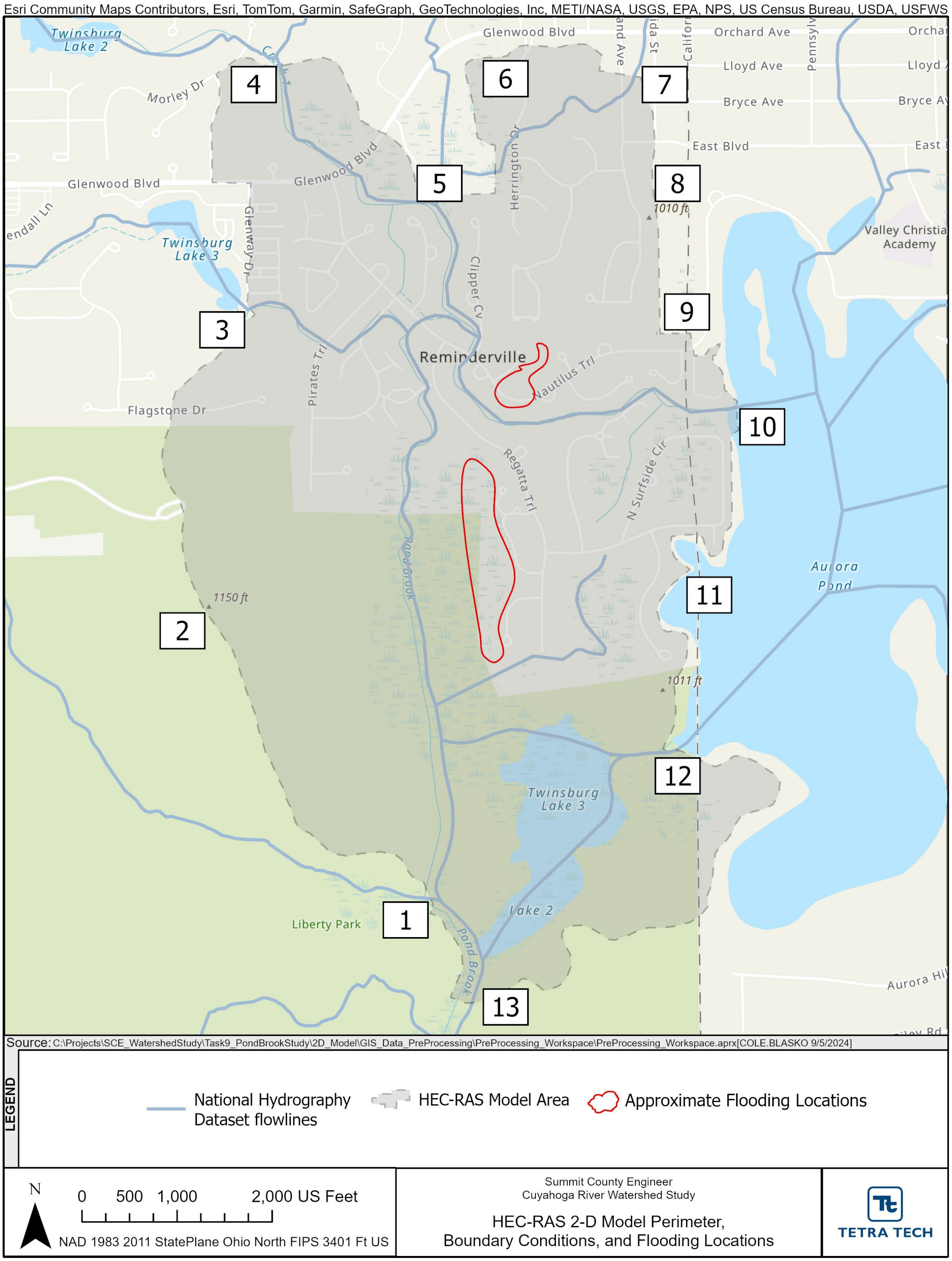


Figure 3. HEC-RAS 2-D model perimeter, boundary condition locations, and locations of flooding

The computational mesh was initially defined with grid side lengths of 40-feet. This spacing provided a balance between good mesh resolution and reasonable simulation times. The computational mesh was modified as needed using breaklines and refinement regions discussed in detail below. An example of the spatial resolution of the computational mesh is provided in **Figure 4**.

Figure 4. Example of computational mesh in location of Clipper Cove aqueduct. Computational mesh is shown as the white grid while breaklines and refinement regions are shown as red lines and polygons, respectively.

Many areas of the model required refinement of the computational mesh to match linear features within the terrain (e.g., the raised crown of a road). The mesh can be refined with breaklines, which when digitized, forces the computational mesh to have edges along the digitized line. Cell size along the line and the number of cells of that size perpendicular to the line are defined by parameters called Near Spacing and Near Repeats, respectively. A parameter called Far Spacing specifies the cell size that will be ramped up to after the near repeats have been inserted. For example, road centerlines were digitized with a Near Spacing of 20 feet, Near Repeats of 1, and a Far Spacing of 40 feet. This means that 20x20 foot cells were inserted along the road, with 1 cell on either side of the road centerline, which then ramp up in size moving away from the road until a cell size of 40x40 feet is achieved.

**Table 1** outlines the various types of breaklines that were digitized when building the 2-D computational mesh for the Aurora Shores model. This table is not comprehensive and other breaklines of various sizes were added in special cases where the mesh needed additional refinement.

Table 1. Typical breaklines and cell sizes used when refining the computational mesh

| | Near Spacing | Near Repeats | Far Spacing |
|---|---|---|---|
| Channel Brook and Pond Brook | Variable to match channel width | Variable to match channel width | 40 |
| Road Centerlines | 20 | 1 | 40 |
| Floodplain Boundary | 20 | 1 | 40 |
| Channels in Wetlands | 10 | 2 | 40 |
| Other Channels | 10 | 2 | 40 |
| Changes in Topography | 40 | 1 | 40 |
| Backyard Edge | 10 | 2 | 40 |
| Select Crossings | 4 | 1 | 10 |

Refinement regions are like breaklines but are digitized as polygons rather than linear features. Cell spacing inside the polygon is defined by the Cell X and Cell Y parameters, while the cell size along the perimeter of the polygon is defined by the Perimeter Spacing parameter. Like breaklines, Near Repeats and Far Spacing parameters are also available for refinements regions.

All buildings within the model area were defined as a refinement region so that computational mesh cell edges were along the building perimeter. This ensured that flows would not pass through buildings. **Table 2** outlines the building refinement region parameters used in the Aurora Shores model.

Table 2. Typical refinement region parameters used to define the computational mesh within buildings

| | Cell X | Cell Y | Perimeter Spacing | Near Repeats | Far Spacing |
|---|---|---|---|---|---|
| Buildings | 10 | 10 | 10 | 1 | 40 |

## 3.3 MANNING'S N ROUGHNESS VALUES

The HEC-RAS model uses Manning's *n* roughness values to quantify the energy losses in the 2-D areas. Manning's n values were spatially assigned based on land cover mapping. Land cover data was not readily available at a fine enough spatial resolution for the study area. Therefore, a land cover dataset was manually digitized based on 60-cm spatial resolution aerial imagery for Summit County (NAIP, 2023). The ESRI Classification Wizard was utilized to create the land use dataset via an object based supervised classification. The following specifications and parameters were used within the ESRI Classification Wizard:

1. How the image is segmented into objects is specified by segmentation parameters. The Spectral Detail and Spatial Detail were assigned the highest possible values and the Minimum Segment Size was set to 200 pixels.
2. The following number of training polygons were digitized to inform the supervised classification: Forest (7), Grass (10), Floodplain (19), Channel (2), Impervious (12), Wetland (2).
3. The Support Machine Vector classification method was used with a maximum of 500 samples per class and segment attributes of Active Chromaticity Color and Mean Digital Number.

Manual modifications of the supervised classification were made where necessary by comparing the resulting land use to the original aerial image. These manual modifications primarily involved reclassification of floodplain and wetlands to grass and forest where the supervised classification was incorrect. Land use was further refined with shapefiles of road centerlines (Summit County GIS Open Data portal), stream banks (FEMA, 2016), building footprints (Summit County GIS Open Data portal), and a manually digitized Pond Brook floodplain. An example of the final land use spatial resolution use is shown in **Figure 5**. Manning's n roughness values used in the model are presented in **Table 3**.

Pond Brook was observed to be generally free of log jams and other obstructions during Tetra Tech's field visit on March 20$^{th}$, 2024. As such, the channel roughness parameters in the model presumes the channel is free of obstructions and no further roughness modifications were made outside of using the land cover to spatially determine Manning's n values.

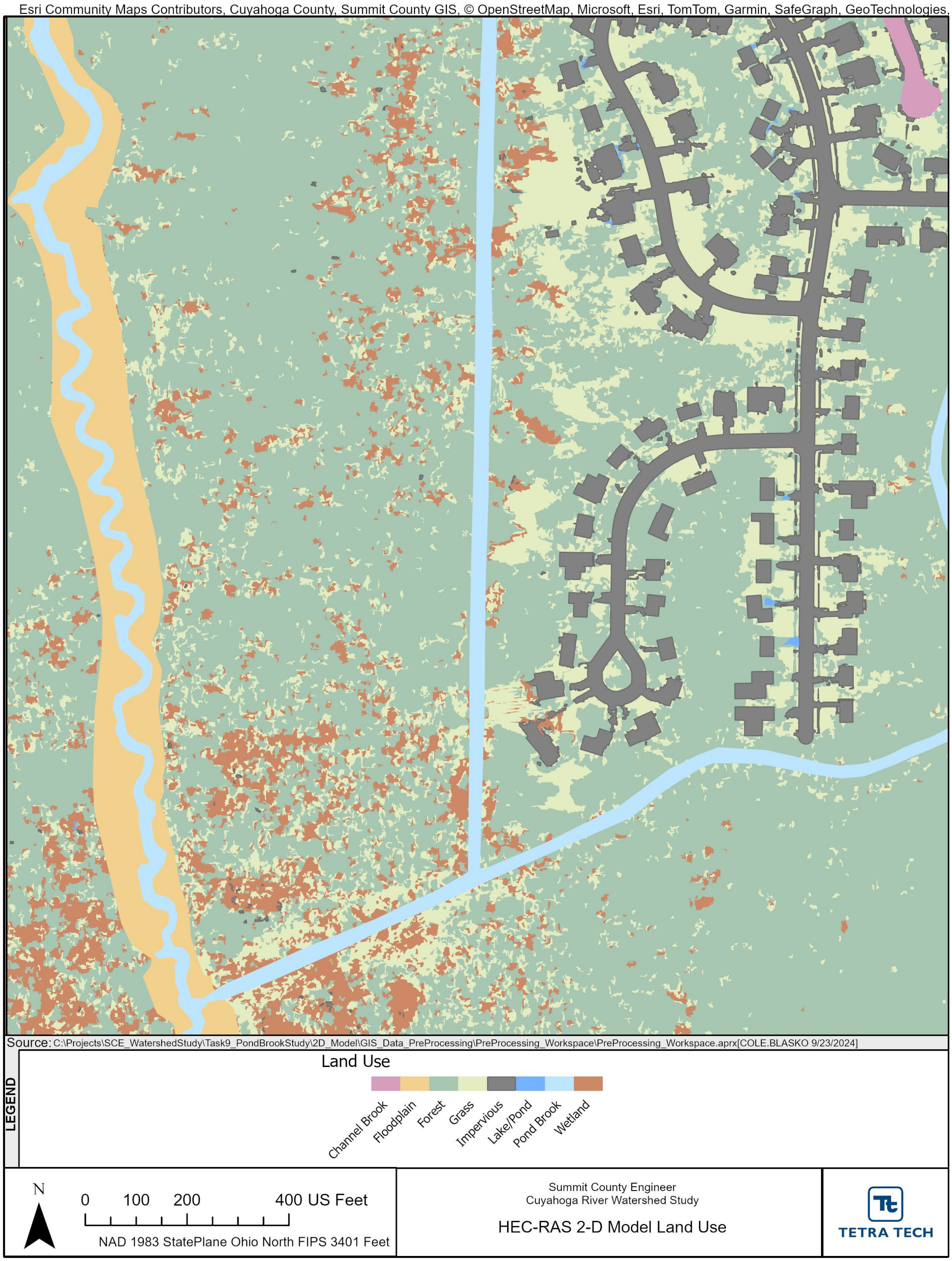


Figure 5. HEC-RAS 2-D model land use zoomed to show spatial resolution

Table 3. Manning's n roughness values for all land use classifications within the model area.

| | Chow, 1959 [a] | HEC-RAS 2-D User's Manual [b] | FEMA FIS [c] | OHM PCSWMM [d] | HEC-RAS Modeled Value |
|---|---|---|---|---|---|
| Forest | 0.08 – 0.120<br>Floodplains > Trees > heavy stand of timber, a few down trees, little undergrowth, flood stage below branches | 0.08 – 0.20<br>NLCD Mixed Forest | -- | -- | 0.140 |
| Grass | 0.025 – 0.035<br>Floodplains > Pasture, no brush > short grass | 0.025 – 0.05<br>NLCD Grassland/Herbaceous | -- | -- | 0.040 |
| Channel Brook | 0.022 – 0.033<br>Excavated or Dredged Channels > Earth, straight, and uniform > with short grass, few weeds | 0.025 – 0.05<br>NLCD Open Water | 0.034 – 0.036<br>Channel Brook | -- | 0.030 |
| Pond Brook | 0.033 – 0.045<br>Main Channels > clean, winding, some pools and shoals | 0.025 – 0.05<br>NLCD Open Water | 0.032 – 0.043<br>Pond Brook, Tributary A, Tributary D | 0.032 – 0.038<br>Pond Brook Channel<br>0.045<br>Pond Brook in restored reach | 0.035 |
| Impervious | 0.016<br>Lined or Constructed Channels > Asphalt > rough | 0.12 – 0.20<br>NLCD Developed High Intensity | -- | -- | 0.016 |

| | Chow, 1959 [a] | HEC-RAS 2-D User's Manual [b] | FEMA FIS [c] | OHM PCSWMM [d] | HEC-RAS Modeled Value |
|---|---|---|---|---|---|
| Wetland | 0.100 – 0.160<br>Floodplains > Trees > heavy stand of timber, a few down trees, little undergrowth, flood stage reaching branches | 0.045 – 0.15<br>NLCD Woody Wetlands | -- | -- | 0.120 |
| Lake/Pond | 0.025 – 0.033<br>Main Channels > clean, straight, full stage, no rifts or deep pools | 0.025 – 0.05<br>NLCD Open Water | -- | -- | 0.030 |
| Floodplain | 0.040 – 0.080<br>Floodplains > Brush > light brush and trees in summer | 0.025 – 0.05<br>NLCD Grassland/Herbaceous | 0.034 – 0.068<br>Overbank of Channel Brook and Pond Brook | 0.049<br>Overbank | 0.060 |

a. Chow, 1959.

b. USACE, 2024.

c. FEMA, 2016.

d. OHM Advisors, 2021.

## 3.4 STRUCTURES

Eleven structures were represented in the model. Locations of these structures are shown in **Figure 6** and parameters used to model these structures are presented in **Table 4** and **Table 5**. These structures were entered into the model with greater detail because these locations are critical for conveying flows and controlling water surface elevations of the main channels. Other structures did not require this level of detail and were considered non-critical because they were not immediately adjacent to the flooded areas, they were on relatively small tributaries and minimally impacted downstream flows, or they could be well-represented hydraulically with simple terrain modifications.

Non-critical structures were represented in the model by manually modifying the terrain itself where needed to ensure conveyance of flows through small culverts in the Aurora Shores neighborhood. These small culverts were not assessed by Tetra Tech in the field and as-built drawings were not available or provided. Dimensions of these culverts and invert elevations were estimated using the existing terrain, the contributing upstream drainage area, nearby or similar culvert dimensions, and professional judgement. These non-critical structures include foot bridges on Ditch 1 and Channel Brook upstream of Glenwood Boulevard. The Nautilus Trail crossing over Channel Brook was also modified in the terrain.

### 3.4.1 Road Crossings

Critical road crossing structures were modeled explicitly using SA-2D connections in the model (**Table 4**). Manning's n values for all concrete box culverts were set to 0.011 for "Closed Conduits Flowing Partly Full" of type "Concrete Culvert, straight and free of debris" (USACE, 2024). Additional parameters not listed in **Table 4** were the same for all road crossings and include the following: Entrance Loss Coefficient = 0.5; Exit Loss Coefficient = 1; Depth to use Bottom n = 0; Depth Blocked = 0. The entrance loss coefficient of 0.5 was informed by USACE, 2024 for "Reinforced Concrete Box Culverts" with "Wingwalls at 10 to 25 degrees to Barrel" with a "Square-edge at crown". The exit loss coefficient is set to 1 as the default by HEC-RAS.

### 3.4.2 Wetland Control Structures

The Agri-Drains located in Ditch 1 adjacent to Wetland Cell 1, and at the end of Ditch 3 which is the outlet of Wetland Cell 2 (**Figure 6**), were both modeled as gates in HEC-RAS (**Table 5**), rather than culverts, to provide the ability to open and close the structures and to set the invert elevations. These adjustable features allow for water-surface elevations to be controlled within the wetland cells and are the primary reason for installation of the Agri-Drains. Overflow gates with a closed top in HEC-RAS best represent how Agri-Drains control flows and water surface elevations. First, weir flow over the gate was specified as sharp crested as this best represents the stoplogs inserted into the Agri-Drain to control the water surface elevation.

