Commonwealth of Pennsylvania
Probable Maximum Precipitation Study

Watershed Analysis and Flood Validation of the July 1942 Smethport Extreme Rainfall Event

Technical Report

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Supporting

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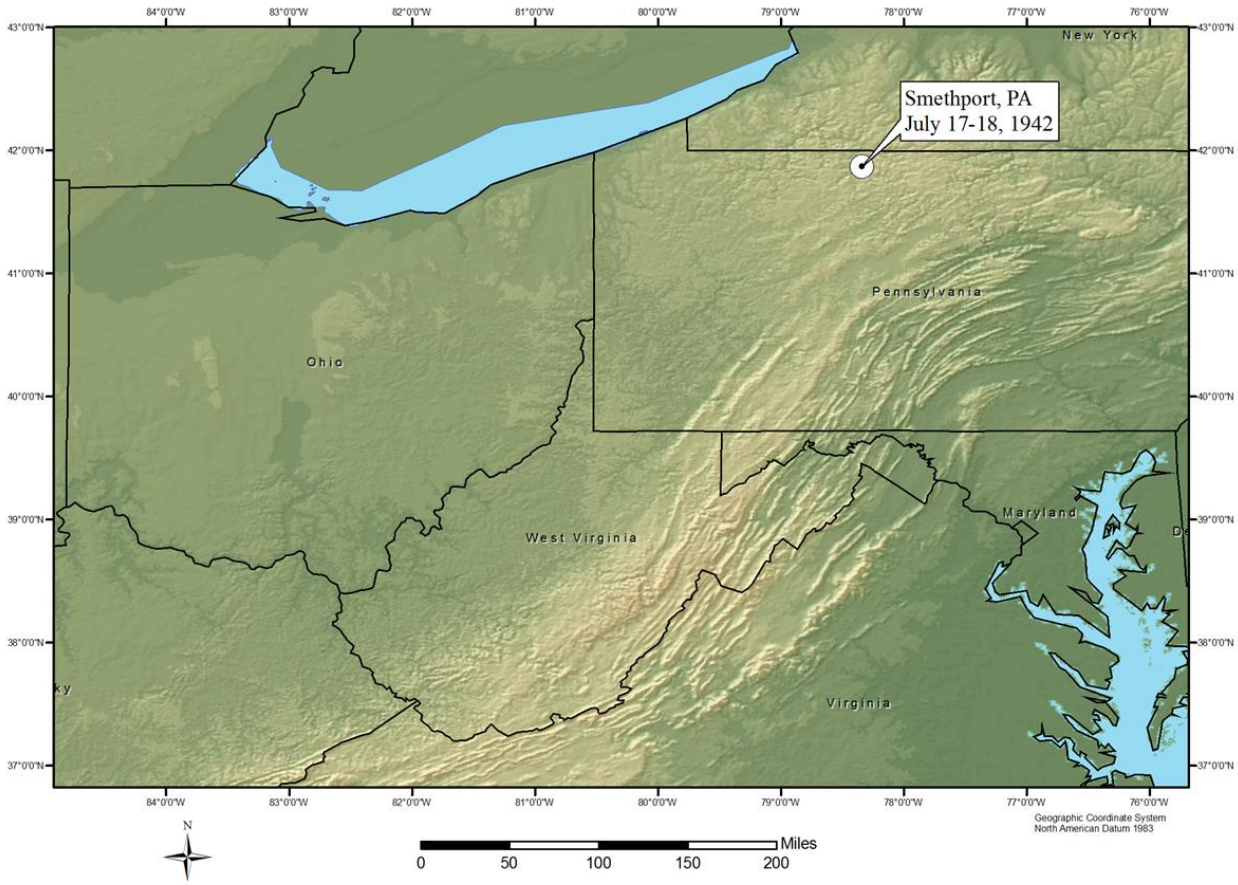
1 Purpose and Background

The Division of Dam Safety, Pennsylvania Department of Environmental Protection (PA DEP), sponsored a Probable Maximum Precipitation (PMP) Study for the Commonwealth of Pennsylvania, led by Applied Weather Associates (AWA). Without an updated study, PMP data are typically obtained from one or more of a series of Hydrometeorological Reports (HMRs) prepared by the National Weather Service (NWS). Areas of the United States east of the 105th meridian are covered by HMR 51 (Schreiner, 1978), which provides generalized depth-area-duration PMP data; with additional generalized temporal and spatial formation in HMR 52 (Hansen, 1982). The outcome of the updated PMP study will enable users in Pennsylvania, many of whom are dam owners, to access site-specific hourly PMP data for areas as small as 1 km^2 to evaluate the impact of the Probable Maximum Flood (PMF) on critical infrastructure (existing or planned), particularly high-hazard dams. The Pennsylvania PMP study uses a storm-based method to transposition and maximize extreme rainfall events in the region to create an envelope of depth-area-duration relationships unique to specific locations in the Commonwealth. Because it is storm-based, PMP depths for Pennsylvania, and much of the larger region covered by HMR 51, are greatly influenced by the exceptional magnitude of a storm that occurred on July 18, 1942 in the region of McKean County (PA), Potter County (PA), and Cattaraugus County (NY). The storm-center occurred in the Smethport/Port Allegany region McKean County, PA. See Figure 1.

Figure 1. Location of July 1942 Storm Center



According to the National Oceanic and Atmospheric Administration (NOAA), the "Smethport" Storm of July 18, 1942, was a world-record setting event for the 3- and 4.5-hour durations at 28.5 and 30.8 inches, respectively (National Oceanic and Atmospheric Administration, 2017). See Figure 2. A

significant number of rainfall observations were reported; however, most were unofficial "bucket surveys" (Eisenlohr, 1952) that have uncertainties in the total reported rainfall and limited temporal information. See Figure 3 for the locations of the hourly gauges in the storm region and Figure 4 for all of the observation points (including bucket surveys) in the study area and vicinity of the storm center. As shown in Figure 5 through Figure 7, the hourly gauges in the areas surrounding the storm center near Smethport and Port Allegany show an initial intense burst of rain near midnight of July 18, 1942 followed by lower intense rainfall then a second significant rainfall period. (Note that midnight of July 18, 1942 corresponds to the end of Index Hour 47 on the hyetographs.) While significant number of total rainfall depths were recorded, including the "bucket surveys", only the scattered hourly gauges shown in Figure 3 and Figure 4 were available to provide temporal information.

Figure 2. Greatest Observed Point Precipitation Values for the World (National Oceanic and Atmospheric Administration, 2017)

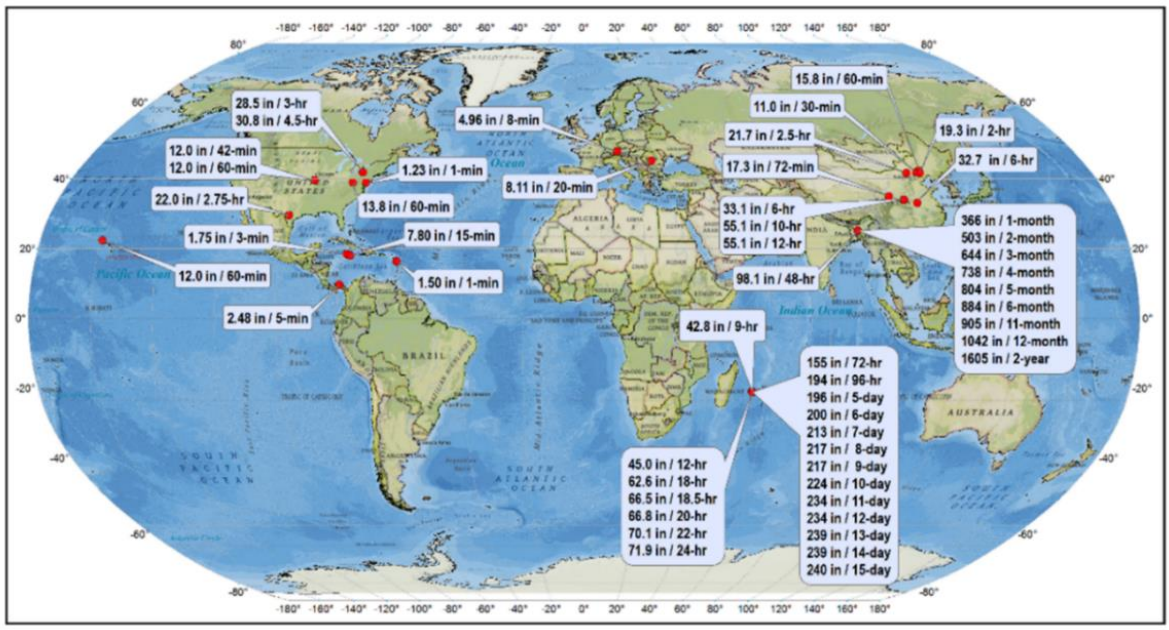


Figure 3. Location of Hourly Rain Gauges in Storm Region New York/Pennsylvania Border

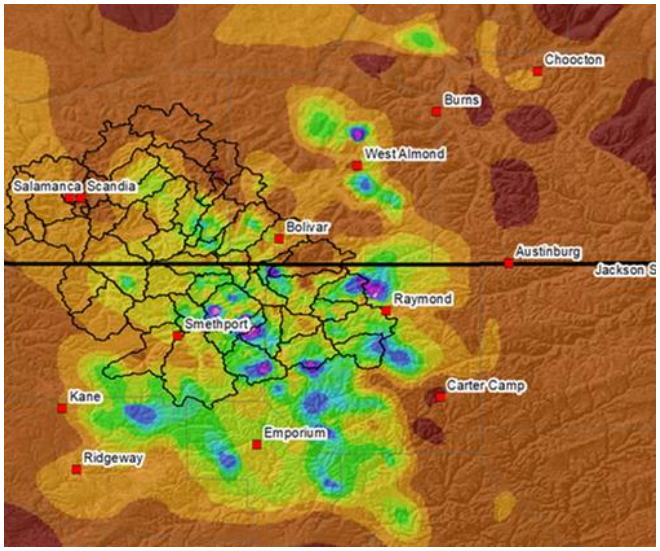


Figure 4. All Rain Gauges in Study Area and Vicinity of Storm Center

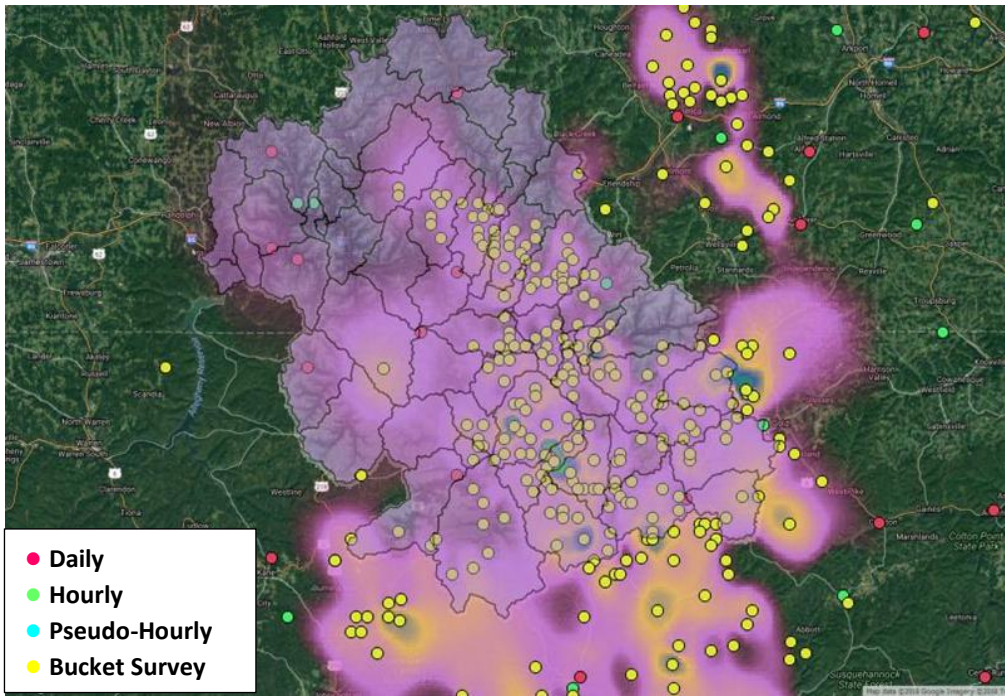


Figure 5. Rainfall Hyetograph at Smethport Hourly Gauge

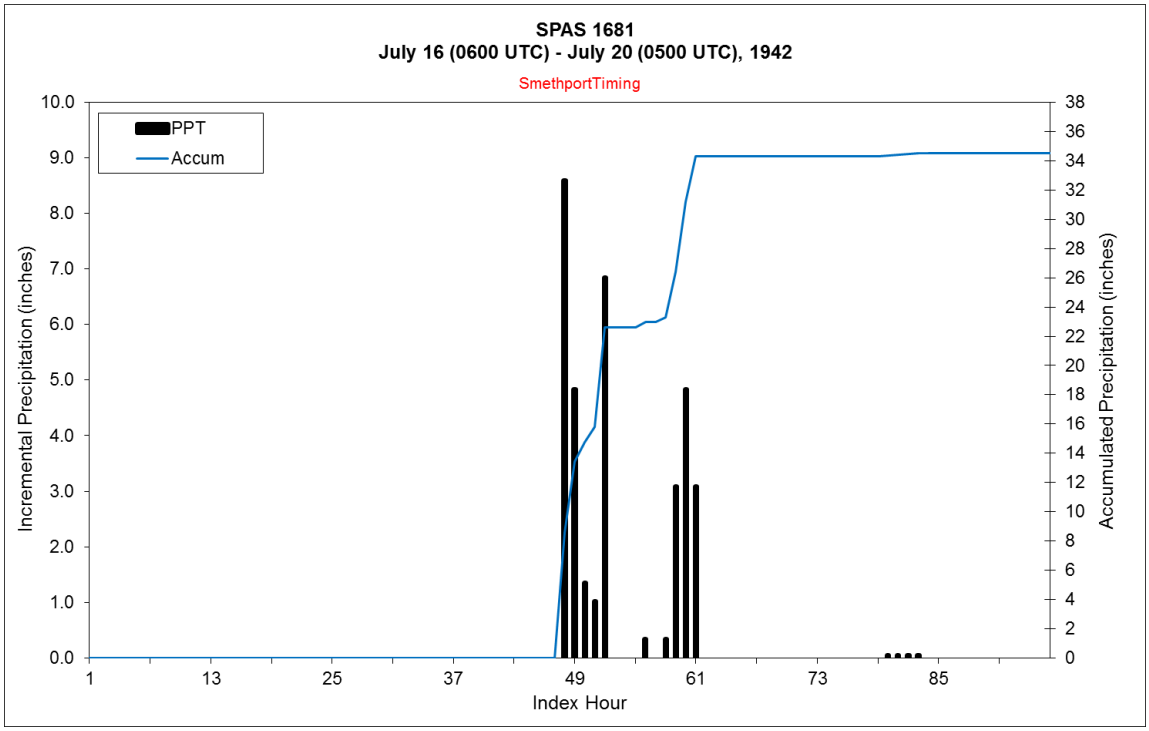


Figure 6. Rainfall Hyetograph at Bolivar Hourly Gauge

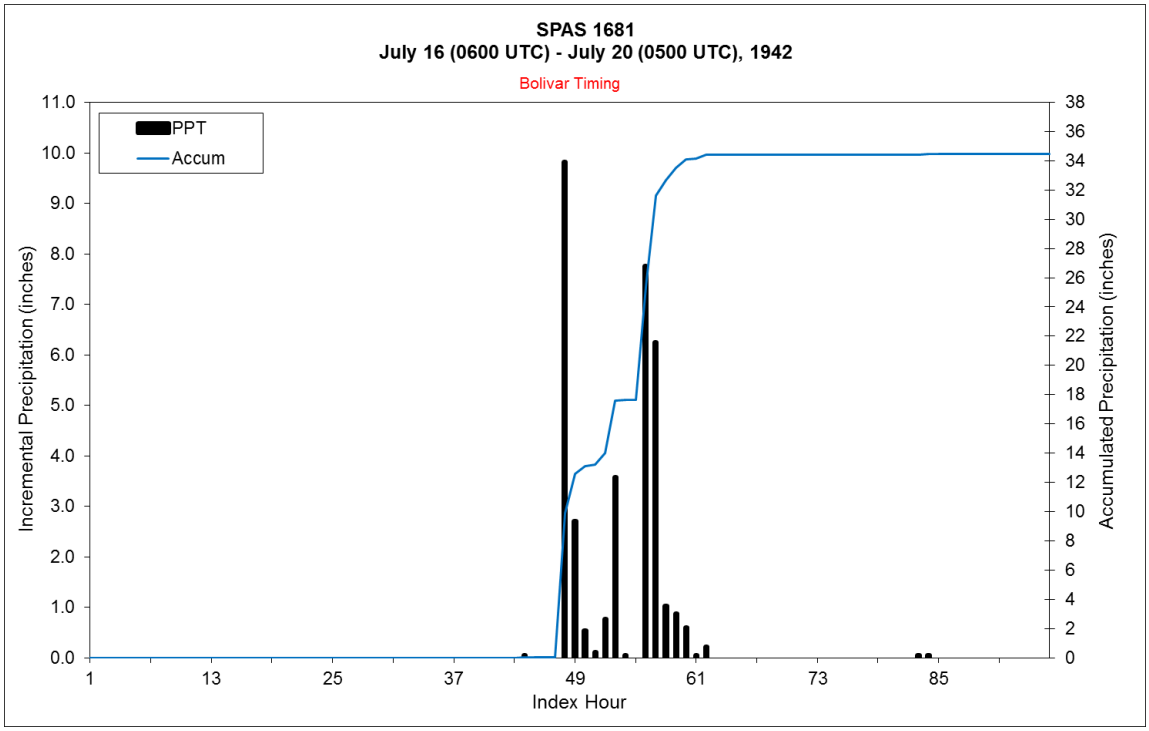
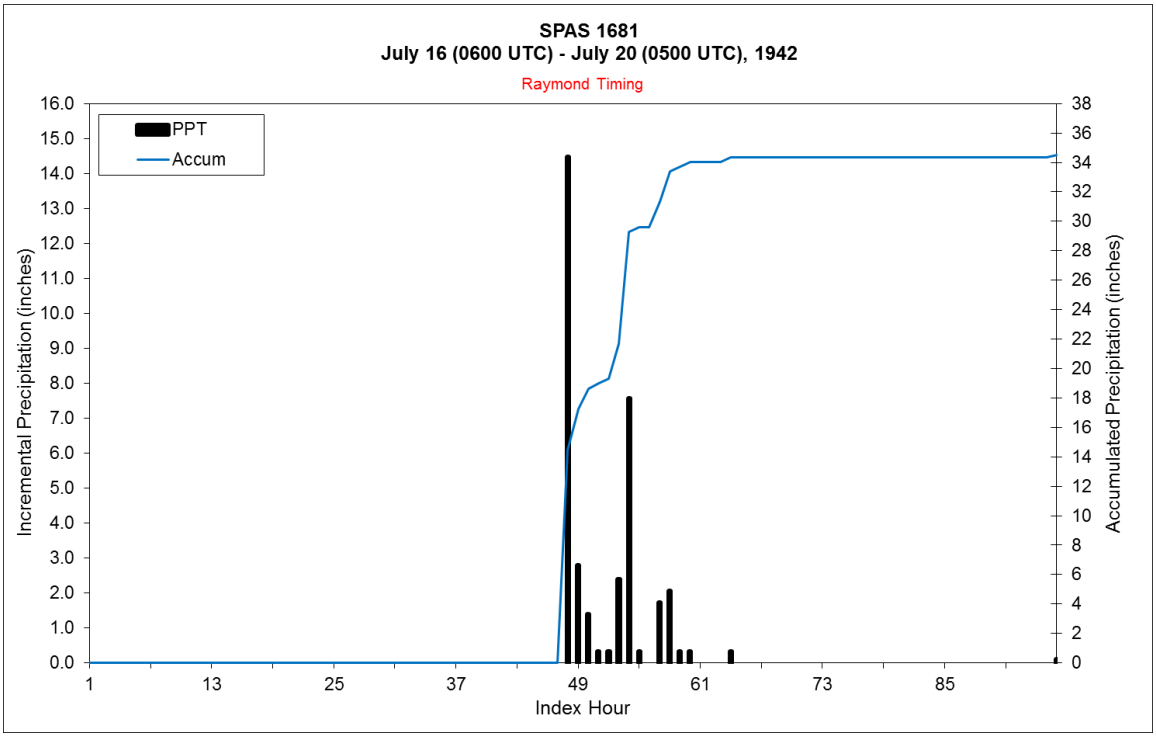


Figure 7. Rainfall Hyetograph at Raymond Hourly Gauge



The focus of this study was on the characteristics of the July 1942 storm and the flood analysis that provided additional insights on the storm's rainfall accumulation patterns and magnitude, utilizing the immense amount of rain gauge observational data and post-flood high-water and peak flow measurements. As discussed previously, many of the rainfall observations are from unofficial "bucket survey" sources, which lack spatial coverage and temporal accumulation information, especially at the hourly level. The hydrologic information provides a way to back-calculate many of the of the unknown rainfall accumulation characteristics that are not captured by the rainfall observations, which were analyzed using AWA's Storm Precipitation Analysis System (SPAS). The outcome of the flood analysis was to substantiate the recorded rainfall or identify, isolate, and quantify observational uncertainties in the recorded rainfall and develop rainfall depth, spatial, and/or temporal patterns that better match observed flood data. The quality and accuracy of the rainfall data was not pre-judged; the flood analysis was conducted to be unbiased and reveal areas where improved accuracy to rainfall magnitude, temporal, and/or spatial patterns can be achieved. The ultimate result of this improved rainfall analysis would be a more accurate representation of the July 1942 rainfall in time, space, and magnitude. This would result in a more accurate estimation of PMP depths and PMF analyses.

2 Flood Model

2.1 Domain

The location of the heaviest rainfall (the storm center) during the July 1942 storm is located in the Upper Allegheny River Watershed, just upstream of the Allegheny Reservoir. The heaviest and most intense rainfall occurred over the Borough of Port Allegany, PA. The storm produced the largest discharges on record at several locations in the upper portions of the Allegheny River, Clarion River, and Sinnemahoning Creek watersheds; shown by the Hydrologic Unit Code Level 8 (HUC-8) watershed boundaries on Figure 8. Discharges diminished in the lower reaches of major streams. See peak flow summary in Table 1 and Figure 9 through Figure 14.

The domain of the flood models focused on the drainage area affected by Port Allegany and surrounding areas and is defined by the Allegheny River 1,780 mi^2 watershed at Red House, NY (discontinued gauge number 03011500). See Figure 15. The location of the Red House gauge moved in October 1964 to its current location in Salamanca, NY, with a gauge number 03011020. The current gauge 03011020 maintains the systematic record prior to October 1964. Review of streamflow gauge records in the region indicate that the July 1942 flood was particularly significant for watersheds less than 500 mi^2, approximately corresponding to the Borough of Eldred, PA and USGS gauge number 03010500 along the Upper Allegheny River.

