Appendix A

Hydrology and Hydraulics

US Army Corps
of Engineers
Philadelphia District

Hydrologic and Hydraulic Analysis

Willowemoc Creek, Little Beaver Kill Creek, and Cattail Brook, Sullivan County, NY

January 2016

PREPARED BY:

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EXISTING CONDITION

1. INTRODUCTION

The Willowemoc Creek, Little Beaver Kill Creek, Cattail Brook, and their tributaries, located in Southeastern New York, have been subject to frequent flooding. Recent and historic floods include:

- Hurricane Connie and Diane, August 11 – 20, 1955
- Event of July 27 – 28, 1969
- Rain/snowmelt event of January 19 - 20, 1996
- Tropical Storm Ivan, September 17 – 18, 2004
- Rain/snowmelt event of April 2 – 3, 2005
- Event of June 26 - 29, 2006
- Event of July 30 – August 2, 2009

Each of these events caused extensive destruction throughout New York, Pennsylvania, and New Jersey. Damages were especially prevalent within Sullivan County, NY with significant damage in the Livingston Manor area. In the case of the June 2006 event, a life was lost due to flooding on Cattail Brook.

The residents of Livingston Manor wish to explore ways to reduce the magnitude and frequency of flooding. Existing conditions hydrologic and hydraulic models were developed in order to recreate and understand different flooding events. The effectiveness of various flood reduction alternatives, along with land use changes, could then be assessed in the future.

Throughout this report, references to 0.99, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.004, and 0.002 probabilities of exceedance correspond to 1, 2, 5, 10, 25, 50, 100, 250, and 500 year recurrence intervals, respectively.

2. BACKGROUND

The Willowemoc Creek, Little Beaver Kill Creek, and Cattail Brook all flow through and converge within Livingston Manor, NY. At the confluence, they have drainage areas of approximately 65 square miles (mi^2), 30 mi^2, and 7 mi^2, respectively with a combined drainage area of 102 mi^2.

The area of focus for this study, for hydraulic purposes, extends downstream from the confluence of the three streams approximately 2 miles. The total drainage area at this point is approximately 104 mi^2.

Figure 2.1 is an orthographic image made available through the United States Department of Agriculture (USDA), dated 2006, overlaid with streams and pertinent geographic information that detail the surrounding area. Figure 2.2 is a close-up that details Livingston Manor and the area of interest.

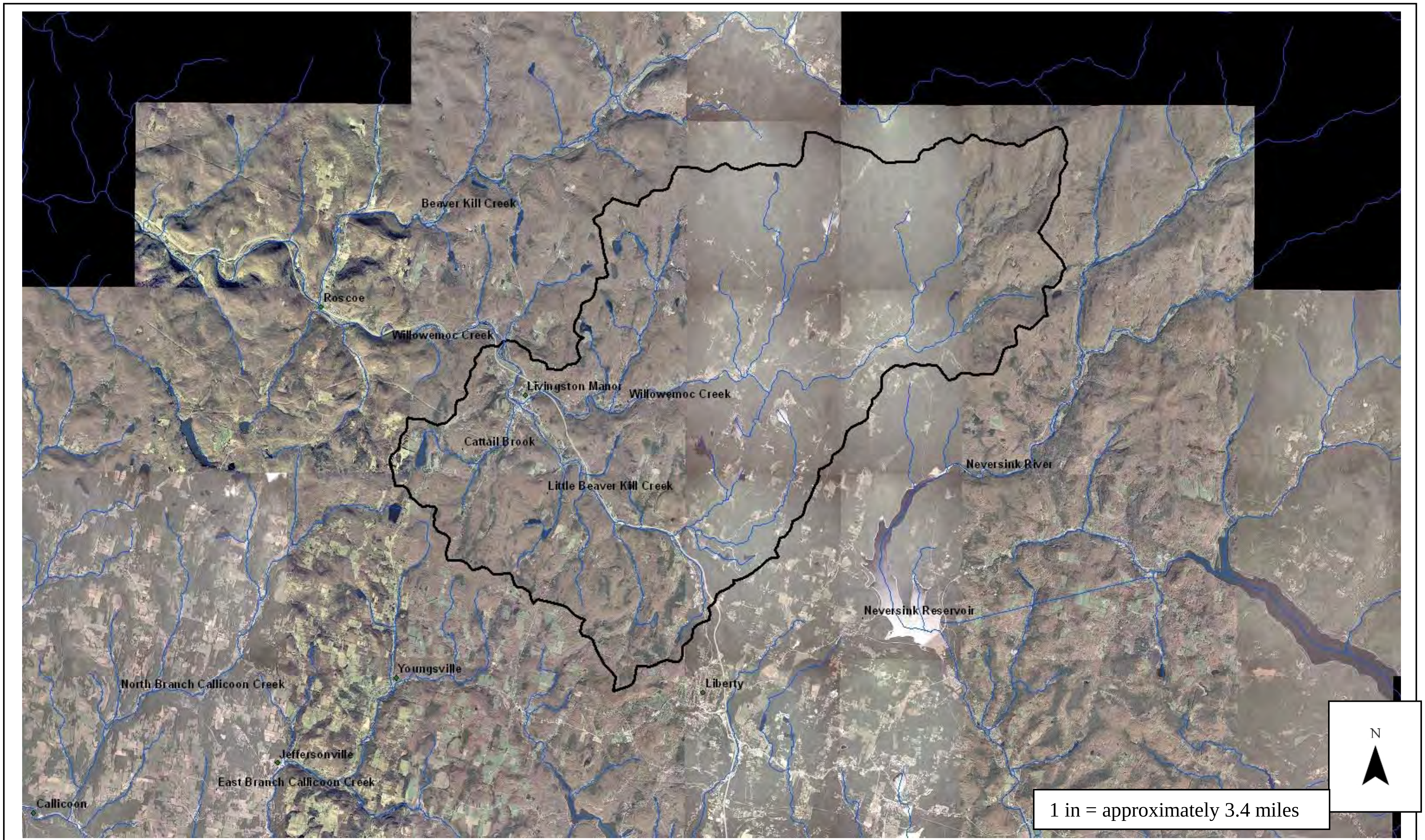


Figure 2.1 Surrounding Area

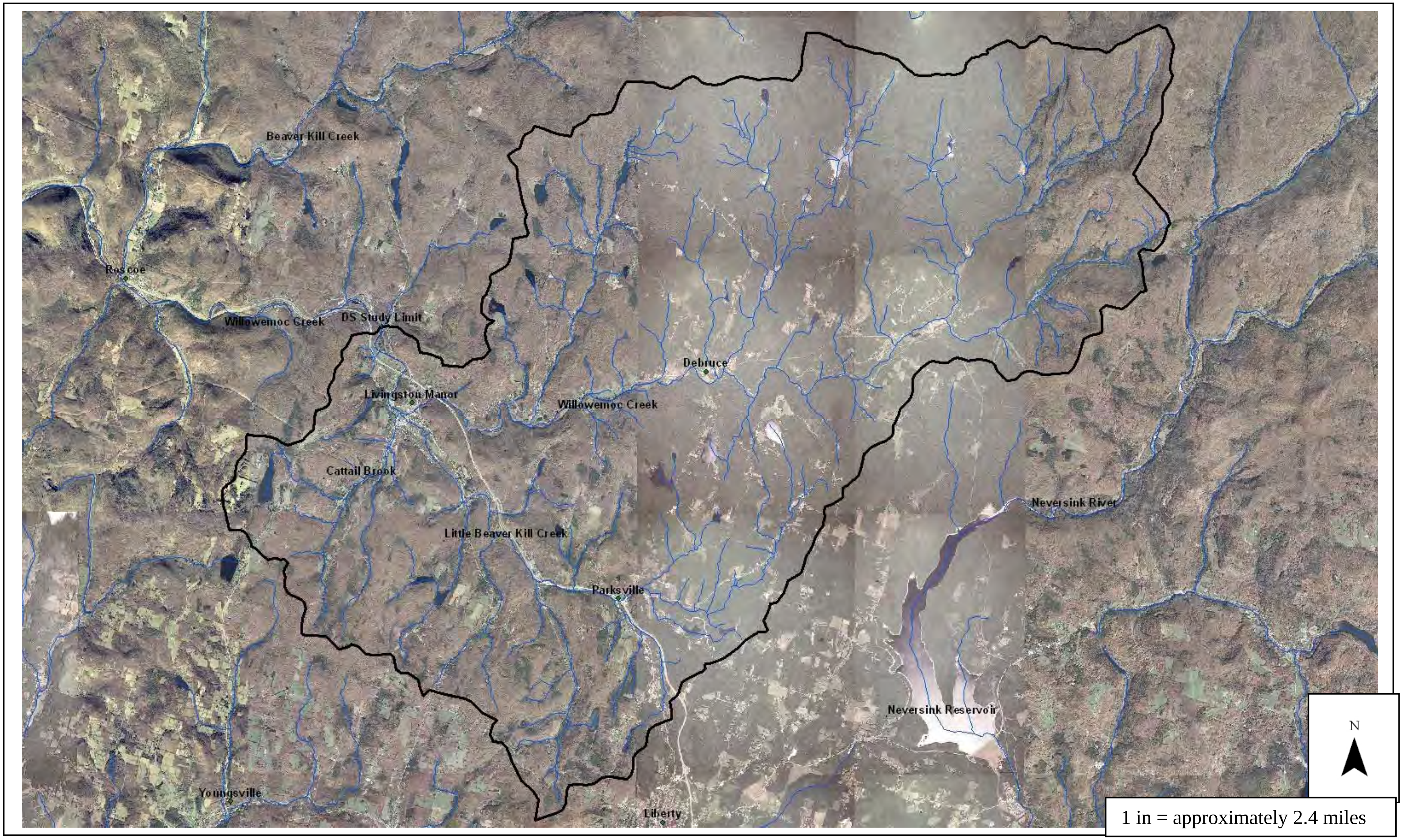


Figure 2.2 Area of interest

3. SPATIAL DATA

This study involved the use of many different types of spatial data. The various data types along with their source, horizontal and vertical datums are shown in Table 3.1. The database of GIS information was managed with ESRI's ARCMAP program, version 9.2. The base layer was the FEMA LIDAR tin and all other data were converted to its horizontal datum. The adjustment to convert ft-NGVD29 to ft-NAVD88 is to subtract 0.49ft from the NGVD elevation.

Table 3.1
Geo-Spatial Data Used in the Study

Data	Organization	Year(s)	Datums	
			Horizontal	Vertical
IMAGERY				
Sullivan County Aerial Photography	NYS GIS	2001	1983 UTM (m) Zone 18N	NA
	USDA	2006	1983 UTM (m) Zone 18N	NA
	USDA	2008	1983 UTM (m) Zone 18N	NA
	USDA	2009	1983 UTM (m) Zone 18N	NA
Quadrangles	USGS	1981-2000	1927 UTM (m) Zone 18N	NGVD (ft) 1929
TOPOGRAPHY				
LIDAR	FEMA	2008	NYSP (ft) 1983	NAVD (ft) 1988
10 meter DEM	USGS	NA	GCS North American1983	NAVD (m) 1988
Field Surveys	USACE	Spring 2010	NYSP (ft) 1983	NAVD (ft) 1988
Channel Surveys	URS	2007	NYSP (ft) 1983	NAVD (ft) 1988
LAND USE & SOIL				
NLCD	MRLC	2001	GCS North American1983	NA
STATSGO	NRCS	2006	GCS North American1983	NA
PRECIPITATION				
NCDC Rain Gages	NOAA	NA	GCS North American1983	NA
HDSG MPE	NOAA	NA	Albers Equal Area Conic	NA
REAL ESTATE				
Structure Inventory	M. Baker	2007	NY State Plane NAD27	NGVD (ft) 1929
Bought Out Properties	Sullivan County (proposed)	2009	NYSP (ft) 1983	NA
	FEMA (completed)	2008	NYSP (ft) 1983	NA
STREAM GAGES				
Little Beaver Kill	USGS	1925-1981 2004	NYSP (ft) 1983	NGVD (ft) 1929
Willowemoc	USGS	1938-1973 2004	NYSP (ft) 1983	NGVD (ft) 1929
Beaver Kill	USGS	1914-2008	NYSP (ft) 1983	NGVD (ft) 1929
HIGH WATER MARKS				
Little Beaver Kill	USGS	2004, 2006	NY E. State Plane NAD83	NGVD , NAVD
Willowemoc	USGS	2004, 2006	NY E. State Plane NAD83	NGVD , NAVD

4. DISCHARGE-FREQUENCY ANALYSIS

The exceedance frequency discharges** for the Willowemoc and the Little Beaver Kill are based on a statistical analysis of Gage 01419500, Willowemoc Creek near Livingston Manor, NY and Gage 01420000, Little Beaver Kill near Livingston Manor, NY. The analysis followed the procedures of "Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Subcommittee", March 1982.

The Willowemoc and the Little Beaver Kill gages were discontinued after water year 1974 and 1981 respectively. Since the region has recently been subject to a series of large events (in 1996, 2004, 2005 and 2006) it was necessary to extend the records of the two gages using the two station comparison procedures of Appendix 7 of Bulletin 17B.

The exceedance frequency discharges for Cattail Brook are based on a Log Discharge vs. Log Drainage Plot involving three gages: Willowemoc, Little Beaver Kill and Beaver Kill. The locations of the three gages relative to Livingston Manor are shown in Figure 4.1and pertinent data for the gages is provided in Table 4.1.

** Exceedance frequency discharge is the percent chance of obtaining in a year a given discharge or greater.

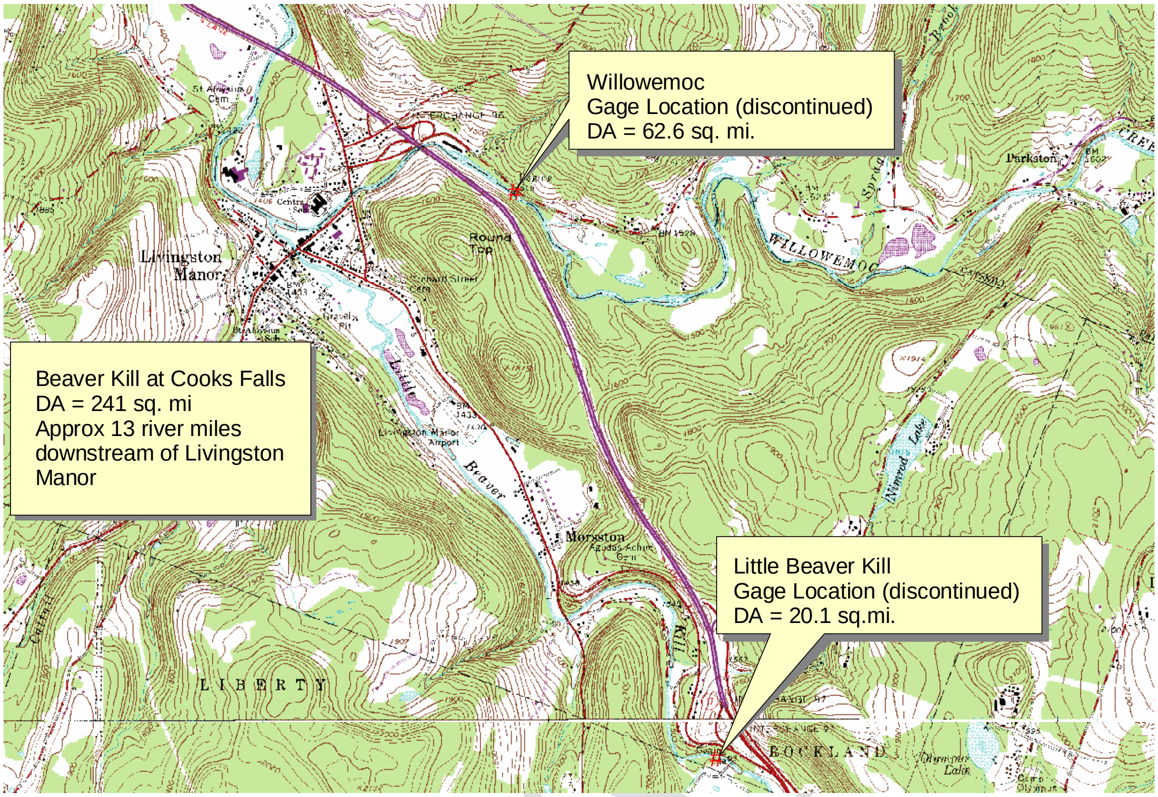


Figure 4.1 USGS Stream Gages Surrounding Livingston Manor

Table 4.1
Pertinent Data for USGS Stream Gages

Gage	Drainage Area (sq mi)	Period of Record	Vertical Datum (ft-NGVD29)
01419500 Willowemoc	62.6	08-11-1938 12-21-1973	1435.85
01420000 Little Beaver Kill	20.1	02-12-1925 05-12-1981	1496.69
1420500 Beaver Kill	241	03-28-1914 07-23-2008	1151.70

The systematic gage records for Willowemoc and Little Beaver Kill are provided on Table 4.2. Dates when the annual peak flows for the two gages coincide are flagged.

Table 4.2
Systematic Gage Records of Willowemoc Creek near Livingston Manor and Little Beaver Kill near Livingston Manor

	Willowemoc		Little Beaver Kill		Common Peak Event for WY
Water Year	Date	Discharge (cfs)	Date	Discharge (cfs)	
1925			02-12-1925	570	
1926			04-09-1926	515	
1927			11-16-1926	1980	
1928			08-26-1928	3420	
1929			03-14-1929	930	
1930			06-10-1930	1630	
1931			07-10-1931	1130	
1931			01-07-1932	750	
1933			08-24-1933	3180	
1934			09-29-1934	1180	
1935			12-01-1934	1430	
1936			03-18-1936	3120	
1937			02-22-1937	3060	
1938	08-11-1938	6200	09-21-1938	2070	
1939	12-06-1938	2720	12-06-1938	1120	X
1940	04-8-1940	3300	04-08-1940	1240	X
1941	12-29-1940	2100	12-29-1940	720	X
1942	09-27-1942	5720	09-27-1942	1630	X
1943	12-30-1942	3090	12-30-1942	1040	X
1944	11-09-1943	3280	11-09-1943	1260	X
1945	03-17-1945	2870	07-29-1945	1080	
1946	03-09-1946	2720	09-24-1946	1140	
1947	04-05-1947	2360	04-05-1947	1230	X
1948	03-22-1948	3490	11-08-1947	1010	
1949	12-30-1948	4490	12-30-1948	1670	X
1950	04-04-1950	2560	04-04-1950	884	X
1951	03-31-1951	10500	11-25-1950	2510	
1952	07-10-1952	4060	07-10-1952	1500	X
1953	12-11-1952	4560	12-11-1952	1450	X
1954	12-07-1953	1500	12-07-1953	540	X
1955	08-19-1955	4300	08-18-1955	2270	X
1956	10-15-1955	3850	10-15-1955	1580	X
1957	01-23-1957	1700	11-02-1956	601	
1958	12-21-1957	8280	12-20-1957	2490	X
1959	04-02-1959	3010	04-02-1959	891	X
1960	11-28-1959	3960	04-04-1960	1210	

Table 4.2 (continued)
Systematic Gage Records of Willowemoc Creek near Livingston Manor and Little Beaver Kill near Livingston Manor

	Willowemoc		Little Beaver Kill		Common Peak Event for WY
Water Year	Date	Discharge (cfs)	Date	Discharge (cfs)	
1961	04-25-1961	3520	04-25-1961	1120	X
1962	04-01-1962	3160	04-07-1962	896	
1963	03-27-1963	1860	03-27-1963	705	X
1964	03-10-1964	2610	03-10-1964	753	X
1965	02-08-1965	1450	02-08-1965	466	X
1966	03-25-1966	1200	06-10-1966	596	
1967	04-03-1967	1660	04-02-1967	500	X
1968	04-25-1968	3460	04-24-1968	1140	X
1969	07-28-1969	15700	07-28-1969	2780	X
1970	04-02-1970	1510	04-02-1970	637	X
1971	10-23-1970	5920	10-23-1970	1560	X
1972	03-22-1972	3190	06-22-1972	1060	
1973	06-29-1973	4380	06-29-1973	2120	X
1974	12- 21-1973	6140	12-21-1973	1790	X
1975			07-20-1975	2040	
1976			01-27-1976	3230	
1977			03-13-1977	2440	
1978			01-09-1978	2200	
1979			09-06-1979	1180	
1980			03-21-1980	2000	
1981			05-12-1981	2000	

A statistical analysis was performed for both gages using the program HEC-FFA and the results are found in Table 4.3. The station skew was weighted with the general skew. A general skew value of 0.0870 was used as taken from a recent study completed by the Hydrologic Engineering Center, "Delaware River Basin Regional Skew Analysis", September 2009. The average prediction error is 0.026.

Table 4.3 Discharge-Frequency based on Systematic Record for Willowemoc and Little Beaver Kill			
		Median Discharge (cfs)	
Exceedance Frequency	Event	Willowemoc	Little Beaver Kill
99	1.01	1010	380
50	2	3250	1300
20	5	5150	2050
10	10	6590	2600
4	25	8630	3380
2	50	10300	4000
1	100	12100	4660
0.4	250	14700	5600
0.2	500	16900	6360

For the Willowemoc, the 1969 peak flow was treated as a high outlier over the period WY 1938 to WY 1996. Examination of the Beaver Kill at Cooks Falls gage record (found below) indicates that there were no large events between 1973 and 1996 at Cooks Falls and presumably no large events between 1973 and 1996 at Willowemoc near Livingston Manor.

Because of recent large floods along the Willowemoc and the Little Beaver Kill, it is likely that a discharge-frequency based on the discontinued gage records is not reflective of the true flood potential of these waterways.

As a sensitivity of gage record length on discharge-frequency estimates, the long term Beaver Kill gage at Cooks Falls (whose record is provided on Table 4.4) was analyzed under five assumptions of record length:

Full record length: WY1914 – WY2008
Record length: WY1938 – WY2008
Record length: WY1938 – WY1973
Record length: WY1925 – WY2008
Record length: WY1925 – WY1981

Water years 1938 and 1973 are the initial and final data years for the Willowemoc gage while water years 1925 and 1981 are the initial and final data years for Little Beaver Kill gage. These record lengths were chosen to show how the discharge-frequency estimates at Cooks Falls vary as a function of the period of record. And since the sensitivity analysis includes the Willowemoc and Little Beaver Kill periods of record, the variation in results at Cooks Falls should be indicative of the expected change in the discharge-frequency of Willowemoc and Little Beaver Kill if their discontinued records were extended.

A statistical analysis was performed for the five record lengths and the discharge-frequency results are provided in Table 4.5

Table 4.4

Systematic Gage Records of Beaver Kill River at Cooks Falls

Water Year	Date	Flow	Water Year	Date	Flow	Water Year	Date	Flow
1914	03-28-1914	7770	1952	04-05-1952	11600	1990	10-20-1989	13300
1915	02-25-1915	6240	1953	12-11-1952	17700	1991	11-10-1990	19600
1916	07-26-1916	5880	1954	12-07-1953	6600	1992	11-23-1991	10500
1917	03-27-1917	7870	1955	08-19-1955	14300	1993	04-11-1993	11900
1918	10-30-1917	9700	1956	10-16-1955	12300	1994	04-13-1994	13900
1919	07-22-1919	3720	1957	01-23-1957	5210	1995	03-08-1995	5480
1920	03-13-1920	5000	1958	12-21-1957	28900	1996	01-19-1996	42900
1921	03-09-1921	5530	1959	04-02-1959	10900	1997	11-09-1996	26600
1922	11-28-1921	7740	1960	11-28-1959	14200	1998	01-08-1998	11900
1923	04-06-1923	7480	1961	04-25-1961	11000	1999	01-24-1999	16600
1924	09-30-1924	13400	1962	04-01-1962	11700	2000	03-12-2000	6600
1925	02-12-1925	7180	1963	03-27-1963	8540	2001	12-17-2000	34400
1926	11-16-1925	4730	1964	03-10-1964	10200	2002	06-07-2002	4900
1927	11-16-1926	14600	1965	02-08-1965	4960	2003	09-04-2003	13200
1928	12-08-1927	7560	1966	03-25-1966	4200	2004	09-18-2004	42100
1929	03-15-1929	6340	1967	04-02-1967	6580	2005	04-03-2005	50800
1930	06-10-1930	4130	1968	04-25-1968	10200	2006	06-28-2006	62400
1931	07-11-1931	5060	1969	07-28-1969	27500	2007	04-16-2007	13900
1932	04-01-1932	4130	1970	04-02-1970	6000	2008	07-23-2008	17000
1933	08-24-1933	19000	1971	10-23-1970	15500			
1934	09-29-1934	4580	1972	04-20-1972	8820			
1935	12-01-1934	11000	1973	06-29-1973	14100			
1936	03-18-1936	21300	1974	12-21-1973	18700			
1937	02-22-1937	15300	1975	12-08-1974	17200			
1938	08-11-1938	19600	1976	01-27-1976	14900			
1939	12-06-1938	9040	1977	03-14-1977	28400			
1940	03-31-1940	11500	1978	01-09-1978	20300			
1941	12-29-1940	7010	1979	01-02-1979	10800			
1942	09-27-1942	20300	1980	03-21-1980	23800			
1943	12-30-1942	10200	1981	02-20-1981	16400			
1944	11-09-1943	10900	1982	04-18-1982	12100			
1945	07-19-1945	9140	1983	04-16-1983	14400			
1946	03-09-1946	9650	1984	04-05-1984	15300			
1947	04-05-1947	9000	1985	09-27-1985	10100			
1948	03-22-1948	15100	1986	03-15-1986	19800			
1949	12-30-1948	16300	1987	04-04-1987	23900			
1950	04-04-1950	8870	1988	03-26-1988	11300			
1951	03-31-1951	31600	1989	05-06-1989	11700			

Table 4.5
Discharge-Frequency based on Various Periods of Record for
Beaver Kill River at Cooks Falls

Exceedance Frequency	Event	Median Discharge (cfs)				
		WY 1914 - WY 2008	WY 1938 - WY 2008	WY 1938 - WY 1974	WY 1925 - WY 2008	WY 1925 - WY 1981
99	1.01	3100	3920	3860	3330	3250
50	2	11500	13400	11200	12400	11000
20	5	19300	21600	16800	20500	17400
10	10	25400	28000	20900	27000	22100
4	25	34400	37200	26500	36300	28600
2	50	42000	44800	30800	44100	33800
1	100	50400	53000	35500	52600	39300
0.4	250	62900	65200	42000	65100	47100
0.2	500	73400	75300	47200	75600	53500

Comparing WY1938-WY2008 to WY1938-WY1973 and WY1925-WY2008 to WY1925-WY1981 shows large differences for the less frequent events. At the Beaver Kill gage the additional years of record with the recent large floods has a dramatic effect on the discharge-frequency relationship and is indicative that for the Willowemoc and the Little Beaver Kill gages the systematic record is unacceptable as the basis of the frequency analysis. The two gages must be extended. The Beaver Kill at Cooks Falls is the long term gage for the two station extension.

Figures 4.2 and 4.3 are regression plots of the Little Beaver Kill and the Willowemoc annual flow record against the corresponding flow record of Beaver Kill at Cooks Falls. The Little Beaver Kill and Willowemoc correlation coefficients are 0.7455 and 0.9602 respectively.

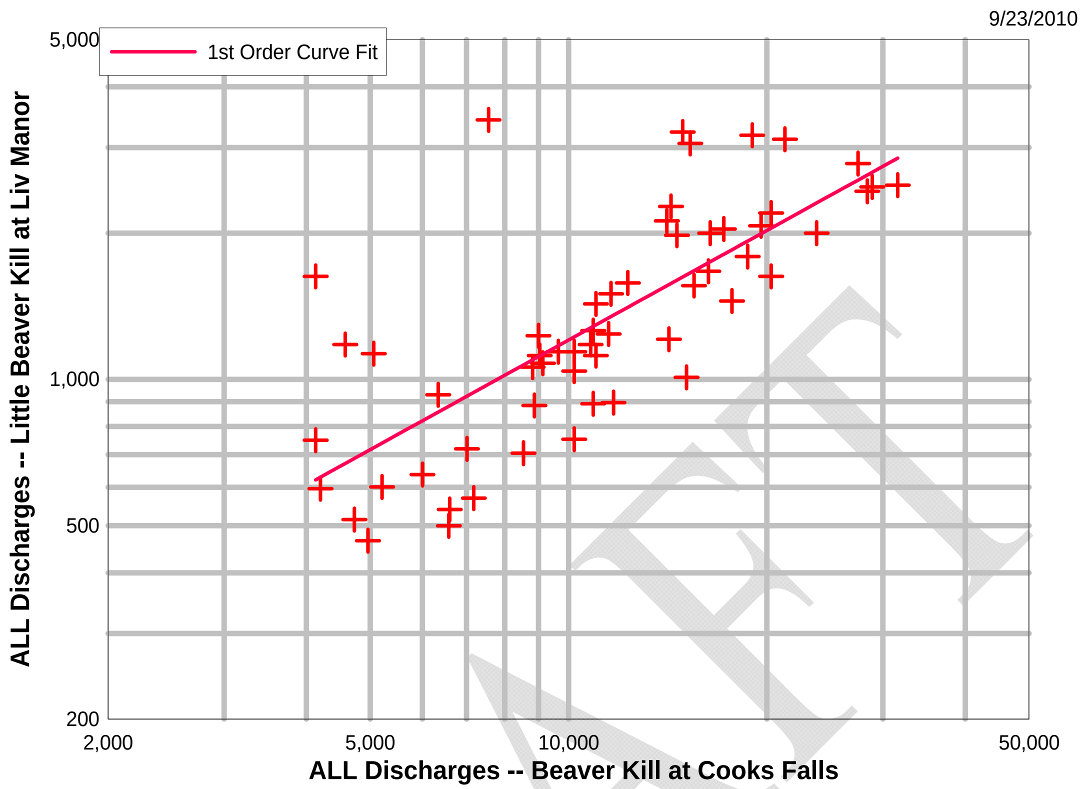


Figure 4.2 Regression Plot of Annual Flow --- Little Beaver Kill vs. Beaver Kill

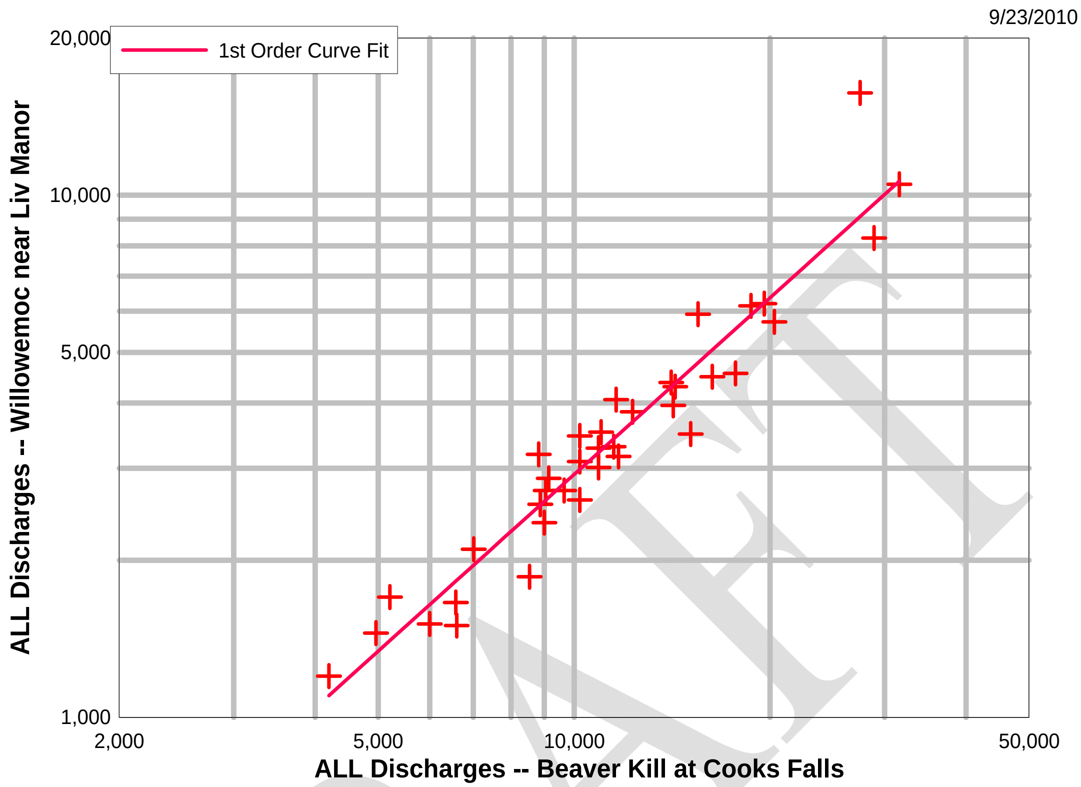


Figure 4.3 Regression Plot of Annual Flow -- Willowemoc vs. Beaver Kill

The two station extension was performed with a spreadsheet that programmed the equations of Appendix 7 of Bulletin 17B. The results are provided in Table 4.6.

