Appendix B. Hydraulic Model Development and Analyses

Appendix B provides engineering calculations associated with the existing and proposed conditions hydraulic (HEC-RAS) models.

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Part I – Completed by Originator

1. Project Name:	Flood Risk and Stormwater Management Planning for the Squaw Run Watershed			
2. Project Type:	☐ Reservoir ☐ Navigation ☐ LPP ☒ Other: Planning FPMS			
3. Watershed Basin:	☒ Allegheny ☐ Monongahela ☐ Ohio		State:	
4. Has design or analysis software been used for this calculation? (*if yes complete below*) ☐ No ☒ Yes				
Software Name:	ArcGIS 10.4.1; HEC-GeoRAS 10.1; HEC-RAS 5.0.7			
Version No.:	See above	ACE-IT Tag No. :	C5307	
5. Has a thorough self-check of this calculation been completed and accurate? ☐ No ☒ Yes				

6. If this is a revision, explain reason for revision: ______________________________

Part II – Completed by Verifier(s):

1. Calculation inputs were correctly selected and incorporated? ______ ☒
2. Significant assumptions are adequately identified, described, justified, reasonable? ______ ☒
3. Numerical calculations are correct and documented?______ ☒
4. Calculation outputs were reasonable compared to inputs______ ☒
5. All pages are legible, references identified and appropriate; document identifier and revision assigned; and acceptable with respect to grammar, spelling and punctuation?______ ☒
6. Each calculation input, information and equations from external sources referenced? ______ ☒

REVIEW COMMENTS:

None

Part III – Approval for Calculations

Originator(s) Print Name	Signature	Date
Howard Kellick, P.E.	KELLICK.HOWARD.BRUCE.1565106755 Digitally signed by KELLICK.HOWARD.BRUCE.1565106755 Date: 2020.10.09 09:59:43 -04'00'	10/09/2020
Verifier(s)	**Signature**	**Date**
Hillary Shipps	SHIPPS.HILLARY.P.1536320373 Digitally signed by SHIPPS.HILLARY.P.1536320373 Date: 2020.10.09 10:12:32 -04'00'	10/09/2020
Water Resources Section Chief	**Signature**	**Date**
Kyle Kaminski, P.E.	KAMINSKI.KYLE.MARTIN.1591285323 Digitally signed by KAMINSKI.KYLE.MARTIN.1591285323 Date: 2020.10.09 11:32:57 -04'00'	10/09/2020

Approval of Section Chief signifies that the document and all required reviews are complete, and the document can be internally released to other sections.

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APPENDICES

1.0 STATEMENT OF PURPOSE

Squaw Run drains an area of approximately 8.6 square miles of land primarily within Fox Chapel Borough and O'Hara Township (Figure 1).

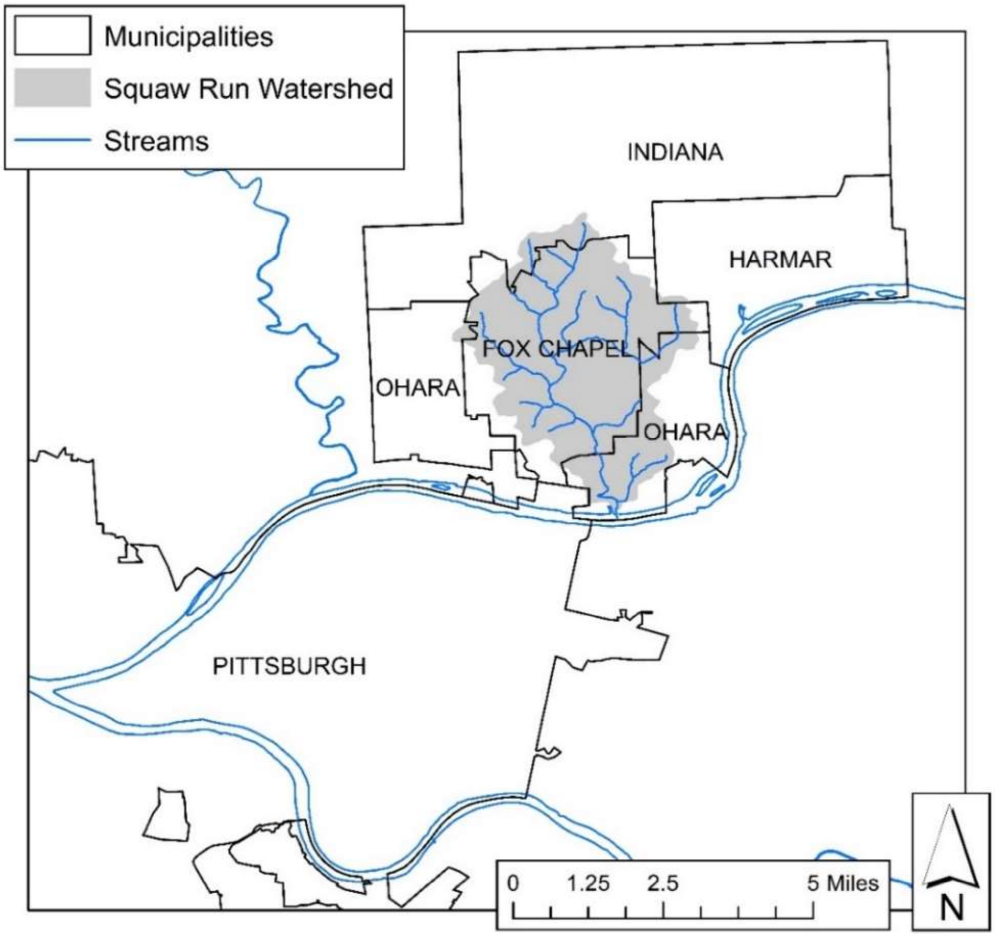

Figure 1: Location of Squaw Run Watershed Relative to the Municipalities It Intersects.

Squaw Run has a history of flooding, including events associated with hurricanes (e.g., Hurricane Ivan, 2004) as well as recent, localized precipitation events (i.e., events of July 2018 and July 2019). Flood waters have reached the first floor of many structures within the floodplain, and certain structures experience flooding.

Flooding events, along with the potential for increased localized flooding, led O'Hara Township and Fox Chapel Borough to jointly request flood risk management assistance from USACE, Pittsburgh District. The goal of this study is to provide the technical tools needed to make informed flood risk management decisions. The specific objectives of this study are to:

1. Create hydrologic and hydraulics (H&H) models to facilitate flood risk and stormwater management planning within the Squaw Run watershed;
2. Characterize nonstructural and structural options to reduce flood risk along Squaw Run;

3. Quantify increased storage at the subbasin level required to alleviate downstream flooding;
4. Meet with local stakeholders to discuss findings and opportunities.

2.0 DESCRIPTION OF METHODOLOGY USED

HEC-RAS MODEL DEVELOPMENT

A hydraulic model was developed using Hydrologic Engineering Center River Analysis System (HEC-RAS), version 5.0.7, as part of this FPMS study. The model extends approximately 9,000 feet and includes Squaw Run from its confluence with the Allegheny River to just downstream of Fox Chapel Road.

The Federal Emergency Management Agency (FEMA) prepared a countywide Flood Insurance Study (FIS) to include all jurisdictions within Allegheny County in October 1995. The FIS report was revised in September 2014. The portion of Squaw Run within Fox Chapel was evaluated in the FIS report.

Existing HEC-2 data for the completed H&H analyses were reviewed prior to beginning work on the new HEC-RAS model for the FPMS study. Model inputs from HEC-2 were reviewed and compared to the publicly available LiDAR data available for the studied portions of Squaw Run and used to help develop the HEC-RAS model. Additionally, a high water mark from a nearby residence within the floodplain was used to help calibrate the model to historic flooding data.

The extents of the modeled reach of Squaw Run are shown in Figure 2.

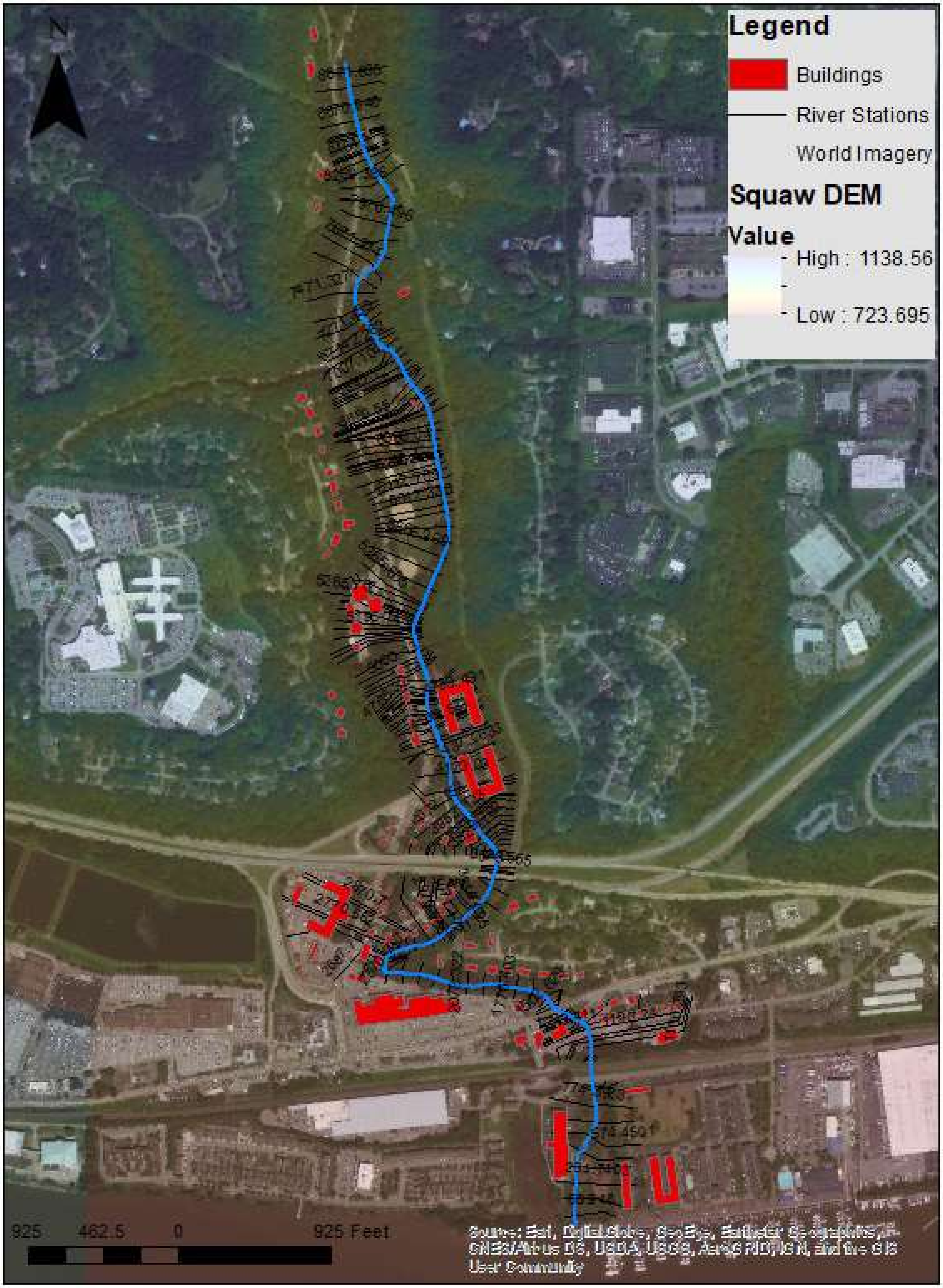

Figure 2: Extents of the HEC-RAS model.

Geometric Data

Existing Conditions Geometric Data: To develop the existing conditions hydraulic models, terrain and river shapefiles were obtained from the Geospatial Section within the Pittsburgh District. Terrain files were built using LIDAR data, dated 2006, from the Pennsylvania Spatial

Data Access (PASDA) website (http://www.pasda.psu.edu/). The LiDAR data used in the model has one-meter resolution. Using ArcGIS 10.4.1 and the HEC-GeoRAS extension, the shapefiles were modified to match the terrain and base maps used in the model. The terrain file was clipped to the modeling extent of Squaw Run, as shown in Figure 2. From this terrain shapefile, HEC-RAS layers for the stream centerline, cross section (XS) cutlines, streambanks, flow paths, bridges/culverts, land use and obstructions were developed per guidance from the HEC-GeoRAS User's Manual (USACE, 2011). Once all the layers were developed, they were imported to HEC-RAS version 5.0.7, using the methodology described in the HEC-GeoRas User's Manual (USACE, 2011). Once in HEC-RAS, model inputs, such as contraction/expansion coefficients, bridge dimensions, and cross section geometries were adjusted based on best engineering judgment. Input data used in the development of the models are summarized below.

Proposed Conditions Geometric Data: The proposed conditions geometric data were developed by modifying the existing conditions geometric data. Specific modifications are discussed in Section 4.0, Calculation Input.

Cross Section Geometry: Cross sections were located at intervals along the stream in order to characterize the flow carrying capability of the stream and its adjacent floodplain. Each cross section extended across the entire floodplain and was placed perpendicular to the direction of flow in accordance with the available topography data. Cross sections were defined from the terrain model looking in the downstream direction from the left bank to the right bank. Cross section spacing was typically set to a maximum distance of 100 feet, in agreement with Samuel's equation (HEC-RAS Reference Manual, 2016), based on the size and slope of the reaches in the study. One cross section, XS 1062.466 was placed approximately 169 feet upstream of both a railroad crossing and Zaenger Drive, neither of which we have sufficient information to model. Additional cross sections were added to capture changes in stream geometry, land cover, and the extents of buildings within the floodplain. Since there is no available bathymetric data for Squaw Run within O'Hara Township, the topography was conservatively left as is, instead of assuming a channel bottom. Assumed wet sections will be discussed further in Section 3.0, Assumptions. Location of all cross sections can be shown on Figure 2.

Manning's n: Selection of an appropriate Manning's n is very significant to the accuracy of the computed water surface elevations. The value of Manning's n is highly variable and depends on factors such as surface roughness, vegetation, channel alignment, channel size, and channel shape. The FIS contains Manning's n values used for the detailed H&H computations. According to the FIS, the Manning's n values were chosen based on engineering judgment and field inspection of the floodplain areas. The Manning's n values included in the FIS and the Manning's n values included in Table 3.1 of the HEC-RAS 5.0 Reference Manual (USACE, 2016) were used as a guide in selecting the appropriate values for the existing conditions model. Aerial imagery was used to determine land use along the channel and overbank for the modeled reach. Manning's n values were then assigned to each land use based on engineering judgment, and incorporated into the model. Manning's n values used for the channel and overbanks were generally kept within the ranges stated in the FIS, though some Manning's n values were adjusted during calibration of the model. The Manning's n values used in the existing conditions are summarized in Table 1 in Section 4.0, Calculation Input.

Contraction/Expansion Coefficients: Losses due to contraction and expansion of flow between cross sections are determined during the standard step profile calculations in the model. Contraction and expansion coefficients are used to compute energy losses associated with changes in the shape of the reach cross sections. Typical contraction/expansion coefficients of 0.1 for contraction and 0.3 for expansion were used for the stream. For cross sections associated with bridges within the models, the contraction and expansion coefficients were increased to 0.3 and 0.5, respectively.

Ineffective Flow Areas: Ineffective flow areas are portions of the cross section that contain the water that is not actively being conveyed (zero flow but ponded water). Ineffective flow areas were defined at bridge structures in order to describe the active flow areas within the model. Ineffective flow area stations were set outside the edges of the bridge opening to allow for the contraction and expansion of flow that occurs in the vicinity of the bridge. At the bounding cross sections of the bridge, stations were placed at a 1:1 expansion and contraction rate. For example, if the upstream cross section is 10 feet from the bridge, the ineffective flow area stations would be 10 feet outside of the bridge opening on both sides. The upstream elevation for the ineffective flow areas corresponded to the elevation where flow passes over the bridge, which in this case is the elevation of the roadway. For the downstream ineffective flow area, elevations were set below the minimum top of roadway. Ineffective flow area elevations were lowered to low ground elevation outside the immediate bridge area where that low ground was below the bridge elevation.

Outside of the cross sections around each bridge, additional ineffective flow stations were placed in areas where the flow would likely not be effective. The Squaw Run watershed includes numerous areas where the ground topography quickly narrows or expands, leading to numerous areas where the water would pool or swirl, but not actively flow downstream. Ineffective flow areas allow water to pond at these locations, but do not include them in the conveyance calculations.

Obstructions: Obstructions decrease the flow area and add wetted perimeter where the water surface comes in contact with the obstruction. An obstruction does not prevent water from going outside of the obstruction. Portions of Squaw Run flow through populated areas with a number of structures within the floodplains of the reaches. Obstructions were modeled to determine the effects the structures have on the water surface elevations within each reach. Obstructions were all set to 15 feet above the average ground elevation, which is representative of a typical one-story building.

Bridges/Culverts: Six bridge/culvert crossings exist within the study area, all of which were modeled in HEC-RAS. Because HEC-GeoRAS only allows the user to assign the locations and widths of bridges and/or culverts, additional properties were assigned in HEC-RAS. Several variables are necessary for inputting bridge/culvert information into the model. Inputs include bridge deck/roadway information, bridge abutments, bridge piers, and the bridge modeling approach. For this study, detailed bridge information was not readily available for all of the bridges. Bridge surveys were not included in the scope of work. PennDOT and Allegheny County provided plans for four bridges: Old Freeport Road, Freeport Road, and both PA 28 bridges. Bridge and culvert information was estimated for the culvert that passes under both Zaenger Road and the railroad just north of Zaenger Road, as well as forRockwood Drive based on measurements from Google Earth and Google Street View. The crossings included in this model are discussed further in Section 4.0, Calculation Input.

STEADY FLOW DATA

A steady mixed flow analysis was performed on the existing conditions models for the FPMS study. A steady flow analysis consists of the number of profiles to be computed, the flow data, the flow regime, and the river system boundary conditions. At least one flow must be entered for every reach within the system. Flow values must be entered for all profiles.

Flood Profiles: The analysis includes seven (7) flood profiles: the 50-percent Annual Chance Exceedance (ACE) (2-year storm), the 10-percent ACE (10-year storm), the 4-percent ACE (25-year storm), the 2-percent ACE (50-year storm), the 1-percent ACE (100-year storm) the 0.5-percent ACE (200-year storm), and the 0.2-percent ACE (500-year storm) storm events. These are the flood profiles calculated as part of the HEC-HMS modeling effort.

Flow Rates: Hydrologic analyses were performed as part of this FPMS calculation. The flows calculated by the HEC-HMS modeling effort are entered into the HEC-RAS existing conditions models.

Boundary Conditions: Upstream and downstream boundary conditions are necessary to establish the starting water surface elevation at the ends of the river system. Since the existing conditions HEC-RAS model runs a mixed flow regime (both subcritical and supercritical flows), boundary conditions must be entered at all ends of the river system. Boundary conditions are summarized in Section 4.0, Calculation Input.

3.0 ASSUMPTIONS AND JUSTIFICATION

1. The model is run as a steady mixed flow regime model. The mixed flow regime will provide the most reasonable approximation of the water surface elevations along Squaw Run for a given flow frequency flood.

2. Flow was determined within "Squaw Run Existing Condition HEC-HMS Model Development" (USACE, 2020) at the upstream end of the modeling reach, for an incoming 0.5-square-mile watershed, labeled RIDCTrib, and for the downstream end of the modeling reach. The flow at the downstream end was conservatively used.

3. The existing conditions flow and proposed conditions flow rate for Stormwater Ponds 1 and 2 are based on curve numbers assuming antecedent moisture content II. This assumption is appropriate for the typical NOAA Atlas 14 storm. This is in contract to the

decision to use antecedent moisture content III for Hurricane Ivan. The decision to use antecedent moisture content III for Hurricane Ivan is due to the overall rainfall for Ivan having a much higher volume of runoff than a comparative NOAA Atlas 14 storm of that peak rainfall would have, and thus a much higher impact to water elevations.

4. It is assumed that flooding along Squaw Run is not coincident with flooding along the Allegheny River. Squaw Run's drainage area is roughly 8.2 square miles, as per "Squaw Run Existing Condition HEC-HMS Model Development" (USACE, 2020). The Allegheny River's drainage area is roughly 11,700 square miles at Allegheny River Lock and Dam 2, the nearest navigational lock and dam (USGS, 2020). Any storm impacting Squaw Run will make its way downstream significantly before it makes i's way along the Allegheny River.

5. The LIDAR terrain files that were imported into HEC-RAS are assumed to be representative of the existing terrain. For most of the stream, the only data available is LIDAR data, which does not include most bathymetry as the light does not typically penetrate much below the water surface.

   Furthermore, Squaw Run is a very small watershed and the channel itself does not flow deeply during normal flows. It is assumed that the loss of channel bathymetry due to LIDAR is minimal and that the channel defined through the LIDAR topography is reasonable.

6. Aerial imagery and engineering judgment were utilized for estimating overbank land cover type. More detailed information was not available for model development.

7. Structures located on cross sections were modeled as obstructions. Obstructions were assumed to be 15 feet high, which is the typical height of a single-story residential building.

8. Contraction/expansion coefficients of 0.3 and 0.5, respectively, were assumed to adequately represent the abrupt changes in the cross sections surrounding bridges and culverts.

9. In July 2019, The Township Engineer for O'Hara Township provided USACE with a high water mark of 2.02 feet above a slab elevation of 752.996 feet at 222 North Margery Drive. The high water event occurred during a rain storm occurring sometimes in the 1980s or 1990s. The Township Engineer further noted that while they weren't sure specifically when the high water event occurred, flooding due to Hurricane Ivan essentially went up to the same high water mark. For lack of more specific data, the high water mark is assumed to have occurred during Hurricane Ivan.

