US Army Corps of Engineers
Philadelphia District

Tookany Creek Flood Risk Reduction Study

Appendix B-1
Hydrologic Modeling Appendix

Cheltenham Township
Montgomery County, PA

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CHELTENHAM TOWNSHIP

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Executive Summary

A hydrologic analysis for the Tookany Creek watershed within Cheltenham Township was conducted to investigate potential project alternatives in an attempt to lessen damages due to flooding. The Gridded Surface Subsurface Hydrologic Analysis (GSSHA) modeling code was used to quantitatively analyze existing conditions as well as various with project conditions.

A statistical flow-frequency analysis was conducted using the United States Geological Survey streamflow gaging station at Adams Ave. The results of this statistical analysis were compared against model results obtained using hypothetical precipitation events. Multiple hypothetical precipitation events were created using point precipitation estimates from the National Oceanic and Atmospheric Administration's Atlas 14 publication. Numerous events were created from frequently occurring, low intensity rain storms to high intensity, rarely occurring rain storms. Varying precipitation durations were also analyzed from 3-hrs to 24-hrs.

These hypothetical precipitation events were used to compare the effects of various with project conditions against existing conditions in order to determine potential flood risk reduction benefits. Computed flow rates were input to a HEC-RAS hydraulic model that was used to estimate water surface elevations (WSELs) for input to an economic analysis, both of which are detailed in additional technical appendices.

With project conditions that were analyzed for their hydrologic impacts included Low Impact Development options, constriction removals, local neighborhood flood walls, and storage areas. Storage areas were found to reduce peak flow rates (and in turn WSELs and flooding damages) to greater magnitudes and extents than any other option that was analyzed. Nine individual storage areas were simulated in addition to three different storage area groupings. A final, all-encompassing storage area grouping of all nine was also analyzed.

Due to the large reductions in peak flow rates predicted to occur due to the all-encompassing storage area grouping, this plan is recommended as the Tentatively Selected Plan.

Throughout this report, references to 50, 20, 10, 4, 2, 1, 0.5, and 0.2% annual chance exceedances correspond to 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals, respectively. Additionally, annual chance exceedance probability or frequency refers to the chance of an event equal to or greater than the stated magnitude occurring in a given year.

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1. Introduction

Tookany Creek has repeatedly flooded causing significant damage within its watershed. Following several flooding events in 2011, the U.S. Army Corps of Engineers (USACE), Philadelphia District partnered with Cheltenham Township to explore possible flood risk reduction measures within the Tookany Creek watershed. The aspects of the study contained in this report details the hydrologic and hydraulic investigations that formed the basis of possible flooding solutions for the Tookany Creek watershed.

1.1. Area of Interest

The Tookany Creek watershed is part of the larger Tookany/Tacony-Frankford (TTF) watershed. The TTF drains approximately 36 square miles (mi^2) from two counties (Philadelphia and Montgomery) and seven municipalities (Cheltenham, Springfield, Abington, Jenkintown, Rockledge, and Philadelphia). The stream is termed "Tookany Creek" above the Cheltenham Township / Philadelphia County boundary, "Tacony Creek" within Philadelphia County and above Castor Avenue, and "Frankford Creek" below Castor Avenue until it empties into the Delaware River near the Betsy Ross Bridge. Major stream systems bordering the TTF watershed include the Pennypack Creek to the east, Delaware River to the south, Wissahickon Creek to the west, and Schuylkill River to the south west.

Tookany Creek drains the majority of Cheltenham Township. Several smaller streams drain to the Tookany Creek, including Baederwood Creek, Jenkintown Creek, and Rock Creek. Since Cheltenham Township is the non-federal sponsor, all flood risk reduction activities must benefit Cheltenham Township. Therefore, the area of interest for this study was delimited above the Cheltenham Township / Philadelphia County boundary near Adams Avenue.

Topographically delineating above this point using a 1 meter (approximately 3.2 ft) horizontal resolution Light Detention and Ranging (LIDAR) elevation coverage resulted in the creation of an approximately 16 mi^2 watershed. However, certain areas within Philadelphia County have been redirected to flow towards Pennypack Creek through various sewer systems. Therefore, these areas no longer contribute runoff to Tookany Creek and were removed from consideration. The resultant study watershed was therefore reduced in size to approximately 15.6 mi^2.

The final study watershed, larger TTF watershed, major stream systems, roadways, and administrative boundaries are shown in Figure 1.1.

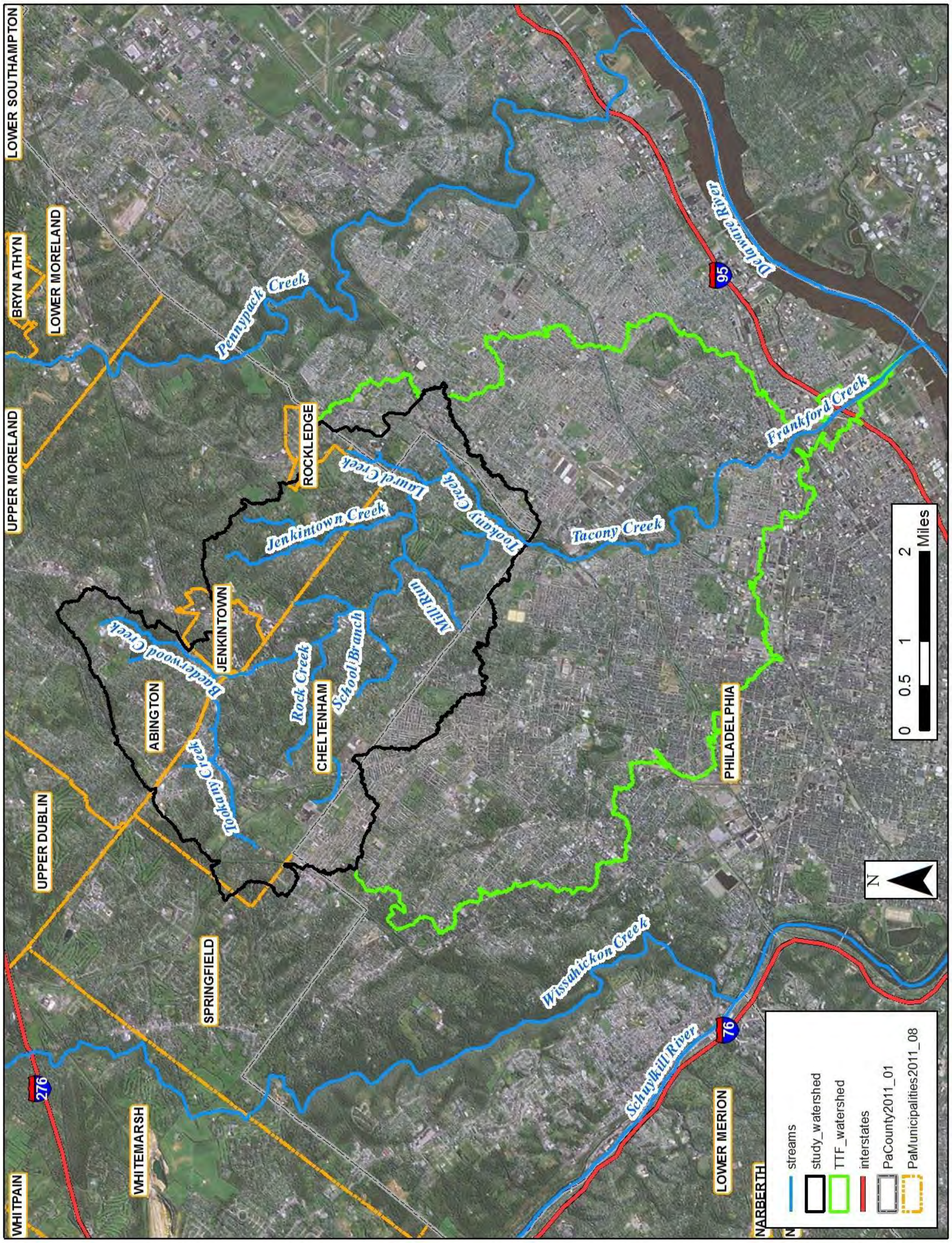

Figure 1.1 – TTF Watershed and Area of Interest
ESRI World Imagery

1.2. Modeling Purpose

Many potential solutions can be implemented to reduce flooding risks within Cheltenham Township. However, some may have unintended consequences to private and public properties as well as the environment. Current USACE guidelines restrict federal cost sharing to projects that contain a benefit to cost ratio (BCR) greater than one (i.e. benefits must outweigh costs on an annualized basis).

In order to predict, quantify, and maximize benefits for any flood risk reduction scheme, hydrologic, hydraulic, and economic modeling was required.

1.3. Previous Investigations

Relevant studies that explored the surface water and groundwater networks in and around the area of interest have been completed by the Philadelphia Water Department (PWD), South Eastern Pennsylvanian Transportation Association (SEPTA), and Pennsylvania Department of Environmental Protection (PADEP), amongst others.

To comply with requirements of the Pennsylvania Stormwater Management Act (Act 167) of 1978, PWD completed an Act 167 Assessment of the TTF watershed.[1] This plan utilized hydrologic modeling to determine water quality and quantity loading originating within the TTF watershed. Updated regulation activities were then determined on a watershed scale in order to adhere to the Act 167 requirements.

SEPTA undertook a hydraulic modeling investigation to alleviate maintenance and operational difficulties of SEPTA property within the area of interest due to flooding along Tookany Creek.[2] Updated bridge designs were developed at three locations near Jenkintown Station.

PADEP has completed several studies of communities within Cheltenham Township and the Tookany Creek watershed. Studies culminating in the construction of altered channel segments, a levee, and a corresponding pumping station have been completed. Finally, a flood risk reduction study authored by PADEP is currently being finalized for the Glenside area.

1.4. Neighborhoods

Several neighborhoods throughout Cheltenham Township were visited during September 26 – 28, 2012. These site visits were executed to document watershed-specific conditions and historic flooding accounts as well as identify potential flood risk reduction measures. Neighborhoods that were visited included Bickley Road, Brookdale Avenue, Brookside Road, Shoemaker Road, Cliff Terrace, Harrison Avenue, High School Road, Mill Road, and Rock Lane. Residents of each neighborhood provided detailed accounts of flooding events including water depths, inundations, and accrued damages.

The neighborhoods that were visited in September 2012 are visually identified in Figure 1.2.

2. Geospatial Data

These analyses / modeling efforts made use of various sources of geospatial data. These data sources were compiled to generate and assess modeling inputs and outputs. The main Geographic Information System (GIS) used to process this data was Environmental Systems Research Institute's (ESRI) ArcGIS ver. 9.3 and 10.0. Additional geospatial manipulations were performed using Aquaveo's Watershed Modeling System (WMS) ver. 9.0 as well as extensions to ArcGIS, namely the Hydrologic Engineering Center's (HEC) HEC-GeoHMS ver. 5.0[3] and 10.0 and HEC-GeoRAS ver. 4.3.93[4] and 10.0 add-ons.

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[1] (Borton-Lawson Engineering, Inc., 2008)
[2] (Urban Engineers, Inc., 2005)
[3] (Hydrologic Engineering Center, 2010)
[4] (Hydrologic Engineering Center, 2010)

The common horizontal datum used in this analysis was the North Atlantic Datum of 1983, while the coordinate systems varied between Pennsylvania South State Plane (feet) and Universal Transverse Mercator (UTM), Zone 18 North. All data that wasn't natively in these coordinate systems/datums was transformed. Furthermore, the vertical datum used in this analysis was North American Vertical Datum of 1988 (NAVD88), feet. Depending upon the age of the original data source, the vertical datum reported for each piece of data varied between NAVD88 and National Geodetic Vertical Datum 1929 (NGVD29). Differences between NGVD29 and NAVD88 vary from location to location. For simplification, a uniform conversion factor of 1.02 ft (i.e. 100 ft NGVD29 = 98.98 ft NAVD88) was used to convert NGVD29 elevation data sources to NAVD88 for the study area.

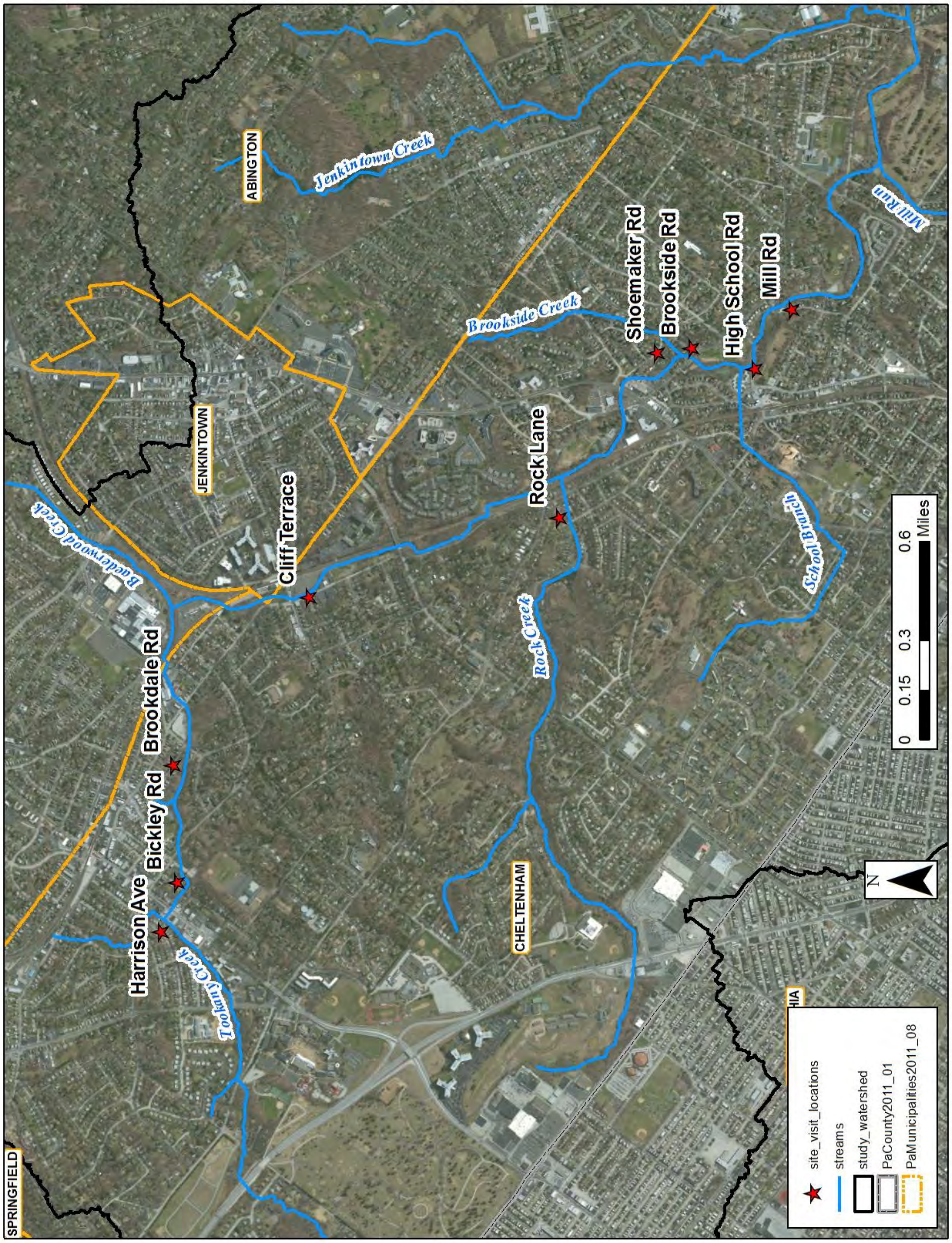

Figure 1.2 – Neighborhoods Visited in September 2012
ESRI World Imagery

3. Study Area Conceptual Model

The three dimensional surface and subsurface system needed to be conceptualized prior to executing a modeling analysis. This was achieved through the use of a "conceptual model" which is a detailed description of the area of interest. The conceptual model is intended to identify the various hydrologic, hydraulic, topographic, and geological features that physically affect the flow of water within the area of interest.

3.1. Topography

Pennsylvania can be divided into several distinct physiographic provinces. The area of interest is contained within two provinces separated by a vague fall line escarpment: the Piedmont and Atlantic Coastal Plain. The Piedmont province is characterized by flat-topped hills and shallow valleys while the Atlantic Coastal Plain is comprised of flat terraces and shallow valleys. Essentially, the latter province is the Delaware River floodplain.[5]

Elevations within the area of interest range from approximately 60 ft near the Cheltenham / Philadelphia County boundary to nearly 430 ft in the northwestern portions of the Tookany Creek watershed. These elevations were sourced from the Pennsylvania Department of Conservation and Natural Resources (DCNR) PAMAP LIDAR elevation coverages, which were representative of 2008 conditions.[6] These coverages, which formed the primary source of elevation data for this study, were sourced as Digital Elevation Models (DEM) with a horizontal resolution of 1 meter. Generally, the accepted vertical accuracy of these coverages is +/- 1 ft. A mosaic was created from each individual DEM using tools within ArcGIS to form a complete elevation model as shown in Figure 3.1.

[5] http://www.dcnr.state.pa.us/topogeo/field/map13/index.htm
[6] http://www.pasda.psu.edu/

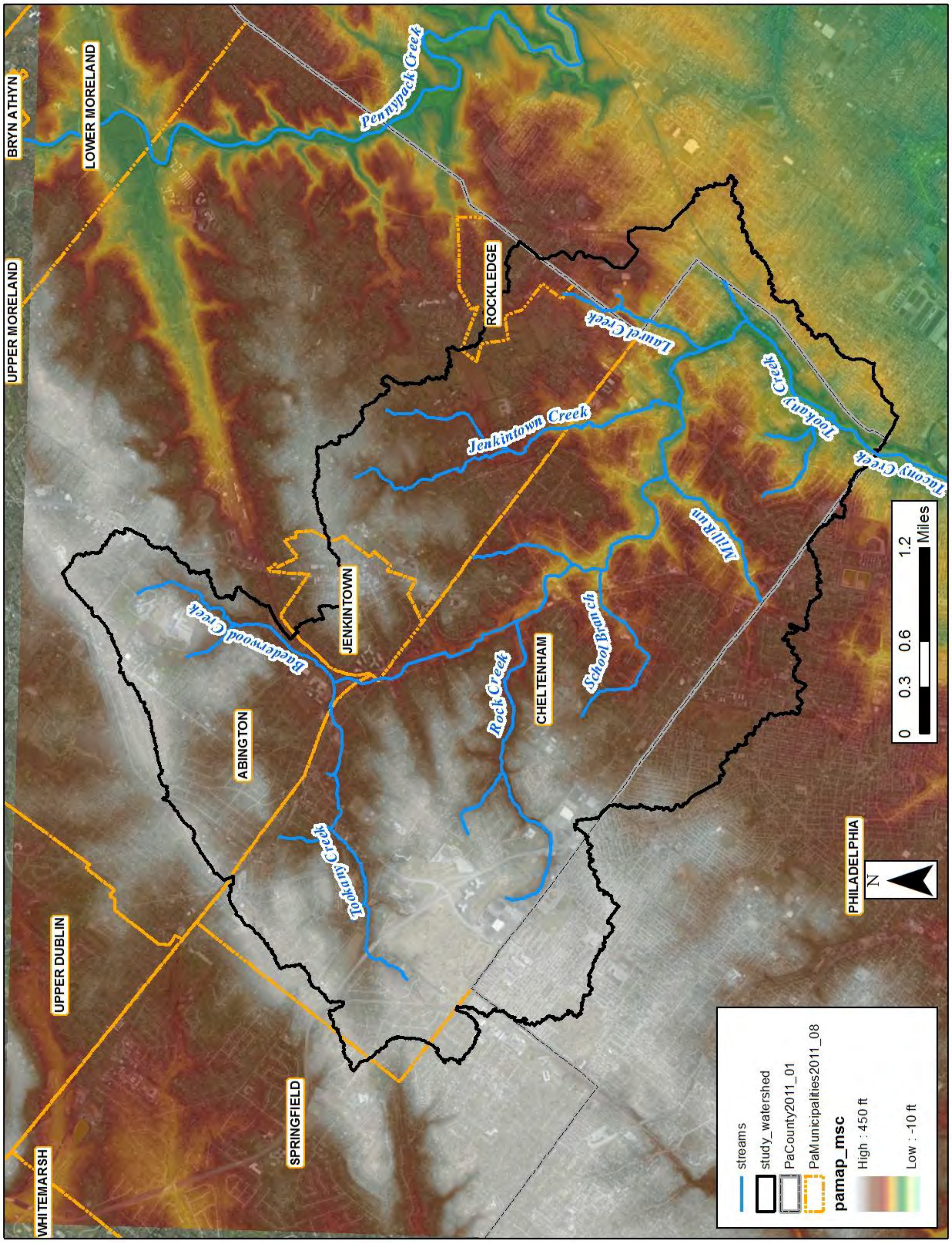

Figure 3.1 – Elevations near the Area of Interest
ESRI World Imagery

3.2. Climate and Precipitation

The Tookany Creek watershed has a climate that is typical of the Piedmont and Coastal Plain provinces. This includes warm and humid summers with wet and variable winters. Residing in a northeastern state, the area of interest is exposed to occasional tropical storms (hurricanes) and extra-tropical storms ("northeasters"). However, thunderstorms, which normally occur during the summer months, are the predominant storm type.

Air temperatures within the area of interest, as recorded at two United States Air Force 14th Weather Squadron (USAF – 14WS) hydrometeorological stations that are near the area of interest, vary from near zero (Fahrenheit) temperatures during the winter months to near 100 degree temperatures during the summer months.

Average annual point rainfall within and around the Tookany Creek watershed, as derived from nearby precipitation gaging stations, usually varies between approximately 30 to 60 inches. Average annual point snowfall within the area of interest can also vary between 10 and 30 inches.[7] These variations are also supplemented by temporal and spatial distributions due to topographic relief (orographic effects) and effective weather patterns.

The previously mentioned precipitation gaging stations near the area of interest are maintained by the National Oceanic and Atmospheric Administration (NOAA) National Climactic Data Center (NCDC)[8] in addition to several gages maintained by PWD. Additionally, three non-recording gages are maintained by Cheltenham Township throughout the area of interest with limited records. The locations of these precipitation gaging stations, as well as the USAF – 14WS locations, are shown in Figure 3.2 and detailed in Tables 3.1 and 3.2.

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[7] http://www.erh.noaa.gov/ctp/features/snowmaps/index.php?tab=norms

[8] http://www.ncdc.noaa.gov/oa/ncdc.html

Table 3.1 – Precipitation Gaging Stations near the Area of Interest

ID	OWNER	LOCATION	LONG	LAT	Type	Period of Record
14	PWD	NEWPC Plant	-75.08	39.99	15min	1990 - current
7	PWD	Public Property Sign Shop	-75.11	40.02	15min	1990 - current
17	PWD	SEPTA Depot	-75.07	40.03	15min	1990 - current
8	PWD	Heinz Tank Farm	-75.12	40.03	15min	1990 - current
11	PWD	Lawncrest Public Library	-75.10	40.05	15min	1990 - current
19	PWD	Emlen Middle School	-75.18	40.05	15min	1990 - current
13	PWD	Northeast High School	-75.07	40.06	15min	1990 - current
10	PWD	Medical Mission Sisters	-75.08	40.08	15min	1990 - current
100	Cheltenham	Township Admin Building	-75.13	40.08	non-recording	2010 - current
101	Cheltenham	Rowland Community Center	-75.10	40.06	non-recording	2010 - current
102	Cheltenham	Brookdale Pump Station	-75.15	40.10	non-recording	2010 - current
201	NCDC	GLASSBORO_2_NE	-75.12	39.70	15min	1984 - 1998
202	NCDC	GLENMOORE	-75.78	40.10	15min	1971 - current
203	NCDC	GRATERFORD_1_E	-75.43	40.23	15min	1976 - 2002
204	NCDC	MT_HOLLY	-74.80	39.98	15min	1971 - current
205	NCDC	PALM_3_SE	-75.50	40.38	15min	1971 - current
206	NCDC	PHILA_INTL_AP	-75.23	39.87	hourly	1900 - current
207	NCDC	NE_PHILA_AP	-75.02	40.08	15min	1984 - 1989
208	NCDC	PHOENIXVILLE_1_E	-75.50	40.12	15min	1984 - 2008
209	NCDC	SELLERSVILLE	-75.33	40.38	15min	1984 - current
210	NCDC	TRENTON_ST_COLLEGE	-74.79	40.27	15min	1977 - 2003
211	NCDC	WINDSOR	-74.58	40.25	15min	1971 - 2009

Table 3.2 – USAF – 14WS Gaging Stations near the Area of Interest

Station	LOCATION	LAT	LONG
KPHL	Philadelphia International AP	39.88	-75.25
KNXX	Willow Grove NAS JR	40.20	-75.15
KPNE	Northeast Philadelphia	40.08	-75.01
KVAY	South Jersey Regional	39.95	-74.85

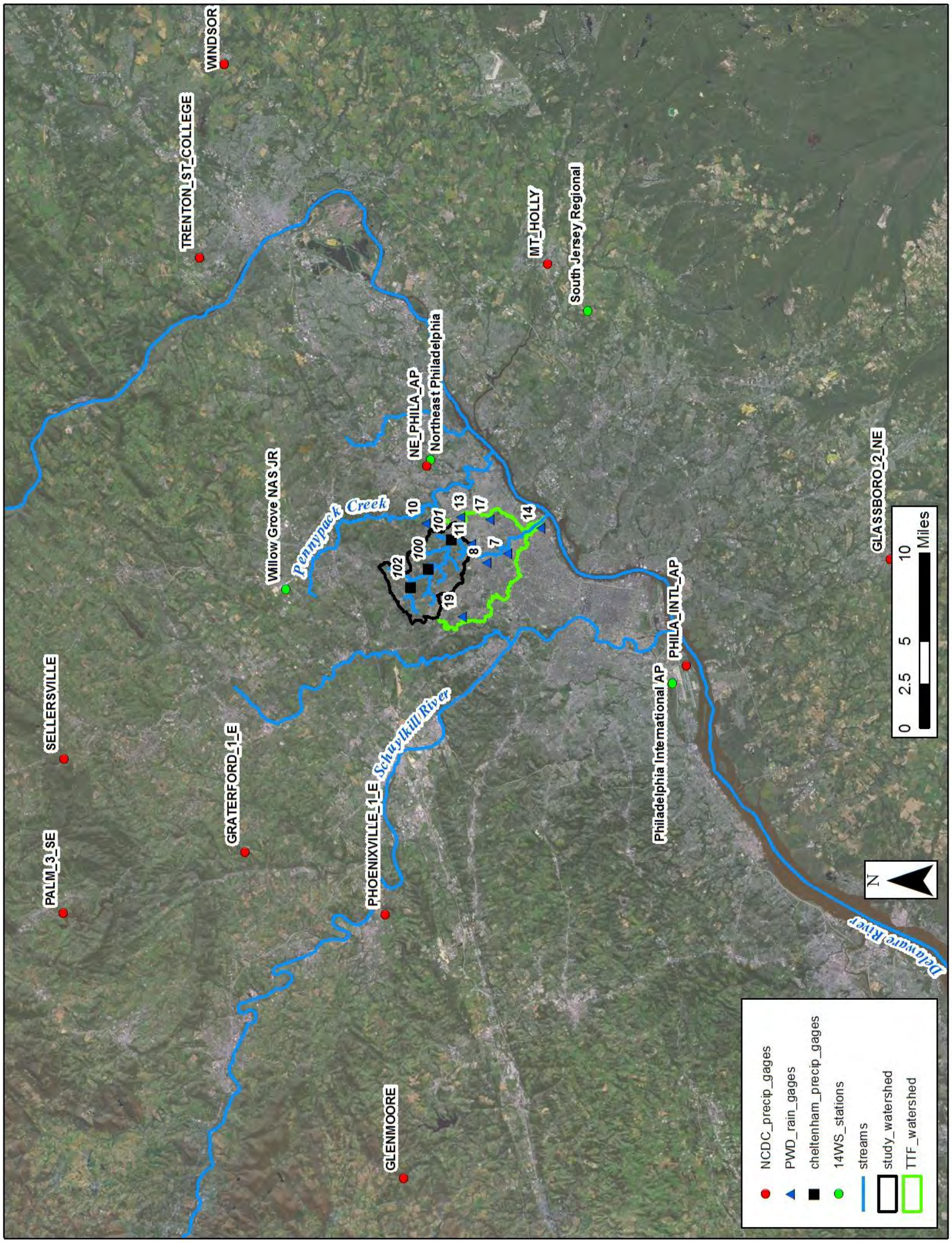

Figure 3.2 – Precipitation and Hydrometeorological Gaging Stations
ESRI World Imagery

3.3. Streamflow

Depending upon the time of year and the composition of the upstream watershed, streamflow is highly variable within the Tookany Creek watershed. In locations where upstream development is heavy, groundwater infiltration has been largely removed and therefore ephemeral streamflow can exist within historically perennial streams. In other locations that have less upstream development, sources of groundwater flow are available to promote nearly constant streamflow to the existing stream network.

In an effort to promote public health as well as increase available real estate for development, several streams (both perennial and ephemeral) have been paved over and confined to sewer systems within the Tookany Creek watershed. This practice was used by all of the municipalities within the area of interest. The most extensive use of this practice was within Philadelphia County, where an extensive combined sanitary and storm sewer system exists. This arrangement can severely degrade water quality during times of heavy rainfall when the system capacity is exceeded and combined sewer overflows (CSO) occur. As was previously mentioned, several portions of the historic Tookany Creek watershed have been diverted to flow to the Pennypack Creek through storm sewer systems.

Two United States Geological Survey (USGS) streamflow gaging stations are currently active within the TTF watershed. The first gage is located near the Cheltenham / Philadelphia County boundary above Adams Avenue (where the “Tookany Creek” transitions to the “Tacony Creek”) and was used to set the downstream limits of the study watershed. While this gage was installed in 1965, the period of record is not continuous with missing discharge records from 1986 – 2005. However, continuous discharge measurements are available since Oct. 2005 at 15 minute intervals.

The second USGS gage is located at Castor Ave (where the “Tacony Creek” transitions to the “Frankford Creek”). This gage was installed in July 1982 with no missing discharge records. Continuous discharge records are available since Oct 1990 at 15 minute intervals.

Additional USGS gages have been historically active within the TTF watershed. These gages were located along tributaries to the Tookany Creek and on the main stem TTF as well. However, due to their short periods of record and age, they were not used as part of this modeling effort. Pertinent data relating to these USGS streamflow gaging stations is detailed in Table 3.3 while the locations of these gages are shown in Figure 3.3.

Table 3.3 – USGS Streamflow Gaging Stations near the Area of Interest

USGS ID	Name	LAT	LONG	Period of Record	Published Drainage Area (mi^2)
01467083	Tacony Creek near Jenkintown, PA	40.09	-75.14	10/1973 - 10/1978	5.25
01467084	Rock Creek ab Curtis Arboretum near Philadelphia	40.08	-75.15	5/1971 - 10/1978	1.15
01467085	Jenkintown Creek at Elkins Park, PA	40.08	-75.11	10/1973 - 10/1978	1.17
01467086	Tacony Creek ab Adams Avenue, Philadelphia, PA	40.05	-75.11	10/1965 - 9/1970, 6/1974 - 9/1986, 10/2005 - current	16.7
01467087	Frankford Creek at Castor Ave, Philadelphia, PA	40.02	-75.10	7/1982 - current	30.4
01467089	Frankford Creek at Torresdale Ave, Philadelphia, PA	40.01	-75.09	10/1965 - 7/1982	33.8

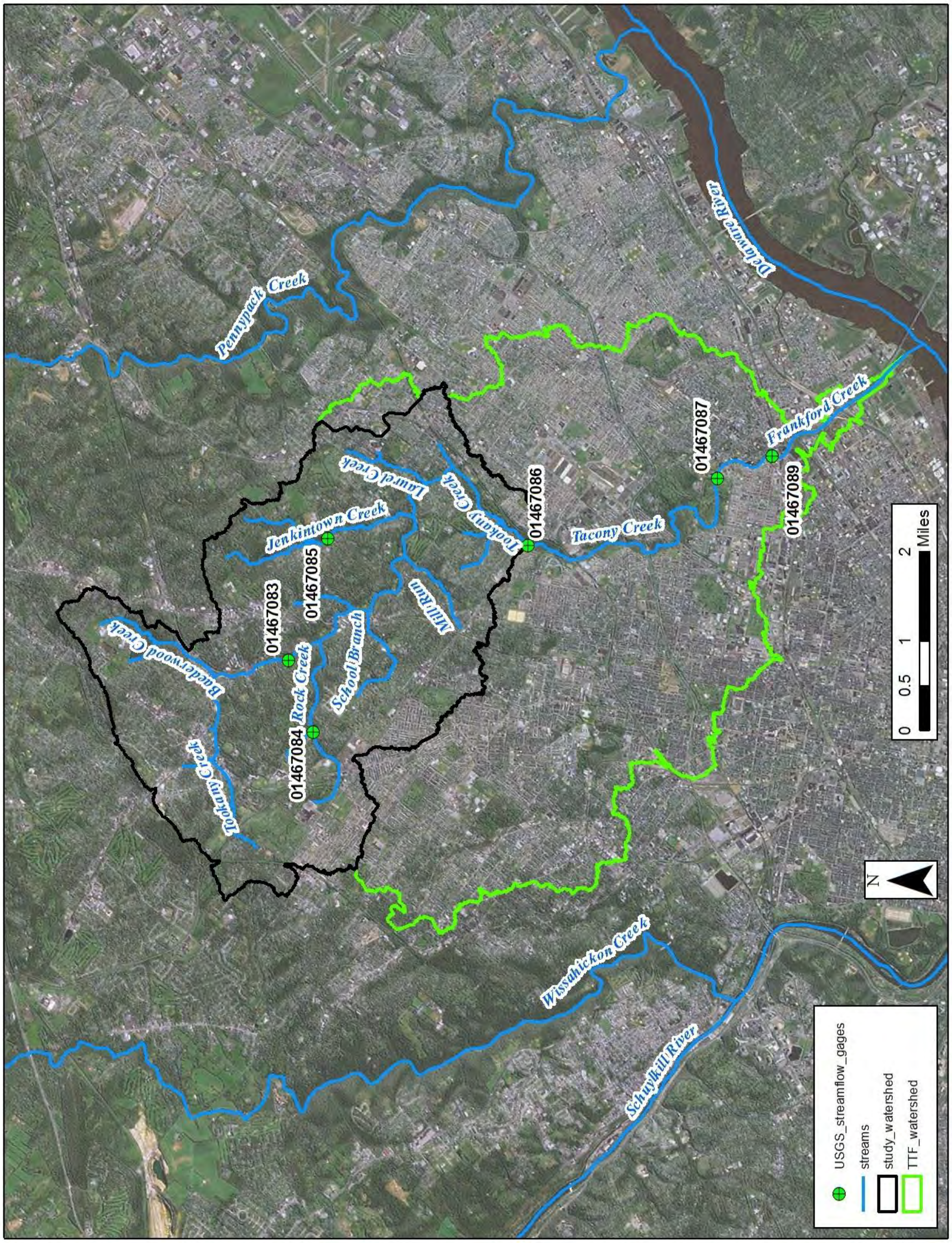

Figure 3.3 – USGS Streamflow Gaging Stations
ESRI World Imagery

3.4. Existing Infrastructure

Man-made infrastructure within the Tookany Creek watershed plays a large role in both the occurrence of flooding and the severity of flooding. Man-made infrastructure includes projects built to reduce flooding risks as well as those that disregarded flooding risks when they were constructed.

Existing flood control projects within the Tookany Creek watershed include storm sewers, channel modifications (channelization), levees, pumping stations, and scattered small scale detention basins.

Approximately 131 channel obstructions within the Tookany Creek watershed were identified by PWD using in-stream surveys. These obstructions included bridges and culverts on the main stem Tookany Creek as well as many tributaries.

As was previously mentioned, an extremely large scale storm sewer system exists within the area of interest. Major individual systems include those along Cheltenham Ave, Cottman Ave, Keswick Ave, and Limekiln Pike.

Several segments of the Tookany Creek have been altered to increase flow capacity. These segments include both concrete lined portions and earthen channels with varying cross sectional shapes including vertical walls and trapezoidal shapes.

The Brookdale Avenue levee was constructed in 1952 to provide reduced flooding risks to the low-lying Brookdale Avenue neighborhood in the Glenside area of Cheltenham Township. Located along the downstream left side of the channel, the alignment stretches approximately 1000 linear feet in length with varying heights up to 5 ft. The top width along the levee crest is approximately 10 ft while side slopes are approximately 1:2 (H:V) on both the stream and landward sides. An accompanying pumping station completed in 1978 consists of three pumps, trash racks, and a backup diesel generator. The location of the pumping station requires interior drainage to move past many homes, thereby raising flooding risks to the "protected" side of the levee. Historically, the trash racks have also become clogged with trash and debris which prevents the effective operation of the pumping station.

The locations of several large storm sewer systems and other identified infrastructure is shown in Figure 3.4 with a more detailed view of the Brookdale Avenue Levee alignment and Pump Station shown in Figure 3.5.

This is by no means a comprehensive description of listing of infrastructure within the Tookany Creek watershed. However, this listing includes the most influential structures that relate to or affect flooding within the Tookany Creek watershed.

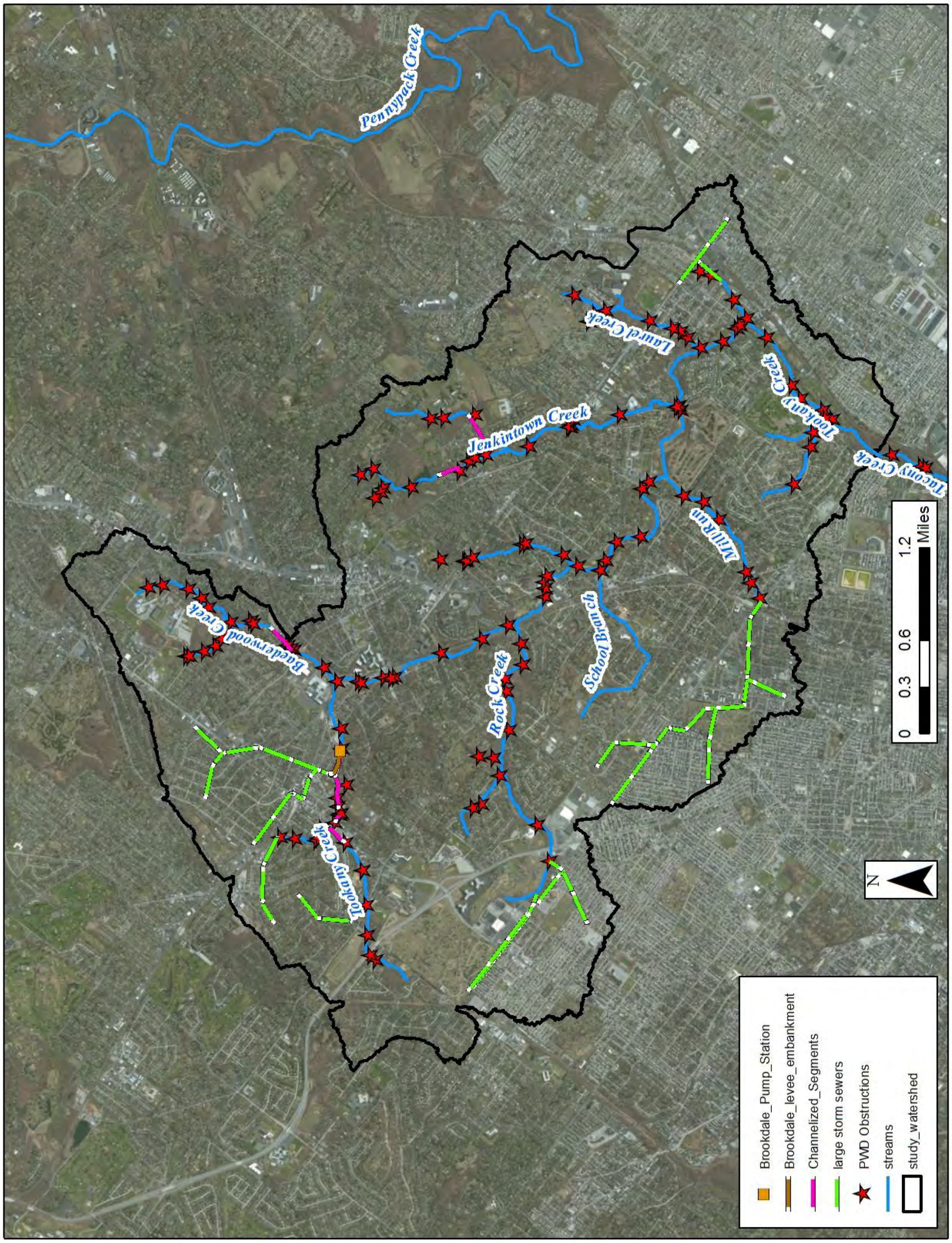

Figure 3.4 – Existing Infrastructure
ESRI World Imagery

Figure 3.5 – Brookdale Avenue Levee
ESRI World Imagery

4. Hydrologic Model Development

In order to ascertain the various hydrologic possibilities within the Tookany Creek watershed in addition to determining distributed frequency discharge rates, a hydrologic model was needed. A hydrologic model simulates precipitation runoff and routing characteristics, both natural and man-made. The essence of a hydrologic model is to transform precipitation (known) into runoff / streamflow (unknown) at given locations and times.