Table 4.6
Adjusted Log Pearson III Statistics Based on Two Station Extension

LPIII Parameters	Willowemoc		Little Beaver Kill	
	Two Sta. Extension	Systematic Record	Two Sta. Extension	Systematic Record
Mean	3.5332	3.5179	3.1319	3.1148
Standard Deviation	0.2803	0.2323	0.2497	0.2339
Years of Record	86	37 systematic 59 historic	73	57 systematic 0 historic
General Skew	0.0870	0.0870	0.0870	0.0870
Station Skew	NA	0.4063	NA	-0.0622
Adopted Skew	0.1460	0.1460	0.0549	0.0549

The adjusted log Pearson III parameters for the Willowemoc and the Little Beaver Kill were input to the HEC-FFA program and the discharge-frequency relationship was generated. The results are shown in Table 4.7; for ease of comparison the systematic results are reproduced in the table. The adjusted discharge-frequency curve for the Willowemoc and Little Beaver Kill are plotted on Figures 4.4 and 4.5 respectively.

Table 4.7
Discharge-Frequency based on Two Station Extension for Willowemoc and Little Beaver Kill

Exceedance Frequency	Event	Median Discharge (cfs)			
		Willowemoc		Little Beaver Kill	
		Two Sta. Extension	Systematic Record	Two Sta. Extension	Systematic Record
99	1.01	815	1010	364	380
50	2	3360	3250	1350	1300
20	5	5850	5150	2190	2050
10	10	7880	6590	2840	2600
4	25	10900	8630	3750	3380
2	50	13500	10300	4490	4000
1	100	16400	12100	5280	4660
0.4	250	20800	14700	6430	5600
0.2	500	24500	16900	7370	6360

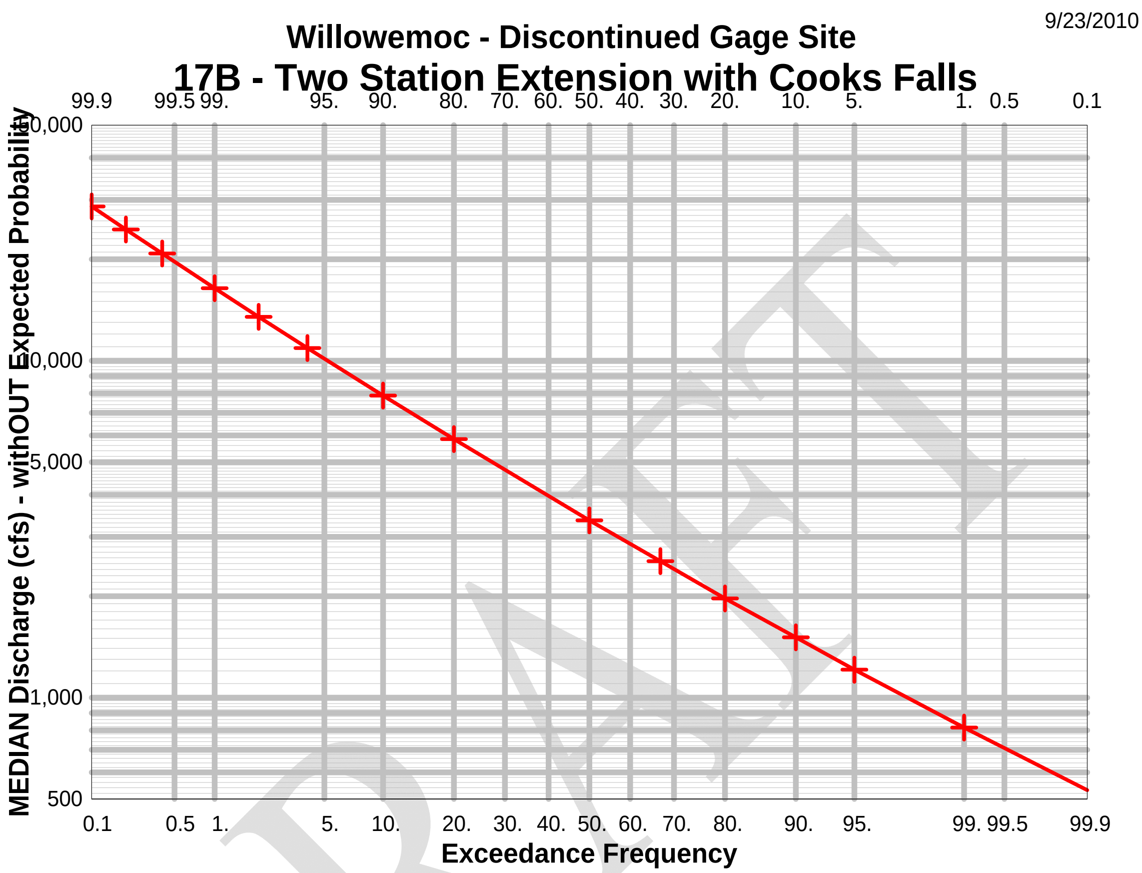


Figure 4.4 Willowemoc Log Q-Exceedance Plot based on Two Station Extension with Cooks Falls

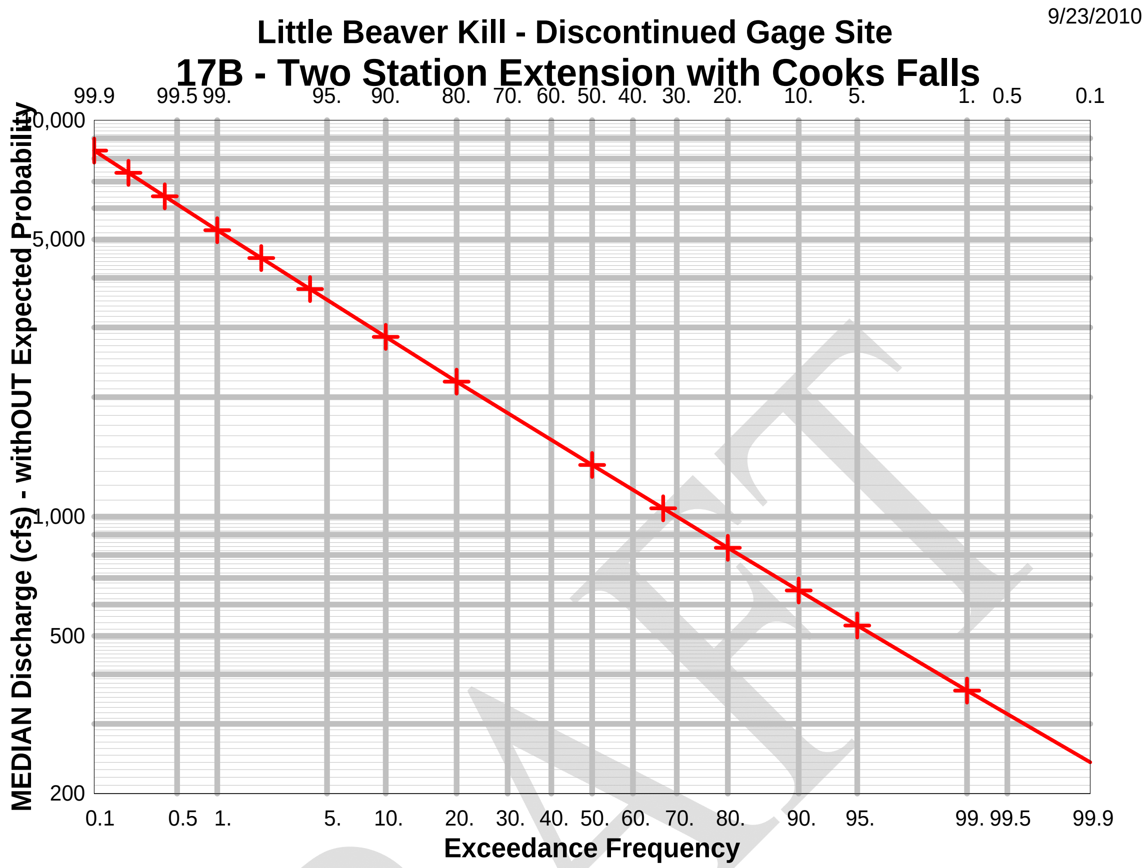


Figure 4.5 Little Beaver Kill Log Q-Exceedance Plot based on Two Station Extension with Cooks Falls

The discharge-frequency curve for Cattail Brook at its mouth (DA=7.43 sq mi) is based on a Log Discharge – Log Drainage Area plot involving the drainage areas of 20.1 sq mi (Little Beaver Kill), 62.6 sq mi (Willowemoc) and 241 sq mi (Beaver Kill at Cooks Falls). The Log Discharge – Log Drainage plot is provided as Figure 4.6. The Cattail Brook extrapolated discharge-frequency relationship along with the best estimates for Willowemoc, Little Beaver Kill and Beaver Kill data are provided in Table 4.8.

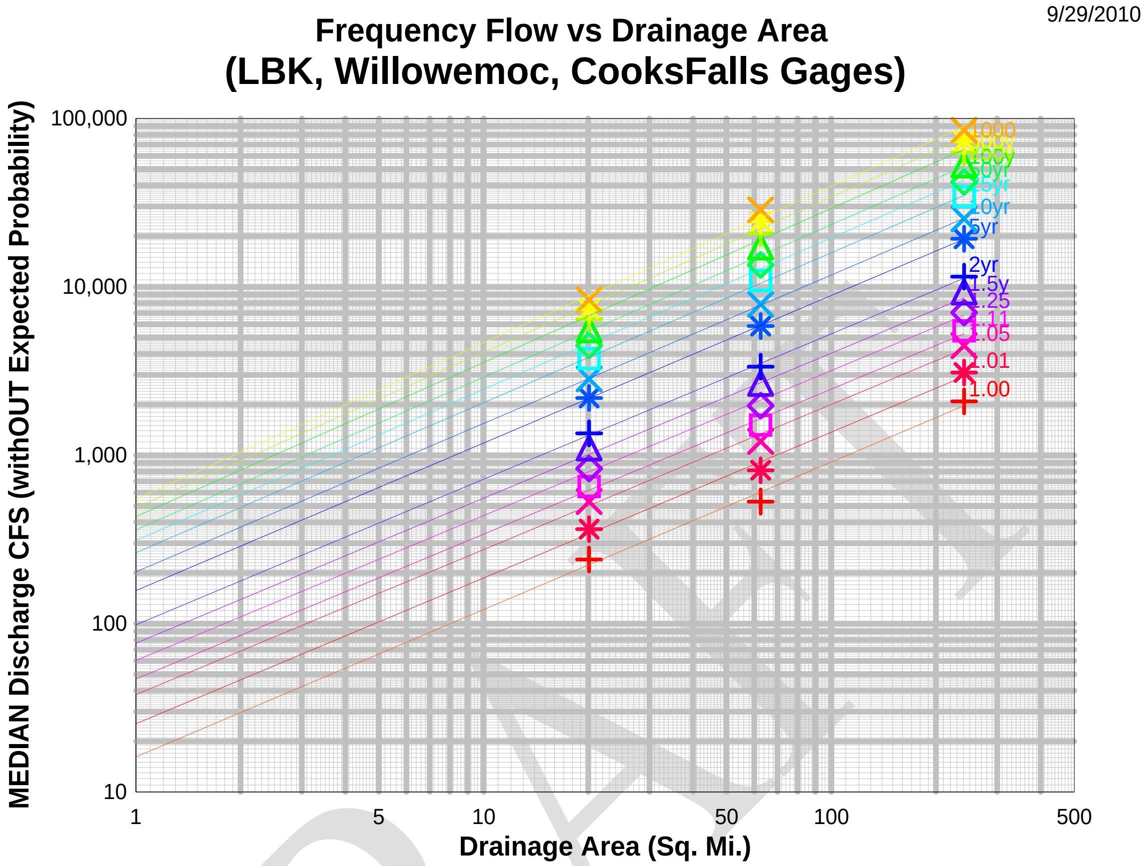


Figure 4.6 Log Q- Log DA Plot to Estimate Discharge-Frequency for Cattail Brook

Table 4.8 **Final Discharge-Frequency at Primary Sites**					
Exceedance Frequency	Event	Median Discharge (cfs)			
		Willowemoc	Little Beaver Kill	Cattail Brook	Beaver Kill
		DA=62.6 sq mi	DA=20.1 sq mi	DA=7.43 sq mi	DA=241 sq mi
99	1.01	815	364	144	3100
50	2	3360	1350	558	11500
20	5	5850	2190	911	19300
10	10	7880	2840	1189	25400
4	25	10900	3750	1575	34400
2	50	13500	4490	1889	42000
1	100	16400	5280	2223	50400
0.4	250	20800	6430	2712	62900
0.2	500	24500	7370	3109	73400

Note: Beaver Kill is at the active Cooks Falls gage site, 01420500.
Willowemoc and Little Beaver Kill are at discontinued gage sites 01419500 and 01420000, respectively.
Cattail Brook is at the confluence with the Willowemoc.

In the study area major changes in drainage area for the Willowemoc, Little Beaver Kill and Cattail Brook were determined and are shown on Figures 4.7, 4.8 and 4.9 respectively. These locations were used as the flow change locations in the hydraulic model.

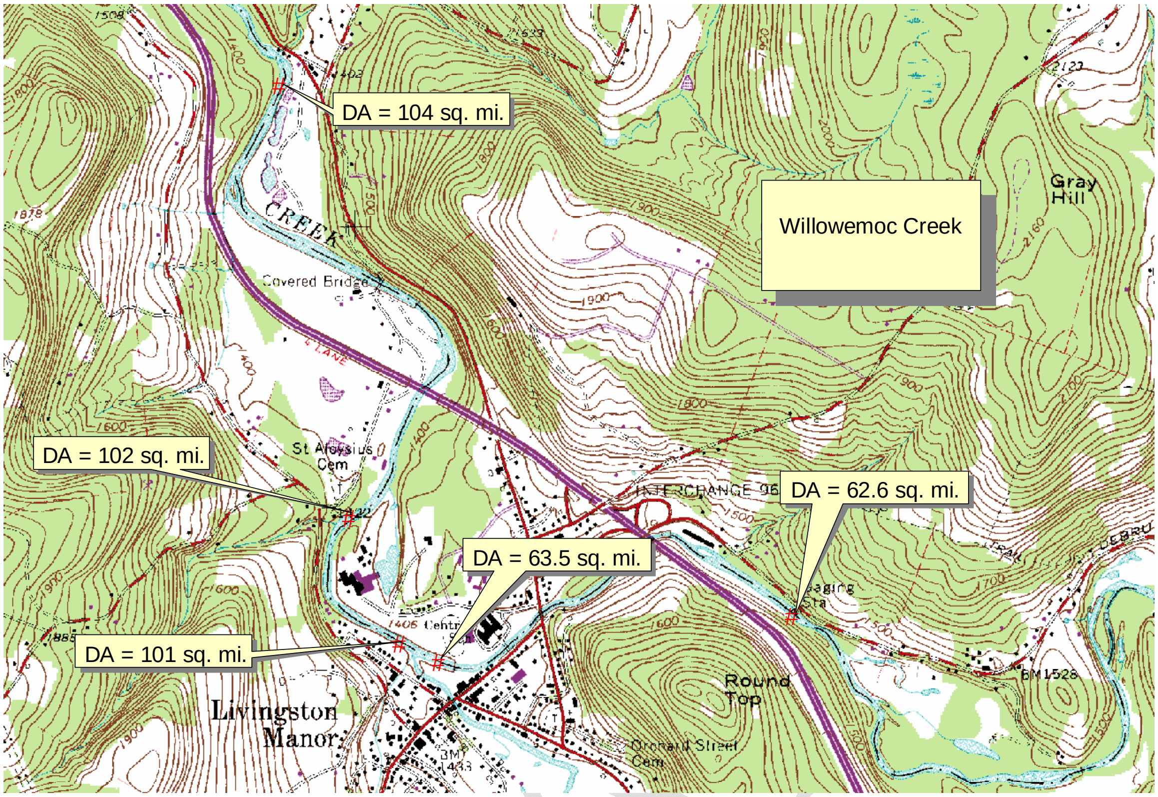


Figure 4.7 Willowemoc – Drainage Areas along Creek

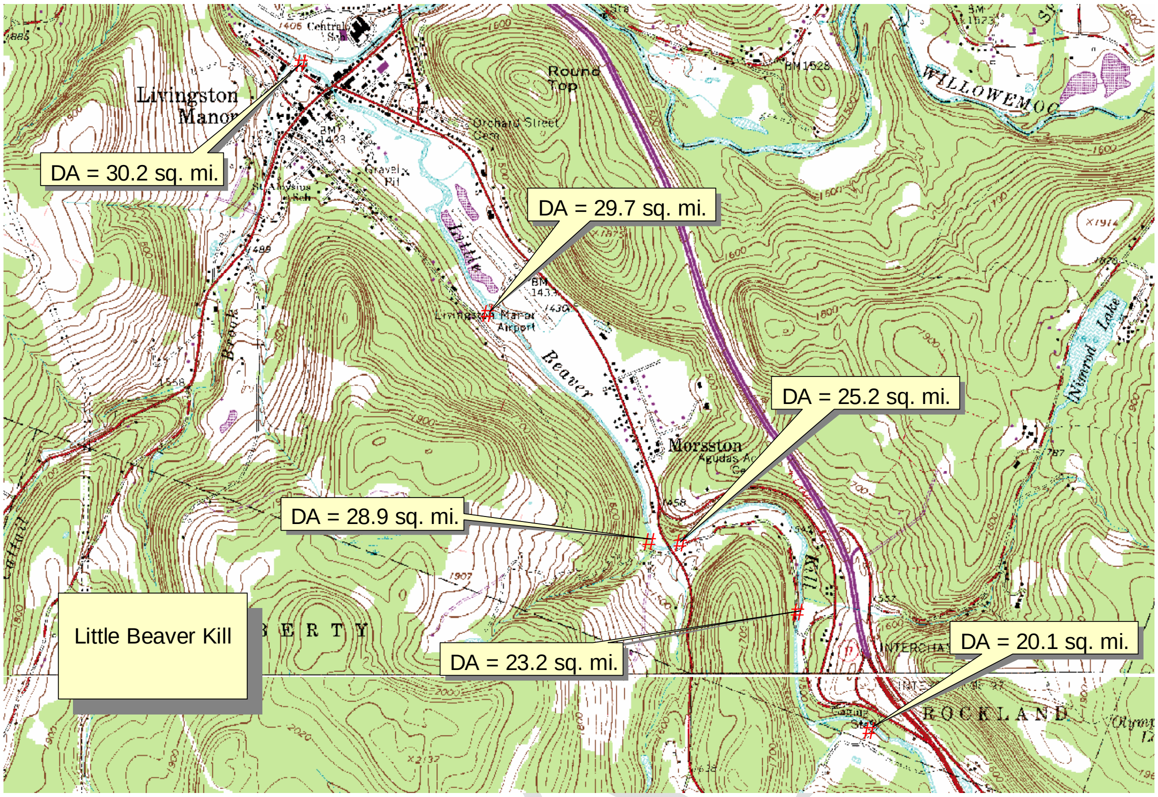


Figure 4.8 Little Beaver Kill – Drainage Areas along Creek

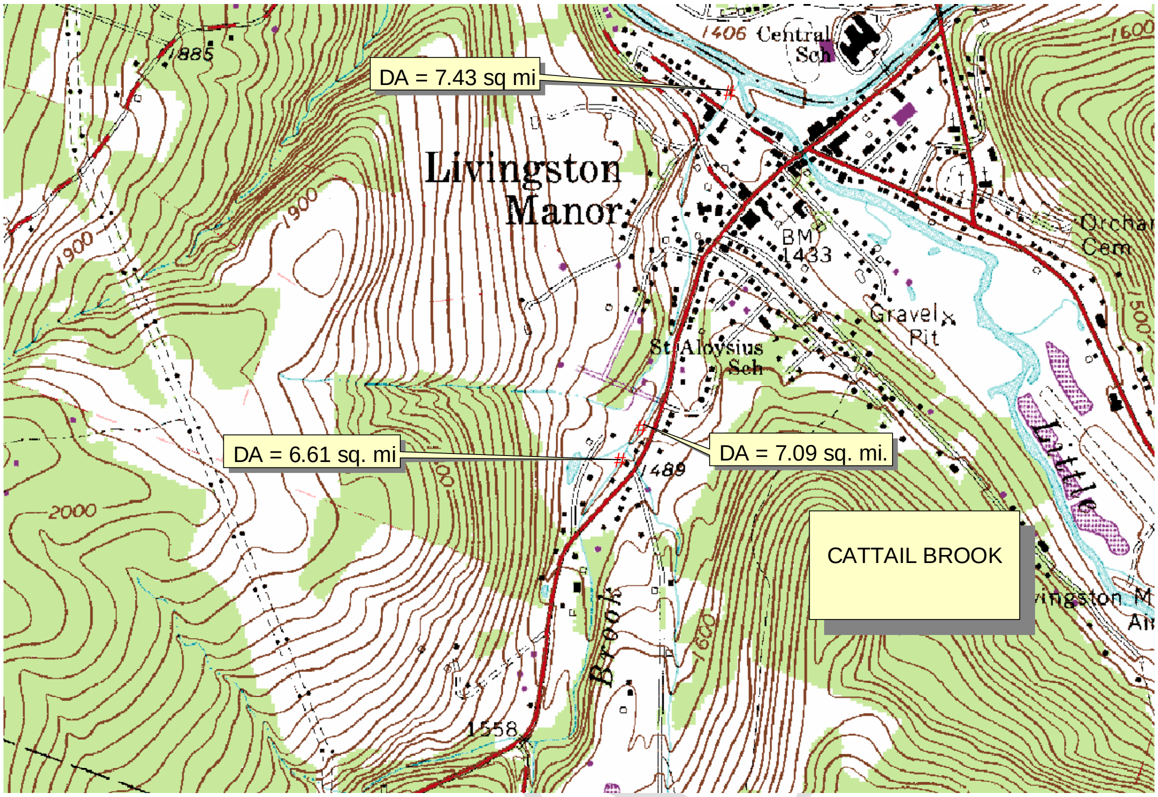


Figure 4.9 Cattail Brook – Drainage Areas along Creek

The discharge-frequency relationship at the required flow change locations were determined by multiplying the discharge-frequency relationship at the primary sites by an adjustment factor. The adjustment factor was calculated using a USGS regional regression relationship documented in the report, "Magnitudes and Frequency of Floods in New York", 2006, SIR2006-5112. The regional regression equation was applied to the primary sites and to the flow change locations and the ratios of the regional frequency flows were the adjustment factors by which the final discharges at the primary site were multiplied. The various ratios used are provided in Table 4.9. For a given location the ratio is constant across the frequency range. StreamStats, a web based GIS implementation of the equations of SIR2006-5112, was used to determine the regional regression discharges. (http://water.usgs.gov/osw/streamstats/new_york.html).

Table 4.9 Ratios to Adjust Primary Discharge-Frequency to Discharge-Frequency at Flow Change Locations	
Willowemoc	
Drainage Area at Flow Changes (sq. mi.)	Ratio to adjust Median Q-Freq at DA=62.6 sq mi
63.5	1.01
102	1.48
104	1.50
Little Beaver Kill	
Drainage Area at Flow Changes (sq. mi.)	Ratio to adjust Median Q-Freq at DA=20.1 sq mi
29.7	1.38
30.2	1.40
Cattail Brook	
Drainage Area at Flow Changes (sq. mi.)	Ratio to adjust Median Q-Freq at DA=7.43 sq mi
6.61	0.89
7.09	0.95

The discharge-frequency relationships at all flow change locations are provided in Table 4.10.

Table 4.10 Discharge-Frequency at Various Drainage Areas for the Willowemoc, Little Beaver Kill and Cattail Brook										
Exceedance Frequency	Event	Median Discharge (cfs)								
		Willowemoc				Little Beaver Kill		Cattail Brook		
		62.6 sq mi	63.5 sq mi	102 sq mi	104 sq mi	29.7 sq mi	30.2 sq mi	6.61 sq mi	7.09 sq mi	7.43 sq mi
99	1.01	815	823	1206	1223	502	510	128	137	144
50	2	3360	3394	4973	5040	1863	1890	497	530	558
20	5	5850	5909	8658	8775	3022	3066	811	865	911
10	10	7880	7959	11662	11820	3919	3976	1058	1130	1189
4	25	10900	11009	16132	16350	5175	5250	1402	1496	1575
2	50	13500	13635	19980	20250	6196	6286	1681	1795	1889
1	100	16400	16564	24272	24600	7286	7392	1978	2112	2223
0.4	250	20800	21008	30784	31200	8873	9002	2414	2576	2712
0.2	500	24500	24745	36260	36750	10171	10318	2767	2954	3109

The existing frequency discharges for existing condition are also appropriate for future conditions. The level of imperviousness is very low for the three stream gages and is unlikely to change in the future. (Per StreamStats the percent imperviousness at the gages is: Little Beaver Kill – 1.2; Willowemoc – 0.1; Beaver Kill – 0.3.) Development potential is limited but if it were to occur frequency flows will not increase due to New York State's storm water management regulations.

A large number of previous studies have published discharge-frequency relationships for the discontinued Willowemoc and Little Beaver Kill gages. Those prior frequency discharges are tabulated in Tables 4.11 and 4.12.

Table 4.11
Previous Estimates of Discharge-Frequency at the Discontinued Willowemoc Creek Gage Site

	DA	1yr	2yr	5yr	10yr	20yr	25yr	50yr	100yr	200yr	250yr	500yr
USACE Recon 1970	None											
USACE Recon 1979	64.0			5370	7140			12400	15300			24400
SPM 83-1 – Skew Study	62.6		3260		6960			11500	13900	16500		20500
USGS Lumia 1991 WRI 90-4197	62.6		3270	5420	7230		9920	12200	14600			21200
1993 FEMA FIS	64.0				7330			12900	16000			24000
USGS Skew Study WRIU 00-4022	None											
USGS Regression Equations, SIR2006-5112	62.6	2160	3300	5370	7080		9560	11700	13800	16100		19100
Prelim May 2009 FEMA Flood Recovery Mapping	62.5				7289			11917	14103			19396
HEC Skew Study, PR70 Sep 2009	None tabulated											

Table 4.12
Previous Estimates of Discharge-Frequency at the Discontinued Little Beaver Kill Gage Site

	DA	1yr	2yr	5yr	10yr	20yr	25yr	50yr	100yr	200yr	250yr	500yr
USACE Recon 1970	None											
USACE Recon 1979	27.0			2550	3300			5400	6500			9650
SPM 83-1 – Skew Study	20.1		1300		2600			3940	4560	5210		6140
USGS Lumia 1991 WRI 90-4197	20.1		1270	2000	2530		3290	3900	4550			6250
1993 FEMA FIS	19.8				2450			3985	4780			6800
USGS Skew Study WRIU 00-4022	None											
USGS Regression Equations SIR2006-5112	20.1	1020	1290	2030	2570		3300	3910	4540	5210		6110
Prelim May 2009 FEMA Flood Recovery Mapping	20.4				2576			3936	4576			6187
HEC Skew Study, PR70 Sep 2009	None tabulated											

A. Climate Uncertainty

USACE guidance requires consideration of changing hydrology through time, specifically possible changes in a project's function if the quantity and the timing of the runoff from the watershed changes in the future. There are two driving forces for such changes: changing landscape (e.g. urbanization) and changing climate. Technically, the issue of changing landscape is described with the term, homogeneity (or lack thereof) and the issue of climate uncertainty or climate change is described with the term, stationarity (or lack thereof).

USACE guidance on the issue of homogeneity has long existed. Engineering Regulation (ER) 1110-2-1450, Hydrologic Frequency Estimates, mandates that the issue of homogeneity be addressed in statistical analysis of gage records and Engineering Manuals (EM) 1110-2-1415, Hydrologic Frequency Analysis and 1110-2-1417 Flood-Runoff Analysis provide techniques for handling non-homogeneity. The homogeneity of the Little Beaver Kill and Willowemoc watersheds was considered and is documented above. Given the very low, present level of imperviousness, the landscape has not materially changed since the USGS gages were first established. It is unlikely that any future development will manifest as non-homogenous stream flows.

In the past, because of lack of knowledge and data, the issue of stationarity was not considered. That is, the climate was assumed stable and future storms were assumed to be of the same type and same magnitude of past storms. However, due to advances in technology, stationarity is now being addressed by the USACE as reflected in, Engineering Construction Bulletin (ECB) 2014-10, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects.

ECB 2014-10 is concerned about climate change because of its direct effect on the hydrology of a watershed. For example, changes in temperature, precipitation and other climate variables such as wind patterns can affect the rainfall over a watershed (rain vs. snow; changes in annual rain total, changes in temporal distribution etc…) such that the base and flood flows of a stream may change. Although the USACE is working on a series of future guidance documents to support a quantitative assessment of climate change, at this time the ECB mandates a qualitative assessment of climate change. The qualitative analysis is to include consideration of both past (observed) changes and well as potential future changes to relevant hydrologic inputs. The qualitative assessment can indicate the predicted direction of change (e.g. more intense downpours) but it cannot determine the magnitude of the change. However, the qualitative assessment has value because it can inform the decision process related to future with and without project conditions.

The USACE commissioned a survey report of climate change studies of the Northeast entitled, Climate Change and Hydrology Literature Synthesis for the US Army Corps of Engineers Missions in the United States – Mid-Atlantic Region (HUC2), October 30, 2014 by CDM Smith. The Mid-Atlantic Literature Synthesis assessed the impact of global climate change to a number of climatologic parameters, but for purposes of this study only precipitation and stream flow are of interest. A majority of the reports predict a moderate increase in both precipitation (annual and monthly) and peak flows. A reasonable consensus exists that the intensity and frequency of extreme storm events will increase in the future. Significant uncertainty exists, however, with respect to the extent of these increases.

Two USGS stream gages exist in or near the project area. Gage, 01419500, Willowemoc Creek near Livingston Manor, NY has a drainage area of 62.6 sq. mi. The period of record is August 11, 1938 to December 21, 1973. Gage 01420000, Little Beaver near Livingston Manor, NY has a drainage area of 20.1 sq. mi. The period of record is

February 12, 1925 to May 12, 1981. Annual Peak and average daily flows are available for the two gages.

Figures 4.10 and 4.11 are plots of annual peak flows for the two gages. However, peak annual flows will not be used to assess possible climate change because of the random nature of large events. A series of large events can as easily be the result of random chance within a stationary time series as be caused by a climate shift. It takes a long period of record to untangle the cause.