Figure 8. HUC-8 Watersheds

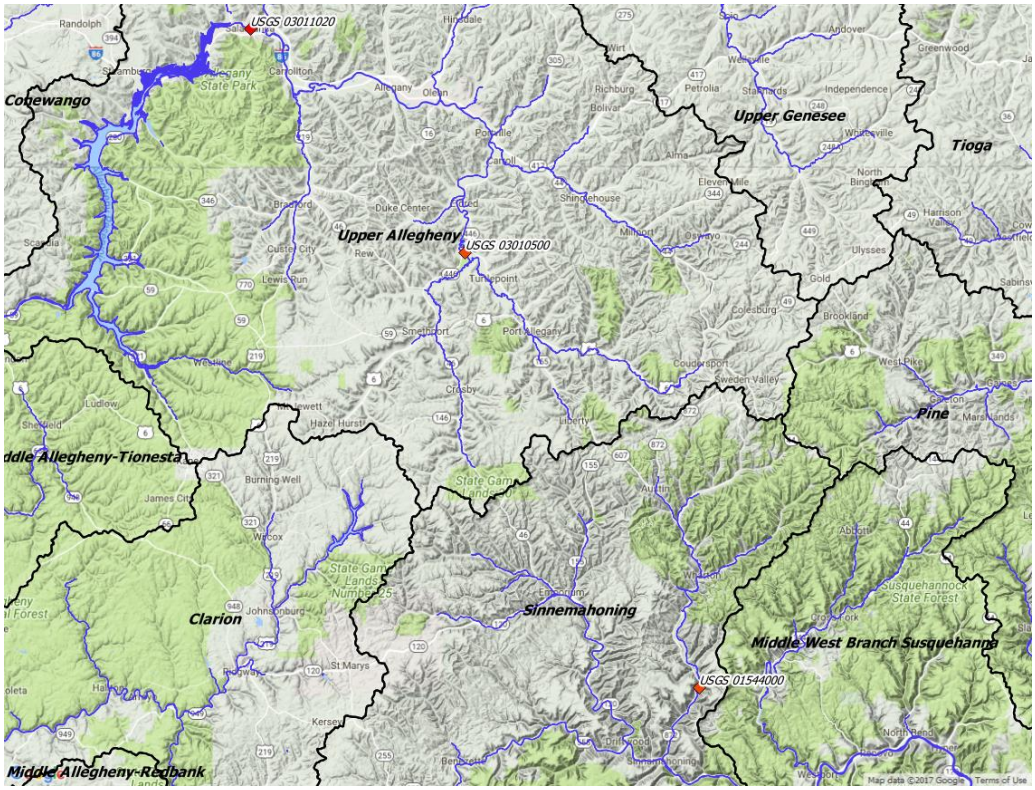


Figure 9. USGS 03011020 Allegheny River at Salamanca, NY (1,608 mi²)

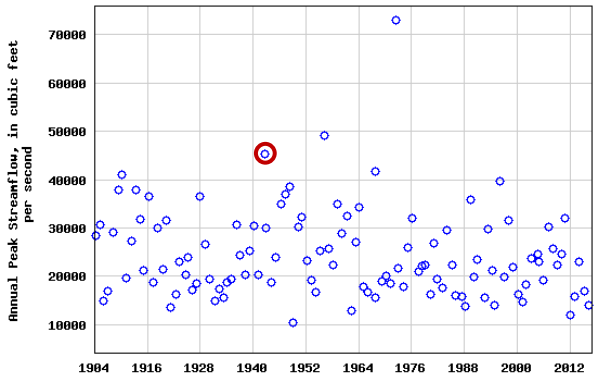


Figure 10. USGS 01543500 Sinnemahoning Creek at Sinnemahoning, PA (685 mi²)

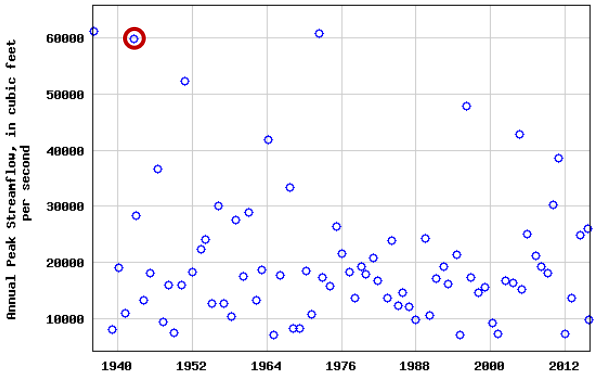


Figure 11. USGS 03010500 Allegheny River at Eldred, PA (550 mi²)

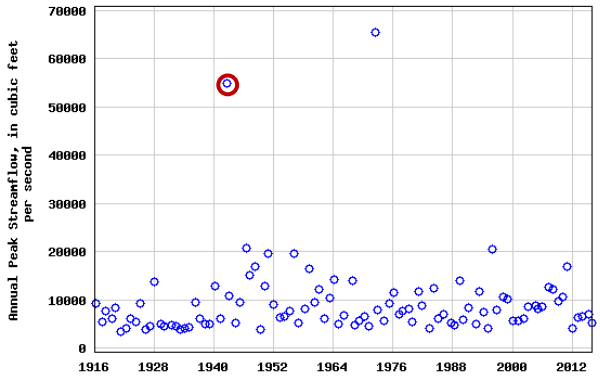


Figure 12. USGS 01543000 Driftwood Bridge Sinnemahoning Creek at Sterling Run, PA (272 mi²)

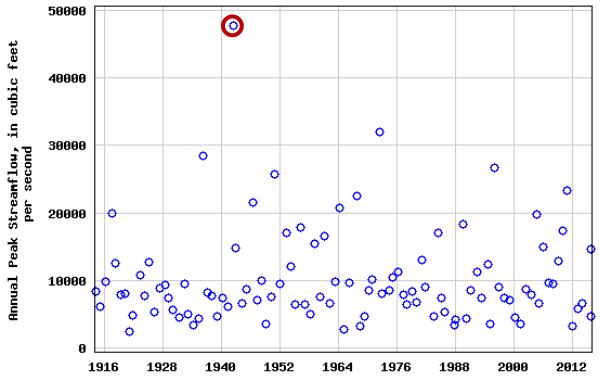


Figure 13. USGS 01544000 1st Fork Sinnemahoning Creek near Sinnemahoning, PA (245 mi²)

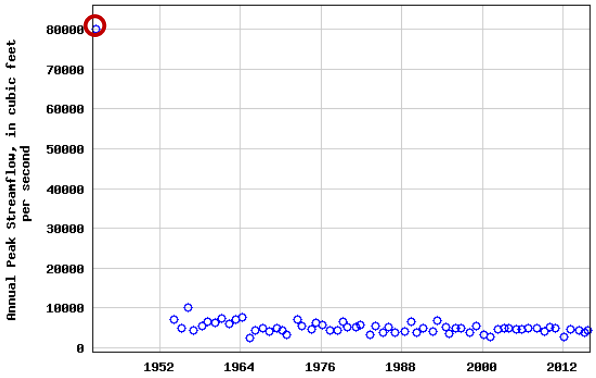


Figure 14. USGS 03007800 Allegheny River at Port Allegany, PA (248 mi²)

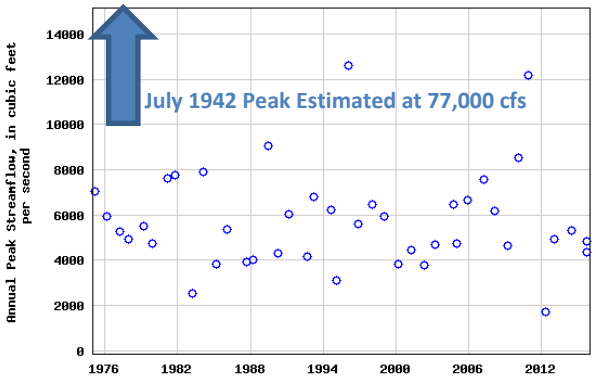
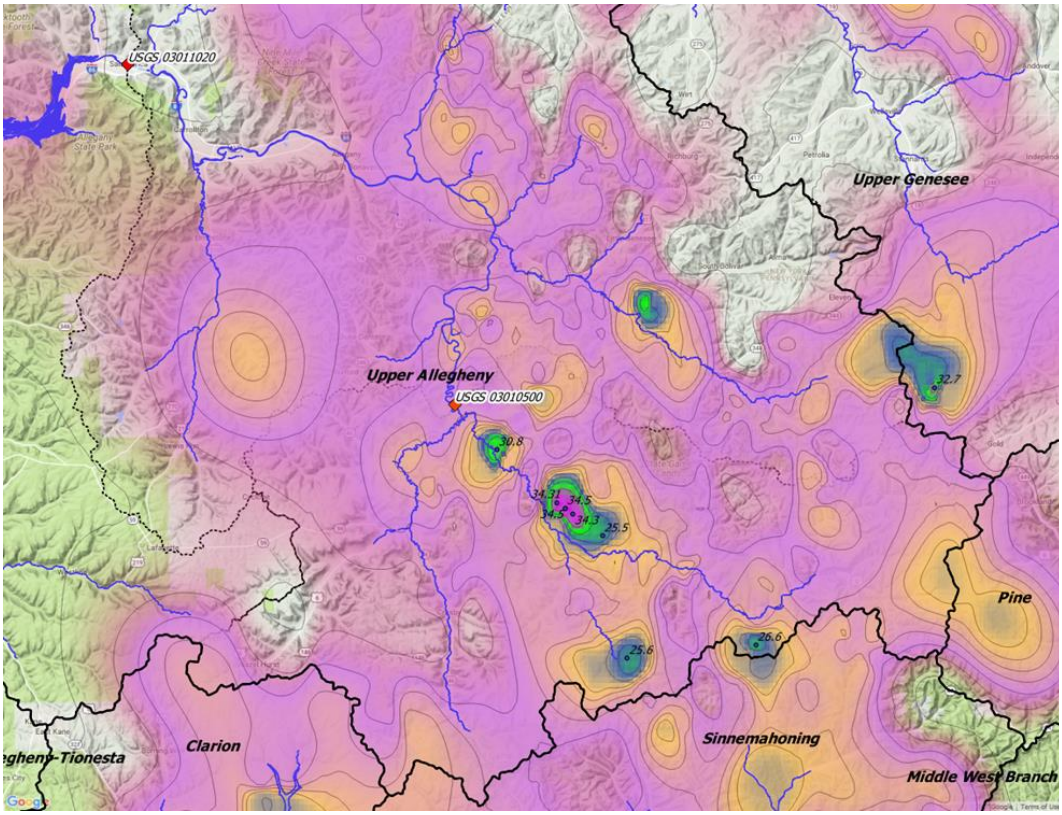


Table 1. Peak Flow Summary

Location	Drainage Area (mi²)	Peak Flow (cfs)	Unit Peak Flow (cfs/mi²)
Port Allegany, PA	251	77,000	307
Eldred, PA	549	55,000	100
Olean, NY	1,167	44,000	38
Red House, NY	1,780	45,300	25

Figure 15. July 1942 Storm Pattern and Flood Model Domain



2.2 Description

The flooding analysis of the 1,780 mi^2 watershed was accomplished using complementary models designed to make optimal use of current computational capacity. The entire study domain, to Red House, NY, was modeled with the USACE's HEC-HMS Version 4.2 software using the Runoff Curve Number (RCN) approach for loss/retention estimation and the Snyder Unit Hydrograph for runoff transformation. As part of the calibration process, the Unit Hydrograph in the HEC-HMS model was adjusted to reconcile the hydrograph from the 2D hydrologic/hydraulic models (discussed further below) and account for a non-linear watershed response in the calibration events. Distributed, 2-dimensional (2D) watershed models were developed for three (3) sub-watersheds within the study domain: Upper Allegheny River watershed Port Allegany, PA (250 mi^2); Oswayo Creek watershed to its confluence with the Allegheny River (248 mi^2); and Tunungwant Creek watershed to its confluence with the Allegheny River (169 mi^2). These are the sub-watersheds, particularly the watershed to Port Allegany, where the most extreme rainfall measurements were recorded. A distributed 2D modeling approach has advantages over conventional lumped and semi-distributed hydrologic models. The distributed 2D modeling approach is more physically-based, making it flexible in modeling hydrologic and hydraulic responses to rainfall events of various magnitudes, intensities, spatial distributions, and temporal distributions. The 2D approach was chosen where the more concentrated rainfall occurred. Another important consideration in using the 2D approach is it reduces concerns over the application of generic non-linearity Unit Hydrograph adjustments in the HEC-HMS model, which introduces an unknown level of inaccuracy. Saghafian (Saghafian, 2006) provides additional discussion regarding non-linearity characteristics of Unit Hydrographs. Mesh sizes were kept relatively small (25 ft to 60 ft, with an average distance between the mesh nodes of 46 ft) to maintain accuracy, particularly to limit artificial

retention of runoff in the watershed. This mesh size limitation made the 2D model computationally impractical for the entire 1,780 mi^2 watershed.

The computer software chosen to provide the distributed 2D watershed simulation was RiverFlow2D, developed by Hydronia, LLC. As stated in the Reference Manual, RiverFlow2D is a "combined hydrologic and hydraulic, mobile bed and pollutant transport finite-volume model for rivers, estuaries and floodplains. The model can integrate hydraulic structures such as culverts, weirs, bridges, gates and internal rating tables. The hydrologic capabilities include spatially distributed rainfall, evaporation, and infiltration." RiverFlow2D solves the shallow water equations (depth averaged/vertical integration of the Navier-Stokes equation) using a finite-volume scheme and, therefore, does not rely on the lumped unit hydrograph approach to estimate flow rates over time (hydrographs). Each triangulated mesh element is assigned individual parameters (rather than homogenous parameters for each sub-basin). Bed stresses use Manning friction law; turbulence and energy losses are implicit in the Manning n-value. Hydrologic capabilities include spatially distributed rainfall, evaporation, and infiltration.

Downstream of Port Allegany PA, 2D hydraulic modeling was also performed along the main-stem Allegheny River using USACE HEC-RAS (Version 5.0.5). The HEC-RAS2D model extended upstream along unnamed and named Allegheny River tributaries, including Potato Creek, Cole Creek, Oswayo Creek, Olean Creek, and Tunungwant Creek, to account for the effects of backwater on flood attenuation. Outflow hydrographs from each HEC-HMS sub-watershed were directly linked, via the HEC-HMS DSS file, to the HEC-RAS2D model along external inflow boundaries with one exception; the outflow hydrograph from the Upper Allegheny RiverFlow2D model (at Port Allegany) was a manual input to HEC-RAS2D at the upstream inflow boundary. HEC-HMS parameters, specifically RCN and Snyder Parameters, were adjusted in the Oswayo Creek and Tunungwant Creek watershed models to achieve a good hydrologic match with RiverFlow2D. The HEC-RAS2D model provided the ability to more accurately account for river and floodplain attenuation and flood profile data for comparison with high-water observations. See Figure 16 and Figure 17 for an illustration on how the HEC-HMS and 2D models relate to cover the watershed.

Figure 18 and Figure 19 show the land use and hydrologic soil groups in the model domain, respectively. Note the apparent discrepancy in hydrologic soil groups (HSG) between PA and NY shown in Figure 19. This is addressed further in Section 2.4.1. Table 2 summarizes the input data collected for the models.

Figure 16. HEC-HMS Model Schematic

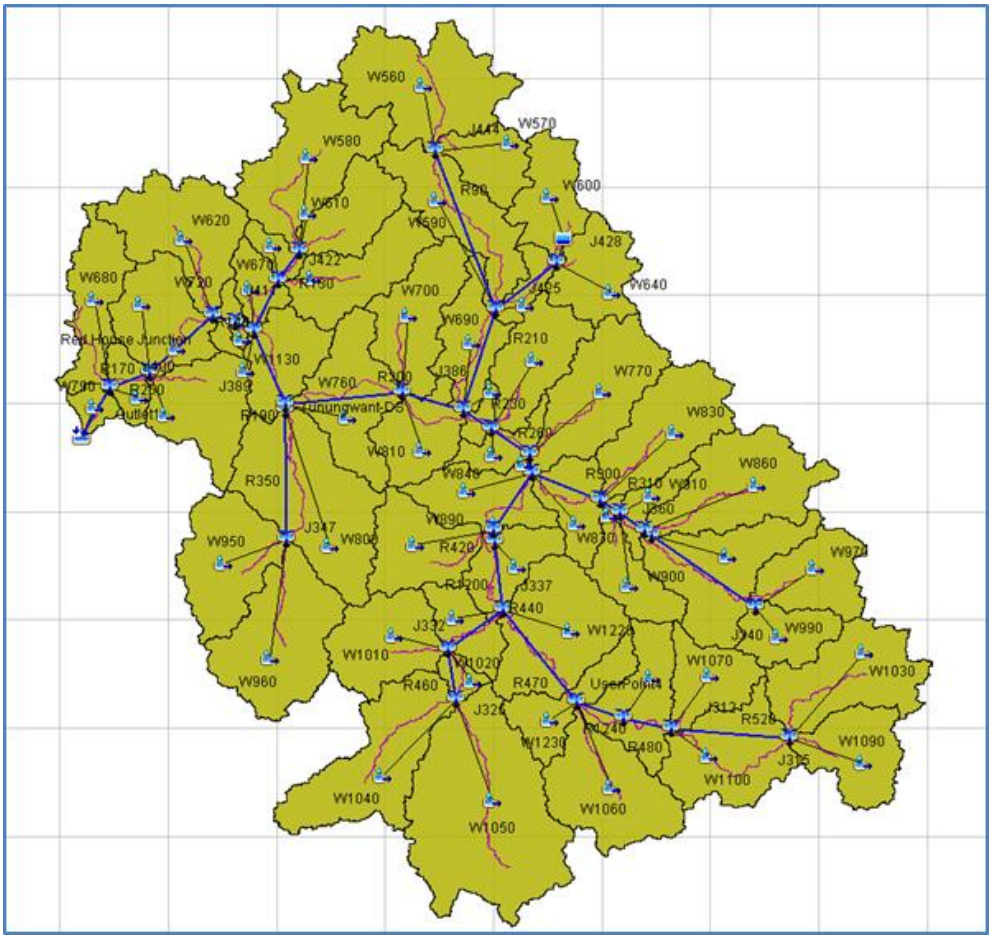


Figure 17. Areas Covered by 2D Models

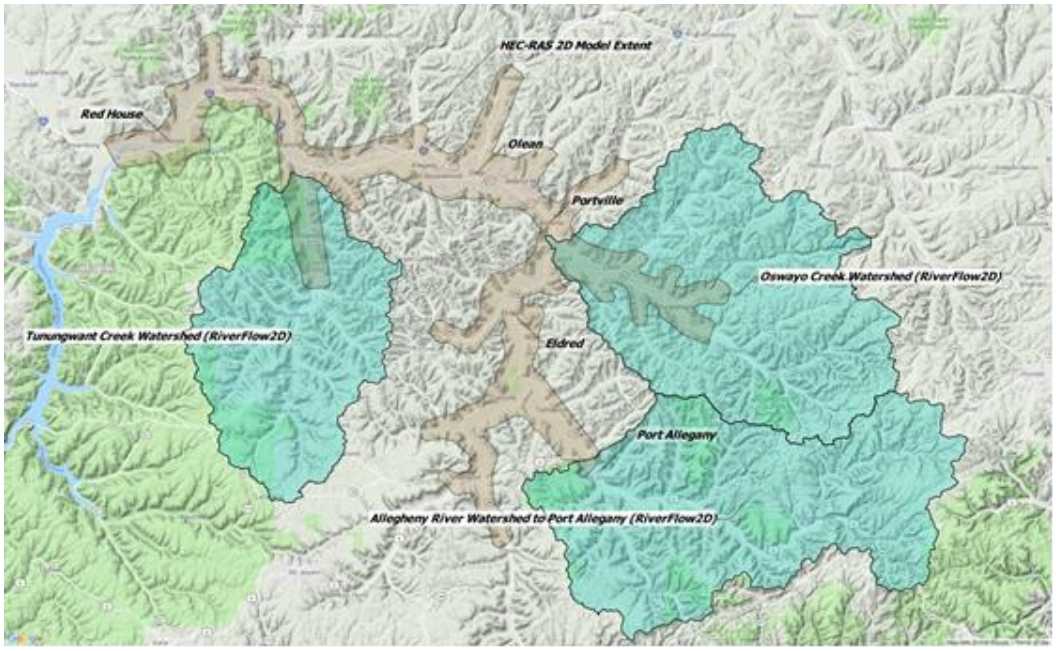


Figure 18. Overview of Land Use

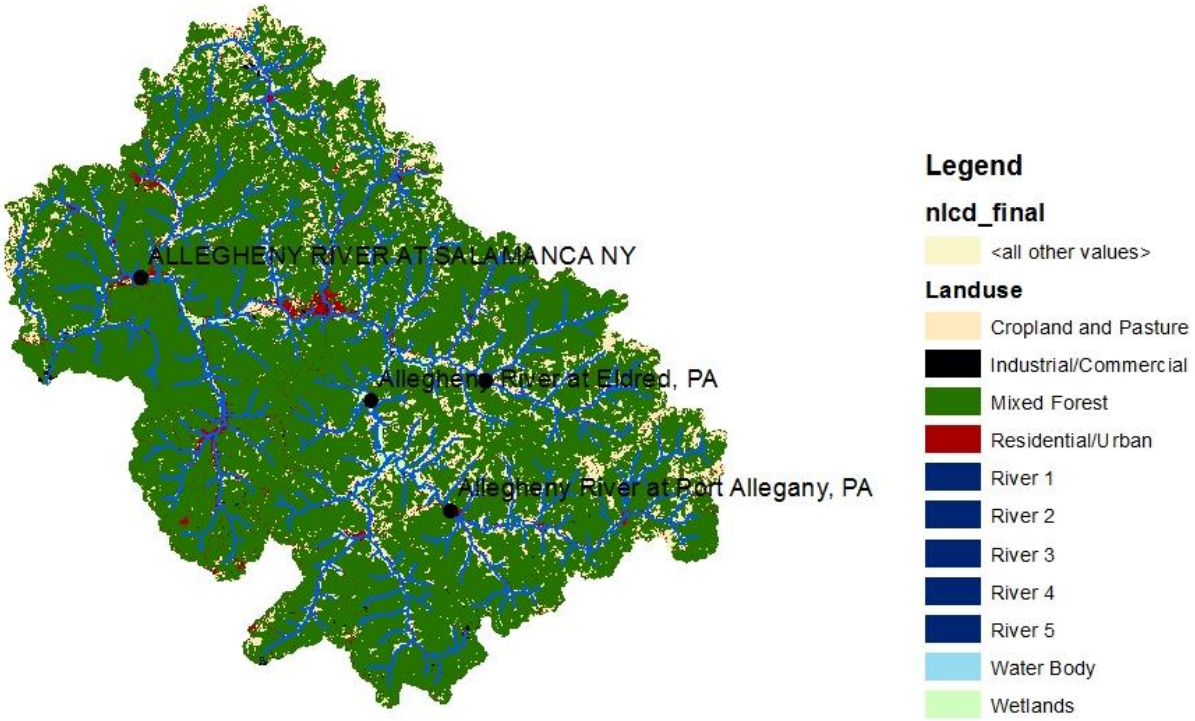


Figure 19. Hydrologic Soil Groups

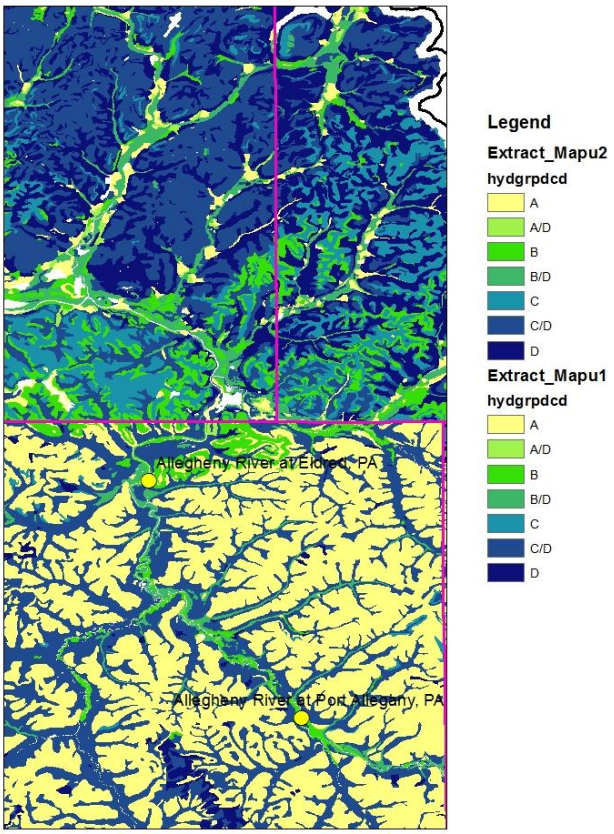


Table 2 - Input Data for the Hydrologic Models

NED elevation data	https://nationalmap.gov/3dep_prodserv.html	Primarily NY DEM
Lidar (PA only)	www.pasda.psu.edu	DEM for PA
1m DEM	LiDAR (PA) and NED (NY)	this is a composite data set of LiDAR and resampled NED from NY.
Buffer of watershed for clipping	HUC 10 plus buffer	Primarily reference
Historical land use (poly)	https://water.usgs.gov/GIS/dsdl/ds240/index.html	Will be used for Agnes storm modeling
NLCD land cover	https://www.mrlc.gov/nlcd2011.php	Ivan storm modeling
HUC 10 & 8 watershed boundaries	https://datagateway.nrcs.usda.gov/	Primarily reference
NHD Streams	https://nhd.usgs.gov/data.html	Primarily reference
Streamgage data - U.S.	water.usgs.gov	Primarily reference
gssurgo data	https://datagateway.nrcs.usda.gov/	Soils Data for Curve Number
Current aerial photographs	www.pasda.psu.edu	For final mapping
Current aerial photographs	https://orthos.dhses.ny.gov/	For final mapping
Historical aerial photographs	www.pasda.psu.edu	For final mapping

2.3 Calibration

The 2D and HEC-HMS models were calibrated using three warm-season flood events in months with full vegetative growth to simulate canopy coverage comparable to July 1942. The September 2004 “Ivan” flood and June 2014 storms were selected as warm-season candidates and run through AWA’s SPAS program to produce the hourly gridded rainfall data. Using post-1996 storm events allows the use of the NEXRAD data, providing a more reliable and comprehensive understanding of the spatial and temporal distribution for the calibration storms. Combining the NEXRAD data with the stream gauge data from these events, processed through SPAS (in 1-hour 1 km2 gridded format), reduces uncertainty and improves the quality of the input data for the 2D and HEC-HMS models. In addition to post-1996 floods, the 1972 “Tropical Storm Agnes” flood was selected for calibration due to its significant effect on the region and availability of a substantial amount of reliable precipitation and flood data. Note that the June 2014 storm was only significant to Port Allegany and, thus, only used to validate the RiverFlow2D model. The results of the June 2014 analysis were similar to the September 2004 “Ivan” analysis and, therefore, are not provided in this report. As indicated in Table 3 and Table 4, all other post-1996 annual peaks occurred in months with potential rain/snowmelt combinations and/or periods with limited or no vegetation canopy.

