HYDROLOGIC ANALYSIS OF PROMPTON DAM USING A PHYSICALLY-BASED RAINFALL RUNOFF MODEL

Wayne County, PA
Lackawaxen River Watershed

By

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STATEMENT BY AUTHOR

This thesis has been submitted in partial fulfillment of requirements for an advanced degree at the Villanova University.

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Thank you.

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Terms of Reference

ACE	Annual Chance Exceedence
AOI	Area of Interest
ArcGIS	ESRI Geographic Information System
CFS	Cubic Feet per Second
CHL	ERDC Coastal and Hydraulics Laboratory
DAD	Depth-Area-Duration
DCNR	Pennsylvania Department of Conservation and Natural Resources
DEM	Digital Elevation Model
DRBC	Delaware River Basin Commission
ER	USACE Engineer Regulation
ERDC	USACE Engineer Research and Development Center
EROS	USGS Earth Resources Observation and Science Center
ESRI	Environmental Systems Research Institute
GIS	Geographic Information System
GSSHA	Gridded Surface Subsurface Hydrologic Analysis
HEC	Hydrologic Engineering Center
HEC-1	HEC Flood Hydrograph Package
HEC-GeoHMS	HEC Geospatial Hydrologic Model System extension
HEC-GeoRAS	HEC Geospatial River Analysis System extension
HEC-HMS	HEC Hydrologic Modeling System
HEC-RAS	HEC River Analysis System
HMR	Hydrometeorological Report
IDF	Inflow Design Flood
LIDAR	Light Detection and Ranging
LSOR	Line Successive Over Relaxation
MPE	Multisensor Precipitation Estimator
NAD83	North American Datum of 1983
NAP	USACE - Philadelphia District
NAVD88	North American Vertical Datum of 1988
NCDC	National Climatic Data Center
NED	National Elevation Dataset

NEXRAD	Next-Generation Radar
NGVD29	National Geodetic Vertical Datum of 1929
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
NY	New York
PA	Pennsylvania
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PMS	Probable Maximum Storm
SDF	Spillway Design Flood
SHG	Standard Hydrologic Grid
SPF	Standard Project Flood
SSURGO	Soil Survey Geographic Database
TREX	Two-dimensional Runoff, Erosion, and Export
UH	Unit Hydrograph
USACE	U.S. Army Corps of Engineers
USAF	U.S. Air Force
USGS	U.S. Geological Survey
WMS	Watershed Modeling System
WSEL	Water Surface Elevation

Abstract

When a dam impounds water upstream of a populated area, a distinct hazard to that area from a possible failure of the dam is created.[1] As such, extreme care is required in every phase of the engineering design, construction, and operation of dams. The dam design and operation should allow for the safe passage of an Inflow Design Flood (IDF) with adequate freeboard. Within the U.S. Army Corps of Engineers (USACE), which manages a portfolio of over 650 dams, the IDF is commonly comprised of the Probable Maximum Flood (PMF) and varying antecedent conditions.

The Gridded Surface Subsurface Hydrologic Analysis (GSSHA) rainfall runoff modeling code was used to estimate the IDF for Prompton Dam, which was designed, owned, and is currently operated by the USACE, Philadelphia District for the primary purpose of flood risk reduction. Following model calibration and validation efforts, this model was used to ascertain the accuracy, effectiveness, and implications of various hydrologic routines, simplifications, and assumptions commonly employed when simulating extreme events. GSSHA uses hydrologic routines that are not available within most commonly employed hydrologic modeling codes which frequently rely on empirical relationships. These physically-based routines can provide for a more defensible prediction of runoff conditions. This is especially helpful when a user is simulating extreme events for use in dam safety studies when there frequently is not any applicable observed data available for comparison that is equally extreme in nature.

The USACE Hydrologic Engineering Center (HEC) developed an HEC-1 model in 1988 during a dam modification study (*1988 HEC Prompton Modification Study*) that analyzed a potential storage reallocation for water supply that would occupy available flood storage. When

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[1] (U.S. Army Corps of Engineers, 1991)

simulating the IDF, a hydrologic deficiency of the dam was found to exist. In particular, the dam was predicted to overtop by approximately 14 feet if overtopping flows were not considered during the IDF, which would likely fail the dam with catastrophic consequences. Prompton Dam was modified from its original design by installing a parapet wall, widening the spillway and approach channel, installing an access bridge across the spillway, and relocating various facilities. These modifications were completed in 2012 at a cost of nearly $25 million.

The 1988 HEC Prompton Modification Study HEC-1 model was compared and contrasted against the GSSHA results. The hydrologic modeling routines used within the 1988 HEC Prompton Modification Study resulted in large differences in the resultant streamflow hydrographs and reservoir pool elevations when compared to the physically-based routines within GSSHA. When simulating the IDF within the GSSHA model, peak pool elevations were found to be approximately 25 feet lower than the results obtained from the 1988 HEC Prompton Modification Study. The GSSHA model predicted approximately 11.5 feet of freeboard between the static peak pool elevation and original dam crest.

The differences in peak pool elevations during the IDF event were primarily due to the larger inflow hydrograph volumes predicted by the 1988 HEC Prompton Modification Study when compared to the GSSHA model results. These larger inflow volumes were a result of the differences in input parameters, infiltration routines, runoff transform routines, and inappropriate baseflow ratio to peak values within the 1988 HEC Prompton Modification Study. However, when changes were made to the 1988 HEC Prompton Modification Study model baseflow routine to better reflect actual baseflow conditions during the IDF event, peak pool elevations were still 22 ft higher than the GSSHA model and 10.5 ft higher than the original dam crest elevation.

The time and effort required to successfully carry out a study using a physically-based model is generally greater than an empirically-based model due to the greater amount of required data and computation times. Throughout this study, efforts were made to record the approximate amount of time necessary to complete each phase of the GSSHA modeling effort, from construction to execution. This will allow future efforts to better estimate the amount of time entailed in a detailed dam safety analysis using GSSHA and decide whether or not the added effort is warranted. A total of 18.5 days (assuming an 8 hour work day) was necessary to complete the GSSHA model set up. Another 20 days were necessary for model calibration using a total of four events and another three days for model validation using one event. Four and a half days were needed to retrieve, build, and format Probable Maximum Storm grids for use in GSSHA while another seven days were needed to execute the necessary PMF, antecedent event, and IDF simulations. A total of 53 days or approximately 10.5 weeks (assuming a 40 hour or five day work week) was needed to complete the tasks specifically related to the construction, calibration, validation, and execution of the GSSHA model in this analysis.

The increase in model development and execution time should be weighed against the potential time and cost savings that can be realized with increased accuracy. The additional cost of a detailed modeling analysis using a physically-based model can be incredibly small compared to the potential costs of dam safety modifications.

This topic of research would benefit from additional studies performed on a multitude of dams and watersheds across the United States. This would allow for the determination of effects due to geographic location as well as differing watershed characteristics and provide a larger data set from which to draw conclusions for future use. Additionally, the inclusion of stochastic modeling techniques, which vary model input parameters that are commonly “fixed”, on top of the physically-based hydrologic modeling routines discussed in this research would allow for a greater understanding of parameter uncertainty and could provide more defensible results.

1. Introduction

U.S. Army Corps of Engineers (USACE) policy requires that dams "designed, constructed, or operated by USACE will not create a threat of loss of life or inordinate property damage".[2] In order for a dam to be considered "hydrologically adequate" and adhere to USACE policy, an appropriate embankment, spillway, and regulating outlet (if necessary) must be designed to safely pass an Inflow Design Flood (IDF) with adequate freeboard. An appropriate IDF is dependent upon the dam's uses, size, and relevant upstream and downstream factors. In most cases, this IDF is determined from the Probable Maximum Precipitation/Storm (PMP/PMS) that can occur over the watershed upstream of the dam. The Probable Maximum Flood (PMF) is then determined through an iterative process of centering and aligning the PMP within the contributing watershed. Finally, the IDF is determined through the use of varying antecedent conditions and/or ratios of the PMF inflow, depending upon site conditions. Instead of an IDF, most USACE dams were originally designed using a Spillway Design Flood (SDF) that made use of site-specific transposed and maximized historical storm events.

IDF's (and SDF's) have almost exclusively been determined using simplified, lumped, empirical rainfall-runoff methodologies. In fact, throughout the world, dam safety modeling guidelines specify or recommend the use of simplified hydrologic routines (such as unit hydrographs).[3,4] The most commonly used modeling approaches within USACE consist of initial and constant losses to generate excess precipitation, unit hydrograph theory to transform excess precipitation to point runoff hydrographs, and ratio to peak / recession constants to add a baseflow component to arrive at the final streamflow hydrograph. During the mid 1900s, when the majority of

[2] (U.S. Army Corps of Engineers, 1991)
[3] (Institute of Hydrology, 1999)
[4] (The Institution of Engineers, Australia, 2001)

USACE dams were designed and built, these approaches helped simplify the complicated rainfall-runoff process so computations could be performed by hand.

It is common knowledge that the three previously mentioned rainfall runoff modeling techniques transform precipitation to runoff hydrographs without detailed consideration of complicated internal processes.[5,6,7] As such, the empirical equations and parameters within each method tend to have limited physical significance. To "ground truth" these routines (in addition to those used within physically-based models), parameters are optimized through a calibration process where model outputs are compared to observed data in order to achieve an adequate "fit". Normally, calibrated infiltration, transform, and baseflow parameters are dependent upon the magnitude of the event being investigated. For instance, according to Sherman, who originally proposed the unit hydrograph concept, the unit hydrograph of a watershed is "…the basin outflow resulting from one unit of direct runoff generated uniformly over the drainage area at a uniform rainfall rate during a specified period of rainfall duration."[8] This implies that ordinates of any hydrograph resulting from a quantity of runoff-producing rainfall of unit duration would be equal to corresponding ordinates of a unit hydrograph for the same areal distribution of rainfall, multiplied by the ratio of rainfall excess values.[9] However, due to differences in areal distributions of rainfall and hydraulic reactions between large and small precipitation events, the corresponding unit hydrographs have not been found to be equal, as implied by unit hydrograph theory.[10]

[5] (Minshall, 1960)
[6] (Mein & Larson, 1973)
[7] (Harrison, 1999)
[8] (Sherman, 1932)
[9] (U.S. Army Corps of Engineers, 1959)
[10] (U.S. Army Corps of Engineers, 1991)

These realizations must also be combined with two factors: 1) Most precipitation events used for calibrating infiltration losses, unit hydrograph transforms, and base flows are normally much less intense than the PMP/PMF for a given area; 2) Engineer Regulation (ER) 1110-8-2 (FR) directs USACE personnel to use loss rates and unit hydrographs that will result in rapid runoff conditions.

To aid in the selection of "conservative" input parameter estimates, various rules of thumb have been in use for over 50 years within the USACE community. Two impactful rules of thumb when simulating PMF events are: 1) modelers are to use minimum infiltration loss rates from the events chosen for model calibration; 2) calibrated unit hydrograph parameters should be peaked by 25 – 50 percent.[11] In most PMF/IDF investigations, the applicability of these rules of thumb is not thoroughly analyzed. For instance, it is unknown whether a 25% peaking factor over or under-predicts the true unit hydrograph of a watershed in response to the PMP. Similarly, it is unknown whether a 50% peaking factor over or under-predicts the true unit hydrograph.

With the advent and proliferation of personal computers, hydrologic models such as those developed by the USACE Hydrologic Engineering Center, HEC-1[12] and HEC-HMS[13], have replaced hand calculations. However, most users still employ lumped, empirically-based rainfall-runoff approaches executed in a single-event fashion. As personal computing power increased, distributed, physically-based hydrologic modeling codes with continuous simulation capabilities have become more popular and used by design engineers to a greater extent. These modeling codes use little to no empirical relationships to approximate rainfall-runoff processes. Instead, they rely on spatially distributed parameters and physical process descriptions to

[11] (U.S. Army Corps of Engineers, 1991)
[12] (Hydrologic Engineering Center, 1998)
[13] (Scharffenberg, 2013)

approximate runoff volumes and hydrographs. However, they still must be calibrated to observed data within the watershed.

Due to the extreme nature of events used in the design of most flood risk management dams in the USACE portfolio, it is unknown whether simplifications commonly used in the design of these dams led to over or underestimates of watershed response and in turn over or underestimates of the chosen design. It is therefore pertinent to investigate these rainfall-runoff process simplifications through the use of physically-based rainfall-runoff processes that were not available during the design and construction of most USACE dams.

Comparisons between empirical, lumped modeling techniques and physically-based, distributed modeling techniques are numerous. Paudel et al. compared runoff consequences due to land use changes in a hypothetical watershed using the physically-based Gridded Surface Subsurface Hydrologic Analysis (GSSHA) rainfall runoff model and empirical routines within the HEC-HMS modeling routine.[14] Booker and Woods compared low and mean flows estimated using the physically-based TopNet rainfall-runoff model against two empirically-based rainfall-runoff models. They then used the empirical modeling results to improve the predictive capability of the physically-based model.[15] Also, Kalin and Hantush contrasted results obtained using the Kinematic Runoff and Erosion model (KINEROS-2) and GSSHA within an experimental watershed.[16]

GSSHA has also been used to simulate large historical flood events throughout the United States. Chintalapudi et al. simulated the June 2002 flood event in the Upper Guadalupe River watershed in south central Texas and contrasted the results using three different types of precipitation

[14] (Paudel, Nelson, & Downer, 2011)
[15] (Booker & Woods, 2014)
[16] (Kalin & Hantush, 2006)

input.[17] Sharif et al. used the same GSSHA model for the Upper Guadalupe River watershed to demonstrate the ability of GSSHA to adequately model the cumulative effect of two rainfall events without the need for significant calibration.[18] Sharif et al. also explored the differences caused by varying grid cell sizes and precipitation input when simulating a flood event in the Bull Creek Watershed in Austin, Texas using the GSSHA modeling code.[19]

England, et al. tested the applicability of a physically-based model when simulating events on the order of the PMF.[20] England, et al. also used the same model to constrain flood frequency curves for the watershed above Pueblo Dam on the Arkansas River in Colorado.[21] These studies made use of the Two-dimensional Runoff, Erosion, and eXport (TREX) rainfall runoff model to simulate extreme events within a large watershed in a semi-arid region of the Midwestern United States. However, these efforts did not delve into the differences in predicted runoff flow rates, volumes, and timing between the TREX model and previous, empirical models used in the original design of Pueblo Dam.

Abdullah also used the TREX modeling code to simulate extreme precipitation events within three different watersheds in Malaysia.[22] While events on the order of the PMP were simulated, this effort mainly focused on the determination of rainfall duration on the magnitude of peak discharges as a function of watershed size and relating peak specific-discharge to watershed size. A limited comparison between an existing empirical model and the TREX model was also made.

[17] (Chintalapudi, Sharif, Yeggina, & Elhassan, 2012)
[18] (Sharif, Hassan, Bin-Shafique, Xie, & Zeitler, 2010)
[19] (Sharif, Hassan, Zeitler, & Hongjie, 2010)
[20] (England, Velleux, & Julien, 2007)
[21] (England, Klawon, Klinger, & Bauer, 2006)
[22] (Abdullah, 2013)

2. Modeling Purpose

This research had three goals: 1) Demonstrate that the GSSHA modeling code can be used to simulate extreme events on the scale of the PMF, 2) Quantify the amount of time required to develop a more advanced physically-based rainfall runoff model capable of simulating extreme events for a specific dam, and 3) Investigate the limitations imposed by commonly employed empirical modeling methods in the realm of dam safety and extreme event hydrology. To achieve these goals, a GSSHA model was built, calibrated, validated and compared against an existing HEC-1 model for the same watershed. Comparisons between the infiltration, runoff transform, and baseflow routines within each model were then made by isolating each routine and comparing the runoff hydrographs at various points. Then, PMF/IDF hydrographs were created using the GSSHA model and compared against PMF/IDF hydrographs that were generated using the same HEC-1 model. Prompton Dam, which was recently modified due to dam safety issues discovered using simplified hydrologic methods, was chosen as the site for this comparison.

3. Prompton Dam

Prompton Dam is located within Wayne County on the West Branch Lackawaxen River near Prompton, Pennsylvania. The dam is owned and operated by USACE – Philadelphia District (NAP). Prompton Dam has a contributing drainage area of approximately 59 mi^2. Along with General Edgar Jadwin Dam, Prompton Dam provides flood risk reduction, in varying degrees, to the Boroughs of Prompton, Honesdale, and Hawley and to smaller communities along the Lackawaxen River. The location of Prompton Dam, in relation to the larger Delaware River watershed and other USACE NAP projects is shown in Figure 3.1.

Prompton Dam was designed and constructed as a 1200 feet long, 140 feet high, zoned earthfill embankment with a 30 feet wide crest originally placed at elevation 1225.37 feet, as referenced to the North American Vertical Datum of 1988 (NAVD88). The embankment is comprised of an upstream compacted earth-fill zone and a downstream compacted random fill zone separated by an inclined drainage zone. Rock fill was placed on the upstream and downstream faces for slope protection. The original spillway was an uncontrolled, 50 foot wide, open channel cut through the right abutment with a crest elevation of 1204.37 feet NAVD88.

The outlet works consist of an uncontrolled drop intake structure, an approximately 550 feet long reinforced concrete conduit, and a stilling basin. The intake is a morning glory structure with a crest at elevation 1124.37 feet NAVD88, which maintains a normal pool for recreational purposes.

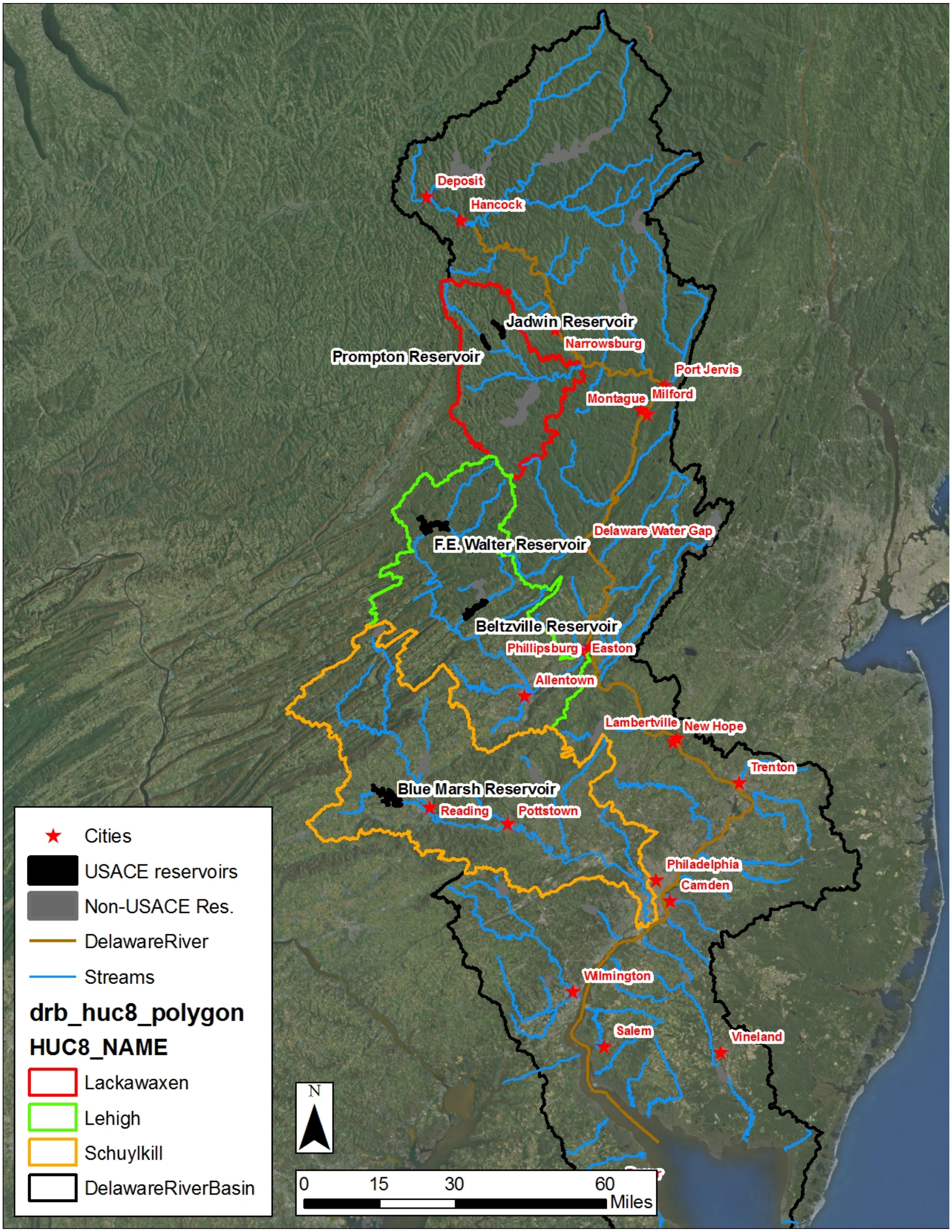

Figure 3.1 Delaware River Basin and Major Streams, Watersheds, Reservoirs, and Cities

ESRI World Imagery

Approximately 51,125 ac-ft of flood storage, which is equivalent to 16 inches of runoff from the contributing watershed, is maintained between 1124.37 ft and 1204.37 ft NAVD88. Pertinent information pertaining to Prompton Dam is shown in Table 3.2 while Figure 3.3 shows the elevation-volume-area relationship for the Prompton Reservoir. A photo of Prompton Dam reaching a record pool elevation during the June 2006 event is shown in Figure 3.4.

Table 3.2 Prompton Dam Pertinent Information

Feature	Elevation (ft NAVD88)	Surface Area (ac)	Storage	
			ac-ft	inches of runoff
streambed	1085.37	0	0	0
top of recreation pool	1124.37	271	3540	1.12
top of flood control pool	1204.37	958	54675	17.21
original top of dam	1225.37	1181	77135	24.28
modified top of dam	1232.37	1309	85800	27.01

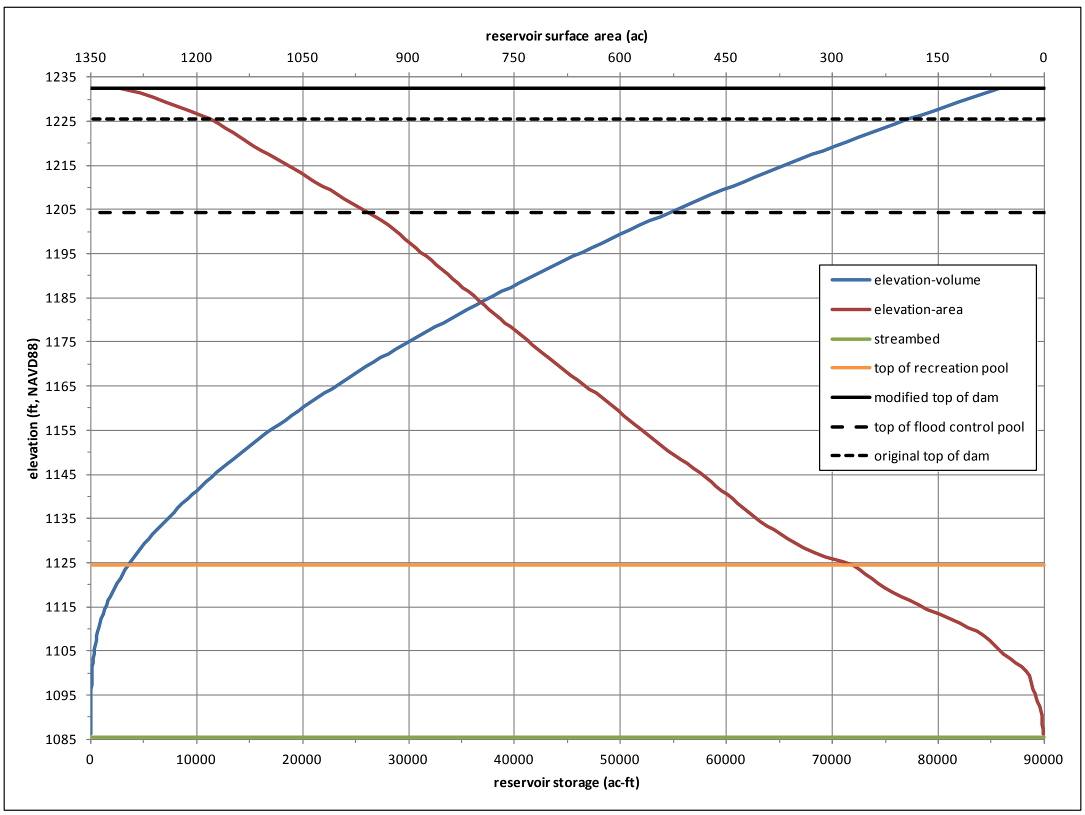

Figure 3.3 Prompton Dam Elevation-Volume-Area

Figure 3.4 Prompton Dam
June 29, 2006

The original design of the Prompton Dam spillway is contained within the 1949 Definite Project Report for Prompton and Dyberry Reservoirs. The SDF was designated as the full PMF, which was based upon the National Weather Service (NWS) Hydrometeorological Report (HMR) 23. The 48-hour PMP was determined to contain 24.15 inches of precipitation which resulted in approximately 23.42 inches of runoff with a peak inflow of 81,500 ft^3/s.[23] Routing the SDF through Prompton Reservoir and using the Standard Project Flood (SPF) as an antecedent pool resulted in a peak spillway outflow of 9200 ft^3/s and pool elevation of 1219.87 ft NAVD88.

[23] (U.S. Army Corps of Engineers - Philadelphia District, 1949)

Adding the required design freeboard resulted in the dam embankment crest being constructed at elevation 1225.37 ft NAVD88.

Design Memorandum Number 10 was completed by USACE NAP in 1966. The SDF was updated using the recently released NWS report HMR 33. The new 48-hour PMP contained 22.02 inches of precipitation which resulted in 19.75 inches of runoff with a peak inflow of 60,000 ft^3/s.[24] Using the SPF as the antecedent event, the reduced inflow volume of the new SDF resulted in additional freeboard being determined during this event.

In the mid-1980s, USACE NAP was asked by the Delaware River Basin Commission (DRBC) to provide additional water supply, low-flow augmentation, and salinity repulsion storage at two of its reservoirs, Prompton Dam and Francis E. Walter Dam. Designs were undertaken to include a control tower and gated outlet works along with a re-allocation of approximately 28,550 ac-ft of flood storage to water supply storage.[25] This additional storage required an increase in the normal conservation pool elevation of approximately 55 feet.

The USACE Hydrologic Engineering Center developed an HEC-1 hydrologic model to determine what ramifications this storage re-allocation would have to the flood risk reduction mission of Prompton Dam. An updated PMF was also developed as part of this effort using the NWS reports HMR 51 and HMR 52. A new 72-hour duration PMP was created that contained 33.65 inches of precipitation which resulted in 29.92 inches of runoff with a peak inflow of 111,000 ft^3/s. Hereafter, this report/model will be referred to as the "*1988 HEC Prompton Modification Study*". Following the completion of preconstruction and design efforts, DRBC

[24] (U.S. Army Corps of Engineers - Philadelphia District, 1966)
[25] (Hydrologic Engineering Center, 1988)

withdrew support and funding for construction of the Prompton and Francis E. Walter Dam Modifications.

In 1993, USACE NAP used the results of the 1988 HEC Prompton Modification model and the recently drafted ER 1110-8-2 (FR) to create an IDF. This IDF (along with an antecedent event equal to ½ of the IDF) was routed through the existing outlet works and spillway using two conditions. The first scenario assumed the dam was raised to avoid any overtopping flows. Using this first assumption, the peak pool elevation was calculated to be approximately 14 feet higher than the existing embankment elevation. The second routing scenario assumed the dam would be overtopped but not fail. Using this second assumption, the peak pool elevation was calculated to be approximately 5.5 feet higher than the existing embankment elevation.[26] This effort determined that the dam was hydrologically deficient and required modifications to align with current USACE hydrologic and hydraulic design criteria.

To rectify the hydrologic deficiency, the embankment and spillway were modified to safely pass the IDF. The earthen dam crest was effectively raised by 7 ft through the installation of a parapet crest wall. Along with the embankment, the spillway was also modified. A more efficient approach channel was excavated along with widening and deepening the spillway to 130 ft and 5 ft, respectively. A five foot tall erodible fuse plug was placed at the control sill to maintain the design flood storage capacity while increasing the available discharge capacity once the pool rises above the fuse plug (the fuse plug is designed to erode once flood storage is exceeded). The original spillway and modified spillway discharge rating curves are shown in Figure 3.5.

[26] (U.S. Army Corps of Engineers - Philadelphia District, 1993)

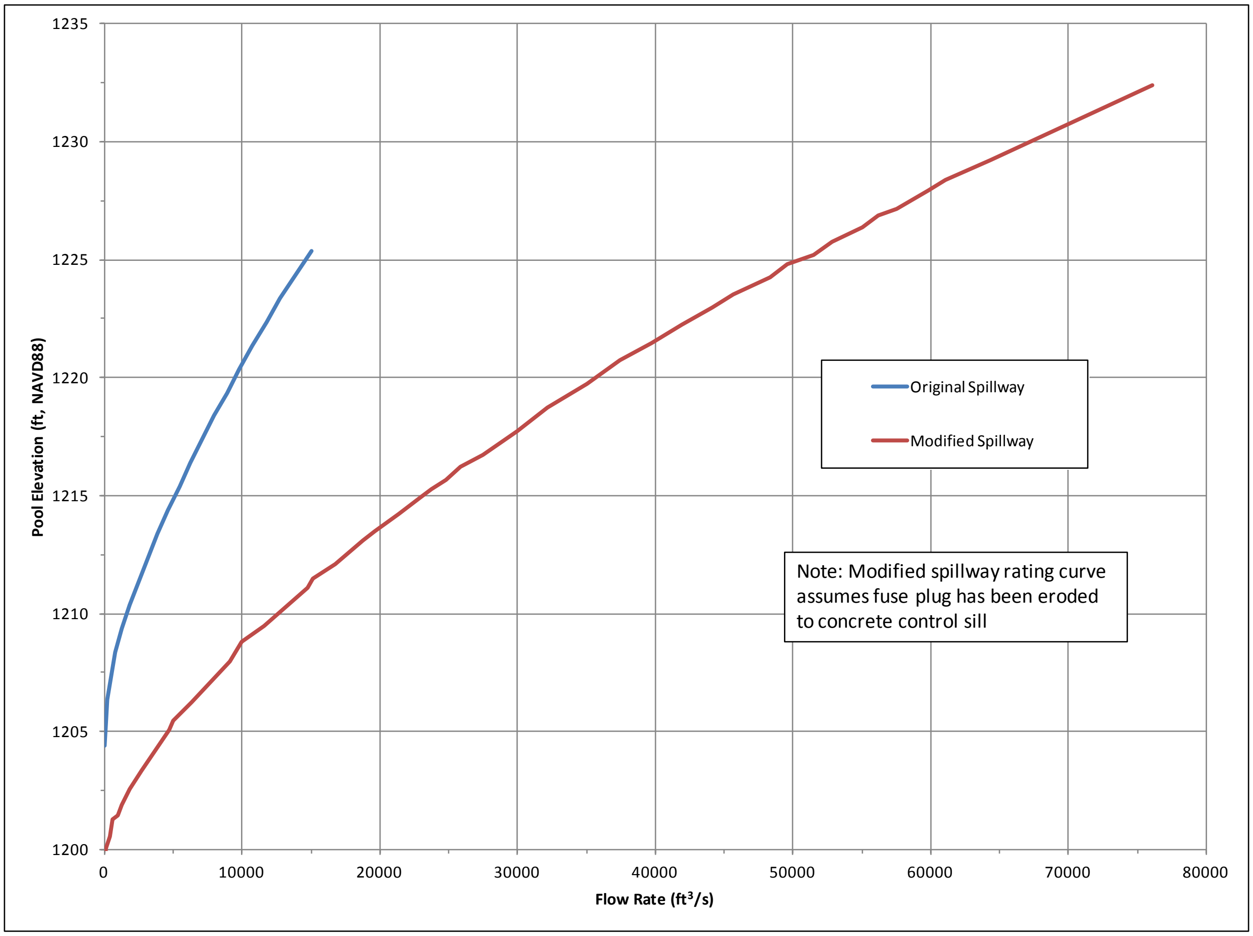

Figure 3.5 Prompton Dam Spillway Rating Curves

Additional modifications included stabilizing the channel below the stilling basin and spillway, the addition of a bridge to cross the spillway, and a new maintenance facility. Construction work was completed in two phases, ending in 2008 and 2012. All together, these modifications and study costs totaled approximately $25 million. The modifications to the spillway and embankment alone totaled approximately $18 million.