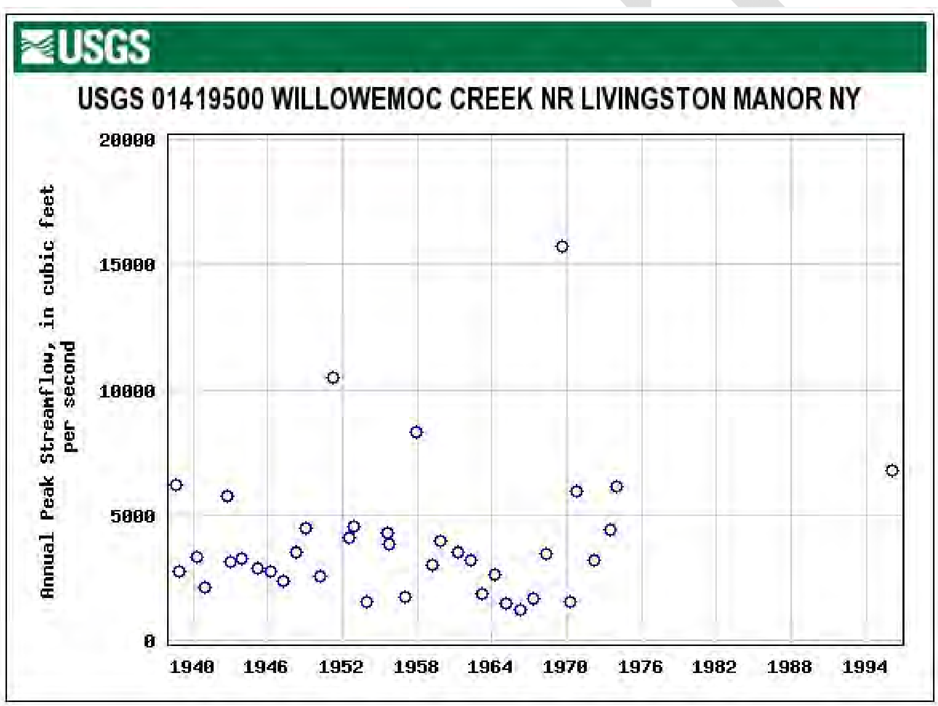


Figure 4.10 Peak Annual Flows for Willowemoc Gage

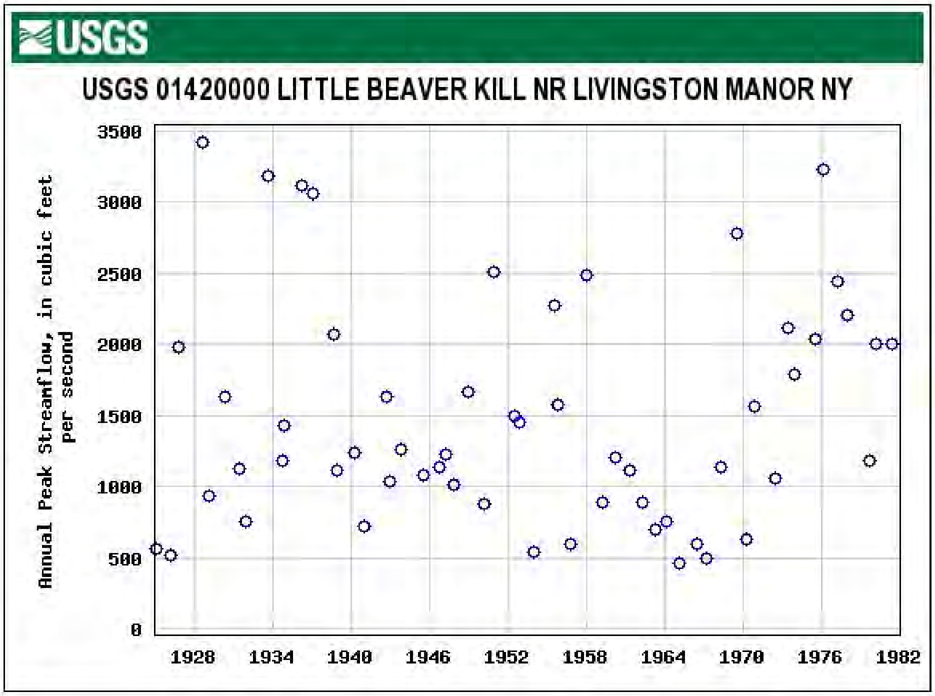


Figure 4.11 Peak Annual Flows for Little Beaver Kill Gage

Figures 4.12 and 4.13 are plots of average daily flows for the two gages. Average daily flow is not a measured flow rate. Rather, it is a proxy for the total volume of water that passes a gage in a 24 hour period. Assuming that the level of imperviousness has not increased with time (see above – it has not), a trend of either increasing or decreasing annual rainfall would manifest as an increasing or decreasing trend of average daily flows with time. Based on visually inspection of Figures 4.12 and 4.13 such a trend is not apparent.

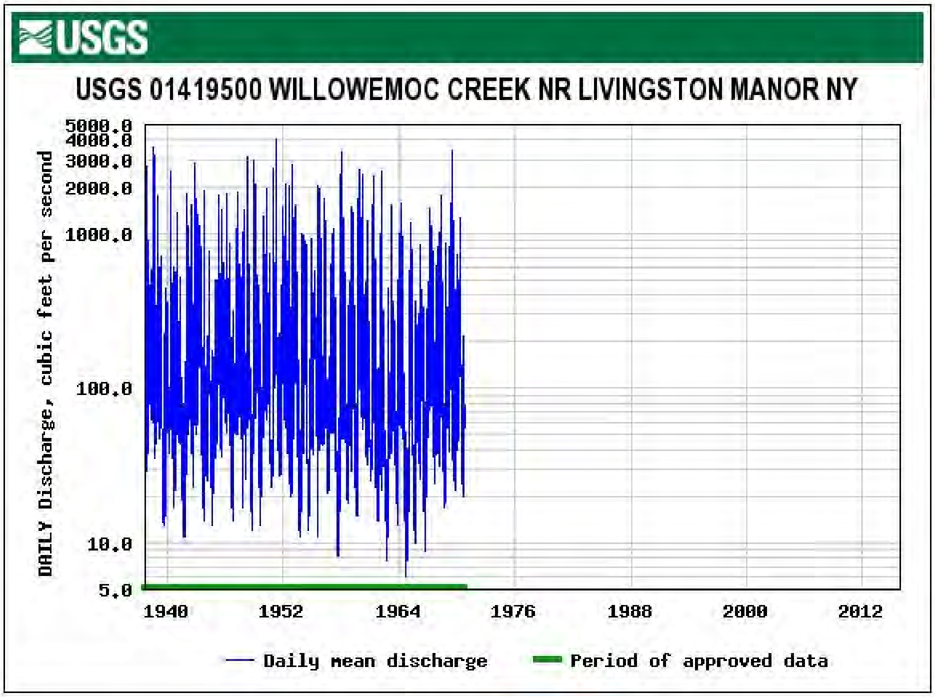


Figure 4.12 Average Daily Flows for Willowemoc Gage

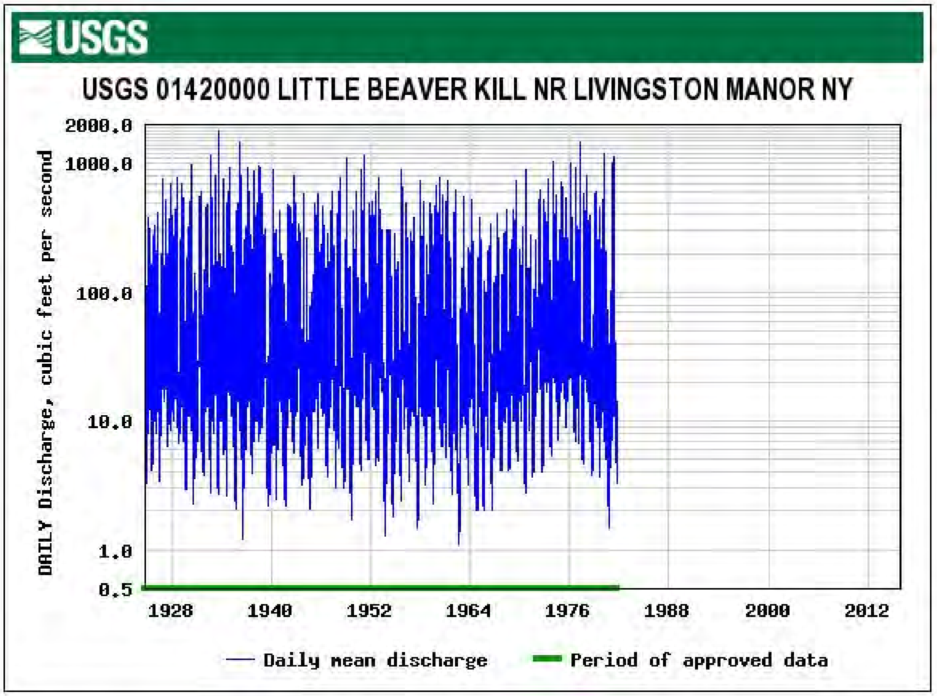


Figure 4.13 Average Daily Flows for Little Beaver Kill Gage

Since the Willowemoc and the Little Beaver Kill gages were discontinued in 1973 and 1981 respectively, they would not reflect recent climate change, if any. Therefore a gage with a longer period of record was assessed. Gage 01420500 Beaver Kill at Cooks Falls, NY is approximately 13 miles downstream of the project area with a drainage area of 241 sq.mi. Its period of record is March 28, 1914 to the present.

Figures 4.14 is a plot of annual peak flows for the Beaver Kill gage. In the last 10 years there appears to be a cluster of large flow events. However, it is premature to view this cluster as a manifestation of climate change. It is more likely an example of hydrologic persistence. That is, a weather pattern persists for a few years and then breaks to be replaced by a different weather pattern but all within the same climate.

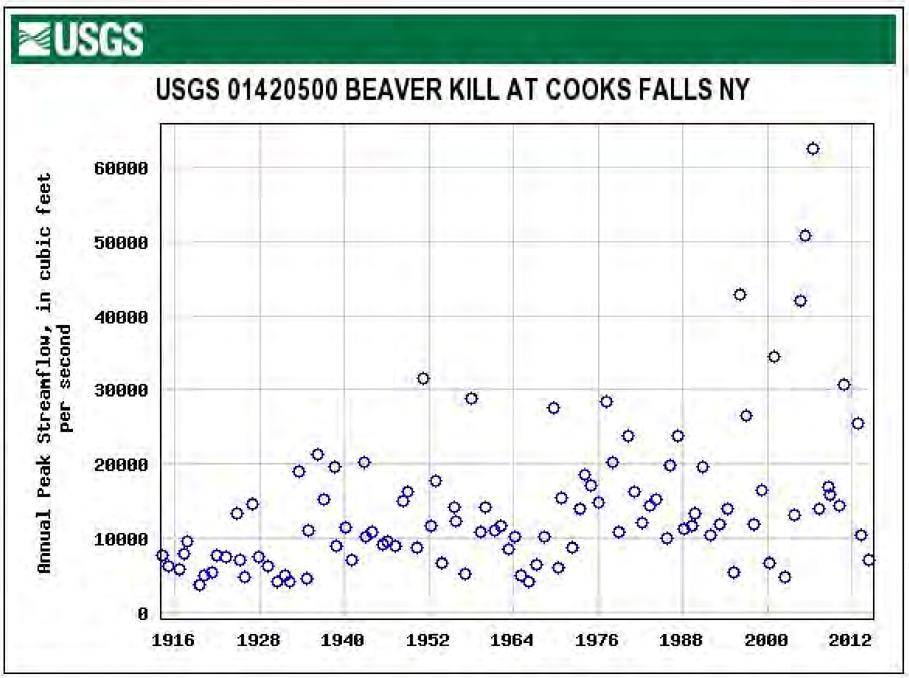


Figure 4.14 Peak Annual Flows for Beaver Kill Gage

Figure 4.15 is a plot of daily flow at the Cooks Falls gage. Visual inspection indicates neither an increasing nor decreasing trend.

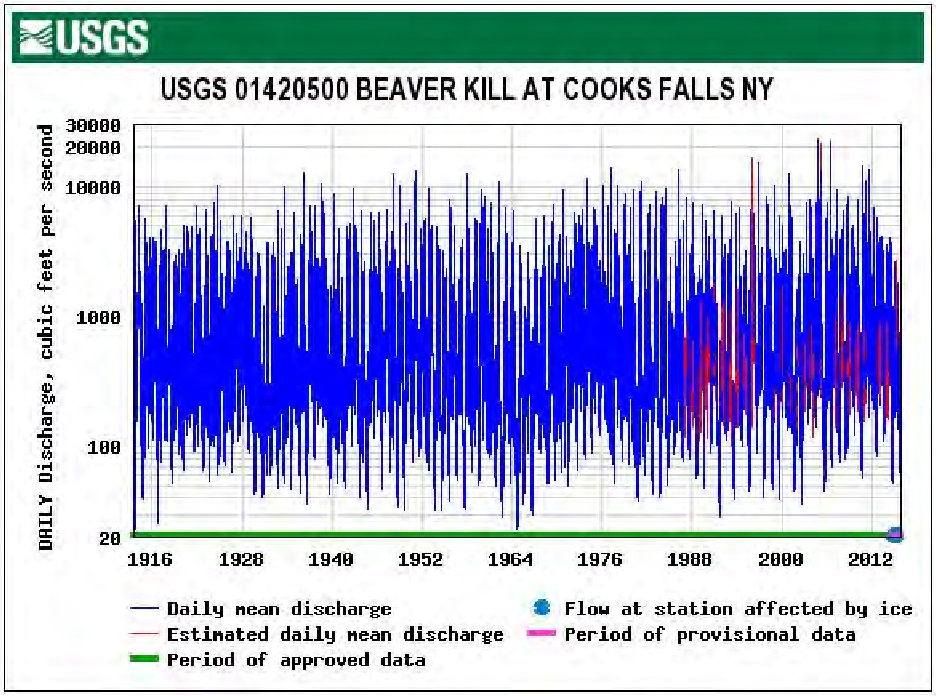


Figure 4.15 Average Daily Flows for Beaver Kill Gage

Figures 4.16 is a plot of instantaneous flow data for the recent period of record for the Beaver Kill gage. Visual inspection indicates a decreasing trend for the last two years. This is likely to be no more than hydrologic persistence.

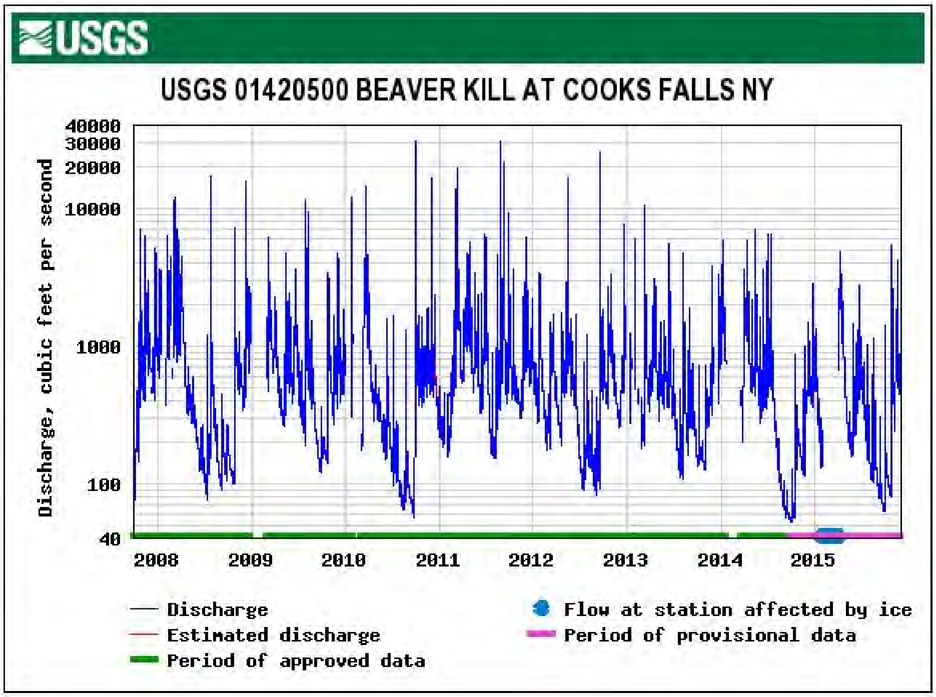


Figure 4.16 Instantaneous Flows for Beaver Kill Gage

Ultimately any climate change would be expected to manifest as a change in precipitation thru time. In place of a statistical analysis of specific rain gages in the project area, readily available rainfall frequency analysis is assessed for temporal trends. There are two sources of information: Technical Paper NO. 40, Rainfall Frequency Atlas of the United States, May 1961 (TP40) and NOAA Atlas 14, Precipitation-Frequency Atlas of the United States, Volume 2, Version 3.0, 2004, Revised 2006 (Atl14). TP40 used rain data through 1958 and Atl14 extended the analysis with rain records through December 2000.

Table 4.13 is a comparison of TP40 and Atl14 frequency-depth-duration rainfall at Livingston Manor, NY. The common frequency events were tabulated because they are less likely to be affected by changes in statistical procedures or the occurrence of large rainfall events that can skew frequency rainfall estimates for rare events.

Table 4.13
Frequency* Depth (Inches) Duration Precipitation at Livingston Manor, PA

Event	30 min		1 hr		3 hr		6 hr		24hr	
	TP40	Atl14	TP40	Atl14	TP40	Atl14	TP40	Atl14	TP40	Atl14
2yr	0.95	0.89	1.20	1.12	2.00	1.68	2.50	2.15	3.20	3.28
5yr	1.30	1.17	1.60	1.47	2.50	2.15	3.00	2.73	4.50	4.19
10yr	1.45	1.40	1.85	1.76	3.00	2.53	3.50	3.21	5.30	4.95

*Partial Duration Series

Table 4.13 shows that all Atl14 frequency rainfall estimates are lower than the TP40 values. This observation is not given much weight because the TP40 values were graphically interpolated from maps. Rather, it is concluded that the common frequency rainfalls are constant thru time. This conclusion is supported by Atlas 14, specifically Appendix A.3 were the possible effects of climate change were examined. To quote (with elision):

“1-day precipitation annual maximum series for stations used in NOAA Atlas 14 Volume 2 were examined for linear trends, linear trends in variance, and shifts in mean. Overall, the 1-day annual maximum time series were free from linear trends and from shifts in mean for most of the stations in the project area. …. the entire historical time series was used in the Atlas.”

The selected plan is a combination of hydraulic solutions whose efficacy is a function of the flow rate. If precipitation and runoff increase in the future such that common events have a higher flow than present flows, the plan will continue to provide some protection but not the level of protection promised.

5. HYDROLOGIC MODEL

The runoff of the Willowemoc Creek and its tributaries was quantified using multiple modeling methods. The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), version 3.4, and Engineering Research and Development Center's (ERDC) Gridded Surface-Subsurface Hydrologic Analysis (GSSHA), version 5.0, were used in conjunction to develop an accurate rainfall-runoff model.

HEC-HMS is a generalized modeling program that is designed to simulate dendritic watershed systems through the use of deterministic mathematical models. HEC-HMS has the ability to model a wide range of watersheds using an equally wide range of rainfall-runoff procedures.

GSSHA is a physically-based modeling program that utilizes a fully distributed parameter routine with two-dimensional overland flow, one-dimensional unsteady diffusion channel routing, and coupled groundwater/surface water interaction. A physically-based model, such as GSSHA, has the ability to produce more accurate historic and frequency-discharge hydrographs than a lumped (or quasi-distributed) unit-hydrograph method, especially in the absence of calibration data.

A. Representative Sub-Basin Comparison

HEC-HMS was chosen to model the entire area of interest due to its ease of use and fast mathematical computation. However, GSSHA was used to determine the most appropriate unit- hydrograph (UHG) transform parameters for use within HEC-HMS. This was accomplished by choosing representative sub-basins within both the Willowemoc and Little Beaver Kill watersheds. One inch of rain without any losses was then applied to the sub-basins for varying amounts of time and the resulting GSSHA hydrographs were compared to hydrographs generated from HEC-HMS using similar inputs but different UHG methods.

The representative sub-basins were chosen because their attributes, such as basin/stream slope and land use, were characteristic of their entire, larger basin. The Little Beaver Kill representative sub-basin had a drainage area of 4.82 mi^2 while the Willowemoc representative sub-basin had a drainage area of 3.53 mi^2. These representative sub-basins and their locations within the area of interest are shown in Figure 5.1.

1. GSSHA

GSSHA solves a diffusive wave form of the St. Venant Equations for overland flow by utilizing a finite-difference scheme. This is accomplished by creating multiple networks of grid cells to assign properties as well as perform the computations. All the properties used in both representative sub-basin models were spatially assigned via the Department of Defense (DOD) Geographic Information System (GIS) preprocessor program Watershed Modeling System (WMS) version 8.3.

For the Little Beaver Kill model, a grid cell size of 25 meters (approximately 0.0005 mi^2) was used while the Willowemoc model used a grid cell size of 12.5 meters (approximately 0.0001 mi^2). Grid cell sizes were based upon the relative sizes of features within the sub-basins in question. The cells in the Little Beaver Kill model were assigned elevation data from a Light Detection and Ranging (LIDAR) elevation model while the cells in the Willowemoc model were derived from a United States Geological Survey (USGS) 10 m Digital Elevation Model (DEM).

The imperfections within the resulting grid networks were smoothed using the Clean Dams program developed by ERDC's Coastal and Hydraulics Laboratory (CHL). Stream networks were derived from USGS Quadrangle Topographic Maps and augmented using aerial photographs. These stream networks were subsequently linked and incised to their complimentary gridded elevation networks. Stream geometries were applied to individual sections using eight-point cross sections that were derived from LIDAR and USGS DEM elevation coverages.

The finalized representative sub-basin GSSHA models are shown in Figures 5.2 and 5.3.

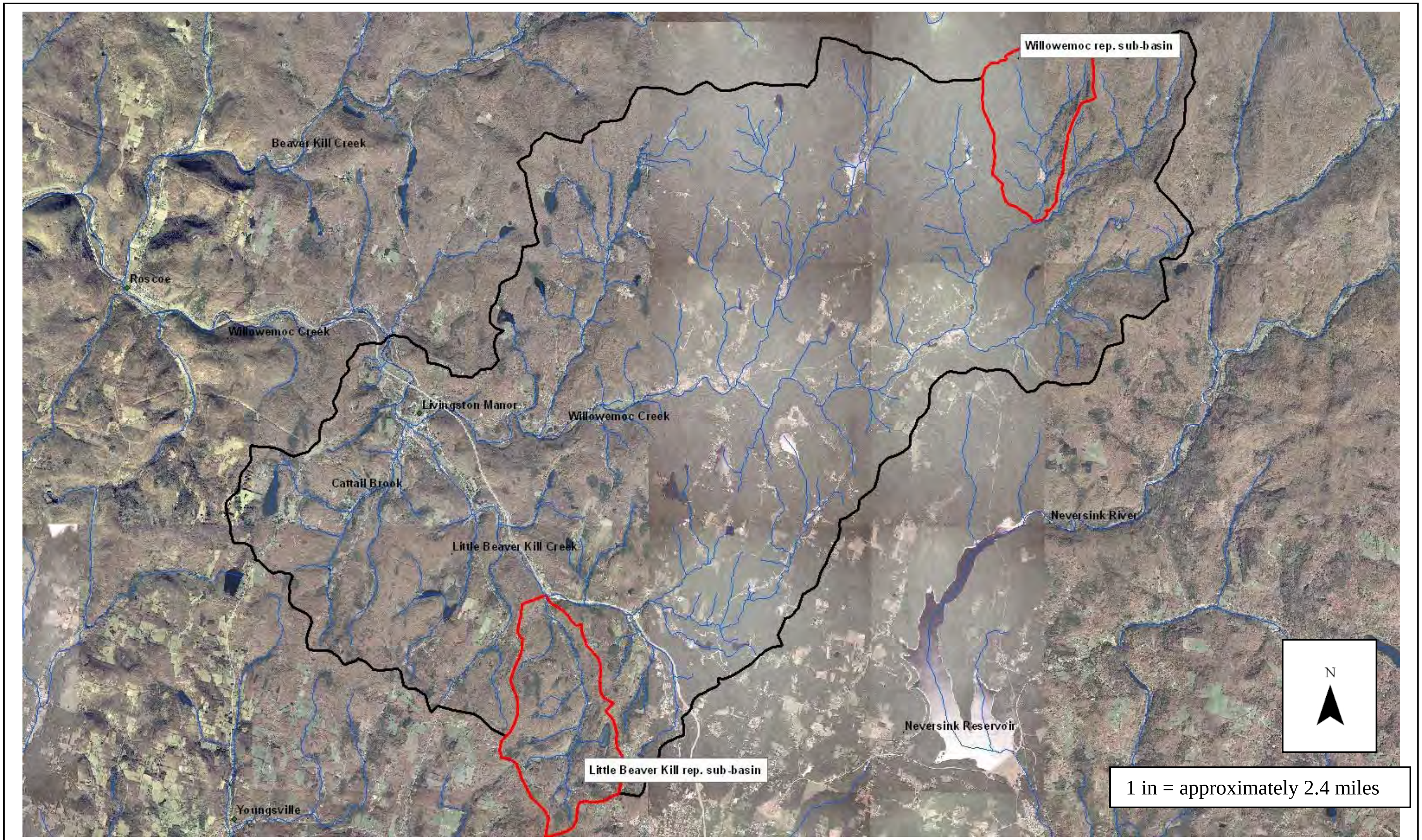


Figure 5.1 Representative Sub-Basins

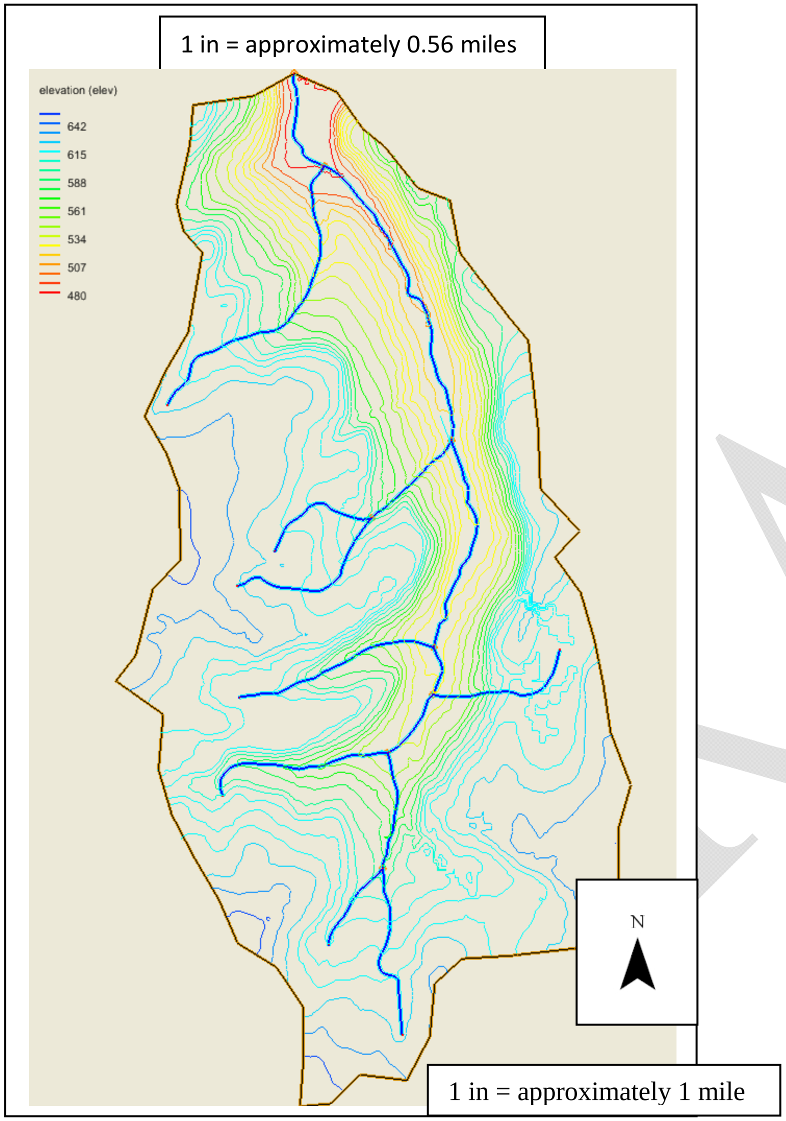


Figure 5.2.Little Beaver Kill Representative Sub-basin Modeled With GSSHA

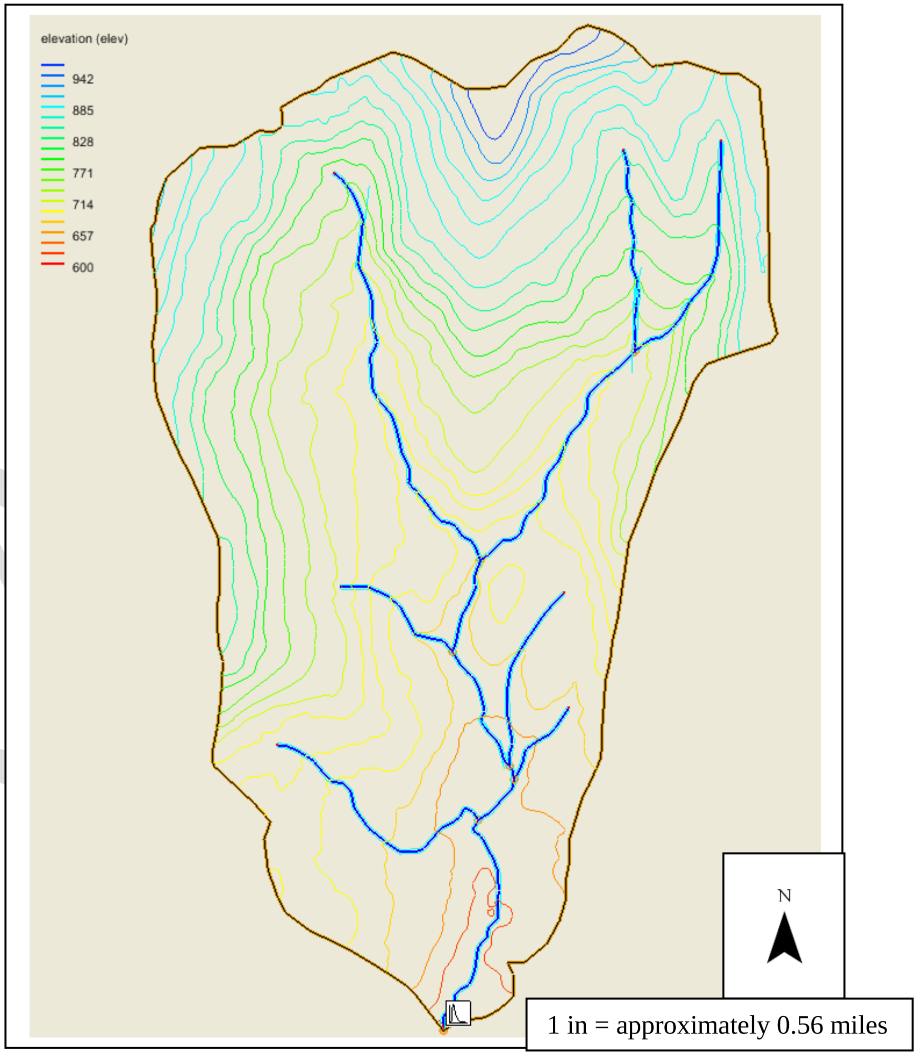


Figure 5.3. Willowemoc Representative Sub-basin Modeled With GSSHA

Land use properties were used to assign overland roughness values to individual cells. Land uses were derived from the Multi-Resolution Land Characteristics Consortium (MRLC) National Land Cover Database (NLCD) 2001. Land uses for both representative sub-basin models and their associated roughness values are detailed in Table 5.1. These values were drawn from the GSSHA Wiki page (http://gsshawiki.com).

Table 5.1

GSSHA Representative Sub-basin Land Uses and Roughness Values

Land Use	Roughness
Reservoirs	0.0001
Developed Space	0.01
Forest	0.2 – 0.25
Scrub	0.25
Grasslands	0.25
Croplands	0.3
Wetlands	0.3

A unit amount of rainfall (one inch) was then distributed over 5, 10, 15, 30, and 60 minutes and input to the model. The resulting hydrographs were then compared to hydrographs developed with HEC-HMS (see the following section).

2. HEC-HMS

The HEC-HMS models for both the Little Beaver Kill and Willowemoc representative sub-basins were developed within WMS and exported to HEC-HMS to be computed. Several different transform methods were selected for analysis. These included Clark, Snyder, and SCS unit hydrographs.

A previous study conducted by HEC entitled "Hydrologic Analysis of Prompton Reservoir Modifications, Lackawaxen River Basin, Pennsylvania" (1988) presented approximate Clark unit hydrograph parameter estimations based upon physical data. Another study conducted by the USACE Baltimore and Philadelphia Districts entitled "Modification of the Francis E. Walter Dam and Reservoir" (1984) presented approximate Snyder unit hydrograph parameter estimations based upon physical data. The Snyder unit hydrograph equations were modified using coefficients contained within WMS. The following equations were used for both the representative sub-basins and the following full-scale model for the entire area of interest.

Clark Unit Hydrograph:

$$T_c = 2.2 \times \left(\frac{L \times L_{ca}}{S^{0.5}} \right)^{0.3}$$

$$\frac{R}{T_c + R} = 0.55 - 0.65$$

$$1.5 \times T_c = R$$

Snyder Unit Hydrograph:

$$T_p = 1.42 \times \left(\frac{L \times L_{ca}}{S^{0.5}} \right)^{0.390}$$

$$C_p = 0.45$$

Where T_c = time of concentration (hours), R = Clark Storage Coefficient (hrs), T_p = Snyder Lag (hrs), C_p = Snyder Peaking Coefficient, L = stream length (mi), L_{ca} = stream length to the basin centroid (mi), and S = stream slope (ft/mi).

SCS Unit Hydrograph:

$$t_L = \frac{L^{0.8}(S+1)^{0.7}}{1900 \times Y^{0.5}}$$

$$t_c = 5/3 \times t_L$$

$$t_p = 0.67 \times t_c$$

$$S = \frac{1000}{CN} - 10$$

Where t_L = lag time (hours), t_c = time of concentration (hrs), t_p = time to peak, S = potential maximum retention, L = watershed length (ft), CN = SCS Curve Number, Y = watershed slope (%).

Both representative sub-basins were modeled in two different ways. Method One comprised of delineating the representative sub-basins at each stream confluence. Channel routing was modeled using Muskingum-Cunge with eight-point cross sections identical to the GSSHA model. Figures 5.4 and 5.5 show the HEC-HMS representative sub-basins for this method. Method Two encompassed delineating each representative sub-basin as one basin. Figures 5.6 and 5.7 show the HEC-HMS representative sub-basins for this method.

A unit amount of rainfall (one inch) was then distributed over 5, 10, 15, 30, and 60 minutes and applied to the model. The resulting hydrographs were then compared to the hydrographs developed with GSSHA.