Once the judgement was made that the RiverFlow2D model was reasonably reliable in hydrologically simulating watershed response, parameters were adjusted in the HEC-HMS model (particularly Curve Number and Snyder Unit Hydrograph (UH) parameters) to establish a match with RiverFlow2D. The rainfall patterns and calibration results for the 1972 (Agnes) and 2004 (Ivan) floods are provided in Figure 20 through Figure 28. Additional tabulation of observed and model data for the 1972 Agnes Flood is provided in Table 5.

It should be noted that, in calibrating the HEC-HMS model to the June 1972 "Agnes" flood, a discrepancy was identified between the USGS's and USACE's estimation of the actual peak flows at the Eldred PA gauge. The USGS's streamflow records (page 283 of the USGS Hurricane Agnes Report (USGS, 1975)) show a peak flow rate of 65,400 cfs, whereas the USACE reported a peak flow rate of 30,300 cfs in Table 2 of their 1974 Report (USACE, 1974). The USACE updated this flow to 35,500 cfs, as reported by FEMA in Section 2.3 of the Effective Flood Insurance Study (FIS) for McKean County PA (FEMA, 2016), which cites a 1976 report from the USACE (USACE, December 1976). It appears the USACE updated the original estimate of 30,300 cfs to 35,500 cfs based on a HEC-2 model developed for the FEMA FIS. Figure 24 shows the recorded USGS flows and approximated hydrograph from the USACE 1976 estimated peak flow. Iterations of the HEC-HMS model indicate that the USACE peak flow estimate is more plausible since a runoff volume estimated using the USGS streamflows exceed the rainfall volume estimate. Therefore, it was concluded that the HEC-HMS calibration should be based on a comparison with an estimated hydrograph that corresponds to the USACE peak flow of 35,500 cfs. See Figure 24 for the approximated hydrograph and HEC-HMS model hydrograph. Figure 24 also shows the hydrograph generated from a "profile line" created at the Eldred gauge in the HEC-RAS2D model, which shows a hydrograph that deviates from the approximated and HEC-HMS hydrographs. Hydrographs generated at selected locations in HEC-RAS2D generally compare well with observed at other locations. Therefore, given the uncertainties surrounding the streamflow estimates at the Eldred gauge, the discrepancies at the Eldred gauge were accepted for the purpose of this study. It is possible that the measured and modelled flows at Eldred are affected by backwater from the confluence between the Allegheny River and Cole Creek, located just downstream of the gauge and the Route 346 bridge.

Table 3 – Annual Maximum Streamflow @ USGS 03007800, Allegheny R @ Pt Allegany, PA (248 mi^2)

Date	Annual Peak Discharge (cfs)
12/1/2010	12,200
1/26/2010	8,530
3/15/2007	7,560
11/30/2005	6,650
1/9/1998	6,480
9/18/2004	6,460
2/7/2008	6,170
1/24/1999	5,940
6/26/2014	5,300
1/31/2013	4,920
9/30/2015	4,820
1/14/2005	4,730
3/22/2003	4,670
3/9/2009	4,660
4/10/2001	4,460
10/1/2015	4,370
2/28/2000	3,840
5/14/2002	3,760
5/8/2012	1,740

Table 4 – Annual Maximum Streamflow @ USGS 03010500, Allegheny R @ Eldred, PA (550 mi^2)

Date	Annual Peak Discharge (cfs)
12/2/2010	17,000
3/16/2007	12,600
2/8/2008	12,200
1/10/1998	10,700
1/27/2010	10,500
1/26/1999	10,100
3/11/2009	9,670
9/19/2004	8,800
3/23/2003	8,610
12/1/2005	8,560
1/15/2005	8,110
4/11/2015	6,890
12/23/2013	6,650
2/2/2013	6,330
5/15/2002	6,030
2/29/2000	5,730
4/11/2001	5,640
12/29/2015	5,270
1/28/2012	4,010

Figure 20. Precipitation Patterns for the June 1972 (Agnes) and September 2004 (Ivan) Floods

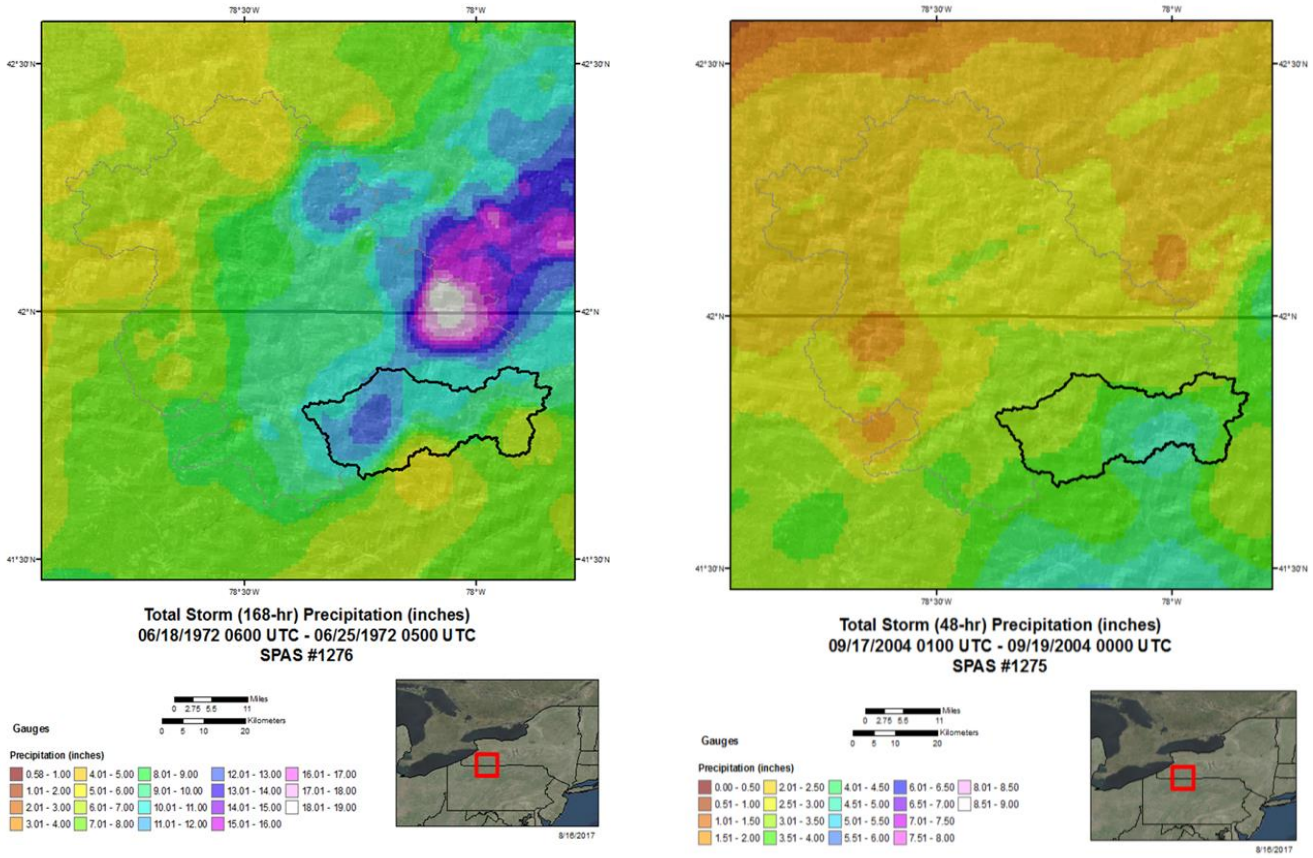


Table 5 – Model Calibration Results for the June 1972 (Agnes) Flood

River Mile[3]	Location	Observed			Model[4]			
		Date/Time	Peak Discharge (cfs)	Peak WSEL (ft, NGVD29)	Date/Time	Peak Discharge (cfs)	Peak WSEL (ft, NAVD88)	Peak WSEL (ft, NGVD29)
	Coudersport (US Rt 6 Br)		5,790	1653.1	6/23/72 12:00 AM	6,348	1655.0	1655.4
	Coudersport (Mill Creek)		3,490			2,865		
298.3	Roulette (Fishing Cr Rd Br)			1527.6			1529.2	1529.7
295.1	Burtville PA (Kim Hill Rd Br)			1509.7			1510.7	1511.2
289.6	Port Allegany (Rte 155 Bridge)			1478.9			1478.4	1478.9
288.9	Port Allegany (W Mill St Br)[2]	6/22/72 9:00 PM	22,000	1475.2	6/22/72 9:00 PM	21,083	1476.6	1477.1
288.0	Port Allegany (Rte 6 Bridge)[2]	6/22/72 9:00 PM		1472.3	6/22/72 9:00 PM	21,325	1473.1	1473.6
269.0	Eldred PA[1,5]	6/23/72 9:00 PM	35,000	1445.5	6/23/72 8:00 AM	35,540	1443.1	1443.6
	Olean NY[1]		59,000	1426.0	6/23/72 9:30 AM	65,143	1427.1	1427.6
233.7	Salamanca NY[1]	6/23/72 1:00 PM	73,000	1381.5	6/23/72 12:45 PM	80,797	1379.2	1379.7

[1] Observed peak discharge value obtained from the FEMA Flood Insurance Study for McKean County (FEMA, 2016), which cites a 1976 USACE study (USACE, December 1976).

[2] Estimated to be 24 hours before the peak at Eldred (6/23/72 9:00 PM at USGS Eldred Gauge), from the HEC-HMS model.

[3] River Miles from USGS report vary from the USACE profile for Agnes.

[4] Results from the 2D model are shown at and upstream of Port Allegany. Results from the HEC-HMS model are shown downstream of Port Allegany.

[5] The "peak discharge" reported in the HEC-HMS model of 32,913 cfs at 8:30 AM on June 23, 1972, appears to be an anomaly. The actual peak appears to be occurring at the magnitude and time shown in the table.

Figure 21. June 1972 (Agnes) HEC-HMS Hydrographs – Allegheny River at Coudersport (Compared to 2D Model)

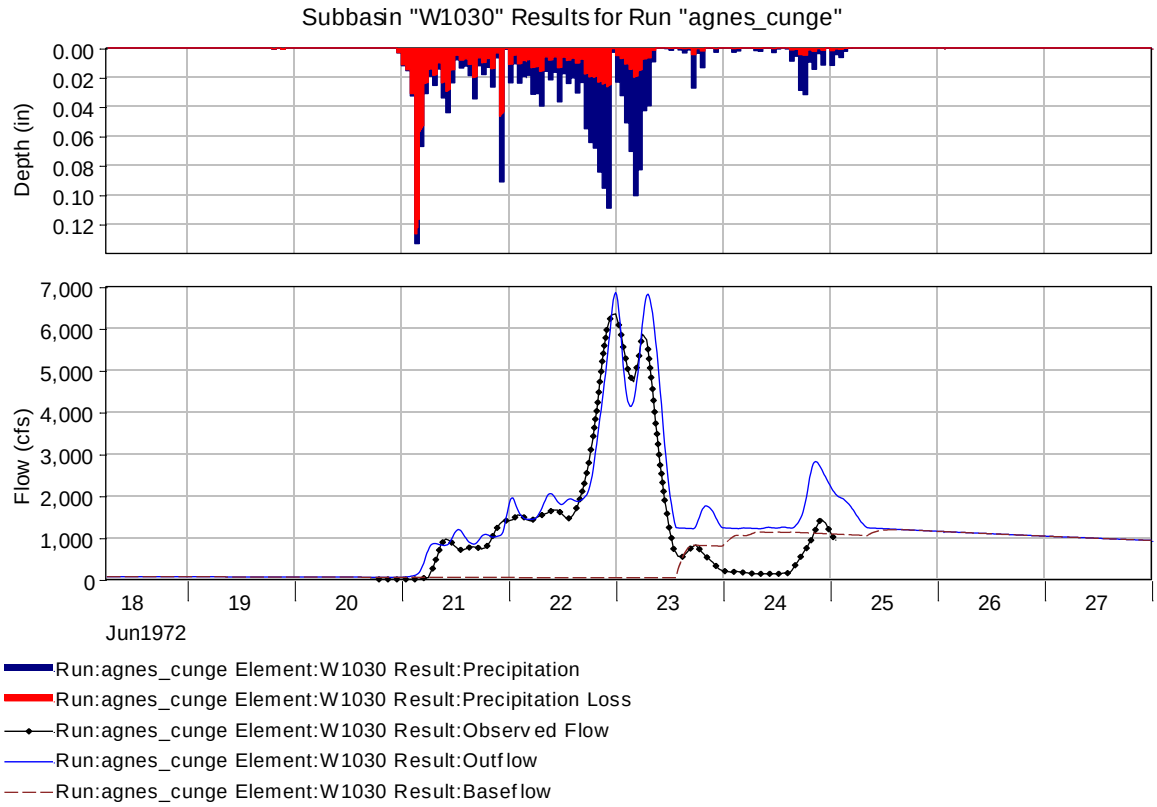


Figure 22. June 1972 (Agnes) HEC-HMS Hydrographs – Mill Creek at Coudersport (Compared to 2D Model)

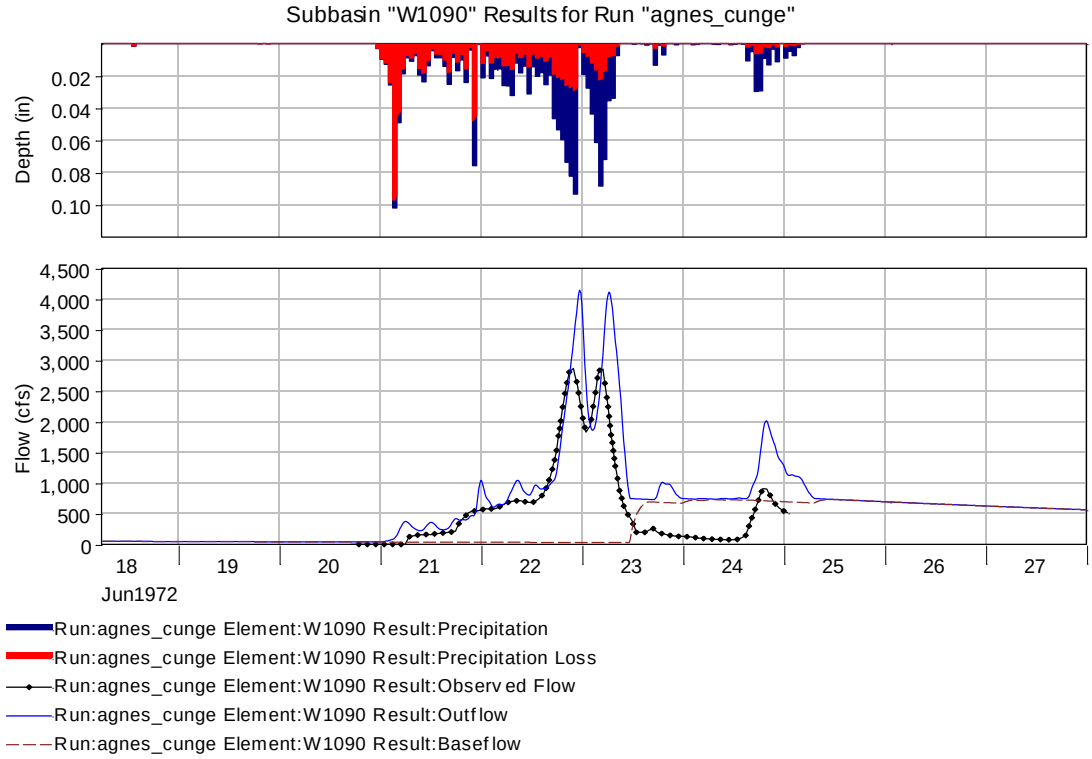


Figure 23. June 1972 (Agnes) HEC-HMS Hydrographs – Allegheny River at Port Allegany (Compared to 2D Model)

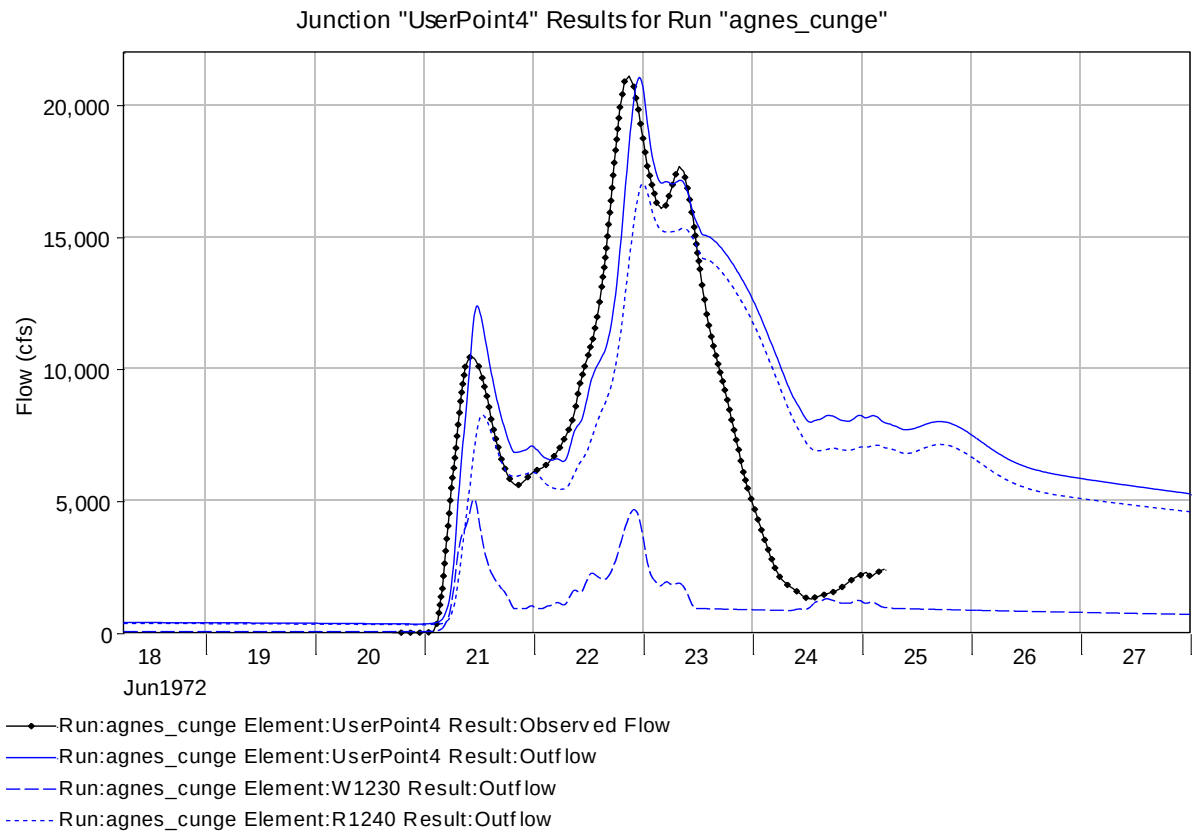


Figure 24. June 1972 (Agnes) HEC-HMS Hydrographs – Allegheny River at Eldred (Streamflow data above 20,000 cfs is approximated based on 35,500 cfs USACE peak flow estimate)

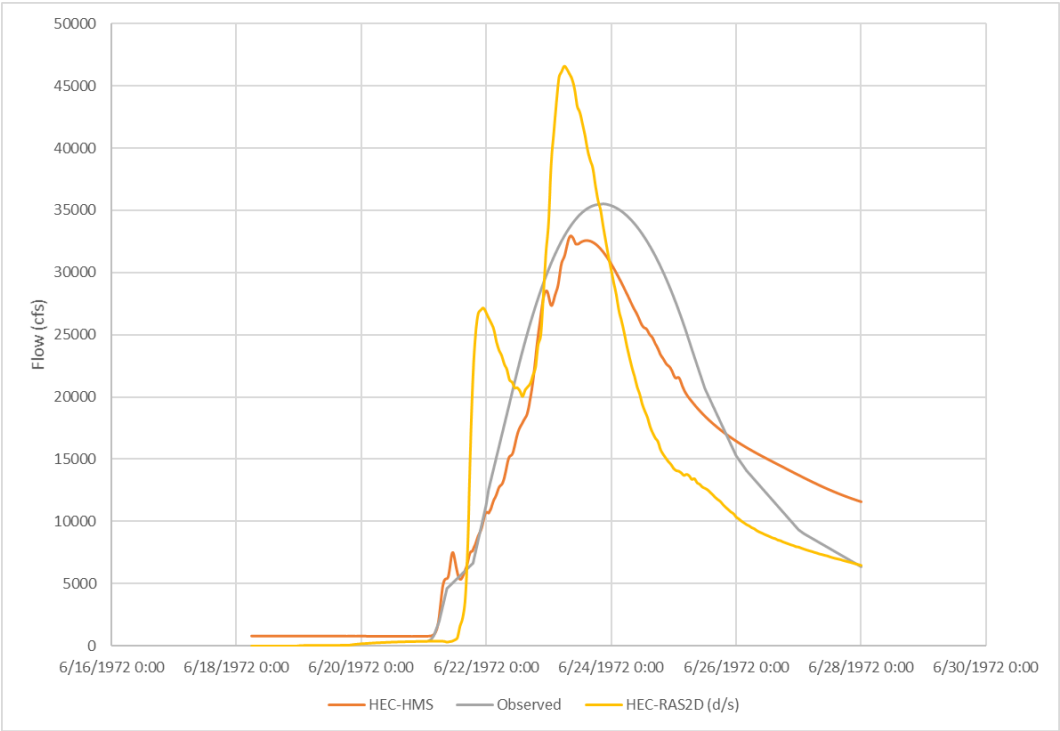


Figure 25. June 1972 (Agnes) Hydrographs – Allegheny River at Olean (Streamflow data approximated from NOAA Report 73-1, page 24)

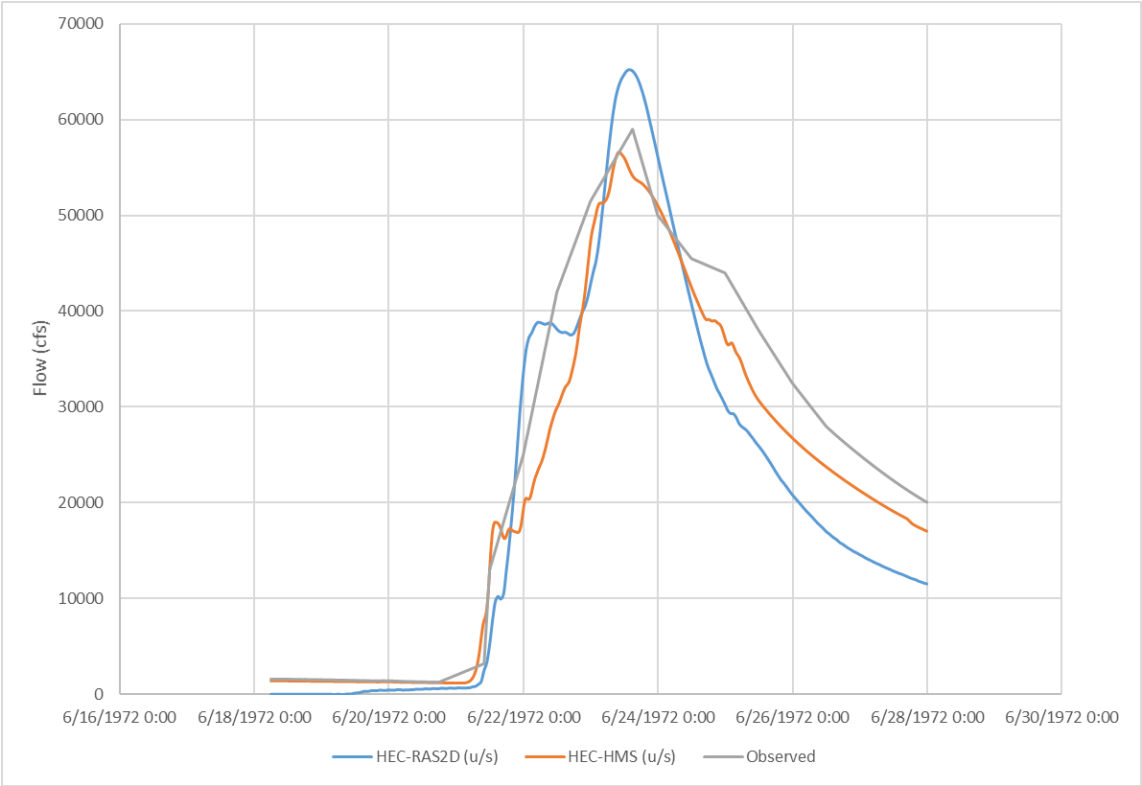


Figure 26. HEC-HMS Model Run for the September 2004 (Ivan) Flood at Port Allegany

