Appendix C

Description of Dam and Typical Operations: Souhegan River Basin

Figures

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C-1.0 INTRODUCTION

C-1.1 Overview

This report describes the methodology, available data, runoff characteristics for the May 2006 and April 2007 runoff events, and reservoir operations in several lakes and reservoirs within the Souhegan River Basin, located in southern New Hampshire. The headwaters of the Souhegan begin in northern Massachusetts and run northeast through the New Hampshire towns of New Ipswich, Greenville, Wilton, Milford, Amherst and Merrimack where it confluences with the Merrimack River. Several of these towns reported heavy flooding in these two events, particularly the towns of Greenville and Wilton during the April 2007 event. Figure C-1 shows this 221 mi^2 watershed and some of the critical data used for this report. For purposes of comparison, this report focuses on the 171 mi^2 upstream of USGS Gage Number 01094000.

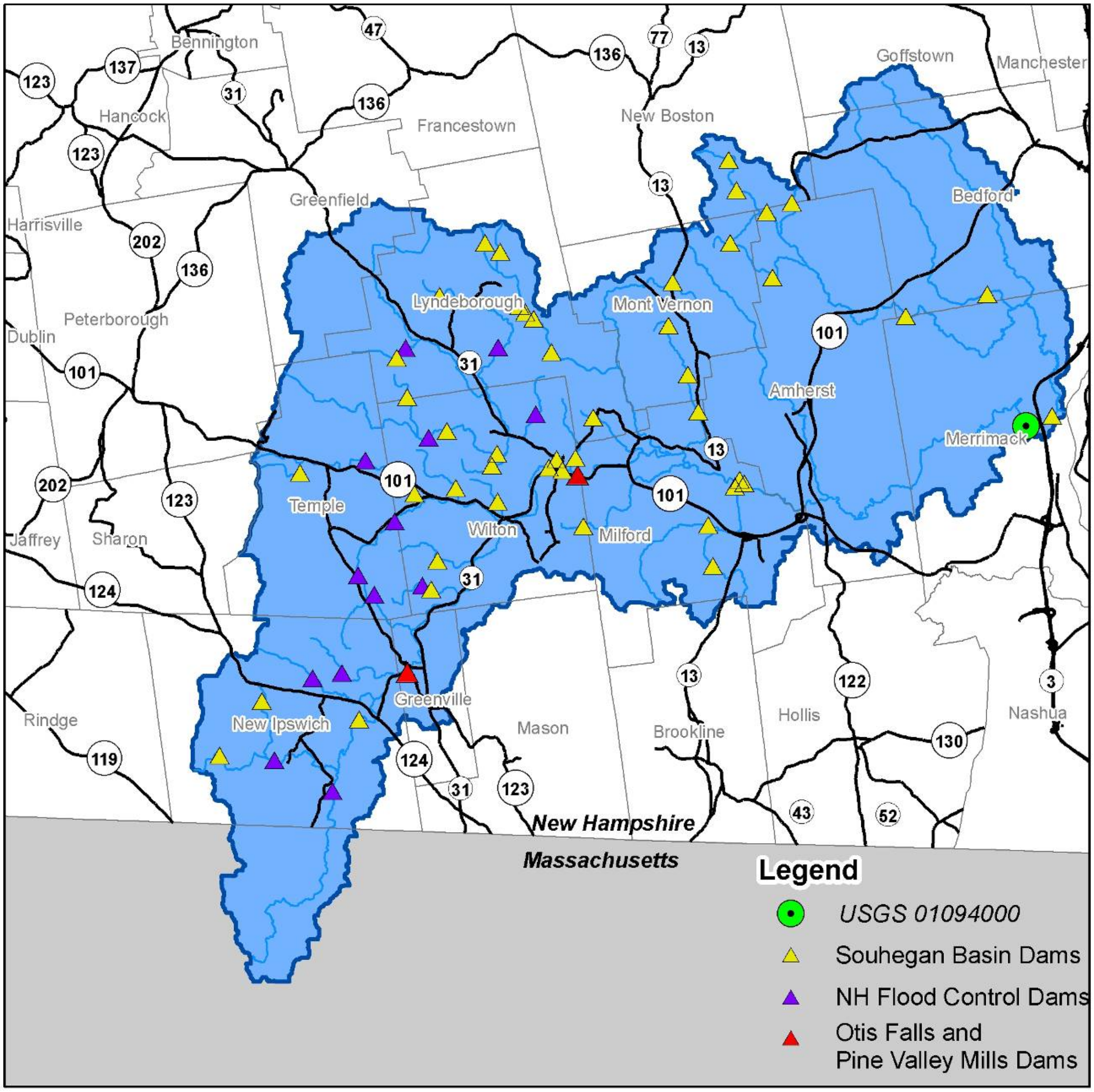

Figure C-1: Souhegan River Basin

Some previous hydrology studies have been conducted of the Souhegan Basin but few have focused on reservoir operations. Zhang (1995) prepared a physically-based distributed rainfall-runoff model with radar data. This study focused more on modeling techniques and procedures rather than flooding implications or reservoir operations for the Souhegan Basin.

FEMA (1979a, 1979b, 1979c, 1979d, 1980, 1991, 1994) studied the Souhegan Basin for the purpose of developing Flood Insurance Rate Maps. For the uncontrolled portions of the watershed, mostly approximate methods and regional regression equations were used. For the controlled portions of the watershed, discharges were obtained from data supplied by the SCS (now the NRCS) (convex routing method) or USACE. No comprehensive modeling effort involving all of the flood control reservoirs and run-of-the-river dams is apparent in any of these studies.

C-1.2 Acronyms

FEMA: Federal Emergency Management Agency

NHDES: New Hampshire Department of Environmental Services

NRCS: Natural Resources Conservation Service (formerly Soil Conservation Service or SCS)

NWS: National Weather Service

USACE: U.S. Army Corps of Engineers

USGS: U.S. Geological Survey

af: acre-feet

cfs: cubic feet per second

C-1.3 Terminology

Channel Capacity: Maximum flow through a river or manmade channel without overtopping.

Curve Number: Number that describes runoff potential of a given drainage area with a given combination of land use and soil type.

Downstream Flooding: Flooding occurring downstream of a dam site. Releases from the dam in certain cases can contribute to downstream flooding.

Flashboards: Bulkheads placed on the crest or top of a channel wall or control structure. Flashboards are sometimes designed to break and wash away under high flow conditions ("to operate") and thus to provide only a temporary diversion. In contrast, stoplogs are intended to be reused.

Flood Control Dams: Large dams constructed for the purpose of attenuating peak discharges and to reduce the effects of flooding.

HEC-HMS: hydrologic computer model developed by the U.S. Army Corps of Engineers used to calculate the flow from a given river basin.

HEC-RAS: hydraulic computer model developed by the U.S. Army Corps of Engineers used to determine the velocity, depth, and flooding effects for flows from a given river basin.

HydroCAD: Computer model used to analyze stormwater and reservoir facilities.

Lag Time (Time of Concentration): Time between the centroid of the precipitation pattern to the peak of the hydrograph. Estimated to be about 0.6 times the time of concentration.

Mean Areal Temperature: Assumed mean temperature over an area, typically a river sub-basin. It is typically estimated from observation at climate sites in the area.

Mean Areal Precipitation: Assumed mean precipitation over an area, typically a river sub-basin. It is typically estimated from observation at climate sites in the area.

Normal Pool Elevation: Typical water elevation of a lake or reservoir. This value might change seasonally.

Precipitation: Rainfall or snowfall onto an area, typical expressed as depth of water over an area.

Recurrence Interval: Time interval in which an event can be expected to occur once on the average.

Rainfall-Runoff Model: Computer model that simulates the effects of rainfall (or snowmelt) onto an area and estimates the resulting runoff into a river or lake.

Snow-water Equivalent: Amount of water contained within the snowpack. It can be thought of as the depth of water that would theoretically result if you melted the entire snowpack instantaneously.

Spillway: A structure used to provide for the controlled release of flood flows from a dam into the dammed river. Spillways release floods so water does not overtop and damage or even destroy the dam.

Stoplogs: A hydraulic engineering control element used in floodgates to adjust the water level and/or flow rate in a river, canal, or reservoir. Stoplogs are typically long rectangular timber beams or boards that are placed on top of each other and dropped into premade slots inside a dam weir (the “stoplog bay”). Placing more stoplogs in a stoplog bay increases the pool elevation of the lake or reservoir and decreases the releases.

Time of Concentration (Lag Time): Time between the centroid of the precipitation pattern to the inflection point of the receding limb of the hydrograph.

Run-of-the-River Dams: small dams used for hydropower, recreation, or water quality that have only a small quantity of storage capacity.

Storage Capacity: Volume of water a lake or reservoir holds at a certain elevation.

Sub-basin: Area draining into a lake or river above a certain point.

Upstream Flooding: Flooding occurring upstream of a dam site due to high reservoir or lake pool elevation.

Winter Drawdown: Difference between the summer normal pool elevation and the winter normal pool elevation.

WISE: GIS-based software program that helps develop and utilize hydrologic and hydraulic data.

C-2.0 METHODOLOGY

The following methodology was used to analyze the Souhegan River Basin. HEC-HMS (Hydrologic Modeling System) developed by U.S. Army Corps of Engineers (USACE 2002) was the model used for the hydrologic analysis.

HEC-HMS has three components: the **basin model**, where drainage area, the unit hydrograph, runoff volume, and reservoir characteristics for each sub-basin are defined; the **meteorologic model**, where the rainfall events are defined; and the **control specifications**, where the time of period being simulated to model the rainfall events is defined. A schematic showing the HEC-HMS representation of the Souhegan River Basin is shown in Figure C-2.

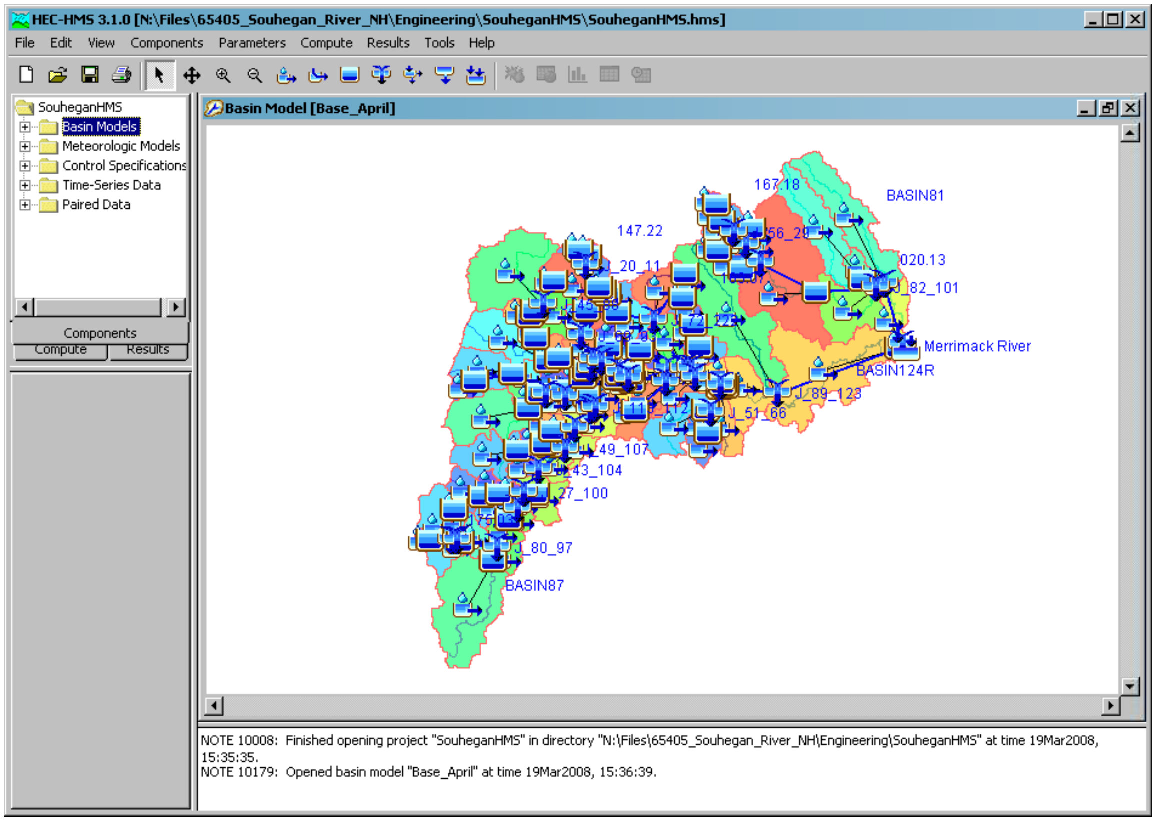

Figure C-2: HEC-HMS Model for the Souhegan River Basin

C-2.1 Basin Model

The Souhegan drainage area was subdivided into 120 subbasins using the automatic drainage area delineation routine in WISE, a GIS-based hydrology and hydraulics software package, and a 10-meter USGS Digital Elevation Model (DEM). The drainage values generally agree with values derived by NHDES in their dam inspection reports. Subbasins were defined at every junction with a major tributary and at every impoundment structure included in this analysis.

A number of unit hydrograph methods are available within HEC-HMS. The unit hydrograph method selected for this analysis was the Snyder's method which is dependent on calibration coefficients. These calibration coefficients were modified so that the runoff for a particular rainfall event produces a hydrograph that is similar to an observed hydrograph for the same event. Refer to Attachment A for a detailed description of the calibration procedure.

For runoff volume computations, the initial and constant-rate method was used. This method uses an initial rainfall abstraction that accounts for interception and depression storage and then an estimate for the ultimate infiltration capacity of the soils. In May 2006, the antecedent conditions were moderately wet with some minor infiltration occurring during the storm event. In April 2007, the antecedent runoff conditions involved heavy rainfall and a substantial amount of snow cover, so very little infiltration occurred during the event.

Base flow, although only a small part of the runoff in events as large as the April 2007 and May 2006 storms, was included using the exponential recession method.

Since the operations of both run-of-river dam and flood control dams have generated a substantial amount of public concern, the HEC-HMS model included every reservoir for which data was available. This resulted in the inclusion of 59 dams in the model. The reservoir characteristics included in HEC-HMS are elevation-storage and elevation discharge curves. These curves are derived using data provided by NHDES and had varying degrees of quality as discussed in the following section.

River routing reaches were also included where one or several sub-basin drains through another sub-basin. The Modified-Puls method was selected for reach routing. The storage-discharge values necessary in applying the Modified-Puls methods were derived using WISE.

C-2.2 Meteorologic Model

The meteorologic model defines the rainstorm distribution type and intensity. As shown on Figure C-3 and Table C-1, four gages provided precipitation data: Everett Dam on the Piscataquog River (WERN3), Nashua River at East Pepperell, Massachusetts (DNSM3), Birch Hill Dam (RYLM3), and the Souhegan River at the Merrimack River (SOHN3). The rainfall for the May 2006 and April 2007 were estimated based on a Thiessen polygon weighting of the rainfall gages within or near the Souhegan River Basin, assuming a single pattern of rainfall for the entire basin.

DNSM3 was excluded from the Thiessen polygon weighting since the rainfall totals for this gage were much smaller and initial model simulations indicated that precipitation in the Souhegan would be underestimated if this gage was used.

C-2.3 Control Specifications

The control specifications, the time periods being simulated, for this project were defined for the dates of May 10-31, 2006, and April 13-30, 2007.