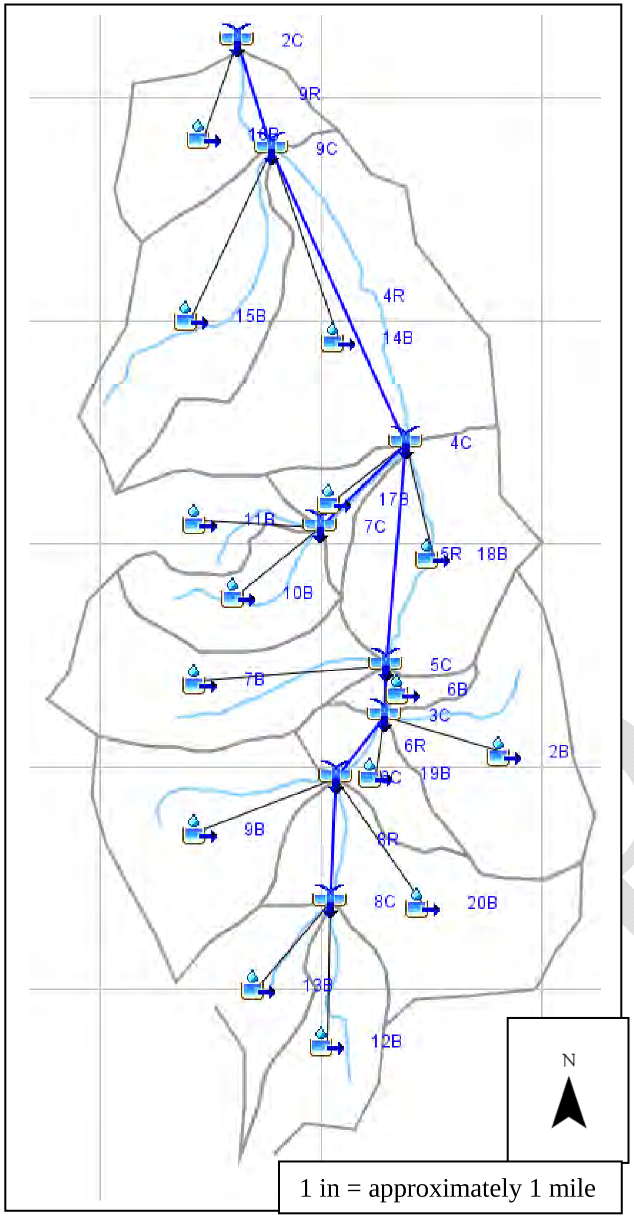


Figure 5.4 Method One - Little Beaver Kill Representative Sub-basin Modeled With HEC-HMS

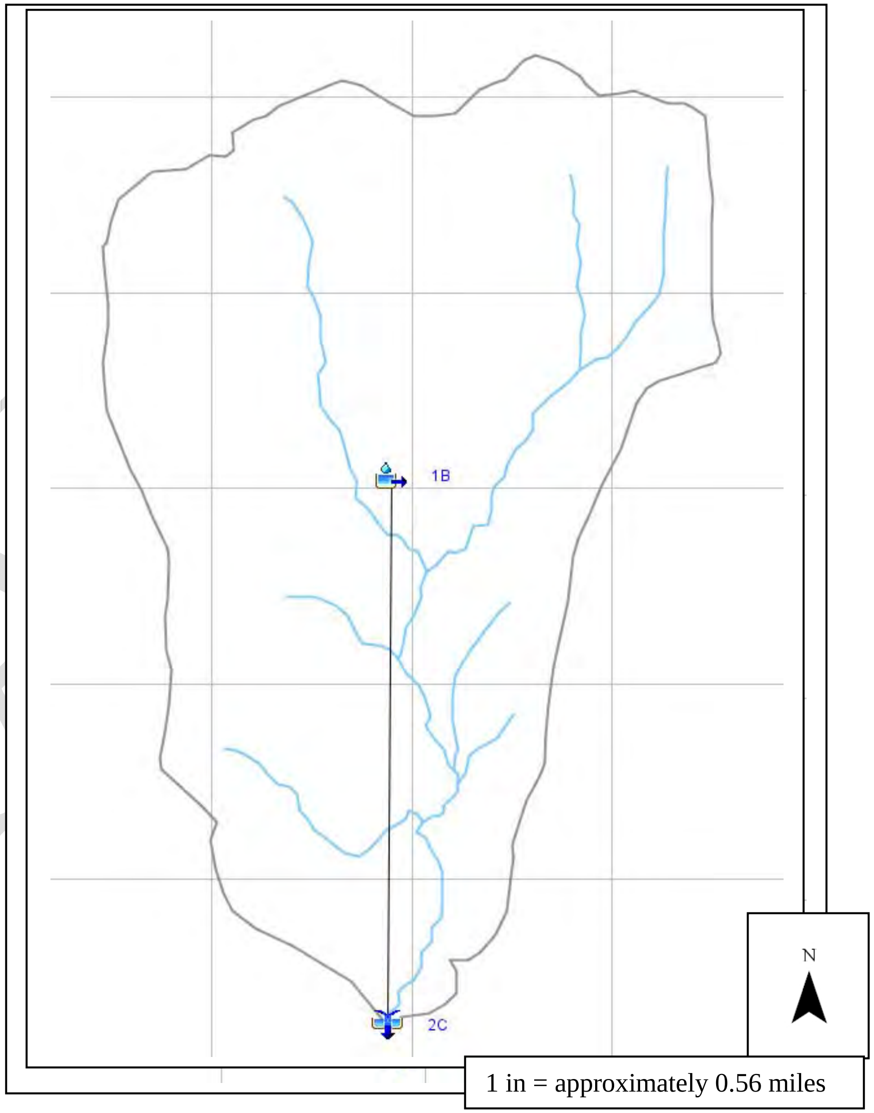


Figure 5.5 Method One - Willowemoc Representative Sub-basin Modeled With HEC-HMS

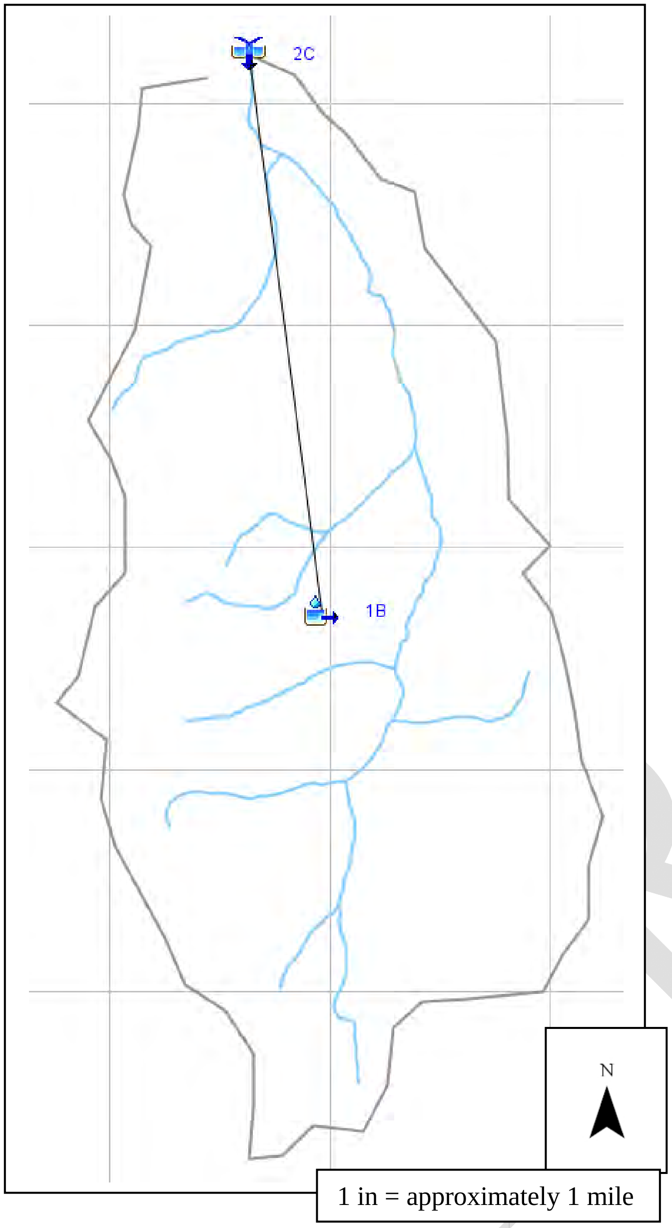


Figure 5.6 Method Two - Little Beaver Kill Representative Sub-basin Modeled With HEC-HMS

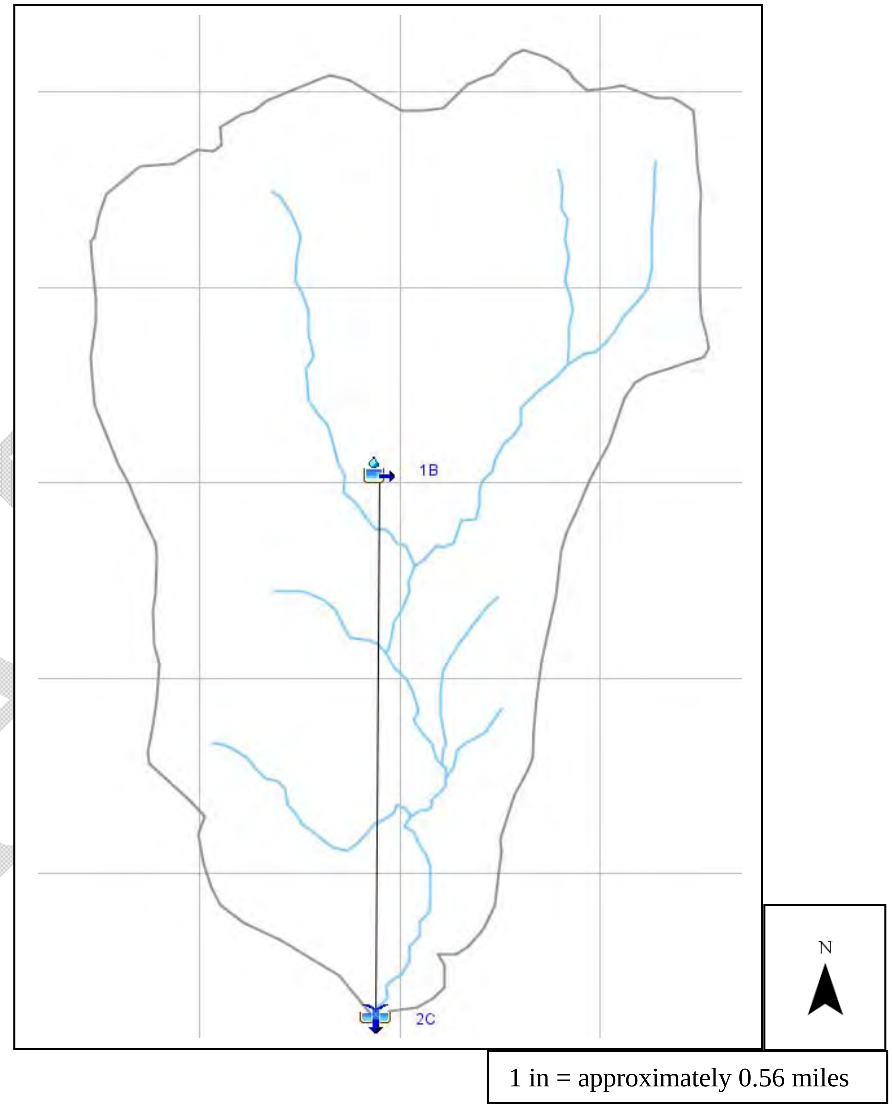


Figure 5.7 Method Two - Willowemoc Representative Sub-basin Modeled With HEC-HMS

3. Results

In both representative sub-basins, the Clark unit hydrograph transform method compared well to the hydrograph generated from GSSHA. In the Little Beaver Kill representative sub-basin, method two did an inadequate job recreating the hydrograph from GSSHA. In the Willowemoc representative sub-basin, the differences in hydrographs between methods one and two are trivial. This is most likely due to the steep nature of the Willowemoc sub-basin.

Overall, the Clark unit hydrograph transform method reproduced the GSSHA hydrographs most closely and was determined to be the most applicable in both cases. Figures 5.8 – 5.11 show the resulting hydrographs from GSSHA and both HEC-HMS methods of delineation utilizing the Clark Unit Hydrograph for each representative sub-basin.

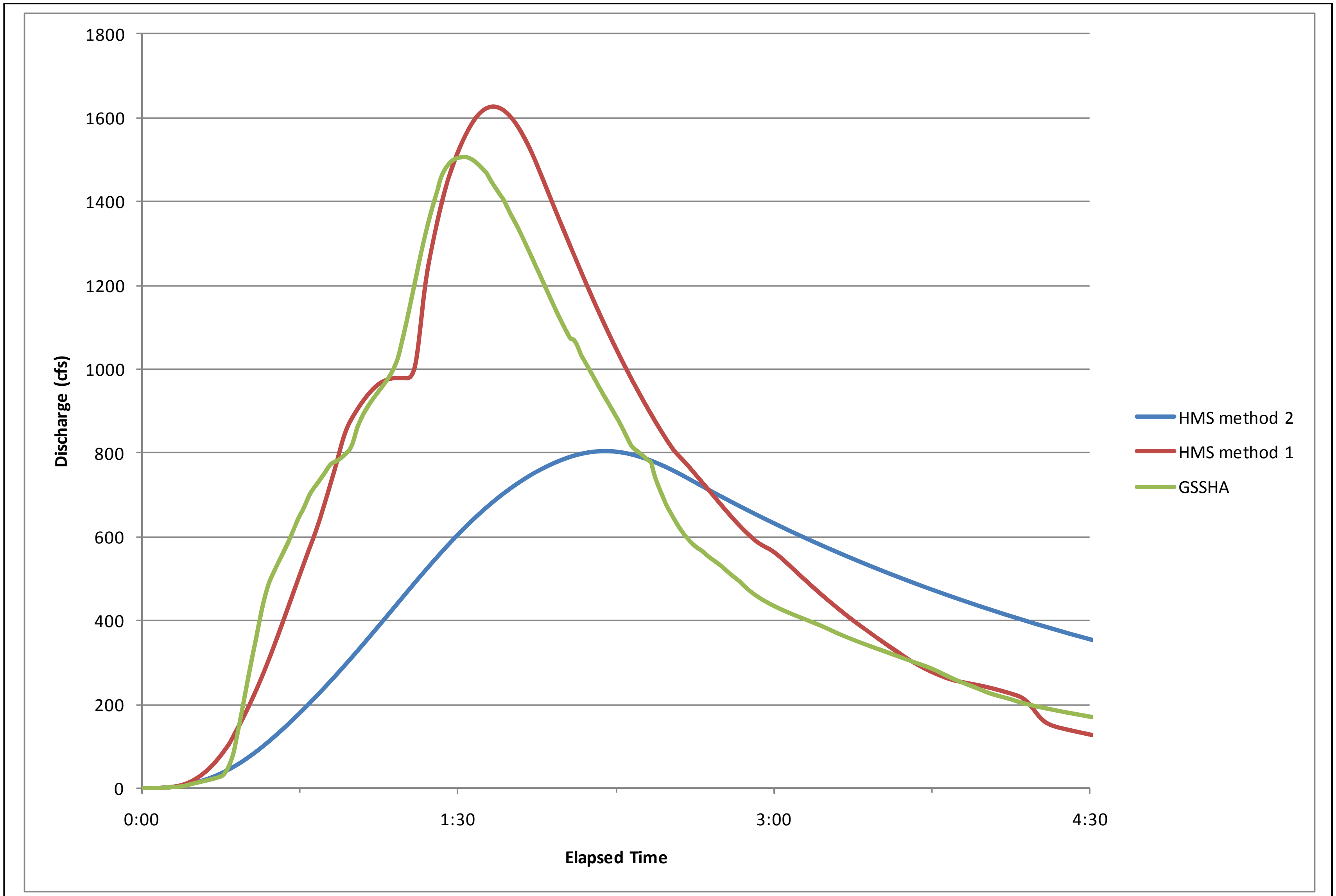


Figure 5.8 GSSHA vs. HMS Comparison - Little Beaver Kill Representative Sub-basin - 1 inch of rainfall in 30 minutes

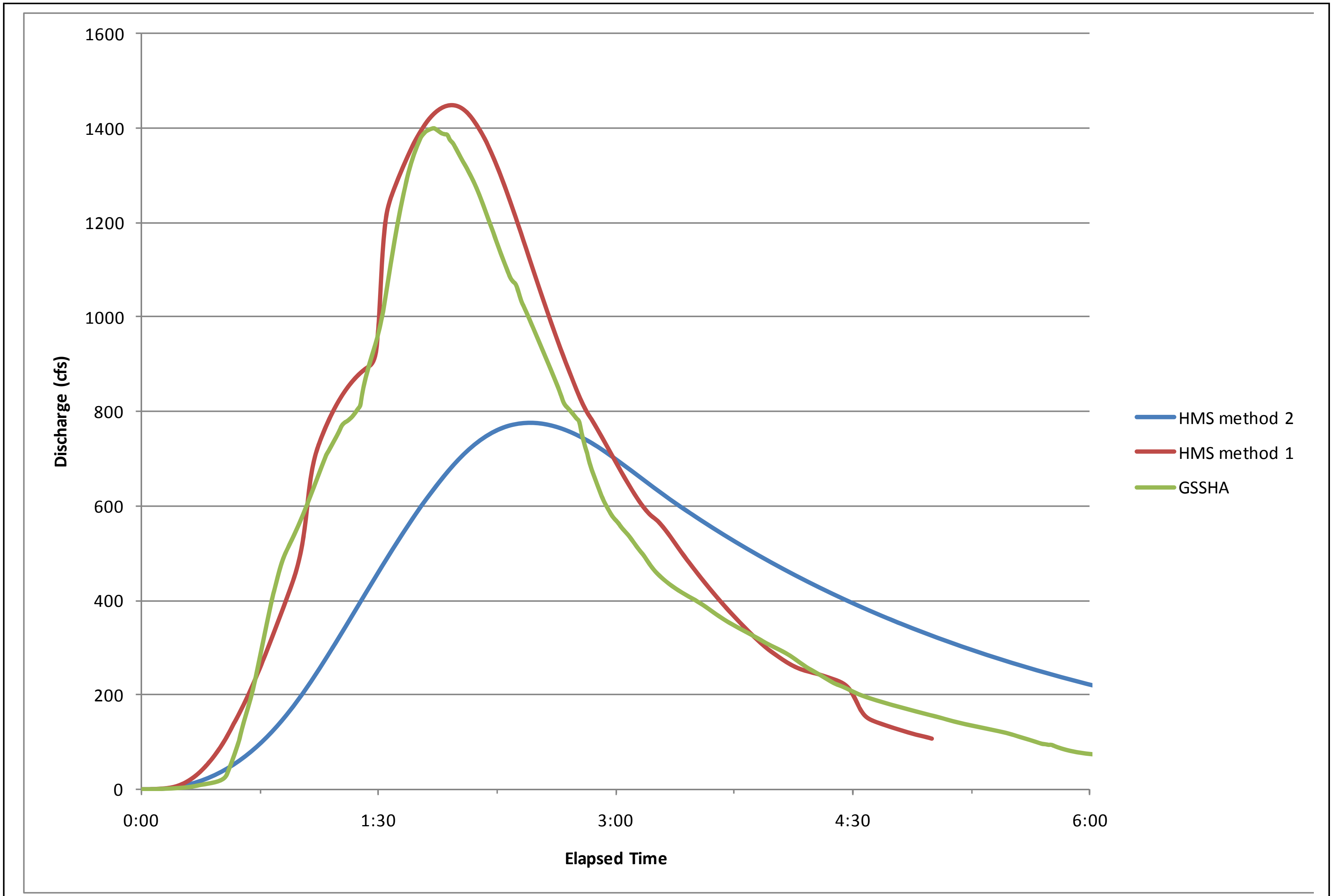


Figure 5.9 GSSHA vs. HMS Comparison - Little Beaver Kill Representative Sub-basin - 1 inch of rainfall in 60 minutes

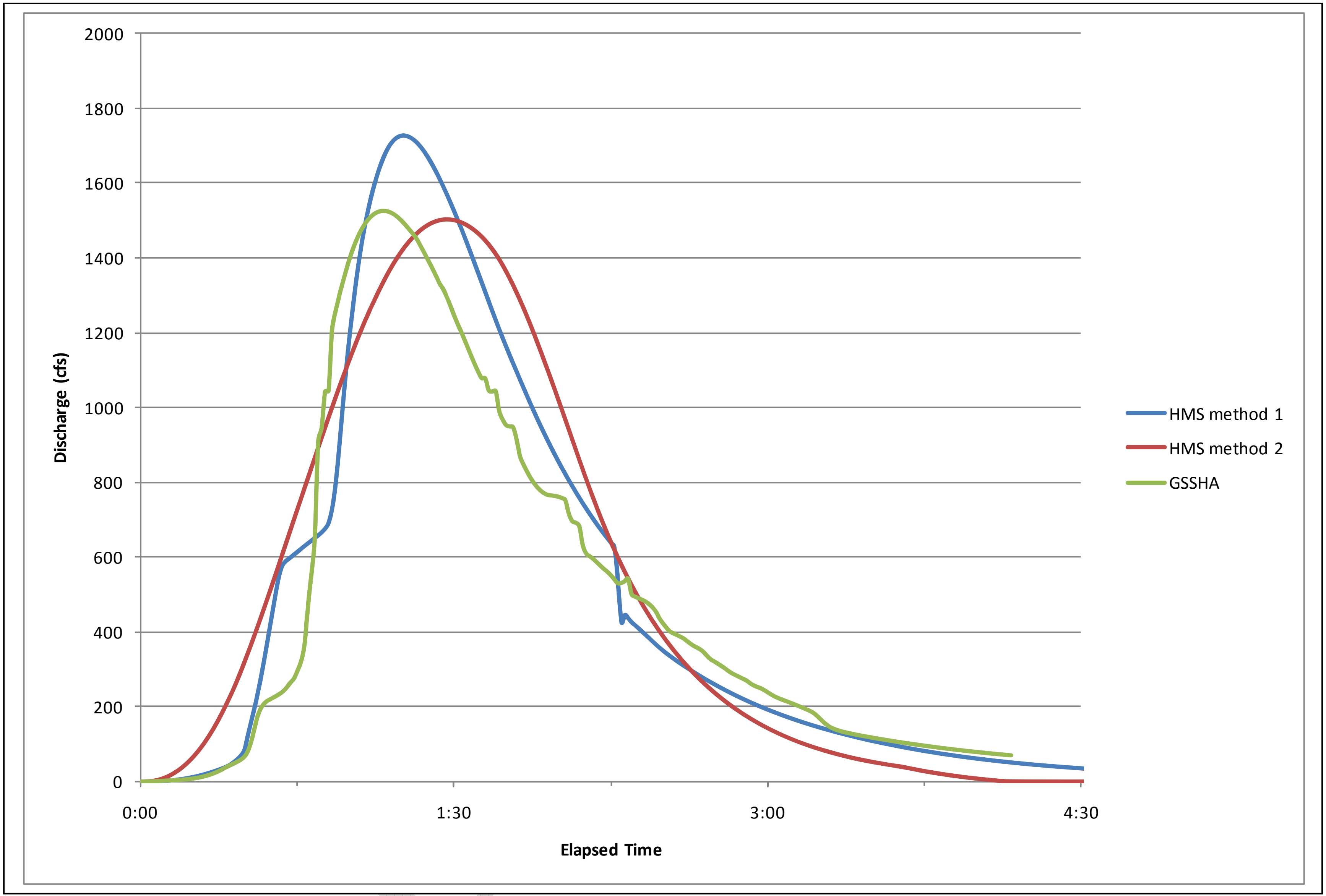


Figure 5.10 GSSHA vs. HMS comparison - Willowemoc Representative Sub-basin - 1 inch of rainfall in 30 minutes

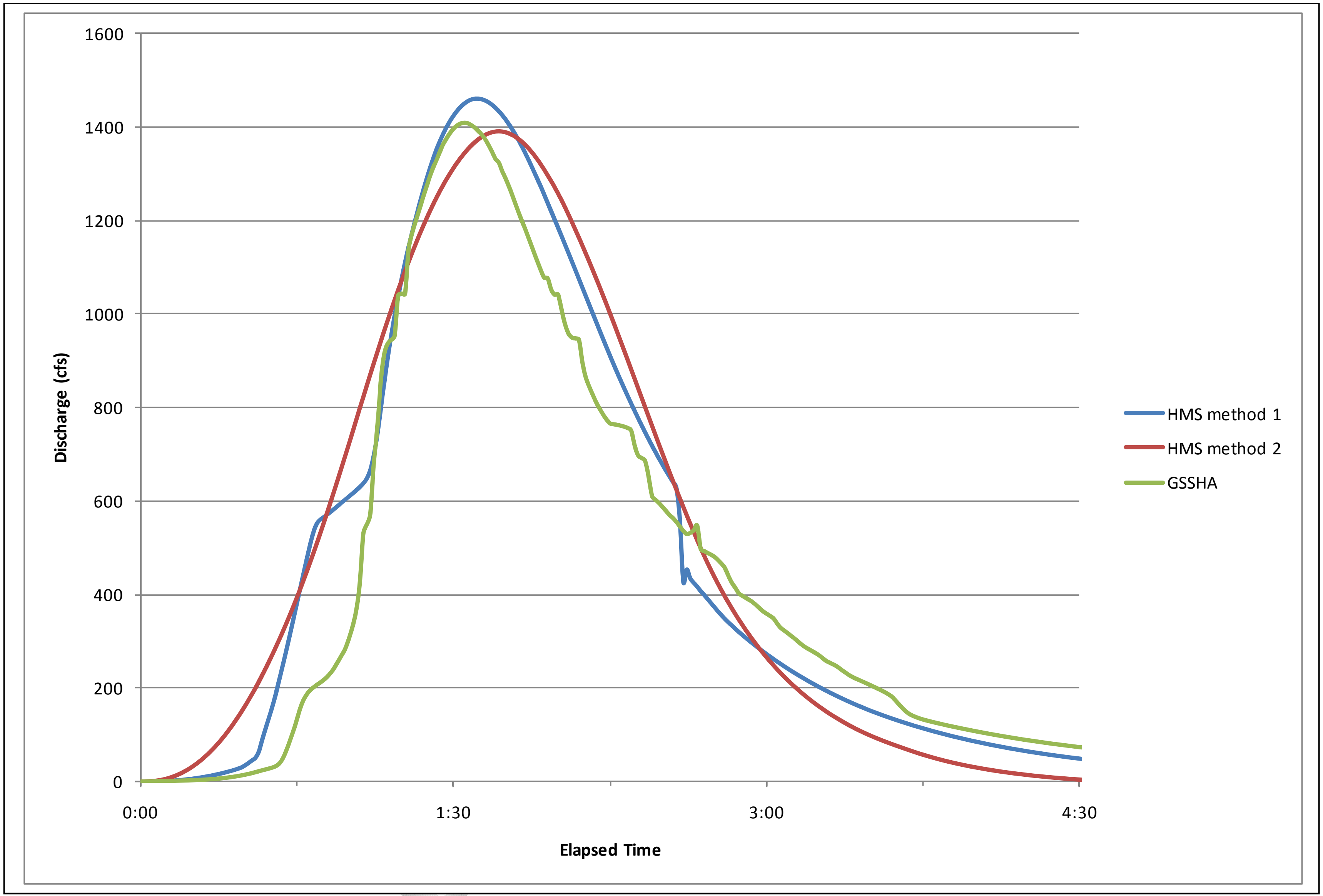


Figure 5.11 GSSHA vs. HMS comparison - Willowemoc Representative Sub-basin - 1 inch of rainfall in 60 minutes

B. Full Scale Hydrologic Model

1. Set Up

Once an appropriate unit hydrograph transform method was selected, the full scale hydrologic model was created. WMS was used to develop the conceptual model.

Elevation data was derived from a USGS 10 m DEM and augmented using LIDAR elevation coverage as well as USGS Quad sheets. Generally, the accepted accuracy of the USGS 10 m DEM is +/- 2.44 m or approximately 8 ft. Contours generated from the USGS 10 m DEM at an interval of 30 ft are shown in Figure 5.12.

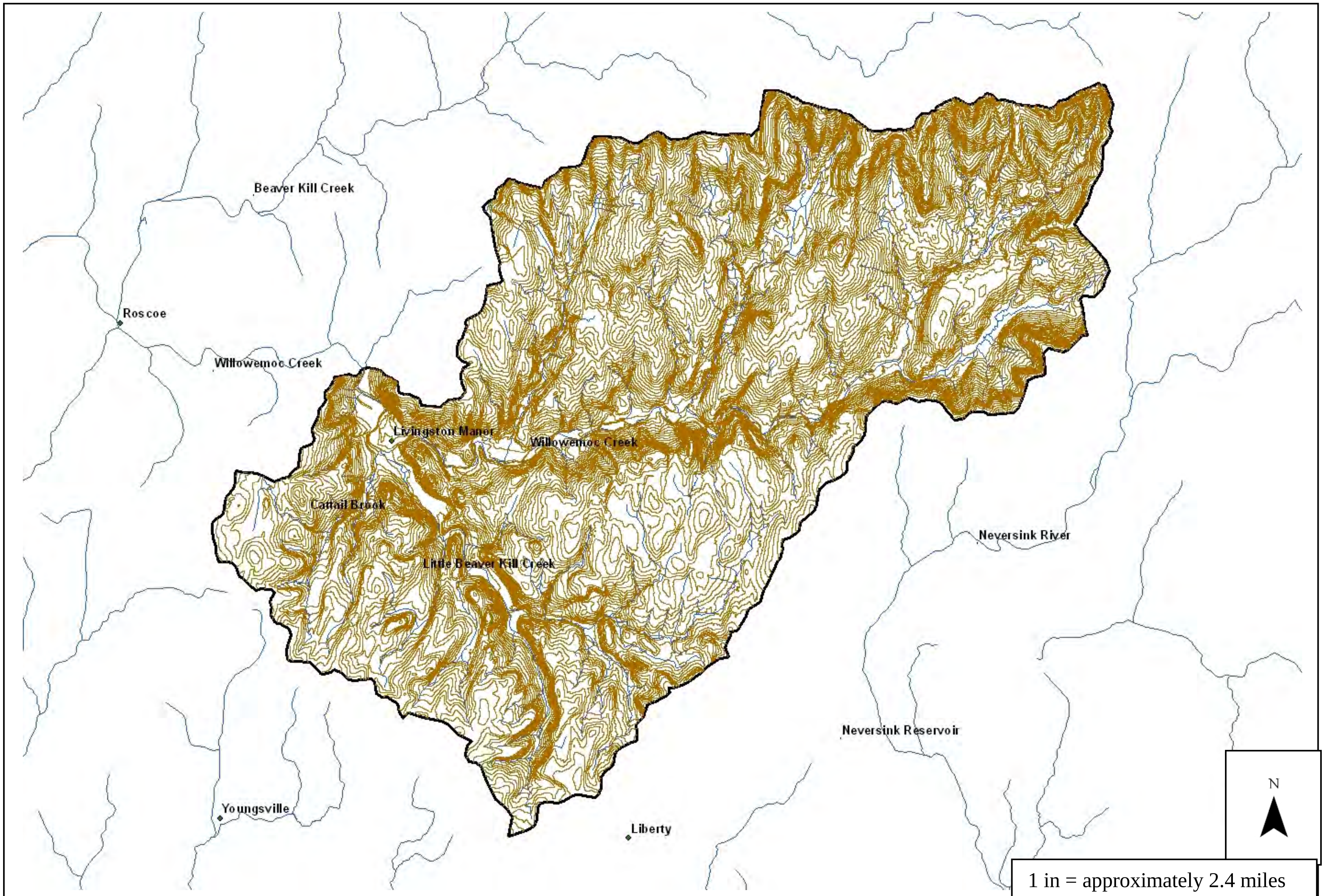


Figure 5.12 30 ft. Contour Intervals Generated From the USGS 10 m DEM

Sub-basin delineations were based on several criteria. These include:

- Stream confluences
- Time step limitations
    - Generally speaking, it is necessary to have at least 4 computation intervals on the rising limb of any hydrograph. Therefore, the smaller the sub-basin, the smaller the time step must be.
- Sites of possible future Land Use changes
- Sites of possible With Project Conditions options

Conforming to these delineation requirements led to the creation of 86 sub-basins for the area of interest. The Willowemoc creek, Little Beaver Kill creek, and Cattail Brook were delineated into 44, 26, and 13, sub-basins, respectively. The remaining three sub-basins were required to account for the inflow of two unnamed tributaries to the Willowemoc creek near the downstream end of the study area. The assorted sub-basins are shown in Figure 5.13.

Pertinent physical information was obtained from the USGS 10 m DEM and used to tabulate Clark Unit Hydrograph parameters. Initial parameter estimates were based on the same equations used in the representative sub-basin models and the results are shown in Tables 5.2 and 5.3.