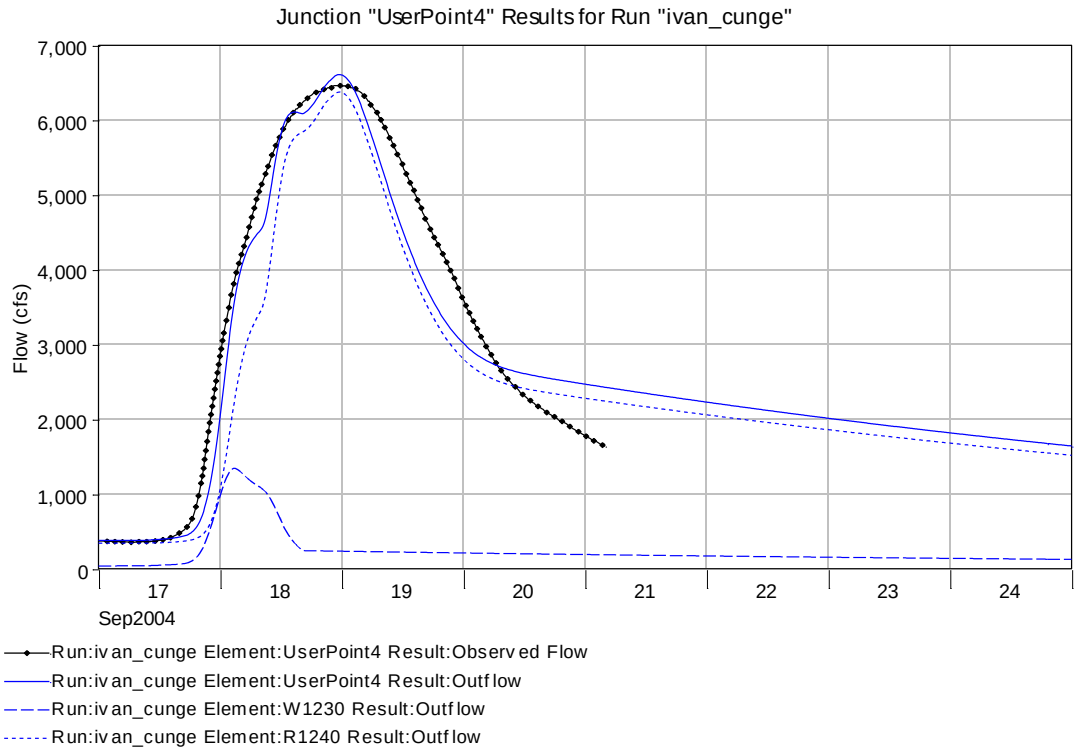


Figure 27. HEC-HMS Model Run for the September 2004 (Ivan) Flood at Eldred

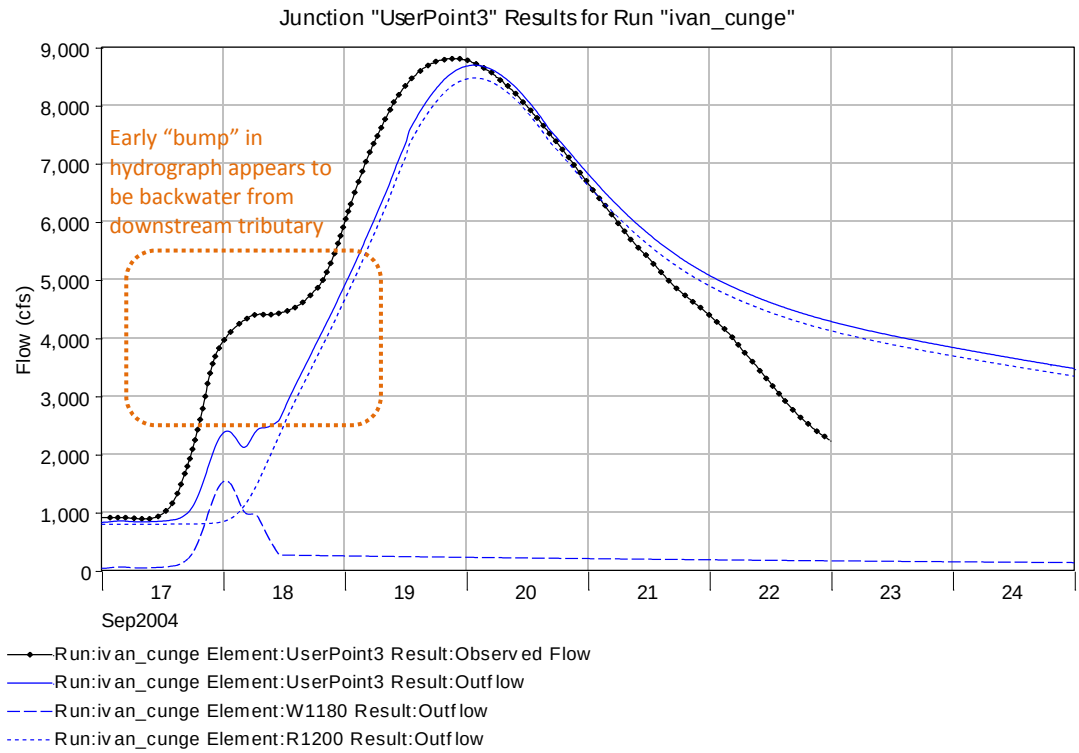
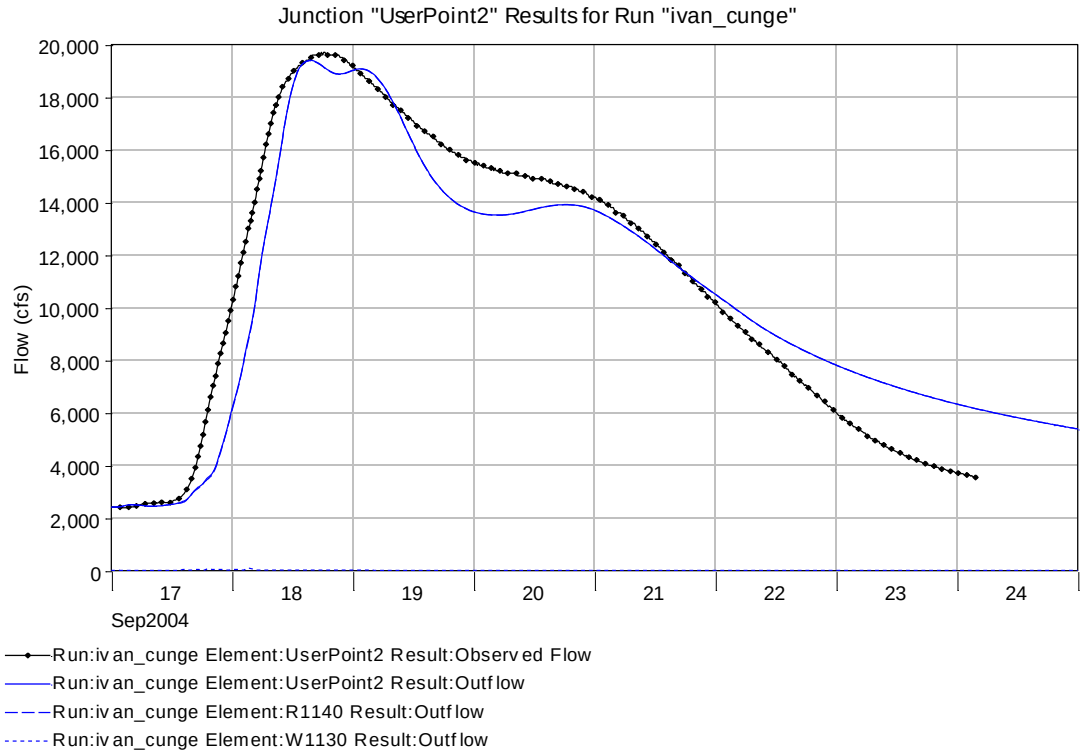


Figure 28. HEC-HMS Model Run for the September 2004 (Ivan) Flood at Salamanca



2.4 Parameters

2.4.1 HEC-HMS Model Parameters

The HEC-HMS model was developed using the following components:

- Loss Method – SCS (Runoff Curve Number)
- Transformation Method – Snyder Unit Hydrograph
- Reach Routing Method – Muskingum-Cunge
- Baseflow Method – Recession

Similar to the 2D model, RCNs were developed based on cover type, hydrologic conditions, and hydrologic soil groups (HSG) obtained from various sources, as described in Section 2.2. The RCN and initial abstraction values were adjusted as part of the calibration process to provide a good fit of the modeled hydrograph with the observed data. Given the homogenous watershed characteristics, the calibration focused on the mixed forest (HSG A) land use type. Table 6 provides the calibrated RCNs and Initial Abstraction for each sub-basin. In developing the HEC-HMS model, discontinuities in the NRCS's HSG Classifications were discovered along the PA-NY border. (See Figure 19.) The majority of the model domain in PA has a "Mixed Forest" land cover with HSG A. Much of the apparent discontinuity is in the PA HSG A and NY HSG C or HSG C/D. Similar to the 2D Model RCN calibration, the HEC-HMS Model calibration produced RCNs in HSG A, "Mixed Forest", areas of PA that correspond closer to HSG B or HSG C.

Table 6 – Final HEC-HMS Runoff Curve Numbers

Sub-Basin	Initial Abstraction (inches)	Curve Number	Impervious (%)
W1000	0.5	79.5	5
W1010	0.5	79.2	5
W1020	0.5	79.7	5
W1030	0.5	64.0	5
W1040	0.5	79.0	5
W1050	0.5	78.8	5
W1060	0.5	62.0	5
W1070	0.5	67.6	5
W1080	0.5	65.0	5
W1090	0.5	64.0	5
W1100	0.5	67.5	5
W1120	0.5	68.5	5
W1130	0.5	66.1	5
W1170	0.5	59.6	5
W1180	0.5	78.3	5
W1220	0.5	76.2	5
W1230	0.5	70.0	5
W560	0.5	76.9	5
W570	0.5	78.3	5
W580	0.5	77.0	5
W590	0.5	77.3	5

Sub-Basin	Initial Abstraction (inches)	Curve Number	Impervious (%)
W600	0.5	78.6	5
W610	0.5	77.5	5
W620	0.5	74.4	5
W630	0.5	72.0	5
W640	0.5	78.4	5
W650	0.5	77.8	5
W660	0.5	77.0	5
W670	0.5	72.0	5
W680	0.5	74.9	5
W690	0.2	78.3	7
W700	0.5	75.6	7
W720	0.5	70.7	5
W730	0.5	70.3	5
W740	0.5	72.9	5
W750	0.5	71.6	5
W760	0.5	78.1	7
W770	0.5	75.0	5
W780	0.5	77.1	7
W790	0.5	70.0	5
W800	0.5	70.0	5
W810	0.5	78.1	7
W820	0.5	85.1	7
W830	0.5	72.9	5
W840	0.5	72.0	5
W850	0.5	72.2	5
W860	0.5	73.8	5
W870	0.5	75.1	5
W880	0.5	80.2	5
W890	0.5	72.1	5
W900	0.5	76.1	5
W910	0.5	73.8	5
W920	0.5	60.0	5
W940	0.5	77.6	5
W950	0.5	75.0	5
W960	0.5	75.0	5
W970	0.5	50.0	2
W990	0.5	50.0	2

The Muskingum-Cunge modeling technique was used to simulate attenuation in the hydrographs due to river channel and floodplain storage. The technique is based on a finite difference solution of a combination of the continuity equation and simplified (diffusion-form) of the momentum equation. The model inputs include an 8-point cross-section configuration, reach length, roughness (n-value) coefficients, and energy slope. The initial value for energy slope was determined from the LiDAR

representation of the channel bottom (which is the base-flow water surface at the time LiDAR data was collected). Early calibration runs of the HEC-HMS model, particularly for the June 1972 "Agnes" flood, showed significant attenuation in the hydrographs that was not represented by observed streamflow data. Typically, velocities and bottom shear increases, resulting in steepening of the energy slope, as flow increases. Therefore, the energy slope was gradually increased to achieve good agreement in the hydrograph peak flow and timing for the "Agnes" flood model.

The Snyder Unit Hydrograph method for runoff transformation consists of two key parameters: Lag Time (t_p) and Peaking Coefficient (C_p). Equation 34 from the HEC-HMS Technical Reference Manual (USACE, 2000) provides the equation for t_p:

$$t_p = CC_t(LL_c)^{0.3}$$

where,

- C = 1 (for English Units)
- C_t = Basin Coefficient
- L = Length of the main stream from the outlet to the divide
- L_c = Length along the main stream from the outlet to a point nearest to the watershed centroid

Citing Bedient and Huber (1992), the HEC-HMS Technical Reference Manual states that C_t typically ranges between 1.8 and 2.2, although lower values have been found in mountainous regions (0.4). For each sub-basin, an initial value for t_p was calculated using a C_t of 2.0. L and L_c were estimated for each sub-basin using GIS. The initial values for Lag Time (t_p) were adjusted to achieve good agreement between the observed and model hydrographs at the Port Allegany, Eldred, and Salamanca streamflow gauges for the September 2004 (Ivan) flood. The Peaking Coefficient (C_p) is reported in the HEC-HMS Technical Reference Manual to range between 0.4 and 0.8. An initial C_P value of 0.6 was selected for each sub-basin, which were then adjusted higher to achieve a good match for the September 2004 (Ivan) hydrographs.

Unit Hydrographs, commonly used to transform runoff volume to a runoff hydrograph, inherently assume that "discharge at any time is proportional to the volume of runoff and that the time factors affecting hydrograph shape are constant" (USDA-NRCS, 2007). This linearity assumption is not strictly true when a Unit Hydrograph is applied to a storm of much higher magnitude than the calibration storm, even when calibrated at a gauged location. As discussed previously, the non-linearity property of lumped Unit Hydrographs was a significant consideration in using a 2D distributed model for part of the study area.

The non-linearity unit hydrograph issue became evident in applying an "Ivan" calibrated HEC-HMS model to the June 1972 "Agnes" storm. Additional adjustments to the "Ivan-calibrated" Snyder Unit Hydrograph parameters were required to achieve an acceptable level of agreement at Coudersport, Port Allegany, Eldred, and Olean for the "Agnes" calibration. Because the June 1972 "Agnes" flood was much larger in magnitude than the September 2004 "Ivan" flood, the "Agnes" calibrated Snyder Unit Hydrograph parameters were initially applied to the July 1942 storm in the HEC-HMS model. The "Ivan-Calibrated" RCNs were reduced by between 0% and 30% to achieve good runoff volume agreement for the "Agnes" flood. HEC-HMS model parameters were adjusted to provide good agreement with both streamflow gauge data and the three calibrated RiverFlow2D models for the "Agnes" flood. Calibration of the RiverFlow2D models also involved adjustments to Manning n-values to provide good agreement with the "Agnes" runoff responses and flood profiles provided by the USACE in their 1974 report (USACE, 1974).

Another "check" in the HEC-HMS model was at the critical location of Port Allegany. An observed hydrograph at Port Allegany was not available for the 1972 "Agnes" flood. However, as discussed in the

previous section, available peak water surface profiles, flows, and timing information was available for the 1972 "Agnes" flood (USACE, 1974), for validating the RiverFlow2D model within the domain at and upstream of Port Allegany, which showed good agreement in the RiverFlow2D model for the 1972 "Agnes" flood. Therefore, the Snyder Unit Hydrograph Lag Times were further reduced by 50% for all sub-basins in HEC-HMS, from the "Ivan" calibrated Lag Times, to achieve good agreement with the RiverFlow2D model at Port Allegany and Coudersport. Table 7 shows the evolution in the development of the Snyder Unit Hydrograph parameters. See also Section 2.3 for the results of the "Agnes" and "Ivan" HEC-HMS calibrations, respectively.

Table 7 - HEC-HMS Snyder Unit Hydrograph Parameters

Basin Parameters					Ivan			Agnes	
Sub-Basin ID	Longest Flowline (ft)	Centroidal Length (ft)	HMS Drainage Area (mi²)	Calculated Lag Time (hrs)	Lag Time (t_p), hr	Ratio of Calc/Final Lag Time	Peaking Coef (C_p)	Lag Time (t_p), hr (½ x t_p^{Ivan})	Peaking Coef (C_p)
W1000	47,226	22,492	18.37	5.96	2.62	0.44	0.80	1.31	0.80
W1010	64,798	30,268	37.02	7.16	2.23	0.31	0.80	1.12	0.80
W1020	37,354	20,747	6.03	5.42	1.88	0.35	0.80	0.94	0.80
W1030	88,123	43,038	47.01	8.73	4.95	0.57	0.60	2.48	0.60
W1040	112,665	48,121	55.61	9.72	4.31	0.44	0.80	2.16	0.80
W1050	129,619	57,755	106.23	10.71	5.32	0.50	0.80	2.66	0.80
W1060	90,953	45,255	47.76	8.95	6.00	0.67	0.60	3.00	0.60
W1070	56,054	29,965	24.49	6.84	3.60	0.53	0.60	1.80	0.60
W1080	77,440	40,094	32.22	8.22	6.07	0.74	0.60	3.04	0.60
W1090	57,353	26,237	31.49	6.62	3.21	0.49	0.60	1.61	0.60
W1100	91,356	25,828	44.02	7.57	6.26	0.83	0.60	3.13	0.60
W1120	26,071	5,016	6.30	3.18	2.52	0.79	0.60	1.26	0.60
W1130	12,748	5,753	1.54	2.67	0.72	0.27	0.80	0.36	0.80
W1170	3,697	2,573	0.10	1.45	0.80	0.55	0.40	0.40	0.40
W1180	59,289	18,866	21.95	6.05	3.06	0.51	0.80	1.53	0.80
W1220	100,241	41,697	53.68	8.99	4.91	0.55	0.80	2.46	0.80
W1230	49,373	5,120	23.59	3.88	4.00	1.03	0.60	2.00	0.60
W560	103,008	32,583	58.88	8.42	6.00	0.71	0.60	3.00	0.60
W570	53,288	31,715	19.88	6.85	3.18	0.46	0.80	1.59	0.80
W580	108,732	51,228	54.61	9.80	8.00	0.82	0.80	4.00	0.80
W590	105,448	48,877	41.01	9.57	6.00	0.63	0.80	3.00	0.80
W600	69,734	31,940	26.59	7.44	2.00	0.27	0.80	1.00	0.80
W610	58,036	31,924	26.12	7.04	4.00	0.57	0.80	2.00	0.80
W620	84,296	41,496	47.07	8.52	4.75	0.56	0.60	2.38	0.60
W630	35,403	16,209	7.36	4.96	1.50	0.30	0.80	0.75	0.80
W640	47,053	14,301	27.63	5.20	2.63	0.51	0.80	1.32	0.80
W650	46,287	20,491	9.15	5.76	1.50	0.26	0.80	0.75	0.80
W660	80,449	36,979	35.01	8.12	4.00	0.49	0.80	2.00	0.80
W670	43,169	22,584	12.72	5.81	2.00	0.34	0.80	1.00	0.80
W680	91,656	42,243	30.59	8.79	8.20	0.93	0.60	4.10	0.60
W690	72,740	34,077	26.84	7.69	4.00	0.52	0.80	2.00	0.80
W700	83,642	34,308	37.60	8.03	3.00	0.37	0.80	1.50	0.80
W720	68,843	38,612	35.61	7.85	4.57	0.58	0.60	2.29	0.60
W730	50,810	24,442	26.52	6.25	3.00	0.48	0.80	1.50	0.80
W740	83,679	30,574	36.96	7.76	4.77	0.61	0.60	2.39	0.60
W750	23,669	5,898	10.17	3.24	1.43	0.44	0.60	0.72	0.60
W760	72,688	29,633	41.83	7.37	3.53	0.48	0.80	1.77	0.80

Basin Parameters					Ivan			Agnes	
Sub-Basin ID	Longest Flowline (ft)	Centroidal Length (ft)	HMS Drainage Area (mi²)	Calculated Lag Time (hrs)	Lag Time (t_p), hr	Ratio of Calc/Final Lag Time	Peaking Coef (C_p)	Lag Time (t_p), hr (½ x t_p^{Ivan})	Peaking Coef (C_p)
W770	88,261	45,497	47.50	8.88	4.79	0.54	0.80	2.40	0.80
W780	73,589	40,037	31.96	8.10	3.00	0.37	0.80	1.50	0.80
W790	29,145	13,241	6.72	4.40	2.17	0.49	0.60	1.09	0.60
W800	110,454	54,647	69.56	10.04	6.99	0.70	0.80	3.50	0.80
W810	48,121	14,919	26.66	5.30	3.00	0.57	0.80	1.50	0.80
W820	23,355	6,446	5.04	3.32	1.00	0.30	0.80	0.50	0.80
W830	86,163	46,874	43.53	8.90	4.56	0.51	0.80	2.28	0.80
W840	44,742	15,749	12.94	5.27	2.00	0.38	0.80	1.00	0.80
W850	6,482	4,192	0.27	1.98	0.25	0.13	0.80	0.13	0.80
W860	101,506	51,421	50.84	9.61	6.74	0.70	0.80	3.37	0.80
W870	62,996	32,604	21.56	7.26	4.66	0.64	0.80	2.33	0.80
W880	17,861	9,138	2.01	3.40	2.62	0.77	0.60	1.31	0.60
W890	78,277	30,155	28.67	7.57	7.22	0.95	0.80	3.61	0.80
W900	67,269	35,364	20.25	7.59	11.23	1.48	0.40	5.62	0.40
W910	48,261	20,661	11.04	5.85	3.65	0.62	0.80	1.83	0.80
W920	102,615	50,492	48.60	9.59	5.50	0.57	0.40	2.75	0.40
W940	77,800	47,691	32.99	8.68	12.92	1.49	0.40	6.46	0.40
W950	61,239	24,329	42.64	6.60	4.17	0.63	0.60	2.09	0.60
W960	84,595	42,665	57.12	8.60	5.00	0.58	0.60	2.50	0.60
W970	60,772	32,351	29.25	7.17	3.00	0.42	0.40	1.50	0.40
W990	49,269	23,011	20.64	6.08	3.00	0.49	0.40	1.50	0.40

2.4.2 2D Model Parameters

Manning's Roughness Coefficients (n-values)

To characterize surface roughness, Manning's roughness coefficients (n-values) were assigned to each land cover type. The initial values were based on Table 5-5 of Open Channel Hydraulics (Chow, 1959) and NRCS (USDA-NRCS, 2016). The values were further adjusted during the calibration process. Manning n-values from Chow's Open Channel Hydraulics are typically applied to one-dimensional flow analyses and inherently "lump" internal and surface energy losses in three-dimensions. Typically, a reduction in n-values for two-dimensional flow would be expected when compared to n-values used in the one-dimensional flow application.

For the RiverFlow2D model, which was used for coupled hydrologic and hydraulic analyses, the roughness n-values had to be increased in the overland areas to compensate for the shallow depth flow (less than one inch) with low velocities, to keep the watershed response from being too "flashy" (compared to streamflow gauge data). The approach used in RiverFlow2D was to differentiate the overland areas from the areas where greater flow depths were expected to vary depending on the storm magnitude. For example, for the June 1972 and July 1942 floods, areas with overland flow were defined as outside the FEMA delineated 500-year floodplain, whereas for the less-intense 2014 storm event the areas with land use classification other than wetlands, river, or waterbody were defined as overland flow. Based on the calibration of the model, the normal n-values were increased by a factor of six in the overland flow areas to achieve acceptable hydrologic (time distribution and magnitude of flow) agreement. Table 8 below provides a summary of the n-values used in the analysis for the respective land use types.

Table 8 – Manning's Roughness Coefficients in the RiverFlow2D Model

Land Use Type	Base n-value	Assigned n-value Overland Areas
River (Alleghany River and Mill Creek)	0.04	N/A
River (Tributaries)	0.08	N/A
Water Body	0.08	N/A
Wetlands	0.1	N/A
Industrial/Commercial	0.1	0.6
Cropland and Pasture	0.15	0.9
Residential/Urban	0.2	1.2
Mixed Forest	0.6	3.6

For the HEC-RAS2D model, Manning n-values were initially based on NRCS guidance (USDA-NRCS, 2016) using land cover information from the National Land Cover Database (NLCD). These n-values lead to good agreement with the USACE peak water surface profile for the June 1972 "Agnes" flood. As discussed further in Section 2.7, "override" regions were assigned with different n-values in areas of the floodplain that experienced significant land use changes between 1942 and present-day (which is similar to the land use conditions during the June 1972 calibration flood). Most of the land use changes, primarily from cultivated farmland in 1942 to present-day and 1972 wooded conditions, occurred between Olean, NY and Port Allegany, PA. In these "override" areas, n-values were set to 0.07. The calibrated n-values used in the HEC-RAS2D model are provided in Table 9.

Table 9 – Manning's Roughness Coefficients in the HEC-RAS2D Model

Land Use Type	Base n-value
Water Body (Allegheny River and tributaries)	0.04
Wetlands	0.07
Industrial/Commercial	0.15
Cropland and Pasture	0.07
Residential/Urban	0.08
Mixed Forest	0.16

Runoff Curve Number (RiverFlow2D Only)

Runoff curve numbers (RCN) were developed based on cover type, hydrologic conditions, and hydrologic soil groups (HSG) obtained from various sources, as described in Section 2.2. The RCN, along with the initial abstraction values, were adjusted as part of the calibration process to provide a good fit of the modeled hydrograph with the observed data. The watershed characteristics of the RiverFlow2D model domain are moderately homogenous, predominantly defined as mixed forest land use. The most prevalent HSG within the model domain is Type A. Given the homogenous watershed characteristics within the model domain, the calibration focused on the mixed forest (HSG A) land use type. Table 10 below provides the initial and the calibrated RCNs for each land use and HSG combination. Initial abstraction was another hydrologic parameter that was adjusted as part of the calibration process. For the 2004 storm event, the final initial abstraction value was set to 0.15, while for the Agnes and Smethport storm events it was reduced to 0.1.