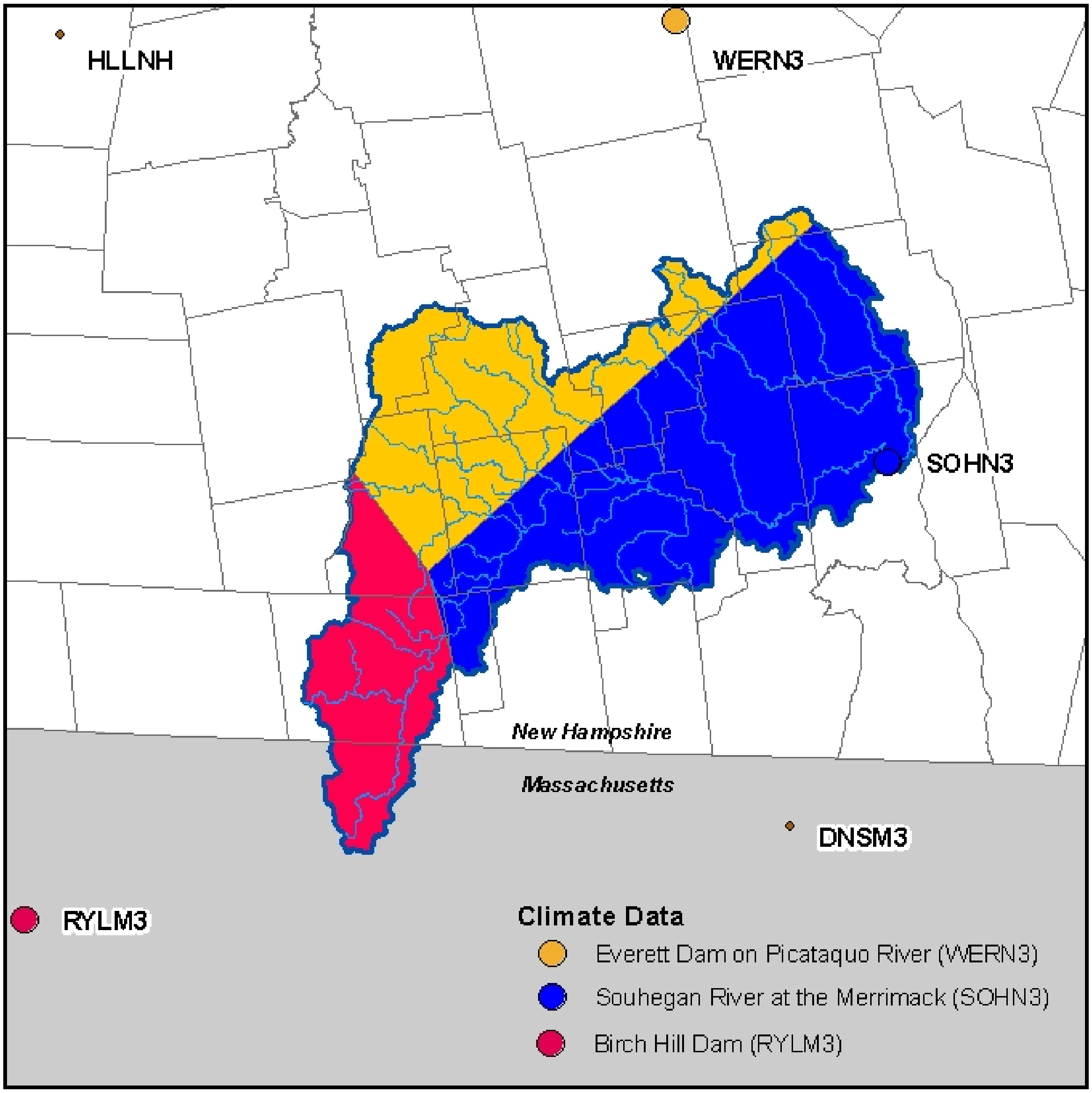

Figure C-3: Thiessen Polygons and Associated Precipitation Gages Basin

Table C-1: Precipitation Data Used for the Souhegan River Basin

Description	Precipitation Gage	May 10-31, 2006 Precipitation (in)	May 10-31, 2006 Max Intensity (in/hr)	April 13-30, 2007 Precipitation (in)	April 13-30, 2007 Max Intensity (in/hr)
Everett Dam on the Piscataquog	WERN3	10.77	0.28	7.07	0.40
Nashua River at East Pepperell	DNSM3	3.72	0.45	4.09	0.20
Birch Hill Dam	RYLM3	5.29	0.39	4.06	0.18
Souhegan River at the Merrimack River	SOHN3	5.43	0.26	4.62	0.31
Thiessen Polygon Weighted Average		6.92	0.29	5.22	0.31

C-2.4 Calibration Procedure

C-2.4.1 Snyder's Method for Unit Hydrograph

The lag time, t_p (in hours), or approximately the time between the rainfall and the peak of the hydrograph, is defined as:

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

where C_t = basin coefficient; L = length of the main stream from the outlet to the divide; and L_c = length along the main stream from the outlet to a point nearest the watershed centroid. C_t is modified during calibration so that the timing of the simulated runoff peak. The peak discharge of the unit hydrograph (in cfs) is determined by the following function:

$$Q_p=C_pA/t_p$$

where C_p = peaking coefficient; A = drainage area in square miles, and t_p is as previously defined. The unit hydrograph is then convoluted with an historical rainfall event to produce an event hydrograph such as the April 2007 or May 2006 rainfall-runoff event.

Since there was only one runoff gage and limited precipitation data, C_t and C_p are assumed to have the same value throughout the Souhegan watershed for both the May 2006 and April 2007 storm events. For the Souhegan Basin, C_p was found to be 3.2. Typically this ranges between 1.8 and 2.2, with values found to range between 0.4 in mountainous regions and 8.0 in extremely flat areas. C_t was found to be 0.8. Typically this ranges between 0.4 and 0.8 (USACE 2000).

C-2.4.2 Initial and Constant-Rate Loss Method

The initial and constant-rate loss method was used in HEC-HMS to simulate runoff volume. This method assumes a maximum potential rate of precipitation loss, f_c, that is constant throughout an event. Therefore a precipitation value of p_t for a time interval of $t+\Delta t$, the excess runoff volume pe_t is given by:

$$pe_t = \begin{cases} p_t - f_c, & if\ p_t > f_c \\ 0, & otherwise \end{cases}$$

An initial loss, I_a, is also included in the model to represent interception and depression storage. In the May 2006 and April 2007 storms events, no initial loss was used in the final calibration. Table C-2 shows the loss rates that were used to calibrate the May 2006 and April 2007 storm. The soil types and areas were determined for each sub-basin using NRCS SURRGO soils data. Then a weighted loss rate was calculated for each sub-basin.

C-2.4.3 Base Flow Method

The base flow for the Souhegan River Basin was estimated using the exponential recession model where

$$Q_t = Q_0 k^t \text{ with}$$

Q_t = the baseflow at anytime t in cfs,

Q_0 = initial value for baseflow in cfs/mi^2,

k = expotential decay constant, and

t = unit time.

The values used for this study are included in Table C-3. The same values are used for all 120 subbasins.

Table C-2: Precipitation Losses Used in Souhegan River Basin HEC-HMS Model

Hydrologic Soil Group	Description	Typical Range of Loss Rates (in/hr)	Loss Rates for May 2006 Storm	Loss Rates for April 2007 Storm
A	Deep sand, deep loss, aggregated silts	0.30-0.45	0.075	0.00
B	Shallow loess, sandy loam	0.15-0.30	0.038	0.00
C	Clay loams, shallow sandy loam, soils in organic content, and soils usually high in clay	0.05-0.015	0.013	0.00
D	Soils that swell significantly when wet, heavy plastic clays and certain saline soils	0.00-0.05	0.000	0.00

Table C-3: Base Flow Values Used in Souhegan River Basin HEC-HMS Model

Storm Event	Initial Discharge (cfs/mi^2)	Recession Constant	Ratio to Peak
May 2006	1	0.9	0.1
April 2007	5	0.9	0.1

Additional analysis was conducted using the NRCS Soil Complex Method as described in NRCS *National Engineering Handbook-4*, although it was determined that Snyder's method provided a better estimate of the storm hydrograph since the NRCS method could not correctly approximate the volume under the hydrograph. Using detailed land use files and NRCS SURRGO Soils data, the overall basin curve number was found to be 64.

C-3.0 AVAILABLE DATA

Data for this analysis was taken from USGS, NWS, NHDES, and USACE.

C-3.1 Climate Data

The climate data available in this study was primarily precipitation from NHDES, continuous discharge data from the USGS, and temperature and snow water equivalent data from NWS. The temperature and snow water equivalent data were used to examine the general climate trends and antecedent conditions for the two storms, but not explicitly included in the modeling effort. These data are typically recorded every hour at climate sites in the region, and provide a reasonable representation of the weather development during the May 2006 and April 2007 flood events.

Figures C-4 and C-5 show the Thiessen polygon weighted precipitation and temperature data from the station located along the Souhegan River at the Merrimack River (SOHN3). Figure C-5 also shows the snow-water equivalent data for the April storm.

For the May 2006 storm, there is no snow-water equivalency since there was no snowpack. The observed peak for the May 2006 storm occurred on May 15 at 10:00 a.m. It rained 4.7 inches in the 48 hours prior to the peak runoff rate arriving at USGS Gage 00190400. Since the mean areal temperature was above freezing several days prior to the storm event, there was no measurable snowmelt contribution to the May 2006 storm. Rainfall characteristics for the May storm are shown in Table C-4.

Table C-4: Rainfall Characteristics of May 2006 Storm

Date of Runoff Peak of 6,150 cfs: 5/15/2006 @ 10:00 a.m.			
Precipitation Value	Rainfall Total (in)	Approximate Recurrence interval	Dates of Occurrence
Pre 48-hour	4.7	--	5/11/2006 4:00 p.m. to 5/13/2006 4:00 p.m.
Peak 1-hour	0.2	<1 year (0.9 inch)	5/13/2006 4:00 p.m.
Peak 6-hour	0.8	< 1 year (1.5 inches)	5/13/2006 3:00 p.m. to 5/13/2006 9:00 p.m.
Peak 12-hour	1.4	< 1 year (2.3 inches)	5/14/2006 3:00 p.m. to 5/15/2006 3:00 a.m.
Peak 24-hour	2.5	< 2 year (2.9)	5/13/2006 8:00 a.m. to 5/14/2006 8:00 a.m.
Peak 48-hour	4.8	~5 year to 10 year	5/13/2006 7:00 a.m. to 5/15/2006 7:00 a.m.
Peak 120-hour	5.9	~10 year	5/12/2006 5:00 p.m. to 5/16/2006 5:00 p.m.

For the April 2007 storm, the snow water equivalent peaked at around 1.36 inches at 96 hours prior to the storm event. By the time the peak flow arrived at USGS Gage 00109400 on April 17 at 3:00 a.m., the snow water equivalent had been reduced to 0.20 inches. This is the equivalent of another 1.16 inches of rain falling during this time period. The mean areal temperature during this time period stayed above freezing, so the high temperatures contributed to significant runoff volume around the same period of heavy rainfall. Table C-5 summarizes the rainfall characteristics for the April 2007 storm. The peak 24-hour rainfall falls between the 2- and 5-year recurrence interval, but these values do not account for the snowmelt contribution.

Table C-5: Rainfall Characteristics of April 2007 Storm

Date of Runoff Peak of 10,550 cfs: 4/17/2007 @ 3:00 a.m.			
Precipitation Value	**Rainfall Total (in)**	**Approximate Recurrence interval**	**Dates of Occurrence**
Pre 48-hour	4.0	--	4/14/2007 7:00 a.m. to 4/16/2007 7:00 a.m.
Peak 1-hour	0.3	< 1 year (0.9 inch)	4/16/2007 7:00 a.m.
Peak 6-hour	1.1	< 1 year (1.5 inches)	4/16/2007 6:00 a.m. to 4/16/2007 12:00 p.m.
Peak 12-hour	2.1	~ 2 year (2.5 inches)	4/16/2007 2:00 a.m. to 4/16/2007 2:00 p.m.
Peak 24-hour	3.3	~2 year (2.9 inches) to 5 year (3.8 inches)	4/15/2007 3:00 p.m. to 4/16/2007 3:00 p.m.
Peak 48-hour	4.1	~5 year	4/15/2007 6:00 a.m. to 4/17/2007 6:00 a.m.
Peak 120-hour	4.5	~2 year to 5 year	4/13/2007 0:00 a.m. to 5/18/2007 0:00 a.m.

The May 2006 storm was caused by a large quantity of rainfall over a long period of time. The longer the duration of rainfall, the more severe the event as is approximated for the 120 hour rainfall. In contrast, the April 2007 storm involved almost as large a quantity of rainfall in a shorter period of time combined with heavy snow melt. As discussed in the following sections, the consequence of these differences was dramatic in some locations throughout the Souhegan Basin. The flooding associated with both of these storms was greater than would be expected if other conditions in the basin had been more normal. However, the high rate of runoff in May was attributable to nearly saturated soil conditions coupled with seasonally high baseflow; while the even higher runoff in April was attributable to very intense rainfall coincident with rapid snowmelt on ground that had now yet thawed with even higher seasonal baseflow.

