US Army Corps of Engineers®
New York District

WESTCHESTER COUNTY STREAMS, BYRAM RIVER BASIN

FLOOD RISK MANAGEMENT FEASIBILITY STUDY

FAIRFIELD COUNTY, CONNECTICUT AND WESTCHESTER COUNTY, NEW YORK

FINAL INTEGRATED FEASIBILITY REPORT & ENVIRONMENTAL IMPACT STATEMENT

APPENDIX B.1:

Hydrology

This Appendix calculates existing and future without project conditions hydrology for the flood risk management feasibility study of the Byram River Basin. Byram River flood flows have been developed for the following design recurrence intervals: 100, 50, 20, 10, 4, 2, 1, 0.5, and 0.2 percent (also known as the 1-yr, 2-yr, 5-yr, 10-yr, 25-yr, 50-yr, 100-yr, 200-yr, and 500-yr respectively). The methodology is based on a calibrated runoff model, and validated by statistical analysis of available stream flow data. The Hydrology analysis described in this appendix is an update to the Byram River peak flood discharges developed by the U.S. Army Corps of Engineers (USACE) in their 1977 Feasibility Report for Flood Control (USACE, 1977).

1. STUDY AREA

The Byram River basin is almost entirely within the extents of the Town of Greenwich in Fairfield County Connecticut with headwaters north across the border in Westchester County, New York. The Byram River, with a length of 13.5 miles, flows south and empties into Long Island Sound. The lower portion of the river, for a length of 1.3 miles, is tidal. At the mouth of the river, the Byram River is the state boundary between Connecticut and New York. The total contributing area at the river mouth is 30 square miles. The riparian zone of the lower three miles of the Byram River is populated with suburban housing and commercial buildings. In the upper reach, generally upstream of the bridge at Bailiwick Road, the area is less densely developed.

2. RAINFALL

Rainfall data was used as input to the runoff model for simulating: (1) historic discharge events and (2) hypothetical design flood events associated with recurrence intervals such as 100-years.

2.1. Observed Rainfall Data

Rainfall collected at the Westchester Airport (WBAN #94745, COOP # 309140) represents the longest sub-daily record in the immediate vicinity of the Byram River basin. **Figure 1** shows the location of the Westchester Airport relative to the Byram River watershed. The airport is 2.5 miles southwest of the centroid of the watershed.

The weather station at Westchester Airport is part of the Automated Surface Observation Systems (ASOS) program maintained by the National Weather Service (NWS) of the National Oceanic and Atmospheric Administration (NOAA). It is the largest and most modern network of weather stations in the country, updating precipitation observations every minute with a 0.01 inch data resolution. One-minute precipitation data is available from March 2005 to present.

It was necessary to use sub-daily rainfall data to accurately simulate runoff from the Byram watershed. Several recent runoff events including the April 2007 Nor'easter and the 2011

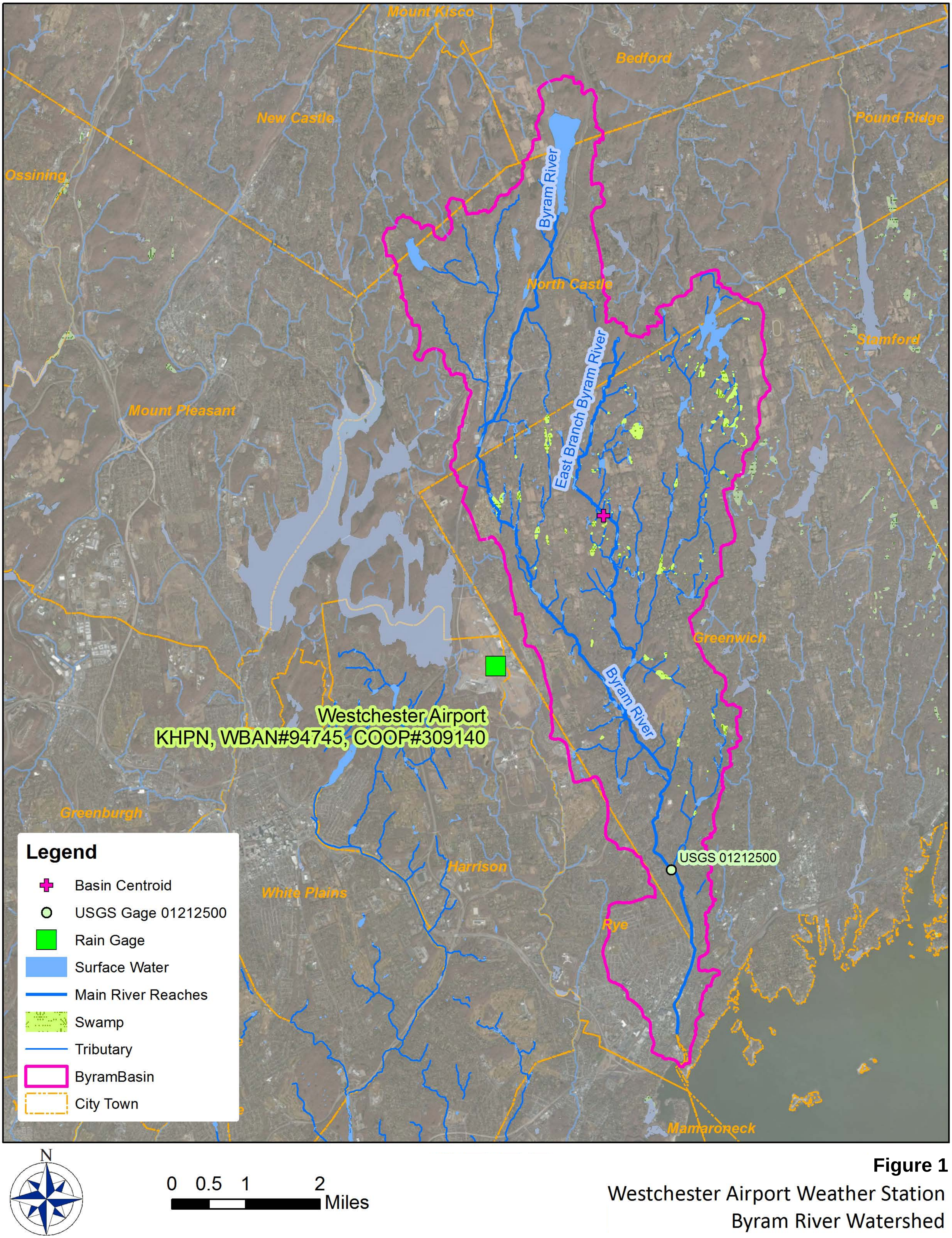


Figure 1
Westchester Airport Weather Station
Byram River Watershed

Hurricane Irene were selected for simulation. As described in Section 6.0, cumulative rainfall curves were developed from the 1-minute ASOS data for each simulation event.

There is also a 3-year long record of 15-minute precipitation data available at the USGS gage on the Byram River at Pemberwick (USGS 01212500). Although the station is within the basin boundaries it is 4.8 miles south of the watershed centroid as shown in **Figure 1**. The short available gage record is used for validation of precipitation inputs for recent runoff simulations.

2.2. Design Storms

The Connecticut Department of Transportation (ConnDOT) Drainage Manual (January 2001) uses the effective duration-frequency depths defined by NWS, as the basis for the 24-hour rainfall-frequency definition for Fairfield County. In the New England region, the effective atlas is Technical Paper No. 40 (TP-40) (Hershfield, 1961). As these depths were published over 50 years ago, a source of updated depths was sought.

The Northeast Regional Climate Center (NRCC) at Cornell University maintains an online atlas that provides precipitation estimates that include data from precipitation gages all over New England over the last fifty years (http://precip.eas.cornell.edu). The 2012 Town of Greenwich Drainage Manual adopted the NRCC rainfall design depths (Fuss and O'Neill, 2012). The NRCC depths are higher than those from TP-40 and will be the basis of the runoff simulations in this analysis.

Table 1 shows the rainfall depths associated with both the NRCC and TP-40 analysis.

Table 1 Design Storm Precipitation Amounts

24-Hour Design Storm Frequency	NRCC Precipitation Amount (inches)	TP-40 Precipitation Amount (inches)
100%	2.9	2.7
50%	3.4	3.3
20%	4.3	4.3
10%	5.1	5.0
4%	6.4	5.7
2%	7.6	6.4
1%	9.1	7.2
0.5%	10.8	N/A
0.2%	13.5	N/A

The temporal distribution of the synthetic storms used in this analysis is based on a Type III synthetic rainfall distribution corresponding to Southwestern Connecticut from Appendix B of

Technical Report 55 (TR-55) (USDA, 1986) published by the Natural Resource Conservation Service (NRCS).

2.3. Historic Events

The Feasibility Report for Flood Control published by the USACE in 1977, documents Byram River discharge estimates for several historic events including the floods in October 1955, June 1972, and September 1975 (USACE, 1977). For calibration and verification purposes, the present analysis performs a similar estimate of these three events described in **Section 6** of this Appendix, and is based on the 3-hour rainfall intensity records used in the 1977 unit hydrograph analysis described in Figures A15 through A17 of the USACE report. The cumulative rainfall plots for these three events are shown on Figures 13 through 15.

3. RIVER GAGE DATA

River gage data from the surrounding region was used to (1) statistically estimate the discharge frequency relationship for the Byram River and for (2) calibrating the runoff model to the observed and recorded recent historic flood hydrographs.

3.1. Local USGS Gages

Nine USGS gages were identified in the immediate area of the Byram River watershed with varying length records and are summarized in **Table 2**.

Table 2 Local USGS Stream Gages

USGS Gage Number	River	Location	Drainage Area (sq. mi.)	Temporal Resolution	Begin Date	End Date	Notes
01212500	Byram River	at Pemberwick, CT	25.6	15 minute	Oct 2009	Present (August 2013)	3 years 10 months
01211600	Byram River	at Riversille, CT	11.7	Daily	Oct 1964	Oct 1965	only 1 year
01211700	E Br Byram River	at Round Hill, CT	1.7	Peaks Only	1960	1975	peak annual only
01212100	E Br Byram River	at Riversille, CT	11.1	Peaks Daily	1962 Oct 1962	1984 Oct 1969	missing 1966 in daily data
01209700	Norwalk River	at South Wilton CT	30.0	Daily 15 minute	Sep 1962 Oct 2007	Present (May 2013)	
01209901	Rippowam River	at Stamford, CT	34.0	Daily 15 minute	Sep 1977 Oct 2007	Present (May 2013)	missing Oct 1982 through Oct 2001

USGS Gage Number	River	Location	Drainage Area (sq. mi.)	Temporal Resolution	Begin Date	End Date	Notes
01300000	Blind Brook	at Rye, NY	8.9	Daily	Oct 1944	Oct 1989	
01300500	Beaver Swamp Brook	at Mamaroneck, NY	4.4	Daily	Oct 1944	Oct 1989	
01301000	Mamaroneck River	at Mamaroneck, NY	23.4	Daily	Oct 1944	Oct 1989	missing Oct 1952 through Oct 1954. Peak discharge available from 2009 to present

The discharge record on the Byram River at Pemberwick (USGS 01212500) is the best representation of Byram River discharge available, representing 85% of the total watershed. The gage is located at the Comly Ave Bridge, upstream of Caroline Pond. While the record is insufficient for deriving discharge-frequency curves, the 3 years of available 15 minute discharge records can be used for calibration of the runoff model for flood events since September 2009. This calibration is described in **Section 6**.

In addition to USGS 01212500, there are three other gages that are located within the Byram River watershed with limited data. The gage on Byram River at Riversille, CT has a daily record of only one year. The gage on the East Branch Byram River at Riversille has a daily record of six non-continuous years, including the water year of 1965. The gage on East Branch Byram River at Round Hill, CT does not have a daily record associated with it. Only a limited record of annual peaks (1960 – 1975).

Two of the other eight gages have records coincident to the Byram River gage: Norwalk River at South Wilton, CT (USGS 01209700) and Rippowam River at Stamford, CT (USGS 01209901). Of these two candidates for basin transposition, Norwalk river has a longer continuous record.

The record on Blind Brook at Rye, NY (USGS 01300000) was the basis of the flood frequency analysis of the Byram River by basin transfer in the 1977 USACE feasibility study (USACE, 1977). The record on Blind Brook ends in 1989 and was not used for the updated analysis because it does not coincide with the recent record on Byram River. **Figure 2** shows the location of all nine gages as well as the corresponding watersheds.

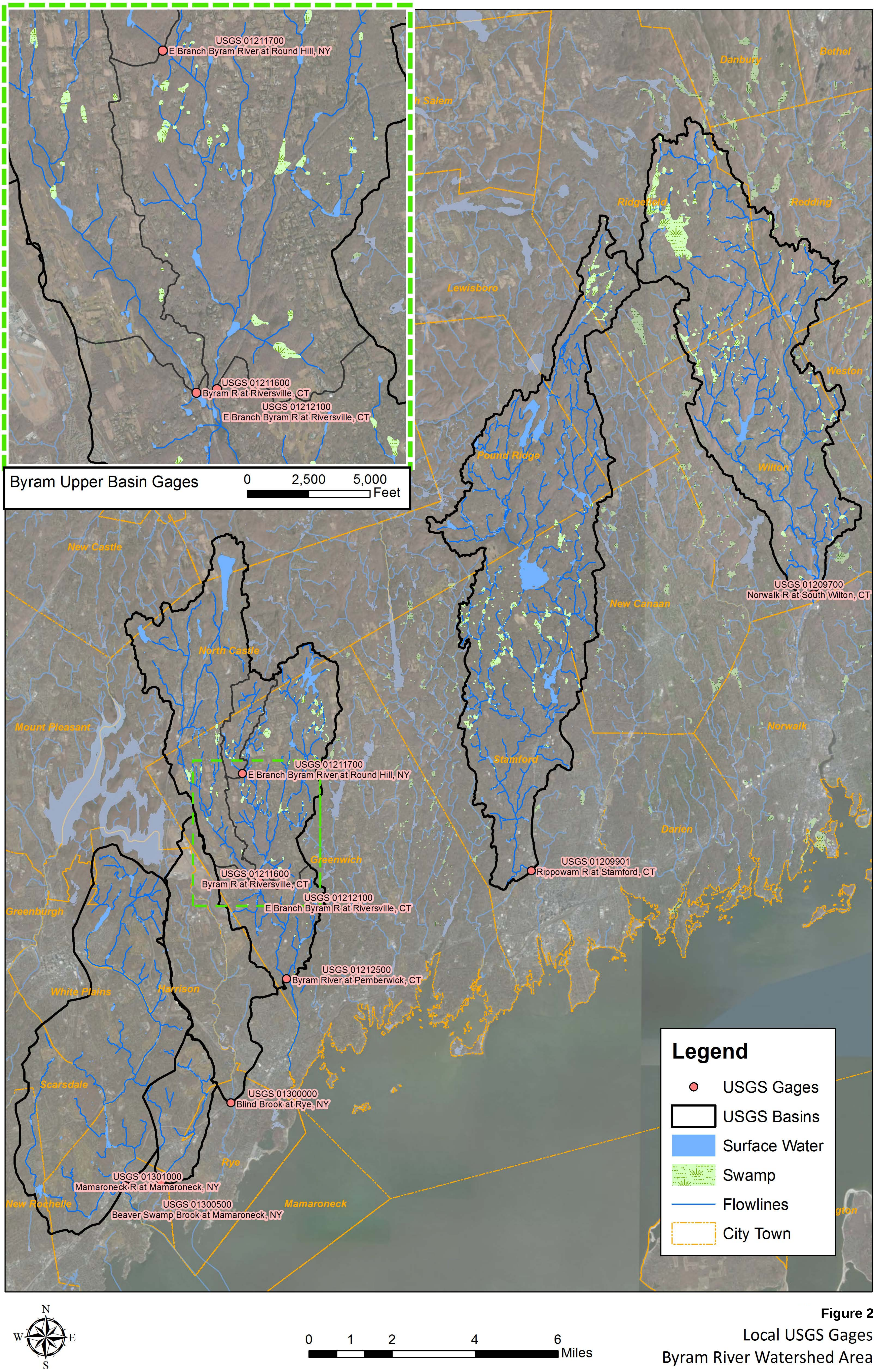


Figure 2
Local USGS Gages
Byram River Watershed Area

3.2. Peak Record Extension of USGS 01212500

The gage record at Pemberwick (USGS 01212500) has a period of record just over three years. This period of record is too short to be able to perform a useful analysis to determine key return periods flows such as the 2 or 1 percent flow. The period of record for this gage was extended using data on the East Branch Byram River at Riversville, CT (USGS 01212100) and the Norwalk River at South Wilton, CT (USGS 01209700).

The East Branch Byram River gage is just upstream of the confluence with the main branch of the Byram River and has 22 years of annual peak flow data from 1963 to 1984. A relationship between the peak flow at the East Branch Byram gage was developed with the peak flow at Pemberwick using fourteen events simulated with the HEC-HMS model described in **Section 5.0**. The rainfall events simulated were the NRCC design storms and nine high flow events used to calibrate the HMS model. The peak HMS flows at each location are summarized in **Table 3**. The relationship between the flows at Riversille and Pemberwick is also shown in **Figure 3**. This relationship was then used to translate the 22 historic annual peak flows at Riversille to annual peak flows at Pemberwick. These 22 annual peaks along with the 3 observed annual peaks will be used together as a basis of comparison to the other methods of extending the record for the Pemberwick gage.