4. Geospatial Data

This analysis / modeling effort 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 (versions 9.3 and 10.0). Extensions to ArcGIS, namely the Hydrologic Engineering Center's (HEC) HEC-GeoHMS (ver. 5.0 and 10.1) and HEC-GeoRAS (ver. 4.3.93 and 10.1) add-ons, were used to perform additional geospatial manipulations. Additional work was done within Aquaveo's Watershed Modeling System (WMS) ver 9.0.

The common horizontal datum used in this analysis was the North Atlantic Datum of 1983, while the coordinate system was specified as Pennsylvania South State Plane (feet). All data that was not natively in this coordinate system/datum was transformed. Furthermore, the vertical datum used in this analysis was the 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 the National Geodetic Vertical Datum 1929 (NGVD29). Differences between NGVD29 and NAVD88 vary from location to location. For simplification, a uniform conversion factor of 0.63 ft (i.e. 100 ft NGVD29 = 99.37 ft NAVD88) was used to convert NGVD29 elevation data sources to NAVD88 for the study area.

5. Study Area Conceptual Model

The three dimensional surface and groundwater system needed to be conceptualized prior to executing the aforementioned modeling analysis. This was achieved through the use of a "conceptual model" which is a detailed description of the occurrence and movement of water within the area of interest (AOI). The conceptual model is intended to identify the various hydrologic, hydraulic, topographic, and geological features that physically affect the flow of water within the AOI. The following sections describe significant features that were used to formulate the conceptual model as well as data sources.

5.1. Hydrography

The West Branch Lackawaxen River begins in Sullivan County, PA near the town of Orson. Draining approximately 61 mi^2, the West Branch Lackawaxen River flows for approximately 21 miles in a southeasterly direction to the village of Prompton. The largest tributary to the West Branch Lackawaxen River is Johnson Creek. Johnson Creek drains approximately 17 mi^2 and flows for 10.5 miles in a southerly direction to its confluence with the West Branch Lackawaxen River just north of the town of Aldenville.

Approximately 0.5 miles below Prompton Dam, Van Auken Creek enters from the right to form the main stem Lackawaxen River. Van Auken Creek drains approximately 22 mi^2. During extreme runoff events, elevated water surface elevations (WSEL) from Van Auken Creek can cause backwater/hysteresis effects at the Prompton Dam outlet works. The Lackawaxen River then flows for approximately 40 miles through the towns of Honesdale (where it meets Dyberry Creek), Hawley (where it meets Wallenpaupack Creek), and Rowland before emptying into the Delaware River near the town of Lackawaxen, PA.

Stream bankfull depths within the West Branch Lackawaxen River watershed range from 2 – 5 feet in some headwater streams to 10 – 20 feet just above Prompton Reservoir. Similarly, stream widths (at a bankfull depth) range from 10 – 25 feet in headwater streams to nearly 100 feet just above Prompton Reservoir. Also, stream slopes vary throughout the contributing watershed. In the extreme headwaters, slopes of less than 10 ft/mile prevail. As the upstream drainage area increases, stream slopes increase with some segments approaching 100 ft/mi. However, downstream of Aldenville, stream slopes gradually mellow to approximately 20 ft/mi.

Prompton Dam has a contributing drainage of approximately 59 mi^2 which equates to approximately 98% of the entire West Branch Lackawaxen River watershed. Prompton Dam works together with General Edgar Jadwin Dam on Dyberry Creek north of Honesdale to reduce the occurrence and magnitude of flooding to downstream communities along the Lackawaxen River.

While there are numerous man-made reservoirs upstream of Prompton, there are only three reservoirs with storage volumes in excess of 1000 ac-ft. All three of these structures are owned by the Pennsylvania Fish Commission and all are used for recreation purposes. Flood runoff is typically passed through these reservoirs with little to no attenuation or translation.

Several USGS streamflow gaging stations are in operation within the AOI. All of the streamflow gages have 15 minute flow measurements available starting in the mid 1980s with daily, intermittent, and instantaneous yearly peak flows dating back much further. Additionally, hourly reservoir elevation time series data is available for Prompton Dam starting in October 1987 with additional intermittent measurements dating back to the start of operations (1960). The streamflow gage near Prompton has been in operation since 1944 while the gage near Aldenville, which is the only gaging station above Prompton Dam, has been operational since October 1986. The locations of these major streams, reservoirs, and gaging stations are shown in Figure 5.1.

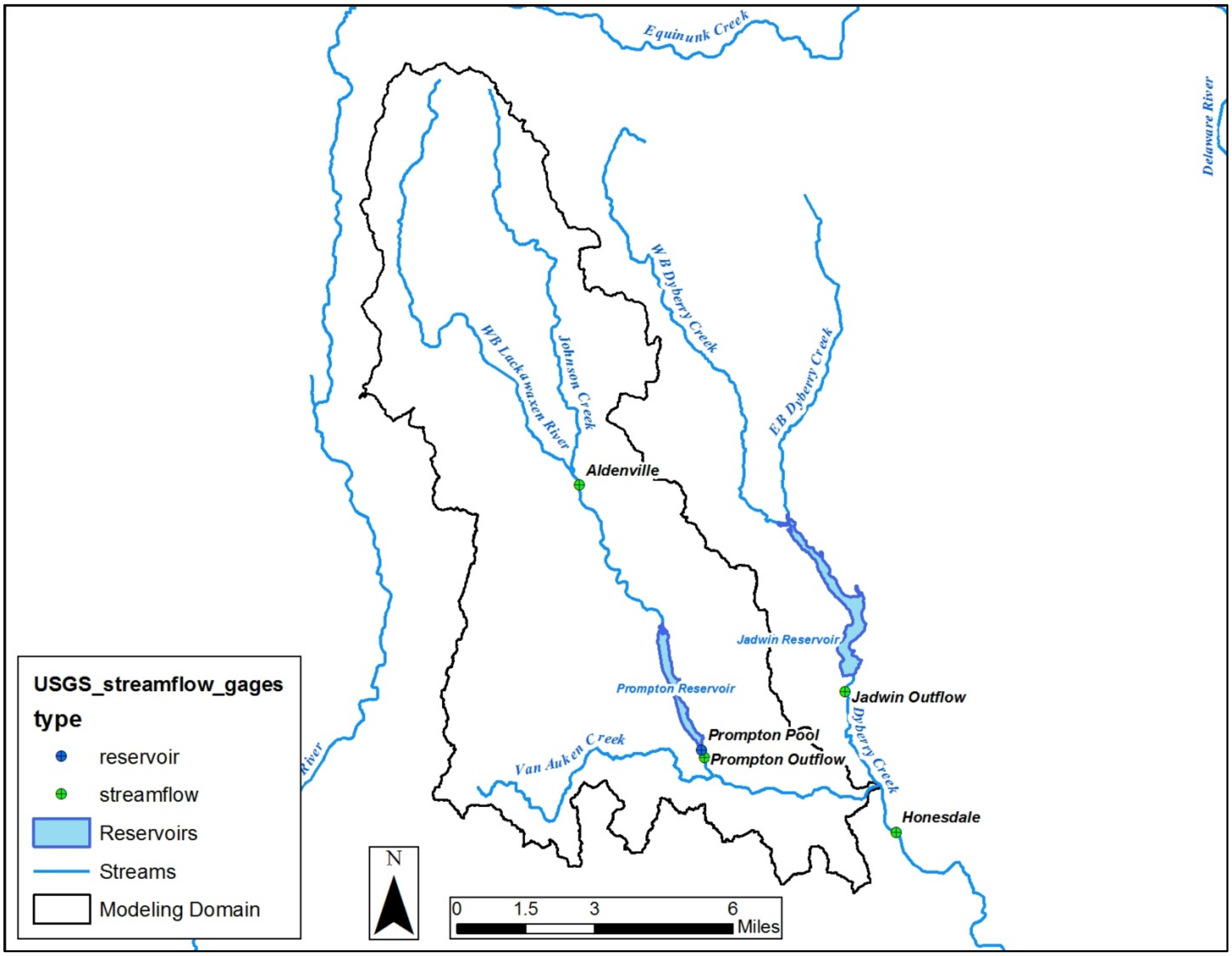

Figure 5.1 Hydrography and Major Streamflow/Reservoir Gaging Stations within the Area of Interest

5.2. Topography

Prompton Dam resides within the Glaciated Low Plateau physiographic province of Pennsylvania. As such, the topography consists of features that have been extensively modified by glacial erosion and deposition.[27] Elevations within the West Branch Lackawaxen River watershed range from approximately 1000 ft NAVD88 near the confluence with the Lackawaxen River to approximately 2500 ft in the headwater tributaries as shown in Figure 5.2. The primary source of elevation data used throughout this study was from the United States Geological Survey (USGS) National Elevation Dataset (NED). Digital Elevation Models (DEM) retrieved from the NED website[28] with a horizontal resolution of 1/9th arc seconds (approximately 3 meters) were mosaicked together to form a complete elevation coverage.

Light Detection and Ranging (LIDAR) elevation data from the Pennsylvania Department of Conservation and Natural Resources' (DCNR) PAMAP program were also used to supplement the NED 3m dataset. These data, with a horizontal resolution of 3.2 ft, were representative of 2008 conditions.[29]

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[27] http://www.dcnr.state.pa.us/topogeo/field/map13/13glps/index.htm
[28] http://ned.usgs.gov/
[29] http://www.pasda.psu.edu/

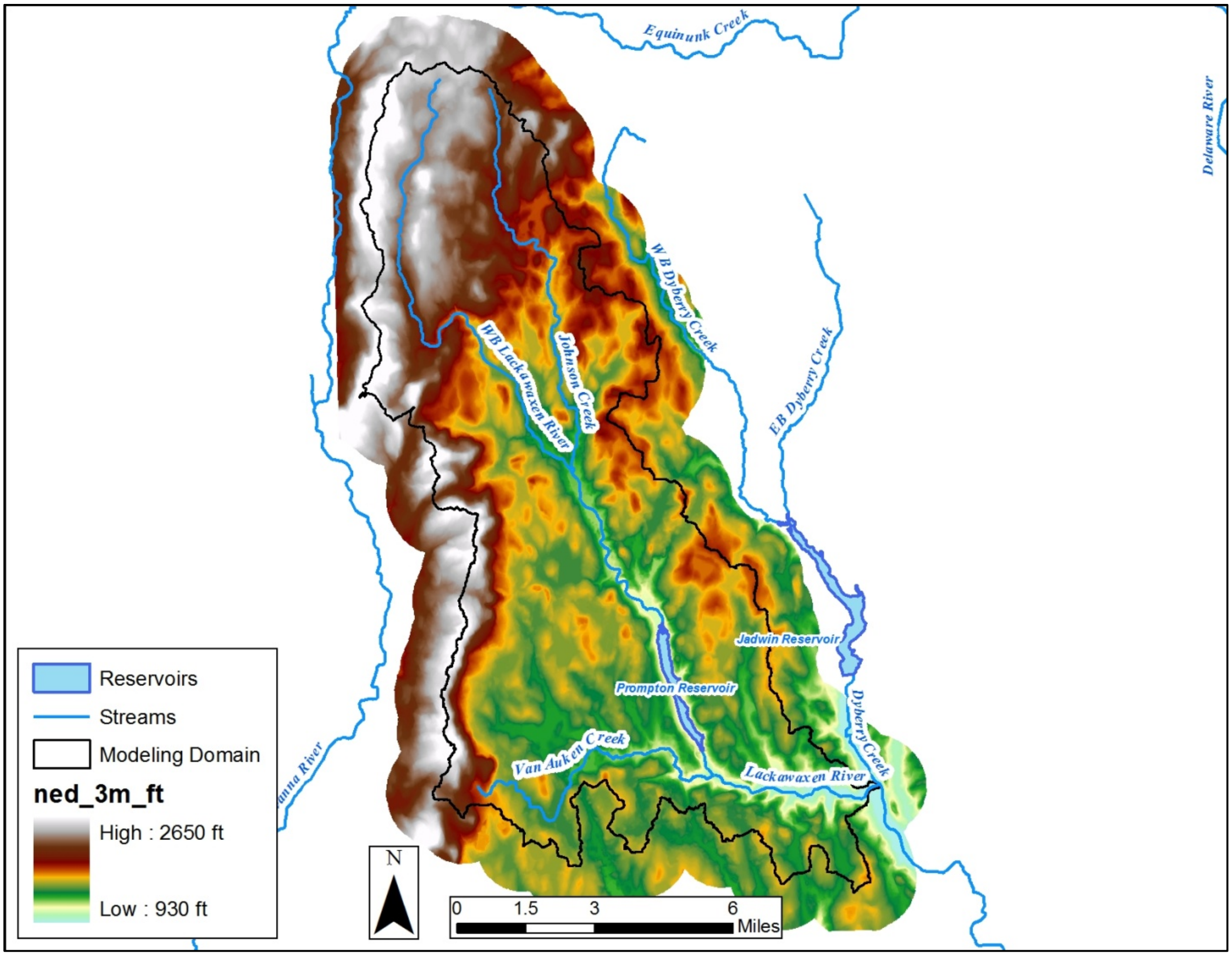

Figure 5.2 Elevations and Hydrography within the West Branch Lackawaxen River Watershed

NED 3m Dataset

5.3. Climate

The AOI has a climate that is typical of the Glaciated Low Plateau physiographic province within PA. This includes relatively warm summers with wet and variable winters. The most common storm type within the AOI is summer-time thunderstorms. However, the area is occasionally subjected to tropical storms (hurricanes) and extratropical storms ("northeasters"). Temporal and spatial distributions of precipitation commonly reflect topographic relief (orographic effects) and the prevailing eastward winds.

Average annual point rainfall within and around the AOI, as derived from precipitation gaging stations, can vary between approximately 20 and 60 inches. Average annual snowfall within the AOI from 1981 – 2000 varies between 61 to 70 inches.[30] Air temperatures near the AOI, as recorded at three United States Air Force 14th Weather Squadron (USAF – 14WS) hydrometeorological stations, vary from sub zero (Fahrenheit) temperatures during the winter months to near 100 degree temperatures during the summer months.

The previously mentioned precipitation gaging stations near the AOI are maintained by the National Oceanic and Atmospheric Administration (NOAA) National Climactic Data Center (NCDC)[31], USACE, and USGS. The locations of these gages, in addition to the three nearby USAF – 14WS hydrometeorological stations, are shown in Figure 5.3. Additional sources of precipitation and hydrometeorological data used to calibrate the hydrologic model are discussed in later sections.

[30] http://www.erh.noaa.gov/ctp/features/snowmaps/index.php?tab=norms
[31] http://www.ncdc.noaa.gov/oa/ncdc.html

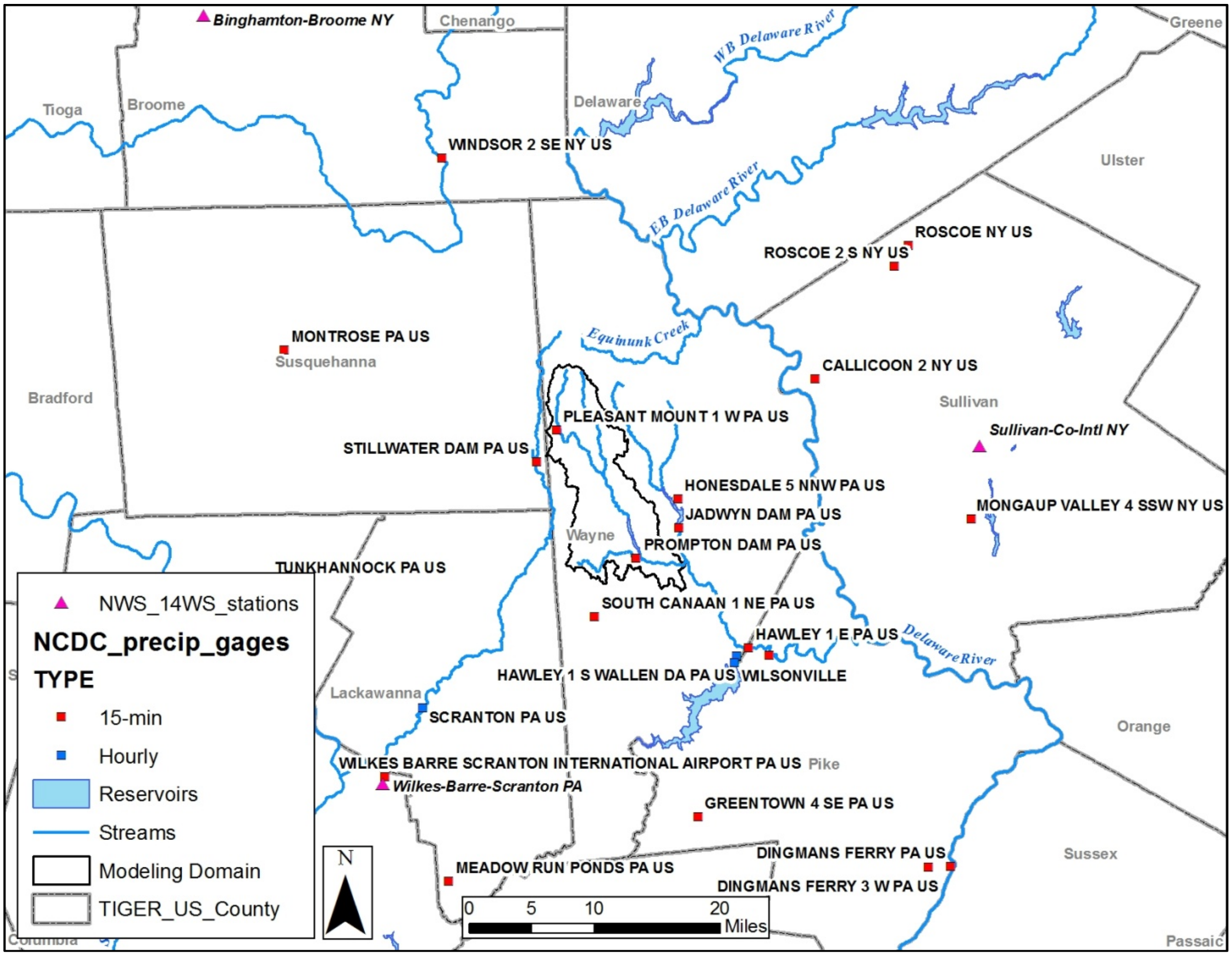

Figure 5.3 Precipitation and Hydrometeorological Gaging Stations Near the Area of Interest

5.4. Land Use

Land uses within the area of interest ranged from low intensity developed areas to woody wetlands, as shown within Figure 5.4. These land uses were derived from the National Land Cover Database (NLCD) 2006 coverages developed by the USGS EROS Center, which makes use of the Anderson land use classification system.[32] These land uses were representative of 2005 conditions.

[32] http://eros.usgs.gov/#/Science/Landscape_Dynamics/Land_Cover-Land_Use/National_Land_Cover

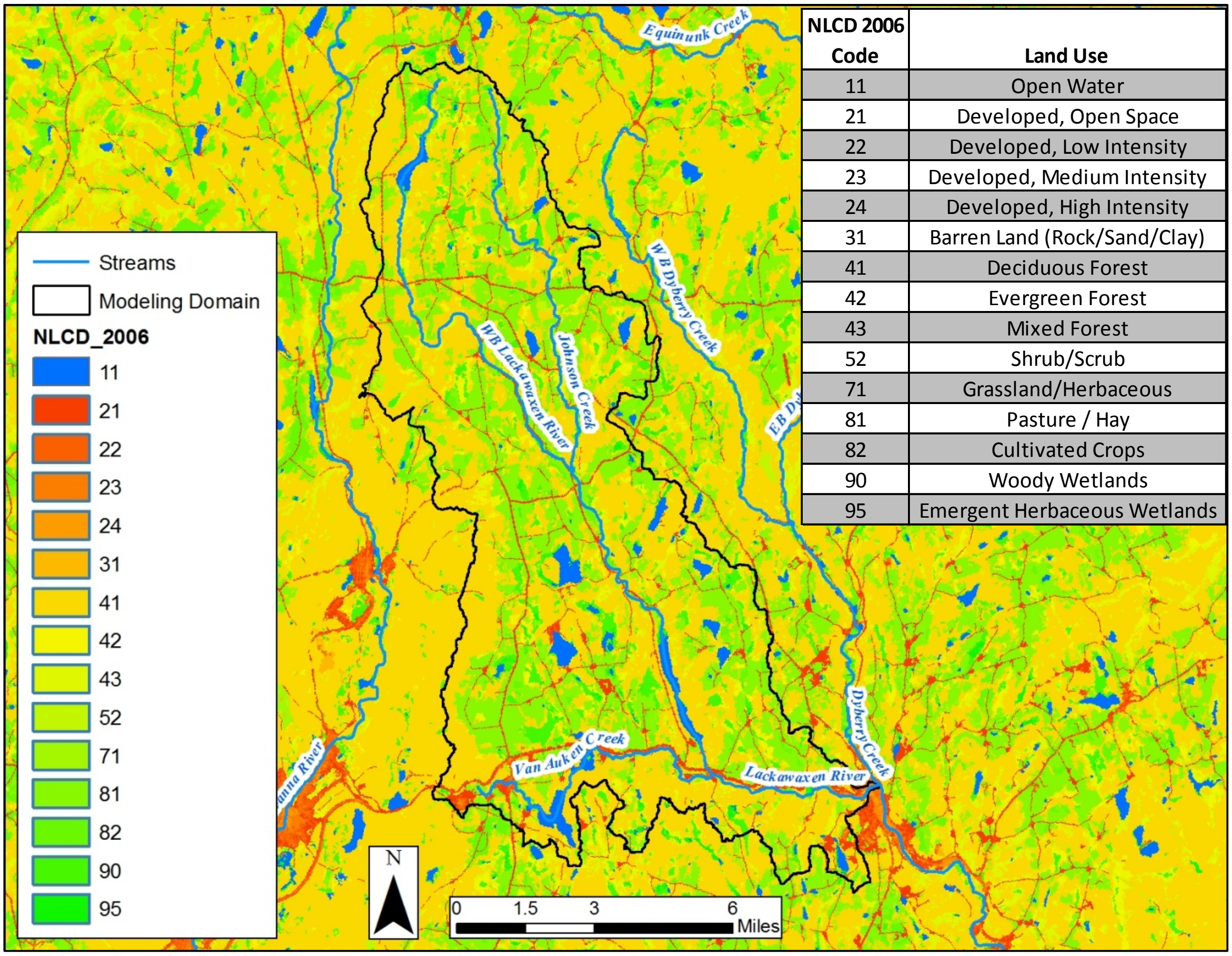

Figure 5.4 NLCD 2006 Land Uses Near the Area of Interest

5.5. Soils

Soils within the area of interest were primarily comprised of sands, silts, and loam combinations. Approximately 12 different surface soil textures were sourced from the Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) database.[33] These soil types were representative of 2004 – 2008 conditions and are shown in Figure 5.5.

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[33] http://soildatamart.nrcs.usda.gov/

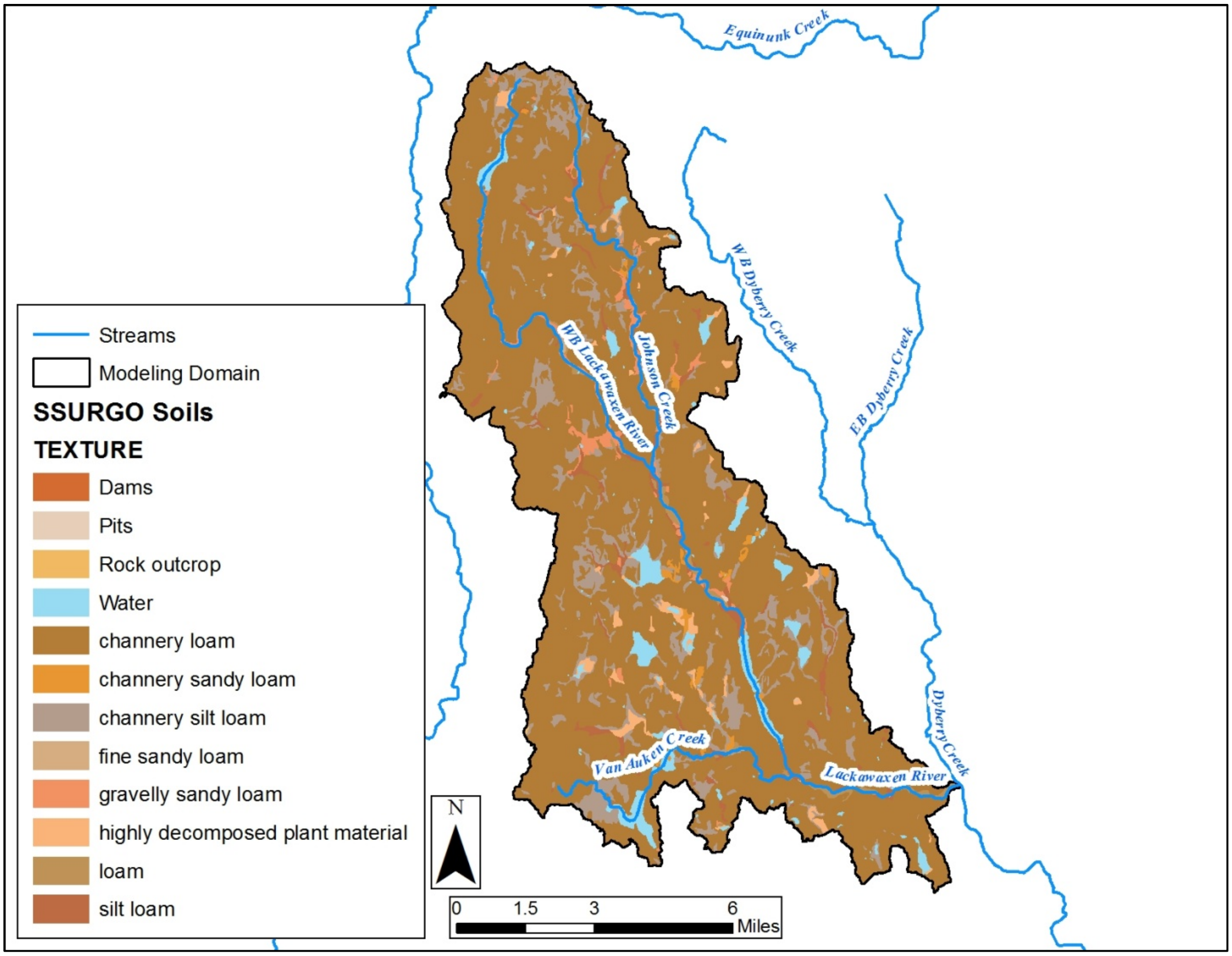


Figure 5.5 SSURGO Soil Textures Near the Area of Interest

5.6. Geology

Geologic formations within the area of interest are primarily members of the Catskill formation.[34] Sandstones, siltstones, shales, and mudstones comprise the majority of deposits found with the Catskill formation and various other geologic arrangements throughout northeastern PA and southwestern NY.

[34] (Berg, Geyer, & Edmunds, 1980)

6. Hydrologic Model Development

A hydrologic model simulates precipitation runoff and routing procedures, both natural and man-made. The essence of a hydrologic model is to transform precipitation (known) into runoff, streamflow, and/or reservoir elevations (unknown) at given locations and times. To determine volumetric flow rates and reservoir elevations during the PMF/IDF at Prompton Dam, the USACE Engineer Research and Development Center's Coastal and Hydraulics Laboratory (ERDC – CHL) Gridded Surface Subsurface Hydrologic Analysis (GSSHA) code (version 6.0) was used.[35] GSSHA is classified as a distributed parameter, physically-based model that can be run for a single-event or in a multi-event, continuous simulation. Conversely, the 1988 HEC Prompton Dam Modification Study model made use of lumped parameter, empirical modeling processes that were executed in a single-event mode.

Table 6.1 summarizes the rainfall-runoff processes used within the 1988 HEC Prompton Dam Modification Study model and those that were used within the GSSHA model. The following sections discuss the specific modeling processes that were used to create and calibrate the GSSHA hydrologic model.

Table 6.1 Rainfall-Runoff Process Comparison

Model	Infiltration	Transform	Baseflow
HEC-1*	Single Event, Lumped Parameter, Initial and Constant Loss Rate	Lumped, Dimensionless, Clark Unit Hydrograph	Uncoupled, Recession Baseflow
GSSHA	Continuous Simulation, Distributed, Green Ampt w/ Moisture Redistribution	Distributed, Two-Dimensional, Diffusive Wave	Coupled, Single-Layer, Two-Dimensional Groundwater Routing

* These were the processes used within the 1988 HEC Prompton Modification Study. HEC-HMS (the successor program to HEC-1) allows users to develop hydrologic models using a wide range of loss, transform, and baseflow routines including several distributed parameter routines.

[35] (Downer, Ogden, & Byrd, 2008)

The time and effort required to successfully carry out a study using a physically-based model is generally greater than an empirically-based hydrologic model. This is due to the greater amount of required data to construct the model as well as the millions more hydrologic computations required to execute a simulation. However, this trade-off has the potential to result in greater accuracy and time/cost savings at a later time. Throughout this study, efforts were made to record the approximate amount of time necessary to complete each phase of the GSSHA modeling effort, from construction to execution. An eight hour work day and 40 hour work week was assumed. This will allow future efforts to better estimate the amount of time entailed in a detailed analysis of this sort and decide whether or not the added effort is warranted.

6.1. Modeling Domain

An all encompassing modeling domain was created to accurately replicate the hydrology of the AOI. This domain was created by topographically delineating the Lackawaxen River watershed above its confluence with Dyberry Creek at Honesdale (totaling approximately 93 mi^2). This was done in an effort to minimize the influence of boundary conditions on results within the AOI. Large tributaries below Prompton Dam (principally Van Auken Creek) can induce backwater effects which influence the tailwater rating during high releases from the dam. This delineation was performed using the previously mentioned NED 3m elevation dataset by means of tools within ArcGIS and HEC-GeoHMS. Approximately 4 hours (1/2 working day) was required to delineate the modeling domain for use in the GSSHA model.

6.2. 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 100 meter x 100 meter (m) grid cell resolution (i.e. each grid cell covers approximately 2.5 acres in area) was chosen as a compromise between definition and run time requirements. The AOI overlain by this grid cell size resulted in the formulation of approximately 24,000 active cells. A no flow boundary was assumed to exist along the lateral edges of the modeling domain, save for 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 NED 3m dataset was used to assign representative elevations to the overland computational grid. Topographic artifacts (sinks) resulting from the conversion of the approximately 3 m x 3 m NED 3m DEMs to the 100 m x 100 m GSSHA grid were selectively removed to reduce computational burdens through visual identification and the CHL program "CleanDam".[36] However, areas that were in fact sinks on the overland network were allowed to remain.

