Appendix A. Hydrologic Model Development and Analyses

Appendix A provides engineering calculations associated with:

Tab 1. Squaw Run Existing Condition Hydrologic (HEC-HMS) Model Development

Tab 2. Squaw Run Proposed Condition Hydrologic (HEC-HMS) Model Development

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Tab 1. Squaw Run Existing Condition HEC-HMS Model Development

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Part I – Completed by Originator

1. Project Name:	Flood Risk and Storm Water 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 Names:	ArcMap 10.5.1; HEC-GeoHMS 10.4; HEC-HMS 4.4
Version Nos.:	As 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: N/A

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 number 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
T.H. Jackson, P.E.	JACKSON.THOMAS.HOFFMAN REYHER.1485301298 Digitally signed by JACKSON.THOMAS.HOFFMAN REYHER.1485301298 Date: 2020.10.05 14:33:37 -04'00'	October 5, 2020
Verifier(s)	**Signature**	**Date**
Howard Kellick, P.E.	KELLICK.HOWARD.BRUCE.1565106755 Digitally signed by KELLICK.HOWARD.BRUCE.1565106755 Date: 2020.10.05 14:40:47 -04'00'	October 5, 2020
Water Resources Section Chief	**Signature**	**Date**
Kyle Kaminski, P.E.	KAMINSKI.KYLE.MARTIN.1591285323 Digitally signed by KAMINSKI.KYLE.MARTIN.1591285323 Date: 2020.10.05 15:24:32 -04'00'	October 5, 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.

TABLE OF CONTENTS

1.0 STATEMENT OF PURPOSE

Squaw Run drains an area of approximately 8.2 square mile watershed to the Allegheny River. The area lies primarily within Fox Chapel Borough and O'Hara Township, as shown in the lower right panel of Figure 1.

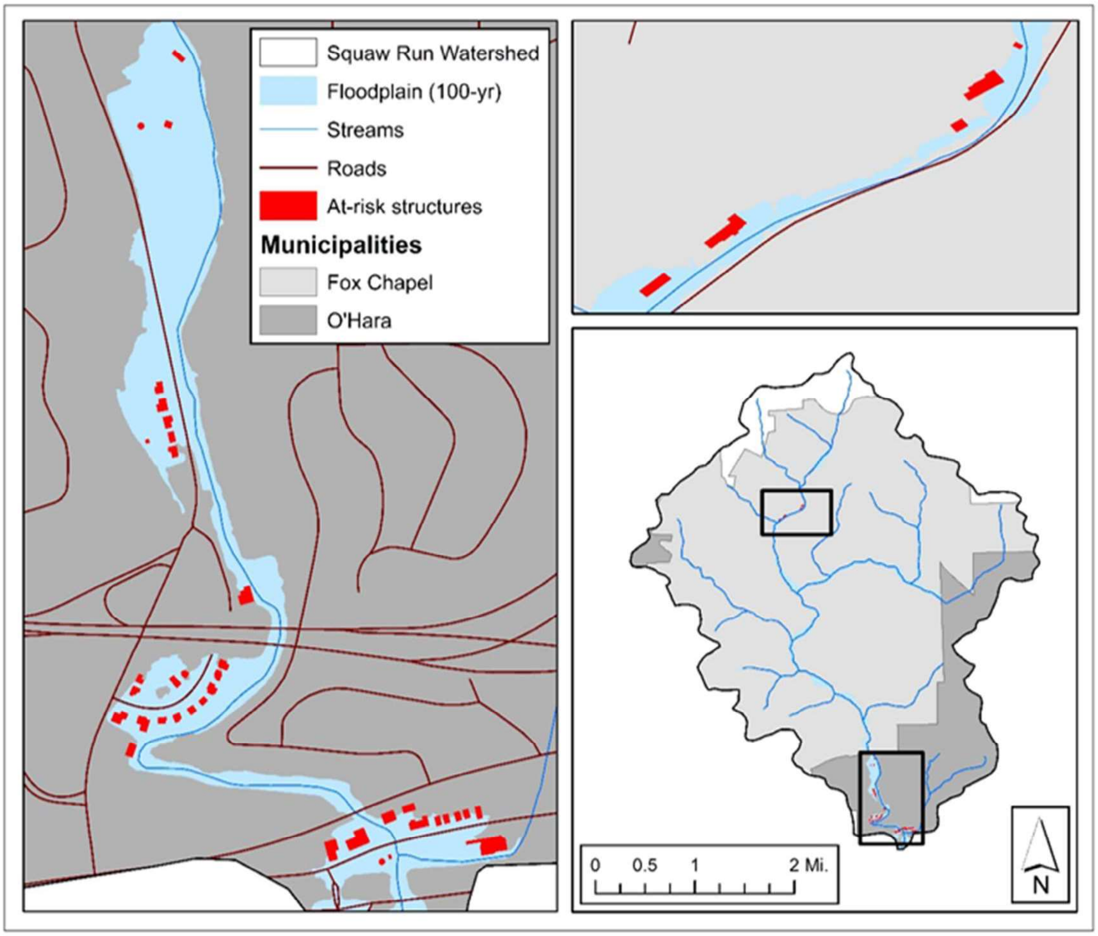

Figure 1. Location of 44 high-risk structures in the 100-year floodplain along Squaw Run within O'Hara Township and Fox Chapel Borough (dark gray and light gray coloring, respectively in lower right panel

Historic flooding on Squaw Run, including Hurricane Ivan in 2004 and intense localized precipitation events (e.g., July 2, 2018) have impacted many structures within the floodplain (see left and upper right panels of Figure 1) with some structures experiencing basement flooding multiple times per year.

Considering the historic flooding and the potential for increased future flooding, O'Hara Township and Fox Chapel Borough have jointly requested flood risk management assistance from USACE, Pittsburgh District through the Floodplain Management Services (FPMS) program.

This calculation brief describes work done to complete Task 1 of the program as described in the FPMS Study Scope, i.e., to develop a HEC-HMS simulation of existing condition hydrology within the Squaw Run watershed for return periods between 2 and 500 years. The HEC-HMS model and associated outputs will be used as inputs to HEC-RAS models of Squaw Run to be developed in subsequent tasks for the preliminary evaluation of alternative structural and non-structural flood mitigation strategies.

2.0 DESCRIPTION OF METHODOLOGY USED

HEC-GeoHMS (version 10.4) in ArcGIS (version 10.5) was used to delineate the Squaw Run watershed and subbasins and principal flow paths based on DEM data as shown in Figure 2.

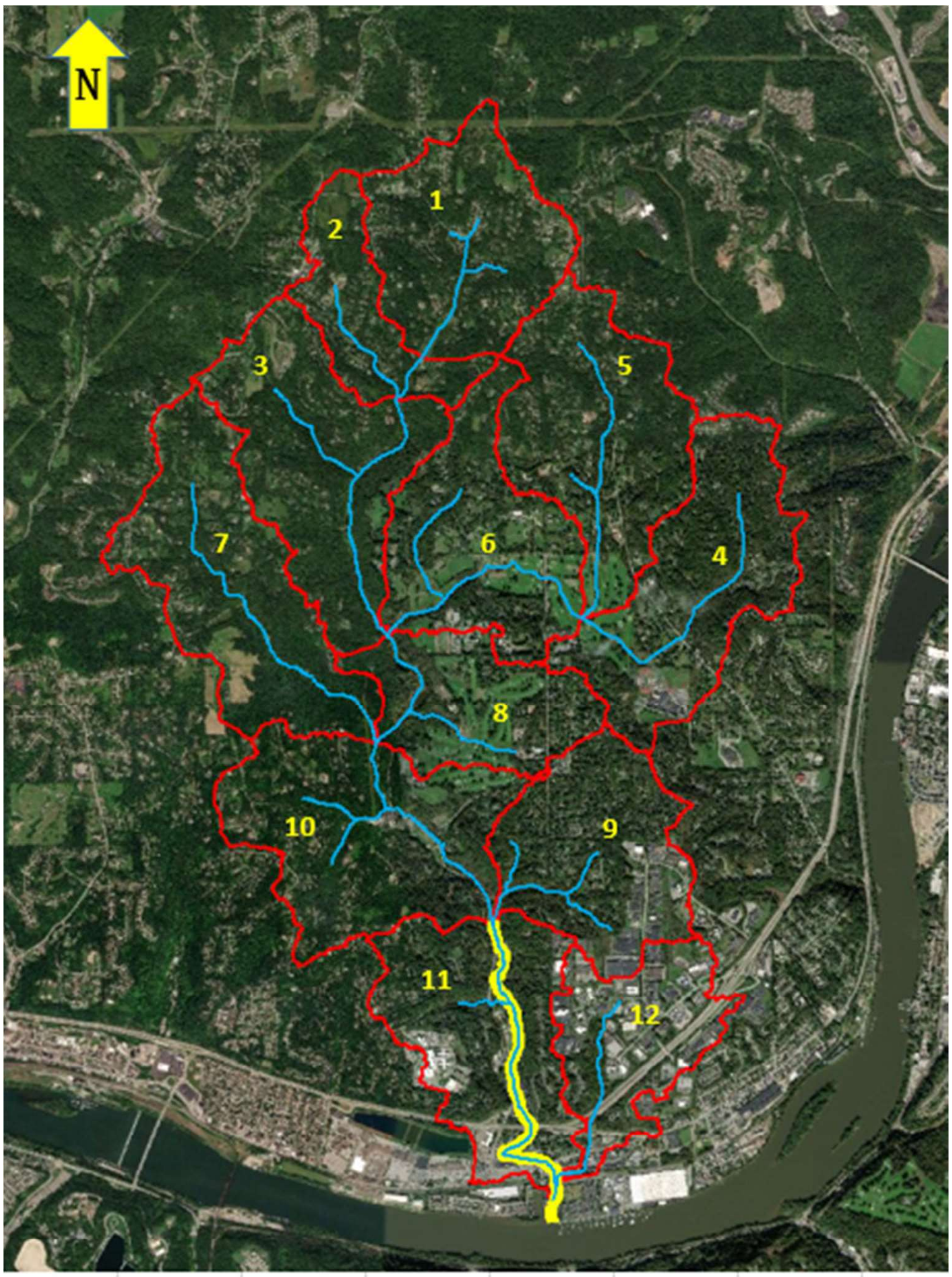

Figure 2: Extents of HEC-HMS Study Area - no scale
(HEC-RAS study reach shown with yellow outline)

The HEC-HMS model (Version 4.4) of the study watershed was developed using data processing utilities accessed from the HEC-GeoHMS extension within ArcGIS in accordance with the HEC-GeoHMS User's Manual (USACE, 2013). Work included the following tasks:

- Delineate the watershed and subbasins using 3-meter terrain data from USGS (see Section 4.0 for more details). When stage-storage-discharge data for the Campbell Lake Stormwater Management Pond (SWM) was subsequently obtained, the headwater subbasin delineated in HEC-GeoHMS was subdivided at the SWM pond outlet, as located in ArcGIS, to form subbasins 1 and 2 (see Figure 2), permitting the modeling of the impact of the structure on downstream flows on Squaw Run.
- Calculate river lengths, river slopes, basin slopes, longest flow path, basin centroid, centroid elevation, and the longest flow path from the centroid to the outlet
- Calculate time of concentration (t_c) with additional input estimating Manning's roughness coefficients and typical channel geometry (See Section 3.0 for more details)
- Calculate effective curve numbers for each subbasin with additional input from soil and land cover datasets (See Section 4.0 for more details)

Within HEC-HMS, the following simulation modules were used:

- Loss: SCS Curve Number
- Transform: SCS Unit Hydrograph
- Base flow: No base flow
- Routing: Muskingum-Cunge Eight Point

GIS-based maps of watershed topography, hydrologic soil and land use mapping, and resultant CN value mapping used in the calculations, as bulleted above, are provided in Appendix A.

The SCS hydrograph method uses lag time, equal to $0.6*t_c$, as a key parameter for hydrograph transformation. For the t_c calculations, the Natural Resources Conservation Service (NRCS) Technical Release 55 (TR-55) velocity methodology was used. The sum of the travel time from sheet flow, shallow concentrated flow, and open channel flow are summed to calculate t_c. There is a worksheet embedded within HEC-GeoHMS that uses this methodology to calculate the t_c. In addition to the Manning's roughness coefficient assumptions, this worksheet requires a wetted perimeter and flow area. The worksheet is set-up to accept a trapezoidal section with an assumed depth.

The routing parameters (i.e., stream length and slope) were primarily calculated within HEC-GeoHMS as outlined above.

Similar to the t_c calculations outlined above, assumptions were made in estimating the typical Manning's roughness coefficients (Section 3.0). Also, representative eight point cross sections for each reach, needed as input for the Muskingum-Cunge eight point routing, were estimated by cutting a cross section in HEC-RAS RasMapper.

Information regarding the Campbell Lake Storm Water Management (SWM) structure was obtained subsequent to the initial model being constructed. The basin was split into two subbasins, one upstream of the Campbell Lake SWM and one downstream. A review of the data showed that the headwater subbasins, delineated in HEC-GeoHMS, was relatively homogenous. Therefore, the same loss parameters were used for the two parts it was divided into.

The impact of the existing Campbell Lake SWM structure was modeled with level pool reservoir routing utilizing the Elevation-Storage-Discharge method, with Storage-Discharge and Elevation-Storage functions based on data provided in a report to the Borough (Partridge, 1988)

The SCS hydrograph method was utilized in HEC-HMS for the parameterization of the rainfall-runoff process in the Squaw Run watershed for this study.

Meteorologic models were developed in HEC-HMS to model the 2-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence interval flows using the Frequency Storm methodology. The following parameters were used: 24-hour storm duration, 5-minute intensity duration, 50% intensity position, TP40 area reduction, and a storm area of 10 square miles. NOAA Atlas 14 data for the centroid of the Squaw Run basin was also added for the appropriate meteorologic model (Section 4.0).

3.0 ASSUMPTIONS AND JUSTIFICATION

Model development was based on the following assumptions.

1. Information regarding the Lake Campbell SWM pond, including the elevation, stage, and outflow relationship were obtained from Table 8 of the design study (Partridge, 1988). Aerial imagery, gathered via Google Earth, aligns with the assumption that the proposed facility was built. The pond was modeled in HEC-HMS assuming that it was constructed with the proposed dimensions and elevations.

2. For the purposes of Muskingum routing, the channel bottom was assumed to be 5 feet wide with side slopes of 3H:1V. The depth was assumed to be 2 feet. That width is in order of magnitude agreement with cross sections determined from the digital terrain data for the development of the Muskingum-Cunge 8-point cross sections used in HEC-HMS (see below).

3. A 24-hour rainfall event was used as input for simulating runoff in the watershed for all return periods evaluated. This complies with the requirement that storm duration must be greater than the time of time of concentration to accurately model the peak flow (USACE, 2000). An additional requirement is that the storm duration should be 3 or 4 times the time of concentration (Placer, 1990). Extending the TR-55 velocity method downstream to the outlet of the basin, yields a t_c estimate of approximately 4.5 hours, which complies with this requirement.

4. For the hydrologic routing along the main channel within HEC-HMS, Manning's roughness coefficients were estimated to be 0.06 for all channels and 0.10 for all overbanks based on engineering judgment. These assumptions impact the tc

calculations within HEC-GeoHMS and the routing calculations within HEC-HMS. The current FEMA FIS for the area of interest used values for the channel ranging from 0.015 to 0.059. Two of the four streams modeled in the FIS used a channel roughness coefficient ranging from 0.055 to 0.059. The overbank roughness coefficients used by the FIS ranged from 0.07 to 0.12.

5. Base flow was assumed to be negligible in comparison with the peak discharges in the Squaw Run watershed, and was set to zero. The watershed is located within the Pittsburgh Low Plateau physiographic Region (DCNR, 2018). A study of base flow generation for small watersheds in Virginia (USGS, 1997) reported median values of base flow for physiographic provinces similar to the Pittsburgh Low Plateau Section of between .50 and .82 cfs/mi^2 (USGS, 1997). Using a conservatively high value 1.0 cfs/mi^2 for the watershed would result in a base flow of approximately 8 cfs, confirming the reasonableness of the assumption that base flow is negligible in comparison with peak flows for the study area which are nearly three orders of magnitude higher.

6. The initial abstraction of rainfall (Ia) for each subbasin was assumed to be equal to 20% of the potential maximum moisture retention after runoff begins (S) in accordance with current guidelines of the Soil Conservation Service (SCS) Runoff Curve Number (CN) methodology (NRCS, 2019).

7. To generate the effective CN for each subbasin, land use and soil data were combined. The CN for each combination of land use and soil type combination is shown in Table 1.