Table 3 Peak HMS Flows

Event	Peak Flow on E. Branch Byram at Riversville (cfs)	Peak Flow on Byram River at Pemberwick (cfs)
50% NRCC Flood	468	796
10% NRCC Flood	1,031	1,734
4% NRCC Flood	1,532	2,558
2% NRCC Flood	2,026	3,367
1% NRCC Flood	2,677	4,426
October 1955 Flood	1,724	3,194
June 1972 Flood	1,150	1,983
September 1975 Flood	2,131	3,414
April 2007 Nor'easter	1,569	2,860
Hurricane Irene	1,650	2,690
September 2011 High Flow	1,102	1,893
March 2011 High Flow	1,015	1,728
April 2011 High Flow	809	1,305
May 2011 High Flow	401	609

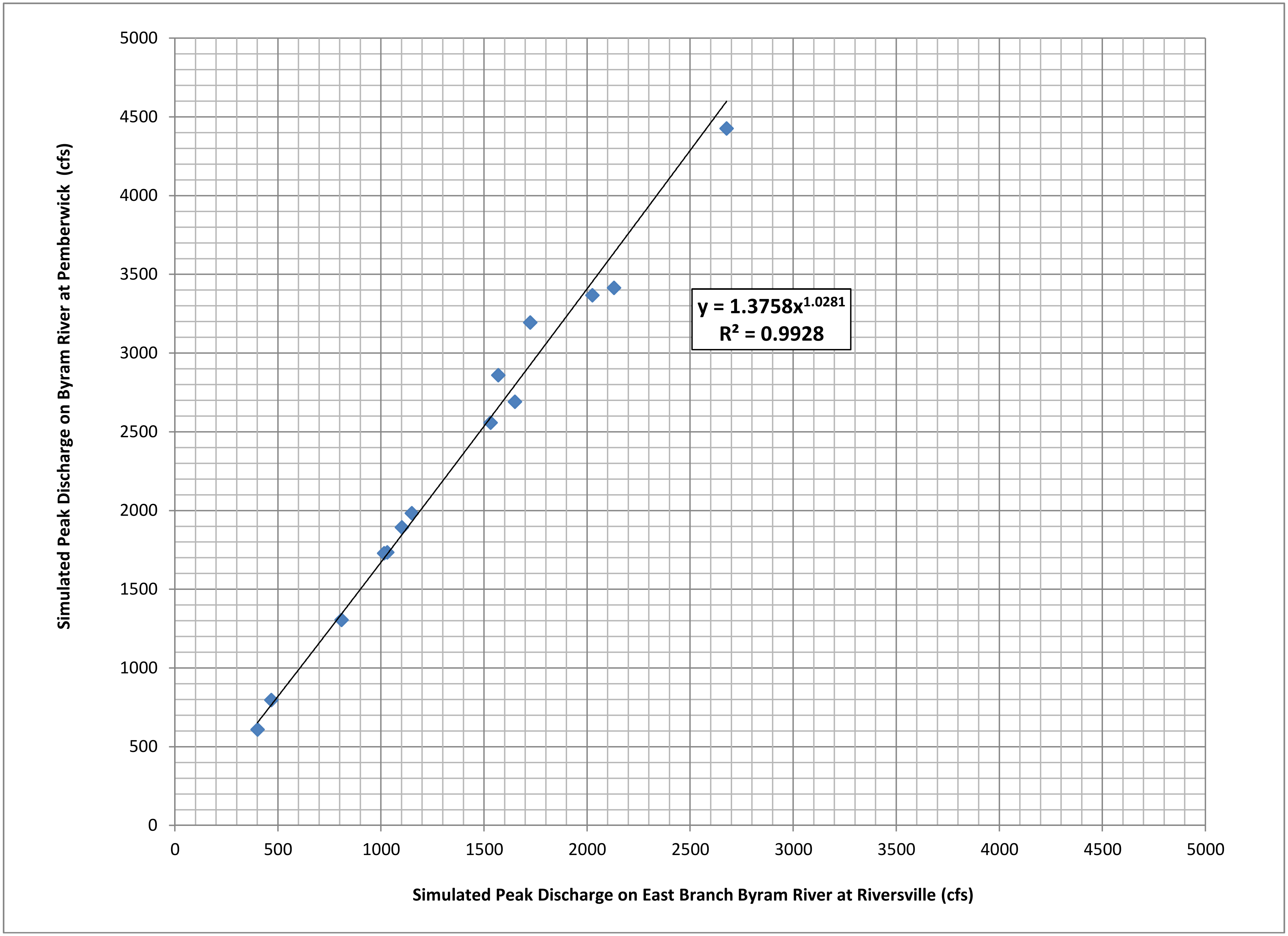


Figure 3

Comparison of simulated peak discharge at two locations of interest on the Byram River

Two different methods were then used to estimate the annual peak flows between 1984 and 2010 at Pemberwick. These were the Area Ratio and the MOVE.2 methods. The Area Ratio method simply adjusts the observed flow at one location to another location by multiplying the observed flow by the ratio of the drainage area for the two corresponding sub-basins. As shown in **Table 2** the Norwalk gage has a drainage area of 30 square miles and the Pemberwick gage has a drainage area of 25.6 square miles. This gives us a ratio of 0.853. This ratio was used to translate the annual peak flows at the Norwalk gage to annual peak flows at the Pemberwick gage. These estimated flows were then compared with the 22 historic annual peaks and 3 observed annual peaks at Pemberwick. A scatter plot showing the Area Ratio estimated annual peak flows against the synthetic and observed annual peak flows is shown in **Figure 4A**. The results of the area ratio extension are shown in **Table 4**.

The MOVE.2 approach analyzes the available set of coincident data to develop a relationship between two time series (Hirsch, 1982). The relationship developed from the coincident peaks at the Norwalk River gage and the Byram River gage was used to extend the record of peaks at the Byram River gage using the longer record of peaks at the Norwalk River gage. **Table 4** shows all of the observed and synthesized peak annual flows used to develop the MOVE.2 record extension.

On the Byram River there are peak annual flows available from the recent record at Pemberwick and 22 peak annual flows available from the historic record at Riversville and transferred to the Pemberwick gage using the relationship in **Figure 3**. For 8 events, the coincident Norwalk River peak annual flow was estimated since the peak annual flow recorded at the Norwalk gage for that year did not correspond to the same event as the peak recorded for the same year on the E. Branch Byram River. The peak flow rate for this event was estimated from the daily record at the Norwalk gage using the antecedent and subsequent daily average flows with a nomograph developed by Langbein (1944)

Once the coincident peaks had been identified the parameters for the MOVE.2 method were calculated and then the Norwalk annual peak flows were translated to Pemberwick. The estimated flows were then compared with the historic and observed annual peak flows at Pemberwick. A scatter plot showing the estimated annual peak flows against the synthetic and observed annual peak flows is shown in **Figure 4B**.

The peak annual flows for the different estimation methods are summarized in **Table 4** and **Figure 5** shows a plot comparing these estimates. These estimates were combined into annual maximum time series to be used in the flood-frequency analysis discussed in **Section 4.0**. When available, the observed annual peaks and the historic peaks estimated from the Riversville gage were used over the MOVE.2 or Area Ratio estimates.

Four peak discharge were estimated at the Pemberwick gage using other analysis and included in the final annual maximum time series:

- The USACE (1977) estimated a October 1955 flood peak of 4,250 cfs for the Byram River at U.S. Route 1 which was corroborated by a comparison of flood marks and initial runs of the HEC-RAS hydraulic profile model described in **Appendix B2**. Using the HEC-HMS runoff model described in **Section 5.0**, the flow at Pemberwick was interpolated from this estimate at U.S. Route 1 to be 4,260 cfs.

- The USACE (1977) estimated the June 1972 flood peak at 2,900 cfs. The flow estimated from the Riversville gage is 2,880 cfs. The average of the two (2,890 cfs) is used for the 1972 peak. This corresponds to a ratio of total excess to total rain of 0.60.

- The USACE (1977) estimated a September 1975 flood peak of 4,400 cfs for the Byram River at U.S. Route 1 which was corroborated by a comparison of flood marks and initial runs of the HEC-RAS hydraulic profile model described in **Appendix B2**. Using the HEC-HMS runoff model described in **Section 5.0**, the flow at Pemberwick was interpolated from this estimate at U.S. Route 1 to be 4,520 cfs. This corresponds to a ratio of total excess to total rain of 0.72.

- A Pemberwick-South Wilton correlation developed by Peter Koch of the USACE estimated a flood peak at Pemberwick to be 3,192 cfs for the April 2007 flood

When it is necessary to extend the Pemberwick gage record using the Norwalk River data, the MOVE.2 record extension is preferred because it better preserves the statistical moments of the observed record on the Byram River (Hirsch, 1982). The area ratio extension is presented for comparison only.

4. DISCHARGE-FREQUENCY RELATIONSHIP

The goal of Task 2.2b of the Project Management Plan is to develop a set of flow regimes that represent the recurrence intervals of flooding on the Byram River. The results from several methods for determining the discharge-frequency curve are presented in this section.

4.1. Analysis of USGS Peak Record extension

The estimated annual peak flows, derived from record extension of the Byram River gage at Pemberwick, and discussed in **Section 3.2** are the basis of a partial duration discharge-frequency analysis. The 51-year record of annual peaks was fit to a Weibull probability distribution using HEC-FFA (USACE, 1992) using Bulletin 17B methodology (IACWD, 1982). **Table 5** shows annual peak discharge-frequency relationship calculated by the FFA software.

Date	Observed Annual Peak Flow from Byram River at Pemberwick (USGS 01212500) (cfs)	Observed Annual Peak Flow from Norwalk River at South Wilton (USGS 01209700) (cfs)	Estimated Annual Peak Flow of Byram River at Pemberwick using record from E. Branch Byram River at Riversville [3] (cfs)	Estimated Annual Peak Flows on Byram R. at Pemberwick		MOVE.2 Annual Peak Series for Byram R. at Pemberwick (cfs)	Area Ratio Annual Peak Series for Byram R. at Pemberwick (cfs)
				Using MOVE.2 [4] (cfs)	Using Area Ratio [5] (cfs)		
10/15/1955	-	-	-	-	-	4260 [6]	-
11/10/1962	-	249 [1]	484	186	213	484	484
11/29/1963	-	261 [1]	237	201	223	237	237
2/8/1965	-	470	173	462	401	173	173
2/13/1966	-	208 [1]	336	135	178	336	336
3/7/1967	-	365	431	330	312	431	431
5/28/1968	-	1020 [2]	1738	1149	870	1738	1738
3/25/1969	-	1100	581	1249	939	581	581
2/3/1970	-	525 [1]	937	530	448	937	937
9/14/1971	-	1360	1738	1574	1161	1738	1738
6/19/1972	-	1690	2880	1986	1442	2890 [7]	2880
2/2/1973	-	1610 [2]	1841	1886	1374	1841	1841
12/21/1973	-	1540	1567	1798	1314	1567	1567
9/26/1975	-	1220 [2]	2014	1399	1041	4140 [8]	2014
8/10/1976	-	827 [1]	751	908	706	751	751
3/22/1977	-	1440	1191	1674	1229	1191	1191
1/26/1978	-	1480	1004	1723	1263	1004	1004
1/21/1979	-	1940	1712	2298	1656	1712	1712
4/10/1980	-	2300	2152	2748	1963	2152	2152
2/20/1981	-	560	517	574	478	517	517
1/4/1982	-	772 [1]	718	839	659	718	718
4/16/1983	-	699 [1]	1004	748	597	1004	1004
7/7/1984	-	1054 [1]	1498	1192	900	1498	1498
9/27/1985	-	969	-	1085	827	1085	827
1/26/1986	-	962	-	1076	821	1076	821
4/4/1987	-	1710	-	2011	1459	2011	1459
2/20/1988	-	400	-	374	341	374	341
5/16/1989	-	1800	-	2123	1536	2123	1536
10/20/1989	-	1010	-	1136	862	1136	862
10/24/1990	-	1360	-	1574	1161	1574	1161
8/18/1992	-	612	-	639	522	639	522
11/23/1992	-	672	-	714	573	714	573
1/28/1994	-	1090	-	1236	930	1236	930
1/20/1995	-	313	-	265	267	265	267
1/27/1996	-	1510	-	1761	1289	1761	1289
12/2/1996	-	1400	-	1624	1195	1624	1195
1/24/1998	-	523	-	528	446	528	446
9/17/1999	-	1720	-	2023	1468	2023	1468
6/7/2000	-	495	-	493	422	493	422
3/30/2001	-	655	-	693	559	693	559
5/14/2002	-	556	-	569	475	596	475
1/2/2003	-	586	-	606	500	606	500
9/18/2004	-	1340	-	1549	1144	1549	1144
4/3/2005	-	892	-	989	761	989	761
4/23/2006	-	2340	-	2798	1997	2798	1997
4/16/2007	-	3490	-	4235	2978	3192 [9]	2978
9/7/2008	-	1320	-	1524	1126	1524	1126
12/12/2008	-	1230	-	1411	1050	1411	1050
3/30/2010	1660	-	-	1736	1272	1660	1660
8/28/2011	2690	-	-	2011	1459	2690	2690
12/8/2011	658	-	-	758	603	658	658

[1] Norwalk River peak flow is estimated for this event since the peak annual flow recorded at the Norwalk gage for this year did not correspond to the same event as the peak recorded for the same year on the E. Branch Byram River. The peak flow rate for this event was estimated from the daily record at the Norwalk gage using the antecedent and subsequent daily average flows with a nomograph developed by Langbein (1944)

[2] Annual peak flow at Norwalk gage was observed one calendar day after annual peak flows at East Branch River gage.

[3] Annual peak flow on the Byram River at Pemberwick was estimated from the observed record from the upstream gage on E. Branch Byram at Riversville shown in Table A.3. Comparing results at the two locations from the HEC-HMS model described in Section A.3, an exponential relationship (R^2=0.9928) between peak flow at each location was developed.

[4] Using the coincident record of estimated and observed peak annual flows at the gages on Norwalk River and Byram River at Pemberwick, the peak annual record on Byram River at Pemberwick was extended using MOVE.2 analysis and the longer record on Norwalk River gage (Hirsch, 1982).

[5] Using area ratio between the gage on Norwalk River at South Wilton (30.0 sq. miles) and the gage on Byram River at Pemberwick (25.6 sq. miles)

[6] The Feasibility Report by USACE (1977) estimated the October 1955 flood peak at 4,520 cfs at Route 1 bridge. Interpolating from this estimate using the HEC-HMS runoff model described in Section A.3 of this Appendix, the peak flow at Pemberwick is 4,260 cfs.

[7] The Feasibility Report by USACE (1977) estimated the June 1972 flood peak at 2,900 cfs. The flow estimated from the Riversville gage is 2,880 cfs. The average of the two (2,890 cfs) is used for the 1972 peak.

[8] The Feasibility Report by USACE (1977) estimated the September 1975 flood peak at 4,400 cfs at Route 1 bridge. Interpolating from this estimate using the HEC-HMS runoff model described in Section A.3 of this Appendix, the peak flow at Pemberwick is 4,520 cfs.

[9] A correlation with the South Wilton gage on the Norwalk developed by Peter Koch of the USACE is used for the April 2007 flood.