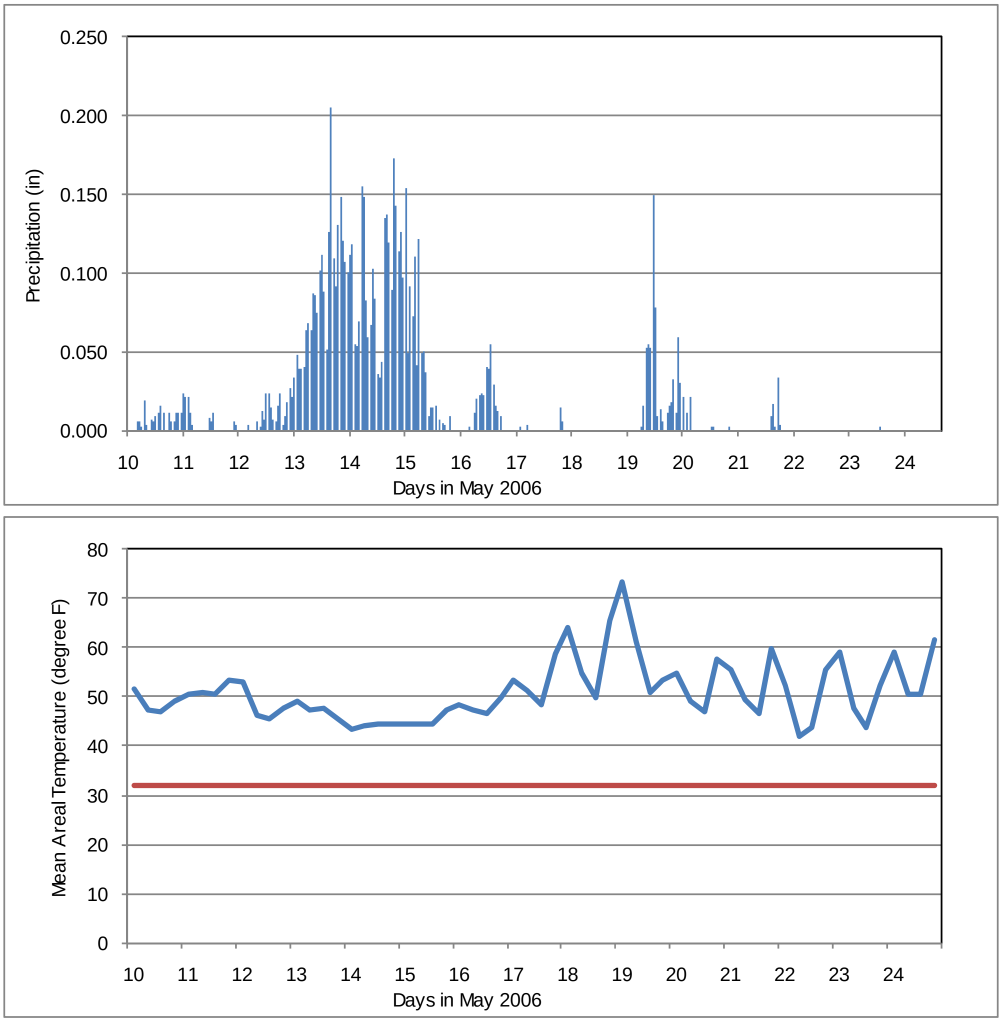

Figure C-4: Precipitation and Temperature during the May 2006 Storm Event

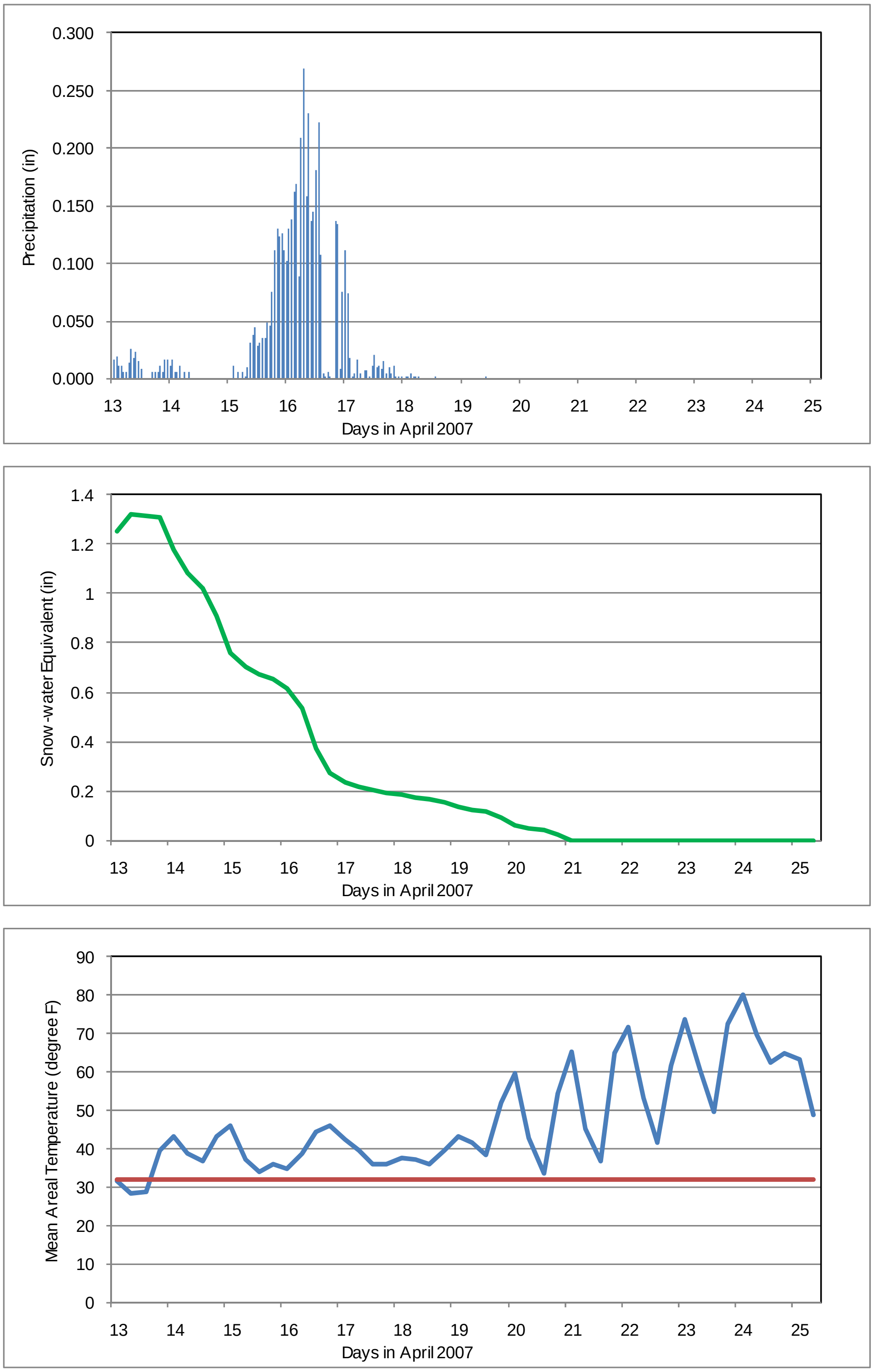

Figure C-5: Precipitation, Snow-Water Equivalent, and Temperature During the April 2007 Storm Event

Observations of streamflow data were obtained primarily from the USGS and USACE. Although five USGS gaging stations exist within the Souhegan Basin, only one of these (USGS Gage 00109400 Souhegan River at Merrimack) has hourly flow records and a sufficient length of record to be included in this study. This gage is located above the Merrimack Village Dam (NHDES# 156.01) and the confluence with Baboosic Brook. All comparisons in this study that examine overall basin results use the inflows to Merrimack Village Dam as a point of analysis. This location is very similar in drainage area with USGS Gage 00109400 (~171 sq mi for both).

The hydrographs of the May 2006 and April 2007 storm events are shown in Figure C-6. The April 2007 event corresponds roughly with the 50 year runoff event and the May 2006 event corresponds roughly with the 10-year runoff event. The timing and magnitude of these hydrographs is included in the HEC-HMS model.

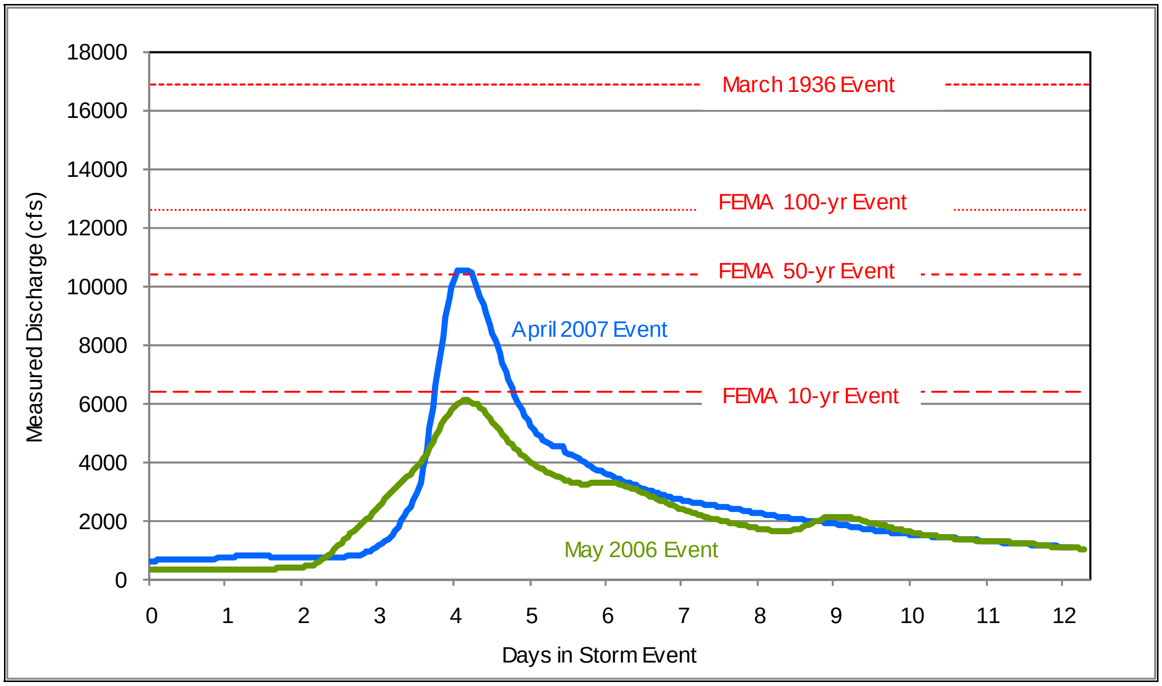

Figure C-6: Comparison of Measured Discharge for May 2006 and April 2007 with FEMA Storm Events

The recurrence interval of rainfall and runoff events is rarely coincident. A rainfall storm of relatively low intensity and low recurrence interval (e.g., 1- or 2-year event) can result in a more significant runoff event with a greater recurrence interval (e.g., 5- or 10-year event) if ground conditions exacerbate the effect of the rainfall on the watershed. However, the disparity in the May 2006 (<1-year, 24-hour rainfall with 10-year runoff) and April 2007 (2- to 5-year rainfall event with a 50-year runoff) storms may indicate the Souhegan Basin (and other similar basins in New Hampshire) is particularly vulnerable to heavy rainfall during the snowmelt season. The rainfall that occurred during both events was heavy but not "historic," yet they caused extensive flood damage. Had the rainfall been more intense, then flooding would likely have been even more widespread.

C-3.2 Reservoir Data

The overall approach to modeling the reservoirs in the Souhegan Basin was to include every dam with available data: both run of the river and flood control dams. Approximately 80 percent of the reservoir storage is accounted for in 12 New Hampshire flood control sites. The necessary data to conduct a reservoir study include: (1) physical data such as storage capacity and dam release capacity and (2) operations data such as operating records and operating rules.

There are 239 dams listed in the NHDES dam database. Only 142 of these are active since many of the dams listed in the database are in ruins, have been breached, have not been built, or have been removed. This study includes 59 of the active dams that are substantial enough in size to require NHDES inspection reports. This includes all dams classified as high or significant hazards and the majority of dams that are classified as low hazard.

NHDES performs varying level of analysis on dams depending on their respective hazard classification and ownership. HydroCAD, a reservoir analysis software package, is used by NHDES engineers to perform analysis on reservoirs for the level of design storm for a particular dam (e.g., 50-year, 100-year, etc.). The data input to the HydroCAD models require the same elevation-storage and elevation-discharge relationships as HEC-HMS, so this data was used wherever available. If no HydroCAD models were available, elevation-storage and elevation-discharge relationships were developed from either NHDES Data Sheets or NHDES Inspection Reports. Table C-6 summarizes the physical data availability (i.e., non-operation related data) for all of the dams included in this study. It is important to note that 96 percent of the reservoir storage in the Souhegan Basin is accounted for with at least a reasonable quality of physical data. Table C-7 provides physical characteristics of the dams as derived from NHDES HydroCAD models; Table C-8 provides characteristics derived from NH Dam Data Sheets, and Table C-9 provides characteristics derived from NH Dam Inspection Reports.

Table C-6: Physical Data Available for Souhegan River Basin Dams

Data Source	Number of Dams	Percentage of Total Basin Storage	Relative Quality of Data	Comments
NHDES HydroCAD Models	37	49	Good	Highly detailed: elevation-storage and elevation-discharge relationship used from HydroCAD models
New Hampshire Dam Data Sheets	7	47	Adequate	Fairly detailed: elevation-storage and elevation-discharge relationship created from available information
New Hampshire Dam Inspection Reports	15	4	Judgment required for estimates	Some information available to estimate elevation-storage and elevation-discharge relationships

Since the majority of dams within the Souhegan River Basin are run-of-the river, few have operation flexibility such as stop logs or gates. Operation sheets are available for some of the dams operated by the state, but these involve little or no operator discretion during storm events aside from clearing debris and simply provide observations such as the presence of ice or water levels. Observations of lake elevations ("pool elevation") were not readily available for the Souhegan Basin aside from sporadic observations made on some of the New Hampshire flood control dams.

The one operation activity that generated public concern during the April 2007 storm was the removal and installation of flashboards, particularly on two run-of-the-river dams located in the mid and upper Souhegan Basin: Otis Falls Dam (NHDES# 101.01) and Pine Valley Mill Dam (NHDES# 254.01). Consequently, much of the simulation effort focuses on these two dams.

Table C-7: Souhegan River Basin Dams with NHDES HydroCAD Models

NHDES#	Dam Name	Height (ft)	Drainage Area[1] (mi^2)	Maximum Storage[2] (af)	Runoff to fill[3] (in)
7.01	JOE ENGLISH POND DAM	5.5	3.13	101	0.61
7.09	VIJVERHOF POND DAM	9.0	0.67	192	5.37
147.13	CURTIS BROOK DAM	10.0	2.23	3	0.02
147.14	PURGATORY BROOK	6.5	2.55	12	0.09
147.18	PURGATORY BROOK DAM	0.0	2.45	19	0.15
147.22	RECREATION POND	4.0	0.16	3	0.33
147.24	WILDLIFE POND	7.5	0.37	13	0.66
147.26	SOUHEGAN RIVER SITE 28 DAM	29.0	1.1	185	3.16
147.28	SOUHEGAN SITE 8 DAM	25.0	4.7	2721	10.86
147.29	MORISON POND	19.0	0.06	15	4.53
147.31	SWARTZ POND DAM	8.0	0.25	42	3.17
147.33	FARM POND	6.0	0.01	2	3.30
147.38	CURTIS BROOK DAM	12.0	3.5	1	0.01
159.01	RAILROAD POND DAM	12.0	10.58	48	0.09
159.04	OSGOOD POND DAM	9.0	5.24	270	0.97
159.05	HARTSHORN POND DAM	14.9	2.55	40	0.29
159.16	COMPRESSOR POND	24.0	2.25	76	0.64
163.02	CURTIS BROOK DAM	5.0	0.41	126	5.77
163.06	TROW DAM	0.0	1.27	1	0.01
163.07	HARTSHORN BROOK II DAM	8.0	0.22	28	2.39
163.12	ROBY POND DAM	3.5	0.34	3	0.17
167.18	BEAVER DAM POND DAM	5.0	0.58	210	6.79
167.29	GARDNER RESERVOIR DAM	8.0	1.16	17	0.27
175.01	SOUHEGAN SITE 14 DAM	35.0	2.1	885	7.90
175.03	PRATT POND DAM	6.5	0.74	110	2.79
175.19	SOUHEGAN RIVER SITE19 DAM	35.5	11.4	2072	3.41
175.20	SOUHEGAN RIVER SITE 13 DAM	13.5	0.8	249	5.84

NHDES#	Dam Name	Height (ft)	Drainage Area[1] (mi^2)	Maximum Storage[2] (af)	Runoff to fill[3] (in)
175.21	SOUHEGAN RIVER SITE 35 DAM	30.0	6.4	647	7.67
175.23	WHEELER POND DAM	5.0	0.25	23	1.73
254.09	NEW WILTON RESERVOIR DAM	24.0	0.4	335	15.70
254.19	PETERS FARM POND DAM	10.0	0.98	6	0.11
254.20	BATCHELDER POND DAM	12.0	1.2	20	0.31
254.21	FROG POND DAM	15.0	0.6	143	4.45
254.30	SOUHEGAN RIVER SITE 15 DAM	13.0	1.1	315	12.75
254.34	SOUHEGAN RIVER SITE 33 DAM	21.0	1	1078	20.21
254.38	RECREATION POND DAM	8.0	0.4	10	0.48
254.43	CAMP POND DAM	11.0	0.76	33	0.80

Notes for Tables C-6, C-7, C-8:

[1] Drainage approximated from WISE or dam inspection report, if available.
[2] Maximum storage in this table is extrapolated to estimated storage above dam to also account for overtopping storage.
[3] Runoff to fill is the ratio of Maximum storage to drainage area as defined in these tables.

Table C-8: Souhegan River Basin Dams with New Hampshire Dam Data Sheets

NHDES#	Dam Name	Height	Drainage Area (mi^2)	Maximum Storage (af)	Runoff to fill (in)
101.01	OTIS FALLS DAM	27.0	29.6	110	0.07
175.09	WATERLOOM POND DAM	22.5	23.1	679	0.55
234.08	SOUHEGAN RIVER SITE 26 DAM	79.0	4.9	1287	4.93
234.11	SOUHEGAN RIVER SITE 12A SOUTH	33.5	5.6	3304	11.06
234.12	SOUHEGAN RIVER SITE 25C DAM	69.0	5.4	1564	5.43
254.01	PINE VALLEY MILL DAM	23.0	97	70	0.01
254.33	SOUHEGAN RIVER SITE 10A DAM	59.0	6.4	2735	8.01

Table C-9: Souhegan River Basin Dams with New Hampshire Dam Inspection Reports

NHDES#	Dam Name	Height (ft)	Drainage Area (mi^2)	Maximum Storage (af)	Runoff to fill (in)
020.09	STOWELL POND	8.0	23.2	26	0.02
020.13	MCQUADE BROOK DAM	14.0	7.9	351	0.83
147.17	BURTON POND DAM	14.0	0.5	2	0.09
156.01	MERRIMACK VILLAGE DAM	20.5	171.0	171	0.02

NHDES#	Dam Name	Height (ft)	Drainage Area (mi^2)	Maximum Storage (af)	Runoff to fill (in)
159.02	GOLDMAN DAM	12.0	137.8	114	0.02
159.03	MCLANE DAM	18.7	138.0	39	0.01
167.17	GREENTREE RES DAM	4.5	0.1	17	2.42
234.04	LEIGHTON POND DAM	10.0	1.1	11	0.19
254.02	WILTON HYDRO DAM	17.0	97.0	18	0.00
254.03	SOUHEGAN RIVER III DAM	19.3	70.3	8	0.00
254.05	STONEY BROOK DAM	20.0	33.5	24	0.01
254.08	OLD WILTON RESERVOIR	17.5	8.3	8	0.02
254.11	MILL BROOK	12.0	6.7	15	0.04
254.18	BLOOD BROOK DAM	18.0	6.6	20	0.06
254.32	ERB WILDLIFE POND DAM	20.0	0.3	16	1.07

C-4.0 MODEL SIMULATION DESCRIPTIONS

Using the available data as described in the previous sections, a HEC-HMS simulation model is developed to examine operational flexibility within the Souhegan Basin. Initially, this model is calibrated so that the simulated inflow hydrograph matches the observed outflow hydrograph within reason. The model is calibrated so that simulated values approximate observed values at USGS Gage 00109400, located just above the confluence with Baboosic Brook, the only location where calibration data is available. The model is a useful tool for evaluating comparisons between different “what if scenarios,” the purpose of this modeling effort. The list of modeling scenarios is outlined in Table C-10. This simulation effort focuses on the **relative** effects of specific dam operating scenarios. This does not minimize or refute the consequences to downstream property owners during these two significant storm events; it only serves as an approximation of one scenario versus another.