4.1. Modeling Code

To more accurately simulate the three dimensional and dynamic nature of flowing water within the area of interest, a physics-based, numerical modeling scheme / code was required. The code chosen was the Gridded Surface Subsurface Hydrologic Analysis (GSSHA) ver. 6.0.[9] This code is developed by the USACE Engineering Research and Development Center's Coastal and Hydraulics Laboratory (ERDC - CHL).

GSSHA allows the user to simulate overland, channel, and groundwater flow processes as well as sediment fate and transport through coupled routines using finite volume approximations. GSSHA has been used to analyze the Upper Mississippi River Valley region,[10] extensively modified agricultural watersheds in Minnesota,[11] and the San Jacinto River Basin in California,[12] amongst others.

The GSSHA modeling code was chosen for several reasons. First, being a dynamic, physics-based model, GSSHA allows water to flow in different directions over time. This is important in watersheds like the Tookany Creek where the overland network has been substantially altered by human activities (i.e. runoff flow directions aren't necessarily constant throughout time). Second, GSSHA has the ability to model underground pipe routing that interacts with the surface water routing routines on a time-step scale. This is vital due to the large-scale storm sewer systems present within the area of interest. Third, due to the relatively short period of record and the location of the USGS streamflow gage at Adams Avenue, frequency-based precipitation was required to determine frequency flow rates throughout the area of interest, especially for events with smaller exceedance probabilities (more intense events). Frequency flow rates would then be used to gauge the effectiveness of with project options. Fourth, with project conditions specifics (types, locations, etc) were not known at the start of this effort. GSSHA allows users more flexibility when analyzing potential with project conditions than most other hydrologic modeling codes that use unit hydrograph approaches requiring sub-basin delineations at potential with project condition locations. Finally, the short and long term effects of possible with project conditions options to reduce flooding risks within the Tookany Creek watershed, such as distributed land form changes, stream channel modifications, and underground alterations, can be explicitly modeled using GSSHA.

4.2. Modeling Domain

The GSSHA modeling code was applied to the modeling domain to accurately replicate the hydrology of the area of interest. As was previously described, this domain was created by topographically delineating the Tookany Creek watershed upstream from the USGS streamflow gaging station at Adams Ave near the Cheltenham Township / Philadelphia County boundary. This delineation was performed using the previously mentioned PAMAP LIDAR elevation dataset by means of tools within ArcGIS and HEC-GeoHMS.

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[9] (Downer, Ogden, & Byrd, 2008)
[10] (Downer, 2008)
[11] (Downer, James, & Eggers, 2002)
[12] (Fong, Downer, & Byrd, 2007)

4.2.1. Overland Grid

GSSHA makes use of a gridded network as the overland computational framework. Within the modeling domain, it was desirable to capture as many surface features as possible, necessitating relatively small grid cell sizes. However, small grid cell sizes tend to be more computationally intensive than larger grid cell sizes, requiring longer run times. Therefore, a 15 meter x 15 meter grid cell resolution (approximately 2400 ft^2 per grid cell) was chosen as a compromise between resolution and run time requirements. The modeling domain overlain by this grid cell size resulted in the formulation of approximately 179,000 active cells. A no flow boundary was assumed to exist along the lateral edges of the modeling domain, except at the watershed outlet.

Representative physical parameters that were required by the GSSHA modeling code for each grid cell included elevation, land use, and soil texture. These parameters were used for the various hydrologic processes within GSSHA. The PAMAP DEMs were used to assign representative elevations to the overland computational grid. Topographic artifacts (sinks) resulting from the conversion of the approximately 1 m x 1 m PAMAP DEM to the 15 m x 15 m GSSHA grid were selectively removed by hand and also through the use of the CHL program "CleanDam" to reduce computational burdens.[13] However, areas that were actually sinks on the overland network were allowed to remain.

4.2.2. Infiltration

Infiltration computations were executed using a modified Green and Ampt routine to account for soil moisture redistribution throughout a simulation. Representative parameters for the infiltration computations were assigned using surficial textural estimates.[14,15] Soils representative of 2005 – 2009 conditions were acquired from the Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database.[16] The various soil types within the modeling domain were aggregated to six individual textures. The primary soil types were found to be loam and silt combinations. The initial Green and Ampt parameter estimates for each SSURGO ID are shown in Table 4.1 while the various soil types within the modeling domain are shown in Figure 4.1.

Table 4.1 – SSURGO Soil Types within the Modeling Domain

Soil Type	Symbol	Saturated Hyd. Conductivity (cm/hr)	Wetting Front Suction Head (cm)	Porosity	Pore Size Distribution Arithmetic Mean	Residual Saturation	Field Capacity Saturation	Wilting Point Saturation
gravelly loam	GR-L	1.09	11.01	0.412	0.378	0.041	0.207	0.095
loam	L	0.66	8.89	0.434	0.252	0.027	0.27	0.117
silt loam	SIL	0.34	16.68	0.486	0.234	0.015	0.33	0.133
slightly decomposed plant material	SPM	0.1	20.88	0.39	0.242	0.075	0.318	0.197
variable / assumed silt loam	VAR	0.34	16.68	0.486	0.234	0.015	0.33	0.133
channery loam	CN-L	1.09	11.01	0.412	0.378	0.041	0.207	0.095

[13] http://www.gsshawiki.com/gssha/Utility_Programs:CleanDam
[14] http://www.gsshawiki.com/gssha/Infiltration:Parameter_Estimates
[15] (Rawls & Brakensiek, 1983)
[16] http://soildatamart.nrcs.usda.gov/

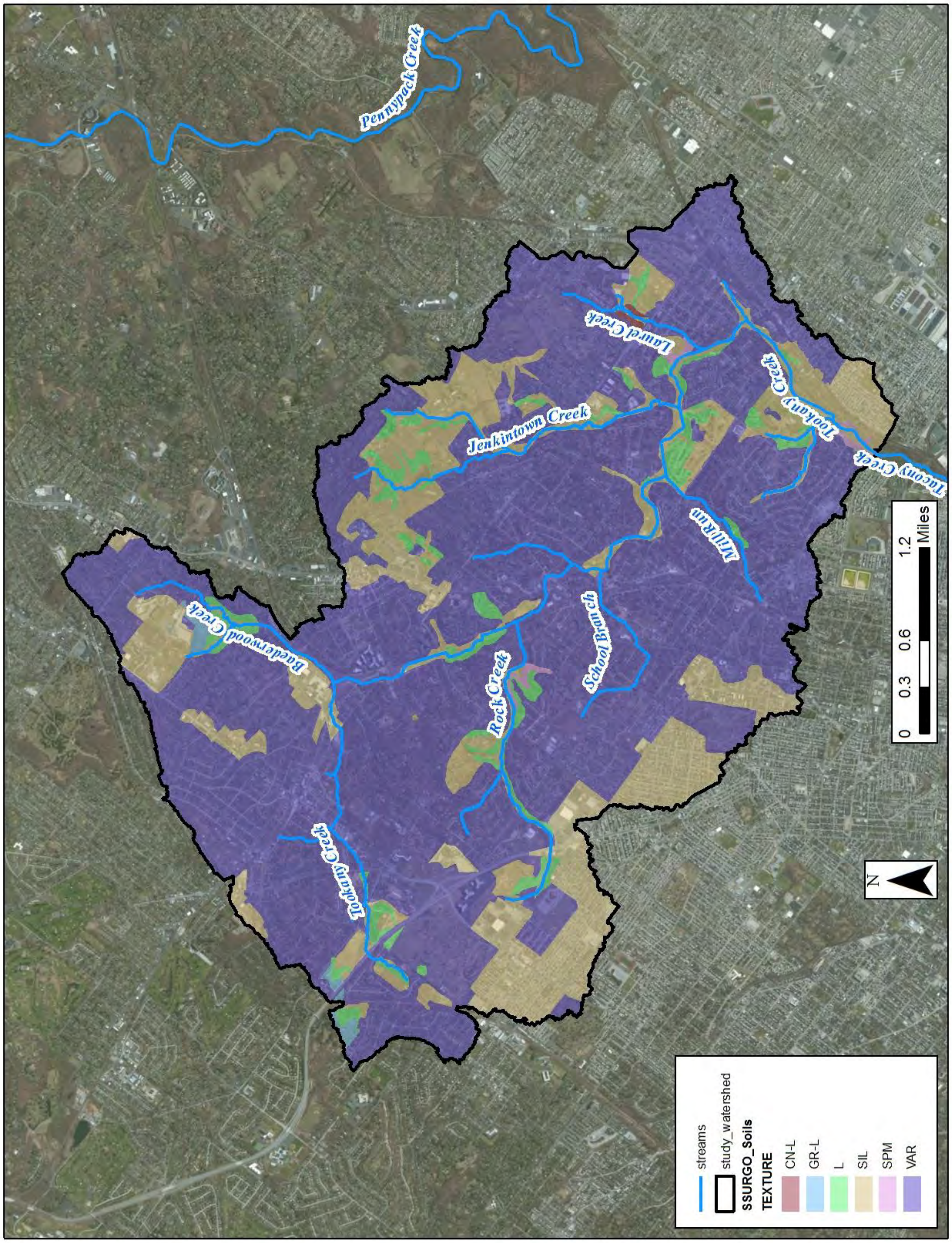

Figure 4.1 – SSURGO Soil Types
ESRI World Imagery

4.2.3. Overland Routing

Overland routing computations were performed using the two-dimensional, alternating direction explicit (ADE), finite volume, diffusive wave overland routing routine. Land uses were used to assign the representative parameters for several overland hydrologic processes. The National Land Cover Database (NLCD, ver. 2006) developed by the USGS Earth Resources Observation and Science (EROS) Center, which makes use of the Anderson land use classification system, was used to assign land uses throughout the modeling domain.[17] These land uses were representative of 2006 conditions.

Initial estimates of overland roughness, area reductions (used to account for impervious surfaces), and retention storage were based upon representative land uses.[18] Table 4.2 details the various Anderson land use classifications as well as the representative GSSHA initial parameter values that were based upon each land use. The land uses throughout the modeling domain are shown in Figure 4.2.

Table 4.2 – NLCD 2006 Land Use within the Modeling Domain

ID	Description	Roughness	Retention Depth (mm)	Impervious Area (%)
11	Open Water	0.001	0	100
21	Developed, Open Space	0.1	2	10
22	Developed, Low Intensity	0.1	1.5	35
23	Developed, Medium Intensity	0.1	1.25	65
24	Developed, High Intensity	0.1	1	90
31	Barren Land (Rock/Sand/Clay)	0.1	1	30
41	Deciduous Forest	0.4	7.5	0
42	Evergreen Forest	0.4	7.5	0
43	Mixed Forest	0.4	7.5	0
52	Shrub / Scrub	0.2	5	0
81	Pasture / Hay	0.25	5	0
90	Woody Wetlands	0.25	5	20
95	Emergent Herbaceous Wetlands	0.25	5	20

[17] http://eros.usgs.gov/#/Science/Landscape_Dynamics/Land_Cover-Land_Use/National_Land_Cover
[18] (MacArthur & DeVries, 1993)

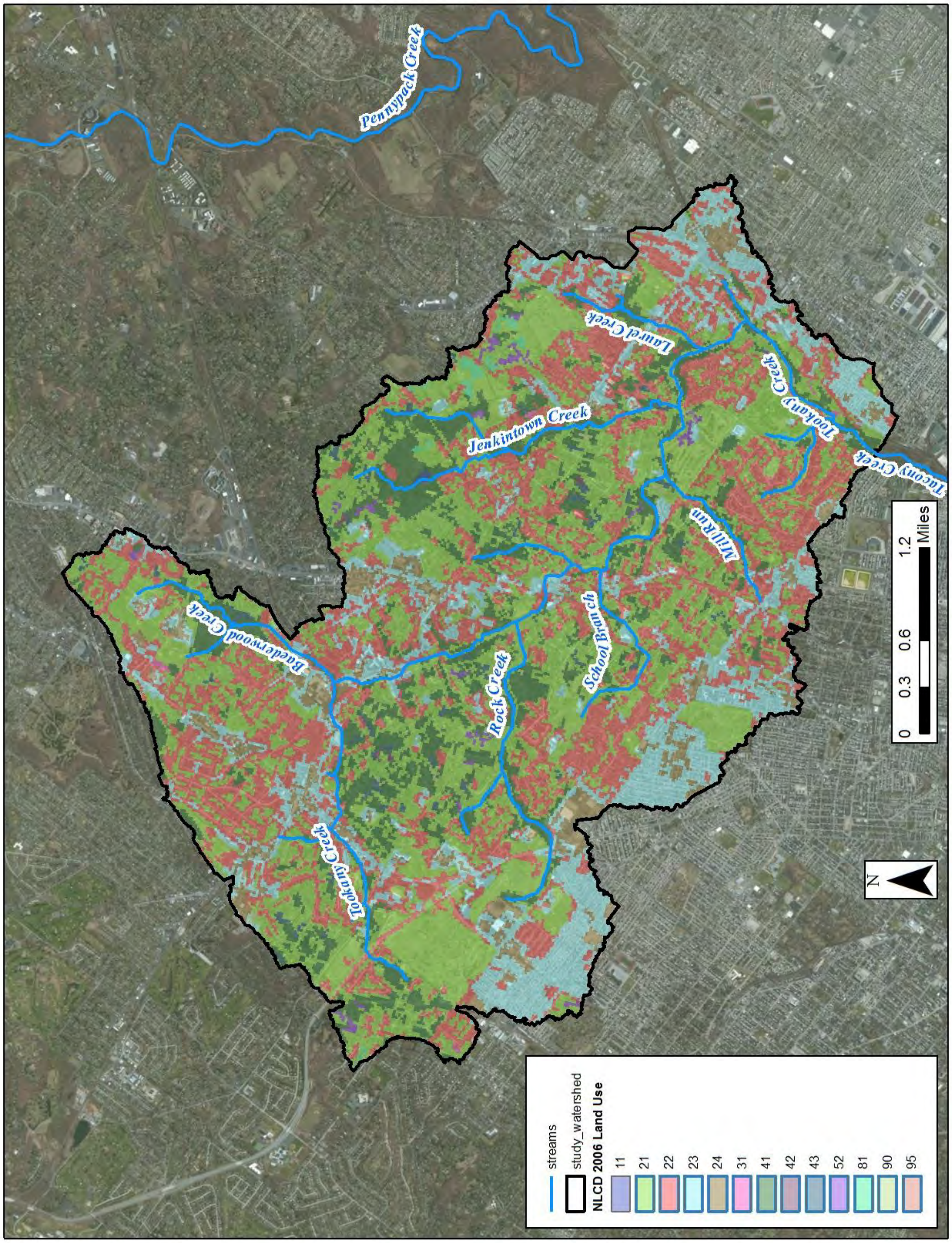

Figure 4.2 – NLCD 2006 Land Uses
ESRI World Imagery

4.2.4. Stream Routing

Stream channel routing was performed using an explicit finite-volume diffusive wave routing routine that is similar to the overland routing routine. However, only one-dimensional flow was assumed to exist within the channel routing portions.

Individual stream channels were identified using tools within ArcGIS and HEC-GeoHMS. The locations of the resulting streams were verified using DCNR PAMAP 2008 orthophotographs, USGS topographic quadrangles, and site visits. Channel sections were subdivided at stream confluences, significant changes in slope, and at large in-stream structures. Ephemeral channels were added as necessary to avoid implausible ponding within low lying areas. This resulted in the creation of 62 channel segments for a total length of approximately 23 stream miles. These streams were linked with the overland grid within WMS.

While the GSSHA code includes an adaptive time step to avoid violating Courant stability criteria, vertices along the channel segments were distributed to appropriate lengths to allow for larger overall time steps. Stream thalwegs were incised and smoothed to remove small-scale depressions and errors within the GSSHA grid.

Most stream segments were assigned a representative cross sectional shape based upon the PAMAP LIDAR and extracted using tools within ArcGIS and HEC-GeoRAS. However, trapezoidal shapes were used to represent the shape of some channel segments. For all stream segments, overland backwater effects were simulated for flow entering the channel during elevated streamflow conditions. Uniform manning's roughness factors for each channel segment were assigned using orthophotographs and site visits.[19] The resultant channels within the GSSHA model are detailed in Table 4.3.

[19] (Chow, 1959)

Table 4.3 – Channel Segments within the GSSHA Model

Stream ID	Channel Shape	Length (m)	Manning's "n"	Bottom Width (m)	Side Slope (H:V)	Max Conveyance Depth (m)
1	Natural	326	0.05	-	-	12.42
2	Trapezoidal	362	0.05	4	2	10
3	Natural	464	0.05	-	-	12.42
4	Natural	881	0.05	-	-	6.33
5	Natural	345	0.05	-	-	5.91
6	Natural	775	0.05	-	-	8.96
7	Natural	482	0.05	-	-	3.85
8	Natural	406	0.05	-	-	9.76
9	Trapezoidal	394	0.05	5	2	10
10	Natural	236	0.05	-	-	9.76
11	Natural	855	0.05	-	-	8.03
12	Natural	696	0.035	-	-	11.58
14	Natural	1131	0.035	-	-	15.77
15	Natural	1428	0.035	-	-	17.43
17	Trapezoidal	243	0.02	7	0.5	10
18	Trapezoidal	463	0.035	4	2	10
19	Natural	670	0.05	-	-	4.83
20	Natural	869	0.035	-	-	11.52
22	Natural	531	0.035	-	-	8.62
23	Natural	227	0.035	-	-	5.28
24	Natural	901	0.02	-	-	9.82
25	Natural	332	0.035	-	-	5.79
27	Natural	412	0.035	-	-	5.79
28	Trapezoidal	105	0.02	5	0.5	10
29	Natural	730	0.035	-	-	5.72
31	Natural	259	0.035	-	-	5.72
32	Natural	1108	0.035	-	-	10.08
33	Natural	1197	0.035	-	-	13.13
34	Natural	689	0.035	-	-	13.25
36	Natural	567	0.035	-	-	7.97
37	Natural	1681	0.035	-	-	4.59
39	Natural	396	0.035	-	-	16.68
40	Natural	929	0.05	-	-	10.96
41	Natural	1787	0.035	-	-	12.67
43	Trapezoidal	136	0.02	5	0.5	10
44	Natural	325	0.035	-	-	9.55
45	Natural	759	0.05	-	-	5.91
46	Natural	994	0.05	-	-	10.92

48	Trapezoidal	379	0.05	5	0.5	10
49	Natural	1597	0.05	-	-	17.76
50	Natural	1716	0.05	-	-	10.53
51	Natural	790	0.05	-	-	11.7
52	Natural	820	0.05	-	-	17.22
53	Trapezoidal	209	0.05	3	1.5	10
54	Natural	675	0.035	-	-	6.73
55	Natural	964	0.05	-	-	12.08
56	Natural	553	0.05	-	-	11.63
57	Natural	510	0.05	-	-	9.99
58	Natural	1437	0.035	-	-	14.8
59	Natural	534	0.05	-	-	9.83
60	Natural	1012	0.05	-	-	6.44
61	Natural	246	0.05	-	-	8.89
62	Natural	597	0.035	-	-	19.53

The extensive storm sewer system within the modeling domain was simplified to include only the largest and most impactful sections. Pipe segments were added and removed using an iterative process to identify those that had large effects on flow rates within the stream system as well as overland depths. Those with negligible effects were removed to ease computational burdens.

Pipe segments were subdivided to have at least four computational vertices. A uniform roughness coefficient for each pipe was assumed to be 0.024, which corresponds to a corrugated metal pipe (CMP) serving as a storm drain.[20] All pipes were assumed to be circular in shape even though some pipes were actually rectangular in shape. Appropriate sizes were determined to minimize changes in flow area, wetted perimeter, and hydraulic radius (and therefore conveyance) using the previously mentioned PWD database. Invert elevations were assigned using maps and data supplied by Abington, Cheltenham, and Jenkintown Municipalities as well as PWD. Drain spacing, which recreates storm sewer inlets, was uniformly set at 100 meter intervals. GSSHA uses a combination of weir and orifice flow equations to determine inlet capacity under low and high flow conditions, respectively. Outlet capacities are determined each time step based upon downstream conditions. Inlet and outlet dimensions were assigned using the municipality and PWD data as well as orthophotographs and site visits. This resulted in the creation of 35 segments totaling to a length of 12.3 miles. Pipe details are shown in Table 4.4.

[20] (Hydrologic Engineering Center, 2010)

Table 4.4 – Pipe Segments within the GSSHA Model

Pipe ID	Pipe Shape	Length (m)	Manning's "n"	Diameter (m)
1	Circular	326	0.024	0.45
2	Circular	362	0.024	1.22
3	Circular	464	0.024	0.91
4	Circular	881	0.024	0.91
5	Circular	345	0.024	0.91
6	Circular	775	0.024	1.01
7	Circular	482	0.024	1.01
8	Circular	406	0.024	1.01
9	Circular	394	0.024	1.01
10	Circular	236	0.024	1.07
11	Circular	855	0.024	1.07
12	Circular	696	0.024	1.94
13	Circular	115	0.024	1.07
14	Circular	1131	0.024	2.71
15	Circular	1428	0.024	1.21
16	Circular	556	0.024	2.42
17	Circular	243	0.024	2.42
18	Circular	463	0.024	0.61
19	Circular	670	0.024	1.37
20	Circular	869	0.024	1.68
21	Circular	770	0.024	0.91
22	Circular	531	0.024	0.91
23	Circular	227	0.024	1.66
24	Circular	901	0.024	1.66
25	Circular	332	0.024	1.07
26	Circular	110	0.024	2.59
27	Circular	412	0.024	2.59
28	Circular	105	0.024	0.91
29	Circular	730	0.024	2.59
30	Circular	627	0.024	2.11
31	Circular	259	0.024	2.13
32	Circular	1108	0.024	0.91
33	Circular	1197	0.024	1.06
34	Circular	689	0.024	1.83
35	Circular	171	0.024	1.83

4.2.5. Structures

Culverts were added to permit the analysis of potential with project conditions (i.e. removal) as well as replicating major flow restrictions / alterations. These culvert structures also included bridges since they are analyzed in a similar fashion within the GSSHA model. No embankments were added for roadways (i.e. no overtopping flow; all flow must pass through the structure). At locations where significant flow could conceivably overtop a roadway, the use of a culvert was deemed inappropriate within the hydrologic model. These structures were better analyzed within the hydraulic model detailed in a separate appendix. A total of nine culverts/bridges were included within the GSSHA model and simulated using the culvert flow routines within GSSHA.

Even though some culverts were actually circular in shape, all modeled culverts were assumed to be rectangular due to simplifications within the GSSHA code. Appropriate sizes were determined to minimize changes in flow area, wetted perimeter, and hydraulic radius (and therefore conveyance) using the previously mentioned PWD database. Approximate lengths were determined using orthophotographs and site visits. Appropriate roughness coefficients (Manning's "n") were assigned based upon pipe materials determined through the PWD database and site visits.[21] Culverts within the GSSHA model are shown in Table 4.5.

Table 4.5 – Culverts within the GSSHA Model

Location	Stream	Shape	Width (m)	Height (m)	Length (m)	Loss Coefficients		Material	Manning's "n"
						Inlet	Exit		
RT 309	Tookany Creek	Box	3.05	2.44	50	0.5	1.0	Concrete	0.02
Easton Rd	Tookany Creek	Box	9.15	1.83	70	0.5	1.0	Concrete	0.02
SEPTA 11.22	Tookany Creek	Box	4.27	4.37	10	0.5	1.0	Masonry	0.025
Wannamaker Rd	Baederwood Creek	Box	3.67	3.67	104	0.5	1.0	Corrugated Metal	0.025
SEPTA 10.12	Tookany Creek	Box	3.66	4.27	10	0.5	1.0	Masonry	0.025
Limekiln Pike / Ogontz Ave	Rock Creek	Box	1.08	1.08	178	0.5	1.0	Corrugated Metal	0.025
Widener Rd	Rock Creek	Box	2.44	3.05	230	0.5	1.0	Concrete	0.02
Shoemaker Rd	Brookside Creek	Box	1.22	3.66	136	0.5	1.0	Masonry	0.025
Elkins Ave	School Branch	Box	5	2.5	270	0.5	1.0	Corrugated Metal	0.025

The Brookdale Levee was also added as an embankment within the modeling domain. Crest elevations were assigned using the PAMAP DEMs. The inclusion of this structure prevented the movement of water from Tookany Creek to the landward side (unless WSELs were greater than the levee crest) as well as from the landward side to Tookany Creek (unless interior WSELs were greater than the levee crest).

[21] (Hydrologic Engineering Center, 2010)

Concerns were raised during the September 2012 site visits and throughout the course of this study that the Brookdale Levee and associated pumping station arrangement does not mitigate interior drainage appropriately. To investigate these claims, the pumping station was not included within the GSSHA model.

The layout of all stream segments, culverts, and cross section locations used within the GSSHA stream routing routine are shown in Figure 4.3.

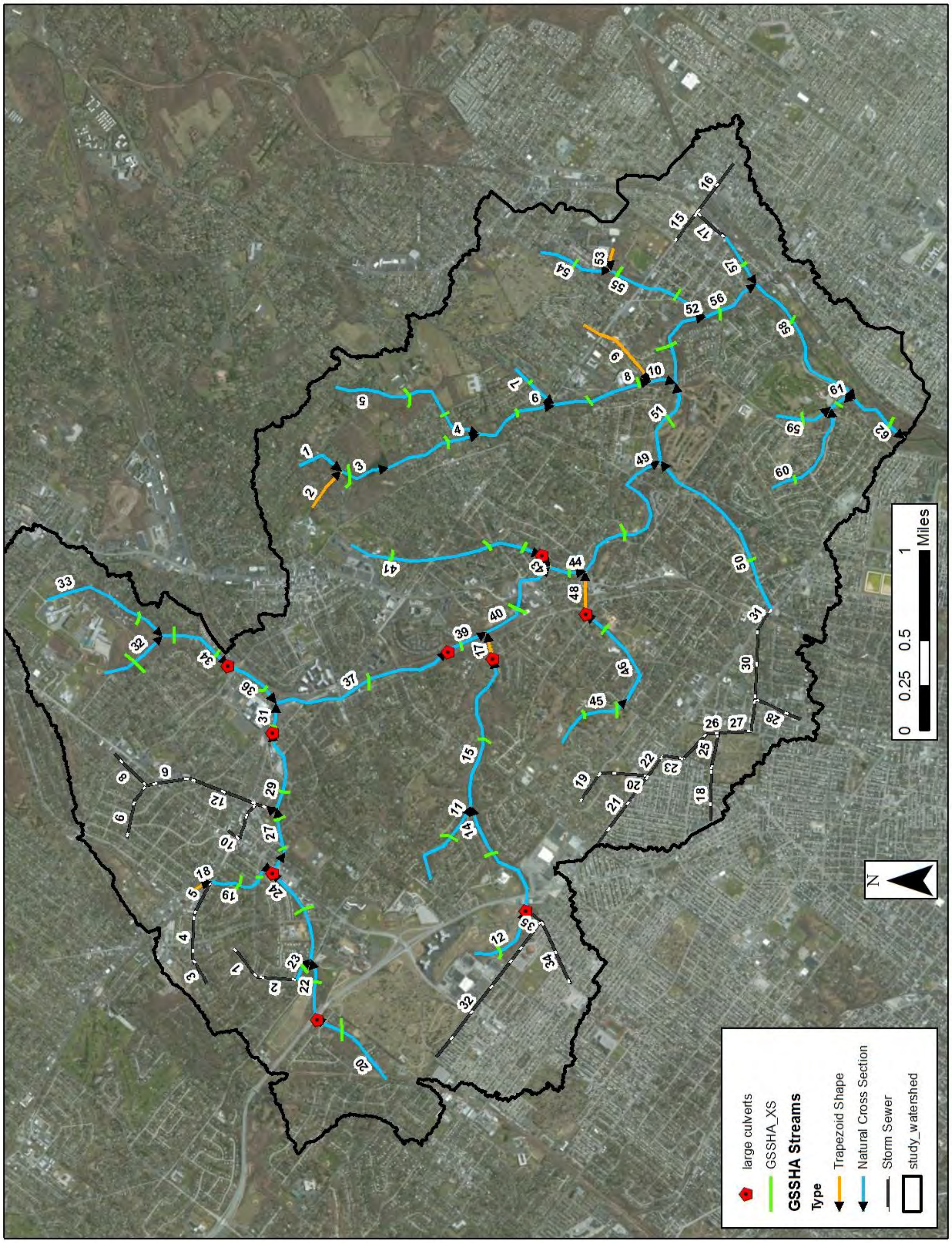

Figure 4.3 – GSSHA Stream Segments, Culverts, and Cross Sections
ESRI World Imagery

4.3. Model Calibration

In order to accurately estimate volumetric flow rates, volumes, and hydrograph timing throughout the Tookany Creek watershed, initial model processes, parameters, and inputs were "ground truthed" through a calibration process. This involved adjusting model parameters to minimize the differences between computed and observed hydrograph shape, peak flow rate, and discharge volume at the USGS streamflow gaging station above Adams Avenue (01467086) for multiple historical storm events.

Since the USGS stream gage was located at the downstream outlet of the modeling domain, additional calibration points were required. This data, which amounted to high water marks, was sourced from eyewitness accounts, pictures, and flooding artifacts observed during the September 2012 site visits in addition to questionnaires distributed to the various neighborhoods throughout Cheltenham Township. Some data was useful when calibrating the GSSHA model. However, other data was more appropriate for use when calibrating the hydraulic model, which is detailed in a separate appendix.

4.3.1. Hurricane Irene and Tropical Storm Lee

Damaging runoff and streamflow rates during August and September 2011 were the result of rainfall associated with Hurricane Irene and Tropical Storm Lee. The first heavy period of rainfall occurred overnight on August 27 – 28, 2011 as Hurricane Irene moved through the northeastern US. Twenty-four hour rainfall accumulations in excess of 7 inches were recorded at the Brookdale Avenue pumping station. This rainfall resulted in peak streamflow rates exceeding previous records by approximately 1500 ft^3/s at the Adams Avenue gage.

A little over one week later, the remnants of Tropical Storm (TS) Lee moved through the northeastern US resulting in even more disastrous flooding within the Delaware River watershed. Cheltenham Township was again hard hit receiving between 9 and 12 inches of precipitation from September 6 – 8, 2011. Similar to Hurricane Irene, the most intense periods of rainfall fell overnight in the early hours of September 8. Peak 2-, 3-, and 6-hour precipitation accumulations were particularly high exceeding 1% annual chance exceedance accumulations (100-yr recurrence interval).[22] This extreme rainfall resulted in peak streamflow rates exceeding the record-setting discharges recorded during Hurricane Irene by approximately 150 ft^3/s.

To ensure that the GSSHA model could adequately predict runoff and streamflow rates during extreme rainfall, these two events were chosen for model calibration. Historical precipitation data used to simulate these events was originally sourced from the National Weather Service (NWS) as Next-Generation Radar (NEXRAD) Multisensor Precipitation Estimator (MPE) coverages. This data is a mosaic of NEXRAD and observed gage point precipitation, recorded each hour, and distributed in approximately 4 square kilometer (km^2) grids for each NWS River Forecast Center. The raw gridded data was then projected and interpolated to an appropriate grid overlaying the model domain using tools available through HEC.

While the MPE data was found to be adequate when simulating the Hurricane Irene event, this precipitation data source was found to underpredict precipitation accumulations during the TS Lee event when compared to the precipitation gages within Cheltenham Township and Philadelphia County. Since the Cheltenham Township rainfall records do not have appropriate temporal distributions, nearby PWD gages were used to create "synthetic" hyetographs that were representative of the precipitation accumulations recorded at the Cheltenham Township gages. To spatially distribute these point precipitation hyetographs throughout the modeling domain, Thiessen polygons were used.

The GSSHA model was calibrated to each event separately. This was done since groundwater and/or evapotranspiration routines were not included within the simulations. Consequently, infiltrated moisture

[22] (Bonnin, et al., 2006)

was not removed from the soil due to natural processes. During short simulation time frames, these processes are not usually necessary to adequately simulate runoff and streamflow. However, during a two week simulation necessary to model both Hurricane Irene and Tropical Storm Lee, these processes can become essential.

The Hurricane Irene simulation was executed from 8/25/2011 000 – 8/30/2011 2400 while the Tropical Storm Lee simulation was executed from 9/5/2011 000 – 9/9/2011 2400. As was previously mentioned, both volumetric streamflow records and high water marks were used to calibrate the GSSHA model. The locations of data used to calibrate the GSSHA model for the Hurricane Irene and TS Lee events are shown in Figure 4.4.

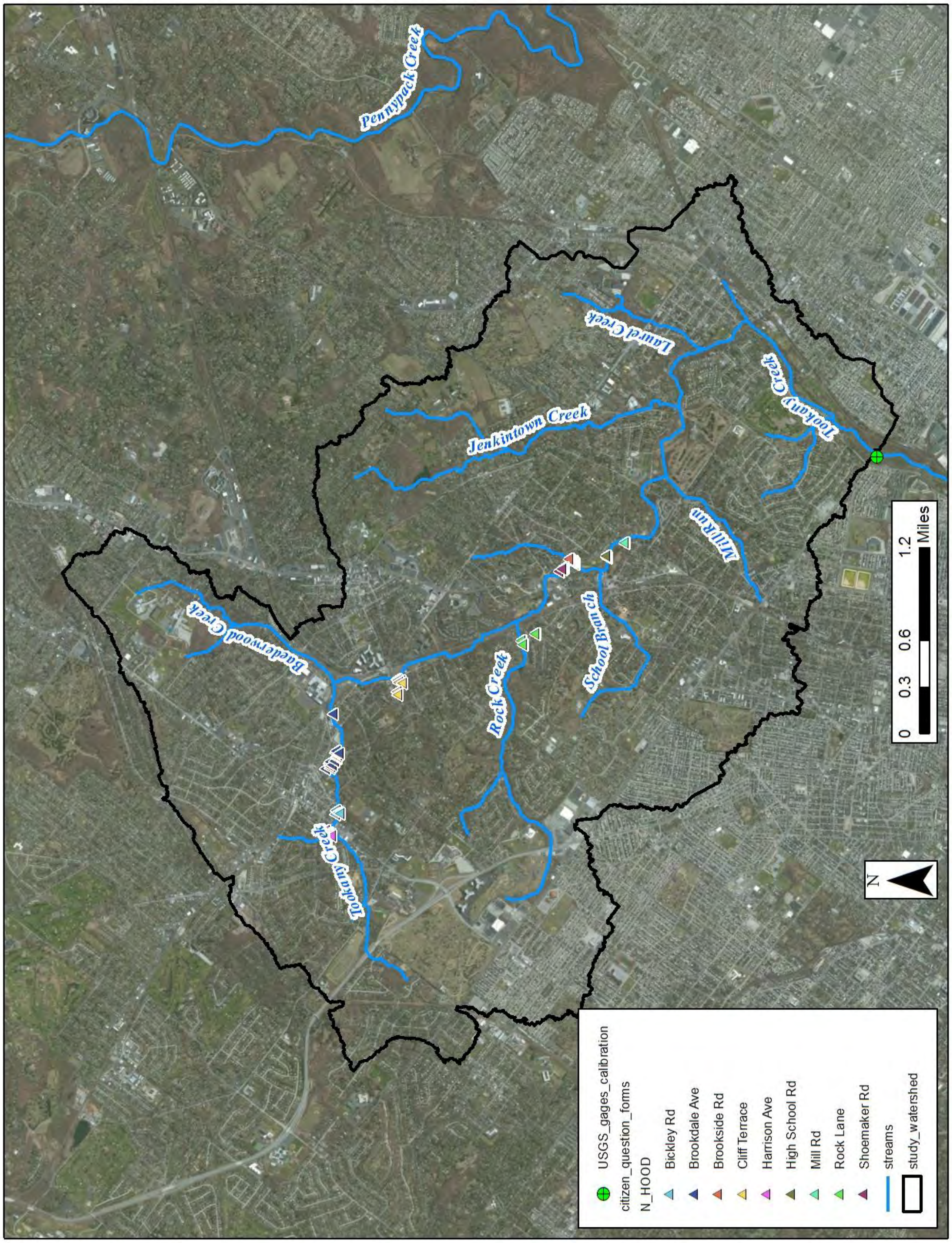

Figure 4.4 – GSSHA Model Calibration Data
ESRI World Imagery

4.3.2. Calibration Results

The calibration results at the Adams Avenue gage for the Hurricane Irene and Tropical Storm Lee events are tabulated in Tables 4.6 and 4.7 while the GSSHA-computed hydrographs at the Adams Avenue gage for both events are shown in Figures 4.5 and 4.6. Peak flow rates and time of peak flows were matched very well to observed data while flow volumes were underpredicted. This is to be expected when no baseflow / groundwater processes are modeled. However, hydrograph shape during the majority of both events adequately matches observed records.