Table 1: CN Look-Up Table

Grid Code	Description	CN for Hydro Soil Group				Assumptions
		A	B	C	D	
1	Water	0	0	0	0	No losses for rain on water
2	Transportation	98	98	98	98	assumed "impervious"
3	Forest	30	55	70	77	Woods - good condition
4	Grasslands	39	61	74	80	Pasture, grassland, or range - good condition
5	Agriculture	64	75	82	85	assumed row crops, straight rows, crop residue cover (SR _ CR) - good condition
6	Low-Density Residential	51	68	79	84	assumed 1 acre as Fox Chapel has minimum of 1 acre lot sizes
7	Medium-Density Residential	61	75	83	87	assumed 1/4 acre lot size (very little medium density in study area)
8	High-Density Residential	77	85	90	92	assumed 1/8 acre lot size
9	Identified Malls	89	92	94	95	Commercial and business
10	Commercial	89	92	94	95	Commercial and business
11	Light Industrial	81	88	91	93	assumed "Industrial"
12	Heavy Industrial	81	88	91	93	assumed " Industrial "
13	Strip Mine	77	86	91	94	assumed "Fallow - bare soil"
14	Non-Vegetative	77	86	91	94	assumed "Fallow - bare soil"

4.0 CALCULATION INPUT

The development of the HEC-HMS model used to calculate design flow rates for the flood control study included utilizing the following input.

Subwatersheds were delineated in HEC-GeoHMS utilizing 3-meter USGS digital elevation model (DEM) 2015 data downloaded from here: https://www.usgs.gov/core-science-systems/ngp/3dep/about-3dep-products-services. See Appendix A, Figure 2 for topography developed from the 3-meter DEM data.

Eight-point cross sections used for estimating channel routing parameters was developed from 1-meter LiDAR terrain data developed between 2006 and 2008, downloaded from: https://www.pasda.psu.edu/uci/DataSummary.aspx?dataset=1247

Soil characteristics required for the calculation of SCS Runoff Curve Numbers were developed from NRCS Web Soil Survey Geographic Database (SSURGO). The SSURGO database contains information about soil as collected by the National Cooperative Soil Survey over the course of a century based on field studies and mapping at scales between 1:12,000 and 1:63,360. (https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm)

Land use characteristics required for the calculation of SCS Runoff Curve Numbers were developed from the Allegheny County 2018 Land Cover dataset (https://catalog.data.gov/dataset/allegheny-county-land-cover-areas-aebce)

Input hyetographs were developed for the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year design storms utilizing the Frequency Storm method implemented in HEC-HMS. Inputs for each storm were based on rainfall depths obtained from 2017 NOAA Atlas 14 data for a 10-square mile storm centered over the Squaw Run watershed (see Table 2).

Table 2: NOAA Atlas 14 Precipitation Depths (inches) for Centroid of Squaw Run Watershed[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.317	0.378	0.457	0.518	0.596	0.655	0.712	0.772	0.85	0.907
10-min	0.492	0.59	0.711	0.8	0.911	0.993	1.07	1.15	1.25	1.32
15-min	0.603	0.721	0.872	0.984	1.13	1.23	1.33	1.43	1.56	1.65
30-min	0.798	0.965	1.2	1.37	1.59	1.76	1.92	2.09	2.31	2.48
60-min	0.974	1.18	1.5	1.74	2.06	2.32	2.57	2.84	3.19	3.48
2-hr	1.12	1.37	1.72	2	2.38	2.68	2.99	3.32	3.76	4.11
3-hr	1.19	1.45	1.81	2.11	2.52	2.85	3.19	3.55	4.04	4.44
6-hr	1.43	1.72	2.14	2.48	2.97	3.36	3.78	4.22	4.84	5.34
12-hr	1.68	2.02	2.49	2.88	3.44	3.9	4.39	4.9	5.65	6.25
24-hr	2.01	2.4	2.93	3.36	3.98	4.49	5.01	5.57	6.36	6.99

Note:

1. Data downloaded from https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html (for Latitude: 40.5226°, Longitude: -79.8863°). See Appendix B

Calculation of Lag Time

The lag time, input to HEC-HMS for use in the transformation of the SCS hydrograph for each subbasin, is based on the time of concentration, calculated as the sum of sheet flow, overland flow, and in-channel flow, as shown in Tables 3a and b.

Table 3-a – Calculation of Lag Time in Subwatersheds 1 to 6 (see Figure 2)

Watershed Name	SquawHW-US	SquawHW-DS	SquawHWMid	GladeRunHW	GladeRunMid	GladeRun
Map ID	1	2	3	4	5	6
Sheet Flow Characteristics						
Manning's Roughness Coefficient	0.4	0.4	0.4	0.4	0.4	0.4
Flow Length (ft)	100	100	100	100	100	100
Two-Year 24-hour Rainfall (in)	2.4	2.4	2.4	2.4	2.4	2.4
Land Slope (ft/ft)	0.034	0.034	0.0899	0.0136	0.1357	0.1295
Sheet Flow Tt (hr) = .007*(nL)^0.8/(P2^0.5*S^0.4)	**0.33**	**0.33**	**0.23**	**0.48**	**0.19**	**0.20**
Shallow Concentrated Flow Characteristics						
Surface Description (1 - unpaved, 2 - paved)	1	1	1	1	1	1
Flow Length (ft)	2800	2400	2931	3620	2324	3920
Watercourse Slope (ft/ft)	0.0499	0.0499	0.0535	0.0611	0.0743	0.0481
Average Velocity - computed (ft/s)	3.60	3.60	3.73	3.99	4.40	3.54
Shallow Concentrated Flow Tt (hr)	**0.22**	**0.18**	**0.22**	**0.25**	**0.15**	**0.31**
Channel Flow Characterisitics						
Depth	2	2	2	2	2	2
Channel width	5	5	5	5	5	5
Side slopes (xH:1V)	3	3	3	3	3	3
Flow Area A = (b+zy)*y	22	22	22	22	22	22
wetted perimeter P = b + 2y*(1 + z^2)^0.5 =	17.6	17.6	17.6	17.6	17.6	17.6
Hydraulic Radius R = A/P	1.25	1.25	1.25	1.25	1.25	1.25
Channel Slope (ft/ft)	0.02	0.005	0.0227	0.0152	0.0171	0.0318
Manning's Roughness Coefficient	0.06	0.06	0.06	0.06	0.06	0.06
Average Velocity (ft/s) = 1.49*(R^(2/3)S^0.5)/n	4.07	2.03	4.33	3.55	3.76	5.13
Flow Length (ft)	2800	3400	7519	7845	7513	5328
Channel Flow Tt (hr)	**0.19**	**0.46**	**0.48**	**0.61**	**0.55**	**0.29**
Watershed Time of travel (hr) = SF +SCF +CF	0.74	0.98	0.93	1.35	0.89	0.79
Watershed Travel time (minutes)	44.47	59.01	55.60	80.93	53.63	47.52
Lag time = 0.6 x Travel Time	26.68	35.41	33.36	48.56	32.18	28.51

Table 3-b – Calculation of Lag Time in Subwatersheds 7 to 12 (see Figure 2)

Watershed Name	StonyCampRun	SquawMid	EastTrib	SquawMidLow	SquawLow	RIDCTrib
Map ID	7	8	9	10	11	12
Sheet Flow Characteristics						
Manning's Roughness Coefficient	0.4	0.4	0.4	0.4	0.4	0.4
Flow Length (ft)	100	100.0001	99.9998	100	99.9999	100.0002
Two-Year 24-hour Rainfall (in)	2.4	2.4	2.4	2.4	2.4	2.4
Land Slope (ft/ft)	0.0443	0.1409	0.0987	0.0414	0.2621	0.2469
Sheet Flow Tt (hr) = .007*(nL)^0.8/(P2^0.5*S^0.4)	**0.30**	**0.19**	**0.22**	**0.31**	**0.15**	**0.15**
Shallow Concentrated Flow Characteristics						
Surface Description (1 - unpaved, 2 - paved)	1	1	1	1	1	1
Flow Length (ft)	3044.2759	3243.4605	4025.0611	2916.321	3513.3683	4199.0024
Watercourse Slope (ft/ft)	0.0376	0.0475	0.0384	0.0573	0.083	0.0421
Average Velocity - computed (ft/s)	3.13	3.52	3.16	3.86	4.65	3.31
Shallow Concentrated Flow Tt (hr)	**0.27**	**0.26**	**0.35**	**0.21**	**0.21**	**0.35**
Channel Flow Characterisitics						
Depth	2	2	2	2	2	2
Channel width	5	5	5	5	5	5
Side slopes (xH:1V)	3	3	3	3	3	3
Flow Area A = (b+zy)*y	22	22	22	22	22	22
wetted perimeter P = b + 2y*(1 + z^2)^0.5 =	17.6	17.6	17.6	17.6	17.6	17.6
Hydraulic Radius R = A/P	1.25	1.25	1.25	1.25	1.25	1.25
Channel Slope (ft/ft)	0.0302	0.0253	0.0337	0.0296	0.0123	0.0375
Manning's Roughness Coefficient	0.06	0.06	0.06	0.06	0.06	0.06
Average Velocity (ft/s) = 1.49*(R^(2/3)S^0.5)/n	5.00	4.58	5.28	4.95	3.19	5.57
Flow Length (ft)	8913.9871	4250.6171	4028.699	6510.7094	7526.004	5511.6433
Channel Flow Tt (hr)	**0.50**	**0.26**	**0.21**	**0.37**	**0.66**	**0.27**
Watershed Time of travel (hr) = SF +SCF +CF	1.07	0.70	0.78	0.88	1.01	0.78
Watershed Travel time (minutes)	63.98	42.21	47.03	53.05	60.78	46.71
Lag time = 0.6 x Travel Time	38.39	25.33	28.22	31.83	36.47	28.02

A summary of the hydrologic parameters used for each subwatershed is provided in Table 4.

Table 4: Hydrologic Parameters Used in HEC-HMS for Each Subwatershed

HEC-HMS name	Map ID (See Fig. 2)	Area, sq. mi.	I_a	CN[1]	Lag, minutes
SquawHW-US	1	0.710	0.78	72.04	34.14
SquawHW-DS	2	0.375	0.78	72.04	53.52
SquawHWMid	3	0.865	0.83	70.75	52.16
GladeRunHW	4	0.799	0.80	71.52	72.52
GladeRunMid	5	0.818	0.87	69.78	53.81
GladeRun	6	0.725	0.83	70.62	39.77
StonyCampRun	7	0.872	0.84	70.54	57.71
SquawMid	8	0.520	0.85	70.46	35.39
EastTrib	9	0.681	0.72	73.41	36.48
SquawMidLow	10	0.923	0.88	69.39	46.08
SquawLow	11	0.375	0.73	73.36	62.03
RIDCTrib	12	0.505	0.57	77.74	38.74
Total area		8.168			

Note:

1 – Area-weighted CN values are reported; impervious area is accounted for in the calculation of CN

Muskingum-Cunge Routing within Squaw Run Channels

The Muskingum Cunge routing method was used to simulate flood wave attenuation within the Squaw Run channel system. The physically-based parameters used in the simulation are summarized in Table 5. Reference flows were taken as 10% of the peak flow within the basin.

Table 5 – Muskingum-Cunge Routing Parameters Used in HEC-HMS Model

Rout reach	Initial type	Length, feet	slope	Manning's n			Index flow, cfs
				Left O.B.	Channel	Right OB	
R100	Discharge = Inflow	2,687.4	0.0169	0.1	0.06	0.1	87.86
R130	Discharge = Inflow	4,890.5	0.0071	0.1	0.06	0.1	98.15
R150	Discharge = Inflow	4,333.3	0.0136	0.1	0.06	0.1	87.54
R160	Discharge = Inflow	1,800.1	0.0113	0.1	0.06	0.1	98.08
R170	Discharge = Inflow	2,447.3	0.0106	0.1	0.06	0.1	271.78
R200	Discharge = Inflow	1,102.8	0.0123	0.1	0.06	0.1	271.48
R210	Discharge = Inflow	1,761.5	0.0095	0.1	0.06	0.1	347.72
R240	Discharge = Inflow	1,714.6	0.0080	0.1	0.06	0.1	346.75
R310	Discharge = Inflow	2,462.8	0.0094	0.1	0.06	0.1	345.73
R330	Discharge = Inflow	2,196.8	0.0051	0.1	0.06	0.1	413.68
R350	Discharge = Inflow	6,074.2	0.0070	0.1	0.06	0.1	410.95

Stormwater Management Pond Simulation

Stage-volume-discharge relations for the Lake Campbell Stormwater Structure were obtained from the design study (Partridge, 1988) as summarized in Table 6 and shown graphically in Figure 3. It is assumed that the proposed spillway and outlet works described by the data provided in Table 8 of the report was constructed as designed. Accordingly, that data was input as the elevation-storage and storage-discharge functions used for level-pool hydrograph routing for the HEC-HMS Reservoir Node “CampbellLake” located at the outlet of the subwatershed “SquawHW-US.”

For the model runs, it was assumed that the riser pipe was not blocked; the initial water surface elevation was estimated as 968 feet, which coincides with a depth of eight feet, a little less than half the volume of the reservoir. As the pond drains quickly, this was considered a reasonably conservative starting point – equivalent to a significant rainfall one or two days before the design event.

Table 6 – Reservoir Routing Parameters for Campbell Lake Simulation

Depth, feet	Elevation, feet [1]	Storage, acre-feet	Discharge, cfs
0	960	0.0	0
2	962	0.4	15
4	964	2.6	26
6	966	6.4	34
8	968	17.1	40
10	970	21.7	45
12	972	32.0	250
13	973	37.4	1360

Note: 1) Depths reported in Partridge, 1988 converted to elevation with assumed datum higher than invert of the R100 cross section

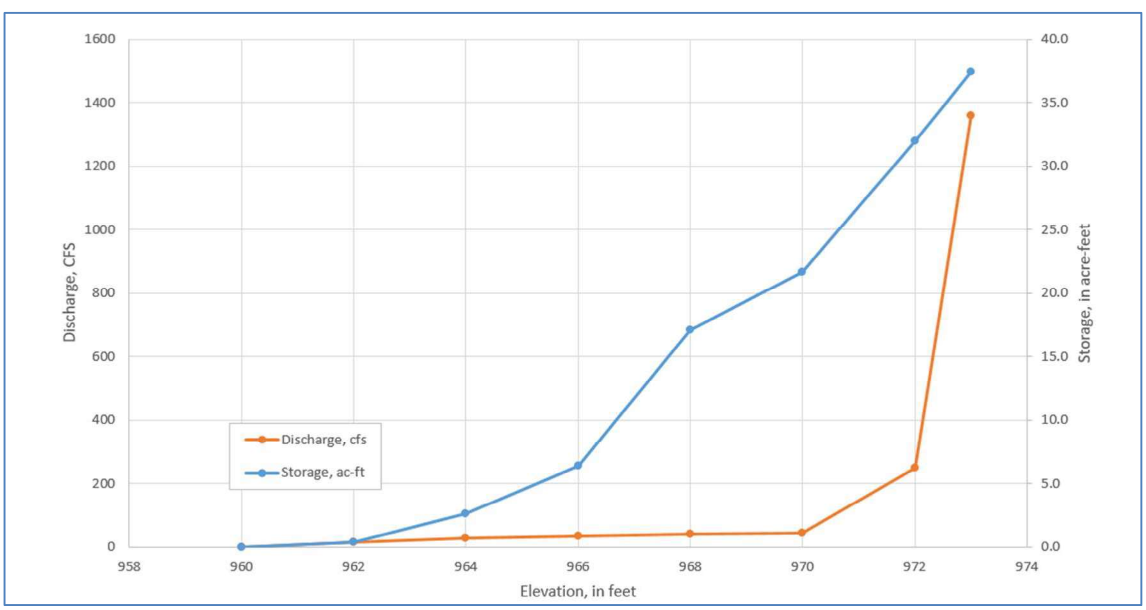

Figure 3 – Stage-Storage-Discharge Parameterization for the Campbell Lake SWM Pond

5.0 CALCULATION OUTPUT/RESULTS

Output is located in the basin model of the HEC-HMS project "***SquawRunModeling.hms***" labeled "RevisedExisting for each simulation. The HEC-HMS project is saved in L:\EC\EC-WH\Planning_FPMS\2020SquawRun\2_Working. An electronic copy of the input and output files is provided as Appendix D of this calculation brief.

5.1 24-HOURFREQUENCY STORMS

Table 7 shows the peak discharges for each subwatershed and at the outlet of the entire watershed as simulated in the HEC-HMS model ***SquawRunModeling.hms*** for the 24-hour 2-, 10-, 25-, 50-, 100-, 200-, and 500-year return period storm events.