10. This calculations include some assumptions required in order to model the proposed structural alternatives. This calculation provides a broad analysis of proposed structural alternatives, without providing a full technical design. The included assumptions were

necessary in order to do an analysis of each proposed alternative and represent engineering judgment, not site-specific or design values.

a. For the proposed channelization structural alternatives, it was assumed that the proposed channel would be riprapped. A Manning's n value of 0.035 was chosen for the channelized sections of each model.
b. For the proposed Floodwall 1, a floodwall was assumed to be placed just east of Fox Chapel Road, at an elevation high enough to protect the nearby buildings from the 500-year flood.
c. For the proposed Floodwall 2, a floodwall was assumed to be placed West of Fox Chapel Road, between Fox Chapel Road and the nearby buildings. The floodwall was assumed to be high enough to protect the nearby buildings from the 500-year flood.

4.0 CALCULATION INPUT

Existing Conditions Input

Manning's n: Manning's n values for the models were determined based on engineering judgment and land uses, as seen in the "World Imagery" aerial layer accessible through ArcGIS.

Manning's n values for the channel of the Squaw Run model were set to 0.049 which characterizes the channel as clean and winding, with ineffective slopes, and stones and vegetation with the channel. This value was based on view of the channel via Google Street View. The Manning's n values were further calibrated based on a high water mark at 222 North Margery Drive. The calibrated Manning's n value for the channel, 0.049, is within 0.02 and 0.059, the range specified for the studied portions of Squaw Run in Table 7 of the effective Allegheny County Flood Insurance Study. Overbank Manning's n values are summarized in Table 1 below and are chosen as average values listed in the HEC-RAS Technical Reference Manual.

Table 1: Manning's n Value Used in HEC-RAS Model.

Land Use	Manning's n value
Brush	0.05
Forest	0.1
Grass / Lawn	0.03
Impervious – Roof, Street	0.012

Bridges/Culverts: Available bridge crossing information gathered from PennDOT and Allegheny County was imported into HEC-RAS. PennDOT and Allegheny County were able to provide information for four crossings: both lanes of Pennsylvania Route 28, Freeport Road, and Old Freeport Road. Crossing information for the culvert that passes under both Zaenger Drive and the railroad north of Zaenger Drive, and for Rockwood Drive were estimated based on measurements from Google Earth and Google Street View. Figure 3 shows the locations of all bridges and culverts included in the model.

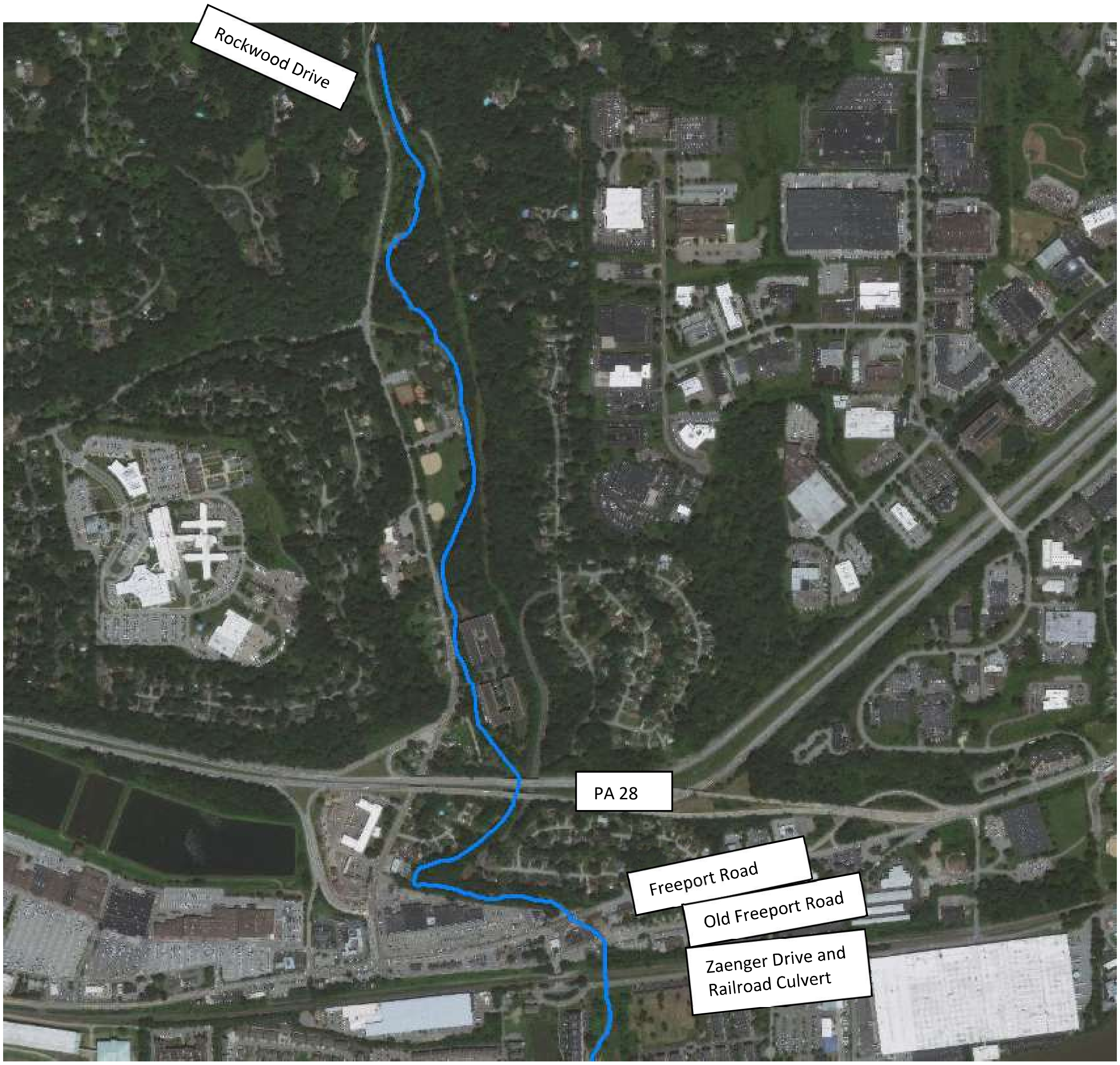

Figure 3: Location of Bridges and Culverts within the Model Limits.

Deck thickness was adjusted to account for the presence of parapets, guide rails, or railings along the bridge. Ineffective flow areas within the bounding cross sections of all bridges were offset from the edges of the opening width using a 1:1 ratio upstream of the crossing. For Freeport Road, the upstream ineffective flow elevations were set at the high chord elevation of the deck/roadway, and the downstream ineffective flow areas were set between the high chord and the edge of the opening. For Old Freeport Road, the deck slopes downward on other side of the bridge. The upstream ineffective flow elevations were set along the high chord, while the downstream ineffective flow locations were set between the high chord and the edge of the opening. For Rockwood Drive, the upstream ineffective flow elevations were set at the high chord elevation of the deck/roadway, and the downstream ineffective flow areas were set between the high chord and the edge of the opening. No ineffective flow stations were set for the Zaenger Road and Railroad Culvert as the culvert is sufficiently big to handle the flow without ineffective areas. Also, no ineffective flow stations were set for Pennsylvania Route 28 as the bridge was significantly higher than even the 500-year water surface elevations. A bridge modeling approach was selected for both Low Flow and High Flow Methods. For low flow conditions (water surface is below the highest point on the low chord of the bridge opening), the energy method was used. For high flow methods (flows that come in contact with the maximum high chord or overtop the bridge), the energy only method was used.

Obstructions: Obstructions were added to the model in areas where structures impacted the water surface elevations. Obstruction footprints were determined based on the "AlleghenyCounty_Footprints2013" shapefile made available by Allegheny County on their Allegheny County GIS data portal. Obstruction heights were set at 15 feet tall, which is the standard height of a single-story residential building. To determine obstruction height, the ground elevations along the obstruction footprint were averaged and extended 15 feet in elevation.

Flows: No historical stream flow data is available for any of the reaches within the study area, and all reaches are ungaged. Flow rates used in this model were developed in a previous calculation, "Squaw Run Existing Condition HEC-HMS Model Development," and are summarized in Table 2.

Table 2: Summary of Peak Discharges for Existing Conditions Models.

2-year (cfs)	**10-year (cfs)**	**25-year (cfs)**	**50-year (cfs)**	**100-year (cfs)**	**200-year (cfs)**	**500-year (cfs)**
496.9	1367.3	2204.9	2959.1	3702.9	4670.3	6150.7

Source: USACE, 2020

Boundary Conditions: For steady flow analyses, the boundary conditions for both the upstream and downstream boundary conditions were set to normal depth. Normal depth slope was calculated as the average slope over the three most upstream and three most downstream cross sections. Table 3 lists the boundary conditions used for this model.

Table 3: Boundary Conditions for Squaw Run HEC-RAS Model.

Location	Boundary Condition	Slope Value
Downstream	Normal Depth	0.0001
Upstream	Normal Depth	0.01345

PROPOSED MODIFICATIONS INPUT

Seven (7) proposed modifications and four (4) combinations of modifications were modeled by incorporating changes to specific cross sections in the calibrated models. Structural modifications were modeled for the areas of interest that are impacted by flooding along Squaw Run. Structural modifications that were modeled include construction of a floodwall, deepening the channel, and bridge modification. Additionally, the impacts the two (2) proposed stormwater management (SWM) ponds developed as a part of “Squaw Run Proposed Conditions HEC-HMS Model Development” (USACE, 2020) have on flood risk were incorporated into the model by revising the flow rates within the model to the proposed flow rates calculated in the above-referenced calculation. The following proposed modifications were considered:

FLOODWALL 1

A floodwall is proposed, as shown in Figure 4, between River Stations 4466.716 and 5064.615, in order to protect seven (7) buildings west of Fox Chapel Road from being flooded. The seven buildings are placed along a flat area west of Fox Chapel Road, such that as soon as Fox Chapel Road floods (roughly the 25-year flood), the buildings themselves flood as well. Fox Chapel Road is roughly 752 ft NAVD 88 along this portion of Fox Chapel Road. The floodwall itself is modeled to go up to 768.5 ft NAVD 88, above the 500-year flood. If implemented, it is recommended that the final selected height of the wall should be a risk-informed decision with an economic cost-benefit analysis.

Figure 4: Location of Floodwall 1.

FLOODWALL 2

A floodwall is proposed, as shown in Figure 5, between River Stations 3332.839 and 2570.757, in order to protect 23 buildings west of Fox Chapel Road from being flooded. The 23 buildings are placed along a flat area west of Fox Chapel Road, such that as soon as Fox Chapel Road floods (roughly the 10-year flood), the buildings themselves begin to flood as well. Fox Chapel Road is between 737 and 741 ft NAVD 88 along this portion of Fox Chapel Road. The floodwall itself is modeled to go up to 758 ft NAVD 88, above the 500-year flood. If implemented, it is recommended that the final selected height of the wall should be a risk-informed decision with an economic cost-benefit analysis.

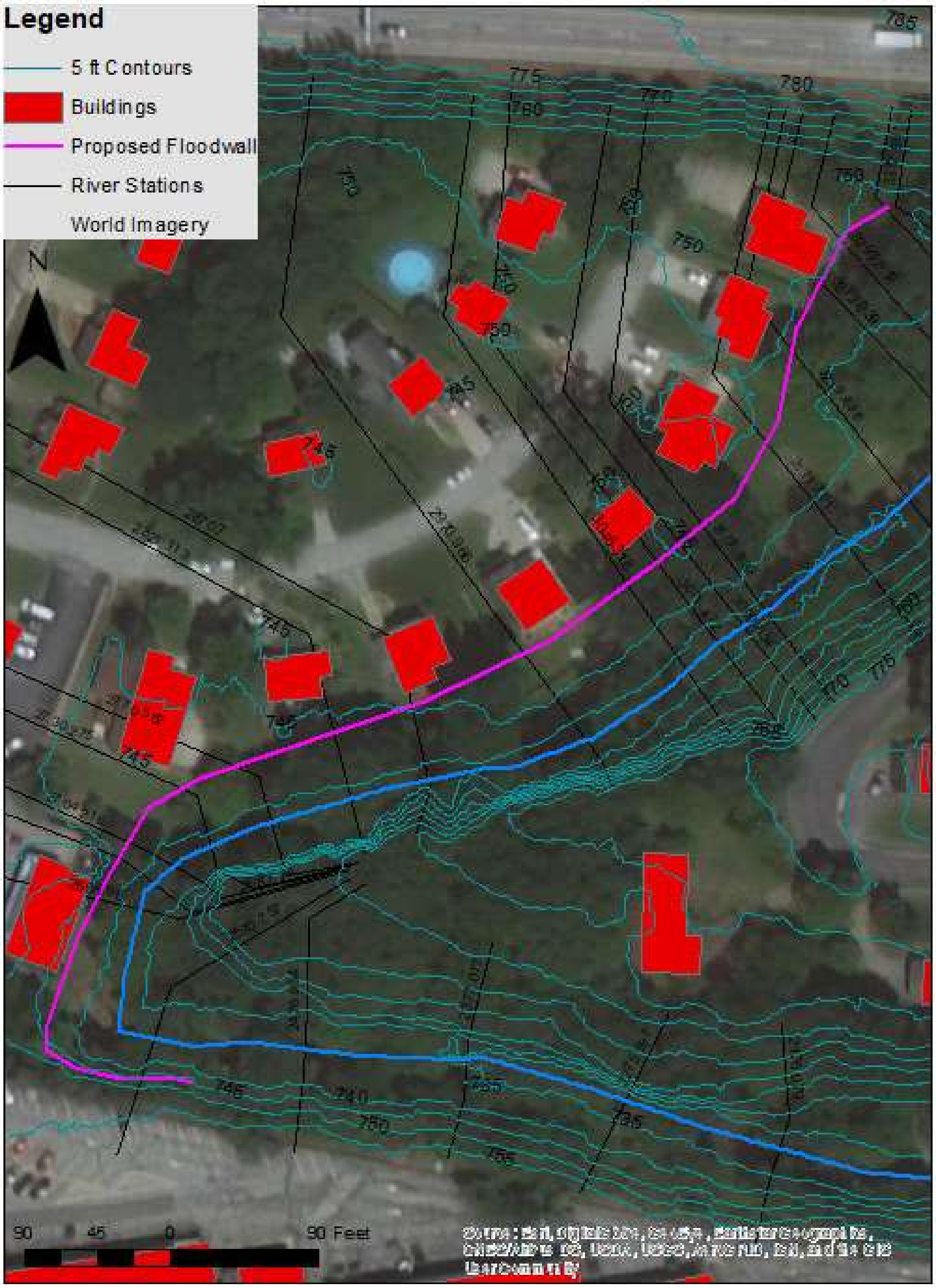

Figure 5: Location of Floodwall 2.

CHANNEL 1

Channelization is proposed between River Stations 4269.647 and 5265.757. The proposed channel is assumed to reduce the channel bottom by roughly two feet, smooth the slope over the channelized reach, and then tie into the effective ground elevations. The average ground slope in Channel 1 is very steep, leading to critical and supercritical flows. It is assumed that the proposed channel will need to be lined with riprap to avoid channel bottom erosion. Figure 6 shows the proposed location of the channel and Figure 7 shows the proposed channel bottom.

Figure 6: Location of Channel 1.

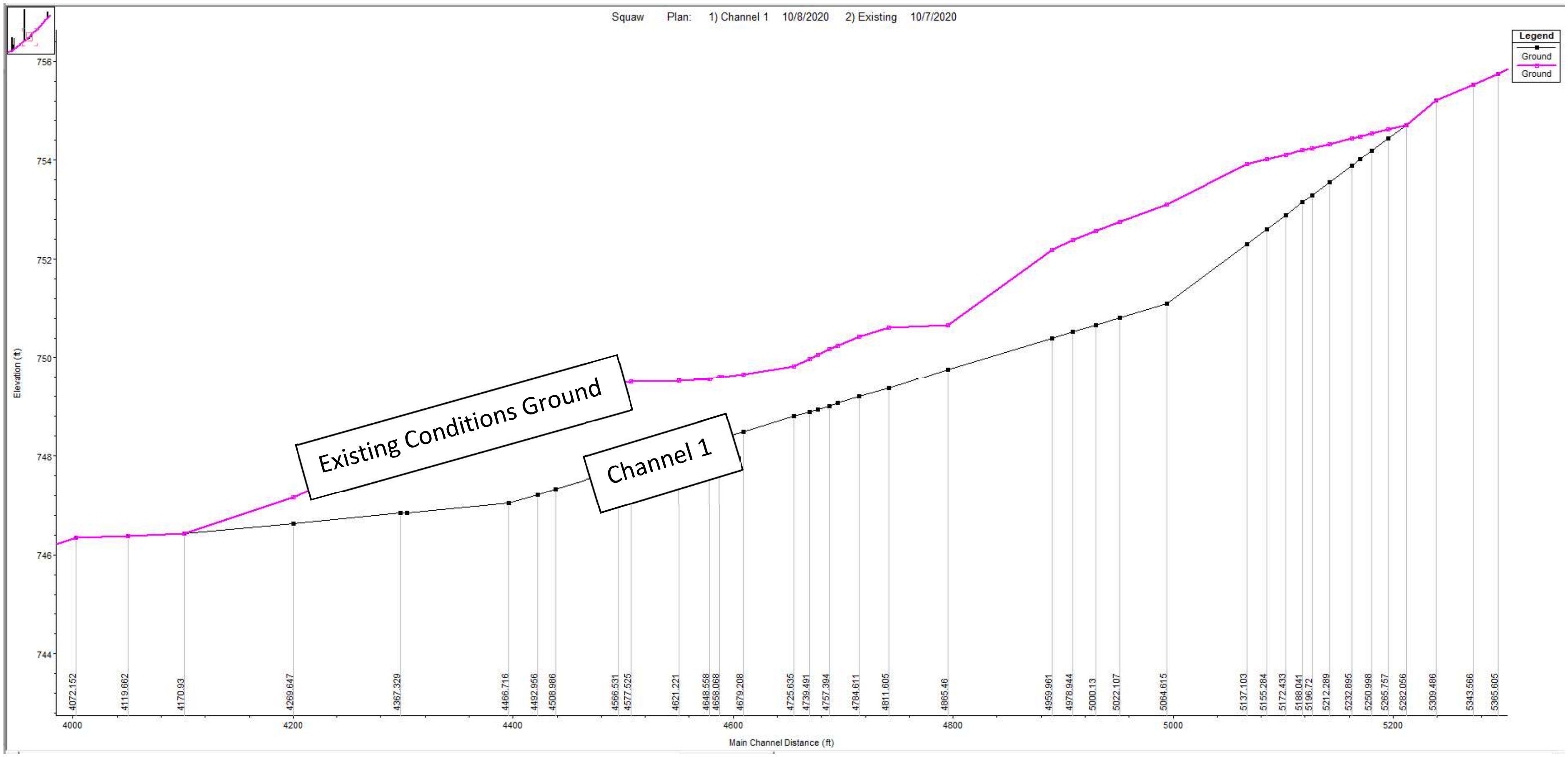

Figure 7: Profile of Channel 1.

CHANNEL 2

Channelization is proposed between River Stations 2377.077 and 3473.246. The proposed channel is assumed to reduce the channel bottom by roughly two feet, smooth the slope over the channelized reach, and be lined with riprap to avoid channel bottom erosion. Figure 8 shows the proposed location of the channel and Figure 9 shows the proposed channel bottom.

Figure 8: Location of Channel 2.