Table 4.6 – GSSHA Hurricane Irene Event Calibration

	Peak Flow Rate (ft^3/s)	Time of Peak	Volume (in)*
Observed	5830	8/28/11 0:45	3.91
GSSHA	5786	8/28/11 0:25	3.15
Difference (%)	-0.75		-19.28

*based upon a contributing drainage area of 15.6 mi^2

Table 4.7 – GSSHA Tropical Storm Lee Event Calibration

	Peak Flow Rate (ft^3/s)	Time of Peak	Volume (in)*
Observed	5990	9/8/11 5:45	4.35
GSSHA	6087	9/8/11 5:20	2.48
Difference (%)	1.63		-42.95

*based upon a contributing drainage area of 15.6 mi^2

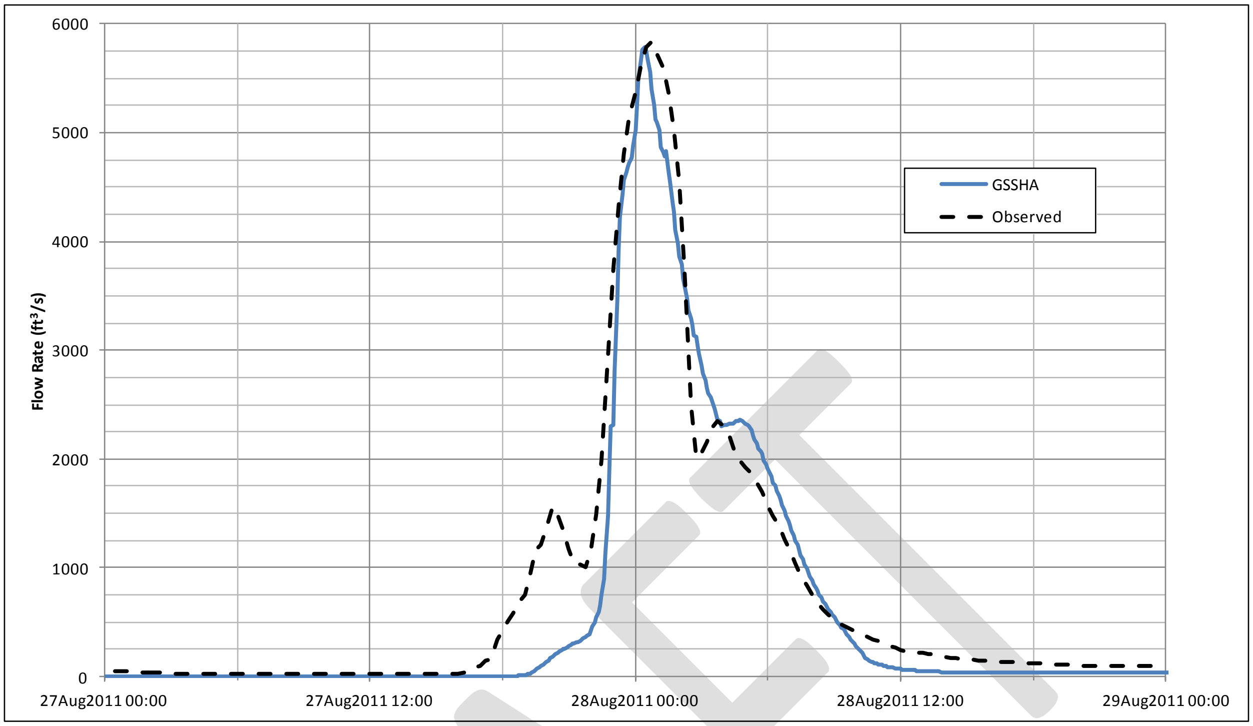

Figure 4.5 – GSSHA Hurricane Irene Event Calibration

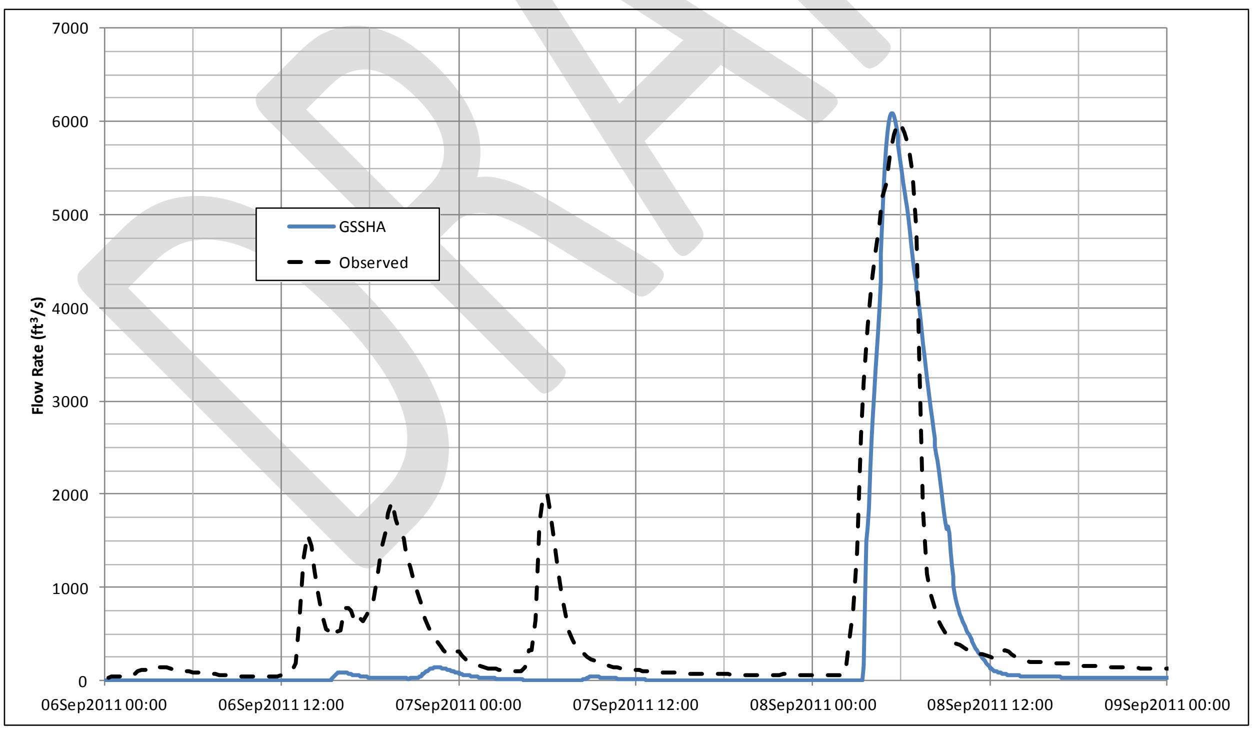

Figure 4.6 – GSSHA Tropical Storm Lee Event Calibration

GSSHA-computed preferential flow paths, areas of ponding, and other flow phenomena were compared against eyewitness accounts and citizen questionnaire data for reasonableness. This included neighborhoods like Harrison Avenue, Bickley Road, and Brookdale Avenue.

The Harrison Avenue neighborhood lies in a slight depression at the bottom of a slope that delivers runoff during storm events that cannot drain into Tookany Creek due to high WSELs. This causes ponding along Harrison Avenue, especially on the southeastern side of the street.

The Bickley Road neighborhood is situated within a bowl-shaped depression similar to Harrison Avenue. During storm events, runoff cannot drain into Tookany Creek and consequently ponds on the street and in backyards.

At the Brookdale Avenue neighborhood, eyewitness accounts detail significant water coming from Abington Township (which is uphill), flowing down Keswick Avenue, under the SEPTA tracks near the Glenside Station, passing into Cheltenham Township, and into the Brookdale Avenue neighborhood. Water then ponds amongst the homes and inflicts serious residential damage, primarily to basement contents and foundations. This damage occurs behind the Brookdale Levee before any interior drainage can be evacuated through the associated pumping station and before any water overtops the levee. These effects are exacerbated by debris and trash clogging the trash racks at the pumping station (trash pickup was scheduled for the morning of 9/8/2011).

Multiple eyewitness accounts assert that the Brookdale Levee was overtopped during TS Lee. However, little to no damage was evident on the levee or surrounding areas. This implies that water overtopping the levee did not have sufficient velocity to cause erosion. This was most likely caused by elevated interior drainage which had ponded within the "leveed" side up to or near the levee crest. As the Tookany Creek overtopped the levee, water did not sufficiently "fall" over the levee, leaving no damage.

These eyewitness accounts matched the results obtained within the GSSHA simulation. GSSHA-computed maximum overland depths within the Harrison Avenue, Bickley Road, and Brookdale Avenue neighborhoods are shown in Figure 4.7.

GSSHA-computed maximum overland depths are compared against data gleaned from citizen questionnaires in Table 4.8. The GSSHA-computed maximum depths shown in Table 5.8 are averages within the close vicinity of the address in question, not depths at a single spot which may reflect localized topographic "pits".

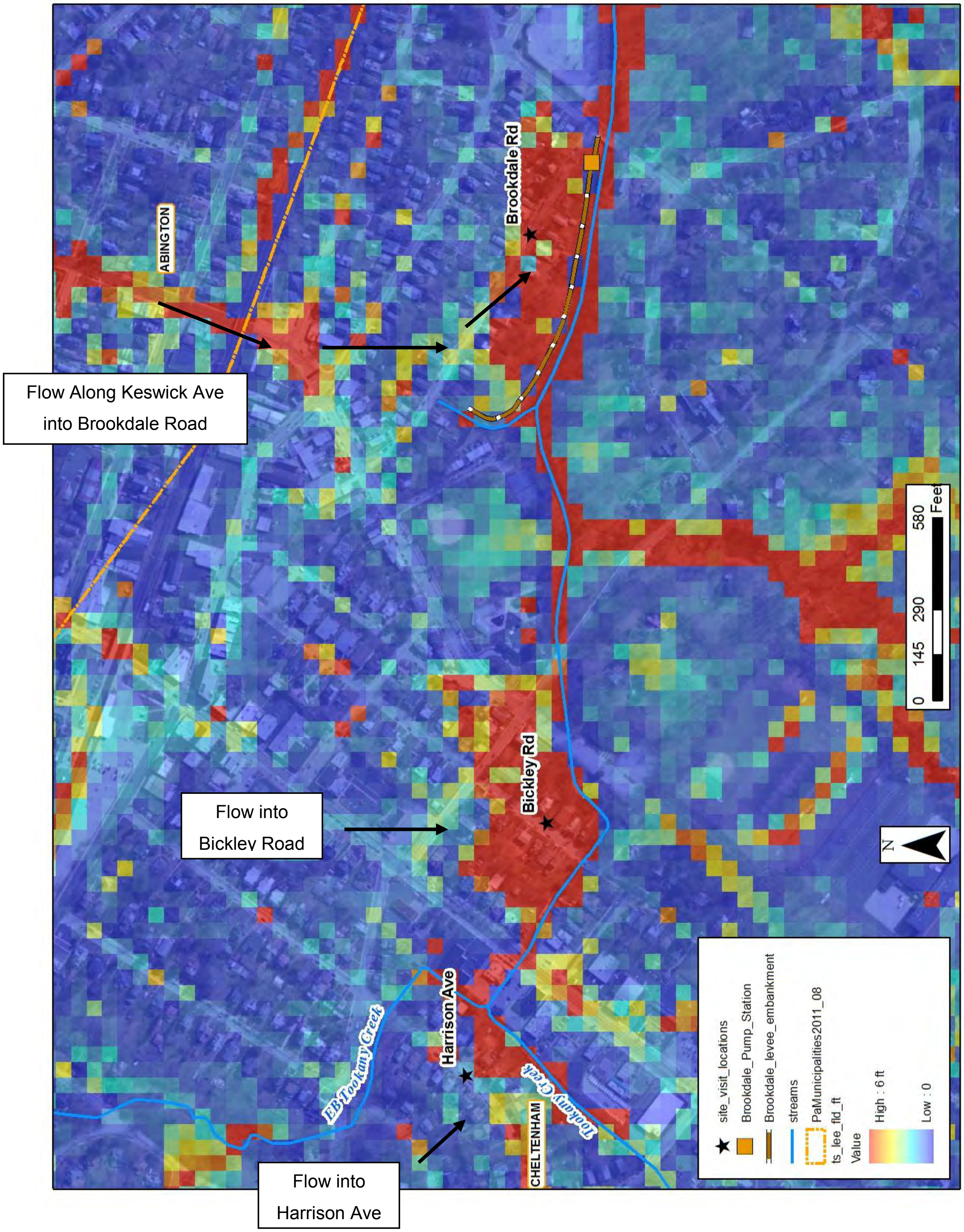

Figure 4.7 – Sample GSSHA Model Results
ESRI World Imagery

Table 4.8 – GSSHA Tropical Storm Lee Event Questionnaire Data Calibration

Citizen Questionnaires				GSSHA-computed Max Depth (ft)**
ID	Address	Neighborhood	Reported Max Depth (ft)*	
1	217 Bickley	Bickley Rd	3	3
2	220 Bickley	Bickley Rd	3	2
3	223 Bickley	Bickley Rd	4.5	3.5
4	229 Bickley	Bickley Rd	1.5	2
5	225 Brookdale	Brookdale Road	0.67	0.5
6	232 Brookdale	Brookdale Ave	2	4.5
7	236 Brookdale	Brookdale Ave	-	4.5
8	239 Brookdale	Brookdale Ave	3	0.5
9	243 Brookdale	Brookdale Ave	-	0.5
10	244 Brookdale	Brookdale Ave	4	4.5
11	300 Brookdale	Brookdale Ave	3.5	2.5
12	316 Brookdale	Brookdale Ave	2.5	2.25
13	320 Brookdale	Brookdale Ave	4.5	3
14	324 Brookdale	Brookdale Ave	4.5	3.25
15	325 Brookdale	Brookdale Ave	4	2.5
16	327 Brookdale	Brookdale Ave	5	2.5
17	328 Brookdale	Brookdale Ave	4	2.5
18	8104 Brookside	Brookside Rd	-	HEC-RAS
19	8108 Brookside	Brookside Rd	-	HEC-RAS
20	8116 Brookside	Brookside Rd	-	HEC-RAS
21	8120 Brookside	Brookside Rd	-	HEC-RAS
22	101 Cliff Terrace	Cliff Terrace	3	HEC-RAS
23	103 Cliff Terrace	Cliff Terrace	1.5	HEC-RAS
24	106 Cliff Terrace	Cliff Terrace	-	HEC-RAS
25	107 Cliff Terrace	Cliff Terrace	0.5	HEC-RAS
26	108 Cliff Terrace	Cliff Terrace	0	HEC-RAS
27	120 Greenwood Ave	Cliff Terrace	-	1.5
28	146 Greenwood Ave	Cliff Terrace	4	4
29	208 Harrison	Harrison Ave	0	negl.
30	211 Harrison	Harrison Ave	-	negl.
31	214 Harrison	Harrison Ave	-	negl.
32	215 Harrison	Harrison Ave	-	3
33	217 Harrison	Harrison Ave	-	3.25
34	219 Harrison	Harrison Ave	3.5	3.25
35	221 Harrison	Harrison Ave	-	3.25
36	8000 Heather	Rock Lane	4	0.5
37	8027 High School	High School Rd	6.5	HEC-RAS
38	8029 High School	High School Rd	7	HEC-RAS
39	8031 High School	High School Rd	0	HEC-RAS
40	7859 Mill Rd	Mill Rd	1.5	HEC-RAS
41	5 North Ave	Brookdale Ave	6	HEC-RAS
42	542 Shoemaker	Shoemaker Rd	5	HEC-RAS
43	536 Shoemaker	Shoemaker Rd	4.5	HEC-RAS
44	538 Shoemaker	Shoemaker Rd	5	HEC-RAS
45	875 Widener Rd	Rock Lane	3	3
46	846 Widener Rd	Rock Lane	3	3

*some Questionnaires were returned with no maximum depth specified

**some HWMs are better suited to the HEC-RAS calibration
**"negl." implies little to no water ponding at this location within the GSSHA simulation

Following model calibration a final set of model parameters was created. Parameters and processes that were changed to achieve model calibration included overland routing, infiltration, and streamflow routing. However, infiltration and streamflow routing parameters required only slight changes to achieve acceptable results for the Hurricane Irene event while the best calibration was achieved through the use of the initial estimates for TS Lee event. Therefore, the infiltration routine was left unchanged while the altered overland routing parameters were used in the final, accepted set of parameters. The altered set of parameters is shown in Table 4.9.

Table 4.9 – Calibrated Overland Routing Parameters

ID	Description	Roughness	Retention Depth (mm)	Impervious Area (%)
11	Open Water	0.001	0	100
21	Developed, Open Space	0.08	1	10
22	Developed, Low Intensity	0.08	0.75	35
23	Developed, Medium Intensity	0.08	0.625	65
24	Developed, High Intensity	0.08	0.5	90
31	Barren Land (Rock/Sand/Clay)	0.08	0.5	0
41	Deciduous Forest	0.32	3.75	0
42	Evergreen Forest	0.32	3.75	0
43	Mixed Forest	0.32	3.75	0
52	Shrub / Scrub	0.16	2.5	0
81	Pasture / Hay	0.2	2.5	0
90	Woody Wetlands	0.2	2.5	20
95	Emergent Herbaceous Wetlands	0.2	2.5	20

4.4. Model Validation

The final set of model parameters was tested through the model validation process to determine the usability and reasonableness of the models for hydrologic prediction in the absence of parameter changes. This process involves the simulation of an independent rainfall event without any adjustment of the controlling model parameters. Computed peak flow rate timing and hydrograph shape should still closely match observed data, however.

4.4.1. June 2006

The June 2006 event was caused by a single period of moderately intense rainfall on June 28, 2006. This event was extremely damaging throughout the Delaware River and Schuylkill River watersheds. However, rainfall in the Tookany Creek watershed was much less intense than elsewhere throughout the Delaware River watershed. Nevertheless, streamflow and runoff rates were still elevated. Therefore this event was chosen for model validation. NEXRAD MPE data was used as the meteorological driver for this event.

The validation results at the Adams Avenue gage for the June 2006 event is tabulated in Table 4.10. The GSSHA-computed hydrograph at the Adams Avenue gage for this event is shown in Figure 4.8. Peak flow rates and time of peak flows were matched very well to observed data while flow volumes were slightly underpredicted. Similar to the results presented in the model calibration sections, this effect is to be expected when no baseflow / groundwater processes are modeled. However, hydrograph shape during the event matches observed records.

Table 4.10 – GSSHA June 2006 Event Validation

	Peak Flow Rate (ft^3/s)	Time of Peak	Volume (in)*
Observed	2650	6/28/06 6:15	1.05
GSSHA	2648	6/28/06 5:30	0.84
Difference (%)	-0.08		-20.05

*based upon a contributing drainage area of 15.6 mi^2

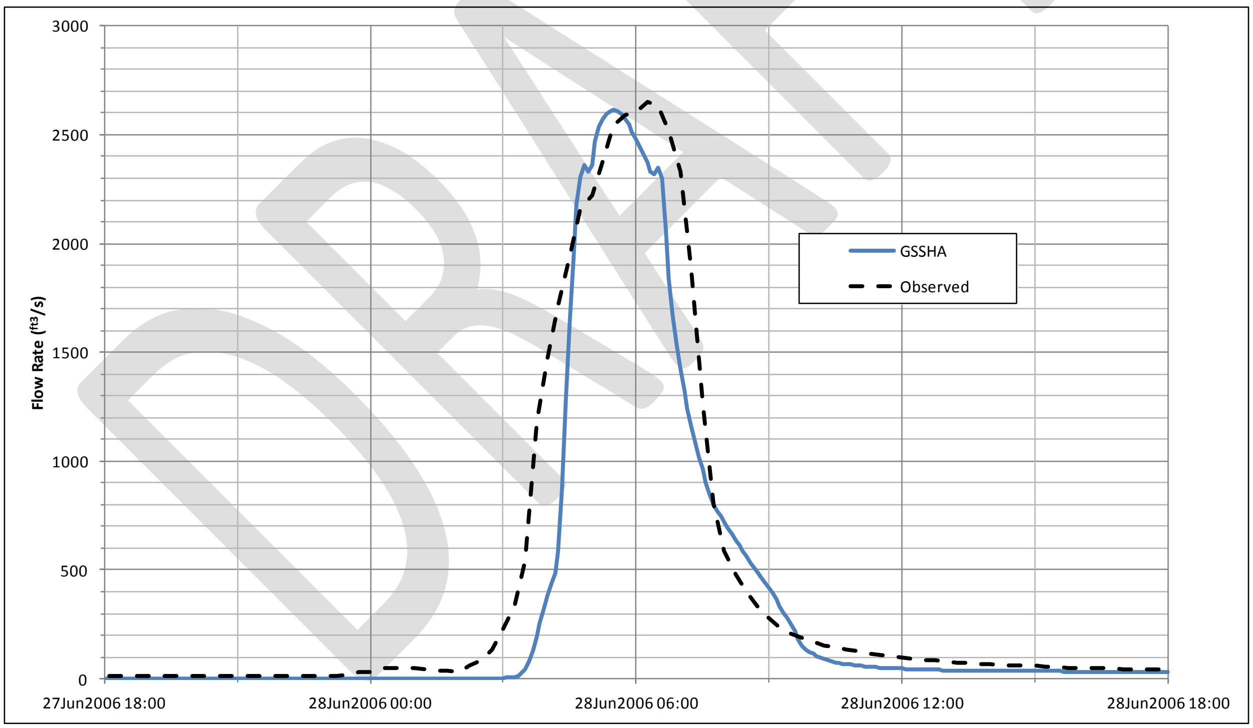

Figure 4.8 – GSSHA June 2006 Event Validation

4.5. Results

The validation results for the June 2006 event demonstrate the ability of the accepted model parameters to predict runoff and streamflow rates. Therefore, this set of parameters was considered adequate for use when assessing various flood risk reduction with project conditions within the Tookany Creek watershed, as described in Section 6.

5. Without Project Conditions

Commonly, frequency discharge rates and water surface elevations (WSELs) are used to gauge the effectiveness of various with project conditions. Multiple methods of determining frequency discharge rates exist, including Bulletin 17B procedures, regional regression equations, and hypothetical event simulation within a hydrologic model.

5.1. Bulletin 17B Analysis

A Bulletin 17B analysis was performed on the USGS streamflow gage above Adams Avenue (01467086) using the HEC Statistical Software Package (HEC-SSP) ver. 2.0.[23] An annual series from July 1966 – Nov. 1985 and Oct. 2005 – Sep 2011 was used. The annual series was assumed to be broken from Nov. 1985 – Oct. 2005. Therefore, the different record segments were analyzed as a continuous record with a period of record equal to the sum of the two parts.[24] This assumption was based on the lack of large-scale physical changes in the watershed between the record segments. This assumption was verified through the comparison of historical aerial photographs of the Tookany Creek watershed dating back 70 years to recent orthophotographs. The annual series for this gage is shown in Table 5.1.

[23] (Hydrologic Engineering Center, 2010)
[24] (Interagency Advisory Committee on Water Data, 1981)

Table 5.1 – Tacony Creek above Adams Avenue (01467086) Annual Series

Ordinate	Date	Observed Peak Flow Rate (ft3/s)
1	7/19/1966	1950
2	8/27/1967	4550
3	6/12/1968	3230
4	7/28/1969	2700
5	8/23/1970	2800
6	8/28/1971	3150
7	6/22/1972	4410
8	2/2/1973	1120
9	8/23/1974	2400
10	7/14/1975	2600
11	5/1/1976	1100
12	8/1/1977	1740
13	8/28/1978	2910
14	5/23/1979	2500
15	9/18/1980	2120
16	10/25/1980	2570
17	7/28/1982	4000
18	9/21/1983	1390
19	7/7/1984	3290
20	9/27/1985	2040
21	11/5/1985	575
22	10/8/2005	4120
23	4/16/2007	2040
24	3/8/2008	2120
25	8/2/2009	3630
26	7/13/2010	4950
27	9/8/2011	5990

A two station comparison / extension was attempted to lengthen the analysis period using nearby streamflow gages on the Frankford, Wissahickon, Pennypack, and Poquessing Creeks. However, correlation between observed peak flow rates throughout the available records was lacking (i.e. peak flow rates at the Adams Ave. gage often occurred on different dates when compared to other gaging stations). This reinforces the previous assumption that short duration summer thunderstorms are the predominant storm type within this area, not regional-scale events that affect large swaths of area. The watershed composition (highly urbanized) can also amplify these effects.

A regional skew of 0.178 and a regional skew mean square error of 0.033 were sourced from a 2009 HEC study of the Delaware River which recommended these values for all gages within the Delaware River watershed between Trenton, NJ and the confluence with Chester Creek.[25] Computed peak flow rates for

[25] (Hydrologic Engineering Center, 2009)

various percent chance exceedances along with 5% and 95% confidence limits at the Adams Avenue gage location are tabulated in Table 5.2. This information is also graphically shown in Figure 5.1.

However, these peak discharges could not be directly applied throughout the area of interest. This was due to the differences in contributing drainage area. Using factors strictly based upon upstream drainage areas at various points of interest was not considered appropriate due to differences in runoff generation, stream characteristics, and common rainfall patterns. Therefore, synthetic precipitation events were used as another means to develop frequency discharge rates throughout the area of interest.

Table 5.2 – Tacony Creek above Adams Avenue (01467086) Bulletin 17B Analysis Results

Percent Chance Exceedance	Return Interval (years)	Computed Peak Flow Rate (ft^3/s)	Confidence Limit Peak Flow Rate (ft^3/s)	
			0.05	0.95
0.2	500	10095	15511	7622
0.5	200	8654	12749	6700
1	100	7635	10877	6030
2	50	6672	9174	5379
4	25	5757	7625	4740
10	10	4603	5784	3899
20	5	3753	4524	3240
50	2	2575	2966	2232
80	1.25	1800	2086	1491
90	1.11	1503	1771	1201
95	1.05	1300	1559	1005
99	1.01	999	1243	723

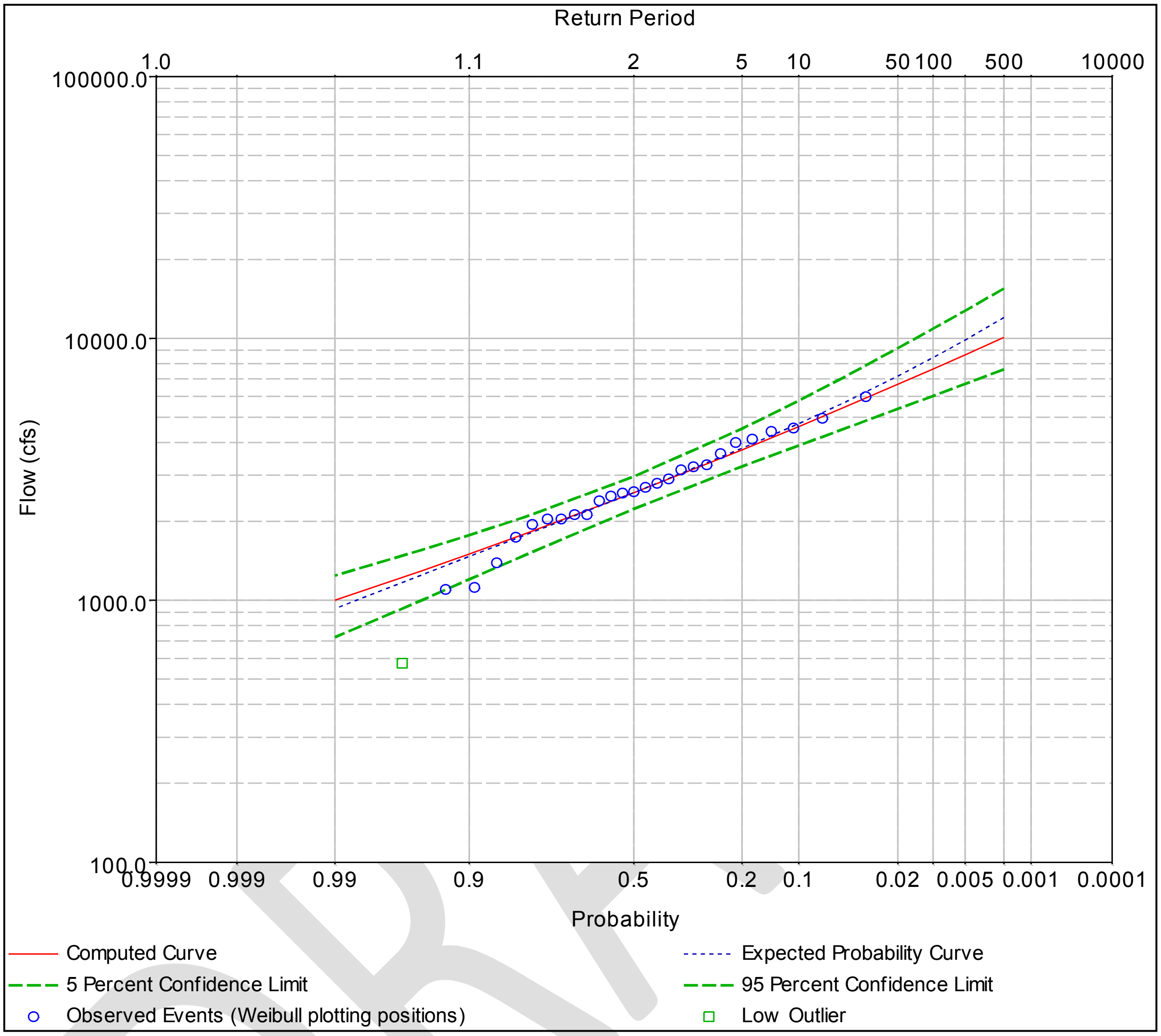

Figure 5.1 – Tacony Creek above Adams Avenue (01467086) Bulletin 17B Analysis Results

5.2. Hypothetical Frequency Rainfall

A cursory examination of the rainfall events that resulted in the peak flow annual series presented in Table 5.1 demonstrated that short duration storms, primarily 1-, 2-, and 3-hr durations, cause the majority of elevated streamflows observed within the area of interest. Furthermore, peak flow rates associated with the Hurricane Irene, Tropical Storm Lee, and June 2006 events were primarily caused by 3-hr periods of intense rainfall nested within longer duration, lighter rainfall. This is due to the predominant weather patterns, location, and land use composition of the area of interest. The location of the area of interest lends itself to summer thunderstorms (and occasional tropical storms) while the intense development of the area of interest results in increased runoff rates as well as short runoff response times. Therefore, synthetic precipitation temporal distributions used within this study for hypothetical frequency events was conceptualized to occur during the summer months over 3-, 6-, 12-, and 24-hr durations.

Partial duration, point-precipitation exceedance – depth – durations for Glenside, PA were acquired from the NWS's Hydrometeorolocial Design Studies Center (HDSC) website.[26] Guidelines presented in NOAA Atlas 14 volume 2, ver. 3 were used to develop each event for input to the hydrologic model.[27]

Normalized, balanced hyetographs for each distribution were then created by centering the largest 1% annual chance exceedance (ACE) 15-minute precipitation depth for each duration and progressively alternating the remaining depths around the largest depth, which approximately matched a 50% second quartile temporal distribution. The normalized temporal distribution for each duration is shown in Figure 5.8. The reported accumulated precipitation depths for each duration and exceedance probability were then multiplied by the normalized hyetographs shown in Figure 5.2 to create 15-minute hyetograph ordinates.

[26] http://www.nws.noaa.gov/ohd/hdsc/
[27] (Bonnin, et al., 2006)

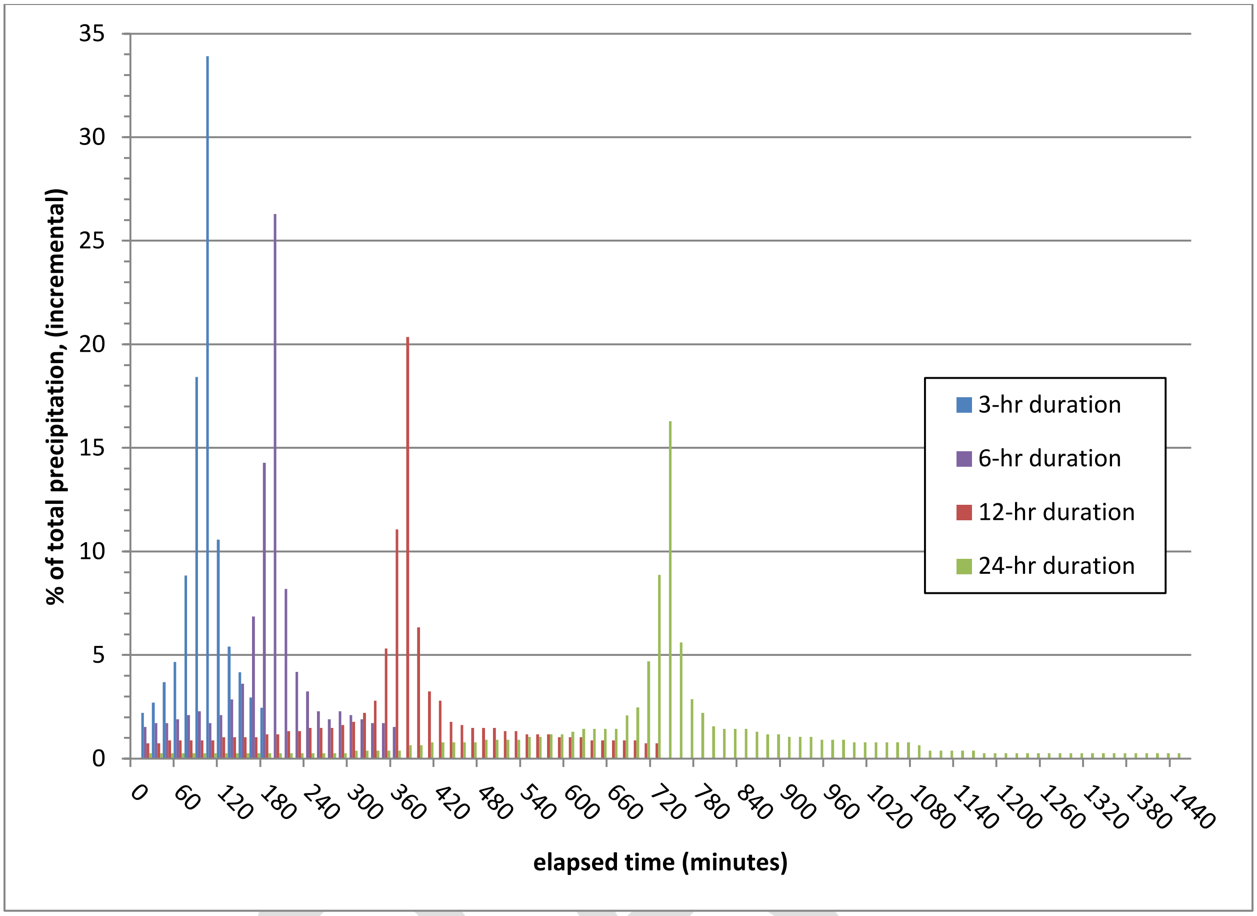

Figure 5.2 – Normalized Hyetographs for 3-, 6-, 12-, and 24-hr Duration Events

A uniform spatial distribution was assumed for each event due to the small size of the modeling domain. Due to similar reasoning, no depth-area reduction factors were used. The finalized precipitation events for each duration and exceedance probability were input to the GSSHA model using the final set of model parameters tested during the validation process.

5.2.1. Results

Generally speaking, peak flow rates and volumes increased as event duration increased and exceedance value decreased throughout the modeling domain. However, the predicted 6-hr duration 10%, 20%, and 50% peak flow rates were greater than the corresponding 12-hr and 24-hr duration events at the Adams Ave. gage location. While this is highly unlikely, it is not impossible. Conversely, runoff volume must always increase as event duration increases and exceedance value decreases, which was demonstrated by the GSSHA model. It should be mentioned that the Bulletin 17B analysis used peak flow rates that were sampled from a multitude of rainfall events with different durations, temporal distributions, and spatial distributions. Therefore, these events are not associated with any duration or flow volume.

The peak discharge rates computed using the hydrologic model are compared to the 17B results at the Adams Avenue in Table 5.3. For most of the hypothetical events, the computed peak discharge rates were within the 5 and 95% confidence limits predicted by the statistical 17B Analysis results. However, the 10, 20, and 50% ACE event peak flow rates computed using the GSSHA model are outside of these confidence limits. This demonstrates the lack of direct connection between frequency precipitation and frequency discharge (i.e. a statistical 17B Analysis might not predict the same flow rates as a hydrologic model using frequency precipitation).

Table 5.3 – Bulletin 17B vs. GSSHA Results at the Adams Ave. Gage

Annual Chance Exceedance	Return Interval (years)	17B Peak Flow Rate (ft^3/s)	Confidence Limits (ft^3/s)		GSSHA-computed Peak Flow Rates (ft^3/s)				GSSHA-computed Runoff Volumes (in)			
			0.05	0.95	3-hr	6-hr	12-hr	24-hr	3-hr	6-hr	12-hr	24-hr
0.2	500	10095	15511	7622	9513	11338	12997	13024	2.92	3.92	5.19	5.59
0.5	200	8654	12749	6700	7767	9014	9949	10032	2.44	3.16	4.02	4.35
1	100	7635	10877	6030	6669	7434	8016	8120	2.09	2.65	3.25	3.50
2	50	6672	9174	5379	5501	6225	6510	6599	1.76	2.18	2.56	2.78
4	25	5757	7625	4740	4316	4722	4851	4905	1.44	1.73	1.98	2.12
10	10	4603	5784	3899	3459	3623	3608	3620	1.06	1.23	1.35	1.45
20	5	3753	4524	3240	2672	2910	2864	2884	0.77	0.90	0.97	1.05
50	2	2575	2966	2232	1743	1924	1872	1876	0.45	0.53	0.58	0.61
99	1	999	1243	723	902	1098	1146	1065	0.24	0.31	0.36	0.37

Flow change locations within the steady state hydraulic (HEC-RAS) model were chosen to correspond with stream confluences, large changes in drainage area, calibration data locations, and locations of possible with project conditions. At these locations, peak flow rates were output from the GSSHA model for export and use within the HEC-RAS model. These locations are tabulated in Table 5.4 and visually identified in Figure 5.3.