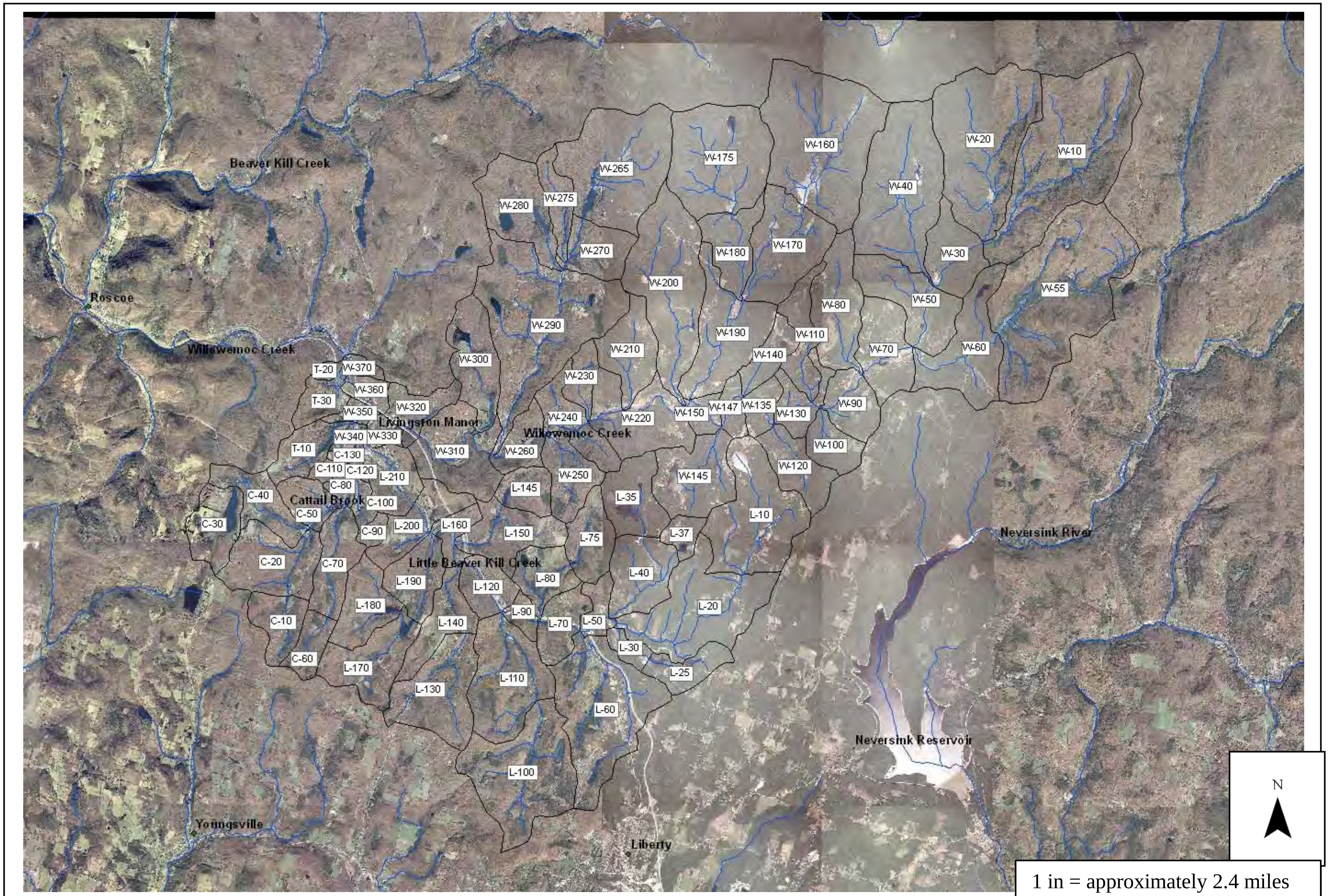


Figure 5.13 Sub-basins Within the Area of Interest

Table 5.2
Sub-basin Physical and Unit Hydrograph Parameters – Part 1

BASIN NAME	BASIN AREA (sq. mi.)	L (mi)	Lca (mi)	S (ft/mi)	Tc (hr)	CLARK R	R / (Tc + R)
C-10	0.77	1.42	0.54	211.74	0.91	1.11	0.55
C-100	0.24	0.89	0.50	456.87	0.69	0.84	0.55
C-110	0.18	0.95	0.52	588.09	0.68	0.84	0.55
C-120	0.04	0.17	0.09	810.30	0.23	0.28	0.55
C-130	0.15	0.45	0.23	556.50	0.43	0.53	0.55
C-20	1.50	1.23	0.93	205.44	1.03	1.26	0.55
C-30	0.97	1.52	0.70	58.67	1.21	1.48	0.55
C-40	0.91	1.83	1.19	226.12	1.23	1.51	0.55
C-50	0.59	0.80	0.62	485.23	0.70	0.86	0.55
C-60	0.54	1.40	0.59	112.38	1.02	1.25	0.55
C-70	1.01	2.13	1.05	200.71	1.26	1.55	0.55
C-80	0.19	0.52	0.19	441.87	0.44	0.54	0.55
C-90	0.18	0.56	0.17	178.16	0.50	0.61	0.55
L-10	2.47	2.67	1.14	104.85	1.53	1.87	0.55
L-100	2.79	2.97	1.36	166.72	1.55	1.90	0.55
L-110	2.03	1.70	1.08	210.78	1.18	1.45	0.55
L-120	0.71	1.08	0.58	419.98	0.77	0.94	0.55
L-130	1.78	2.28	1.19	169.58	1.37	1.68	0.55
L-140	0.92	2.89	1.04	215.07	1.37	1.67	0.55
L-145	0.59	1.03	0.30	140.51	0.74	0.90	0.55
L-150	1.22	1.23	1.30	308.78	1.07	1.31	0.55
L-160	0.42	1.17	0.53	272.98	0.82	1.00	0.55
L-170	1.36	2.10	1.03	135.08	1.33	1.62	0.55
L-180	1.01	1.52	0.50	127.73	0.98	1.20	0.55
L-190	0.93	1.57	0.85	259.30	1.04	1.27	0.55
L-20	2.60	3.19	1.84	135.71	1.79	2.19	0.55
L-200	0.98	0.57	0.29	408.98	0.52	0.64	0.55
L-210	0.89	1.46	0.89	353.41	0.99	1.21	0.55
L-25	0.65	1.67	0.78	208.35	1.07	1.31	0.55
L-30	0.28	0.92	0.51	286.49	0.75	0.92	0.55
L-35	1.10	1.52	0.67	128.07	1.07	1.30	0.55
L-37	0.34	0.81	0.27	149.32	0.66	0.81	0.55
L-40	1.21	2.04	1.03	213.67	1.23	1.50	0.55
L-50	0.40	0.56	0.46	407.75	0.60	0.73	0.55
L-60	3.03	3.84	1.71	117.00	1.89	2.32	0.55
L-70	0.75	0.74	0.40	250.45	0.67	0.82	0.55
L-75	0.88	1.49	0.60	197.56	0.96	1.18	0.55
L-80	0.79	1.24	0.92	354.93	0.95	1.16	0.55
L-90	0.27	0.65	0.38	399.68	0.59	0.72	0.55
T-10	0.75	2.01	1.05	371.22	1.13	1.39	0.55
T-20	0.10	0.44	0.17	1171.22	0.35	0.43	0.55
T-30	0.49	1.03	0.02	649.15	0.26	0.32	0.55

Table 5.3

Sub-basin Physical and Unit Hydrograph Parameters – Part 2

BASIN NAME	BASIN AREA (sq. mi.)	L (mi)	Lca (mi)	S (ft/mi)	Tc (hr)	CLARK R	R / (Tc + R)
W-10	4.55	4.79	2.62	275.89	2.02	2.47	0.55
W-100	0.64	1.32	0.70	412.16	0.87	1.06	0.55
W-110	0.83	2.45	1.10	273.82	1.28	1.56	0.55
W-120	0.94	1.80	0.93	280.01	1.10	1.35	0.55
W-130	0.62	0.94	0.55	319.51	0.76	0.93	0.55
W-135	0.43	0.77	0.41	478.06	0.62	0.75	0.55
W-140	0.59	1.81	0.79	391.73	1.00	1.22	0.55
W-145	1.48	2.29	1.09	283.96	1.24	1.51	0.55
W-147	0.36	0.71	0.39	341.63	0.62	0.76	0.55
W-150	0.33	0.88	0.07	447.65	0.38	0.46	0.55
W-160	3.87	2.85	1.26	363.16	1.33	1.63	0.55
W-170	1.81	2.19	1.37	273.12	1.32	1.61	0.55
W-175	2.69	2.47	1.27	403.22	1.26	1.54	0.55
W-180	0.96	1.62	1.01	338.07	1.06	1.30	0.55
W-190	1.85	2.08	1.35	247.57	1.31	1.60	0.55
W-20	3.53	3.52	2.01	295.20	1.68	2.06	0.55
W-200	2.84	4.33	2.34	250.02	1.93	2.35	0.55
W-210	1.10	2.17	1.14	353.47	1.20	1.46	0.55
W-220	1.65	1.98	0.62	397.56	0.95	1.17	0.55
W-230	0.44	1.59	0.81	378.16	0.97	1.19	0.55
W-240	0.94	0.90	0.64	438.53	0.75	0.91	0.55
W-250	0.91	1.72	0.87	336.24	1.04	1.27	0.55
W-260	0.64	1.08	0.67	255.03	0.87	1.06	0.55
W-265	2.93	2.73	1.32	290.46	1.38	1.69	0.55
W-270	0.89	0.95	0.13	434.34	0.47	0.57	0.55
W-275	0.60	2.21	1.10	296.35	1.22	1.49	0.55
W-280	1.50	2.42	1.66	134.55	1.60	1.96	0.55
W-290	4.18	3.63	2.69	181.23	2.00	2.44	0.55
W-30	1.05	1.29	0.75	254.49	0.95	1.16	0.55
W-300	1.31	2.77	1.57	310.93	1.44	1.76	0.55
W-310	1.65	2.09	0.95	205.43	1.22	1.49	0.55
W-320	0.66	0.42	0.50	532.14	0.54	0.66	0.55
W-330	0.25	0.65	0.39	137.79	0.70	0.85	0.55
W-340	0.18	0.42	0.21	733.16	0.39	0.48	0.55
W-350	0.21	0.37	0.22	537.92	0.41	0.50	0.55
W-360	0.33	0.66	0.43	382.46	0.62	0.75	0.55
W-370	0.24	0.47	0.12	539.78	0.36	0.44	0.55
W-40	3.05	3.62	1.97	328.13	1.66	2.03	0.55
W-50	1.51	1.28	1.11	293.39	1.04	1.28	0.55
W-55	4.71	3.44	0.17	56.25	1.01	1.24	0.55
W-60	1.81	1.95	1.31	118.28	1.42	1.74	0.55
W-70	1.24	0.99	0.61	295.37	0.81	0.98	0.55
W-80	1.45	2.78	1.42	327.16	1.39	1.70	0.55
W-90	0.82	0.99	0.55	484.14	0.73	0.89	0.55

Loss estimates were quantified using Green & Ampt infiltration. Green & Ampt computes precipitation loss on pervious areas in a given time interval as:

$$f_t = K \times \frac{1 + (\phi - \theta_i) \times S_f}{F_t}$$

where f_t = loss during time period *t*, K = saturated hydraulic conductivity, $(\phi - \theta_i)$ = volumetric moisture deficit, S_f = wetting front suction head, and F_t = cumulative loss at time *t*. HEC-HMS also allows for the addition of an initial abstraction, which was not used in this model.

Initial Green & Ampt parameter estimates were derived from a combination of land use and soil texture properties. Land use coverage was created from the MRLC NLCD 2001. The individual 30+ land use values were aggregated to eight. The land uses present in the area of interest include:

- Open Water (reservoirs, ponds, streams, etc.)
- Developed/Urban Space
- Barren Land
- Forest (both deciduous and coniferous)
- Scrub
- Grassland
- Cropland
- Wetland

Soil coverage was taken from Natural Resources Conservation Service (NRCS) State Soil Geographic (STATSGO) maps that were updated in 2006. Many different soil textures were present, but the dominating soil textures were combinations involving silt, loam, bedrock, and peat.

Initial estimates for the Green & Ampt infiltration parameters were based upon soil texture-specific hydraulic conductivities. These initial estimates came from Rawls, Brakensiek, and Miller (1983). They are:

Soil Texture	Sat. Hydraulic Conductivity (in/hr)
Silt	0.26
Loam	0.2
Bedrock	0
Peat	0.01

Percent impervious for each sub-basin was based upon land use and altered using aerial photographs. Developed space was given a percent impervious value of 85% while the other land uses were given percent impervious values ranging between 0 and 50%. WMS was used to calculate weighted sub-basin specific infiltration parameter values. Initial estimates for these parameters are shown in Tables 5.4 and 5.5.

Table 5.4

Initial Green & Ampt Infiltration Parameters Values – Part 1

Subbasin	Initial Loss (in)	Moisture Deficit	Suction (in)	Conductivity (in/hr)	Impervious (%)
C-10	0	0.222	4.74	0.42	0
C-100	0	0.217	4.74	0.413	3.33
C-110	0	0.175	4.74	0.385	0
C-120	0	0.25	4.74	0.427	16.67
C-130	0	0.179	4.74	0.382	7.14
C-20	0	0.197	4.74	0.402	0
C-30	0	0.052	4.74	0.294	0
C-40	0	0.167	4.74	0.379	0
C-50	0	0.235	4.74	0.43	0
C-60	0	0.181	4.74	0.39	0
C-70	0	0.237	4.74	0.431	0
C-80	0	0.175	4.74	0.385	0
C-90	0	0.182	4.74	0.391	0
L-10	0	0.222	4.74	0.42	0.34
L-100	0	0.244	4.74	0.436	0.94
L-110	0	0.244	4.74	0.436	0
L-120	0	0.243	4.74	0.428	9.21
L-130	0	0.235	4.74	0.429	0.51
L-140	0	0.185	4.74	0.393	0
L-145	0	0.212	4.74	0.413	0
L-150	0	0.204	4.74	0.405	2.11
L-160	0	0.24	4.74	0.424	10.42
L-170	0	0.211	4.74	0.412	0
L-180	0	0.202	4.74	0.405	0
L-190	0	0.232	4.74	0.427	0
L-20	0	0.25	4.74	0.44	1.01
L-200	0	0.216	4.74	0.414	1.72
L-210	0	0.193	4.74	0.393	7.55
L-25	0	0.223	4.74	0.42	1.35
L-30	0	0.25	4.74	0.438	3.33
L-35	0	0.164	4.74	0.377	0
L-37	0	0.25	4.74	0.441	0
L-40	0	0.25	4.74	0.438	3.68
L-50	0	0.25	4.74	0.438	4
L-60	0	0.247	4.74	0.434	5.2
L-70	0	0.233	4.74	0.426	2.33
L-75	0	0.245	4.74	0.437	0
L-80	0	0.199	4.74	0.401	2.27
L-90	0	0.25	4.74	0.435	7.14
T-10	0	0.191	4.74	0.397	0
T-20	0	0.208	4.74	0.41	0
T-30	0	0.161	4.74	0.375	0

Table 5.5

Initial Green & Ampt Infiltration Parameter Values – Part 2

Subbasin	Initial Loss (in)	Moisture Deficit	Suction (in)	Conductivity (in/hr)	Impervious (%)
W-10	0	0.25	4.74	0.441	0
W-100	0	0.25	4.74	0.441	0
W-110	0	0.25	4.74	0.441	0
W-120	0	0.25	4.74	0.441	0
W-130	0	0.25	4.74	0.44	1.35
W-135	0	0.228	4.74	0.425	0
W-140	0	0.227	4.74	0.424	0
W-145	0	0.208	4.74	0.41	0
W-147	0	0.206	4.74	0.408	0
W-150	0	0.235	4.74	0.43	0
W-160	0	0.238	4.74	0.432	0
W-170	0	0.245	4.74	0.437	0
W-175	0	0.242	4.74	0.435	0
W-180	0	0.25	4.74	0.441	0
W-190	0	0.228	4.74	0.424	0
W-20	0	0.25	4.74	0.441	0
W-200	0	0.245	4.74	0.437	0
W-210	0	0.242	4.74	0.435	0
W-220	0	0.203	4.74	0.406	0
W-230	0	0.231	4.74	0.427	0
W-240	0	0.2	4.74	0.404	0
W-250	0	0.228	4.74	0.424	0
W-260	0	0.186	4.74	0.393	0
W-265	0	0.238	4.74	0.432	0
W-270	0	0.24	4.74	0.434	0
W-275	0	0.21	4.74	0.411	0
W-280	0	0.212	4.74	0.413	0
W-290	0	0.22	4.74	0.419	0
W-30	0	0.25	4.74	0.441	0
W-300	0	0.224	4.74	0.422	0
W-310	0	0.213	4.74	0.412	1.58
W-320	0	0.2	4.74	0.399	5.71
W-330	0	0.161	4.74	0.357	21.43
W-340	0	0.125	4.74	0.343	6.25
W-350	0	0.063	4.74	0.299	4.17
W-360	0	0.184	4.74	0.381	13.16
W-370	0	0.217	4.74	0.41	6.67
W-40	0	0.249	4.74	0.44	0
W-50	0	0.25	4.74	0.44	0.6
W-55	0	0.25	4.74	0.441	0
W-60	0	0.25	4.74	0.441	0
W-70	0	0.25	4.74	0.439	2.34
W-80	0	0.25	4.74	0.44	0.62
W-90	0	0.25	4.74	0.438	3.41

Base flow for each sub-basin was modeled as a function of area using the Exponential Recession model. It computes base flow according to the function:

$$Q_t = Q_o \times k^t$$

where Q_t = base flow at any time t, Q_o = initial base flow, and k = exponential decay constant. According to the USGS report entitled "Hydrogeology of the Beaver Kill Basin in Sullivan, Delaware, and Ulster Counties, New York" (2005), the mean annual discharge for the Willowemoc basin is approximately 2.42 cfs / mi^2. Therefore, that value was used within HEC-HMS as the initial discharge / area value for every sub-basin. The HEC-HMS Technical Reference Manual, dated March 2000, advises setting the recession constant for surface runoff between 0.3 and 0.8. It was set at 0.5 for every sub-basin. The threshold at which the recession model defined the total flow was set at 0.1.

Muskingum-Cunge hydrologic channel routing was chosen due to its ability to define eight-point cross sections and is arguably the most realistic method within HEC-HMS. Routing reach extents were based upon uniformity of slope and length. Any abrupt changes in either parameter called for a new routing reach. The routing reach parameters were calculated by WMS based upon the USGS 10 m DEM and Quadrangle sheets. Embankment edges that crossed each stream were not modeled within HEC-HMS; they included within the hydraulic model. The locations and names of the initial routing reaches are shown in Figures 5.14 and 5.15.

The routing cross sections were cut from the USGS 10 m DEM and simplified into 8-point cross sections. Manning's n values for the right / left overbanks were set between 0.075 and 0.12 while channel n values were set at 0.045 for every routing segment. The locations of these cross sections are shown in Figure 5.16 while Tables 5.6 and 5.7 contain the physical parameters for each reach.

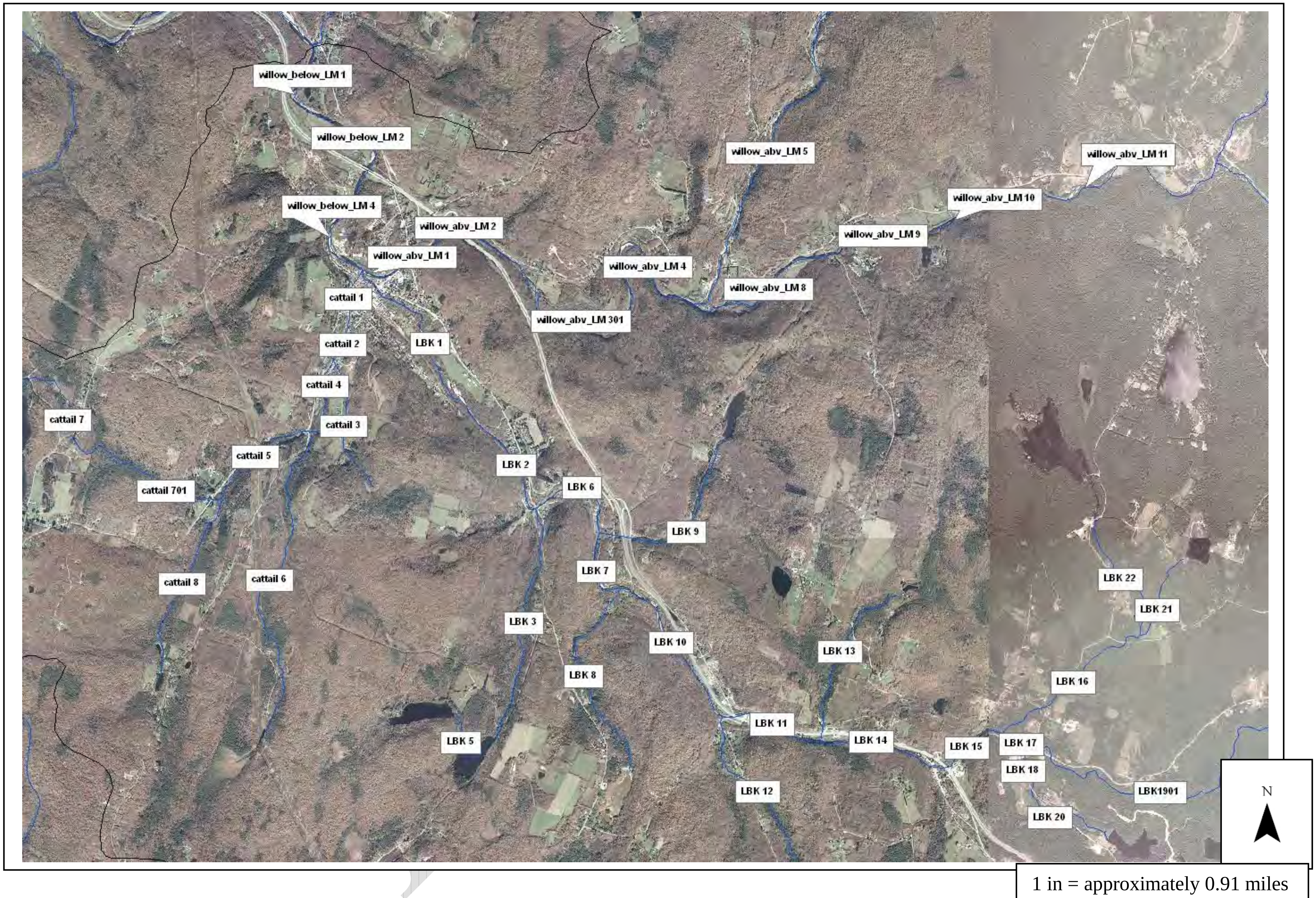


Figure 5.14 Muskingum-Cunge Routing Reaches – Part 1

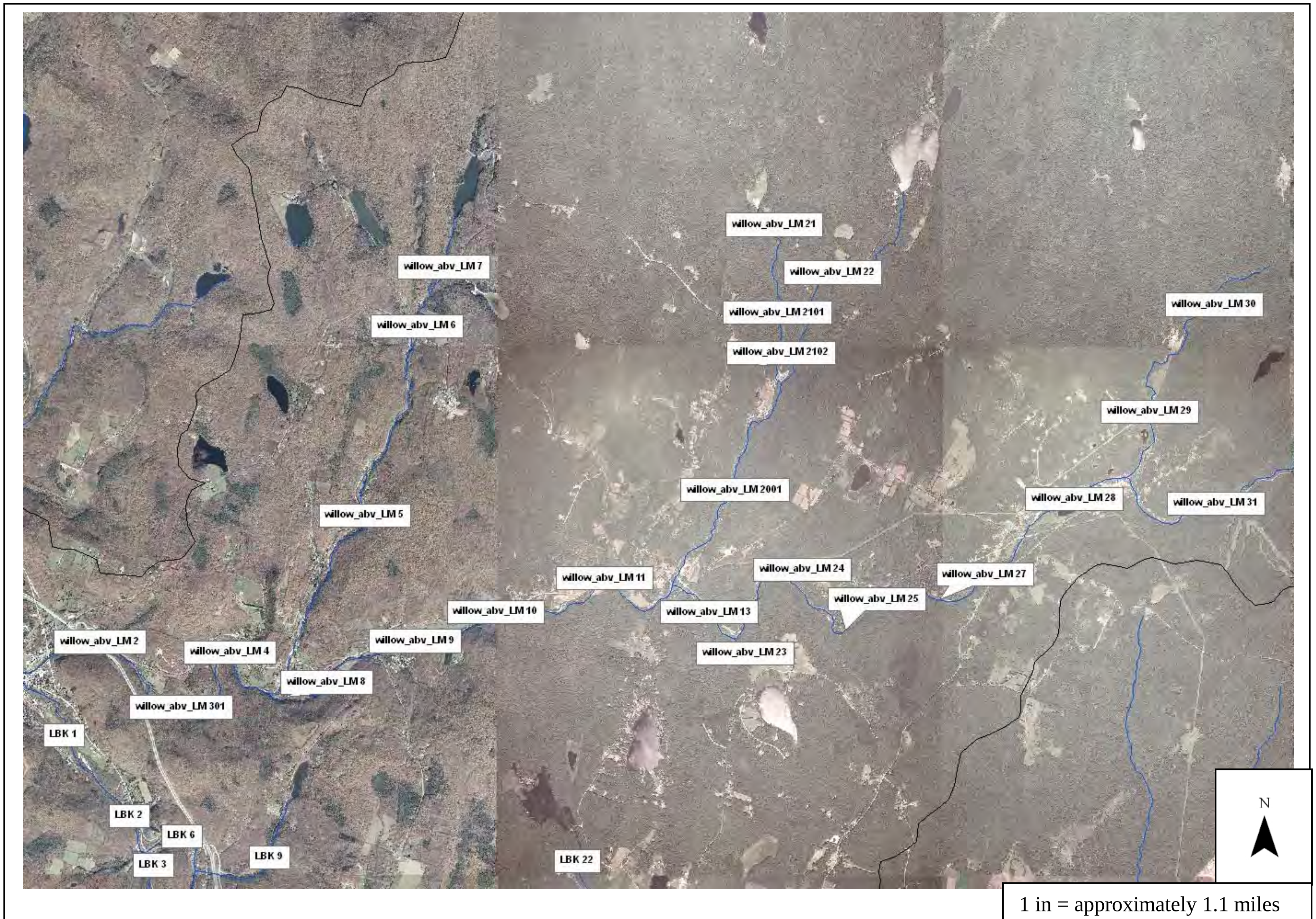


Figure 5.15 Muskingum-Cunge Routing Reaches – Part 2

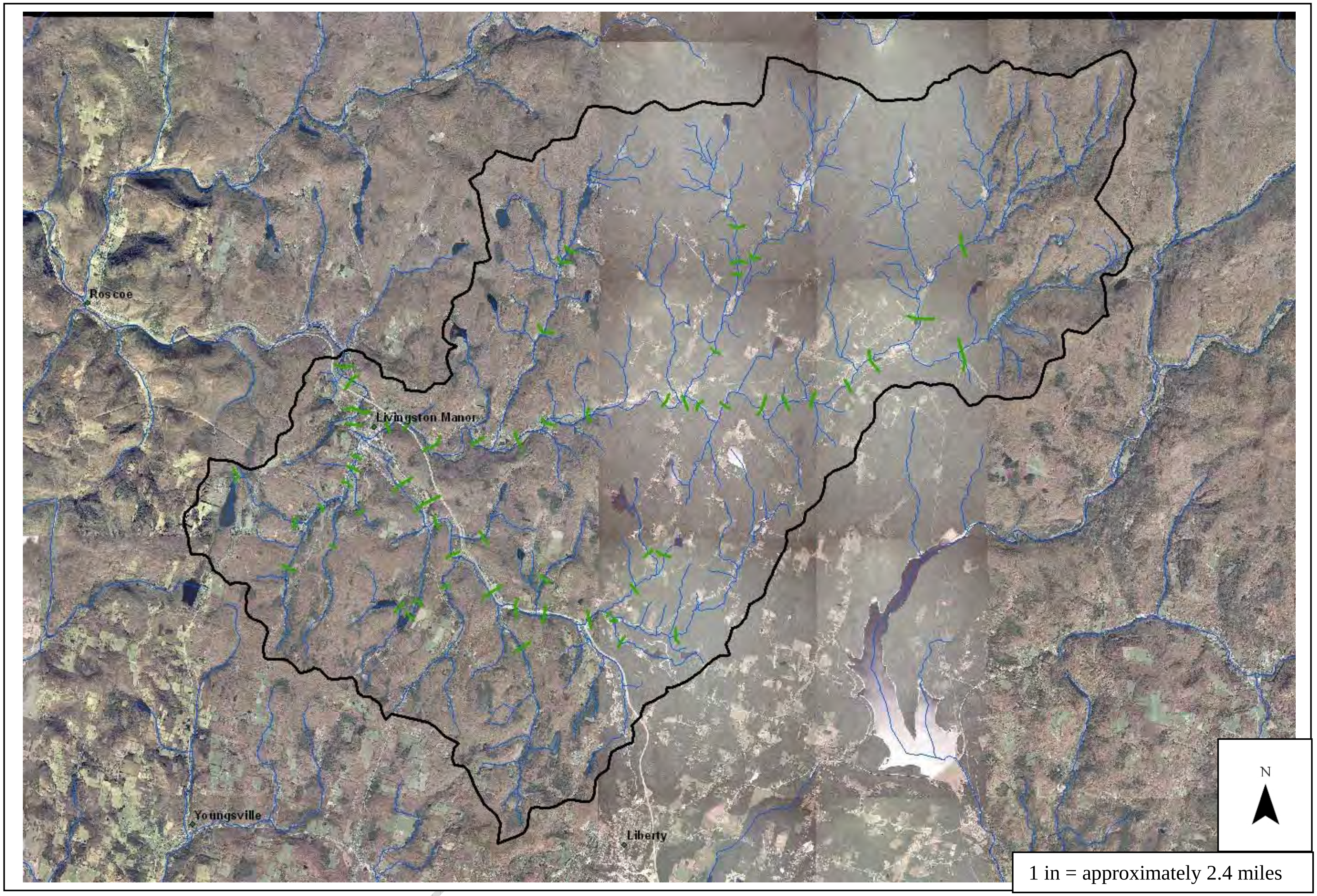


Figure 5.16 Muskingaum-Cunge Hydrologic Cross Section Locations

Table 5.6

Muskingum-Cunge Routing Reach Parameters – Part 1

Reach	length (ft)	slope (ft/ft)
cattail 1	2370	0.027
cattail 2	902	0.020
cattail 3	4723	0.093
cattail 4	2762	0.027
cattail 5	4228	0.037
cattail 6	11249	0.037
cattail 7	5679	0.010
cattail 701	3939	0.072
cattail 8	6489	0.025
LBK 1	7699	0.005
LBK 10	5680	0.008
LBK 11	3420	0.013
LBK 12	8975	0.016
LBK 13	6568	0.057
LBK 14	3926	0.019
LBK 15	2979	0.019
LBK 16	6157	0.045
LBK 18	411	0.100
LBK 2	3020	0.007
LBK 20	4839	0.017
LBK 21	3179	0.023
LBK 22	4588	0.023
LBK 3	8308	0.044
LBK 5	1644	0.044
LBK 6	3545	0.005
LBK 7	2623	0.010
LBK 8	7622	0.039
LBK 9	6511	0.053
LBK1901	16830	0.007

Table 5.7

Muskingum-Cunge Routing Reach Parameters – Part 2

Reach	length (ft)	slope (ft/ft)
willow_abv_LM 1	3448	0.006
willow_abv_LM 10	3275	0.002
willow_abv_LM 11	7208	0.009
willow_abv_LM 12	854	0.008
willow_abv_LM 13	3811	0.009
willow_abv_LM 2	2236	0.008
willow_abv_LM 2001	10953	0.017
willow_abv_LM 21	3357	0.035
willow_abv_LM 2101	2261	0.036
willow_abv_LM 2102	2927	0.027
willow_abv_LM 22	11568	0.007
willow_abv_LM 23	3726	0.005
willow_abv_LM 24	4079	0.010
willow_abv_LM 25	4149	0.005
willow_abv_LM 26	822	0.002
willow_abv_LM 27	4281	0.009
willow_abv_LM 28	5227	0.007
willow_abv_LM 29	6778	0.026
willow_abv_LM 30	6800	0.016
willow_abv_LM 301	8338	0.007
willow_abv_LM 31	10267	0.004
willow_abv_LM 4	2699	0.0001
willow_abv_LM 5	19138	0.023
willow_abv_LM 6	661	0.023
willow_abv_LM 7	4376	0.029
willow_abv_LM 8	5725	0.018
willow_abv_LM 9	4723	0.093
willow_below_LM 1	2491	0.0001
willow_below_LM 2	3472	0.006
willow_below_LM 3	1970	0.0001
willow_below_LM 4	2224	0.0001

Storage areas within the area of interest were analyzed to determine their relative effects on downstream flows. Table 5.8 contains all the storage areas within the Willowemoc sub-basin that were analyzed while Table 5.9 contains all the storage areas within the Little Beaver Kill sub-basin. Storage areas that were not modeled had negligible effects on downstream hydrograph attenuation. The lakes that were modeled include:

- Denman Lake
- Lenape Lake
- Lilly Pond
- Matawa Lake
- Mongaup Pond
- Nimrod Pond
- Orchard Lake
- Paramount Pond
- 2nd Pond @ Parksville
- Tanzman Lake
- Shandelee Lake

An Elevation – Area – Discharge approach was used to model outflow. Elevation – Area curves were created using contours generated from the USGS 10 m DEM. The Elevation – Area curve was then converted to an Elevation – Storage curve within HEC-HMS using the Conic Formula. Elevation – Discharge curves were created based upon dam crest and spillway elevations. Dam crest heights and spillway dimensions were determined from the USACE National Inventory of Dams, updated in 2007. If the spillway capacity was exceeded, water was allowed to flow over the dam crests without failure. All dam crests / spillways were modeled as broad-crested weirs according to the weir equation:

$$Q = C \times L \times H^{1.5}$$

Where C = dimensionless weir coefficient, Q = flow (cfs), L = effective crest length (ft), and H = energy head (ft). The weir coefficients were set at 2.6 while initial discharges were set at zero.