Surveyed bathymetry beneath the surface of Prompton Reservoir was incised into the grid to account for any possible subsurface groundwater/surface water routing effects. The GSSHA grid was manipulated until surveyed storage values updated using bathymetric surveys from March 2009 were recreated (shown in Figure 3.3). The final GSSHA grid is shown (rotated) in Figure 6.2. Approximately 40 hours (5 working days) was required to set up, alter, and finalize the computational grid for use in the GSSHA model.

[36]http://www.gsshawiki.com/gssha/Utility_Programs:CleanDam

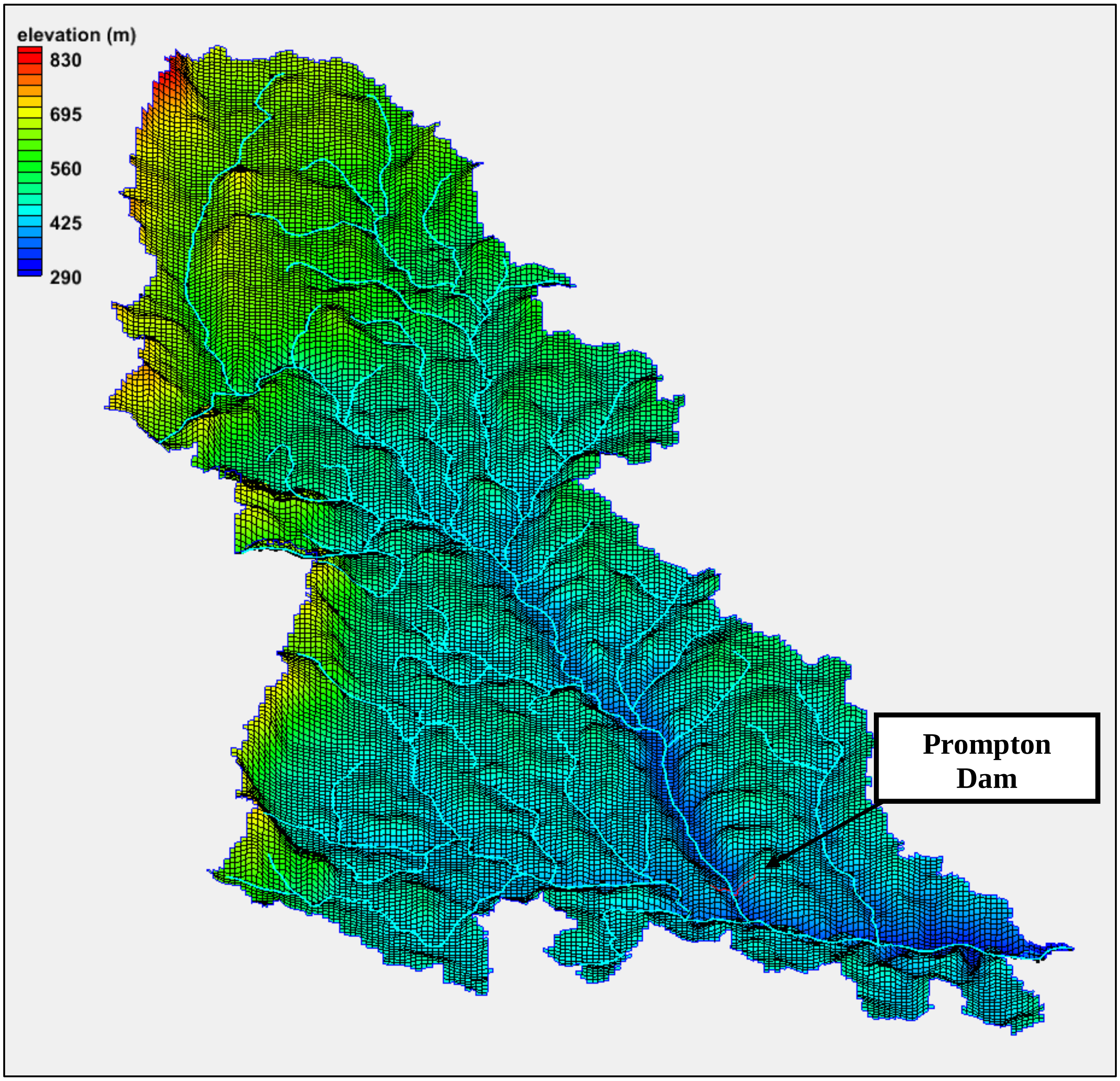


Figure 6.2 GSSHA Overland Grid

6.3. Infiltration

Infiltration computations were executed using a modified Green Ampt routine that allowed for soil moisture redistribution during times of little to no rainfall. Surface soil textures were used to assign the representative initial parameter values for the Green Ampt infiltration computations using the previously shown SSURGO soil types.[37,38]

[37] http://www.gsshawiki.com/gssha/Infiltration:Parameter_Estimates

The initial parameter estimates for each SSURGO ID are shown in Table 6.3. Approximately 20 hours (2.5 working days) was required to source, format, and import the soils data to the GSSHA model.

Table 6.3 Initial Green Ampt Infiltration Parameter Estimates

SSURGO ID	Soil Type	Sat. Hydraulic Conductivity (cm/hr)	Capilary Head (cm)	Effective Porosity	Pore Index	Residual Saturation	Field Capacity	Wilting Point
1	Dams	1.09	11.01	0.412	0.189	0.041	0.207	0.095
2	Pits	1.09	11.01	0.412	0.189	0.041	0.207	0.095
4	Rock outcrop	0.1	1	0.3	0.1	0.01	0.1	0.01
6	Water	0.1	1	0.3	0.1	0.01	0.1	0.01
7	channery loam	1.09	11.01	0.412	0.189	0.041	0.207	0.095
8	channery sandy loam	1.09	11.01	0.412	0.189	0.041	0.207	0.095
9	channery silt loam	0.34	16.68	0.486	0.117	0.015	0.33	0.133
13	fine sandy loam	1.09	11.01	0.412	0.189	0.041	0.207	0.095
15	gravelly sandy loam	1.09	11.01	0.412	0.189	0.041	0.207	0.095
17	decomposed plant material	1.09	11.01	0.412	0.189	0.041	0.207	0.095
18	loam	0.66	8.89	0.434	0.126	0.027	0.27	0.117
20	silt loam	0.34	16.68	0.486	0.117	0.015	0.33	0.133

6.4. 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 using the previously shown NLCD 2006 coverage.

Estimates of percent impervious cover were assigned using the National Land Cover Database (NLCD) 2006 developed by the USGS Earth Resources Observation and Science (EROS) Center. Additionally, Prompton Reservoir (and any additional major water body) was considered impervious area that contributed runoff at the rate of precipitation. Penman-Monteith

[38] (Rawls & Brakensiek, 1983)

evapotranspiration parameters, which play a critical part in determining water budgets and antecedent conditions during long term simulations, were also based upon land uses.[39]

Table 6.4 details the various Anderson land use classifications as well as the representative GSSHA initial parameter values that were based upon each land use. Approximately 20 hours (2.5 working days) was required to source, format, and import the land use data to the GSSHA model.

Table 6.4 Initial Overland Routing, Impervious Area, and Evapotranspiration Parameter Estimates

NLCD 2006 Code	Land Use	Initial Overland Roughness	Impervious Area (%)	Albedo	Vegetation Height (m)	Vegetation Radiation Coefficient	Canopy Resistance
11	Open Water	0.001	100	0.5	0	0	0
21	Developed, Open Space	0.1	10	0.2	20	0.18	120
22	Developed, Low Intensity	0.1	35	0.2	20	0.18	120
23	Developed, Medium Intensity	0.1	65	0.2	20	0.18	120
24	Developed, High Intensity	0.1	90	0.2	0.1	0	86
31	Barren Land (Rock/Sand/Clay)	0.1	0	0.05	0	0	0
41	Deciduous Forest	0.4	0	0.15	20	0.18	120
42	Evergreen Forest	0.4	0	0.2	20	0.18	120
43	Mixed Forest	0.4	0	0.2	20	0.18	120
52	Shrub/Scrub	0.2	0	0.2	20	0.18	120
71	Grassland/Herbaceous	0.4	0	0.2	20	0.18	120
81	Pasture / Hay	0.25	0	0.2	0.5	0.1	100
82	Cultivated Crops	0.3	0	0.2	1	0.18	50
90	Woody Wetlands	0.25	0	0.3	20	0.18	120
95	Emergent Herbaceous Wetlands	0.25	0	0.3	20	0.18	120

[39] http://www.gsshawiki.com/gssha/Obtaining_Data:Obtaining_Data

6.5. 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 portion.

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 land use, and at large in-stream structures. This resulted in the creation of 92 channel segments for a total length of approximately 115 stream miles. These streams were linked with the overland grid and its associated elevations 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 within the GSSHA grid.

Each stream segment was assigned an appropriate cross sectional shape based upon the NED 3m dataset and extracted using tools within ArcGIS and HEC-GeoRAS. Overland backwater effects were simulated for flow entering the channel during elevated streamflow conditions. Manning's roughness factors for each channel segment were assigned using orthophotographs and site visits.[40] These initial roughness values ranged between 0.035 and 0.05.

Stream channel losses were modeled based upon the Darcy equation which required two input parameters: stream bottom material hydraulic conductivity and stream bottom material thickness.

[40] (Chow, 1959)

Stream channel bottom hydraulic conductivity was globally set at 2.5 cm/hr while the material thickness of each channel segment was set at 1 meter.

The locations of all of the routing reaches used within the hydrologic model are detailed in Figure 6.5. Approximately 40 hours (5 working days) was required to identify, draw, initialize, and alter stream data for use in the GSSHA model. The majority of that time was spent extracting cross sections and smoothing the stream thalwegs.

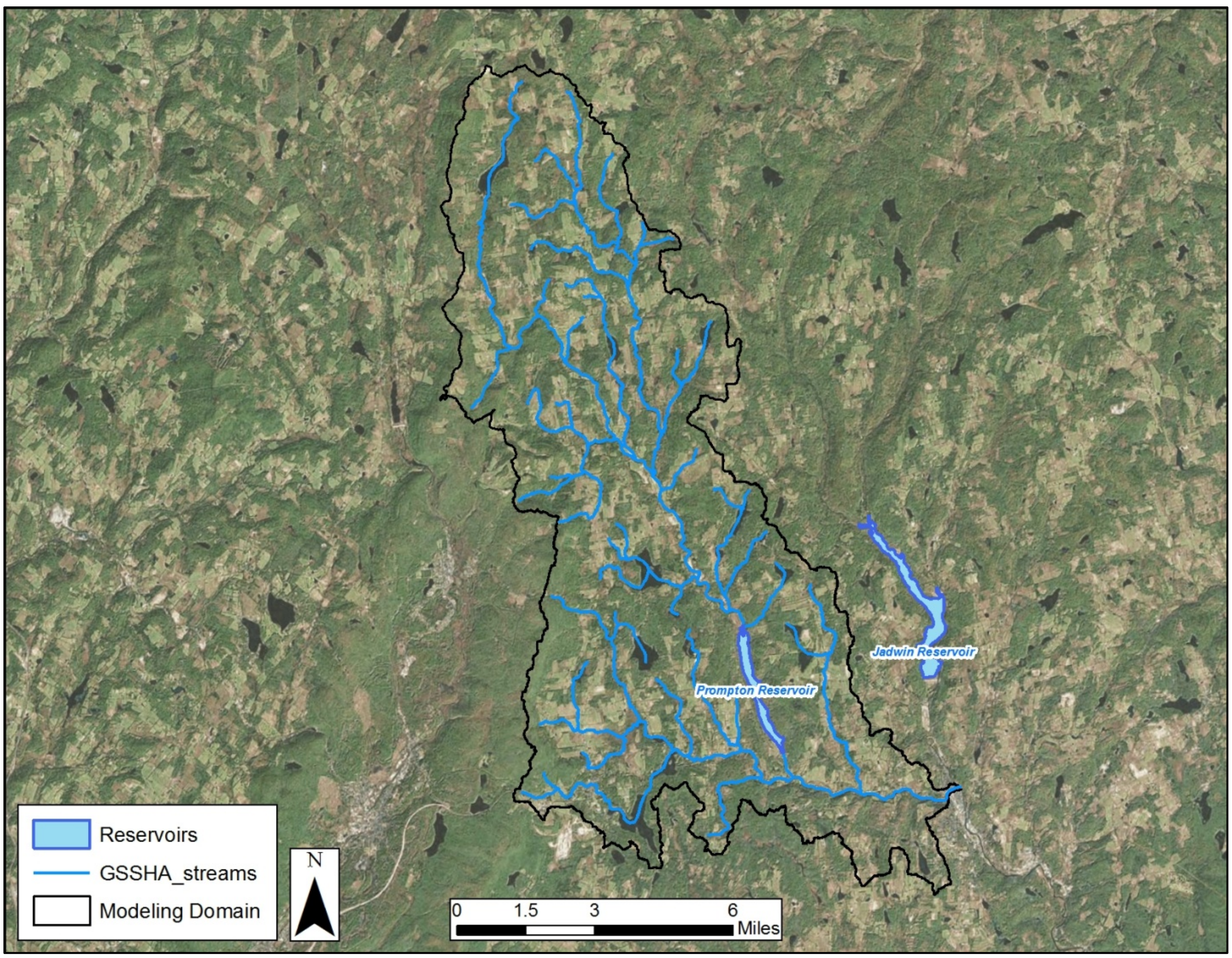


Figure 6.5 GSSHA Routing Reaches

ESRI World Imagery

6.6. Groundwater Routing

A simplified groundwater network was created to link the overland network and stream channels. The one layer, saturated, two-dimensional horizontal flow calculations were solved using the line successive over relaxation (LSOR) method. Dynamic recharge to the groundwater network and discharge from the groundwater network was calculated using the previously mentioned Green Ampt infiltration/exfiltration routine.

A single material zone within the groundwater network was identified (Catskill Formation). Therefore, the groundwater network was initially set to have spatially uniform (horizontal) saturated hydraulic conductivity and porosity values of 0.2 cm/hr and 0.3, respectively. A confining unit (no flow boundary) was assumed to exist approximately 50 meters below the ground surface.

Approximately 12 hours (1.5 working days) was required to source and format the groundwater data for use in the GSSHA model. Another 12 hours (1.5 working days) was necessary to build and initialize the groundwater network within the GSSHA model.

6.7. Prompton Dam Releases

The Prompton Dam conduit and spillway were modeled as known elevation-discharge relationships.[41] The conduit and spillway rating curves were summed for computational ease. The total elevation-discharge relationship input to the GSSHA model is shown in Figure 6.6.

[41] (U.S. Army Corps of Engineers - Philadelphia District, 1993)

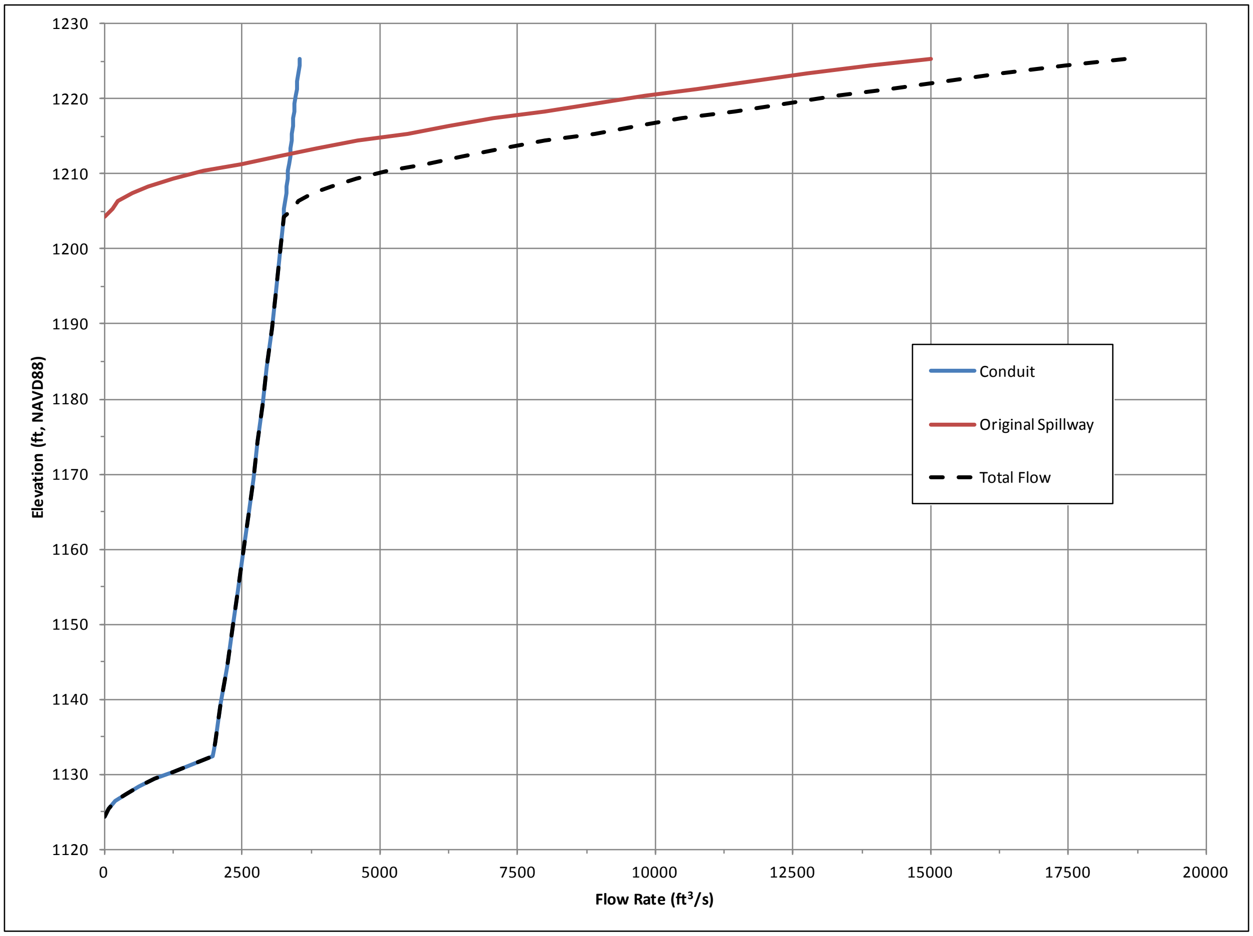


Figure 6.6 Prompton Dam Release Rating Curve

6.8. Model Calibration

In order to accurately estimate runoff responses during hypothetical events as needed within this analysis, initial model processes, parameters, and inputs were "ground-truthed" through a calibration process which adjusted several parameters to reflect watershed conditions during historic precipitation events. First, model parameters were adjusted to minimize the differences between computed and observed hydrograph shape, peak flow rates, and discharge volumes at the USGS streamflow gaging stations on the West Branch Lackawaxen River near Aldenville (01428750; if available) and Prompton (01429000) as well as pool elevations within Prompton Lake (01428900).

While no events within recorded history resulted in discharges or pool elevations close to those from a PMF or IDF event, it was still desirable to use the most extreme events possible for calibration due to the nature of this analysis. Therefore, events resulting in the highest observed runoff rates and pool elevations were chosen for use in this analysis. Events chosen for model calibration included those occurring in June 2006 and October – November 2006. Additionally, the Tropical Storm Irene and Tropical Storm Lee event of August – September 2011 was chosen for model validation, which is described in Section 6.10.

Also, the same events that were used for model calibration during the 1988 HEC Prompton Modification Study were used to determine how input parameter values varied due to the differences in modeling techniques. These two events occurred in August 1955 and November 1950. It should be noted that the peak inflow rates for Prompton Dam (had it existed at that time) were less than half that experienced during the June 2006 event.

The instantaneous peak streamflow at Aldenville, inflow to Prompton Dam, outflow from Prompton Dam, and Prompton Dam peak pool elevation period of record is shown in Figure 6.7. The events used for calibration during this analysis as well as those used during the HEC 1988 Prompton Modification Study are also detailed in this figure.

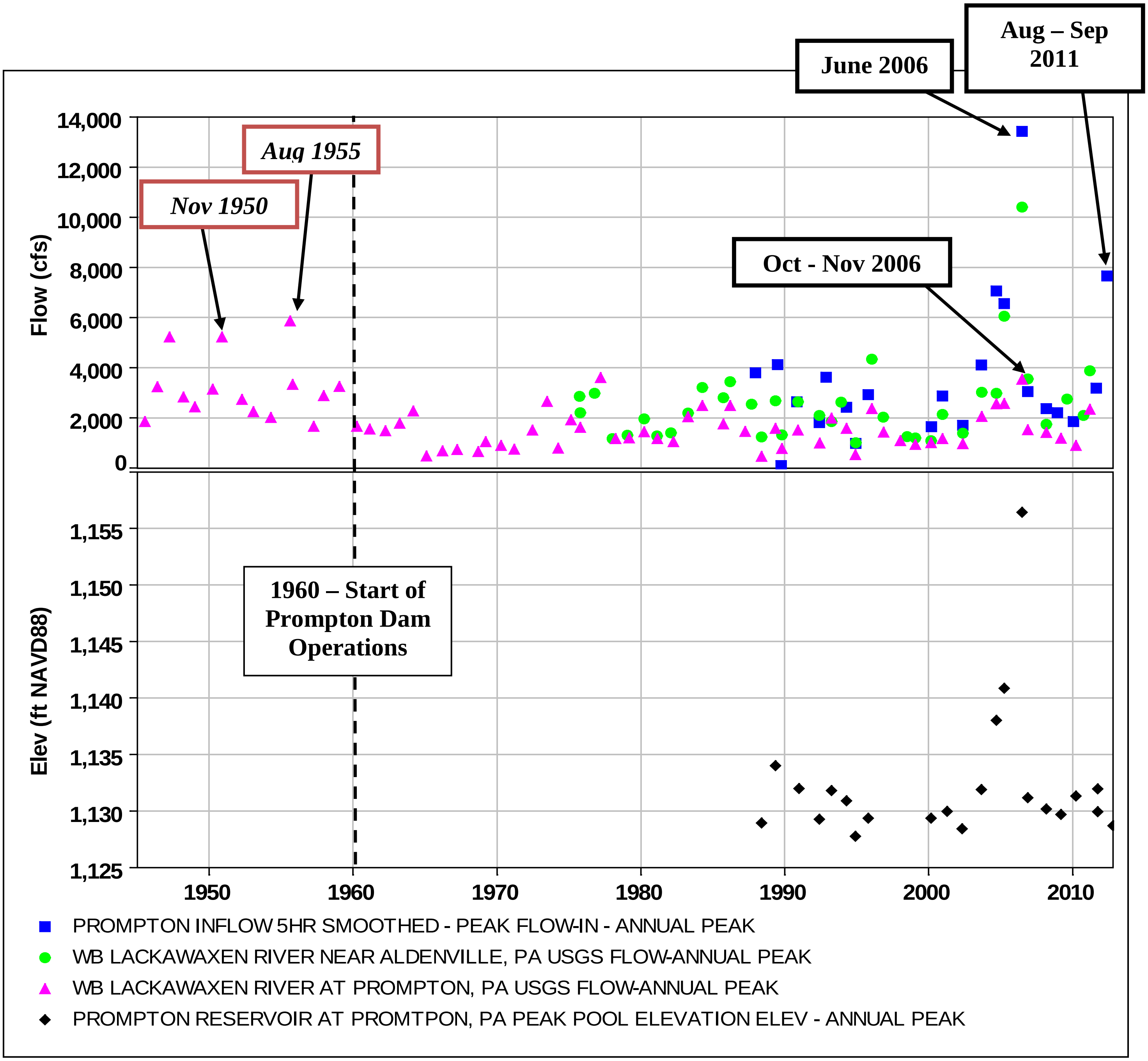


Figure 6.7 Streamflow at Aldenville, Prompton Inflow, Prompton Outflow, and Prompton Pool Elevation Period of Record

Historical precipitation data used for each historical event was compiled from two sources. First, the NWS Next-Generation Radar (NEXRAD) Multisensor Precipitation Estimator (MPE) coverages were used when available. 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 a Standard Hydrologic Grid with 500 m x 500 m resolution (SHG500) using tools available through HEC. The second source of precipitation that was used to supplement the MPE coverages came from the previously shown NWS precipitation gages (Figure 5.3).

Hydrometeorological data used within the Penman-Monteith evapotranspiration routine was sourced from the three previously mentioned USAF – 14WS stations near the AOI. These stations are located at airports near Wilkes Barre-Scranton, Sullivan County, NY, and Binghamton-Broome County, NY (Figure 5.3).

6.9. Calibration Results

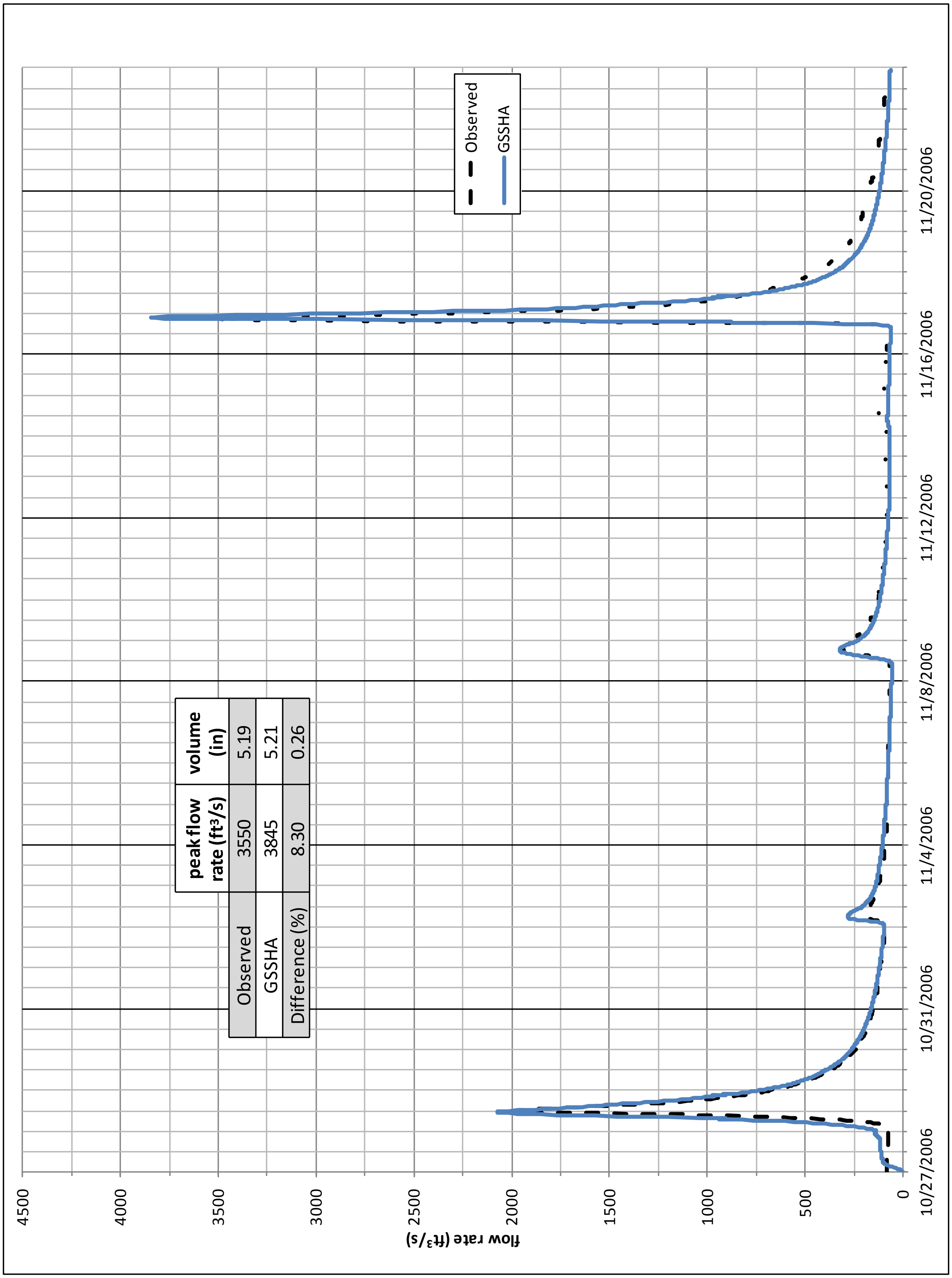
Prior to the execution of the calibration events, the groundwater network was allowed to "equilibrate" from the estimated initial conditions (which were described in Section 6.6) by repeatedly inputting rainfall from March – November 2006. The groundwater table reached a stable condition and sustained baseflow rates within the stream network were simulated that reasonably matched observed baseflow rates and expected groundwater conditions. These simulations required a total of approximately 40 hours (5 working days) to execute.

Following the groundwater network equilibration, the calibration events were simulated. These simulations used time steps in the range of 5 – 10 seconds to increase accuracy and aid model stability. Each simulation was executed in approximately 4% of real time. Depending upon the desired simulation time frame, the total time required to execute a simulation varied from approximately 2.5 hours to 28 hours for a 2.5 day and month-long simulation, respectively.

Generally, the GSSHA model results compared favorably with the observed data in hydrograph peak flow rate, peak flow rate timing, flow volume, and hydrograph shape. The computed flow hydrograph at Aldenville matched the observed records extremely well for the October – November 2006 event. The computed flow volume was within 0.3 inches of the observed flow volume while the computed peak flow rate was within 300 ft^3/s of the observed peak flow rate, as shown in Figure 6.8. Also, the peak reservoir stage was within 0.3 ft of the observed peak reservoir stage, which is shown in Figure 6.9.

During the June 2006 event, the Aldenville gage received heavy debris damage that caused the gage to malfunction and eventually stop recording. Therefore, the observed Prompton pool stage hydrograph was given greater credence than the observed flow hydrograph at Aldenville. As such, model inputs were adjusted until the computed reservoir stage hydrograph matched the observed stage hydrograph reasonably well and the peak pool stage was within 0.1 ft as compared to the observed peak pool stage, as shown in Figures 6.10 and 6.11.