Table C-10: Simulations for the Souhegan River Basin

Simulation	May 2006	April 2007	Description
Base	X	X	All reservoirs at normal pool with no flashboard operation
1	X	X	All reservoirs initially empty
2	X	X	Removal of all New Hampshire Flood Control Dams
3	X	X	Increase flashboards by 3 feet at all New Hampshire Flood Control Dams
4	X	X	Double storage at Otis Falls and Pine Valley impoundments
5		X	Otis Falls and Pine Valley with Flashboards holding throughout
6		X	Otis Falls and Pine Valley with Flashboards lowering at 0.5 foot of surcharge
7		X	Otis Falls and Pine Valley with flashboards at midnight on April 16
8		X	Otis Falls and Pine Valley with flashboards removed at 6:00 p.m. on April 15 prior to the peak
9		X	Removal of 5 Flashboards on Otis Falls at start of event and all removed by 6:00 p.m. April 15th; Pine Valley set to lower with 1 foot surcharge; both outlets on Pine Valley Mills fully opened.
10		X	Otis Falls panels lowered at 11:00 a.m. on April 16th; Pine Valley set to lower with 1 foot surcharge; both outlets on Pine Valley fully opened.
11		X	Otis Falls panels lowered at 11:00 a.m. on April 16th; Pine Valley set to lower at 6:00 a.m. April 16th
12		X	Otis Falls panels lowered at 11:00 a.m. on April 16th; Pine Valley set to lower at 9:30 a.m. April 16th

The first four simulations examine the impact that specific elements have on the overall basin: initial reservoir water levels; the impact of New Hampshire flood control cams; and the storage capacities of Otis Falls and Pine Valley Mills Dams. Simulations 5 through 12 focus on the various operating scenarios outlined in Table C-9.

C-5.0 OVERALL BASIN ANALYSIS

C-5.1 General Description

The purpose of the base simulation and Simulations 1 through 4 is to provide general conclusions on a basin wide basis. All comparisons are made at USGS Gage 00109400 on the Souhegan River just upstream of its confluence with Baboosic Brook. Later sections discuss more localized flooding impacts.

C-5.2 Observations during Flood Events

The general consensus from public comments and climatologic observations during the two storms is the April 2007 storm was much more severe. In particular there was a general concern that poorly executed dam operations and a general lack of operation policy was the main distinction between the severity in the May 2006 and April 2007 storms.

C-5.3 Simulations

C-5.3.1 Base: May 2006 and April 2007 Storms

The base simulations attempt to simulate actual conditions. As shown in Table C-11, the timing and volume of both the hydrographs match the observed hydrograph within a reasonable range (See Figures C-7 and C-8). These base runs are adequate for examining "what-if scenarios."

Table C-11: Base Run Comparison for the Souhegan River Basin

Storm Event	HEC-HMS Peak (cfs)	Measured Peak (cfs)	Difference from Measured
May 2006	6,300	6,150	2.3%
April 2007	10,415	10,550	1.3%

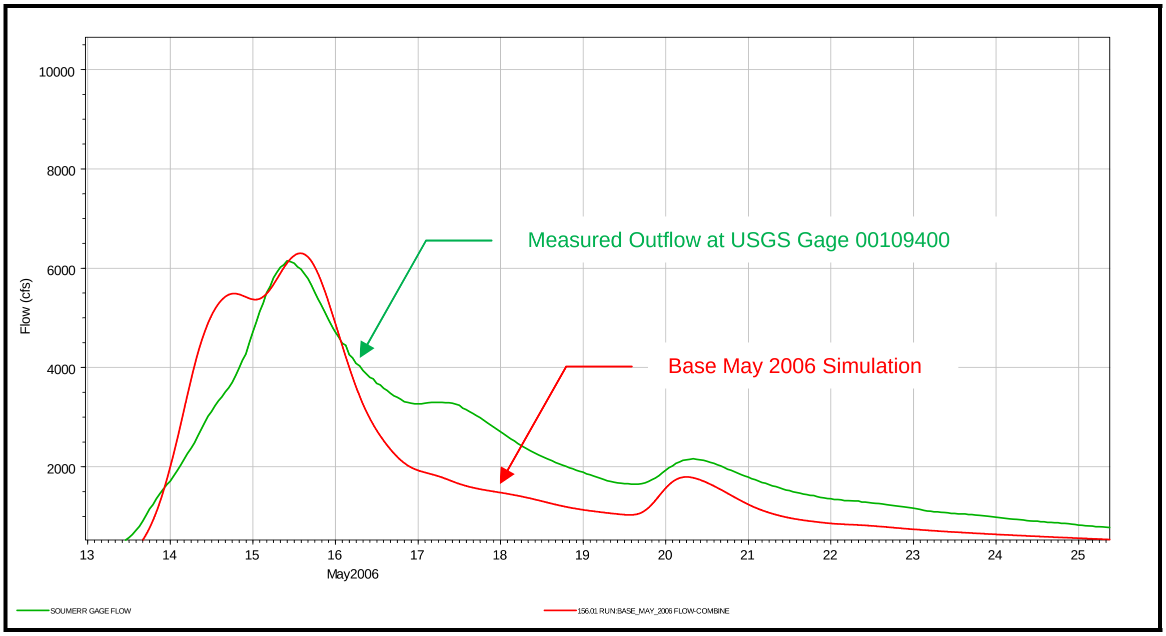

Figure C-7: Base Simulation and Measured Flows for May 2006 Storm Event

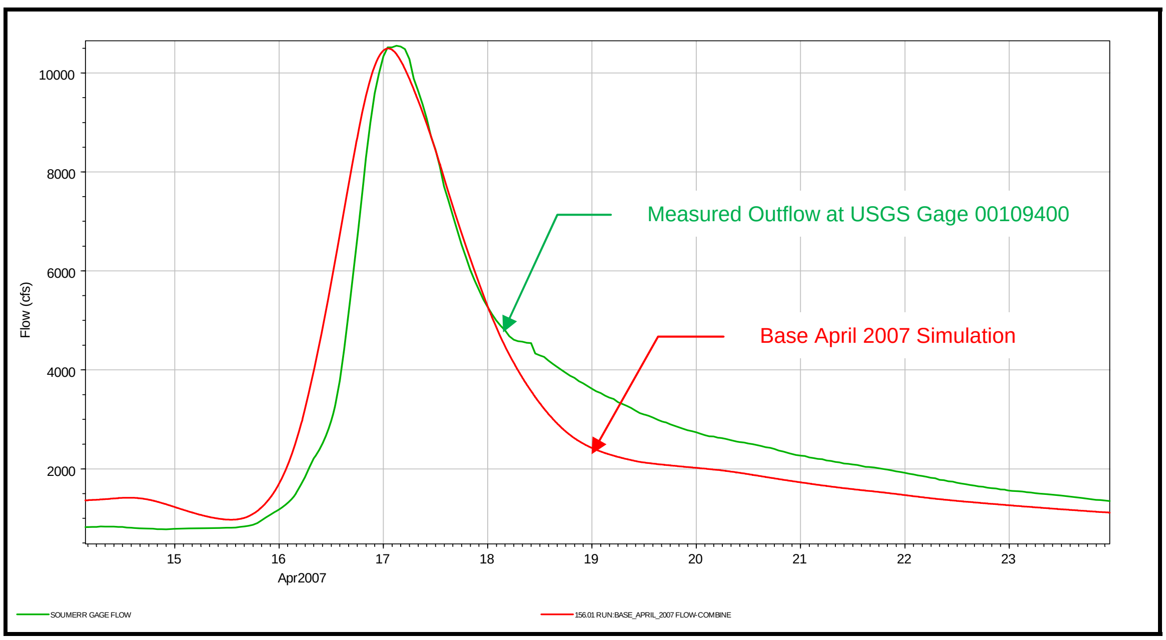

Figure C-8: Base Simulation and Measured Flows for April 2007 Storm Event

C-5.3.2 Simulation 1: All Reservoirs Initially Empty

Simulation 1 was developed to examine the range of operating flexibility in terms of operating pool levels or seasonal discharge requirements. The simulation was designed to assess whether there would be a flood control benefit if the water depth in the reservoirs in the basin were shallower, thus having more room to store runoff and therefore reduce downstream flooding. The results of these simulations are shown in Table C-12, Figure C-9, and Figure C-10.

Table C-12: Base Run Comparison for the Souhegan River Basin

Storm Event	Base Run Peak (cfs)	Simulation 1 (Assuming Reservoirs Empty, cfs)	Difference from Base
May 2006	6,300	6,290	1.5%
April 2007	10,415	10,389	1.3%

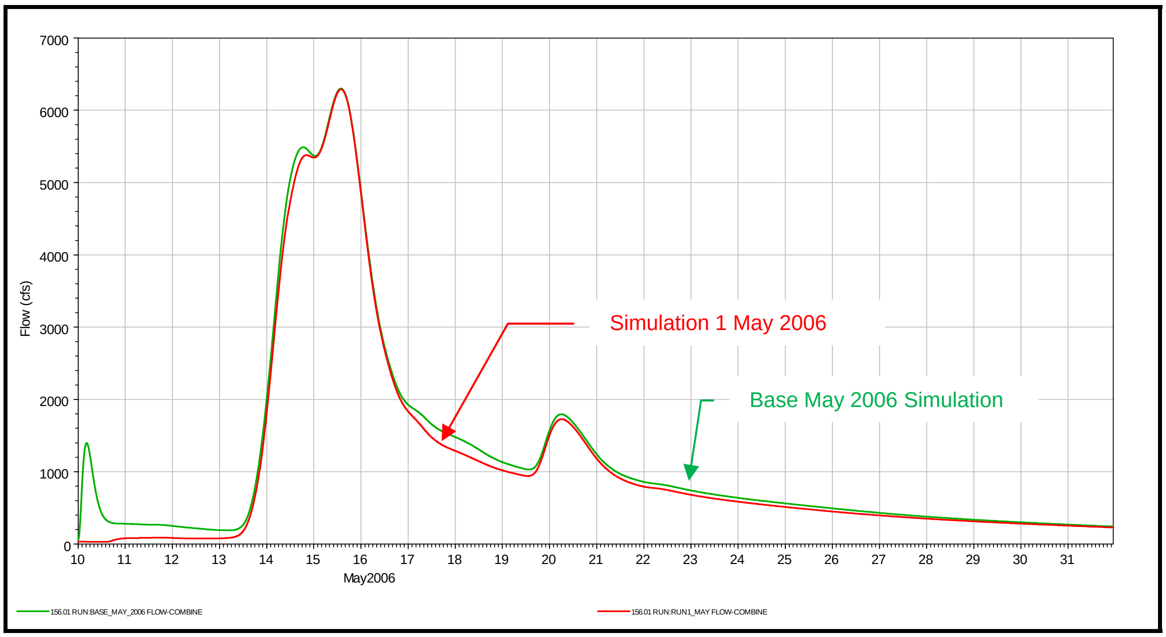

Figure C-9: Base Simulation and Simulation 1 (Assuming "Empty" Reservoirs) for May 2006 Storm Event

To examine the greatest possible effects that maximum water levels might have on the study flood events, Simulation 1 was conducted under the assumption that every reservoir was completely empty prior to arrival of both the May 2006 and April 2007 events. This would involve removing all storage in the Souhegan Basin that currently supports environmental flows, water levels for lakeside properties, and hydropower generation. Although this is not a technically realistic alternative, it does define the maximum range of operating possibilities for the Souhegan Basin.

Under this idealized set of circumstances, there is a negligible difference in peak flows even if all reservoirs in the basin were empty prior to the storms. Initially, between May 10 and May 14, there is a reduction in discharge as the reservoirs begin to fill, but the storage capacity and potential flood discharge attenuation of the reservoirs is maximized prior to the peaks of both events arriving.

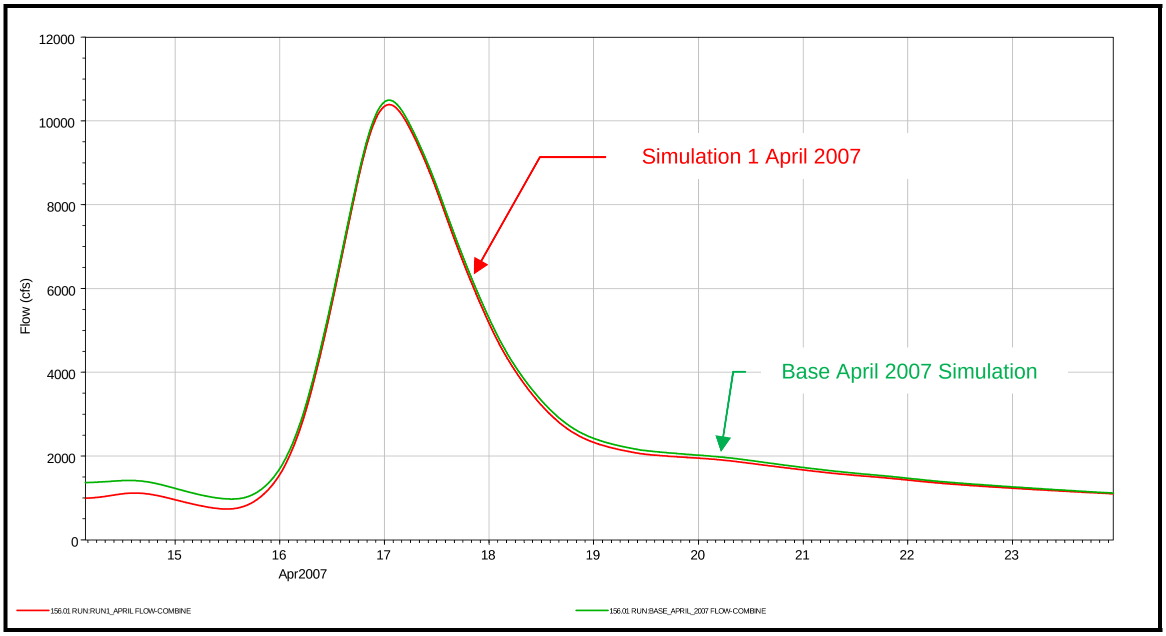

Figure C-10: Base Simulation and Simulation 1 (Assuming "Empty" Reservoirs) for the April 2007 Storm Event

C-5.3.3 Simulation 2: Assessing the Effect of the New Hampshire Flood Control Dams

The State of New Hampshire operates 12 flood control dams located in the upper end of the Souhegan Basin (refer to Figure C-1). These dams were originally built by the National Resources Conservation Service and turned over the State. Since these dams were designed for flood control, the contribution and benefit in the May 2006 and April 2007 storm events was questioned. These dams are summarized in Table C-13 and shown in Figure C-1.

Table C-13: Summary of Souhegan River Basin Dams Operated by the State of New Hampshire

NHDES#	Dam Name	Height (ft)	Drainage Area[1] (mi^2)	Maximum Storage[2] (af)	Runoff to fill[3] (in)
147.26	SOUHEGAN RIVER SITE 28 DAM	29.0	1.1	185	3.16
147.28	SOUHEGAN SITE 8 DAM	25.0	4.7	2721	10.86
175.01	SOUHEGAN SITE 14 DAM	35.0	2.1	885	7.90
175.19	SOUHEGAN RIVER SITE19 DAM	35.5	11.4	2072	3.41
175.20	SOUHEGAN RIVER SITE 13 DAM	13.5	0.8	249	5.84
175.21	SOUHEGAN RIVER SITE 35 DAM	30.0	6.4	647	7.67
234.08	SOUHEGAN RIVER SITE 26 DAM	79.0	4.9	1287	4.93
234.11	SOUHEGAN RIVER SITE 12A DAM S	33.5	5.6	3304	11.06
234.12	SOUHEGAN RIVER SITE 25B DAM	69.0	5.4	1564	5.43
254.30	SOUHEGAN RIVER SITE 15 DAM	13.0	1.1	315	12.75
254.33	SOUHEGAN RIVER SITE 10A DAM	59.0	6.4	2735	8.01
254.34	SOUHEGAN RIVER SITE 33 DAM	21.0	1	1078	20.21

Notes:

[1] Drainage approximated from WISE or dam inspection report, if available.