Table 7: Peak Discharges within the Squaw Run Watershed for Existing Conditions (see Figure 2)

Subwatershed	2-year Qpeak, cfs	10-year Peak Q, cfs	25-year Peak Q, cfs	50-year Peak Q, cfs	100-year Peak Q, cfs	200-year Peak Q, cfs	500-year Peak Q, cfs
SquawHW-US	125.7	309.9	440.6	551.7	665.8	790.8	965.6
SquawHW-DS	49.9	130.9	189.7	240.4	292.7	351.2	432.3
SquawHWMid	106.2	289.4	427.2	546.5	670.4	807.1	996.8
GladeRunHW	83.0	219.7	321.0	409.7	503.0	606.4	750.5
GladeRunMid	91.6	261.8	390.7	503.0	619.8	749.0	928.5
GladeRun	96.6	265.1	390.9	499.2	611.5	735.1	906.1
StonyCampRun	95.3	262.5	388.3	498.7	613.9	741.4	918.9
SquawMid	71.0	198.6	292.8	375.1	460.6	554.4	684.3
EastTrib	122.7	302.4	429.4	537.0	647.3	767.8	932.8
SquawMidLow	99.2	289.0	433.6	559.7	691.1	836.5	1038.9
SquawLow	57.2	141.4	201.5	252.9	305.8	363.8	443.6
RIDCTrib	136.6	290.1	393.1	478.5	564.8	658.0	783.8
Squaw Run (at the outlet)	496.9	1367.3	2204.9	2959.1	3702.9	4670.3	6150.7

Table 8 shows the peak discharges simulated for each event at HEC-HMS model junctions (junctions) along Squaw Run (see Figure 4) based on Muskingum-Cunge routing through the main channel. Junctions J81, J121, and J78 (as named in HEC-geoHMS) are within the study reach to be modeled in HEC-RAS. There is a slight reduction in the peak discharge between J81 and J121 due to flow attenuation along the reach.

For steady-state modeling, it is appropriate to use the peak flow at J81 at the upstream end of the study reach and the peak flow at J78 one or two sections below the downstream boundary of the study reach.

Table 8: Peak Discharges along Squaw Run for Existing Conditions (see Figure 4)

Hydrologic Model Element	2-year Qpeak, cfs	10-year Peak Q, cfs	25-year Peak Q, cfs	50-year Peak Q, cfs	100-year Peak Q, cfs	200-year Peak Q, cfs	500-year Peak Q, cfs
Junction-1	40	42	116	196	373	599	862
J96	50	131	190	240	293	351	432
J107	50	131	190	241	294	352	433
J86	286	885	1,335	1,758	2,232	2,753	3,455
J113	286	894	1,345	1,790	2,263	2,746	3,467
J99	337	1,096	1,731	2,331	2,981	3,692	4,712
J116	336	1,080	1,708	2,303	2,958	3,670	4,656
J118	334	1,068	1,689	2,276	2,929	3,655	4,644
J81	**461**	**1,280**	**2,015**	**2,674**	**3,487**	**4,419**	**5,747**
J121	453	1,257	1,995	2,649	3,457	4,371	5,700
J78	**497**	**1,367**	**2,205**	**2,959**	**3,703**	**4,670**	**6,151**

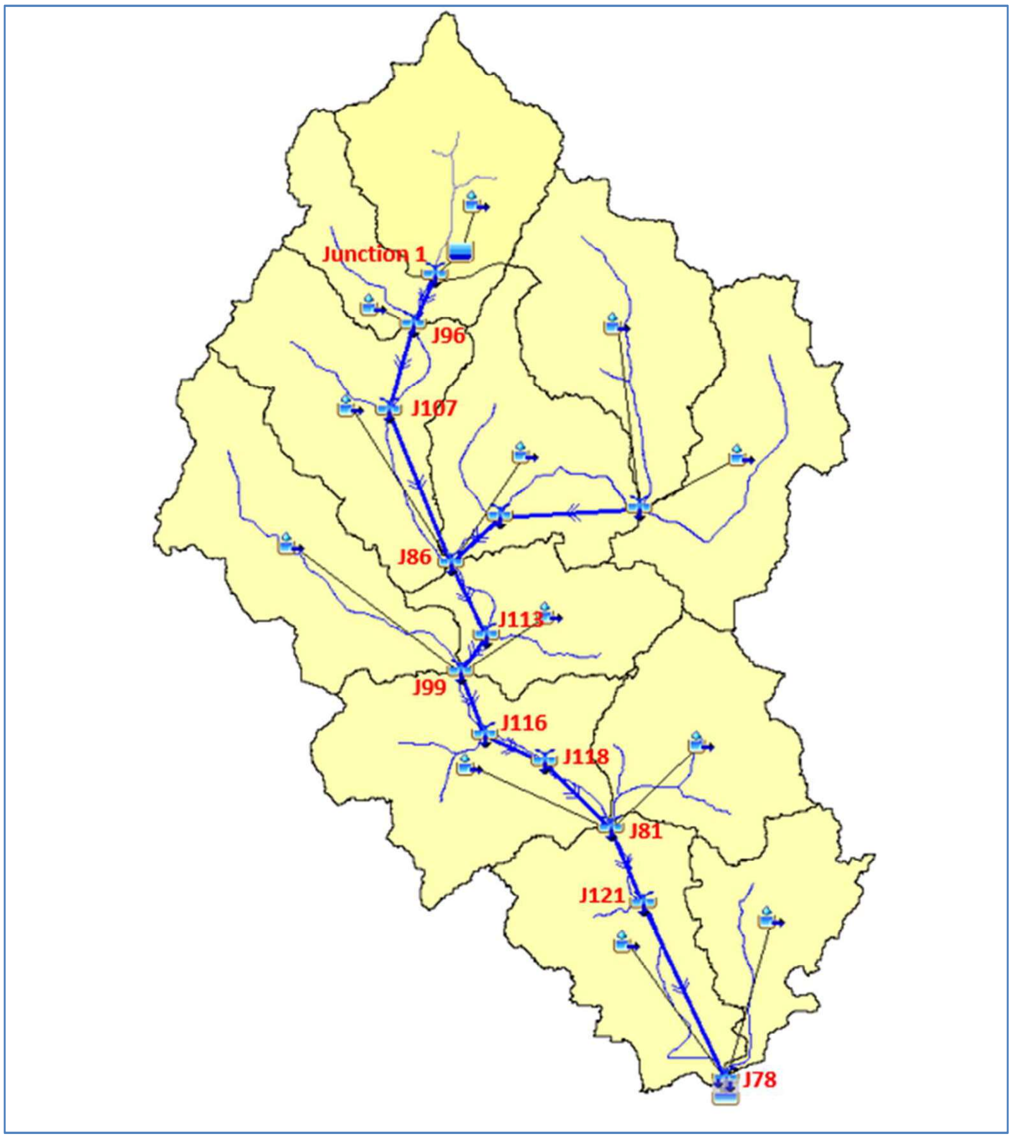

Figure 4 – HEC-HMS Model Junction Location Map

5.2 Evaluation of Model Reliability

There is no USGS gage within the Squaw Run watershed, and the applicable Flood Insurance Study reports that no discharge information is available for the streams studied within the Squaw Run watershed (FEMA, 2014).

To provide a check on model validity, therefore, the 2-, 10-, 50-, 100-, and 500-year peak discharges obtained from the HEC-HMS model were compared with:

- Values from the regression equations used in the USGS StreamStats program for Region 4 of Pennsylvania
- Peak floods determined from HEC-SSP flood frequency analysis of two nearby gauged streams similar to Squaw Run -
    - Little Pine Creek (USGS Gauge No. 03049800)
    - Thompson Run (USGS Gauge 03084800)

Normalization of the peak flows by dividing by the upstream drainage area allows the comparison of the four resultant peak flow series in cfs/mi^2, as shown in Table 9 and Figure 5.

Table 9 – Comparison of Annual Peak Discharge Estimates for Squaw Run

T	USACE HEC-HMS[1]		USGS StreamStats[2]		Little Pine Creek[3]		Thompson Run[4]	
years	Qp, cfs	cfs/sq. mi	Qp, cfs	cfs/sq. mi	Qp, cfs	cfs/sq. mi	Qp, cfs	cfs/sq. mi
2	497	61	407	48	230	40	743	41
10	1,367	167	956	112	903	156	2,094	116
50	2,959	362	1,650	194	2,496	432	3,450	192
100	3,703	453	2,010	236	3,726	645	4,073	227
500	6,151	753	3,050	358	9,019	1,560	5,635	313

Notes:

1 - HEC-HMS modeling with drainage area delineated in ArcMap as 8.17 sq.mi.
2 - StreamStats estimate for Region 4 for drainage area of 8.52 sq. mi. and urbanization at 59%, USGS 2020
3 – HEC-SSP Estimate for USGS Gauge 03049800 with drainage area of 5.78 sq. mi. (57 years of data, with one high outlier)
4 - HEC-SSP Estimate for USGS Gauge 03084800 with drainage area of 17.98 sq. mi. (10 years of data – minimum acceptable)

Inspection of Table 9 and Figure 5 shows that the annual peak flow per square mile modeled in HEC-HMS are bracketed by annual peaks calculated from observed annual peak flow gauge data from nearby streams utilizing HEC-SPP (USACE, 2019). The model values are less than the values calculated at Little Pine Creek (USGS Gage 03049800) and higher than the values calculated for Thompson Run (USGS Gage 03084800)

The flood frequency calculations for Little Pine Creek are based on 57 years of observed data, a period which contains a single significant high outlier than may bias the peak values calculated for high return periods.

The flood frequency calculations for Thompson Run are based on only 10 years of data (the minimum period length that can be run in HEC-SSP), a period in which there was no significant flood event.

The annual peaks calculated for Thompson Run are essentially equal to the values estimated for Squaw Run using the StreamStats regression equations, but the USGS notes that the regression equations for Region 4 do not include an explicit parameter for percent urbanization, so that "effects from urbanization may not be captured" (USGS, 2008). This could partially explain the discrepancy between the values in HEC-HMS and StreamStats – especially at higher discharges.

Based on the fact that the peak discharges per square mile simulated in HEC-HMS fall within the ranges of values observed at nearby sites, it is concluded that the HEC-HMS values are reasonable and conservative and are appropriate for evaluating flood control alternatives.

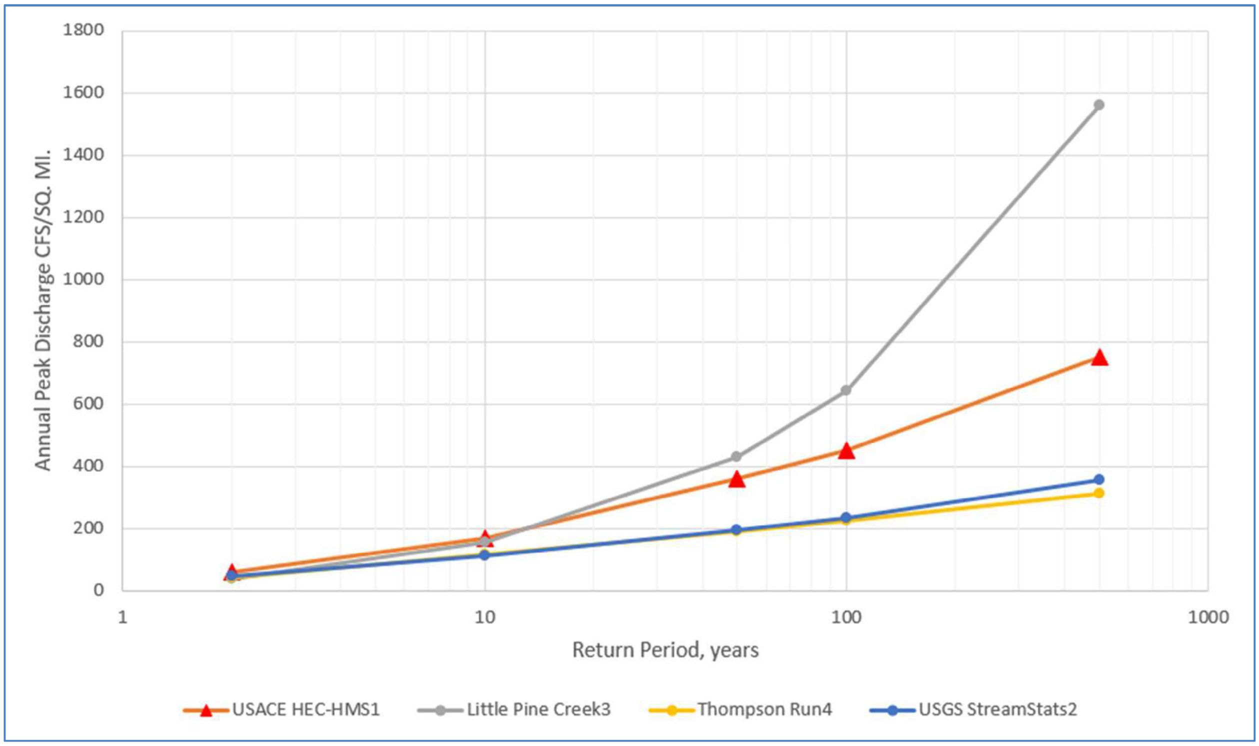

Figure 5 – Comparison of Area-Corrected Discharge Estimates for Squaw Run

6.0 CONCLUSION/SUMMARY

A hydrologic model of existing conditions within the Squaw Run watershed was produced for O'Hara Township and Fox Chapel Borough utilizing available digital topography, soil, and land use data evaluated in a GIS-environment in accordance with standard engineering practice.

There is no record of observed flows on Squaw Run available for calibration or verification of the model, but the peak discharges from the model are considered to be reasonable and conservative, based on a comparison with annual peak flows calculated from nearby stream gauges and from USGS Regression Equations for Region 4 of Pennsylvania.

Output from the HEC-HMS model will be used to provide peak discharges for input to HEC-RAS modeling of existing conditions and proposed conditions along the study reach.