The total time required to adequately calibrate the GSSHA model to the October – November 2006 and June 2006 historic events was approximately 120 hours (15 working days). The amount of time necessary to achieve adequate calibration is dependent upon the event being simulated (rainfall duration, rainfall intensity, etc) and the desired level of accuracy.



	peak flow rate (ft³/s)	volume (in)
Observed	3550	5.19
GSSHA	3845	5.21
Difference (%)	8.30	0.26

Figure 6.8 Streamflow Calibration at Aldenville – Oct-Nov 2006 Event

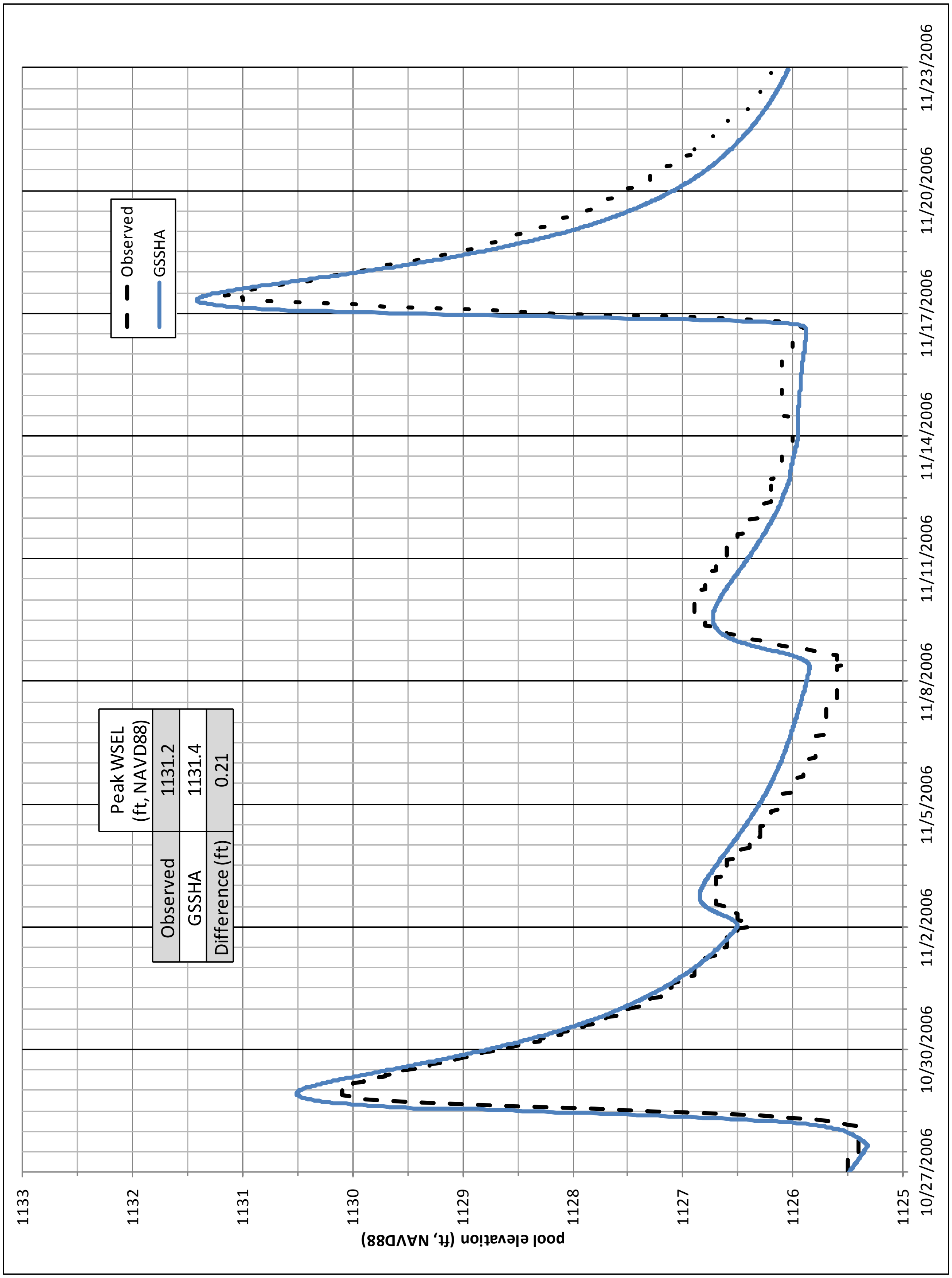


Figure 6.9 Prompton Dam Pool Elevation Calibration – Oct-Nov 2006 Event

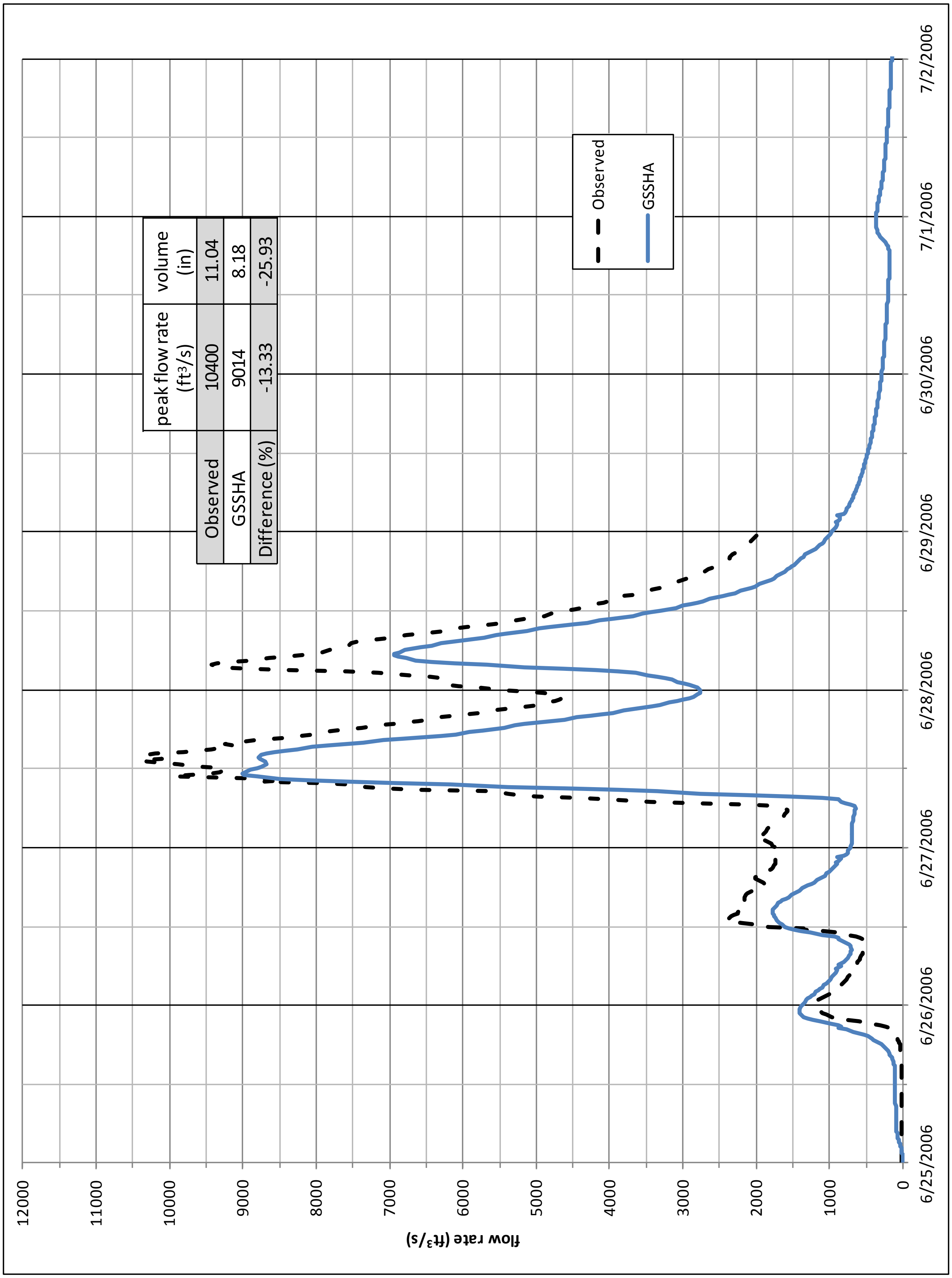


Figure 6.10 Streamflow Calibration at Aldenville – June 2006 Event

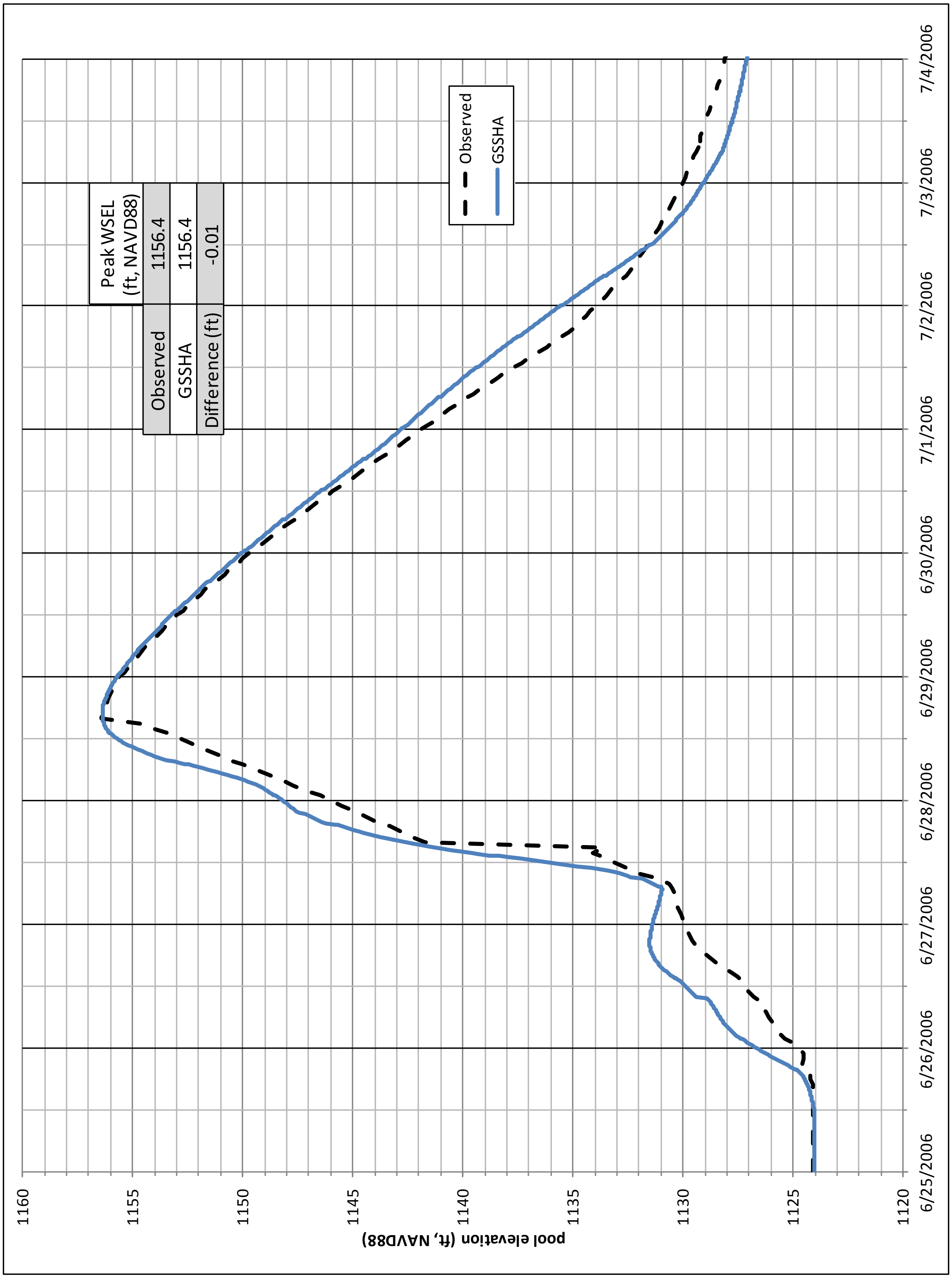


Figure 6.11 Prompton Dam Pool Elevation Calibration – June 2006 Event

In order to calibrate the GSSHA model to the same events used during the 1988 HEC Prompton Modification Study, significant computational grid modifications were required in the vicinity of Prompton Dam. This was due to the fact that Prompton Dam was not built at the time of the November 1950 and August 1955 events. Also, the Aldenville gage was not operational; however, the gage at Prompton, PA was operational. Gage precipitation records, observed streamflow, and GSSHA-computed streamflow for the November 1950 and August 1955 events are shown in Figures 6.12 and 6.13, respectively.

The GSSHA model was able to recreate both events matching peak flow rates, volumes, and hydrograph shape reasonably well. However, the timing of runoff initiation and peak flow rate for the November 1950 event lagged by approximately 2 hours. Unreasonable GSSHA parameter values were required to match the observed streamflow. Therefore, this discrepancy was accepted. It should be noted that the 1988 HEC Prompton Modification Study showed the same lag in model response compared to the observed streamflow data hinting at probable inconsistencies in either observed streamflow or precipitation.

The total time required to adequately calibrate the GSSHA model to the November 1950 and August 1955 events was approximately 40 hours (5 working days). These events required less time to calibrate due to their shorter duration and lower rainfall intensities leading to overall smaller simulation times.

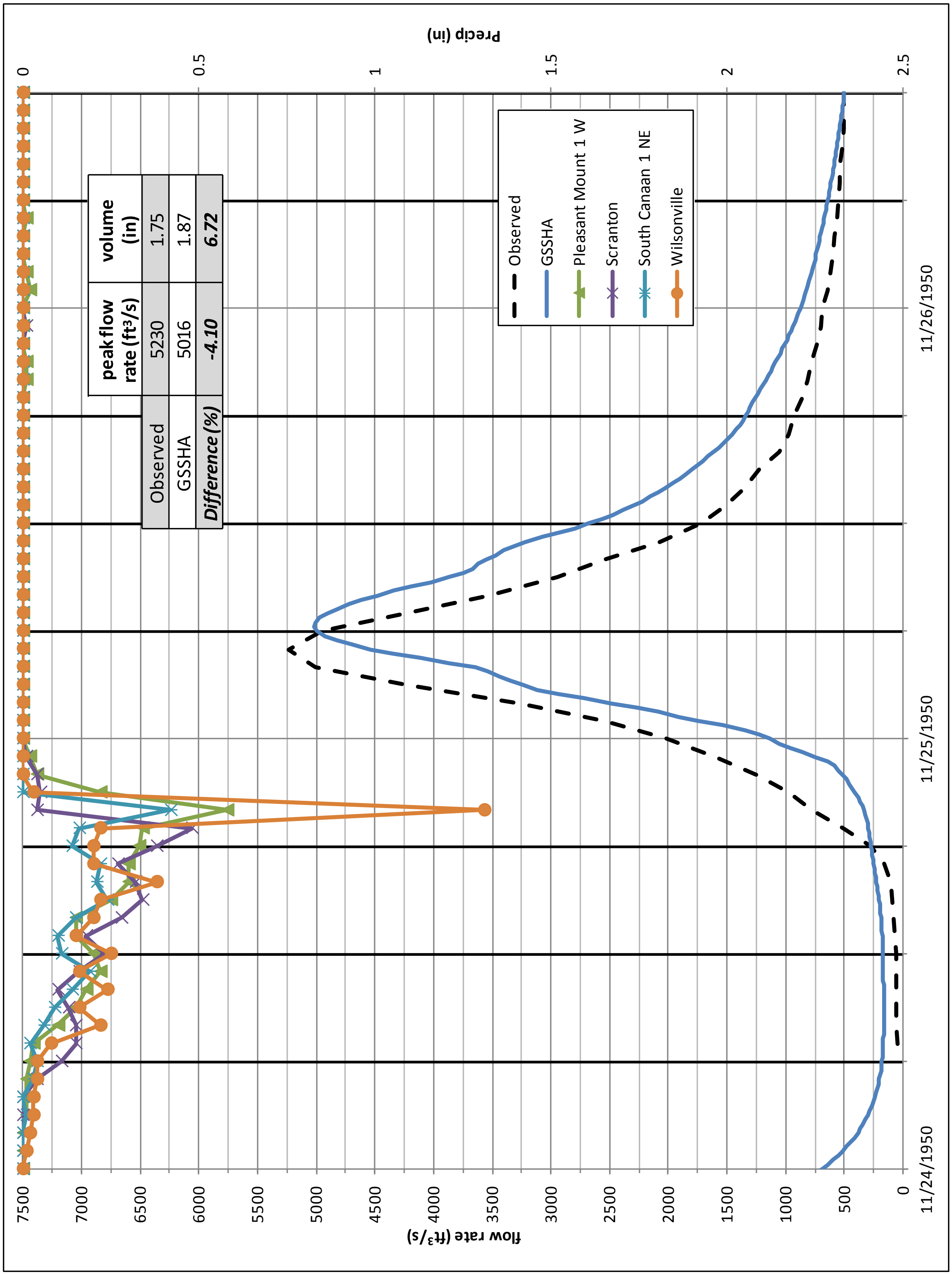


Figure 6.12 Rainfall Input and Streamflow Calibration Results at Prompton – Nov 1950 Event

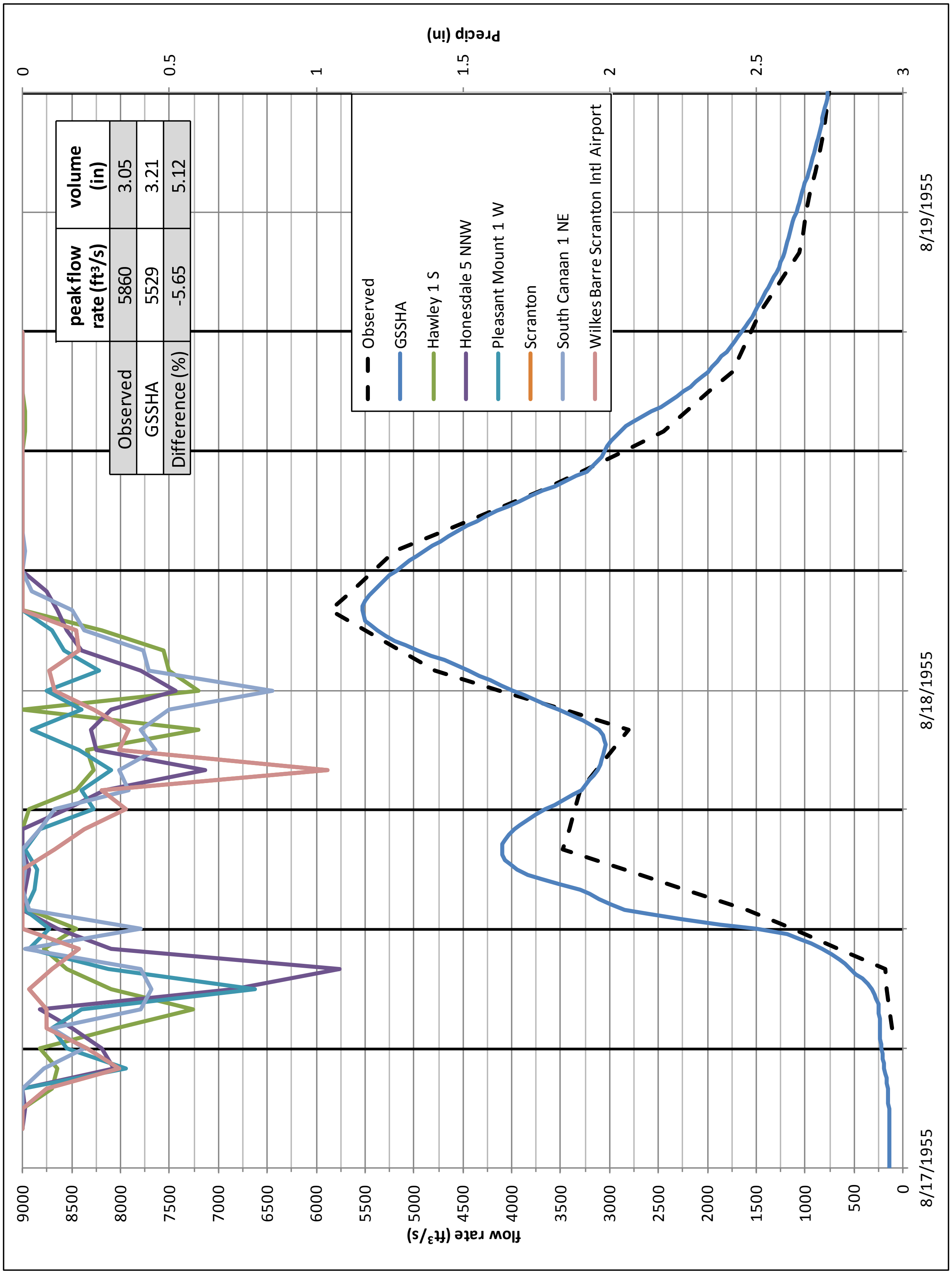


Figure 6.13 Rainfall Input and Streamflow Calibration Results at Prompton – Aug 1955 Event

The calibrated model inputs for the overland routing, infiltration, groundwater routing, and stream routing routines for each event is shown within Table 6.14 – 6.17

Table 6.14 Calibrated Overland Roughness Values

NLCD 2006 Code	Land Use	Calibrated Overland Roughness:		
		June 2006 & Oct - Nov 2006 Events		Nov 1950 & August 1955 Events
11	Open Water	0.001		0.0005
21	Developed, Open Space	0.15		0.05
22	Developed, Low Intensity	0.15		0.05
23	Developed, Medium Intensity	0.15		0.05
24	Developed, High Intensity	0.15		0.05
31	Barren Land (Rock/Sand/Clay)	0.15		0.05
41	Deciduous Forest	0.6		0.2
42	Evergreen Forest	0.6		0.2
43	Mixed Forest	0.6		0.2
52	Shrub/Scrub	0.3	.	0.1
71	Grassland/Herbaceous	0.6		0.2
81	Pasture / Hay	0.375		0.125
82	Cultivated Crops	0.45		0.15
90	Woody Wetlands	0.375		0.125
95	Emergent Herbaceous Wetlands	0.375		0.125

Table 6.15 Selected Calibrated Green Ampt Infiltration Values

Soil Type	June 2006 Event		Oct - Nov 2006 Event		Nov 1950 Event		Aug 1955 Event	
	K_{sat} (cm/hr)	Capil. Head (cm)	K_{sat} (cm/hr)	Capil. Head (cm)	K_{sat} (cm/hr)	Capil. Head (cm)	K_{sat} (cm/hr)	Capil. Head (cm)
Dams	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
Pits	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
channery loam	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
channery sandy loam	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
channery silt loam	0.05	12.51	0.08	16.68	0.10	16.68	0.03	12.51
fine sandy loam	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
gravelly sandy loam	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
decomposed plant material	0.16	8.26	0.25	11.01	0.33	11.01	0.11	8.26
loam	0.10	6.67	0.15	8.89	0.20	8.89	0.07	6.67
silt loam	0.05	12.51	0.08	16.68	0.10	16.68	0.03	12.51

Table 6.16 Calibrated Groundwater Routing Values

	June 2006 Event	Oct - Nov 2006 Event	Nov 1950 Event	Aug 1955 Event
Groundwater Horizontal Hydraulic Conductivity (cm/hr)	0.2	0.35	0.35	0.35
Groundwater Porosity	0.45	0.45	0.45	0.45

Table 6.17 Calibrated Stream Roughness Values

	June 2006 Event	Oct - Nov 2006 Event	Nov 1950 Event	Aug 1955 Event
Stream Roughness	0.09	0.085	0.075	0.085

Once the GSSHA model was calibrated to the June 2006 and Oct – Nov 2006 events, a representative set of model parameters was created by averaging the calibrated parameters. This calibrated set of model parameters was then validated using another historical event, which is discussed in the Section 6.10.

Another set of model parameters was created by averaging the GSSHA model parameters from the November 1950 and August 1955 calibration events. The creation of this set of model parameters allowed for additional comparisons between the 1988 HEC Prompton Modification Study results which are discussed in Sections 7.2 – 7.5.

The two biggest parameter differences were found within the overland routing (surface roughness, "N") and Green Ampt infiltration (saturated hydraulic conductivity, "K_{sat}") routines. The calibrated values for surface roughness and saturated hydraulic conductivity are shown in Figures 6.18 and 6.19. Overland roughness was found to vary by approximately 80% between the two calibrated parameter datasets while saturated hydraulic conductivity was found to differ by approximately 6.5%.

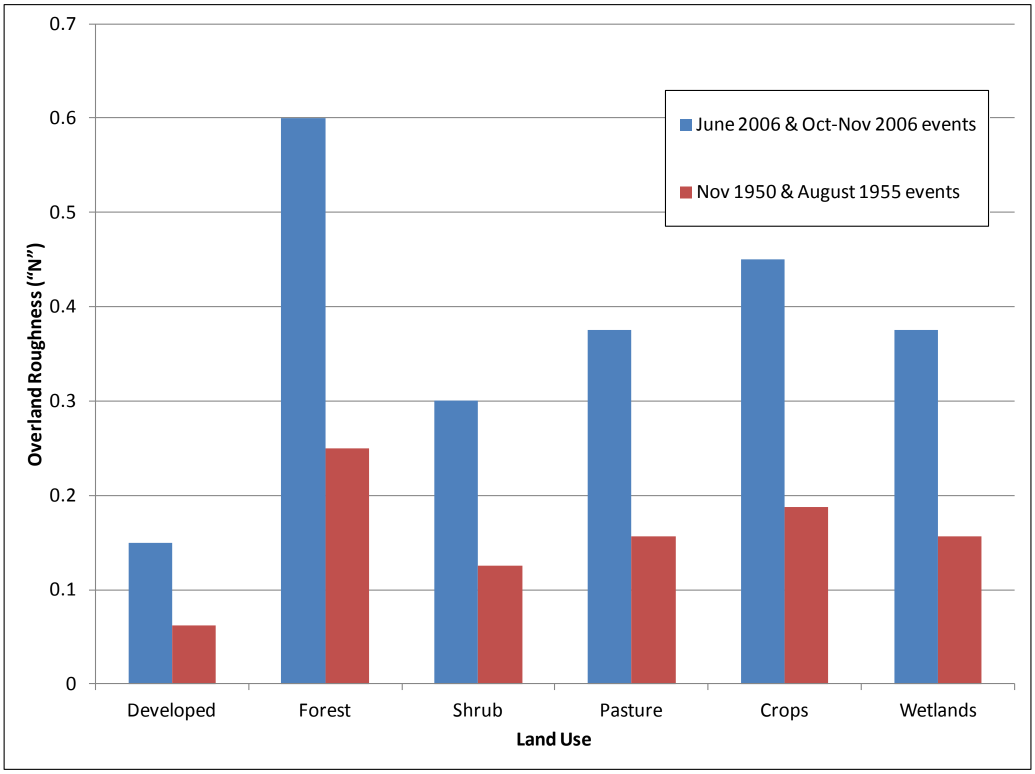


Figure 6.18 Calibrated Overland Roughness Comparison

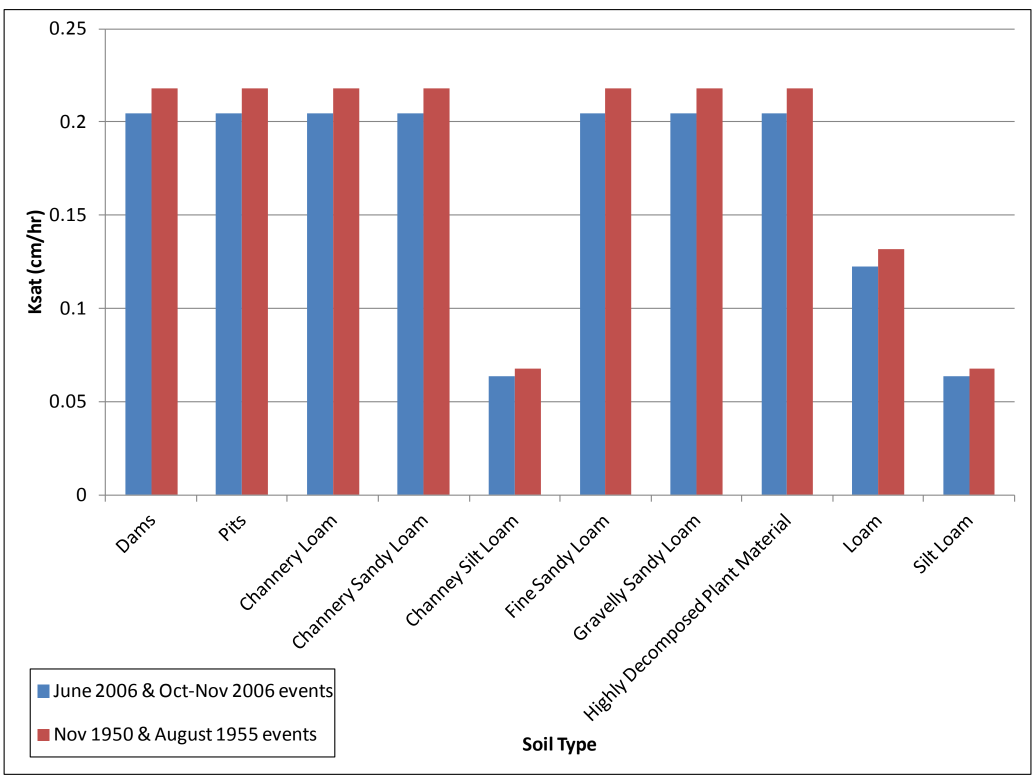


Figure 6.19 Calibrated Saturated Hydraulic Conductivity Comparison

6.10. Model Validation

The representative set of model inputs determined during the calibration process from the June 2006 and Oct – Nov 2006 events was tested using another historical event without changing any model parameters. This historical event occurred during August and September 2011 which was due to the remnants of Tropical Storm Irene and Tropical Storm Lee.

6.11. Validation Results

As shown in Figure 6.20, the GSSHA model slightly over predicted the peak flow rate during Tropical Storm Irene (August 2011) while under predicting the peak flow rate and runoff volume during Tropical Storm Lee (September 2011) at the Aldenville gage. However, these differences only resulted in Prompton Dam peak pool elevation differences of approximately 2 ft and 0.1 ft for the Tropical Storm Irene and Lee events, respectively, as shown in Figure 6.21.

Despite the differences between observed and predicted streamflow and pool elevation, the GSSHA model acceptably reproduced the runoff response of these two events. Therefore, the GSSHA model was considered validated. It should be noted that the 1988 HEC Prompton Modification Study did not include any model validation efforts.

The total time required to validate the GSSHA model to the Tropical Storm Irene and Lee events was approximately 24 hours (3 working days). Model validation typically requires less time due to the inherent lack of model parameter changes.