The water leveling pipe between Wetland Cells 1 and 2 was modeled as a simple culvert in HEC-RAS (**Table 5**). Manning's n was set to 0.021 for "Corrugated Metal Pipe" (USACE, 2024). Parameters not listed in **Table 5** include the following: Entrance Loss Coefficient = 0.9; Exit Loss Coefficient = 1; Depth to use Bottom n = 0; Depth Blocked = 0. The entrance loss coefficient of 0.9 was informed by USACE, 2024 for "Corrugated Metal Pipe Projected from fill (no headwall). The exit loss coefficient is set to 1 as the default by HEC-RAS.

The reinforced spillway at the downstream end of Wetland Cell 3 was modeled as a weir/embankment in HEC-RAS. It was modeled as a broad crested weir with a weir coefficient of 3.0 (**Table 5**). The embankment on either side of the weir was set to an elevation of 994 feet while the weir itself has an elevation of 992.5 feet.

Parameters for all wetland control structures were informed by Wetlands Resource Center et al. (2006).

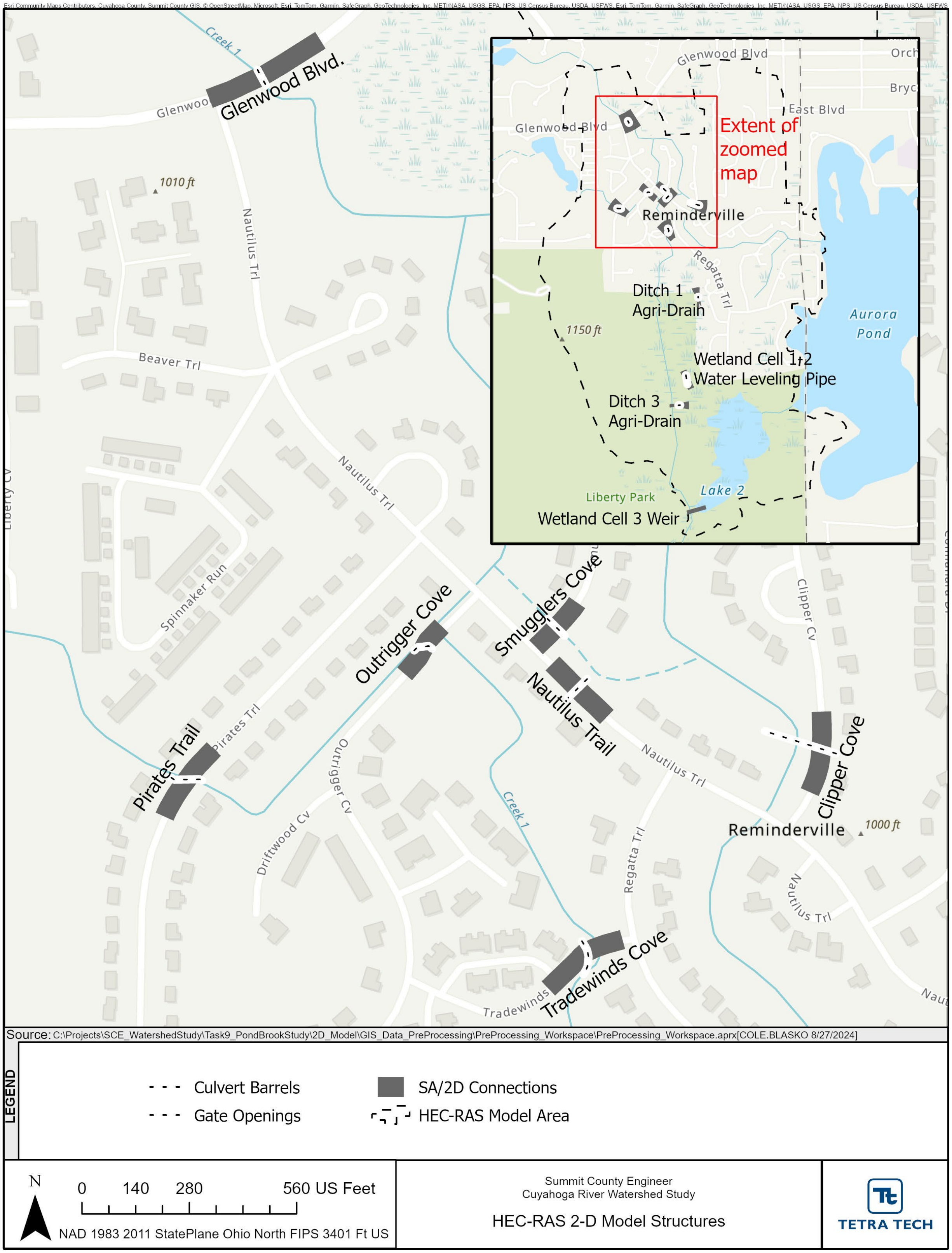


Figure 6. Locations and names of structures modeled in HEC-RAS

Table 4. Parameters of road crossing structures modeled explicitly in HEC-RAS

| | Chart # | Scale # | Shape | Span (feet) | Rise (feet) | Length (feet) | Upstream Invert (feet) | Downstream Invert (feet) | Manning's n Top | Manning's n Bottom |
|---|---|---|---|---|---|---|---|---|---|---|
| Clipper Cove | 58-Rectangular concrete | 2-Side Tapered; More favorable edges | Box | 16 | 4 | 185 | 991.1 | 990.9 | 0.011 | 0.011 |
| Glenwood Blvd. | 58 – Rectangular concrete | 2 – Side Tapered; More favorable edges | Box | 22 | 5 | 45.33 | 999.85 | 999.3 | 0.011 | 0.03 [a] |
| Nautilus Trail - Pond Brook | 58 – Rectangular concrete | 2 – Side Tapered; More favorable edges | Box | 16 | 4 | 80 | 990.84 | 990.5 | 0.011 | 0.011 |
| Outrigger Cove | 58 – Rectangular concrete | 2 – Side Tapered; More favorable edges | Box | 10 | 5 | 58 | 991.23 | 991 | 0.011 | 0.011 |
| Pirates Trail | 58 – Rectangular concrete | 2 – Side Tapered; More favorable edges | Box | 12 | 4 | 90 | 993.75 | 993.2 | 0.011 | 0.011 |
| Smugglers Cove | 55 – Circular culvert | 1 – Smooth tapered inlet throat | Circle | 3 | 3 | 48 | 992.01 | 991.8 | 0.024 | 0.024 |
| Tradewinds Cove | 58 – Rectangular concrete | 2 – Side Tapered; More favorable edges | Box | 16 | 6 | 78.7 | 989.32 | 989 | 0.011 | 0.011 |

a. The Glenwood Blvd. crossing has a natural channel bottom and therefore the Manning's n was set to match that of Channel Brook.

Table 5. Parameters of wetland control structures modeled explicitly in HEC-RAS

| | Structure Type | Chart # | Scale # | Height (feet) | Width (feet) | Invert (feet) | Length (feet) | Manning's n Top | Manning's n Bottom |
|---|---|---|---|---|---|---|---|---|---|
| Ditch 1 Agri-Drain | Gate – Overflow (closed top) | -- | -- | 2 | 2 | 992.5 | -- | -- | -- |
| Ditch 3 Agri-Drain | Gate – Overflow (closed top) | -- | -- | 2 | 2 | 992.5 | -- | -- | -- |
| Wetland Cell 1-2 Water Leveling Pipe | Culvert | 2 – Corrugated Metal Pipe Culvert | 3 – Pipe projecting from fill | 2.5 | 2.5 | Upstream: 993.25<br>Downstream: 993.25 | 193.21 | 0.021 | 0.021 |
| Wetland Cell 3 Weir | Weir / Embankment | -- | -- | 992.5 | 20 | -- | 12 | -- | -- |

## 3.5 BOUNDARY CONDITIONS

Boundary condition locations are shown in **Figure 3** and listed in Section 3.2. The model spin-up time and flows required to reach antecedent baseflow conditions were established prior to running scenarios with the model. There are no stream gages within the modeled area and as such, various data sources and methods for developing these steady baseflow boundary conditions were utilized. Steady baseflow volumes were informed using PCSWMM model data (OHM Advisors, 2021), USGS regression equations (Jennings et al., 1994), previous studies or other documentation (Buckeye Engineering, 2021), professional judgement based on the size of the contributing drainage area, and iterative model runs. Steady baseflow discharges ranged from 2 to 8 cubic feet per second (cfs) for small tributaries. Channel Brook and the Aurora Lake spillway were applied steady baseflow discharges of 15 cfs and 50 cfs, respectively. With these discharges, the model took 12 hours for streams to reach equilibrium and for the wetland cells to fill up to antecedent conditions.

Section 3.5.1 through Section 3.5.4 describe boundary conditions in more detail.

### 3.5.1 Unsteady Flow Hydrograph Boundary Conditions

Multiple tributaries contribute flow to the HEC-RAS 2-D modeled area. Tributary flows were applied as hydrograph boundary conditions (input flows), each of which use various data sources and rationale for establishing unsteady flow volumes. These tributaries are discussed in detail in Section 3.5.1.1 through Section 3.5.1.4 below.

#### 3.5.1.1 Channel Brook

Channel Brook (Location 4 in **Figure 3**) is ungaged but has been assessed by previous studies that have estimated its 100-year peak flow as 1,440 cfs (Buckeye Engineering, 2021). A multi-step process was used to establish the time series of unsteady flow boundary conditions for Channel Brook (**Figure 7**):

1. The PCSWMM subbasins representative of the ungaged Channel Brook watershed were identified. A hydrograph of average flow from all representative PCSWMM subbasins was calculated at a one-minute time step.
2. This hydrograph was then unitized by dividing the flows by the average effective precipitation from all representative PCSWMM subbasins.
3. To scale this unit hydrograph up to the size of the ungaged Channel Brook watershed, the unit hydrograph flows were multiplied by the ratio of the 100-year peak flow of Channel Brook to the 100-year peak flow of the representative PCSWMM subbasins. This unit hydrograph therefore describes the response of the ungaged Channel Brook watershed to one unit of rainfall.
    a. Channel Brook 100-year flow of 1,440 cfs (Buckeye Engineering, 2021) divided by representative PCSWMM 100-year flow of 91 cfs (OHM Advisors, 2021) equals a ratio of 15.8.
4. Finally, this unit hydrograph of Channel Brook was multiplied by the effective rainfall to get the unsteady flow time series of 100-year design storm for the Channel Brook watershed.

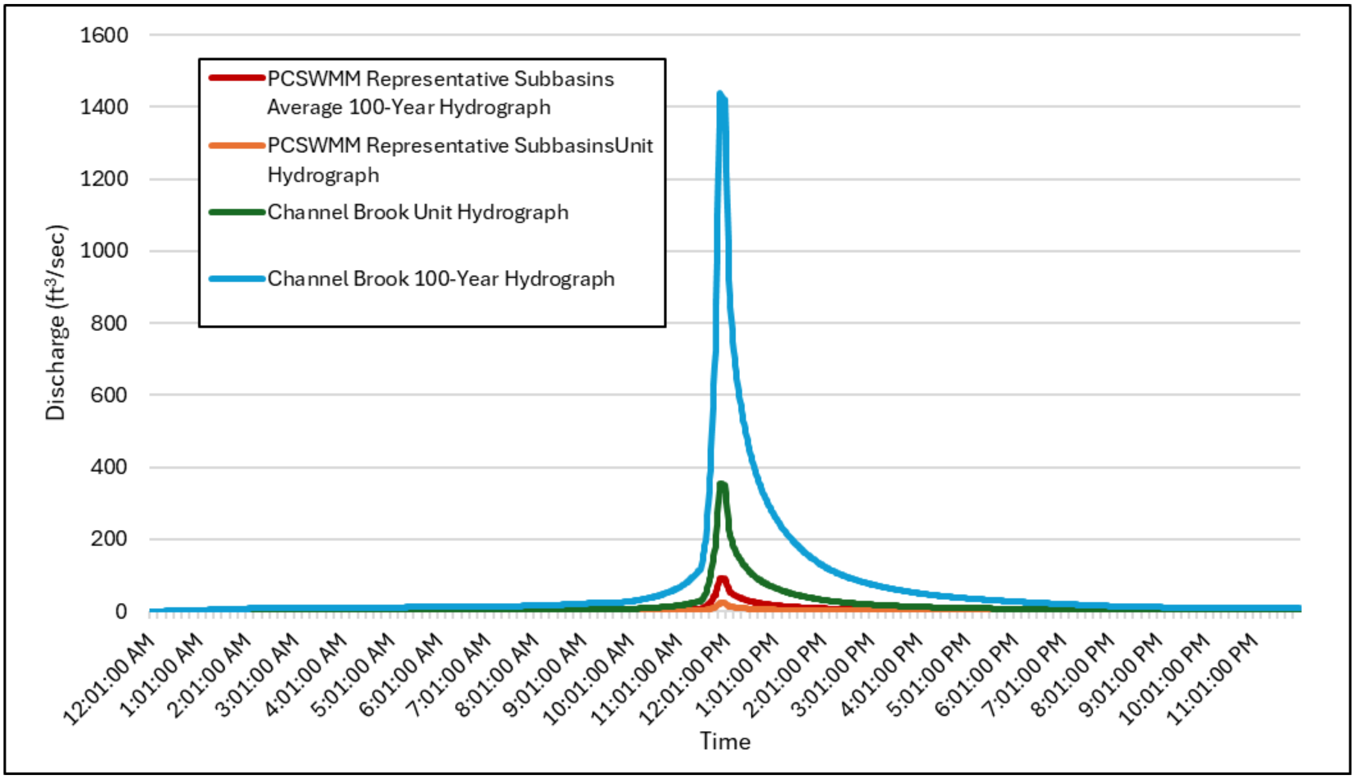


Figure 7. Channel Brook unsteady flow boundary condition development

### 3.5.1.2 PCSWMM Subbasins

The PCSWMM model developed by OHM Advisors (2021) was used for multiple boundary conditions in the northern and eastern edges of the HEC-RAS model extent. Extra care was taken to ensure the perimeter of the HEC-RAS model matched the boundaries of subbasins modeled in PCSWMM. Therefore, the contributing drainage areas modeled by PCSWMM and the associated subbasin outflows could be used directly as the unsteady flow boundary conditions in the HEC-RAS 2-D model for all 24-hour design storms. This procedure was performed for five tributaries shown as Locations 5, 6, 7, 8, and 9 in **Figure 3**.

### 3.5.1.3 Ungaged West Tributaries

Two ungaged tributaries enter the HEC-RAS 2-D modeled area from the west. The northern tributary (Location 3 in **Figure 3**) drains 1.4 square miles of residential, forested, and golf course land uses. Multiple retention basins exist throughout the watershed, the largest of which has its spillway directly at the HEC-RAS 2-D model boundary. Tetra Tech contacted the Ohio Department of Natural Resources (ODNR) Dam Safety Program and the City of Reminderville engineer and no information on the stage-storage or stage-discharge relationships were available for this retention basin. This series of retention basins makes estimating the downstream flows coming into the HEC-RAS 2-D model area extremely difficult to accurately estimate. As such, no unsteady flows for any of the design storms were created. However, these design storm time series for unsteady flow boundary conditions were not required to run any of the scenarios described in Section 4.0 below.

The southern tributary (Location 1 in **Figure 3**) drains one square mile, is largely a natural channel, and flows through Liberty Park east of Liberty Road before its confluence with Pond Brook at the downstream end of the model. USGS regression equations (Jennings et al., 1994) can be used to estimate the 100-year flows from this ungaged tributary.

Developing unsteady flow boundary condition time series can be achieved using the same methods employed for Channel Brook described in Section 3.5.1.1 above. However, these design storm time series for unsteady flow boundary conditions were not required to run any of the scenarios described in Section 4.0 below.

#### 3.5.1.4 Aurora Lake Spillway

The ODNR[2] provided Tetra Tech with as-built drawings and HEC-1 model results for Aurora Lake and its spillway (Location 12 in **Figure 3**). The HEC-1 model results include stage-storage and stage-discharge relationships for Aurora Lake. However, design storm time-series for unsteady flows out of the Aurora Lake spillway was not required to run any of the scenarios described in Section 4.0 below and were therefore not developed; however, such time-series can be added later, if necessary.

### 3.5.2 Stage Hydrograph Boundary Condition

The Channel Brook stream network has one primary inflow and one primary outflow (Locations 4 and 10 in **Figure 3**, respectively). As such, the stage of Aurora Lake greatly controls the flows at the downstream end of the Channel Brook boating canal. The stage in Aurora Lake is a function of many factors including inflows from Channel Brook and other tributaries outside of the HEC-RAS modeled area, the outflow at the dam spillway controlled by a gate of varying elevations, precipitation directly to the lake, evaporation from the lake, and losses to groundwater seepage. Calculating a mass balance or developing a stage-storage relationship in conjunction with Channel Brook discharge is therefore complicated and outside the scope of this modeling project.