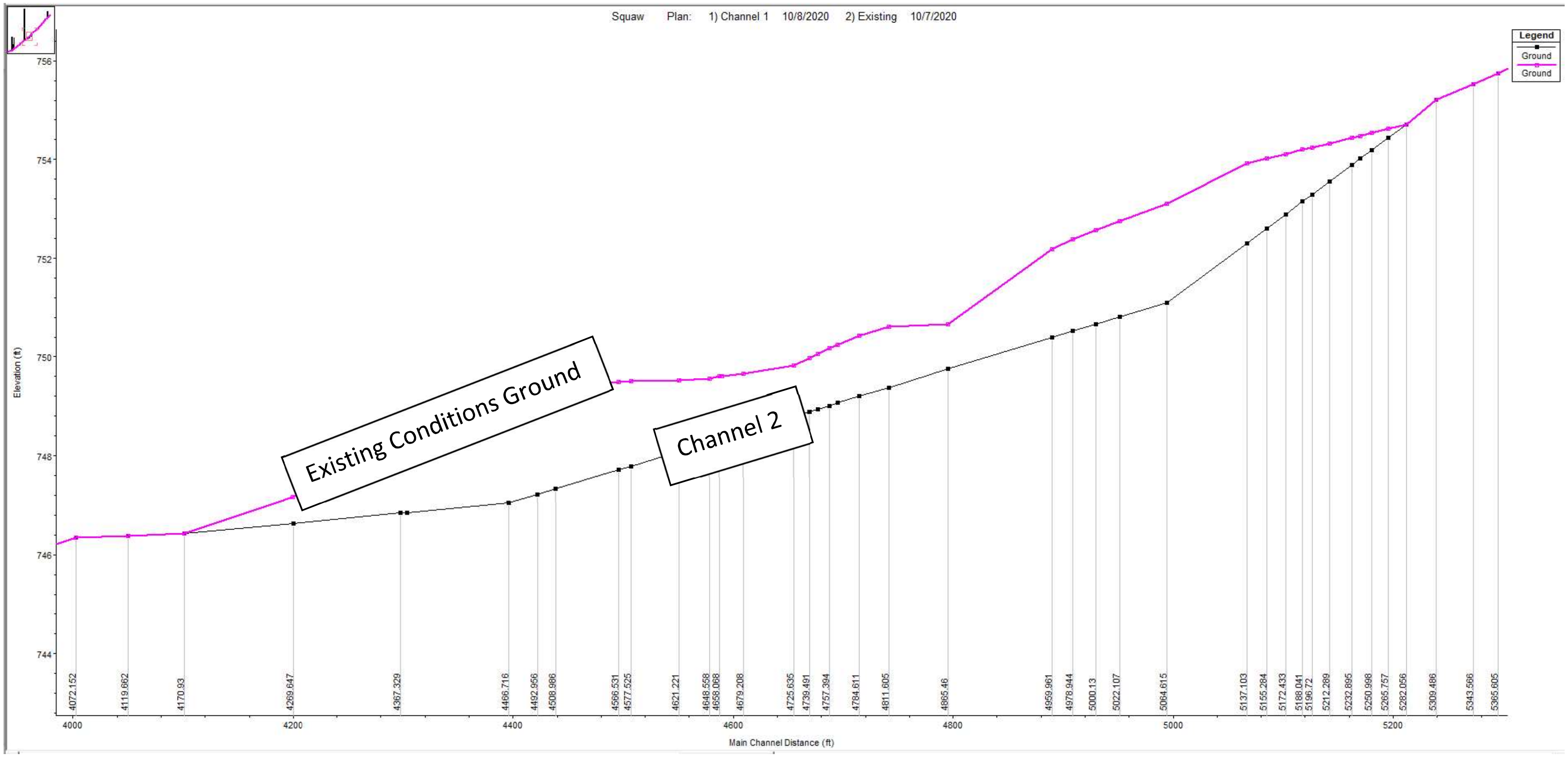

Figure 9: Profile of Channel 2.

REPLACEMENT OF OLD FREEPORT ROAD

There is flooding between Old Freeport Road and Freeport Road, as well as significant flooding behind Old Freeport Road. As a means to see whether replacing Old Freeport Road would impact flood risks, a replacement of Old Freeport Road was considered. As shown in Figure 10, there are parking lots along both sides of Old Freeport Road, making widening Old Freeport Road more difficult. The one potential improvement to Old Freeport Road which was considered was raising the road. The proposed Old Freeport Road bridge includes raising the top of the bridge 1.25 feet over the existing top of bridge. Figure 11 shows the existing bridge top and Figure 12 shows the proposed bridge top.

Figure 10: Close-up of Squaw Run along Freeport Road and Old Freeport Road.

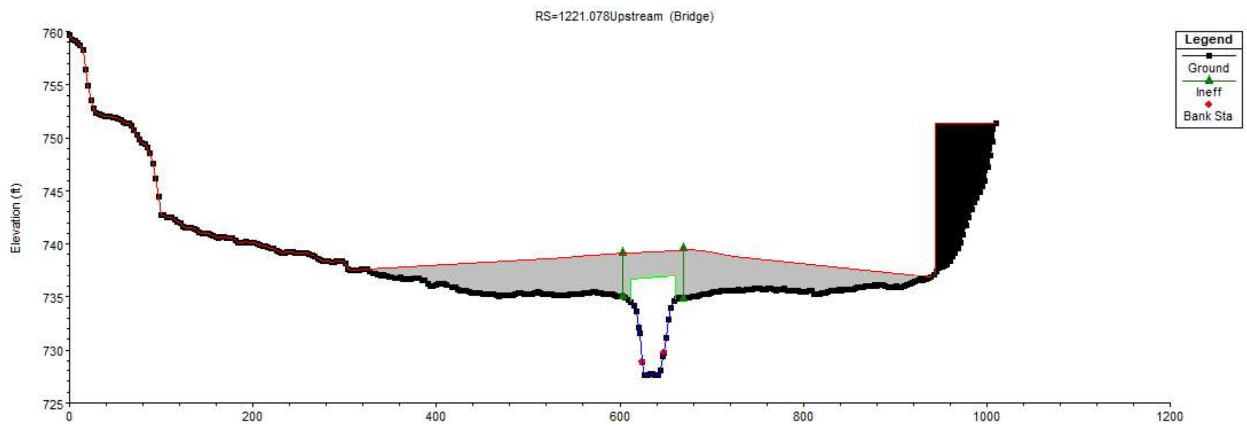

Figure 11: Existing Old Freeport Road Bridge.

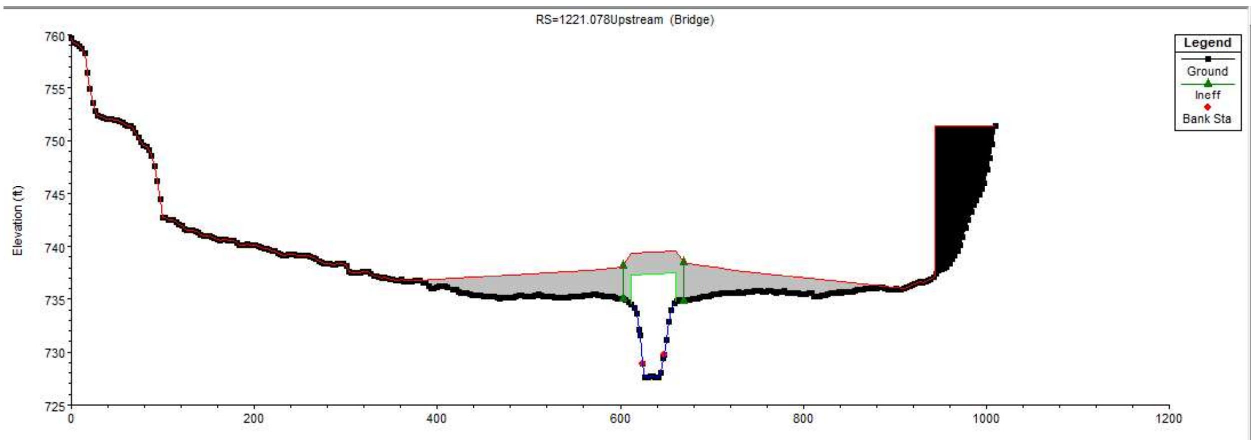

Figure 12: Existing Old Freeport Road Bridge.

STORMWATER PONDS 1 AND 2

The "Squaw Run Proposed Conditions HEC-HMS Model Development" (USACE, 2020) calculation brief describes the impact that two proposed stormwater ponds, one along Glad Run and one along Stony Camp Run, have on flow rates along Squaw Run. The impact that those two stormwater ponds have on flood elevations along Squaw Run is considered as part of this calculation. Table 4 lists the flow rates determined at the confluence with the Allegheny River for Stormwater Ponds 1 and 2 (USACE, 2020).

Table 4: Summary of Peak Discharges for Proposed Conditions Models.

Stormwater Pond	2-year (cfs)	10-year (cfs)	25-year (cfs)	50-year (cfs)	100-year (cfs)	200-year (cfs)	500-year (cfs)
1	477	1278.8	2072.3	2798.4	3566.4	4555.6	6036.2
2	391.2	1134.6	1880.2	2635.9	3452.6	4392.7	5800.5

COMBINATIONS OF STRUCTURAL MEASURES

Table 5 lists the combinations of structural measures, discussed above, which were modeled as a part of this calculation.

Table 5: Proposed Channel Modifications to Squaw Run Model.

Plan Name	XS Modified	Description
Floodwalls 1 and 2	5064.615 - 4466.716 and 3332.839 – 2570.757	The combination of Floodwall 1 and Floodwall 2.
SWM Pond 2 and Floodwall 1	5064.615 - 4466.716	Combination of SWM Pond 2 and Floodwall 1.
SWM Pond 2 and Floodwall 2	3332.839 – 2570.757	Combination of SWM Pond 2 and Floodwall 2.
SWM Pond 2 and Floodwalls 1 and 2	5064.615 - 4466.716 and 3332.839 – 2570.757	The combination of SWMM Pond 2, Floodwall 1, and Floodwall 2.

5.0 ANALYSIS OR NUMERICAL CALCULATIONS

HEC-RAS version 5.0.7 was used to run a steady flow analysis of the models using a mixed flow regime, and US customary measurements. No FIS profile exists for Squaw Run along the studied reach, but in July 2019, the Township Engineer for O'Hara Township provided USACE with a high water mark of 2.02 feet above a slab elevation of 752.996 feet at 222 North Margery Drive. Manning's n values and contraction/expansion coefficients were adjusted until there was a satisfactory match between hydraulic model results and known water surface elevations for the one high water mark, and to represent current site conditions under the appropriate flows. The calibrated HEC-RAS models have water surface profiles within one (1) percent of the high water mark along 222 North Margery Drive. Table 8 in Section 7.0, Results, summarized the results of the calibration. Table 10 in Section 7.0, Results, summarizes the results of the proposed modifications.

CALIBRATION OF HEC-RAS MODEL TO HIGH WATER MARK

As noted in the Section 3.0, Assumptions and Justifications, it is assumed that the high water mark of 755.02 occurred during Hurricane Ivan. Hurricane Ivan impacted the site approximately throughout September 17, 2004. Fifteen-minute NEXRAD data was downstream from the 3 Rivers Wet Weather Website (3 Rivers Wet Weather, 2020). The rainfall, tabulated in Table 6, was then input into the existing HEC-HMS model developed as a part of "Squaw Run Existing Conditions HEC-HMS Model Development" (USACE, 2020).

Table 6: 15-Minute NEXRAD Precipitation for Hurricane Ivan.

Timestamp	Precipitation (in)	Timestamp	Precipitation (in)	Timestamp	Precipitation (in)	Timestamp	Precipitation (in)
9/17/2004 1:15	0	9/17/2004 6:45	0.016	9/17/2004 12:15	0.164	9/17/2004 17:45	0.02
9/17/2004 1:30	0.003	9/17/2004 7:00	0.029	9/17/2004 12:30	0.088	9/17/2004 18:00	0.021
9/17/2004 1:45	0	9/17/2004 7:15	0.052	9/17/2004 12:45	0.106	9/17/2004 18:15	0.0509
9/17/2004 2:00	0	9/17/2004 7:30	0.039	9/17/2004 13:00	0.103	9/17/2004 18:30	0.056
9/17/2004 2:15	0	9/17/2004 7:45	0.022	9/17/2004 13:15	0.116	9/17/2004 18:45	0.05
9/17/2004 2:30	0.001	9/17/2004 8:00	0.027	9/17/2004 13:30	0.036	9/17/2004 19:00	0.037
9/17/2004 2:45	0.001	9/17/2004 8:15	0.018	9/17/2004 13:45	0.037	9/17/2004 19:15	0.059
9/17/2004 3:00	0	9/17/2004 8:30	0.03	9/17/2004 14:00	0.132	9/17/2004 19:30	0.027
9/17/2004 3:15	0	9/17/2004 8:45	0.035	9/17/2004 14:15	0.136	9/17/2004 19:45	0.01
9/17/2004 3:30	0	9/17/2004 9:00	0.027	9/17/2004 14:30	0.244	9/17/2004 20:00	0.012
9/17/2004 3:45	0	9/17/2004 9:15	0.039	9/17/2004 14:45	0.2839	9/17/2004 20:15	0.004
9/17/2004 4:00	0.009	9/17/2004 9:30	0.037	9/17/2004 15:00	0.1019	9/17/2004 20:30	0.017
9/17/2004 4:15	0.02	9/17/2004 9:45	0.0509	9/17/2004 15:15	0.08	9/17/2004 20:45	0.03
9/17/2004 4:30	0.008	9/17/2004 10:00	0.078	9/17/2004 15:30	0.149	9/17/2004 21:00	0.02
9/17/2004 4:45	0.008	9/17/2004 10:15	0.06	9/17/2004 15:45	0.125	9/17/2004 21:15	0.026
9/17/2004 5:00	0.006	9/17/2004 10:30	0.153	9/17/2004 16:00	0.233	9/17/2004 21:30	0.023
9/17/2004 5:15	0.006	9/17/2004 10:45	0.158	9/17/2004 16:15	0.343	9/17/2004 21:45	0.035
9/17/2004 5:30	0.003	9/17/2004 11:00	0.077	9/17/2004 16:30	0.277	9/17/2004 22:00	0.063
9/17/2004 5:45	0.005	9/17/2004 11:15	0.088	9/17/2004 16:45	0.304	9/17/2004 22:15	0.107
9/17/2004 6:00	0.005	9/17/2004 11:30	0.057	9/17/2004 17:00	0.116	9/17/2004 22:30	0.111
9/17/2004 6:15	0.005	9/17/2004 11:45	0.036	9/17/2004 17:15	0.099	9/17/2004 22:45	0.053
9/17/2004 6:30	0.008	9/17/2004 12:00	0.0869	9/17/2004 17:30	0.068	9/17/2004 23:00	0.057

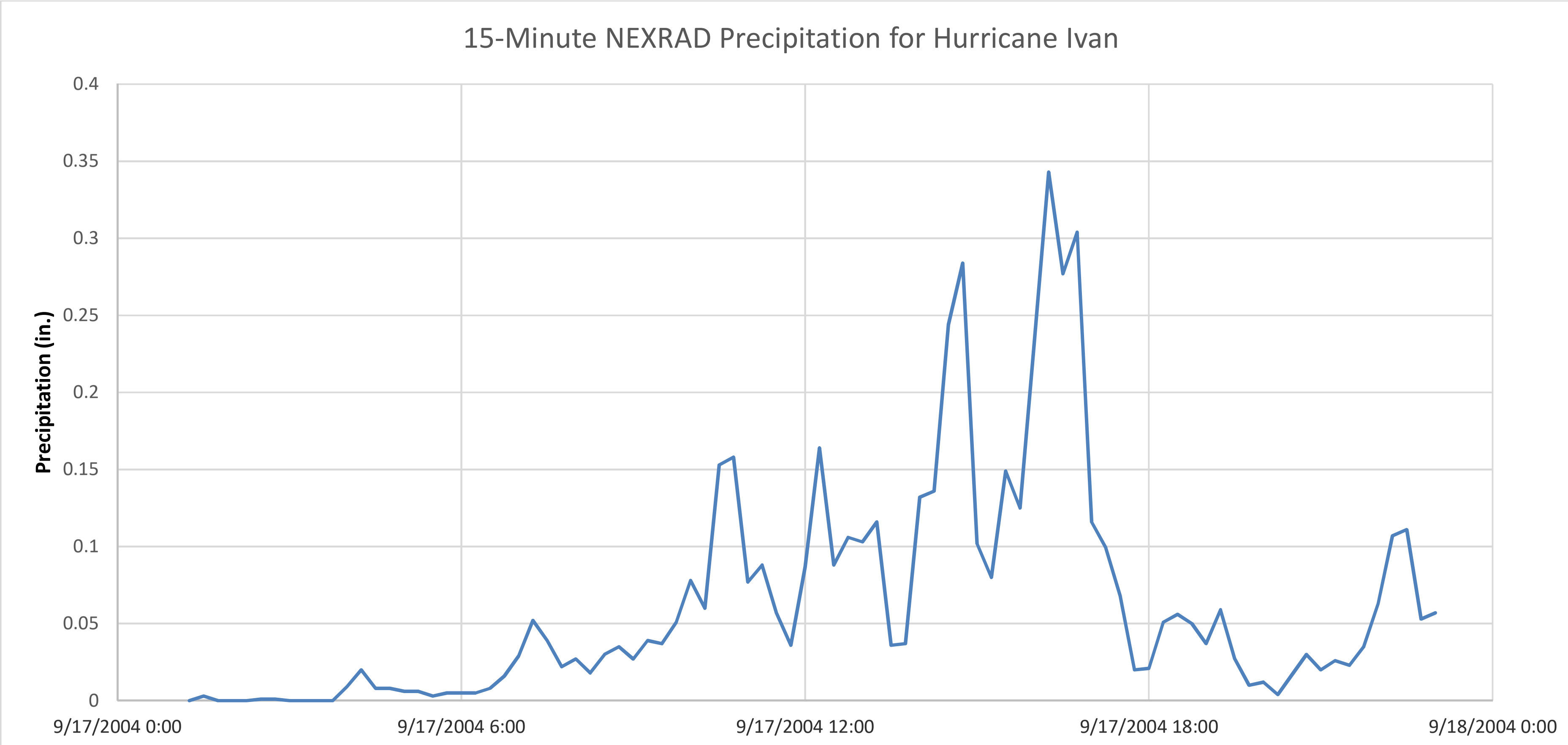

Figure 13: 15-Minute NEXRAD Gage Corrected Precipitation Data for Hurricane Ivan

As per Section 630.1001(d) of the NRCS National Engineering Handbook (NEH), variability in the curve number results from variability in rainfall intensity and duration, total rainfall, soil moisture conditions, cover density, stage of growth, and temperature. As Hurricane Ivan was a highly intense storm, the curve numbers were increased to antecedent runoff condition III (ARC III). Table 10-1 of the NEH provides the conversion between whole number curve numbers and curve numbers for ARC III. To get the appropriate curve numbers for the revised model, the curve numbers for each subbasin were rounded to the nearest whole number and then Table 10-1 was consulted to determine the appropriate curve number for ARC III. The revised ARC III curve numbers were input into the revised HEC-HMS model to determine the flow rate for Hurricane Ivan. Table 7 includes the revised curve numbers used in the revised HEC-HMS model.

Table 7: Revised Curve Numbers for Hurricane Ivan, Assuming ARC III.

HEC-HMS name	Existing CN	CN (ARC III)
SquawHW-US	72.04	86.00
SquawHW-DS	72.04	86.00
SquawHWMid	70.75	86.00
GladeRunHW	71.52	86.00
GladeRunMid	69.78	85.00
GladeRun	70.62	86.00
StonyCampRun	70.54	86.00
SquawMid	70.46	85.00
EastTrib	73.41	87.00
SquawMidLow	69.39	84.00
SquawLow	73.36	87.00
RIDCTrib	77.74	90.00

The revised HEC-HMS model provides a flow rate for Hurricane Ivan at Junction J78 of 3468.5 cfs. This flow was entered into HEC-RAS and the Manning's n values were adjusted until the model approached the high water mark noted at 222 North Margery Drive.

6.0 CALCULATION OUTPUT

Input and output files for the calibrated and proposed modifications HEC-RAS models are located in the following folder: L:\EC\EC-WH\Planning_FPMS\2020SquawRun\2_Working\HEC-RAS. Input and output files for the revised HEC-HMS models are located in the following folder: L:\EC\EC-WH\Planning_FPMS\2020SquawRun\2_Working\HEC-HMS\SquawRunExisting - ARC III.

7.0 RESULTS

CALIBRATED MODELS

The calibrated model for Squaw Run was modeled as a steady state flow analysis. The water surface profiles calculated in HEC-RAS compare favorably to the historical high water mark at 222 North Margery Drive. Table 8 summarizes the results of the calibration for Squaw Run.

Table 8: Calibrated Model for Squaw Run.

HEC-RAS Station	3871.54
HEC-RAS Water Depth (ft NAVD 88)	755.03
Historical High Water Mark (ft NAVD 88)	755.02
Depth above Slab Elevation (ft)	2.02
HEC-RAS Calculated Depth	2.03
Difference (ft)	0.01
Difference (%)	0.50%

IMPACTS TO NEARBY BUILDINGS

The primary concern of this project is that numerous structures within the floodplain are damaged when there is sufficient rain. To compare how effective proposed alternatives are, the number of structures impacted by flood waters for each analyzed storm is summarized in Table 9. Only those structures that see impacts stemming from overflow of Squaw Run itself were used for this analysis.

Table 9: Buildings Impacted by Flood Waters.

Return Period of Flow	Number of Buildings Impacted by Flood
2-year	1
10-year	7
25-year	27
50-year	36
100-year	46
200-year	52
500-year	59

PROPOSED MODELS

Results from the proposed modifications models that were described in Section 4.0, Calculation Input, were compared with results from the calibrated models. Average change in water surface elevation, cumulative change in overbank water volume, and reduction of structures flooded from each structural modification in the areas of interest along Squaw Run are summarized in Table 10 below.

Table 10: Summary of Benefits from Structural Modifications on Squaw Run.