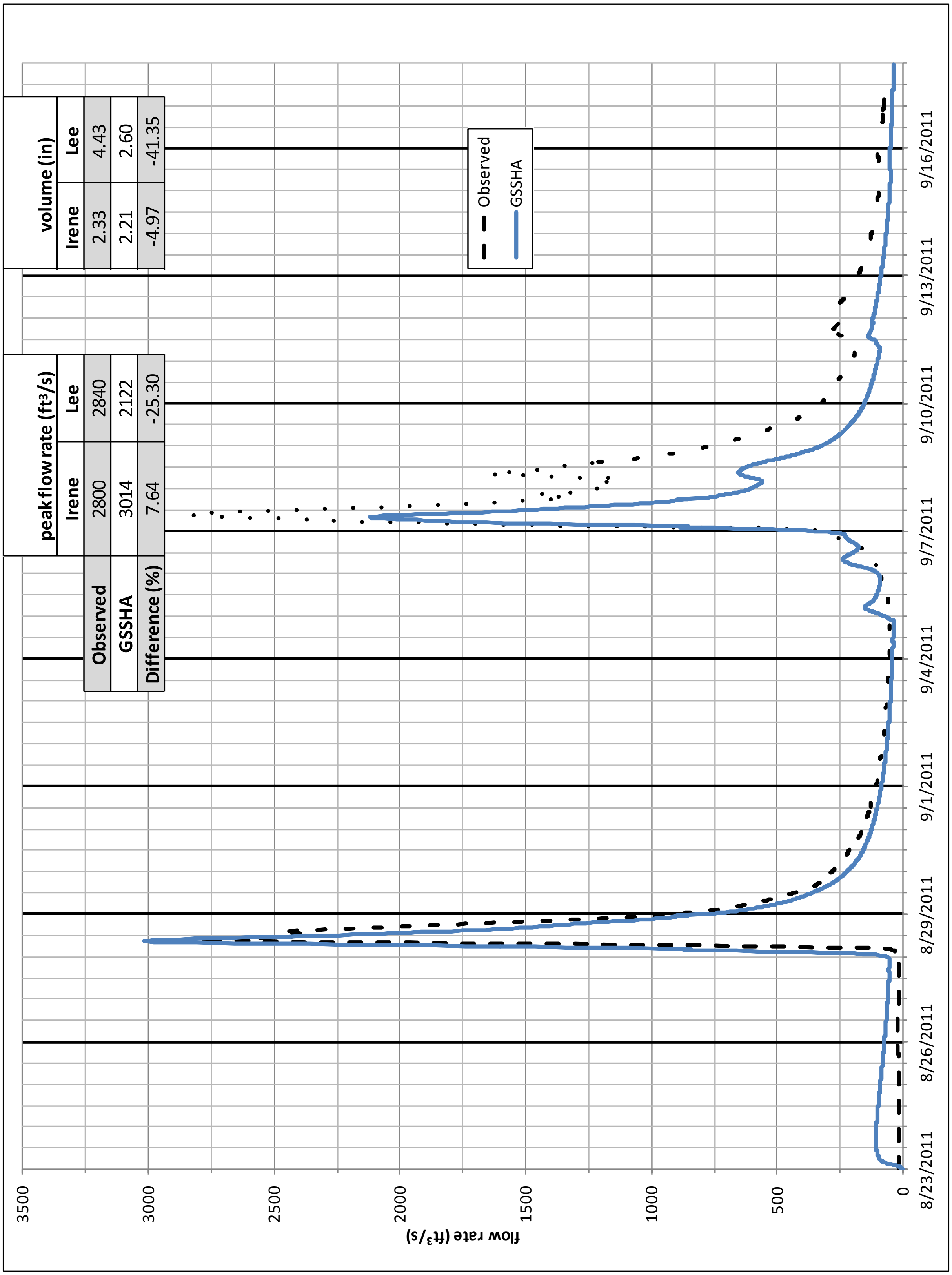


Figure 6.20 Streamflow Validation at Aldenville – Aug-Sep 2011 Event

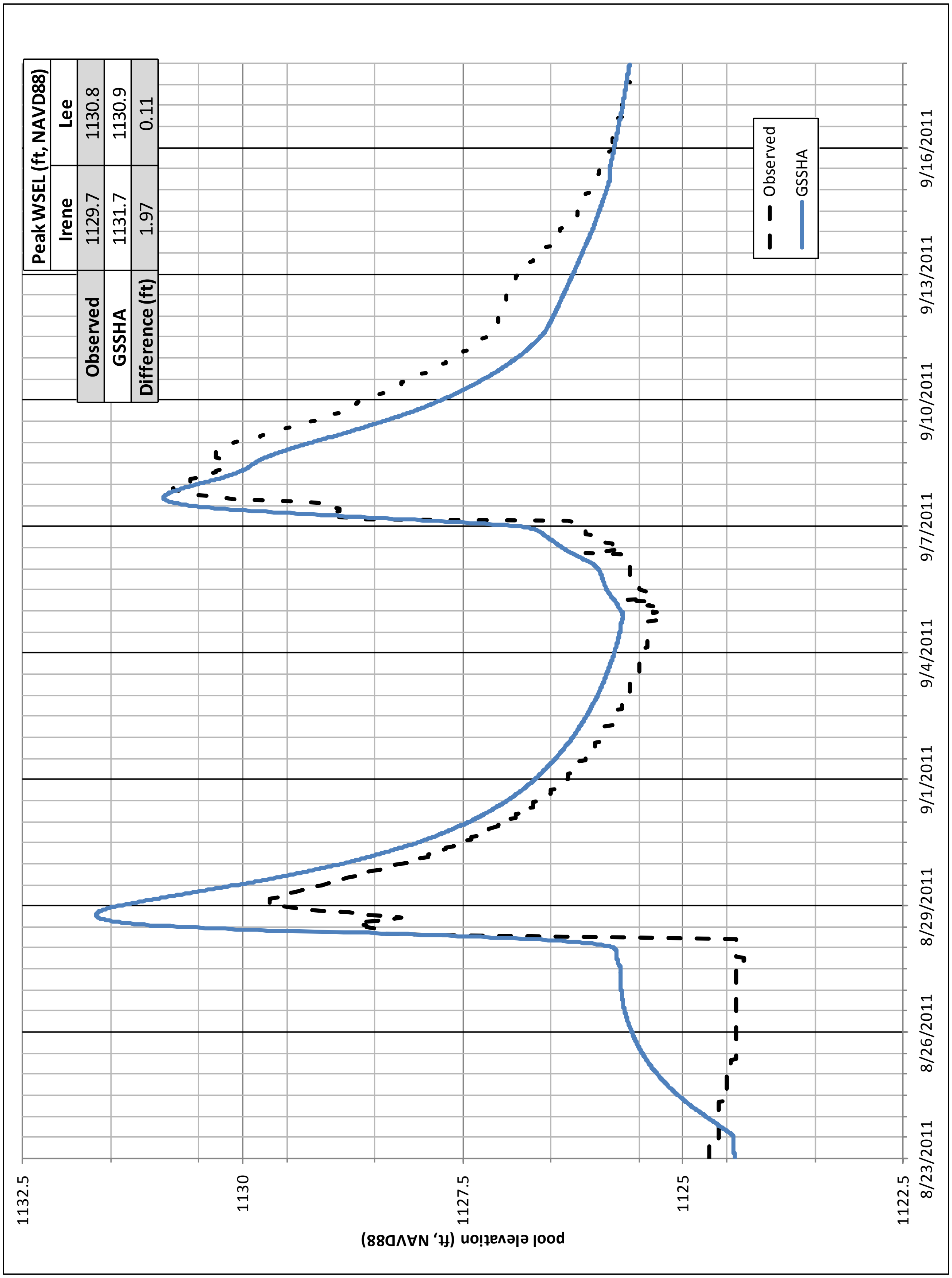


Figure 6.21 Prompton Dam Pool Elevation Validation – Aug-Sep 2011 Event

7. Modeling Processes Comparison

Following the successful calibration and validation of the GSSHA model to various historic events, the individual "components" of the rainfall-runoff process could be compared to the previously developed HEC 1988 Prompton Dam Modification model.

In order to ascertain the differences between varying hydrologic processes (such as infiltration, runoff transform, and baseflow), it was necessary to isolate each process. For all comparisons, the parameter datasets determined during the 1988 HEC Prompton Modification Study were used within the HEC model while the GSSHA model made use of the parameter dataset from the Nov. 1950 and August 1955 calibration events. Using these datasets that were derived from the same calibration events allowed for a more direct comparison of the modeling processes. The following sections detail the results of those comparisons.

7.1. Distributed vs. Lumped

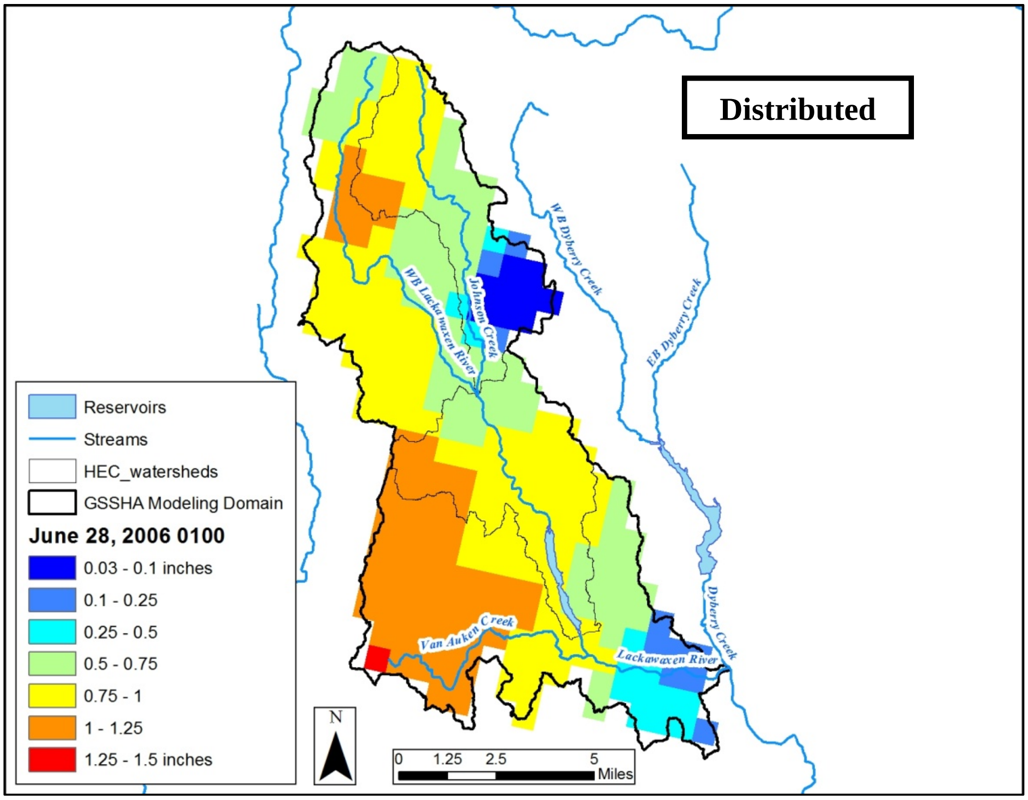
When spatial variations of watershed characteristics, inputs, and modeling processes are considered explicitly, the model may be termed as *distributed*. Conversely, when spatial variations in inputs and processes are ignored or averaged over large areas, the model is termed as *lumped*.[42] The 1988 HEC Prompton Modification Study model made use of lumped parameter estimations while the GSSHA model used spatially distributed parameter estimates. It should be noted that there are quasi-distributed parameter/processes within HEC-HMS (the successor program to HEC-1). However, the vast majority of models used in dam safety studies do not make use of these capabilities.

Generally speaking, distributed models allow for greater accuracy due to fewer errors associated with spatial parameter averaging. An example of this averaging is shown within Figures 7.1 and

[42] (Feldman, 2000)

7.2. These figures show NWS NEXRAD MPE precipitation distributions during the June 2006 event within a single hour (28Jun2006 01:00 EST, Figure 7.1) as well as the total event accumulation (Figure 7.2). During one hour ending on June 28, 2006 01:00 EST, precipitation accumulations of 0.03 and 1 inch were recorded within approximately 2.5 miles in the headwaters of the West Branch Lackawaxen River. However, since the 1988 HEC Prompton Modification Study model did not have subbasins delineated in accordance with this particular precipitation spatial distribution, these large differences in spatial precipitation accumulations are lost. Similarly, total event accumulations of 6.4 and 11 inches occurred within approximately 2.5 miles. Again, these large differences in spatial precipitation distribution are lost due to the averaging within the 1988 HEC Prompton Modification Study model.

Another example of the differences between lumped and distributed parameter models is illustrated within Figure 7.3. This figure shows the cumulative infiltration depths from 26 June 2006 00:00 – 29 June 2006 00:00 and overland depths at 29 June 2006 00:00 within each grid cell during the June 2006 event simulation. Infiltration at a location is dependent upon more than just precipitation and the predominant soil type; infiltration is also reliant upon the amount of water that has been routed to that point. For instance, if two grid cells have the same soil type and underlying infiltration properties but different amounts of water flow to the two different grid cells due to topographical influences, the location with the larger amount of runoff will infiltrate more water over time. Combined with spatially varying precipitation inputs, differing topography, differing groundwater conditions, and differing streamflow conditions, one can see that truly distributed parameter models can allow for much greater accuracy during complicated rainfall-runoff simulations.



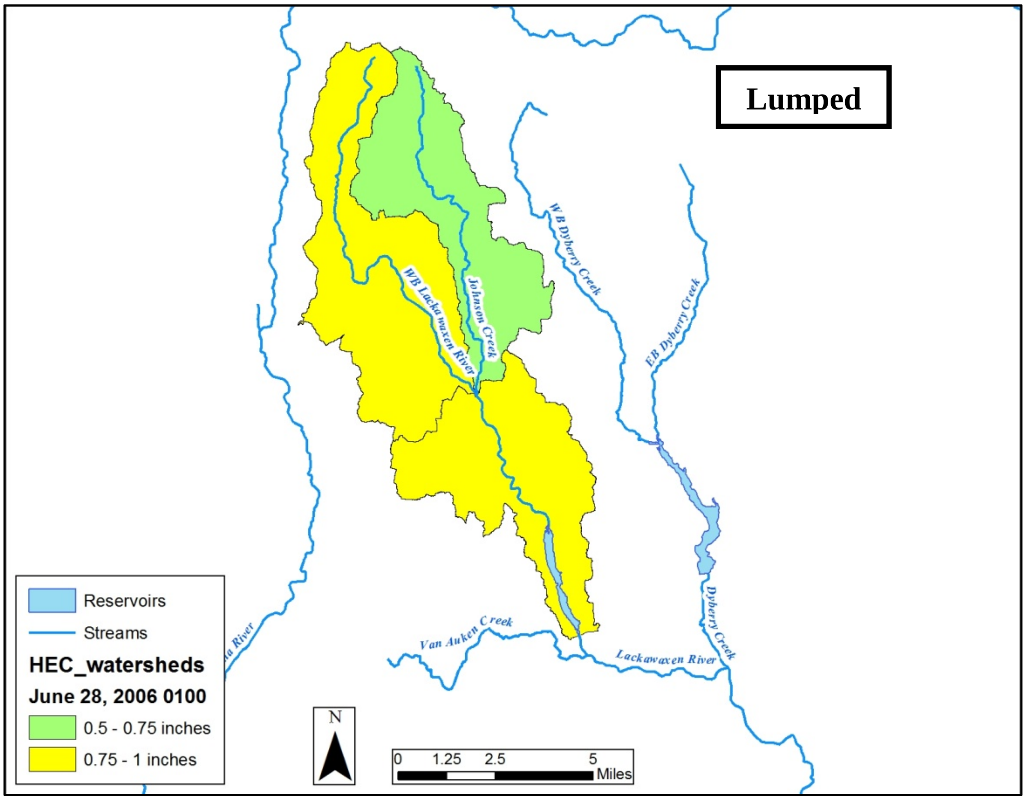


Figure 7.1 June 28, 2006 00:00 – 01:00 EST Precipitation Comparison

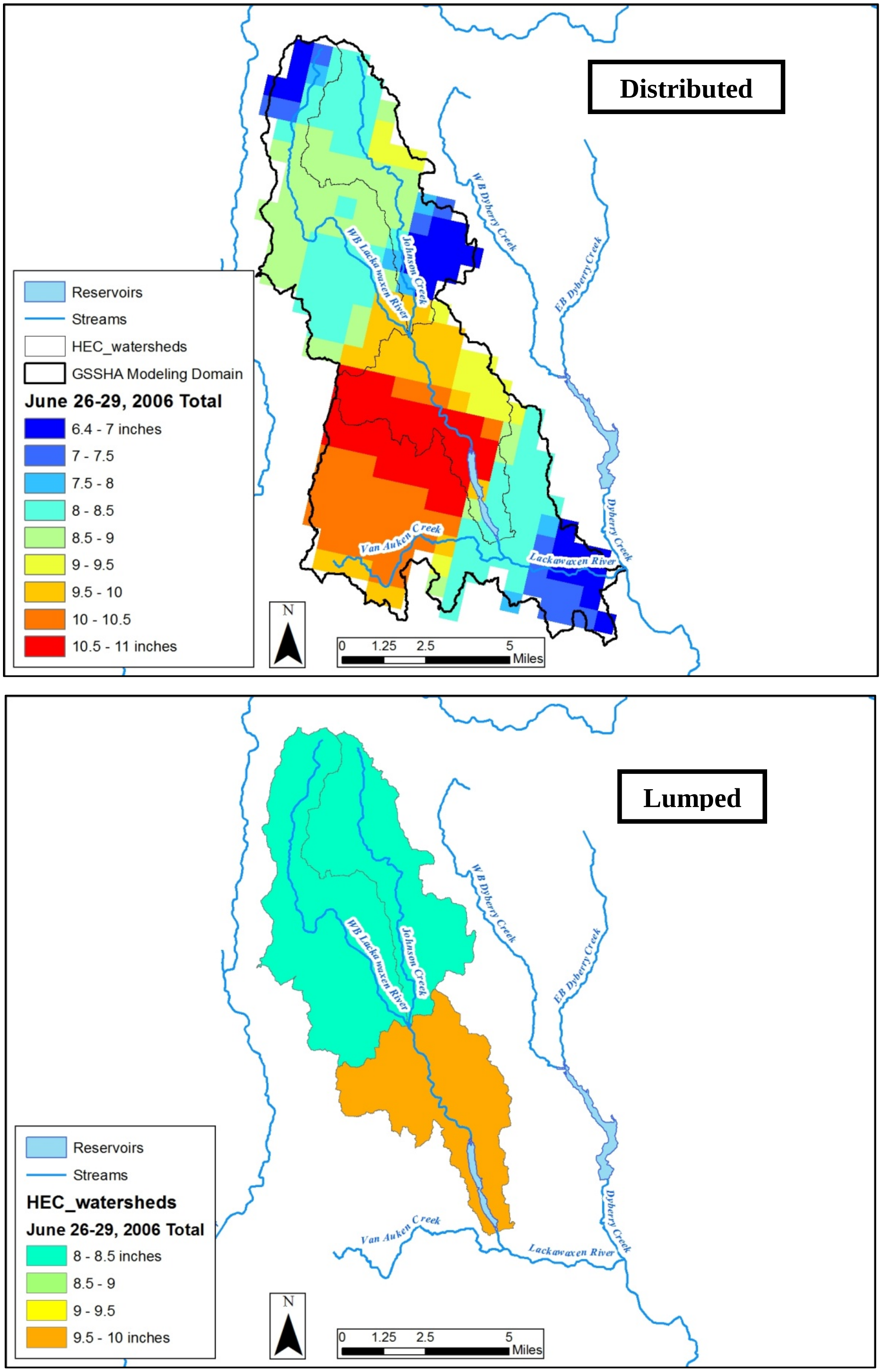


Figure 7.2 June 26 – 29, 2006 Precipitation Accumulation Comparison

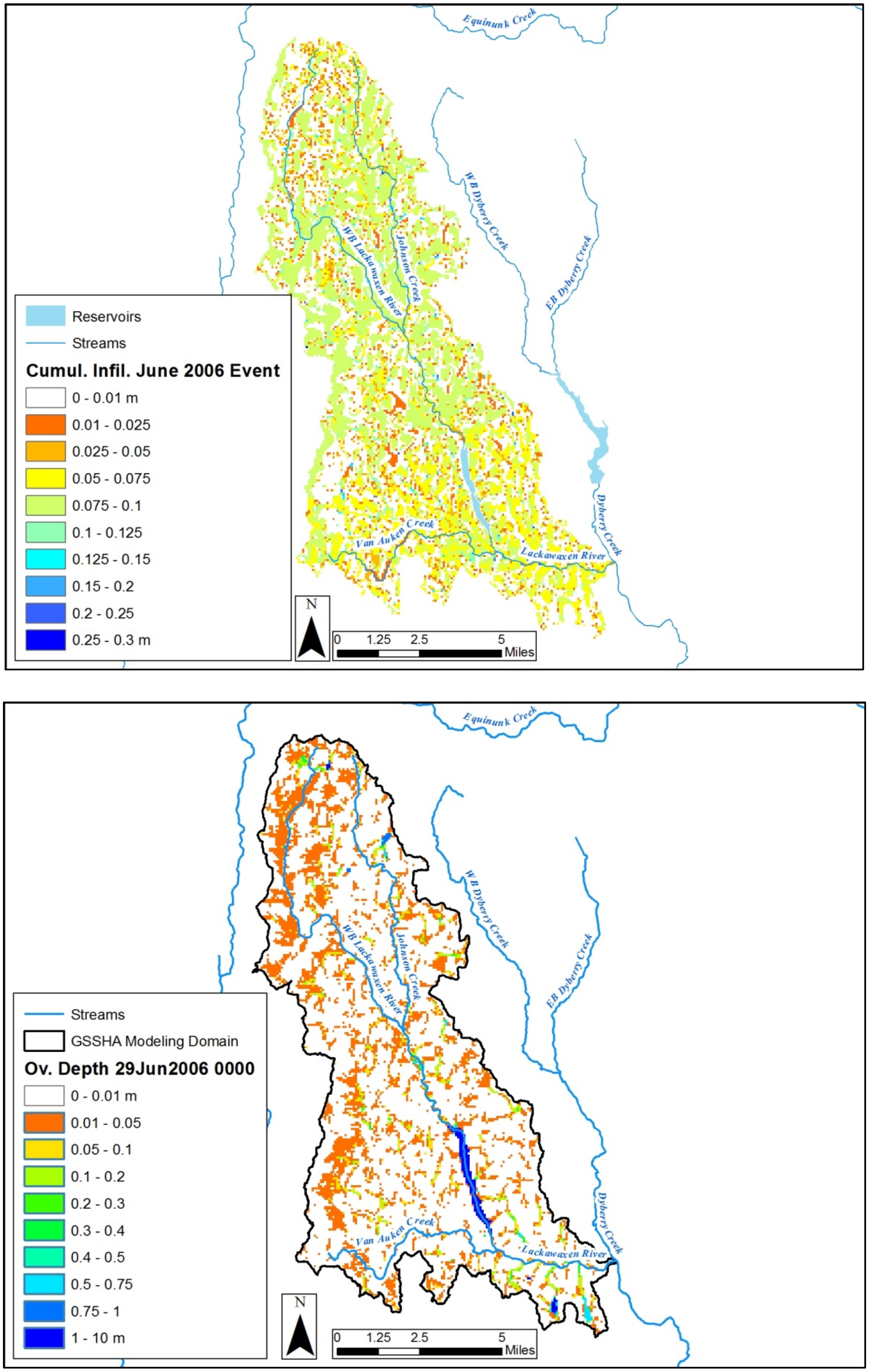


Figure 7.3 Cumulative Infiltration from June 26, 2006 00:00 – June 29, 2006 00:00 (top) and Overland Depths at June 29, 2006 00:00 (bottom)

7.2. Infiltration Comparison

The infiltration algorithm chosen for a hydrologic simulation is extremely important in the realm of dam safety and extreme event simulation. The operation and performance of all reservoirs is dependent upon both flow rates and volumes. However, flow volumes generally play a greater role in pool elevations attained within a reservoir that is operated for flood risk reduction due to common outlet works operations (outflow is greater than inflow for long periods of time) and limited available storage volumes (i.e. most reservoirs cannot store all runoff).

Various simplifications to the complicated infiltration process have been used in hydrologic models in order to predict the volume and timing of runoff. The infiltration routine used within the HEC 1988 Prompton Dam Modification model (initial and constant losses) is the most commonly employed infiltration routine used within dam safety studies, mostly due to its simplicity and ease of use. While numerous studies have shown that infiltration rates decrease as greater volumes of rainfall are infiltrated,[43,44,45] the initial and constant loss method assumes that infiltration proceeds at a constant rate once initial losses are satisfied.[46] Also, as employed within most hydrologic modeling codes, excess precipitation is not subjected to additional infiltration computations using the initial and constant loss routine. Once rainfall is considered "excess", it is not subject to additional infiltration throughout the remainder of the simulation (unless user-input loss rates are simulated within a channel routing segment).

The Green Ampt infiltration routine within the GSSHA model uses a physically-based simplification of the governing Richards Equation.[47] Being a physically-based process, the

[43] (Green & Ampt, 1911)
[44] (Rawls & Brakensiek, 1983)
[45] (Mein & Larson, 1973)
[46] (Feldman, 2000)
[47] (U.S. Army Corps of Engineers, 1994)

Green Ampt routine is not subject to the same inherent limitations as the initial and constant loss rate routine. Infiltration can occur anywhere and in the modeling domain including the overland grid, within stream segments, and reservoirs. Also, infiltration can occur at any time; losses continue to impact runoff throughout the length of the simulation as the infiltration process does not stop once precipitation is termed “excess”.

In order to compare the results from the two different infiltration routines, a hypothetical storm event was used which eliminated any differences in spatial and/or temporal rainfall distribution. Only infiltration and runoff transform processes were simulated in both models; baseflow was not simulated in the HEC model and groundwater effects were neglected in the GSSHA model. This event was comprised of precipitation at a constant rainfall rate of one inch/hour for three hours totaling three inches. The previously mentioned representative set of model parameters determined during the calibration process from the Nov. 1950 and August 1955 events were used within the GSSHA model while the 1988 HEC Prompton Modification Study model HEC-1 model made use of the final parameter set developed during the 1988 HEC Prompton Modification Study.

It is important to note that the GSSHA model allows for a dynamic pool which rises and falls in relation to the inflow, outflow, and available storage. Two scenarios were modeled: one in which routing through the length of pool was simulated using a static pool (“GSSHA – Static Pool”) and one in which the dynamic pool was allowed to fluctuate (“GSSHA – Dynamic Pool”). The first scenario allowed for the direct measurement of flow at the dam embankment while the second scenario required inflow to be indirectly determined through changes in outflow and storage since the actual “inflow point” changed throughout the simulation. Determining inflow through changes in storage and outflow can lead to slight underestimates of flow volumes, especially during the receding limb of the inflow hydrograph. The 1988 HEC Prompton Modification Study model made use of a routing reach within the Prompton Reservoir.

The runoff hydrographs at the Aldenville gage location and Prompton Reservoir inflow are shown in Figures 7.4 and 7.5. The hydrographs at both the Aldenville gage location and Prompton Dam inflow demonstrate that the 1988 HEC Prompton Modification Study model estimates greater inflow volumes from direct runoff than the GSSHA model. These differences are due to the dissimilarities between the Green Ampt routine used within the GSSHA model and the initial and constant loss rate routine used within the 1988 HEC Prompton Modification Study model as well as input parameter values.

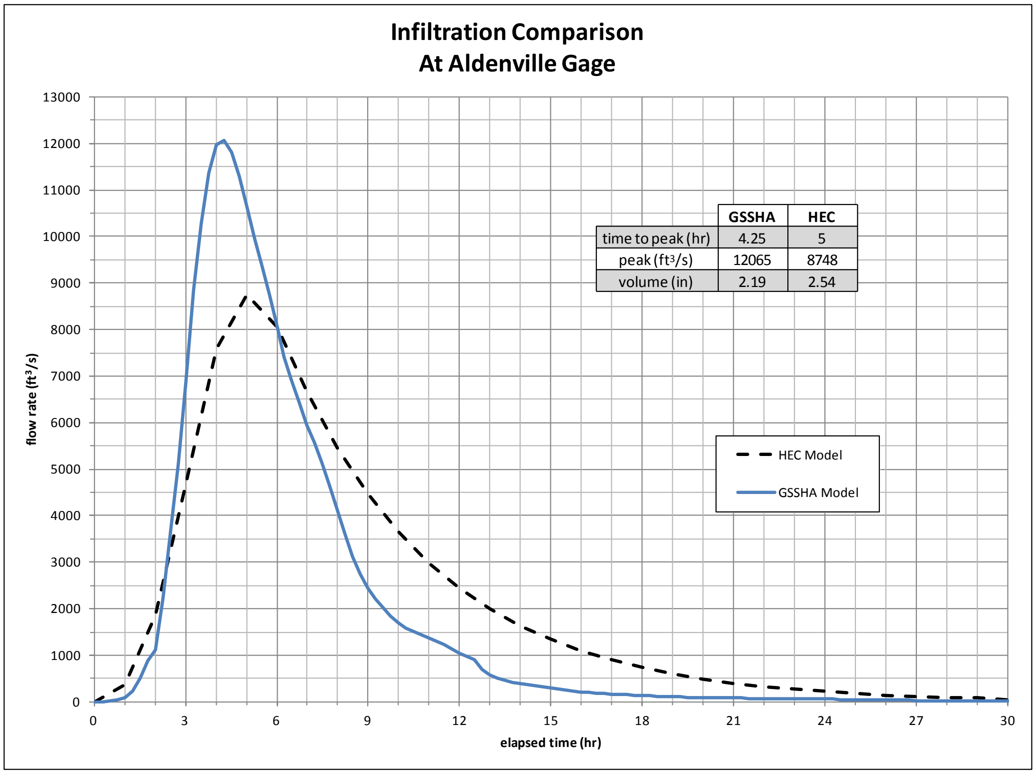


Figure 7.4 Infiltration Comparison at Aldenville Gage

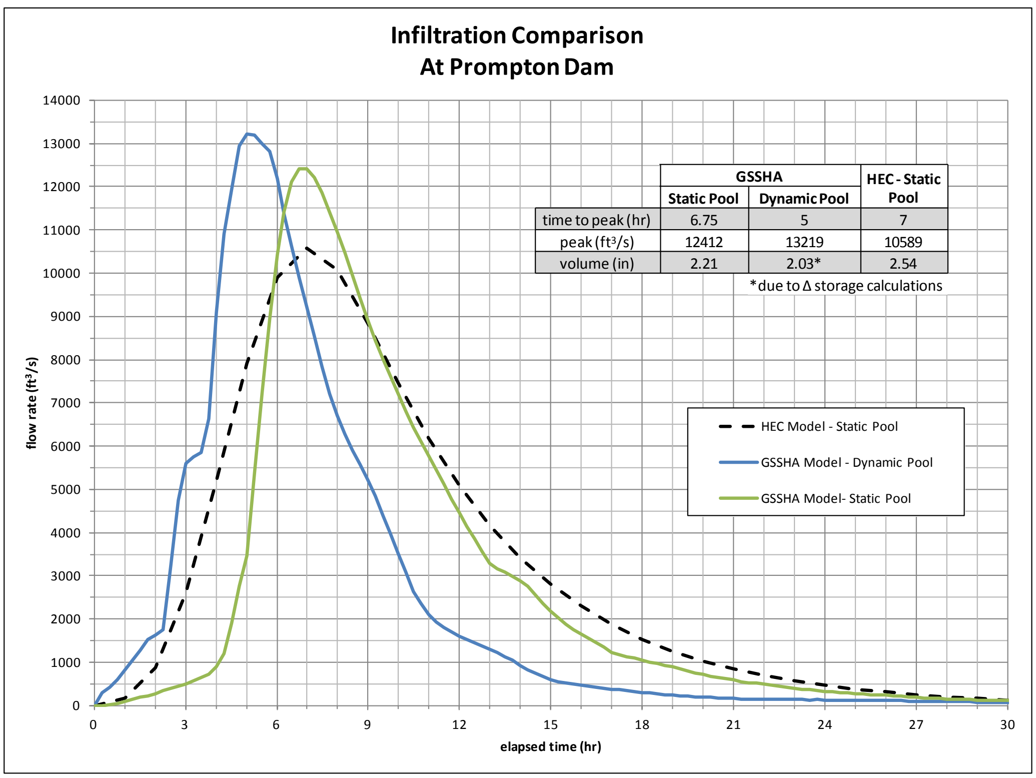


Figure 7.5 Infiltration Comparison for Prompton Dam Inflow

7.3. Unit Hydrograph Comparison

Unit hydrographs are used to predict the timing and magnitude of runoff following infiltration. Unit hydrographs relate excess precipitation and the resultant runoff without detailed consideration of complicated internal processes.[48] As such, the equations and parameters within unit hydrograph transforms tend to have limited physical significance. To "ground truth" these routines, parameters are optimized through a calibration process where model outputs are compared to observed data in order to achieve an adequate "fit".

Normally, calibrated unit hydrographs are dependent upon the magnitude of the event being investigated. This is a consequence of the derivation of a unit hydrograph. According to Sherman, who originally proposed the unit hydrograph concept, the unit hydrograph of a watershed is "…the basin outflow resulting from one unit of direct runoff generated uniformly over the drainage area at a uniform rainfall rate during a specified period of rainfall duration."[49] This implies that ordinates of any hydrograph resulting from a quantity of runoff-producing rainfall of unit duration would be equal to corresponding ordinates of a unit hydrograph for the same areal distribution of rainfall, multiplied by the ratio of rainfall excess values.[50] However, due to differences in areal distributions of rainfall and hydraulic reactions between large and small precipitation events, the corresponding unit hydrographs have not been found to be equal, as implied by unit hydrograph theory. This realization must also be combined with two factors: 1) Most precipitation events used for calibrating infiltration losses and unit hydrograph transforms are normally much less intense than the PMP for a given area; 2) ER 1110-8-2 directs USACE personnel to use loss rates and unit hydrographs that will result in rapid runoff

[48] (Feldman, 2000)
[49] (Sherman, 1932)
[50] (U.S. Army Corps of Engineers, 1959)

conditions. As such, a rule of thumb has been in use for over 50 years within the USACE community to peak unit hydrographs when determining a PMF/IDF by 25 – 50 percent.[51] In most PMF/IDF investigations, the applicability of this peaking factor is not explicitly analyzed. It is unknown whether a 25% peaking factor over-predicts the true unit hydrograph of a watershed in response to the PMP. Also, it is unknown whether a 50% peaking factor under-predicts the true unit hydrograph.