Boundary conditions for unsteady flows to Aurora Lake are addressed by a *Scenario 2* and discussed in more detail in Section 4.1.2. For steady baseflow conditions, an Aurora Lake stage of 1001.0 feet was used for the model period. This elevation was informed by multiple sources of data:

1. As-built drawings provided by the ODNR shows the gate opening range as being between 996.5 feet and 1002.1 feet.
2. Buckeye Engineering (2021) lists the Aurora Lake normal pool elevation as 1002.1 feet.
3. Bathymetry maps from the Aurora Shores Homeowners Association website has depths listed relative to a water surface elevation of 1000.9 feet.
4. Terrain used in the model has levee elevations on either side of Channel Brook as low as 1001.5 feet in select locations.
5. The Aurora Shores Homeowners Association monitoring[3] shows water surface elevations in August of 2024 at 1001.3 feet.
6. As-built drawings for the Nautilus Trail bridge lists the boating canal water surface elevation as 1001.0 feet.

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[2] Email communication with Kathleen McDaniel of Ohio Department of Natural Resources on July 17th, 2024. Files provided to Tetra Tech include “Misc. Correspondence.pdf” and “1213-036 - As-Builts - 2009.pdf”.
[3] Aurora Shores Homeowners Association monitoring website: https://dashboard.hobolink.com/public/28914/Aurora%20Lake%20Dam?fbclid=IwAR0Li_vWO7u3B2nyZOhJTu-OHJKMH4pajg7-HwyThysbNyd4tnQOT7xTu1w#/

Ultimately, an elevation of 1001.0 feet was selected as the steady baseflow stage for Aurora Lake. Data sources that support this elevation are the most recent, most reliable, and most applicable to this study. This elevation is also supported by aerial imagery showing the water in the channel being at approximately this elevation when compared to the terrain being used in the model. Additionally, lake stages may vary slightly based on where they are measured (i.e., at the spillway versus at the mouth of Channel Brook). As such, the as-built drawing for the Nautilus Trail bridge was used as the primary data source because this stage is most representative of lake elevations at the Channel Brook mouth.

### 3.5.3 Downstream Boundary Condition

The water-surface elevations at the downstream boundary (outflow) of the model was specified using the "Normal Depth" option in HEC-RAS. The outflow of the model is the downstream end of Pond Brook (Location 13 in **Figure 3**) where the local bed slope of 0.00023 ft/ft was applied as the energy slope.

### 3.5.4 Precipitation Boundary Conditions

Per the Summit County Stormwater Drainage Manual, an SCS Type II temporal distribution pattern was used to simulate rainfall in the HEC-RAS model (Brubaker, 2020). The PCSWMM model precipitation for the 100-year event was used as the HEC-RAS rain-on-grid boundary condition. Infiltration was not explicitly modeled in HEC-RAS. Rather, the total infiltration modeled in PCSWMM was used to decrease the total amount of precipitation of the design storm to calculate the effective rainfall. OHM Advisors provided infiltration depths on a subbasin scale for all design storms modeled in PCSWMM. As such, two values for total infiltration were considered for use: (1) the average infiltration among all PCSWMM subbasins; or (2) the infiltration in the subbasin of interest. In this case, the subbasin of interest is PCSWMM model subbasin 43 because it encompasses the flooding occurring the backyards of homes on Windjammer Trail and Sea Ray Cove. Additionally, the land use in this subbasin is representative of the Aurora Shores neighborhood as a whole. Finally, it was assumed that infiltration rates of the soils within the HEC-RAS modeled area are the same throughout.

The 100-year design storm precipitation depth is 5.52-inches and the total infiltration of the PCSWMM model subbasin is 1.86-inches. Therefore, effective rainfall applied to the 24-hour SCS Type II precipitation event is 3.66-inches (**Figure 8**).

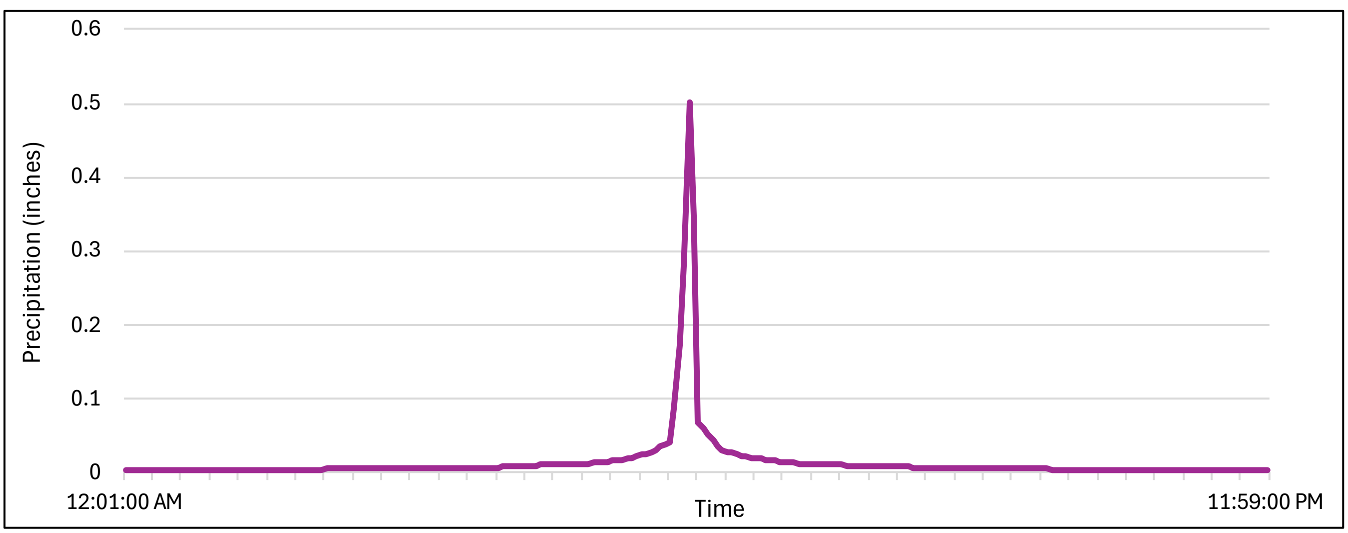



Figure 8. 100-year design storm hyetograph with cumulative effective rainfall of 3.66-inches

# 4.0 SCENARIOS

Due to the two separate drainage networks shown in **Figure 1**, scenarios were set up and run separately for each network to gain a better understanding of the source of observed flooding and potential solutions. Flooding has occurred in two primary locations within the Aurora Shores neighborhood. The flooding in the vicinity of the Nautilus Trail and Anchorage Cove intersection was evaluated by the OHM Advisors (2021) study and resulted in the replacement of the Clipper Cove aqueduct in Spring 2024. The second location of flooding occurs in the backyards of homes on Windjammer Trail and Sea Ray Cove, east of Wetland Cell 1 and Ditch 1.

## 4.1 CHANNEL BROOK

One potential source of the flooding in the Aurora Shores neighborhood is due to the Channel Brook stream network (**Figure 1**). As such, three scenarios were set up and run (Section 4.1.1 through Section 4.1.3) to evaluate the following:

1. Channel Brook overtopping its banks due to 100-year flows entering upstream of Glenwood Boulevard.
2. Channel Brook overtopping its banks due to backwater effects from Aurora Lake.
3. Large flows from the Aurora Lake spillway creating a backwater effect throughout the wetland cells adjacent to Pond Brook to the extent that water surface elevations rise and inundate roads or structures within the Aurora Shores neighborhood.

Running these scenarios provided insight into the likely causes of the Aurora Shores flooding and potential solutions, such as lowering the water level of Aurora Lake prior to storm events to provide additional storage within the drainage network.

### 4.1.1 Scenario 1 - Channel Brook 100-Year Event

*Scenario 1* was developed to assess whether Channel Brook overtops during the 100-year event, and if so, to determine if Channel Brook alone is the isolated source of the observed flooding. The inflows for the 100-year flow are discussed in detail in Section 3.5.1.1. All other inflows were simulated as steady baseflow discharges, and the downstream Channel Brook stage was set at a constant elevation of 1001.0 feet (representing normal stage of Aurora Lake).

The 100-year peak-flow entering the model via Channel Brook in this scenario is 1,440 cfs (**Figure 9**). The predicted maximum extent and depth of inundation is shown in **Figure 10**. The Glenwood Boulevard crossing does not have the capacity to convey these flows and therefore ponding occurs upstream. Moving from upstream to downstream, the following can be noted this scenario:

- No structures are inundated in this scenario, except that one home on Morley Drive (in the northwest corner of **Figure 10**) experiences approximately 3-inches of water abutting the rear of the structure.
- Glenwood Boulevard experiences a maximum of 1-inch of inundation at the road crest along its centerline and 3-inches of inundation along its shoulder. A minimal amount of flow (approximately 3 cfs) overtops Glenwood Boulevard to reach the forested area southeast of the road before entering the Pond Brook tributary (**Figure 10**).
- The maximum water surface elevation of Channel Brook immediately downstream of Glenwood Boulevard is 1003.1 feet and decreases to 1002.4 feet about 1,300 feet downstream (**Figure 10**).
- The 100-year Channel Brook peak flow of 1,440 cfs attenuates to 451 cfs at Glenwood Boulevard, a reduction about 69% (**Figure 9**).

Through Tetra Tech's review of previous studies and reports, there is no known history of Channel Brook overtopping. This scenario substantiates the finding that Channel Brook will not overtop its levees in the Aurora Shores neighborhood due to the 100-year flows from Channel Brook upstream of Glenwood Boulevard.

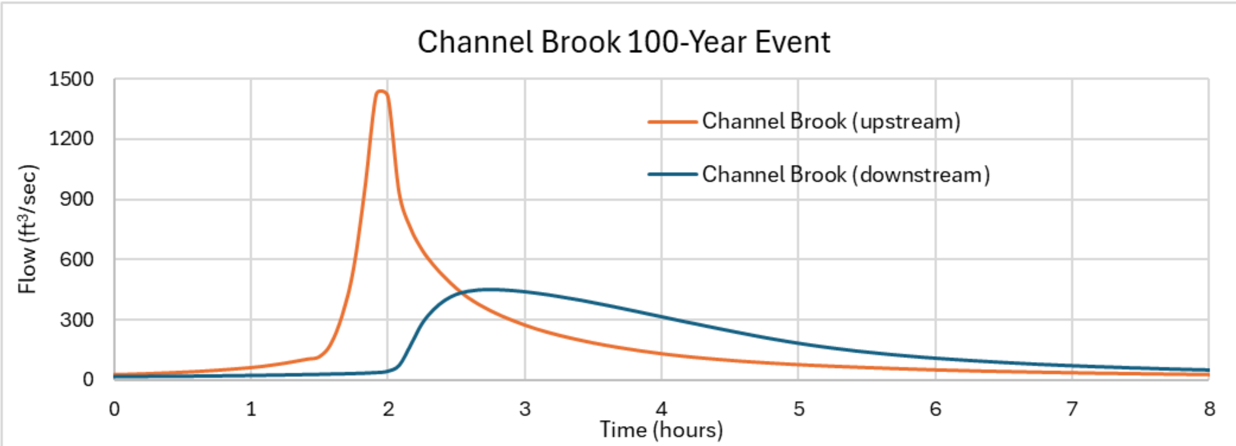



Figure 9. Hydrograph response to the *Channel Brook 100-Year Event* scenario

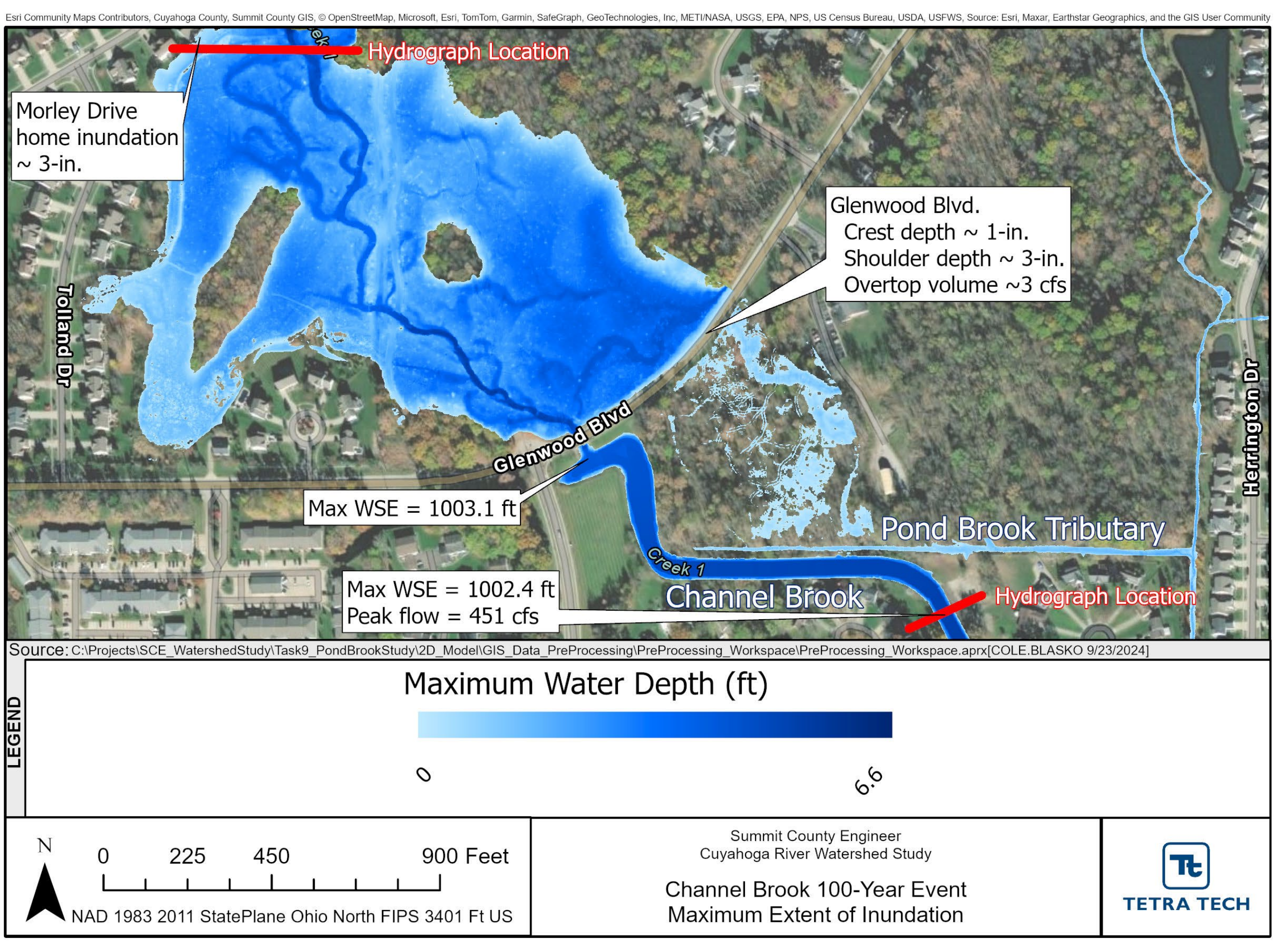



Figure 10. Maximum extent of inundation during the *Channel Brook 100-Year Event* scenario

### 4.1.2 Scenario 2 - Aurora Lake Stage Increase

As shown by the previous scenario, the model predicts Channel Brook does not overtop its levees in response to the 100-year flows coming from Channel Brook upstream of Glenwood Boulevard. Another possible source of Channel Brook flooding is from the downstream end where the stage of Aurora Lake may cause backwater flooding sufficient to overtop the Channel Brook levees. This scenario was set up with all flow hydrograph boundary conditions simulated at steady baseflow discharges. However, the stage of Aurora Lake was steadily increased from 1000.0 feet to 1003.0 feet over 36 hours. Setting up the scenario in this way will show locations in which the Channel Brook levee is overtopped and the associated stage of Aurora Lake when the flooding occurs. The lake stage associated flooding can then provide information about how to manage the lake in the future (e.g., release water before storms) to mitigate potential flooding.

The Channel Brook levee is overtopped in six locations as the stage of Aurora Lake is increased from 1000.0 feet to 1002.0 feet. Locations of each point of overtopping and the associated Aurora Lake stage are shown in **Figure 11**. Flooding first occurs at 1001.5 feet as the south bank of Channel Brook is overtopped 400 feet upstream of its confluence with Aurora Lake. As the stage was increased from 1002.0 to 1003.0 feet, multiple locations downstream of Nautilus Trail were overtopped nearly simultaneously.