7.0 REFERENCES

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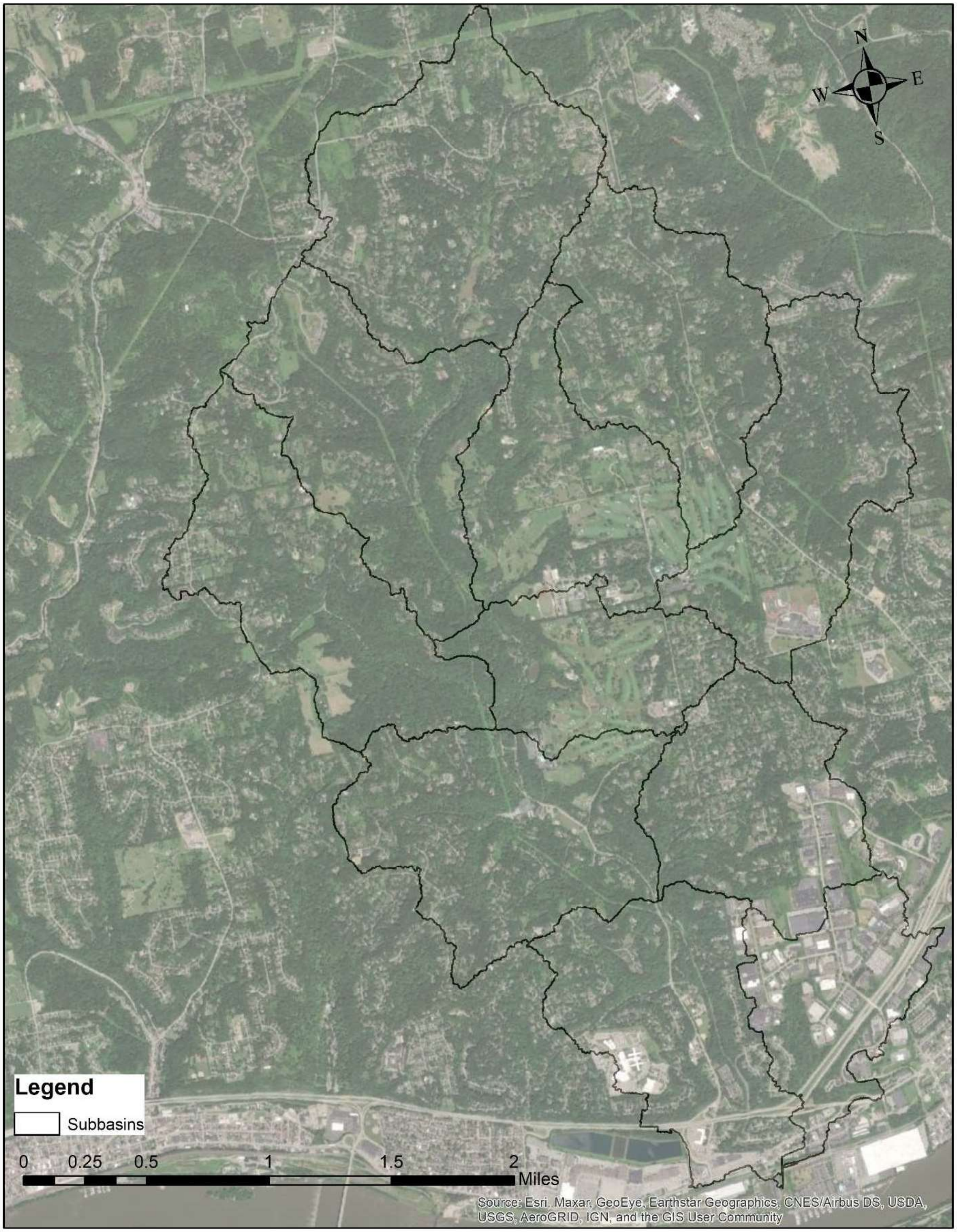
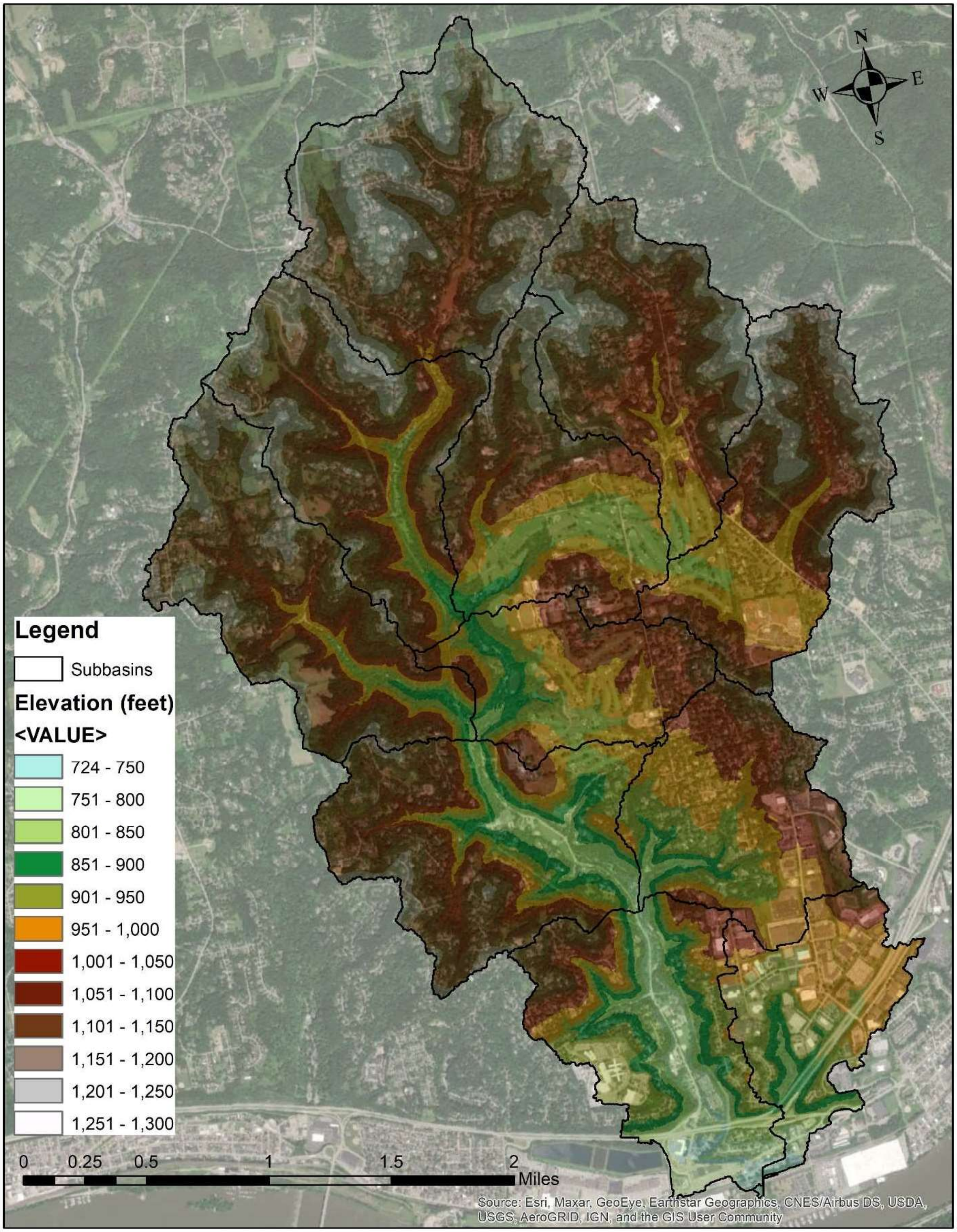
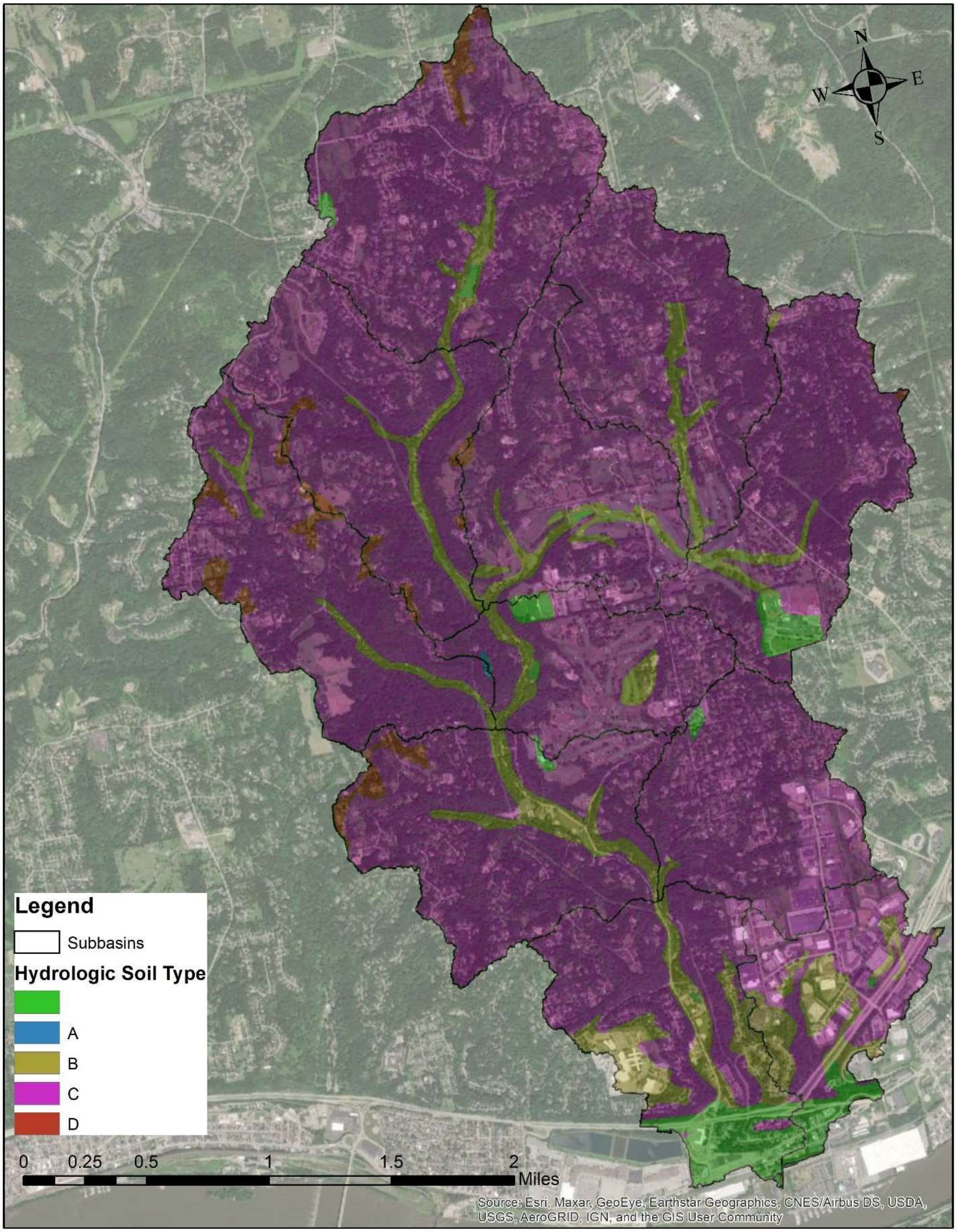
APPENDIX A - GIS-Based Mapping of Squaw Run Watershed Characteristics



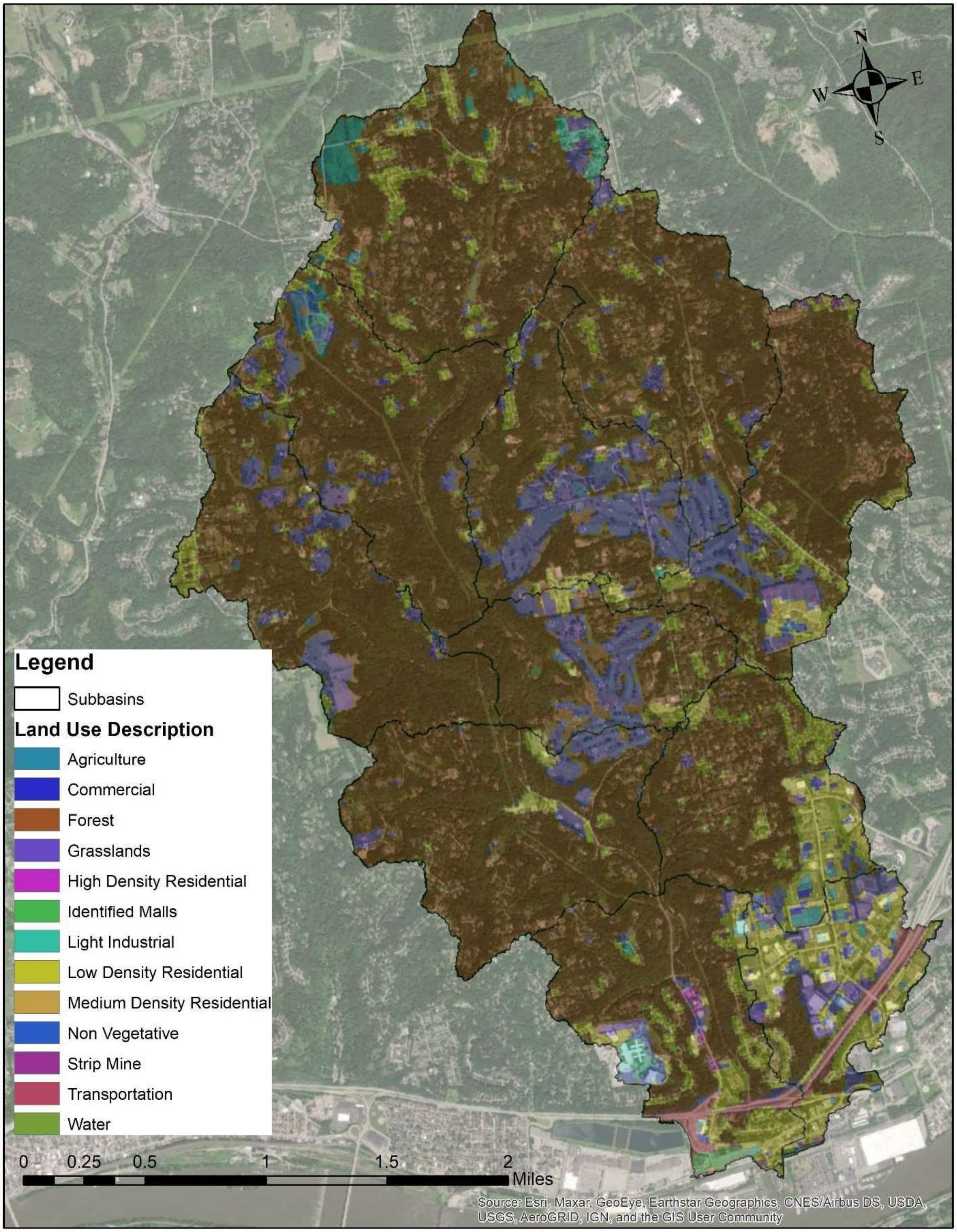
Appendix A, Figure 1- GIS-based World Imagery, Squaw Run Watershed



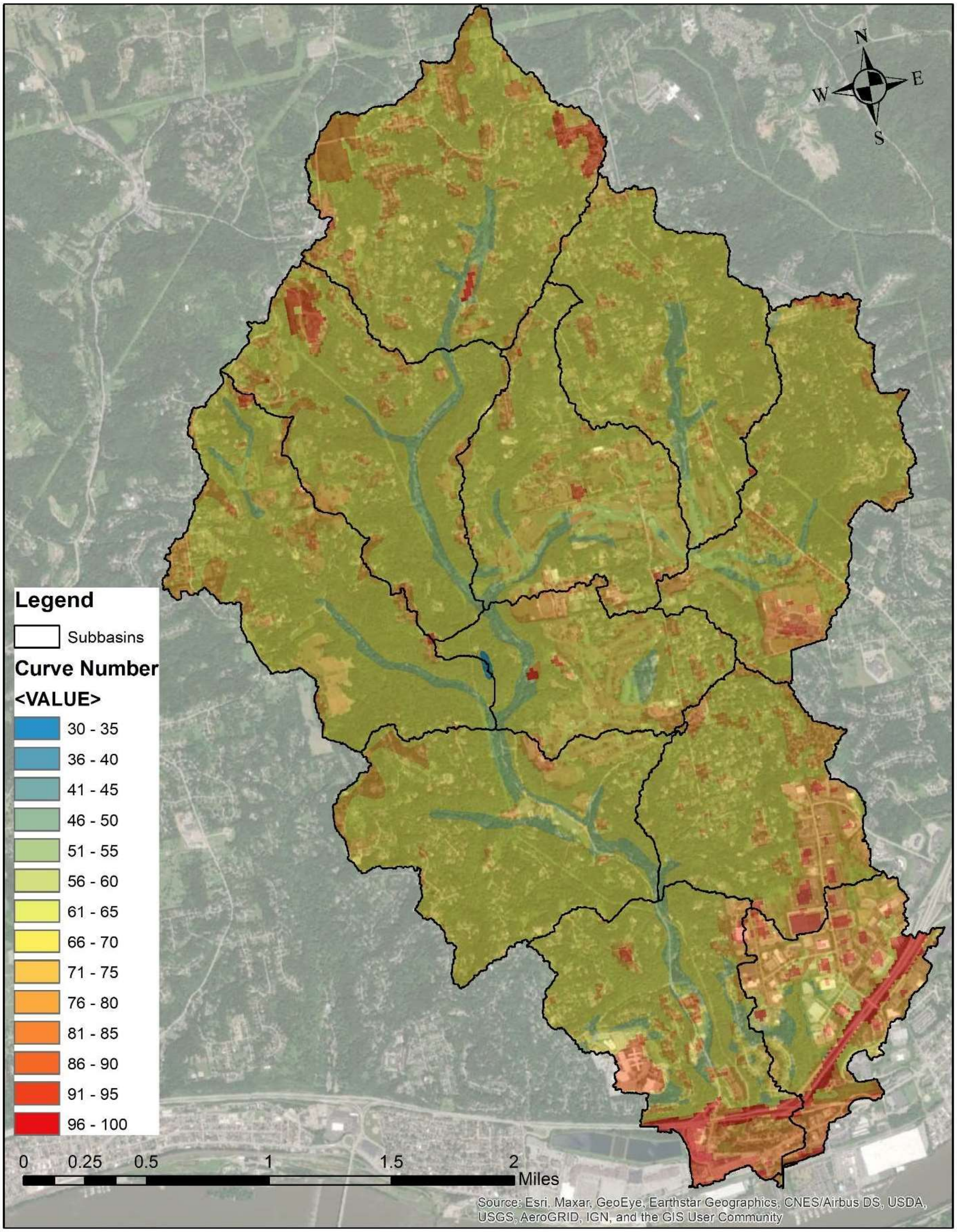
Appendix A, Figure 2- GIS-based Topographic Mapping, Squaw Run Watershed



Appendix A, Figure 3- GIS-based Hydrologic Soil-type Mapping, Squaw Run Watershed