The locations of all modeled reservoirs are shown in Figure 5.17.

Table 5.8

Storage Areas Within Willowemoc Sub-Basin

POND / LAKE	TOTAL DRAINAGE AREA TO POND (mi^2)	NORMAL SURFACE AREA of the POND (mi^2)	ASSUMED STORAGE DEPTH (ft)	INCHES OF RUNOFF FROM SUB-BASIN
Maple Lake	0.542	0.017	1	0.37
			5	1.83
			10	3.66
White Roe Lake	0.278	0.028	1	1.23
			5	6.13
			10	12.25
Forest Lake	0.251	0.052	1	2.50
			5	12.51
			10	25.02
Lake Uncas	1.269	0.051	1	0.48
			5	2.42
			10	4.84
Orchard Lake	2.931	0.056	1	0.23
			5	1.16
			10	2.31
Hodge Pond	0.255	0.029	1	1.35
			5	6.74
			10	13.48
Frick Pond (incls DA of Hodge Pond)	2.693	0.011	1	0.05
			5	0.25
			10	0.50
Sand Pond	0.333	0.022	1	0.80
			5	3.99
			10	7.99
Long Pond	0.255	0.027	1	1.29
			5	6.47
			10	12.94
Hunter	0.531	0.110	1	2.49
			5	12.43
			10	24.86
Knickerbocher Pond	0.383	0.026	1	0.83
			5	4.14
			10	8.28
Trojan Lake	0.451	0.049	1	1.29
			5	6.46
			10	12.91
Thomas Cole Lake	0.234	0.016	1	0.84
			5	4.19
			10	8.38
Mongaup Pond	3.839	0.148	1	0.46
			5	2.31
			10	4.62

Table 5.9

Storage Areas Within Little Beaver Kill Sub-Basin

POND / LAKE	TOTAL DRAINAGE AREA TO POND (mi^2)	NORMAL SURFACE AREA (mi^2)	ASSUMED STORAGE DEPTH (ft)	INCHES OF RUNOFF FROM SUB-BASIN
Revonah Lake	0.358	0.058	1	1.93
			5	9.64
			10	19.29
Melbern Lake	0.236	0.014	1	0.71
			5	3.54
			10	7.08
Kleins Hillside Lake	0.686	0.007	1	0.12
			5	0.60
			10	1.21
Cranberry Pond	0.335	0.030	1	1.08
			5	5.41
			10	10.81
Mud Pond	0.236	0.009	1	0.45
			5	2.24
			10	4.48
Lilly Pond (includes DA from MUD POND)	1.105	0.127	1	1.38
			5	6.89
			10	13.79
Olympus Lake	0.073	0.017	1	2.80
			5	14.00
			10	28.01
Pearl Lake	0.889	0.003	1	0.04
			5	0.20
			10	0.39
Nimrod Lake	0.610	0.026	1	0.51
			5	2.57
			10	5.15
North Pond	0.416	0.086	1	2.49
			5	12.43
			10	24.86
Spring Lake	2.777	0.005	1	0.02
			5	0.10
			10	0.21
Denman Lake	1.777	0.032	1	0.21
			5	1.07
			10	2.14
Lenape Lake	1.009	0.039	1	0.46
			5	2.31
			10	4.62
Matawa Lake (does not include DA from LENAPE LAKE)	1.350	0.038	1	0.34
			5	1.68
			10	3.36
1st Pond @ Parksville	0.292	0.023	1	0.94
			5	4.72
			10	9.44
Tanzman Lake (includes DA from 1st Pond)	0.660	0.034	1	0.63
			5	3.13
			10	6.26
2nd Pond @ Parksville (includes DA from 1st Pond and Tanzman)	0.788	0.007	1	0.10
			5	0.51
			10	1.02
Paramount Pond (includes DA from 1st Pond, Tanzman Lake, and 2nd Pond)	0.938	0.006	1	0.08
			5	0.39
			10	0.78

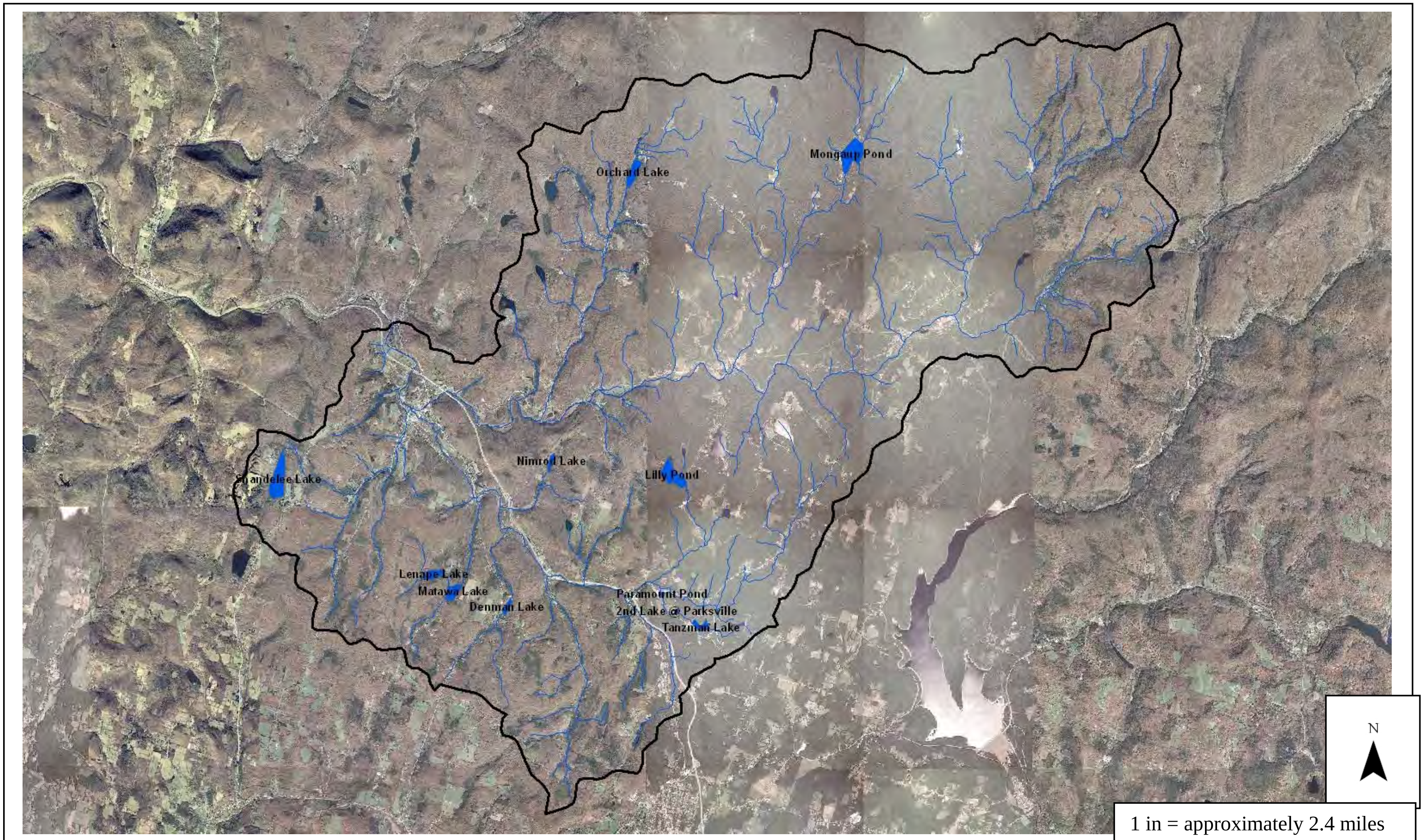


Figure 5.17 Modeled Reservoirs

2. Precipitation

a. Historic Events

Historic precipitation events were chosen based on availability of precipitation data, USGS stream gage data, and high water marks for calibration. Events that occurred due to possible snowmelt, such as the January 1996 and April 2005 event, were not modeled because the inclusion of snowmelt makes calibration more difficult.

Table 5.10 contains the historic events that were modeled and all available data types that could be used for calibration.

Table 5.10

Modeled Historic Events

EVENT	CALIBRATION DATA AVAILABLE			
	USGS STREAM GAGES		HIGH WATER MARKS	NEWSPAPER ACCOUNTS
	DISCHARGE	STAGE		
8 / 1955	X	X		X
7 / 1969	X	X	X	X
9 / 2004		X	X	X
6 / 2006			X	X
7 - 8 / 2009				X

Due to the smaller amount of calibration data available, the June 2006 and July – August 2009 events were used for validation purposes.

Precipitation magnitudes and temporal distributions for the 1955, 1969, 2006, and 2009 events were derived from surrounding National Oceanic and Atmospheric Administration (NOAA) National Climactic Data Center (NCDC) gaging stations and multiple publications. (The 2004 event was modeled with NEXRAD precipitation.) NCDC precipitation gages, both recording and non-recording that were initially screened for use include:

- Butternut Brook
- Callicoon
- Claryville
- Craigie Clair
- Downsville
- Ellenville
- Greentown
- Lewbeach
- Mary Smith
- Mongaup Valley
- Neversink
- Prompton Dam
- Roscoe
- Slide Mountain
- Tannersville

Their locations in relation to the area of interest are shown in Figure 5.18.

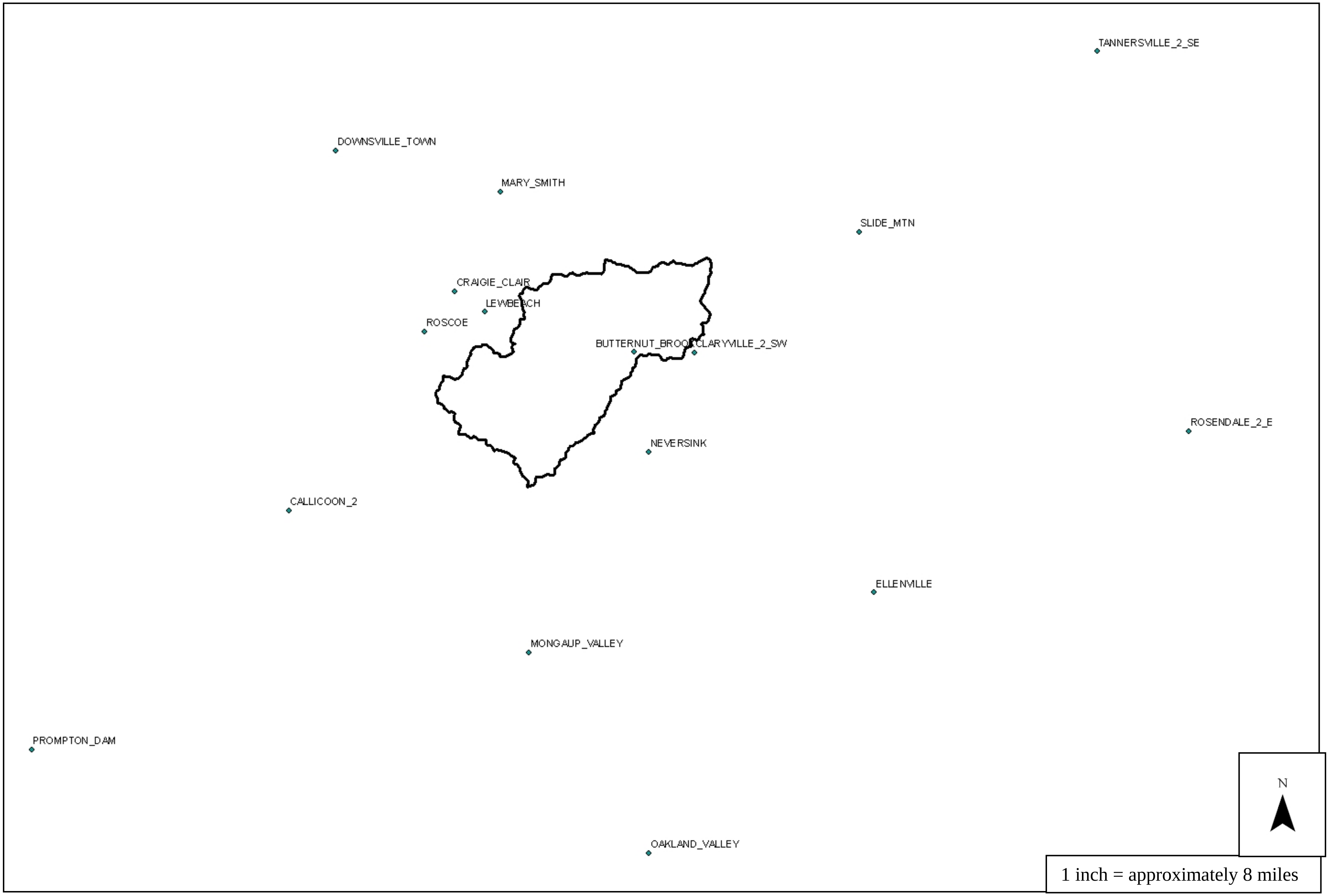


Figure 5.18 NCDC Precipitation Gages in the Surrounding Area

Table 5.11 contains all the gages that were used for each historic event along with pertinent information on each.

Table 5.11

NCDC Precipitation Gages Used

STATION	PERIOD OF RECORD	MEASUREMENT INTERVAL (hr)	EVENTS USED			
			1955	1969	2006	2009
Butternut Brook	1948 - 1959	24	X			
Callicoon	1980 - current	15				X
Claryville	1948 - current	0.25			X	X
Craigie Clair	1948 - 1959	24	X			
Ellenville	1949 - 1984	1	X	X		
Lewbeach	1948 - 1959	24	X			
Mongaup Valley	1974 - current	0.25				X
Prompton	1966 - current	0.25				X
Tannersville	1976 - current	0.25				X

A temporal distribution at an hourly interval for the August 1955 and July 1969 events was derived from the Ellenville NCDC precipitation gage. The Ellenville gage hyetographs and cumulative rainfall amounts for each event are shown in Figures 5.19 and 5.20. The non-recording NCDC gages Craigie Clair, Lewbeach, and Butternut Brook were used to assign magnitudes and spatial distributions to the 1955 event while being augmented by the US Department of the Interior (DOI) paper titled “Floods of August 1955 in the Northeastern States” (1956). The total depths applied to each sub-basin for the 1955 event are shown in Tables 5.12 and 5.13.

Magnitudes and spatial distributions for the 1969 event were assigned using the USGS paper entitled “Flood of July 27 – 28, 1969 in Southeastern New York” which was superseded by the US DOI paper entitled “Summary of Floods in the United States

During 1969”.   The total depths applied to each sub-basin for the July 1969 event are shown in Tables 5.14 and 5.15.

A temporal distribution at a 15 minute interval for the June 2006 event was derived from the Claryville NCDC precipitation gage. The Claryville gage hyetograph and cumulative rainfall amount are shown in Figure 5.21. Magnitudes and spatial distributions were assigned using the USGS paper entitled “Flood of June 26 – 29, 2006, Mohawk, Delaware, and Susquehanna River Basins, New York”. The total depths applied to each sub-basin are shown in Tables 5.16 and 5.17.

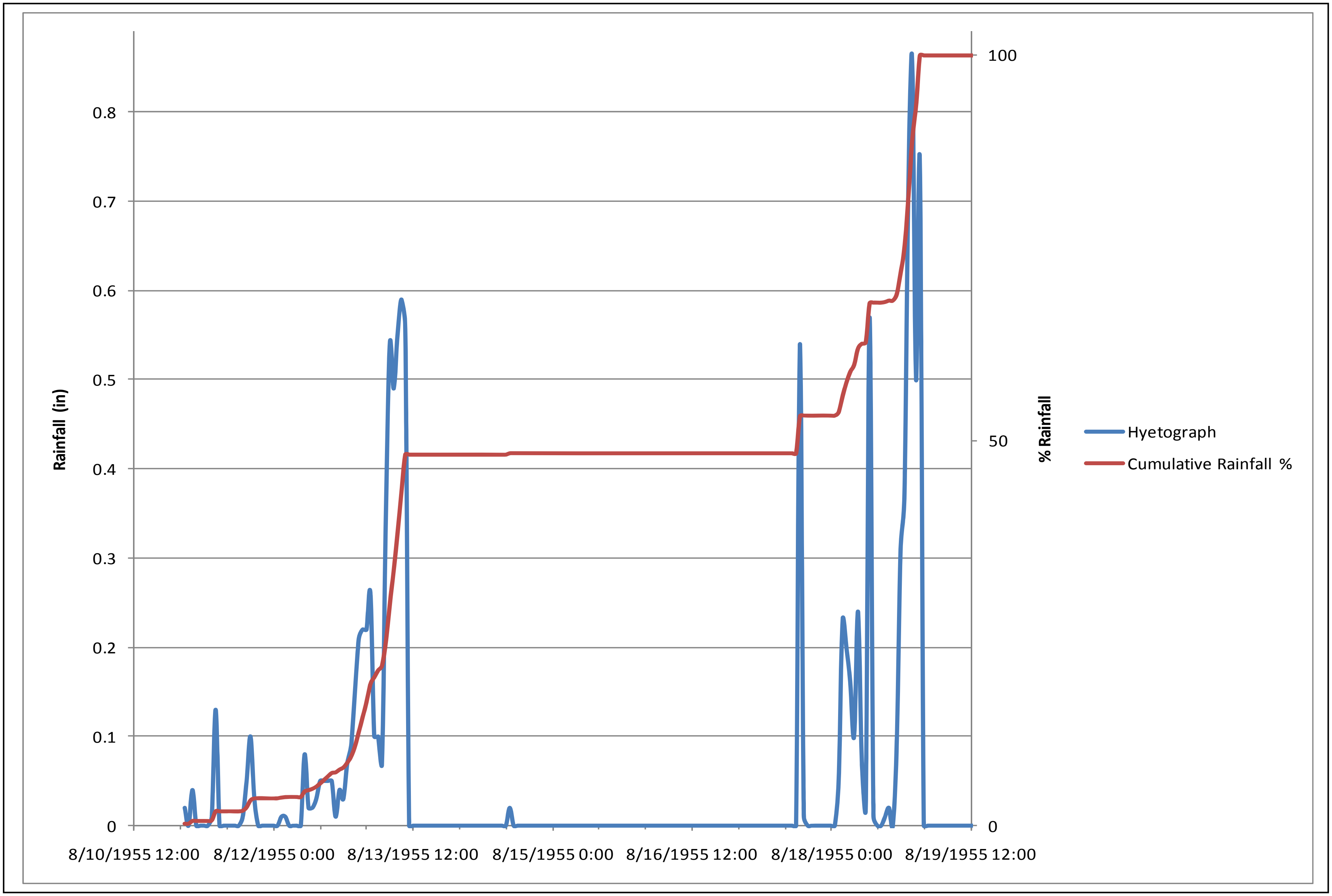


Figure 5.19 August 1955 Event - Ellenville Hyetograph and % Rainfall

Table 5.12 August 1955 Event – Sub-basin Precipitation Totals – Part 1

Subbasin	Total Depth (in)
C-10	5
C-100	5
C-110	4.75
C-120	5
C-130	4.75
C-20	5
C-30	4.75
C-40	4.75
C-50	4.75
C-60	5.25
C-70	5
C-80	5
C-90	5.25
L-10	6
L-100	6
L-110	5.5
L-120	5.5
L-130	5.5
L-140	5.5
L-145	5.5
L-150	5.5
L-160	5.25
L-170	5.25
L-180	5.25
L-190	5.25
L-20	6
L-200	5.25
L-210	5
L-25	6.5
L-30	6
L-35	5.5
L-37	5.5
L-40	6
L-50	6
L-60	6
L-70	5.5
L-75	5.5
L-80	5.5
L-90	5.5
T-10	4.75
T-20	4.75
T-30	4.75

Table 5.13 August 1955 Event – Sub-basin Precipitation Totals – Part 2

Subbasin	Total Depth (in)
W-10	5.25
W-100	5.5
W-110	5.5
W-120	5.5
W-130	5.5
W-135	5.5
W-140	5.5
W-145	5.5
W-147	5.5
W-150	5.5
W-160	5.25
W-170	5.25
W-175	5.25
W-180	5.25
W-190	5.5
W-20	5.25
W-200	5.25
W-210	5.25
W-220	5.5
W-230	5.25
W-240	5.25
W-250	5.5
W-260	5.25
W-265	5.25
W-270	5.25
W-275	5.25
W-280	5.25
W-290	5.25
W-30	5.25
W-300	5
W-310	5
W-320	5
W-330	5
W-340	4.75
W-350	4.75
W-360	4.75
W-370	4.75
W-40	5.25
W-50	5.5
W-55	5.5
W-60	5.25
W-70	5.5
W-80	5.5
W-90	5.5

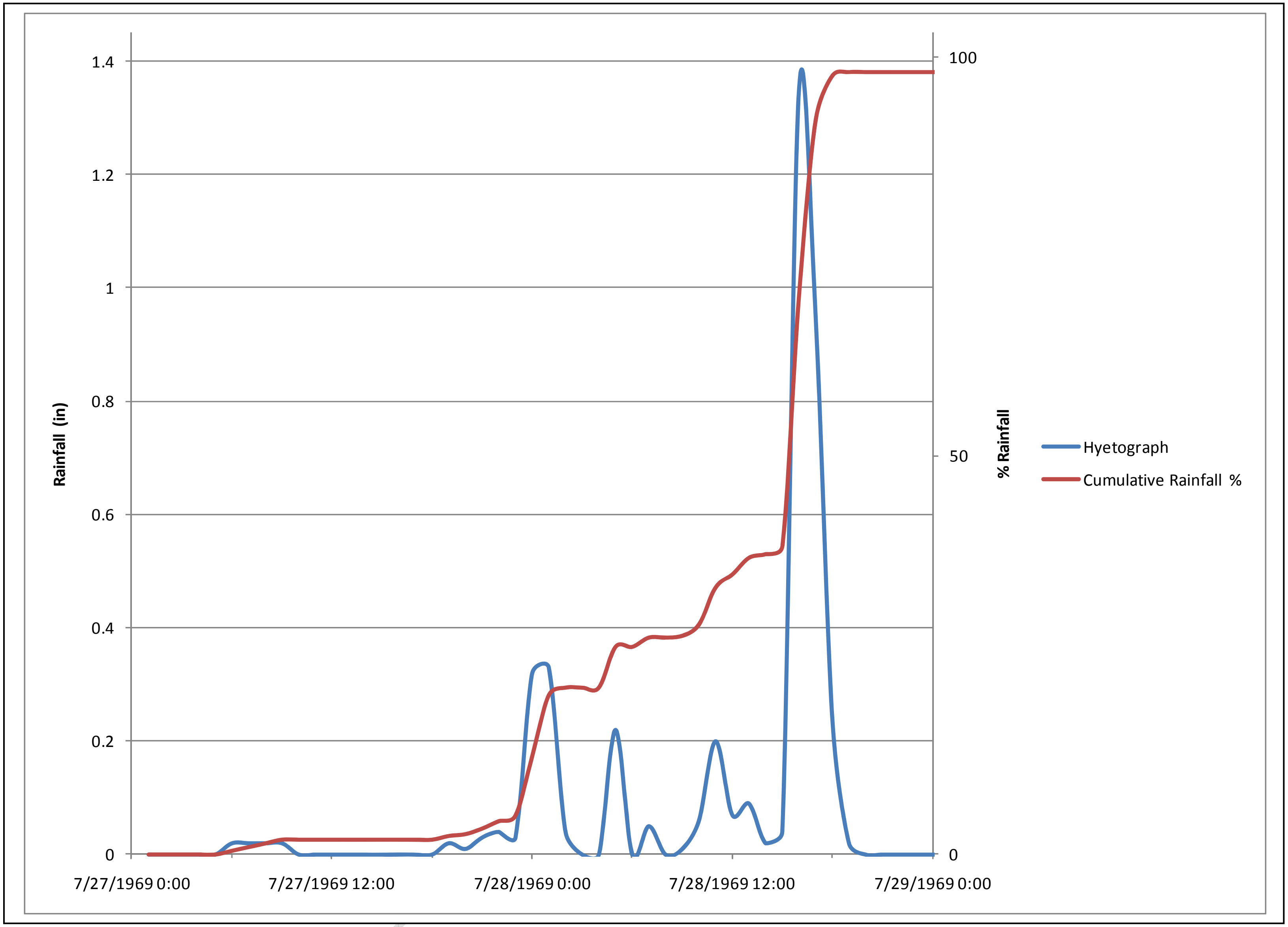


Figure 5.20 July1969 Event - Ellenville Hyetograph and % Rainfall

Table 5.14

July 1969 Event – Sub-basin Precipitation Totals – Part 1

Subbasin	Total Depth (in)
C-10	5
C-100	5
C-110	4.75
C-120	5
C-130	4.75
C-20	4.5
C-30	4.25
C-40	4.25
C-50	4.5
C-60	5
C-70	5
C-80	5
C-90	5
L-10	6.5
L-100	5.5
L-110	5.5
L-120	5.25
L-130	5.25
L-140	5.25
L-145	5.25
L-150	5.25
L-160	5.25
L-170	5.25
L-180	5.25
L-190	5.25
L-20	6
L-200	5.25
L-210	5
L-25	6
L-30	6
L-35	6
L-37	6
L-40	6
L-50	5.5
L-60	5.75
L-70	5.5
L-75	5.5
L-80	5.5
L-90	5.5
T-10	4.5
T-20	4.5
T-30	4.5

Table 5.15

July 1969 Event – Sub-basin Precipitation Totals – Part 2

Subbasin	Total Depth (in)
W-10	7
W-100	7
W-110	7
W-120	7
W-130	7
W-135	6.5
W-140	6.75
W-145	6.25
W-147	6.25
W-150	6.25
W-160	7
W-170	6.75
W-175	6
W-180	6.25
W-190	6.5
W-20	7
W-200	6
W-210	5.5
W-220	5.5
W-230	5.5
W-240	5.5
W-250	5.5
W-260	5.5
W-265	5.5
W-270	5.5
W-275	5
W-280	5
W-290	5.5
W-30	7
W-300	5
W-310	5
W-320	5
W-330	5
W-340	4.75
W-350	4.75
W-360	4.75
W-370	4.75
W-40	7
W-50	7
W-55	7
W-60	7
W-70	7
W-80	7
W-90	7

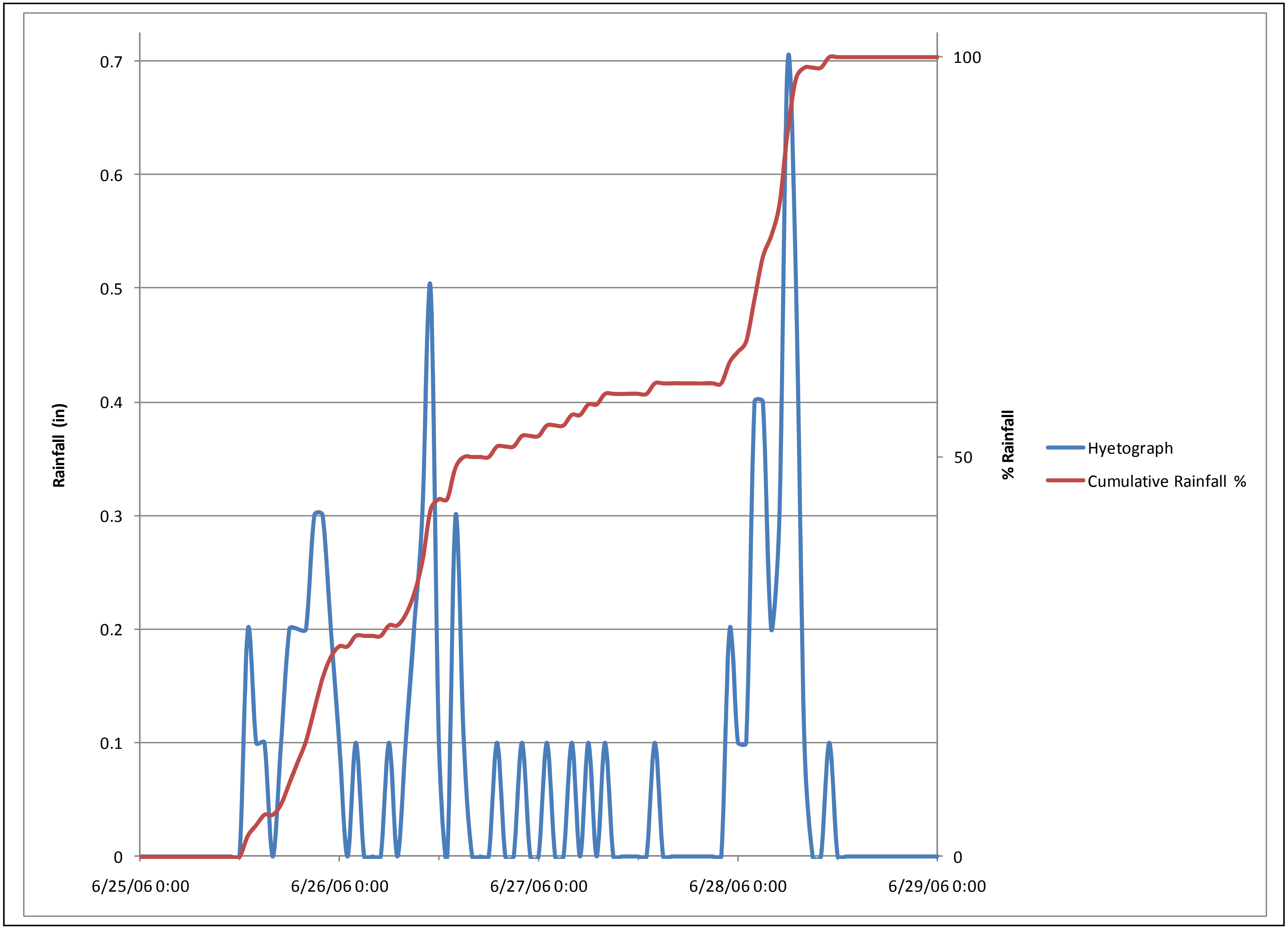


Figure 5.21 June 2006 Event - Claryville Hyetograph and % Rainfall

Subbasin	Total Depth (in)
C-10	8.5
C-100	8.5
C-110	8.5
C-120	8.5
C-130	8.5
C-20	8.5
C-30	9
C-40	9
C-50	8.5
C-60	8.5
C-70	8.5
C-80	8.5
C-90	8.5
L-10	8
L-100	8
L-110	8
L-120	8
L-130	8
L-140	8
L-145	8
L-150	8
L-160	8
L-170	8
L-180	8
L-190	8
L-20	8
L-200	8
L-210	8
L-25	8
L-30	8
L-35	8
L-37	8
L-40	8
L-50	8
L-60	8
L-70	8
L-75	8
L-80	8
L-90	8
T-10	9
T-20	9
T-30	9

Table 5.16

June 2006 Event – Sub-basin Precipitation Totals – Part 1

Subbasin	Total Depth (in)
W-10	8
W-100	8
W-110	8
W-120	8
W-130	8
W-135	8
W-140	8
W-145	8
W-147	8
W-150	8
W-160	8
W-170	8
W-175	8
W-180	8
W-190	8
W-20	8
W-200	8
W-210	8
W-220	8
W-230	8
W-240	8
W-250	8
W-260	8
W-265	8
W-270	8
W-275	8
W-280	8
W-290	8
W-30	8
W-300	8
W-310	8
W-320	8.5
W-330	8.5
W-340	8.5
W-350	8.5
W-360	8.5
W-370	8.5
W-40	8
W-50	8
W-55	8
W-60	8
W-70	8
W-80	8
W-90	8

Table 5.17

June 2006 Event – Sub-basin Precipitation Totals – Part 2

NOAA National Weather Service (NWS) Hydrologic Data Systems Group (HDSG) Multisensor Precipitation Estimator (MPE) Data was available for the Tropical Storm Ivan event of 2004. This type of precipitation uses NEXRAD radar precipitation estimates and adjusts the magnitudes and spatial/temporal distributions to agree with ground based precipitation gaging stations.