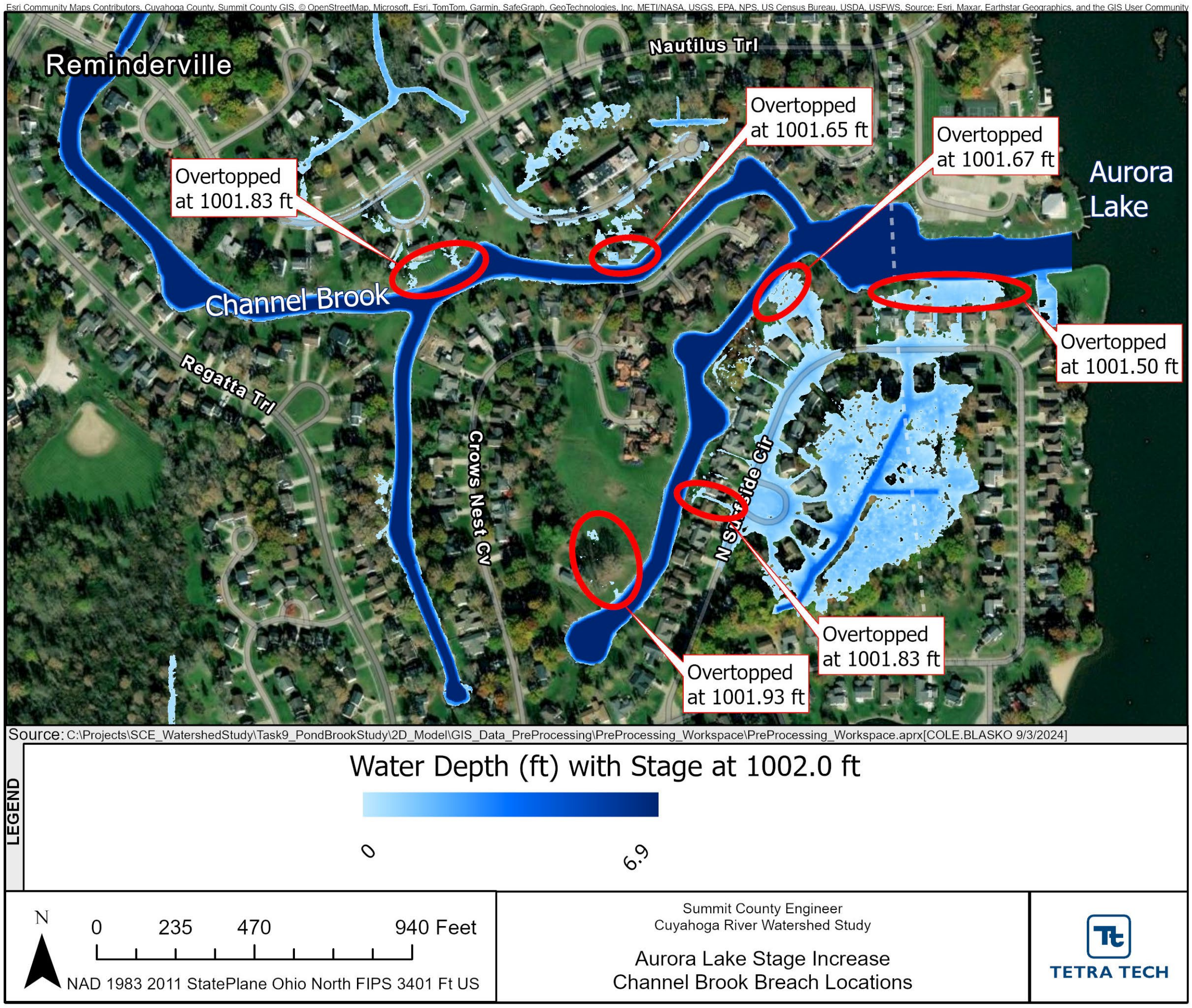



Figure 11. Extent of inundation and overtopping locations with Aurora Lake stage at 1002.0 ft

Knowing that flooding occurs at an Aurora Lake stage of 1001.5 feet, Tetra Tech was able to calculate the amount of storage needed in Aurora Lake to eliminate the flooding. The amount of storage can be back-calculated using the HEC-1 results provided by the ODNR[4] for the lake. The approach is outlined below:

1. First, flow volumes from all tributaries entering Aurora Lake must be estimated for the 100-year event.
    a. The Channel Brook 24-hour 100-year event (**Figure 7**) contributes 5,741,420 feet$^3$ or 131.8 acre-feet of water to Aurora Lake.

[4] Email communication with Kathleen McDaniel of Ohio Department of Natural Resources on July 17th, 2024. Files provided to Tetra Tech include “Misc. Correspondence.pdf” and “1213-036 - As-Builts - 2009.pdf”.

b. The routing and schematic diagram of the stream network in the HEC-1 model indicates that six subbasins contribute flow to Aurora Lake. An approximation of the volume Channel Brook contributes compared to the total flow volume of all Aurora Lake subbasins was made using the peak flows of the 0.12 PMF (probable maximum flood) event provided by the HEC-1 model. This approach requires the assumption that the ratio of peak flows is equal to the ratio of the total flow volumes from each subbasin. See Section 5.0 for discussion on this assumption. Channel Brook (subbasin 1 in the HEC-1 results) has a peak flow of 1,138 cfs while the combined Aurora Lake subbasins have a peak flow of 3,078 cfs. As such, Channel Brook contributes approximately 37% of the total flow to Aurora Lake.

c. Using this ratio of 37%, the total flow volume to Aurora Lake can be extrapolated from the Channel Brook flow volume. Aurora Lake therefore receives approximately 356.2 acre-feet of flow during the 100-year event.

2. Next, the stage-storage relationship for Aurora Lake was established using the HEC-1 results (**Figure 12**). Six discreet state-storage points were used to establish a linear regression where for every one-foot of increase in stage, approximately 471 acre-feet of storage are added to the lake.

3. Finally, the 100-year flow volumes coming into the lake (352.2 acre-feet) were compared to the additional storage provided by lowering the lake stage by one foot (471 acre-feet).

The *Aurora Lake Stage Increase* scenario run in HEC-RAS established that Channel Brook first overtops at an elevation of 1001.5 feet near its downstream end 400 feet away from its confluence with Aurora Lake. The calculations outlined above indicate that lowering Aurora Lake by one foot to a stage of 1000.5 feet provides approximately 471 acre-feet of storage, which is more than enough to store the estimated 356.2 acre-feet of flow volume coming into Aurora Lake from all its tributaries. The gate of the Aurora Lake spillway is operable between elevations of 996.5 feet and 1002.1 feet. As such, the stage of Aurora Lake can be lowered well within the spillway's operational limit and can provide more than enough storage prior to the 100-year precipitation event to prevent Channel Brook from overtopping.

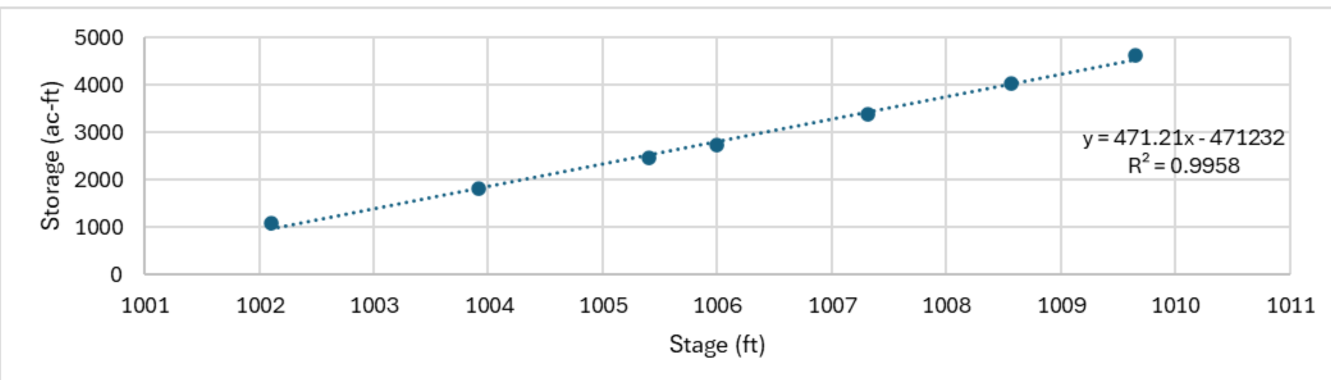



Figure 12. Stage-Storage relationship of Aurora Lake

### 4.1.3 Scenario 3 - Aurora Lake Spillway Backwater

The last potential source of flooding from the Channel Brook stream network is where flows from the Aurora Lake spillway causes a backwater effect within the wetland cells that is great enough for upstream water surface elevations to rise and inundate roads or structures within the Aurora Shores neighborhood. A scenario was set up with discharge from the Aurora Lake spillway increasing linearly to a maximum of 779 cfs before linearly returning to baseflow conditions (**Figure 13**). All other flow hydrograph boundary conditions were simulated at steady baseflow discharges and the downstream Channel Brook stage was set at a constant elevation of 1001.0 feet.

The magnitude of the Aurora Lake spillway peak flow is equal to the HEC-1 model maximum outflow from the dam during the 0.12 PMF. The volume of water released from the reservoir over this 24-hour period is 822 acre-feet. Using information from the *Aurora Lake Stage Increase* scenario above, this volume of water would lower the stage of the reservoir by 1.7 feet and provide 2.3 times the volume needed to store the flow volume of the 100-year event. Therefore, this magnitude of flow is not expected under normal dam operations and would likely only occur if Aurora Lake was close to overtopping its emergency spillway. This discharge should be used as a worst-case scenario where large volumes of water are required to be released from the reservoir in a short amount of time through lowering of the spillway gate.

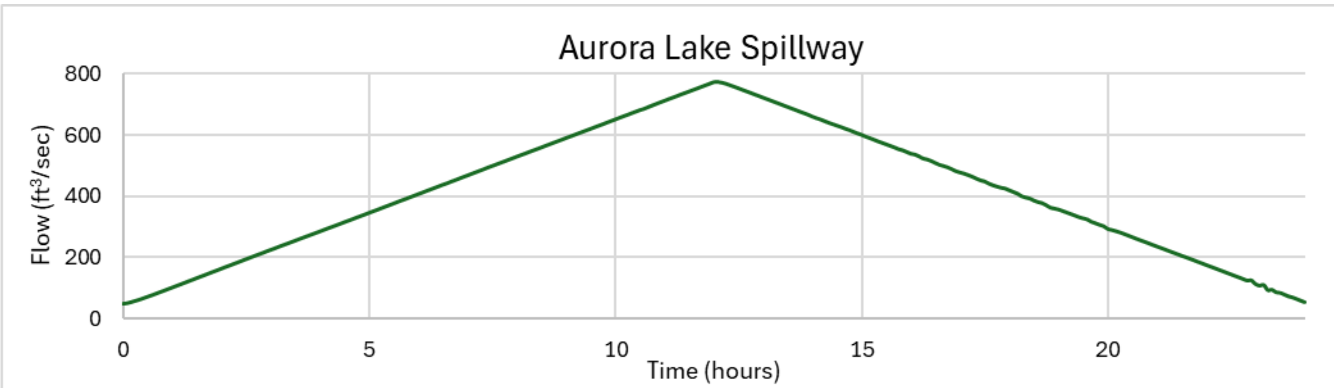



Figure 13. Aurora Lake spillway discharge

The depth and extent of flooding in response to the dam release described above is shown in **Figure 14**. The inundated extent resulting from this scenario approaches buildings on Regatta Trail, Windjammer Trail, and Sea Ray Cove and is comparable to the FEMA 100-year flood plain extent. Considering this discharge from the Aurora Lake spillway as a worst-case scenario, it is unlikely that flooding in the Aurora Shores neighborhood is due to these backwater effects.

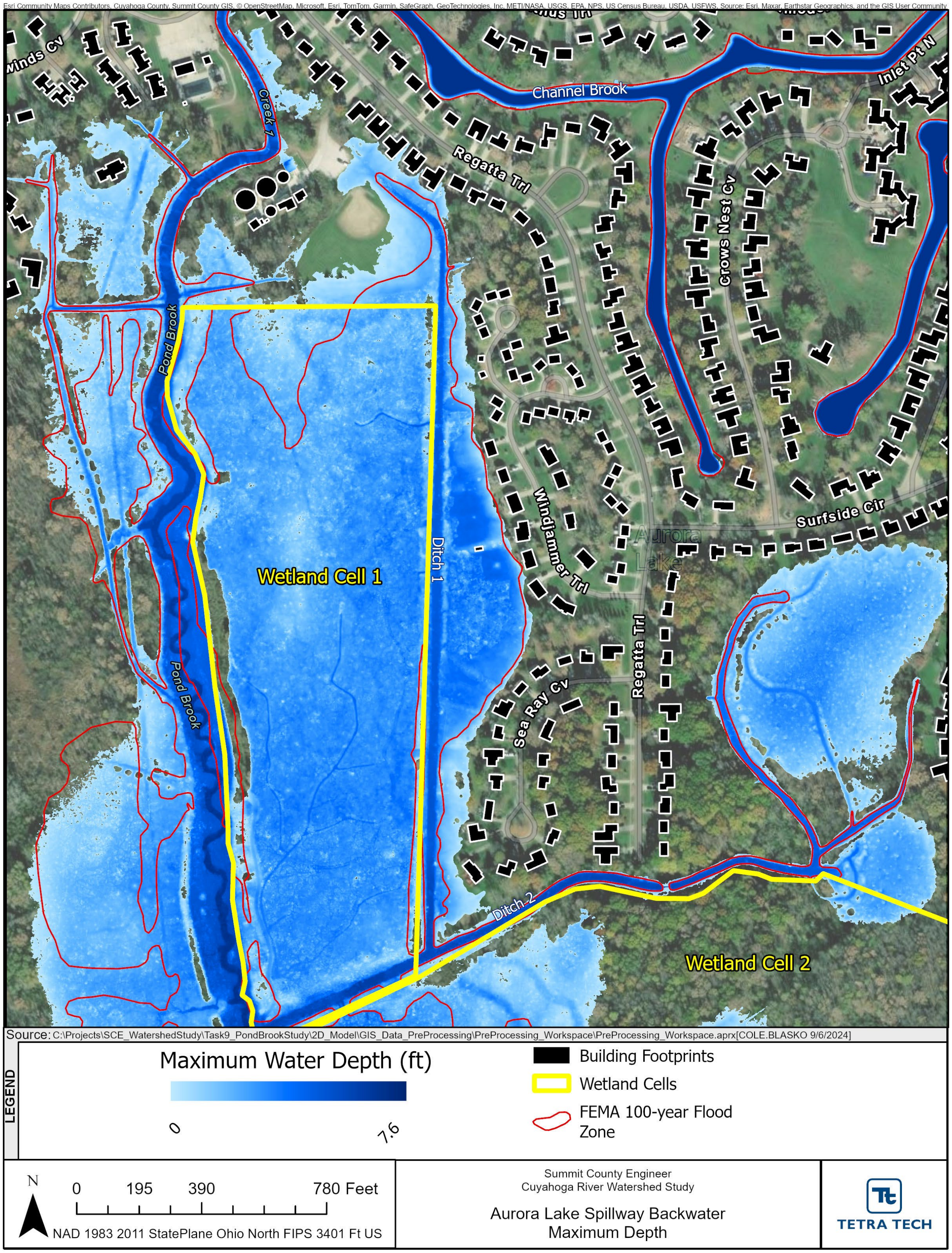



Figure 14. Maximum water depth relative to the FEMA 100-year flood zone and building footprints resulting from the Aurora Lake spillway release.

## 4.2 POND BROOK

Another potential source of flooding in the Aurora Shores neighborhood is the Pond Brook stream network (**Figure 1**). As such, two scenarios were set up and run (*Scenario 4* and *5*) to test whether adjustments of the Agri-Drain stoplogs that control the water surface elevations in the wetland cells could be used to mitigate the flooding in the backyards of homes on Windjammer Trail and Sea Ray Cove.

The Agri-Drain nearest to the flooding in question is in Ditch 1 and was installed in conjunction with a diversion embankment that is designed to divert flow from the north end of Ditch 1 into Wetland Cell 1 (Wetlands Resource Center et al., 2006; **Figure 15** and **Figure 16**). The Agri-Drain and diversion embankment are located west of Windjammer Trail and Windjammer Cove. There is no active tributary or outfall from the nearby wastewater treatment plant contributing flow to the north end of Ditch 1. Rather, the ditch primarily serves as conveyance for surface runoff coming from the Aurora Shores neighborhood from the north and east during and after a precipitation event. As such, both Pond Brook scenarios were set up using the 100-year precipitation boundary condition described in Section 3.5.4. All other flow and stage hydrograph boundary conditions were simulated with steady baseflow values to gain insight as to whether precipitation alone is the source of this flooding and how modification of the Agri-Drain elevations may provide a solution.

### 4.2.1 Scenario 4 - Design Agri-Drain Elevation = 992.5 feet

The Agri-Drains were designed to operate at an elevation of 992.5 feet (Wetlands Resource Center et al., 2006). This scenario was set up to represent those design conditions. Section 3.4.2 discusses in greater detail how this Agri-Drain was modeled in HEC-RAS.

The maximum depth of inundation in the backyards on Windjammer Trail and Sea Ray Cove is approximately two feet while the extent of inundation is approximately equal to that of the FEMA 100-year floodplain (**Figure 15**). The homes themselves are not inundated under this scenario, only the backyards are. Note that the depth and extent of inundation shown in **Figure 15** are at hour 27 of the simulation which is three hours after peak rainfall of the 100-year event. At this 27-hour time step, rainfall intensity has already decreased and the flood wave has begun receding because it is being conveyed by Ditch 1 and through Wetland Cell 1.

### 4.2.2 Scenario 5 - Raised Agri-Drain Elevation = 994.5 feet

The Agri-Drain and diversion embankment control the amount of flow that travels from the upstream end of Ditch 1 to the south where the yards are flooded on Windjammer Trail and Sea Ray Cove. As such, it was believed that if the elevation of the Agri-Drain was raised, more flow would be diverted to Wetland Cell 1 and less flow would be available to travel south into the flooded area. This scenario was set up with the invert elevation of the Agri-Drain at 994.5 feet, which is two feet higher than the design conditions. Section 3.4.2 discusses in greater detail how this Agri-Drain was modeled in HEC-RAS.

When the Agri-Drain invert is raised to an elevation of 994.5 feet, the depth and extent of inundation is similar to the scenario with the Agri-Drain set to the design elevation of 992.5 feet. A comparison of these two scenarios is discussed in Section 4.2.3.