Return Event (yr)	Plan Name	Avg. Change in WSE* (ft)	Cumulative Change in Overbank Volume (ac-ft)	Reduction of Structures Flooded
2	Floodwall 1	0.003	-0.17	0
	Floodwall 2	0.000	0	0
	Floodwalls 1 and 2	0.003	-0.18	0
	Channel 1	-0.321	-0.41	0
	Channel 2	-0.258	-0.33	0
	Replace Ol Freeport	0.000	0	0
	SWM Pond 1	-0.075	-0.44	0
	SWM Pond 2	-0.410	-2.26	0
	SWM Pond 2 and Floodwall 1	-0.410	-2.26	0
	SWM Pond 2 and Floodwall 2	-0.410	-2.26	0
	SWM Pond 2 and Floodwalls 1 and 2	-0.409	-2.33	0
10	Floodwall 1	0.024	-2.57	1
	Floodwall 2	-0.003	-0.31	3
	Floodwalls 1 and 2	0.022	-2.87	3
	Channel 1	-0.234	-2.15	0
	Channel 2	-0.216	-2.36	3
	Replace Ol Freeport	0.000	0	1
	SWM Pond 1	-0.226	-3.41	1
	SWM Pond 2	-0.597	-8.50	3
	SWM Pond 2 and Floodwall 1	-0.597	-8.50	3
	SWM Pond 2 and Floodwall 2	-0.598	-8.63	5
	SWM Pond 2 and Floodwalls 1 and 2	-0.579	-10.38	5
25	Floodwall 1	0.025	-6.92	7
	Floodwall 2	0.033	-0.99	7
	Floodwalls 1 and 2	0.058	-7.92	14
	Channel 1	-0.134	-2.40	-2

	Channel 2	-0.033	-2.60	4
	Replace Ol Freeport	-0.027	-0.47	1
	SWM Pond 1	-0.243	-7.09	1
	SWM Pond 2	-0.697	-17.96	10
	SWM Pond 2 and Floodwall 1	-0.697	-17.96	16
	SWM Pond 2 and Floodwall 2	-0.697	-18.73	14
	SWM Pond 2 and Floodwalls 1 and 2	-0.663	-23.39	20
50	Floodwall 1	0.047	-8.26	7
	Floodwall 2	0.090	-2.80	11
	Floodwalls 1 and 2	0.137	-11.06	18
	Channel 1	0.111	3.87	-1
	Channel 2	-0.072	-2.15	0
	Replace Ol Freeport	-0.014	-0.01	1
	SWM Pond 1	-0.191	-8.09	2
	SWM Pond 2	-0.445	-18.16	3
	SWM Pond 2 and Floodwall 1	-0.351	-26.15	10
	SWM Pond 2 and Floodwall 2	-0.380	-19.94	13
	SWM Pond 2 and Floodwalls 1 and 2	-0.331	-27.93	20
100	Floodwall 1	0.105	-9.05	7
	Floodwall 2	0.142	-7.16	18
	Floodwalls 1 and 2	0.247	-16.21	25
	Channel 1	0.279	9.77	0
	Channel 2	-0.066	-3.06	4
	Replace Ol Freeport	-0.001	0.55	1
	SWM Pond 1	-0.132	-7.1	2
	SWM Pond 2	-0.332	-15.33	5
	SWM Pond 2 and Floodwall 1	-0.250	-24.24	12
	SWM Pond 2 and Floodwall 2	-0.208	-20.71	21
	SWM Pond 2 and Floodwalls 1 and 2	-0.126	-29.62	28
200	Floodwall 1	0.192	-8.89	7
	Floodwall 2	0.190	-18.07	21
	Floodwalls 1 and 2	0.383	-26.95	28

	Channel 1	0.522	19.5	-1
	Channel 2	-0.045	-2.49	0
	Replace Ol Freeport	0.000	0.56	0
	SWM Pond 1	-0.100	-6.45	0
	SWM Pond 2	-0.366	-18.99	1
	SWM Pond 2 and Floodwall 1	-0.199	-28.00	8
	SWM Pond 2 and Floodwall 2	-0.189	-33.94	22
	SWM Pond 2 and Floodwalls 1 and 2	-0.022	-42.94	29
500	Floodwall 1	0.338	-6.92	5
	Floodwall 2	0.309	-34.82	22
	Floodwalls 1 and 2	0.645	-41.79	27
	Channel 1	0.823	34.53	-2
	Channel 2	-0.040	-2.16	1
	Replace Ol Freeport	-0.002	0.56	0
	SWM Pond 1	-0.314	17.15	2
	SWM Pond 2	-0.689	--35.91	3
	SWM Pond 2 and Floodwall 1	-0.387	-43.42	9
	SWM Pond 2 and Floodwall 2	-0.365	-61.09	25
	SWM Pond 2 and Floodwalls 1 and 2	-0.063	-68.60	31

* WSE = water surface elevation

8.0 CONCLUSION/SUMMARY

The calibrated model for Squaw Run represents existing conditions. No errors or problematic warnings have been reported with the model.

Structural improvements to the stream reaches were analyzed as part of the proposed conditions modeling effort in order to assess the flood reduction impacts of the proposed improvements.

Seven (7) proposed structural alternatives and four (4) combinations of structural alternatives were modeled separately to determine the extent of flood reduction along a given reach. The recommended structural modifications include either proposed floodwall, or both floodwalls togetherand the SWM Pond 2 modification, which reduces flows by roughly 15 percent and led to the overall highest average decrease in water surface elevations over the seven proposed structural alternatives. A combination of SWM Pond 2 and both floodwalls together would lead the overall highest reduction in flood risk for the modeled portions of Squaw Run. If implemented, it is recommended that the final selected height of the wall should be a risk informed decision with an economic cost-benefit analysis.

9.0 REFERENCES

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Zhang, J., Howard, K., Langston, C., Kaney, B., Qi, Y., Tang, L., ... & Arthur, A. (2016). Multi-Radar Multi-Sensor (MRMS) quantitative precipitation estimation: Initial operating capabilities. Bulletin of the American Meteorological Society, 97(4), 621-638. [MRMS data available at: https://mtarchive.geol.iastate.edu/, Accessed: August 2020].

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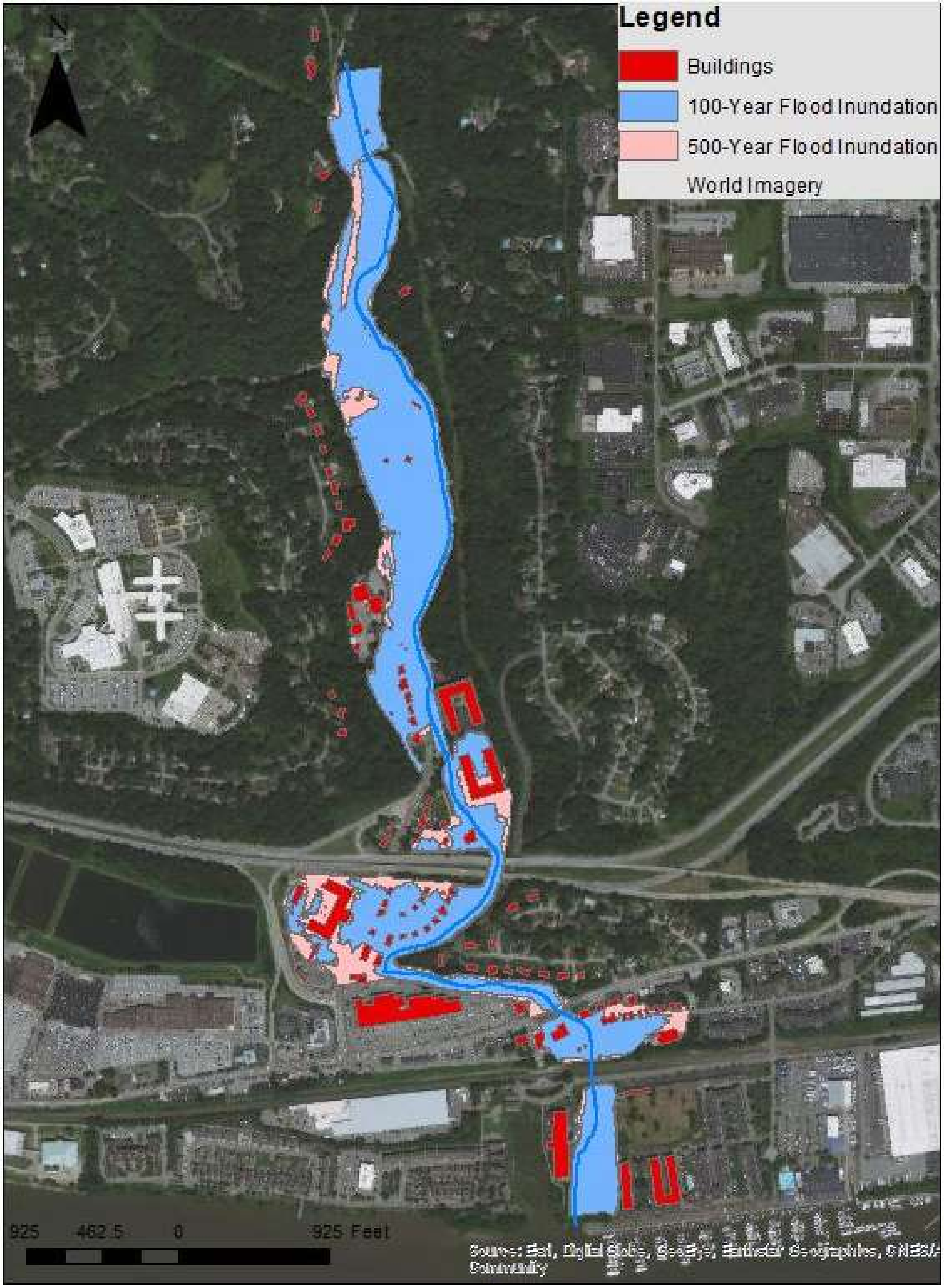
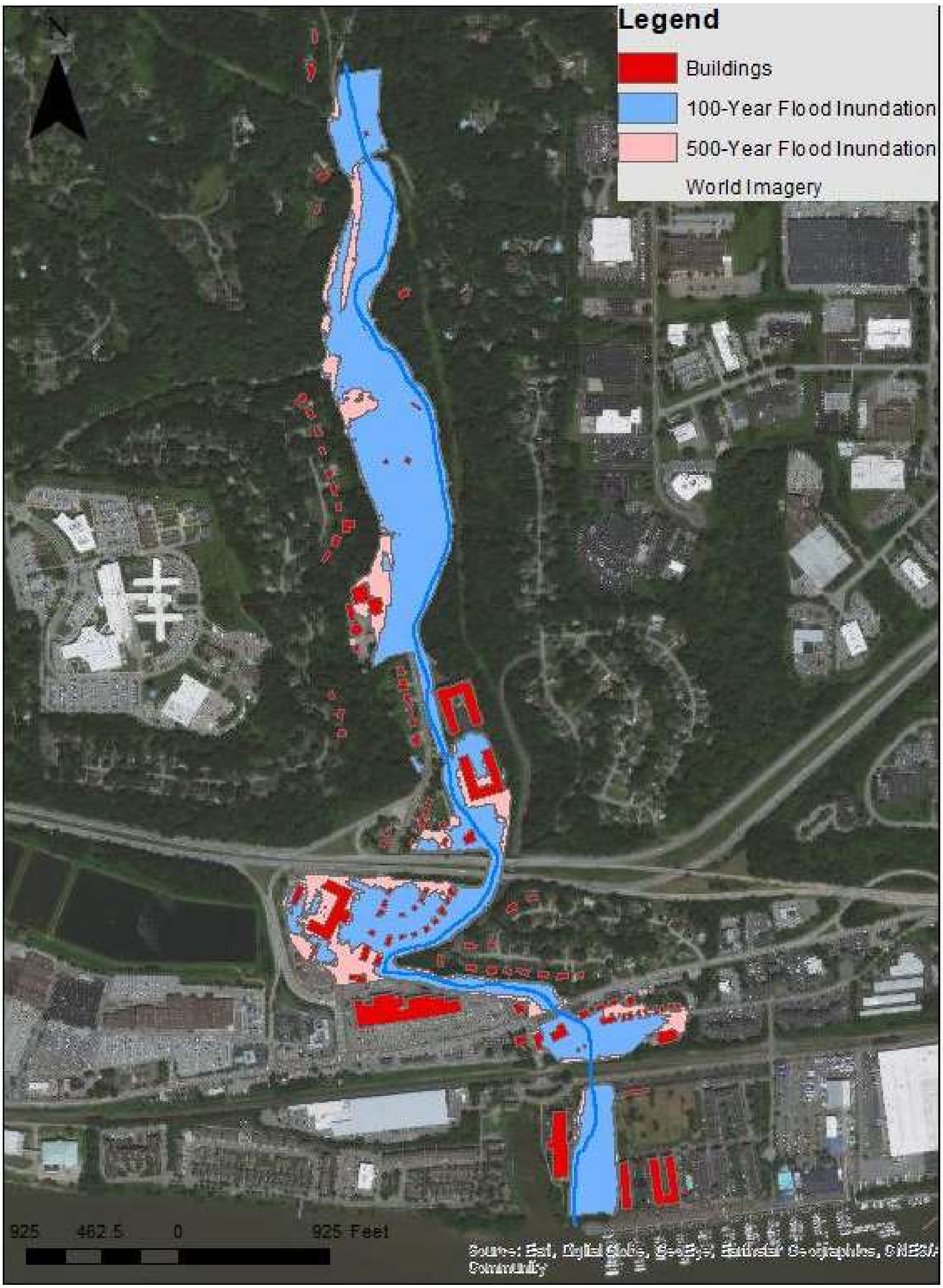
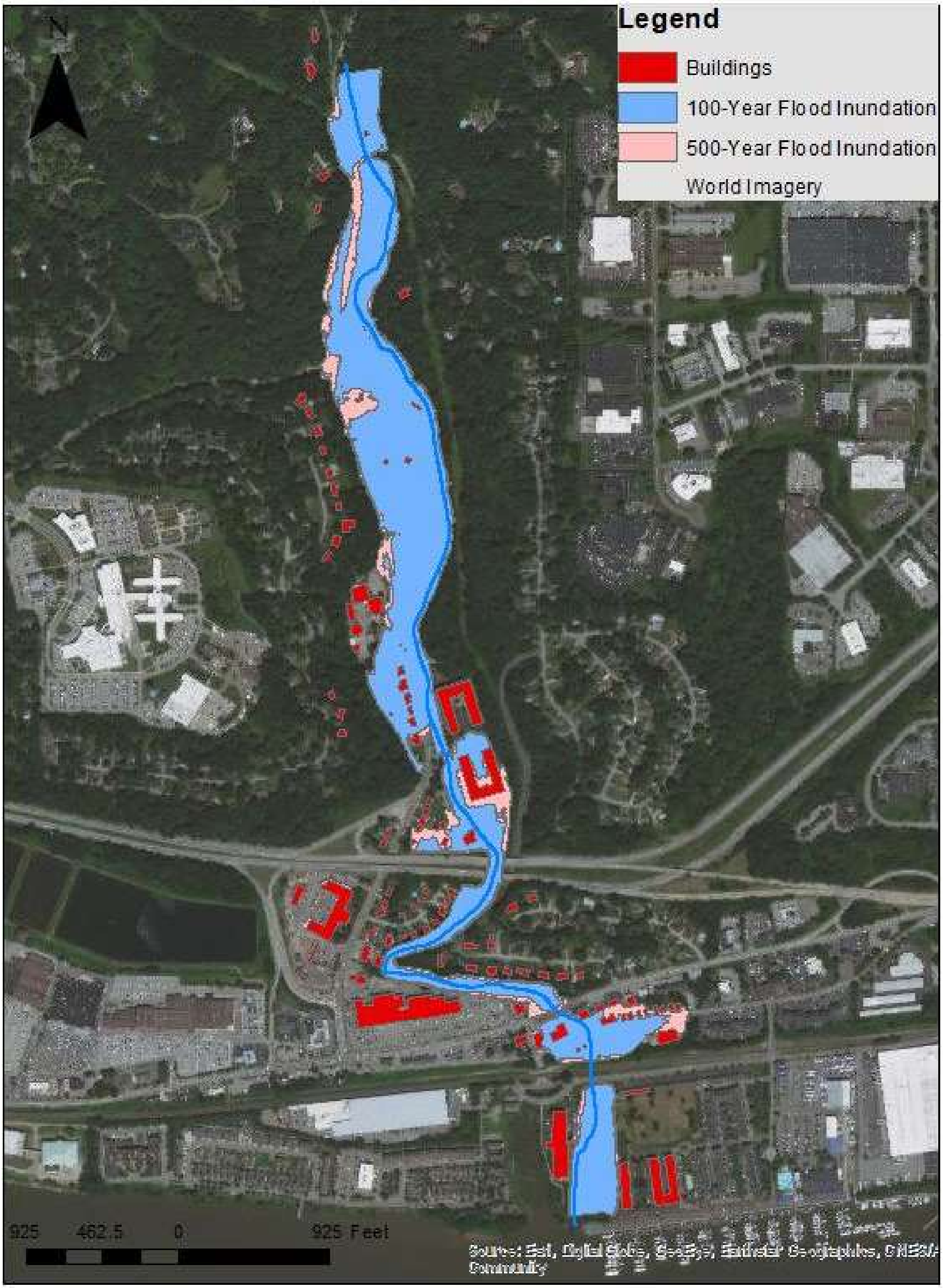
Inundation Map – Existing Conditions