Table 10 – Runoff Curve Numbers

Land Use Type	HSG	Initial RCN	Calibrated RCN
River	All	100	100
Water Body	All	100	100
Wetlands	All	98	98
Industrial/Commercial	B	90	90
Industrial/Commercial	C	92	92
Industrial/Commercial	D	95	95
Cropland and Pasture	A	45	45
Cropland and Pasture	B	60	60
Cropland and Pasture	C	75	75
Cropland and Pasture	D	85	85
Residential/Urban	A	55	55
Residential/Urban	B	70	70
Residential/Urban	C	80	80
Residential/Urban	D	85	85
Mixed Forest	A	30	55
Mixed Forest	B	55	55
Mixed Forest	C	70	70
Mixed Forest	D	78	78

2.5 Floodwater Retarding Dam Considerations

Dams contained in the USACE National Inventory of Dams (NID) (USACE, 2016) database were queried to identify dams within the HEC-HMS model domain. The location of the dams is shown in Figure 29 and the summary of dam information is provided in Table 15, ordered from largest to smallest storage volumes. Due to map scale, Figure 29 does not show all the dams (some are clustered together). Most of the dams are in sub-basins that drain to the Allegheny River at and downstream of Olean, NY. NID Identification differentiated between dams constructed prior to the July 1942 flood and between the July 1942 and June 1972 "Agnes" floods (red represent dams constructed prior to the July 1942 flood and blue were constructed between the July 1942 and June 1972 floods). Three "hypothetical" dams (representing the largest dams, lumped together for modeling purposes) were incorporated into the HEC-HMS model for the June 1972 "Agnes" flood to assess the effect of the dams on the flood hydrographs in the Allegheny River. Simplified assumptions were made for the sensitivity HEC-HMS runs, including an outlet structure consisting only of a broad-crested weir (weir discharge coefficient of 3.0), no tailwater conditions, a linear stage-storage relationship, an average embankment height and spillway width, and spillway crest 8 feet below the top of dam. See Table 11 for the hypothetical dam parameters established for the June 1972 HEC-HMS model. The results of the sensitivity analysis indicate that the dams have a relatively minor effect on the peak flow rates at and downstream of Olean NY, decreasing peak flows by approximately 9,000 cfs (or 10%) at the confluence of the Allegheny River and Olean Creek. This reduction brings the peak flow closer to the observed peak flow of 59,000 cfs.

However, most of the dams constructed before the June 1972 "Agnes" flood did not exist during the July 1942 flood. The only substantial dam constructed prior to the July 1942 flood is the Cuba Lake Dam located in the Olean Creek watershed (NID Identification NY00455 and NY00456), which has the following NID parameters:

- Year Completed = 1872
- Drainage Area = 25.3 mi^2
- Dam Height = 55 feet
- Dam Length = 1,750 feet

- Maximum Storage = 16,498 acre-feet

While not expected to significantly impact the July 1942 hydrographs in the Allegheny River downstream of Olean NY, the Cuba Lake Dam was incorporated into the July 1942 HEC-HMS model.

Figure 29. National Inventory of Dams within the HEC-HMS Model Domain

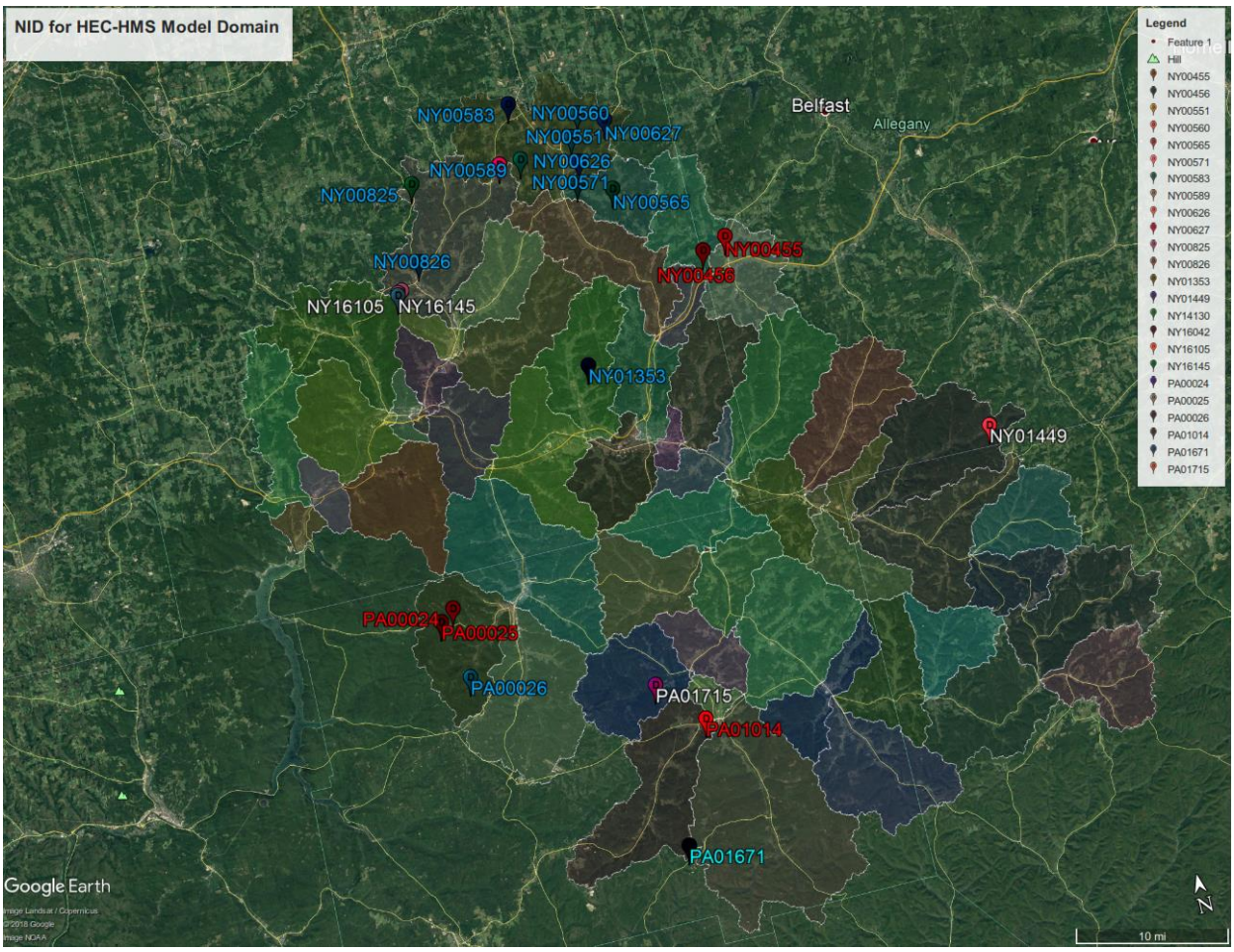


Table 11 - Hypothetical Dams used in the "Agnes" HEC-HMS Model

Parameters	NY00565-NY00627	NY00455-NY00456	PA00024-PA00026
Total Drainage Area (mi^2)[1]	43.20	25.30	15.89
Total Storage (acre-feet)	10,442	16,498	4,652
Average Spillway Width (feet)	286	204	80
Average Height (feet)	41	32	53
Assumed Spillway Height (feet)	33	24	46

[1] Total Drainage Areas excludes duplications for dams in series.

2.6 Baseflow Considerations

Review of the 2D modeling results suggested that the watershed is temporarily retaining runoff and gradually releasing volume from the storm in the later portion of the flood hydrograph. This delayed gradual release does not appear to be coming from floodwater retention structures/dams. Runoff being absorbed into a highly permeable upper layer of soil, including in the floodplain areas, and released during the receding side of the runoff hydrograph was considered as a possible explanation. The following features could provide possible explanations for this phenomenon:

- Unconsolidated glacial sediment deposits along the floodplain in the study reach. See Figure 30.
- Formation of boulder and "kame" fields and other features along the glacial edge. See W. D. Seven (W.D. Seven, 1999) for further discussion.
- Fragipans – "dense subsurface soil layers that severely restrict water flow and root penetration" (J.G. Bockheim, 2012). See Figure 31 and E. J. Ciolkosz, et. al. (Edward J. Ciolkosz, 2000) and J. G. Bockheim, et. al. (J.G. Bockheim, 2012) for further discussion.

It was hypothesized that some of the storm volume, represented in the model as a "loss", enters the riverine system via subsurface flow through highly permeable material overlain (e.g., unconsolidated deposits, "kame" fields, etc.) on a shallow layer of low permeable material (e.g., Fragipan). The hydrologic models do not physically represent this potential surface-subsurface flow interaction. The HEC-HMS model incorporates the "recession baseflow" technique to simulate potential entry of subsurface flow from storm volume. Therefore, the receding side of the HEC-HMS model hydrographs show a more gradual "tail". Comparatively, the RiverFlow2D model is only representing direct surface runoff and, therefore, shows a more rapidly declining receding side of the hydrographs. The effect of the subsurface flow on the calibration results appears to diminish with larger floods. This is evident by the improved performance of the 2D model for the June 1972 "Agnes" flood.

Figure 30. Glacial Deposits of Pennsylvania (W.D. Seven, 1999)

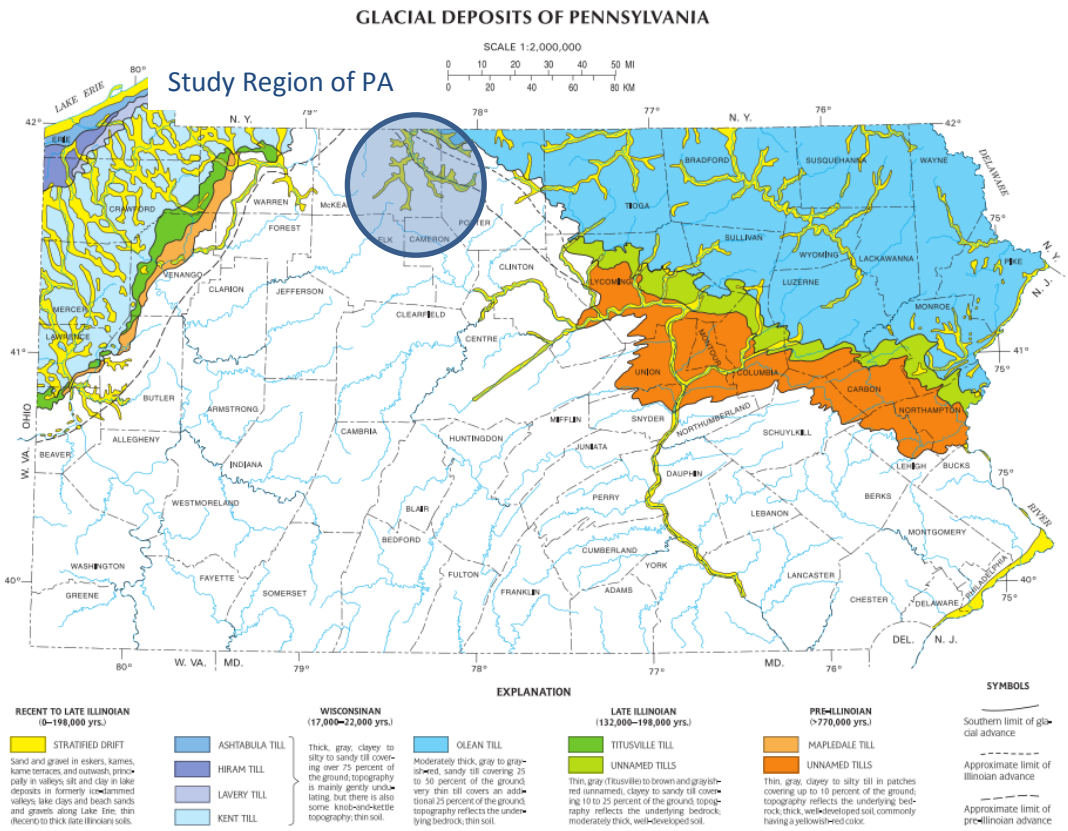
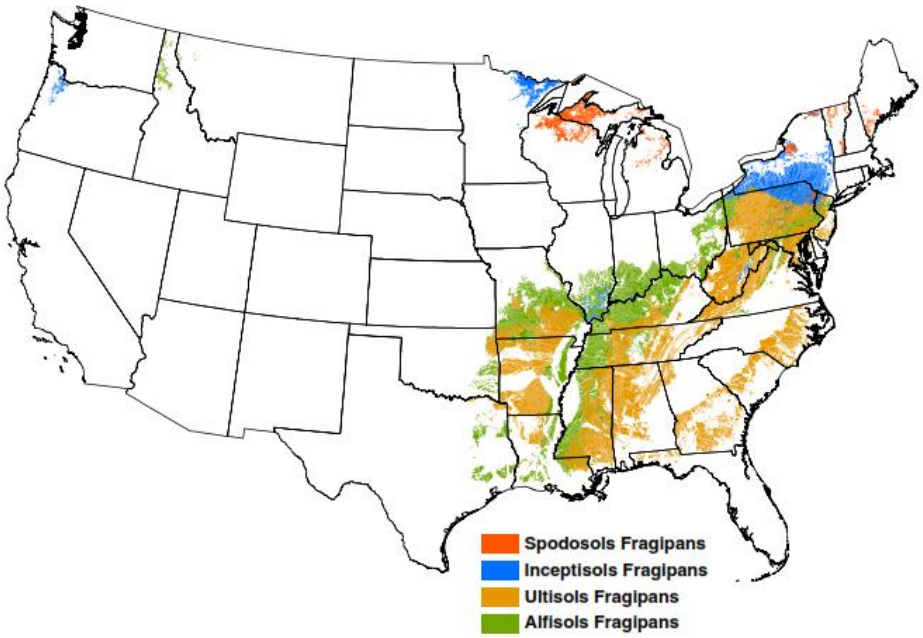


Figure 31. Distribution of Soil Mapping Units with Soils Containing Fragipans in the US (derived from National Survey Laboratory STATSGO Database) (J.G. Bockheim, 2012)



2.7 Post-Calibration Model Adjustments to Account for 1942 Conditions

Recognizing that some conditions between the calibration storms (particularly the June 1972 flood) and the July 1942 flood may vary (e.g. land use, structures, etc.), post-calibration adjustments were made to the models, as described below, prior to applying the July 1942 rainfall. These adjustments, listed below, were made to reduce concerns that flow discrepancies can be attributed to factors other than uncertainties in the rainfall data.

- Reduced the "Ratio to Peak" for the baseflow regression to 0.2 of the values established for the calibration floods due to the significantly higher peak flows in portions of the watershed for the July 1942 flood.
- Manning n-values in the HEC-RAS2D model were originally based on National Land Cover Database (NLCD). These n-values lead to good agreement with the USACE peak water surface profile for the June 1972 "Agnes" flood. However, n-value adjustments were made for the July 1942 HEC-RAS2D model to account for significant land use changes within the floodplain, particularly between Olean, NY and Port Allegany, PA. Changes were primarily from cultivated farmland in 1942 to present-day and 1972 wooded conditions.
- Adjusted the approach embankment elevations and width of the Port Allegany Route 6 Bridge, which collapsed during the 1942 flood, from drawings obtained from PennDOT. See Figure 32.
- Reductions were made to Curve Numbers in the HEC-HMS model (by approximately 20% to 30%), from those calibrated for the "Agnes" flood, to achieve good runoff volume agreement for the July 1942 flood; except for sub-watersheds upstream of Port Allegany, PA (W1030, W1060, W1070, W1080, W1090, W1100, W1230) and the Oswayo Creek upstream of Shinglehouse, PA (W860, W920, W970, and W990). Curve Numbers for these sub-watersheds remained the same for both storms (between approximately 55 and 70). For much of the watershed, except the Upper Allegheny River (upstream of the confluence with Potato Creek) and the upper portion of the Oswayo Creek watershed, had basin-wide average Curve Numbers that were generally consistent with the gridded Curve Numbers in the RiverFlow2D models.
- Due to fast-rising nature of the July 1942 hydrograph at Port Allegany, HEC-RAS2D runs were done using the "Full Momentum" equations to incorporate the "unsteady, advection, and viscous terms" (USACE Hydrologic Engineering Center, 2016) that are disregarded for the "Diffusion Wave" equations. Results from the "Full Momentum" runs show a slower rising limb of the hydrograph, which partially corrects the peak timing discrepancy.

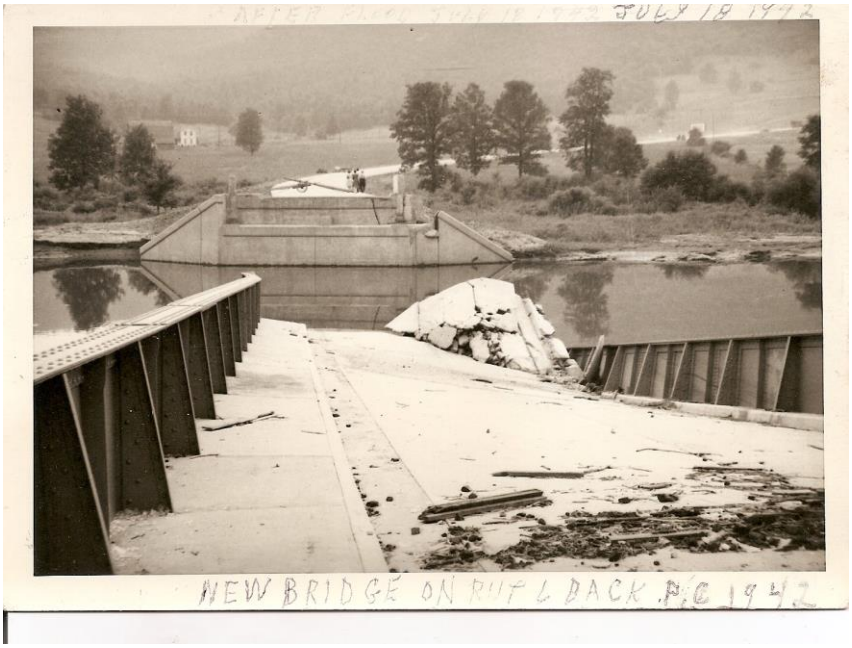


Figure 32 – Photo Looking along the destroyed Route 6 Bridge

2.8 Modeling Observations and Limitations

The following summarizes the observations and limitations in the 2D and HEC-HMS models. These observations and limitations were considered when judging refinements to the July 1942 rainfall.

- The RiverFlow2D model appears to perform well for the more intense rainfall events (given the relative comparison between the "Agnes" and "Ivan" floods).
- Subsurface conditions, in the watershed and/or floodplain, appear to be causing attenuation in the flood flows, particularly downstream of Eldred, that are not reflected in the models.
- The RiverFlow2D and HEC-HMS models generally appear to be representing peak flow timing well.
- The Unit Hydrograph in the HEC-HMS model needed adjustment to reconcile the hydrograph from the RiverFlow2D model at Port Allegany and account for a non-linear watershed response in the calibration events. The non-linear response was a key reason for using RiverFlow2D to simulate watershed response in key sub-watersheds and adjusting HEC-HMS parameters to match the RiverFlow2D hydrographs.
- Adjustments to Manning n-values were not constrained by conventional or "textbook" limits in overland flow areas to get the RiverFlow2D model to calibrate.
- Backwater conditions appear to be influencing the observed streamflow hydrograph for the September 2004 (Ivan) flood.
- The effect of hysteresis was considered when comparing HEC-RAS2D hydrographs with observed hydrographs at Eldred and Red House for the July 1942 flood. HEC-RAS2D generates cumulative flow for grid cell faces along the user-defined "profile line" in RAS Mapper to produce a hydrograph, which inherently accounts for the effect of hysteresis. The observed hydrograph, reported on Figure 42 of Water Supply Paper (WSP) 1134-B (Eisenlohr, 1952), was likely developed by an observer or gauge that recorded stage, which was then converted to flow using a pre-defined stage-discharge rating curve. The stage-discharge rating curve likely did not

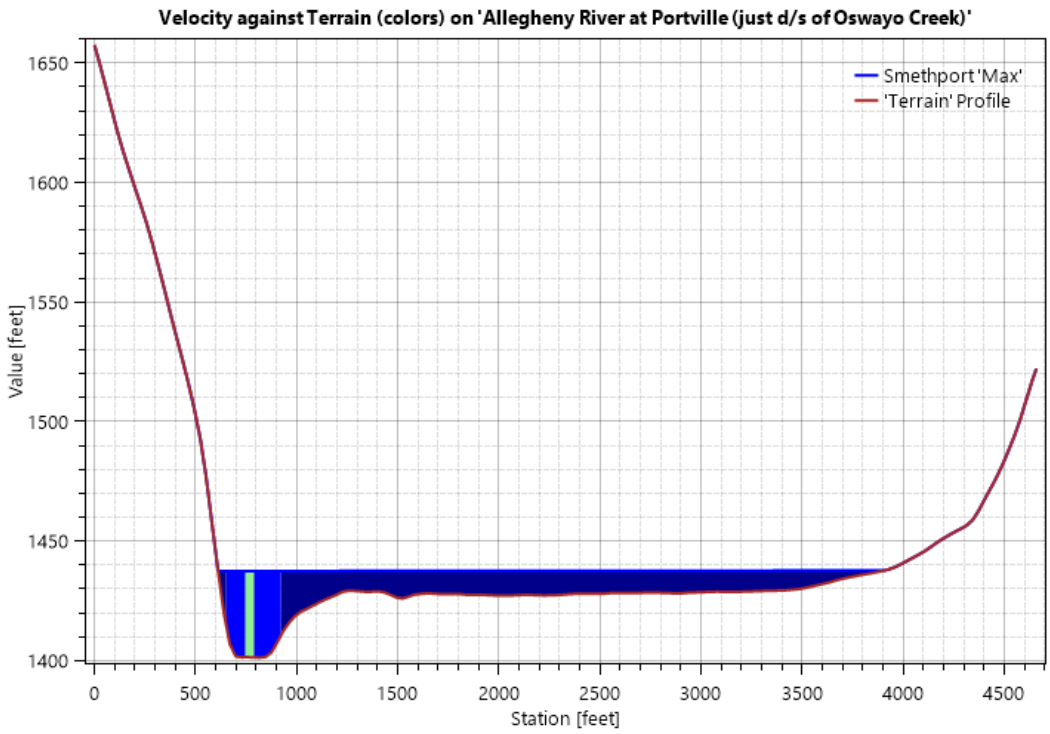
account for hysteresis effect at higher flows. This was considered when judging acceptability of the final hydrographs at Red House, NY and Eldred, PA.

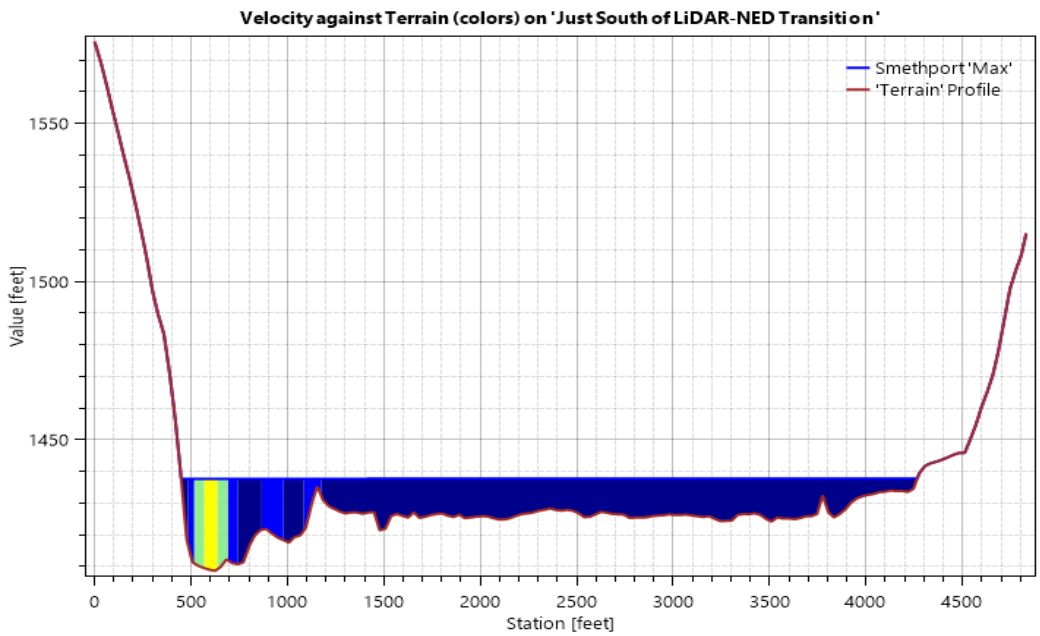
- Early HEC-RAS2D runs were done using the "Diffusion Wave" equations to reduce model time. Due to fast-rising nature of the hydrograph at Port Allegany for the July 1942 flood, HEC-RAS2D runs were revised to use the "Full Momentum" equations to incorporate the "unsteady, advection, and viscous terms" (USACE HEC-RAS, Hydraulic Reference Manual) that are disregarded for the "Diffusion Wave" equations. Results from the "Full Momentum" runs show a slower rising limb of the hydrograph, which partially corrects the peak timing discrepancies at Eldred and Port Allegany, PA.
- There appears to be greater variability than what was expected in the hydrologic response between the storms (September 2004 "Ivan", June 1972 "Agnes", and July 1942 "Smethport" floods), as represented by Curve Number and Snyder parameters in the HEC-HMS model. Curve Number and Snyder parameters needed to vary in the HEC-HMS model to achieve good agreement with observed flood data (USGS gauges, newspaper records, etc.) and the calibrated RiverFlow2D models. Sensitivity analyses shows that the potential July 1942 rainfall inaccuracies would not explain the different responses. This was particularly evident in the Potato Creek Watershed (containing the Smethport Borough) where July 1942 Snyder Lag Times were longer (closer to values developed using the SCS regression equation) and Peaking Coefficients lower than the June 1972 calibrated values.
- The model domain contains several levee systems. Table 12 provides information on these systems, obtained from the USACE National Levee Database. Most, except for the Eldred levee, were constructed after the July 1942 flood but before the June 1972 "Agnes" flood. The terrain built for the HEC-RAS2D model includes these levees. However, for the systems in New York (except a portion of the "South of Dodge Creek" levee in Portville, NY), the perception of the levees in the HEC-RAS2D terrain is limited by the resolution of the DEM. Where levees are perceived, the terrain was not manually adjusted to remove the levees for the July 1942 flood, although flooding is permitted to occur behind the levees. While there may be a minor local effect on the HEC-RAS2D model results (particularly for the PA levee systems where LiDAR is available and the levees are well defined in the DEM), a judgement was made that refinements to the DEM to remove the levees would not significantly affect the outcome of the July 1942 flood analysis (and related decisions regarding rainfall) and is not warranted at this time.
- Differences between observed and model water surface elevations in the HEC-RAS2D and portions of the Oswayo Creek RiverFlow2D models may be attributed to the lower resolution NED DEM in New York. As discussed previously, LiDAR is not available in New York so lower resolution NED was used to create the DEM for parts of the model in New York. Initial comparisons at the LiDAR-NED transition in the DEM shows that more floodplain storage and attenuation may be available than currently represented by the NED. See Figure 33 below. The top figure is just on the NED side of the LiDAR-NED transition and the bottom figure is just on the LiDAR side of the LiDAR-NED transition.
- At Bradford, PA, LiDAR shows significantly different channel and floodplain topographic characteristics than in 1942 due primarily to the construction of Route 219 through the city. The DEM was not manually adjusted to account for this difference.
- At some observation points along the Allegheny River, it is not clear if the peak water surface elevations were reported upstream or downstream of bridges. Therefore, some discrepancies may be expected at the bridges simply due to differing data point locations and bridge hydraulics.