Appendix A, Figure 4- GIS-based Land Use Mapping, Squaw Run Watershed



Appendix A, Figure 5- GIS-based Curve Number Mapping, Squaw Run Watershed

APPENDIX B – NOAA Atlas 14 Input

NOAA Atlas 14, Volume 2, Version 3
Location name: Fox Chapel, Pennsylvania, USA*
Latitude: 40.5226°, Longitude: -79.8863°
Elevation: 1024.2 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.317 (0.286-0.350)	0.378 (0.342-0.418)	0.457 (0.414-0.506)	0.518 (0.467-0.571)	0.596 (0.536-0.657)	0.655 (0.588-0.721)	0.712 (0.637-0.783)	0.772 (0.686-0.847)	0.850 (0.752-0.930)	0.907 (0.799-0.994)
10-min	0.492 (0.445-0.543)	0.590 (0.534-0.652)	0.711 (0.643-0.786)	0.800 (0.721-0.882)	0.911 (0.820-1.00)	0.993 (0.891-1.09)	1.07 (0.958-1.18)	1.15 (1.02-1.26)	1.25 (1.11-1.37)	1.32 (1.16-1.45)
15-min	0.603 (0.545-0.666)	0.721 (0.653-0.797)	0.872 (0.789-0.965)	0.984 (0.887-1.09)	1.13 (1.01-1.24)	1.23 (1.10-1.35)	1.33 (1.19-1.46)	1.43 (1.27-1.57)	1.56 (1.38-1.71)	1.65 (1.46-1.81)
30-min	0.798 (0.722-0.881)	0.965 (0.874-1.07)	1.20 (1.08-1.32)	1.37 (1.23-1.51)	1.59 (1.43-1.75)	1.76 (1.58-1.93)	1.92 (1.72-2.11)	2.09 (1.86-2.29)	2.31 (2.04-2.53)	2.48 (2.18-2.71)
60-min	0.974 (0.881-1.08)	1.18 (1.07-1.31)	1.50 (1.36-1.66)	1.74 (1.57-1.92)	2.06 (1.86-2.27)	2.32 (2.08-2.55)	2.57 (2.30-2.83)	2.84 (2.52-3.11)	3.19 (2.83-3.50)	3.48 (3.06-3.81)
2-hr	1.12 (1.02-1.23)	1.37 (1.25-1.50)	1.72 (1.57-1.89)	2.00 (1.81-2.18)	2.38 (2.15-2.59)	2.68 (2.42-2.92)	2.99 (2.69-3.25)	3.32 (2.96-3.59)	3.76 (3.33-4.06)	4.11 (3.62-4.44)
3-hr	1.19 (1.09-1.31)	1.45 (1.32-1.59)	1.81 (1.66-1.99)	2.11 (1.92-2.31)	2.52 (2.29-2.74)	2.85 (2.57-3.10)	3.19 (2.87-3.46)	3.55 (3.17-3.84)	4.04 (3.59-4.37)	4.44 (3.91-4.79)
6-hr	1.43 (1.32-1.57)	1.72 (1.58-1.89)	2.14 (1.97-2.34)	2.48 (2.27-2.71)	2.97 (2.71-3.23)	3.36 (3.05-3.65)	3.78 (3.41-4.09)	4.22 (3.78-4.55)	4.84 (4.29-5.21)	5.34 (4.70-5.74)
12-hr	1.68 (1.55-1.84)	2.02 (1.86-2.21)	2.49 (2.29-2.72)	2.88 (2.64-3.14)	3.44 (3.13-3.73)	3.90 (3.53-4.22)	4.39 (3.95-4.74)	4.90 (4.39-5.28)	5.65 (4.99-6.06)	6.25 (5.49-6.68)
24-hr	2.01 (1.89-2.16)	2.40 (2.25-2.57)	2.93 (2.74-3.14)	3.36 (3.15-3.61)	3.98 (3.71-4.25)	4.49 (4.17-4.79)	5.01 (4.64-5.34)	5.57 (5.13-5.92)	6.36 (5.81-6.74)	6.99 (6.35-7.41)
2-day	2.34 (2.20-2.50)	2.78 (2.62-2.98)	3.37 (3.17-3.61)	3.85 (3.61-4.12)	4.51 (4.22-4.81)	5.05 (4.70-5.38)	5.61 (5.21-5.96)	6.18 (5.72-6.56)	6.98 (6.41-7.40)	7.61 (6.95-8.07)
3-day	2.52 (2.37-2.68)	2.99 (2.81-3.18)	3.59 (3.38-3.83)	4.08 (3.84-4.35)	4.76 (4.46-5.06)	5.31 (4.96-5.64)	5.88 (5.47-6.23)	6.46 (5.99-6.84)	7.26 (6.69-7.69)	7.89 (7.24-8.36)
4-day	2.69 (2.54-2.86)	3.19 (3.01-3.39)	3.81 (3.60-4.05)	4.31 (4.07-4.58)	5.01 (4.71-5.31)	5.57 (5.22-5.90)	6.15 (5.74-6.51)	6.73 (6.27-7.12)	7.54 (6.98-7.97)	8.18 (7.53-8.65)
7-day	3.22 (3.06-3.40)	3.80 (3.61-4.02)	4.49 (4.26-4.74)	5.03 (4.77-5.31)	5.77 (5.46-6.09)	6.36 (5.99-6.69)	6.94 (6.53-7.31)	7.54 (7.07-7.93)	8.33 (7.77-8.76)	8.93 (8.30-9.40)
10-day	3.71 (3.54-3.90)	4.37 (4.17-4.60)	5.10 (4.86-5.37)	5.69 (5.42-5.99)	6.47 (6.15-6.80)	7.08 (6.71-7.43)	7.69 (7.27-8.07)	8.29 (7.82-8.70)	9.09 (8.53-9.54)	9.69 (9.06-10.2)
20-day	5.20 (4.97-5.45)	6.10 (5.82-6.40)	7.03 (6.71-7.38)	7.76 (7.41-8.14)	8.73 (8.30-9.14)	9.47 (8.99-9.91)	10.2 (9.65-10.7)	10.9 (10.3-11.4)	11.8 (11.1-12.4)	12.5 (11.7-13.1)
30-day	6.54 (6.26-6.85)	7.64 (7.31-8.01)	8.73 (8.34-9.15)	9.58 (9.16-10.0)	10.7 (10.2-11.2)	11.5 (11.0-12.1)	12.3 (11.7-12.9)	13.1 (12.5-13.7)	14.1 (13.4-14.8)	14.8 (14.0-15.5)
45-day	8.38 (8.04-8.74)	9.76 (9.36-10.2)	11.0 (10.6-11.5)	12.0 (11.5-12.5)	13.2 (12.6-13.8)	14.1 (13.5-14.7)	15.0 (14.3-15.6)	15.8 (15.1-16.5)	16.8 (16.0-17.5)	17.5 (16.6-18.3)
60-day	10.1 (9.72-10.5)	11.7 (11.3-12.2)	13.2 (12.6-13.7)	14.2 (13.7-14.8)	15.6 (14.9-16.2)	16.6 (15.9-17.2)	17.5 (16.7-18.1)	18.3 (17.5-19.0)	19.3 (18.5-20.1)	20.0 (19.1-20.8)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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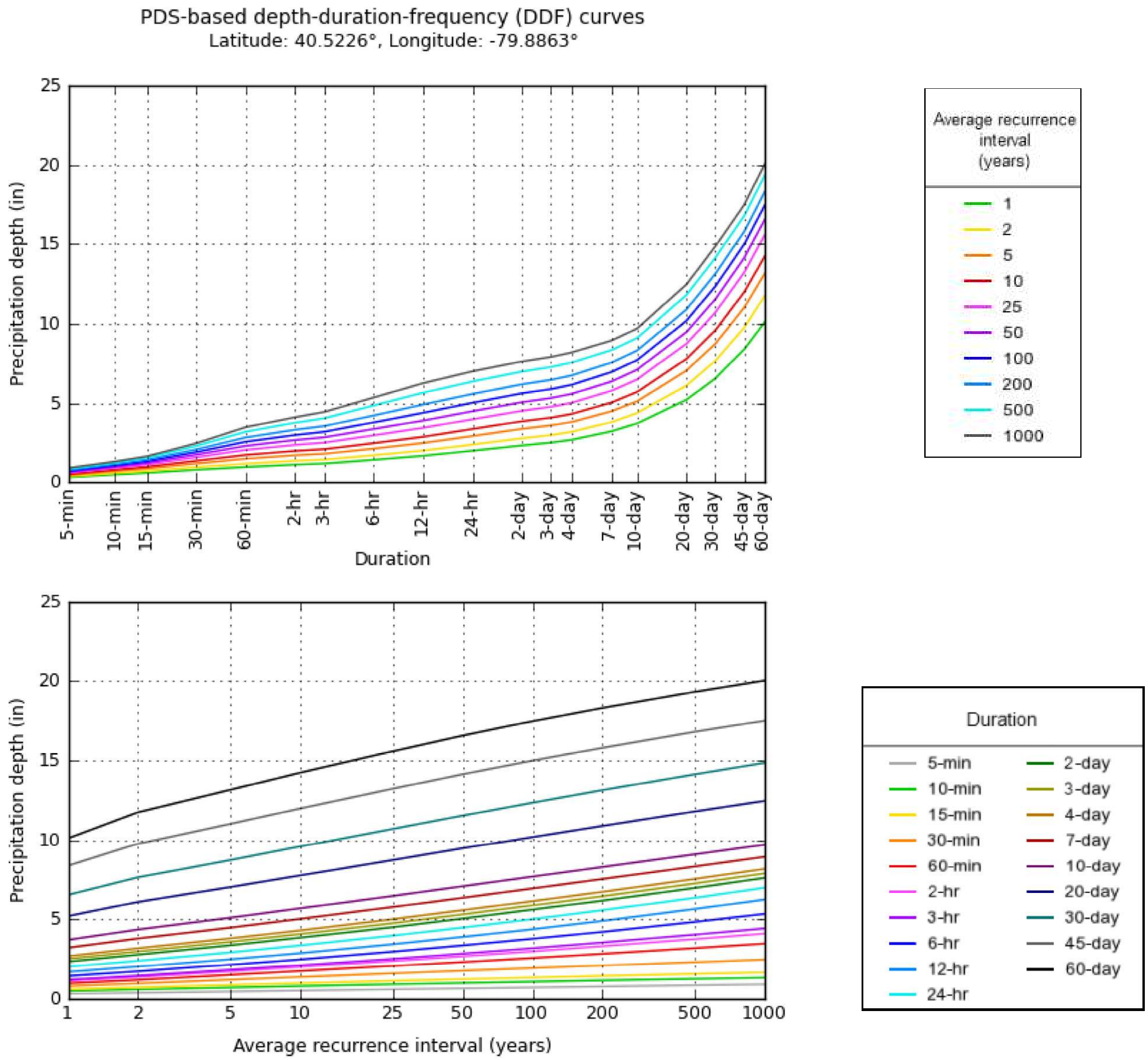
PF graphical



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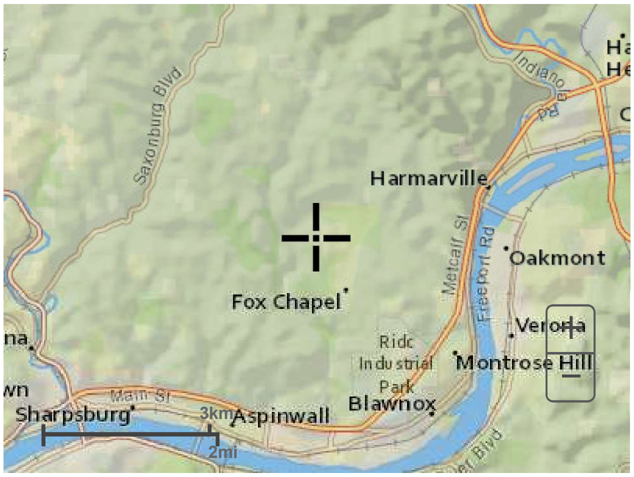
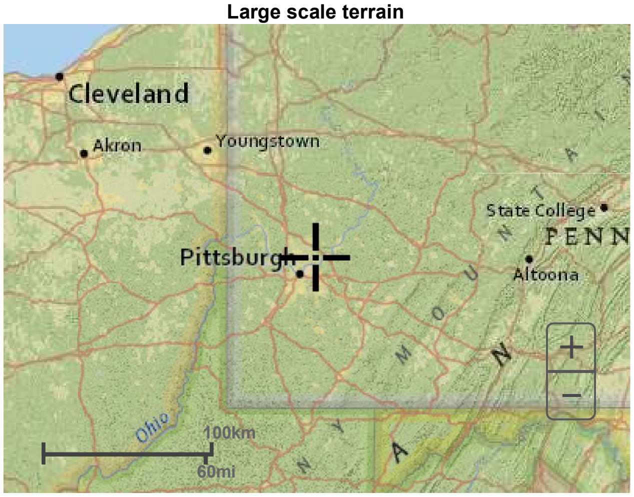
Maps & aerials

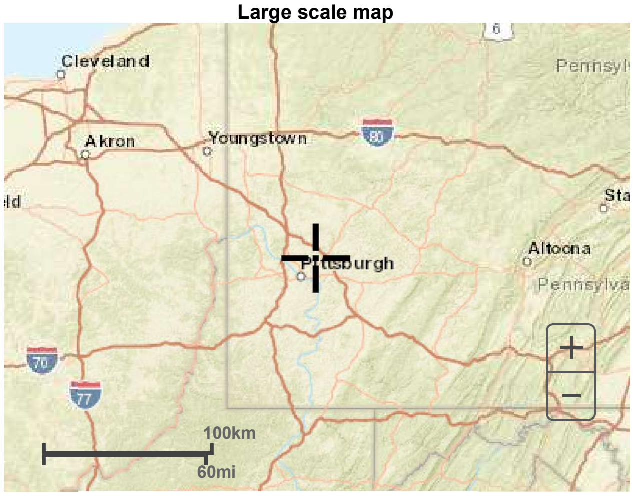
Small scale terrain



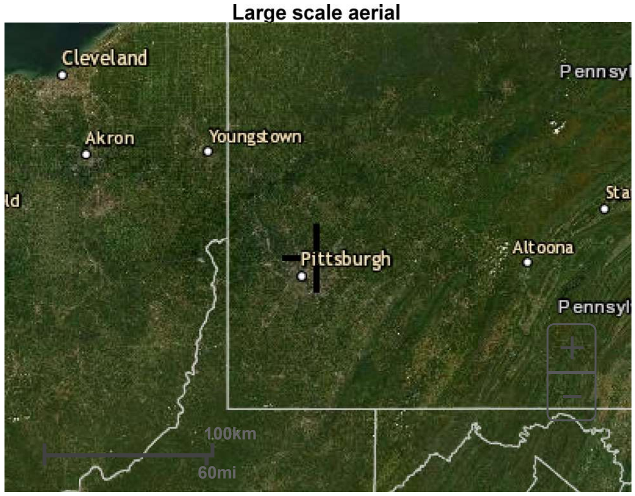
Large scale terrain



Large scale map



Large scale aerial



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US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

APPENDIX C – StreamStats Report for Squaw Run Watershed

StreamStats Report

Region ID: PA
Workspace ID: PA20200428161622946000
Clicked Point (Latitude, Longitude): 40.48763, -79.87756
Time: 2020-04-28 12:16:40 -0400

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	8.52	square miles
ELEV	Mean Basin Elevation	1041.3	feet
PRECIP	Mean Annual Precipitation	39	inches
FOREST	Percentage of area covered by forest	28	percent
URBAN	Percentage of basin with urban development	59	percent
CARBON	Percentage of area of carbonate rock	0	percent

Peak-Flow Statistics Parameters[Peak Flow Region 4]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	8.52	square miles	0.92	1720

Peak-Flow Statistics Flow Report[Peak Flow Region 4]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SE	SEp	Equiv. Yrs.
2 Year Peak Flood	407	ft^3/s	28	28	4
5 Year Peak Flood	706	ft^3/s	26	26	7
10 Year Peak Flood	956	ft^3/s	28	28	10
50 Year Peak Flood	1650	ft^3/s	33	33	13
100 Year Peak Flood	2010	ft^3/s	38	38	13
500 Year Peak Flood	3050	ft^3/s	49	49	12

Peak-Flow Statistics Citations

Roland, M.A., and Stuckey, M.H.,2008, Regression equations for estimating flood flows at selected recurrence intervals for ungaged streams in Pennsylvania: U.S. Geological Survey Scientific Investigations Report 2008-5102, 57p. (http://pubs.usgs.gov/sir/2008/5102/)