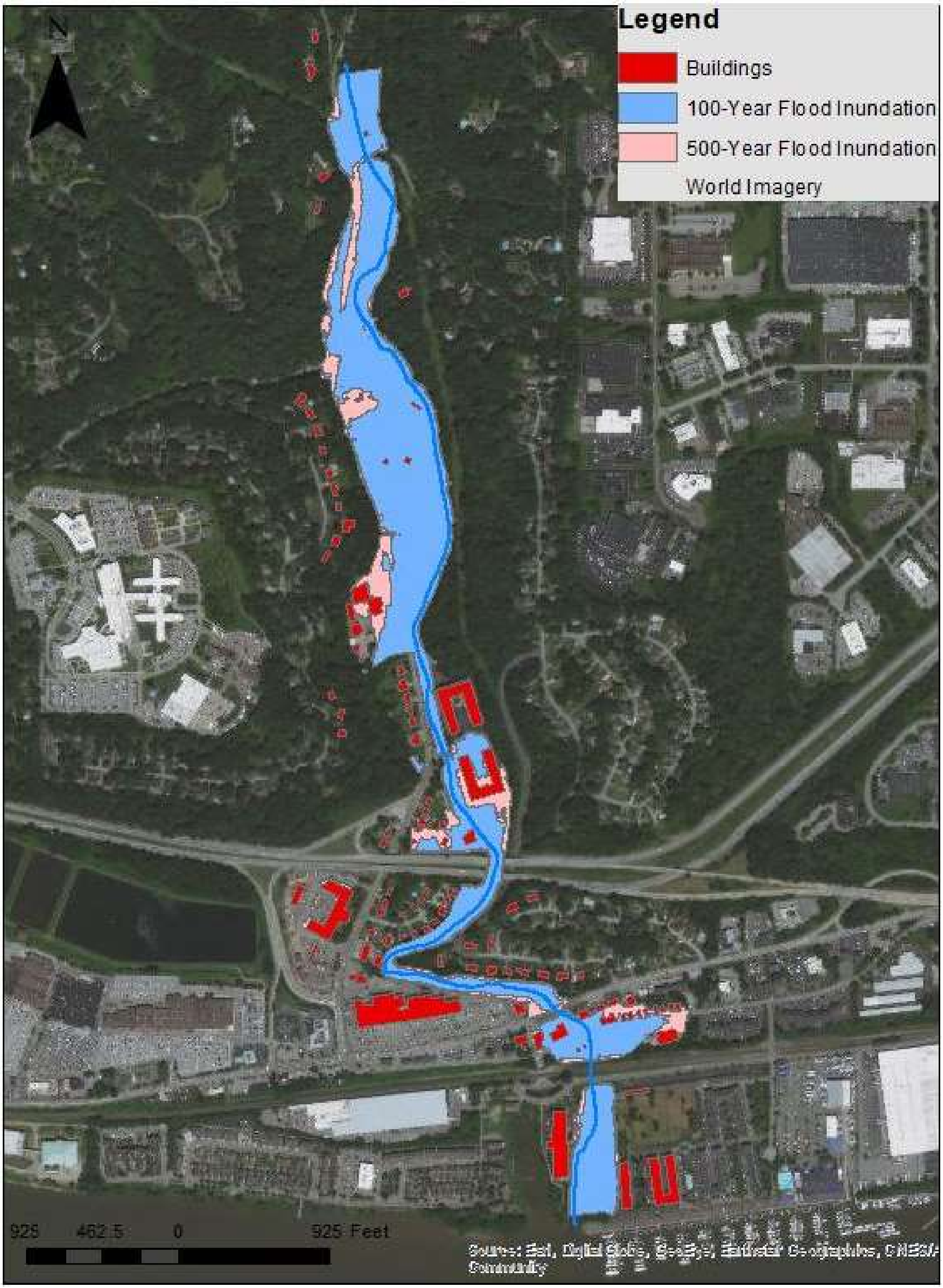
Inundation Map – Floodwall 1



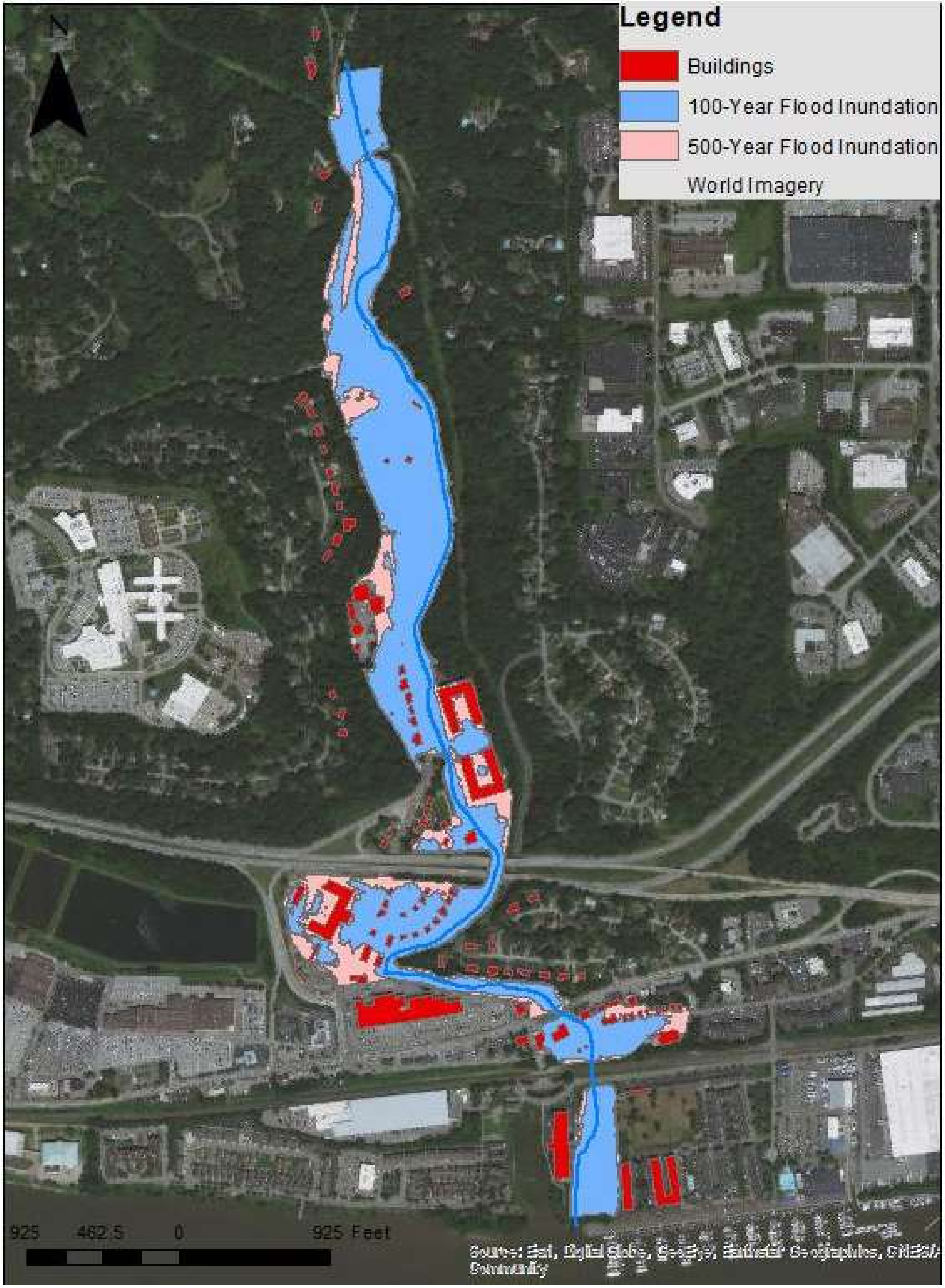
Inundation Map – Floodwall 2



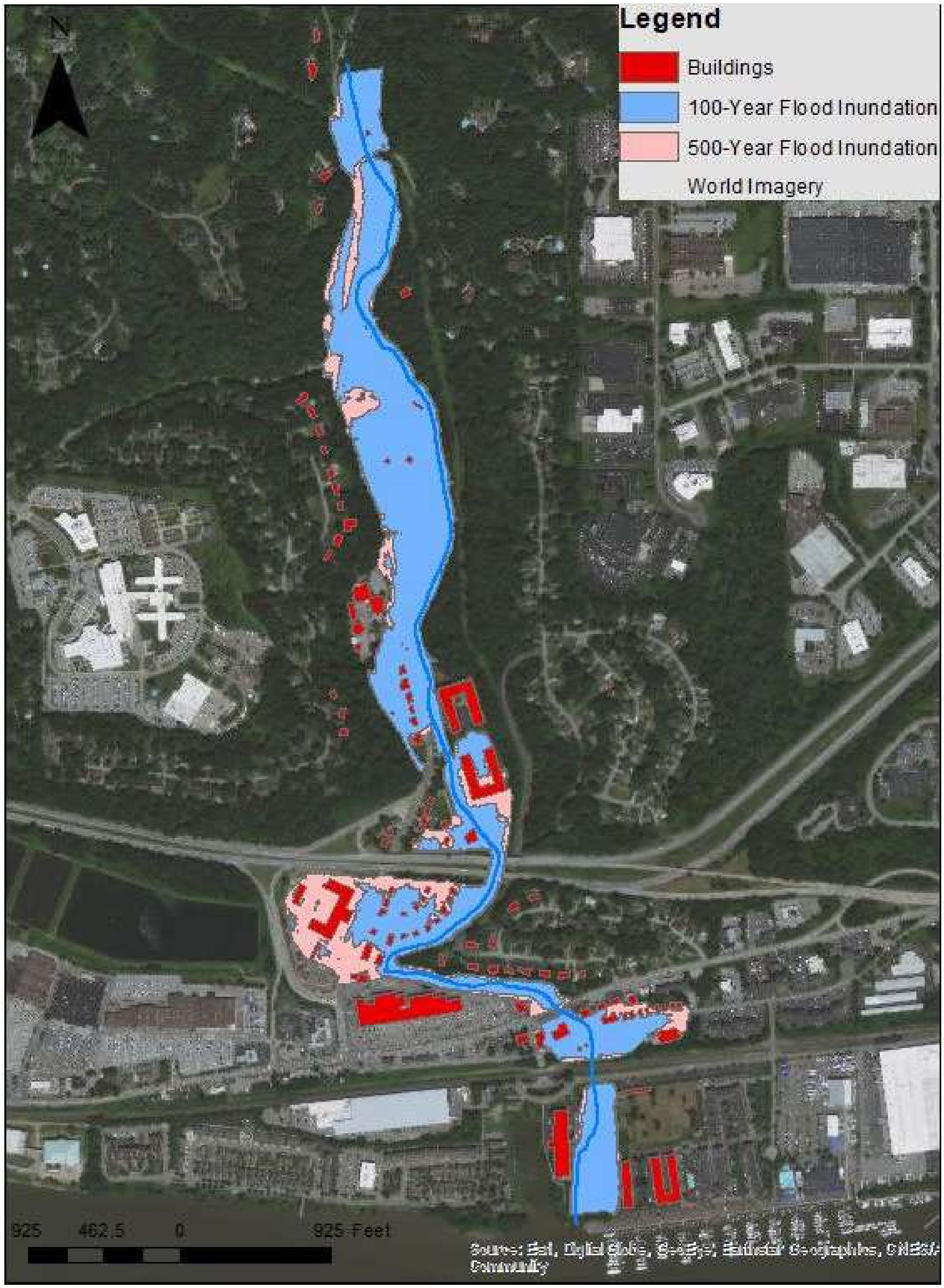
Inundation Map – Floodwalls 1 and 2



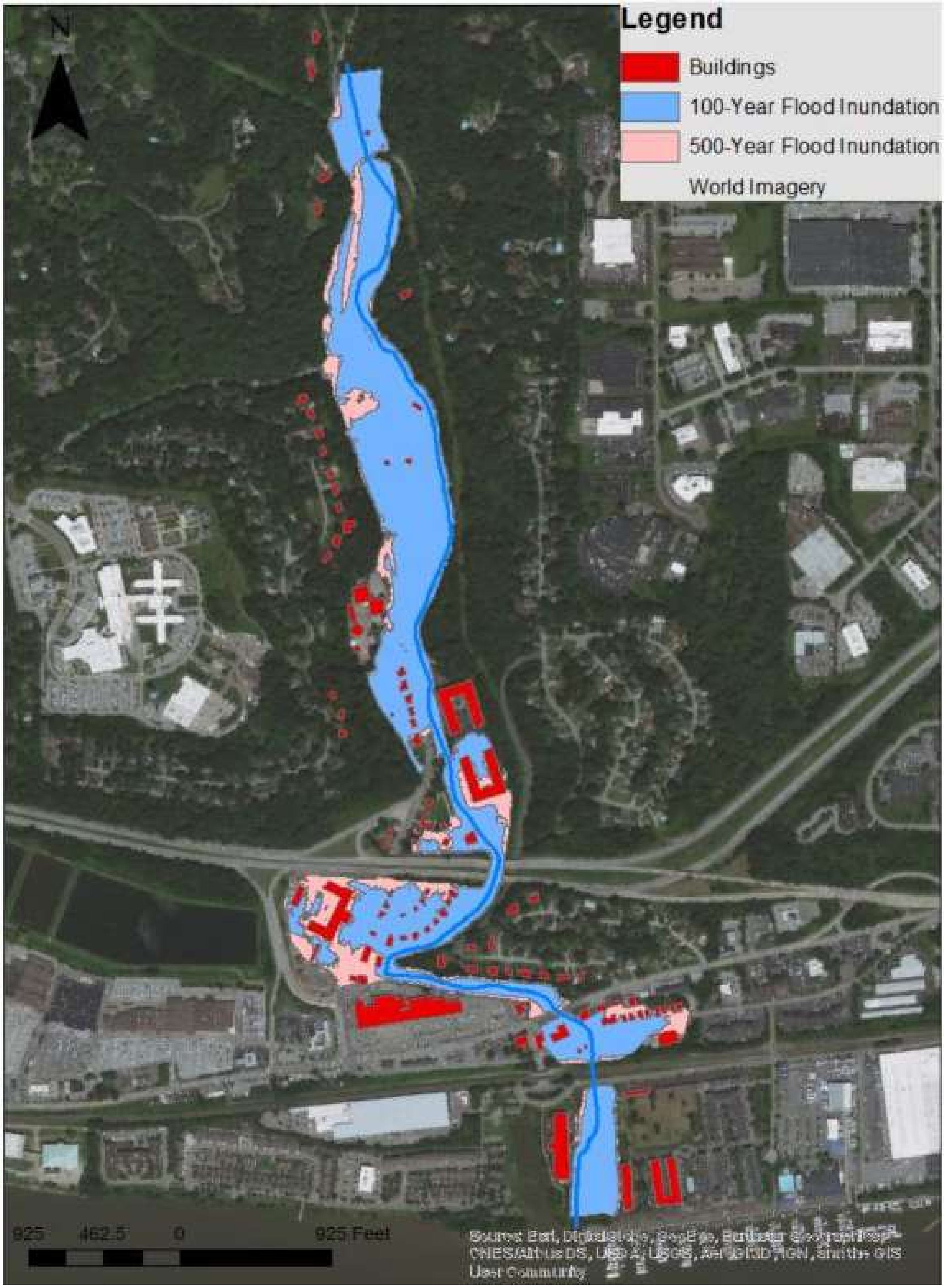
Inundation Map – Channel 1



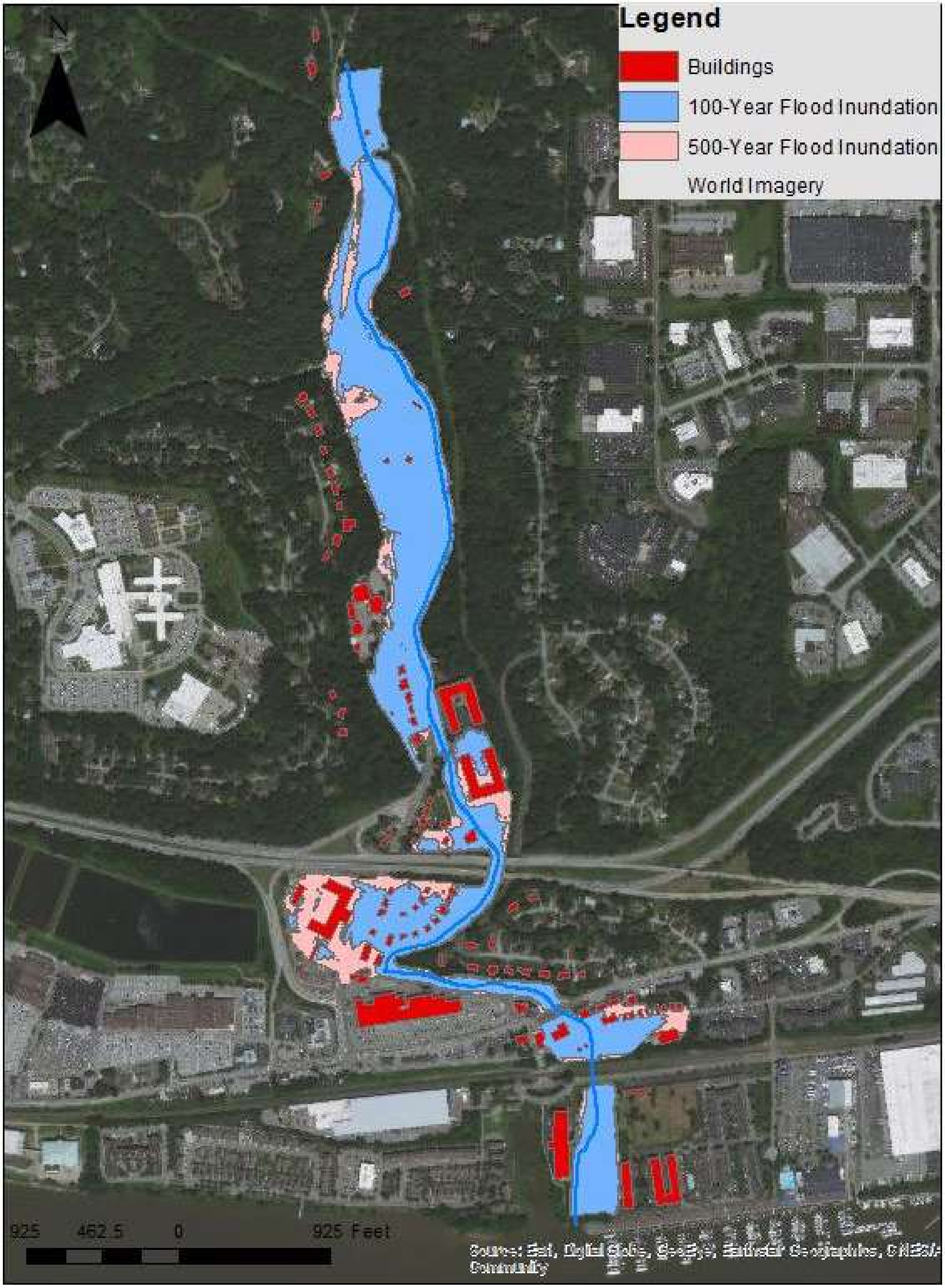
Inundation Map – Channel 2



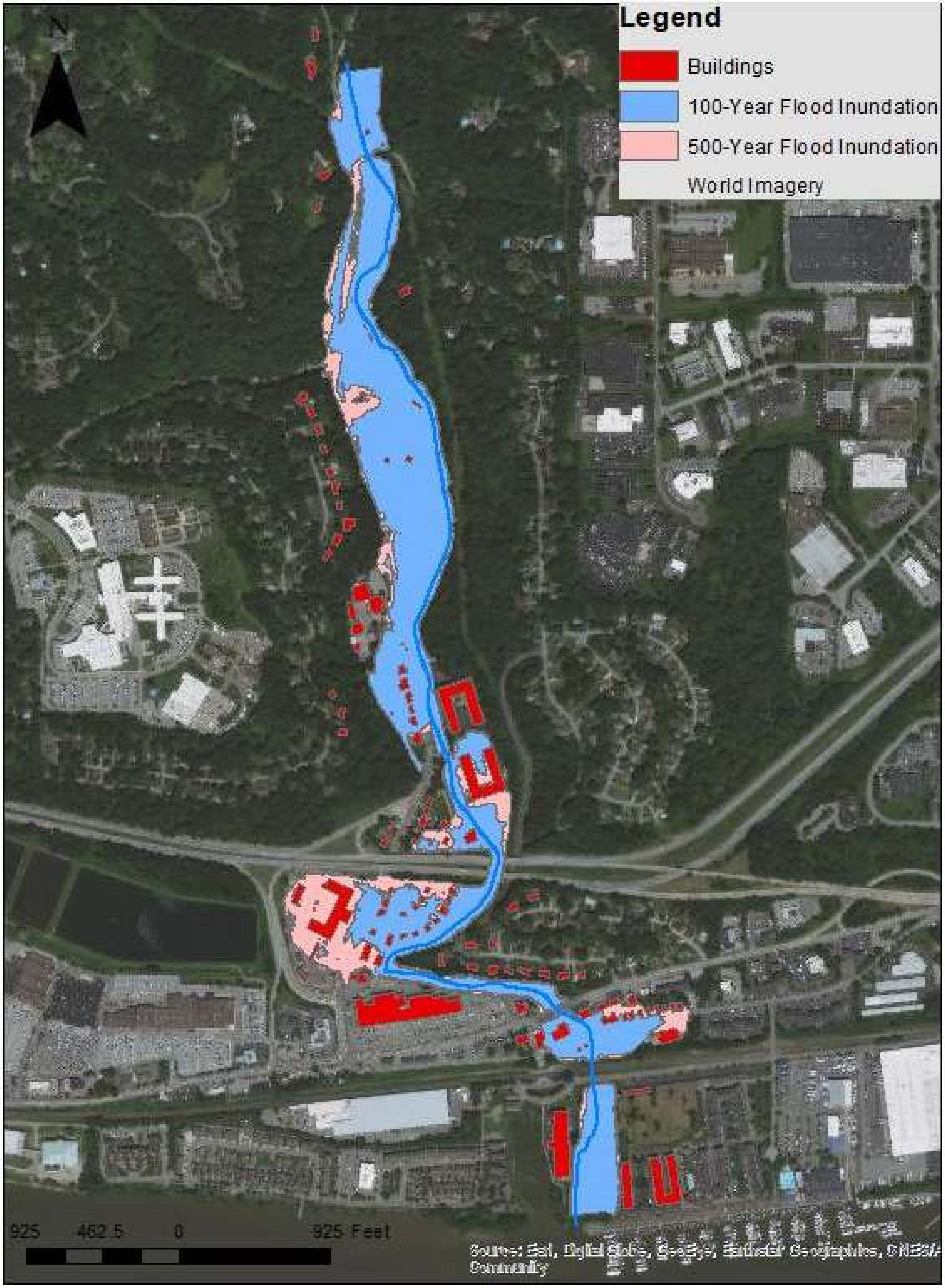
Inundation Map – Replacing Old Freeport Road