Table 4 - Extension of Peak Annual Flow Record of Byram River at Pemberwick

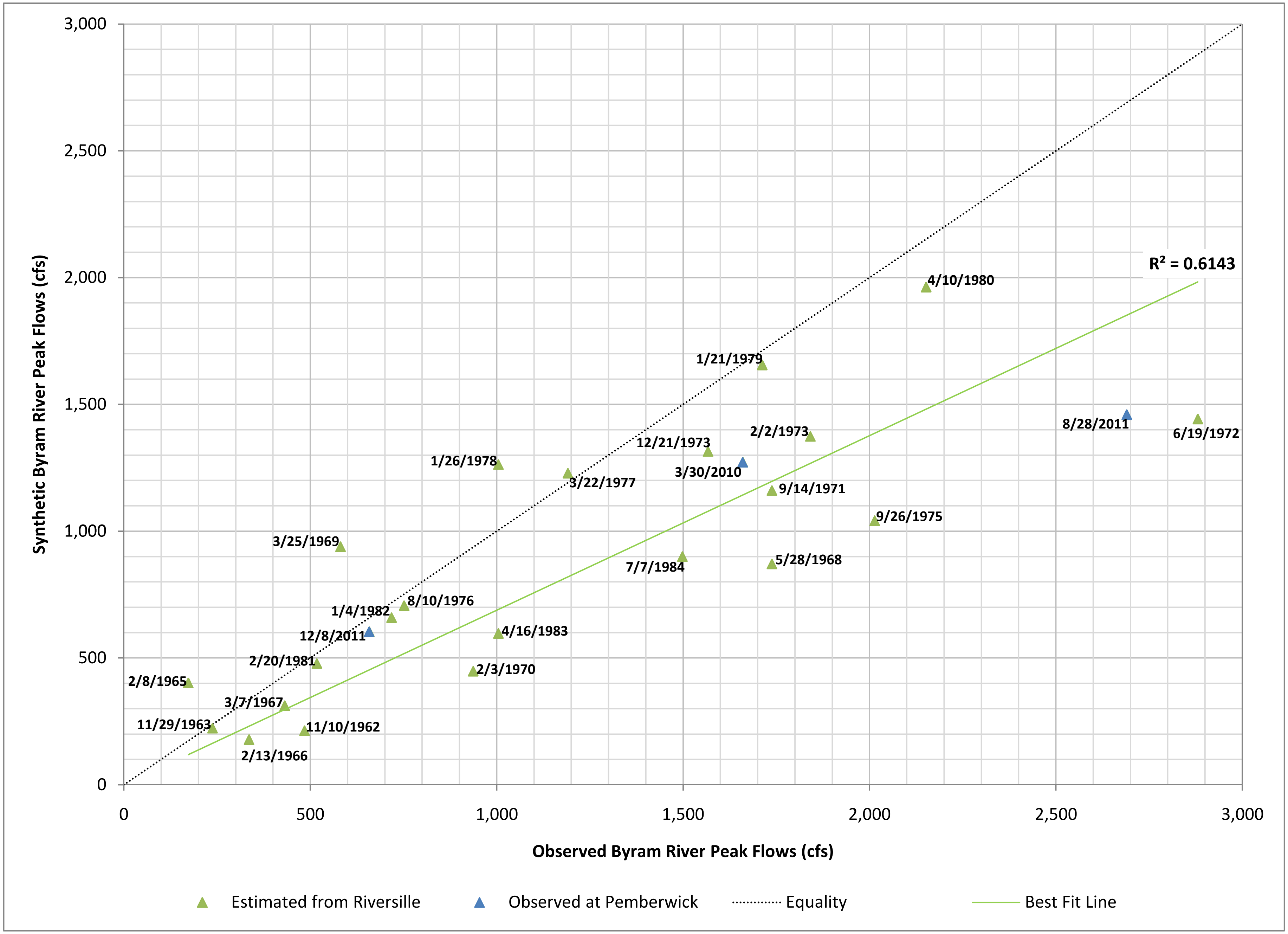


Figure 4A

Comparison of Area Ratio Synthetic and Observed Annual Peak Flows

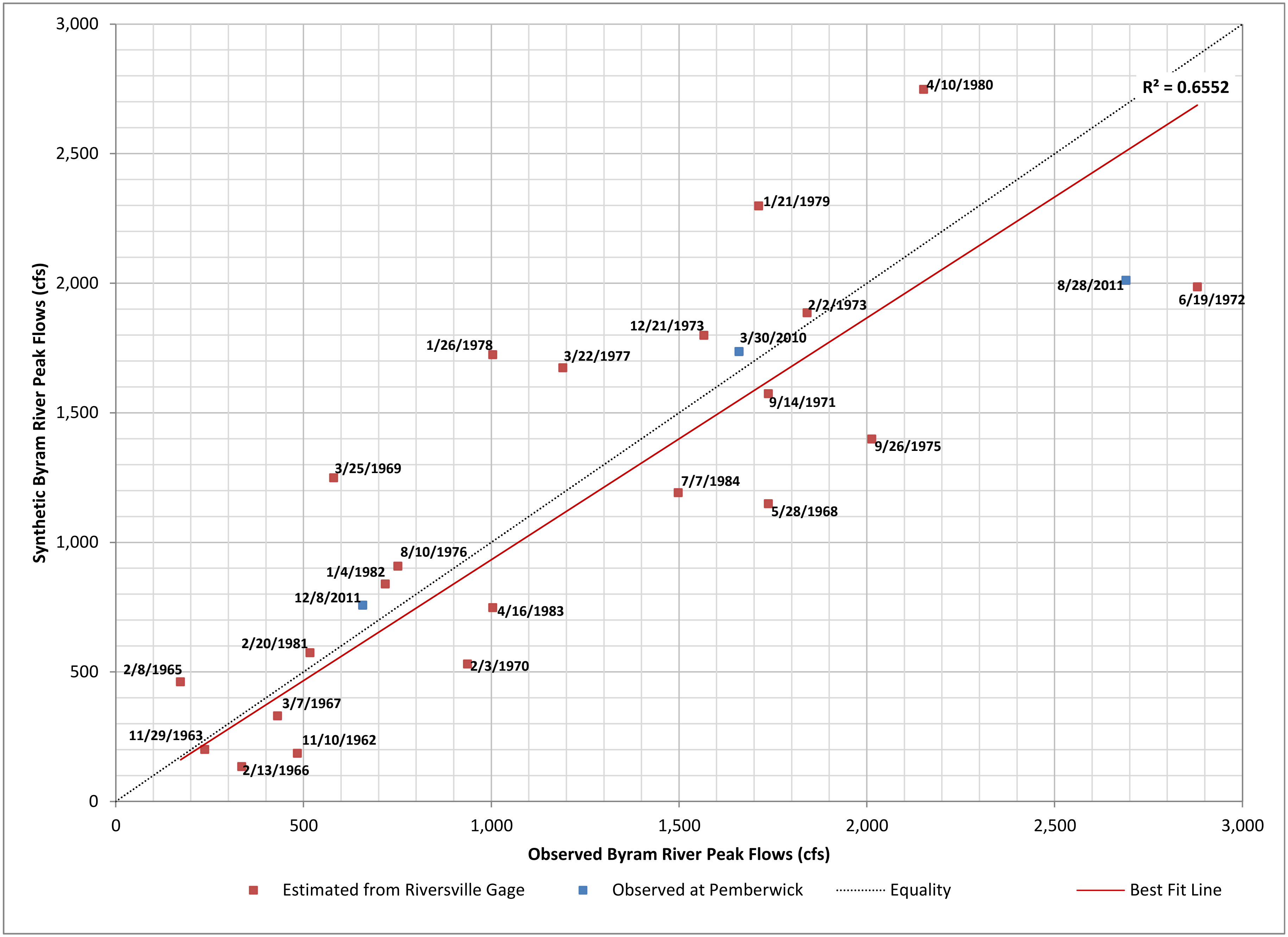


Figure 4B

Comparison of MOVE2 Synthetic and Observed Annual Peak Flows

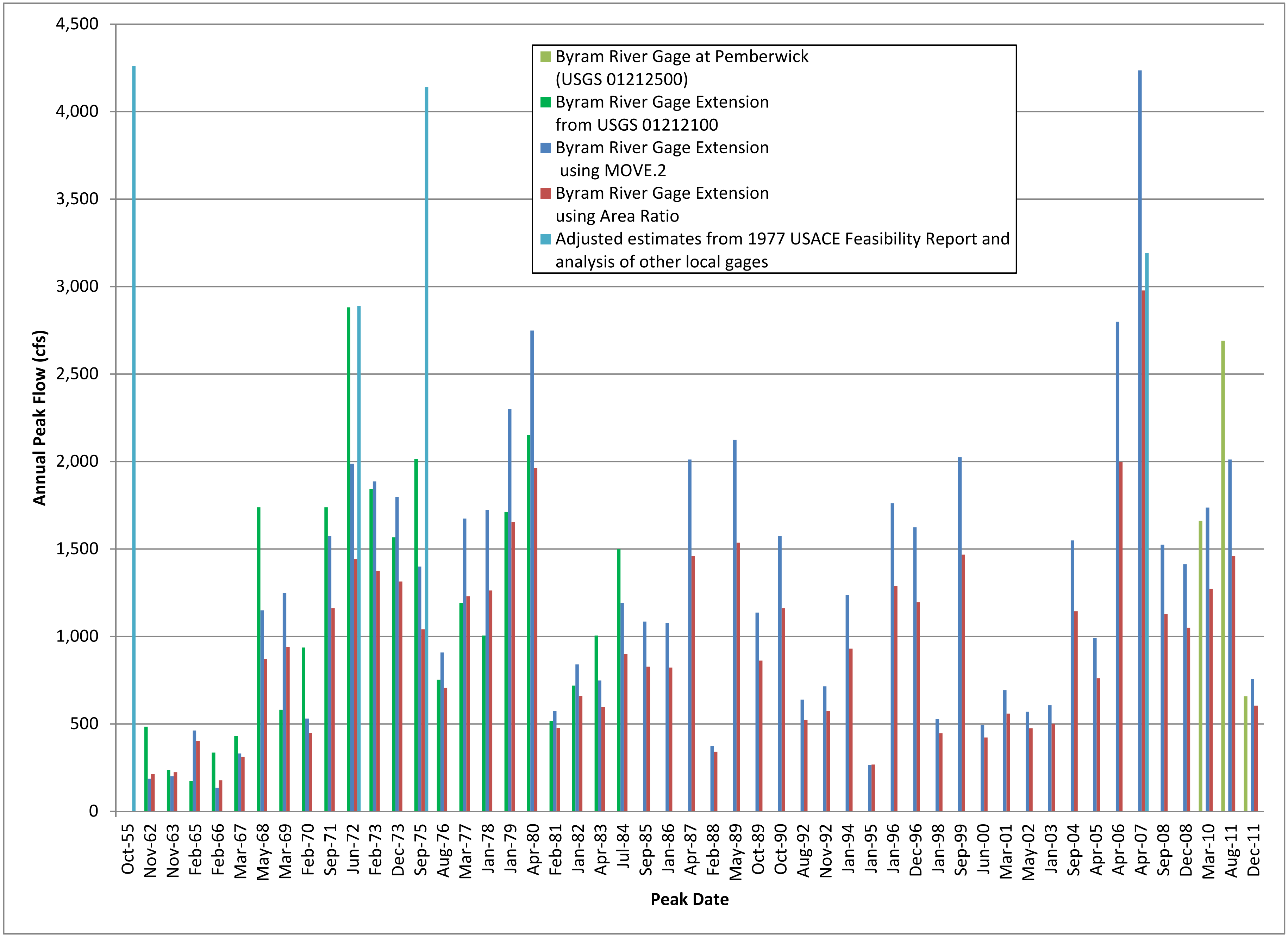


Figure 5

Byram River Extended Record Annual Peak Flows

Table 5 Discharge-Frequency for Byram River at USGS 01212500 (25.6 sq. miles)
MOVE.2 Peak Record Extension with adjustments and Uncertainty Analysis Confidence Limits

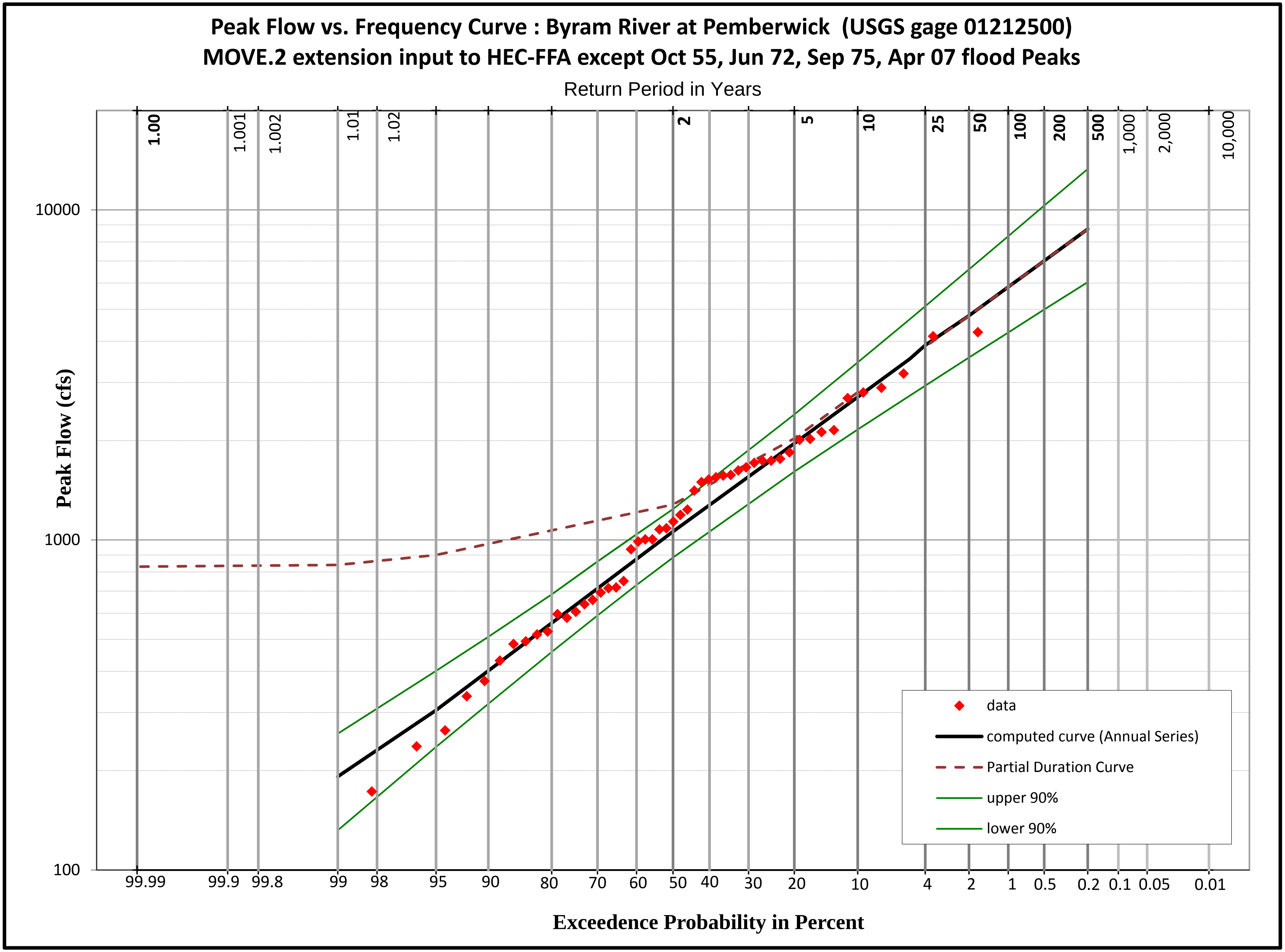
Annual Exceedance Probability	Return Period (yr)	Computed Peak Annual Discharge (cfs)	95% Confidence Limit 51 Year Equivalent		95% Confidence Limit 30 Year Equivalent		Partial Duration Curve (cfs)
			Lower Limit (cfs)	Upper Limit (cfs)	Lower Limit (cfs)	Upper Limit (cfs)	
0.9999	1.00						830
0.990	1.01	192	130	256	113	277	840
0.500	2	1,060	889	1,260	844	1,326	1,280
0.200	5	1,960	1,630	2,450	1,551	2,637	2,030
0.100	10	2,710	2,200	3,530	2,081	3,870	2,800
0.040	25	3,850	2,980	5,230	2,818	5,885	3,850
0.020	50	4,780	3,660	6,800	3,416	7,744	4,780
0.010	100	5,840	4,360	8,600	4,054	9,929	5,840
0.005	200	7,010	5,130	10,700	4,739	12,478	7,010
0.002	500	8,750	6,230	13,900	5,718	16,477	8,750

The partial duration curve was estimated from the peak annual discharge curve using a relationship described by Beard (1964). **Figure 6** shows a plot of the peak discharge vs. frequency curve.

4.2. Flow Uncertainty Analysis

As mentioned above, the period of flow record at the Byram River gage was extended to 51 years. This extended record was used with HEC-FFA, to estimate the flood frequency flows and confidence intervals (see **Table 5**). That analysis provided estimates of both the suite of peak discharges associated with flood frequency events as well as the associated confidence intervals. The flow uncertainty analysis builds on the earlier one, but reduces the period of record considered to 30 years. The reduction in number of years is in keeping with Table 4-5 (USACE, 1996) for a synthetically extended period of record, distributed using a calibrated rainfall/runoff model.

The Army Corps of Engineers Program HEC-SSP Statistical Software Package (USACE, 2016) was used for the uncertainty analysis. A “general frequency analysis” was developed to replicate the original flood frequency flows and confidence intervals. The replicated one includes the use of the systematic record of 50 years, as well as a single historic flow in 1956. Once the previous analysis was replicated, the “User Statistics – Equivalent Years of Record” was activated to modify the number of events to be equal to 30 years.



Peak Flow vs. Frequency Curve : Byram River at Pemberwick (USGS gage 01212500)
MOVE.2 extension input to HEC-FFA except Oct 55, Jun 72, Sep 75, Apr 07 flood Peaks
Return Period in Years
1.00
1.001
1.002
1.01
1.02
2
5
10
25
50
100
200
500
1,000
2,000
10,000
Peak Flow (cfs)
10000
1000
100
99.99
99.9
99.8
99
98
95
90
80
70
60
50
40
30
20
10
4
2
1
0.5
0.2
0.1
0.05
0.01
Exceedence Probability in Percent
data
computed curve (Annual Series)
Partial Duration Curve
upper 90%
lower 90%

Figure 6

Table 5 was revised to include the confidence limits associated with the uncertainty analysis. The result of this analysis is also presented graphically in **Figure 7**. These revised confidence limits can be utilized in the economic analysis.

4.3. NSS

The peak discharge frequency relationship was also estimated using the USGS National Streamflow Statistics (NSS) program (Ries et al., 2007). This program uses a multi-parameter regional regression to estimate the peak discharges at varying return periods (Ahearn, 2004). These parameters include the drainage area of the basin, the average basin elevation, and the 24-hour rainfall for varying return periods. The NRCC rainfall estimates discussed in **Section 2.2** and summarized in **Table 1** were used for the input parameters. **Table 6** summarizes the peak discharges found at relevant locations using this method.

Table 6 Peak Annual Flood Flows – USGS National Streamflow Statistics

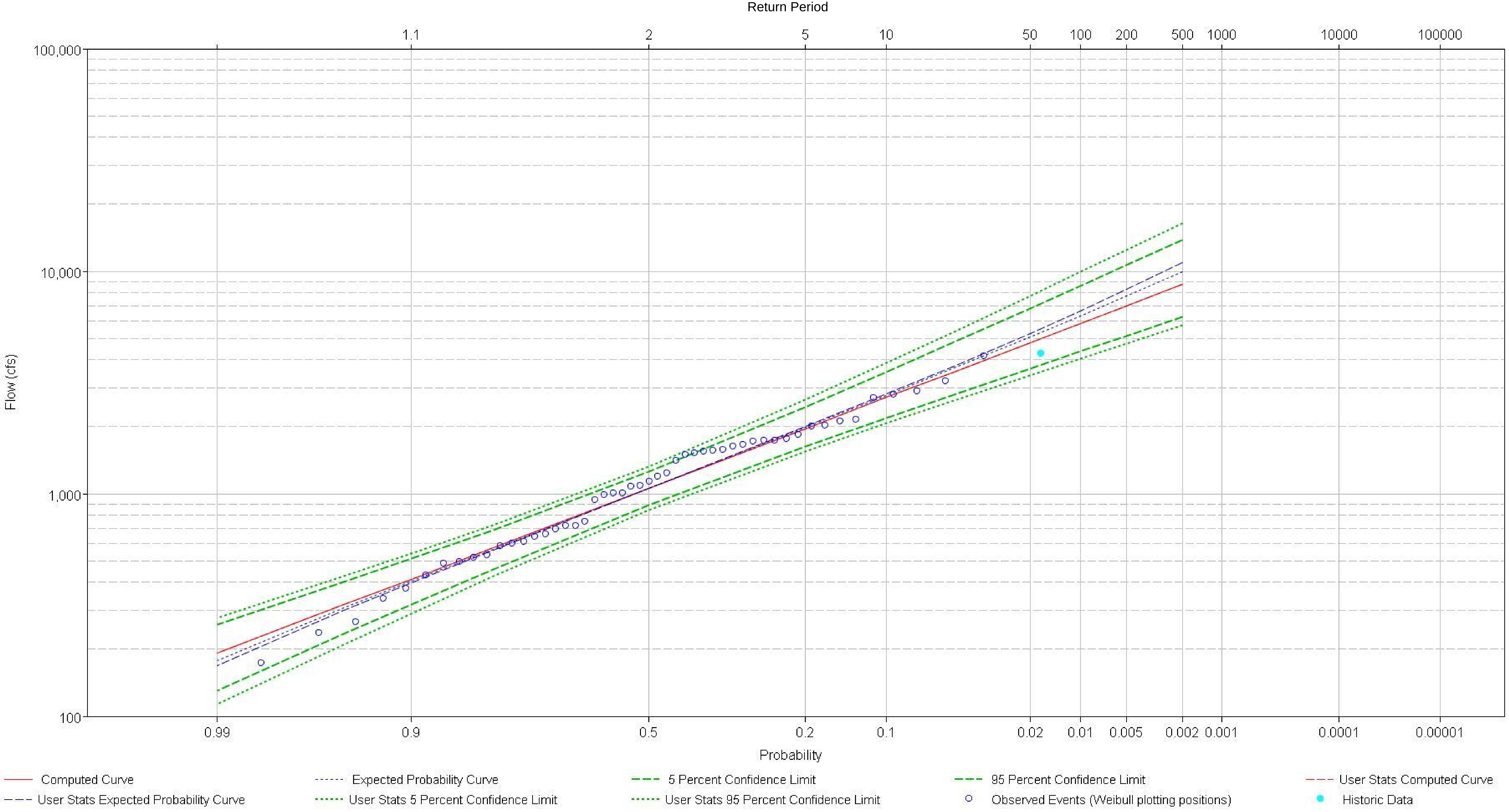
Location	Drainage Area (sq. miles)	Peak Discharge (cfs)					
		50% Flood	10% Flood	4% Flood	2% Flood	1% Flood	0.2% Flood
East Branch Byram River at confluence with Byram River	11.1	380	891	1,220	1,500	1,820	2,400
Byram River at north end of Toll Gate Pond	24.6	722	1,700	2,340	2,880	3,530	4,600
Byram River at Comly Ave. (upstream of Pemberwick Brook)	26	729	1,720	2,370	2,920	3,580	4,680
Byram River at Rte 1 West Putnam Ave	28.1	766	1,810	2,500	3,080	3,780	4,940
Byram River at Railroad crossing	28.4	758	1,790	2,470	3,060	3,750	4,910

4.4. Previous Studies

Previous estimates of the peak discharge-frequency relationship are available at relevant locations in the Byram River watershed. While the prior studies are often based on shorter records from neighboring basins, they provide a comparison for the discharge-frequency relationships developed in this study.

There are effective flood flows in the FEMA Flood Insurance Study (FIS) at three locations in the watershed (FEMA, 2010): (1) Byram River at Railroad Crossing, (2) Byram River at the north end of Toll Gate Pond, and (3) East Branch Byram River at its confluence with the main branch. The peak discharge estimates on the main branch Byram River are based on analysis performed by the USACE (USACE, 1964). The statistical analysis transposes 13 years of gage data on the Blind Brook at Rye, NY (USGS 01300000) to the Byram River watershed and

Figure 7 Exceedance Probability for Byram_Flow_03252018-FLOW-PEAK



assumes that the logarithms of the annual peak flows are normally distributed. The peak discharge estimates on the East Branch Byram River are based on a regional regression analysis developed for Connecticut (Weiss, 1977). The discharges are shown in **Table 7**.