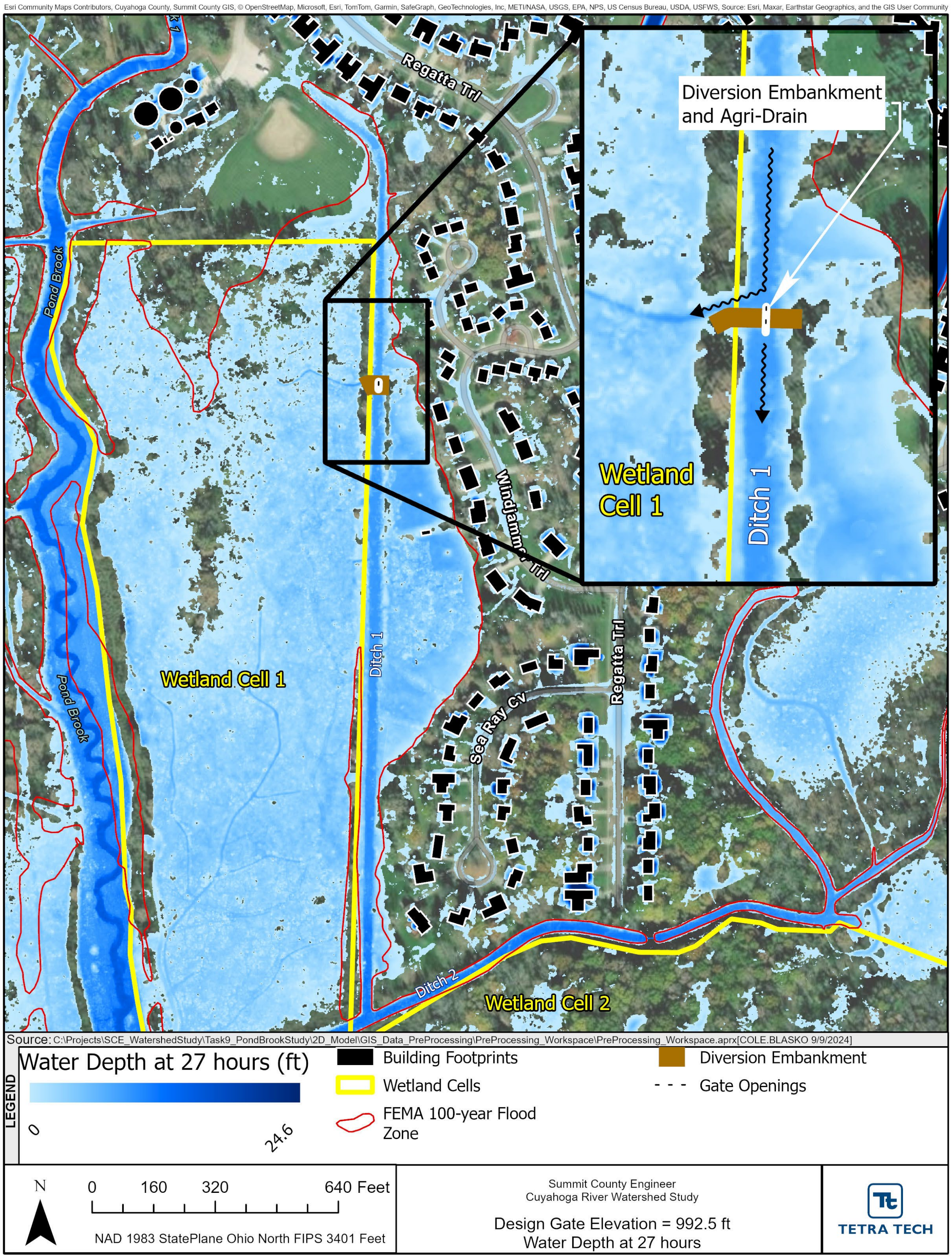



Figure 15. Predicted depth and extent of inundation with Agri-Drain invert elevation set to design conditions at 992.5 feet.

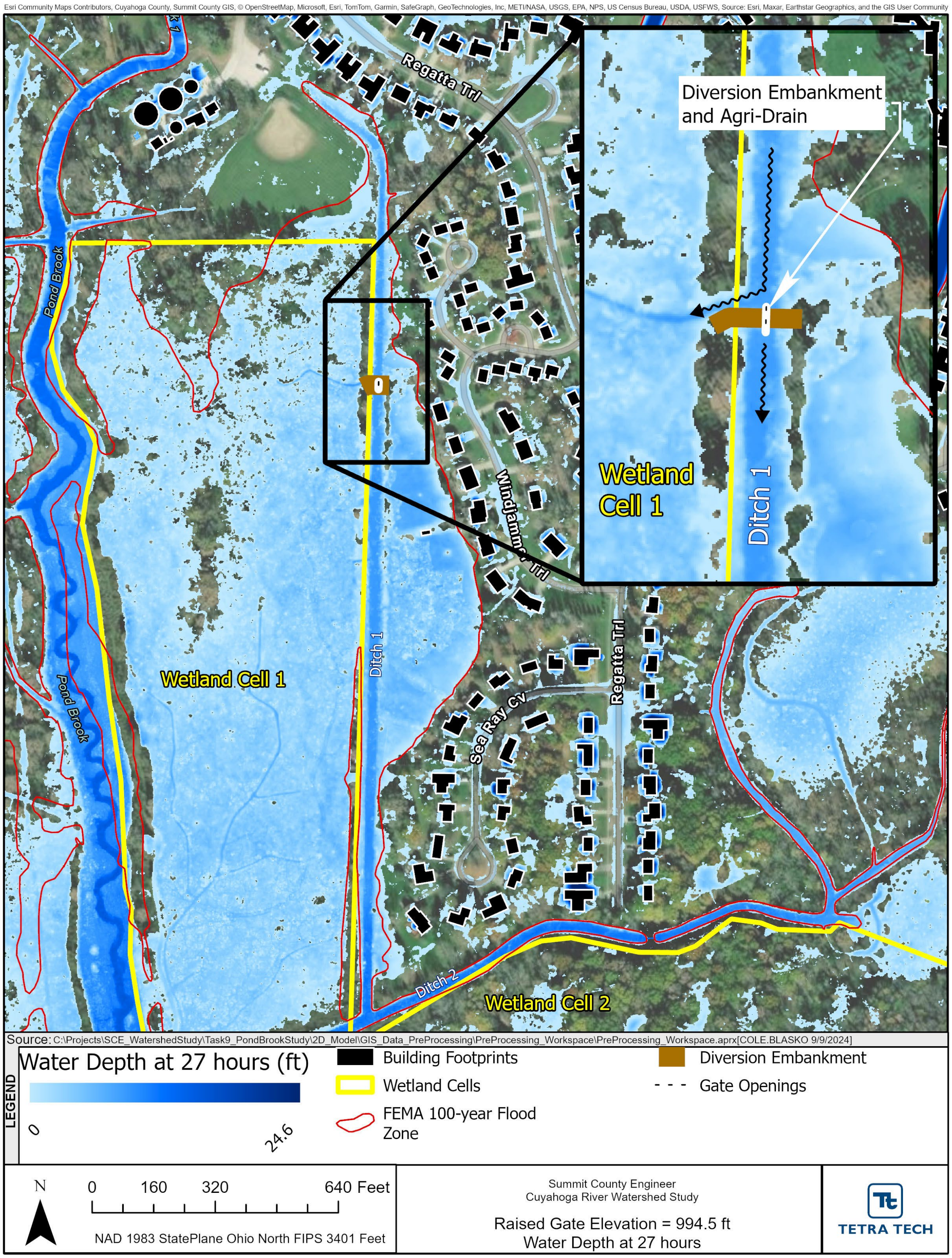



Figure 16. Predicted depths and extent of inundation with Agri-Drain invert elevation set to raised conditions at 994.5 feet.

### 4.2.3 Agri-Drain Elevation Comparison

The depth and extent of flooding is nearly identical between the *Design Agri-Drain Elevation* and *Raised Agri-Drain Elevation* scenarios discussed above (**Figure 15** and **Figure 16**). However, there are substantial differences between the scenarios regarding how much flow is diverted into Wetland Cell 1 versus how much flow is conveyed south via Ditch 1. When the Agri-Drain is set at its design elevation of 992.5 feet, two additional acre-feet of cumulative flow is conveyed via Ditch 1 to the south compared to when the Agri-Drain invert elevation is raised to 994.5 feet (**Figure 17**). This difference is only appreciable after the period of high intensity precipitation of the 100-year event that occurs at the 24-hour mark of the model run. Additionally, the amount of flow being diverted to Wetland Cell 1 is negative under the *Design Agri-Drain Elevation* scenario (**Figure 17**). This indicates that water is traveling from the Wetland Cell 1 into Ditch 1. Conversely, when the Agri-Drain invert elevation is raised to 994.5 feet, approximately 0.35 acre-feet of cumulative flow is diverted from Ditch 1 and into Wetland Cell 1. This assessment shows that the flooded area of concern receives less water under the *Raised Agri-Drain Elevation* scenario.

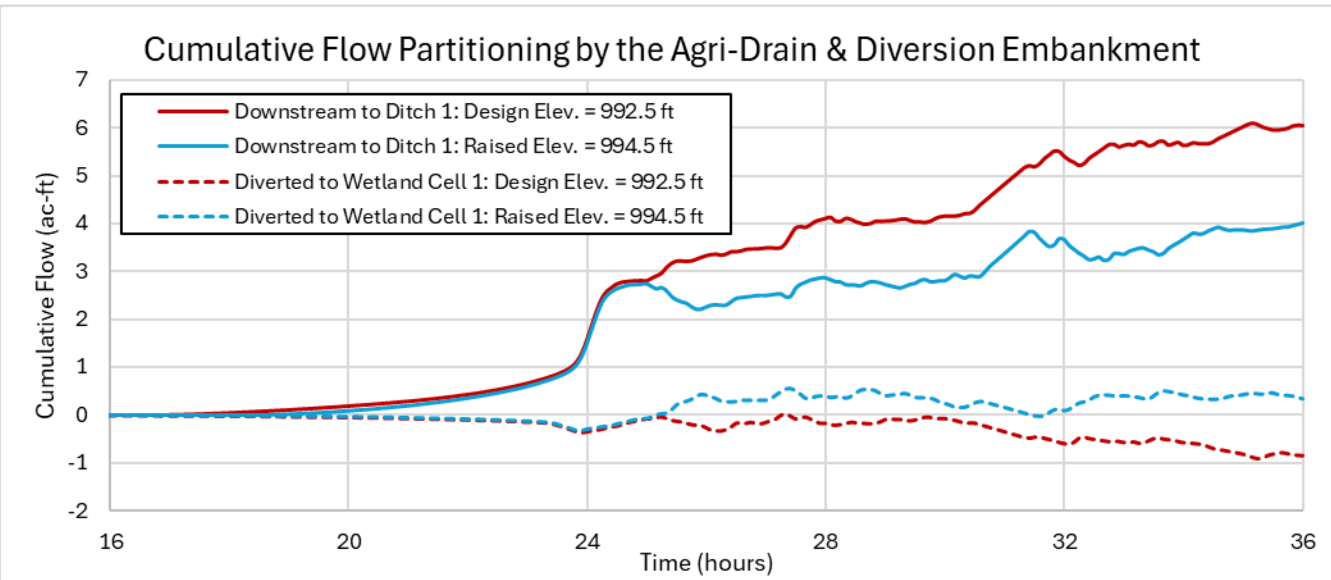



Figure 17. Cumulative flows partitioned either south via Ditch 1 or diverted west to Wetland Cell 1

Water surface elevations upstream and downstream of the diversion embankment were also assessed under both scenarios. Downstream of the diversion embankment, the water surface elevation is nearly identical under both scenarios for low flow conditions prior to the period of high intensity precipitation and for high flow conditions after the period of high intensity precipitation (**Figure 18**). This may appear contrary to the findings discussed above where cumulative flows in Ditch 1 were greater when the Agri-Drain invert elevation was at design conditions of 992.5 feet. However, this shows that Ditch 1 can convey these larger flows without appreciably raising the water surface elevation downstream of the embankment. The capacity of Ditch 1 to convey these greater flows with no increase in water surface elevation may be attributed to not being constricted in its connection to the downstream network of Ditch 2 and Pond Brook.

Upstream of the embankment, water surface elevations are greater when the Agri-Drain elevation is raised to an invert of 994.5 feet but only prior to the high intensity period of precipitation (**Figure 18**). This is because the Agri-Drain at this higher elevation is not receiving any water during these low flows and the embankment itself is impounding water behind it and diverting more flow to Wetland Cell 1. After the period of high intensity precipitation of the 100-year event, water surface elevations are nearly identical regardless of Agri-Drain invert elevation or location relative to the embankment. This is because at the ~24.5-hour mark of the simulation, the embankment is completely overtopped by water within Ditch 1 at a water surface elevation of 994.5 feet. Finally, the volume of water passed by the two-foot diameter Agri-Drains are negligible compared to the total volume in Ditch 1 under these high flow conditions and therefore have negligible impacts.

It should be noted that under the *Raised Agri-Drain Elevation* scenario, flow into the Agri-Drain is only accessed at these peak water surface elevations and effectively provides no additional conveyance because the embankment is also being overtopped at this time. In other words, the *Raised Agri-Drain Elevation* scenario is set up such that the Agri-Drain is not present at all. This was not expected when the scenario was first set up and was only realized after the results were assessed. However, the intention of the scenario remains true where the idea of raising the elevation of the Agri-Drain would result in more flow being diverted to Wetland Cell 1 and less flow would be available to travel south into the flooded area.

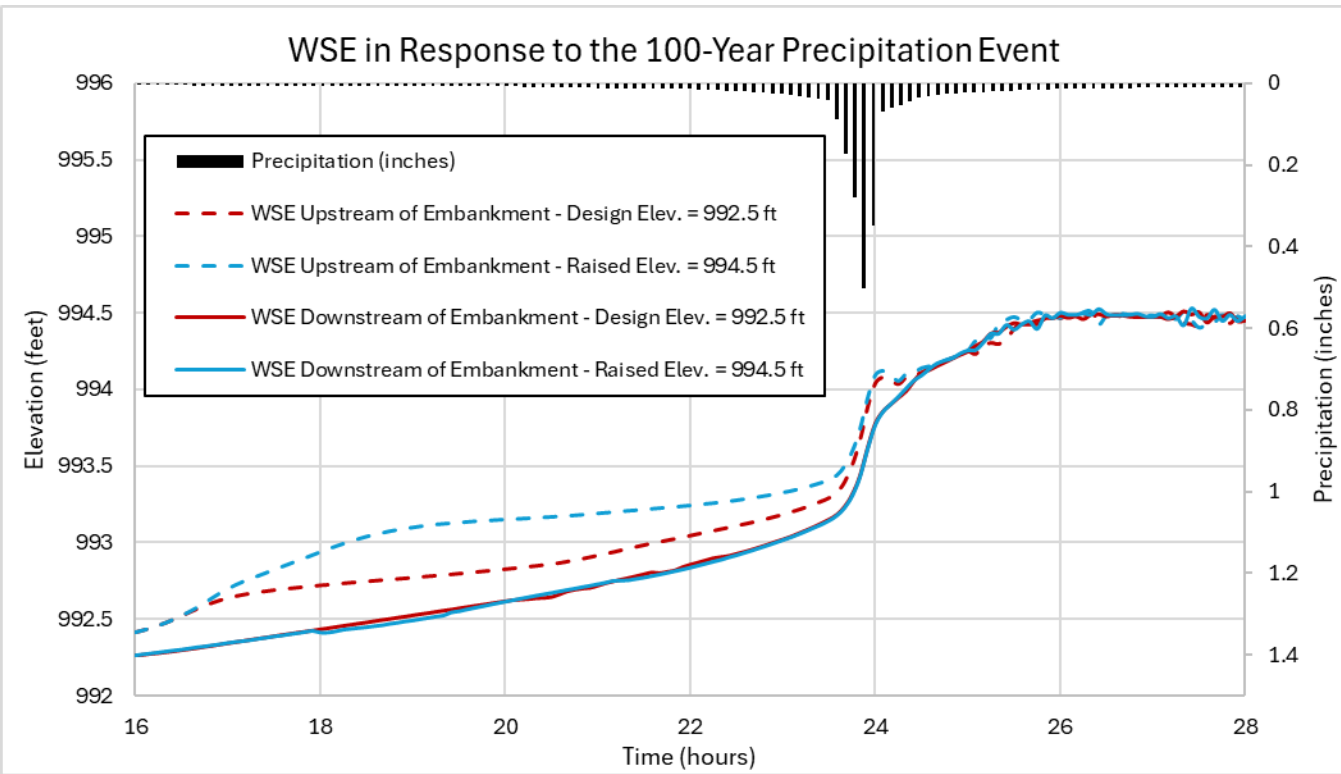



Figure 18. Water surface elevations relative to the diversion embankment under both Agri-Drain scenarios.

The final component assessed using the Agri-Drain elevation scenarios involves comparison of the elevations of Wetland Cell 1, Ditch 1, and the flooded backyards. **Figure 19** shows a representative cross section cut in a west-east direction from Wetland Cell 1 to the homes on Windjammer Trail at peak water surface elevations. Two north-south embankments run parallel to Ditch 1 on both sides. However, these embankments are not continuous and are overtopped in select locations of lower elevations when water surface elevations reach approximately 993.4 feet (**Figure 15** and **Figure 16**). As such, the water surface elevation is equilibrated across Wetland Cell 1, Ditch 1, and the flooded backyards. Elevations in the back yards of Windjammer Trail and Sea Ray Cove are as low as 992.2 feet which is only 0.3 feet higher than the channel bottom of Ditch 1 at 991.9 feet. The yards are also lower than the typical elevation of 993.0 feet in Wetland Cell 1.