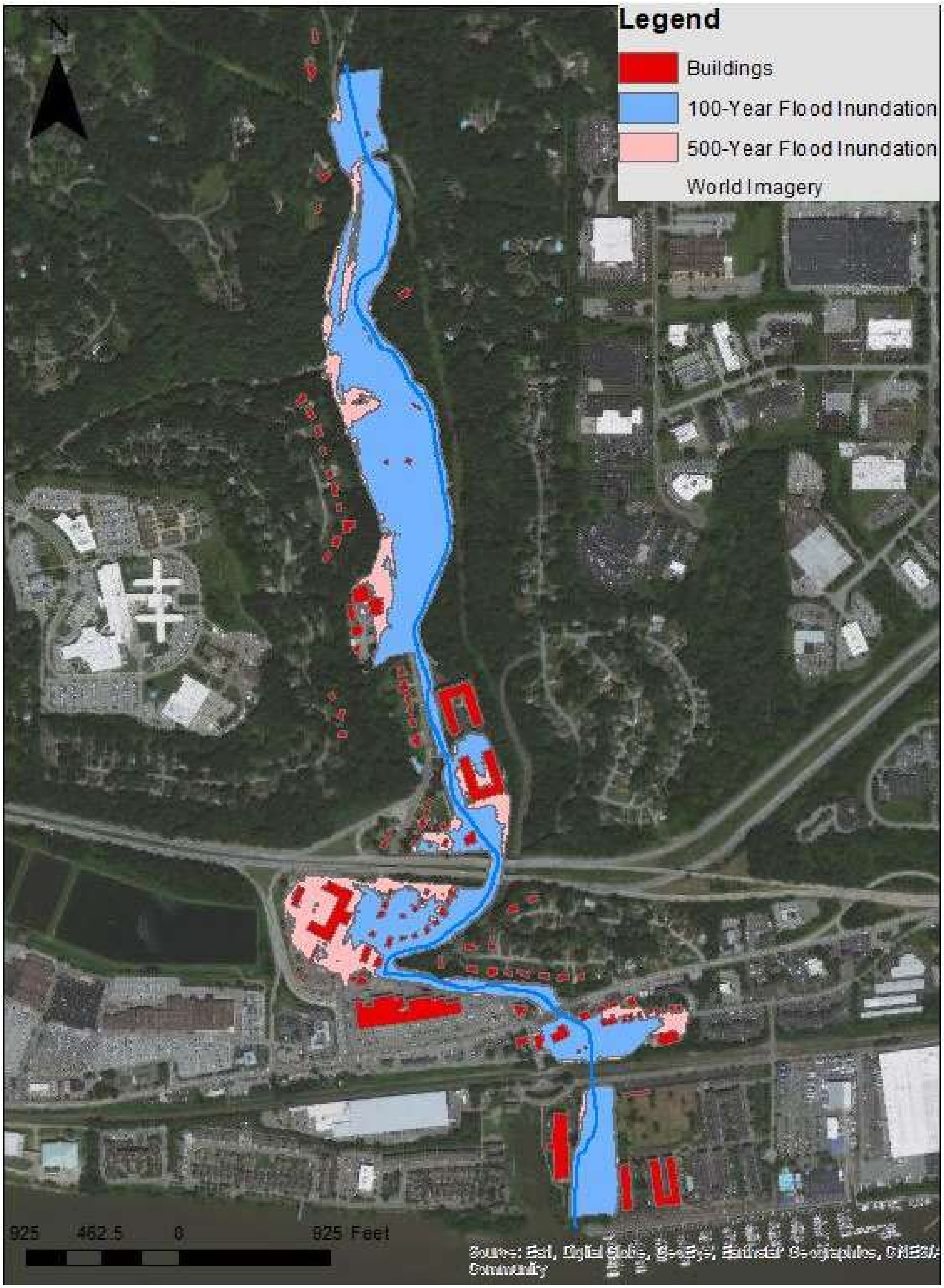
Inundation Map – Stormwater Pond 1



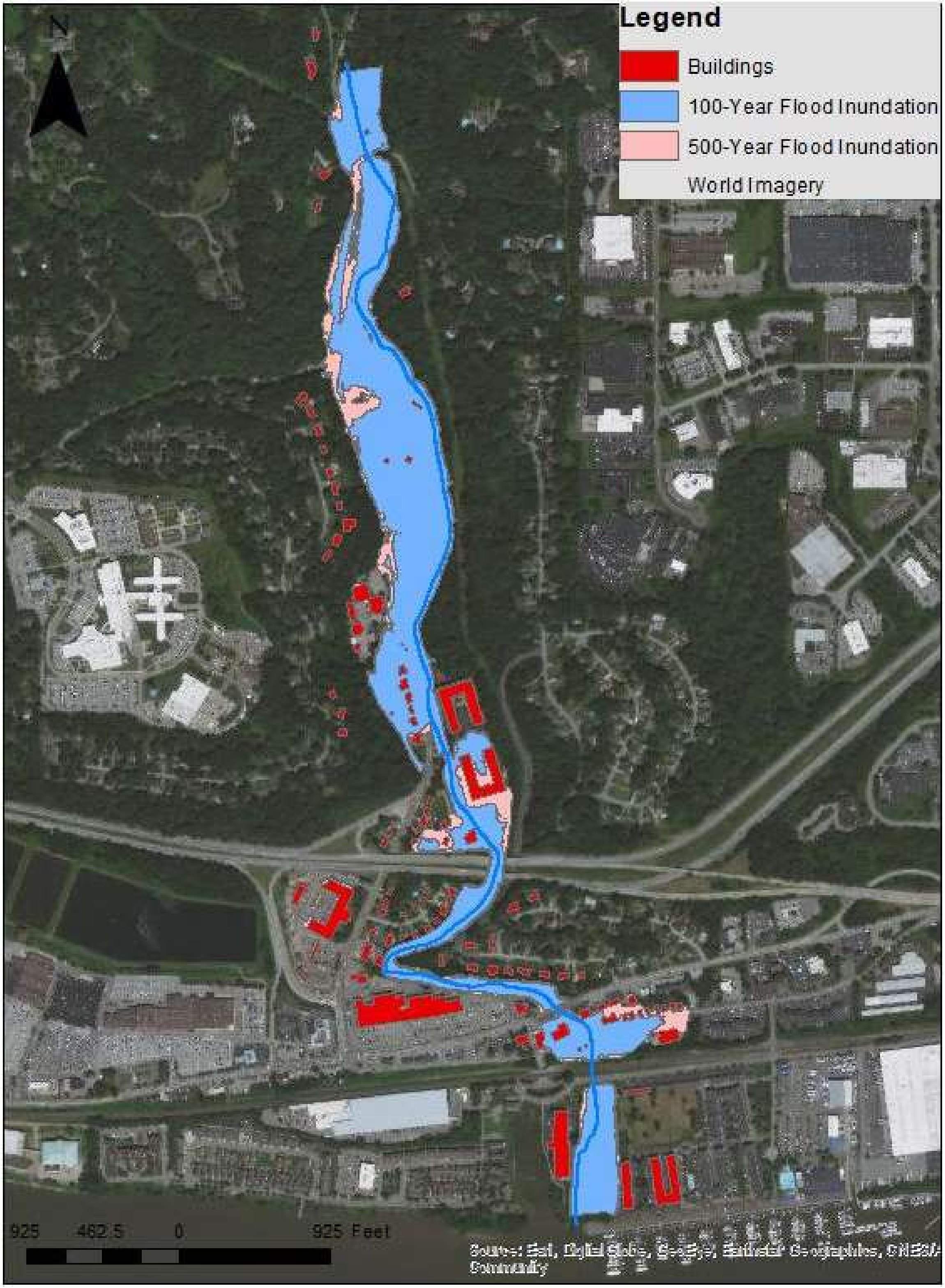
Inundation Map – Stormwater Pond 2



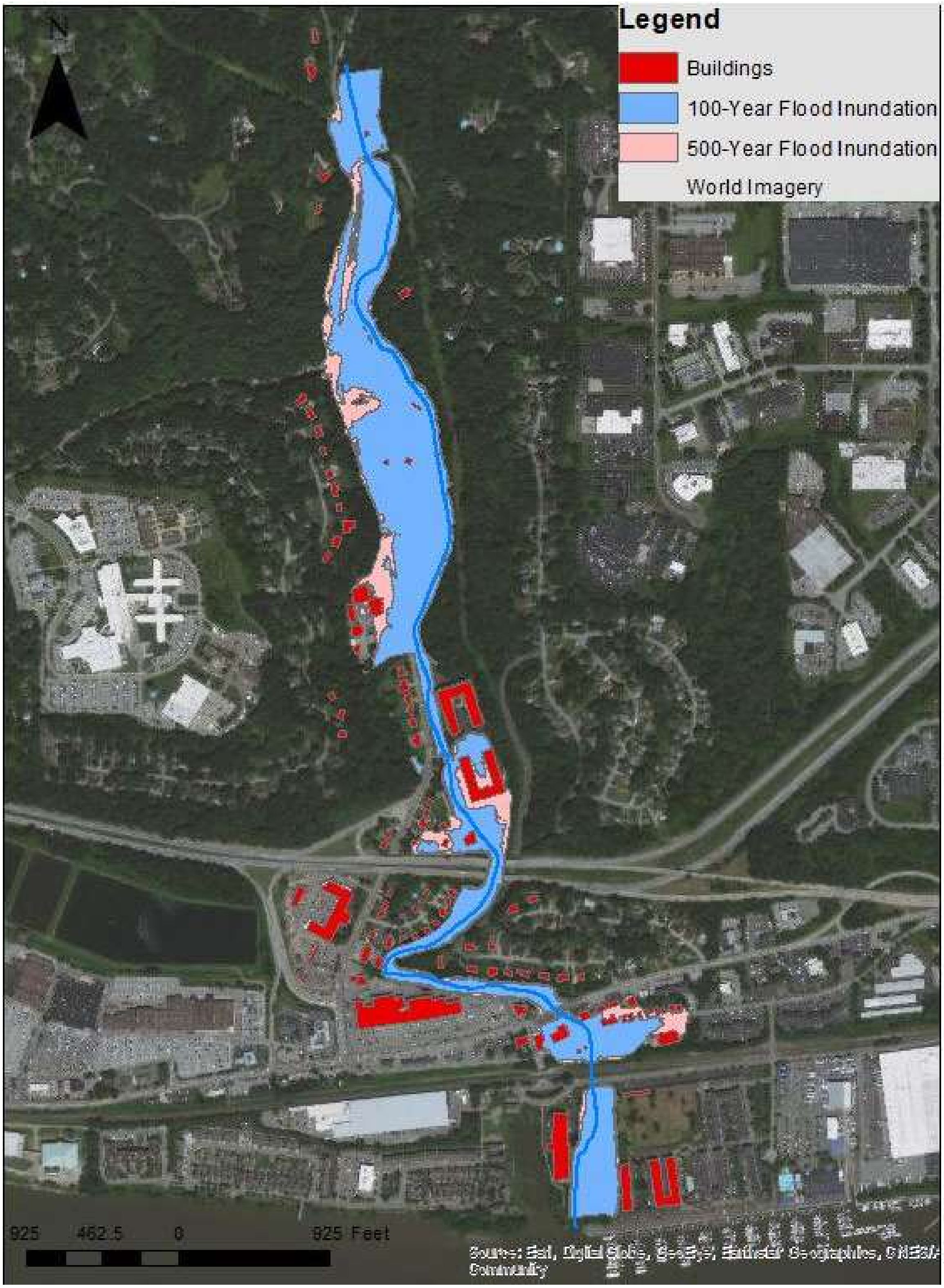
Inundation Map – Stormwater Pond 2 and Floodwall 1



Inundation Map – Stormwater Pond 2 and Floodwall 2

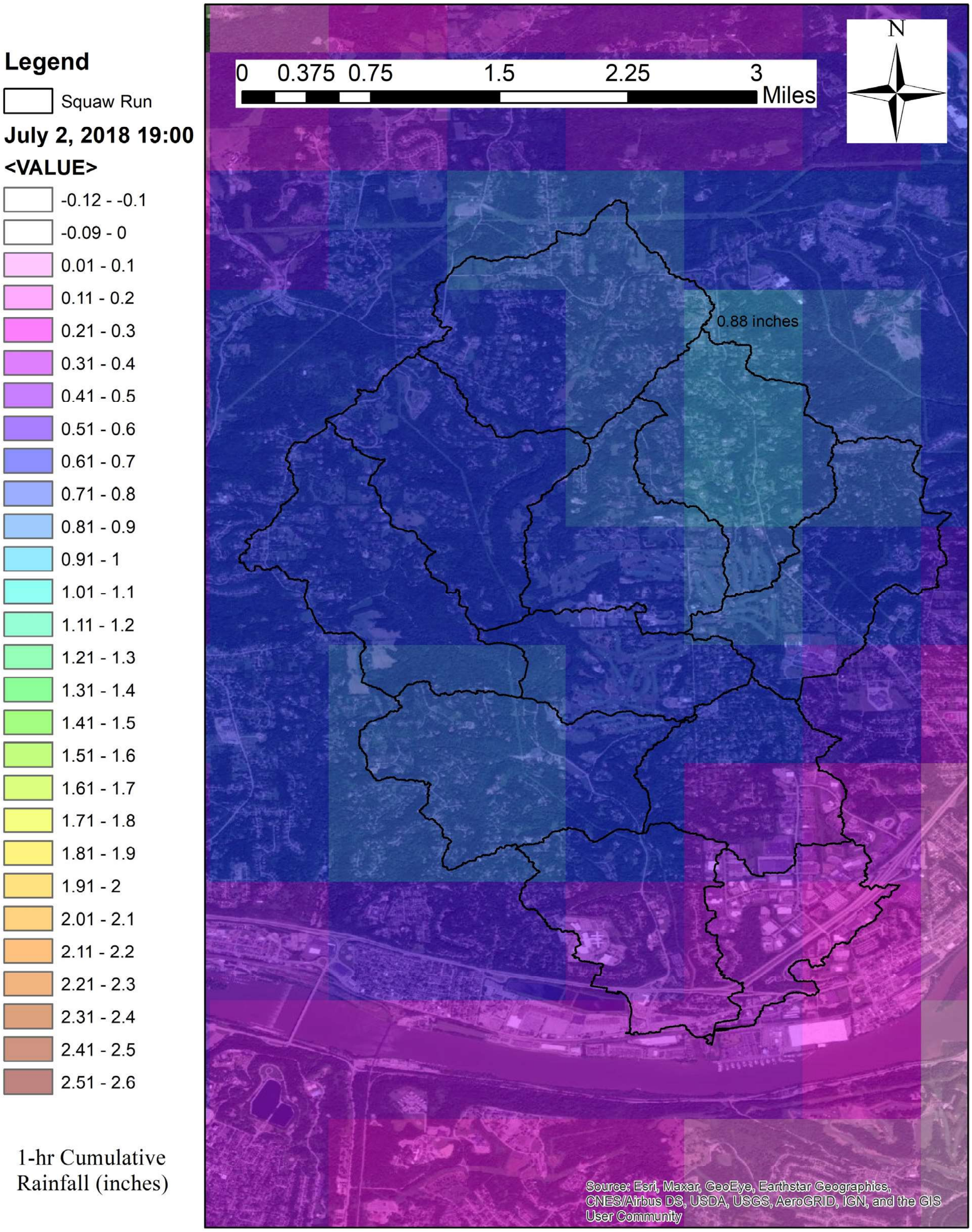


Inundation Map – Stormwater Pond 2 and Floodwalls 1 and 2

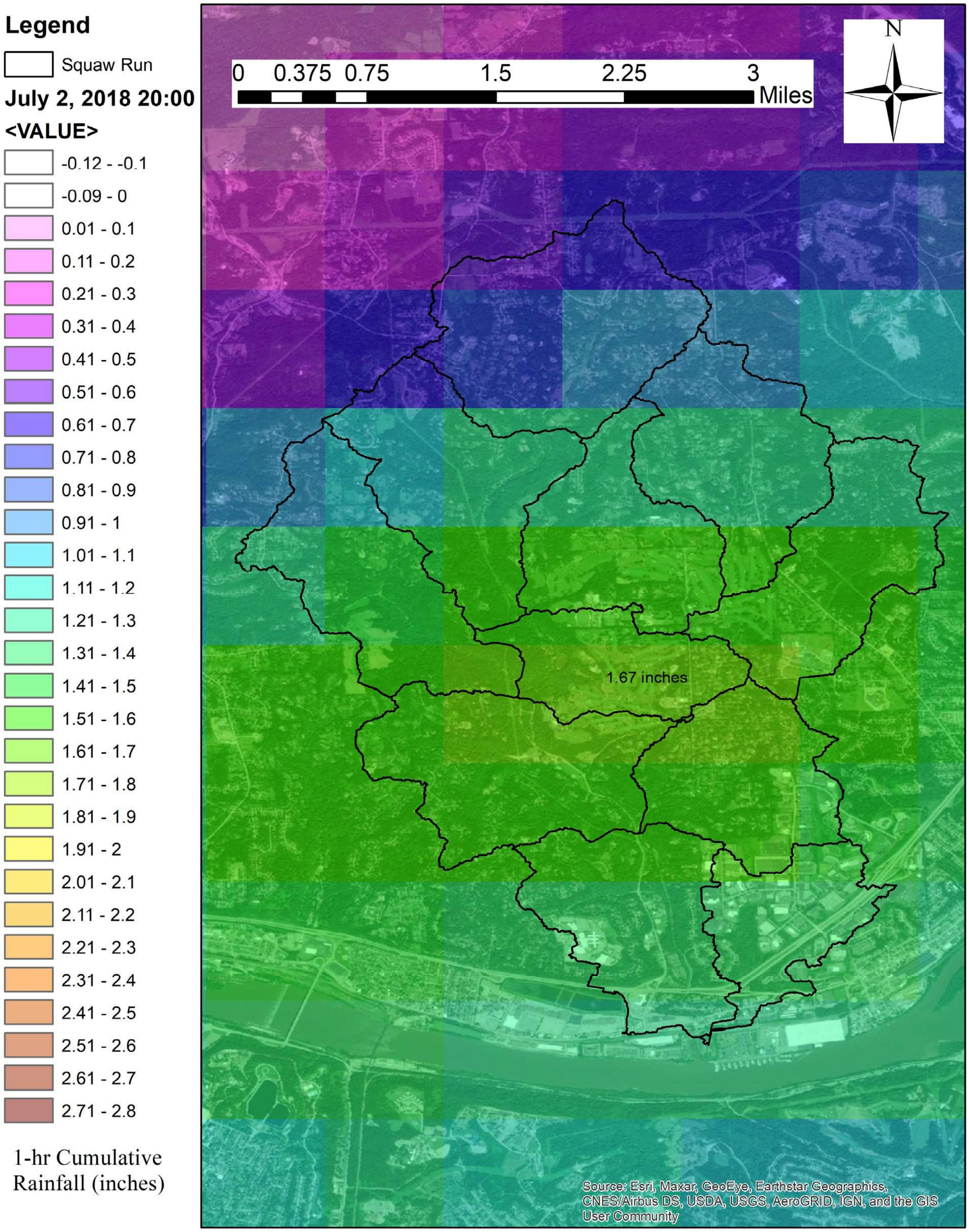


Data shown on the subsequent figures are the MRMS gage-corrected quantitative precipitation estimate (QPE) from July 2, 2018 over Squaw Run. The MRMS quantitative precipitation system currently integrates about 180 operational radars and creates a seamless 3D radar mosaic across the conterminous United States (CONUS) and southern Canada at very high spatial (1 km) and temporal (2 min) resolution. The radar-base data are integrated with atmospheric environmental data, satellite data, and lightning and rain gauge observations to generate a suite of severe weather and QPE products. Figures below utilized the MRMS gage-corrected quantitative precipitation estimate (QPE) product (ID: GaugeCorrQPE01H) with resolution of 1 km and 1 hour (Zhang et al., 2016).

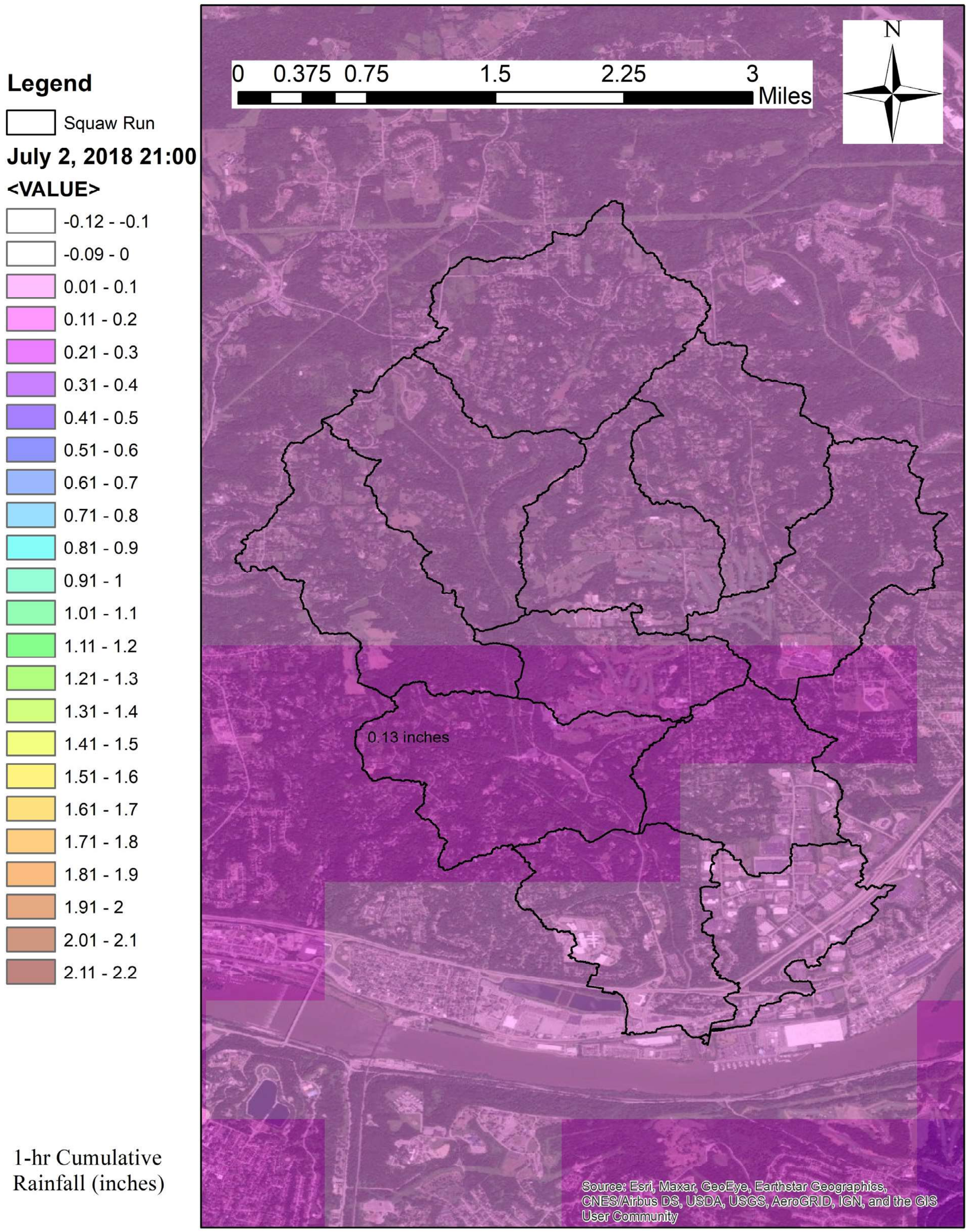
NEXRAD 1-hour Cumulative Rainfall – July 2, 2018 19:00