Annual Flow Statistics Parameters[Statewide Mean and Base Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	8.52	square miles	2.26	1720
ELEV	Mean Basin Elevation	1041.3	feet	130	2700
PRECIP	Mean Annual Precipitation	39	inches	33.1	50.4
FOREST	Percent Forest	28	percent	5.1	100
URBAN	Percent Urban	59	percent	0	89
CARBON	Percent Carbonate	0	percent	0	99

Annual Flow Statistics Flow Report[Statewide Mean and Base Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SE	SEp
Mean Annual Flow	11.8	ft^3/s	12	12
Harmonic Mean Streamflow	2.62	ft^3/s	38	38

Annual Flow Statistics Citations

Stuckey, M.H.,2006, Low-flow, base-flow, and mean-flow regression equations for Pennsylvania streams: U.S. Geological Survey Scientific Investigations Report 2006-5130, 84 p. (http://pubs.usgs.gov/sir/2006/5130/)

Base Flow Statistics Parameters[Statewide Mean and Base Flow]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	8.52	square miles	2.26	1720
PRECIP	Mean Annual Precipitation	39	inches	33.1	50.4
CARBON	Percent Carbonate	0	percent	0	99
FOREST	Percent Forest	28	percent	5.1	100
URBAN	Percent Urban	59	percent	0	89

Base Flow Statistics Flow Report[Statewide Mean and Base Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SE	SEp
Base Flow 10 Year Recurrence Interval	3.41	ft^3/s	21	21
Base Flow 25 Year Recurrence Interval	3.02	ft^3/s	21	21
Base Flow 50 Year Recurrence Interval	2.8	ft^3/s	23	23

Base Flow Statistics Citations

Stuckey, M.H.,2006, Low-flow, base-flow, and mean-flow regression equations for Pennsylvania streams: U.S. Geological Survey Scientific Investigations Report 2006-5130, 84 p. (http://pubs.usgs.gov/sir/2006/5130/)

Bankfull Statistics Parameters[Statewide Bankfull Noncarbonate 2018 5066]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	8.52	square miles	2.62	207
CARBON	Percent Carbonate	0	percent		

Bankfull Statistics Flow Report[Statewide Bankfull Noncarbonate 2018 5066]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	SE
Bankfull Area	67.7	ft^2	64
Bankfull Streamflow	293	ft^3/s	74
Bankfull Width	38.2	ft	59
Bankfull Depth	1.81	ft	56

Bankfull Statistics Citations

Clune, J.W., Chaplin, J.J., and White, K.E.,2018, Comparison of regression relations of bankfull discharge and channel geometry for the glaciated and nonglaciated settings of Pennsylvania and southern New York: U.S. Geological Survey Scientific Investigations Report 2018–5066, 20 p. (https://doi.org/10.3133/sir20185066)

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Application Version: 4.3.11

Tab 2. Squaw Run Proposed Condition HEC-HMS Model Development

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Part I – Completed by Originator

1. Project Name:	Flood Risk and Storm Water 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:	ArcMap 10.5.1; HEC-GeoHMS 10.4; HEC-HMS 4.4			
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: N/A

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
T.H. Jackson, P.E.	JACKSON.THOMAS.HOFFMAN REYHER.1485301298 Digitally signed by JACKSON.THOMAS.HOFFMAN REYHER.1485301298 Date: 2020.10.15 11:17:14 -04'00'	October 15, 2020
Verifier(s)	**Signature**	**Date**
Howard Kellick, P.E.	KELLICK.HOWARD.BRUCE.1565106755 Digitally signed by KELLICK.HOWARD.BRUCE.1565106755 Date: 2020.10.15 12:20:56 -04'00'	October 15, 2020
Water Resources Section Chief	**Signature**	**Date**
Kyle Kaminski, P.E.	KAMINSKI.KYLE.MARTIN.1591285323 Digitally signed by KAMINSKI.KYLE.MARTIN.1591285323 Date: 2020.10.15 14:42:05 -04'00'	

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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1.0 STATEMENT OF PURPOSE

Squaw Run drains an approximately 8.2 square mile watershed to the Allegheny River. The area lies primarily within Fox Chapel Borough and O'Hara Township, as shown in the lower right panel of Figure 1.

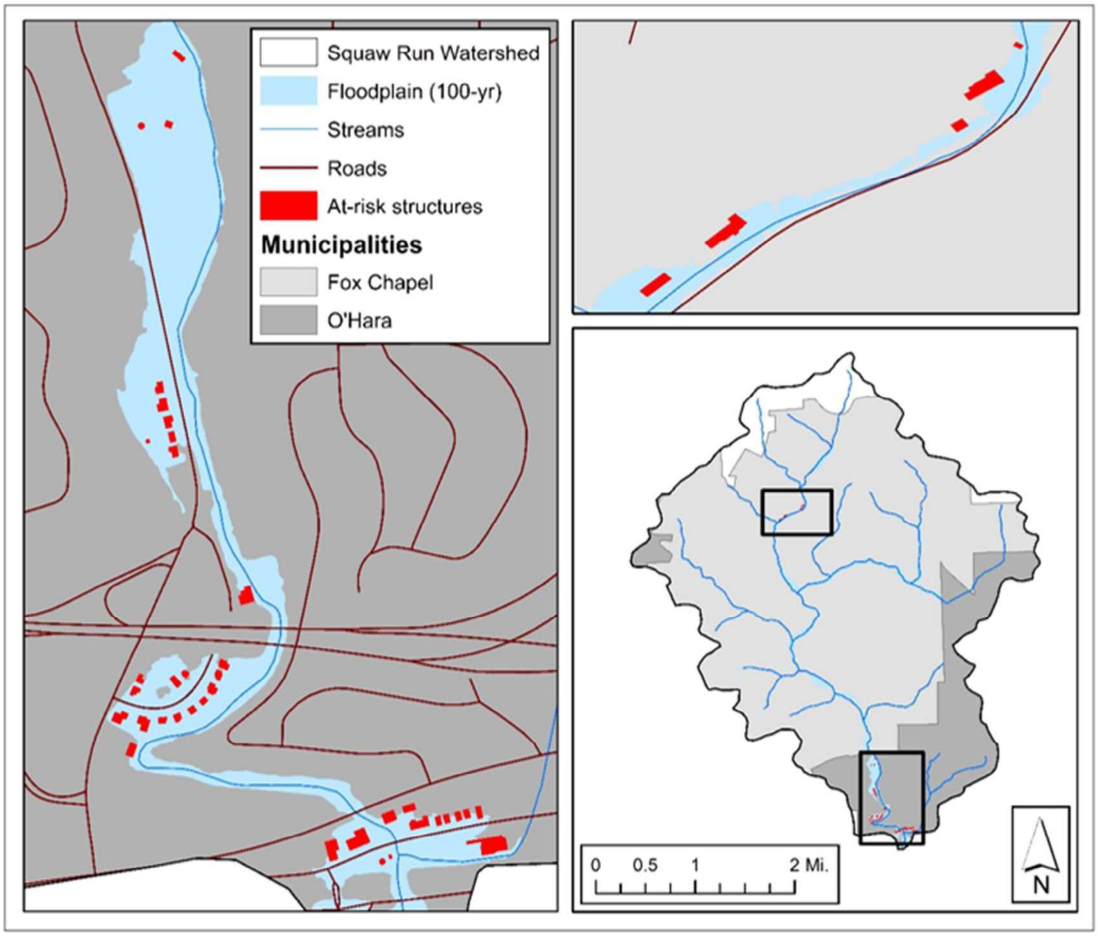

Figure 1 - Location of 44 High-Risk Structures in the 100-year Floodplain along Squaw Run within O'Hara Township and Fox Chapel Borough (Dark Gray and Light Gray coloring, respectively in Lower Right Panel)

Historic flooding on Squaw Run, including Hurricane Ivan in 2004 and intense localized precipitation events (e.g., July 2, 2018) have impacted many structures within the floodplain (see left and upper right panels of Figure 1) with some structures experiencing basement flooding multiple times per year.

Considering the historic flooding and the potential for increased future flooding, O'Hara Township and Fox Chapel Borough have jointly requested flood risk management assistance from USACE, Pittsburgh District through the Floodplain Management Services (FPMS) program.

The development of existing conditions hydrology for the watershed was described in a previous calculation brief, "Squaw Run Existing Condition HEC-HMS Model Development" (USACE, 2020). This calculation brief describes work done to complete Objective 3, as described in the FPMS Study Scope, i.e., to develop HEC-HMS simulations of the impact of potential SWM projects in the upper reaches of the Squaw Run watershed for return periods between 2 and 500 years. The associated outputs are used as inputs to HEC-RAS models of Squaw Run developed in separate tasks for the preliminary evaluation of alternative structural and non-structural flood mitigation strategies.

2.0 DESCRIPTION OF METHODOLOGY USED FOR HEC-HMS MODEL DEVELOPMENT

HEC-GeoHMS (version 10.4) in ArcGIS (version 10.5) was used to delineate the Squaw Run watershed and existing condition subbasins (USACE, 2013) as shown in Figure 2.

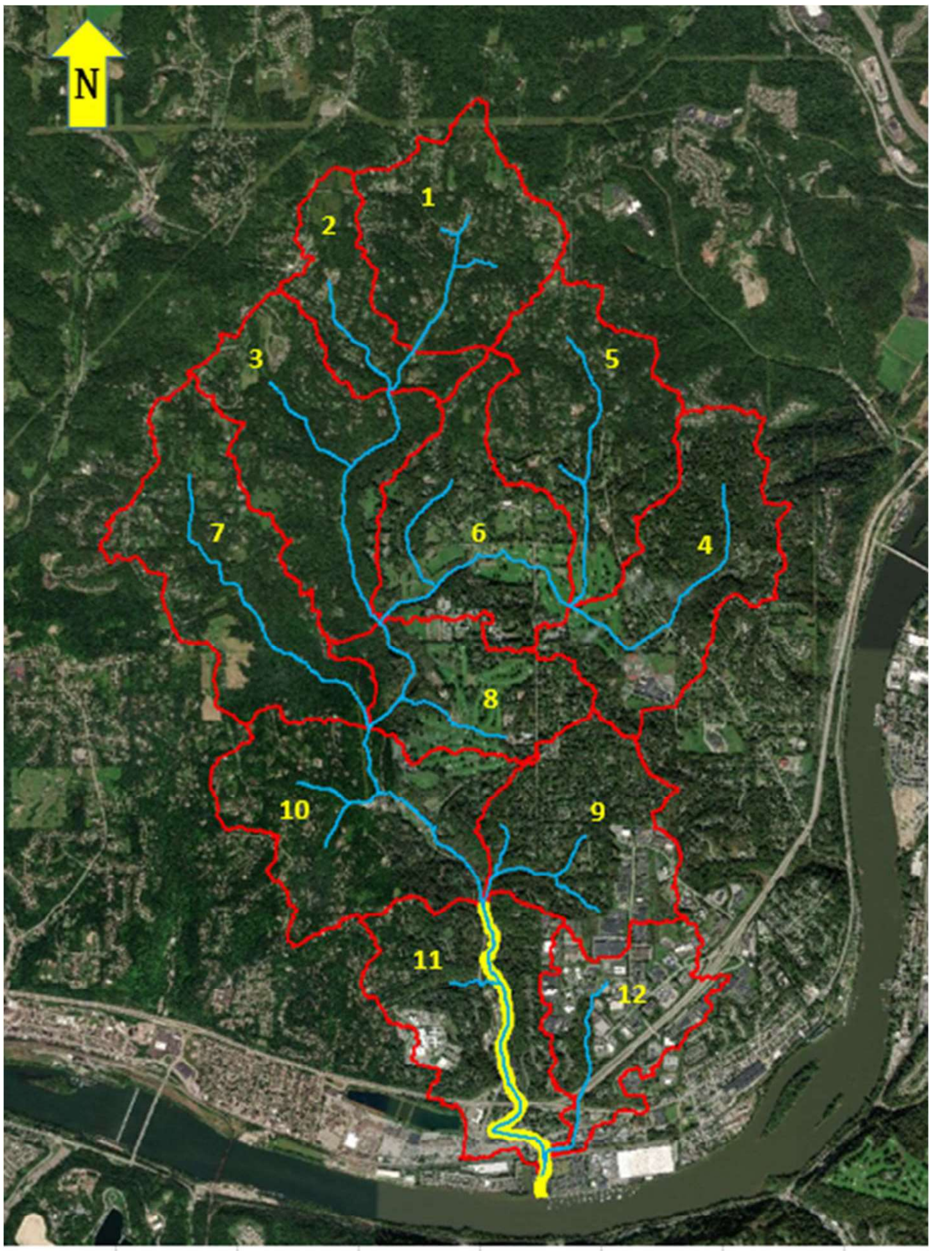

Figure 2 – Extents of HEC-HMS Study Area – No Scale (HEC-RAS Study Reach Shown with Yellow Outline)

Parameters developed in HEC-GeoHMS were imported to HEC-HMS (USACE, 2000) for use in developing a hydrologic model for the simulation of existing conditions. The following simulation modules were used in HEC-HMS:

- Loss: SCS Curve Number
- Transform: SCS Unit Hydrograph
- Routing: Muskingum-Cunge Eight Point

Details concerning development of the existing conditions simulation are provided in a previous calculation brief (USACE, 2020).

Based on the results of the existing conditions modeling and a review of aerial photography and contours developed from 3-meter USGS DEM data in ArcGIS, two physically and institutionally feasible[1] sites for the construction of storm water management (SWM) ponds were identified, as shown in Figure 3:

- SWM Pond No. 1 is located at the confluence of an unnamed tributary with Glade Run (Subbasin No. 6 in Figure 2). It would regulate an approximately 0.24 mi^2 upstream catchment area.
- SWM Pond No. 2 is located at the confluence of Stony Camp Run with Squaw Run (Subbasin No. 7 in Figure 2). It would regulate an approximately 0.87 mi^2 upstream catchment area

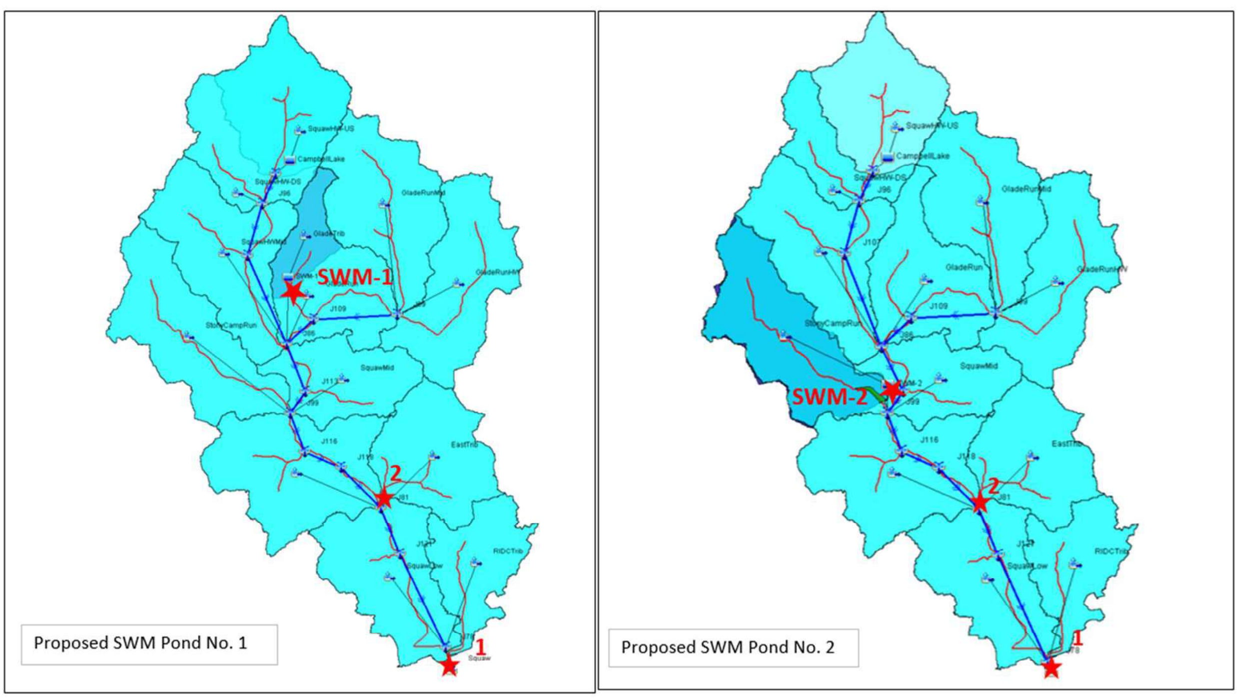

Figure 3 - Location Maps for Proposed SWM Ponds 1 and 2

[1] "Physically feasible" in the sense that there are no topographic constraints and "Institutionally feasible" in the sense that the land is either already owned, can be bought, or easements obtained as required and there are no objections in the community that could rise to an injunction against proceeding.