Identical water surface elevations across the wetland cell, ditch, and backyards also indicates that removal of the Agri-Drain and the embankment in their entirety will likely not impact water surface elevations. This is because the flood waters are overtopping local low-elevation areas regardless, as confirmed by the equilibrated water surface elevation (WSE) shown in **Figure 19**.

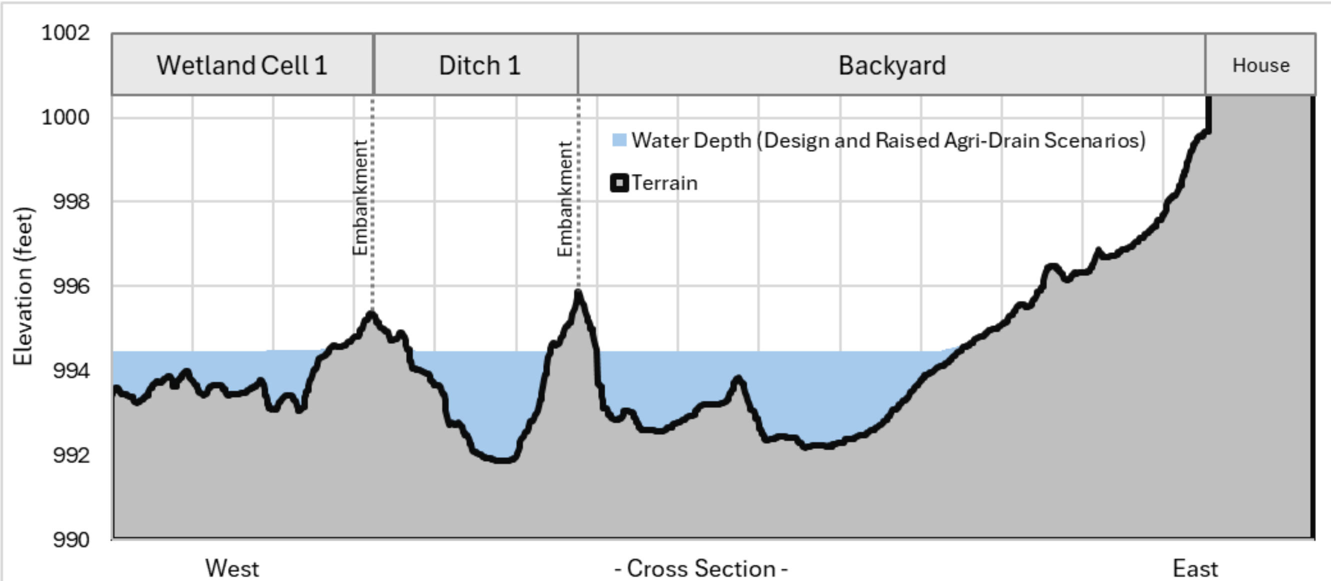



Figure 19. Cross section of terrain and water surface elevations downstream of the Agri-Drain and embankment structure

These results show that the Agri-Drains do convey water through the diversion embankment at a controlled rate during low flows and can be modified to divert larger volumes of water into Wetland Cell 1. However, the water surface elevation in the flooded backyards is largely a function of the low elevation of the backyards and the surface runoff that flows here from the upland areas of the Aurora Shores neighborhood; specifically, Windjammer Cove, Windjammer Trail, and Sea Ray Cove. Furthermore, Ditch 1 conveyed an additional 2 acre-feet of cumulative flow with no increase in water surface elevation as shown by **Figure 17** and **Figure 18**. As such, increased flows in Ditch 1 likely do not worsen the flooding in the backyards. Rather, precipitation, lack of infiltration capacity, and the flat/low lying nature of the backyards are the main drivers of the flooding in this area.

## 4.3 INCREASED HYDRAULIC CAPACITY DOWNSTREAM

State Route 82 (East Aurora Road) and the Wheeling and Lake Erie Railway Company[5] railroad bridge are approximately 0.94-miles and 2.08-miles downstream of the area modeled by the 2-D HEC-RAS domain described in Section 3.0, respectively. While this downstream reach of Pond Brook was not modeled by Tetra Tech, the Summit County Engineer wanted to evaluate whether the S.R. 82 or railroad bridge crossings create a backwater effect that extends to the Aurora Shores neighborhood and contributes the localized flooding. To test this theory, a 1-D HEC-RAS model developed by Stantec in 2017 (Stantec, 2017) was evaluated because it includes the S.R. 82 and railroad crossings. The extents of the Tetra Tech 2-D and Stantec 1-D models are shown in **Figure 20**.

[5] Online maps identify these rail lines as owned by the Norfolk & Western (N&W) Railway that is a predecessor to Norfolk Southern Railway.

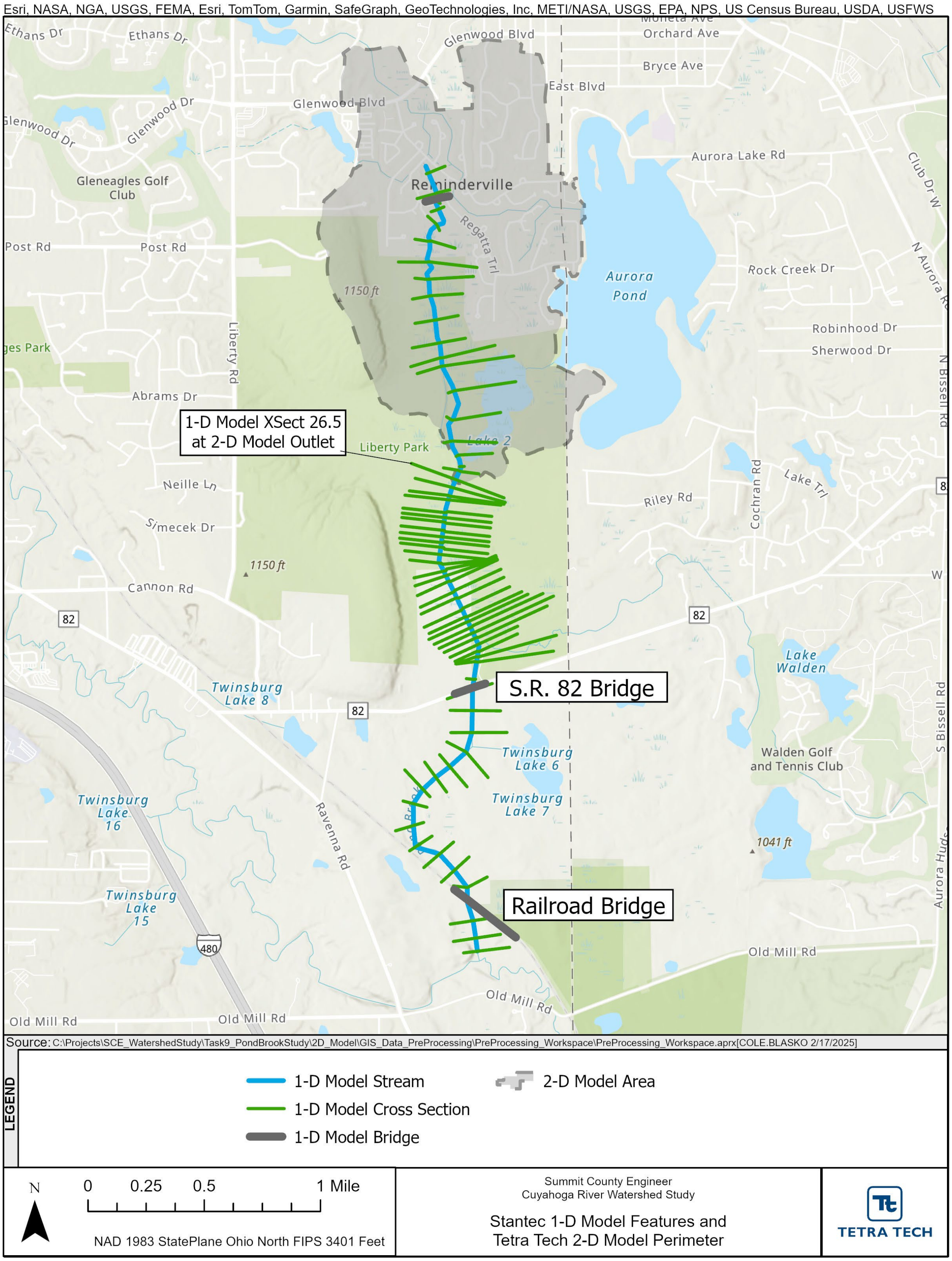



Figure 20. Extents of the 1-D Stantec HEC-RAS model and the 2-D Tetra Tech HEC-RAS model.

Tetra Tech evaluated the potential to reduce flooding in the Aurora Shores neighborhood by increasing the hydraulic capacity at the two bridge crossings in the 1-D model using four scenarios:

1. **As-Built** - Bridges were simulated under as-built conditions represented in the Stantec 1-D model with an opening width between the bridge abutments of 66.5-ft.
2. **20-ft Abutment Expansion** - The opening width between the bridge abutments was increased by 20-ft (10-ft in both directions relative to as-built conditions). This scenario was run to test whether a small, but feasible increase in the hydraulic capacity of the bridges would result in improved backwater elevations. Ineffective flow areas up- and downstream were adjusted proportionally to those in the As-Built scenario.
3. **Floodplain Abutment Expansion** – Bridge abutments were widened to the width of the floodplain as estimated by the underlying terrain in the cross sections (**Figure 21**). This resulted in opening widths between abutments of 300-ft and 450-ft for the S.R. 82 and Railroad bridges, respectively. Bridges of this size are likely cost-prohibitive and therefore not feasible from an implementation standpoint. However, this scenario was run to test whether any increase in bridge hydraulic capacity would result in improved backwater elevations. Ineffective flow areas up- and downstream were adjusted proportionally to those in the As-Built scenario.
4. **Bridges Removed** – Bridge cross sections, including their abutments and ineffective flow areas, were removed in their entirety from the model.

The steady flow boundary conditions representative of the 100-year event in the Stantec model were used in the four scenarios described above.

Water surface elevations upstream of the two bridges are shown in **Figure 22**. Modification of the hydraulic capacity of the bridges resulted in minimal changes in water surface elevations within the Liberty Park Pond Brook Conservation Area. Namely, at cross section 21 (shown in **Figure 22**) the elevation difference between the As-Built scenario and Bridges Removed scenario is 1.5-inches. Therefore, the backwater effect from the bridges under existing conditions is minimal in the reach immediately upstream of the S.R. 82 bridge. Further north, at the downstream extent of the 2-D model (shown in **Figure 22**) the water surface elevation differences between scenarios is reduced. Under the Bridges Removed scenario, water surface elevations are lowered by 0.5-inches. Under the 20-ft Abutment Expansion scenario, water surface elevations are lowered by 0.1-inches relative to the As-Built scenario. These results show that any potential backwater effects caused by the two bridges in question are negligible in the 2-D HEC-RAS modeled area. As such, re-running the 2-D model with modified downstream boundary condition parameters was not required.

Direct comparison of the 1-D and 2-D models for a given design storm is not possible because of the differences in model extent and inflow boundary conditions. However, as a final check, the stage-discharge relationships of the 1-D and 2-D models were compared at the location where the normal depth boundary condition of the 2-D model was assigned which corresponds to the cross section at river station 26.5 in the 1-D model. After elevation adjustments to account for differences in vertical datums, the stage-discharge relationships were shown to be similar between the two models.

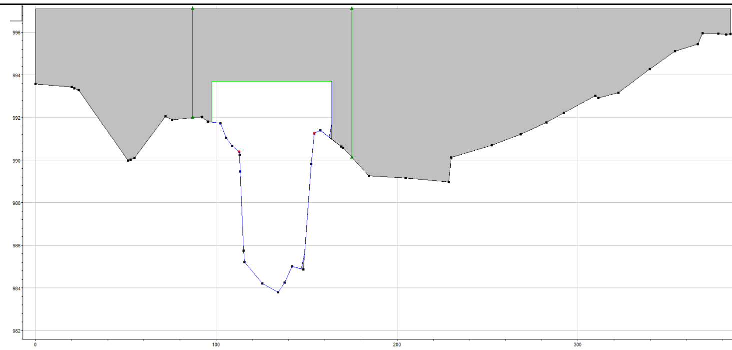
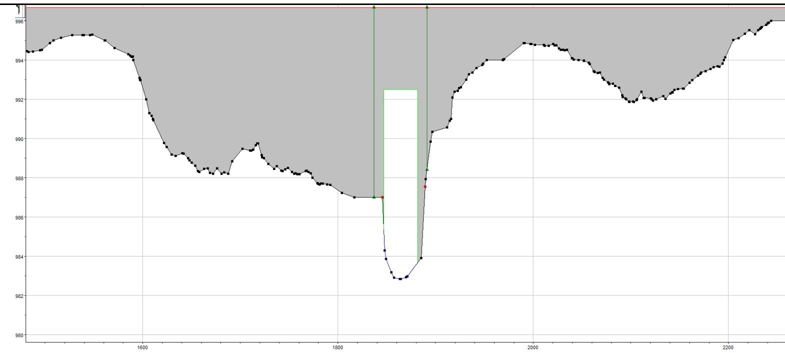
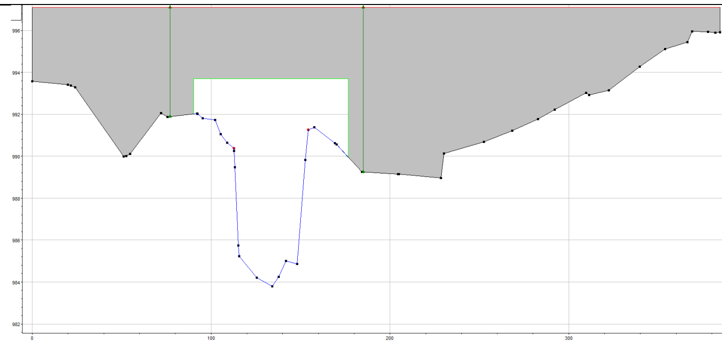
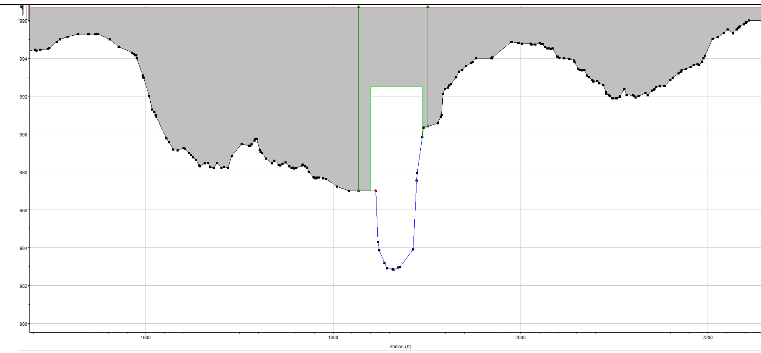
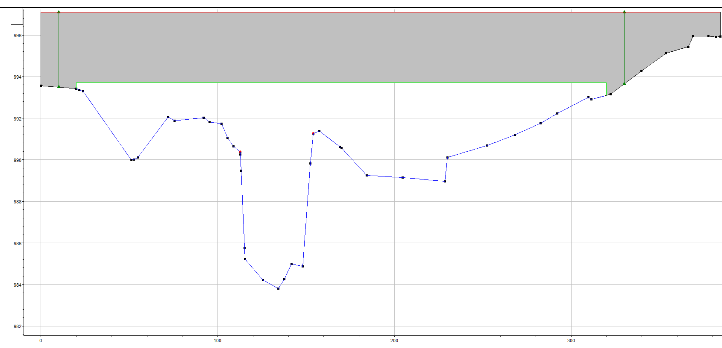
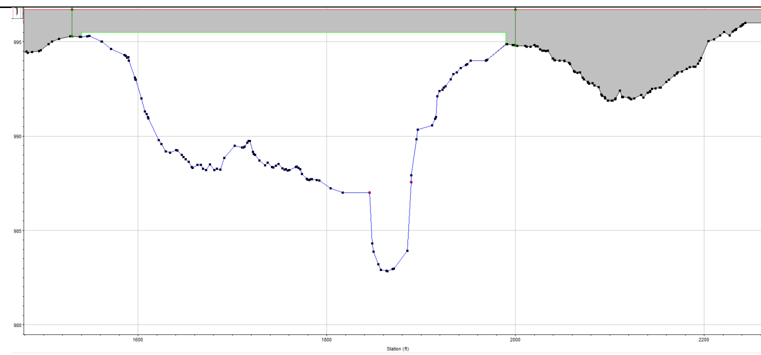
| S.R. 82 Bridge Upstream Cross Section | Railroad Bridge Upstream Cross Section |
| --- | --- |
| <br>As-Built Scenario | <br>As-Built Scenario |
| <br>20-ft Abutment Expansion Scenario | <br>20-ft Abutment Expansion Scenario |
| <br>Floodplain Abutment Expansion Scenario | <br>Floodplain Abutment Expansion Scenario |

Figure 21. Cross sections of bridges under each scenario (Bridges Removed scenario not shown)