Table 5.4 – GSSHA – HEC-RAS Model Linkage Points

ID	Location
1	Adams Ave. Gage
1A	U/S of Trib near Melrose Creek
2	U/S of Central Ave
3	Just U/S of Jenkintown Creek
4	Just U/S of Mill Creek
4A	Just U/S of Trib near High School
5	Just U/S of Trib along Brookside Ave
6	Just U/S of Rock Creek
6A	Just U/S of Green St. Culvert
6B	Just D/S of Baederwood Creek
7	Just D/S RR (just U/S of Baederwood Creek)
8	Just U/S of Keswick Culvert
9	U/S of Easton Road
10	Just U/S of Springhouse Lane
10A	Just D/S of Rt 152 (just U/S of trib on ROB)
11	Just U/S of Rt 73
12	Just D/S of Rt 309
13	Harrison Ave - EB Tookany crossing
14	Waverly Rd - EB Tookany crossing
15	Lynwood Ave - EB Tookany crossing
16	SEPTA RR Embankment - EB Tookany
17	Most upstream point along EB Tookany
18	Rock Creek - Washington Lane crossing
19	U/S end Off-Channel pond along Rock Creek
20	Rock Creek culvert inlet

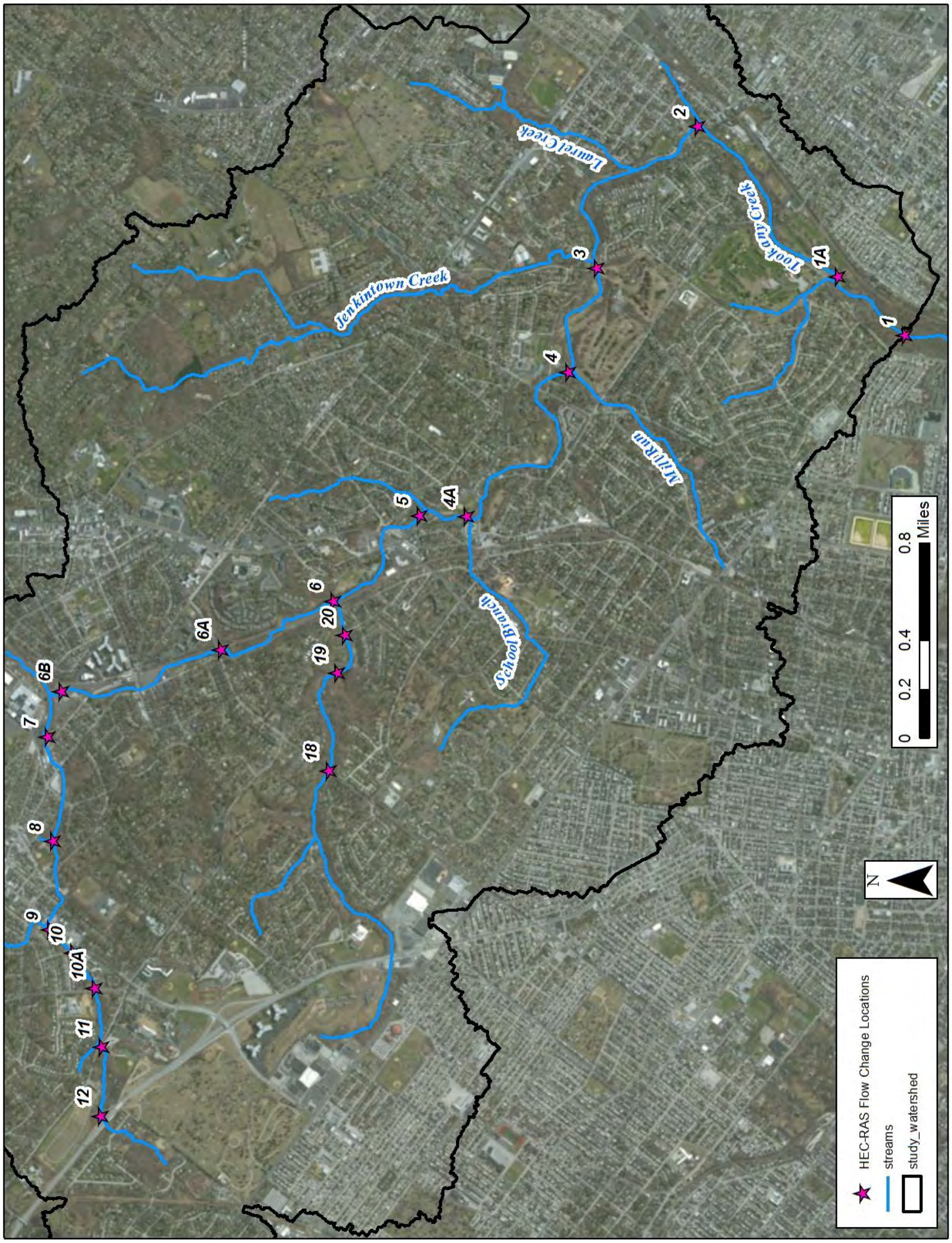

Figure 5.3 – GSSHA – HEC-RAS Model Linkage Points
ESRI World Imagery

Using these points and the 1% ACE Event, 24-hr duration results, one can track the runoff hydrographs as the hypothetical event floodwave moves downstream past multiple locations, as shown in Figure 5.4. Starting just downstream of the RT 309 crossing, the hydrograph is somewhat broad. The hydrograph shape is significantly more peaked near Springhouse Lane due to the influence of the highly incised channel and local runoff. The hydrograph is then attenuated and translated into a more broad shape as it passes through the two bridges / culverts at Easton Road and Bickley Road. After the hydrograph moves through Glenside, runoff emanating from Baederwood Creek enters Tookany Creek and combines to enlarge the hydrograph peak. This hydrograph shape is then held relatively constant, only increasing in peak flow with additional tributary and local runoff, as it moves downstream to the Adams Ave gage location near the Philadelphia / Cheltenham Township boundary.

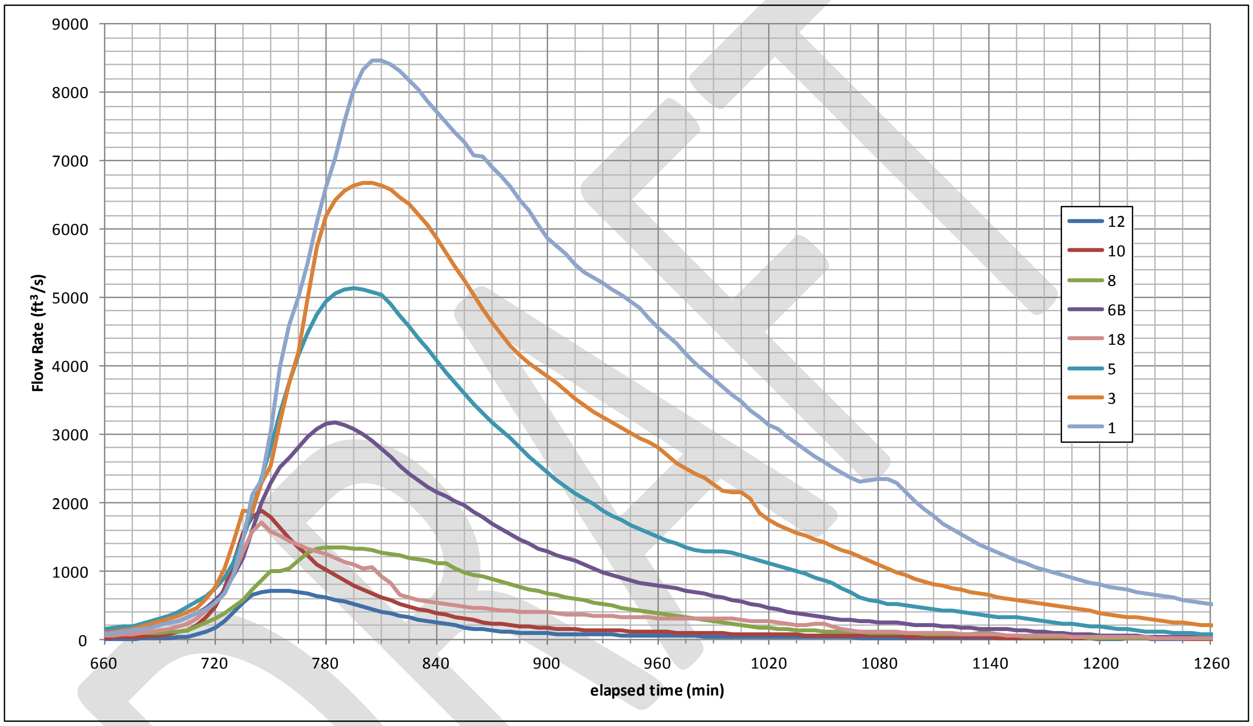

Figure 5.4 – Existing Condition GSSHA Hydrographs
1% ACE Event, 24-hr Duration

6. With Project Conditions

The previously described final set of model parameters developed during the model calibration and tested during the validation process was used to assess the effectiveness of various flood risk reduction with project conditions throughout the Tookany Creek watershed. Multiple with project condition types, groupings, and arrangements were investigated. Flood risk reduction measures that were investigated included low impact development options, existing infrastructure modifications, localized flood walls, and storage areas. The conceptual development and hydrologic results of these with project conditions are detailed in the following sections.

6.1. Low Impact Development

Generally speaking, low impact development (LID) concepts attempt to infiltrate and/or evapotranspirate as much water as possible at the source of runoff instead of treating or storing water elsewhere. Common LID measures (commonly referred to as Stormwater Control Measures, SCMs) include stormwater wetlands, rain barrels, rain gardens, infiltration trenches, and porous pavements. All of these SCMs act to reduce runoff volumes, promote groundwater recharge, and reduce pollutant loadings from small – medium (frequently occurring) rainfall events due to their relative sizes. Two specific SCMs were investigated using the GSSHA model.

6.1.1. Rain Barrels

Rain barrels (or cisterns) offer a convenient way to store runoff from roofs or other impervious areas. Conceptually, rain barrels are connected to a building's downspouts and intercept the "first flush" of pollutants that is commonly associated with the first 0.1 – 0.25 inches of rainfall during a common precipitation event (approximately 95% of all rainfall events that occur in a given year within the Philadelphia area).

To model this SCM, various assumptions were required. First, each building within the modeling domain was assumed to have approximately 2500 ft^2 of roof space. Second, five 50 gallon capacity (approximately 6.68 ft^3) rain barrels were hypothesized to be linked to each building. As such, each 50 gallon rain barrel can hold 0.032 inches of runoff from a 2500 ft^2 building if it is completely empty at the initiation of runoff. Therefore, with five barrels linked to 2500 ft^2 of roof space, approximately 0.15 inches of runoff can be stored from each building before the rain barrels would overflow. Finally, buildings were assumed to be located in each grid cell that was associated with a Developed Low, Medium, and/or High Intensity classification, according to the previously mentioned NLCD 2006 land use coverage.

Adhering to these assumptions results in the conceptual placement of over 24,000 rain barrels within the modeling domain. The assumptions used within this with project condition set up favored greater reductions in runoff than would be realistic.

6.1.2. Porous Pavement

Porous (or pervious) pavements offer another means of infiltrating rainfall at the source of runoff. Similar to other LID SCMs, porous pavements are frequently designed to intercept the "first flush" of pollutants from a common rainfall event. These SCMs work best in parking lots or other locations where vehicles only park and little to no traffic passes over the porous surface. Also, fine sediments and other materials must be kept off of the pavement surface so it doesn't clog up the porous materials. Therefore, these SCMs work best in big parking lots on the tops of hills; not at the bottom where there may be large sediment loads associated with runoff.

Each porous pavement area was hypothesized to have a gravel bed underneath with 0.5 inches of storage capacity for the pre-determined GSSHA grid cell size (approximately 2400 ft^2). Then, porous pavement areas were placed in grid cells with impervious cover equal to or greater than 75% since these locations coincided with large parking lots. This resulted in the conceptual placement of over 600 acres of porous pavement throughout the modeling domain. Similar to the previously mentioned rain barrel with project condition, these assumptions favored greater reductions in runoff than are likely with this SCM.

6.1.3. Results

To further accentuate the effects of these SCMs on flow rates and volumes throughout the modeling domain, 3-hr duration hypothetical storm events were used as the meteorological input to the GSSHA model. Using the 3-hr duration events in lieu of the 24-hr duration events resulted in less rainfall volume and therefore greater reductions in flow rates due to the limited storage space provided by these SCMs.

As expected, flow rate reductions, compared to existing conditions, were greater for the more frequently-occurring events (i.e. 99% and 50% ACE rainfall). However, for less frequently occurring rainfall events (such as the 10% - 0.2% ACE events) flow rate reductions became smaller. Hydrographs comparing the existing condition, w/ rain barrels, and w/ porous pavement runoff responses for the 10% ACE, 3-hr duration event are shown in Figures 6.1 and 6.2 near the upstream end of the Brookdale Ave levee and Rock Creek confluence, respectively. As shown in Figures 6.1 and 6.2, flow rate reductions using these SCMs are minimal for this event.

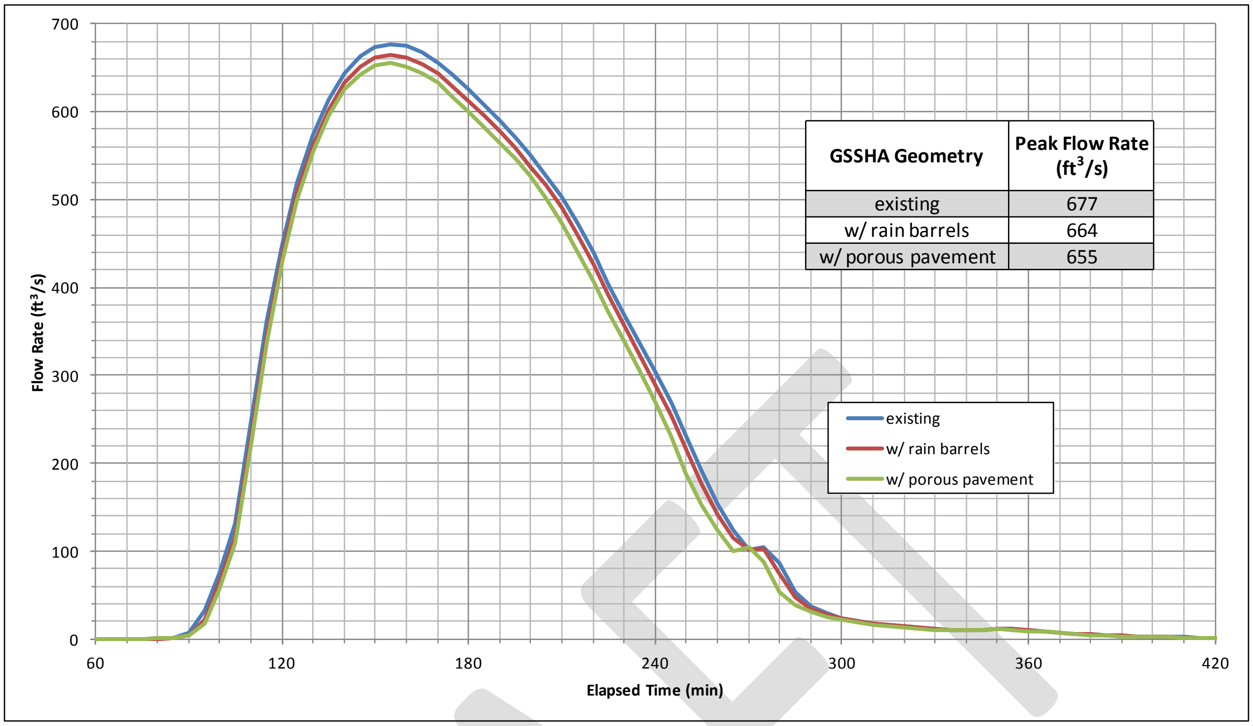

Figure 6.1 – Low Impact Development Options Results – Upstream of Keswick Culvert Confluence
10% ACE Event, 3-hr Duration

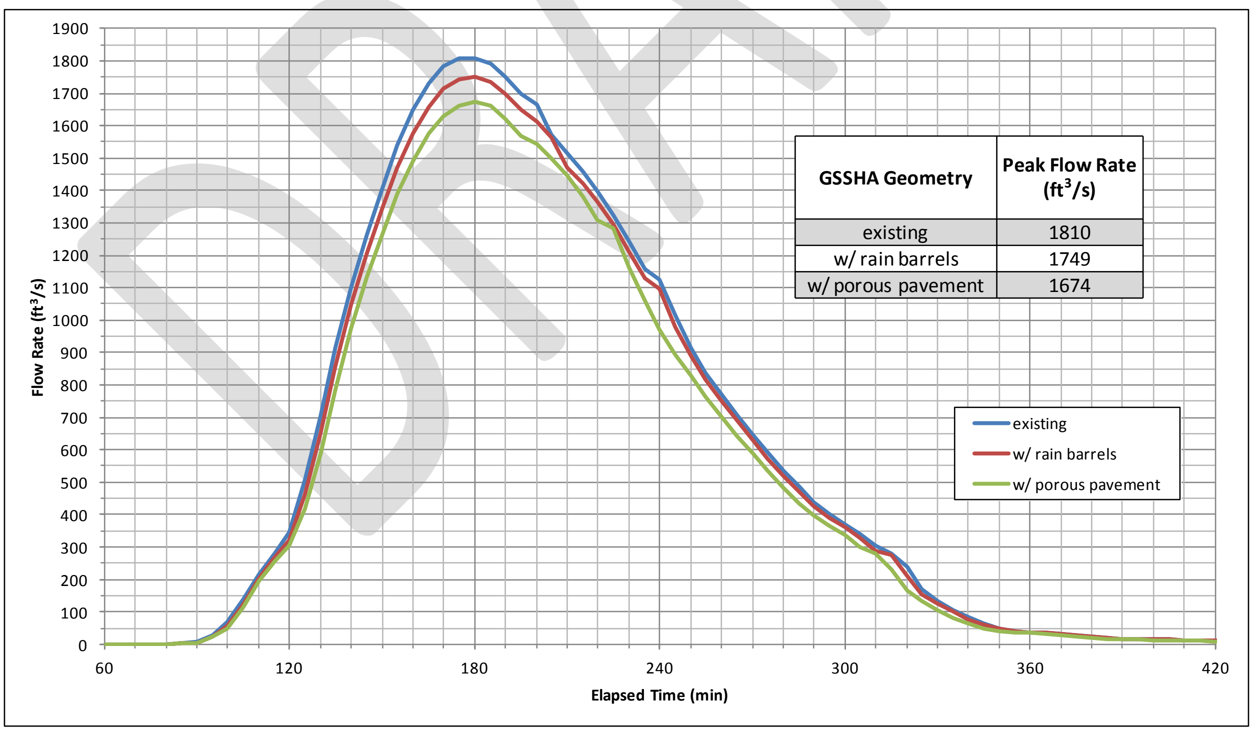

Figure 6.2 – Low Impact Development Options Results – Upstream of Rock Creek Confluence
10% ACE Event, 3-hr Duration

LID SCMs are best implemented to reduce pollutant loadings and runoff volumes for frequently occurring events. In this instance, they don't particularly store large volumes of runoff less frequently-occurring events, which is the purpose of this study. Therefore, these options were not considered for further detailed analysis.

6.2. Constriction Removals

Multiple existing bridge and culverts span Tookany Creek throughout Cheltenham Township. The vast majority of these crossings affect the movement of water. Flows can be “throttled” or constricted by these bridges and crossings leading to elevated WSEL upstream of the bridge that can then negatively impact infrastructure, residences, and various property. However, by temporarily storing water upstream, these bridges and culverts also act to reduce peak flow rates downstream. Reducing or eliminating these constrictions can reduce WSEL upstream and lessen flooding damages and/or increase peak flow rates and damages downstream.

Several major constrictions throughout Tookany Creek were included within the without project condition model geometry which influenced the development of flow-frequency relationships. These constrictions are detailed in Table 5.5. Three bridges/culverts were removed from the GSSHA model geometry and compared against the without project conditions results to determine their potential consequences. These included the Easton Road culvert, the SEPTA 11.22 culvert, and the Rock Creek culvert at Widener Road, as shown in Figure 6.3.

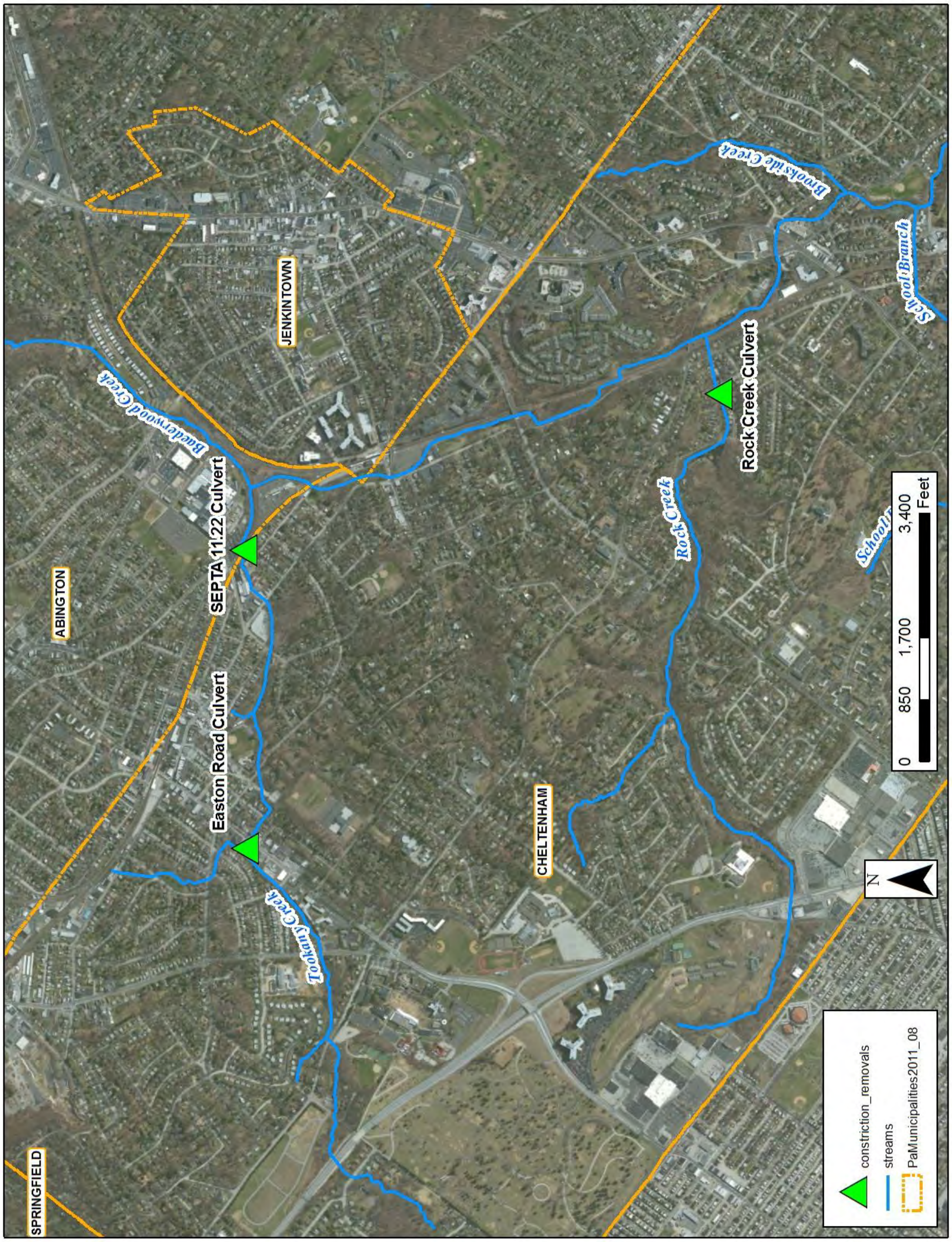

Figure 6.3 – Constriction Removal Options
ESRI World Imagery

Figures 6.4 – 6.7 show the resulting hydrographs due to the removal of each constriction (individually) in addition to the existing conditions hydrograph at various locations throughout the affected areas. This hydrologic analysis shows that peak flow rates are increased due to the removal of these constrictions due to the lack of "storage" induced by these constrictions, causing peak flow rates to occur sooner and with greater magnitudes. While, upstream WSEL (from each constriction) were calculated to occur due to the removal of these constrictions within the GSSHA model, these changes are best estimated using a dedicated hydraulic modeling code, as is described in the Hydraulic Modeling Appendix.

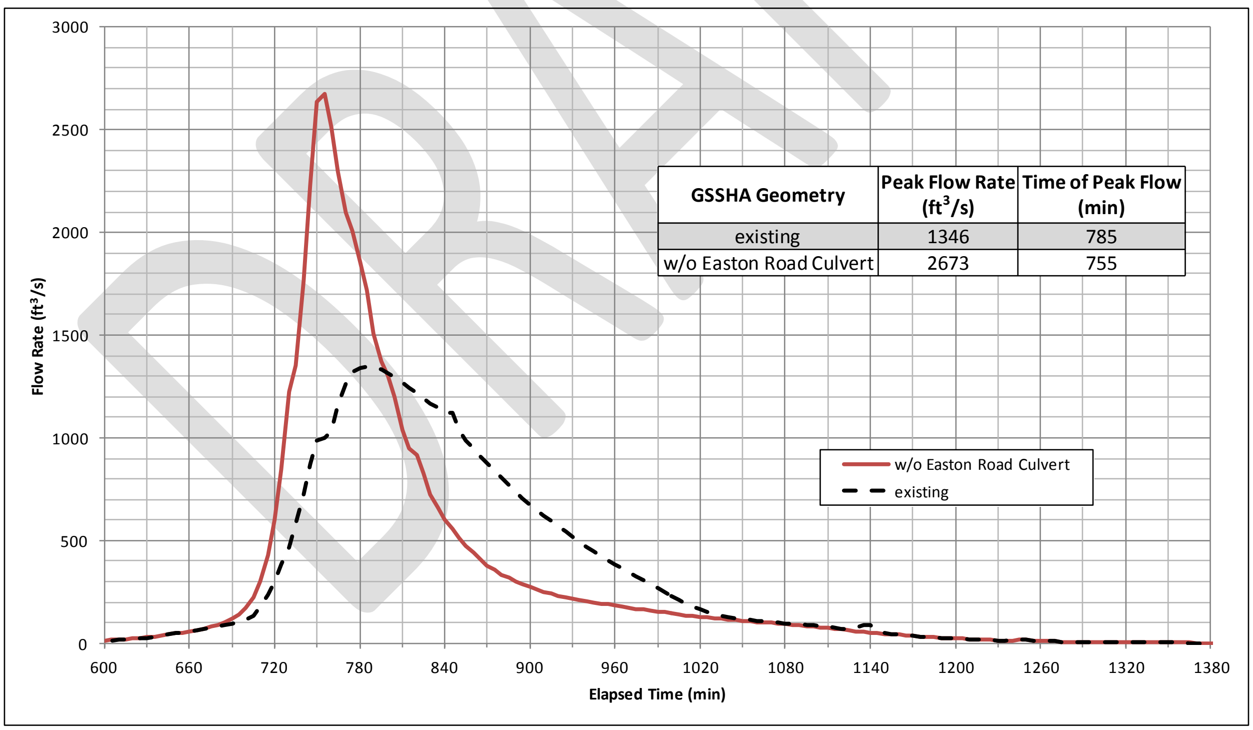

Figure 6.4 – Constriction Removal Results – Upstream of Keswick Culvert
1% ACE Event, 24-hr Duration

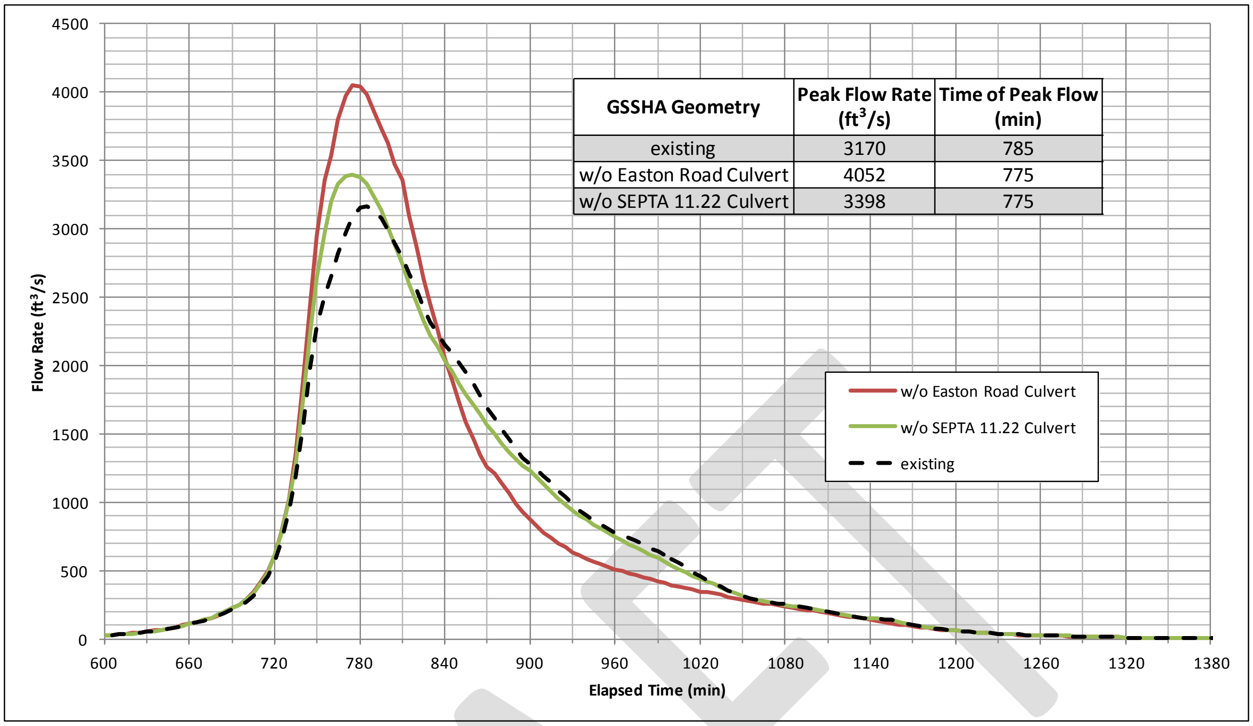

Figure 6.5 – Constriction Removal Results – Downstream of the Baederwood Creek Confluence
1% ACE Event, 24-hr Duration

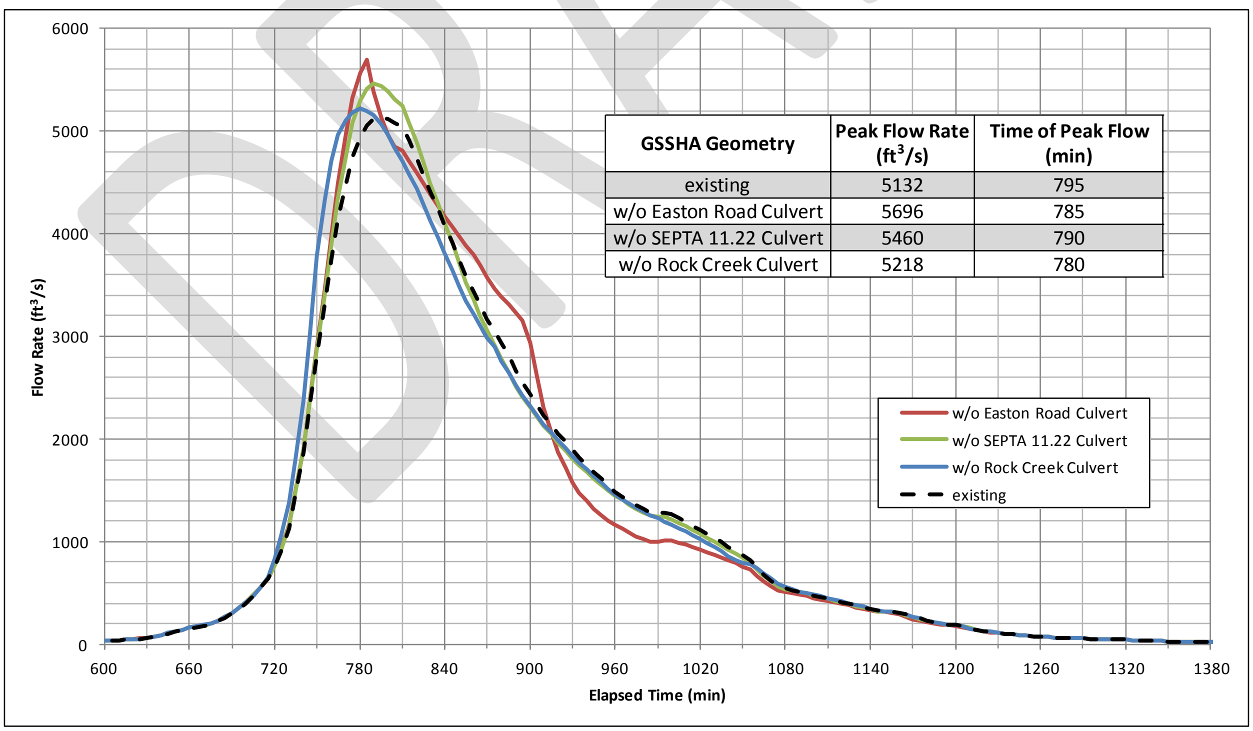

Figure 6.6 – Constriction Removal Results – Upstream of the Brookside Creek Confluence
1% ACE Event, 24-hr Duration

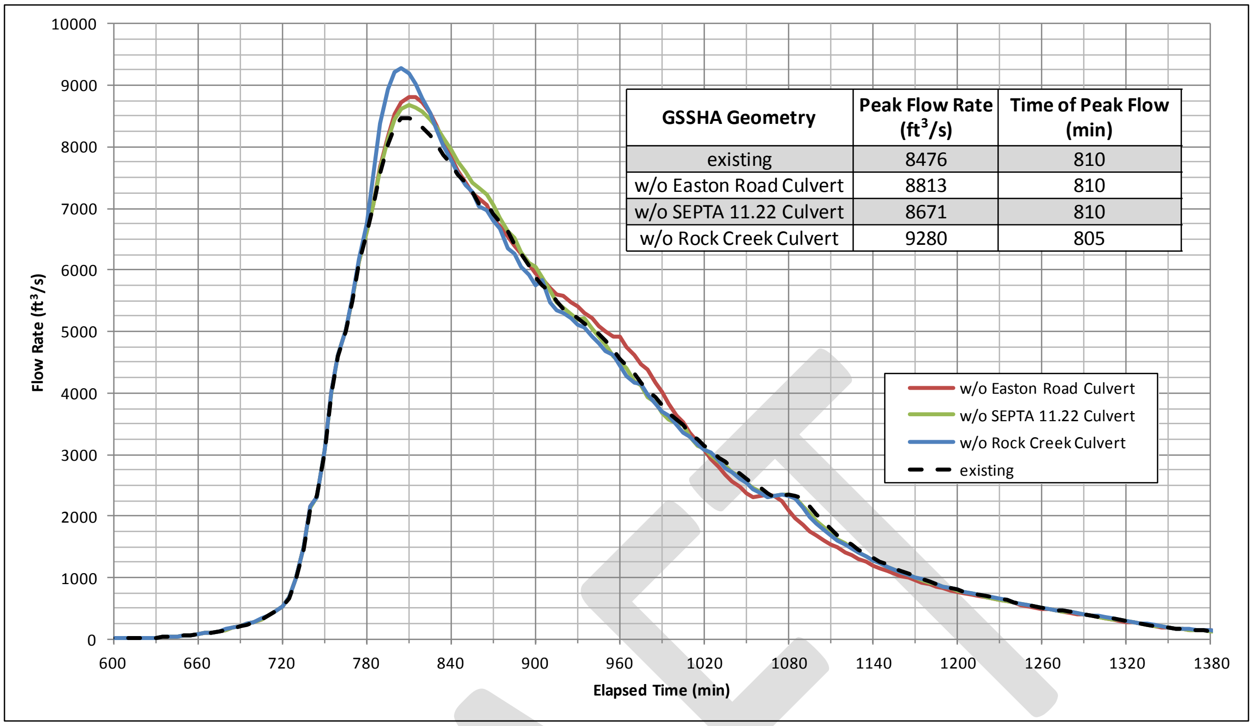

Figure 6.7 – Constriction Removal Results – Adams Ave Gage Location
1% ACE Event, 24-hr Duration

6.3. Local Neighborhood Flood Walls

While high flows and WSELs within Tookany Creek and other streams in Cheltenham Township can cause flooding damages, local runoff emanating from hillsides can also cause flooding damages. This is evident by the runoff patterns in and around the Harrison Avenue, Bickley Road, and Brookdale Road neighborhoods, which was described in Section 4.3.1 and shown in Figure 5.7. In an attempt to reduce the flooding risks due to these local runoff patterns at the Harrison Avenue, Bickley Road, and Brookdale Road neighborhoods, localized floodwalls were conceptualized and input to the GSSHA model geometry. For simplicity, these structures were input as floodwalls. However, these structures could be implemented as raised roadways, earthen berms, or movable miter gates which may have less negative socio-economic impacts than floodwalls.

Within the Harrison Avenue neighborhood, floodwalls were conceptualized to exist from the Springhouse Lane – Harrison Ave intersection to the Springhouse Lane bridge crossing Tookany Creek. Another floodwall was laid out along Lismore Ave. These floodwalls were placed to limit the ingress of localized runoff emanating from the hillside north of the Harrison Avenue neighborhood as well as from the east. The total length of floodwalls conceptualized within this neighborhood was approximately 550 ft.

Within the Bickley Road neighborhood, floodwalls were conceptualized along South Easton Street and East Waverly Road. These floodwalls were placed to limit the ingress of localized runoff emanating from the northeast of the Bickley Road neighborhood. The total length of floodwalls conceptualized within this neighborhood was approximately 960 ft.

Within the Brookdale Road neighborhood, floodwalls were placed along the east and west side of Keswick Avenue starting at the SEPTA railroad underpass and ending at Parkside Lane near Renninger Park. These floodwalls would act to direct flows along Keswick Avenue to Renninger Park to enter the Tookany Creek instead of entering the Brookdale Road neighborhood. The total length of floodwalls conceptualized within this neighborhood was approximately 1500 ft. The layout of these floodwall structures is shown in Figure 6.8.

Figure 6.8 – Local Neighborhood Floodwall Locations
ESRI World Imagery

Model simulations were executed to determine the minimum floodwall heights needed to exclude local runoff generated during the 50%, 4%, 1%, and 0.2% ACE, 24-hr duration events from entering the aforementioned neighborhoods. These options did not include other geometry modifications. Additional infrastructure would likely need to be implemented at the same time for these structures to be effective at reducing flooding risks. For instance, stabilization measures in Renninger Park would be needed to reduce the erosion potential of flows along Keswick Avenue as they enter Tookany Creek. Similar measures would likely be needed within the Harrison Avenue and Bickley Road neighborhoods. The maximum heights needed to exclude local runoff from entering the three neighborhoods are shown in Table 6.1.

Table 6.1 – Maximum Local Neighborhood Floodwall Heights

Neighborhood	Section	Maximum Height (ft) Needed to Exclude**:			
		50% ACE*	4% ACE*	1% ACE*	0.2% ACE*
Harrison	western	0.7	1	1.15	1.3
	northern	0.7	1	1.15	1.3
Bickley	eastern	0.75	1.3	1.6	2
Brookdale	Keswick “corridor”	1.25	2	2.25	3

*24-hr Rainfall Duration

**No freeboard Allotment

These with project conditions were not investigated further since USACE drainage area requirements were not met and BCRs were expected to be below 1.0.

6.4. Storage Areas

Embankments and regulating outlets placed across the Tookany Creek or sizable tributaries offer a way of temporarily storing volumes of water within pre-determined extents and reducing downstream flow rates through floodwave attenuation and translation. In order to effectively reduce peak flow rates downstream, embankments and regulating outlets should be situated upstream of impact areas, specifically at locations that control a large drainage area relative to the downstream impact areas. Additionally, in order to be cost-effective, embankments and regulating outlets should be located in positions that offer large amounts of flood storage space with respect to the upstream drainage area that can be realized with minimal excavation or embankment construction costs. Multiple locations along the Tookany Creek, Baederwood Creek, Rock Creek, and other smaller tributaries were identified upstream of multiple impact areas and offered appreciable storage space with relatively large contributing drainage areas. These included embankments / storage areas located at or near:

1) Tookany Creek near Doe Lane
2) Tookany Creek near West Waverly Road
3) Tookany Creek near Church Road
4) Tookany Creek near Limekiln Pike
5) East Branch Tookany Creek near Grove Park
6) Tookany Creek at or near the George Perley Bird Sanctuary
7) An unnamed tributary to the Tookany Creek near the Highland Ave – Mt. Carmel Ave intersection
8) Baederwood Creek near Baeder Road
9) An unnamed tributary to Baederwood Creek near Highland Ave (East)
10) Baederwood Creek near Highland Ave (West)
11) An unnamed tributary to Rock Creek at Greenwood Ave
12) Rock Creek near Limekiln Pike / Ogontz Ave
13) Rock Creek near Washington Lane

At each embankment / storage area location, the existing stream invert was designated as the invert of the regulating outlet. The regulating outlets were conceptualized to be of such a size to allow the maximum non-damaging discharge to pass through the embankment unimpeded. Once streamflows exceed the maximum non-damaging discharge, additional excess flows should be stored. This maximized the amount of flood control storage space available above each embankment during a large runoff event.

The location and upstream drainage area for each storage area is shown in Table 6.2. The embankment and corresponding existing flood storage as well as the maximum flood storage available with excavation is shown in Table 6.3.