Proposed condition basin models were developed for SWM ponds at each of these sites by modifying the existing condition model by 1) the addition of a reservoir elements representing each SWM pond, with associated stage-storage and storage-discharge relationships input in the Paired Data module of HEC-HMS and 2) revision of the appropriate hydrologic parameters of the circumscribing subbasins for the Existing Conditions basin model, as described in Section 4.

The stage-storage relations for each site were determined from DEM raster data “clipped” to the approximate footprint of the reservoir high water mark, with the measurement of areas for each contour interval determined from the Contour tool in the 3D Analyst “tool box” selected from the assumed elevation of the toe of slope of the embankment up to a maximum assumed top of dam elevation twenty feet above the base.

Stage-discharge relationships were developed using a spreadsheet formulation allowing trial-and-error selection of the dimensions and crest elevation of low-flow and principal spillway drop weirs to maximize the use of the available storage without the use of an emergency spillway, which would be included in the design but was not sized for this exercise.

The Meteorologic Model components developed for the existing conditions HEC-HMS model for the 2-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence interval flows (USACE, 2020) were utilized for the proposed condition HEC-HMS model without modification.

3.0 ASSUMPTIONS AND JUSTIFICATION

1. Stage-storage relationships for the potential SWM Ponds are calculated based on existing topography, assuming that the only earthwork done is to impound the water at the outlet and construct the outlet works.
2. It is assumed that a reasonable order of magnitude estimate of the construction cost of the SWM projects can be obtained from consideration of the volume of the embankment needed to provide the storage of water needed to maximize the attenuation of the flood hydrograph from the upstream contributing area for an embankment no higher than 20 feet in elevation (a "dam construction" permit through PADEP would be required if it exceeds the thresholds established for Pennsylvania).
3. It was assumed that the foundation conditions would be adequate for the placement of an earth fill dam without the extensive use of grouting or flowable fill.
4. It was assumed that the outlet structure for each SWM pond would consist of a low-flow and a principal spillway weir and that the outlet pipe would be sized with a diameter sufficient to ensure barrel flow for all design conditions. The volume of concrete required for the outlet structure can be estimated from the perimeter and height of the structure assuming a wall thickness of 12 inches and a foundation 36 inches thick. The length of the outlet pipe was not determined, but could be assumed to be twice the bottom width of the embankment.
5. The diameter of low-flow pipe required to drain the reservoir within 72 hours was not determined for this calculation. Its additional cost is negligible and it is assumed that it would not add significantly to storage capacity of the dam.

6. A weir coefficient of 3.1 was assumed for the sharp-crested low flow and principal spillway structures as a reasonably conservative value for pre-feasibility study (normal range between 2.6 and 3.3).
7. It was assumed that the emergency spillway could be cut into natural ground at one end of the embankment and would not add significantly to the cost of the structure. Based on the assumed lay-out, the emergency spillway does not operate for the return periods modeled.
8. It was assumed that easements and land for SWM ponds at either of the proposed locations would be available because aerial photography did not show any structures within the footprint of the proposed facilities.
9. It was assumed that structures with the dimensions of the SWM Ponds proposed for future conditions would be institutionally feasible because they are similar in magnitude to the existing Lake Campbell SWM pond (Partridge, 1988).
10. It was assumed that the CN values for the subdivision of the existing Glade Run subbasin above and below SWM Pond 1 were identical because land use appears to be homogeneous within the original subbasin area. This was a reasonable level of accuracy for pre-feasibility level modeling.
11. It is assumed for the calculation of travel times in the two subbasins resulting from splitting the Glade Run subbasin into two parts that land slopes are similar.

4.0 CALCULATION INPUT

The Proposed Conditions HEC-HMS models were developed from the Existing Conditions HEC-HMS model as described in a previous calculation brief (USACE, 2020). No additional data was utilized.

Calculation of Design Parameters for SWM Pond No. 1

The location of the potential SWM Pond No. 1 on the unnamed tributary to Glade Run is shown in Figure 4.

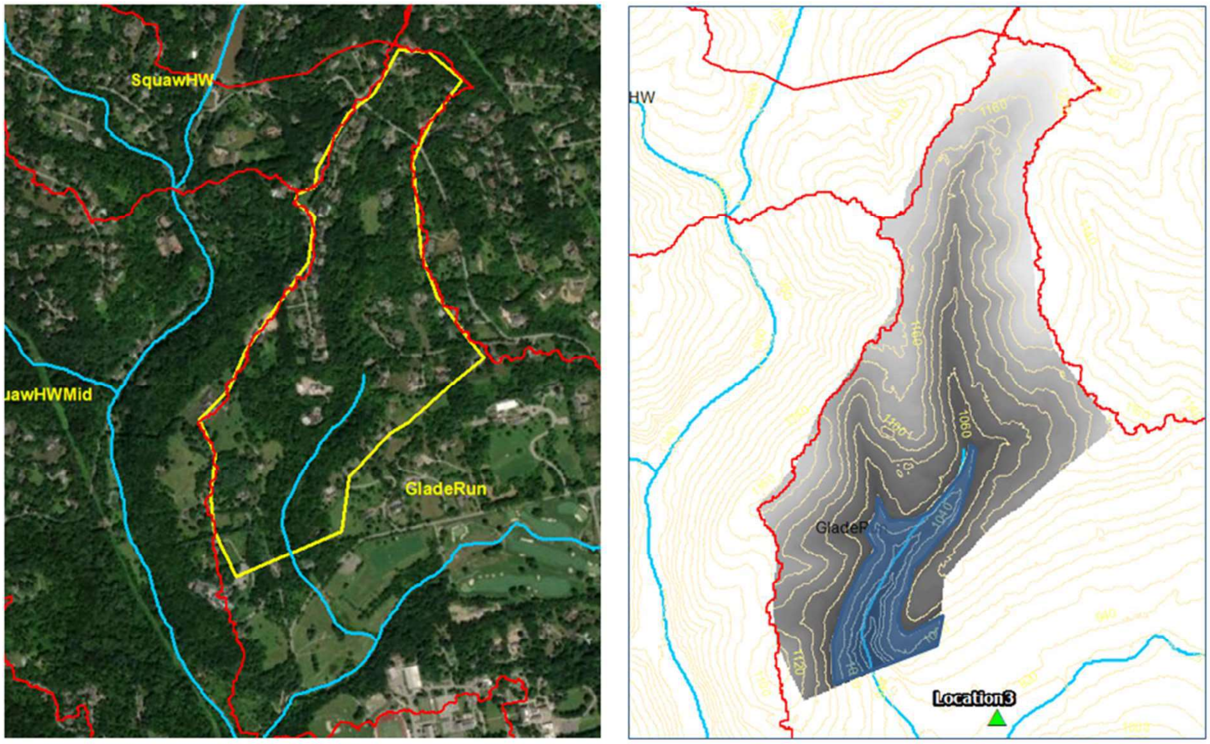

Figure 4 - Aerial Photo View at Left; 20-foot Contours at Right with Approximate Footprint of SWM Pond No. 1 Reservoir at Maximum Depth

The elevation-area data calculated in ArcGIS was used to develop a stage-volume curve for the reservoir that would be impounded behind a 20-foot high structure along the presumed alignment, as summarized in Table 1.

Table 1 - Stage-Volume Calculation for SWM Pond No. 1

Stage, feet	Elevation, feet	Surface Area, ft^2	Volume of Slice^1, ft^3	Cumulative Volume, ft^3	Cumulative Volume, Ac-ft
0	960	35,945	0	0	0.00
5	965	47,105	207,625	207,625	4.77
10	970	65,015	280,300	487,926	11.20
15	975	82,297	368,279	856,204	19.66
20	980	99,687	454,958	1,311,163	30.10

Note:

1) Slice volume ≈ $0.5(SA_2 - SA_1)*(Elev_2 - Elev_1)$

A pre-feasibility level stage-discharge relationship was developed for an outlet structure presumed to consist of a low-flow and a principal spillway weir. It was assumed that the outlet pipe would be sized with a diameter sufficient to ensure barrel flow for all design conditions.

The dimensions and crest elevations of the weirs were adjusted to optimize the use of the available storage (maximizing attenuation of the incoming flood hydrograph) without overtopping a 20-foot structure. The calculation is summarized in Table 2.

Table 2 - Calculation of Stage-Discharge Relation for SWM Pond No. 1

Water Surface[1] Elevation, ft	Low-flow Weir[2] Discharge, ft^3/s	Principal Spillway[3] Weir Discharge, ft^3/s	Total outlet discharge, ft^3/s
960	0.0	0.0	0.0
962	0.0	0.0	0.0
964	0.0	0.0	0.0
966	0.0	0.0	0.0
968	0.0	0.0	0.0
970	8.8	0.0	8.8
972	24.8	0.0	24.8
973	34.7	0.0	34.7
975	57.4	0.0	57.4
980	128.9	277.3	406.1

Notes:

1) Areas and volumes for elevations between values shown in Table 1 determined by interpolation
2) Low-flow weir $Q \approx 3.1*L*H^{1.5}$, with L = 1 ft; H = max(0, WSL – Crest Elev = 968.0 ft)
3) Principal Spillway weir $Q \approx 3.1*L*H^{1.5}$, with L = 8 ft; H = max(0, WSL – Crest Elev = 975.0 ft)

The data in Tables 1 and 2 were used to develop the paired input data for the elevation-storage and storage-discharge functions in the utilized for flood routing in HEC-HMS as input, labeled SWM1Z-V and SWM1V-Q, respectively.

The storage and discharge are plotted as functions of the stage in Figure 5.

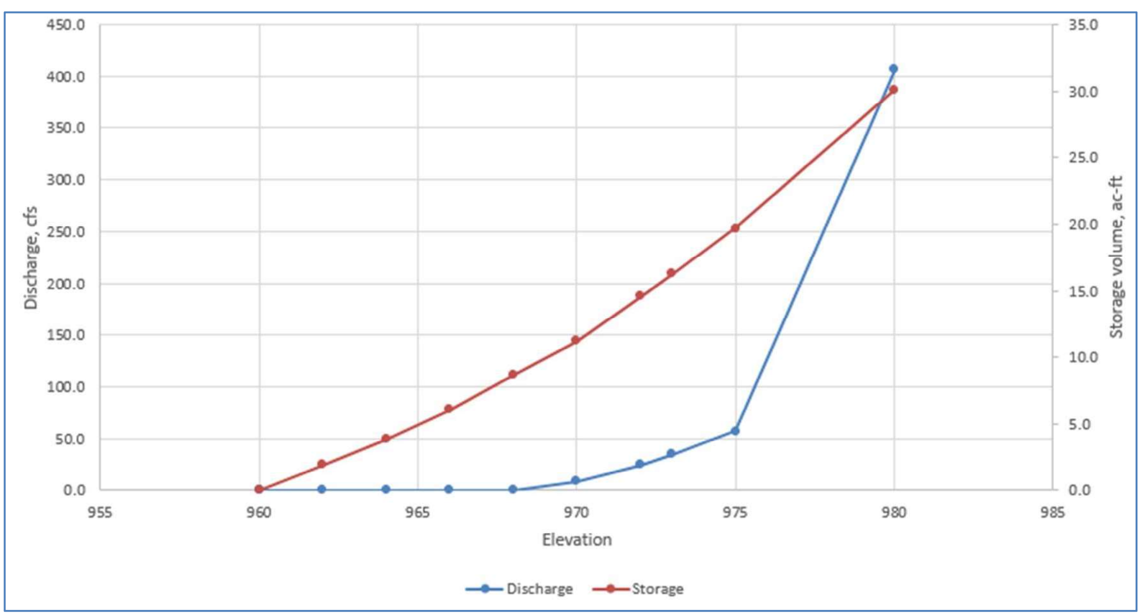

Figure 5 - Elevation-Storage and Elevation-Discharge Relations for SWM Pond No. 1

The placement of SWM Pond No. 1 on the tributary to Glade Run led to the subdivision of its catchment area (Area No. 6 in Figure 2). Table 3 shows the hydrologic parameters utilized for modeling the two resultant subbasins. Table 4 includes the calculation of the lag time, performed using the spreadsheet generated in HEC-GeoHMS.

Table 3 - Hydrologic Parameters for Subbasins Resulting from Placement of SWM Pond No. 1

Subbasin	Glade Run (existing)	Glade Run (revised)[1]	Glade Trib
Area, mi^2	.7252	.4875	.2376
I_A, inches	.8321	.8321	.8321
CN	70.62	70.62	70.62
Lag time, minutes	28.5	28.5	18.8

Notes:

1) The revised subbasin had a reduced area but the values of I_A, CN, and % impervious were assumed to be unchanged due to the uniform land use in the subbasin. The lag time remained the same because the longest flow path is contained in the revised subbasin.
2) The area of Glade Trib was delineated and its area was measured in GIS. The values of I_A and CN were assumed to be the same as for Glade Run because of the uniform land use in the two basins. The % impervious area was estimated as 5% to account for the average extent of the pond. The lag time was taken as 0.6 times the time of concentration calculated for the subbasin (see Table 4)

Table 4 - Calculation of Lag Time for New Subbasin "Glade Trib"

Overland Flow Time, hours[1]	SCF - Shallow Concentrated Flow time, hours[2]	Channel Flow time, hours[3]	Total Travel Time, hours[4]	Lag Time, minutes[5]
0.196	0.249	0.080	0.52	18.8

Notes (see USDA, 1986):

1) Sheet flow = $0.007*(nL)^{0.8}/(P_2^{0.5}*S^{0.4})$ with Roughness n = 0.4, flow length L = 100 ft, 2-yr ppt depth, P2 = 2.4 inches, slope, S = 0.1295
2) SCF = L/V where L = 3176 ft and V = $16.1345*S^{0.5}$ for unpaved surface, with land slope, S = 0.0481
3) Channel Flow = L/V where L = 1400 ft and V per Mannings = $1.49(A/P)^{2/3}S^{0.5}/n$ for A = 22 ft^2, P = 17.6 ft, S = 0.0318 and n = 0.060
4) Total time = 1) OFT + 2) SCFT + 3) CFT in hours
5) Lag time = 0.6*Total Time * (60 minutes/hour)

Calculation of Design Parameters for SWM Pond No. 2

The location of the potential SWM Pond No. 2 is near the confluence of Stony Camp Run with Squaw Run, as shown in Figure 6.

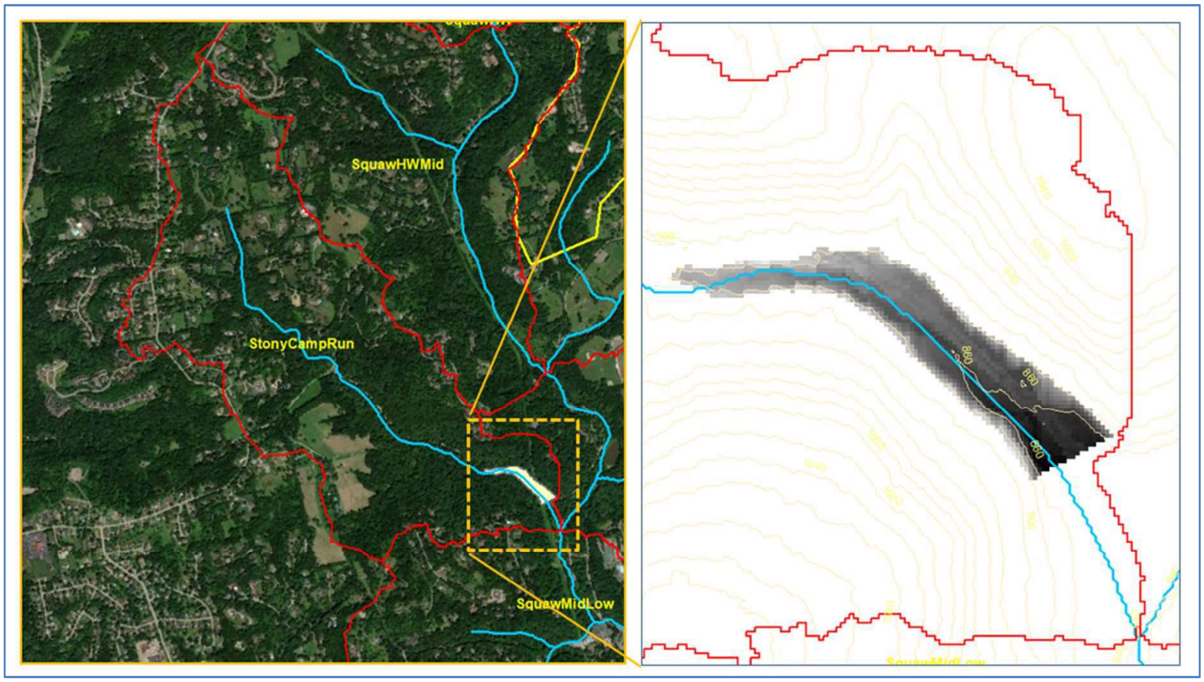

Figure 6 - Aerial Photo View at Left; 20-foot Contours at Right with Footprint of SWM Pond No. 2 Reservoir at Maximum Depth

The elevation-area data calculated in ArcGIS was used to develop a stage-volume curve for the reservoir that could be impounded behind a 20-foot high structure along the presumed alignment, as summarized in Table 5.