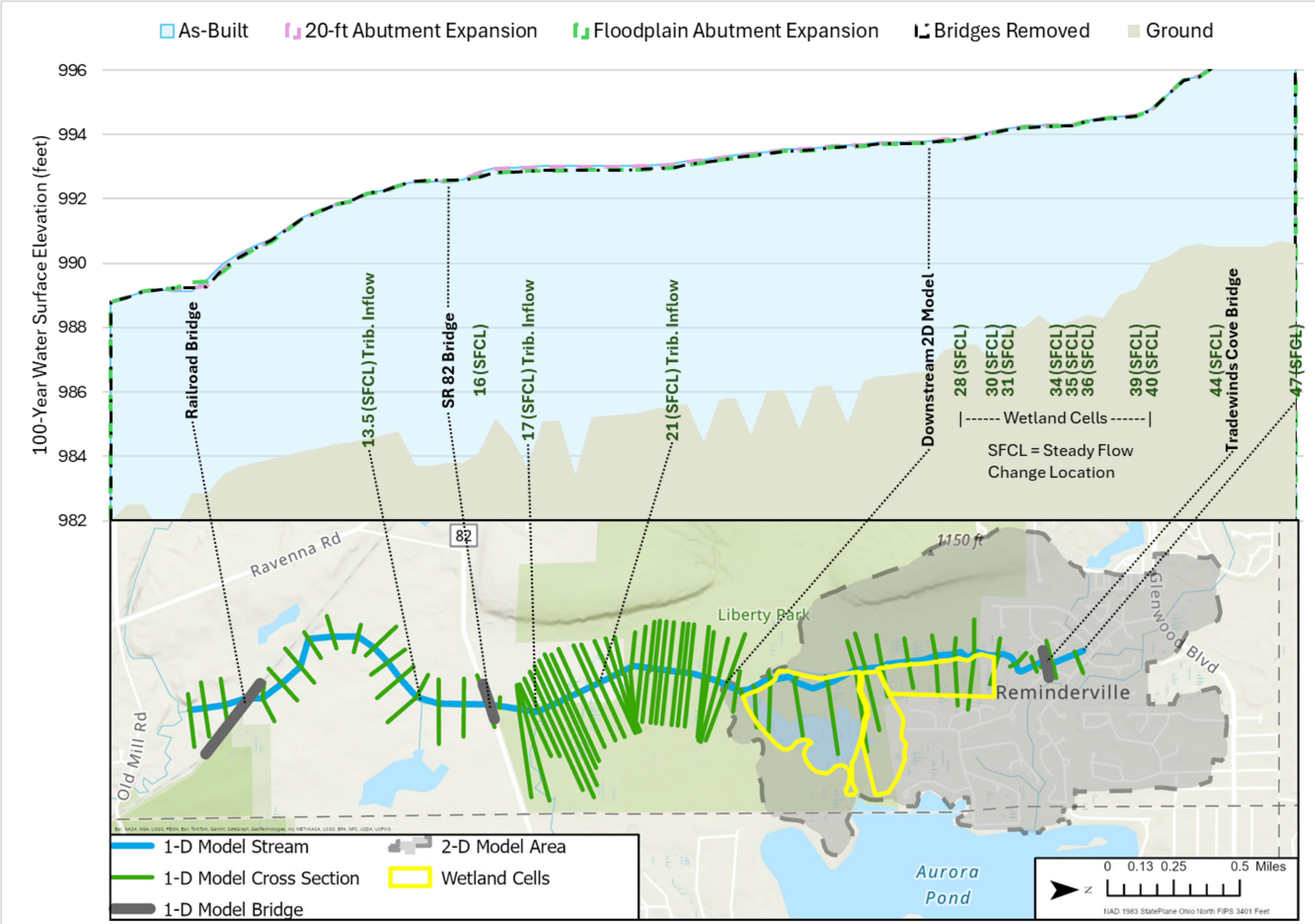



Figure 22. Water surface elevation profile along the extent of the 1-D model compared to the extent of the 2-D model domain.

# 5.0 MODEL ACCURACY AND CONFIDENCE

Tetra Tech was not able to perform a formal calibration of the model because there are no stream gages in the watershed. However, Tetra Tech performed a reasonableness check of the model results using other sources of information including the FEMA 100-year floodplain and other anecdotal descriptions of flooding extent and depths provided by homeowners in Aurora Shores. First, the inundated extent in response to the 100-year precipitation event under the *Design* and *Raised Agri-Drian Elevation* scenarios approximately matches that of the FEMA 100-year floodplain (**Figure 15** and **Figure 16**). Second, the anecdotal descriptions of inundated locations under past precipitation events provided by Aurora Shores homeowners were compared against model results. While these descriptions do not necessarily correspond to a specific scenario discussed herein, development of the model included consideration of these locations and results showed flooding in the expected areas. More specifically, these areas include the Nautilus Trail and Anchorage Cove intersection and the Pirates Trail crossing of the unnamed west tributary.

Other simplifications and assumptions made when setting up the model are presented below:

- The Aurora Shores WWTP located at 10200 Regatta Trail with NPDES permit No. 3PG00030*KD outfalls to Pond Brook to the northwest of the WWTP and not to the ditch that runs parallel to Windjammer Trail and Sea Ray Cove. The Ohio EPA lists the permit as expired as of June 30$^{th}$, 2021. However, permits are usually still applicable to the facilities after expiration and are still administratively continued. Due to its expiration and likely minimal impact on the flows in Pond Brook from its effluent, the Aurora Shore WWTP was not included in the model as a boundary condition.
- In the *Aurora Lake Stage Increase* scenario (*Scenario 2*), the HEC-1 model results for Aurora Lake were used to estimate the volume of flow entering Aurora Lake from all its tributaries during the 100-year event and the storage provided by the associated changes in stage. Two aspects of this approach required assumptions:
  - First, Teta Tech assumed that the ratio of Channel Brook peak flow to the total peak flow entering Aurora Lake from all tributaries was equal to the ratio of Channel Brook flow volume to the total flow volume from all tributaries. This assumption is not always true due to the varying degree of flashiness of each subbasin but is the only data available to draw the conclusions presented by *Aurora Lake Stage Increase* scenario. However, Tetra Tech believes that this is a reasonable assumption to make considering that the total flow volume coming into the lake during the 100-year event is on the order of 356 acre-feet while the storage available in the lake is on the order of 471 acre-feet. Therefore, the possible error introduced by this assumption is likely minimal compared to volume available for additional storage.
  - Second, the stage-storage relationship established for Aurora Lake by the HEC-1 model results used data for stages on the order of 1002 feet to 1009 feet, which is higher than the elevation in which Channel Brook overtops. This stage-storage relationship was extrapolated to lower stages on the order of 1001 feet to draw the conclusions provided by this scenario.
- In the *Design* and *Raised Agri-Drain Elevation* scenarios, infiltration rates of soil are the same throughout entire modeled area due to the assumption that PCSWMM subbasin 43 infiltration depths were applied to precipitation boundary condition. This is a reasonable assumption because this rain on grid boundary condition was only used in the scenario where backyards are flooded and that infiltration depth is representative for that area of interest.
- The design of Agri-Drains with adjustable stop logs are not readily modeled in HEC-RAS. In the *Design* and *Raised Agri-Drain Elevation* scenarios, the Agri-Drains were simulated using gates with adjustable invert elevations that

best represent the operation and function of the Agri-Drains. These gates are square in nature rather than circular pipes. This simplification however is not expected to have appreciable impacts on the scenario results as it was shown that water surface elevations downstream of the Agri-Drain and embankment are nearly identical regardless of scenario set up.

# 6.0 REVIEW OF HISTORIC RECORDS

Homeowners in Aurora Shores have reported that flooding has as occurring more frequently in recent years, particularly after the stream restoration of Pond Brook that occurred from 2004 to 2009. As previously discussed, *Scenario 1* through *Scenario 3* show that flooding is not predicted to be from the Channel Brook stream network. Additionally, it is assumed that the Clipper Cove aqueduct replacement will relieve the flooding observed at the Nautilus Trail and Anchorage Cove intersection. *Scenario 4 and Scenario 5* suggest that flooding in the backyards of homes on Windjammer Trail and Sea Ray Cove is due to direct precipitation rather than the overtopping of adjacent channels. The low elevations of these backyards and poorly draining underlying soils do not allow surface runoff or infiltration to drain the area which exacerbates the issue.

As such, historic records available for the study area were assessed to gain insight as to whether the observed flooding can be partially or fully attributed to changes in climatic influences, such as precipitation, or anthropogenic land use changes within the watershed. Such insights can be useful when developing potential solutions to the flooding issues at hand.

## 6.1 HISTORIC PRECIPITATION

To assess the changes in precipitation over time, hourly data was downloaded from the Global Historical Climatology Network (GHCN)[6]. Criteria for selecting the monitoring location included availability of hourly data, a long-term period of record, location relative to the study area, and the number of missing values. The Ravenna monitoring location (Station ID: USC00336949) was chosen because it has a long period of record (1948 – 2020), had few missing values (94% data coverage), and was close to the study area (approximately 13 miles away) compared to other monitoring locations with hourly data. This station is located at the Ravenna Water Treatment Plant. Other precipitation monitoring locations considered include the Cleveland Hopkins International airport, Akron-Canton airport and Burke Lakefront airport. However, these locations were farther away from the study area compared to the Ravenna location.

Hourly data was processed by taking a running 12-hour sum of precipitation depths. These 12-hour depths were compared against the depths of 12-hour design storms with recurrence frequencies ranging from the 1-year event to the 100-year event. The rainfall depths of these events are based on NOAA Atlas 14[7]. Precipitation events in which any 12-hour period exceeded the design storm were summed by decade to show how often design storms are exceeded over time (**Figure 23**).

[6] Global Historical Climatology Network hourly (GHCNh) – National Centers for Environmental Information (NCEI) – National Oceanic and Atmospheric Administration (NOAA). https://www.ncei.noaa.gov/products/global-historical-climatology-network-hourly
[7] Atlas 14. National Oceanic and Atmospheric Administration (NOAA) National Weather Service Hydrometeorological Design Center. Precipitation Frequency Data Server (PFDS) https://hdsc.nws.noaa.gov/pfds/

The 100-year event was exceeded twice since data collection began at the Ravenna location, once in the 1970s (September 14th, 1979) and once in the 1980s (March 8th, 1980; **Figure 23**). Other than these two large 100-year events, the 1960s, 1970s, and 1980s are relatively free from large precipitation events where only eleven storms across this 30-year period had 12-hour precipitation depths greater than the 1-year event. Conversely, 32 storms in the following three decades (1990s, 2000s, and 2010s) have had 12-hour precipitation depths in which the 1-year event was exceeded. Furthermore, 41% of these precipitation events (13 of the 32 storms) in these latter three decades exceeded the 2-year event, while 16% (5 of the 32 storms) exceeded the 5-year event.

The 2020 decade has fewer than ten years of data available and is therefore underrepresented in **Figure 23** compared to all prior decades. Nevertheless, two large storm events have already occurred in this decade, both of which are estimated to have been larger than any storm of the previous 60 years of recorded data. First, the storm that occurred on September 7th, 2020 (Labor Day) was measured as producing 3.7-inches of rain within a 12-hour period at the Ravenna location which equates to the 10-year design storm. However, the Ravenna location is approximately 13-miles away from the study area and cannot accurately be used to predict exact precipitation depths due to geographic heterogeneities in precipitation intensity and duration. Residential rain gauges in the Aurora Shores neighborhood anecdotally recorded 6.5-inches of rain during the 2020 Labor Day storm (OHM Advisors, 2021) which equates to the 500-year storm.

The second large storm of this decade occurred on July 17th, 2021. The Ravenna location was not recording data at this time and Tetra Tech has received no known precipitation depths from homeowner rain gauges for this storm. However, photographs of inundated areas are available in the Aurora Shores Homeowner's Association Annual Report[8]. These photographs indicate the approximate depth and extent of flooding observed during this 2021 storm match those observed during the Labor Day storm of 2020. While the exact precipitation depths are not available for these storms in 2020 or 2021, both events are estimated to have been larger than the 100-year event based on residential rain gauges, observed flooding, and extrapolation of data from nearby official precipitation monitoring locations.

[8] Aurora Shores Homeowner's Association Annual Report ("ASHA Annual Report.pdf") provided by Summit County Engineer to Tetra Tech.

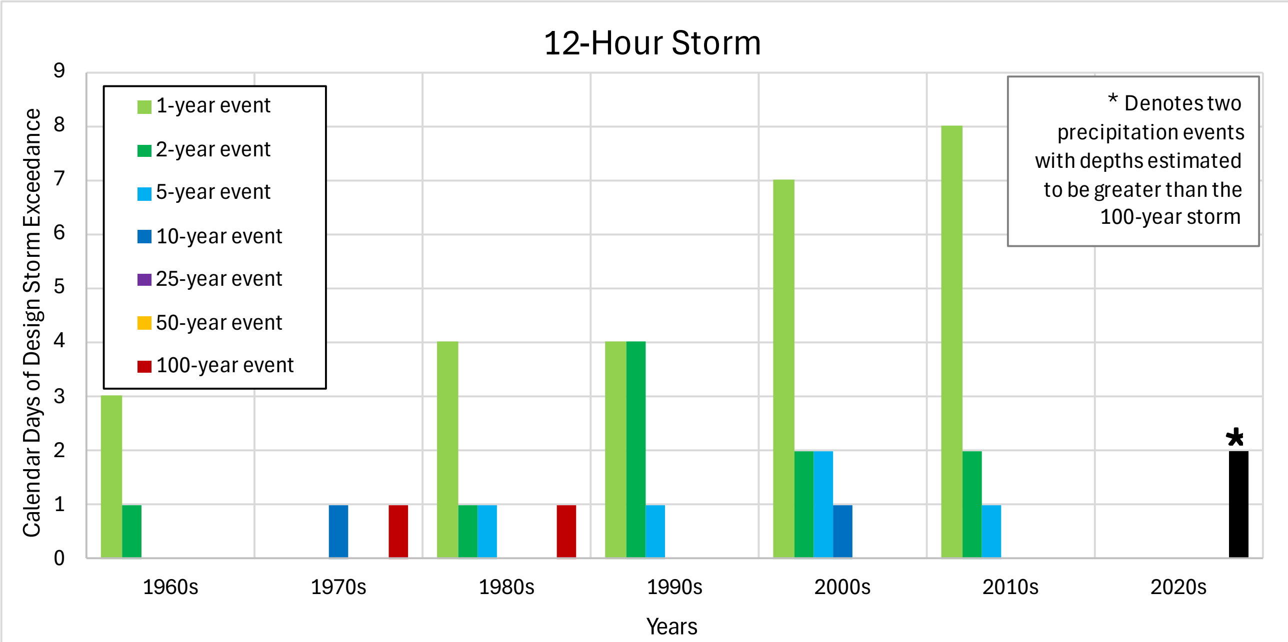



Figure 23. Calendar days in which design storm frequencies were exceed by the 12-hour duration precipitation event

## 6.2 HISTORIC MAPPING

**Figure 24** and **Figure 25** show multiple spatial datasets within the Aurora Shores neighborhood related to prior land use and floodplain mapping. Present day road and building footprints are provided for comparison. **Figure 24** is a United States Geological Survey (USGS) topographic map from 1963 which is prior to the development of the area. The delineation of designated swamp area overlaps large portions of where roads and buildings exist in the present day. These swamps were drained prior to and during the construction of the Aurora Shores neighborhood via a system of ditches. The ditches were designed at a slightly lower elevation than the surrounding swamp to allow for proper drainage. However, development of Aurora Shores occurred on top of the prior swamp and remains at a relatively low elevation. Additionally, the footprint of Aurora Lake has approximately doubled in response to a four-foot increase in water surface elevation (**Figure 24**).

**Figure 25** shows the FEMA 100-year flood zones from 1990 and 2016 (FEMA, 1990; FEMA, 2016). A Physical Map Revision (PMR) request initiated the Summit County Flood Insurance Study (FIS) in 2016 and resulted in a full revision of the FIS which supersedes the previous study (FEMA, 2016). The 2016 FIS leverages new information collected through multiple studies throughout the county. As expected, the 100-year flood zone roughly matches the area and extent of swamp in the 1963 USGS topographic map.

The FEMA 2016 100-year flood zone has a smaller extent compared to the 1990 flood zone, particularly in the wetland cells south of Aurora Shores (**Figure 25**). This difference may be due to modeling approaches used by FEMA and the data available at the time of each study. However, a more likely explanation of this difference is the restoration of Pond Brook that happened between 2004 and 2009. The FEMA 2016 100-year flood zone therefore accounts for this restoration and any impacts it may have on the water surface elevations and inundated extent during the 100-year event. These results show that the restoration of Pond Brook has not increased, but rather decreased the inundated extent during the 100-year event. This interpretation is substantiated by the inundated extents shown in this memorandum (**Figure 15** and **Figure 16**) which were the results of the HEC-RAS 2-D model built by Tetra Tech.

In addition to the historic extents of USGS designated swamp area and FEMA 100-year flood zones, the conclusion that this location in Aurora Shores has historically flooded and will continue to flood is substantiated by the natural underlying soils shown in **Figure 26**. SSURGO map unit “Wt” is the Willette Muck profile of the Willette Series of soils and occupies the flooded backyards on Windjammer Trail and Sea Ray Cove. The soil is described as consisting of “very poorly drained organic soils that formed in muck deposits 16 to 42 inches thick”, having “slow permeability” with a “high water table for long periods unless they are drained” (Ritchie and Steiger, 1974). These soil characteristics are indicative of flooding and must be considered during development of recommendations to mitigate such flooding.