Table 6.2 – Potential Storage Area Locations and Upstream Drainage Area

Embankment / Storage Area	Storage Area Group	Latitude	Longitude	Upstream Drainage Area (mi^2)
Doe Lane	Upper Tookany	40.093	-75.173	0.29
West Waverly Rd		40.095	-75.171	0.62
Church Rd		40.094	-75.167	0.74
Limekiln Pike		40.095	-75.163	0.99
Grove Park		40.100	-75.158	0.48
			effective SUM	***1.47***
George Perley Bird Sanctuary	Middle Tookany	40.097	-75.146	2.92
Highland - Mt Carmel		40.100	-75.142	0.21
			effective SUM	***N/A***
Baeder Rd	Baederwood Creek	40.106	-75.132	0.75
Highland East		40.108	-75.131	0.45
Highland West		40.108	-75.134	0.30
			effective SUM	***0.75***
Limekiln - Ogontz	Rock Creek	40.078	-75.162	0.90
Trib - Greenwood		40.085	-75.155	0.15
Washington Lane		40.082	-75.147	1.58
			effective SUM	***1.58***

Table 6.3 – Potential Storage Area Embankment and Storage Initial Dimensions

Embankment / Storage Area	Storage Area Group	Stream Invert (ft NAVD88)	Embankment Top (ft NAVD88)	Embankment Height (ft)	Existing Flood Storage Space (ac-ft)	Max. Flood Storage Space With Excavation (ac-ft)
Doe Lane	Upper Tookany	289	303	14	8.96	24.24
West Waverly Rd		277	287	10	16.86	50.16
Church Rd		263	274	11	10.96	32.96
Limekiln Pike		248	264	16	24.81	82.01
Grove Park		242	252	10	10.08	28.42
				effective SUM	***71.66***	***217.80***
George Perley Bird Sanctuary	Middle Tookany	221	234	13	29.49	49.03
Highland - Mt Carmel		213	223	10	15.65	60.30
				effective SUM	***45.14***	***109.33***
Baeder Rd	Baederwood Creek	231	243	12	10.37	28.86
Highland East		249	263	12	16.46	45.12
Highland West		264	280	16	12.09	43.17
				effective SUM	***38.92***	***117.14***
Limekiln - Ogontz	Rock Creek	285	300	15	49.78	108.58
Trib - Greenwood		285	305	20	22.71	67.36
Washington Lane		218	235	17	42.86	124.35
				effective SUM	***115.35***	***300.29***

The 50% ACE event was preliminarily set equal to the maximum non-damaging discharge at each embankment location. A simplified backwater analysis using HEC-RAS was executed to determine the appropriate regulating outlet size that fit these criteria. All regulating outlets were assumed to be rectangular, reinforced concrete culverts for modeling purposes. The initial dimensions of the regulating outlets for each storage area are shown in Table 6.4.

Table 6.4 – Potential Storage Area Regulating Outlet Initial Dimensions

Embankment / Storage Area	Storage Area Group	50% ACE Peak Flow at Location (ft^3/s)	Regulating Outlet Width and Height (ft)
Doe Lane	Upper Tookany	130	4
West Waverly Rd		150	4.5
Church Rd		190	5
Limekiln Pike		270	5.5
Grove Park		170	4.75
George Perley Bird Sanctuary	Middle Tookany	500	7
Highland - Mt Carmel		120	4
Baeder Rd	Baederwood Creek	230	5
Highland East		120	4
Highland West		100	4
Limekiln - Ogontz	Rock Creek	250	5.5
Trib - Greenwood		80	2
Washington Lane		390	6

Originally, the previously mentioned 13 potential embankments / storage areas were considered for inclusion within the GSSHA model. The GSSHA model geometry was altered to include an embankment and regulating outlet for each storage area location. Simulations were executed using these altered geometries with the eight previously mentioned hypothetical frequency rainfall events as the meteorological driver. The hypothetical frequency rainfall events were executed using 3- and 24-hr rainfall durations to explore the multitude of rainfall events that can occur within the Tookany Creek watershed. Following these initial model simulations, four embankments / storage areas were removed from consideration including those located at or near:

- Tookany Creek at or near the George Perley Bird Sanctuary
- An unnamed tributary to the Tookany Creek near the Highland Ave – Mt. Carmel Ave intersection
- An unnamed tributary to Rock Creek at Greenwood Ave
- Rock Creek near Limekiln Pike / Ogontz Ave

These embankments / storage areas were removed from consideration due to a lack of appreciable storage in relation to the upstream drainage area, a lack of downstream flow reductions, and/or environmental considerations. This left nine potential storage areas that were considered for further investigation through supplementary GSSHA model simulations. The final nine storage areas and three storage area groupings are shown in Figure 6.9. Elevation-storage-area relationships for each storage area were developed using the previously mentioned PAMAP LIDAR datasets.

Elevation-discharge relationships for each embankment were created using simplified physical routings performed within HEC-HMS and exported as tables. Roughness coefficients for each conduit were set to 0.012, which corresponded to smooth concrete. Conduit lengths were estimated using the PAMAP

LIDAR datasets and orthophotographs. Outlet invert elevations were estimated using PAMAP LIDAR datasets, existing stream slopes, conduit lengths, and engineering judgment.

Both the elevation-storage area relationships and elevation-discharge relationships are shown for each storage area in Figures 6.10 – 6.27.