In rare instances, this rule of thumb peaking factor is not applied when sufficient justification is supplied. The 1988 HEC Prompton Modification Study did not use a unit hydrograph peaking factor since the unit hydrographs for the three modeled subbasins above Prompton Dam were estimated from a (believed) very conservative relationship.[52]

To compare the predicted unit hydrographs from both models at various locations, one inch of rainfall over varying durations, without any losses or baseflow/groundwater contribution, was input to both the GSSHA model and the HEC 1988 Prompton Dam Modification model. The results were compared at the Aldenville gage location and inflow to the Prompton Reservoir.

Figures 7.6 and 7.7 compare the HEC-determined one-hour unit hydrographs at the Aldenville gage location and Prompton Reservoir inflow to the GSSHA-determined unit hydrographs. The GSSHA model predicted a unit hydrograph with an approximately 30% higher peak flow rate than the HEC model at the Aldenville gage location with both unit hydrographs having a similar time to peak flow. A peaking factor of approximately 40% is required to match the GSSHA unit hydrograph at this location for a one-hour rainfall duration. However, at the Prompton Reservoir, the differences between the HEC and GSSHA models become less pronounced with

[51] (U.S. Army Corps of Engineers, 1991)
[52] (Hydrologic Engineering Center, 1988)

peak flow rates being approximately equal and the HEC model unit hydrograph time to peak flow falling between the two GSSHA model simulations. These attenuation and translation effects are due to the drastic changes in stream and overland slopes that occur downstream of the Aldenville gage.

As expected, the differences between the GSSHA model and HEC model decreases with unit precipitation duration. This is due to the fact that the simulation becomes less “dynamic” over time as unit precipitation duration increases (i.e. less of a pronounced flood wave, etc).

Figures 7.8 – 7.13 detail the comparisons between the GSSHA model and HEC model for 3-hr, 6-hr, and 12-hr durations.

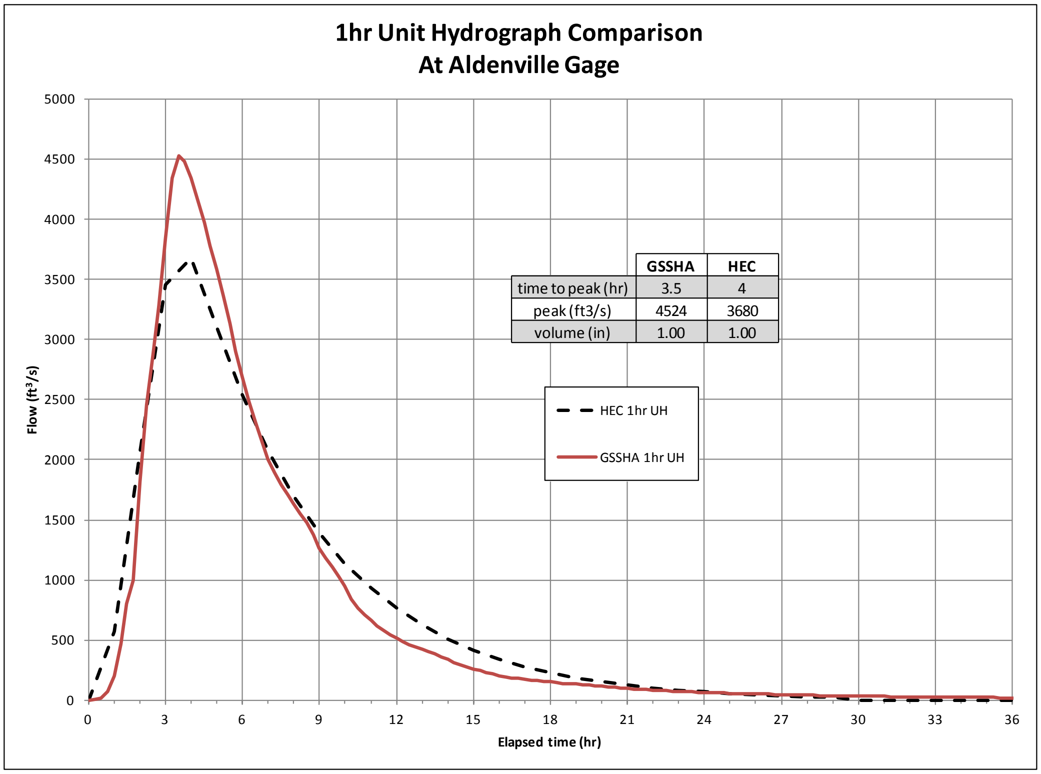


Figure 7.6 1hr Unit Hydrograph Comparison at Aldenville Gage

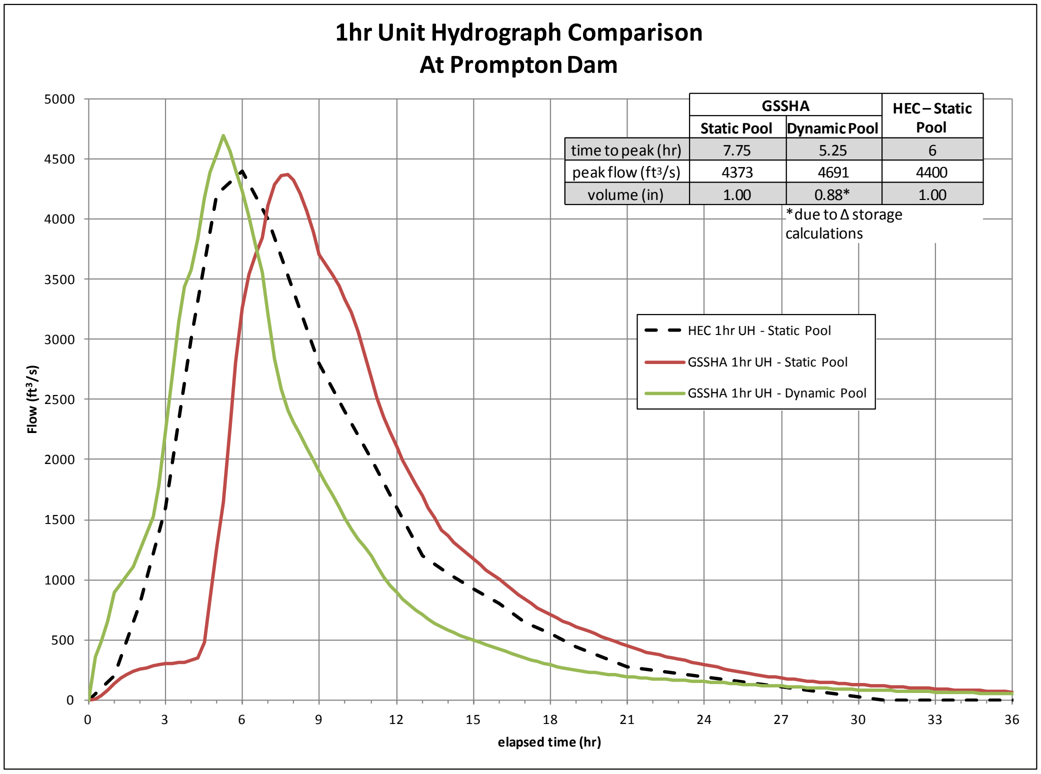


Figure 7.7 1hr Unit Hydrograph Comparison for Prompton Dam Inflow

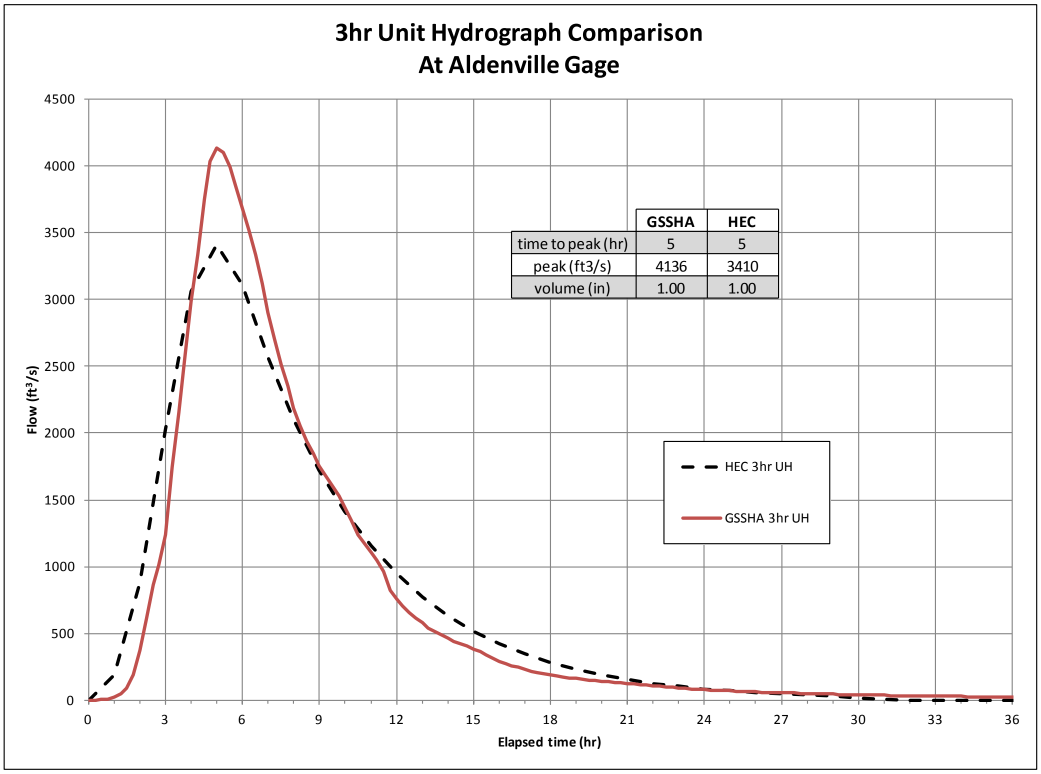


Figure 7.8 3hr Unit Hydrograph Comparison at Aldenville Gage

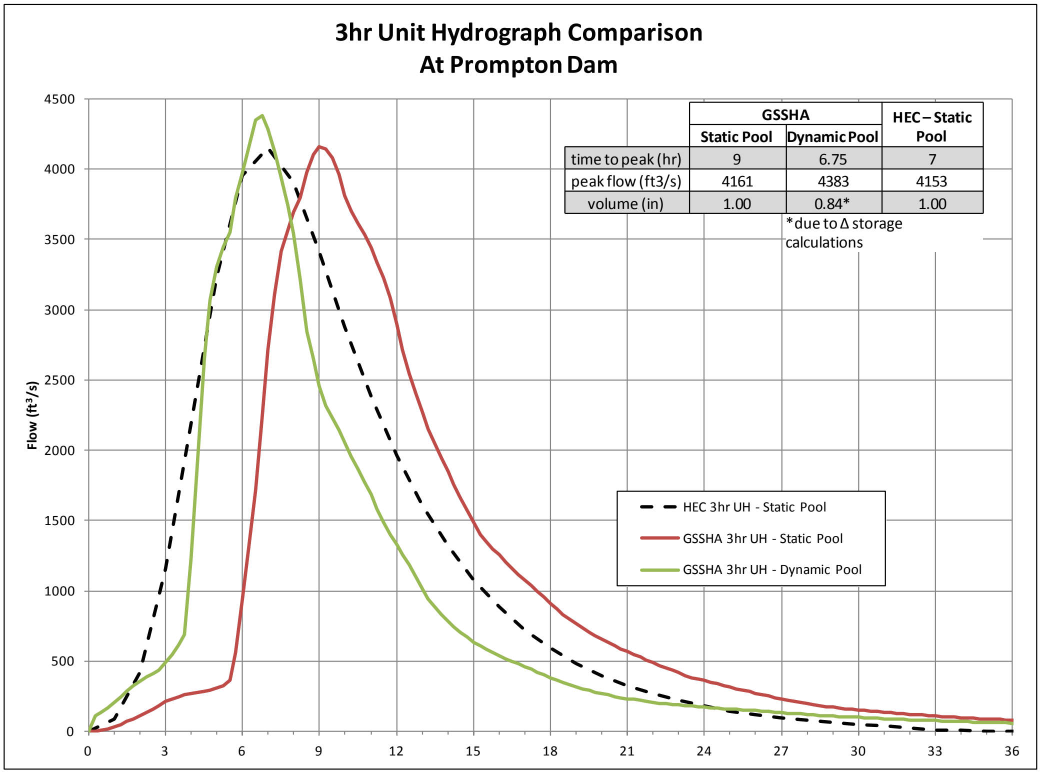


Figure 7.9 3hr Unit Hydrograph Comparison for Prompton Dam Inflow

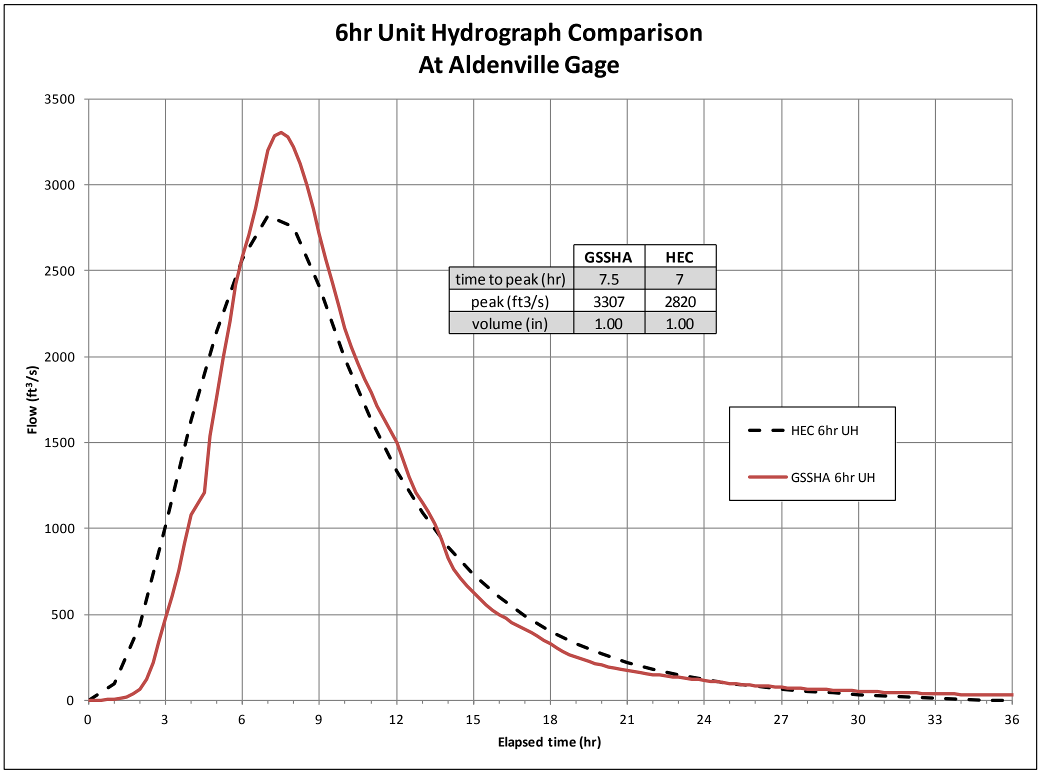


Figure 7.10 6hr Unit Hydrograph Comparison at Aldenville Gage

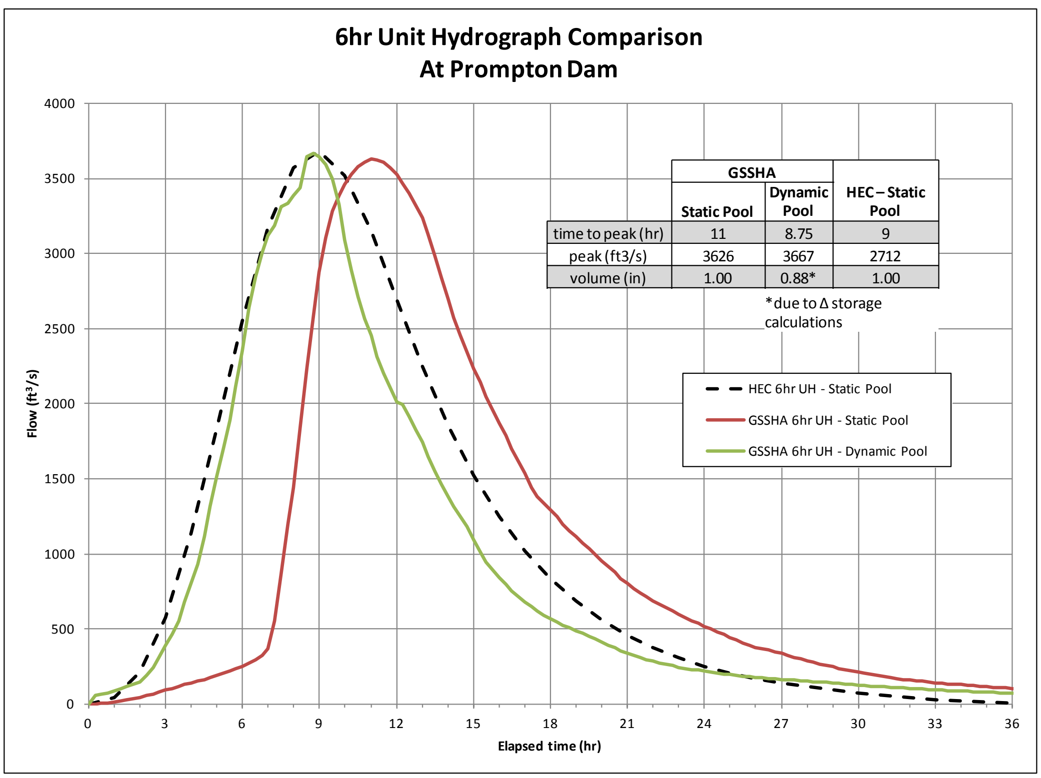
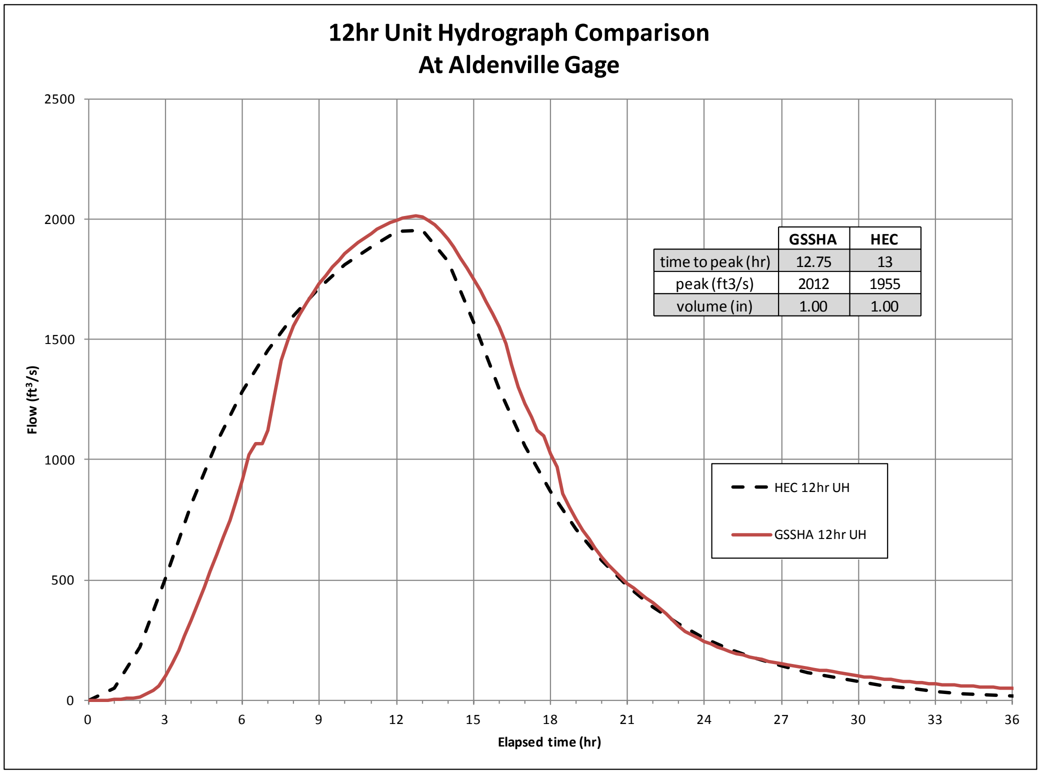
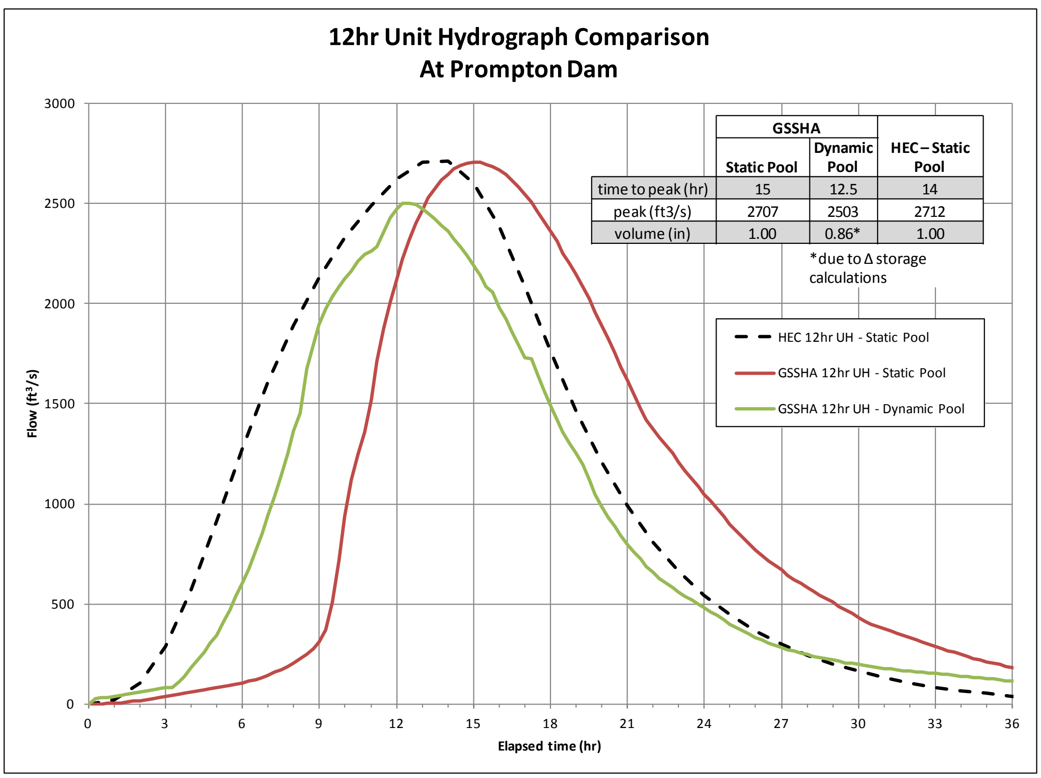


Figure 7.11 6hr Unit Hydrograph Comparison for Prompton Dam Inflow



	GSSHA	HEC
time to peak (hr)	12.75	13
peak (ft3/s)	2012	1955
volume (in)	1.00	1.00

Figure 7.12 12hr Unit Hydrograph Comparison at Aldenville Gage



	GSSHA		HEC – Static Pool
	Static Pool	Dynamic Pool	
time to peak (hr)	15	12.5	14
peak (ft3/s)	2707	2503	2712
volume (in)	1.00	0.86*	1.00

Figure 7.13 12hr Unit Hydrograph Comparison for Prompton Dam Inflow

7.4. Normalized One-Hour Storm Comparisons

The linear assumptions contained within unit hydrograph theory are the primary reason that USACE requires unit hydrograph parameters be peaked by 25 to 50 percent for dam / spillway design studies. The implicit linearity of the unit hydrograph theory can be demonstrated by routing hour long precipitation amounts greater than one inch and dividing the resulting hydrographs by the input precipitation amount. For instance, if one were to input a spatially uniform two inch/hour hyetograph for a total of one hour to a watershed with no losses, use unit hydrograph theory to route the runoff, and then divide the resulting hydrograph at any point by two, the ensuing hydrograph would be equal to the one-hour unit hydrograph of the same watershed.

However, inputting the same hour-long hyetograph to a watershed within GSSHA and dividing by the rainfall amount does not result in the watershed's unit hydrograph. For increasing rainfall amounts, significant runoff is generated earlier, peak flow rates are higher, and flow rates on the recession limb of the hydrograph are quicker to decrease. This non-linear result is shown in Figure 7.14. For the purposes of this comparison, the hydrograph resulting from a two inch/hour hyetograph divided by two is referred to as "UH2", three inches/hour divided by three as "UH3", etc.

It should be noted that analyses comparing a watershed's unit hydrograph to the response of increasingly more intense precipitation amounts over a single hour can be completed in various hydrologic and hydraulic models. For example, the two-dimensional routing capabilities within HEC's River Analysis System (HEC-RAS) have been used to perform an analysis similar to the one presented here for Success Dam in California.

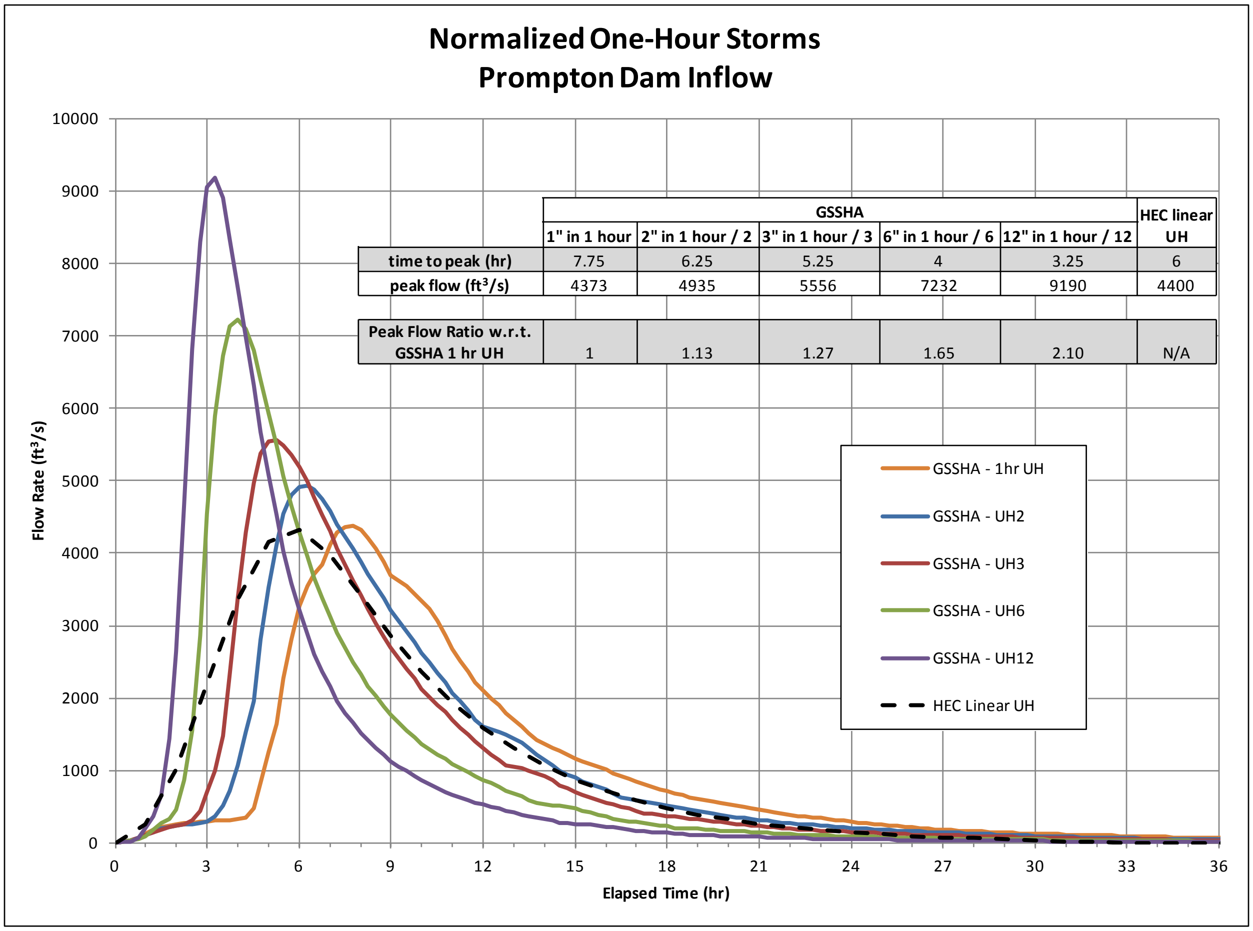


Figure 7.14 Normalized One-Hour Storms – Inflow to Prompton Dam

In addition to better demonstrating the short comings of unit hydrograph theory, this type of analysis is especially useful in the realm of dam safety. For instance, modelers are required to determine the effects of many different PMP/PMS arrangements in order to determine the storm that results in the peak inflow rate, maximum inflow volume, and other critical flow/volume combinations to a specific reservoir. Using these rainfall intensity-appropriate unit hydrographs can allow a modeler to quickly determine the resulting runoff in a spreadsheet-type hydrologic model therefore saving potentially significant computing time.

Additionally, dam safety analyses oftentimes require the variation of many different parameters, including precipitation amounts, temporal distributions, seasonality (time of year), antecedent pool elevation, etc. The effects of these parameters on the ultimate pool elevation are best investigated through the use of Monte Carlo techniques, which may require many thousands of model iterations.[53]

Finally, these hydrographs can be used to predict more appropriate unit hydrograph peaking coefficients for use in extreme event simulations, as required by ER 1110-8-2 (FR). For instance, if the PMP/PMS to a specific dam is comprised of periods of rainfall accumulations between three inches/hour and six inches/hour, it may be useful to compare the peak flow rate of the UH3 hydrograph (three inches/hour) to the normal unit hydrograph (one inch/hour) and the UH6 hydrograph (six inches/hour) to the normal unit hydrograph. For instance, the results shown in Figure 7.14 show that Prompton Dam has a UH3 / unit hydrograph ratio of 1.27 and a UH6 / unit hydrograph ratio of 1.65. Depending upon the temporal and spatial distribution of the PMP, unit hydrograph peaking factors ranging from 1.25 – 1.65 are warranted.

7.5. Groundwater Routing/Baseflow Comparison

For reasons similar to the chosen infiltration algorithm, the groundwater flow/baseflow algorithm is extremely important in the realm of dam safety and extreme event simulation. A streamflow hydrograph is commonly broken down into two distinguishable parts: direct runoff of excess precipitation and baseflow. Baseflow is commonly designated as the sustained runoff that was temporarily stored within the watershed prior to a runoff event in addition to the delayed subsurface runoff from the current storm. As such, more complicated flow processes such as interflow are commonly lumped into either direct runoff or baseflow.[54]

[53] (Stedinger, Heath, & Thompson, 1996)
[54] (Chow, Maidment, & Mays, 1988)

The recession baseflow model, as implemented in HEC-1 and HEC-HMS, defines the relationship of baseflow at any time during the simulation to an initial value which displays an exponential decay (a.k.a. “recession”) over time.[55] The recession baseflow model also uses an initial flow value at time = 0 and a user-specified ratio-to-peak value to delineate the time when the recession model defines the total streamflow. Summing baseflow and direct runoff together results in the total streamflow hydrograph. There is no direct connection between infiltrated water and baseflow. As a result, there is no guarantee that volume is conserved throughout a simulation. The user is required to verify that baseflow rates and volumes do not exceed those that are physically possible.