[2] Maximum storage in this table is extrapolated to estimated storage above dam to also account for overtopping storage.

[3] Runoff to fill is the ratio of maximum storage to drainage area as defined in these tables.

To examine the effect of these reservoirs, Simulation 2 assumed that none of these reservoirs had been constructed.

Simulation 2 was compared with the base run to examine the effects the New Hampshire flood control reservoirs had in the storm events. The peak flows at USGS Gage 00109400 are about 25 percent less than they would have been if the reservoirs were not built (refer to Table C-14 and Figures C-11 and C-12). Although the reservoir storage in the Souhegan Basin is relatively small, these 12 flood control reservoirs do serve the purpose of reducing flooding.

Table C-14: Base Run Comparison for the Souhegan River Basin

Storm Event	Base Run Peak (cfs)	Simulation 2 (No NH Flood Control Dams, cfs)	Difference from Base
May 2006	6,300	7,916	25.7%
April 2007	10,415	13,289	27.6%

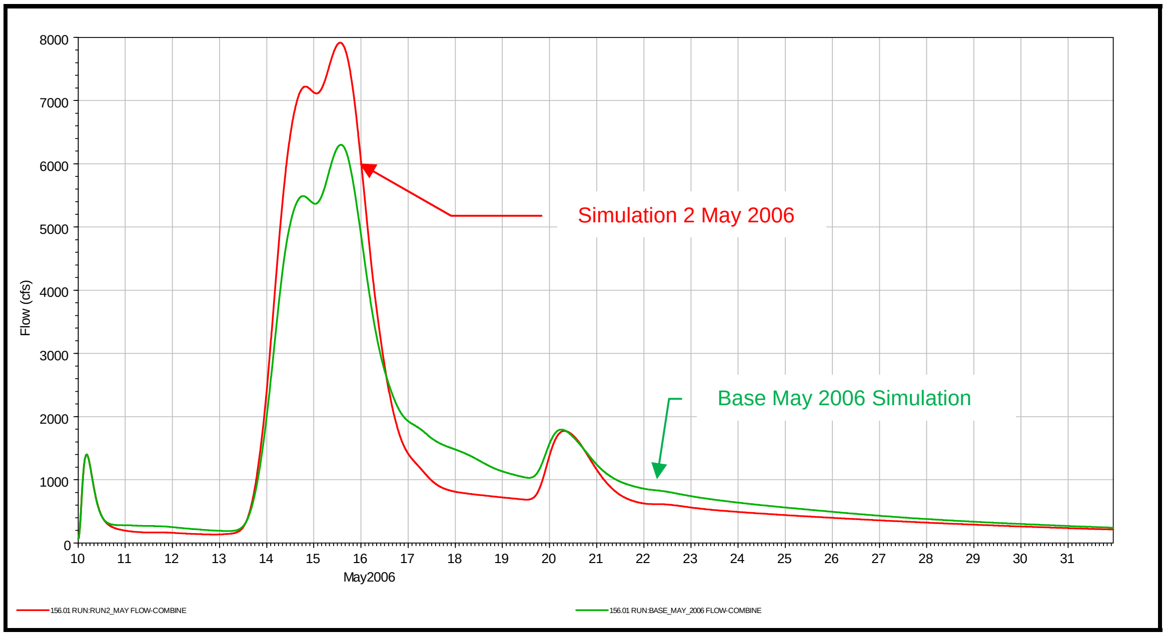

Figure C-11: Base Simulation and Simulation 2 (No New Hampshire Flood Control Dams) for May 2006 Storm Event

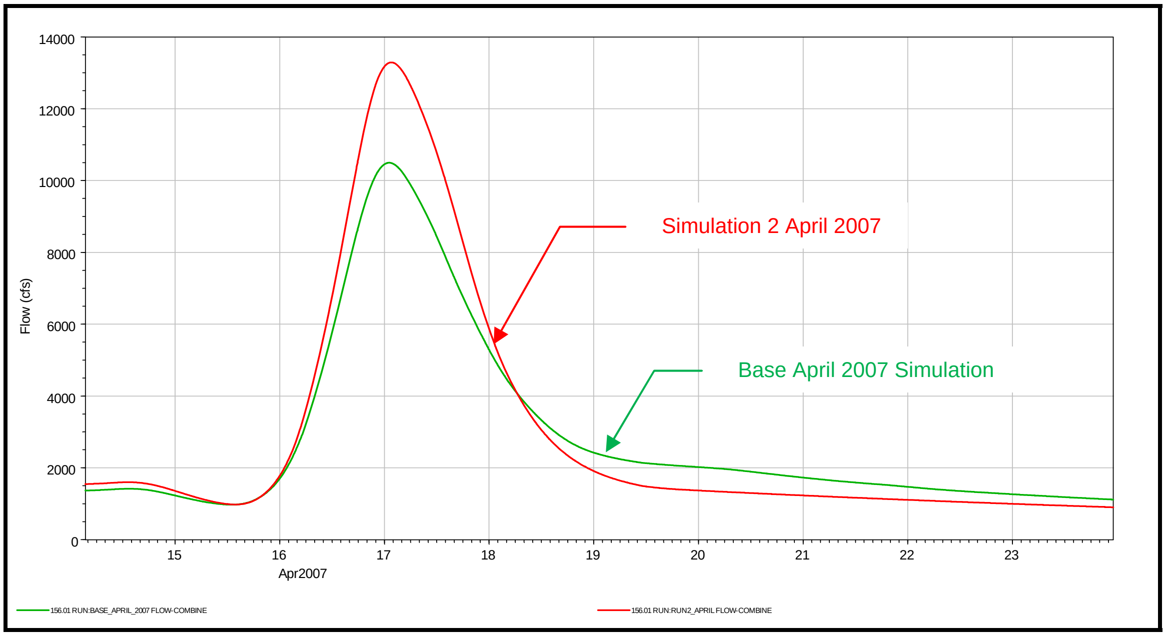

Figure C-12: Base Simulation and Simulation 2 (No New Hampshire Flood Control Dams) for April 2007 Storm Event

C-5.3.4 Simulation 3: Use of flashboards on New Hampshire Flood Control Dams

Simulation 2 established that the New Hampshire Flood Control Dams reduced peak flows by a substantial quantity. The question was then explored: could these dams be used to store more water and further decrease downstream flooding? To examine the incremental effect that greater storage might have had, Simulation 3 was developed with the assumption that 3-foot flashboards were installed on the emergency spillways of all of the New Hampshire Flood Control Dams. This would increase the storage capacity of each reservoir.

Compared to the Base Simulations for both the May 2006 and April 2007 storm events, there was virtually no impact on the overall basin by slightly increasing the storage capacity on the New Hampshire flood control dams. As shown in Table C-15 and Figures C-13 and C-14, there is no measureable difference between Simulation 3 and the Base Simulation.

Table C-15: Base Run Comparison for the Souhegan River Basin

Storm Event	Base Run Peak (cfs)	Simulation 3 (Flashboards Added at NH flood control dams, cfs)	Difference from Base
May 2006	6,300	6,279	0.3%
April 2007	10,415	10,480	0.6%

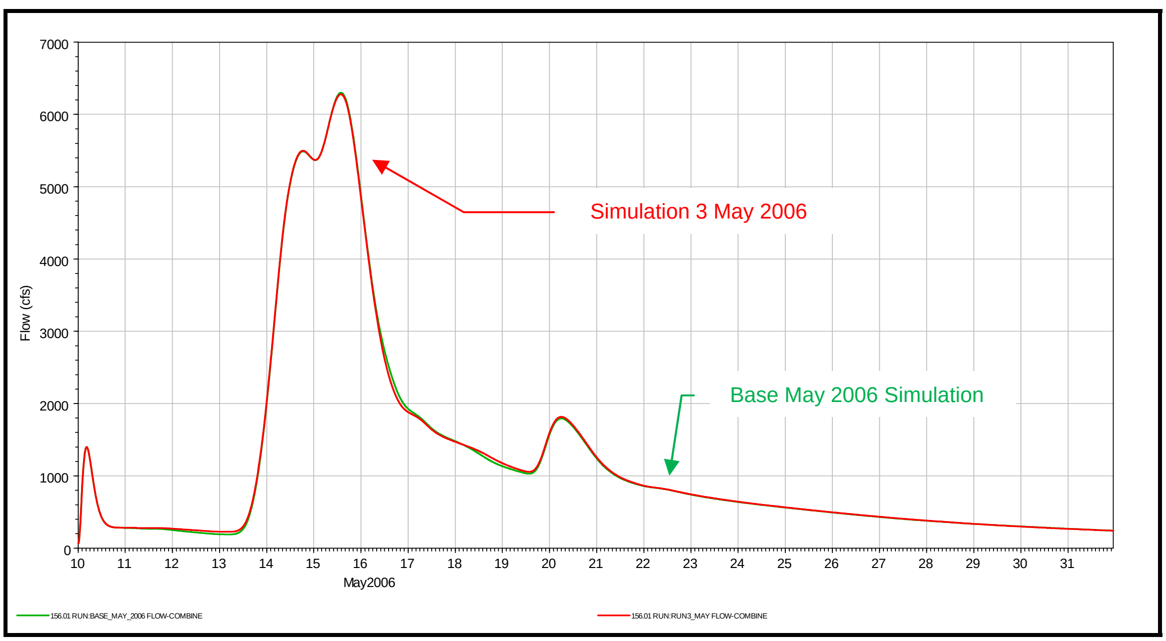

Figure C-13: Base Simulation and Simulation 3 (Flashboards added at New Hampshire Flood Control Dams) for May 2006 Storm Event

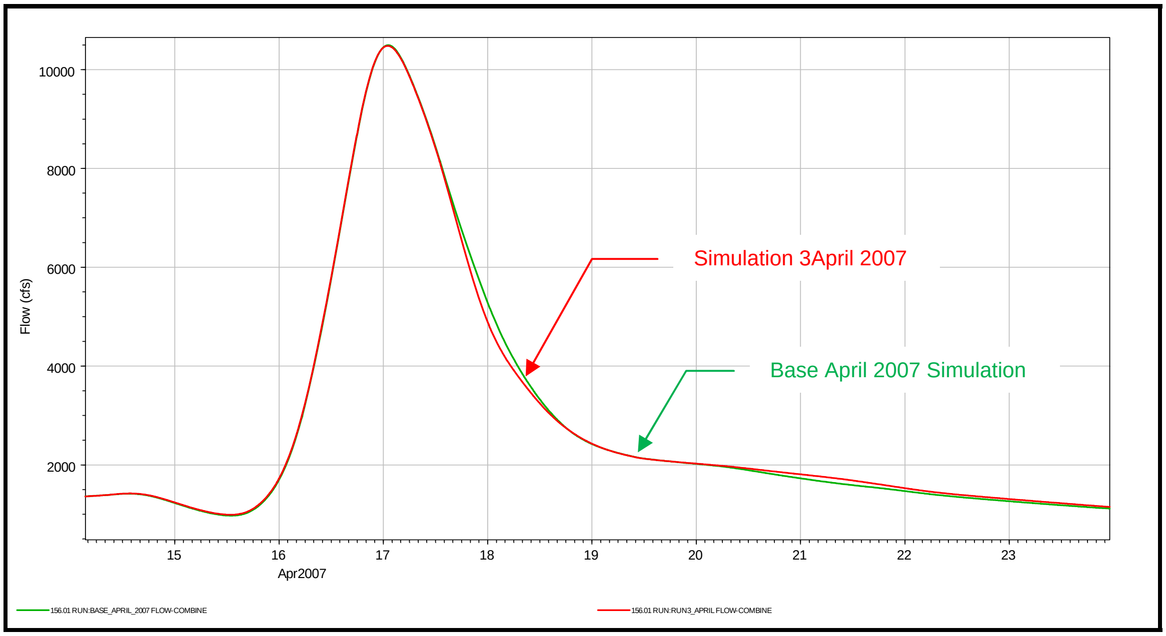

Figure C-14: Base Simulation and Simulation 3 (Flashboards added at New Hampshire Flood Control Dams) for April 2007 Storm Event

C-5.3.5 Simulation 4: Double the storage on Otis Falls and Pine Valley Mill Dam

Much public concern has been expressed regarding the operation of Otis Falls Dam (OFD) and Pine Valley Mill Dam (PVD), two run-of-the-river dams located in the upper half of the Souhegan Basin. The effect that these dams have on the overall Souhegan Basin is examined in Simulation 4. In Simulation 4, the storage capacity of both Otis Falls and Pine Valley Mill Dam is doubled from 105 af to 210 af and from 70 af to 140 af respectively. Given the small amount of storage of these two dams relative to the rest of the Souhegan Basin (~12,000 af), the results from Simulation 4 do not vary from the Base Simulation by any significant quantity for either storm event (see Table C-16 and Figures C-15 and C-16).

It is important here to distinguish between the overall basin perspective in Simulation 4 and the localized effect of these dams in Simulations 5 through 12. Simulation 4 is compared to the Base Simulation at USGS Gage Number 001094000, 20 miles downstream from Pine Valley Dam and 29 miles downstream of Otis Falls Dam. Simulations 5 through 12 focus on the towns of Greenville, Wilton, and Milford, where impacts from these dams' operation are more localized and direct.

Table C-16: Base Run Comparison for the Souhegan River Basin

Storm Event	Base Run Peak (cfs)	Simulation 4 (Increased storage at OFD and PVD, cfs)	Difference from Base
May 2006	6,300	6,297	0.0%
April 2007	10,415	10,493	0.1%

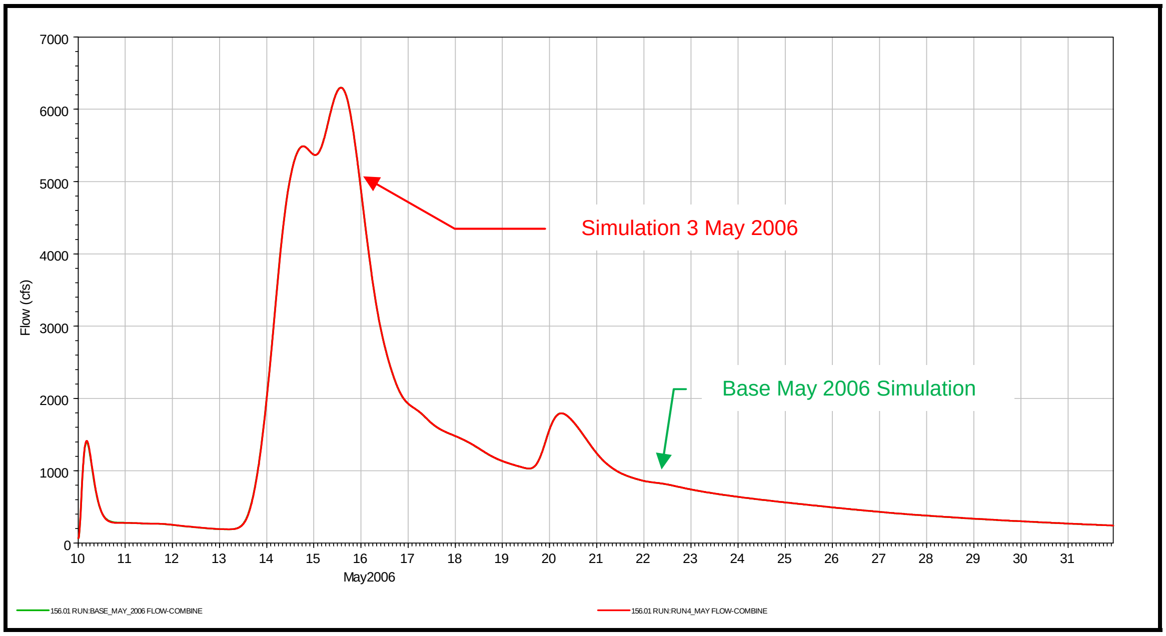

Figure C-15: Base Simulation and Simulation 3 (Increased storage at OFD and PVD) for May 2006 Storm Event

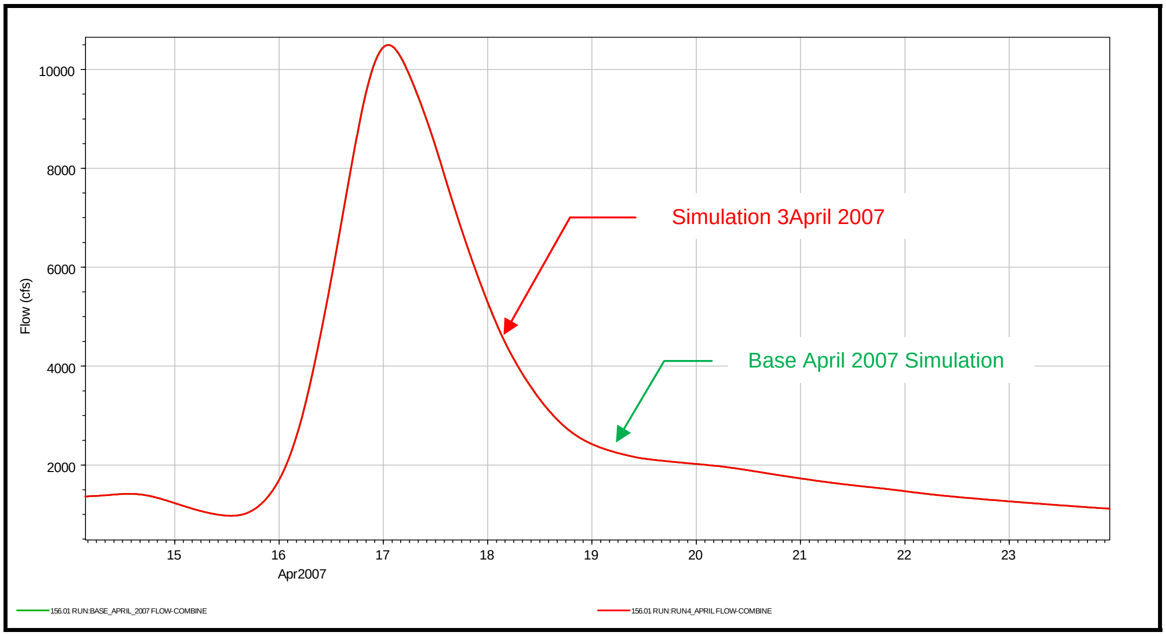

Figure C-16: Base Simulation and Simulation 3 (Increased storage at OFD and PVD) for May 2006 Storm Event

C-5.4 Evaluation of the Results

Comparison between the Base Simulation and Simulation 1 (where all impoundments in the basin are assumed to be empty at the beginning of the storms) shows the entire operating envelope for the Souhegan Basin. Even if it were possible to empty all of the impoundments in the basin prior to these storms, there would have been no flood control benefit.