Table 12. Summary of Levee Systems in Study Area

Municipality	Description	Year Construction Completed
Coudersport PA	Right Bank Mill Creek	1955
Coudersport PA	Left Bank Allegheny River	1955
Port Allegany PA	Lillibridge Creek – Allegheny River	1950 (approx.)
Eldred PA	Right Bank Allegheny River & Right Bank Barden Brook	1987
Shinglehouse PA	Oswayo Creek	Unknown
Portville NY	North of Dodge Creek & Right Bank Allegheny River	1951
Portville NY	South of Dodge Creek & Right Bank Allegheny River	1951
Olean NY	Left Bank Olean Creek & Right Bank Kings Creek	1952
Olean NY	Right Bank Allegheny River & Olean Creek	1952
Olean NY	Left Bank Kings Creek	1952
Salamanca NY	Left Bank Allegheny River	1971
Salamanca NY	Left Bank Allegheny River	1971
Salamanca NY	Right Bank Allegheny River – West Salamanca	1971

Figure 33. Velocity and Terrain Plot at Cross-Section at LiDAR/NED Transition





3 July 1942 Storm and Flood Analysis

3.1 Collection of Flood Data

Readily available historical information was collected and reviewed to support the July 1942 watershed/flooding analyses. Sources of flood data for the July 1942 flood included USGS streamflow gauge records (at Eldred and Salamanca) and scientific reports from government agencies on the flood that contained peak flows, peak water surface elevations, time-to-peak, and flow hydrographs at key locations along the Allegheny River, its tributaries and small drainages at the storm center. Most of the official government data came from the USGS Water Supply Paper 1134-B (Eisenlohr, 1952), Pennsylvania Department of Forestry and Waters Report (Commonwealth of Pennsylvania, Department of Forestry and Waters, 1943), and newspaper articles from an internet search.

The historical data collection and review included a site visit on August 24 and 25, 2017 to inspect key locations identified during the desktop review, including high water mark locations, areas of greatest impact from the flooding, and other locations determined to be critical to the analysis. The team met with individuals and historic societies with knowledge of or records of the event for additional insight. The site visit focused on populated areas most severely affected by the flood, particularly Port Allegany, Coudersport, Smethport, Eldred, and Portville, NY. Newspaper articles and photos provided visual markers of the flood and depth and time information. Information was geo-referenced to allow for comparison with the July 1942 flood models. See Figure 34.

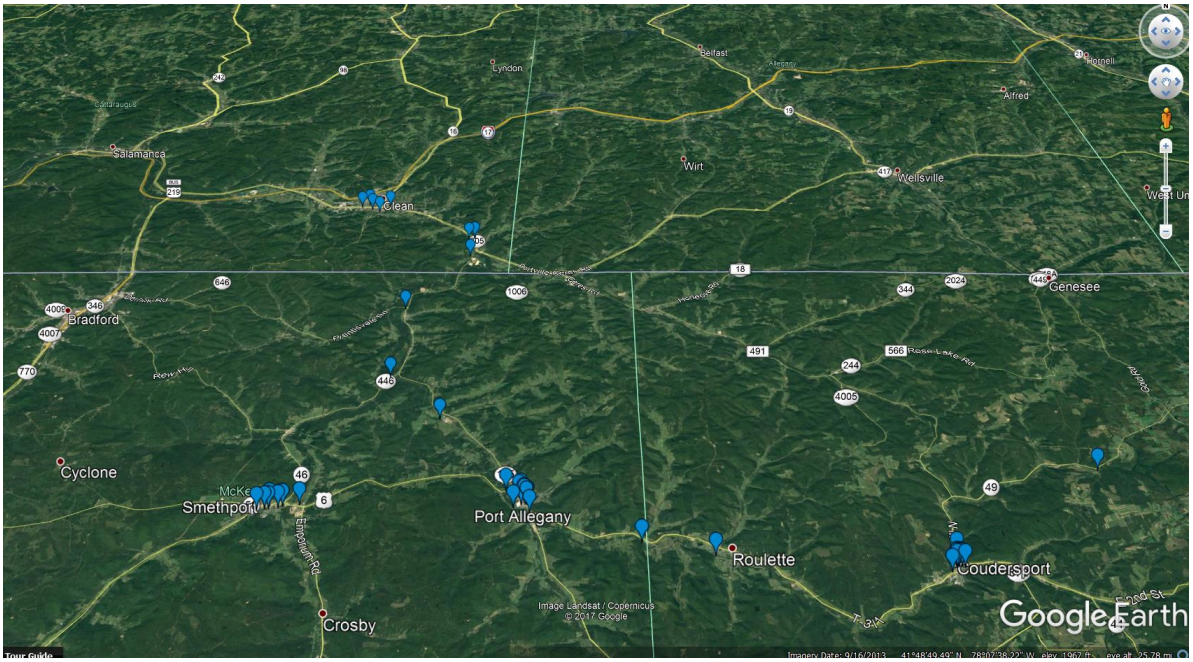


Figure 34 - Markers Showing Locations of Field and Desktop Data from the July 1942 Flood

3.2 Initial Findings

The 1-hour gridded (1 km^2) precipitation of the 1942 storm, generated using AWA's SPAS analysis of reported rainfall, was used as input in the flood models. The purpose of this task was to essentially replicate the 1942 flood with the hydrologic and hydraulic models, duplicating the stream and watershed conditions at that time. The results of the model, specifically flow, flood stage, and timing information, were compared with observations from historic records and provided insights on how well flood data corresponds to rainfall data. The objective was to identify watershed regions where reported rainfall agrees with the estimated runoff (flow rates and timing) and observed flooding, or regions

where the historic records and model predictions are in disagreement (e.g., rainfall versus peak runoff). Consideration was given to the modeling observations and limitations, discussed in Section 2.8, when making comparisons to inform the rainfall adjustments. Below is a summary of the rainfall observations made in reviewing the initial modeling results:

1. The original rainfall temporal distribution in the sub-watersheds downstream of Couderport and in the Port Allegany region is front loaded (peak intensity occurs early). The initial RiverFlow2D runs show peak flows along local tributaries within the Allegheny River watershed at Port Allegany occurring much earlier than when the peak was reported to have occurred. Since the response or lag time is short and directly correlated to the most intense rainfall period, this suggests that the peak rainfall intensity should be closer to center weighted; approximately 8 hours later than in the original rainfall temporal pattern. See Figure 35 for an illustration at Lillibridge Creek in Port Allegany.
2. The original RiverFlow2D results did not accurately predict the location and magnitudes of the peak flows in Twomile Run and Lillibridge Creek (reported to be 15,000 cfs and 16,000 cfs, respectively). This indicates that adjustments are warranted to the magnitude and spatial and temporal pattern at the Port Allegany storm center. See Figure 36 and Section 3.5 for additional analysis of the storm center rainfall.
3. The RiverFlow2D model peak time at Seven Bridges matches well with what was observed. However, the peak time in the model is early by approximately 4.5 hours downstream of the confluence between the Allegheny River and Mill Creek at Coudersport. The peak water elevations in Coudersport from the model are also consistently very high, between approximately 6 and 8 feet, from the elevations observed. As shown in Figure 37, the temporal patterns of the Mill Creek sub-watershed inflow (and the associated rainfall) appear to be contributing to the cause. While there is hourly rainfall data available from a gauge located at the boundary of the Allegheny River watershed (Raymond, PA), the temporal distribution within the Mill Creek sub-watershed may vary from the gauge temporal distribution.
4. The RiverFlow2D model shows good peak timing and water surface elevation at Roulette, suggesting that the effect of rainfall issues upstream of Coudersport dampen downstream to Roulette and the rainfall patterns between these locations, including the intense rainfall cell just to the south and west of Coudersport, are stable.
5. The peak flow along the Allegheny River at the Route 6 Bridge, generated by the original RiverFlow2D model, is early (by approximately 8 hours) and underpredicting the peak flow (61,000 cfs versus the observed peak of 77,000 cfs). The underprediction of the peak flow was attributed to the temporal rainfall patterns in the Allegheny Portage watershed (particularly the intense rainfall cell near Liberty PA). As with other parts of the watershed, the Allegheny Portage watershed rainfall is front loaded.
6. Inaccuracies in the broader temporal rainfall patterns over the Oswayo Creek and Tunungwant Creek watersheds appear to be contributing to high and early peak flows along the Allegheny River downstream of Portville, NY (at the Allegheny River's confluence with Oswayo Creek). See Figure 38.

Figure 35. Initial Hyetograph and RiverFlow2D Hydrograph for Lillibridge Creek at Main Street

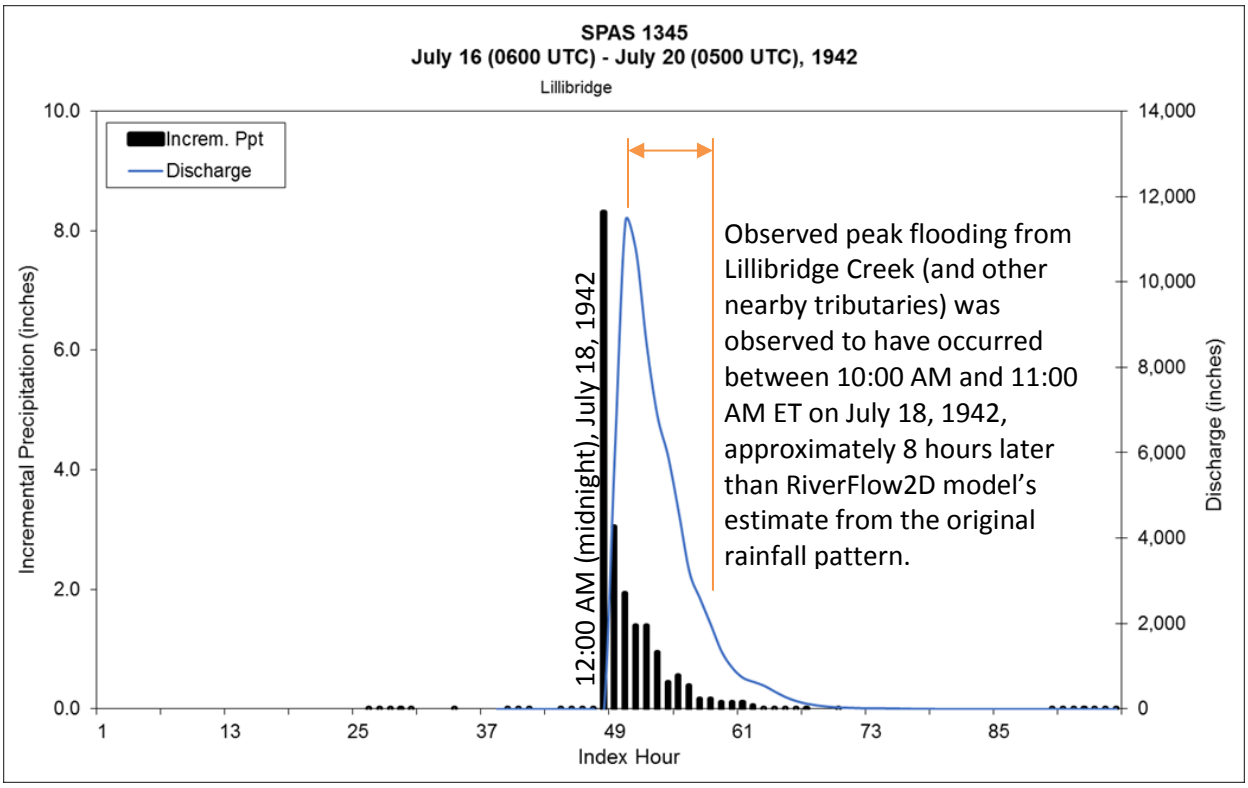


Figure 36. Extreme Rainfall at Port Allegany

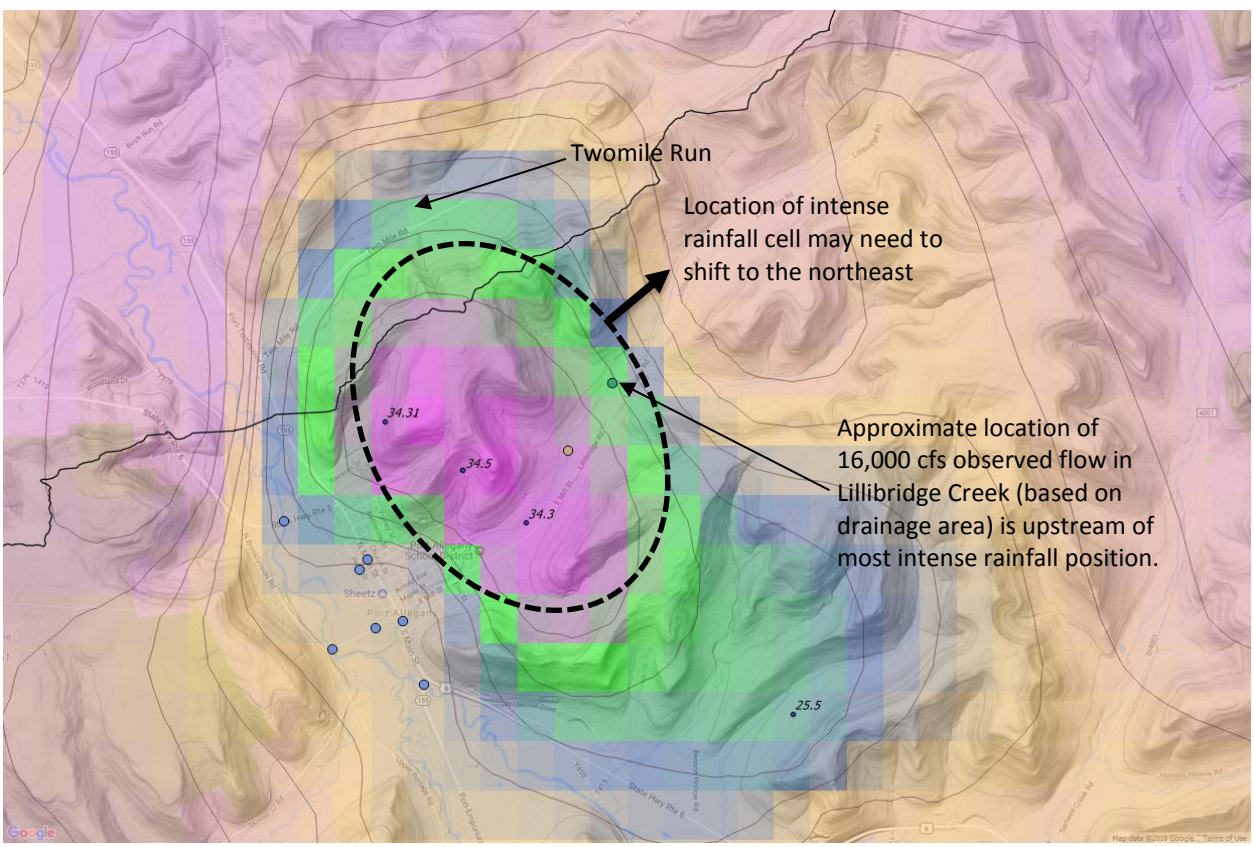


Figure 37. Initial Stage and Flow Hydrographs at Coudersport

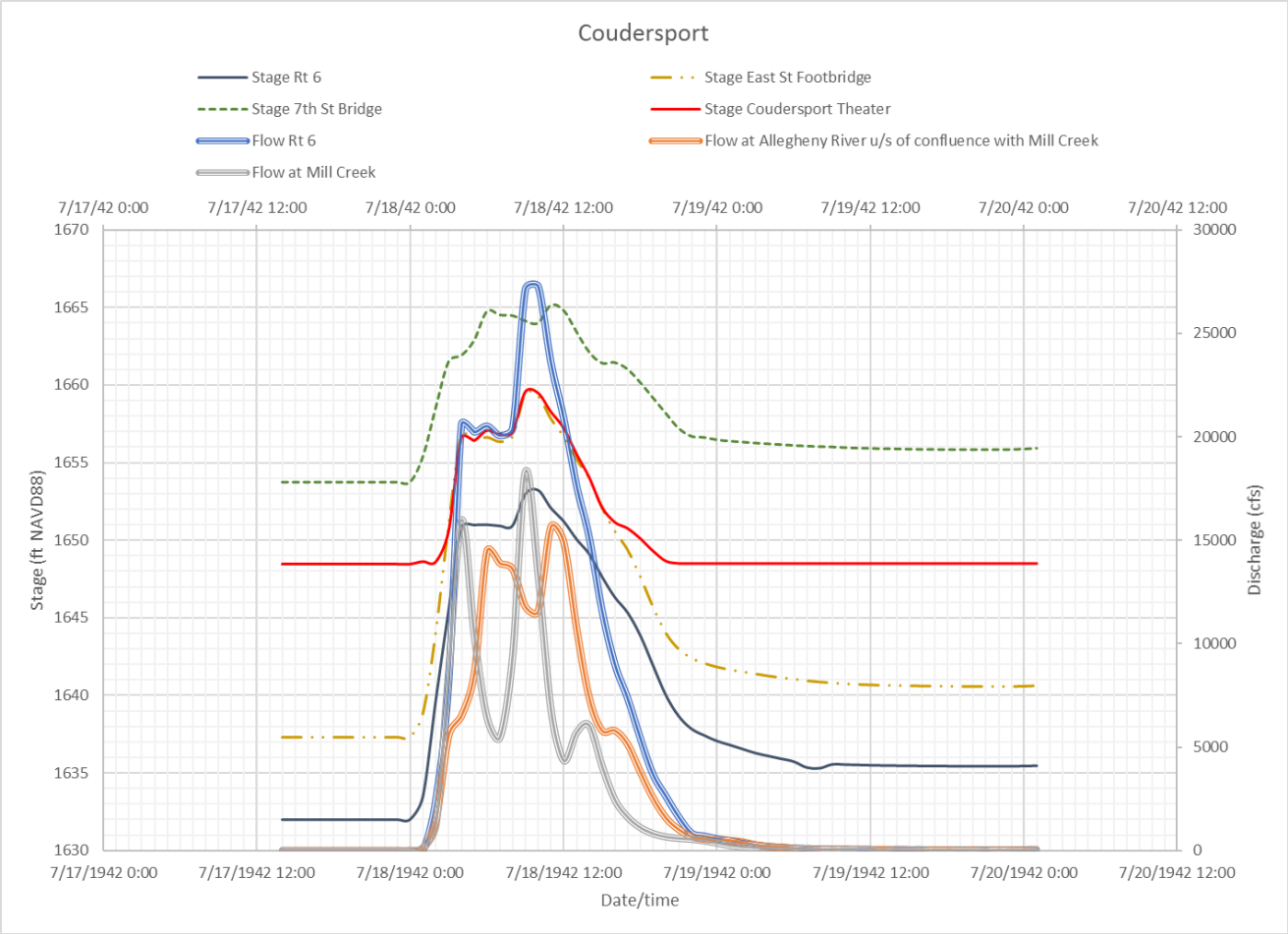
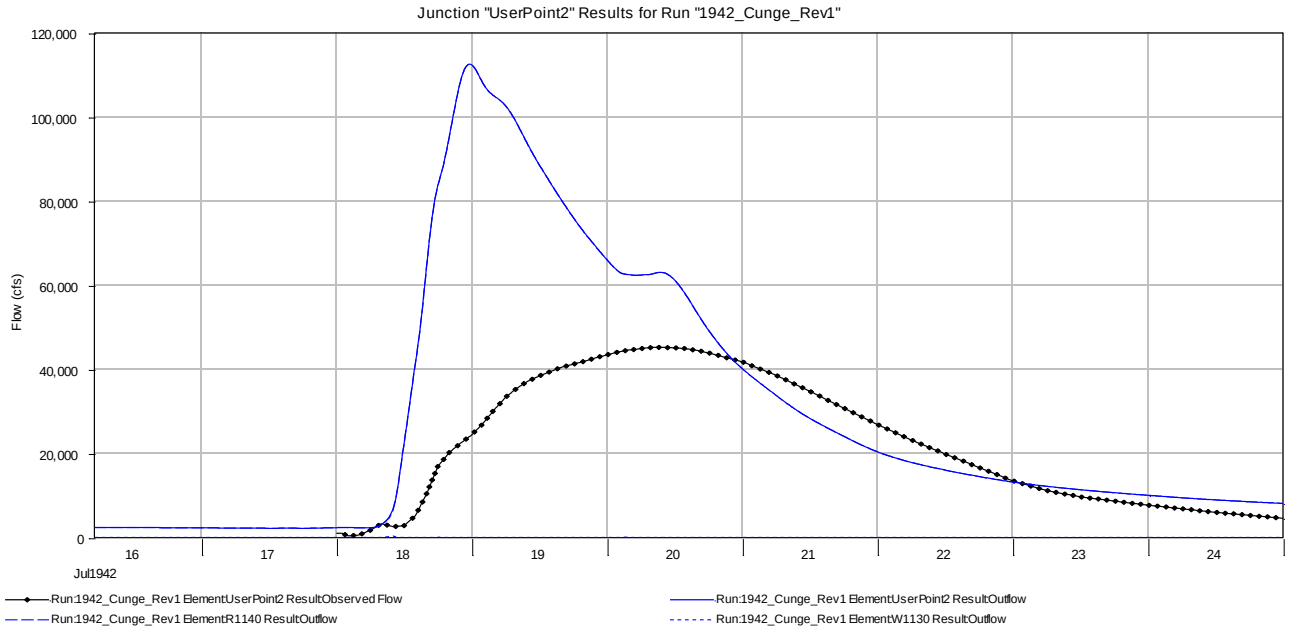


Figure 38. Initial Flow Hydrographs at Red House



3.3 Rainfall Adjustments

The evaluation of model and observed flood data, discussed above, led to iterating adjustments to the SPAS-generated rainfall data for the storm. These included adjustments to the timing, magnitude, and spatial patterns of the rainfall accumulation between observed data points. Each of these adjustments were made to better reconcile rainfall with the hydrology, informed by the calibrated flood models. All changes made to the previous rainfall accumulation patterns and magnitude were explicitly evaluated considering the acceptance of the Smethport rainfall as a world-record rainfall at the 4.5 and 6-hour durations. Most of the flood observations and records were at flood peaks (flows, water surface elevations, and time-to-peak). While peak flood data was helpful in corroborating or adjusting rainfall, a time-distributed representation of the flood (in the form of flow hydrographs) was only available at two USGS gauge locations along the main-stem Allegheny River; Eldred (PA) and Red House (NY). Because the Red House watershed encompassed the entire study domain and key rainfall locations, it represented a key comparison point in judging acceptance. With the rainfall and post-calibration model adjustments, the modeled flow hydrograph at Red House was able to improve as shown in Figure 39. Considering the modeling limitations discussed in Section 2.8, good overall agreement along the Allegheny River for peak water surface profile was also achieved, as indicated in Figure 40.