The MPE data was gathered from the HDSG website in XMRG format at hourly intervals and transformed into Standard Hydrologic Grid (SHG) precipitation at 2000 x 2000 meter grid sizes (approximately 1.54 miles) using tools available through HEC. The MPE precipitation distribution for 9/18/2004 at 2:00 PM is shown in Figure 5.22, where precipitation increases in intensity from blue to red.

This type of precipitation data requires the use of a grid network within HEC-HMS. ESRI's ArcMap version 9.2 was used to create the associated grid file. Figures 5.23 shows the grids placed over the area of interest. Tables 5.18 and 5.19 contain the Tropical Storm Ivan event total precipitation amounts for each sub-basin.

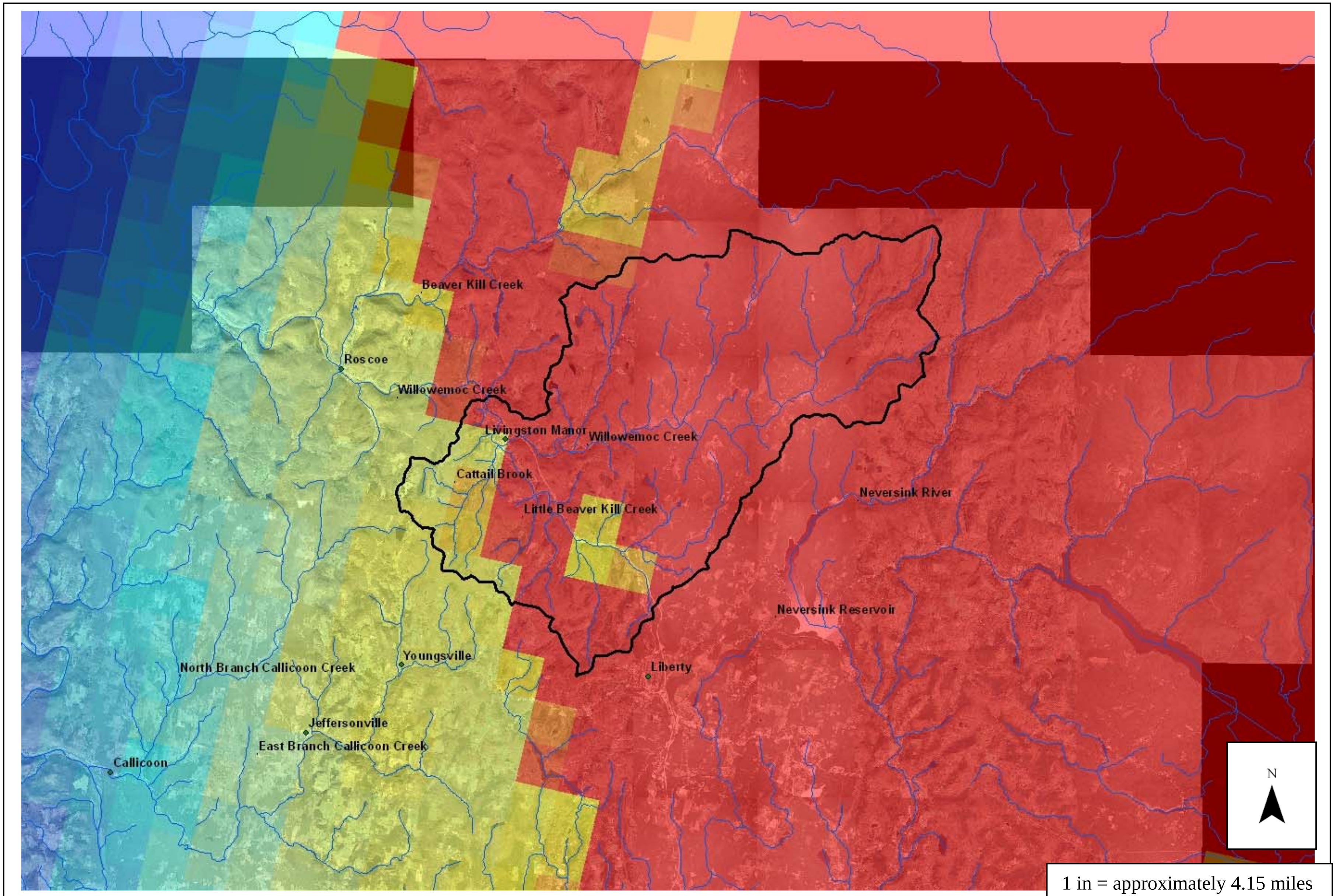


Figure 5.22 MPE Precipitation Distribution - 9/18/2004 2:00 PM

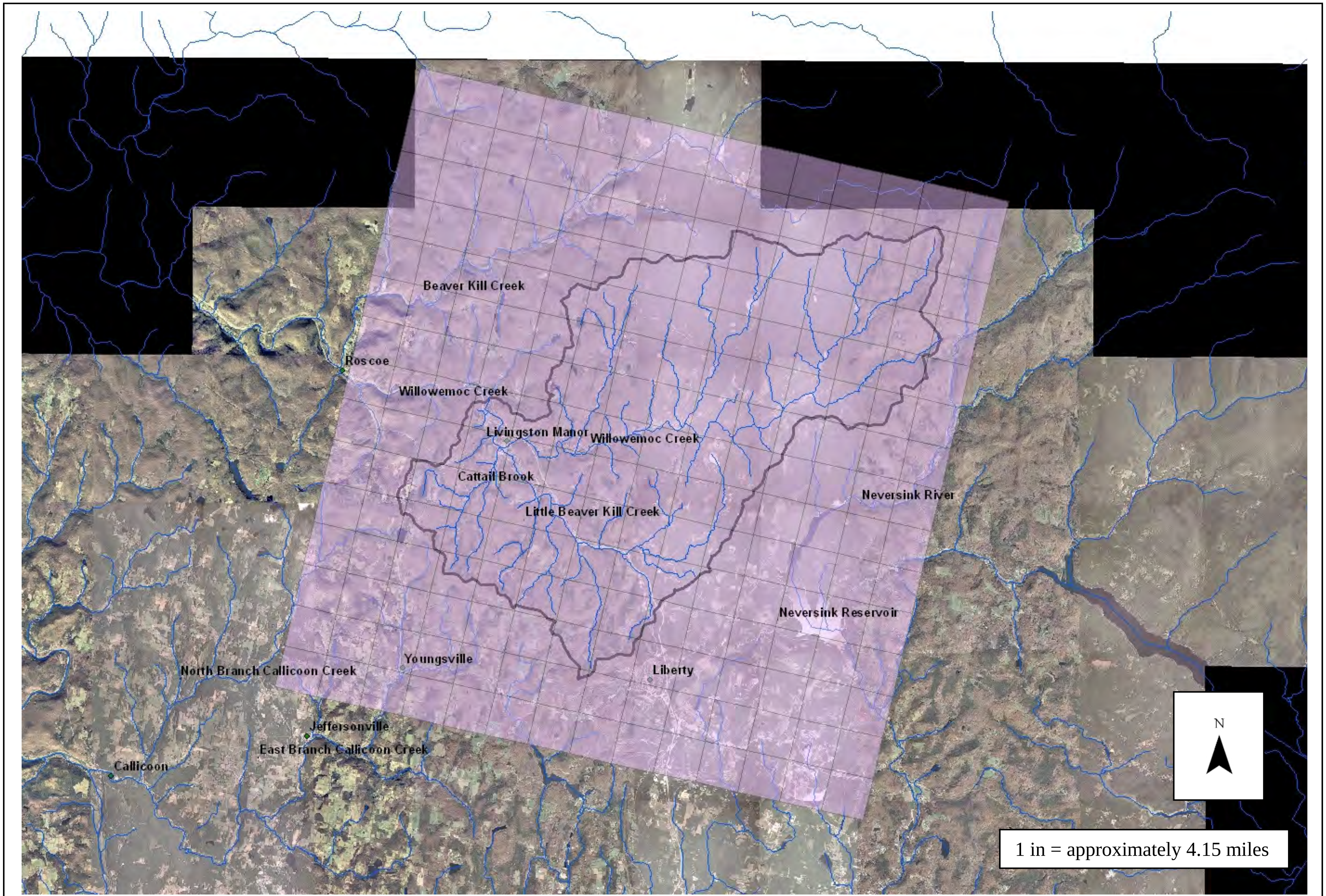


Figure 5.23 SHG 2000 x 2000 m Grids

Tables 5.18

Tropical Storm Ivan Event - Sub-basin Precipitation Totals – Part 1

Subbasin	Total Precipitation (in)
C-10	5.55
C-100	5.95
C-110	5.90
C-120	5.88
C-130	5.97
C-20	5.92
C-30	5.91
C-40	5.77
C-50	5.90
C-60	5.87
C-70	5.63
C-80	5.74
C-90	5.54
L-10	5.70
L-100	5.89
L-110	5.59
L-120	5.80
L-130	5.81
L-140	5.76
L-145	5.70
L-150	5.65
L-160	5.65
L-170	5.58
L-180	5.63
L-190	5.76
L-20	5.90
L-200	6.10
L-210	5.75
L-25	5.81
L-30	5.73
L-35	5.56
L-37	5.71
L-40	5.71
L-50	5.84
L-60	5.91
L-70	5.97
L-75	5.98
L-80	5.69
L-90	5.85
T-10	5.67
T-20	5.82
T-30	5.89

Tables 5.19

Tropical Storm Ivan Event - Sub-basin Precipitation Totals – Part 2

Subbasin	Total Precipitation (in)
W-10	5.55
W-100	5.95
W-110	5.90
W-120	5.88
W-130	5.97
W-135	5.92
W-140	5.91
W-145	5.77
W-147	5.90
W-150	5.87
W-160	5.63
W-170	5.74
W-175	5.54
W-180	5.70
W-190	5.89
W-20	5.59
W-200	5.80
W-210	5.81
W-220	5.76
W-230	5.70
W-240	5.65
W-250	5.65
W-260	5.58
W-265	5.63
W-270	5.76
W-275	5.90
W-280	6.10
W-290	5.75
W-30	5.81
W-300	5.73
W-310	5.56
W-320	5.71
W-330	5.71
W-340	5.84
W-350	5.91
W-360	5.97
W-370	5.98
W-40	5.69
W-50	5.85
W-55	5.67
W-60	5.82
W-70	5.89
W-80	5.83
W-90	5.98

Gridded precipitation was also used for the July – August 2009 event. While MPE data was not available for this event, several recording gages with ¼ hour recording intervals were available within close proximity to the area of interest. The five gages that were used include:

- Callicoon
- Claryville
- Mongaup Valley
- Prompton Dam
- Tannersville

Each gage's temporal and spatial distributions along with their associated magnitudes were distributed using an Inverse Distance Weighting scheme. Tools available through HEC were used to create gridded precipitation datasets. The five hyetographs and cumulative rainfall amounts are shown in Figures 5.24 and 5.25 while Tables 5.20 and 5.21 contain the total precipitation amounts for each sub-basin.

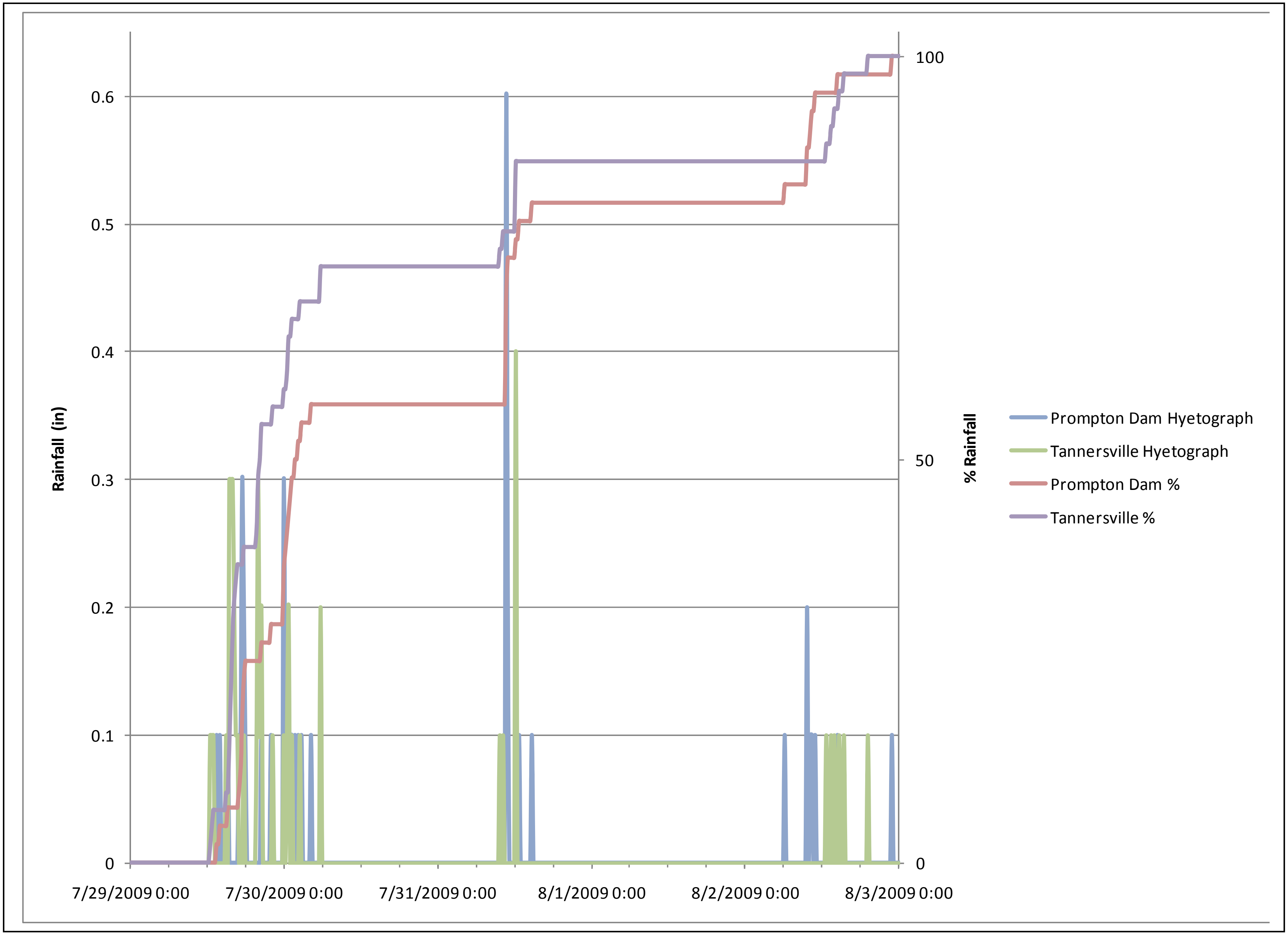


Figure 5.24 July – August 2009 Event - Hyetographs and Cumulative % Rainfall – Part 1

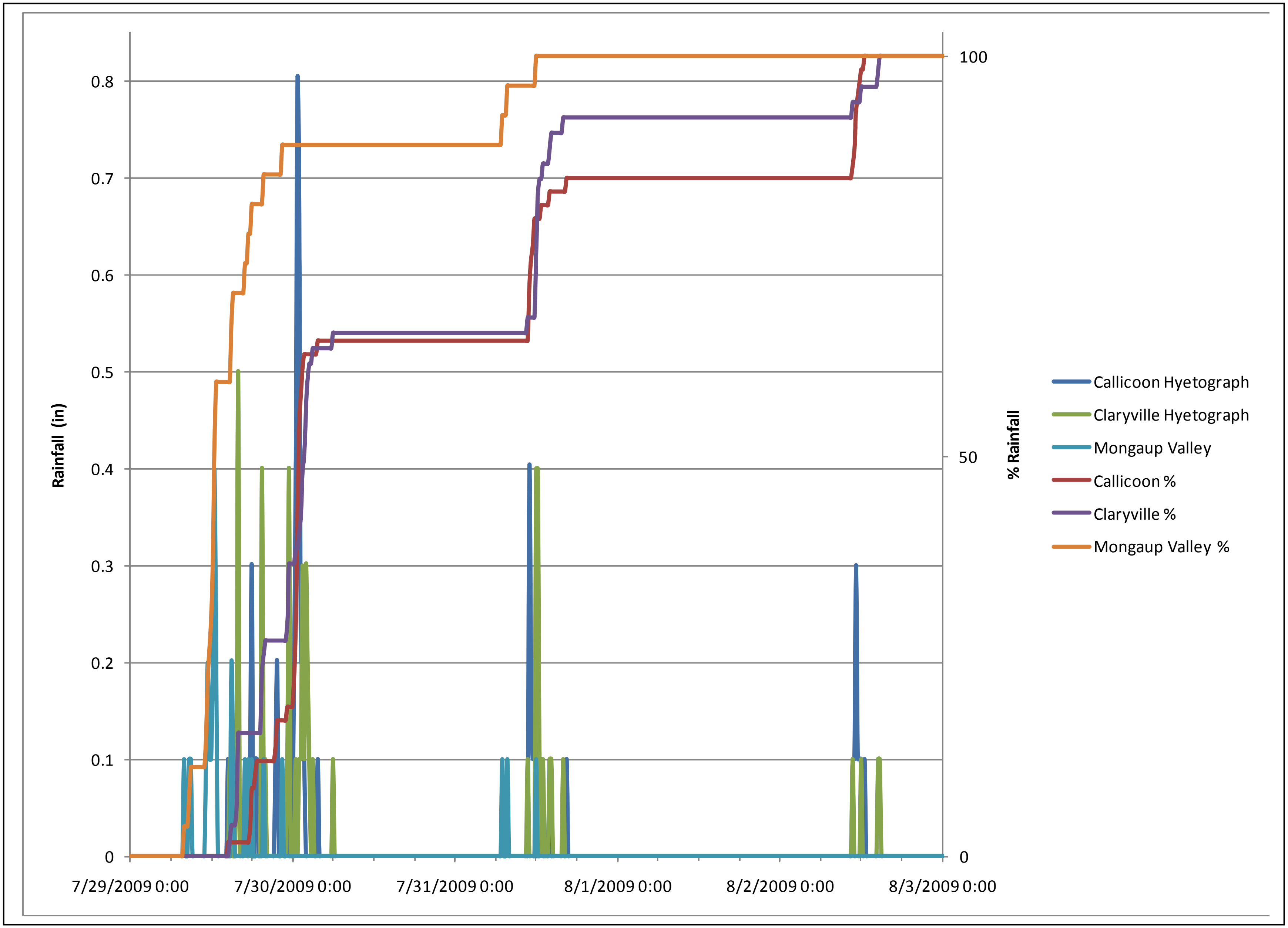


Figure 5.25 July – August 2009 Event - Hyetographs and Cumulative % Rainfall – Part 2

Table 5.20

July – August 2009 Event - Sub-basin Precipitation Totals – Part 1

Subbasin	Total Precipitation (in)
C-10	4.90
C-100	4.88
C-110	4.88
C-120	4.88
C-130	4.88
C-20	4.90
C-30	4.92
C-40	4.90
C-50	4.89
C-60	4.89
C-70	4.89
C-80	4.88
C-90	4.88
L-10	4.84
L-100	4.79
L-110	4.82
L-120	4.85
L-130	4.84
L-140	4.84
L-145	4.85
L-150	4.85
L-160	4.85
L-170	4.87
L-180	4.87
L-190	4.86
L-20	4.84
L-200	4.87
L-210	4.86
L-25	4.82
L-30	4.82
L-35	4.84
L-37	4.83
L-40	4.84
L-50	4.83
L-60	4.80
L-70	4.83
L-75	4.84
L-80	4.84
L-90	4.83
T-10	4.89
T-20	4.88
T-30	4.89

Table 5.21

July – August 2009 Event - Sub-basin Precipitation Totals - Part 2

Subbasin	Total Precipitation (in)
W-10	4.93
W-100	4.88
W-110	4.88
W-120	4.86
W-130	4.88
W-135	4.86
W-140	4.87
W-145	4.85
W-147	4.86
W-150	4.86
W-160	4.87
W-170	4.87
W-175	4.87
W-180	4.87
W-190	4.87
W-20	4.91
W-200	4.86
W-210	4.86
W-220	4.85
W-230	4.86
W-240	4.86
W-250	4.85
W-260	4.85
W-265	4.86
W-270	4.86
W-275	4.86
W-280	4.86
W-290	4.86
W-30	4.92
W-300	4.86
W-310	4.86
W-320	4.87
W-330	4.87
W-340	4.88
W-350	4.88
W-360	4.89
W-370	4.89
W-40	4.90
W-50	4.91
W-55	4.98
W-60	4.94
W-70	4.91
W-80	4.89
W-90	4.90

3. Calibration - HMS

Two USGS stream gages that were active within the area of interest and time period in question are USGS (01420000) Little Beaver Kill near Livingston Manor, NY and USGS (01419500) Willowemoc Creek near Livingston Manor, NY. Gage heights (referenced to a gage-specific datum) were available for both gages. Daily average and yearly peak flow values were measured and/or extrapolated for both gages. Daily flows were used to calibrate the initial base flow estimates while the yearly peak flow and gage height values were used to calibrate / verify event specific conditions.

The Little Beaver Kill gage has a period of record dating from WY 1925 – 1981 with a high water mark reported on 9/18/2004 while the Willowemoc gage has a period of record dating from 1938 – 1973 with high water marks for 1/19/1996 and 9/18/2004. The location of these gages within the area of interest is shown in Figure 5.26.

The USGS paper entitled "Summary of Floods in the United States During 1969" contains a stage hydrograph and calculated flow hydrograph for the Little Beaver Kill gage from July 27 – 30. It also contained a stage hydrograph for the Willowemoc gage from July 27 – 30 which was converted into a flow hydrograph using the applicable rating table.

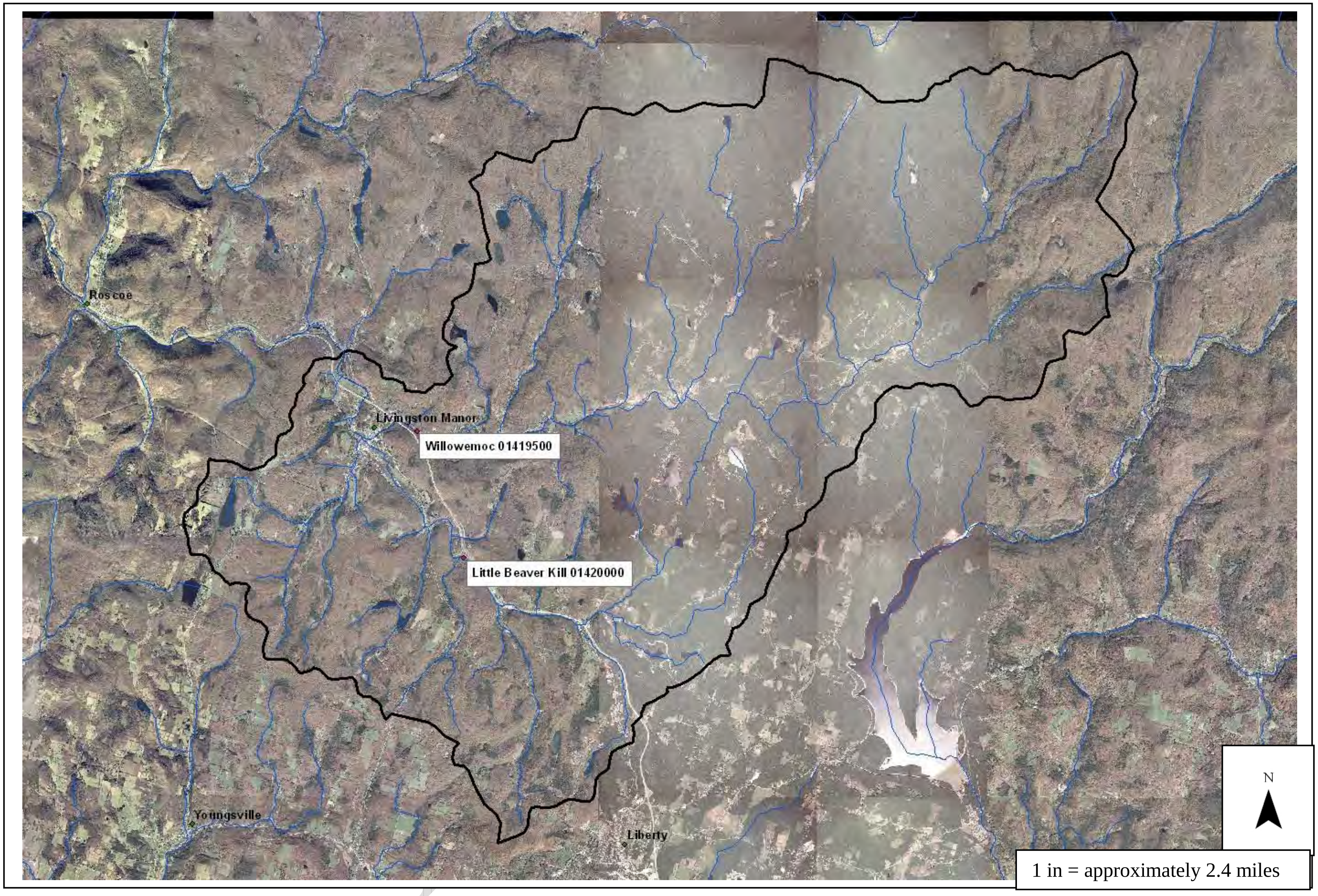


Figure 5.26 USGS Stream Gage Locations

The three historic precipitation events were applied to the HEC-HMS model and the corresponding flow values were compared to the available calibration data. Changes that were made to recreate event-specific conditions include:

- August 1955 event
  - Decreasing the impervious area from 2001 conditions by 75%
  - Condensing the event time period from August 11 – 20 to August 17 – 20
    - This was due to the inability of HEC-HMS's Green & Ampt infiltration model to redistribute moisture between precipitation events. HEC recommends a maximum time period of 7 days between events.
  - Reducing the volumetric moisture deficit (increased saturation) by 75%
    - This accounted for the rainfall during August 11 – 15.

- July 1969 event
  - Decreasing the impervious area from 2001 conditions by 50%
  - Reducing the volumetric moisture deficit (increased saturation) by 50%
    - This accounted for rainfall (½ - 1 inch) that occurred 1-2 weeks before the event

- Tropical Storm Ivan event, September 2004
  - Reducing the volumetric moisture deficit (increased saturation) by 50%
    - This accounted for rainfall from Hurricane Frances 10 days earlier
  - Reduced saturated hydraulic conductivity by 35%

Clark Storage values (R) were changed in an effort to match the USGS stream flow gage records. These changes were based upon the HEC paper entitled "Hydrologic Analysis of Prompton Reservoir Modifications, Lackawaxen River Basin, Pennsylvania" (1988). The Little Beaver Kill creek sub-basin "R" values were raised to create an $R / (T_c + R)$ relationship equal to 0.65.

The Muskingum-Cunge routing reach, willow_abv_LM 2001, was split into three separate routing reaches, along with their respective eight-point cross sections. This was done to represent drastic changes in topography.

Calibrated hydrographs for each event are shown in Figures 5.27 – 5.30.

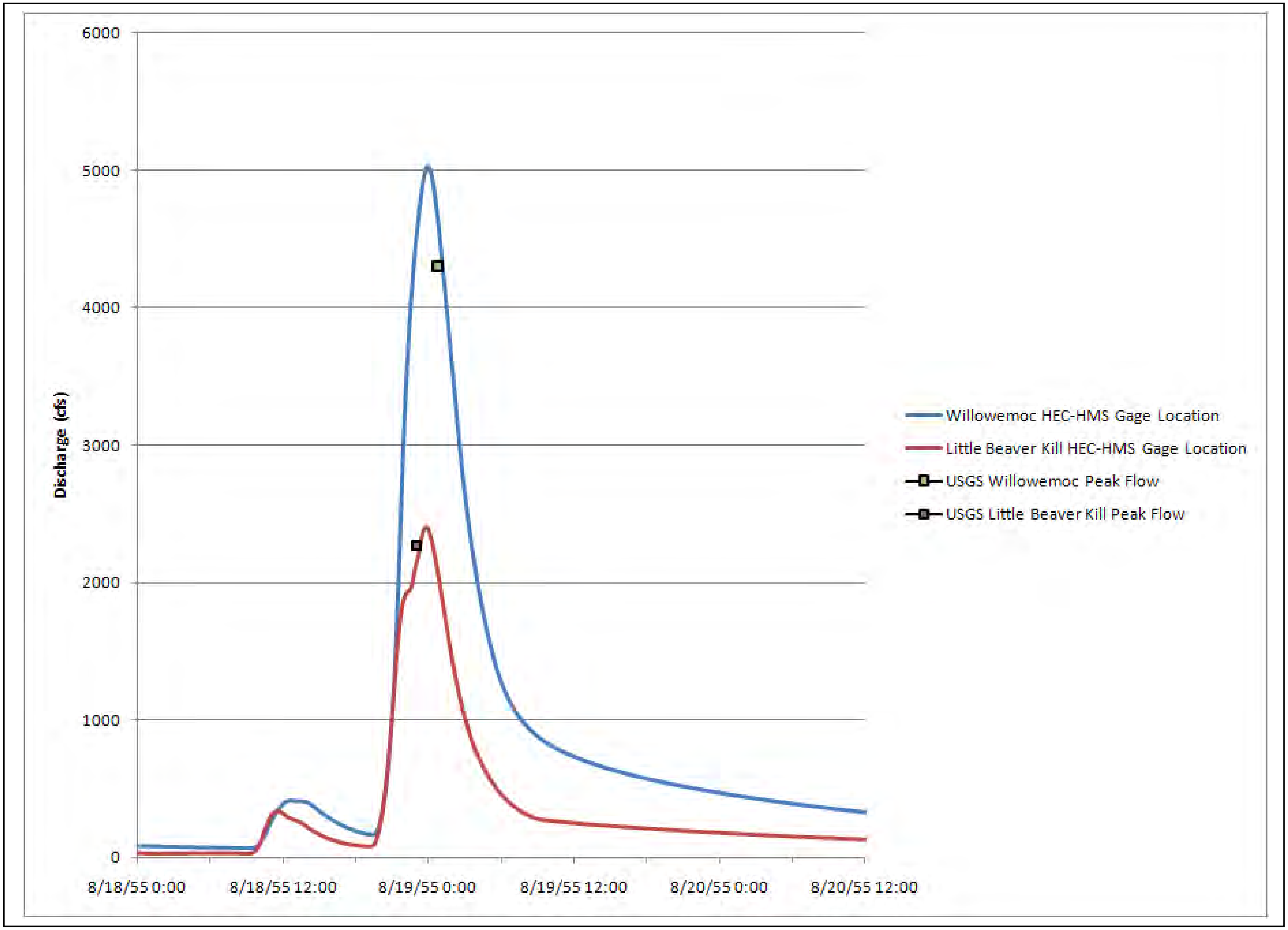


Figure 5.27 August 1955 Event - HEC-HMS Willowemoc and Little Beaver Kill Gage Location Hydrographs