Only Simulation 2, which assumes none of the New Hampshire flood control dams were built, showed any measureable difference in the flood discharges. If there were not flood control dams, discharges at would have been over 25 percent greater during these events at the USGS gage. It is evident that the New Hampshire flood control dams prevented a substantial amount of flooding; unfortunately, an increase in storage capacity (implied by the use of flashboard in Simulation 3) for the New Hampshire flood control dams has little additional flood control benefit.

Ultimately, the operation of run-of-the-river dams has no effect at the USGS gage as shown in Simulation 4. Even if the storage capacity of these facilities is doubled, there is no measureable difference at this location.

C-6.0 OTIS FALLS DAM (NHDES# 101.01)

C-6.1 General Description

Otis Falls Dam is a run-of-the-river dam located in the upper Souhegan River Basin in the town of Greenville. It was constructed in 1936 and its primary current use is for the development of hydropower. To maximize hydropower output, 3-foot wooden flashboards are installed above the emergency spillway as shown in Figure C-17.

Figure C-17: Otis Falls Dam during March 8, 2008 Rainfall Event

No operable outlet works exist on Otis Falls dam, although two inoperable inlet openings in the forebay area and an intake sluice gate exist as shown in Figure C-18. It does not appear that these provide any functionality nor do they appear to offer any operating flexibility to improve flood control performance.

For the development of hydropower, a FERC license is required to be maintained and periodically updated. Part of the licensing involves the use of flashboards which are required to be used in accordance with the provisions in the license. To avoid deleterious environmental impacts, Otis Falls operators are required to maintain a relatively constant level behind the flashboards.

The informal operating rules on Otis Falls Dam require field personnel to manually remove the flashboards prior to the arrival of flood event (Greenwood 2008). This requires field personnel to visit the dam site either prior or during the event and accurate forecasting data. As discussed in section C-6.2, there is some controversy about the exact procedure that was followed during the April 2007 storm event.

<u>Otis Falls Critical Data</u>

- Maximum storage at top of dam embankment: 105 af
- Drainage area: 30 mi^2
- 3-foot manual flashboards
- Takes only 0.07 inch of runoff to fill

Figure C-18: Otis Falls Plan View

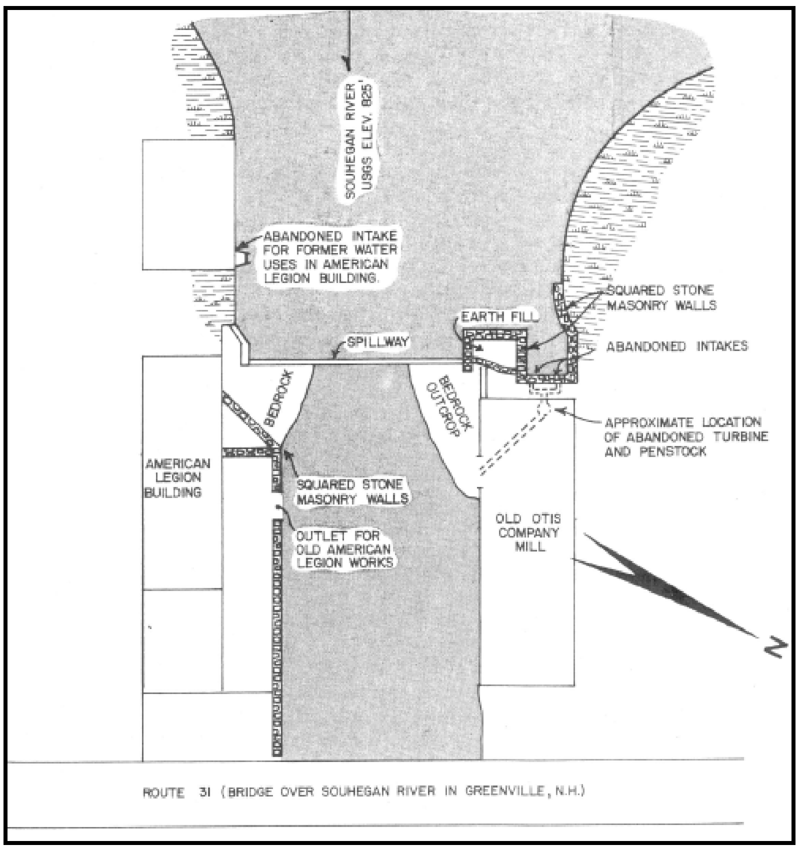

Figure C-19: Otis Falls Plan Schematic (NHDAMS Data Sheet 2007)

C-6.2 Observations During April 2007 Flood Event

Based on a letter to New Hampshire State Representative Peter Leishman dated January 7, 2008 from James Gallagher, Chief Engineer of NHDES, there are slightly varying accounts of the flashboard operation for Otis Falls Reservoir, as described in Table C-17.

Table C-17: Observed Operations at Otis Falls Dam (NHDES# 101.01)

Account of Operations	Source
5 of 24 panels of flashboard removed on Saturday, April 14th; More panels removed on Sunday Morning, April 15th; All flashboards lowered by end of day Sunday April 15th.	Mr. Robert Greenwood
Operator seen lowering panels at 10:30 a.m. on Monday, April 16th	Fire Chief at Wilton
No operations made at 10:30 a.m. on Monday April 16th; for purposes of modeling, it was assumed that "no operations made" meant that all flashboard were in place until 11:00 a.m. on Monday April 16th, at which time they were all removed.	Dam safety engineer with NHDES Dam Bureau

Scenarios were examined evaluating all of these accounts on potential flood impacts downstream of the dam.

C-6.3 Simulations

Simulations 5 through 10 are applicable to the examination of Otis Falls Dam.

In Simulation 5, where the flashboards are simulated to stay in place throughout the entire event, there is an increase in overall pond elevation but there is little to no effect on the discharges since no sudden release or rapid pond draining occurs. There has been little demonstrated public concern over the upstream elevation along the shoreline of the impoundment. However, Figure C-20 shows the general elevation trend: the longer the flashboards are in place, the higher the upstream elevation.

In Simulation 6, where the flashboards are set to be removed with a given depth of overflow, the flashboards fall very early in the storm with a negligible effect on downstream flow.

Simulations 7 and 8 demonstrate that removing the flashboards close to the peak can generate an increase in the downstream discharge rates.

Simulations 9 and 10 demonstrate the time window when the flashboards may have been removed either sometime during April 15th (Simulation 9) or just before noon on April 16th (Simulation 10). Simulation 10 is a worst case scenario since it assumes that no flashboards were removed and all water stored behind the dam was released instantaneously. And since the release on April 16th is closer to the arrival of the peak flow, it has the greatest consequence.

Assuming the worst case condition (Simulation 10), there would have been a large increase in flows (from 1,230 cfs to 2,330 cfs for a difference of 1,100 cfs), and an increase in elevation of about 2.5 feet immediately downstream of the dam (between Otis Falls Dam and Chamberlain Dam). However, this difference diminishes quickly as the flow traverses the downstream floodplain. The differences in peak flow converge as floodplain attenuation stores the additional water caused by the removal of the flashboards. Using the Modified Puls routing method in HEC-HMS, it is estimated that the difference in flow between the Base Simulation (no flashboards) and Simulation 10 is reduced to less than 100 cfs with a corresponding elevation difference of less than 0.1 foot at the point where the Souhegan River intersects Old Wilton Road in the town of Greenville. Using the same methodology, there is no difference in flow by the time these flow arrive at Pine Valley Mills Dam, 9 miles downstream. The approximate area of impact, between Main Street where the dam is located and the intersection of Fitchburg Road (State Route 31) and Old Wilton Road, is shown on Figure C-21.

The effects of this peak were also analyzed by using an approximate unsteady flow approach with HEC-RAS, a widely used hydraulics model (USACE 2002). Immediately downstream of Otis Falls, there is almost a 2-foot increase in water surface elevation. However, within 4,900 feet downstream of Otis Falls Dam, the difference between the Base Run and Simulation 10 (the simulation with the greatest change in flow) become negligible as floodplain storage attenuates the increase in peak flow. Thus, although this increase in peak water surface level is significant to property owners adjacent to Otis Falls Dam, the effect is most likely not noticeable by the time the water arrives at Pine Valley Mills Dam 9 miles downstream.

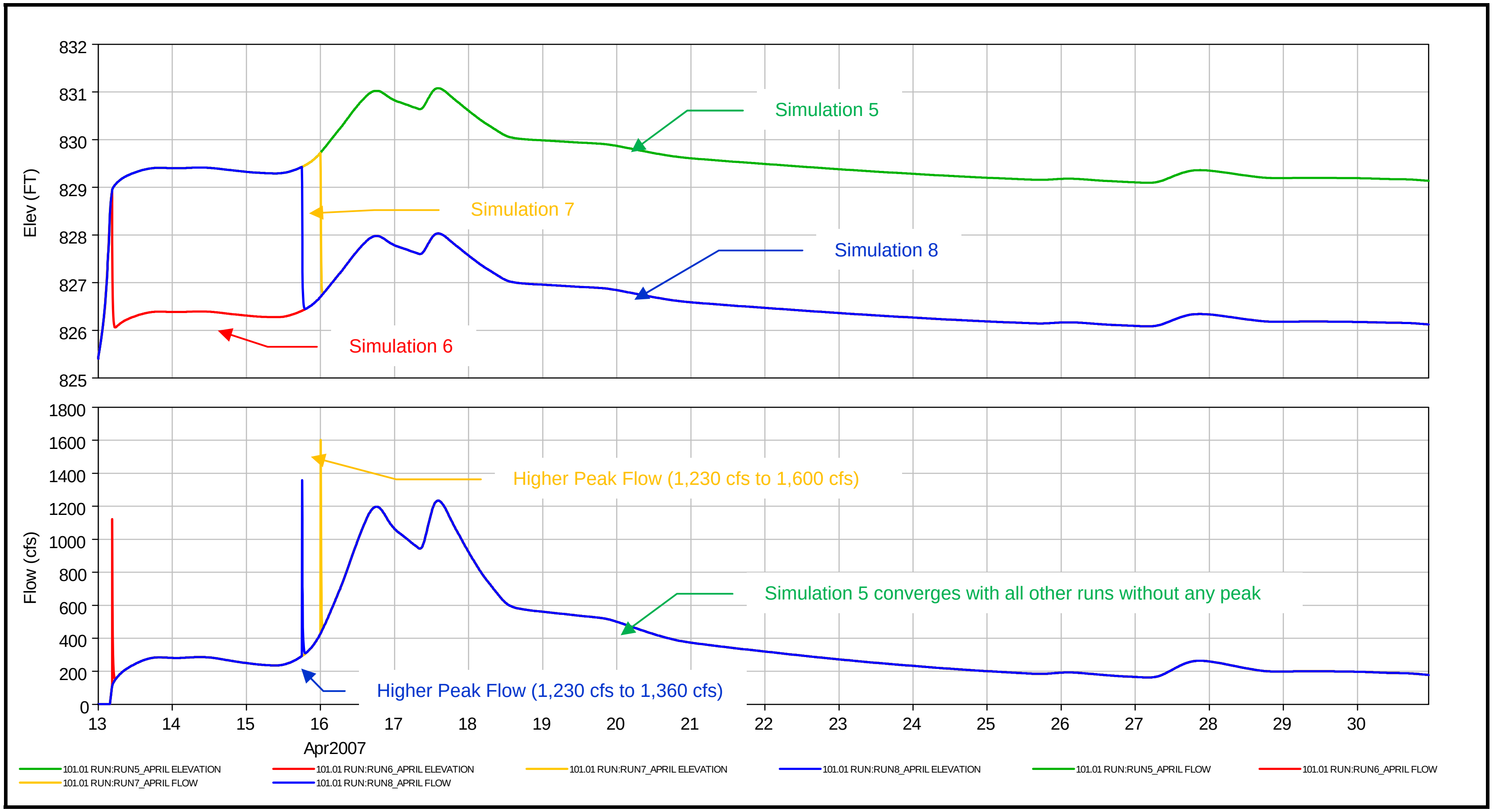

Figure C-20: Otis Falls Elevation and Discharge for Simulations 5, 6, 7, and 8

Simulation 5: Flashboards remain in place
Simulation 6: All flashboards removed when overtopped by 0.5 foot
Simulation 7: All flashboards removed at midnight on April 16
Simulation 8: Flashboards removed at 6 p.m. on April 15