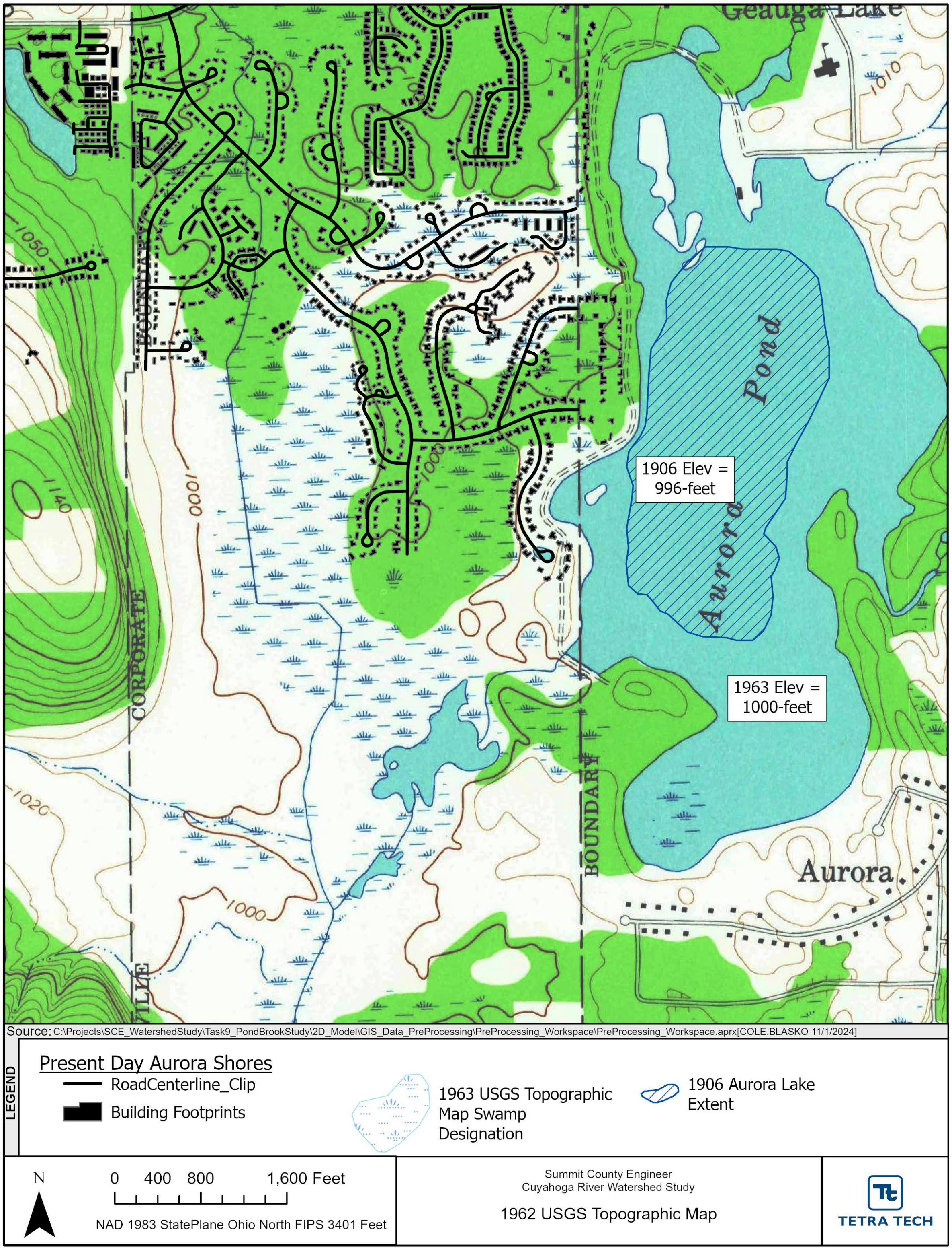



Figure 24. Present day building footprints and roads within the Aurora Shores neighborhood overlapped with a 1963 USGS topographic map

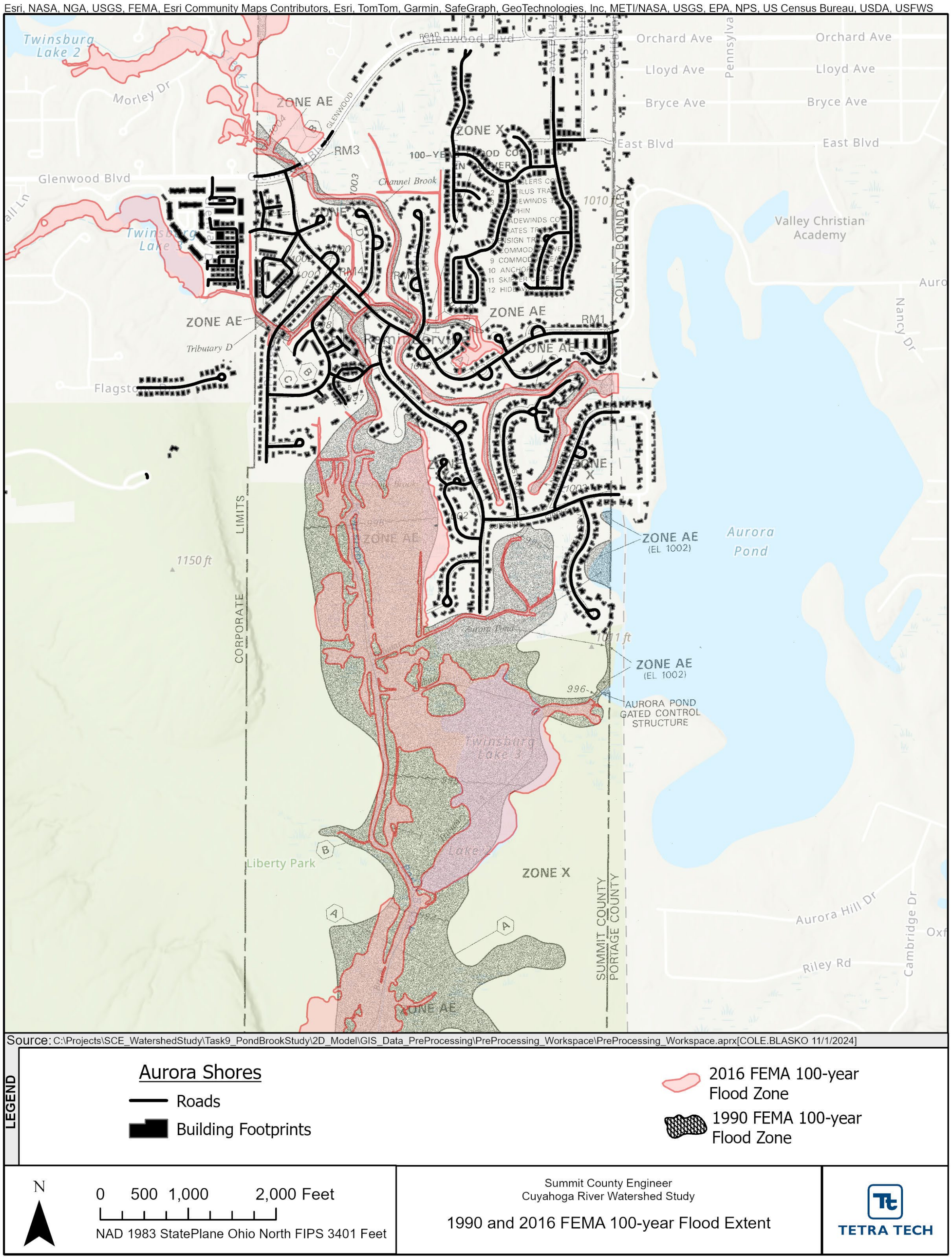



Figure 25. Present day building footprints and roads within the Aurora Shores neighborhood overlapped FEMA 100-year flood zones from 1990 and 2016

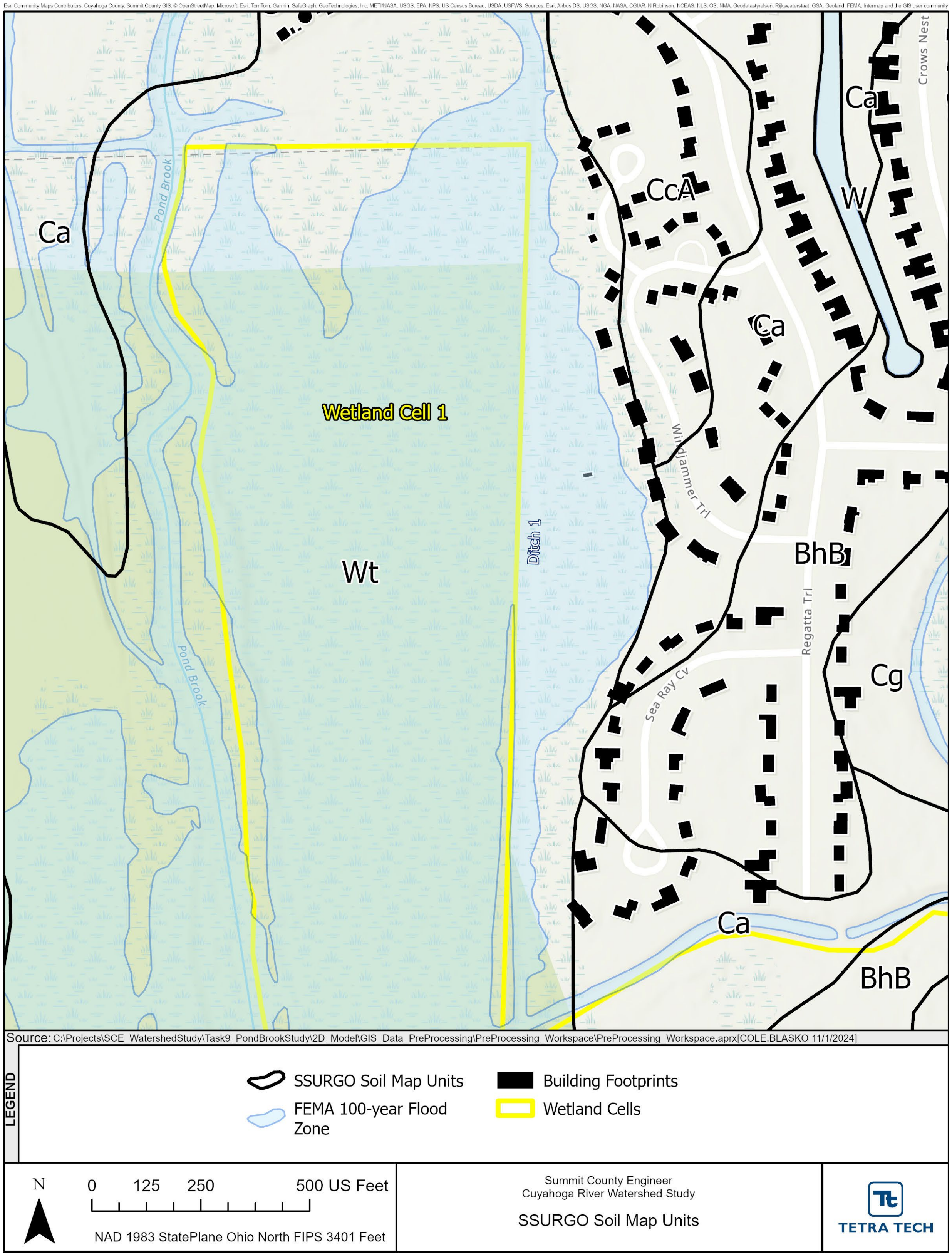



Figure 26. Dominant SSURGO soil map units in the location of localized flooding of backyards on Windjammer Trail and Sea Ray Cove

# 7.0 CONCLUSIONS AND RECOMMENDATIONS

Flooding has been documented as occurring in two primary locations within the Aurora Shores neighborhood: (1) upstream of the Clipper Cove aqueduct that is east of the Anchorage Cove – Nautilus Trail intersection and, (2) the west-facing backyards of homes on Windjammer Trail and Sea Ray Cove, east of Wetland Cell 1 (**Figure 3**). The Clipper Cove culvert replacement (OHM Advisors, 2021) completed in spring of 2024 is presumed to provide the required increase in conveyance to relieve the flooding at the Anchorage Cove – Nautilus Trail intersection. A holistic approach to identifying the source and magnitude of flooding under various scenarios was implemented by Tetra Tech using a 2-D HEC-RAS model. Key takeaways from each scenario are outlined below:

- <u>*Channel Brook 100-Year Event*</u>
  - Model Parameters – Discharge from the Channel Brook watershed was simulated at the 100-year event flow volumes while keeping all other parameters at baseflow conditions.
  - Conclusion - The 100-year discharge from the headwaters of the Channel Brook watershed does not overtop the Channel Brook levees.
  - Recommendation – Maintain the dimensions of the Glenwood Boulevard crossing as this constriction attenuates the flood wave downstream.
- <u>*Aurora Lake Stage Increase*</u>
  - Model Parameters - The Aurora Lake spillway gate is operable between a range of 996.5-feet and 1002.1-feet with a normal operating elevation of 1001.0-feet which sets the surface area of the lake at 377-acres. The stage of Aurora Lake was linearly increased to achieve overtopping of boating canal levees.
  - Conclusion – The Channel Brook boating canal first overtops at an elevation of 1001.5 feet near its downstream end 400 feet away from its confluence with Aurora Lake.
  - Recommendation – Control the stage in Aurora Lake such that storage is available prior to precipitation events to prevent Aurora Lake stage from reaching 1001.5 feet. Lowering the stage of Aurora Lake by 1-foot provides 471 acre-feet of storage which is greater than the 100-year flow volumes coming into the lake (352.2 acre-feet).
- <u>*Aurora Lake Spillway Backwater*</u>
  - Model Parameters – Informed by HEC-1 modeling by ODNR, 779 cfs (349,639 gallons per minute) were released from the Aurora Lake spillway. These flows are approximately 10-times greater than Pond Brook baseflow and equal to 12% of the probable maximum flood in the HEC-1 model. This resulted in a released volume of 822 acre-feet and lowered the stage of the lake by 1.7-feet which is 2.3 times greater than the volume needed to store the 100-year event volumes.
  - Conclusion – Reservoir releases of this magnitude represent a worst-case scenario where Aurora Lake is close to overtopping its emergency spillway and is not expected under normal dam operation. These flows result in a backwater effect approximately equal to the inundated extent of the FEMA 100-year floodplain.
  - Recommendation – Control releases from the Aurora Lake spillway such that discharge of this magnitude is not reached under normal operations.

- <u>*Design Agri-Drain Elevation = 992.5 feet*</u> and <u>*Raised Agri-Drain Elevation = 994.5 feet*</u>
    - Model Parameters – Agri-Drains were set at two different elevations to evaluate their potential to reduce localized flooding. The 100-year precipitation event was applied to the model and all other unsteady flow hydrograph boundary conditions were set to steady baseflow conditions.
    - Conclusion – Agri-Drains at design elevations provide discharge at a controlled rate to pass through the embankment during low flows but water surface elevations during the 100-year precipitation event flood backyards. Cumulative flows downstream of the embankment via Ditch 1 are reduced when the Agri-Drain invert elevation is raised but water surface elevations are not improved because the embankment is overtopped regardless. Equilibrated water surface elevations across the wetland, ditch, and backyards indicate removal of the Agri-Drain and the embankment in their entirety will not resolve inundation of the 100-year event.
    - Recommendation –As part of the Cuyahoga River Watershed Study, Tetra Tech has provided three concept plans that outline the proposed recommended alternatives to control the flooding in the backyards of homes on Windjammer Trail and Sea Ray Cove. These concept plans are provided as an appendix to the Cuyahoga River Watershed Study and are summarized below.
        1. Alternative 1 proposes maintaining the existing conditions and allow the backyards to flood within the FEMA 100-year floodplain.
        2. Alternative 2 proposes the installation of a system of field drains in an east-west direction to convey water to the adjacent ditch. This alternative is not expected to solve the flooding issue but rather, will decrease the time in which the yards are flooded. Feasibility depends on a detailed elevation survey to ensure adequate grade exists between the yards and the ditch. As such, the exact design of the drains has not yet been established.
        3. Alternative 3 proposes removal of water from the area using the following three components: (1) Install field drains in a north-south direction to convey water out of the backyards; (2) Construct two retention basins with impermeable lined bottoms; and (3) Install a pump station to remove water from the retention basins into the adjacent ditch system. This alternative will require the acquisition of three properties where elevations are most suitable (Parcel IDs 6600902, 6600903 and 6600904). Feasibility and design specifications for the proposed alternative have not yet been established.
- *Increased Hydraulic Capacity Downstream*
    - Model Parameters – The S.R. 82 and Railroad bridges were adjusted in the Stantec (2017) 1-D model under four scenarios: (1) As-Built; (2) 20-ft Abutment Expansion; (3) Floodplain Abutment Expansion; and (4) Bridges Removed.
    - Conclusion – Modifications to the hydraulic capacity of the bridges via changes in the abutment opening widths resulted in negligible changes in the water surface elevations upstream, even when bridges were removed in their entirety.
    - Recommendation – Increasing the hydraulic capacity of the S.R. 82 or Railroad bridges is not necessary.

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