Table 13 shows the peak timing comparison between observed and results from the 2D models. Good peak timing agreement was achieved with the rainfall adjustments; except between Eldred and Olean, where the flood peak is several hours earlier than what was reported. Sensitivity analyses based on broad variations in temporal rainfall patterns led to the conclusion that the peak timing discrepancies along this reach of the Allegheny River were not associated with rainfall inaccuracies. The discrepancy abruptly begins at Eldred, then recovers once the flood wave reaches Salamanca and Red House. The peak timing discrepancy at Eldred remains unresolved since it did not appear to be attributed to rainfall. There is speculation that a natural or man-made feature not represented by the HEC-RAS2D model is providing significant storage and attenuation in the Potato Creek sub-watershed or in the Allegheny River near its confluence with Potato Creek.

A summary of rainfall adjustments are as follows:

- Revised the rainfall temporal pattern in the sub-watersheds between Coudersport and Port Allegany, deviating from front-loaded storm (timed based on HMR-56) to a pattern more consistent with the surrounding hourly gauges. See discussion in Section 3.4 for a description of additional rainfall refinements at the storm center.
- For the Mill Creek sub-watershed (just upstream of Coudersport PA), factors were applied to further adjust rainfall by reducing the 2 peak hourly depths and redistributing to the other hours to maintain the total rainfall volume. Also, Basin #5 (W1090) bucket surveys were reduced by 20%.
- After reviewing the quality of rainfall data, the spatial extent of the "Bradford 2A" gauge in the Tunungwant Creek Watershed was reduced. This gauge is located in the Bradford, PA area were rainfall collection was sparse. Spatial extend of other high-rainfall gauges seem to show a tighter spatial distribution.
- For the Oswayo Creek Watershed, re-distributed the 2 hours for the second peak over 4 hours in sub-watershed W830 and resolved high “ΔP’s” (difference between the SPAS generated rainfall with observed).

Figure 39. Post-Adjustment Allegheny River Hydrographs @ Red House, NY - July 18 to 25, 1942

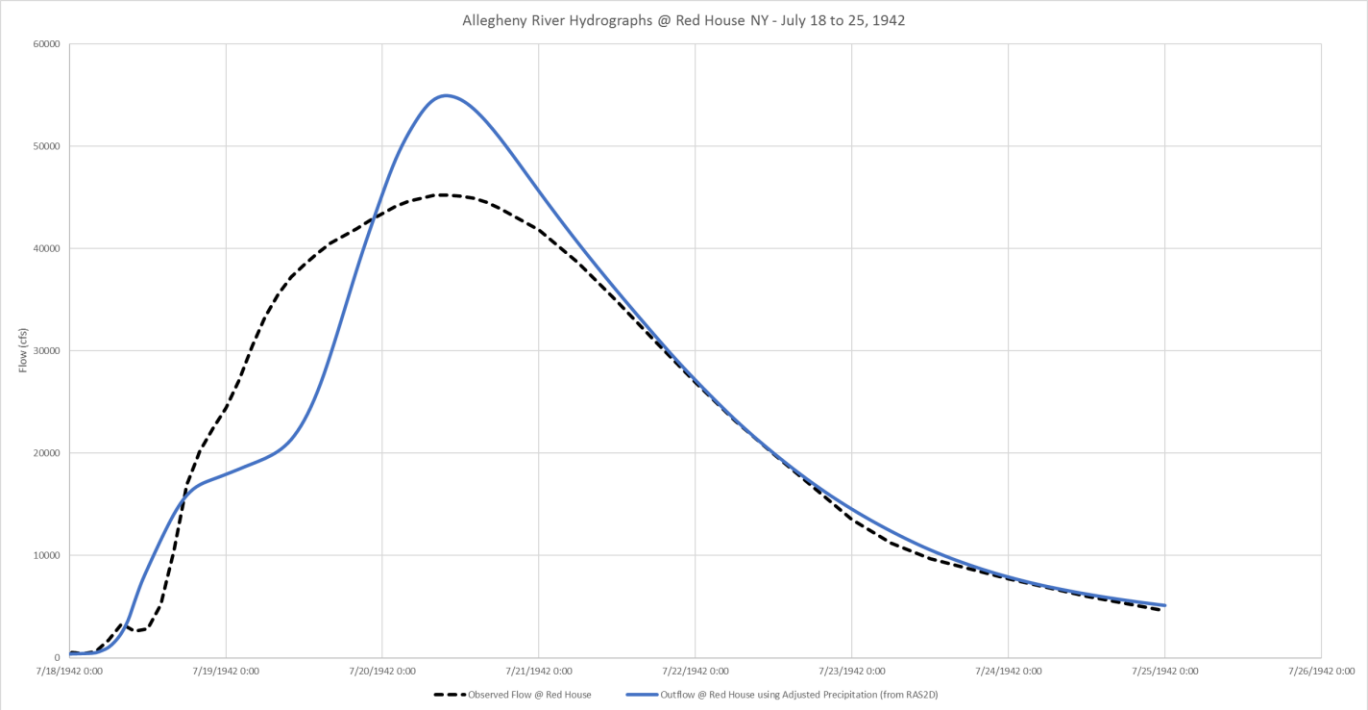
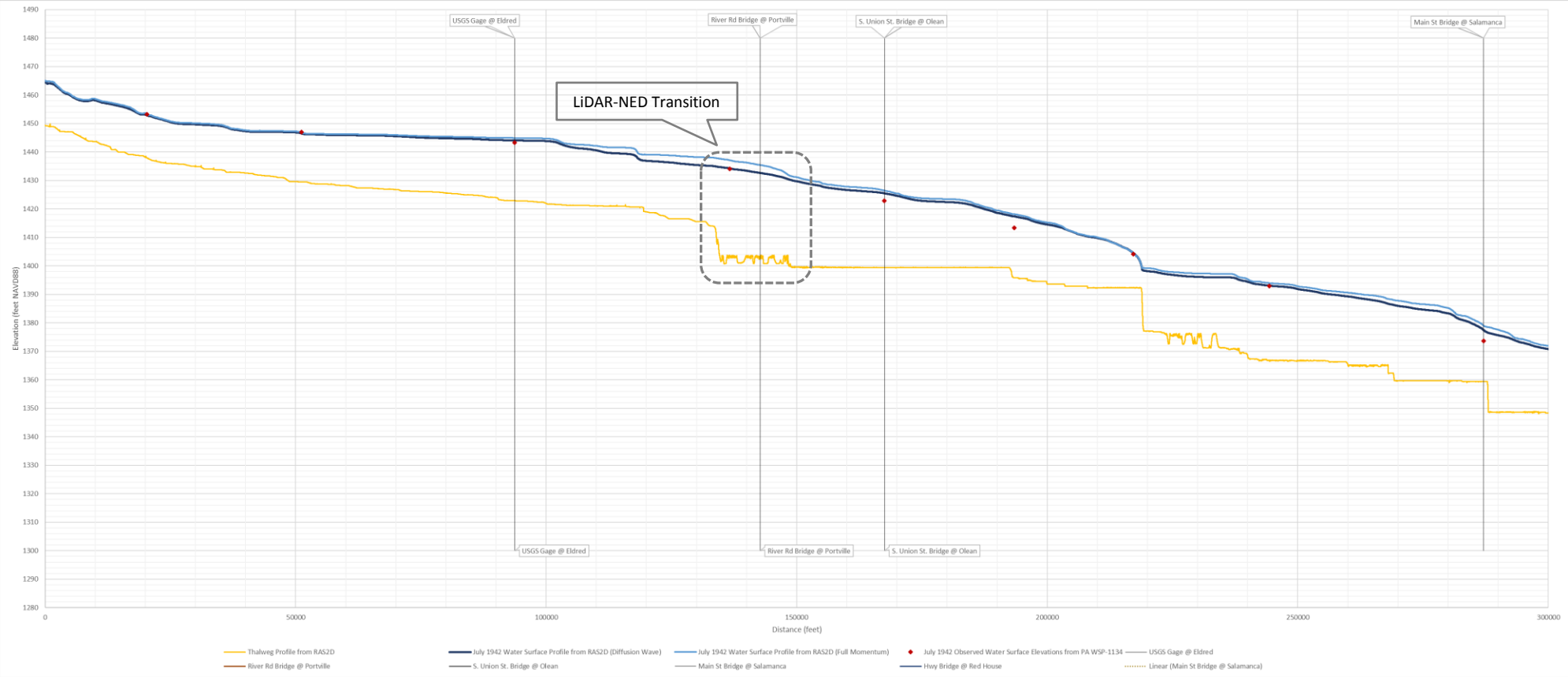


Table 13. Peak Timing Comparison with Rainfall Adjustments

Location	Peak Date/Time (Observed)	Peak Date/Time (2D Models using Ver 10 Rainfall)
Localized/Tributary Flooding:		
Lillibridge Creek - Main St, Port Allegany	7/18/42 10:30 AM	7/18/42 10:00 AM
Two Mile Run	7/18/42 10:30 AM	7/18/42 10:30 AM
Sartwell	7/18/42 10:30 AM	7/18/42 10:00 AM
Allegheny River Flooding:		
Above Roulette (466)	7/18/42 2:00 PM	7/18/42 12:00 PM
Roulette	7/18/42 2:00 PM	7/18/42 1:00 PM
Port Allegany (Route 6 Bridge)	7/18/42 3:30 PM	7/18/42 2:00 PM
Eldred	7/19/42 9:30 AM	7/19/42 12:00 AM
Portville (NY)	7/19/42 3:00 PM	7/19/42 5:00 AM
Olean (NY)	7/19/42 6:30 PM	7/19/42 12:00 PM
Salamanca (NY)	7/20/42 5:00 AM	7/20/42 7:00 AM

Figure 40. Post-Adjustment Allegheny River Peak Water Surface Profile for the July 1942 Flood



3.4 Localized Refinements at the Port Allegany Storm Center

Additional analysis of the rainfall and hydrologic record, particularly for the smaller watersheds, was conducted to refine the understanding of the magnitude and temporal patterns of rainfall in and around Port Allegany; the storm center and location of the most significant rainfall observation at Site 275 (Appolt Farm), where the 30.8-inches in approximately 5 hours was estimated. The estimated timing for this observation is shown in Figure 45. The timing applied at the Site 275 location in RiverFlow2D produces reasonable agreement with observed flood data at tributaries near Port Allegany (specifically, Lillibridge Creek and Twomile Creek). However, when this timing is allowed to influence a larger region, significantly higher flows and water surface elevations are produced in the Allegheny River near Port Allegany. From this, it was concluded that Site 275 timing would need to be significantly restricted in its influence and not allowed to influence the broader watersheds in the Port Allegany region.

Additional iterations were conducted to improve agreement in Twomile Creek, Lillibridge Creek, and Allegheny River; while attempting to hydrologically validate the Site 275 rainfall volume and timing. The additional iterations lead to the development of three (3) rainfall timing zones around the storm center (denoted as Storm Center Zones or SCZs); illustrated in Figure 41. Deviating from the original HMR-56 timing (Figure 42), SCZ 1 rainfall corresponds closest to the "Bolivar" hourly gauge and covers the broader watersheds in the Port Allegany and Coudersport region (Figure 43). With other minor adjustments, the SCZ 1 "Bolivar" timing generally produced reasonable agreement between the model and observed flood data, both in the tributaries and main-stem Allegheny River, with one exception; the Twomile Creek flows was significantly underestimated in RiverFlow2D. Furthermore, as indicated in Figure 43, applying bucket surveyed rainfall to the "Bolivar" timed temporal pattern does not produce cumulative rainfall depths that correspond to the heaviest rainfall observation at Site 275 (Appolt Farm) of 30.8-inches in approximately 5 hours. Therefore, the SCZ 1 ("Bolivar" timed) rainfall was further adjusted locally, creating SCZ 2 and SCZ 3 rainfall timing, while honoring the Site 275 observation and other nearby bucket surveys and achieving reasonable agreement with observed flows in the tributaries and main-stem Allegheny River near Port Allegany. SCZ 3 (at the storm center) is timed to the Site 275 observation (Figure 45), with spatial limits defined in Figure 41. SCZ 2, developed as a transition from SCZ 1 to SCZ 3, is timed as a modified "Bolivar" hourly gauge (Figure 44) and was based on two key observations:

1. There was no record of high flows occurring in the early (overnight) hours of July 18 along Twomile Creek and Lillibridge Creek. The RiverFlow2D model shows that significant flows would have occurred in these tributaries as a direct result of the first intense rainfall (occurring between 12:00 AM and 1:00 AM on July 18) included in the "Bolivar" timed rainfall.
2. Page 67 of WSP-1134-B (Eisenlohr, 1952) states "the observer who recorded more than 30.8 inches of rain in 4 ¾ hours stated that it seemed to fall at a tremendous rate, but quite uniformly, for the greater part of the time. Also, the drops seemed to be exceptionally large and very close together. From her statement and the record of total rainfall at that point, it may be assumed that the rainfall at no time exceeded a rate of about 10 inches per hour and that there was no "streaming" for that rate and for that size drop."

Consequently, the SCZ 2 rainfall was developed by shifting 5 inches of the "Bolivar" timed rainfall from the first hour (between 12:00 AM and 1:00 AM) to the second heavy 2 hours of rainfall (between 8:00 AM and 10:00 AM) to set the rainfall in this period at 10 inches per hour. As discussed previously, the early burst of rain in the SCZ 1 timing, as indicated by the Bolivar gauge (along with other hourly gauges in the region), appears consistent with the hydrology of the broader watershed. Applying SCZ 2 or SCZ 3 timing (i.e., shifting more rainfall later in the storm) for the broader watershed increases runoff and produces overestimated flows and levels since, given the exponential-shaped loss function associated with the NRCS Direct Runoff Equation, higher runoff occurs later in the storm. The final iterations

(Version 10) produced reasonably close matches to flood data while honoring the Site 275 and other bucket surveys in the Port Allegany Region. Modeled peak flows along the tributaries near Port Allegany were converted to unit (cfs per mi^2) flows and plotted (Figure 46) against observed unit flows in the same region (similar to Figure 43 of WSP-1134-B) showing good agreement.

Figure 41. AWA's Total Storm Precipitation (96-hours) at Port Allegany, PA

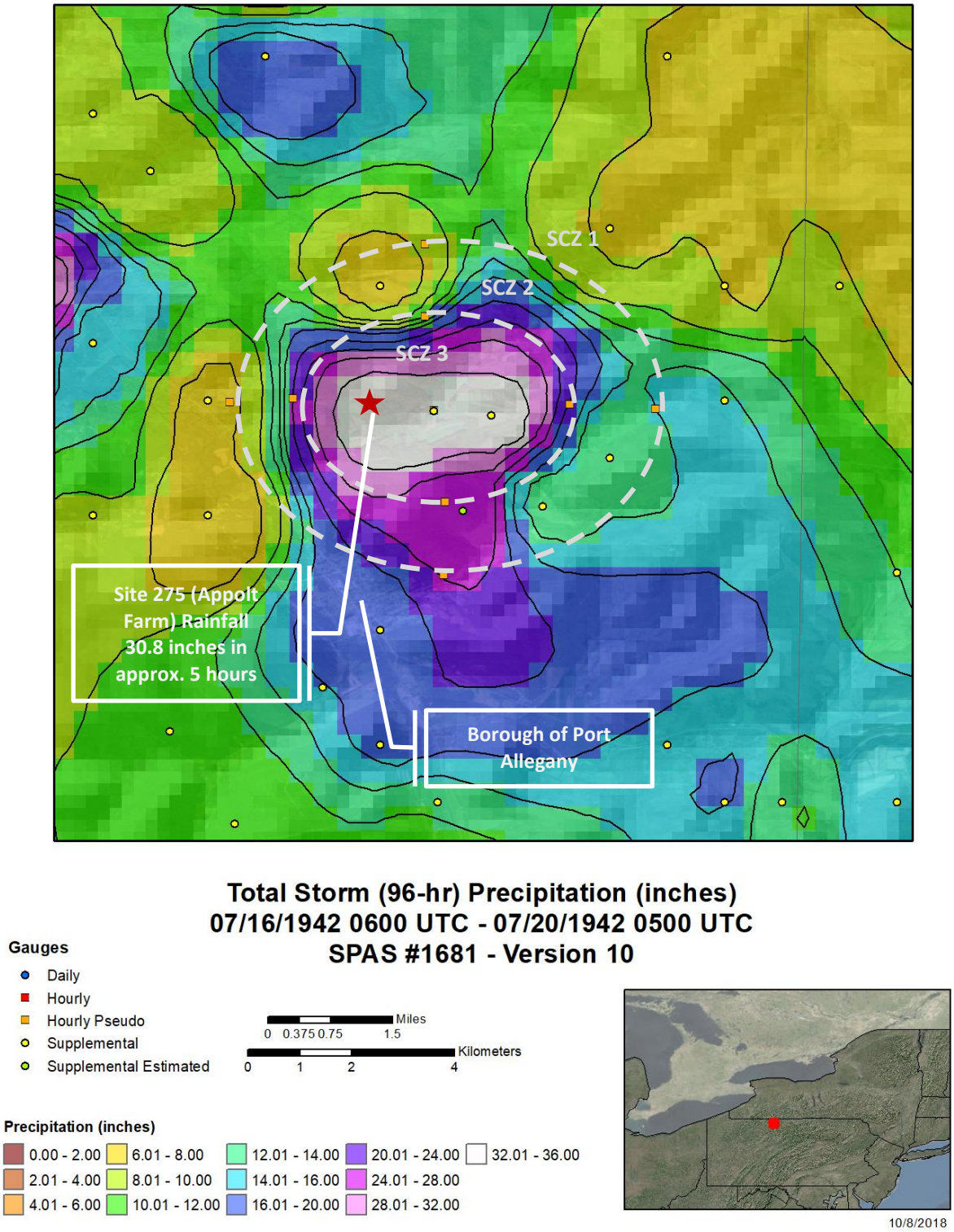


Figure 42. Original Temporal Pattern in Port Allegany, PA, Region (based on HMR-56 Timing)

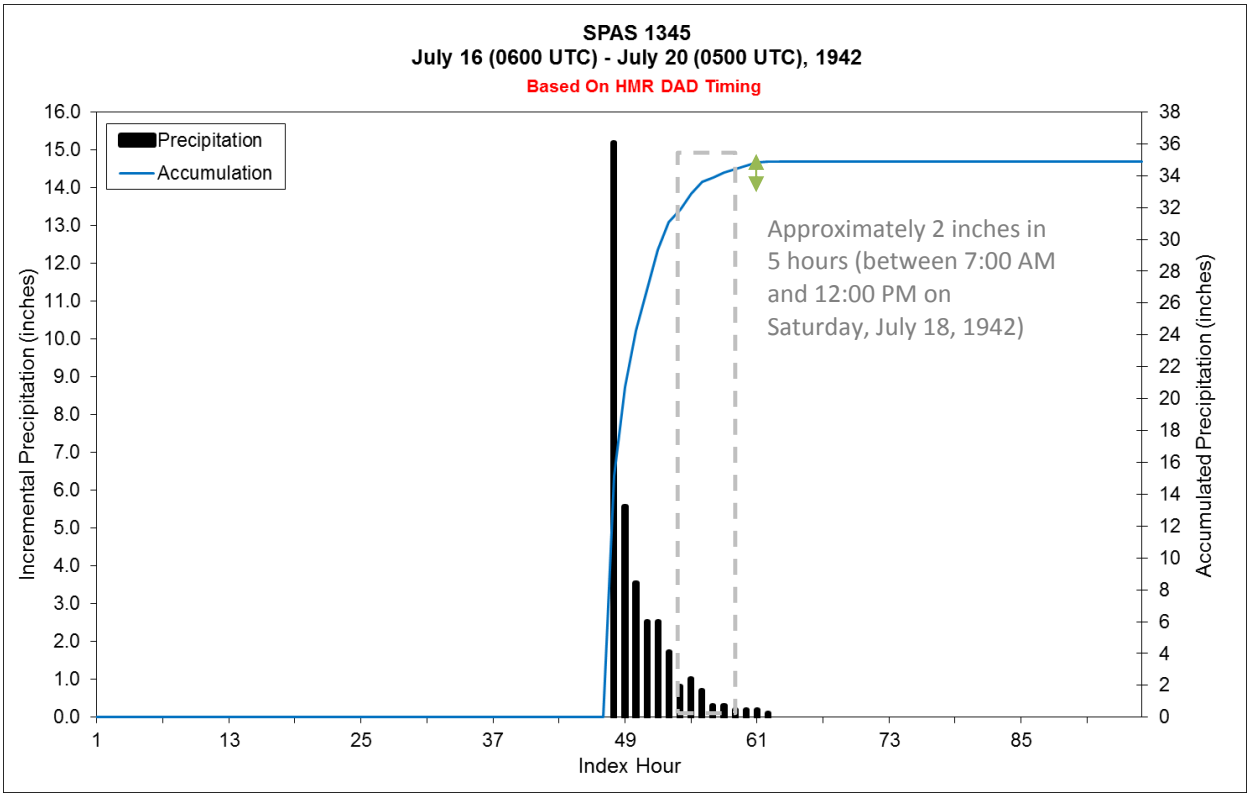


Figure 43. SCZ 1 Temporal Pattern (based on Bolivar Hourly Gauge)

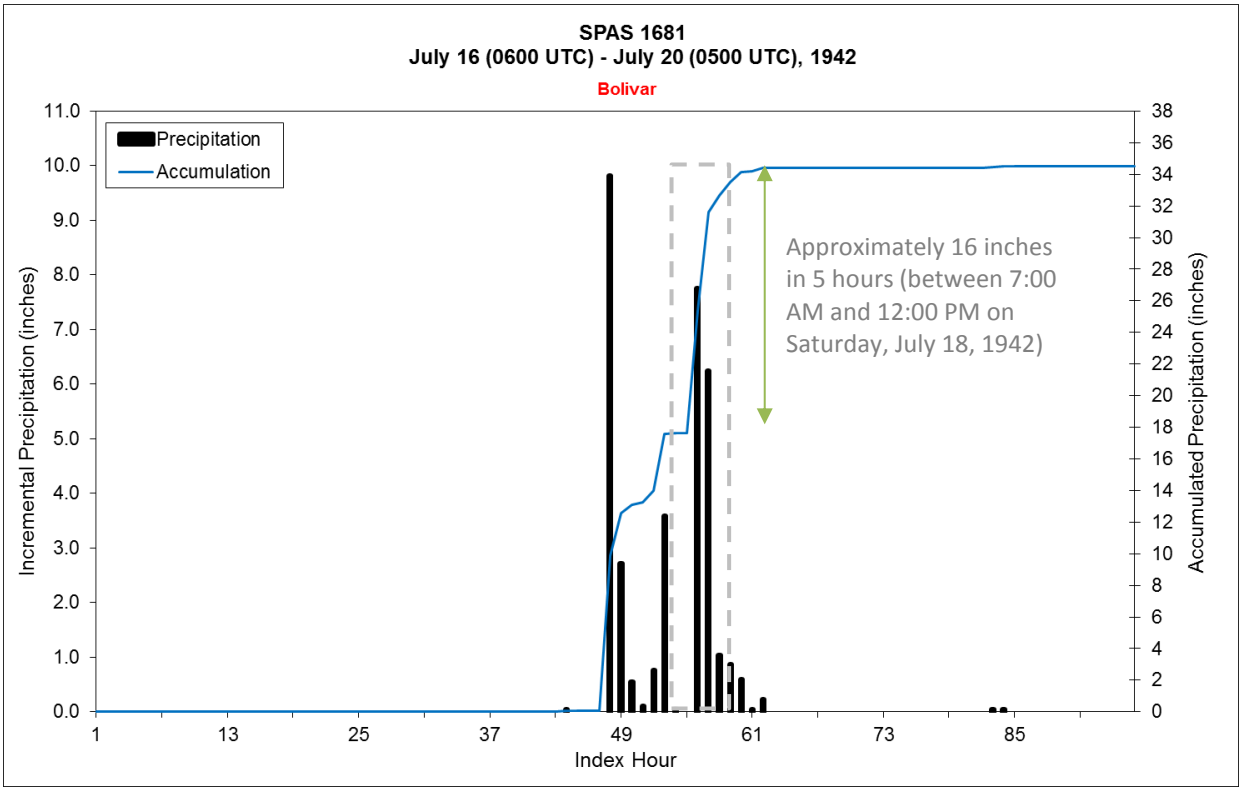


Figure 44. SCZ 2 Temporal Pattern (modified Bolivar Hourly Gauge)