Table 7 Peak Annual Flood Flows – Prior Studies

Location	Drainage Area (sq. miles)	Analysis	10% Flood Discharge (cfs)	2% Flood Discharge (cfs)	1% Flood Discharge (cfs)	0.2% Flood Discharge (cfs)
East Branch Byram River at Confluence with Byram River	11.1	(Weiss, 1977)	1,093	1,578	1,835	2,520
		(Ahearn, 2003)	1,450	2,490	3,000	4,340
Byram River at north end of Toll Gate Pond	25.6 (FEMA) 24.6 (CDM Smith)	(USACE, 1964)	2,950	4,660	5,500	8,090
Byram River at Rte 1 West Putnam Ave	29.1 (USACE) 28.1 (CDM Smith)	(USACE, 1977)	5,090	5,780	6,900	8,890
Byram River at Railroad Crossing	28.5 (FEMA) 28.4 (CDM Smith)	(USACE, 1964)	3,130	4,950	5,850	8,600

Using a unit hydrograph derived from the neighboring gaged basin of Blind Brook at Rye, NY, USACE simulated the discharge associated with hypothetical rainfall depths taken from the TP-40 design storm depths (Hershfield, 1961) as described in **Section 2.2**. The peak annual discharges estimated by this method for the Byram River at the U.S. Route 1 Bridge are shown in **Table 7**.

There is a discrepancy between the drainage areas reported in the FIS and those delineated by the project team in this study as described in **Section 5.1**. Notably the drainage area at the railroad crossing reported by FEMA (28.5 sq. miles) is very similar to the drainage area calculated by the project team (28.4 sq. miles), while both areas are smaller than the drainage area at the upstream Rte 1 Bridge of 29.1 sq. miles used in the unit hydrograph analysis by the USACE in their discharge estimates of historic storms (USACE, 1977).

In 2003, the USGS updated its discharge-frequency analysis of the East Branch Byram River at the gaged confluence with the Byram River (USGS 01212100) using the 19-year peak discharge record available (Ahearn, 2003). The improved estimate is also shown in **Table 7**.

4.5. Discussion

The discharge-frequency relationship developed from the record extension at Byram and described in **Section 4.1** was selected as the most appropriate representation of the peak discharge at the Pemberwick gage. The peak flows in the partial duration curve shown in **Table 5** are the design flows to which the HMS runoff model, described in the following **Section 5.0**, was calibrated.

5. RUNOFF MODEL

A runoff model of the of the Byram River watershed was developed from available data and was used at two stages of the analysis.

First, to develop a preliminary relationship between the record at USGS 0121500 and the record on the East Branch Byram River (USGS 01212100) which is necessary for the peak record extension described in **Section 3.2**. This formulation relied on physically-based runoff loss parameters described in this section (**Section 5.2**). This model was not calibrated and was not the model used to generate peak discharges for the alternative analysis.

A second version of the HMS model was revised and calibrated to the peak discharge regime generated from the statistical gage analysis at USGS 0121500 described in **Section 4.1**. Both "Initial and Constant" runoff parameters (**Section 5.2**) and Baseflow and Recession parameters (**Section 5.6**) were adjusted for each design and historic event to match the peak discharge described in **Table 3**. The calibrated model was then used to represent the routed storm event along the study reach and generate peak discharge at multiple locations upstream and downstream of the gage analysis.

The existing HEC-HMS runoff model built for the 2008 Byram River Drainage Evaluation Report (CDM, 2008) was updated to Version 3.5 of HEC-HMS. As described in this section, enhanced features were added to the existing model including calibrated runoff loss and unit hydrograph parameters, improved river reach routing methodology based on updated hydraulic modeling, and reservoir objects to represent attenuation of flood peaks at dams and other constrictions. **Figure 8** shows a schematic of the updated HEC-HMS runoff model.

The peak discharges from points along the main reach of the Byram River in simulations of the design floods are used as the steady flow regime input to the HEC-RAS hydraulic model described in **Appendix B2**.

5.1. Basin Delineation

At its mouth, the Byram River watershed is 30.0 square miles in area. For the purpose of runoff modeling, the watershed was divided into 14 sub-basins. The ArcHydro extension (version 2.0) to ArcGIS was used to delineate the sub-basins from topographic data obtained from the Town of Greenwich. Where the watershed extends into New York state, the 1/3 arc-cell digital elevation map (DEM) from the USGS was used. **Figure 9** shows the 14 sub-basins. **Table 8** shows the drainage area of each sub-basin.

5.2. Rainfall Infiltration Losses

To build the preliminary runoff model used in the gage transposition analysis described in Section 3.2, rainfall infiltration losses for each sub-basin were calculated using the NRCS runoff curve number (CN) approach (USDA, 2004). The CN of an area is a function of the predominate soils and the land use in the sub-basin. Geospatial soils data for the watershed was downloaded from the Soil Survey Geographic (SSURGO) database maintained by NRCS. Land use data was downloaded from the U.S. Geologic Survey (USGS) and reclassified to the categories in the curve number tables in TR-55 (USDA, 1986). The categorization of residential land use codes was validated by spatial imperviousness data also obtained from the USGS.

Spatially averaged curve numbers were calculated for each sub-basin from the geospatial intersection of the soils and land use data. The initial abstraction was calculated for each sub-basin using methodology in TR-55 (USDA, 1986). **Table 8** shows the NRCS curve numbers for each sub-basin in the watershed. The spatially averaged curve number upstream of the Comly Avenue Bridge is 70.1. The spatially averaged curve number at the mouth of the Byram River is 70.8.

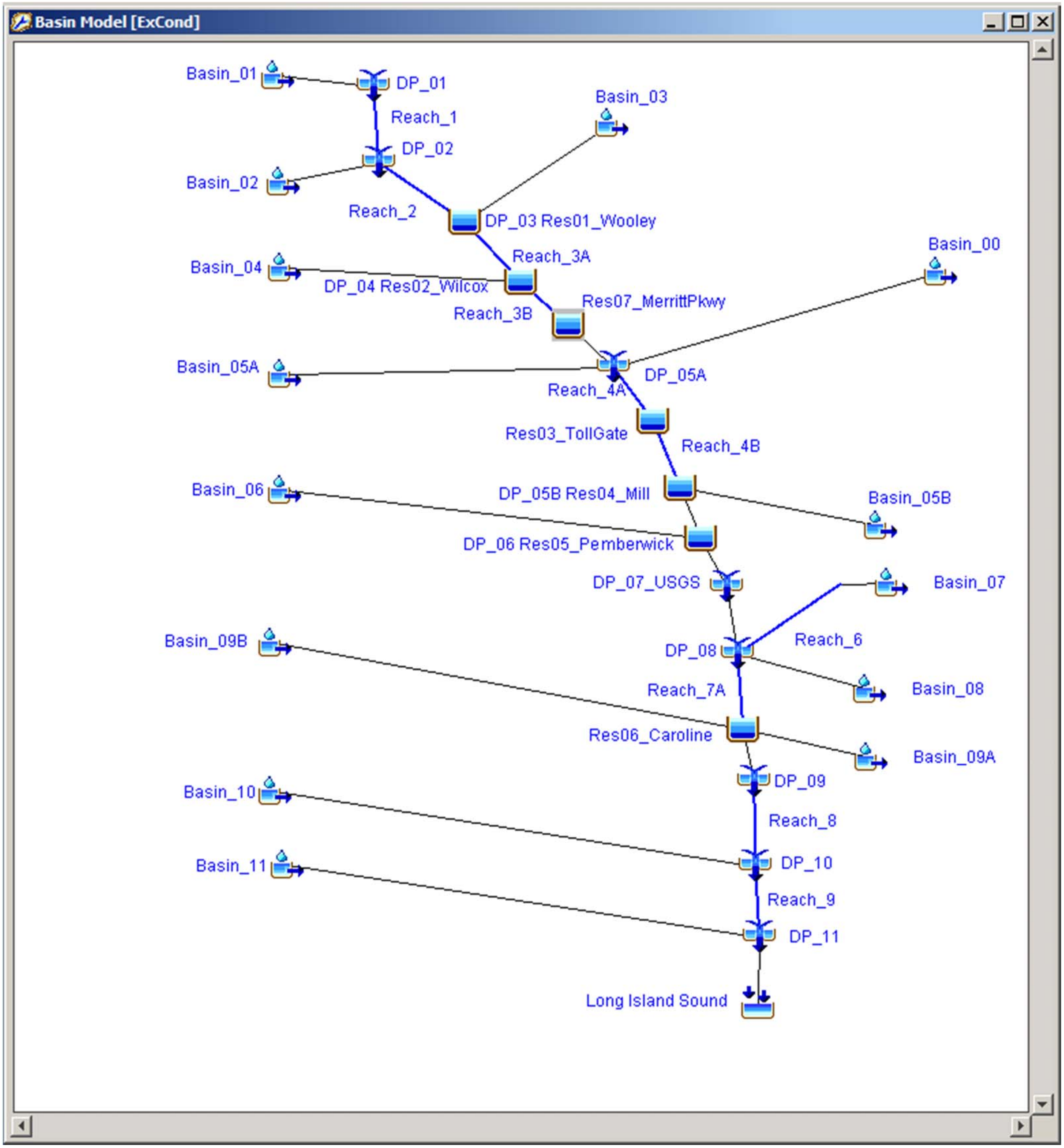


Figure 8

HEC-HMS Runoff Model
Network Schematic
Byram River Watershed

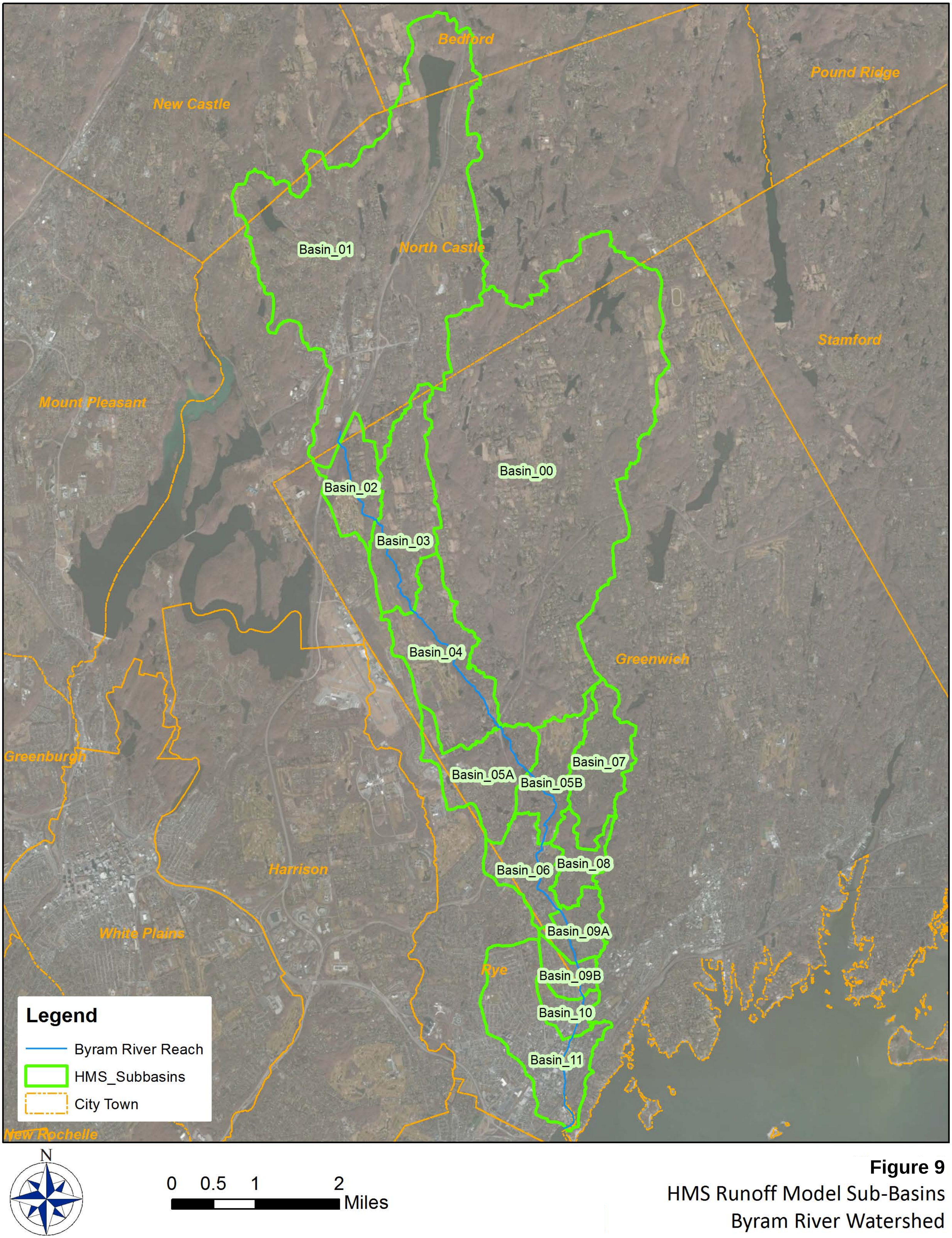


Figure 9
HMS Runoff Model Sub-Basins
Byram River Watershed

Table 8 HEC-HMS Sub Basin Runoff Characteristics

HMS Sub-basin	Area (sq. miles)	Average Percent Impervious (%)	NRCS Curve Number (Existing)	Initial Abstraction (in)
Basin_00	11.2	2.6 %	71	0.817
Basin_01	8.5	11.4 %	69	0.899
Basin_02	0.7	6.8 %	71	0.817
Basin_03	1.5	1.9 %	70	0.857
Basin_04	1.6	9.5 %	70	0.857
Basin_05A	1.1	12.4 %	72	0.778
Basin_05B	0.7	14.4 %	64	1.125
Basin_06	0.7	25.2 %	73	0.740
Basin_07	1.0	3.6 %	63	1.175
Basin_08	0.4	8.5 %	68	0.941
Basin_09A	0.3	21.6 %	74	0.703
Basin_09B	0.4	37.4 %	77	0.597
Basin_10	0.3	35.2 %	82	0.439
Basin_11	1.6	30.8 %	82	0.439

The final version of the runoff model was calibrated to the discharge-frequency analysis at the extended gage record described in Section 4.1. For these runs, the physically-based NRCS curve number loss method was replaced with the Initial Deficit and Constant Loss method, where a single set of loss parameters were selected across all of the basins. For comparison to observed storms as described in **Section 6**, a different set of loss parameters was selected for each event such that they would match the observed peak. For the design storms as described in **Section 7**, the loss parameters were selected to match the peak design flows. **Table 9** lists all of the different Initial Deficit and Constant Loss parameters used in all of the runoff simulations.

Table 9 HEC-HMS Loss Parameters

Simulation	Initial Deficit (in)	Constant Loss (in/hr)
Flood of October 1955	0	0.001
Flood of June 1972	0	0.084
Flood of September 1975	0	0.056
Flood April 2007	0	0.235
March 2011	0	0.130
April 2011	0.8	0.280
May 2011	0	1.300

Simulation	Initial Deficit (in)	Constant Loss (in/hr)
Hurricane Irene (2011)	0	0.440
Tropical Storm Lee (2011)	0.3	0.460
100% Flood Event	0	0.614
50% Flood Event	0	0.440
20% Flood Event	0	0.272
10% Flood Event	0	0.212
4% Flood Event	0	0.184
2% Flood Event	0	0.183
1% Flood Event	0	0.208
0.5% Flood Event	0	0.246
0.2% Flood Event	0	0.310

In the Initial Deficit and Constant Loss method, sub-basins differed by imperviousness as listed in **Table 8**.

5.3. Basin Response

The runoff response from each of the sub-basins was simulated in HEC-HMS using the Clark Unit Hydrograph approach (Clark, 1945). The Clark Unit Hydrograph is defined by two parameters: (1) time of concentration and (2) storage. It was necessary to use the Clark Unit Hydrograph to accurately represent the recession limb of the flood hydrograph observed in the available gage record. Without gage data available for each of the 14 sub-basins, the parameters for each individual Clark unit hydrograph were initially estimated using the physically based NRCS Unit Hydrograph approach (USDA, 2007), and then calibrated in aggregate to the available gage record on the Byram River at Pemberwick as described in **Section 3.0**.

Time of Concentration for the NRCS Unit Hydrograph was calculated for each sub-basin using the velocity method as described by the NRCS (USDA, 2010). The total Time of Concentration for each sub-basin is the sum of the travel times associated with sheet flow and shallow concentrated flow. The ArcHydro GIS extension was used to determine the longest path of flow for each catchment.

Sheet Flow occurs over the first 300 feet of the drain line (USDA, 1986). The upstream and downstream elevations of the sheet flow path in each sub-basin were calculated using the available topographic data. The associated slope was used to calculate the travel time for sheet flow (Overton and Meadows, 1976). A Manning roughness of n=0.13 and n=0.07 was selected to represent the forested areas and the suburban residential areas in the watershed respectively (USDA, 2010).

After the first 300 feet of the drain line, it was assumed that overland flow transitioned into Shallow Concentrated Flow. The longest path of each sub-basin was subdivided into segments of similar slope and land use. The travel time of each segment was calculated using the slope, length, and velocity coefficient associated with the land use as described in the NRCS National Engineering Handbook Part 630.1502(b) Table 15-3 (USDA, 2010 and Kent, 1964). The total Shallow Concentrated Flow travel time is for each sub-basin the sum of all of the segment travel times.

Table 10 shows the total Time of Concentration for each sub-basin. The Lag Time used to define the unit hydrograph response for each sub-basin was assumed to be 60% of the Time of Concentration based on Equation 15-3 in the NRCS National Engineering Handbook Part 630.1501(e) (USDA, 2010 and Simas, 1996). The Lag Time for each sub-basin was input to the HEC-HMS model.