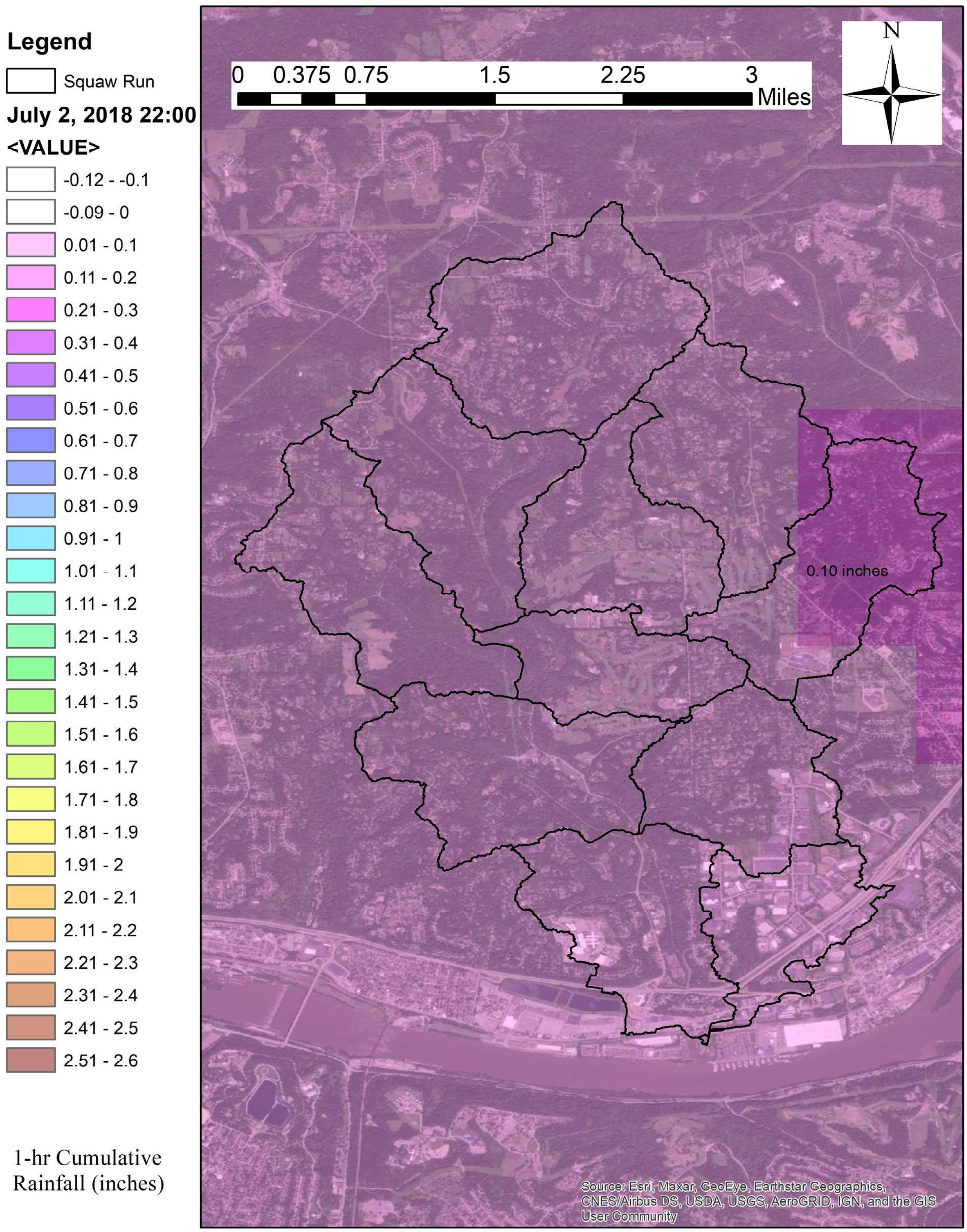
NEXRAD 1-hour Cumulative Rainfall – July 2, 2018 20:00



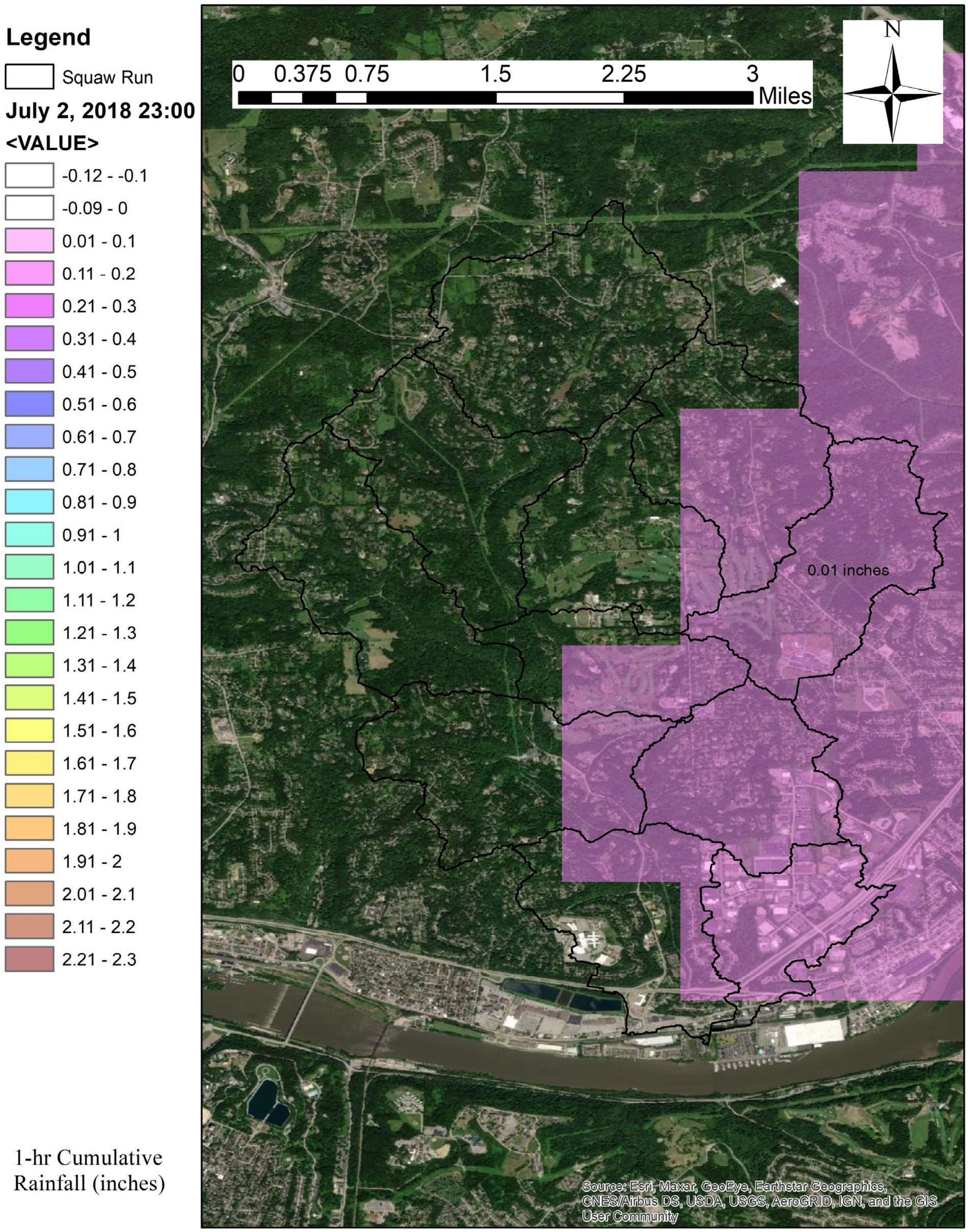
NEXRAD 1-hour Cumulative Rainfall – July 2, 2018 21:00



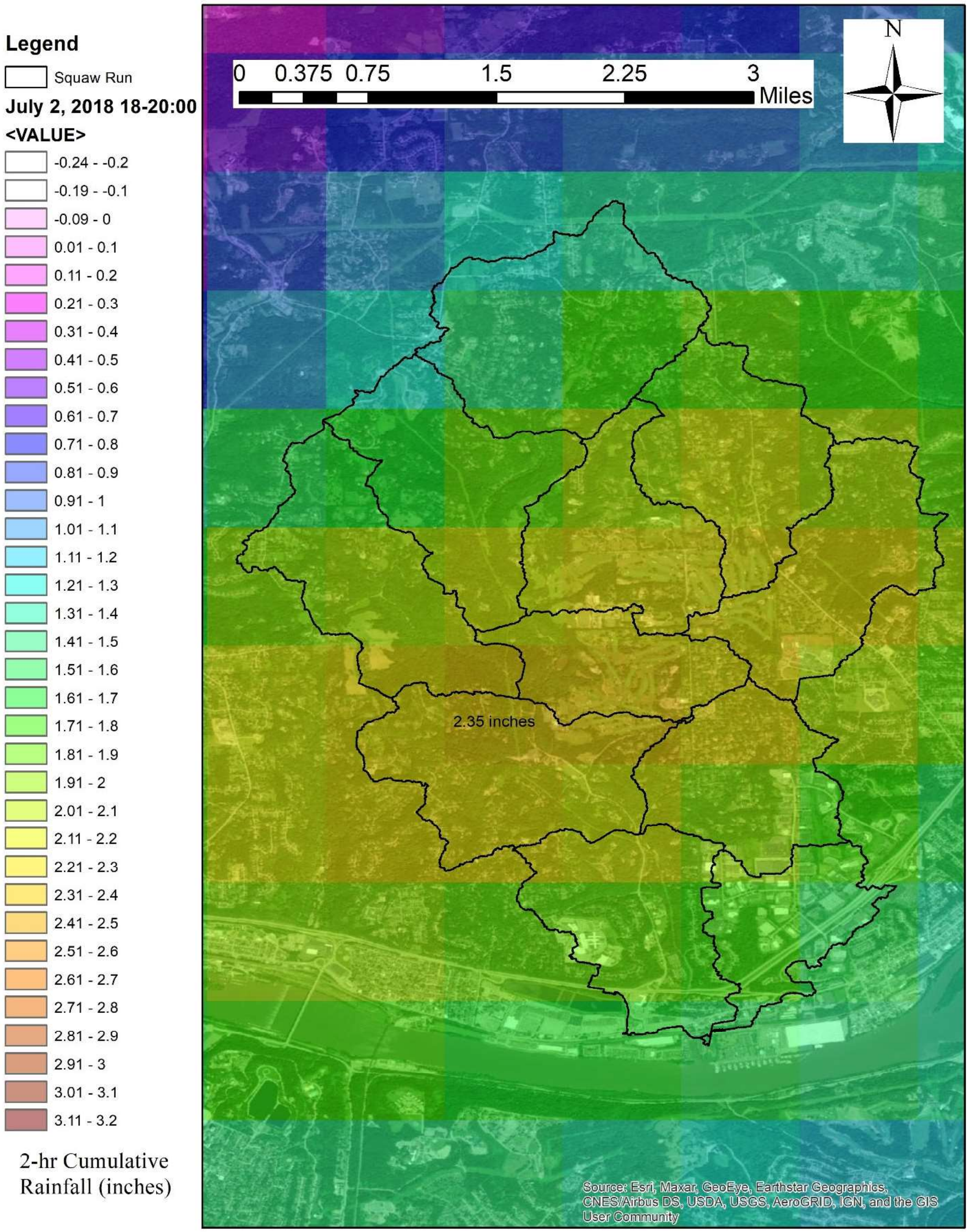
NEXRAD 1-hour Cumulative Rainfall – July 2, 2018 22:00



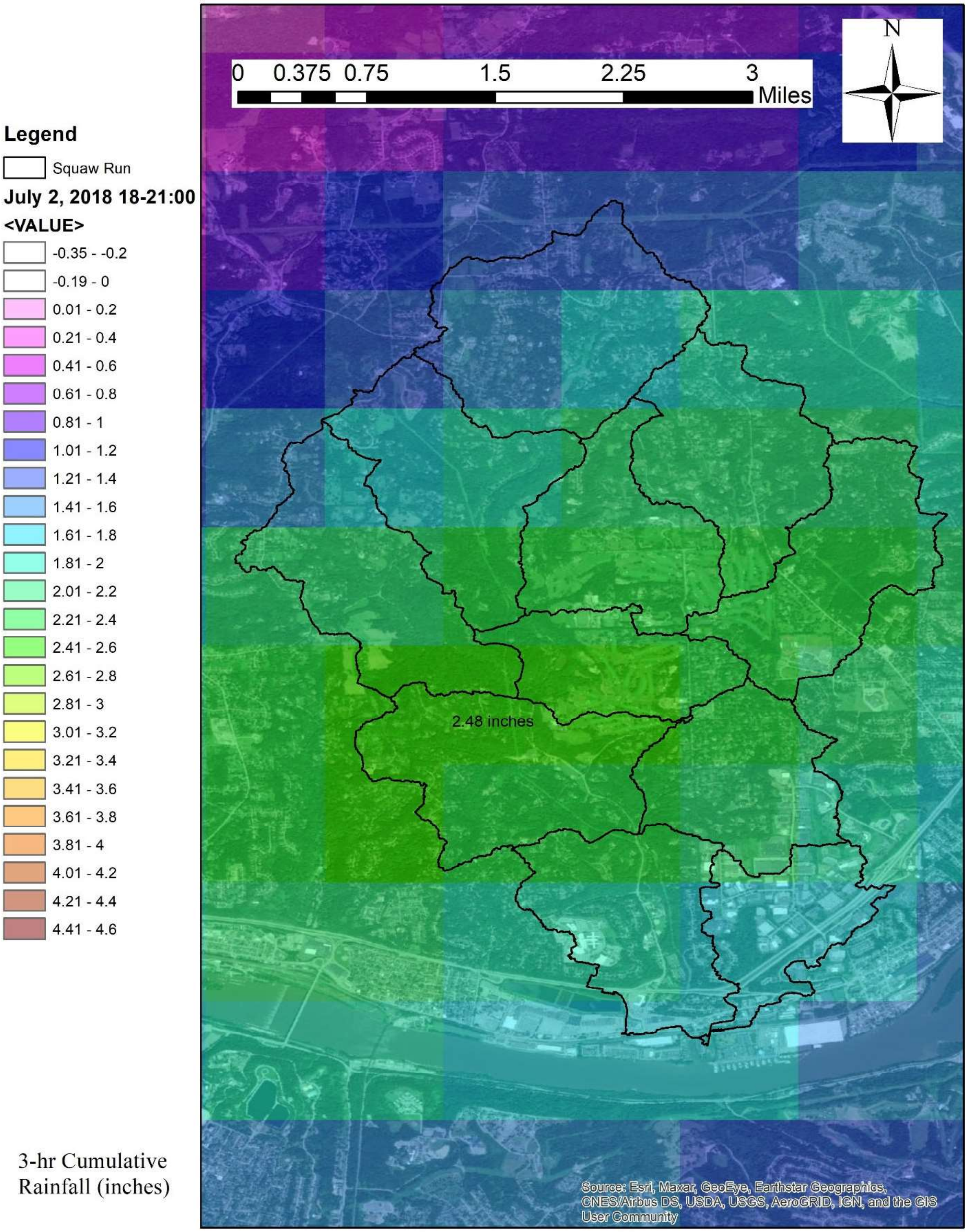
NEXRAD 1-hour Cumulative Rainfall – July 2, 2018 23:00

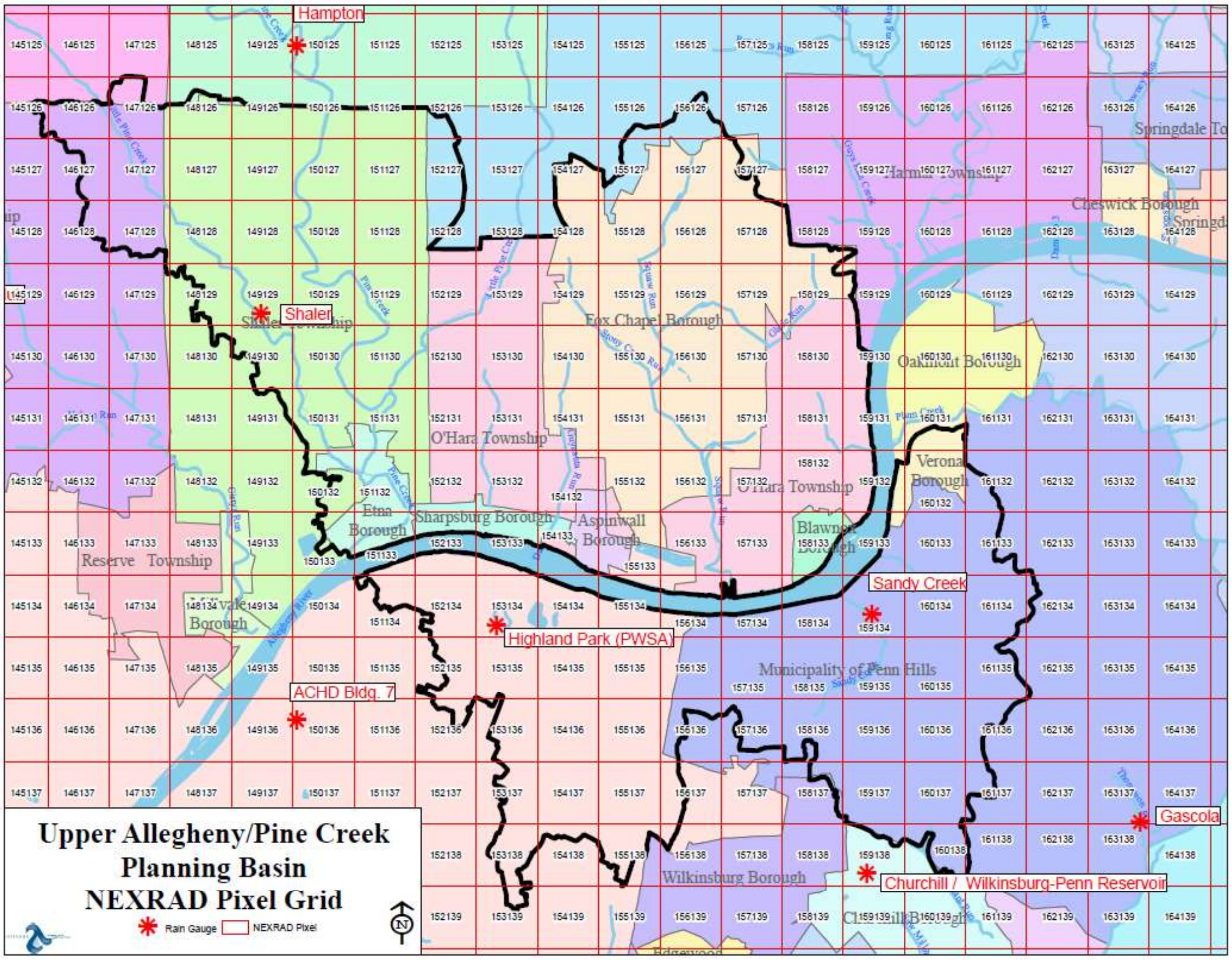


NEXRAD 2-hour Cumulative Rainfall – July 2, 2018 20:00



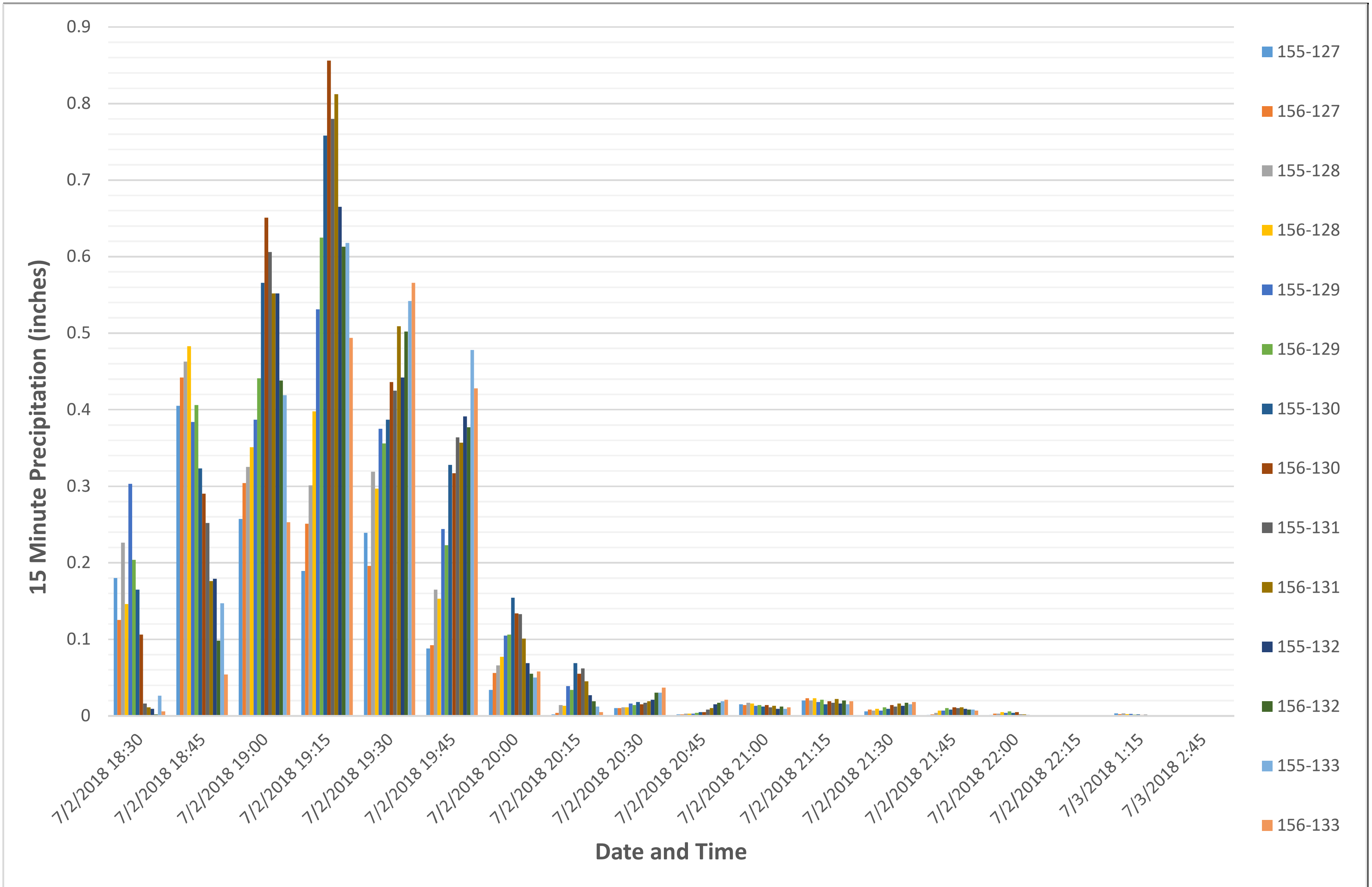
NEXRAD 3-hour Cumulative Rainfall – July 2, 2018 21:00