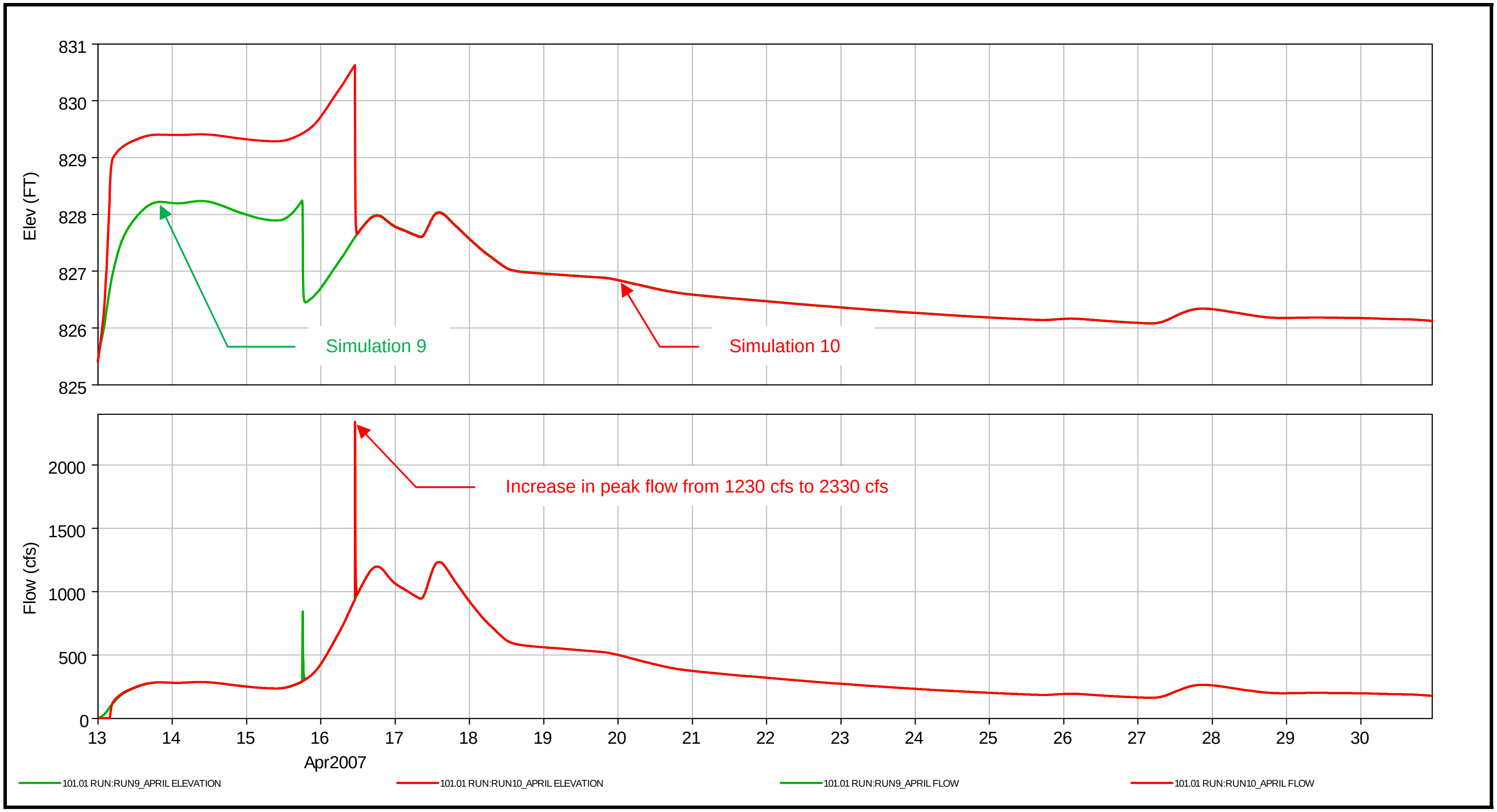

Figure C-21: Otis Falls Elevation and Discharge for Simulations 9 and 10

Simulation 9: Flashboards removed starting April 15th

Simulation 10: Flashboards removed 11 a.m. April 15th

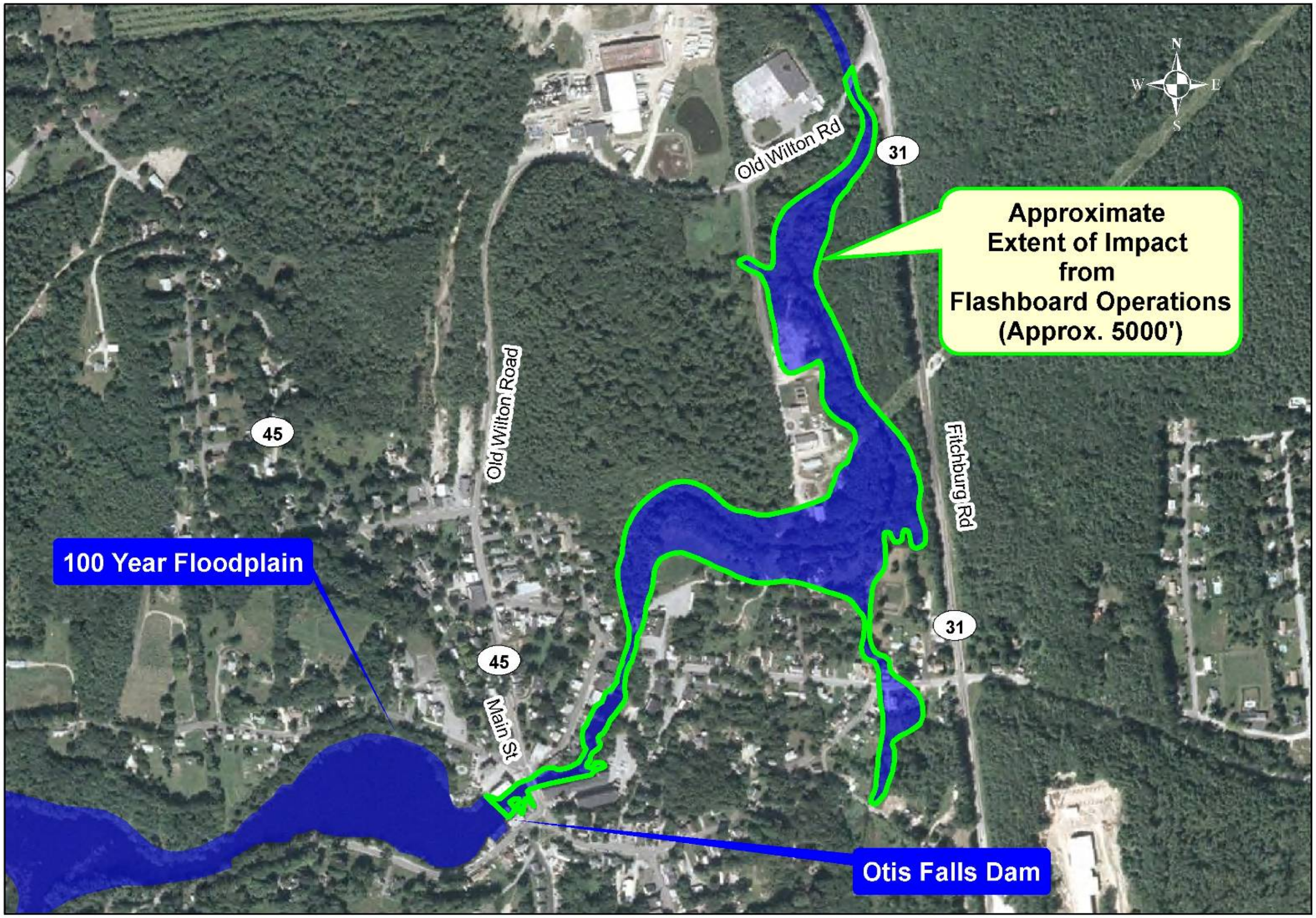

Figure C-22: Approximate Area of Impact from Flashboard Operation on Otis Falls Dam

C-6.4 Evaluation of Results

Simulations 5 through 10 demonstrated that the timing of the flashboard removal impacts the areas immediately downstream of Otis Falls Dam. If the flashboards were gradually removed at the beginning of the storm and totally removed by the evening of April 15th, the peak water surface elevation levels immediately downstream of Otis Falls would not have been affected; if no flashboards were removed until 11:00 a.m. on April 16th, there would have been a substantial increase in discharge and peak water surface elevation immediately downstream from the dam.

Since there are no gages within this area, it would be very difficult to deduce the precise operations of Otis Falls Dam during the April 2007 storm event. Regardless, more formal operating procedures that require the early removal of these particular flashboards may help to protect the downstream properties with the area of impact (shown in Figure C-22).

Because of floodplain attenuation, the effects of the flashboard removal are less noticeable downstream; the effects of the flashboard removal are minor as the Souhegan leaves the town of Greenville and enters Wilton.

C-7.0 PINE VALLEY MILLS DAM (NHDES# 254.01)

C-7.1 General Description

Pine Valley Mills Dam is a run-of-the-river dam located in mid Souhegan River Basin in the town of Wilton. It was constructed in 1912 and its primary current use is for the development of hydropower. To maximize hydropower output, 4-foot automatic flashboards are installed above the emergency spillway. The plan view is shown in Figure C-23. A schematic of the site is shown in Figure C-24.

Two operable outlet works exist on Pine Valley Mills dam, as shown in Figure C-23. Some functionality and operating flexibility to improve flood control performance exists with these outlet works.

For the development of hydropower, a FERC license is required to be maintained and periodically updated. Part of the licensing involves the use of flashboards which are required to be used in accordance with the provisions in the license. Pine Valley Mills operators are also required to maintain a relatively constant level behind the flashboards.

Pine Valley Mills Critical Data

- Maximum storage to top of embankment: 70 af
- Drainage area: 97 mi^2
- 4-foot automatic flashboards
- 0.01 inch of runoff fills lake

Figure C-23: Plan View of Pine Valley Mills Dam

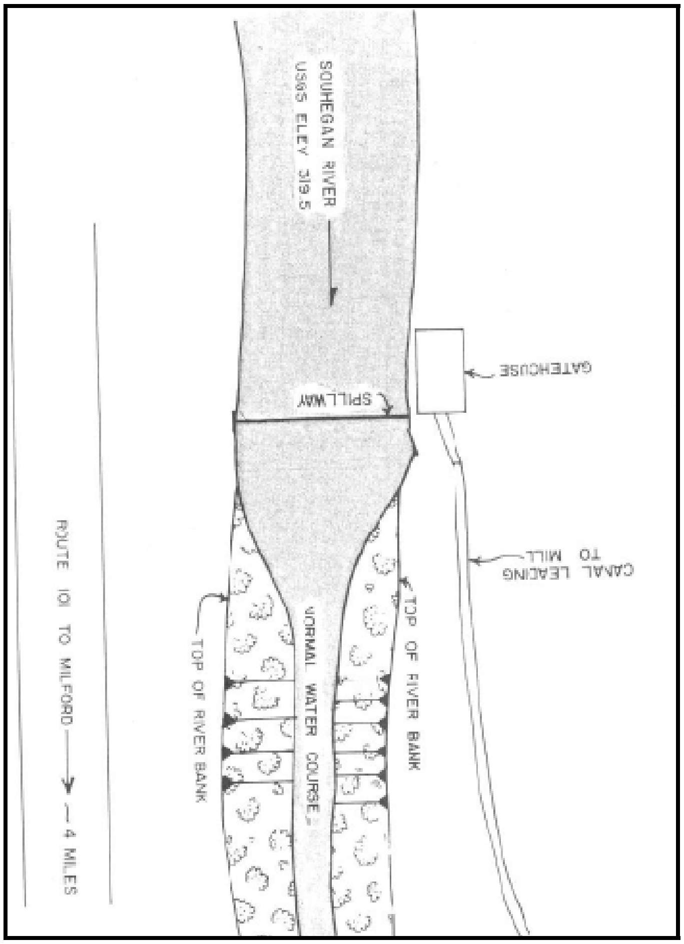

Figure C-24: Pine Valley Mills Plan Schematic (NHDAMS Data Sheet 2007)

C-7.2 Observations During April 2007 Flood Event

Based on the letter to New Hampshire State Representative Peter Leishman dated January 7, 2008, from James Gallagher, Chief Engineer of NHDES, there are slightly varying accounts of the flashboard operation for Pine Valley Mills Reservoir, as described in Table C-18.

Table C-18: Observed Operations at Pine Valley Mill Dam (NHDES#254.01)

Account of Operations	Source
Mr. Greenwood notified Mr. Young at 11:00 a.m. on Sunday April 15th that releases were going to be made from Otis Falls; Mr. Young opened waste gates at 5:00 p.m. on Sunday, April 15th; regardless, the water level caused the flashboards to fall over by 6:30 a.m. on April 16th.	Mr. Michael Young
Flashboards fall over between 9:00 a.m. and 10:00 a.m. on April 16th.	Fire Chief at Wilton

C-7.3 Simulations

Simulations 5 through 8, 11, and 12 are applicable to the examination of Pine Valley Mills Dam. The time window of accounts for the lowering of the automatic flashboards range from 6:30 a.m. to 10:00 a.m. on April 16th. Figures C-25 and C-26 demonstrate the effects of discharge when the flashboards are removed within this time frame.

In Simulation 5, the flashboards are assumed to hold throughout the entire storm event. The resulting peak elevation at the dam is 332.0, but there is no effect on downstream flows since the volume of water behind the flashboards is not released.

In Simulation 6, the flashboards on Pine Valley Mills Dam are set to lower with a 0.5-foot overtopping versus the 1.0-foot overtopping depth that is estimated to be currently installed. This results in an instantaneous peak flow increase from 1,500 cfs to 3,600 cfs, but the subsequent peak flow of over 5,500 cfs is not affected by the early release.

Simulations 7 and 8 simulate the removal the flashboards between 6 p.m. on April 15th and at midnight on April 16th. The earlier removal of flashboards in Simulation 8 is more important with Pine Valley Mills Dam since the lower flows at Pine Valley Mills Dam can be released using the waste gates available on the dam. When the flashboards are removed at 6:00 p.m., there is barely a noticeable difference between elevations and discharges; when the flashboards are removed 6 hours later, there is more noticeable spike in elevation and peak flows (~900 cfs) because the waste gates are at capacity during this time frame.

Simulation 11 and 12 duplicate the account of operations in the table. As shown in Figure C-25, there are noticeable increases in reservoir elevation and discharge with later flashboard removal. However, for Simulations 7, 8, 11, and 12, none of increases in discharges exceed the subsequent peak flow of over 5,500 cfs.

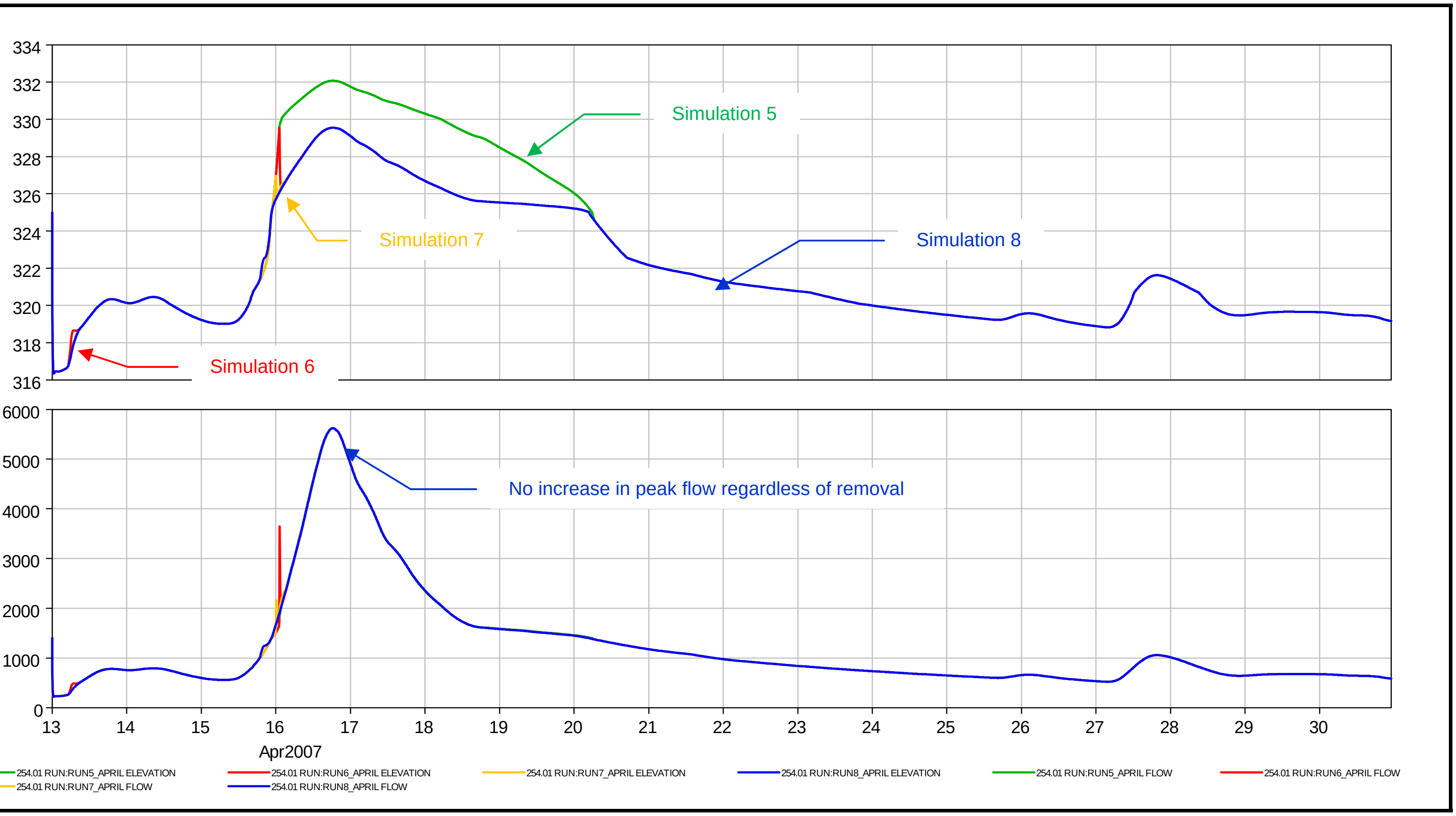

Figure C-25: Pine Valley Mills Elevation and Discharge for Simulations 5, 6, 7, and 8

ulation 5: Flashboards remain in place throughout event
ulation 6: Flashboards lower when overtopped by 0.5 foot
ulation 7: Flashboards removed at midnight on April 16th
ulation 8: Flashboards removed at 6 p.m. on April 15th