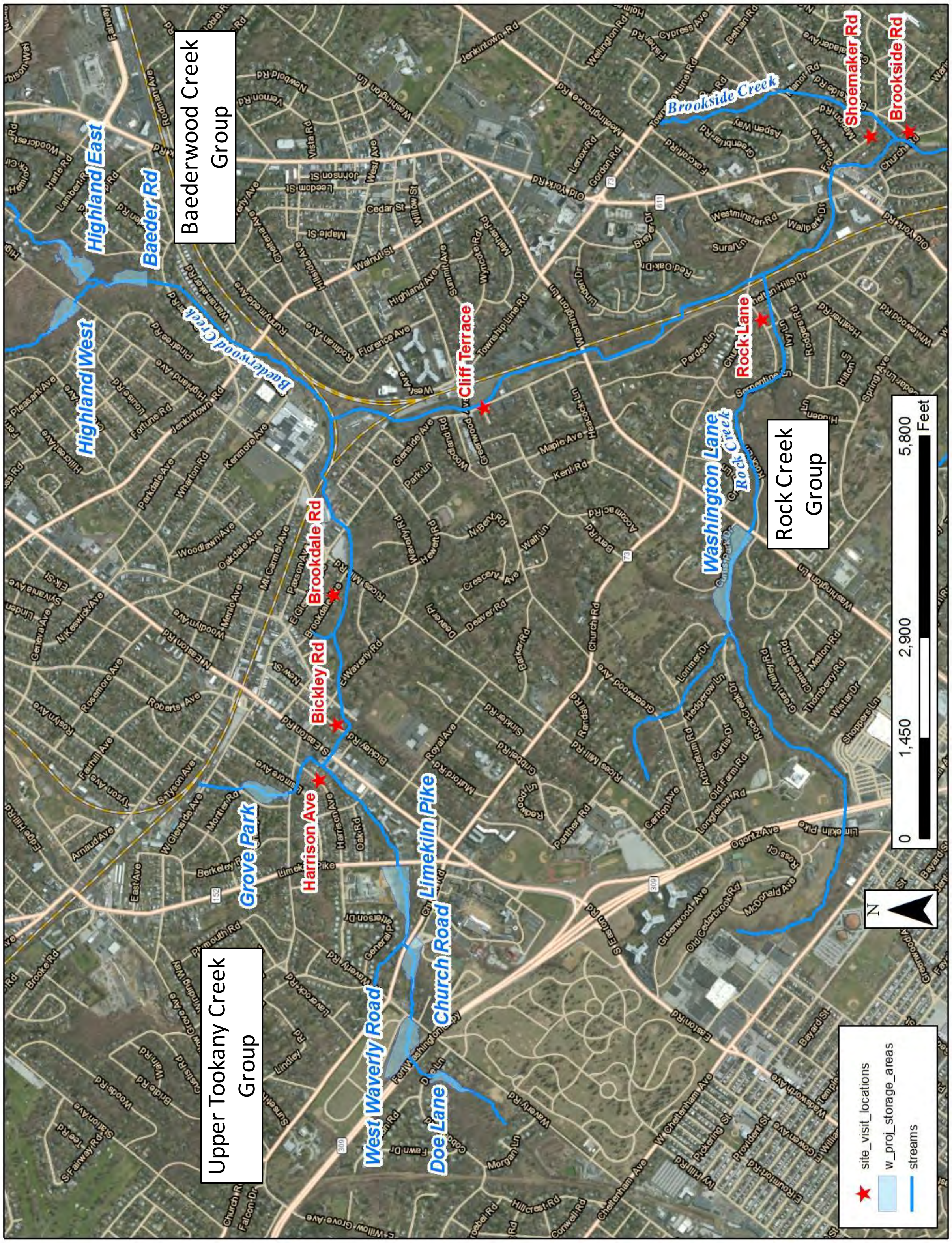

Figure 6.9 – Storage Areas and Storage Area Groups
ESRI World Imagery

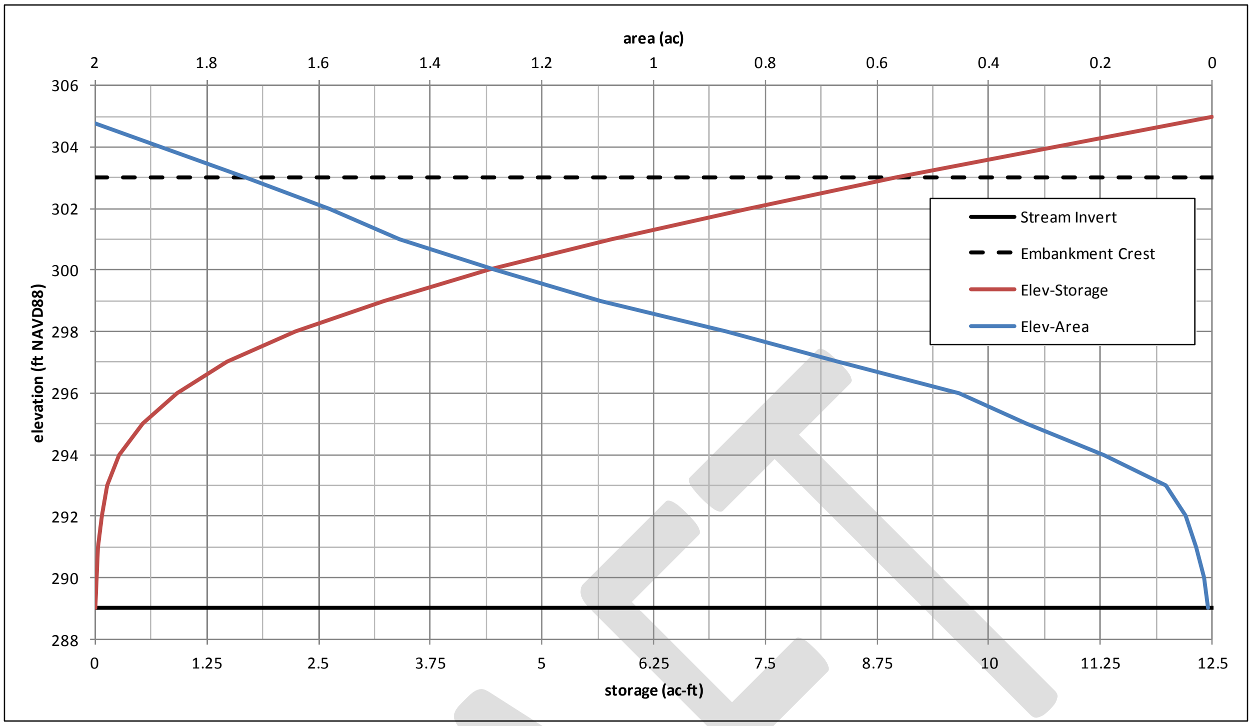

Figure 6.10 – Doe Lane Elevation-Storage-Area

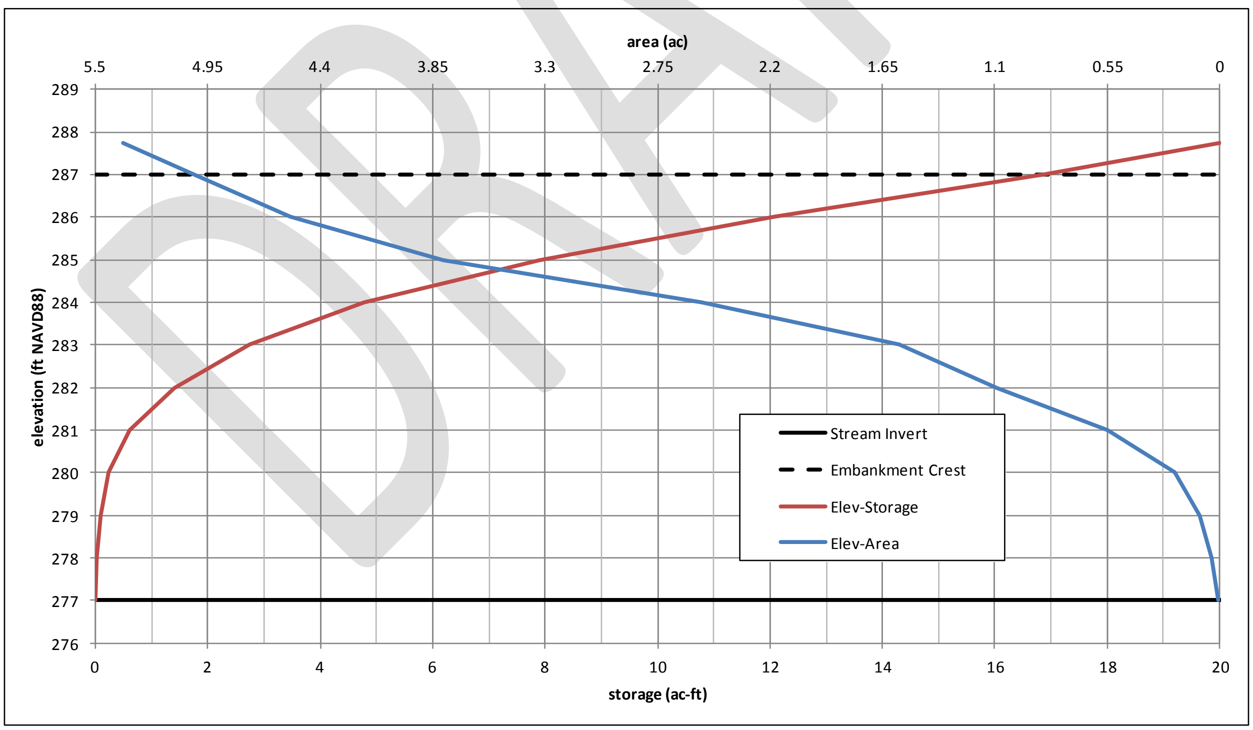

Figure 6.11 – West Waverly Road Elevation-Storage-Area

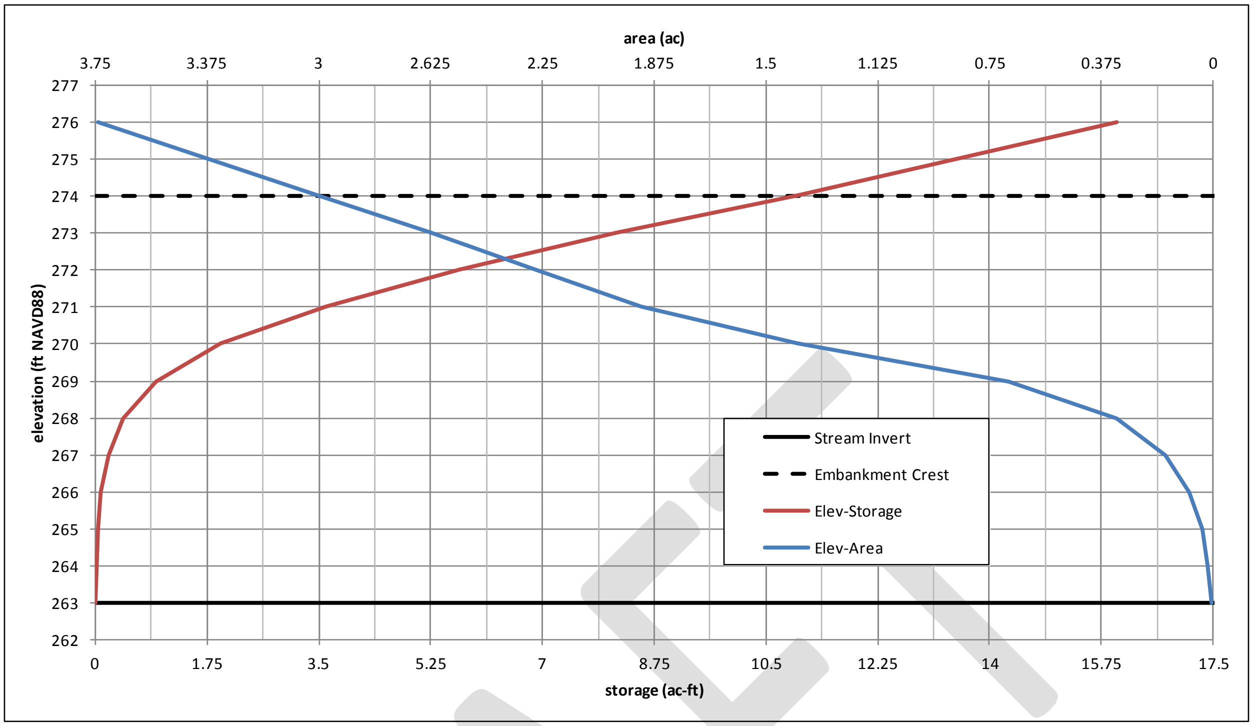

Figure 6.12 – Church Road Elevation-Storage-Area

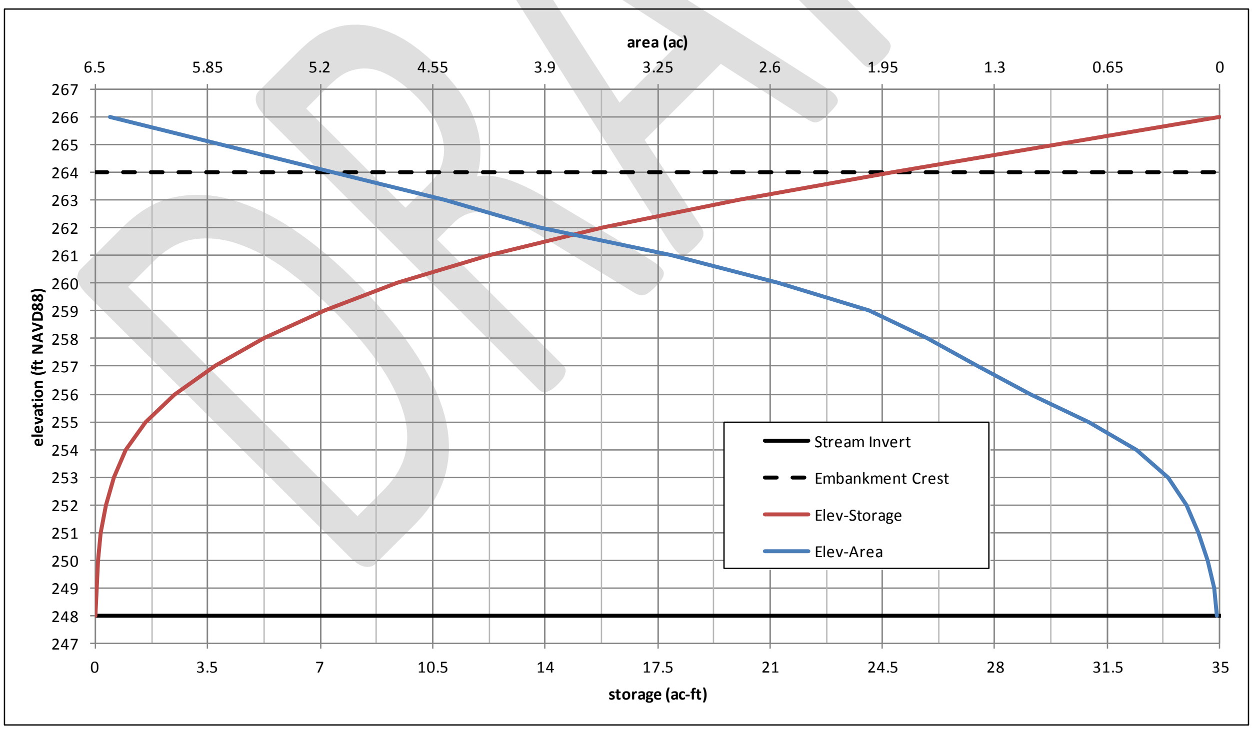

Figure 6.13 – Limekiln Pike Elevation-Storage-Area

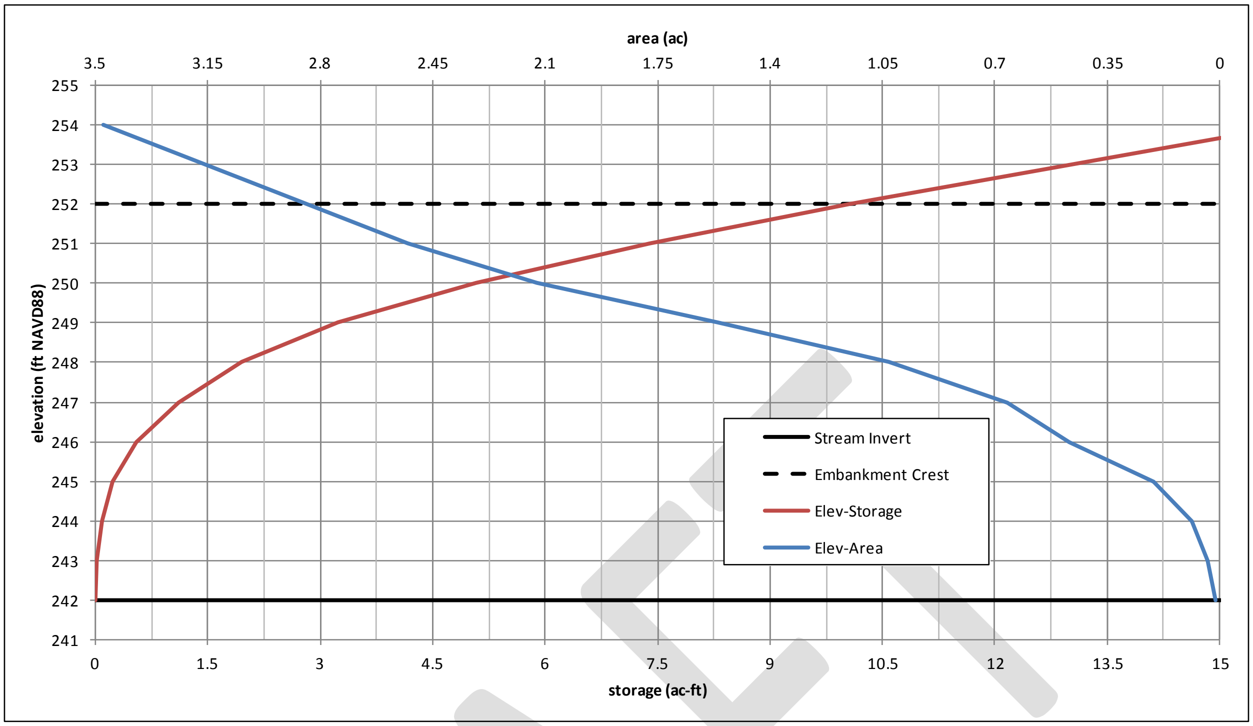

Figure 6.14 –Grove Park Elevation-Storage-Area

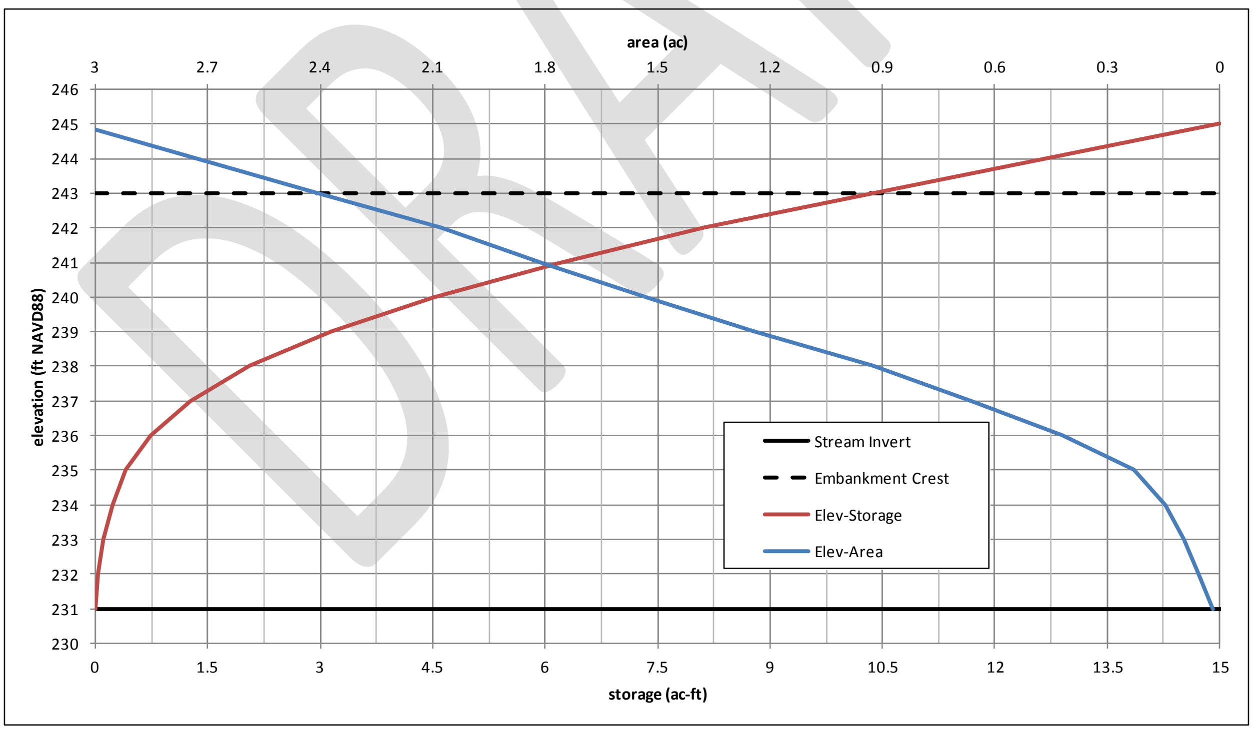

Figure 6.15 – Baeder Road Elevation-Storage-Area

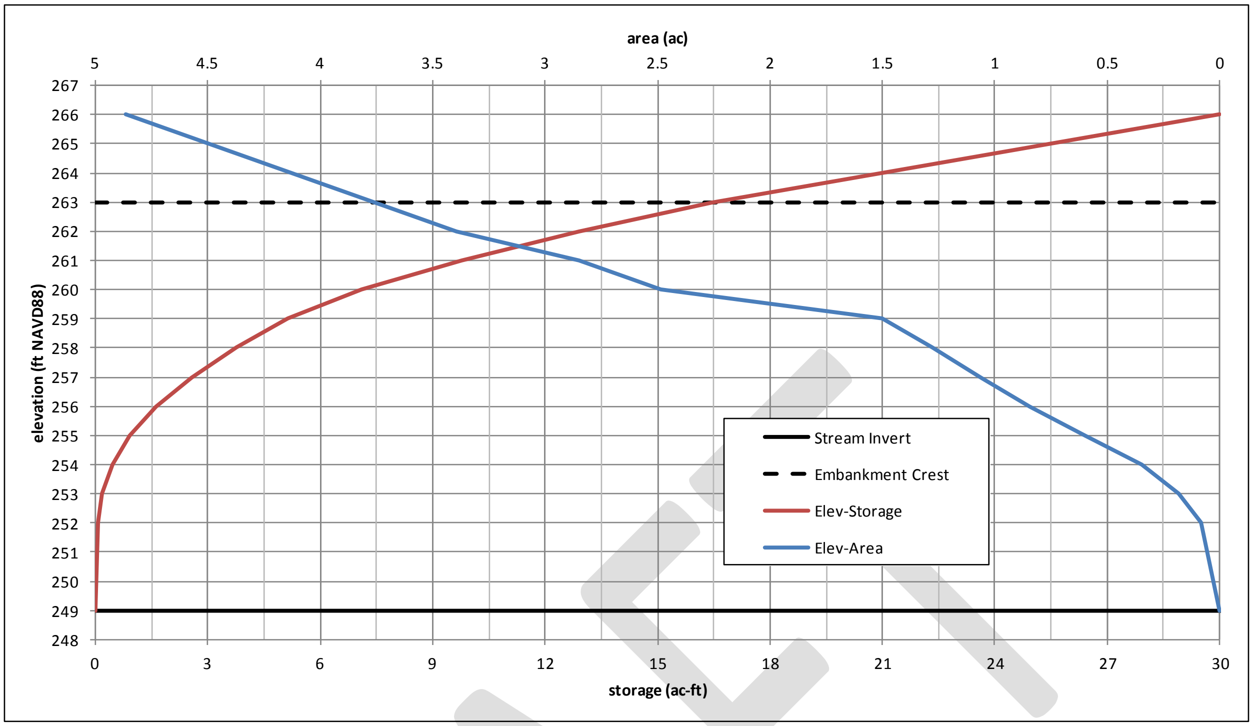

Figure 6.16 – Highland East Elevation-Storage-Area

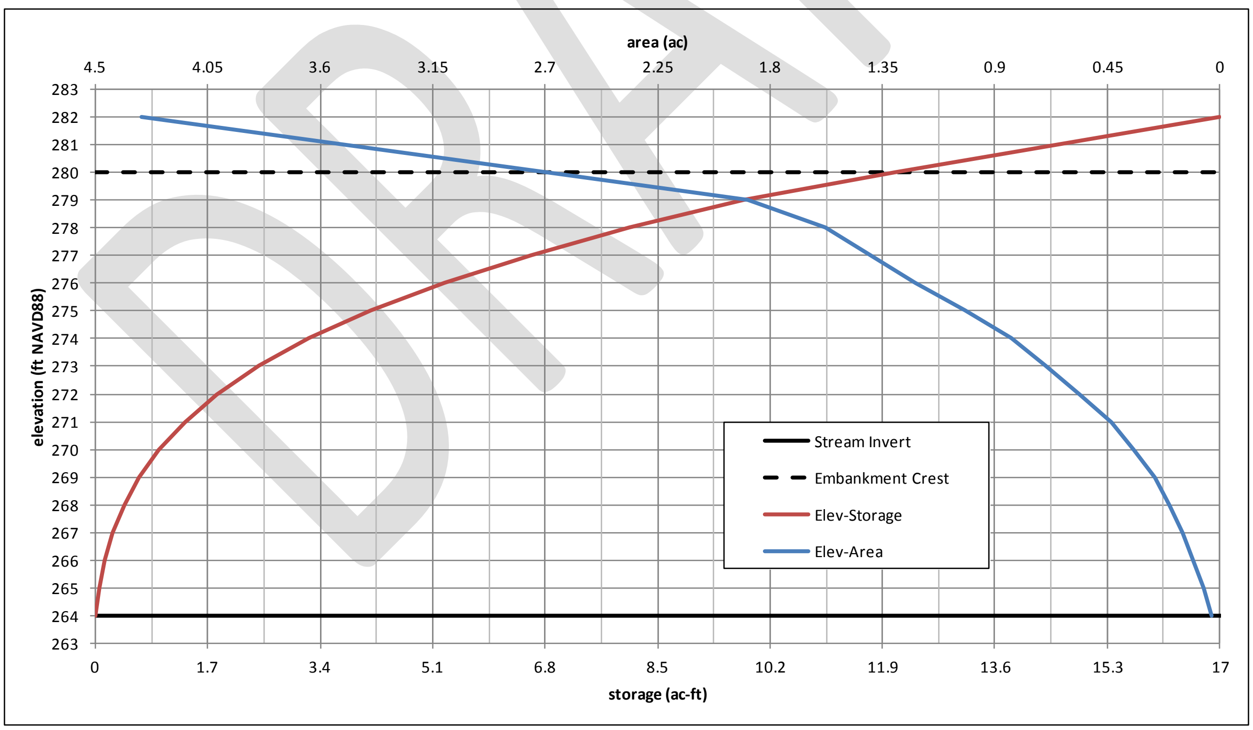

Figure 6.17 – Highland West Elevation-Storage-Area

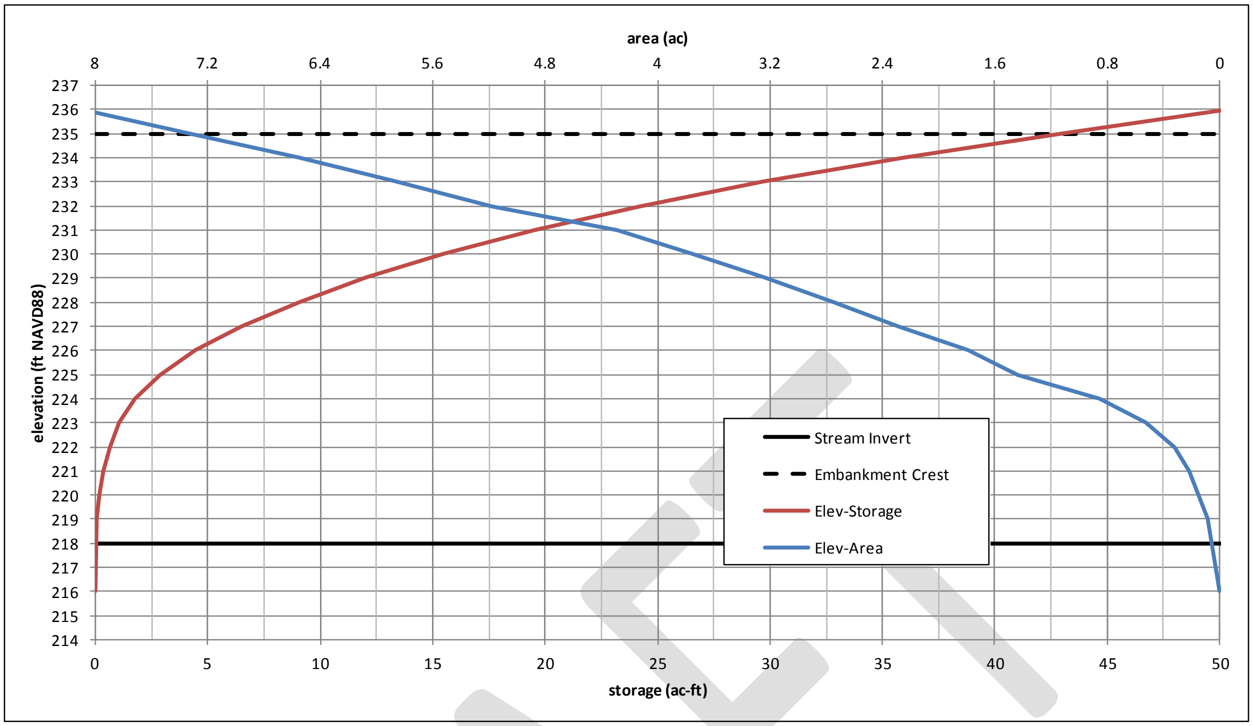

Figure 6.18 – Washington Lane Elevation-Storage-Area

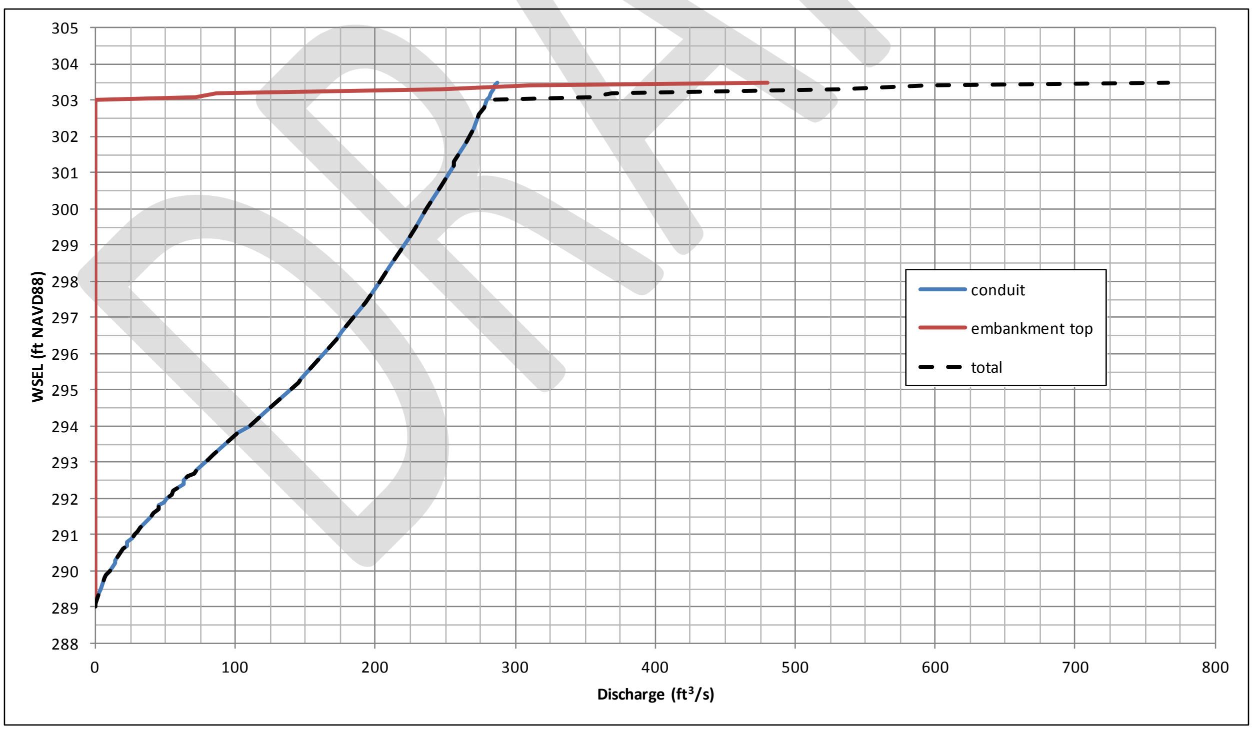

Figure 6.19 – Doe Lane Elevation-Discharge

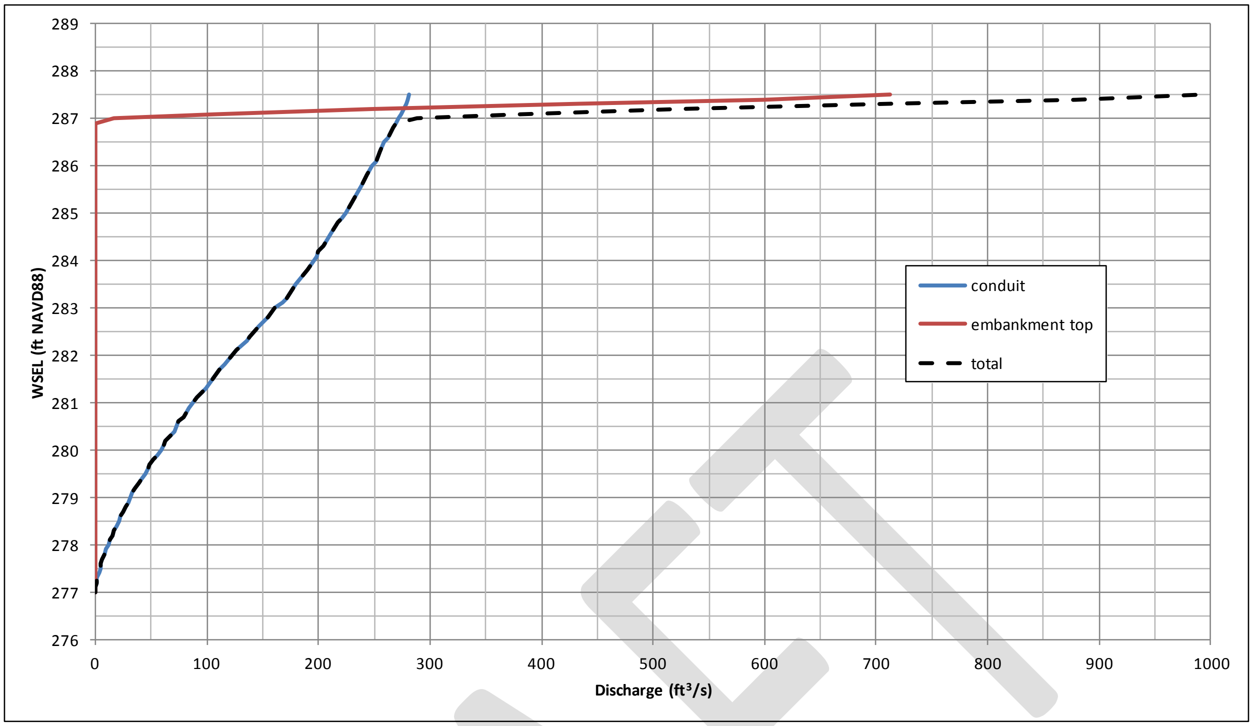

Figure 6.20 – West Waverly Road Elevation-Discharge

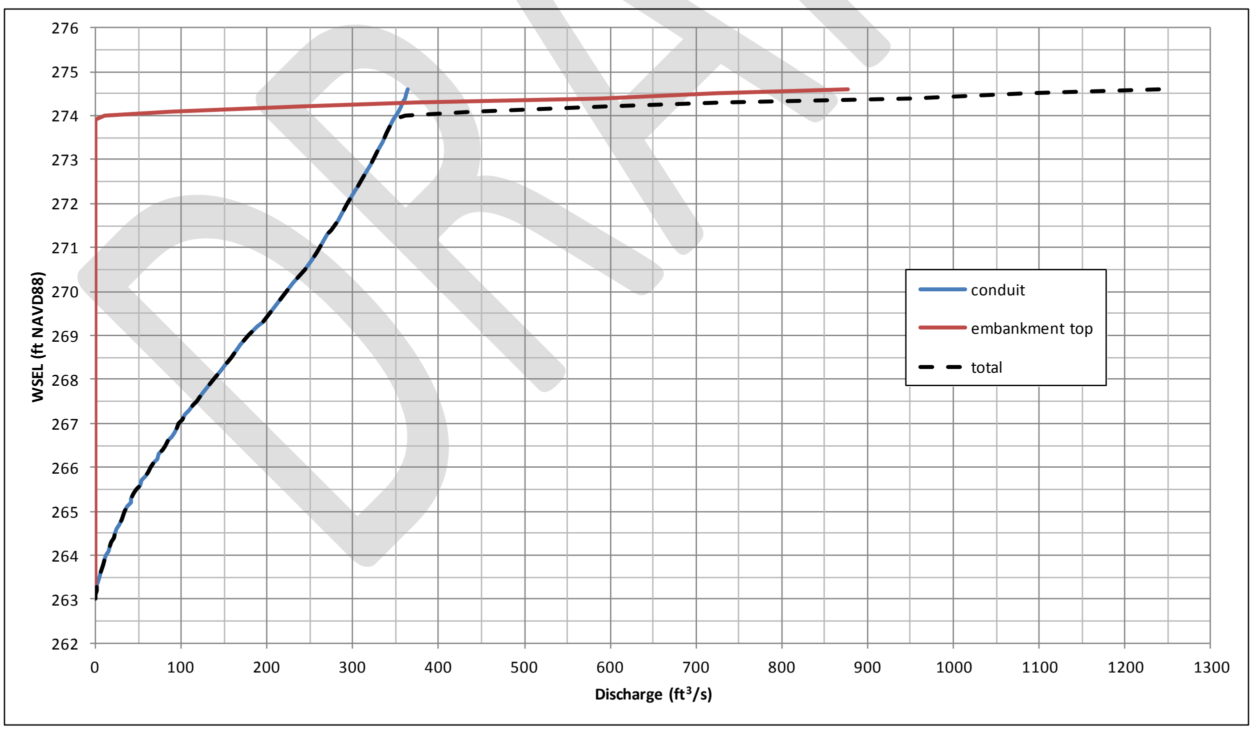

Figure 6.21 – Church Road Elevation-Discharge

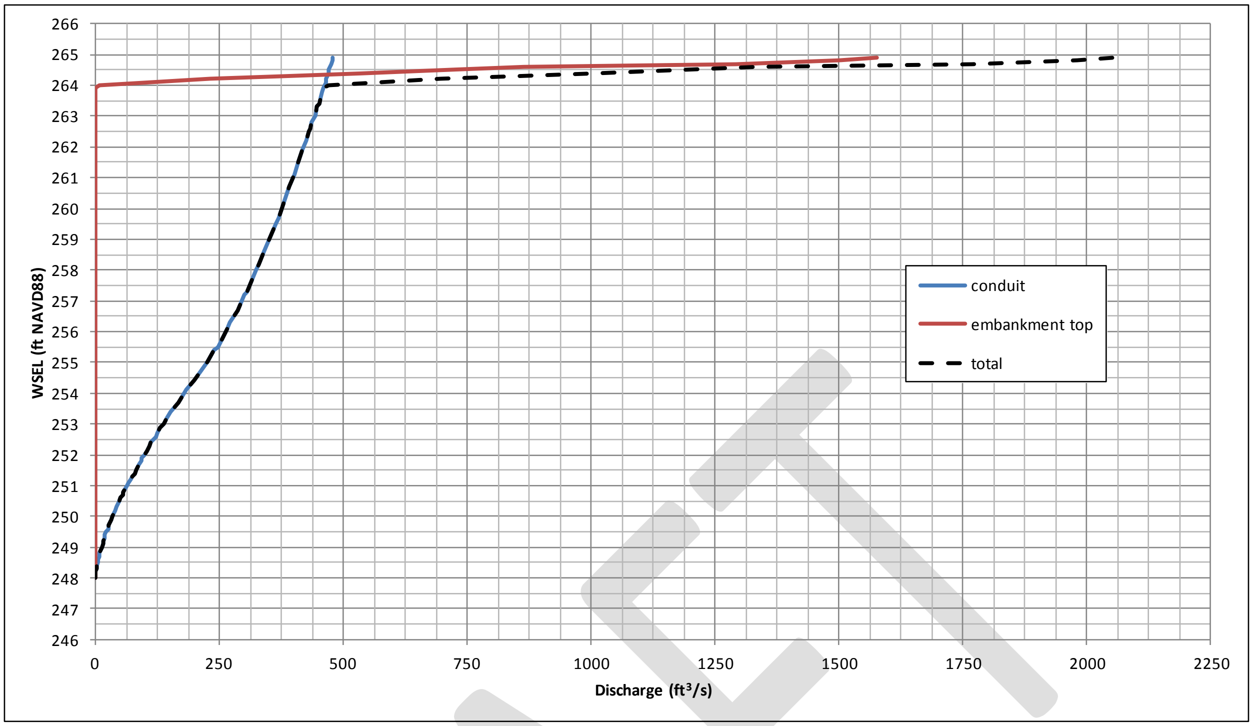

Figure 6.22 – Limekiln Pike Elevation-Discharge

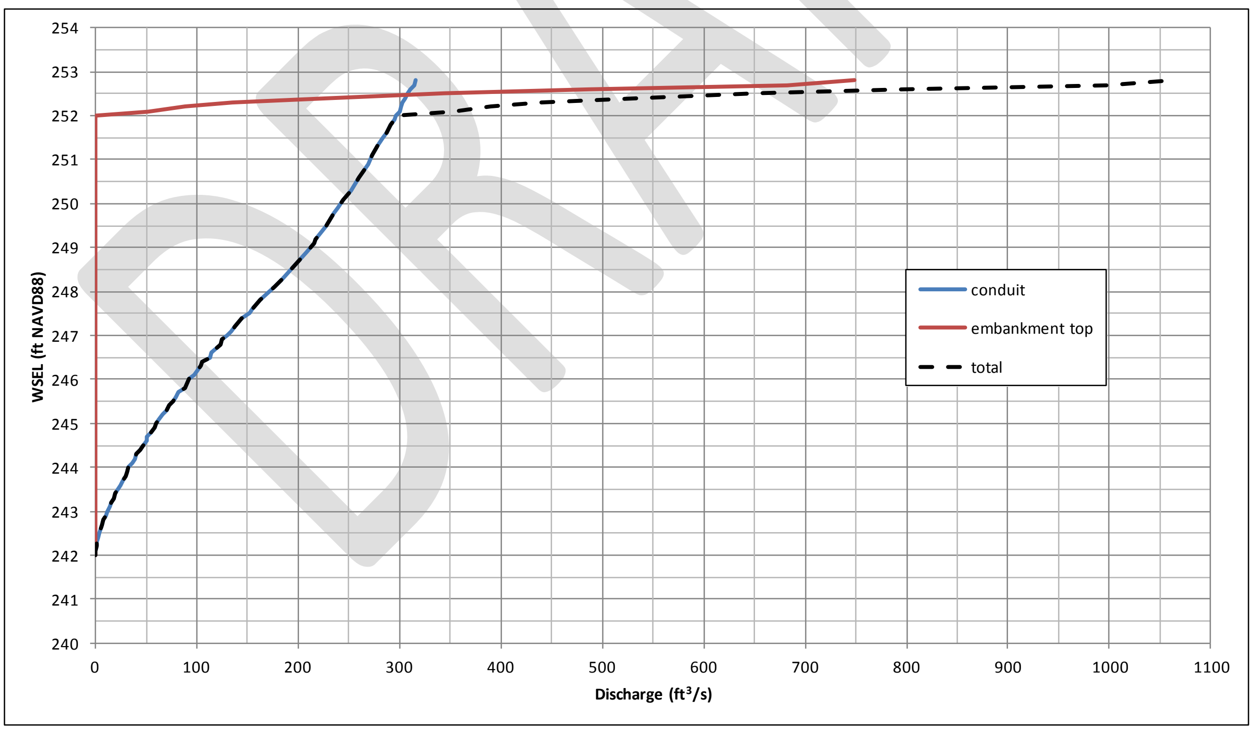

Figure 6.23 – Grove Park Elevation-Discharge

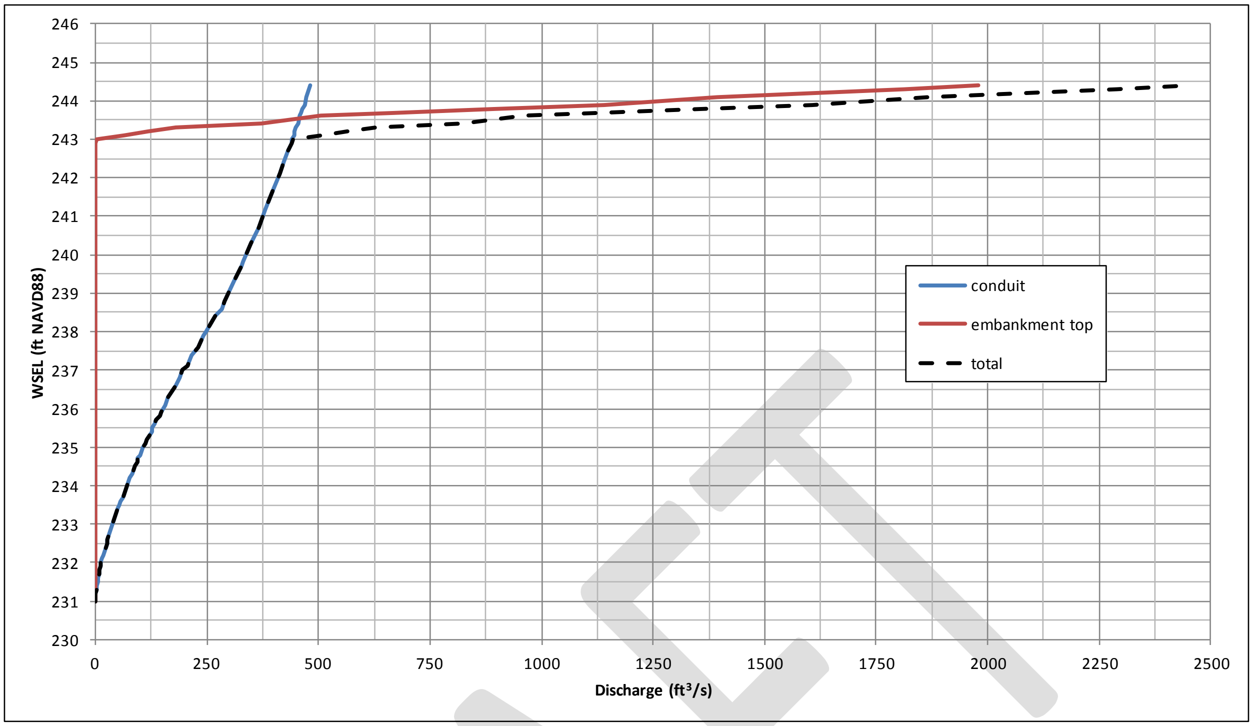

Figure 6.24 – Baeder Road Elevation-Discharge

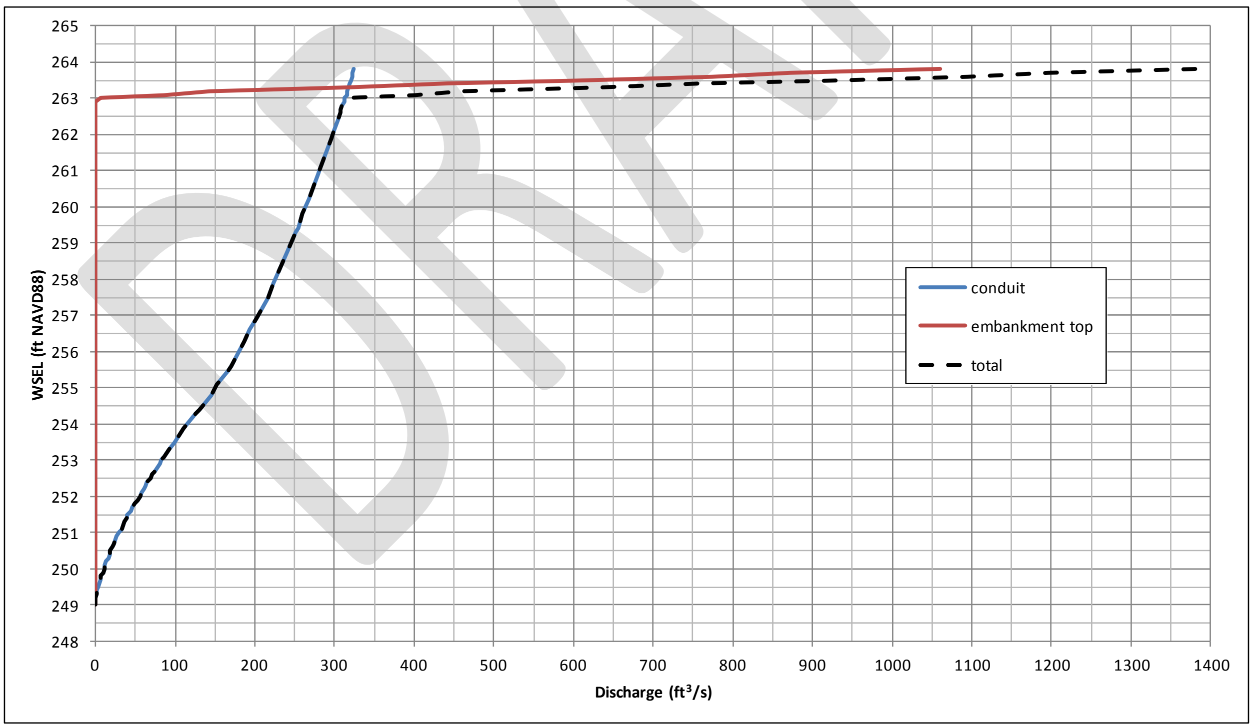

Figure 6.25 – Highland East Elevation-Discharge

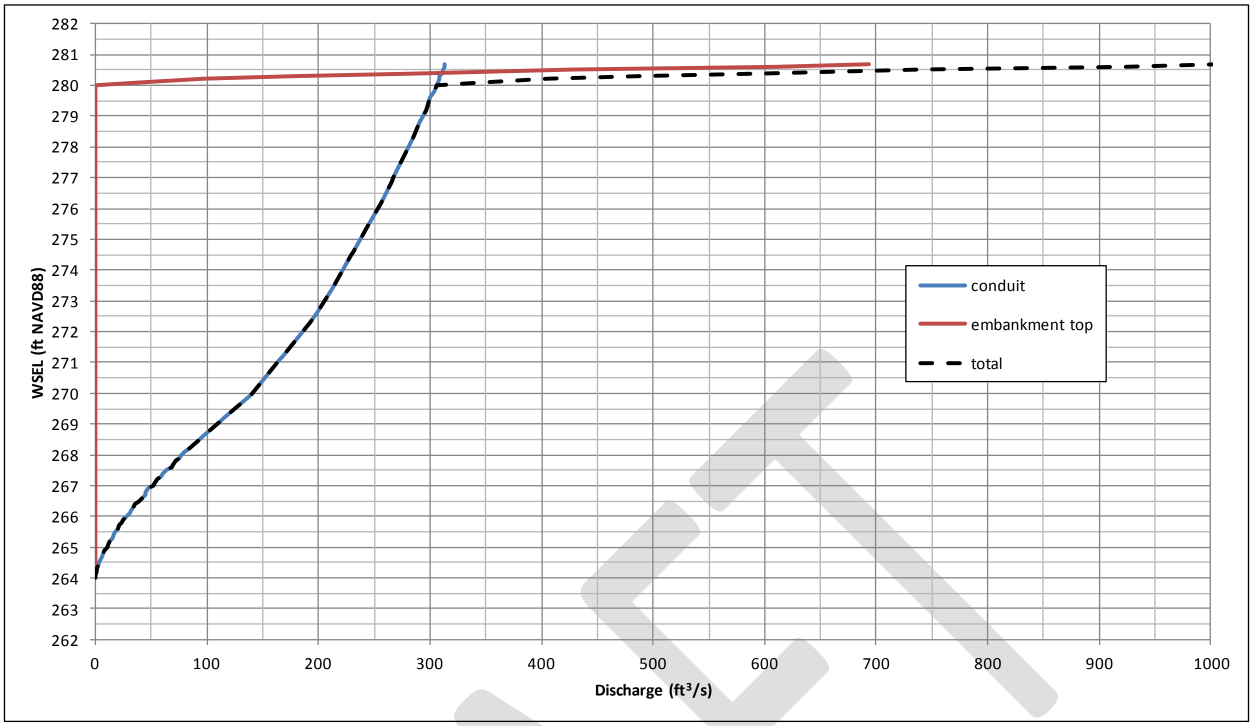

Figure 6.26 – Highland West Elevation-Discharge

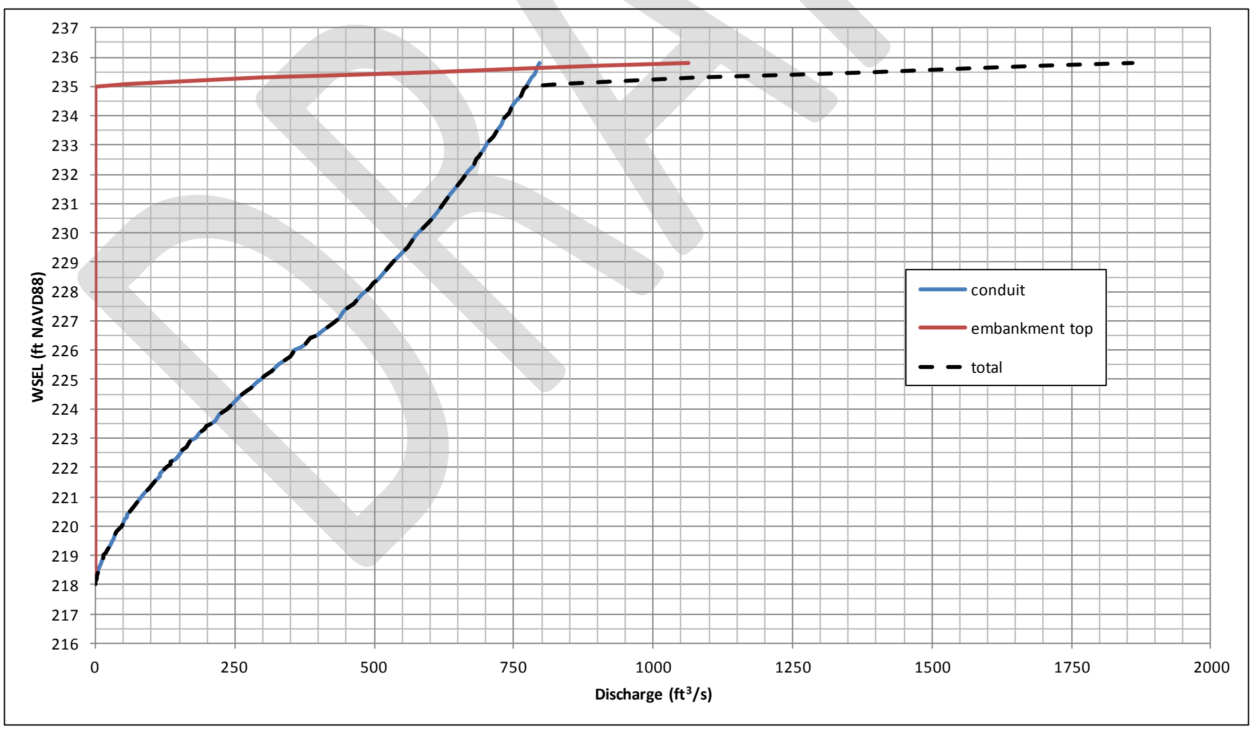

Figure 6.27 – Washington Lane Elevation-Discharge

The nine embankments were input to the GSSHA model geometry using embankment arcs. Embankment overtopping flows were allowed to proceed according to the broad crested weir equation using a weir coefficient of approximately 2.6 for each embankment (default within GSSHA). The previously shown conduit elevation-discharge relationships were input to simulate flow through the regulating outlets. Overland elevations were slightly altered (as necessary) in and around the storage areas due to requirements within the GSSHA code. Simulations were executed for each storage area individually as well as in storage area "groupings". Four groupings were simulated: Upper Tookany Creek, Baederwood Creek, Rock Creek, and an all-encompassing group. Regulating outlet dimensions were modified from those previously shown in Table 6.4 to make better use of available flood storage.

The first storage area grouping that was simulated was the Upper Tookany Creek group. This group was arranged to primarily provide flood risk reduction benefits to neighborhoods in Glenside (i.e. Harrison Ave, Bickley Road, Brookdale Ave). This grouping was comprised of five storage areas: Doe Lane, West Waverly Road, Church Road, Limekiln Pike, and Grove Park.

The second storage area group was the Baederwood Creek group. Three storage areas along Baederwood Creek (Highland West, Highland East, and Baeder Road) made up this group. Each storage area that is part of this group is entirely located within Abington Township. This storage area group is meant to provide flood risk reduction benefits to neighborhoods along Tookany Creek below the Baederwood Creek confluence (i.e. Cliff Terrace neighborhood). Benefits were not explicity quantified within Abington Township at the request of Cheltenham Township officials. However, this storage area group provides flood risk reduction along Baederwood Creek, in addition to communities along Tookany Creek below Baederwood Creek.

The third storage area group consisted solely of the Washington Lane storage area, which was the only remaining storage area that was part of the original Rock Creek group. This storage area is meant to provide flood risk reduction benefits to neighborhoods along Rock Creek and Tookany Creek below the Rock Creek confluence (i.e. Rock Lane, Shoemaker Road, Brookside Road, High School Road, Mill Road).

The final all-encompassing grouping consisted of all nine storage areas. Combined, the nine storage area's individual flood risk reduction benefits "overlapped" and provided benefits to a greater degree as well as to a greater extent (further downstream) than individually or as the three individual storage area groupings.

6.4.1. Upper Tookany Creek Group

Within the following descriptions, figures, and tables, the Upper Tookany Creek storage area group will be called "with project condition D1". The 1% ACE event, 24-hr duration streamflow hydrographs at several locations throughout the Upper Tookany Creek are shown in Figures 6.28 – 6.31 while the five storage area inflows, outflows, and pool elevations are shown in Figures 6.32 – 6.36. Within GSSHA, the extent of each storage area is allowed to fluctuate throughout a simulation as inflows are temporarily stored. As such, the exact location of the storage area "inflow" fluctuated as well. Change in storage and outflow relationships were used to calculate inflow and then smoothed using a 3-hr centered moving average scheme. Finally, the "performance" of each storage area (peak inflow rate, peak outflow rate, and peak WSEL) for the 1% ACE event, 24-hr duration is shown in Table 6.5.

Figure 6.37 shows the maximum overland depths during the 1% ACE event, 24-hr duration simulation within the GSSHA model. This figure helps to explain the complicated runoff patterns within this part of the Tookany Creek watershed. During this event, inflows are stored within the Doe Lane storage area with only a small amount of water exceeding flood storage and spilling over the embankment crest (for approximately 30 minutes). However, the Doe Lane storage area reduces peak inflows by approximately 50%. Significant "uncontrolled" runoff then enters Tookany Creek below the Doe Lane storage area along the eastern edge of the RT 309 roadway embankment. This additional inflow, along with Doe Lane outflows are then stored by the West Waverly Road storage area where peak inflows are reduced by approximately 35%. A relatively large ephemeral channel then enters the Church Road storage area on the downstream right side near Arcadia University. These inflows and West Waverly Road outflows are stored within the Church Road storage area. Peak flow rates are reduced by approximately 35% within this storage area. Immediately downstream of the Church Road storage area, an unnamed tributary of the Tookany Creek enters on the left side, which then enters the Limekiln Pike storage area. Peak inflow rates are reduced by approximately 50% within the Limekiln Pike storage area. Meanwhile, Grove Park reduces peak flow rates on the East Branch Tookany Creek by approximately 40% while excess inflows overtopped the embankment for approximately 30 minutes.

Immediately downstream of the Limekiln Pike storage area, an ephemeral channel conveys a relatively large amount of uncontrolled runoff into Tookany Creek from the right side, which leads to a relatively large increase in flow rates moving downstream. However, peak flow rates (as compared to existing conditions) are reduced by approximately 40% and 25% near the Keswick Culvert confluence and just upstream of Baederwood Creek, respectively. From that point downstream, peak flow rate reductions become negligible due to increasing, intervening, uncontrolled drainage areas. With project condition and existing condition flow rates are essentially the same at the Adams Ave gage location. The decreases in peak flow rates due to this with project condition are shown in Table 6.6. Negative peak flow rate reductions indicate a slight increase in peak flow rate due to this with project condition. However, these increases are only predicted for frequently occurring events where attenuated and translated flood waves (due to the storage within the with project condition storage areas) combines with uncontrolled runoff from other areas.

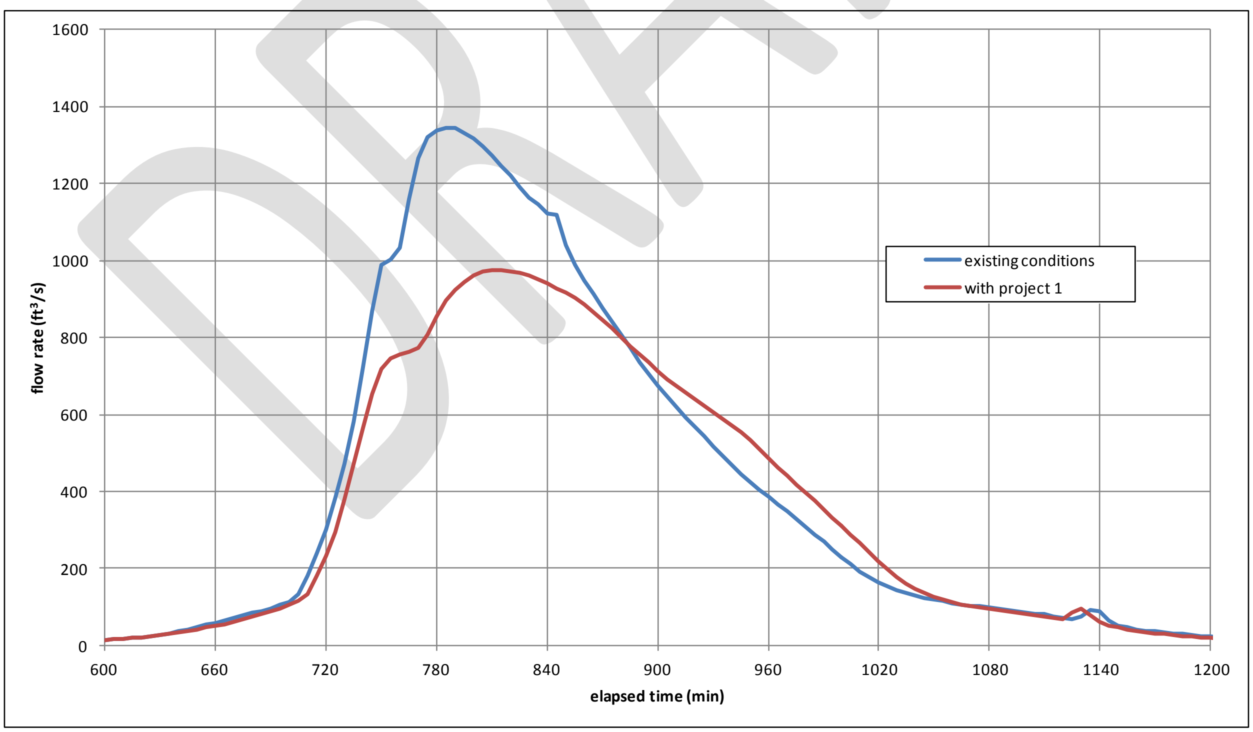

Figure 6.28 – With Project Condition D1 Results – Upstream of Keswick Culvert Confluence
1% ACE Event, 24-hr Duration

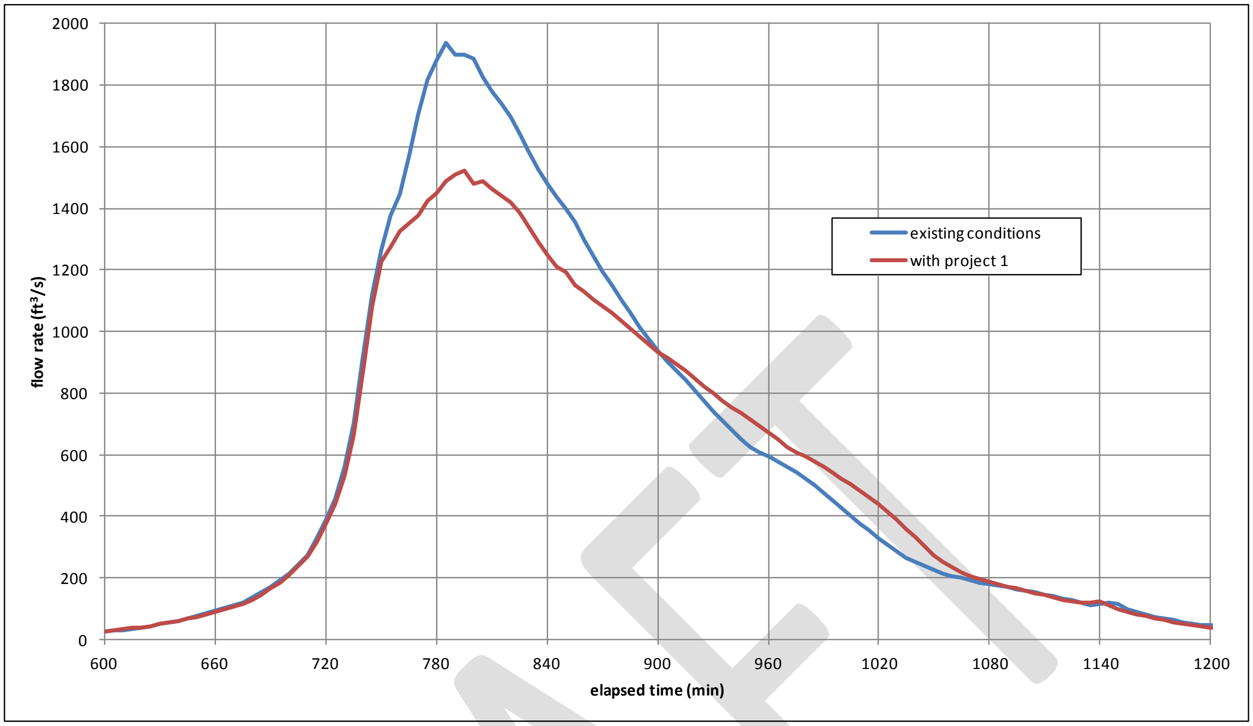

Figure 6.29 – With Project Condition D1 Results – Upstream of Baederwood Creek Confluence
1% ACE Event, 24-hr Duration

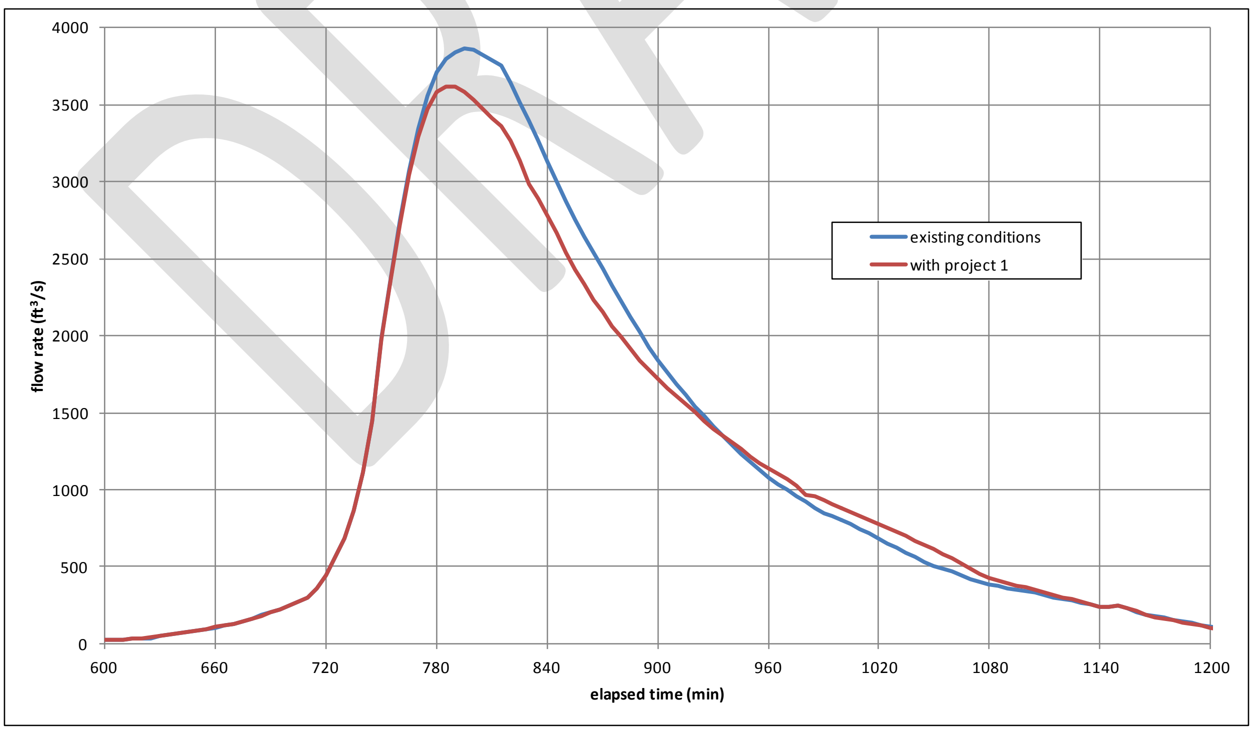

Figure 6.30 – With Project Condition D1 Results – Upstream of Rock Creek Confluence
1% ACE Event, 24-hr Duration

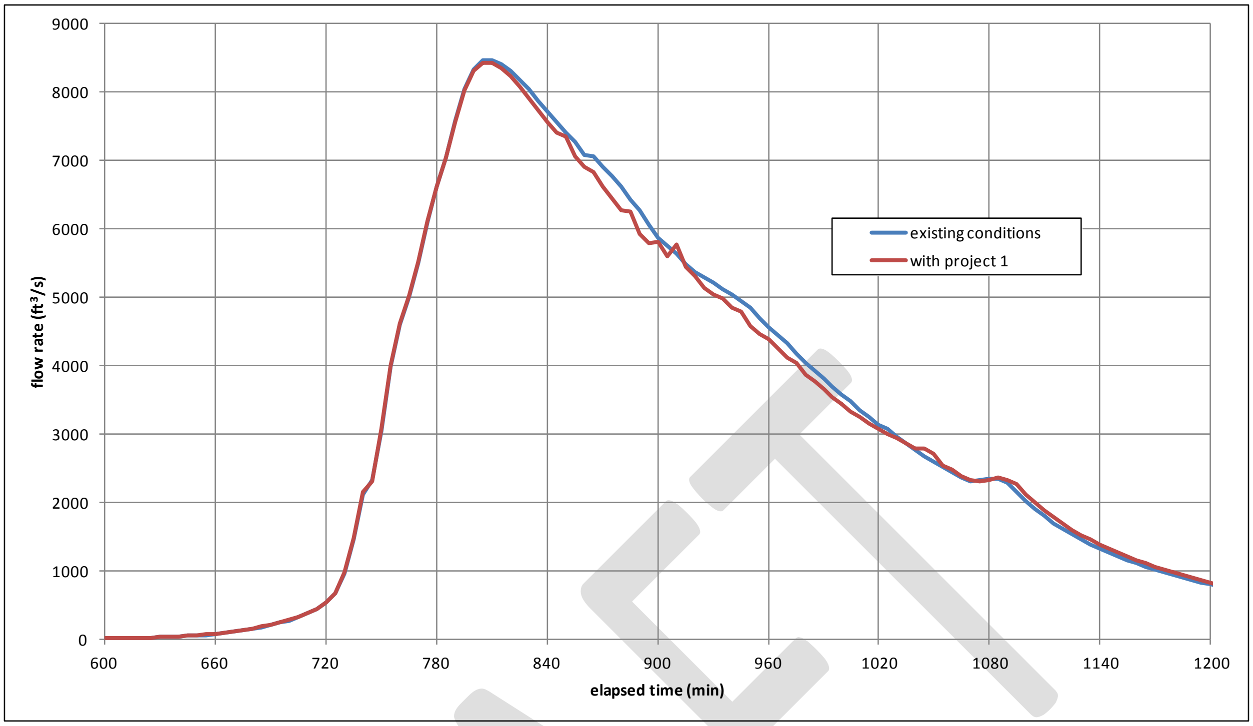

Figure 6.31 – With Project Condition D1 Results – Adams Ave Gage Location
1% ACE Event, 24-hr Duration