Upper Allegheny/Pine Creek
Planning Basin
NEXRAD Pixel Grid
Rain Gauge
NEXRAD Pixel
Hampton
Shaler
Highland Park (PWSA)
ACHD Bldg. 7
Sandy Creek
Gascola
Churchill / Wilkinsburg-Penn Reservoir
Shaler Township
Fox Chapel Borough
O'Hara Township
Oakmont Borough
Verona Borough
Etna Borough
Sharpsburg Borough
Aspinwall Borough
Blawnox Borough
Reserve Township
Millvale Borough
Municipality of Penn Hills
Wilkinsburg Borough
Churchill Borough
Cheswick Borough
Harmar Township
Springdale Township
Allegheny River

June 2, 2018 Rainfall Data

Timestamp	155-127	156-127	155-128	156-128	155-129	156-129	155-130	156-130	155-131	156-131	155-132	156-132	155-133	156-133
7/2/2018 18:30	0.18	0.125	0.226	0.146	0.303	0.2039	0.165	0.106	0.016	0.011	0.009	0.002	0.026	0.006
7/2/2018 18:45	0.405	0.442	0.463	0.483	0.384	0.406	0.323	0.29	0.252	0.176	0.179	0.098	0.147	0.054
7/2/2018 19:00	0.257	0.304	0.325	0.351	0.387	0.441	0.5659	0.651	0.606	0.552	0.552	0.438	0.419	0.253
7/2/2018 19:15	0.189	0.251	0.301	0.398	0.531	0.625	0.758	0.856	0.78	0.812	0.665	0.613	0.618	0.494
7/2/2018 19:30	0.239	0.196	0.319	0.297	0.375	0.356	0.387	0.436	0.425	0.509	0.442	0.502	0.542	0.5659
7/2/2018 19:45	0.088	0.092	0.165	0.153	0.244	0.223	0.328	0.317	0.364	0.357	0.391	0.377	0.478	0.428
7/2/2018 20:00	0.034	0.056	0.066	0.077	0.105	0.106	0.154	0.134	0.133	0.101	0.069	0.055	0.05	0.058
7/2/2018 20:15	0.002	0.004	0.014	0.013	0.039	0.034	0.069	0.055	0.062	0.045	0.027	0.019	0.012	0.005
7/2/2018 20:30	0.01	0.01	0.011	0.011	0.016	0.014	0.018	0.015	0.017	0.019	0.021	0.03	0.03	0.037
7/2/2018 20:45	0.002	0.002	0.003	0.003	0.003	0.004	0.005	0.005	0.008	0.01	0.015	0.017	0.019	0.021
7/2/2018 21:00	0.015	0.014	0.017	0.016	0.013	0.014	0.012	0.014	0.011	0.013	0.009	0.012	0.009	0.011
7/2/2018 21:15	0.02	0.023	0.02	0.023	0.018	0.021	0.015	0.019	0.017	0.022	0.016	0.02	0.015	0.019
7/2/2018 21:30	0.006	0.008	0.007	0.009	0.007	0.011	0.009	0.014	0.012	0.016	0.013	0.017	0.015	0.018
7/2/2018 21:45	0.001	0.002	0.004	0.007	0.007	0.01	0.008	0.011	0.01	0.011	0.009	0.008	0.008	0.007
7/2/2018 22:00	0.001	0.003	0.003	0.005	0.004	0.006	0.004	0.005	0.002	0.002	0.001	0	0	0
7/2/2018 22:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7/3/2018 1:15	0.0031	0.0023	0.0031	0.0022	0.0027	0.0017	0.002	0.0011	0.0013	0.0007	0.0008	0.0004	0.0005	0.0002
7/3/2018 2:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	1.4521	1.5343	1.9471	1.9942	2.4387	2.4766	2.8229	2.9291	2.7163	2.6567	2.4188	2.2084	2.3885	1.9771



15 Minute Precipitation (inches)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
7/2/2018 18:30
7/2/2018 18:45
7/2/2018 19:00
7/2/2018 19:15
7/2/2018 19:30
7/2/2018 19:45
7/2/2018 20:00
7/2/2018 20:15
7/2/2018 20:30
7/2/2018 20:45
7/2/2018 21:00
7/2/2018 21:15
7/2/2018 21:30
7/2/2018 21:45
7/2/2018 22:00
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Date and Time
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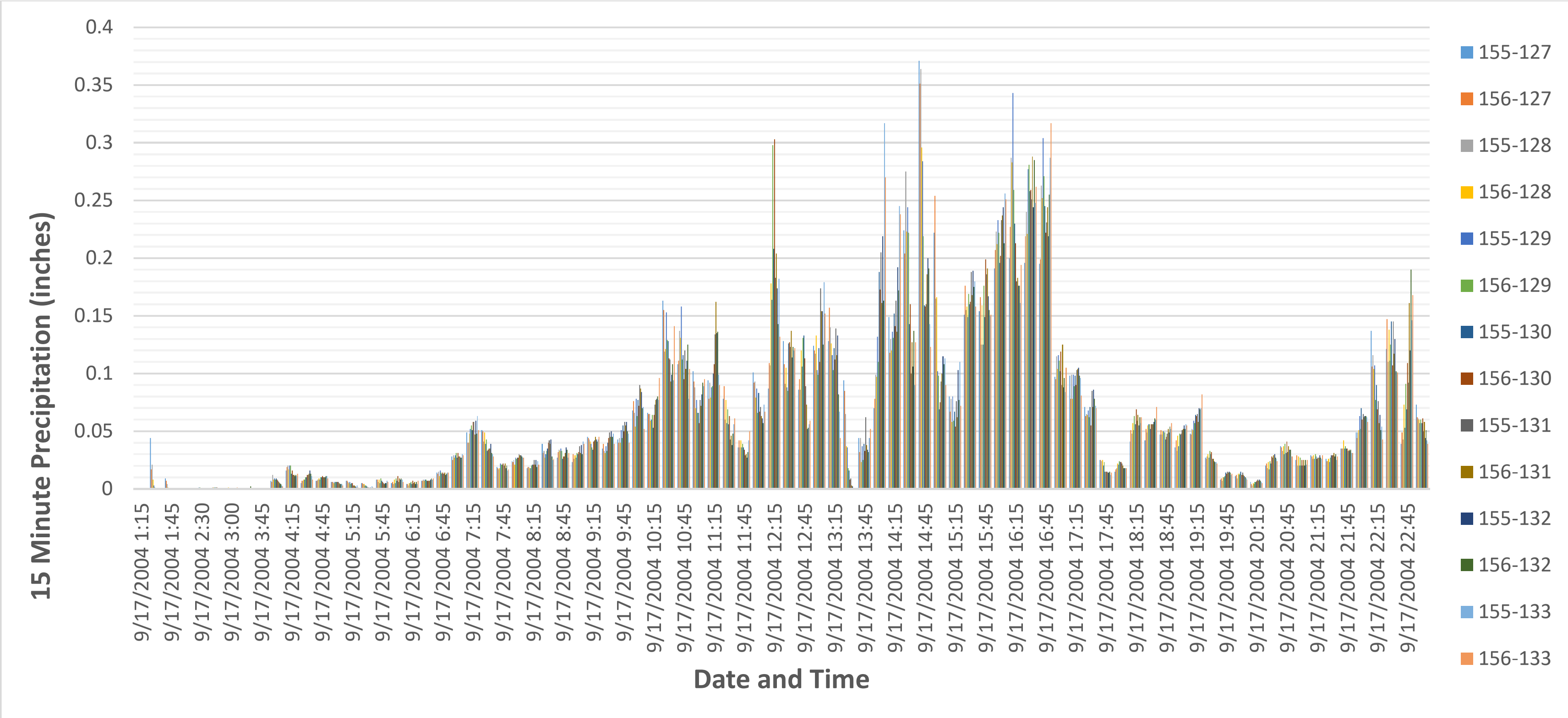
Hurricane Ivan Rainfall – September 17, 2014

Timestamp	155-127	156-127	155-128	156-128	155-129	156-129	155-130	156-130	155-131	156-131	155-132	156-132	155-133	156-133
9/17/2004 1:15	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/17/2004 1:30	0.044	0.017	0.021	0.008	0.003	0.001	0	0	0	0	0	0	0	0
9/17/2004 1:45	0.009	0.007	0.004	0.001	0	0	0	0	0	0	0	0	0	0
9/17/2004 2:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/17/2004 2:30	0	0	0	0	0.001	0.001	0	0	0	0	0	0	0	0
9/17/2004 2:45	0	0	0.001	0.001	0.001	0.001	0.001	0.001	0	0	0	0	0	0
9/17/2004 3:00	0	0	0	0.001	0	0	0	0	0	0	0	0	0	0.001
9/17/2004 3:15	0	0	0	0	0	0	0	0	0	0	0	0.002	0	0
9/17/2004 3:45	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9/17/2004 4:00	0.007	0.006	0.012	0.008	0.009	0.008	0.009	0.008	0.007	0.006	0.005	0.004	0.003	0.002
9/17/2004 4:15	0.016	0.019	0.02	0.017	0.02	0.02	0.013	0.016	0.012	0.012	0.011	0.012	0.011	0.013
9/17/2004 4:30	0.005	0.007	0.007	0.008	0.008	0.01	0.01	0.012	0.012	0.013	0.016	0.013	0.011	0.008
9/17/2004 4:45	0.007	0.007	0.008	0.009	0.008	0.01	0.01	0.011	0.01	0.01	0.01	0.011	0.011	0.009
9/17/2004 5:00	0.006	0.006	0.006	0.005	0.006	0.006	0.006	0.006	0.006	0.005	0.004	0.004	0.004	0.003
9/17/2004 5:15	0.007	0.007	0.006	0.005	0.006	0.004	0.005	0.005	0.003	0.003	0.002	0.001	0.003	0.001
9/17/2004 5:30	0.005	0.005	0.004	0.004	0.003	0.003	0.002	0.001	0.001	0	0.001	0	0.002	0.001
9/17/2004 5:45	0.008	0.008	0.006	0.008	0.005	0.009	0.007	0.006	0.005	0.004	0.005	0.005	0.007	0.006
9/17/2004 6:00	0.005	0.006	0.004	0.008	0.005	0.009	0.007	0.011	0.006	0.009	0.008	0.009	0.008	0.006
9/17/2004 6:15	0.004	0.004	0.003	0.005	0.005	0.006	0.004	0.007	0.005	0.006	0.004	0.006	0.004	0.007
9/17/2004 6:30	0.007	0.007	0.008	0.007	0.008	0.008	0.007	0.007	0.007	0.007	0.008	0.009	0.008	0.009
9/17/2004 6:45	0.014	0.013	0.015	0.011	0.016	0.014	0.013	0.014	0.013	0.014	0.012	0.013	0.014	0.014
9/17/2004 7:00	0.028	0.025	0.029	0.027	0.029	0.031	0.027	0.031	0.028	0.028	0.027	0.027	0.03	0.029
9/17/2004 7:15	0.049	0.04	0.04	0.048	0.052	0.055	0.0509	0.058	0.058	0.047	0.059	0.048	0.063	0.0509
9/17/2004 7:30	0.0509	0.05	0.041	0.049	0.039	0.0429	0.033	0.034	0.035	0.035	0.039	0.031	0.03	0.028
9/17/2004 7:45	0.019	0.018	0.017	0.018	0.022	0.021	0.021	0.022	0.02	0.022	0.018	0.02	0.016	0.017
9/17/2004 8:00	0.024	0.023	0.025	0.021	0.027	0.026	0.027	0.027	0.03	0.029	0.029	0.028	0.027	0.027
9/17/2004 8:15	0.018	0.019	0.019	0.018	0.018	0.02	0.021	0.021	0.025	0.021	0.025	0.019	0.024	0.021
9/17/2004 8:30	0.039	0.031	0.033	0.027	0.03	0.033	0.035	0.04	0.042	0.039	0.0429	0.028	0.025	0.026

9/17/2004 8:45	0.027	0.032	0.034	0.033	0.035	0.033	0.026	0.028	0.028	0.031	0.036	0.034	0.027	0.031
9/17/2004 9:00	0.023	0.03	0.026	0.029	0.027	0.031	0.03	0.032	0.037	0.031	0.038	0.03	0.041	0.039
9/17/2004 9:15	0.045	0.044	0.0429	0.041	0.039	0.036	0.034	0.041	0.042	0.045	0.0429	0.041	0.0429	0.045
9/17/2004 9:30	0.035	0.039	0.033	0.031	0.037	0.033	0.04	0.044	0.049	0.045	0.05	0.045	0.048	0.039
9/17/2004 9:45	0.0429	0.04	0.044	0.04	0.0509	0.044	0.055	0.05	0.058	0.055	0.058	0.05	0.048	0.04
9/17/2004 10:00	0.068	0.076	0.066	0.054	0.078	0.063	0.077	0.072	0.09	0.0869	0.084	0.07	0.0709	0.052
9/17/2004 10:15	0.066	0.065	0.065	0.059	0.06	0.064	0.053	0.064	0.073	0.077	0.078	0.08	0.077	0.096
9/17/2004 10:30	0.163	0.155	0.119	0.121	0.153	0.129	0.128	0.113	0.112	0.093	0.099	0.108	0.094	0.1409
9/17/2004 10:45	0.108	0.111	0.137	0.131	0.158	0.112	0.116	0.095	0.12	0.104	0.111	0.125	0.078	0.104
9/17/2004 11:00	0.1019	0.093	0.088	0.07	0.077	0.066	0.066	0.057	0.077	0.072	0.082	0.092	0.089	0.095
9/17/2004 11:15	0.094	0.078	0.092	0.079	0.088	0.089	0.1	0.108	0.134	0.162	0.135	0.136	0.099	0.09
9/17/2004 11:30	0.078	0.089	0.06	0.077	0.057	0.069	0.056	0.063	0.0429	0.046	0.038	0.048	0.056	0.061
9/17/2004 11:45	0.036	0.042	0.036	0.042	0.036	0.039	0.036	0.034	0.029	0.03	0.027	0.032	0.042	0.05
9/17/2004 12:00	0.101	0.092	0.093	0.079	0.0869	0.066	0.083	0.067	0.0709	0.063	0.061	0.057	0.073	0.067
9/17/2004 12:15	0.0869	0.109	0.107	0.178	0.164	0.298	0.208	0.303	0.183	0.2039	0.1739	0.1429	0.182	0.132
9/17/2004 12:30	0.128	0.108	0.091	0.105	0.088	0.085	0.126	0.127	0.123	0.137	0.114	0.123	0.12	0.122
9/17/2004 12:45	0.0859	0.095	0.0859	0.12	0.106	0.131	0.133	0.113	0.089	0.073	0.052	0.053	0.056	0.059
9/17/2004 13:00	0.124	0.12	0.117	0.133	0.103	0.099	0.122	0.11	0.1739	0.154	0.154	0.125	0.179	0.152
9/17/2004 13:15	0.128	0.157	0.14	0.126	0.116	0.103	0.122	0.112	0.139	0.116	0.133	0.082	0.067	0.044
9/17/2004 13:30	0.094	0.085	0.065	0.037	0.036	0.018	0.016	0.008	0.009	0.003	0.002	0	0	0
9/17/2004 13:45	0.044	0.032	0.044	0.023	0.037	0.025	0.039	0.033	0.062	0.038	0.034	0.032	0.044	0.052
9/17/2004 14:00	0.07	0.078	0.099	0.097	0.132	0.11	0.188	0.1729	0.205	0.161	0.219	0.163	0.317	0.27
9/17/2004 14:15	0.149	0.118	0.13	0.12	0.136	0.131	0.152	0.1409	0.163	0.136	0.192	0.1719	0.245	0.238
9/17/2004 14:30	0.224	0.2039	0.275	0.223	0.244	0.222	0.1429	0.16	0.1	0.127	0.106	0.137	0.09	0.127
9/17/2004 14:45	0.371	0.351	0.364	0.296	0.2839	0.219	0.159	0.158	0.16	0.186	0.2	0.191	0.1429	0.123
9/17/2004 15:00	0.222	0.254	0.165	0.166	0.1019	0.098	0.069	0.075	0.093	0.1	0.115	0.108	0.113	0.09
9/17/2004 15:15	0.08	0.067	0.078	0.058	0.08	0.059	0.067	0.054	0.076	0.062	0.103	0.077	0.11	0.072
9/17/2004 15:30	0.151	0.176	0.155	0.158	0.149	0.169	0.16	0.162	0.188	0.168	0.189	0.175	0.18	0.158
9/17/2004 15:45	0.154	0.166	0.125	0.159	0.125	0.176	0.149	0.199	0.186	0.191	0.167	0.155	0.15	0.151
9/17/2004 16:00	0.191	0.207	0.223	0.212	0.233	0.222	0.196	0.202	0.233	0.237	0.244	0.213	0.256	0.251
9/17/2004 16:15	0.2	0.227	0.287	0.2829	0.343	0.259	0.23	0.213	0.18	0.183	0.176	0.176	0.161	0.194
9/17/2004 16:30	0.196	0.219	0.24	0.221	0.277	0.281	0.258	0.259	0.251	0.288	0.244	0.2849	0.248	0.262

9/17/2004 16:45	0.195	0.199	0.263	0.252	0.304	0.271	0.245	0.222	0.231	0.244	0.219	0.255	0.287	0.317
9/17/2004 17:00	0.097	0.095	0.115	0.104	0.116	0.111	0.1019	0.119	0.09	0.125	0.088	0.096	0.084	0.105
9/17/2004 17:15	0.098	0.078	0.099	0.078	0.099	0.089	0.098	0.09	0.103	0.104	0.105	0.098	0.096	0.081
9/17/2004 17:30	0.0709	0.063	0.065	0.064	0.068	0.062	0.0709	0.055	0.085	0.0709	0.0859	0.078	0.072	0.07
9/17/2004 17:45	0.025	0.025	0.024	0.025	0.02	0.023	0.015	0.015	0.015	0.014	0.014	0.015	0.012	0.014
9/17/2004 18:00	0.015	0.017	0.018	0.022	0.021	0.024	0.023	0.023	0.022	0.02	0.018	0.018	0.018	0.018
9/17/2004 18:15	0.041	0.0509	0.048	0.057	0.0509	0.063	0.056	0.069	0.059	0.064	0.055	0.062	0.057	0.062
9/17/2004 18:30	0.042	0.052	0.049	0.055	0.056	0.056	0.052	0.056	0.056	0.056	0.058	0.061	0.06	0.0709
9/17/2004 18:45	0.047	0.0509	0.047	0.05	0.05	0.049	0.0429	0.045	0.048	0.052	0.049	0.054	0.052	0.057
9/17/2004 19:00	0.036	0.042	0.033	0.046	0.037	0.048	0.047	0.048	0.0509	0.052	0.055	0.052	0.056	0.055
9/17/2004 19:15	0.048	0.047	0.052	0.0509	0.059	0.057	0.064	0.063	0.065	0.058	0.07	0.069	0.07	0.082
9/17/2004 19:30	0.027	0.026	0.028	0.03	0.027	0.033	0.03	0.032	0.026	0.026	0.024	0.023	0.023	0.022
9/17/2004 19:45	0.008	0.009	0.007	0.01	0.01	0.011	0.014	0.013	0.015	0.014	0.015	0.014	0.012	0.013
9/17/2004 20:00	0.011	0.012	0.009	0.013	0.012	0.013	0.015	0.012	0.014	0.012	0.011	0.01	0.009	0.008
9/17/2004 20:15	0.006	0.004	0.003	0.005	0.004	0.006	0.006	0.007	0.008	0.007	0.008	0.007	0.006	0.005
9/17/2004 20:30	0.02	0.022	0.018	0.023	0.017	0.024	0.022	0.028	0.027	0.029	0.03	0.027	0.026	0.024
9/17/2004 20:45	0.037	0.036	0.033	0.037	0.03	0.039	0.031	0.041	0.032	0.037	0.034	0.034	0.028	0.028
9/17/2004 21:00	0.025	0.029	0.02	0.028	0.02	0.027	0.02	0.025	0.02	0.025	0.02	0.025	0.021	0.025
9/17/2004 21:15	0.029	0.028	0.026	0.029	0.026	0.03	0.028	0.026	0.027	0.027	0.029	0.027	0.026	0.029
9/17/2004 21:30	0.026	0.024	0.024	0.026	0.023	0.027	0.029	0.029	0.029	0.031	0.028	0.03	0.025	0.028
9/17/2004 21:45	0.035	0.035	0.035	0.042	0.035	0.037	0.035	0.035	0.033	0.034	0.033	0.034	0.031	0.031
9/17/2004 22:00	0.049	0.044	0.049	0.0509	0.063	0.052	0.07	0.06	0.065	0.062	0.063	0.063	0.062	0.058
9/17/2004 22:15	0.137	0.106	0.116	0.104	0.107	0.077	0.09	0.069	0.076	0.057	0.064	0.0509	0.054	0.0429
9/17/2004 22:30	0.121	0.147	0.11	0.138	0.111	0.125	0.145	0.107	0.145	0.117	0.13	0.1019	0.101	0.101
9/17/2004 22:45	0.039	0.049	0.0429	0.073	0.053	0.091	0.069	0.109	0.092	0.161	0.12	0.19	0.146	0.168
9/17/2004 23:00	0.073	0.062	0.061	0.06	0.057	0.06	0.057	0.061	0.05	0.058	0.044	0.0509	0.042	0.039
TOTAL	5.4174	5.4367	5.4507	5.3877	5.5345	5.4229	5.2225	5.2668	5.3856	5.3717	5.3536	5.1934	5.3237	5.2796


15 Minute Precipitation (inches)
0.4
0.35
0.3
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9/17/2004 1:15
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Date and Time
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