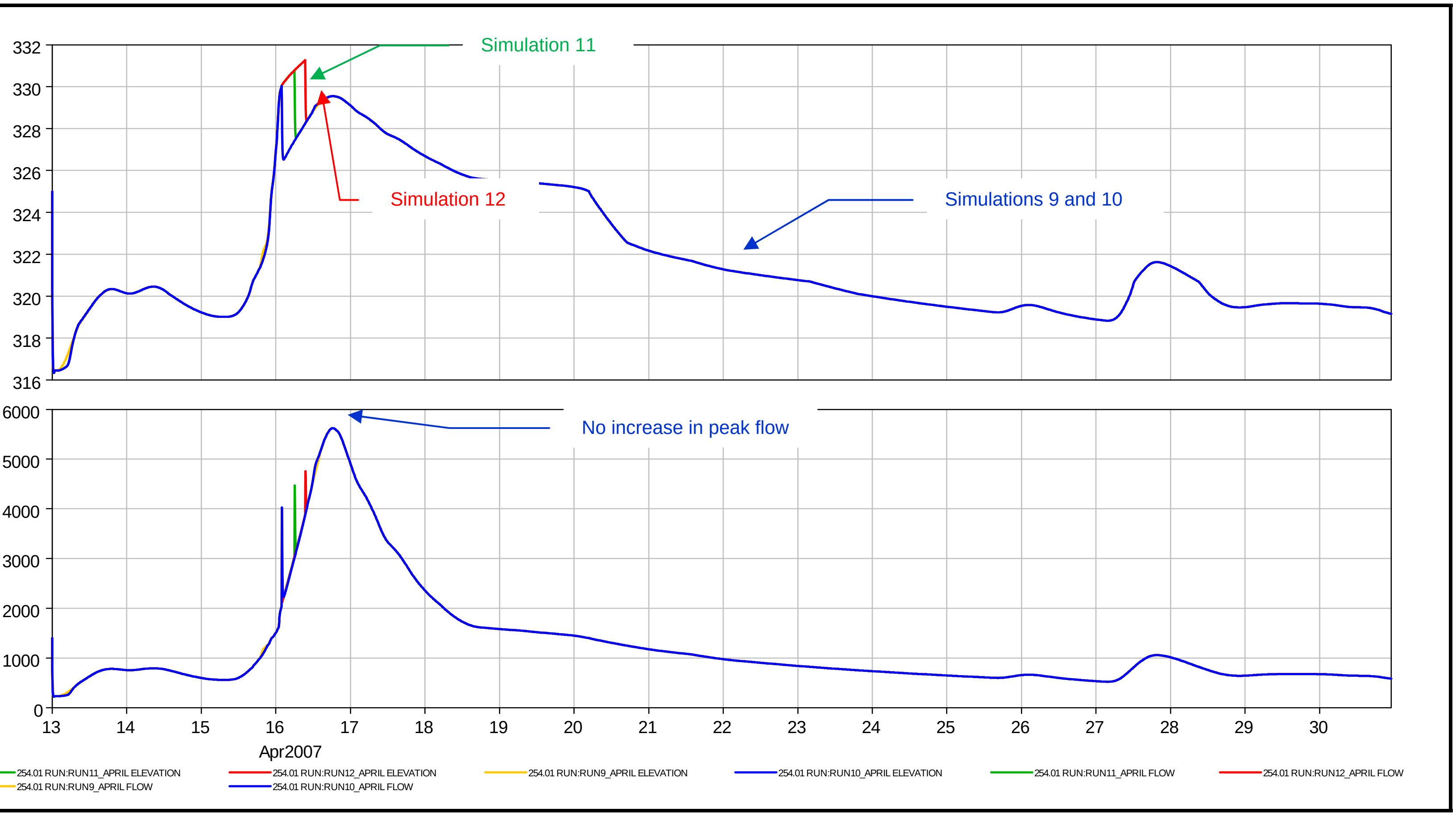

Figure C-26: Pine Valley Mills Elevation and Discharge for Simulations 9, 10, 11, and 12

ulation 9: Impact of operations at Otis Pond not seen at Pine Valley
ulation 10: Impact of operations at Otis Pond not seen at Pine Valley
ulation 11: Flashboards at Pine Valley lowered at 6 a.m. April 16th
ulation 12: Flashboards at Pine Valley lowered at 9:30 a.m. April 16th

C-7.4 Evaluation of the Results

From an observer's perspective located immediately downstream of Pine Valley Mills dam, the removal of the flashboard involves an immediate rush of water; a more substantial rush of water is observed if the corresponding water surface elevation is higher. However, from this same perspective it would also be easy to confuse the peak discharge associated with the removal of flashboards with the subsequent discharge associated with the arrival of the upstream peak discharge.

The location of Pine Valley Mills reservoir relative to Souhegan Basin decreases the impact of its operations. There were increased discharges and elevated water surfaces downstream of the dam because of its operations. However, the ultimate peak flood discharge and the peak downstream water surface elevations were unaffected by these operations

C-8.0 CONCLUSIONS AND RECOMMENDATIONS

1. It is apparent from analyzing climatic and runoff data that the Souhegan Basin incurred a significant runoff event in both the May 2006 and April 2007 storms. The runoff was not as attributable to significant rainfall as it was to the combination of meteorologic factors that existed in the basin prior to onset of the majority of rainfall. In May 2006, this involved moderate antecedent rainfall saturating the soil a few days prior to the event and then moderate intensity rainfall over a long period of time (over 3 days) during the event. In April 2007, this involved substantial antecedent rainfall and snowmelt being followed by a rainfall of significant intensity (between the 2- and 5-year rainfall event for a 24-hour period) over a shorter period of time falling on saturated soil.

2. Given the relatively low recurrence interval of rainfall (1- to 2-years) and the consequent high recurrence interval of runoff (10- to 50-years), any basin-wide policy for flood control protection and floodplain management should account for the influence of antecedent conditions. In particular, snowmelt scenarios should be accounted for in any hydrology and hydraulics analysis that is used for public policy purposes (such as Emergency Action Plans).

3. A comprehensive model was developed for the Souhegan River Basin to examine the effects of reservoir operation in the May 2006 and April 2007 storm events. Several simulations were developed to analyze the effects of both flood control and run-of-the-river impoundments. From an overall basin perspective, the flood control structures operated by the State of New Hampshire reduced flood discharges by substantial quantities, while the overall basin effect of the run-of-the-river impoundments was minor.

4. The localized effects of flashboard operation of the run-of-the-river dams was substantial during the April 2007 storm event and evidenced by the public concern over the flashboard operation and several of the simulation developed in this study. An overall watershed policy for the use and removal of these devices should be developed that considers timing, maintenance, and coordination with FERC permitting.

C-9.0 REFERENCES

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C-10.0 ATTACHMENT A: CALIBRATION PROCEDURE

Snyder's Method for Unit Hydrograph

The lag time, t_p (in hours), or approximately the time between the rainfall and the peak of the hydrograph, is defined as:

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

where C_t = basin coefficient; L = length of the main stream from the outlet to the divide; and L_c = length along the main stream from the outlet to a point nearest the watershed centroid. C_t is modified during calibration so that the timing of the simulated runoff peak. The peak discharge of the unit hydrograph (in cfs) is determined by the following function:

$$Q_p=C_pA/t_p$$

where C_p = peaking coefficient; A = drainage area in square miles, and t_p is as previously defined. The unit hydrograph is then convoluted with an historical rainfall event to produce an event hydrograph such as the April 2007 or May 2006 rainfall-runoff event.

Since there was only one runoff gage and limited precipitation data, C_t and C_p are assumed to have the same value throughout the Souhegan watershed for both the May 2006 and April 2007 storm events. For the Souhegan Basin, C_p was found to be 3.2. Typically this ranges between 1.8 and 2.2 with values found to range between 0.4 in mountainous regions and 8.0 in extremely flat areas. C_t was found to be 0.8. Typically this ranges between 0.4 and 0.8 (USACE 2000).

Initial and Constant-Rate Loss Method

The initial and constant-rate loss method was used in HEC-HMS to simulate runoff volume. This method assumes a maximum potential rate of precipitation loss, f_c, that is constant throughout an event. Therefore a precipitation value of p_t for a time interval of $t+\Delta t$, the excess runoff volume pe_t is given by:

$$pe_t = \begin{cases} p_t - f_c, & if\ p_t > f_c \\ 0, & otherwise \end{cases}$$

An initial loss, I_a, is also included in the model to represent interception and depression storage. In the May 2006 and April 2007 storms events, no initial loss was used in the final calibration. Table A1 shows the loss rates that were used to calibrate the May 2006 and April 2007 storm. The soil types and areas were determined for each sub-basin using NRCS SURRGO soils data. Then a weighted loss rate was calculated for each sub-basin.

Base Flow Method

The base flow for the Souhegan River Basin was estimated using the exponential recession model where

$$Q_t = Q_0k^t \text{ with}$$

Q_t = the baseflow at anytime t in cfs,

Q_0 = initial value for baseflow in cfs/mi^2,

k = expotential decay constant, and

t = unit time.

The values used for this study are included in Table A2. The same values are used for all 120 subbasins.

Table A1: Precipitation Losses Used in Souhegan River Basin HEC-HMS Model

Hydrologic Soil Group	Description	Typical Range of Loss Rates (in/hr)	Loss Rates for May 2006 Storm	Loss Rates for April 2007 Storm
A	Deep sand, deep loss, aggregated silts	0.30-0.45	0.075	0.00
B	Shallow loess, sandy loam	0.15-0.30	0.038	0.00
C	Clay loams, shallow sandy loam, soils in organic content, and soils usually high in clay	0.05-0.015	0.013	0.00
D	Soils that swell significantly when wet, heavy plastic clays and certain saline soils	0.00-0.05	0.000	0.00

Table A2: Base Flow Values Used in Souhegan River Basin HEC-HMS Model

Storm Event	Initial Discharge (cfs/mi^2)	Recession Constant	Ratio to Peak
May 2006	1	0.9	0.1
April 2007	5	0.9	0.1

Additional analysis was conducted using the NRCS Soil Complex Method as described in NRCS *National Engineering Handbook-4*, although it was determined that Snyder's method provided a better estimate of the storm hydrograph since the NRCS method could not correctly approximate the volume under the hydrograph. Using detailed land use files and NRCS SURRGO Soils data, the overall basin curve number was found to be 64.

C-11.0 ATTACHMENT B: DAM DATA SUMMARY

Table B1: Souhegan River Basin Dams with NHDES HydroCAD Models

NHDES#	Dam Name	Height (ft)	Drainage Area[1] (mi^2)	Maximum Storage[2] (af)	Runoff to fill[3] (in)
7.01	JOE ENGLISH POND DAM	5.5	3.13	101	0.61
7.09	VIJVERHOF POND DAM	9.0	0.67	192	5.37
147.13	CURTIS BROOK DAM	10.0	2.23	3	0.02
147.14	PURGATORY BROOK	6.5	2.55	12	0.09
147.18	PURGATORY BROOK DAM	0.0	2.45	19	0.15
147.22	RECREATION POND	4.0	0.16	3	0.33
147.24	WILDLIFE POND	7.5	0.37	13	0.66
147.26	SOUHEGAN RIVER SITE 28 DAM	29.0	1.1	185	3.16
147.28	SOUHEGAN SITE 8 DAM	25.0	4.7	2721	10.86
147.29	MORISON POND	19.0	0.06	15	4.53
147.31	SWARTZ POND DAM	8.0	0.25	42	3.17
147.33	FARM POND	6.0	0.01	2	3.30
147.38	CURTIS BROOK DAM	12.0	3.5	1	0.01
159.01	RAILROAD POND DAM	12.0	10.58	48	0.09
159.04	OSGOOD POND DAM	9.0	5.24	270	0.97
159.05	HARTSHORN POND DAM	14.9	2.55	40	0.29
159.16	COMPRESSOR POND	24.0	2.25	76	0.64
163.02	CURTIS BROOK DAM	5.0	0.41	126	5.77
163.06	TROW DAM	0.0	1.27	1	0.01
163.07	HARTSHORN BROOK II DAM	8.0	0.22	28	2.39
163.12	ROBY POND DAM	3.5	0.34	3	0.17
167.18	BEAVER DAM POND DAM	5.0	0.58	210	6.79
167.29	GARDNER RESERVOIR DAM	8.0	1.16	17	0.27
175.01	SOUHEGAN SITE 14 DAM	35.0	2.1	885	7.90
175.03	PRATT POND DAM	6.5	0.74	110	2.79
175.19	SOUHEGAN RIVER SITE19 DAM	35.5	11.4	2072	3.41
175.20	SOUHEGAN RIVER SITE 13 DAM	13.5	0.8	249	5.84
175.21	SOUHEGAN RIVER SITE 35 DAM	30.0	6.4	647	7.67
175.23	WHEELER POND DAM	5.0	0.25	23	1.73
254.09	NEW WILTON RESERVOIR DAM	24.0	0.4	335	15.70
254.19	PETERS FARM POND DAM	10.0	0.98	6	0.11
254.20	BATCHELDER POND DAM	12.0	1.2	20	0.31
254.21	FROG POND DAM	15.0	0.6	143	4.45
254.30	SOUHEGAN RIVER SITE 15 DAM	13.0	1.1	315	12.75
254.34	SOUHEGAN RIVER SITE 33 DAM	21.0	1	1078	20.21
254.38	RECREATION POND DAM	8.0	0.4	10	0.48
254.43	CAMP POND DAM	11.0	0.76	33	0.80

Notes for Tables B1, B2, and B3:
[1] Drainage approximated from WISE or dam inspection report, if available.
[2] Maximum storage in this table is extrapolated to estimated storage above dam to also account for overtopping storage.
[3] Runoff to fill is the ratio of maximum storage to drainage area as defined in these tables.

Table B2: Souhegan River Basin Dams with New Hampshire Dam Data Sheets

NHDES#	Dam Name	Height	Drainage Area (mi^2)	Maximum Storage (af)	Runoff to fill (in)
101.01	OTIS FALLS DAM	27.0	29.6	110	0.07
175.09	WATERLOOM POND DAM	22.5	23.1	679	0.55
234.08	SOUHEGAN RIVER SITE 26 DAM	79.0	4.9	1287	4.93
234.11	SOUHEGAN RIVER SITE 12A SOUTH	33.5	5.6	3304	11.06
234.12	SOUHEGAN RIVER SITE 25B DAM	69.0	5.4	1564	5.43
254.01	PINE VALLEY MILL DAM	23.0	97	70	0.01
254.33	SOUHEGAN RIVER SITE 10A DAM	59.0	6.4	2735	8.01

Table B3: Souhegan River Basin Dams with Inspection Report Data Only

NHDES#	Dam Name	Height (ft)	Drainage Area (mi^2)	Maximum Storage (af)	Runoff to fill (in)
020.09	STOWELL POND	8.0	23.2	26	0.02
020.13	MCQUADE BROOK DAM	14.0	7.9	351	0.83
147.17	BURTON POND DAM	14.0	0.5	2	0.09
156.01	MERRIMACK VILLAGE DAM	20.5	171.0	171	0.02
159.02	GOLDMAN DAM	12.0	137.8	114	0.02
159.03	MCLANE DAM	18.7	138.0	39	0.01
167.17	GREENTREE RES DAM	4.5	0.1	17	2.42
234.04	LEIGHTON POND DAM	10.0	1.1	11	0.19
254.02	WILTON HYDRO DAM	17.0	97.0	18	0.00
254.03	SOUHEGAN RIVER III DAM	19.3	70.3	8	0.00
254.05	STONEY BROOK DAM	20.0	33.5	24	0.01
254.08	OLD WILTON RESERVOIR	17.5	8.3	8	0.02
254.11	MILL BROOK	12.0	6.7	15	0.04
254.18	BLOOD BROOK DAM	18.0	6.6	20	0.06
254.32	ERB WILDLIFE POND DAM	20.0	0.3	16	1.07