Table 10 HEC-HMS Sub Basins

HMS Sub-basin	Area (sq. miles)	Longest Length (ft)	Slope	NRCS Unit Hydrograph Method (Physically-based)				Clark Unit Hydrograph (Calibrated)	
				Sheet Flow Travel Time (hr)	Shallow Concentrated Flow Travel Time (hr)	Time of Concentration (hr)	Lag Time (min)	Time of Concentration (hr)	“R” - Storage (hr)
Basin_00	11.2	40,350	0.0102	0.30	12.2	12.5	450	2.85	11.8
Basin_01	8.5	32,200	0.0115	0.15	12.4	12.6	456	2.14	8.9
Basin_02	0.7	8,300	0.0080	0.31	4.1	4.4	156	0.66	2.7
Basin_03	1.5	15,650	0.0126	0.20	3.6	3.8	138	0.96	4.0
Basin_04	1.6	14,700	0.0239	0.25	2.2	2.4	90	0.68	2.8
Basin_05A	1.1	19,300	0.0126	0.53	5.3	5.8	210	1.23	5.1
Basin_05B	0.7	7,900	0.0235	0.26	1.0	1.2	42	0.37	1.6
Basin_06	0.7	9,000	0.0293	0.20	1.3	1.5	54	0.38	1.6
Basin_07	1.0	12,900	0.0216	0.21	2.3	2.5	90	0.63	2.6
Basin_08	0.4	6,000	0.0250	0.24	1.0	1.3	48	0.27	1.1
Basin_09A	0.3	9,150	0.0182	0.18	2.3	2.5	90	0.48	2.0
Basin_09B	0.4	5,150	0.0400	0.16	0.6	0.8	30	0.18	0.8
Basin_10	0.3	5,550	0.0297	0.20	0.9	1.0	36	0.23	1.0
Basin_11	1.6	11,300	0.0196	0.22	3.3	3.5	126	0.58	2.4

Once the NRCS time of concentration parameters were estimated, Clark Unit Hydrograph parameters were selected to match the shape of the NRCS Unit Hydrograph for each sub-basin. Then the Clark Unit Hydrograph parameters were calibrated in aggregate to the observed discharge record. The calibrated Clark Unit Hydrograph parameters are shown in **Table 10**.

5.4. Reach Routing

Ten reaches in the Byram River watershed were identified for flow routing in the basin runoff model. The availability of the detailed hydraulic HEC-RAS model of the Byram River, described in **Appendix B2** of this report, provided detailed storage-discharge relationships for each reach, which allowed for the Modified Puls routing method (Chow, 1964 and Henderson, 1966) to be applied for reach routing. **Figure 10** shows the spatial location of the reaches and **Table 11** shows associated river stations in the HEC-RAS model.

Table 11 HEC-HMS Reaches

HMS Reach	RAS River Station Boundaries		Length (ft)	Slope (ft/ft)	HEC-RAS Storage for sample discharge (acre-feet)		
	Upstream (ft)	Downstream (ft)			Q = 500 cfs	Q = 1500 cfs	Q = 3000 cfs
Reach_1	52,659.2	46,613.6	6,240	0.0018	705.6	2,998.4	4,123.9
Reach_2	46,613.6	41,417.6	4,569	0.0156	19.1	42.4	69.7
Reach_3A	39,898.8	32,609.7	7,735	0.0136	18.1	37.0	65.9
Reach_3B	31,995.9	29,709.2	1,953	0.0103	13.0	33.8	54.0
Reach_4A	28,543.1	26,652.0	1,914	0.0035	18.5	34.3	60.1
Reach_4B	25,807.0	22,258.2	3,738	0.0035	16.7	31.5	62.6
Reach_6	N/A	N/A	4,430	0.0120	N/A	N/A	N/A
Reach_7A	14,991.7	13,544.3	1,414	0.0105	2.9	6.2	9.9
Reach_8	9,190.9	6,524.4	2,813	0.0005	41.9	44.6	52.0
Reach_9	6,524.4	3,928.3	2,478	0.0023	170.0	170.4	171.6

The HEC-RAS model simulated a range of 15 steady flow rates including those exceeding the expected 500-year discharge. The total storage in the reach was determined for each of the 15 steady flow rates to create a storage-discharge curve for each reach. **Table 11** shows the storage in acre-feet for a representative 3 of the 15 simulated flow rates.

By far the most available flood storage is in "Reach_1", which has the lowest slope of the non-tidally influenced reaches and flat wide flood banks extending into adjacent wetlands. The downstream boundary of the reach is defined by a narrowing of the river banks and flood plain at a small privately owned dam. Just upstream of the dam is the Bedford Street culvert, which has a 14 ft wide by 8 ft high arch-shaped opening that restricts conveyance during flooding events.

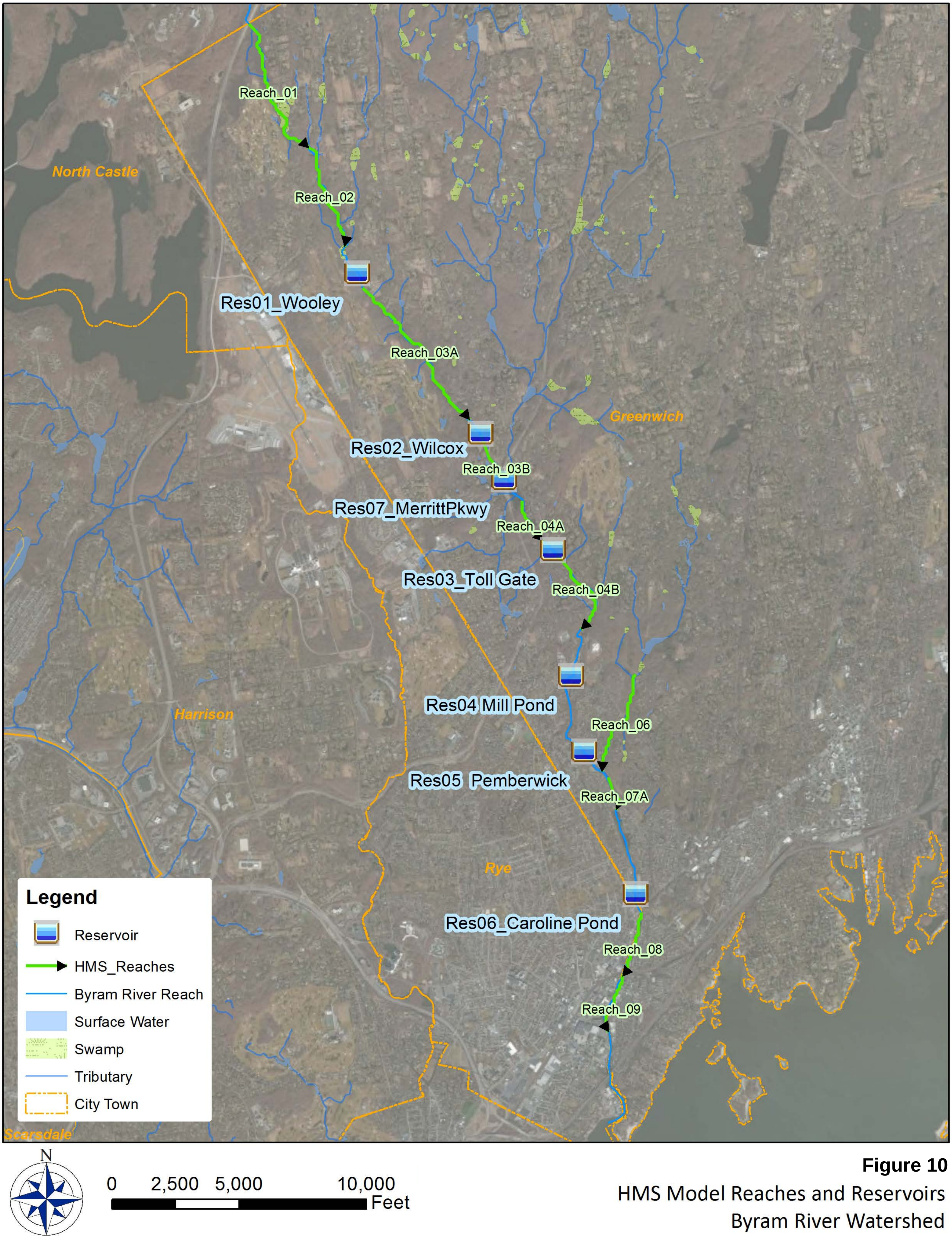


Figure 10

HMS Model Reaches and Reservoirs

Byram River Watershed

"Reach_6" is the only HEC-HMS reach that is not on the Byram River, and is not represented by the HEC-RAS model. The storage-discharge curve for "Reach_6" could not be calculated from HEC-RAS results. Instead a simple lag time was estimated with Manning's velocity equation, an assumed hydraulic radius (R=3.4 ft), and a similar roughness coefficient to those in the Byram River (n=0.04). The lag time used for the reach was 9.2 minutes.

5.5. Reservoirs

Reservoirs were added to the model network at locations along the main reach where either an inline structure or channel narrowing might create storage during a large event that would attenuate the peak discharge downstream. Seven locations were identified and are summarized in **Table 12** and shown by location in **Figure 10**.

Table 12 HEC-HMS Reservoirs

HMS Reservoir	Location Name	RAS River Station of Reservoir Outlet (ft)	Outlet Min. Elev. (ft NAVD88)
Res01	Wooley Pond	RS. 39,867.0	El. 280.0
Res02	Wilcox Pond	RS. 31,994.6	El. 179.8
Res03	Toll Gate Pond	RS. 25,919.5	El. 136.3
Res04	Mill Pond (American Felt Co. Dam)	RS. 19,750.6	El. 109.3
Res05	Pemberwick Dam	RS. 16,211.1	El. 70.9
Res06	Caroline Pond (US of Rte. 1 bridge)	RS. 9,476.7	El. 0.6
Res07	US of Merritt Pkwy	RS. 29,915.3	El. 141.3

The stage-area relationship upstream of each reservoir outlet was defined by the topographic data available from the Town of Greenwich using Spatial Analyst. The stage-discharge relationship of each reservoir outlet was defined by the rating curve of the associated cross section in the HEC-RAS hydraulic model. An iterative process was applied wherein the set of steady state flows used to develop the rating curve in HEC-RAS was adjusted based on the peak flow rates resulting from the improved HEC-HMS model runs.

Since the focus of the proposed alternatives is between Caroline Pond and the Rte 1 bridge, it is necessary to adjust the storage of the area for hydrologic simulations of different alternatives. The associated reservoir object in the model ("Res06") was adjusted to use a stage-storage relationship instead of the stage-area relationship used for the other reservoirs. The stage-storage relationship was developed from the associated cross sections in the HEC-RAS model runs.

5.6. Baseflow and Recession Parameters

In each historic and design simulation, the baseflow contribution was modeled using a recession constant and ratio to peak methodology with parameters selected to match the observed Byram River discharge at Pemberwick. The design storm simulations use the baseflow parameters of Hurricane Irene. **Table 13** shows the baseflow and recession parameters used in each simulation. **Section 6.0** describes the historic simulations and includes calibration plots.

Table 13 HEC-HMS Baseflow and Recession Parameters

Simulation	Initial Flow (cfs/sq-mile)	Recession Constant	Ratio to Peak
Flood of October 1955	0.67	0.9307	0.0145
Flood of June 1972	4.30	0.8814	0.0716
Flood of September 1975	0.57	0.9247	0.0608
Flood April 2007	4.37	0.8110	0.0774
March 2011	3.88	0.6955	0.0900
April 2011	1.19	0.8270	0.1200
May 2011	1.77	0.9000	0.1000
Hurricane Irene (2011)	0.47	0.9048	0.0223
Tropical Storm Lee (2011)	2.27	0.8366	0.0700
Design Storms (100% through 0.2%)	0.47	0.9048	0.0223

5.7. Future Without Project

For Future Without Project, the runoff model loss parameter for each sub-basin was adjusted to reflect expected future land use based on input from town planners. As described in **Section 5.2**, existing land use category spatial data was the basis of the existing conditions loss parameters for each sub-basin. Planners from the Town of Greenwich modified the spatial land use data to reflect expected future development, increasing the estimated curve number and impervious values for 7 of the 14 sub-basins. The other 7 sub-basins are not expected to see additional development in the future. The Existing Conditions and Future Without Project CN values and % impervious are shown in **Table 14**.

Table 14 CN Values for Existing and Future Conditions

HMS Sub-basin	Area (sq. miles)	Existing Conditions		Future Conditions	
		CN Value	% Impervious	CN Value	% Impervious
Basin_00	11.2	71	2.6 %	72	4.3 %
Basin_01	8.5	69	11.4 %	69	11.4 %
Basin_02	0.7	71	6.8 %	72	8.5 %
Basin_03	1.5	70	1.9 %	70	1.9 %
Basin_04	1.6	70	9.5 %	71	11.2 %
Basin_05A	1.1	72	12.4 %	74	15.8 %
Basin_05B	0.7	64	14.4 %	66	17.8 %
Basin_06	0.7	73	25.2 %	74	26.9 %
Basin_07	1.0	63	3.6 %	67	10.5%
Basin_08	0.4	68	8.5 %	68	8.5 %
Basin_09A	0.3	74	21.6 %	74	21.6 %
Basin_09B	0.4	77	37.4 %	77	37.4 %
Basin_10	0.3	82	35.2 %	82	35.2 %
Basin_11	1.6	82	30.8 %	82	30.8 %

There are no other changes to the Existing Conditions model in the Future without Project model.

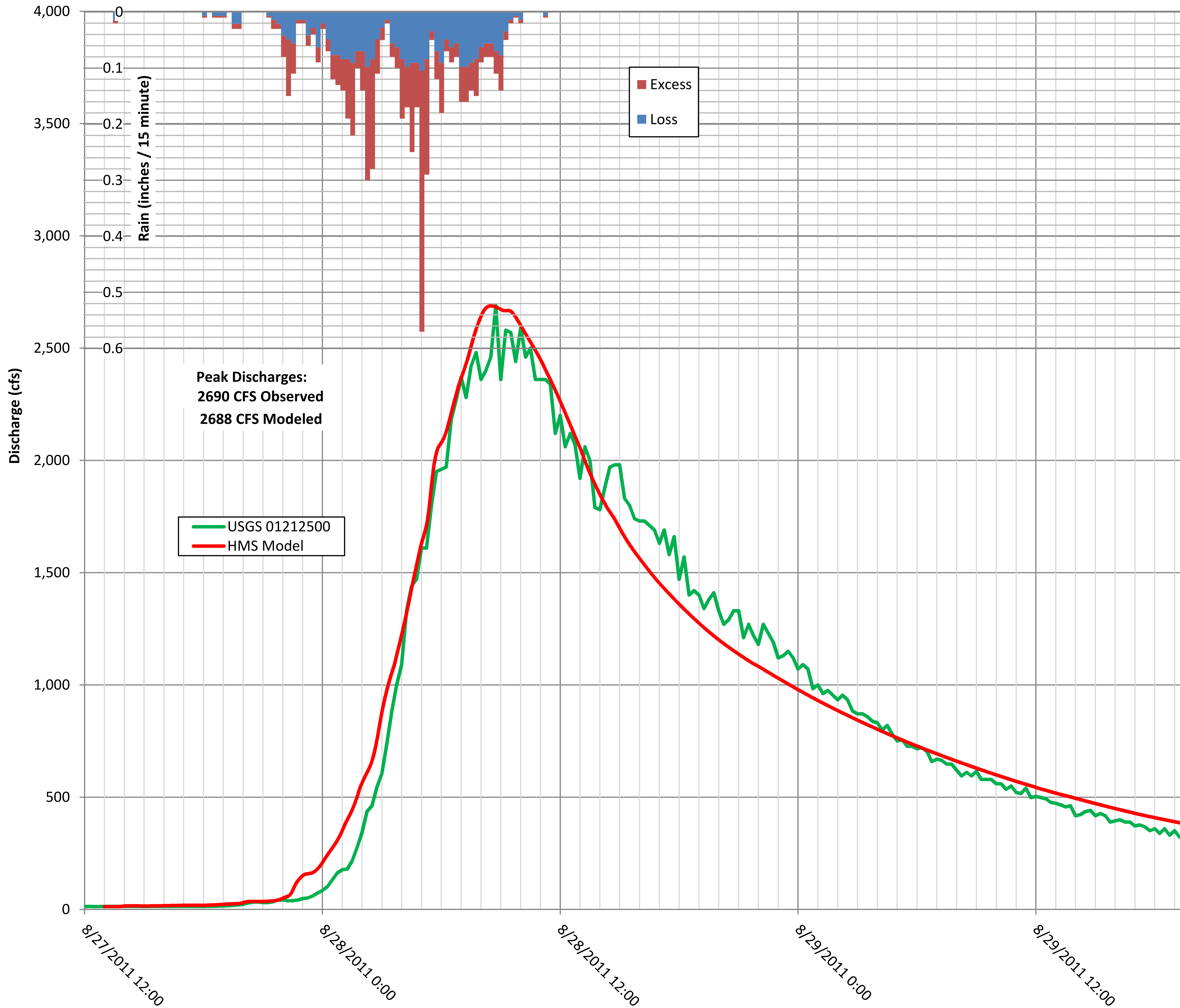
6. RUNOFF SIMULATIONS OF HISTORIC EVENTS

A series of historic flood events was simulated using the HEC-HMS runoff model described in **Section 5.0** for the purpose of calibrating hydraulic model parameters to observed discharge data and estimates of historic discharge from prior studies. The simulated discharge from these historic HMS runs was used as input to the HEC-RAS model, described in **Appendix B2**, to compare the resulting peak water surface elevations to observed high water marks.

6.1. Hurricane Irene (August 2011)

The first event studied for calibration of the HMS model was Hurricane Irene in 2011, for which there was a discharge record of 15-minute data available on the Byram River at Pemberwick, CT (USGS 01212500), as described in **Section 3.1**.

Figure 11 shows the modeled and observed hydrographs for this event on the Byram River at Pemberwick corresponding to a drainage area of 25.6 sq. miles. The cumulative rainfall used for this event, as discussed in **Section 2.1,** is also shown.



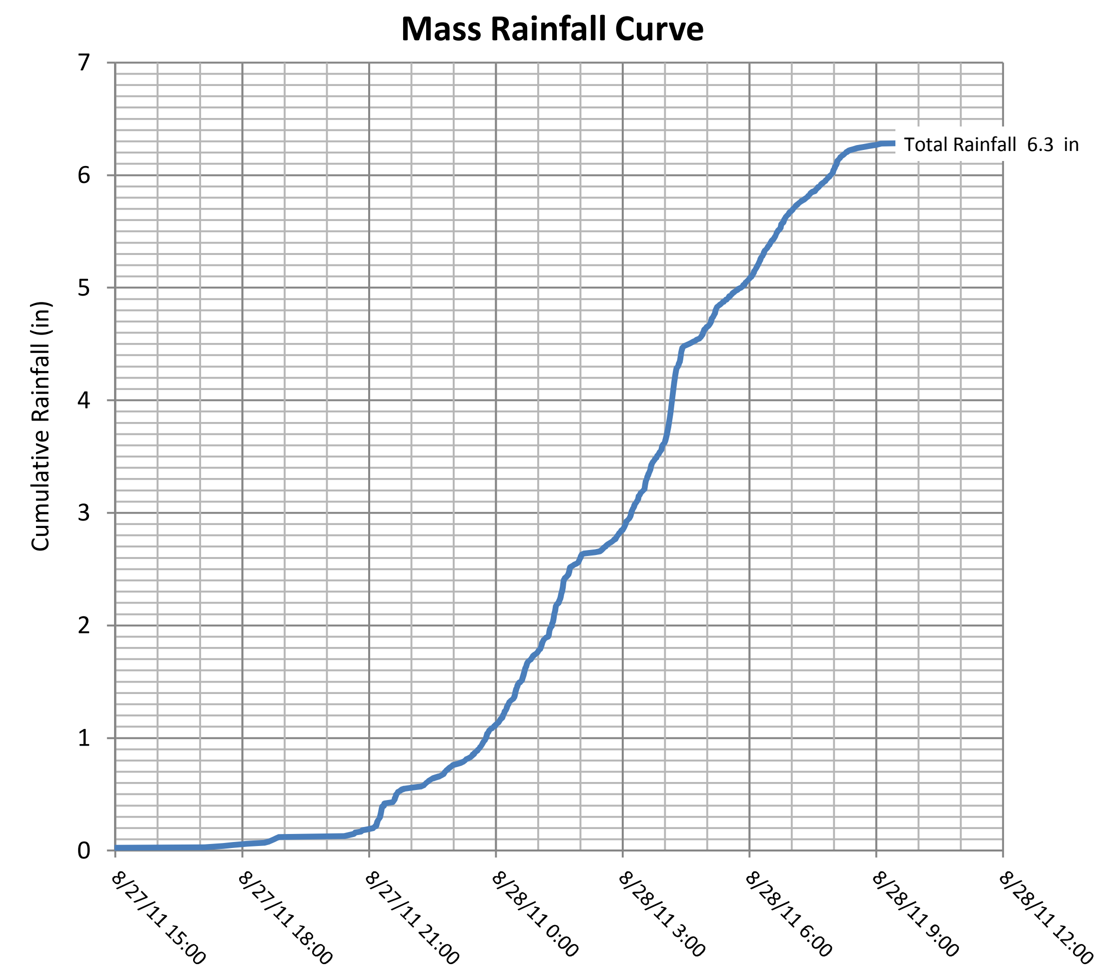


Figure 11

Hurrican Irene Calibration Plot

Byram River at USGS 01212500 (25.6 sq. miles)

The peak flows observed at USGS 01212500 were 2,690 cfs. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak within 1% of the observed.