Table 5 - Stage-Volume Calculation for SWM Pond No. 2

Stage, feet	Elevation, feet	Surface Area, ft^2	Volume of Slice[1], ft^3	Cumulative Volume, ft^3	Cumulative Volume, Ac-ft
0	860	33,210		0	0.00
2	862	45,002	78,211	78,211	1.80
4	864	57,215	102,217	180,428	4.14
5	865	64,051	60,633	241,062	5.53
10	870	103,226	418,191	659,252	15.13
15	875	130,851	585,192	1,244,445	28.57
20	880	157,946	721,994	1,966,438	45.14

Note:

1) Slice volume ≈ $0.5(SA_2 – SA_1)*(Elev_2 – Elev_1)$

A pre-feasibility level stage-discharge relationship was developed for an outlet structure presumed to consist of a low-flow and a principal spillway weir.

The dimensions and crest elevations of the weirs were adjusted to optimize the use of the available storage (maximizing attenuation of the incoming flood hydrograph) without overtopping a 20-foot structure. The calculation is summarized in Table 6.

Table 6 - Calculation of Stage-Discharge Relation for SWM Pond No. 2

Water Surface[1] Elevation, ft	Low-flow Weir[2] Discharge, ft^3/s	Principal Spillway[3] Weir Discharge, ft^3/s	Total outlet discharge, ft^3/s
860	0.00	0.00	0.00
862	0.00	0.00	0.00
864	0.00	0.00	0.00
866	0.00	0.00	0.00
868	8.77	0.00	8.77
870	24.80	0.00	24.80
872	45.56	65.76	111.32
873	57.41	120.81	178.22
875	83.70	259.94	343.64
880	162.39	735.23	897.62

Notes:

1) Areas and volumes for elevations between values shown in Table 1 determined by interpolation
2) Low-flow weir $Q \approx 3.1*L*H^{1.5}$, with L = 1 ft; H = max(0, WSL – Crest Elev = 866.0 ft)
3) Principal Spillway weir $Q \approx 3.1*L*H^{1.5}$, with L = 7.5 ft; H = max(0, WSL – Crest Elev = 870.0 ft)

The data in Tables 5 and 6 were used to develop the paired input data for the elevation-storage and storage-discharge functions in the utilized for flood routing in HEC-HMS as input, labeled SWM2Z-V and SWM2V-Q, respectively.

The storage and discharge are plotted as functions of the stage in Figure 7.

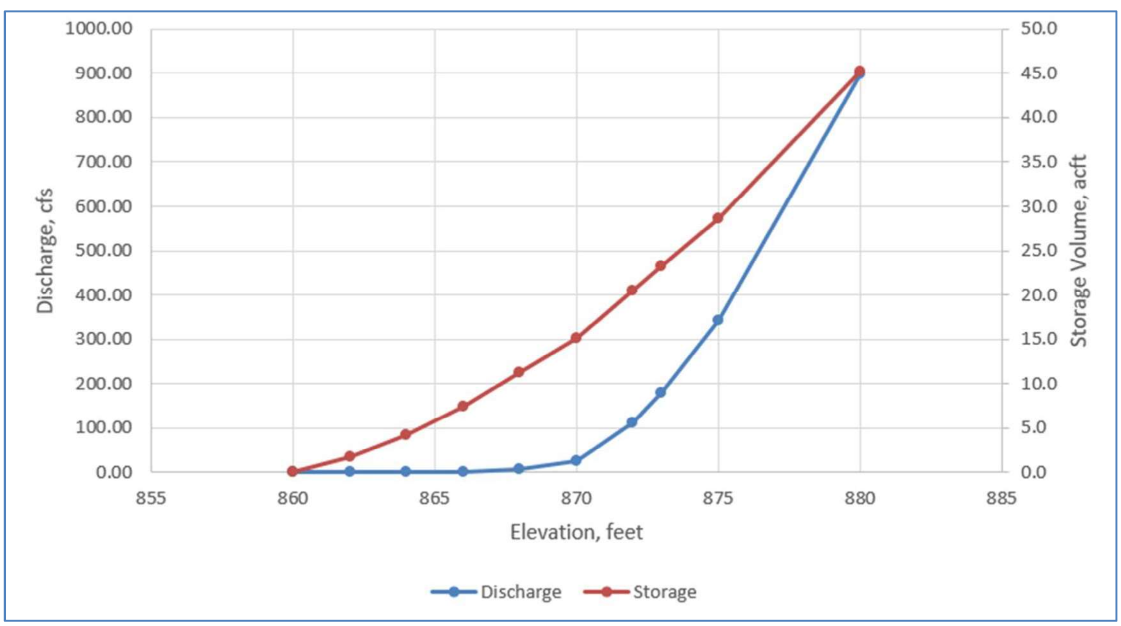

Figure 7 - Elevation-Storage and Elevation-Discharge Relations for SWM Pond No. 2

SWM Pond No. 2 was placed along Stony Camp Run, nearly at the confluence with Squaw Run. As the proposed SWM Pond is nearly at the confluence, the Stony Camp Run watershed (Area No. 7 in Figure 2) was not subdivided.

5.0 CALCULATION OUTPUT/RESULTS

Output files are located within the HEC-HMS project "***SquawRunModeling.hms***" for each simulation in the basin models labeled "Proposed-1" and "Proposed-2". The HEC-HMS project is saved in L:\EC\EC-WH\Planning_FPMS\2020SquawRun\2_Working. An electronic copy of the files is provided as Appendix A of this calculation brief.

Adequate pond performance was confirmed by noting that a 20-foot high embankment would not be overtopped for either SWM Pond No. 1 or 2 for the 100-year event, as shown in Figure 8.

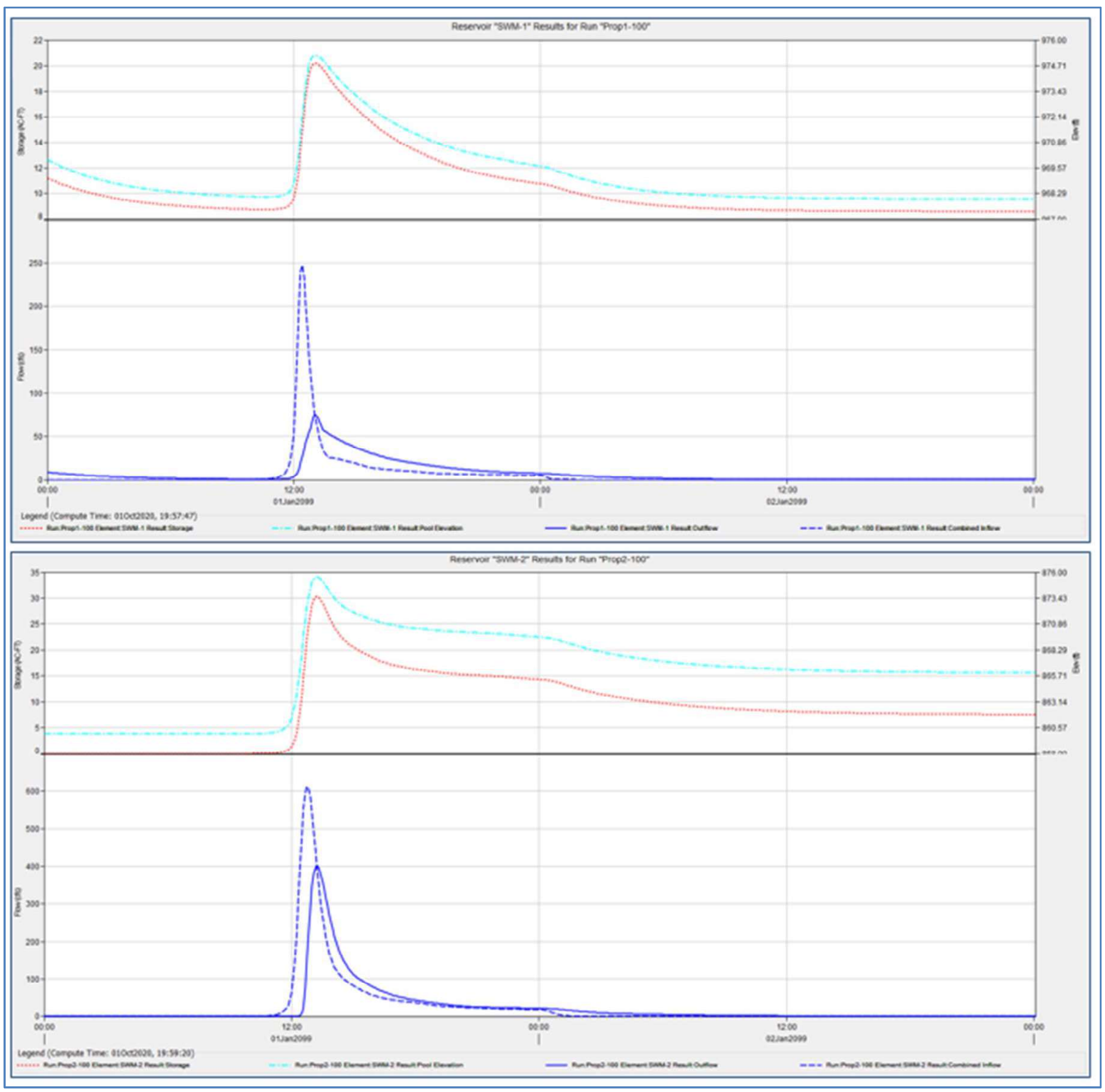

Figure 8 - Hydraulic Performance of SWM Pond No. 1 (upper) and SWM Pond No. 2 (lower) as Simulated in HEC-HMS

In order to provide an order of magnitude estimate of the volume of fill required for each embankment located as assumed, three feet of freeboard was added to the maximum water surface elevation behind each pond for the 100-year event as simulated in HEC-HMS.

Calculations for the estimated fill volume of each embankment are summarized in Table 7.

Table 7 - Order of Magnitude Estimate of Embankment Fill Volume for SWM Ponds 1 and 2

Geometric Element	SWM-1	SWM-2
Peak WS Elevation, ft	975.4	876.9
Freeboard, ft	3	3
Top of embankment Elev., ft	978.4	879.9
Toe of Slope Elevation, ft	960	860
Height, ft	18.4	19.9
Crest width, ft	12	12
Side slope 1:Z	3	3
Base width[1], ft	122.4	131.4
Embankment length[2], ft	600	350
Rough Volume[3], CY	27,477	18,496

Note:

1) Base width, BW = crest width + 2*Z*Height
2) Embankment length estimated in ArcGIS
3) A rough estimate of volume in cubic yards = 0.5*(BW + CW)*Heigth*Length/27

Design Discharges

Figure 3 within Section 2.0 shows the location of the HEC-HMS nodes at the outlet of each SWM pond and at the up- and down-stream ends of the study reach for which discharges are presented in the following discussion.

Table 8 shows the performance of the two SWM Ponds as measured by inflow and outflow for the 24-hour 2-, 10-, 25-, 50-, 100-, 200-, and 500-year return period events. The locations of the two ponds are shown as SWM-1 and SWM-2 in Figure 9.

Table 8- Performance of Potential SWM Ponds Nos. 1 and 2 (Flows in CFS)

Return period, years	SWM Pond NO. 1			SWM Pond NO. 2		
	Peak Inflow	Peak Outflow	% Attenuation	Peak Inflow	Peak Outflow	% Attenuation
2	40	9	-77.5%	95	11	-88.4%
10	110	21	-80.9%	263	85	-67.7%
25	161	34	-78.9%	388	189	-51.3%
50	204	48	-76.5%	498	292	-41.4%
100	248	74	-70.2%	614	402	-34.5%
200	296	122	-58.8%	741	524	-29.3%
500	362	184	-49.2%	919	689	-25.0%

Table 9 shows that the impact that each of the SWM Ponds would have on the peak discharge at the downstream end of the HEC-RAS study peak (if only one was installed) for the 24-hour 2-, 10-, 25-, 50-, 100-, 200-, and 500-year peak discharges at the downstream end of the HEC-RAS study reach (Location 1 in Figure 3).

Table 9 - Peak Discharges at the Downstream End of Study Reach for Proposed SWM Ponds 1 and 2 (Not Combined)

Return period, years	Peak Discharge at Outlet, cfs (Location 1)				
	Existing Conditions	Proposed SWM 1	%Change	Proposed SWM 2	%Change
2	497	477	-4.0%	391	-21.3%
10	1,367	1,279	-6.4%	1,135	-17.0%
25	2,205	2,072	-6.0%	1,880	-14.7%
50	2,959	2,798	-5.4%	2,636	-10.9%
100	3,703	3,566	-3.7%	3,453	-6.8%
200	4,670	4,556	-2.4%	4,393	-5.9%
500	6,151	6,036	-1.9%	5,801	-5.7%

Table 10 shows the impact that each of the SWM Ponds would have on the peak discharge at the upstream end of the HEC-RAS study peak (if only one was installed) for the 24-hour 2-, 10-, 25-, 50-, 100-, 200-, and 500-year peak discharges at the downstream end of the HEC-RAS study reach (Location 2 in Figure 3).

Table 10 - Peak Discharges at the Upstream End of Study Reach for Proposed SWM Ponds 1 and 2 (Not Combined)

Return period, years	Peak Discharge at J81, cfs (Location 2)				
	Existing Conditions	Proposed SWM 1	%Change	Proposed SWM 2	%Change
2	497	443	-10.9%	367	-26.2%
10	1,367	1,185	-13.3%	1,098	-19.7%
25	2,205	1,910	-13.4%	1,818	-17.6%
50	2,959	2,616	-11.6%	2,548	-13.9%
100	3,703	3,410	-7.9%	3,331	-10.0%
200	4,670	4,340	-7.1%	4,204	-10.0%
500	6,151	5,676	-7.7%	5,494	-10.7%

6.0 SUMMARY AND RECOMMENDATIONS

Proposed Condition hydrologic modeling was developed to evaluate the impact of potential SWM projects in the upper reaches of the Squaw Run watershed on flows in the downstream study reach for return periods between 2 and 500 years.

Two physically and institutionally feasible sites for the construction of SWM ponds were identified for evaluation based on a desk-top study of aerial photography and topographic mapping developed from USGS 3-meter DEM data. To facilitate an estimation of the cost for each SWM pond, an order of magnitude estimate of the volume of fill the embankment anticipated for each site is provided in Table 7.

Proposed Condition models were developed by modifying the existing conditions HEC-HMS model (USACE, 2020) to obtain a preliminary order of magnitude estimate of the maximum attenuation that could be achieved for each return period based on flood control storage that could be developed for a 20-foot high dam with outlet works performance based on weir control structures with dimensions and crest elevations selected to maximize storage of the incoming flood hydrograph for the upstream catchment.

A separate model run was developed for each SWM Pond installed as a single alternative based on the assumption that the marginal benefit of a second pond (expressed in terms of reduced annual expected damage) would not exceed the marginal annual cost of constructing a SWM pond at a second site.

The output from each SWM project was routed through the watershed to provide design discharges at the up- and down-stream ends of the study reach of Squaw Run for the HEC-RAS models developed in separate tasks for the preliminary evaluation of alternative structural and non-structural flood mitigation strategies, as summarized in Table 11. HEC-RAS modeling is used to evaluate the impact of in-channel improvement with and without an upstream SWM pond in a separate calculation.

Table 11 – Design Discharges for HEC-RAS models of the Study Reach of Squaw Run

Return period, years	SWM Pond 1		SWM Pond 2	
	Q peak (cfs) U/S of Study reach	Q peak (cfs) D/S of Study reach	Q peak (cfs) U/S of Study reach	Q peak (cfs) D/S of Study reach
2	443	477	367	391
10	1,185	1,279	1,098	1,135
25	1,910	2,072	1,818	1,880
50	2,615	2,798	2,548	2,636
100	3,410	3,566	3,331	3,453
200	4,341	4,556	4,204	4,393
500	5,676	6,036	5,494	5,801

Based on the results of simulations run for each SWM Pond as a single alternative, SWM Pond No. 2 (on Stony Camp Run) would be the recommended single upstream project.

7.0 REFERENCES

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