The GSSHA model has the ability to link surface water processes to a simplified two-dimensional groundwater flow network. Water can be infiltrated in one location, percolate through the underlying soil column, enter the saturated groundwater network, be routed to another location, exit the groundwater network, and be discharged to the stream system, soil column, and/or surface (depending upon prevailing conditions). This allows users to create an explicit link between infiltrated precipitation/runoff and baseflow thereby conserving volume during a simulation. It should be noted that HEC-HMS has the ability to link infiltration rates and volumes to baseflow flow rates and volumes using the linear reservoir baseflow routing routine. This baseflow routine was designed to conserve volume.

Similar to the infiltration comparison, a hypothetical storm event comprised of precipitation at a constant rainfall rate of one inch/hour for three hours totaling three inches was used to compare the results from the two different groundwater routing/baseflow routine. While the HEC model allowed for the direct output of the baseflow component of the streamflow hydrograph, this

[55] (Feldman, 2000)

component needed to be indirectly computed for the GSSHA model results. This was done by subtracting the hydrographs computed during the infiltration comparison from those determined in this comparison.

At the Aldenville gage location the HEC model predicted a small increase in peak flow rate due to the addition of a baseflow component, as shown in Table 7.15. However, an additional two inches of runoff volume was calculated due to the addition of baseflow. Conversely, the GSSHA model predicted a peak flow and runoff volume increase of 213 ft^3/s and 0.17 inches, respectively.

The HEC model predicted a Prompton Dam peak inflow increase of 56 ft^3/s and runoff volume increase of 1.91 inches due to baseflow. The GSSHA model predicted an increase in peak flow rate; however the predicted increase in runoff volume due to baseflow was much less (only 0.43 inches).

Table 7.15 Peak Flow Rate and Runoff Volume Differences due to Baseflow

Aldenville Gage				***Prompton Dam Inflow***		
				GSSHA		**HEC - Static Pool**
Parameter	**GSSHA**	**HEC**		**Static Pool**	**Dynamic Pool**	
peak flow rate (ft^3/s)	213	39		280	-129*	56
volume (in)	0.17	2.0		0.43	0.11	1.9

*due to Δ storage calculations

The runoff hydrographs at the Aldenville gage location and Prompton Reservoir inflow from this event for both the 1988 HEC Prompton Modification model and GSSHA model are shown in Figures 7.16 and 7.17. In these graphs, the baseflow components for both models are shown as dotted lines.

These plots easily demonstrate the differences between the groundwater routing/baseflow routines used within the HEC model and GSSHA model. The GSSHA model shows only relatively small baseflow contributions to the total streamflow hydrograph at both Aldenville and Prompton Dam. On the other hand, the HEC model (using the input parameters determined during the 1988 HEC Prompton Dam Modification Study) shows a drastic difference in streamflow rates and volumes due to baseflow. At both locations, total flow volumes of approximately 4.5 inches are not physically possible from a 3 inch rainstorm event. This displays the lack of physical connection between infiltration and baseflow when using the initial and constant loss rate and recession baseflow routines; the user is required to verify the applicability of the model outputs.

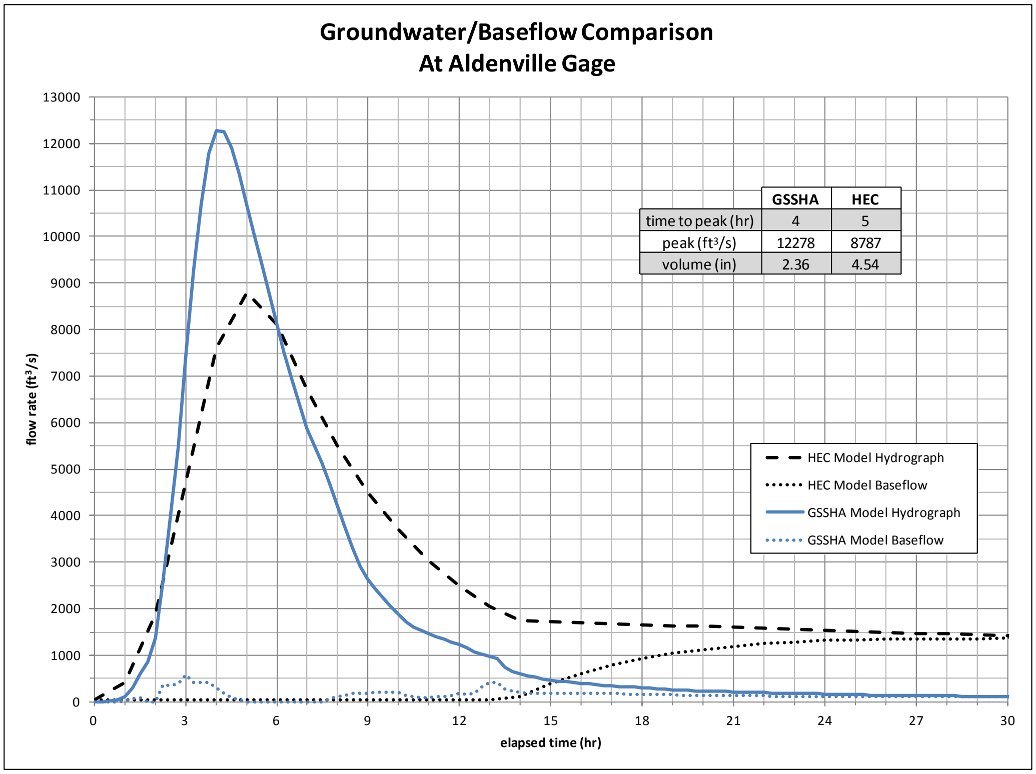


Figure 7.16 Baseflow Comparison at Aldenville Gage

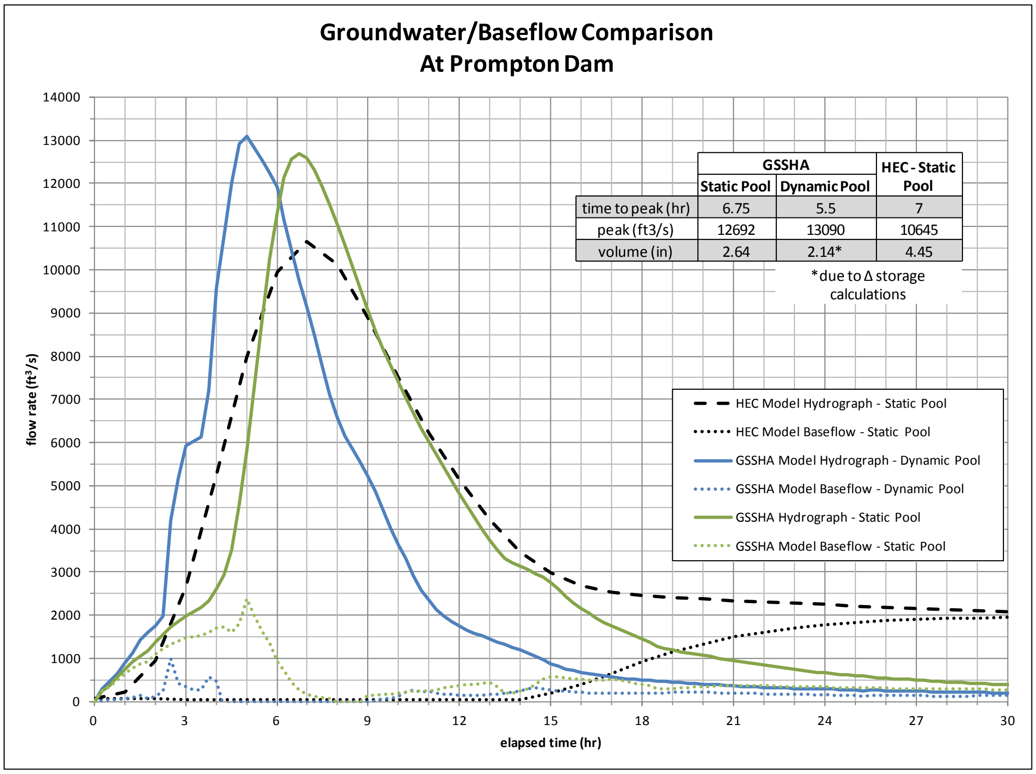


Figure 7.17 Baseflow Comparison for Prompton Dam Inflow

8. Extreme Event Simulation Comparison

The PMF and IDF events were simulated within the GSSHA model and compared against the results obtained from the 1988 HEC Prompton Modification Study. Due to the uses, size, and potential consequences of a failure of Prompton Dam, the IDF was selected as the full PMF.[56] For these simulations, the input parameter datasets determined during the 1988 HEC Prompton Modification Study were used within the HEC model. Meanwhile, the GSSHA model made use of the minimum calibrated parameter estimates obtained from both the June 2006 and Oct. – Nov. 2006 calibration events. This is in accordance with ER 1110-8-2 (FR) that directs USACE personnel to use rainfall to runoff conversions that correspond to patterns favorable for rapid concentrations of runoff from the drainage basin.[57]

Since different events were used to develop the input parameters for both models, this doesn't necessarily result in a true "apples-to-apples" comparison. However, this does allow for a better prediction of the runoff response during the PMF event using the GSSHA model due to the higher intensity of the June 2006 and Oct. – Nov. 2006 events as well as the better data used for model calibration.

In all of these simulations, if the pool elevation exceeded the dam crest elevation, flow over the top of the dam was not allowed. This is analogous to assuming that the dam is raised to prevent overtopping.

8.1. Probable Maximum Storm Creation

A Probable Maximum Storm (PMS) was created for use in the GSSHA model. Efforts were made to ensure the resultant precipitation was the same as the 1988 HEC Prompton

[56] (U.S. Army Corps of Engineers - Philadelphia District, 1993)
[57] (U.S. Army Corps of Engineers, 1991)

Modification Study. However, due to the differences in precipitation input between the two models (basin-averaged vs. spatially-distributed), steps needed to be taken to recreate the PMS in a usable format.

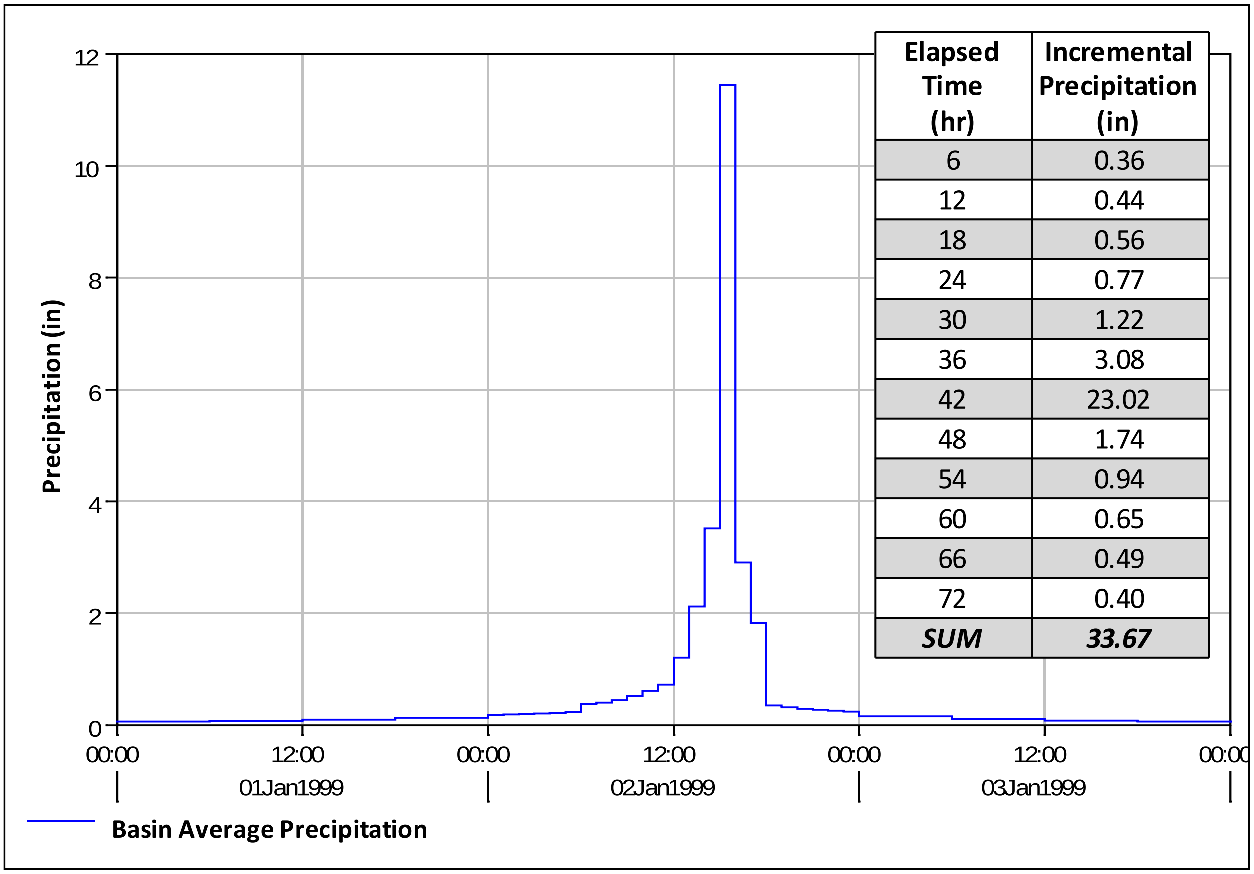
First, all-season Depth-Area-Duration (DAD) values were taken from HMR 51 and the 1988 HEC Prompton Modification Study. These DAD values needed to be increased by 5% due to Prompton Dam's location within the stippled region of HMR 51, as per recommendations from the NWS. The resultant DAD values for 6- through 72-hr durations and 10 through 20,000 square mile areas are shown in Table 8.1.

These DAD values were then input to the HEC program HMR52 to create input hyetographs.[58] First, a basin-averaged hyetograph was created for the contributing watershed to Prompton Dam. This hyetograph is shown in Figure 8.2 and was compared to the basin-averaged hyetograph presented in the HEC 1988 Prompton Modification Study to ensure that there were no differences in precipitation amounts.

Table 8.1 Prompton Dam Basin Average 72-hr Probable Maximum Precipitation – Depth-Area-Duration Values

Duration (hr)	**Area (mi^2)**					
	10	**200**	**1000**	**5000**	**10000**	**20000**
6	27.3	20.0	14.2	7.9	6.0	4.2
12	31.5	22.1	16.8	11.6	9.5	7.4
24	33.6	25.2	20.0	14.2	11.6	9.5
48	37.8	28.4	23.1	17.3	14.2	12.1
72	38.9	29.4	24.2	18.4	15.5	13.1

[58] (Hydrologic Engineering Center, 1987)



Elapsed Time (hr)	Incremental Precipitation (in)
6	0.36
12	0.44
18	0.56
24	0.77
30	1.22
36	3.08
42	23.02
48	1.74
54	0.94
60	0.65
66	0.49
72	0.40
SUM	***33.67***

Figure 8.2 Prompton Dam Basin Average 72-hr Probable Maximum Precipitation Hyetograph

One-Hour Ordinates

Then, an SHG1000 grid was used to create "individual subbasins" for input to the HMR52 program. A total of 720 grid cells were required to completely encompass the contributing area to Prompton Dam. Consequently, 720 individual hyetographs were created by the HMR52 program upon execution. These hourly hyetographs were then transformed into hourly ASCII grids for input to the GSSHA program, which resulted in a total of 72 ASCII grids. These ASCII grids were summed over the 72-hr period and converted to an ESRI grid, which is shown in Figure 8.3. The maximum and minimum 72-hr precipitation accumulation for a single grid cell throughout the modeling domain was 35.7 and 6.9 inches, respectively.

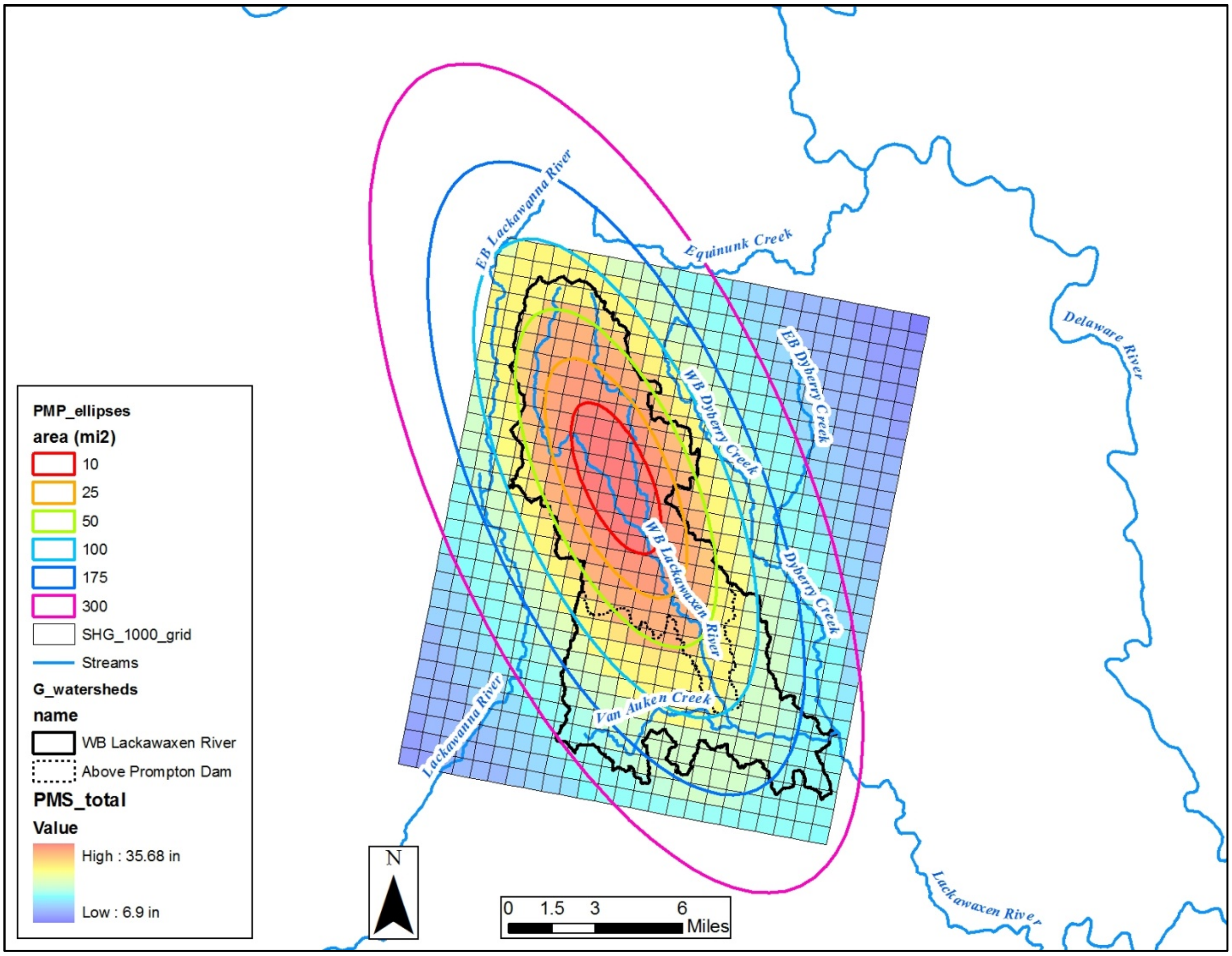


Figure 8.3 Prompton Dam Probable Maximum Storm Total Precipitation

Approximately 12 hours (1.5 working days) was required to source and format the PMP data for input to HMR52. Another 24 hours (3 working days) was necessary to execute HMR52, construct the precipitation grids, and format the precipitation grids for use in the GSSHA model.

8.2. Probable Maximum Flood Routing

The PMF event was simulated within the GSSHA model using the previously mentioned PMS. Additionally, the PMS hyetographs taken from the 1988 HEC Prompton Modification Study were input to the HEC-1 model to recreate results for comparison. These results were verified to match the available output from the 1988 HEC Prompton Modification Study report. Both models used an antecedent pool elevation of 1124.63 ft NAVD88 (normal pool).

The PMF routings at the Aldenville gage location and the Prompton Dam inflow are compared in Figures 8.4 and 8.5, respectively. As shown in these figures, peak flow rates and times of peak flow are relatively similar between the HEC and GSSHA models. However, the 1988 HEC Prompton Modification Study model runoff volumes are much larger than the GSSHA model results.

The 1988 HEC Prompton Modification Study model inflow, pool elevation, and outflow results are shown in Figure 8.6 while the GSSHA inflow, pool elevation, and outflow results are shown in Figure 8.7. The 1988 HEC Prompton Modification Study model predicted that the pool would overtop the original dam crest by 0.8 ft while the GSSHA model predicted that the peak pool elevation was approximately 20 ft below the original dam crest elevation (approximately 0.8 ft above the spillway crest). The differences in peak pool elevations are mostly due to the different inflow volumes, not the peak inflow rates. In fact, the HEC PMF has nearly fifteen more inches of runoff volume that reaches Prompton Dam compared to the GSSHA PMF (37.83 inches vs. 22.94 inches). A sizable portion of this volume difference is contained within the time period between time = 42 hours to time = 54 hours. The HEC pool elevation demonstrates a sharper, earlier, and sustained rise during this time due to the differences in flow rates.