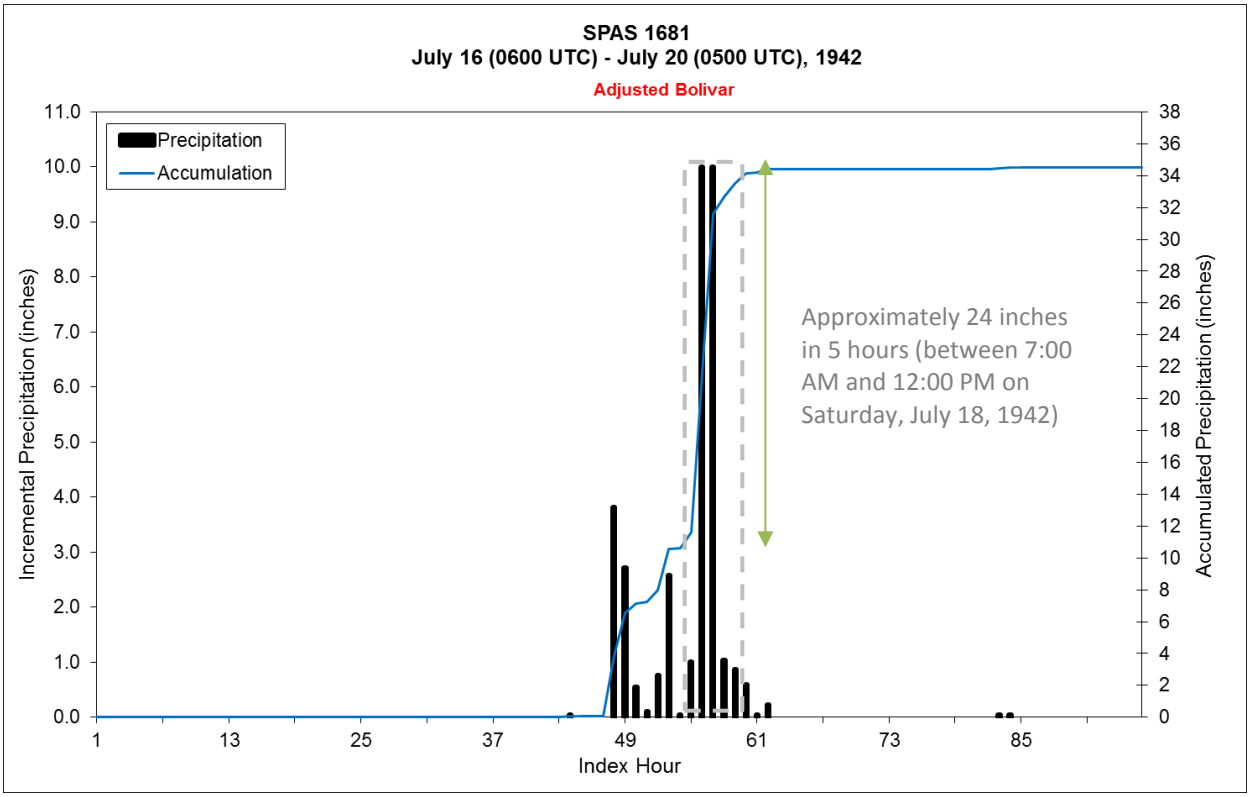


Figure 45. SCZ 3 Temporal Pattern (based on Site 275 (Appolt Farm) Report)

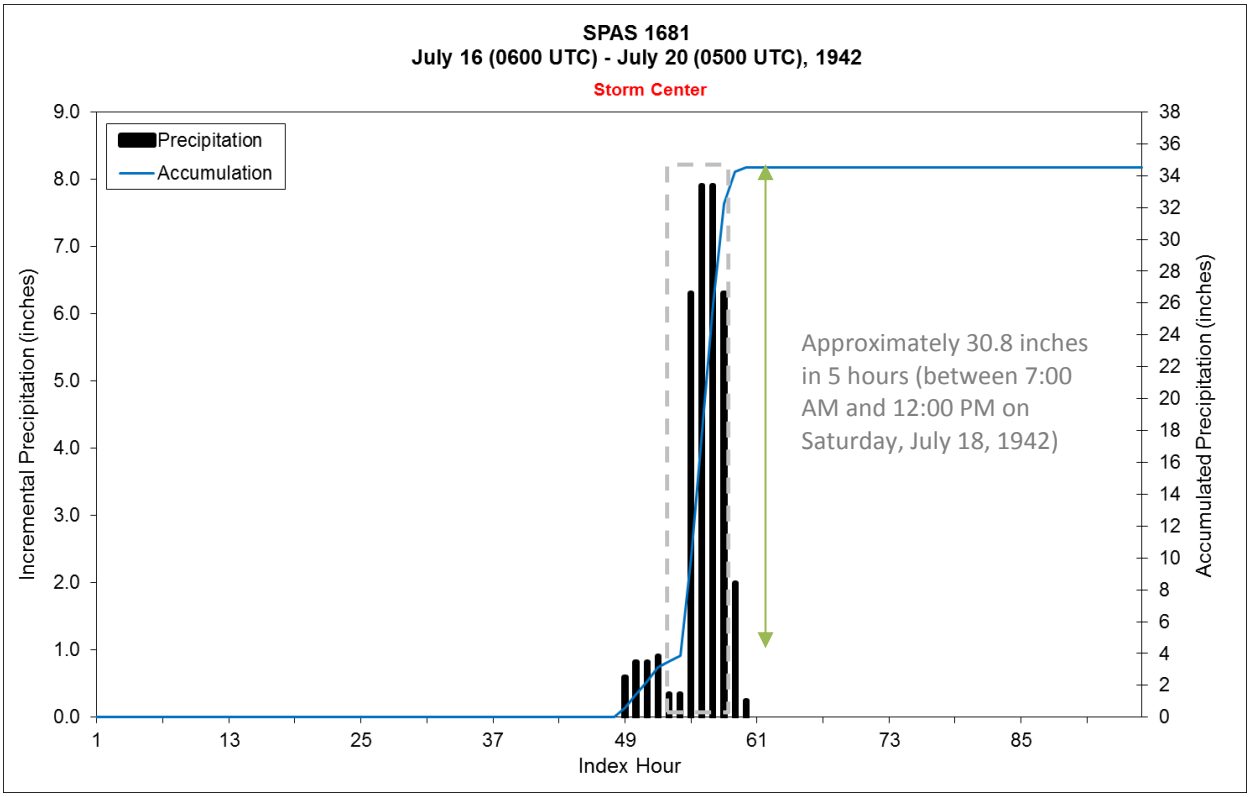
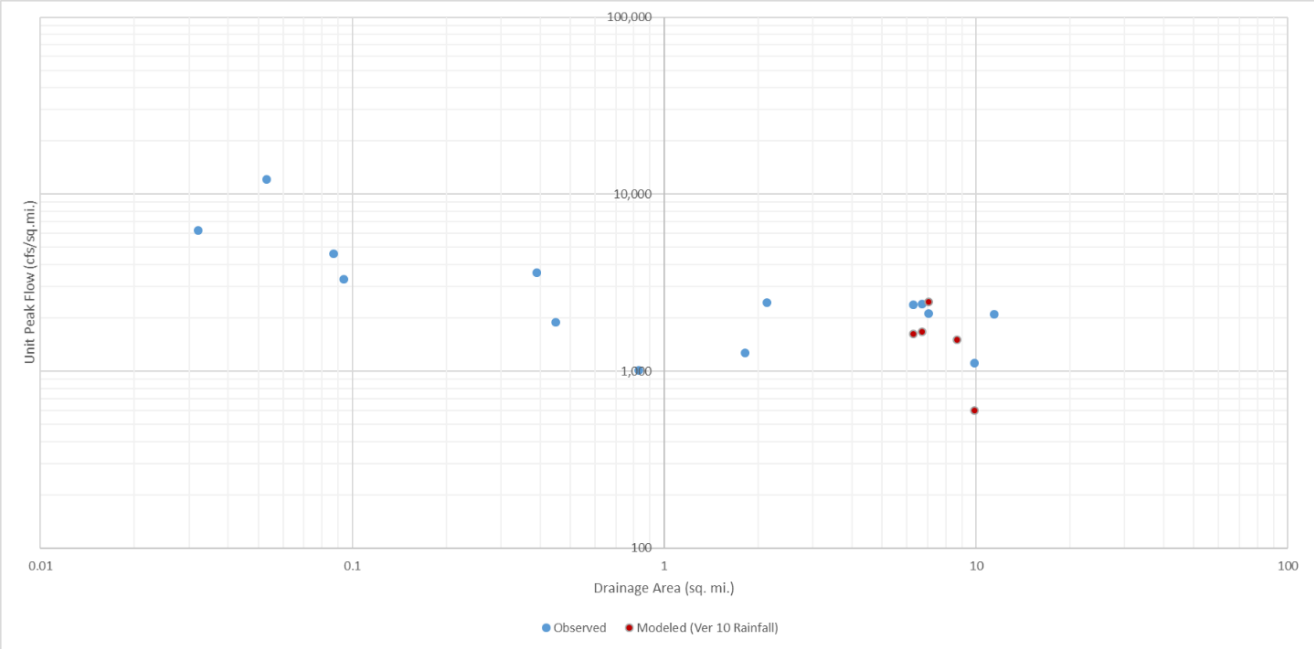


Figure 46. Observed & Modeled Unit Peak Flows vs. Drainage Area for Watersheds near Port Allegany



3.5 Insights into the Most Extreme Rainfall Observation (Site 275)

Even after establishing hydrologically viable rainfall patterns for tributaries and the main-stem Allegheny River near the storm center at Port Allegany, an additional analysis was conducted to further assess the hydrologic viability of the Site 275 observation. As discussed above, the Site 275 timing does produce good agreement with observed flows in Twomile Creek and Lillibridge Creek but significantly overestimates flooding in the main-stem Allegheny River when broadly applied. The additional analysis utilizes observed flows in small drainages and assisted in defining the limits of SCZ 3 in Figure 45. The flow observation locations from small drainages and the Site 275 rainfall observation are shown on Figure 47. Estimated using the NRCS lag time equation, the smallest of these drainage areas have lag times well below 1 hour. As such, observed peak flows are likely governed by sub-hourly rainfall intensities.

Since sub-hourly rainfall patterns are not being defined by the AWA SPAS analysis of the July 1942 storm, an analysis was conducted using the Rational Method (with the Runoff Coefficient (C) calibrated to RiverFlow2D results) to estimate the hourly rainfall intensities needed to produce the observed flows at each location. (See Figure 47 for locations of observed flow locations.) The results, shown in Table 14, indicate that significant rainfall intensities (between 17 and 45 inches per hour) could have occurred at flow locations 016.20, 016.21, and 016.22, located near the Site 275 rainfall observation. Rainfall intensities for other surrounding flow locations, including within the Twomile Creek, Lillibridge Creek, and Sartwell Creek watersheds, were estimated to be between approximately 6 and 16 inches per hour; which is consistent with the “Bolivar” and “modified Bolivar” timing in Figure 43 and Figure 44, respectively. The significant rainfall intensities needed to produce observed flows at locations 016.20, 016.21, and 016.22 suggest that the Site 275 (Applot) observation is viable but probably included a combination of steady heavy rainfall (consistent with the statement on page 67 of WSP-1134-B, above) and significant short-bursts at intensities between 17 to 45 inches per hour, accumulating to 30.8 inches between 7:00 AM to 12:00 PM on July 18. These extreme bursts may seem to contradict the statement on page 67 of WSP-1134-B but it is likely that the extreme bursts occurred at very localized areas in the headwaters of the small drainages, where no direct observations were made. See Figure 49 for an illustration.

Figure 47. Water Supply Paper 1134-B, Plate 2 (Map of Flood Area showing Locations of Stream-Gaging Stations, Rainfall-Measurement Points, and Isohyetal Lines for July 17-18, 1942)

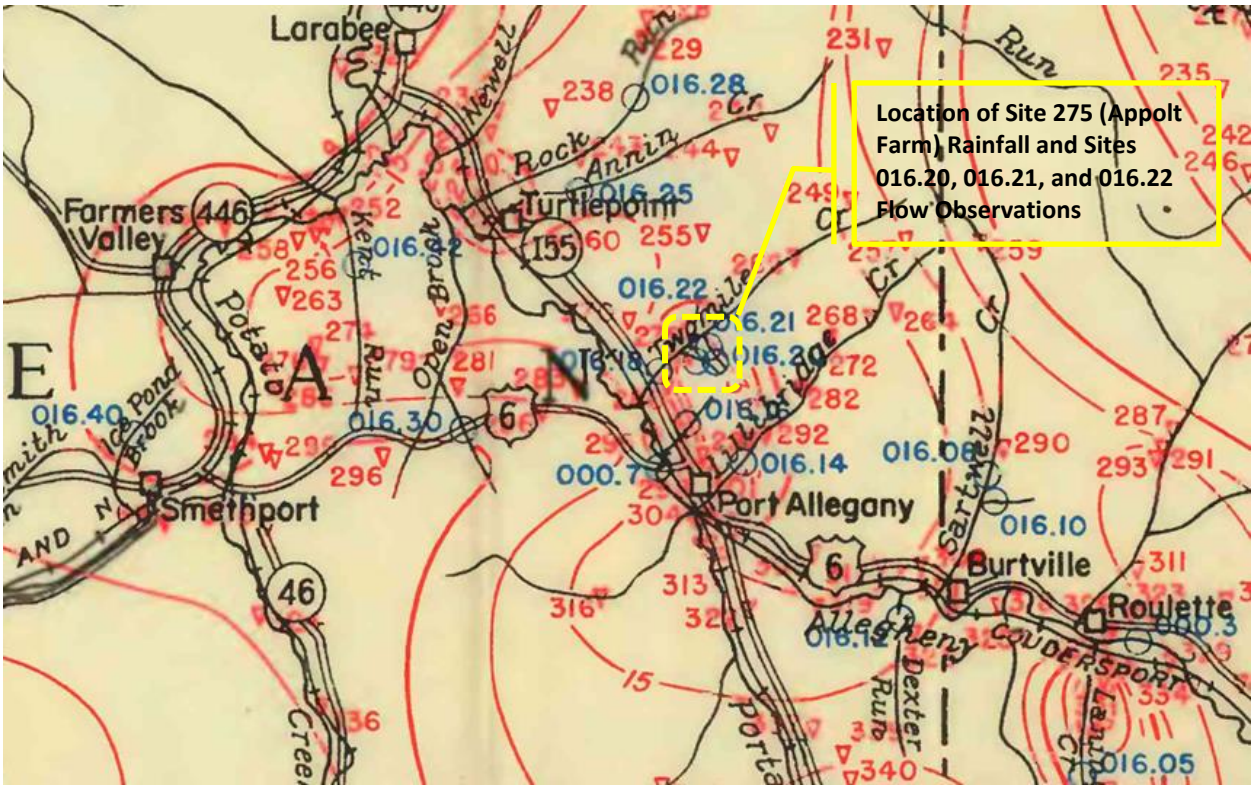


Figure 48. Map of Rainfall and Flow Observations at Storm Center

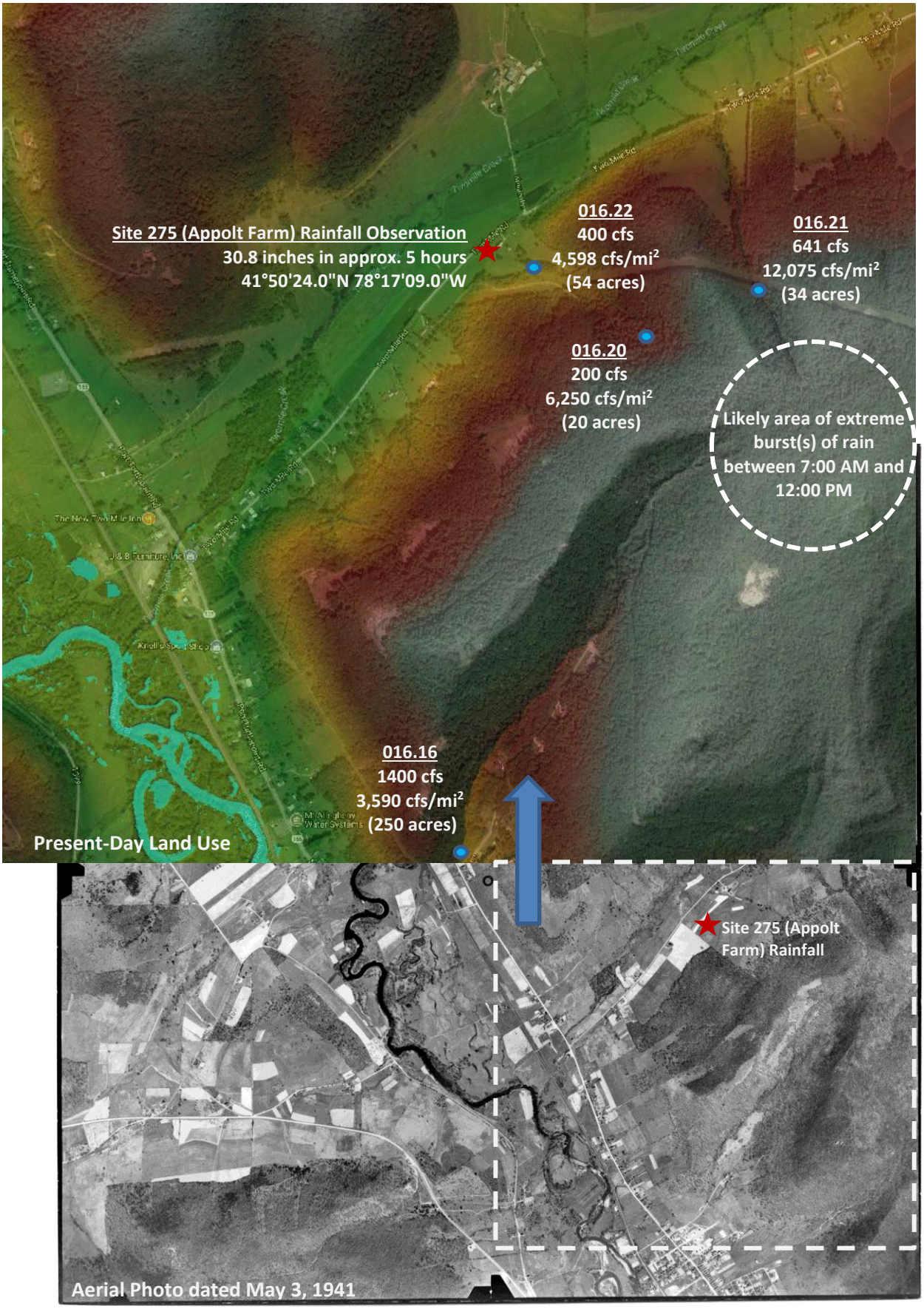


Table 14. Estimate of Rainfall Intensities needed to produce Observed Flows at Small Drainages near Port Allegany (based on Rational Method)

Watershed	Point #	Rational Runoff Coef (C)	Peak Intensity (in/hr)	Drainage Area (acres)	Peak Flow (cfs)	Flow per Sq Mile (cfs/mi^2)
Port Allegany	016.16	0.35	16.1	250	1,406	3,606
Two Mile Run	016.20	0.42	23.2	20	200	6,236
Two Mile Run	016.21	0.42	45.0	34	641	12,096
Two Mile Run	016.22	0.42	17.1	56	400	4,596
Sartwell Creek	016.10	0.32	16.1	60	310	3,297

4 Conclusions

PMP depths across much of the region covered by HMR 51 are greatly influenced by the exceptional July 1942 storm in the Smethport/Port Allegany region of north-central Pennsylvania. The rainfall measurement dataset for this storm includes several “bucket surveys”, which significantly influence the depth-area-duration characteristics of the storm. However, the quality of the “bucket survey” measurements is uncertain. Given the significance of this world-record-setting event in developing PMP values, an analysis of the resulting flood (using advanced modeling techniques and observed flood data) provided key insights into the rainfall observations. In some areas, the flood analysis corroborated the rainfall observations. In other areas, such as the upper Allegheny River (at and upstream of Port Allegany), Tunungwant Creek, and upper Oswayo Creek watersheds, flood data did not fully support the magnitude, spatial, and/or temporally information provided in the HMRs or as reported in hourly and “bucket survey” rainfall data.

Considering uncertainties in the flood models and quality of the flood data in addressing hydrologic differences, adjustments were made to the rainfall data until reasonable agreement was reached between the flood models, flood observations, and rainfall analysis. This combined the best aspects of the meteorological and hydrological analyses to produce the most accurate representation of the rainfall accumulation possible given the data available. Of particular focus was the location of the storm center near Port Allegany, PA, where the world-record-setting “bucket survey” rainfall that exceeded 30 inches in 4.5 hours was observed. From the flood analysis of the tributaries and small drainages at the storm center, it was concluded that the reported rainfall could have occurred, but its influence was very limited and there was a high-degree of spatial variability. The analysis led to refinements to the temporal and spatial patterns of the rainfall at the highly significant storm center. In the end, the flood analysis resulted in a more accurate depth-area-duration representation of this very important storm in Pennsylvania’s PMP development.

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6 National Inventory of Dams Database – Summary Table

Table 15 - National Inventory of Dams Database Summary

NID ID	Dam Name	Owner Name	Primary Purpose	Dam Type	River	City	County	State	Dam Length (Ft.)	Dam Height (Ft.)	Hydraulic Height (Ft.)	Volume	Year Completed	Spillway Width	Max Storage	Drainage Area
NY00456	Cuba Lake Outlet Spillway Dam	NYS Office Of General Services; Cuba Lake District	Recreation	Concrete	Cuba Lake Outlet	Maplehurst	Allegany	NY	136	9	0	0	1919	102	16,498	25.30
NY00455	Cuba Lake Dam	NYS Office Of General Services; Cuba Lake District	Recreation	Earth	Oil Creek	Cuba	Allegany	NY	1,750	55	0	0	1872	102	16,498	25.30
NY00571	Ischua Creek Watershed Dam #6a	Cattaraugus County	Flood Control	Earth	Gates Creek	Franklinville	Cattaraugus	NY	1,043	63	0	0	1971	488	3,890	19.00
NY00583	Ischua Creek Watershed Dam #1	Cattaraugus County	Flood Control	Earth	Ischua Creek	Machias	Cattaraugus	NY	490	27	0	0	1964	530	3,677	13.10
PA00026	Bradford City No 5 Dam	Bradford City Water Authority	Water Supply	Earth	West Branch Tunungwant Creek	-	Mckean	PA	1,200	68	68	544,000	1957	0	3,390	6.60
NY00565	Ischua Creek Watershed Dam #5	Cattaraugus County	Flood Control	Earth	Tr-Gates Creek	Franklinville	Cattaraugus	NY	1,693	54	0	0	1961	376	1,643	6.40
NY00626	Ischua Creek Watershed Dam #4	Cattaraugus County	Flood Control	Earth	Saunders Creek	Franklinville	Cattaraugus	NY	900	51	0	0	1961	309	1,011	4.10
NY16042	Bentley Wildlife Marsh Dam	Martyn Z. & Joan M. Bentley	Fish and Wildlife Pond	Earth	Bakerstand Creek	Machias	Cattaraugus	NY	1,100	10	0	0	2001	80	910	5.15
PA00024	Bradford City No 2 Dam	Bradford City Water Authority	Water Supply	Earth	Gilbert Run	-	Mckean	PA	850	44	44	166,222	1886	0	760	4.49
NY00560	Ischua Creek Watershed Dam #2	Cattaraugus County	Flood Control	Earth	Johnson Creek	Franklinville	Cattaraugus	NY	1,400	42	0	0	1961	160	647	2.80
NY00551	Ischua Creek Watershed Dam #3	Cattaraugus County	Flood Control	Earth	Tr-Ischua Creek	Franklinville	Cattaraugus	NY	1,220	38	0	0	1966	330	646	3.70
PA00025	Bradford City No 3 Dam	Bradford City Water Authority	Water Supply	Earth	Marilla Brook	-	McKean	PA	770	47	47	170,897	1898	0	502	4.80
NY00627	Harwood Lake Dam	NYS Dec Region 9	Recreation	Earth	Tr-Ischua Creek	Franklinville	Cattaraugus	NY	1,070	22	0	0	1963	110	350	0.00
NY01449	Beaver Lake Dam	Alma Rod & Gun Club	Recreation	Earth	Honeoye Creek	Alma	Allegany	NY	180	7	0	0	-	6	320	0.00
NY00589	Camp Lakeland Pond Dam	The Woods At Bear Creek, Llc	Recreation	Earth	Tr-Bear Creek	Franklinville	Cattaraugus	NY	850	47	0	0	1964	75	221	0.50
NY16145	Tannenbaum Reservoir Dam	Win-Sum Ski Corporation	Other	Earth	-	Ellicottville	Cattaraugus	NY	3,000	31	0	0	2006	0	220	0.00
PA01014	Hamlin Lake Park Dam	Borough of Smethport	Recreation	Earth	Marvin Creek	-	McKean	PA	653	10	10	8,465	1915	0	144	56.70
NY16105	Holimont Upper Reservoir Dam	Holimont Inc	Other	Earth	None	Ellicottville	Cattaraugus	NY	0	35	0	0	2003	0	129	0.00
NY00825	Edgar Ploetz Recreational Pond Dam	David Ploetz	Recreation	Earth	Beaver Meadow Creek	Ashford	Cattaraugus	NY	380	22	0	0	1969	17	91	0.60
PA01671	Clark Dam	Albert Clark	Recreation	Earth	Warner Brook	-	McKean	PA	600	16	16	17,778	1966	0	55	0.39
NY01353	Vee Pond Dam	Mary C Schlosser	Other	Earth	Morgan Hollow Run	Allegany	Cattaraugus	NY	245	16	0	0	1947	30	50	0.93
NY00826	William O Nannen Pond Dam	John D Northrup	Recreation	Earth	Tr-Great Valley Creek	Ellicottville	Cattaraugus	NY	1,230	15	0	0	1964	3	36	5.60
NY14130	Sunset Saddle Dam	Holimont Inc	Other	Earth	None	Ellicottville	Cattaraugus	NY	750	20	0	0	-	26	34	0.01
PA01715	Elk Lick Scout Reservation Dam	Allegheny Highlands Council	Recreation	Earth	Tr South Branch Cole Creek	-	McKean	PA	415	12.5	12.5	7,925	-	0	18	1.42