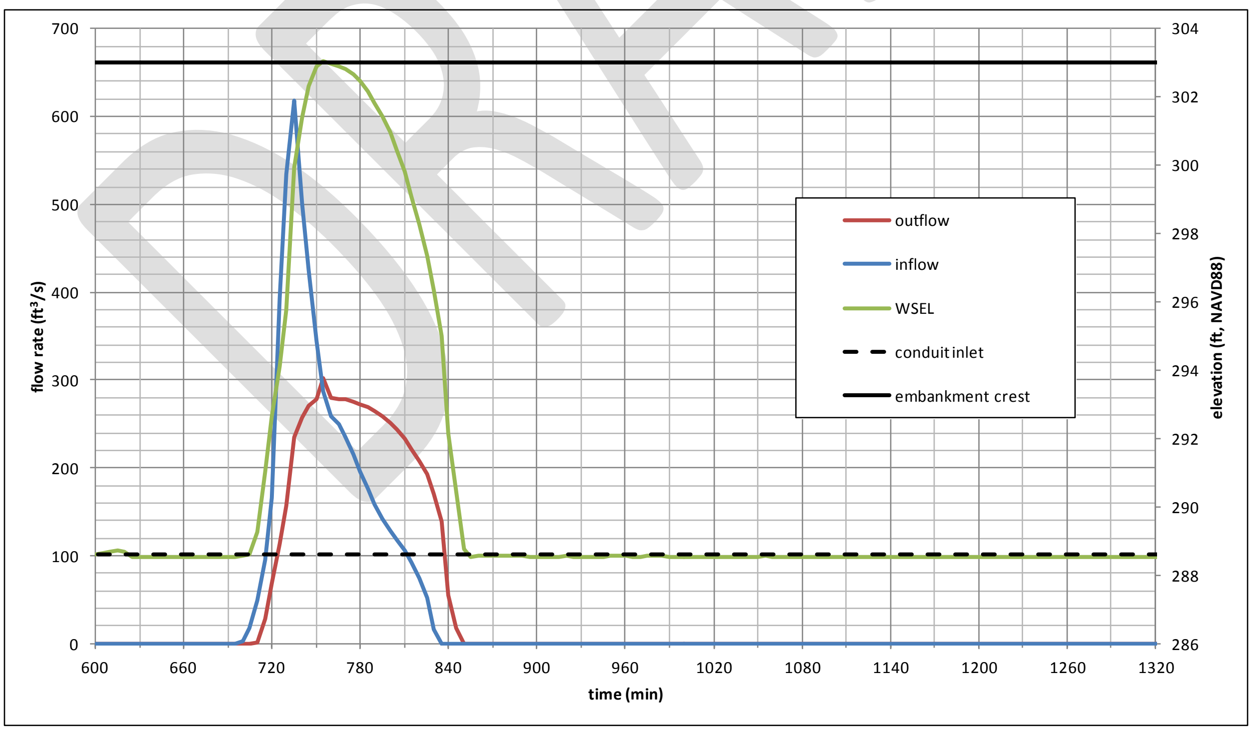

Figure 6.32 – With Project Condition D1 Results – Doe Lane Performance
1% ACE Event, 24-hr Duration

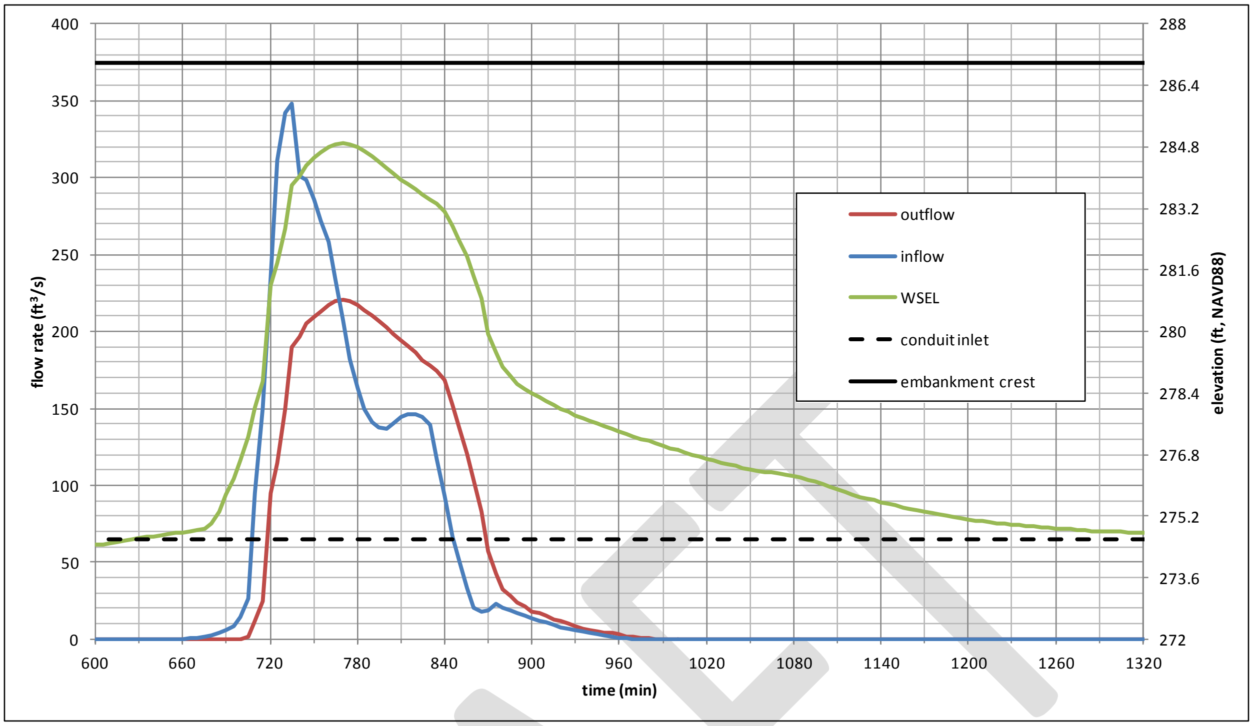

Figure 6.33 – With Project Condition D1 Results – West Waverly Road Performance
1% ACE Event, 24-hr Duration

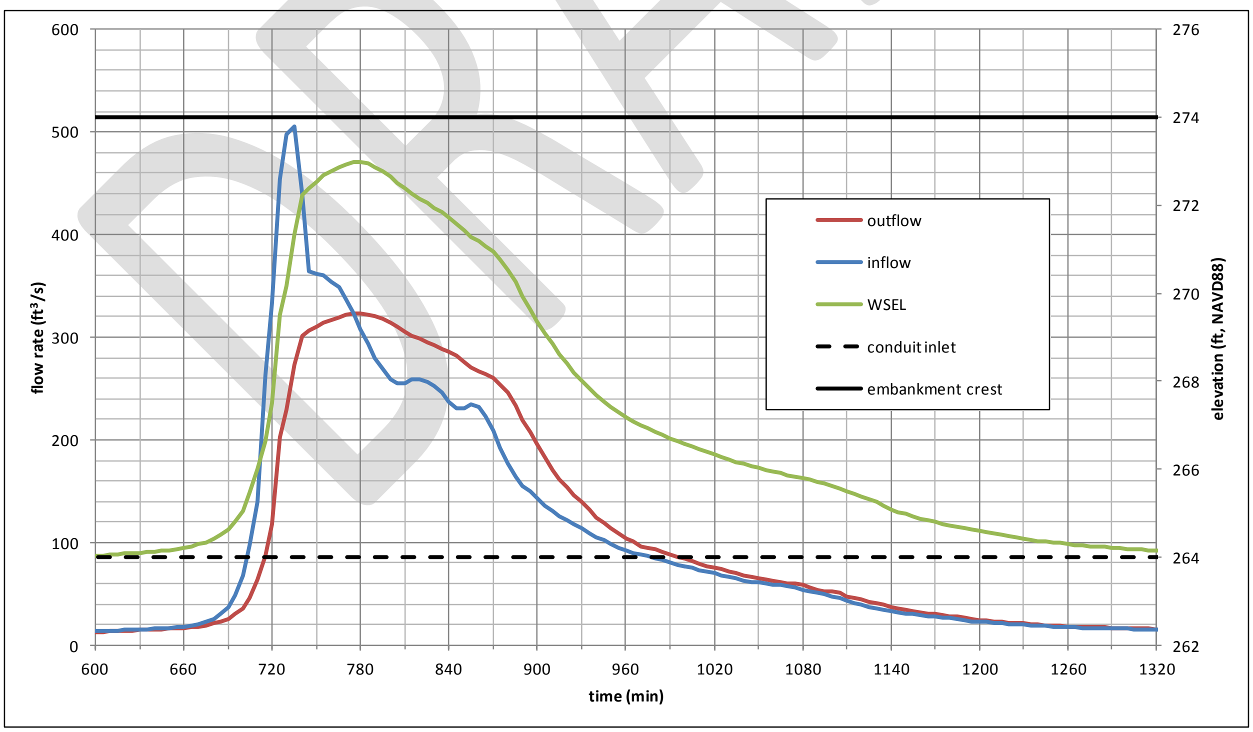

Figure 6.34 – With Project Condition D1 Results – Church Road Performance
1% ACE Event, 24-hr Duration

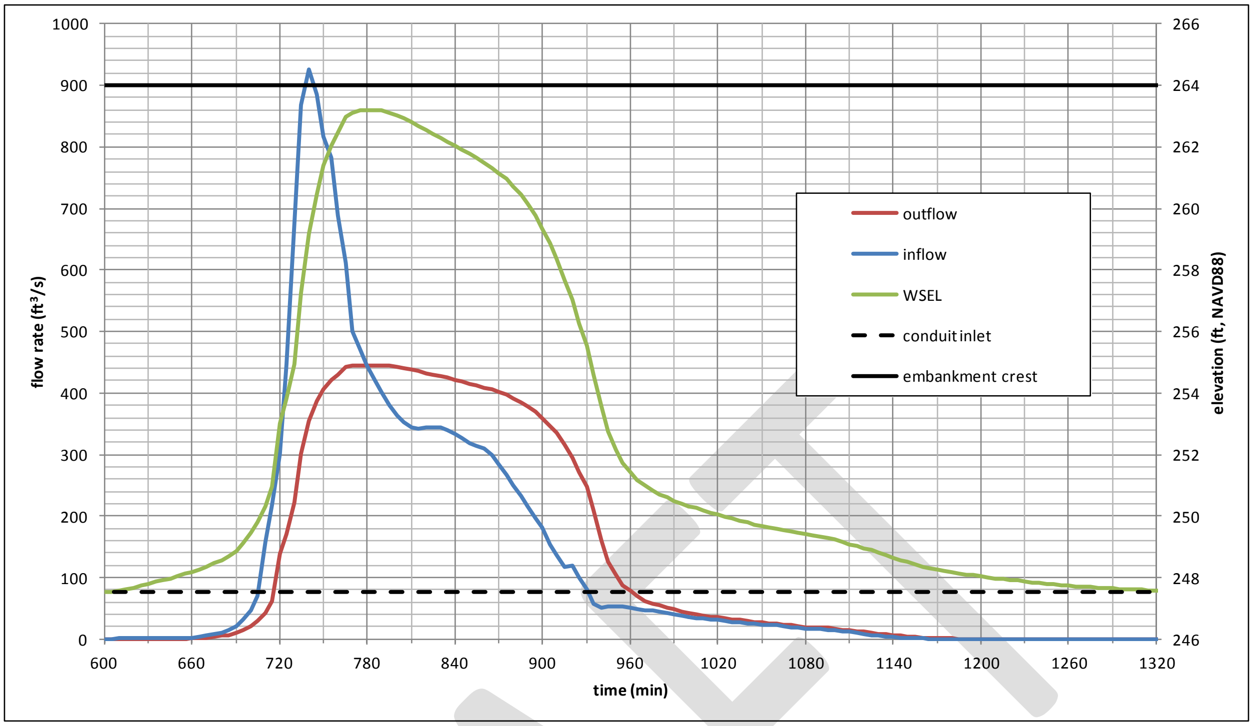

Figure 6.35 – With Project Condition D1 Results – Limekiln Pike Performance
1% ACE Event, 24-hr Duration

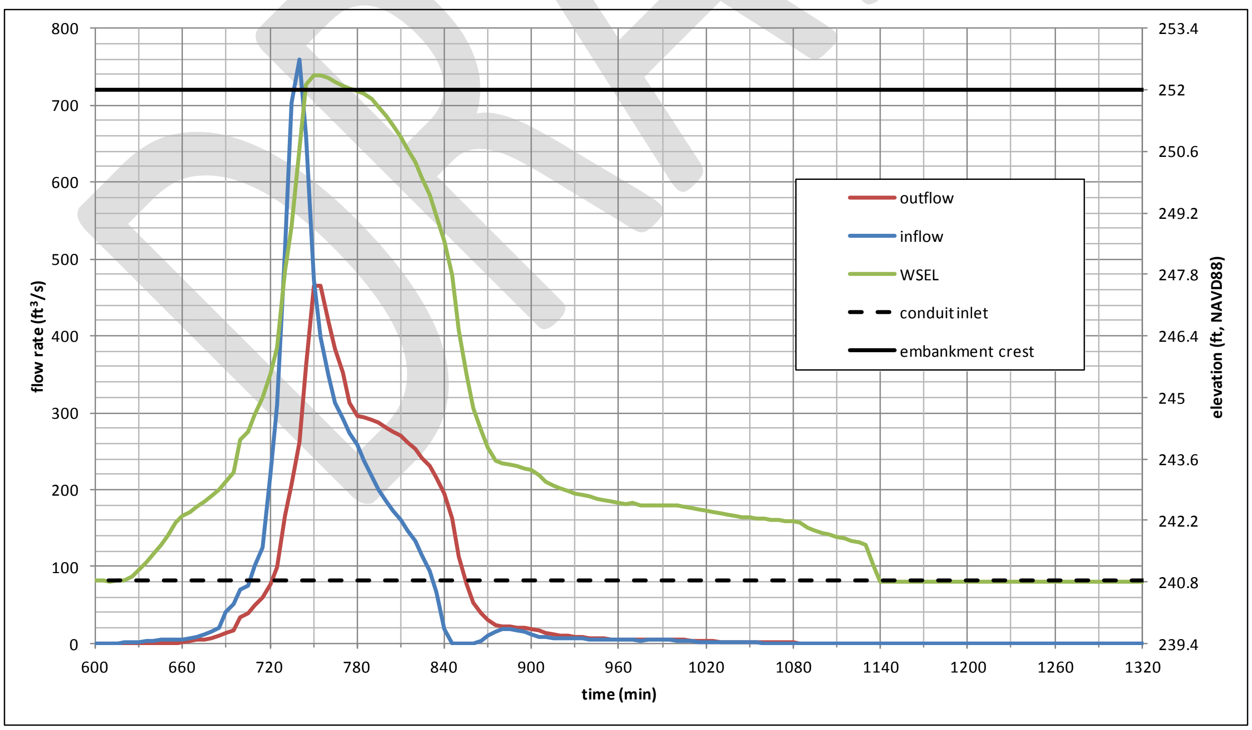

Figure 6.36 – With Project Condition D1 Results – Grove Park Performance
1% ACE Event, 24-hr Duration

Table 6.5 – With Project Condition D1 Results – Storage Area Performance
1% ACE Event, 24-hr Duration

Storage Area	peak flow rate (ft^3/s) inflow	peak flow rate (ft^3/s) outflow	peak WSEL (ft, NAVD88)
Doe Lane	618	302	303.0
West Waverly Road	348	221	284.9
Church Road	505	323	273.0
Limekiln Pike	925	446	263.2
Grove Park	759	466	252.3

Table 6.6 – With Project Condition D1 Peak Flow Rate Reductions

EVENT (ACE)	Peak Flow Reductions (ft^3/s) at location:													
	1	1A	2	3	4	4A	5	6	6A	6B	7	8	9	10
99%	-5	-7	-6	13	21	27	28	36	42	44	51	58	76	73
50%	-9	-8	-8	13	27	43	49	65	80	80	79	86	111	118
20%	-15	-16	-14	1	22	50	69	83	94	99	106	130	248	284
10%	-16	-16	-16	38	56	63	74	116	144	143	166	161	321	383
4%	59	64	71	80	78	80	92	175	219	236	285	238	515	669
2%	30	34	32	60	79	106	131	207	315	353	366	317	691	830
1%	43	32	29	87	127	165	201	240	419	472	413	371	885	1035
0.50%	65	65	64	80	97	136	170	179	307	308	441	373	1475	1109
0.20%	80	88	88	76	9	-47	-50	-107	322	578	400	399	550	1273

EVENT (ACE)	Peak Flow Reductions (%) at location:													
	1	1A	2	3	4	4A	5	6	6A	6B	7	8	9	10
99%	-0.5	-0.6	-0.6	1.3	2.7	3.7	4.1	6.4	8.2	8.8	14.7	27.3	74.9	75.0
50%	-0.5	-0.4	-0.4	0.8	2.0	3.5	4.3	7.5	10.4	10.6	16.7	26.3	49.6	53.8
20%	-0.5	-0.5	-0.5	0.0	1.1	2.5	3.9	5.9	8.1	8.7	15.8	28.4	76.5	87.7
10%	-0.4	-0.4	-0.4	1.3	2.1	2.4	3.1	6.3	9.8	9.8	20.2	29.4	78.7	93.5
4%	1.2	1.3	1.4	1.8	2.0	2.1	2.7	6.9	11.2	12.3	27.0	35.7	98.8	128.0
2%	0.4	0.5	0.5	1.1	1.6	2.3	3.1	6.7	13.6	15.6	29.0	39.1	105.1	117.5
1%	0.5	0.4	0.4	1.3	2.1	3.0	4.1	6.6	15.4	17.5	27.1	38.1	111.8	121.4
0.50%	0.6	0.6	0.6	1.0	1.3	2.1	2.9	4.1	9.3	9.2	24.3	32.1	147.8	99.4
0.20%	0.6	0.7	0.7	0.8	0.1	-0.6	-0.8	-2.1	8.4	14.0	16.6	28.3	36.4	89.8

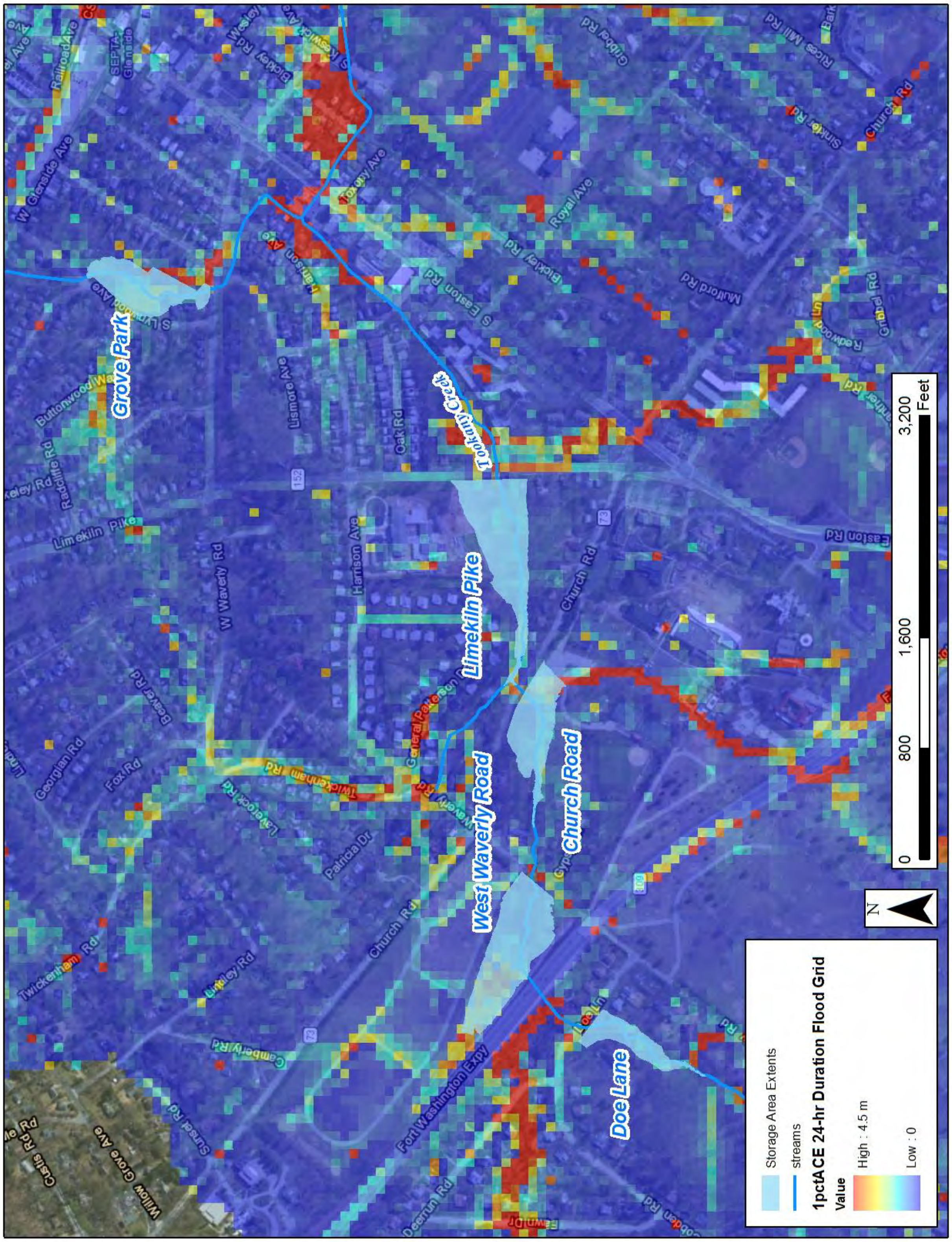

Figure 6.37 – With Project Condition D1 – Max Overland Depth
1% ACE Event, 24-hr Duration – ESRI World Imagery

6.4.2. Baederwood Creek Group

Within the following descriptions, figures, and tables, the Baederwood Creek storage area group will be called "with project condition D9". The 1% ACE event, 24-hr duration streamflow hydrographs at several locations throughout the impacted area are shown in Figures 6.38 – 6.41 while the three storage area inflows, outflows, and pool elevations are shown in Figures 6.42 – 6.44. Change in storage and outflow relationships were used to calculate inflow and then smoothed using a 3-hr centered moving average scheme. The "performance" for each storage area (peak inflow rate, peak outflow rate, and peak WSEL) for the 1% ACE event, 24-hr duration is shown in Table 6.7.

During this event, inflows cause the pools to rise within the Highland West, Highland East, and Baeder Road storage areas and exceed flood storage for approximately 20 minutes, 40 minutes, and 80 minutes, respectively. However, the Highland West storage area reduces peak inflows by approximately 22% while the Highland East storage area reduces peak inflows by approximately 17%. Uncontrolled runoff enters Baederwood Creek and joins outflows from the two Highland storage areas and enters the Baeder Road storage area. Peak inflows are reduced by approximately 5% within this storage area.

Immediately downstream of the Baeder Road storage area, Baederwood Creek enters an existing culvert which passes underneath Madison Manor Apartments. Due to restrictive culvert dimensions, excessive flows can exceed the culvert capacity and inflict substantial flooding damages. These three storage areas act to reduce peak flows (during the 1% ACE, 24-hr duration event) by approximately 25% at this culvert inlet. While WSEL, damages, and benefits were not explicitly calculated along Baederwood Creek during this analysis, it is believed that this reduction in peak flow rates due to with project condition D15 would result in substantial benefits to residences, business, and other property along Baederwood Creek.

Downstream of the Baederwood Creek confluence with the main stem Tookany Creek, minor peak flow rate reductions are predicted due to changes in flood wave timing. Upstream of the Rock Creek confluence, peak flow rate reductions of approximately 6% are predicted to occur during the 1% ACE, 24-hr duration event with this with project condition. However, peak flow rate reductions become negligible below this confluence due to large, intervening, uncontrolled drainage areas. With project condition and existing condition flow rates are essentially the same at the Adams Ave gage location. The decreases in peak flow rates due to this with project condition are shown in Table 6.8.

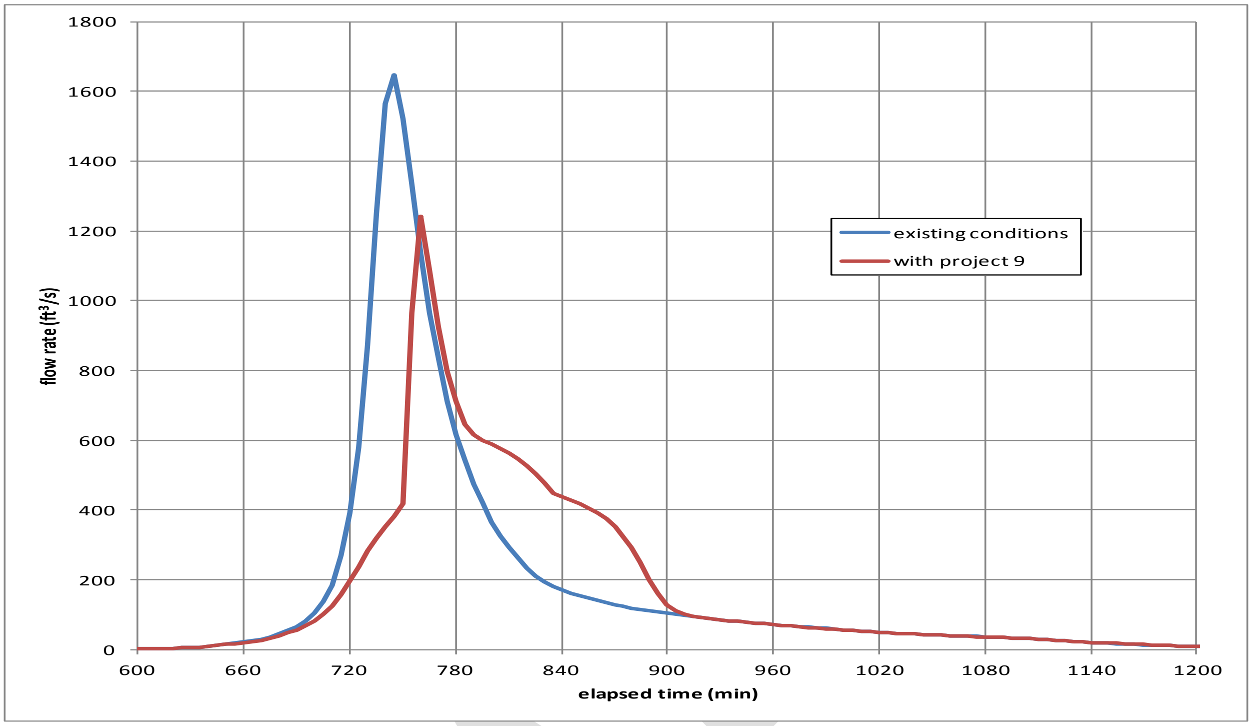

Figure 6.38 – With Project Condition D9 Results – At Baeder Road
1% ACE Event, 24-hr Duration

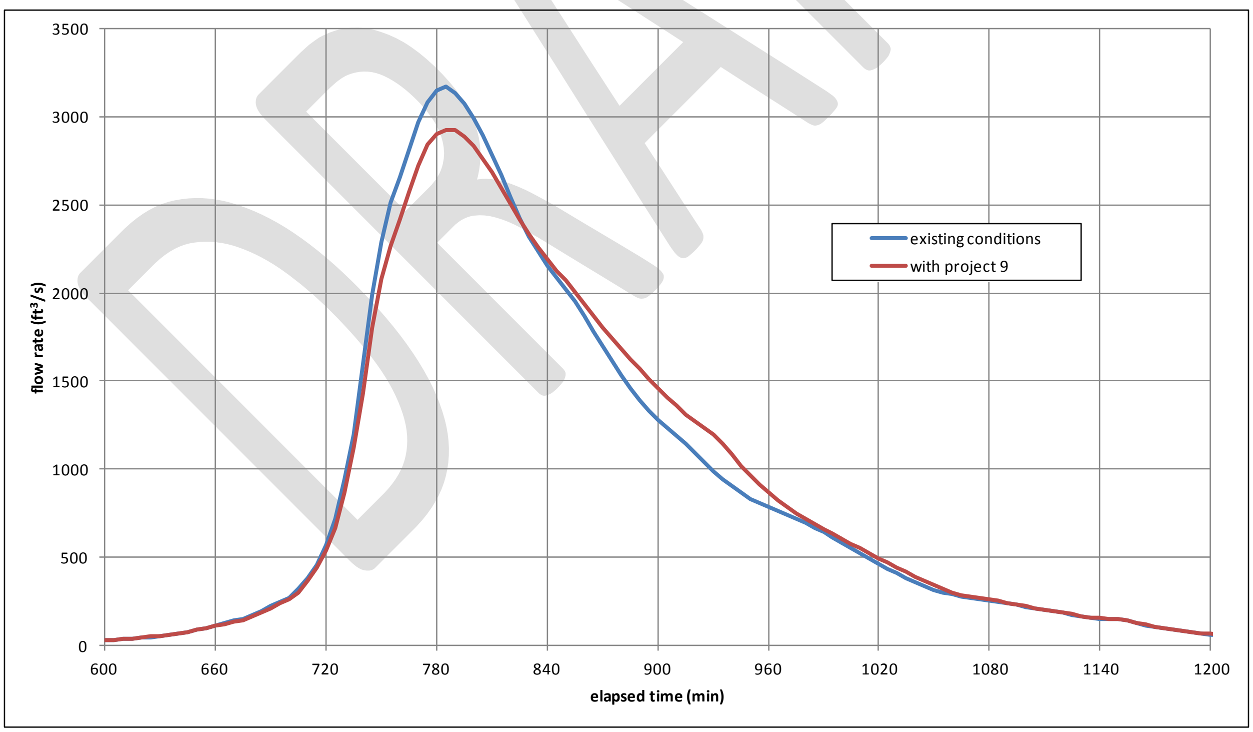

Figure 6.39 – With Project Condition D9 Results – Downstream of Baederwood Creek Confluence
1% ACE Event, 24-hr Duration

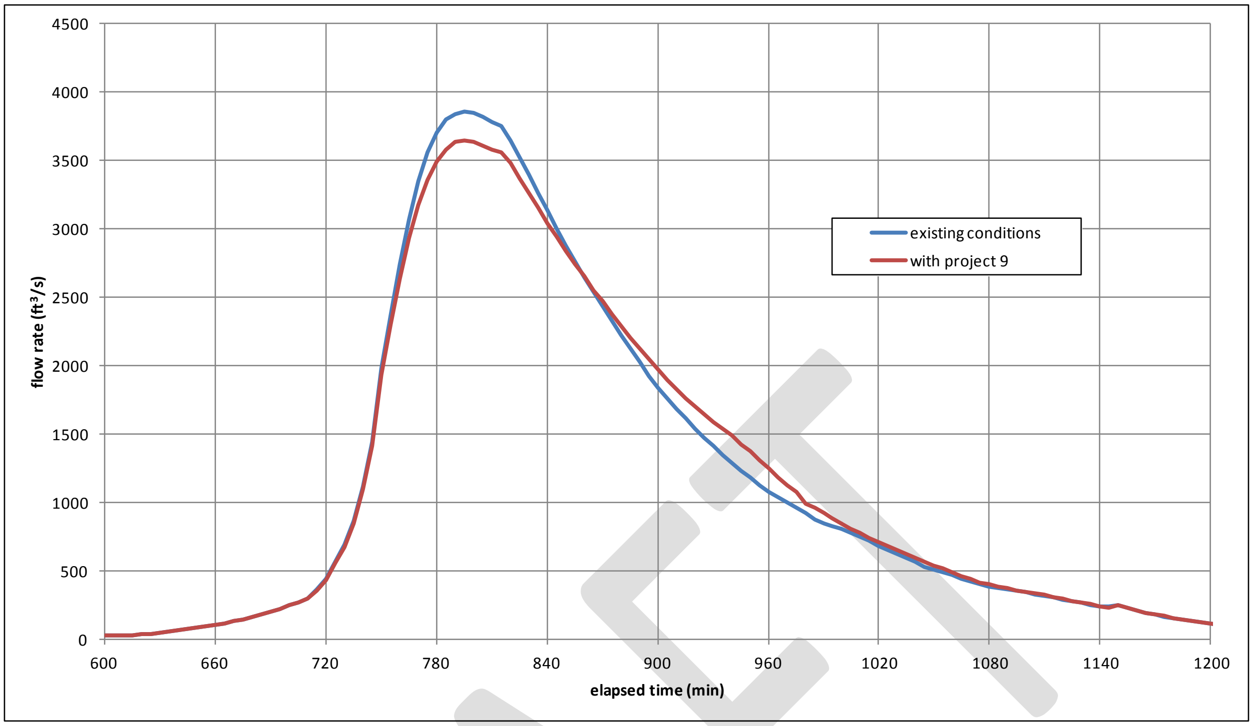

Figure 6.40 – With Project Condition D9 Results – Upstream of Rock Creek Confluence
1% ACE Event, 24-hr Duration

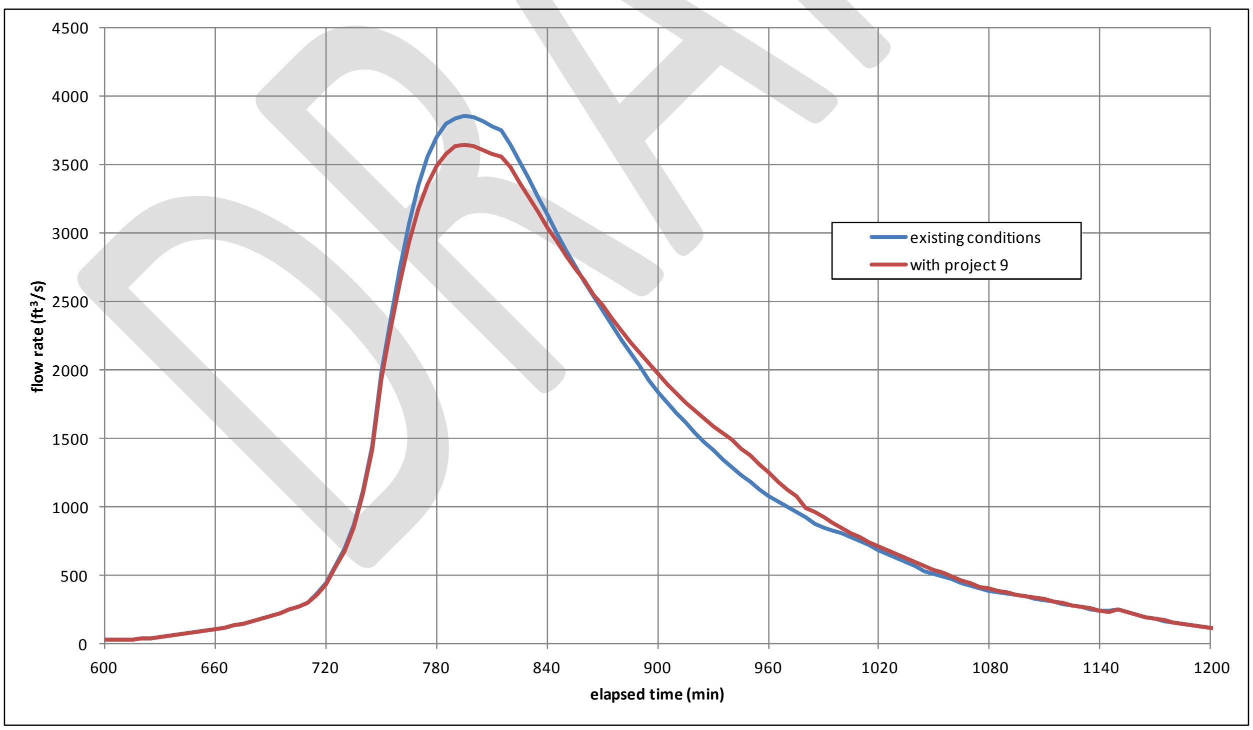

Figure 6.41 – With Project Condition D9 Results – Adams Ave Gage Location
1% ACE Event, 24-hr Duration

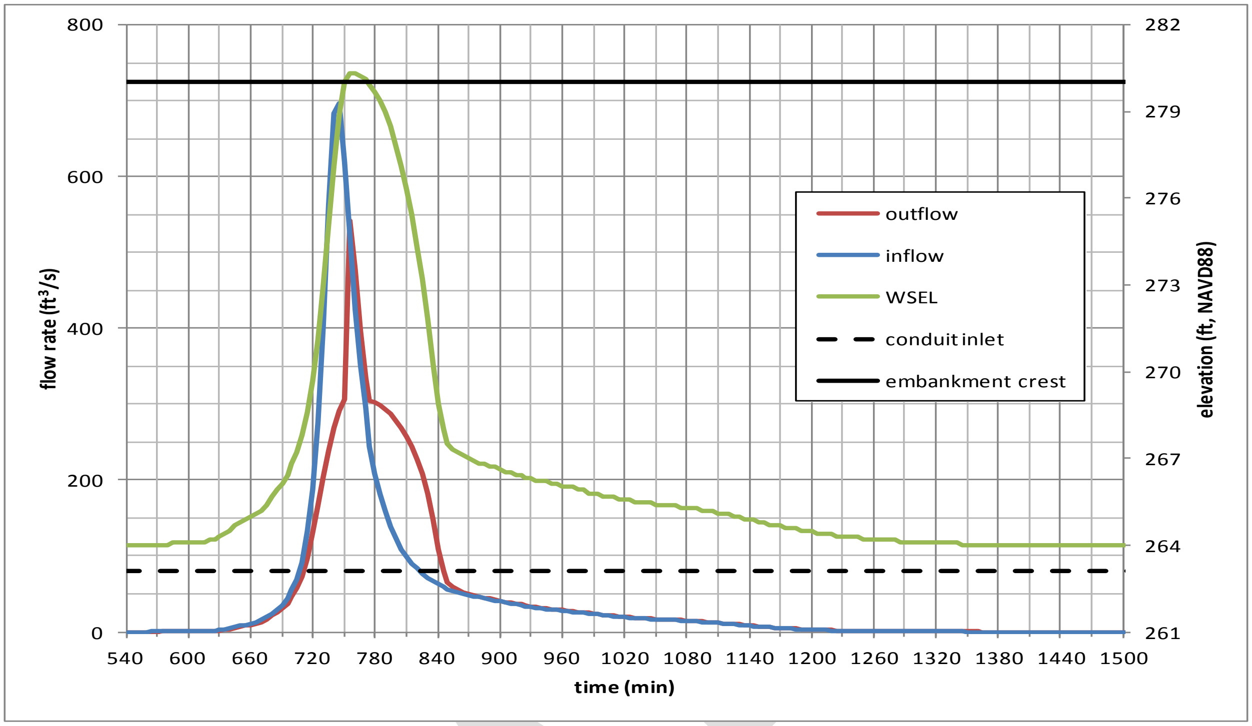

Figure 6.42 – With Project Condition D9 Results – Highland West Performance
1% ACE Event, 24-hr Duration