6.2. April 2007 Flood (Nor'easter)

The Nor'easter of April 2007 caused major flooding in Southern Connecticut including along the Byram River. Because the event occurred prior to the October 2009 installation of the USGS gage on the Byram River at Pemberwick, CT (USGS 01212500), there is no discharge record available for calibration. The discharge hydrograph at Pemberwick (Comly Ave Bridge) simulated with the HEC-HMS runoff model is shown in **Figure 12**. The cumulative rainfall used for this event, as discussed in **Section 2.1,** is also shown.

While there is no discharge record, there are some observed high water marks available of varying quality, which were used to calibrate the runoff model using the rating curves from the HEC-RAS hydraulic model described in **Appendix B2**.

6.3. October 1955 Flood

The storm of October 1955 caused major flooding on the Byram River. At the time there was no USGS gage on the Byram River. In 1977, the USACE estimated the discharge on the Byram River associated with the Flood of 1955 with a synthetic unit hydrograph derived from the neighboring gaged basin of Blind Brook at Rye, NY (USGS 01300000) which has a drainage area of 9.2 sq. miles (USACE, 1977); the USGS lists the same gage with a drainage area of 8.9 sq. miles. With the derived unit hydrograph, USACE estimated the peak discharge on the Byram River at the U.S. Route 1 Bridge as 4,520 cfs. The report states that the drainage area USACE assumed for the Byram River at the U.S. Route 1 Bridge is 29.1 sq. miles, although using the basin delineation described in **Section 5.1**, the project team calculated the drainage area for this location is 28.1 sq. miles.

The USACE Feasibility Report also references a peak discharge estimate at the American Felt Co. Dam of 3,320 cfs. Using area ratio, the USACE provided a second estimate of the discharge at U.S. Route 1 of 4,200 cfs (USACE, 1977). The report states that the drainage area USACE assumed for this location was 23 sq. miles, while the updated drainage area calculated by the project team for this location is 25.3 sq. miles. Using the two updated drainage areas for these two locations, the area ratio transposed discharge at the U.S. Route 1 bridge is 3,690 cfs.

Using the same 3-hour rainfall intensity record as in the USACE Feasibility Report and described in **Section 2.3**, the discharge hydrograph of the Flood of 1955 was simulated using the HEC-HMS runoff model. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak within 1% of the USACE estimate at the U.S. Route 1 bridge (4,520 cfs). The hydrograph at the U.S. Route 1 Bridge (DP_09) is shown in **Figure 13** along with the hydrograph calculated by USACE with a

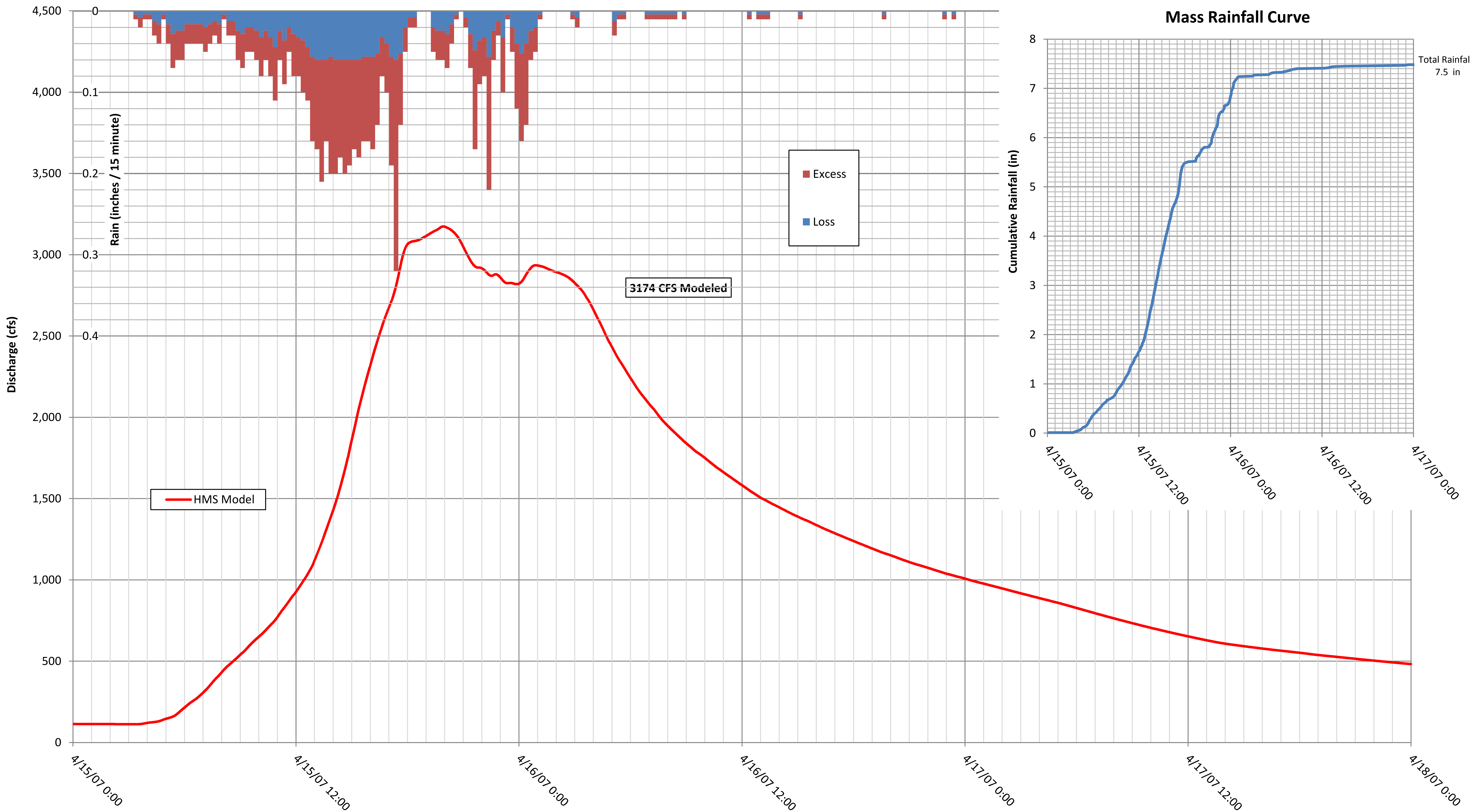
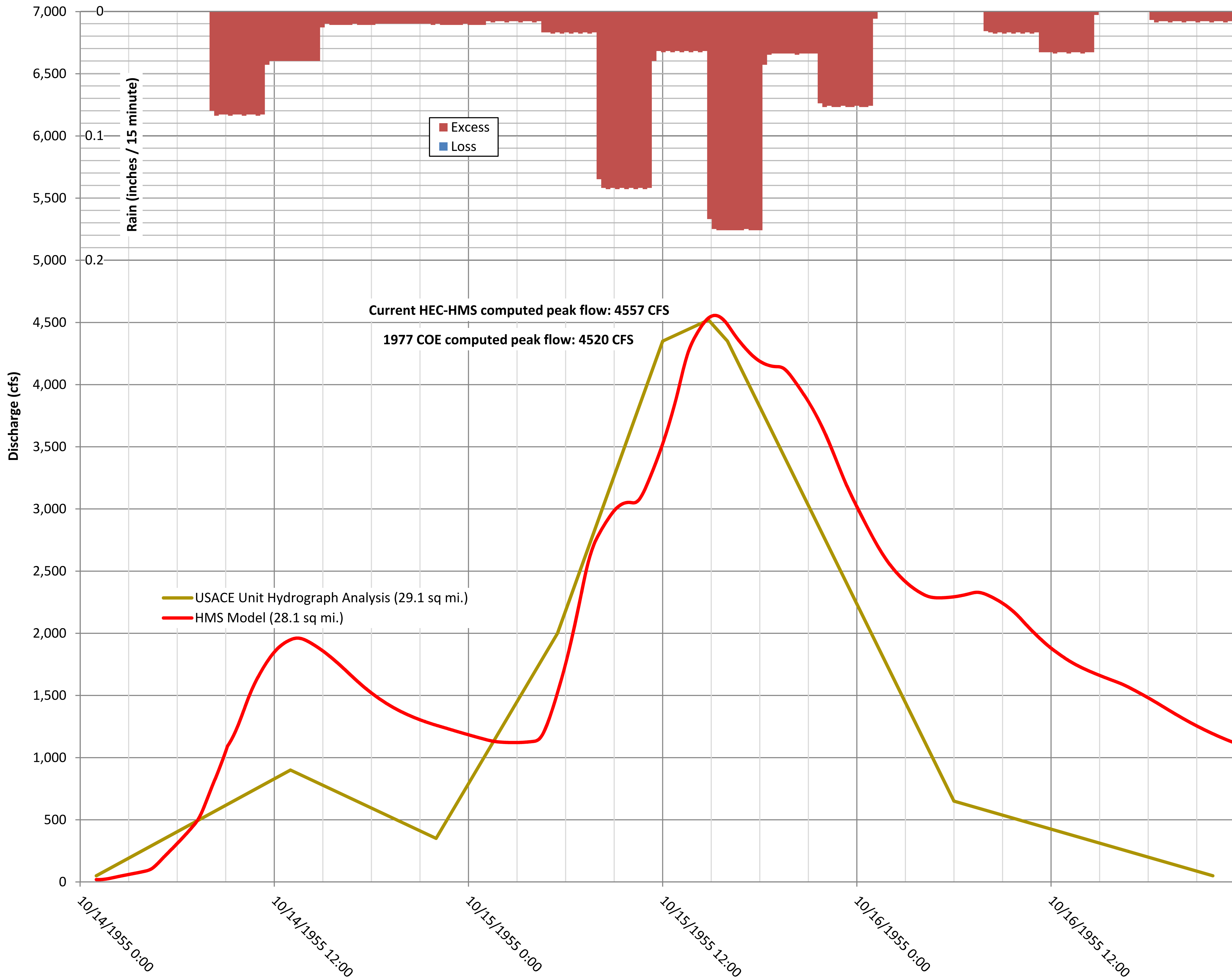


Figure 12

April 2007 Nor'easter Calibration Plot

Byram River at USGS 01212500 (25.6 sq. miles)



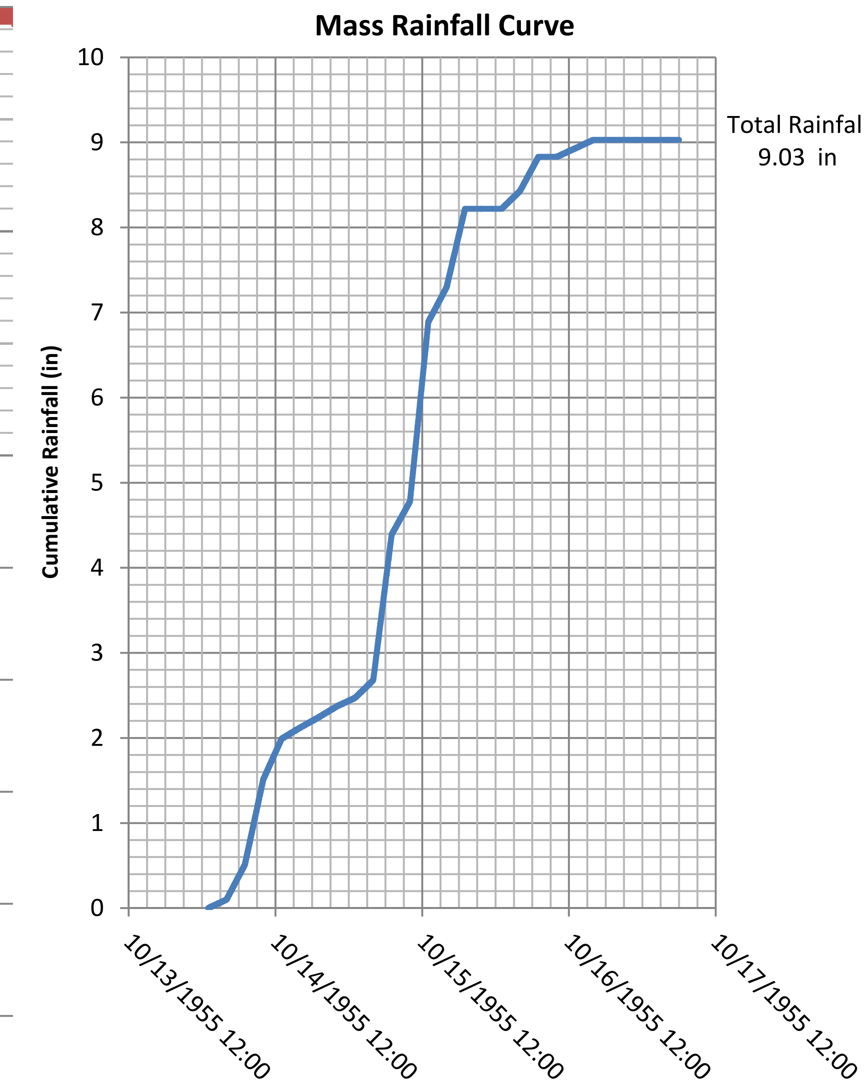


Figure 13

Flood of October 1955

HEC HMS model vs. USACE Analysis

Byram River at Rte 1 Bridge

transposed unit hydrograph. The modeled discharge at the American Felt Co. Dam (DP_05B) is 4,050 cfs.

The total rainfall representing the storm of October 1955 used for both the HEC-HMS runoff model and the UASCE unit hydrograph approach was 9.03 inches. In order to match the 1977 USACE estimate of peak discharge, the simulated loss depth for the HMS model is 0 inches.

6.4. June 1972 Flood (Tropical Storm Agnes)

Tropical Storm Agnes in June 1972 caused major flooding on the Byram River. At the time the only USGS gage in the basin was on a small tributary to the East Branch Byram River at Round Hill, CT (USGS 01211700). The peak discharge recorded at this gage on June 19, 1972 was 245 cfs, but since this gage drains only a small fraction of even the East Branch area (1.7 sq. miles of 11.2 sq. miles) it was not used to calibrate even the sub-basin representing the East Branch (Basin_00).

The USACE used the unit hydrograph derived from Blind Brook to estimate the discharge hydrograph of the Byram River associated with Tropical Storm Agnes. The analysis estimates the peak discharge at the U.S. Route 1 Bridge as 3,120 cfs (USACE, 1977). The total rainfall depth was 5.5 inches. The assumed rainfall infiltration loss depth was 3.1 inches.

Using the same 3-hour rainfall intensity record as in the USACE Feasibility Report and described in **Section 2.3**, the discharge hydrograph of the 1972 Flood was simulated with the HEC-HMS runoff model. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak within 5% of the USACE estimate at the U.S. Route 1 bridge (3,265 cfs). The runoff model hydrograph at the U.S. Route 1 Bridge (DP_09) is shown in **Figure 14** along with the hydrograph calculated by USACE with a transposed unit hydrograph. The adjusted rainfall infiltration loss was 2.0 inches.

6.5. September 1975 Flood (Hurricane Eloise)

Rainfall from Hurricane Eloise in September 1975 resulted in the highest flood on record for all of the neighboring basins that were gaged at the time. The USACE used the unit hydrograph derived from Blind Brook to estimate the discharge hydrograph of the Byram River associated with Hurricane Eloise. The analysis estimates the peak discharge at the U.S. Route 1 Bridge as 4,400 cfs (USACE, 1977). The total rainfall depth was 9.1 inches. The assumed rainfall infiltration loss depth was 3.7 inches.

Using the same 3-hour rainfall intensity record as in the USACE Feasibility Report and described in **Section 2.3**, the discharge hydrograph of the 1975 Flood was simulated with the HEC-HMS runoff model. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak within 8% of the USACE estimate at the U.S. Route 1 bridge (4,680 cfs). The runoff model hydrograph at the U.S. Route 1 Bridge (DP_09) is shown in **Figure 15** along with the hydrograph calculated by USACE with a transposed unit hydrograph. The rainfall infiltration loss was 5.2 inches.