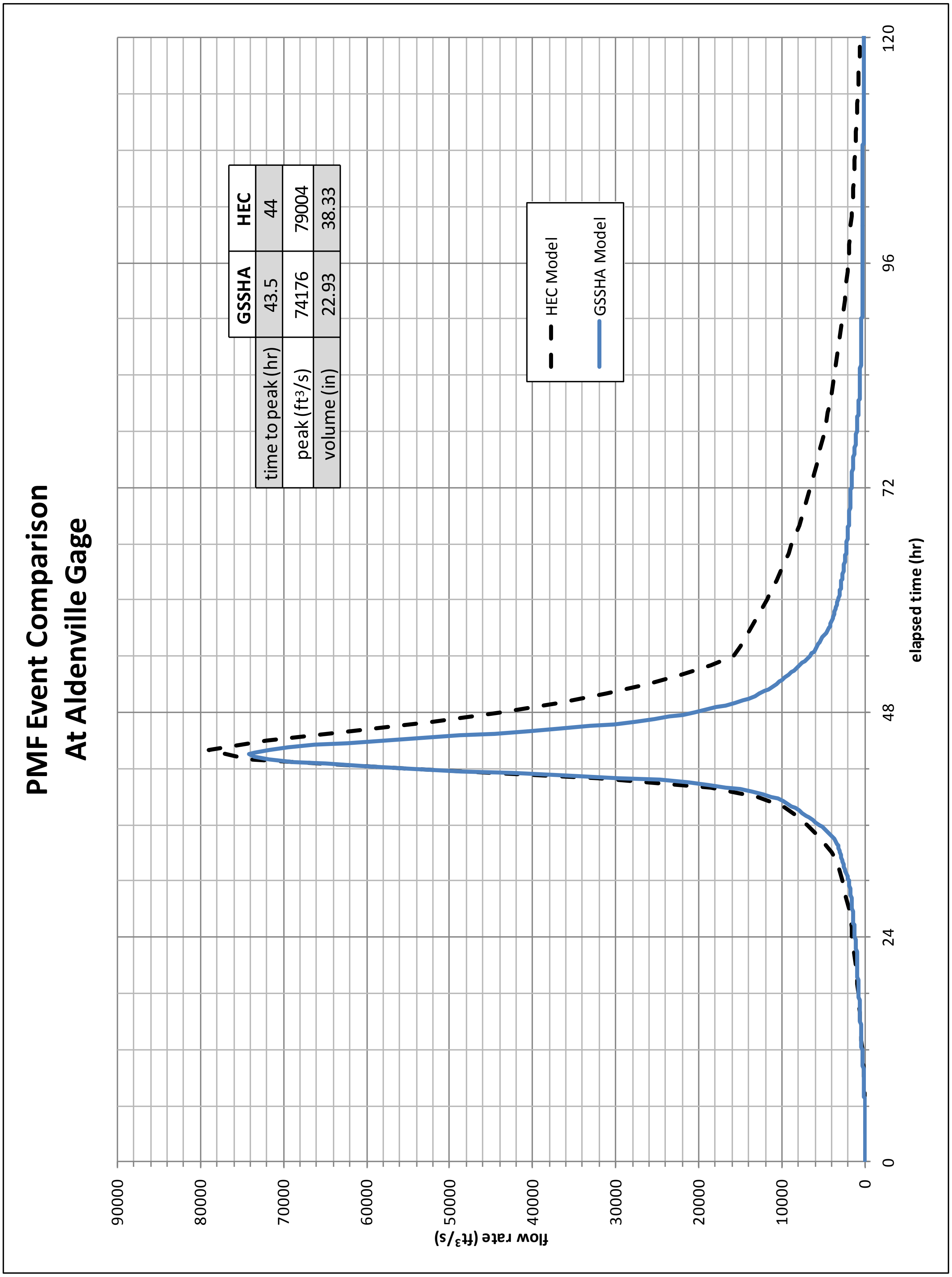


Figure 8.4 PMF Routing Comparison at Aldenville Gage

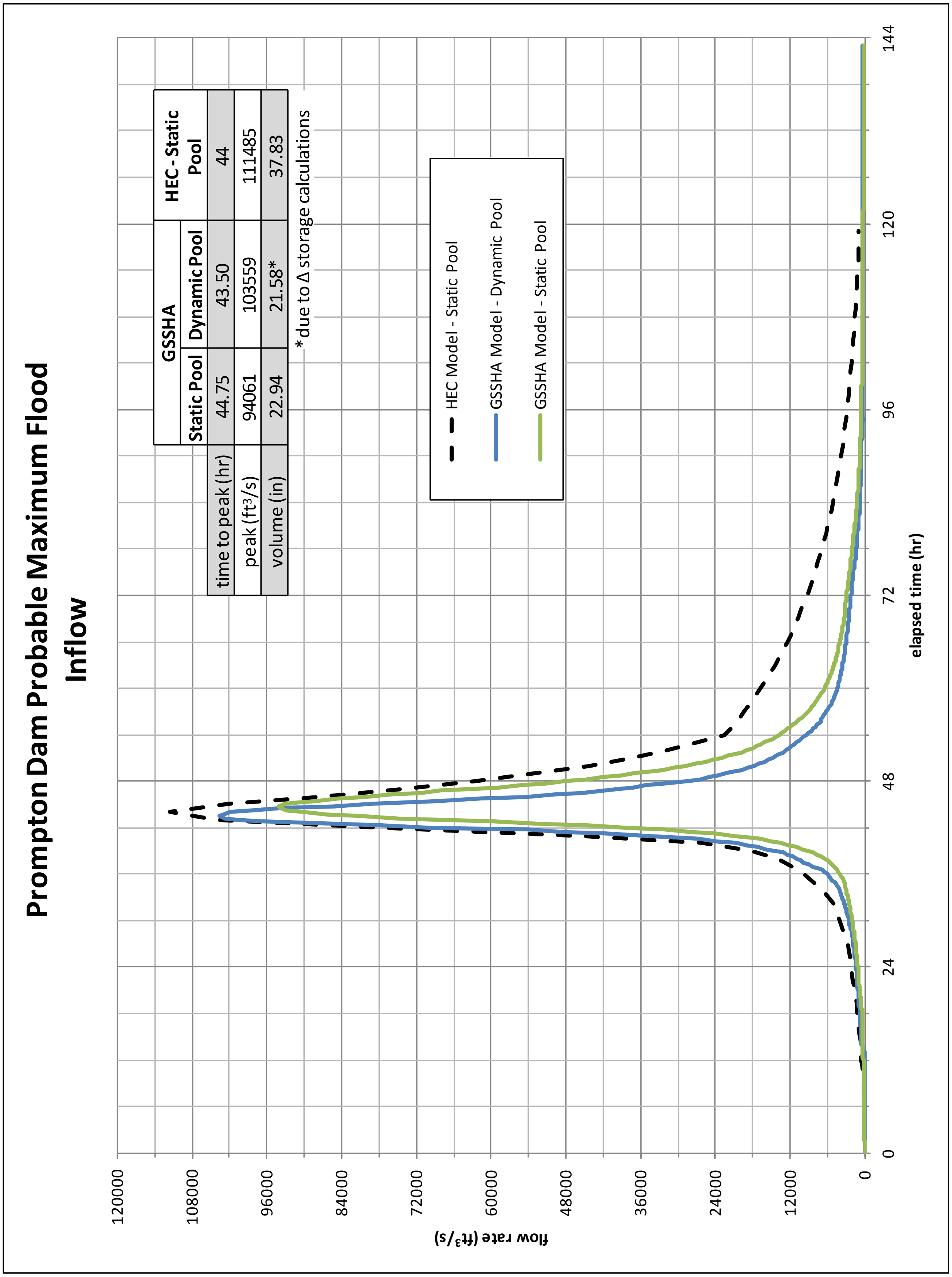


Figure 8.5 PMF Routing Comparison for Prompton Dam Inflow

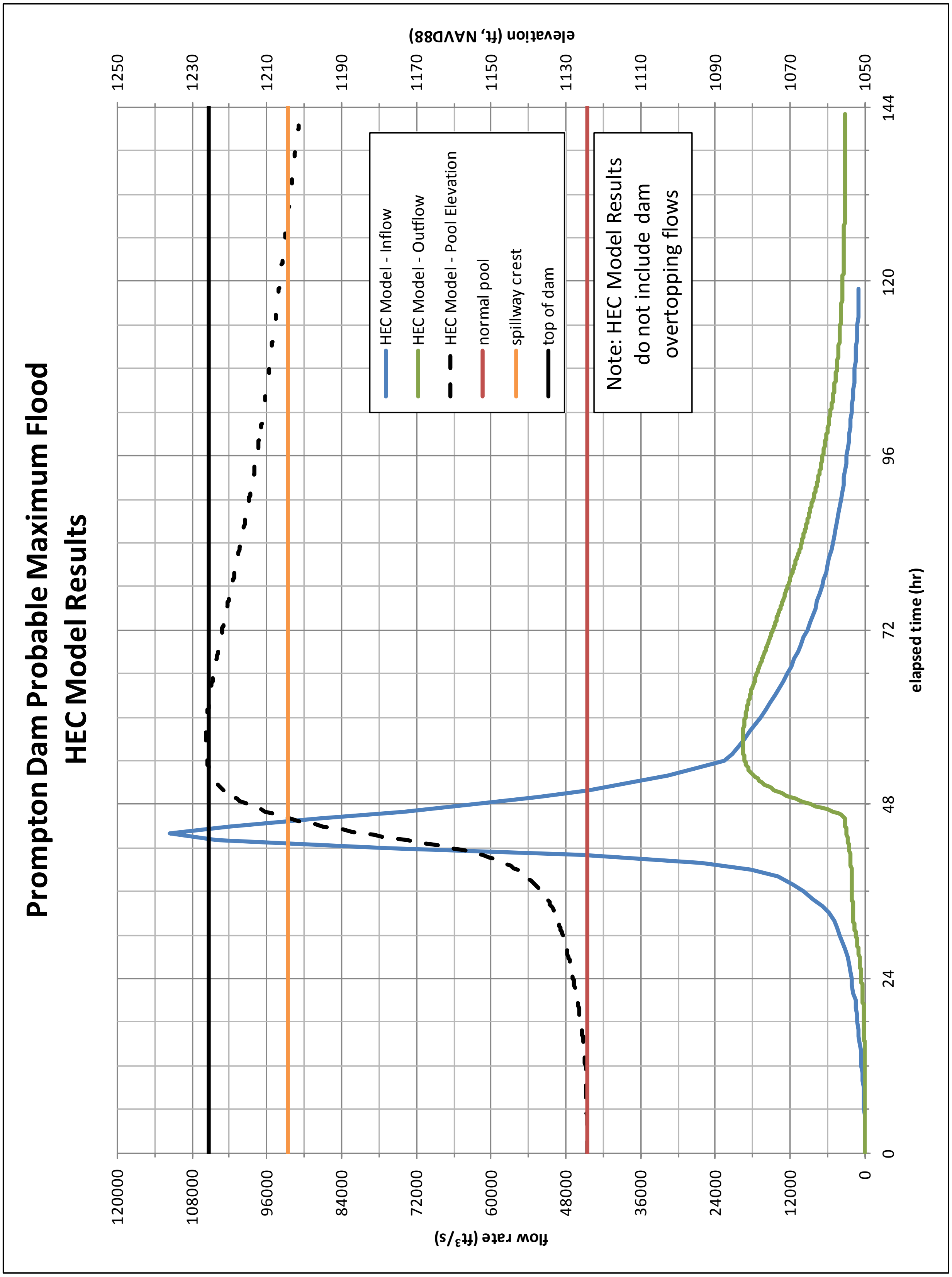


Figure 8.6 Prompton Dam PMF Routing – HEC Model Results

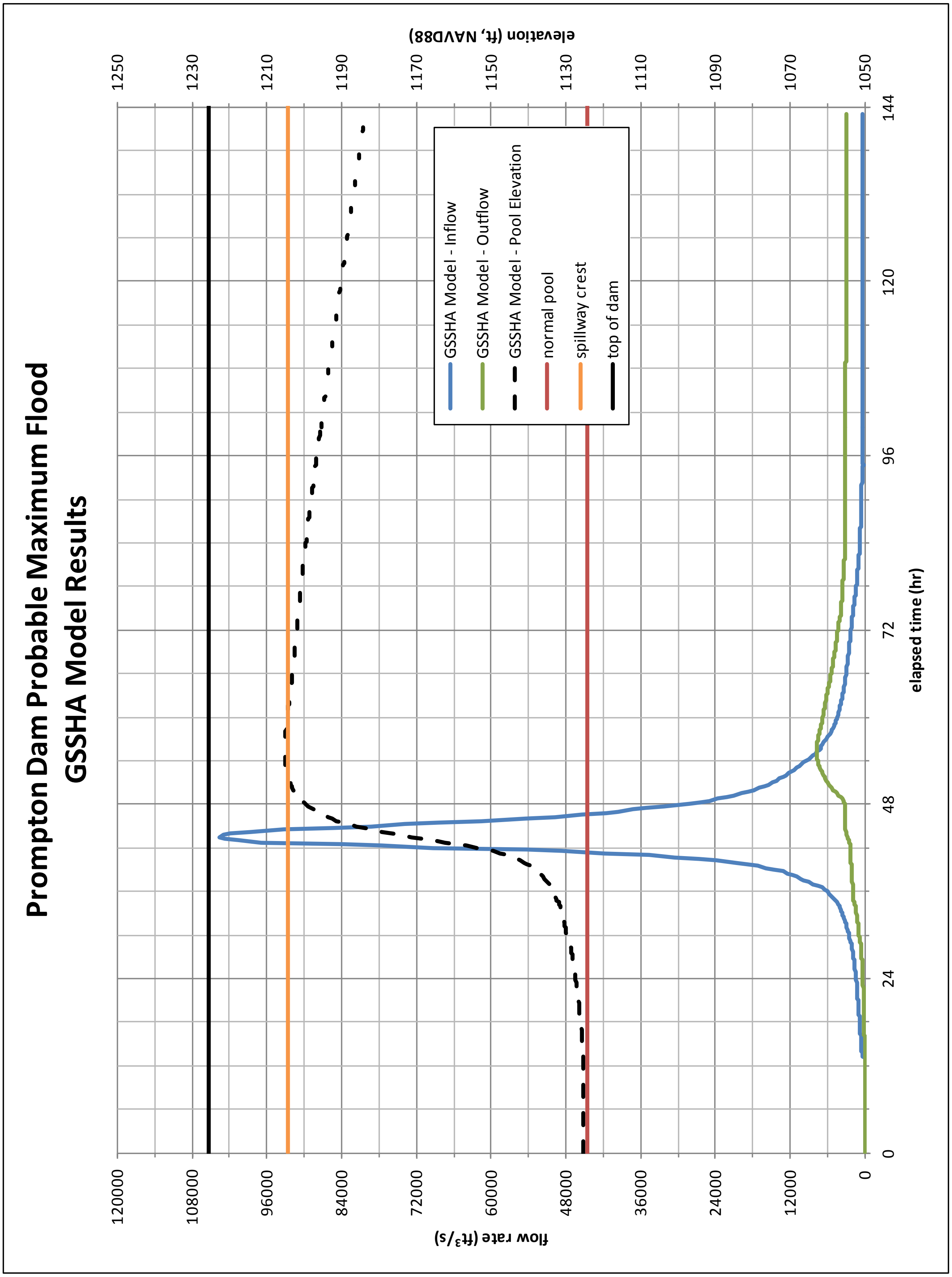


Figure 8.7 Prompton Dam PMF Routing – GSSHA Model Results

Also, upon further investigation, the HEC PMF inflow volume of 37.82 inches is approximately 12% higher than the precipitation volume of 33.67 inches, as shown in Table 8.8. This would not occur during a real-world event of this magnitude, but was allowed to occur within the 1988 HEC Prompton Modification Study HEC-1 model. This demonstrates the potential problems due to the lack of a "physical connection" between infiltration and baseflow when using the initial and constant infiltration routine and recession baseflow method. It is dependent upon the user to determine appropriate baseflow parameters that are physically possible while providing acceptable results. This lack of physical connection is not possible within GSSHA since baseflow is totally driven by initial groundwater conditions and infiltration during a simulation.

Approximately 3.5 hours was required to execute each PMF simulation due to the high intensity rainfall and 5-day simulation period. A total of 20 hours (2.5 working days) was required to complete the PMF simulations.

Table 8.8 Prompton Dam PMF Routing – HEC Model Results

HEC Subbasin	Name	Area (mi^2)	Precip (in)	Loss (in)	Excess (in)	Baseflow (in)	Total Discharge (in)
WSTBR	West Branch Lackawaxen above Johnson Creek	24.6	34.03	3.66	30.37	7.87	38.24
JOHNCR	Johnson Creek	16.8	34.2	3.66	30.55	7.9	38.44
	Aldenville Gage	***41.4****	***34.10***	***3.66***	***30.44***	***7.88***	***38.32***
PRMLOC	Local Drainage area above Prompton Dam	18.2	32.71	3.29	29.42	7.25	36.67
	Prompton Dam Inflow	***59.6****	***33.67***	***3.55***	***30.13***	***7.69***	***37.82***

*Contributing drainage area included within the HEC model

8.3. Antecedent Event Routing

USACE regulations require that the antecedent pool elevation of the reservoir in question prior to the execution of the IDF event be set using one of two conditions: 1) The full flood control pool or 2) The pool elevation prevailing five days after the last significant rainfall of a storm that produces one-half the IDF. [59] The antecedent pool elevation that is most appropriate is dependent upon the dam in question. In the case of Prompton Dam, there is approximately 51,125 ac-ft of flood storage available between the normal pool and spillway crest elevation which equates to approximately 16 inches of runoff from the contributing drainage area. As shown in the PMF routing results using the GSSHA model (Figure 8.7), the PMF inflow volume fills the available flood storage and the peak pool eclipses the spillway crest by less than 1ft. Therefore, the antecedent pool elevation was determined using the second method using the routing results of an antecedent flood equal to ½ of the IDF. If the antecedent pool elevation were set to the spillway crest (full flood control pool), this would imply that an event on the order of the PMF had occurred immediately prior to the start of the IDF, which is overly conservative.

The ½ IDF event inflow hydrograph was determined by dividing each ordinate of the PMF inflow hydrograph from both the 1988 HEC Prompton Modification Study as well as the GSSHA model. This ensured that the both the inflow volumes and peak flow rates were exactly half that of the PMF. These ½ IDF event inflow hydrographs were then routed through the Prompton Reservoir using an elevation-volume-discharge routing using the relationships shown in Figures 3.3 (elevation-volume) and 6.6 (elevation-outflow).

The routing results of both the 1988 HEC Prompton Modification Study and the GSSHA model are shown in Figures 8.9 and 8.10. The pool elevation prevailing at time = 120 hrs (equal to the

[59] (U.S. Army Corps of Engineers, 1991)

start of the IDF simulation) was found to be 1192.2 ft and 1158.2 ft for the 1988 HEC Prompton Modification Study and GSSHA model, respectively. The 34 ft higher pool elevation realized in the 1988 HEC Prompton Modification Study is due to the much larger inflow volume and excessively high baseflow ratio to peak value (ratio to peak = 0.2). Quick tests were performed using a more realistic ratio to peak value of 0.05 and found to result in a pool elevation at time = 120 hrs of approximately 1181.3 ft NAVD88.

Approximately 3.5 hours was required to execute each antecedent event simulation. A total of 16 hours (2 working days) was required to complete the antecedent event simulations.

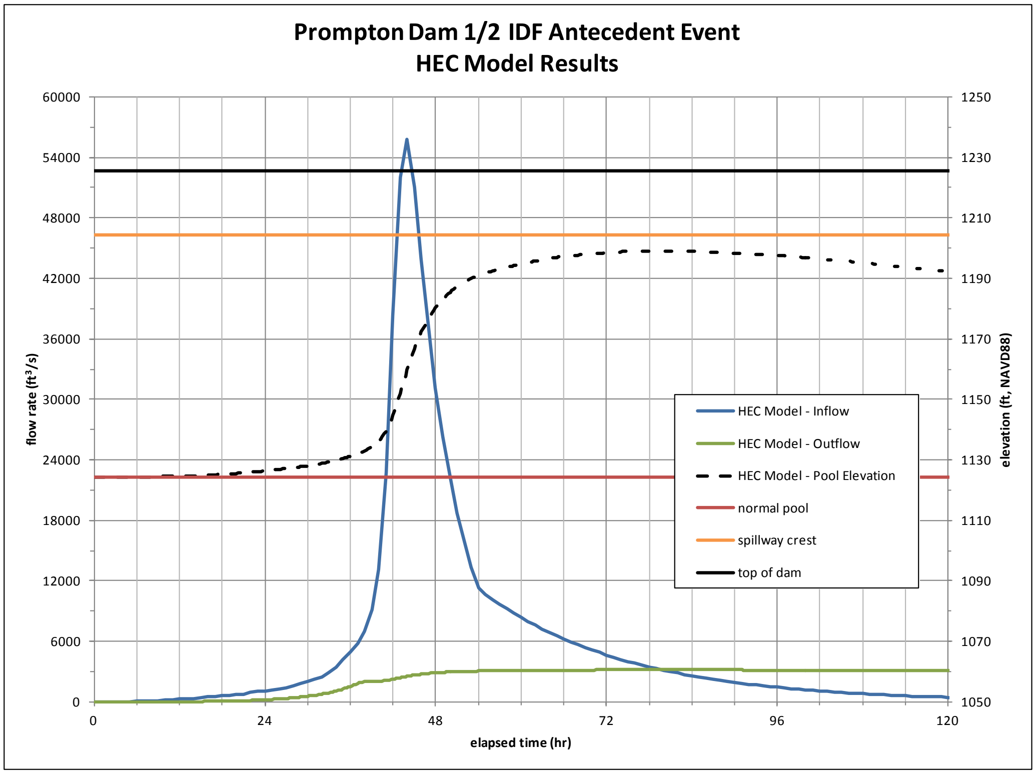


Figure 8.9 Prompton Dam ½ IDF Routing – HEC Model Results

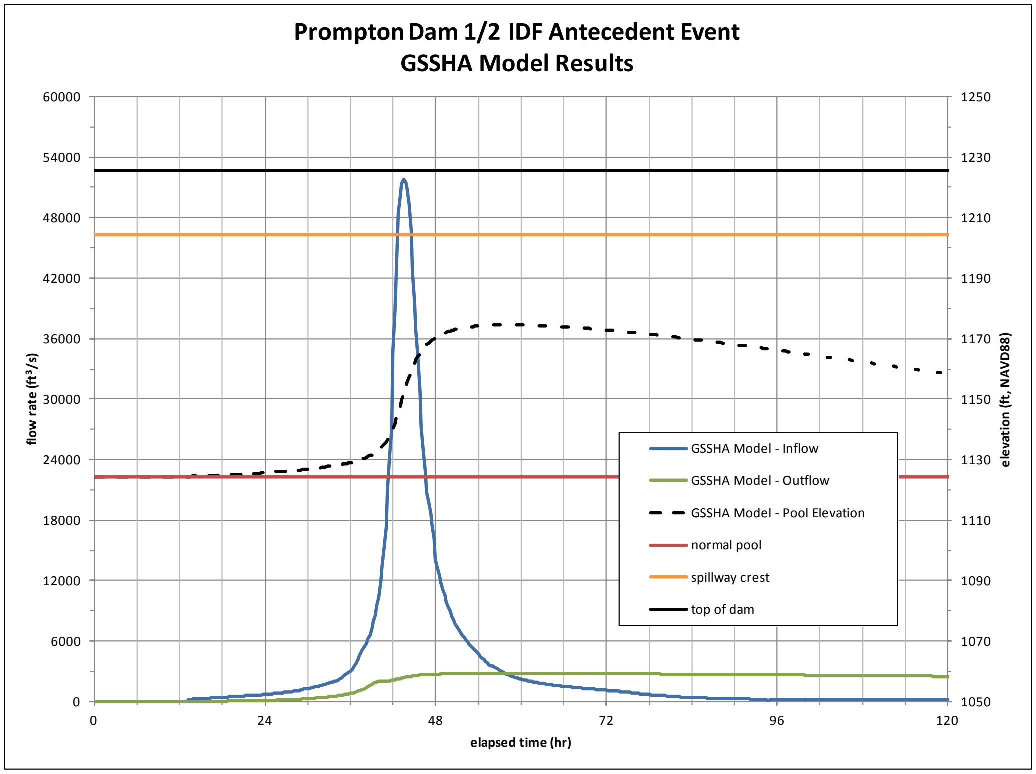


Figure 8.10 Prompton Dam ½ IDF Routing – GSSHA Model Results

8.4. Inflow Design Flood Routing

Using the results of the antecedent event routing, the IDF event was simulated using the previously determined starting pool elevations. The IDF event was simulated within the GSSHA model using the previously mentioned PMS. The PMS hyetographs taken from the 1988 HEC Prompton Modification Study was input to the HEC-1 model to recreate results for comparison. These results were verified to match the available output from the 1988 HEC Prompton Modification Study report.

The HEC IDF routing results showing the inflow hydrograph, pool elevation, and total outflow are shown in Figure 8.11. Similarly, the IDF routing results for the GSSHA model are shown in Figure 8.12. The single most important comparison is made in the pool elevation results shown in Figure 8.13: the HEC IDF routing results in a peak pool elevation of 1239.4 ft NAVD88, which is approximately 14 ft higher than the original dam crest elevation. This is what drove the original "hydrologic deficiency" determination made following the 1988 HEC Prompton Modification Study results. If the dam were to overtop by 14 ft, it would likely fail. However, the GSSHA IDF routing results in a peak pool elevation of only 1213.9 ft NAVD88, which is approximately 25.5 ft below the peak pool elevation determined during the 1988 HEC Prompton Modification Study and approximately 11.5 ft below the original dam crest elevation.

The HEC IDF was also simulated using an antecedent pool elevation of 1181.3 ft NAVD88 and subbasin baseflow ratio to peak values of 0.05, as was discussed in Section 8.3. The peak pool elevation was predicted to be 1235.9 ft NAVD88, which is still approximately 10.5 ft higher than the original dam crest elevation.

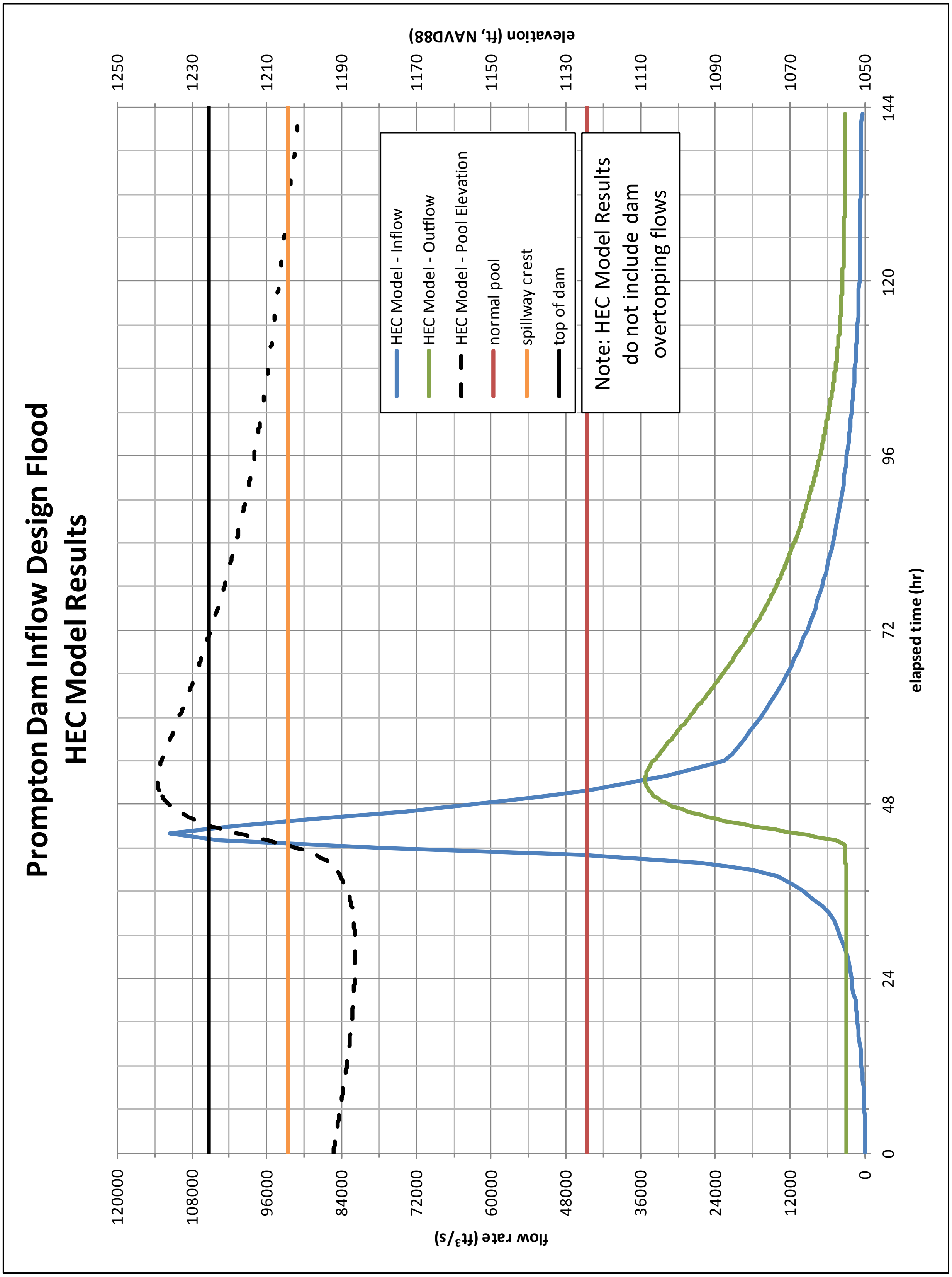


Figure 8.11 Prompton Dam IDF Routing – HEC Model Results

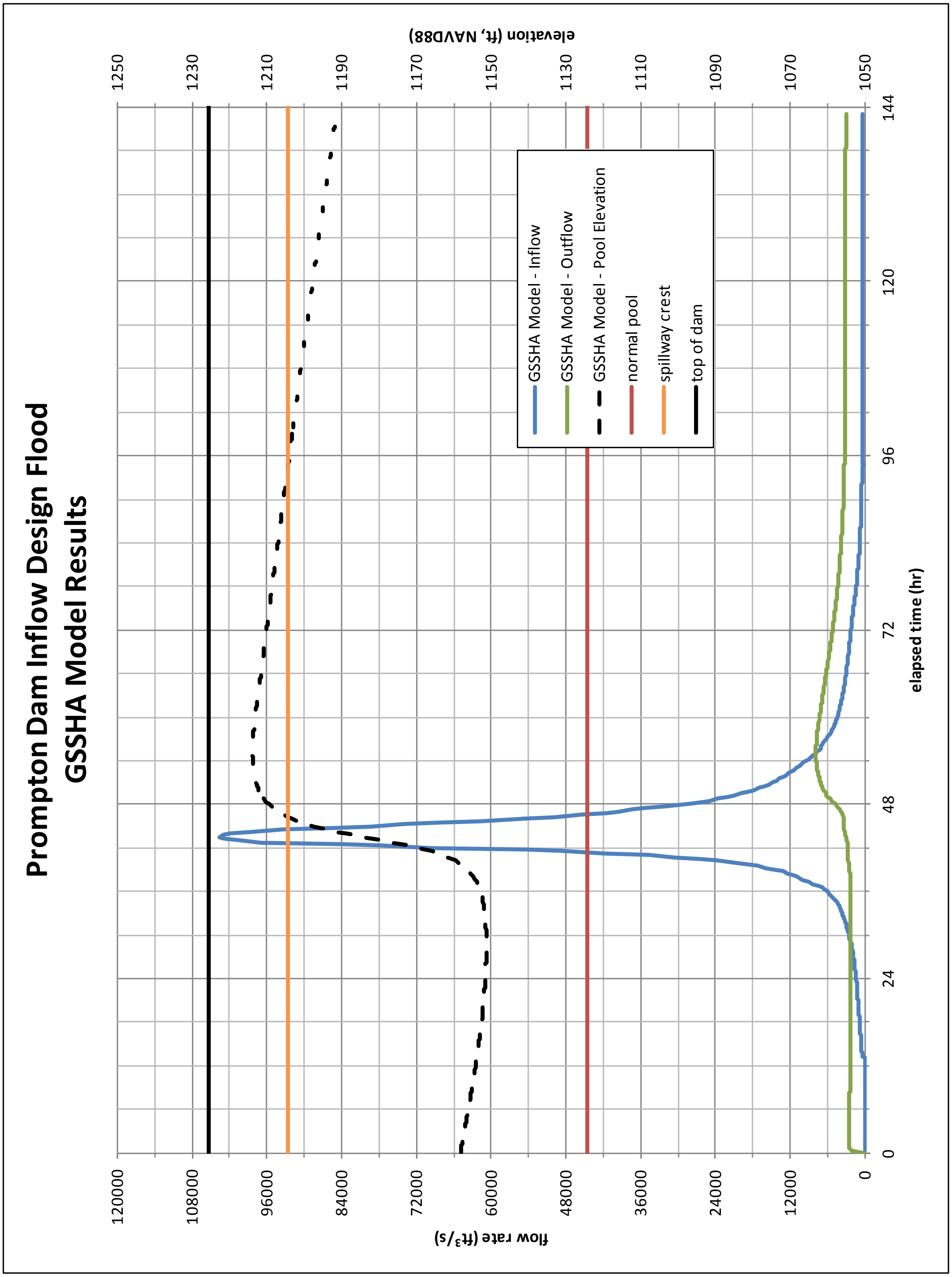


Figure 8.12 Prompton Dam IDF Routing – GSSHA Model Results

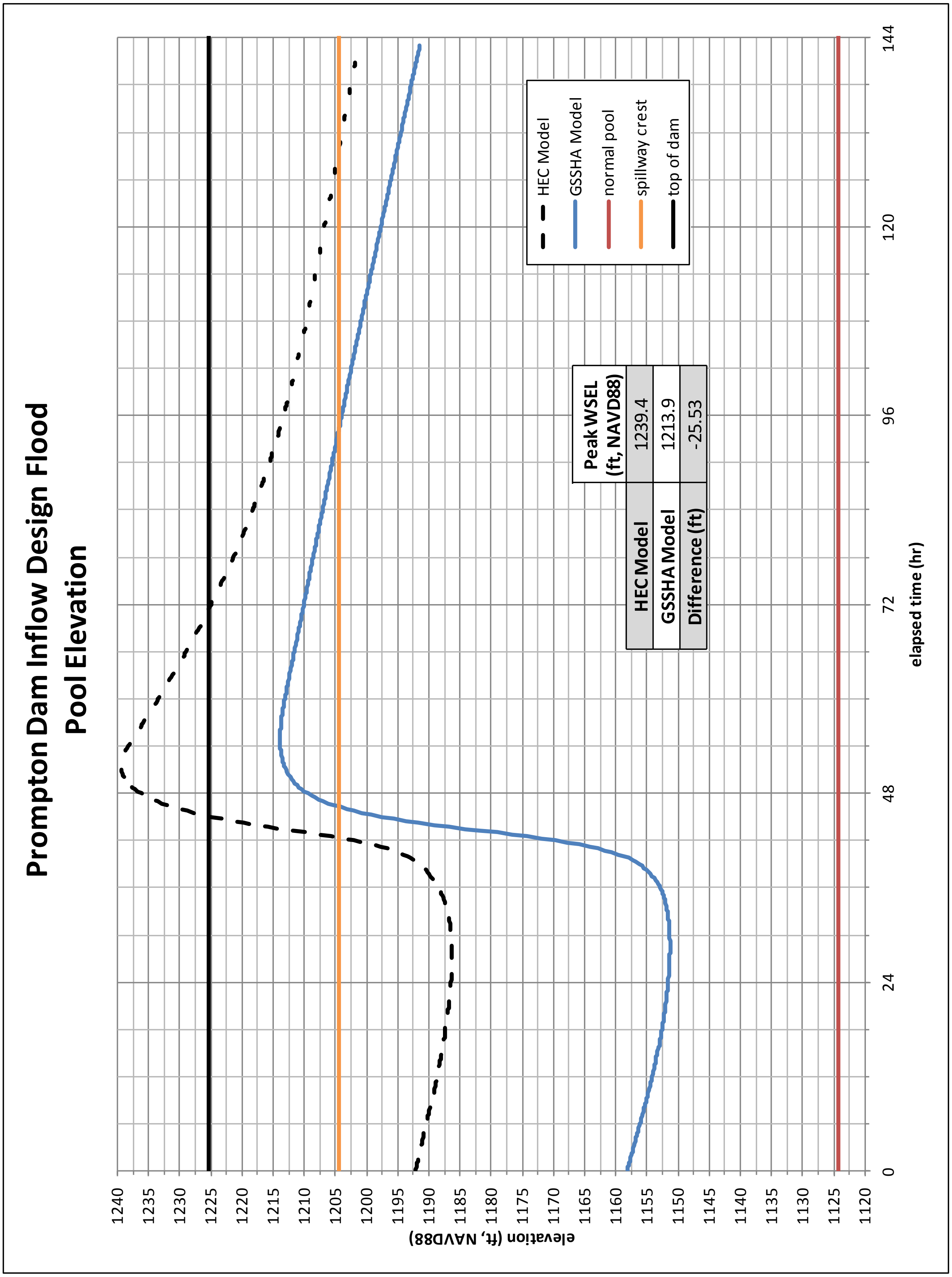


Figure 8.13 Prompton Dam IDF Routing – Pool Elevation Comparison

Approximately 3.5 hours was required to execute each IDF simulation due to the high intensity rainfall and 5-day simulation period. A total of 20 hours (2.5 working days) was required to complete the IDF simulations.

While the GSSHA model predicts that the dam crest would not be overtopped during the IDF event, these results should not be considered the "final" answer. These results are indicative of the need for simulations that are more detailed than those provided by the 1988 HEC Prompton Modification Study. For instance, parameter uncertainty can play a large role in pool elevations that could occur. These parameters could include (but are not limited to) the seasonality of the PMF/IDF, the magnitude of the PMF/IDF, the temporal and spatial distributions of the PMS, antecedent pool elevation, and antecedent precipitation. Uncertainty in these parameters is best investigated using stochastic simulations which vary parameters with pre-set bounds and sample from the results using Monte Carlo sampling procedures. Studies using stochastic modeling techniques have been performed on multiple dam sites including Folsom Dam in California.[60]

[60] (MGS Engineering Consultants, Inc., 2005)

9. Conclusions

The Gridded Surface Subsurface Hydrologic Analysis (GSSHA) rainfall runoff modeling code was used to investigate the differences between various hydrologic routines when simulating extreme events on the order of the Probable Maximum Flood for Prompton Dam (designed, owned, and operated by the U.S. Army Corps of Engineers, Philadelphia District). An HEC-1 model used to design a dam modification for Prompton Dam was compared against the GSSHA results.

During the 1988 HEC Prompton Modification Study, a hydrologic deficiency was found to exist by the HEC-1 model. In particular, the dam was predicted to overtop by approximately 14 ft (not including overtopping flows) during the Inflow Design Flood (IDF), which would likely fail the dam with catastrophic consequences. Prompton Dam was modified from its original design by installing a parapet wall, widening and deepening the spillway and approach channel, installing an access bridge across the spillway, and relocating various facilities. These modifications were completed in 2012 at a cost of nearly $25 million.

The GSSHA model for Prompton Dam was used to ascertain the accuracy/effectiveness of various hydrologic routines, simplifications, and assumptions commonly employed when simulating extreme events. The simplified hydrologic modeling routines used within the 1988 HEC Prompton Dam Modification Study resulted in large differences in the resultant streamflow hydrographs and reservoir pool elevations when compared to the physically-based routines within GSSHA. When simulating the IDF within the GSSHA model, peak pool elevations were found to be approximately 25 ft lower than the results obtained from the 1988 HEC Prompton Dam Modification Study. Approximately 11.5 ft of freeboard between the static peak pool elevation and original dam crest was predicted within the GSSHA model. The differences in peak pool elevation during the IDF event were primarily due to the larger inflow hydrograph volumes predicted by the 1988 HEC Prompton Modification Study when compared to the GSSHA model results.

The time and effort required to successfully carry out a study using a physically-based model is generally greater than an empirically-based model due to the greater amount of required data and computation times. Throughout this study, efforts were made to record the approximate amount of time necessary to complete each phase of the GSSHA modeling effort, from construction to execution. This will allow future efforts to better estimate the amount of time entailed in a detailed dam safety analysis using GSSHA and decide whether or not the added effort is warranted. A total of 18.5 days (assuming an 8 hour work day) was necessary to complete the GSSHA model set up. Another 20 days were necessary for model calibration using a total of four events and another three days for model validation using one event. Four and a half days were needed to retrieve, build, and format Probable Maximum Storm grids for use in GSSHA while another seven days were needed to execute the necessary PMF, antecedent event, and IDF simulations. A total of 53 days or approximately 10.5 weeks (assuming a 40 hour or five day work week) was needed to complete the tasks specifically related to the construction, calibration, validation, and execution of the GSSHA model in this analysis.

The increase in model development and execution time should be weighed against the potential time and cost savings that can be realized with the increased accuracy. The additional cost of a detailed modeling analysis using a physics-base model can be incredibly small compared to the potential costs of dam safety modifications.

This topic of research would benefit from additional studies performed on a multitude of dams and watersheds across the United States. This would allow for the determination of effects due to geographic location as well as differing watershed characteristics and provide a larger data set from which to draw conclusions for future use. Additionally, the inclusion of stochastic modeling techniques, which vary model input parameters that are commonly “fixed”, on top of the physically-based hydrologic modeling routines discussed in this research would allow for a greater understanding of parameter uncertainty and could provide more defensible results.

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