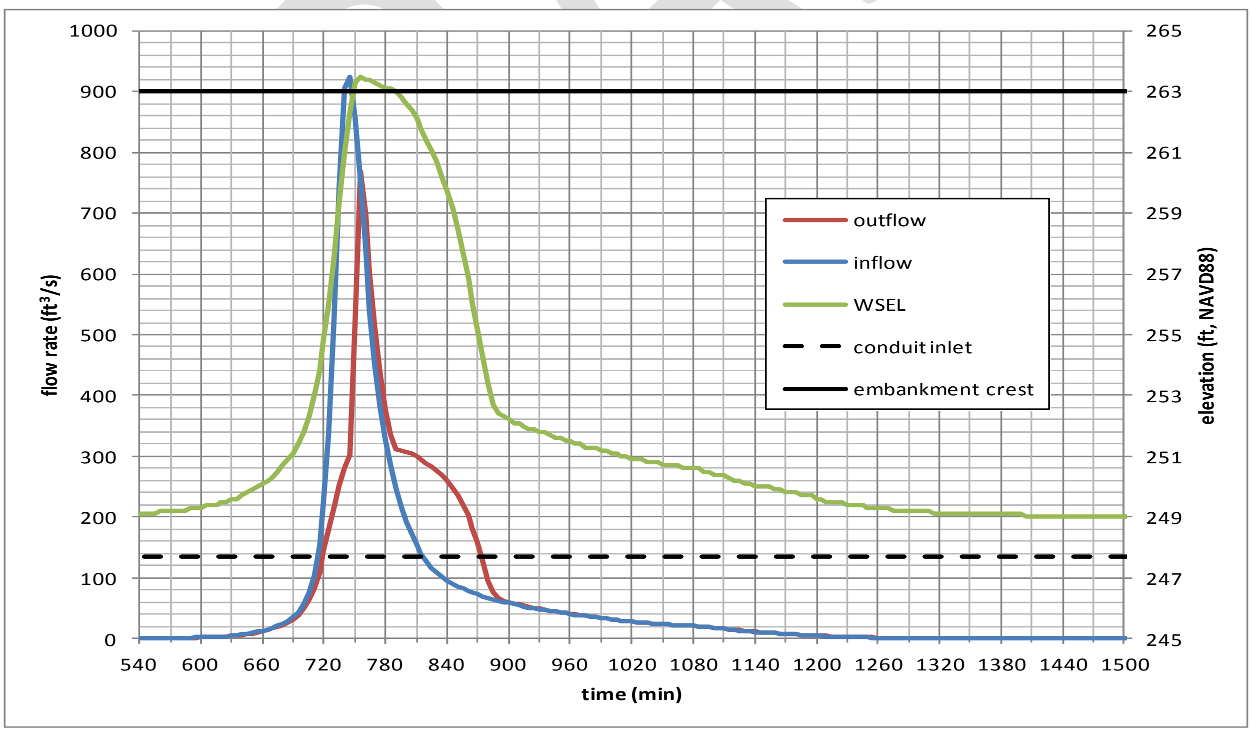

Figure 6.43 – With Project Condition D9 Results – Highland East Performance
1% ACE Event, 24-hr Duration

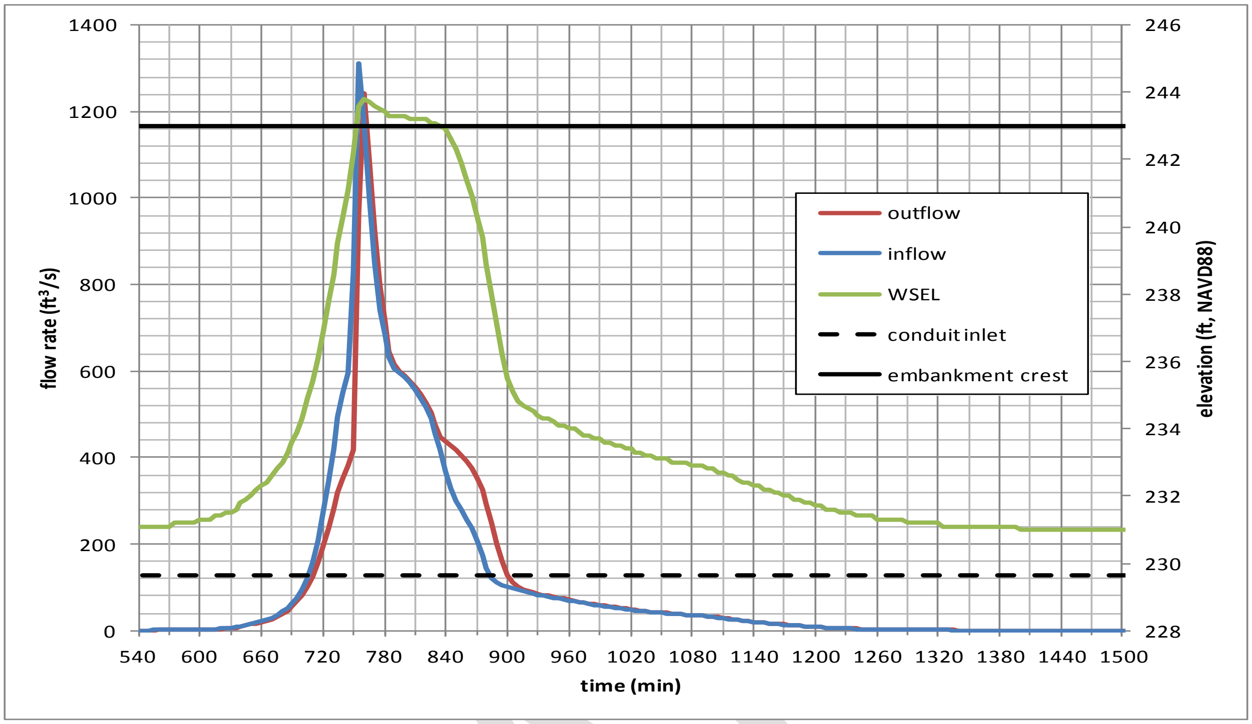

Figure 6.44 – With Project Condition D9 Results – Baeder Road Performance
1% ACE Event, 24-hr Duration

Table 6.7 – With Project Condition D9 Results – Storage Area Performance
1% ACE Event, 24-hr Duration

Storage Area	peak flow rate (ft^3/s) inflow	peak flow rate (ft^3/s) outflow	peak WSEL (ft, NAVD88)
Highland West	696	542	280.3
Highland East	923	768	263.5
Baeder Road	1311	1241	243.8

Table 6.8 – With Project Condition D9 Peak Flow Rate Reductions

EVENT (ACE)	Peak Flow Reductions (ft^3/s) at location:									
	1	**1A**	**2**	**3**	**4**	**4A**	**5**	**6**	**6A**	**6B**
99%	3	2	5	17	14	13	13	4	2	1
50%	7	9	13	24	20	20	22	6	-8	-8
20%	6	7	11	19	18	32	9	16	15	15
10%	-11	-12	-12	71	73	86	78	65	52	53
4%	146	158	213	155	150	144	144	144	123	125
2%	101	105	121	154	169	181	186	189	177	186
1%	118	103	97	180	192	212	209	213	214	240
0.50%	131	130	117	219	262	275	260	246	163	171
0.20%	114	114	102	182	138	37	-63	83	185	415

EVENT (ACE)	Peak Flow Reductions (%) at location:									
	1	**1A**	**2**	**3**	**4**	**4A**	**5**	**6**	**6A**	**6B**
99%	0.3	0.2	0.5	1.7	1.7	1.8	1.9	0.7	0.3	0.1
50%	0.4	0.5	0.7	1.5	1.5	1.6	1.9	0.6	-0.9	-0.9
20%	0.2	0.2	0.4	0.8	0.9	1.6	0.5	1.1	1.2	1.2
10%	-0.3	-0.3	-0.3	2.4	2.7	3.3	3.3	3.4	3.3	3.4
4%	2.9	3.2	4.3	3.6	3.9	3.9	4.3	5.6	6.0	6.2
2%	1.5	1.6	1.8	2.8	3.5	4.0	4.5	6.1	7.2	7.6
1%	1.4	1.2	1.2	2.8	3.3	3.9	4.3	5.8	7.3	8.2
0.50%	1.3	1.3	1.1	2.8	3.7	4.4	4.6	5.7	4.7	4.9
0.20%	0.8	0.9	0.8	1.9	1.6	0.5	-1.0	1.7	4.7	9.6

6.4.3. Rock Creek Group

Within the following descriptions, figures, and tables, the Rock Creek storage area group (Washington Lane) will be called "with project condition D15". The 1% ACE event, 24-hr duration streamflow hydrographs at several locations throughout the impacted area are shown in Figures 6.45 – 6.47 while the storage area inflow, outflow, and pool elevation for this event are shown in Figures 6.48. Change in storage and outflow relationships were used to calculate inflow and then smoothed using a 3-hr centered moving average scheme. Finally, the storage area "performance" (peak inflow rate, peak outflow rate, and peak WSEL) for the 1% ACE event, 24-hr duration are shown in Table 6.9.

Peak inflow rates are reduced by approximately 50% within the Washington Lane storage area during this event while excess inflows overtopped the embankment for approximately 30 minutes. Peak flow rates (as compared to existing conditions) near the Rock Creek culvert inlet are reduced by approximately 50%. On the main stem Tookany Creek, peak flow rate reductions are not as drastic due to the large uncontrolled drainage area upstream. However, due to flood storage within the Washington Lane storage area, flood wave timing from Rock Creek is offset against the main stem Tookany Creek leading to noticeable peak flow rate reductions of approximately 7.5% and 6% at the Shoemaker Road neighborhood and Adams Ave gage location, respectively. The decreases in peak flow rates throughout the impacted area due to this with project condition are shown in Table 6.10.

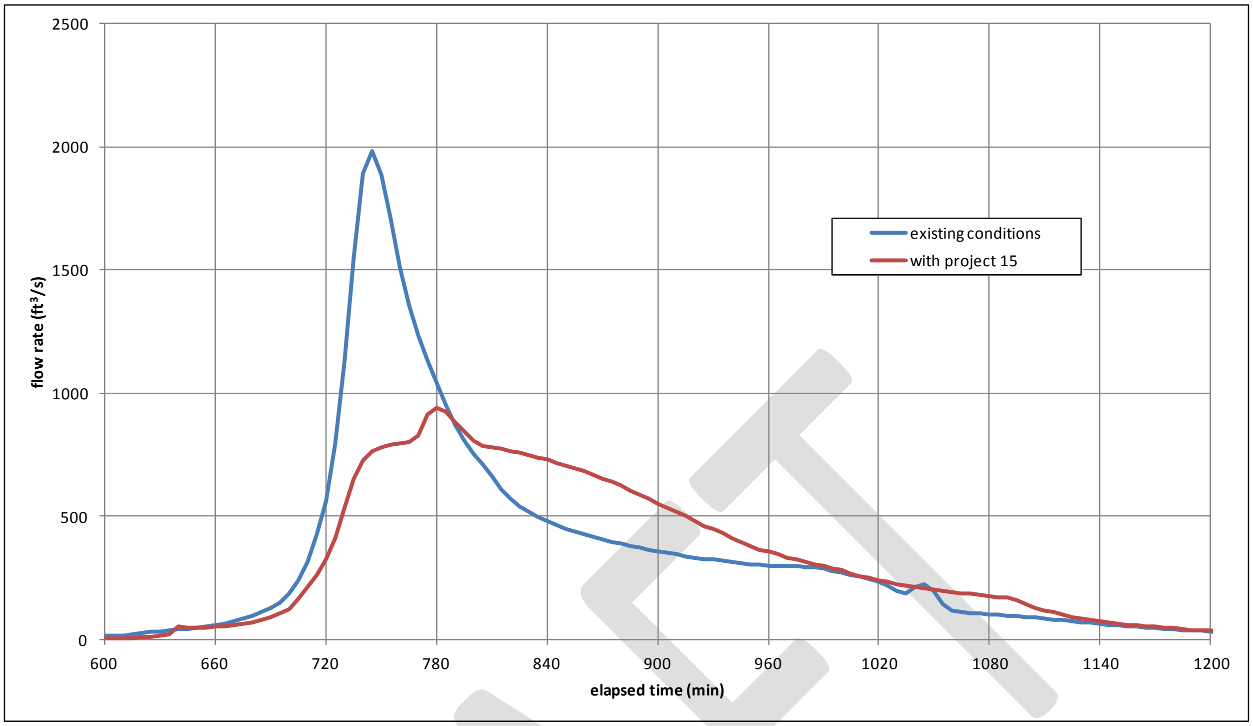

Figure 6.45 – With Project Condition D15 Results – Near Rock Creek Culvert Inlet
1% ACE Event, 24-hr Duration

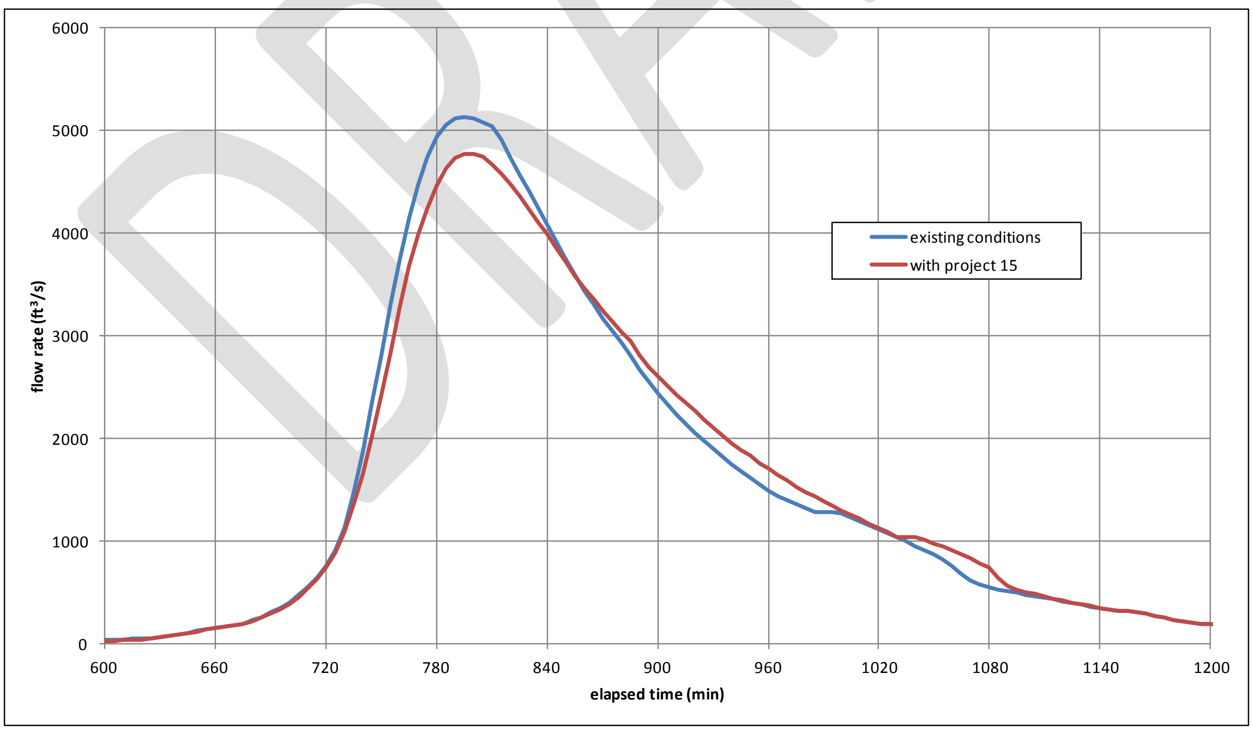

Figure 6.46 – With Project Condition D15 Results – Near Shoemaker Road Neighborhood
1% ACE Event, 24-hr Duration

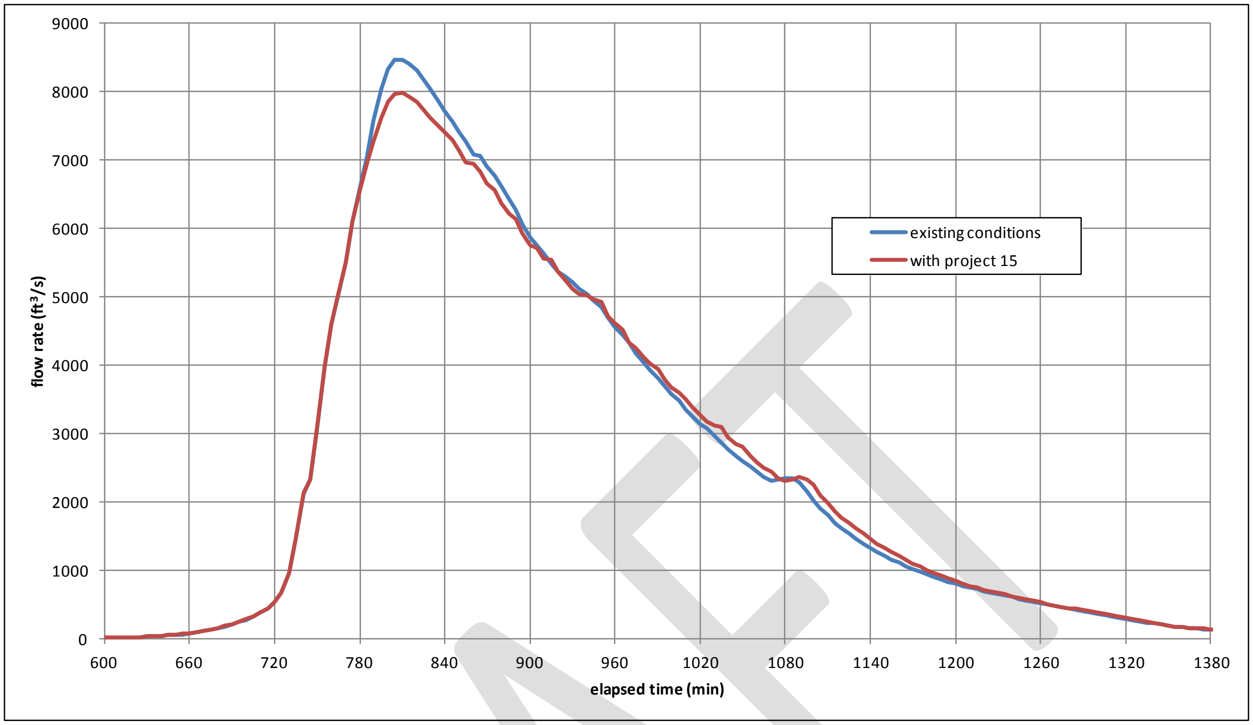

Figure 6.47 – With Project Condition D15 Results – Adams Ave Gage Location
1% ACE Event, 24-hr Duration

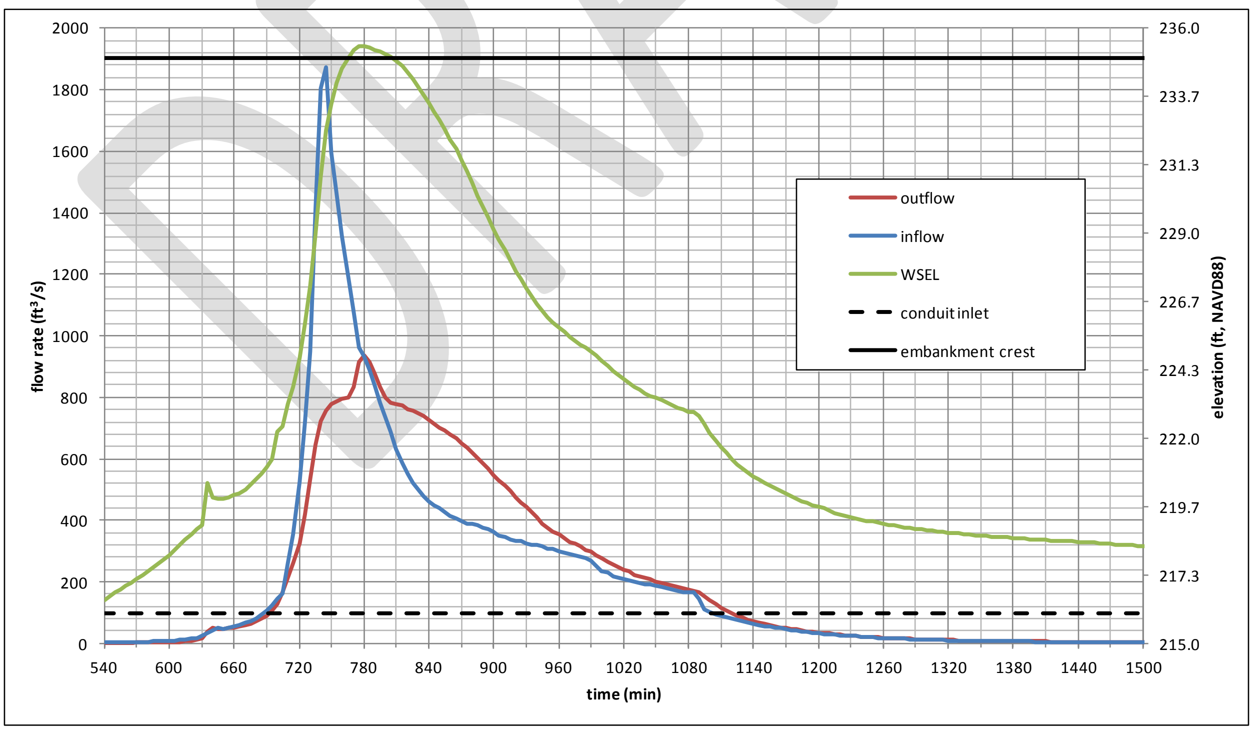

Figure 6.48 – With Project Condition D15 Results – Washington Lane Performance
1% ACE Event, 24-hr Duration

Table 6.9 – With Project Condition D15 Results – Washington Lane Performance
1% ACE Event, 24-hr Duration

Storage Area	peak flow rate (ft^3/s) inflow	peak flow rate (ft^3/s) outflow	peak WSEL (ft, NAVD88)
Washington Lane	1873	936	235.4

Table 6.10 – With Project Condition D15 Peak Flow Rate Reductions

EVENT (ACE)	Peak Flow Reductions (ft^3/s) at location:						
	1	1A	2	3	4	4A	5
99%	-31	-34	-38	-45	-49	-50	-49
50%	7	8	11	-6	-26	-33	-40
20%	71	79	91	46	-5	-53	-55
10%	34	34	34	117	65	29	3
4%	404	417	474	361	278	220	173
2%	537	547	615	485	444	394	350
1%	492	492	515	442	414	374	355
0.50%	443	453	487	346	290	232	225
0.20%	384	386	428	350	299	266	279

EVENT (ACE)	Peak Flow Reductions (%) at location:						
	1	1A	2	3	4	4A	5
99%	-2.7	-3.0	-3.2	-4.4	-5.7	-6.2	-6.4
50%	0.4	0.4	0.5	-0.4	-1.8	-2.5	-3.3
20%	2.5	2.7	3.2	1.9	-0.2	-2.6	-2.9
10%	0.9	0.9	0.9	4.0	2.4	1.1	0.1
4%	8.5	8.8	10.1	8.8	7.5	6.1	5.3
2%	8.5	8.8	9.9	9.5	9.7	9.1	8.8
1%	6.2	6.2	6.6	7.1	7.3	7.1	7.4
0.50%	4.4	4.5	4.9	4.5	4.1	3.6	3.9
0.20%	2.9	3.0	3.3	3.6	3.5	3.7	4.5

6.4.4. All-Encompassing Group

Within the following descriptions, figures, and tables, the all-encompassing storage area grouping will be called "with project condition D27". The 1% ACE event, 24-hr duration streamflow hydrographs at several locations throughout the impacted area are shown in Figures 6.49 – 6.52. Storage area inflows, outflows, pool elevations, and "performances" are unchanged from previous figures and tables and are thus not shown again.

Peak flow rates (as compared to existing conditions for the 1% ACE, 24-hr duration event) near the Cliff Terrace neighborhood and Shoemaker Road neighborhood are reduced by approximately 25% and 20%, respectively. Reductions in peak flow rates become less dramatic from this point downstream due to the influence of the increasingly large uncontrolled drainage area. However, there is still approximately an 8% decrease in peak flow rate at the Adams Ave gage location for this event. The decreases in peak flow rates due to this with project condition are shown in Table 6.11.

Generally speaking, when simulated individually, all of the embankments were first overtopped during the 2% ACE, 24-hr duration event or the 1% ACE, 24-hr duration event. However, flood waves were still attenuated and translated for events where flood storage was exceeded. Also, when grouped together, the most upstream storage areas in each group may exceed flood storage during a particular event, but this action commonly prevents downstream storage areas from exceeding flood storage, thus greatly reducing peak flow rates at critical damage locations. Finally, in general, all flood storage from each storage area is evacuated (i.e. storage areas empty) 12-18 hours after the peak pool is achieved.

The all-encompassing storage area group (with project condition D27) reduces peak flow rates and flood damages to a greater degree and extent than any other with project condition.

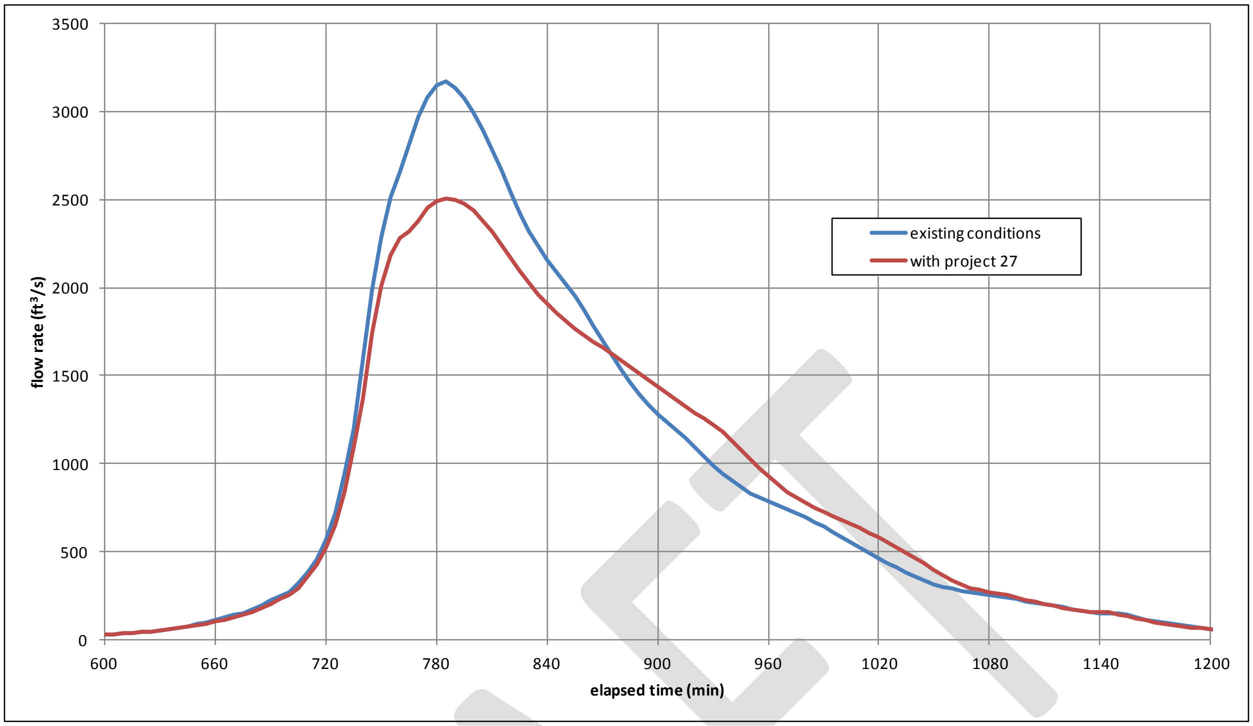

Figure 6.49 – With Project Condition D27 Results – Downstream of Baederwood Creek Confluence
1% ACE Event, 24-hr Duration

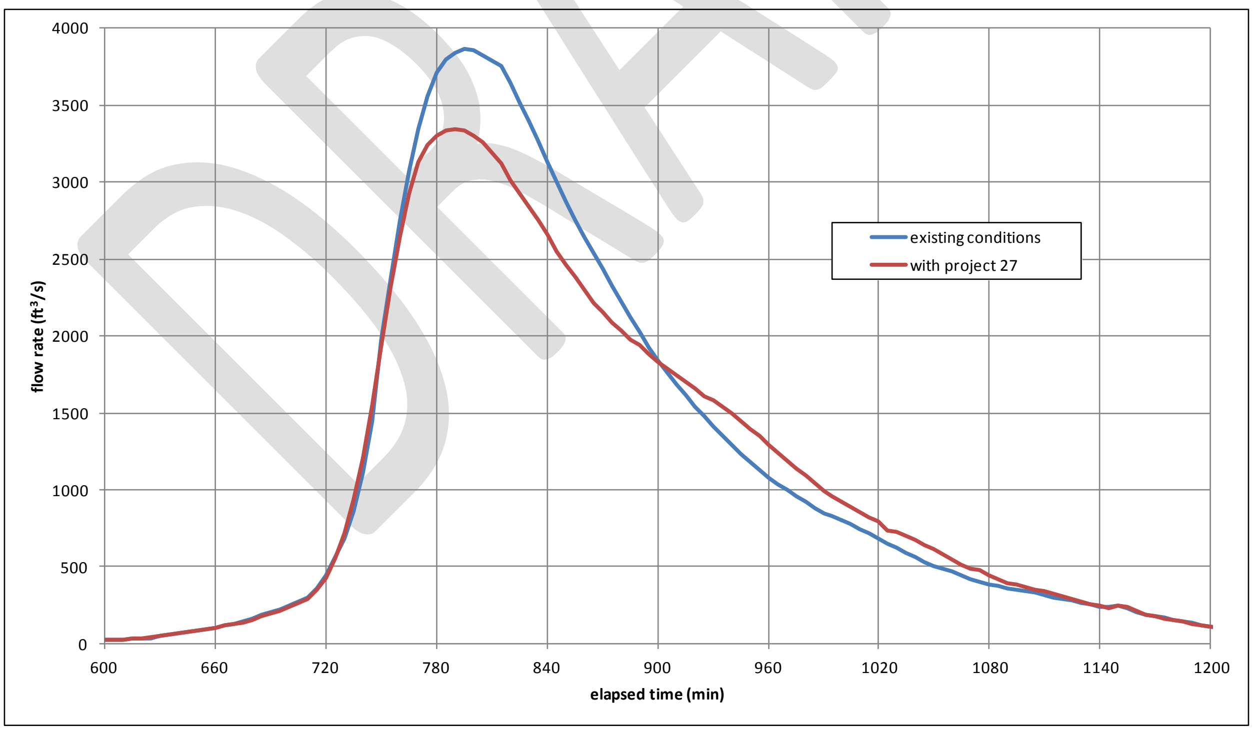

Figure 6.50 – With Project Condition D27 Results – Upstream of Rock Creek Confluence
1% ACE Event, 24-hr Duration

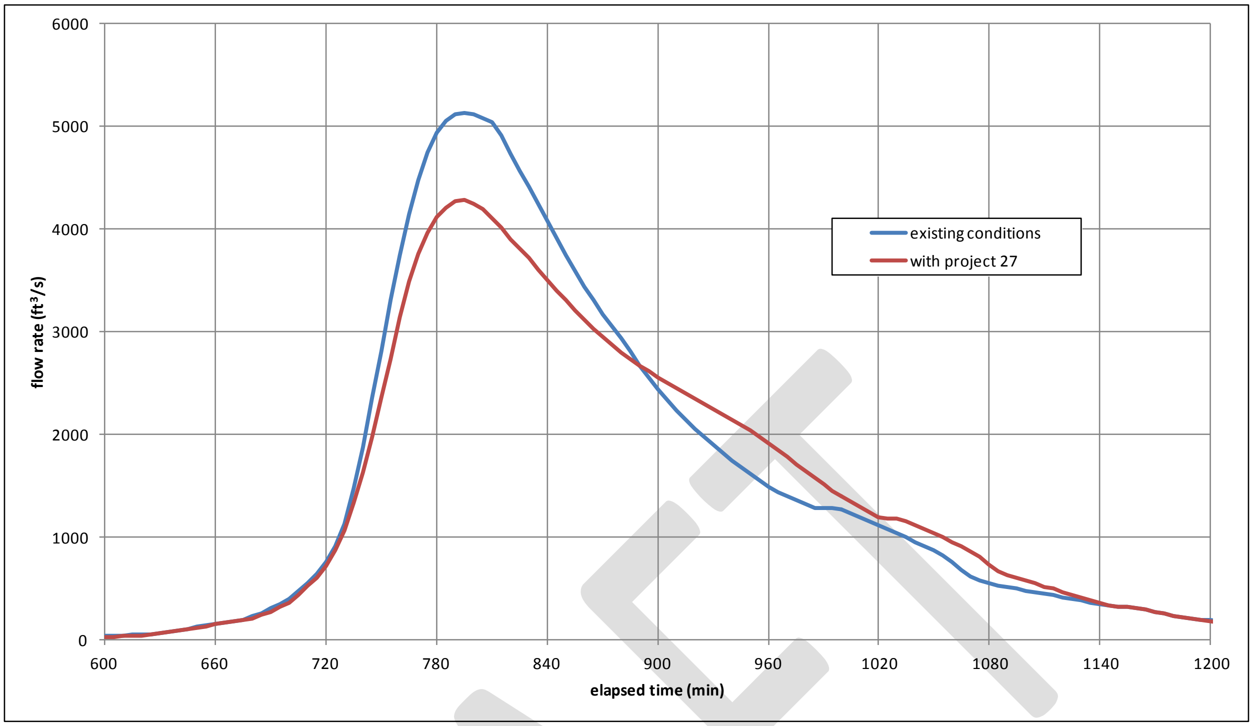

Figure 6.51 – With Project Condition D27 Results – Near Shoemaker Road Neighborhood
1% ACE Event, 24-hr Duration

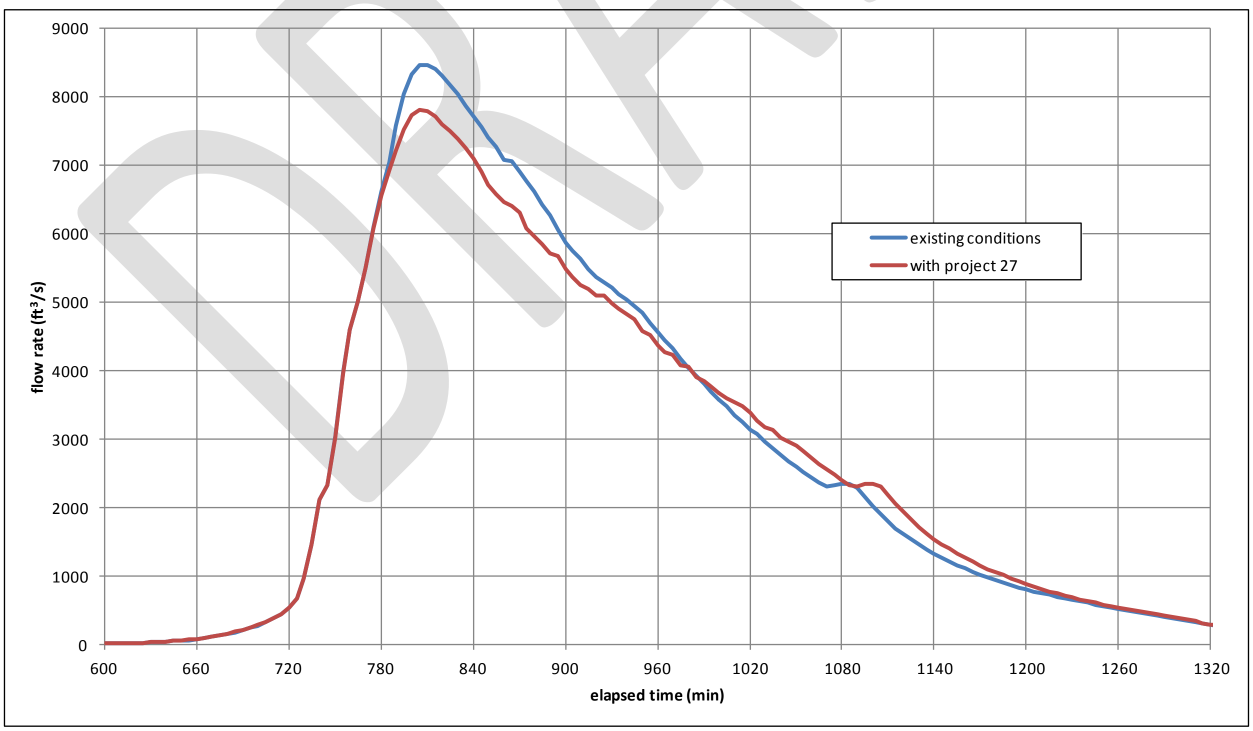

Figure 6.52 – With Project Condition D27 Results – Adams Ave Gage Location
1% ACE Event, 24-hr Duration

Table 6.11 – With Project Condition D27 Peak Flow Rate Reductions

EVENT (ACE)	Peak Flow Reductions (ft^3/s) at location:										
	1	1A	2	3	4	4A	5	6	6A	6B	7
99%	19	18	17	21	21	21	22	54	58	59	52
50%	65	67	70	79	70	68	63	86	88	88	80
20%	128	135	115	135	105	108	108	137	139	141	116
10%	42	41	41	266	261	255	236	248	251	250	172
4%	521	548	633	647	596	522	535	410	396	410	300
2%	740	760	850	810	795	784	779	488	510	542	381
1%	655	649	665	777	799	849	853	518	605	662	409
0.50%	636	645	663	723	738	764	776	532	567	605	458
0.20%	565	577	598	618	533	353	204	81	447	871	435

EVENT (ACE)	Peak Flow Reductions (%) at location:										
	1	1A	2	3	4	4A	5	6	6A	6B	7
99%	1.7	1.6	1.6	2.3	2.6	2.8	3.2	9.9	11.7	12.2	15.0
50%	3.4	3.5	3.7	5.2	5.4	5.6	5.6	10.0	11.6	11.7	17.0
20%	4.5	4.8	4.1	6.0	5.3	5.7	6.2	10.2	12.5	12.9	17.6
10%	1.1	1.1	1.2	9.6	10.3	10.3	10.5	14.4	18.4	18.5	21.1
4%	11.2	11.9	14.0	17.0	17.5	15.9	18.2	17.8	22.3	23.5	28.8
2%	12.1	12.6	14.3	16.9	18.8	19.8	22.1	17.3	24.1	26.1	30.6
1%	8.4	8.4	8.6	13.2	15.2	17.7	19.9	15.5	23.9	26.4	26.8
0.50%	6.4	6.6	6.8	9.8	11.3	13.1	15.0	13.3	18.7	19.8	25.5
0.20%	4.4	4.5	4.7	6.6	6.4	5.0	3.3	1.7	12.0	22.7	18.4

7. Conclusions

A hydrologic analysis for the Tookany Creek watershed within Cheltenham Township was conducted to investigate potential project alternatives in an attempt to lessen damages due to flooding. A conceptual model of the Tookany Creek watershed upstream of the Cheltenham / Philadelphia boundary was constructed to better understand the flow of water within the area of interest. The Gridded Surface Subsurface Hydrologic Analysis (GSSHA) modeling code was used to quantitatively analyze existing conditions as well as various with project conditions. The GSSHA model was built using various up-to-date, high-resolution datasets and calibrated to two recent, high flow events that occurred in August and September 2011. The model was then validated using an independent dataset from June 2006.

A statistical flow-frequency analysis was conducted using the United States Geological Survey streamflow gaging station at Adams Ave. The results of this statistical analysis were compared against results obtained using hypothetical precipitation events. Multiple hypothetical precipitation events were created using point precipitation estimates from the National Oceanic and Atmospheric Administration's Atlas 14 publication. Numerous events were created from frequently occurring, low intensity rain storms to high intensity, rarely occurring rain storms. Varying precipitation durations were also analyzed from 3-hrs to 24-hrs.

These hypothetical precipitation events were used to compare the effects of various with project conditions against existing conditions in order to determine their flood risk reduction benefits. Flow rates were passed onto an HEC-RAS hydraulic model that was used to estimate water surface elevations for input to an economic analysis, both of which are detailed in additional technical appendices.

With project conditions that were analyzed for their hydrologic impacts included Low Impact Development options, constriction removals, local neighborhood flood walls, and storage areas. Storage areas were found to reduce peak flow rates (and in turn WSELs and flooding damages) to greater magnitudes and extents than any other option that was analyzed. Nine individual storage areas were simulated in addition to three different storage area groupings. A final, all-encompassing storage area grouping of all nine was also analyzed.

Due to the large reductions in peak flow rates predicted to occur due to the all-encompassing storage area grouping, this plan is recommended as the Tentatively Selected Plan.

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