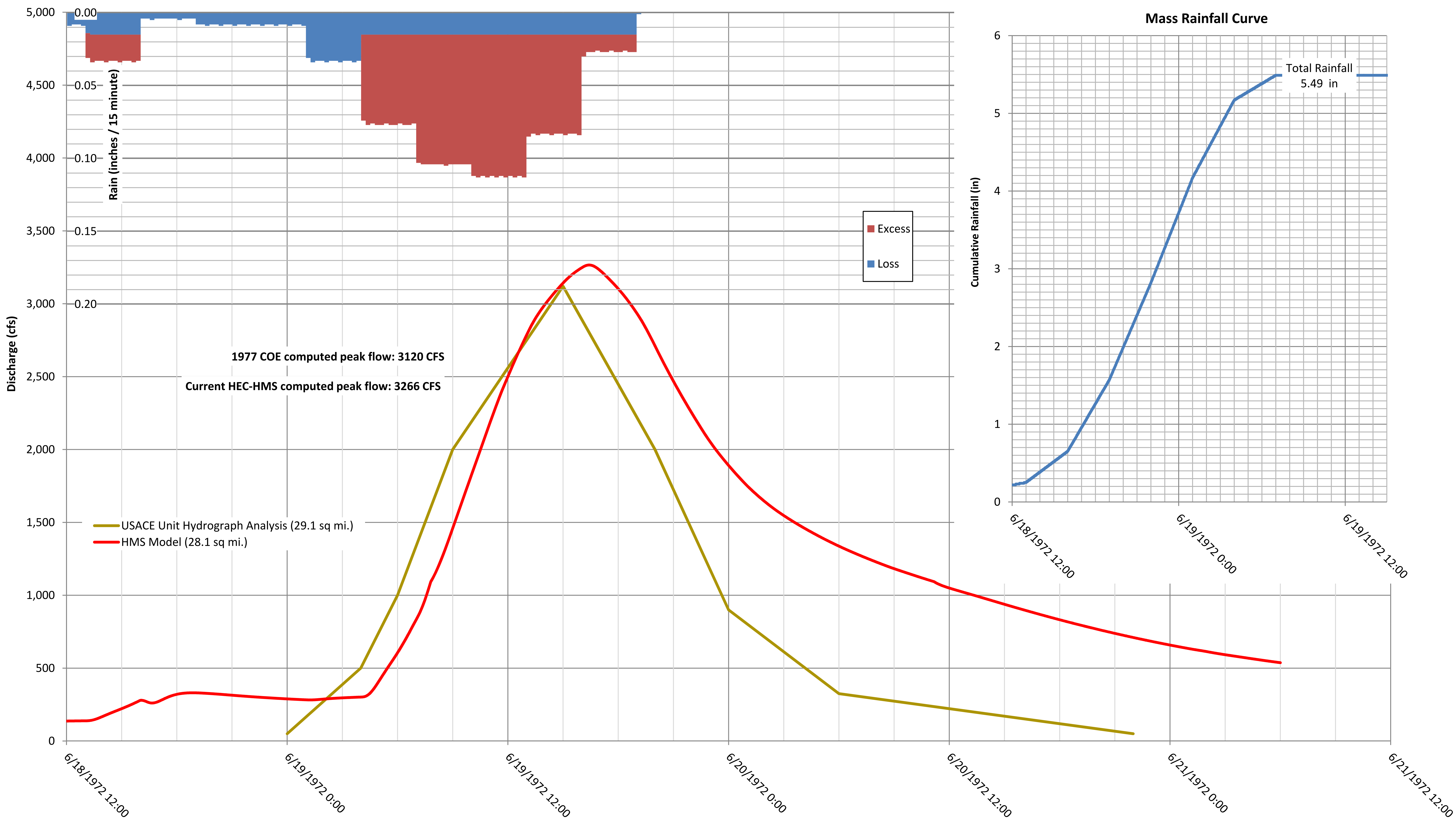


Figure 14

Flood of June 1972

HEC HMS model vs. USACE Analysis

Byram River at Rte 1 Bridge

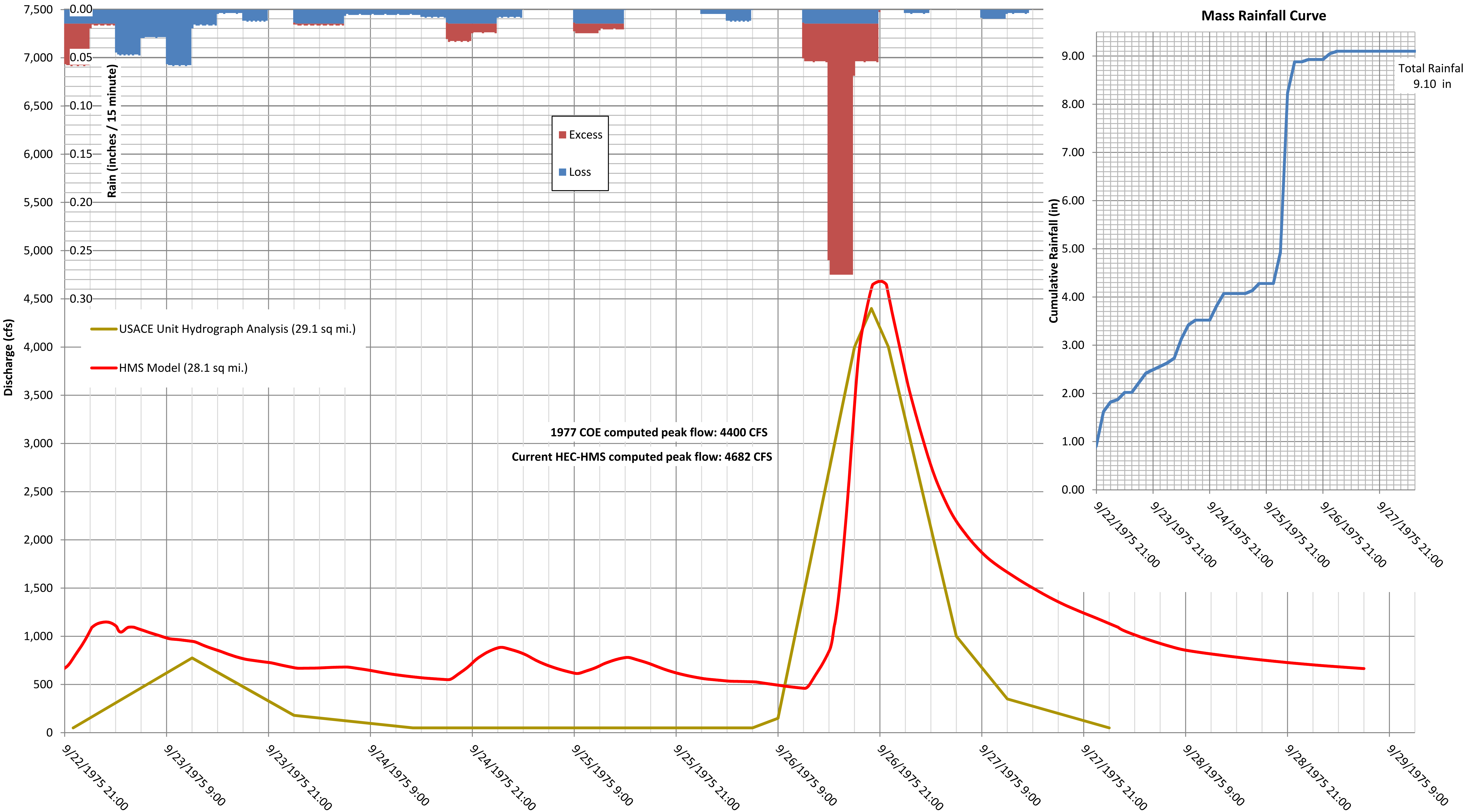


Figure 15
Flood of September 1975
HEC HMS model vs. USACE Analysis
Byram River at Rte 1 Bridge

6.6. Other Events (2009 - 2013)

Several other events coinciding with the recent 3-year discharge record on the Byram River at Pemberwick, CT (USGS 01212500) were also simulated to inform the calibration process. High flow events were selected from the gage record for which there was adequate rainfall data to drive the HMS model. All the additional events that were identified occurred in 2011.

Two of the events selected occurred within days of each other and were analyzed together. The first of these events occurred on March 7 and the second was on March 11. **Figure 16** shows the calibration plot and the cumulative rainfall for these events. The peak flow observed for the March 7 event was 1,800 cfs and the peak flow observed for the March 11 event was 1,920 cfs. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak of the March 11 event within 1% of the USGS observation at the Pemberwick gage. The simulated peak at Pemberwick for the March 7 event (2,073 cfs) is 15 % greater than the observed.

A rainfall gage at the USGS gage 01212500 representing the southern-most extent of the contributing basin observed 1.2 inches less rainfall than at the ASOS gage at the Westchester Airport only 3.6 miles northwest of the gage and only 2 miles west of the basin centroid during the same event. This may mean that the March 2011 storms were very spatially variable, and without more detailed precipitation data it is difficult to accurately represent the hydrograph with a runoff model. While acknowledged that NEXRAD data may be available for a portion of the basin in New York state, it is assumed that the other simulated events sufficiently represent the historic record without a detailed spatial analysis of the March 2011 event.

The next event occurred on September 8, shortly after Hurricane Irene. **Figure 17** shows the calibration plot and the cumulative rainfall for this event. The observed peak flow for this event was 1,840 cfs. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak of the September 8 event (1,953 cfs) within 8% of the USGS observation at the Pemberwick gage.

The next event occurred on April 17. **Figure 18** shows the calibration plot and the cumulative rainfall for this event. The observed peak flow for this event was 1,540 cfs. The initial loss and deficit parameters selected for this event as described in **Section 5.2** and **Table 9** brought the simulated peak of the April 17 event (1,600 cfs) within 4% of the USGS observation at the Pemberwick gage.

6.7. Runoff Calibration Summary

As described in **Sections 5.2** and **5.6**, the loss and baseflow parameters in the HEC-HMS runoff model were adjusted to match the observed discharge record on the Byram River (USGS 01212500) or the historic peak discharge previously estimated by USACE (1977). **Figure 19** shows a comparison of simulated and observed peak discharges for the 9 events discussed in **Section 6.0**.

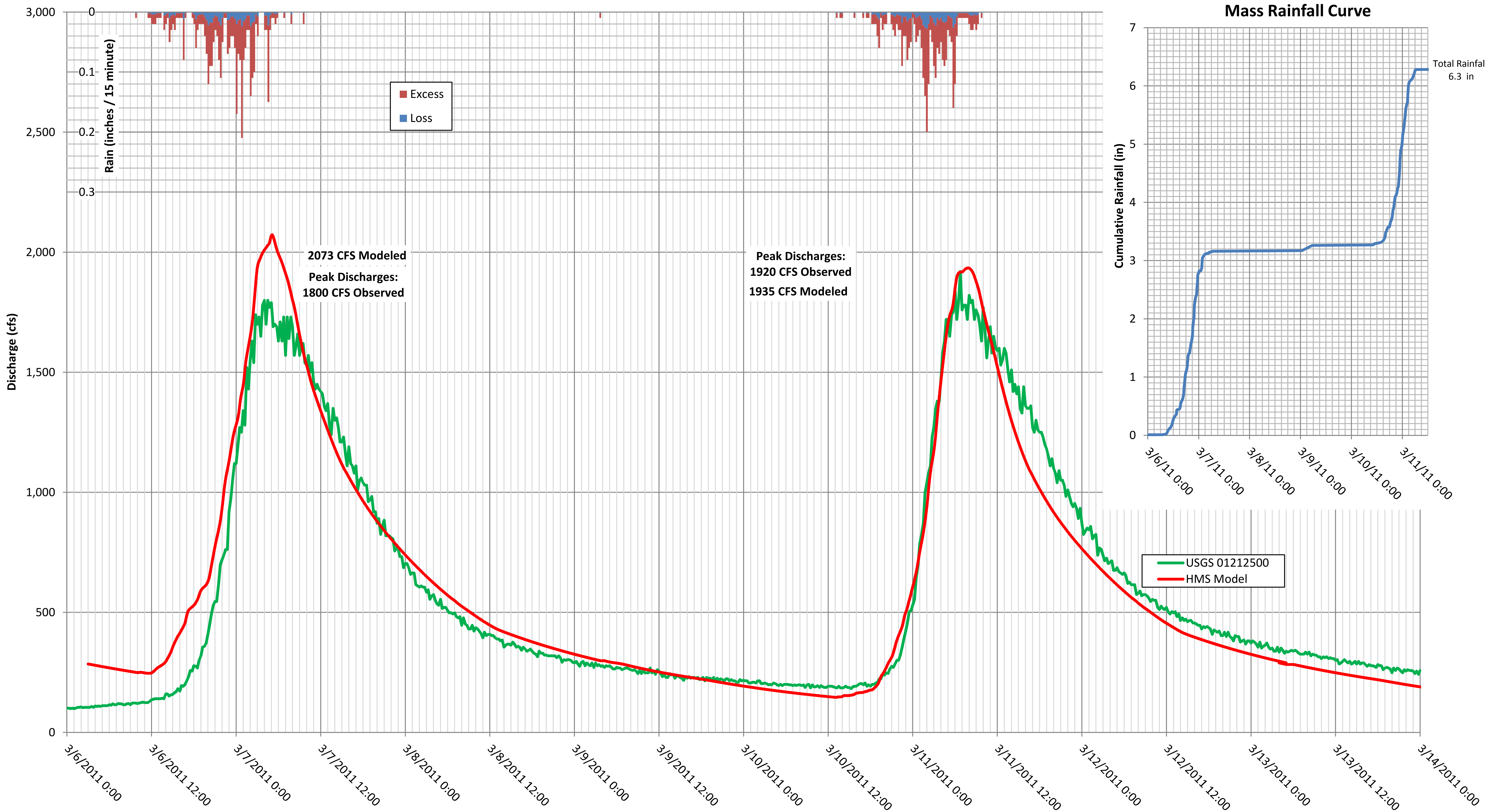


Figure 16

March 2011 Calibration Plot

Byram River at USGS 01212500 (25.6 sq. miles)

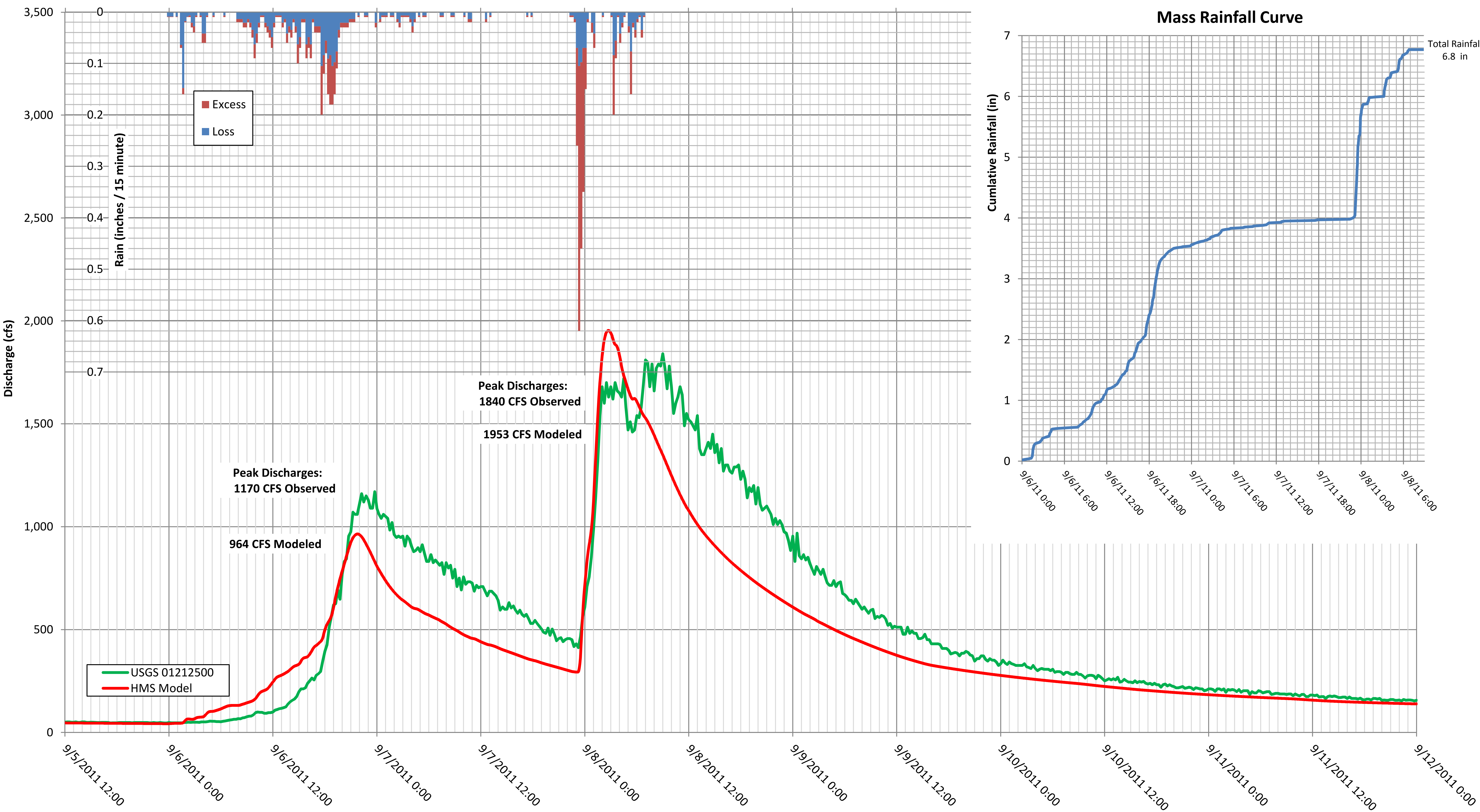


Figure 17

September 2011 Calibration Plot

Byram River at USGS 01212500 (25.6 sq. miles)

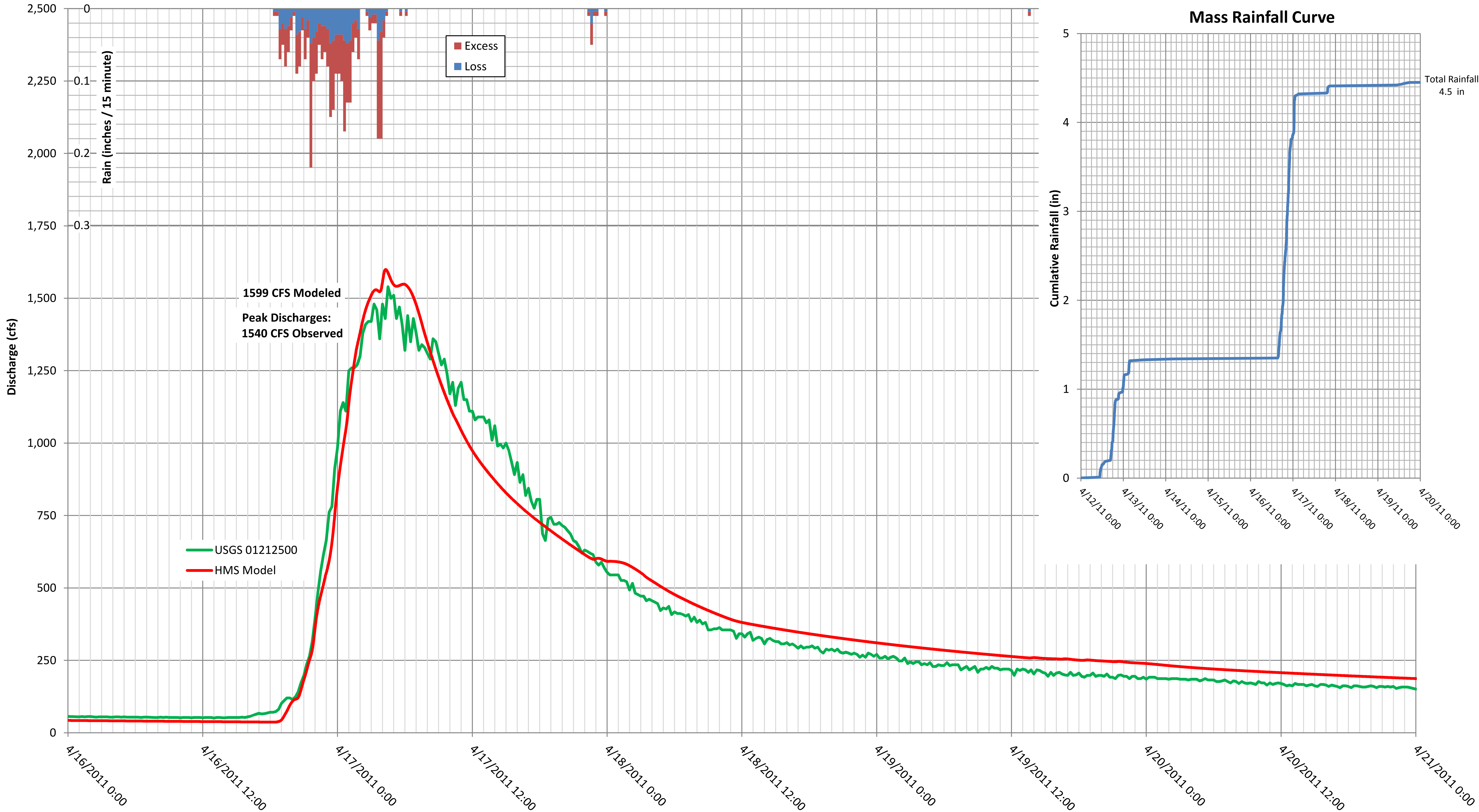


Figure 18

April 2011 Calibration Plot

Byram River at USGS 01212500 (25.6 sq. miles)

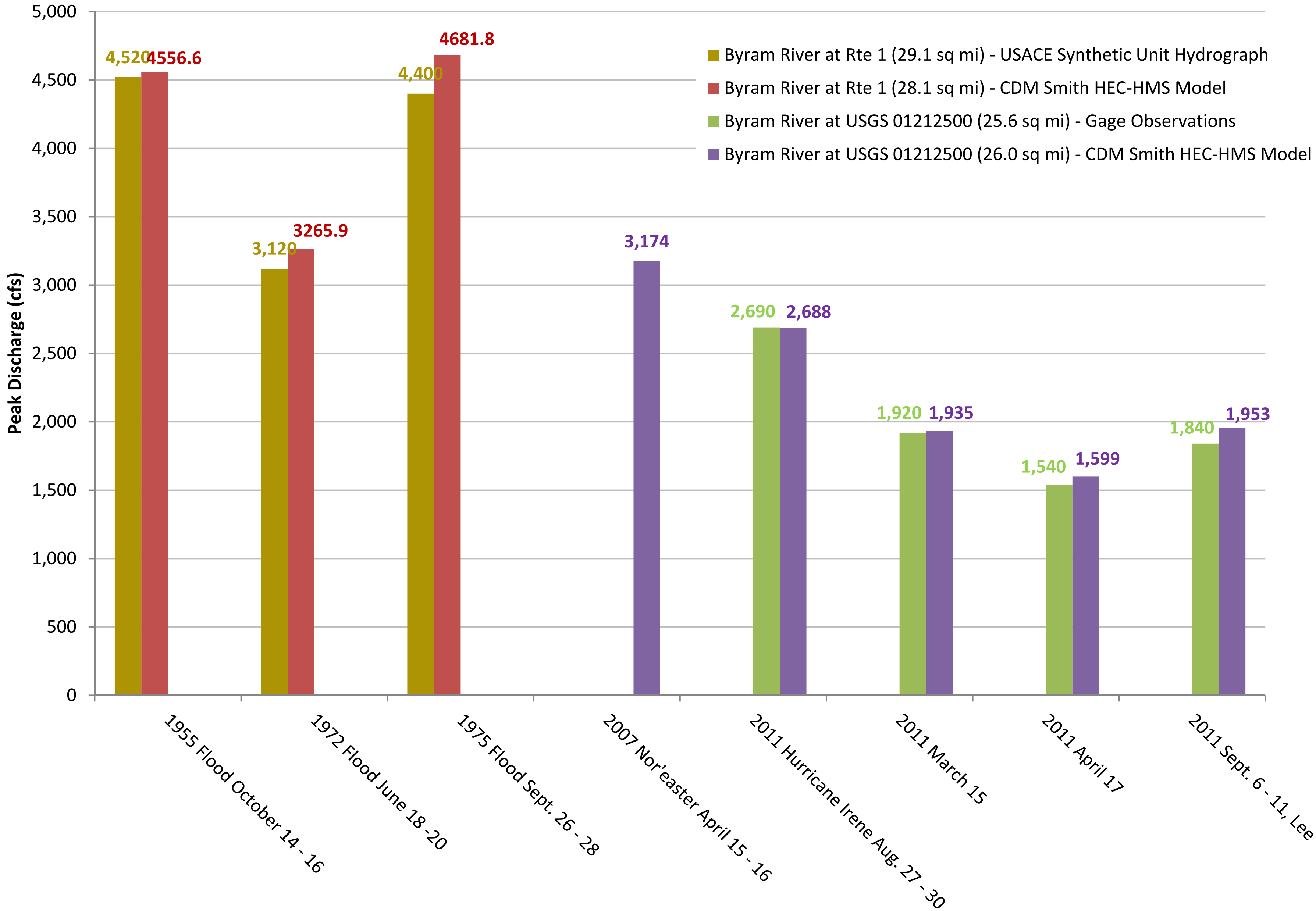


Figure 19
Comparison of Peak Flows for Modeled Historic Events
to Observations at USGS Gage
and to USACE Derived Unit Hydrograph

7. PEAK DESIGN FLOWS

As described in **Section 4.3**, the initial loss and deficit parameters in the HMS model were adjusted to match the peak flows from the partial duration discharge frequency curve described in **Table 5**, when simulating the design storms discussed in **Section 2.2**. The simulated hydrographs for the NRCC design storms are shown in **Figure 20**.

The basis of the peak design flows for flood profile modeling (described in **Appendix B2**) is the regime of peak discharges associated with HEC-HMS runoff model simulations described in **Section 5.2**, using the rainfall design depths published by NRCC, described in **Section 2.2**. The runoff model output provides peak discharge at every node of the network shown in **Figure 6**. For each design interval there is a peak discharge for a portion of the main channel of the Byram River. **Table 15** shows the flow regime for the 100, 50, 20, 10, 4, 2, 1, 0.5, and 0.2 percent design storms at each flow change location in the HEC-RAS model. The design flows are shown for both the Existing Conditions and Future Without Project described in **Section 5.7**. **Figure 21** shows the peak discharge plotted against contributing drainage area for the Existing Conditions flow regimes.

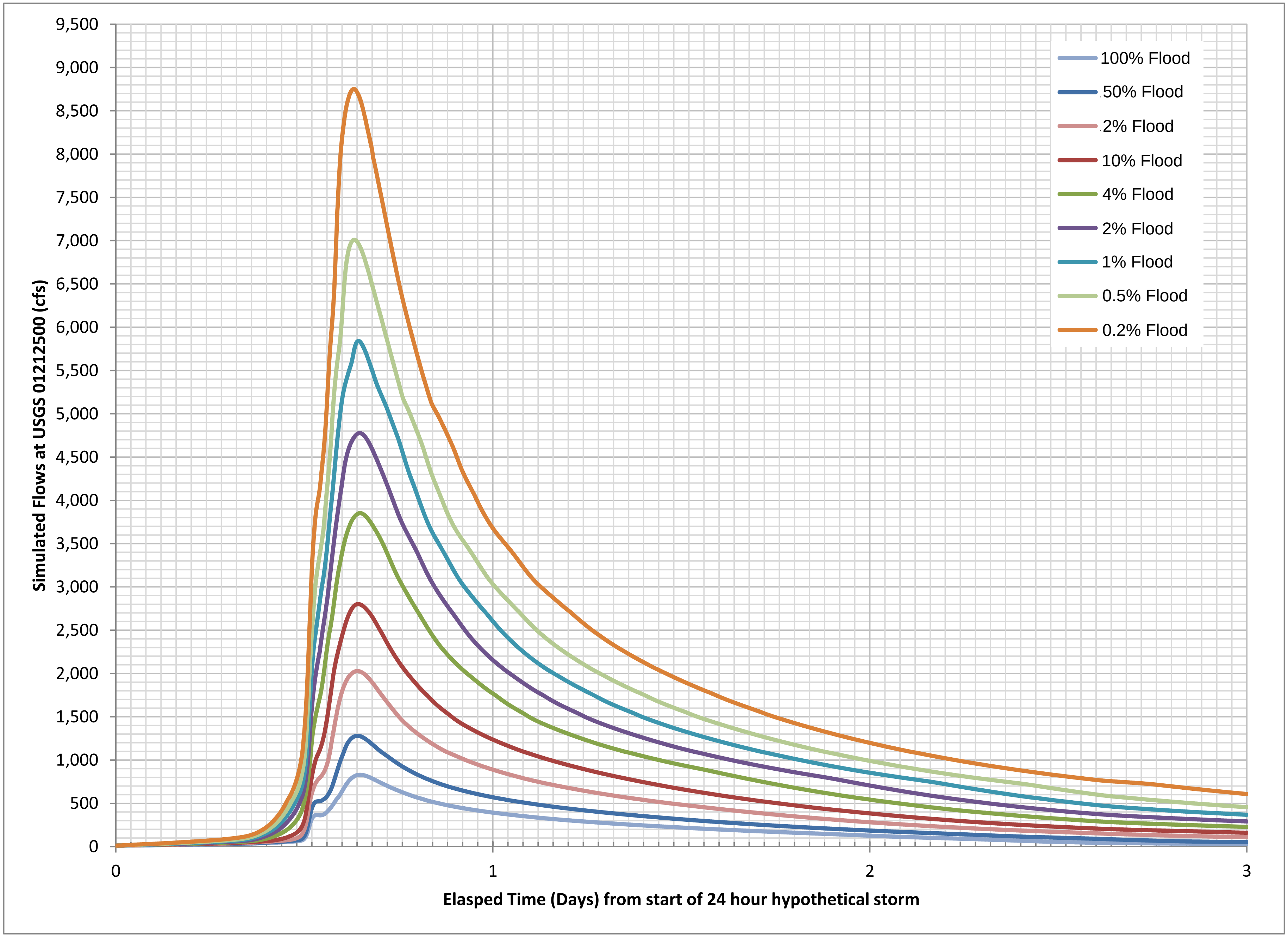


Figure 20

Simulated Flows for Byram River at USGS 01212500 (26.0 sq. miles)

Drainage Area (sq mi)	HEC-RAS River Stations (ft)		Downstream Location	HMS Object	Design Storm Steady Flows - Existing Condition (cfs)								
	US	DS			100%	50%	20%	10%	4%	2%	1%	0.5%	0.2%
8.5	52,659.2	47,122.8		DP_01	406	604	948	1,288	1,770	2,175	2,630	3,122	3,879
9.2	47,122.8	42,121.6		DP_02	158	214	315	425	557	644	740	844	1,002
10.7	42,121.6	39,898.8	Wooley Pond - Inflow	DP_03 (Res01) Inflow	236	369	585	784	1,060	1,286	1,543	1,819	2,238
12.3	32,019.8	29,709.2	Merrit Pkwy	DP_04 (Res02) Outflow	375	591	967	1,324	1,805	2,206	2,654	3,139	3,854
24.6	29,709.2	26,652.0	Confluence with East Branch	DP_05A	774	1,196	1,930	2,673	3,675	4,517	5,445	6,430	8,017
24.6	26,209.8	22,258.2	Toll Gate Dam	(Res03) Outflow	760	1,175	1,880	2,598	3,565	4,429	5,360	6,355	7,928
25.3	22,258.2	19,783.8	American Felt Co. Dam	DP_05B (Res04) Inflow	791	1,222	1,941	2,685	3,692	4,571	5,585	6,664	8,317
26.0	19,783.8	16,242.1	Pemberwick Dam	DP_06 (Res05) Inflow	834	1,284	2,030	2,804	3,854	4,781	5,852	7,015	8,759
26.0	16,242.1	14,991.7	USGS 01212500	DP_07 USGS	830	1,280	2,028	2,801	3,852	4,777	5,841	7,010	8,752
27.4	14,991.7	13,544.3		DP_08	891	1,383	2,199	3,049	4,208	5,232	6,397	7,747	9,681
28.1	13,544.3	9,526.8		(Res06) Inflow	926	1,433	2,274	3,149	4,353	5,415	6,607	8,036	10,053
28.1	9,526.8	6,805.3	Rte 1 Putnam Ave	DP_09	894	1,391	2,244	3,065	4,218	5,274	6,387	7,728	9,792
28.4	6,805.3	3,928.3	Rail Road Bridge	DP_10	904	1,406	2,266	3,090	4,250	5,316	6,437	7,785	9,877
30.0	3,928.3	321.6	I-95 Overpass	DP_11	1,021	1,573	2,524	3,437	4,676	5,852	7,072	8,569	10,913

Drainage Area (sq mi)	HEC-RAS River Stations (ft)		Location	HMS Object	Design Storm Steady Flows - Future with No Improvements (cfs)								
	US	DS			100%	50%	20%	10%	4%	2%	1%	0.5%	0.2%
8.5	52,659.2	47,122.8		DP_01	406	604	949	1,288	1,770	2,175	2,630	3,122	3,879
9.2	47,122.8	42,121.6		DP_02	159	215	315	425	557	644	740	844	1,002
10.7	42,121.6	39,898.8	Wooley Pond - Inflow	DP_03 (Res01) Inflow	239	371	588	785	1,061	1,288	1,544	1,821	2,240
12.3	32,019.8	29,709.2	Merrit Pkwy	DP_04 (Res02) Outflow	383	599	974	1,329	1,809	2,210	2,659	3,144	3,861
24.6	29,709.2	26,652.0	Confluence with East Branch	DP_05A	799	1,221	1,956	2,694	3,694	4,537	5,469	6,458	8,053
24.6	26,209.8	22,258.2	Toll Gate Dam	(Res03) Outflow	785	1,200	1,905	2,618	3,588	4,450	5,384	6,384	7,965
25.3	22,258.2	19,783.8	American Felt Co. Dam	DP_05B (Res04) Inflow	819	1,250	1,968	2,708	3,718	4,595	5,615	6,696	8,358
26.0	19,783.8	16,242.1	Pemberwick Dam	DP_06 (Res05) Inflow	864	1,313	2,059	2,828	3,882	4,806	5,886	7,050	8,803
26.0	16,242.1	14,991.7	USGS 01212500	DP_07 USGS	859	1,309	2,056	2,825	3,879	4,802	5,875	7,045	8,796
27.4	14,991.7	13,544.3		DP_08	930	1,422	2,237	3,082	4,240	5,265	6,443	7,794	9,738
28.1	13,544.3	9,526.8		(Res06) Inflow	965	1,472	2,312	3,182	4,383	5,447	6,654	8,083	10,111
28.1	9,526.8	6,805.3	Rte 1 Putnam Ave	DP_09	932	1,431	2,283	3,089	4,250	5,306	6,428	7,780	9,851
28.4	6,805.3	3,928.3	Rail Road Bridge	DP_10	943	1,446	2,304	3,113	4,282	5,347	6,477	7,838	9,935
30.0	3,928.3	321.6	I-95 Overpass	DP_11	1,060	1,613	2,563	3,453	4,707	5,882	7,113	8,623	10,972

Table 15 - Design Flood Flow Regime for HEC-RAS Modeling

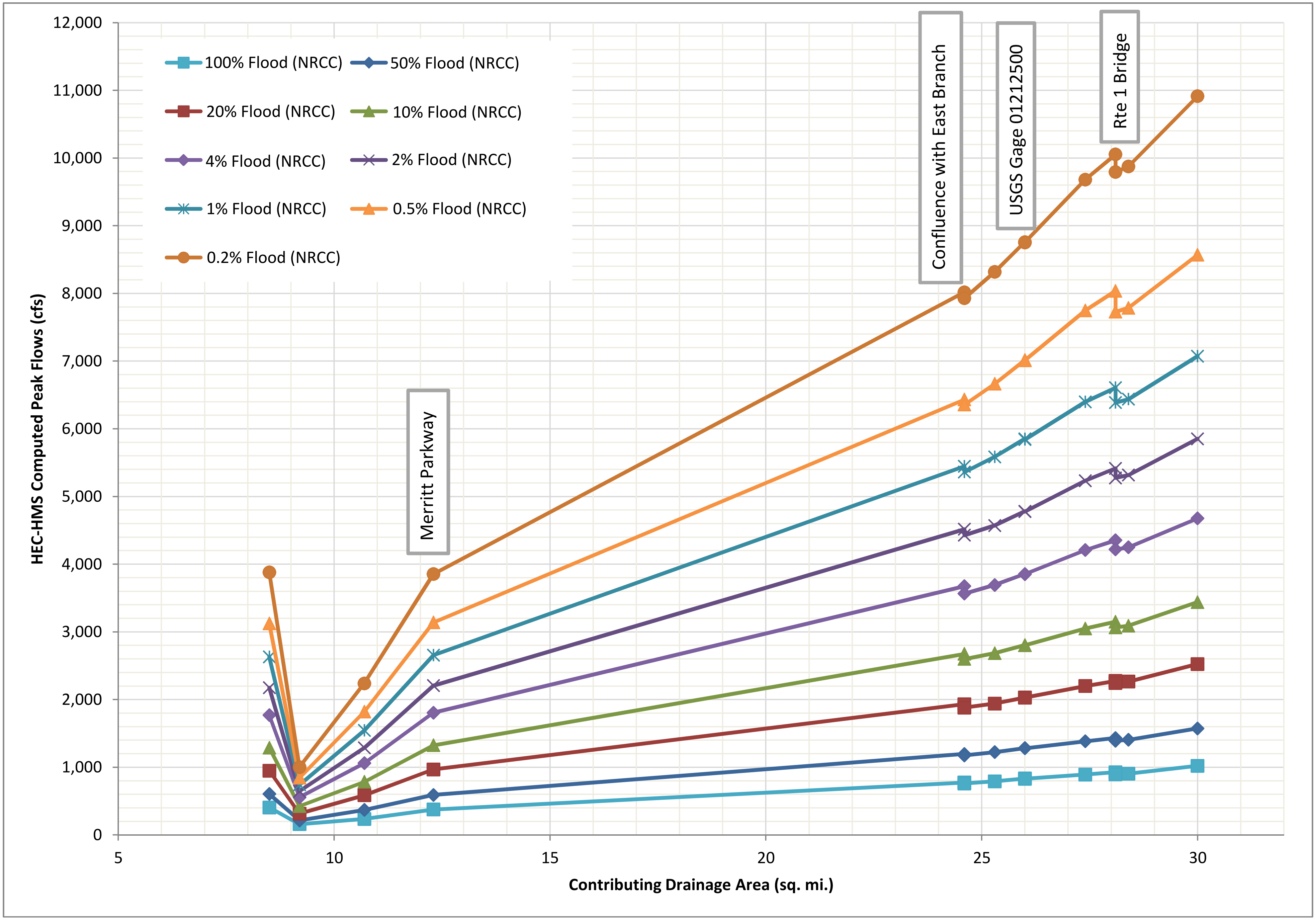


Figure 21
Modeled Peak Discharge by Contributing Drainage Area
Existing Conditions

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