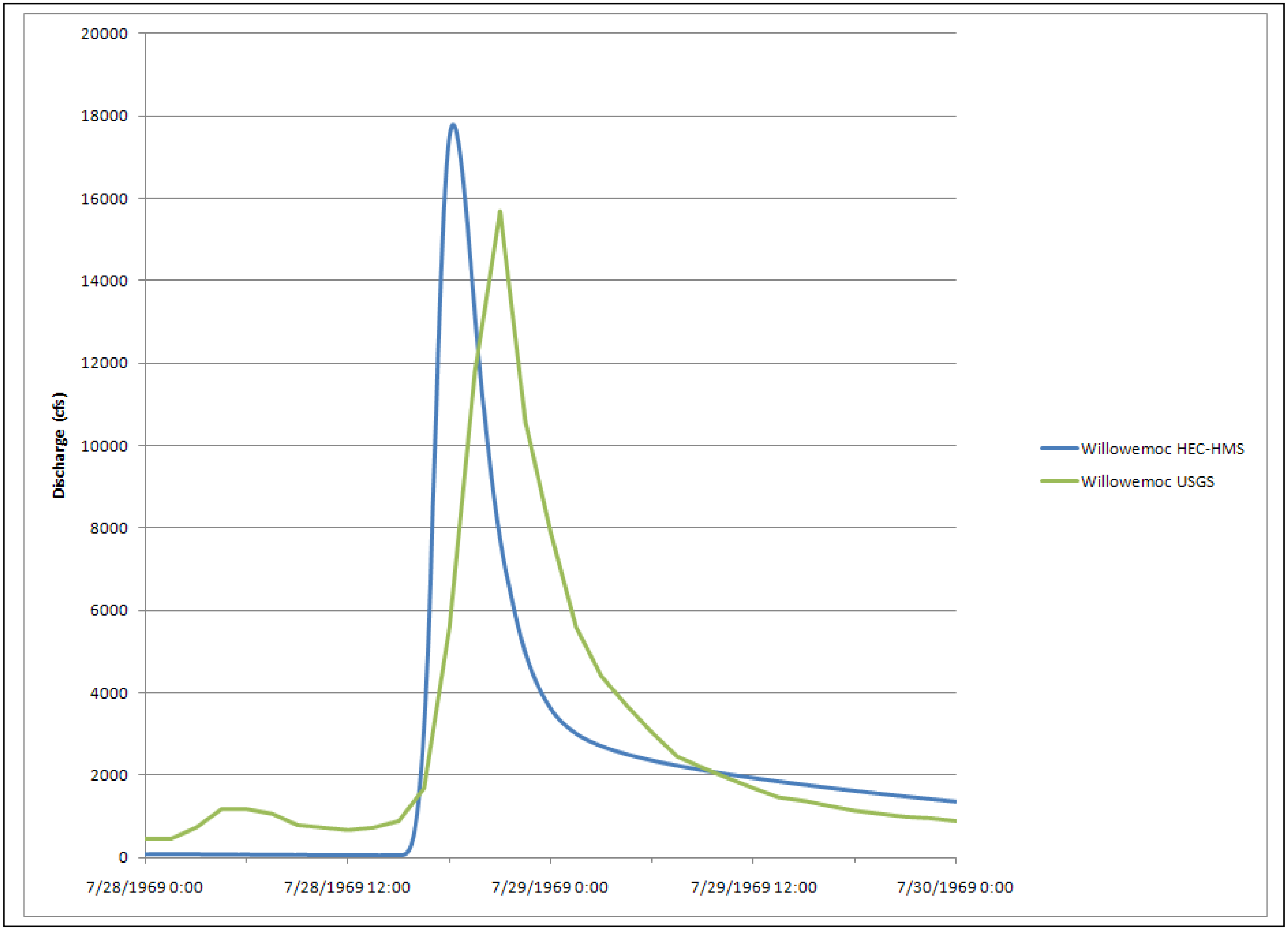


Figure 5.28 July 1969 Event - HEC-HMS Willowemoc Gage Location Hydrographs

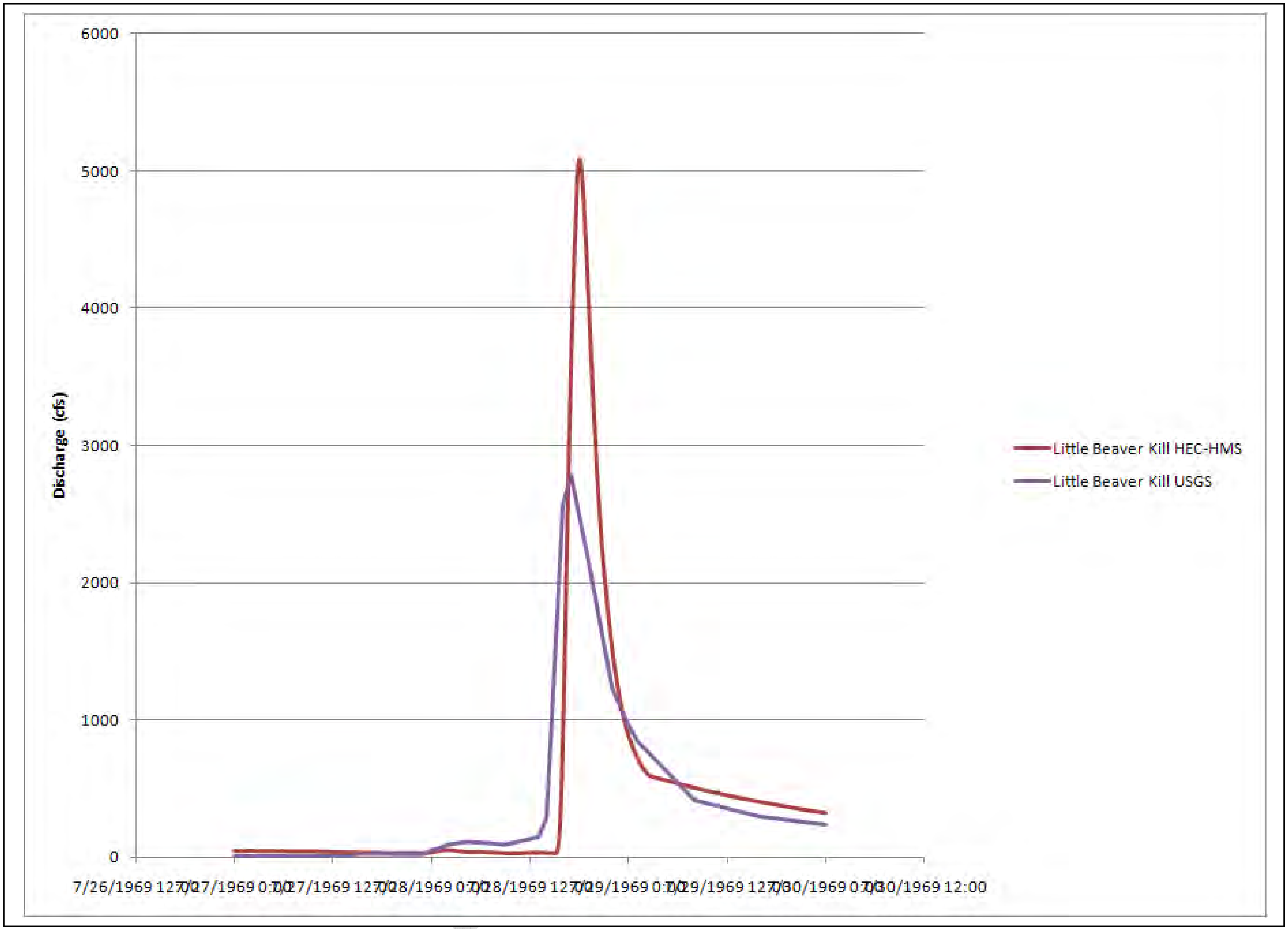


Figure 5.29 July 1969 Event - HEC-HMS Little Beaver Kill Gage Location Hydrographs

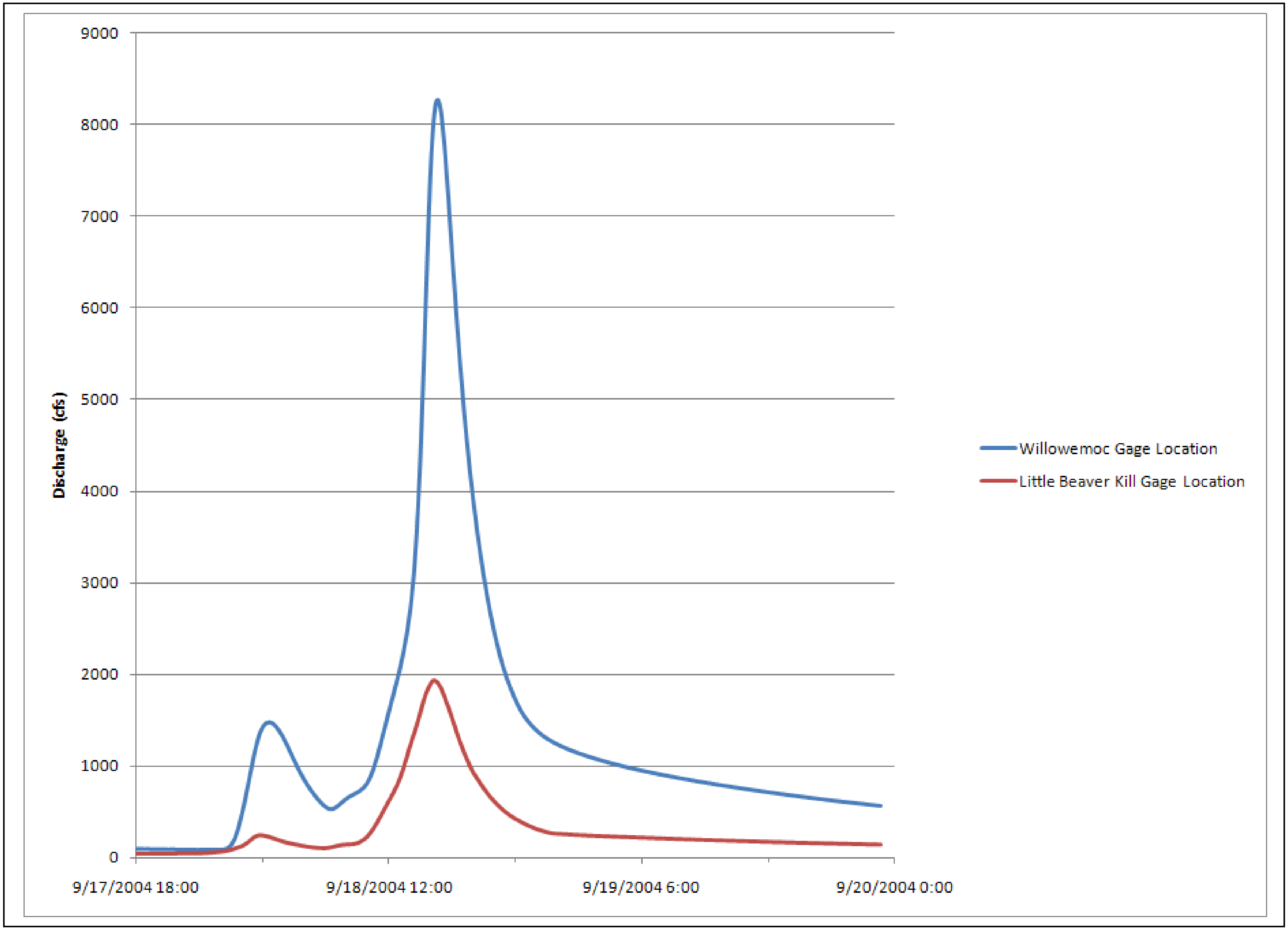


Figure 5.30 Tropical Storm Ivan Event - HEC-HMS Willowemoc and Little Beaver Kill Gage Location Hydrographs

a. Results - HMS

Calibration success for each historic event, compared to the published USGS stream gage data at both sites, was mixed. The differences between the August 1955 and July 1969 event peak flow values at both gage sites are shown in Tables 5.22 and 5.23.

The high water marks at each gage site that were taken on 9/18/2004 were linearly extrapolated into flow values using the last rating table available for each site. The resulting flows for the Little Beaver Kill and Willowemoc USGS stream gage sites were 2400 cfs and 10615 cfs, respectively. The difference between the USGS and HEC-HMS estimated flows were 473 and 2350 cfs for the Little Beaver Kill and Willowemoc, respectively. However, these values should be considered as rough estimates. Substantial changes to stream geometries at both sites during the 20+ years between the creation of the last rating table and 2004 could render those estimates moot.

While the differences seem large, the peak flows from each of the historic precipitation events were used in hydraulic models and compared to measured high water marks (see Hydraulic Model section). These high water marks were at both the Willowemoc and Little Beaver Kill USGS gages and throughout the area of interest. The high water marks compared well and validated the differences seen between the HEC-HMS peak flows and those extrapolated / calculated at the USGS gages.

Tables 5.22

Differences Between HEC-HMS Output and Willowemoc USGS Data

Willowemoc Creek						
	HEC-HMS		USGS		Peak Flow Rate % Difference	Time Difference (hr)
EVENT	Peak Flow Rate (cfs)	Time	Peak Flow Rate (cfs)	Time		
August 1955	5027	8/18/55 11:55 PM	4300	8/19/55 12:45 AM	16.91	0.83
July 1969	17769	7/28/69 6:15 PM	15700	7/28/69 9:00 PM	13.18	2.75

Tables 5.23

Differences Between HEC-HMS Output and Little Beaver Kill USGS Data

Little Beaver Kill Creek						
	HEC-HMS		USGS		Peak Flow Rate % Difference	Time Difference (hr)
EVENT	Peak Flow Rate (cfs)	Time	Peak Flow Rate (cfs)	Time		
August 1955	2408	8/18/55 11:45 PM	2270	8/18/55 11:00 PM	6.08	0.75
July 1969	5086	7/28/69 6:00 PM	2780	7/28/69 5:00 PM	82.95	1

b. Sources of Error - HMS

There were several limitations for calibrating the HEC-HMS model to the three chosen historic precipitation events. These include:

- The USGS Willowemoc and Little Beaver Kill stream gages only recorded gage heights relative to the individual gage's datum. These stages were then converted to a flow using a rating curve for each gage.
  - The 1955 event's peak flow rate at the Willowemoce gage of 4300 cfs was extrapolated from the last calculated flow value of 1100 cfs.
  - In the case of the July 1969 event, the extrapolated flow value at the Willowemoc gage was **eight** times greater than the last calculated flow value.
  - Errors could arise from the extrapolation of the rating curve to reach the extreme flows seen for these events.
- There were no recording precipitation gages within the area of interest for any of the historic precipitation events used.
  - For both the 1955 and 1969 events, the temporal rainfall pattern was taken from the NCDC precipitation gage at Ellenville, which is over 18 miles from the area of interest. Though the magnitudes and spatial distributions were taken from gages and published rainfall figures, the temporal pattern could vary from the Ellenville gage to the area of interest.
  - Though the MPE data is, theoretically, the best possible rainfall data available, it is not without limitations. Elevation changes, dust / particulates, and availability of recording precipitation gages to adjust radar estimates are just some of the sources of possible error.

4. Validation - HMS

Validation runs were preformed to assess the legitimacy of the calibrated model parameters. Since Tropical Storm Ivan was the most recent event with a large amount of calibration data, the calibrated HEC-HMS model conditions for that event were used for the validation runs. The two events chosen for validation were:

- June 2006 event
- July – August 2009 event

HEC-HMS flow hydrographs for each event at both the Little Beaver Kill and Willowemoc USGS stream gage sites are shown in Figures 5.31 and 5.32.

No flow values for reference were available for either event. However, high water marks were available at certain locations throughout the area of interest. The flows from each event were used as input to hydraulic models. The resulting water surfaces were compared to these high water marks (see Hydraulic Model section).

The HEC-HMS model is acceptable for the purposes of this technical effort.

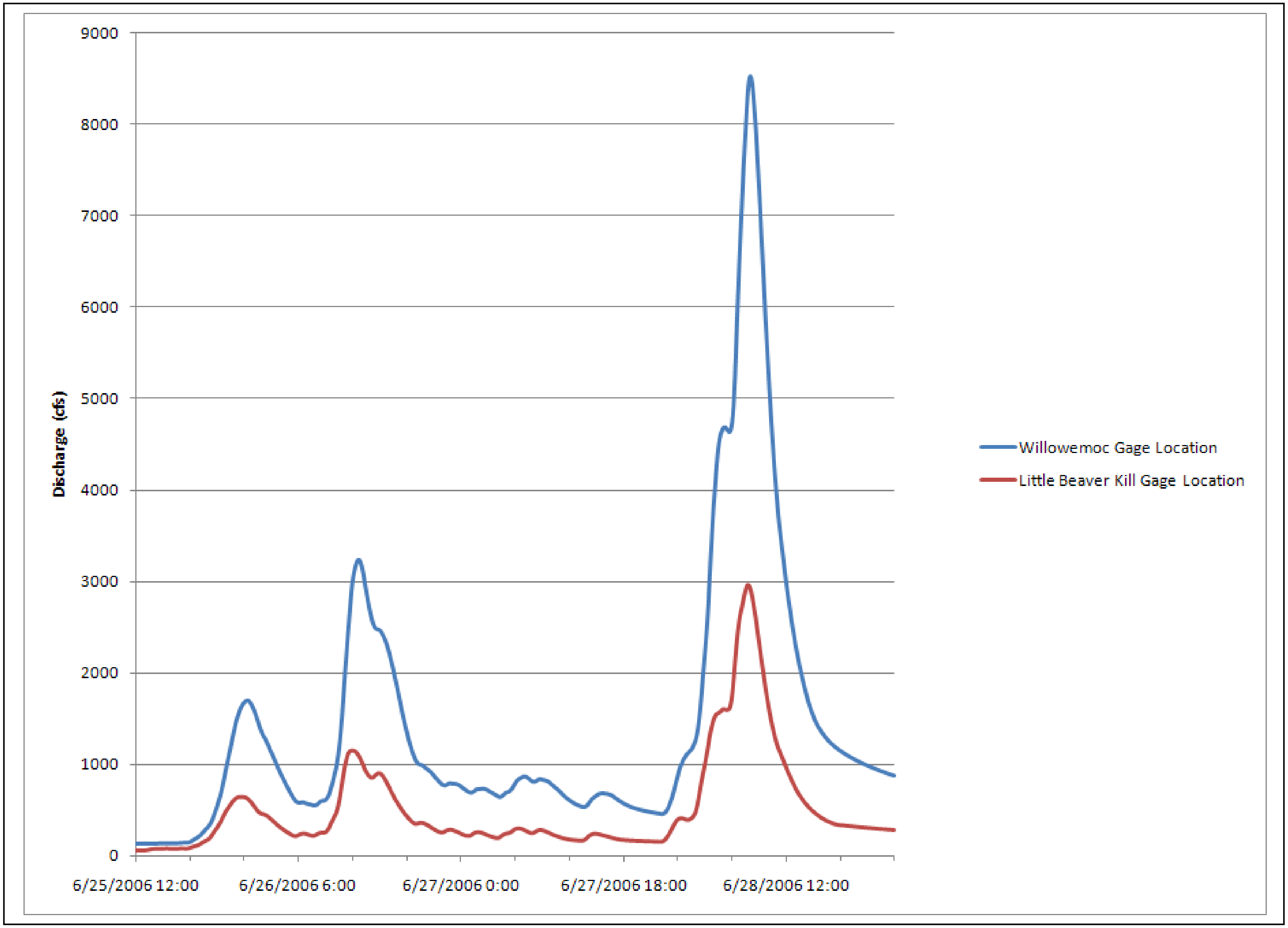


Figure 5.31 June 2006 Event - Gage Location Hydrographs

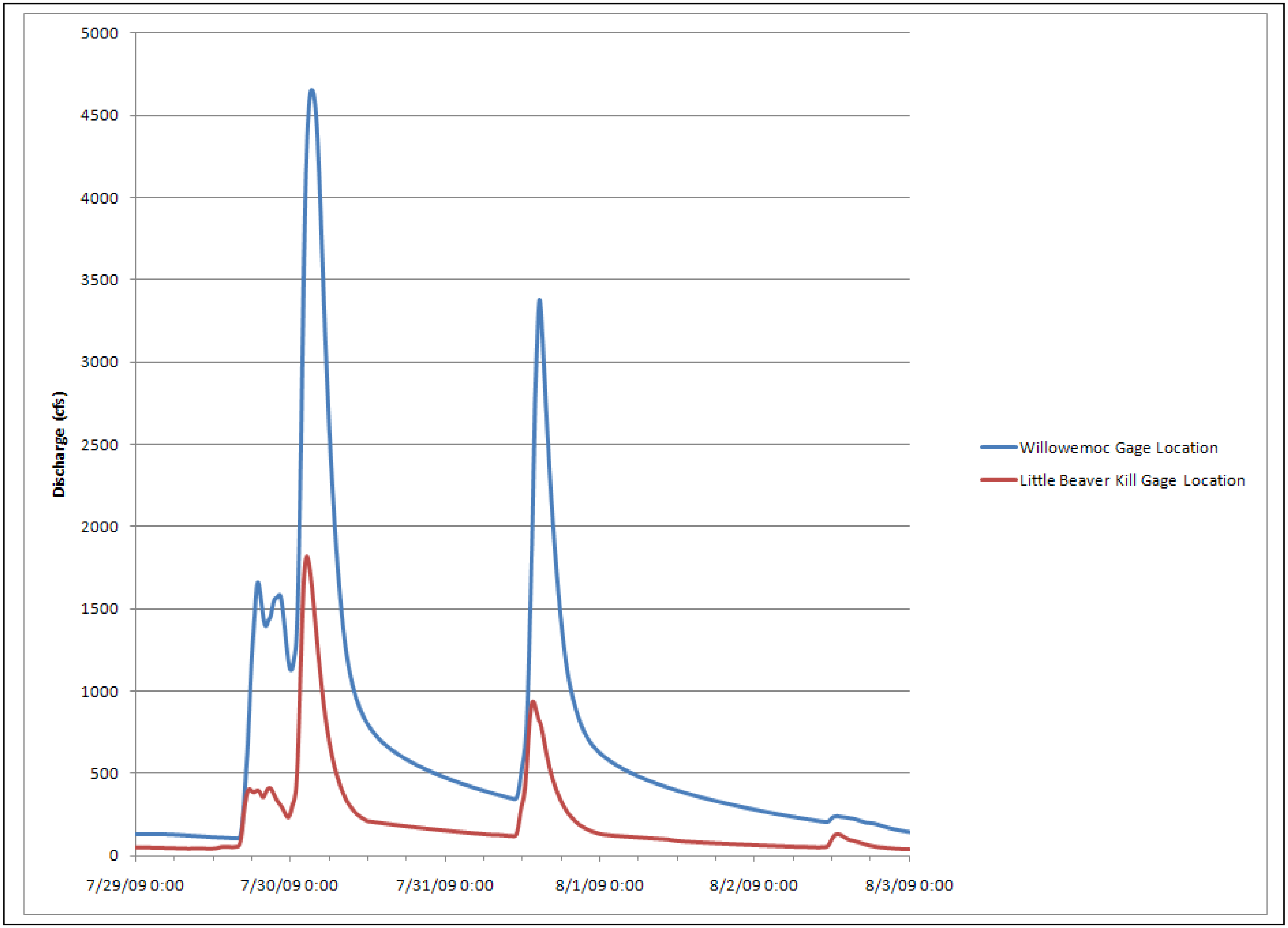


Figure 5.32 July – August 2009 Event - Gage Location Hydrographs

5. Frequency Precipitation

Annual depth-duration-frequency precipitation was tabulated using Point Precipitation Frequency Estimates from NOAA Atlas 14. New York does not have available Atlas 14 point precipitation data online. Therefore, the closest point in Pennsylvania (approximately 19 miles away) was used. The 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.005, and 0.002 chance events were created using the precipitation values contained within Table 5.24.

Table 5.24

Annual Depth-Duration-Frequency Precipitation

Event (year)	5min	10min	15min	30min	60min	2hr	3hr	6hr	12hr	24hr
2	0.33	0.51	0.62	0.83	1.02	1.22	1.33	1.69	2.12	2.51
5	0.41	0.64	0.79	1.08	1.36	1.62	1.76	2.21	2.78	3.30
10	0.47	0.73	0.90	1.25	1.59	1.91	2.07	2.59	3.28	3.91
25	0.55	0.84	1.04	1.47	1.90	2.33	2.52	3.15	4.01	4.83
50	0.61	0.93	1.15	1.64	2.16	2.69	2.91	3.65	4.66	5.66
100	0.68	1.02	1.27	1.83	2.45	3.09	3.36	4.21	5.41	6.64
200	0.75	1.12	1.39	2.03	2.76	3.55	3.88	4.86	6.28	7.80
500	0.86	1.26	1.58	2.34	3.23	4.28	4.68	5.89	7.65	9.67
1000	0.95	1.38	1.73	2.59	3.64	4.92	5.40	6.81	8.90	11.41

6. HYDRAULIC MODEL

Discharges, both historic and frequency, were transformed into water surface elevations with the USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 4.1.

HEC-RAS is a gradually varied flow model capable of analyzing both steady and unsteady state flow conditions. This study ran all of the RAS models in steady state.

A. Set Up

HEC-GeoRAS is a pre-processor program for HEC-RAS. It is a geo-spatial extension for ArcMap version 9.2 and was used to prepare and refine the HEC-RAS input files. Three primary HEC-RAS models were created within the area of interest, Willowemoc creek, Little Beaver Kill creek, and Cattail Brook with lengths of 14641, 6972, and 5975 feet, respectively.

Cross section and bridge geometries were initially drawn from LIDAR elevation models. Elevations were then modified as necessary using various bridge surveys, recent field surveys, aerial photographs, and previous FEMA FIS HEC-RAS models.

Portions of Livingston Manor along the Willowemoc creek are protected by levees on both the left and right overbanks. However, for some events the levees are flanked at the upstream end and/or overtopped. Under such conditions the water surface elevations in the main channel of the Willowemoc are not the same as the water surface elevations in the "back channels" behind the levees.

The Willowemoc floodplain was analyzed with three models: a main stem Willowemoc model and two back channel models. The back channel on the right overbank is labeled, "channel behind the school". This channel extends from Willowemoc cross-section 7489 to 9956.   The back channel on the left overbank is labeled, “channel behind the levee on the LOB”. This channel extends from Willowemoc cross-section 6969 to 7920. The limits of the five RAS models were set to encompass all known damage locations and all locations of possible hydraulic solutions.

The main stem Willowemoc HEC-RAS model required the use of 62 cross sections and 5 bridge crossings. These bridges include:

- Covered Bridge Road
- Route 17 bridge below Livingston Manor
- Foot Bridge leading to High School
- Old Route 17
- Route 17 bridge above Livingston Manor

The levees along the Willowemoc are modeled as lateral structures. The levees' elevations, which were field surveyed, determine the discharges for the two back channel models and correspondingly the flow that remains in the main channel of the Willowemoc creek downstream of the diversion points. As such, the modeling of the levees is critical to accurate water surface elevations throughout Livingston Manor.

Pertinent information for the levees modeled as lateral structures is provided in Table 6.1. The levees provide approximately a 50 year level of protection and historically the levees have not failed from seepage. While larger events may fail the levee in spots, the maximum water surface profile along the river before failure is likely to be the water surface elevations that obtain under the assumption of steady state overtopping without failure. Hence for purposes of this analysis the levees were assumed not to fail during overtopping and interior water surface elevations were assumed not to exert a backwater effect on the levee.

The locations of the cross sections, bridges and lateral structures are shown in Figure 6.1 along with contours at a 2ft interval.

Table 6.1
Dimensions of Lateral Structures

Lateral Structure	Side of Creek	Bounding X-sections	Length (feet)	Weir Coeff	Minimum Elevation (ft-NAVD88)	Arithmetic Average Elevation (ft-NAVD88)
10078	Right	ROB old Rt. 17	440	2.6	1431.2	1432.4
9957	Right	9956 - 9854	102	2.6	1440.0	1440.0
9853	Right	9854 - 9754	136	2.6	1425.3	1426.7
9753	Right	9754 – 9579	180	2.6	1426.8	1427.1
9578	Right	9579 – 9459	126	2.6	1427.1	1427.2
9458	Right	9459 - 9350	108	2.6	1427.1	1427.1
9351	Right	9350 – 9269	81	2.6	1428.5	1428.5
9268	Right	9269 – 9141	120	2.6	1427.5	1427.8
9140	Right	9141 – 8900	219	2.6	1422.6	1426.1
8899	Right	8900 – 8696	196	2.6	1421.9	1422.6
8695	Right	8696 – 8426	235	2.6	1421.3	1421.9
8425	Right	8426 – 8226	166	2.6	1421.6	1422.1
8225	Right	8226 – 8043	175	2.6	1420.0	1421.4
8042	Right	8043 – 7920	120	2.6	1421.3	1421.4
7919	Right	7920 – 7704	213	2.6	1419.0	1420.2
7703	Right	7704 – 7489	205	2.6	1416.0	1418.9
8033	Left	U/S end of Channel behind LOB Levee	261	2.6	1415.8	1419.2
7910	Left	7920 – 7704	221	2.6	1419.4	1420.6
7694	Left	7704 – 7489	226	2.6	1419.0	1419.9
7479	Left	7489 – 7214	278	2.6	1415.0	1417.8
7204	Left	7214 - 6969	250	2.6	1408.7	1414.4

The channel behind the school has no bridges and required 24 cross-sections.
The channel behind the levee on the left over bank has no bridges and required 11 cross-sections. The locations of the cross-sections for the right overbank and left overbank back channels are shown on Figures 6.2 and 6.3 respectively. One foot contours are shown on both figures.

The Little Beaver Kill HEC-RAS model required the use of 30 cross sections and 1 bridge. The bridge is:

- Main Street

The locations of every cross section and the bridge are shown in Figure 6.4. Two foot contours are shown.

The Cattail Brook HEC-RAS model required the use of 37 cross sections and 7 bridges crossings. These bridges include:

- River Street
- An Access Road approximately 198 feet upstream
- Creamery Road
- Finch Street
- A Private Road approximately 469 feet upstream
- Hoos Road
- Main Street (County Road 149)

The locations of every cross section and bridge are shown in Figure 6.5. Two foot contours are shown.

Figure 6.1-Part 1, Willowemoc River – HEC-RAS Features (USDA 2008 Orthographic Image)

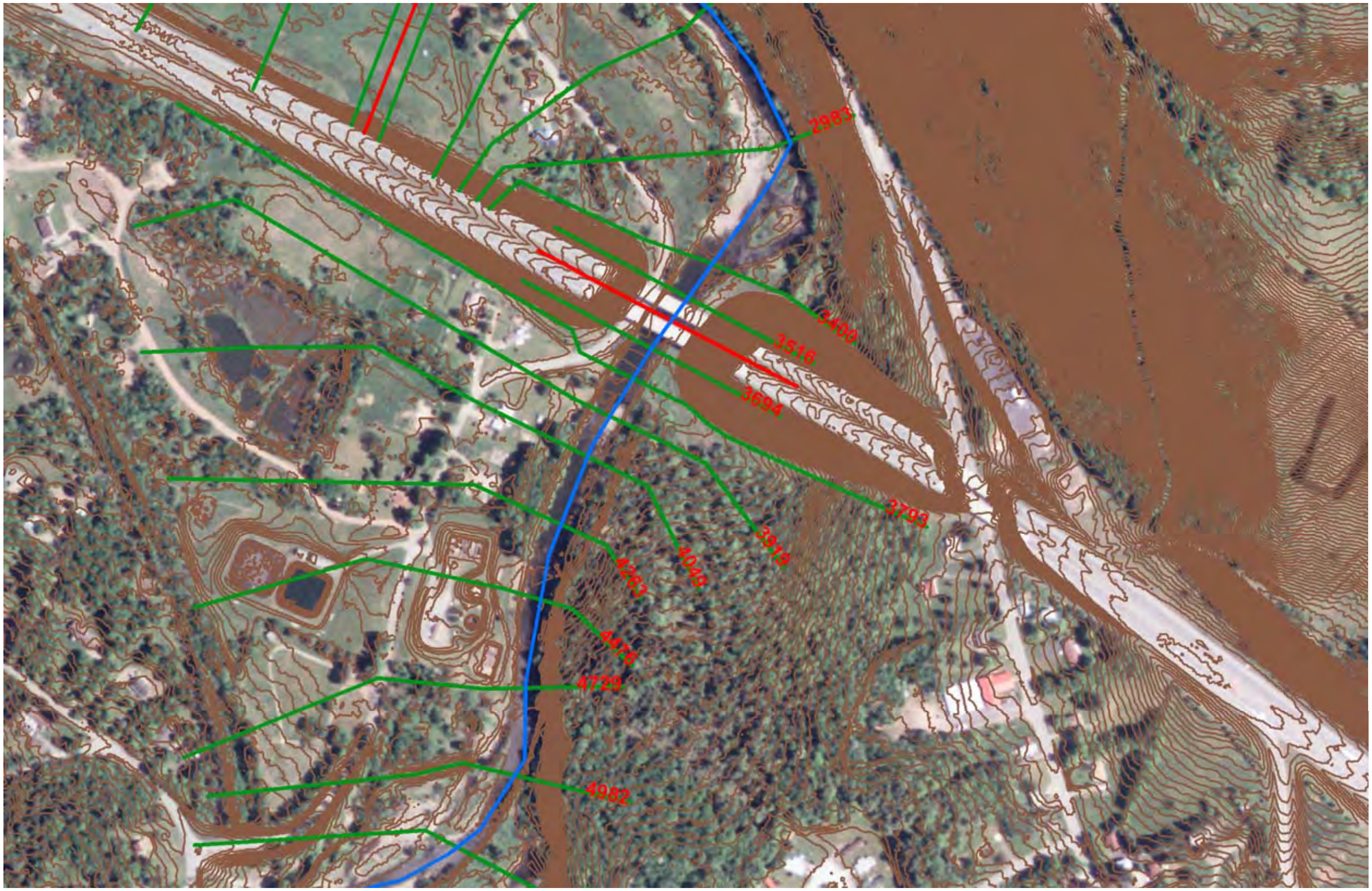


Figure 6.1-Part 2, Willowemoc River – HEC-RAS Features (USDA 2008 Orthographic Image)

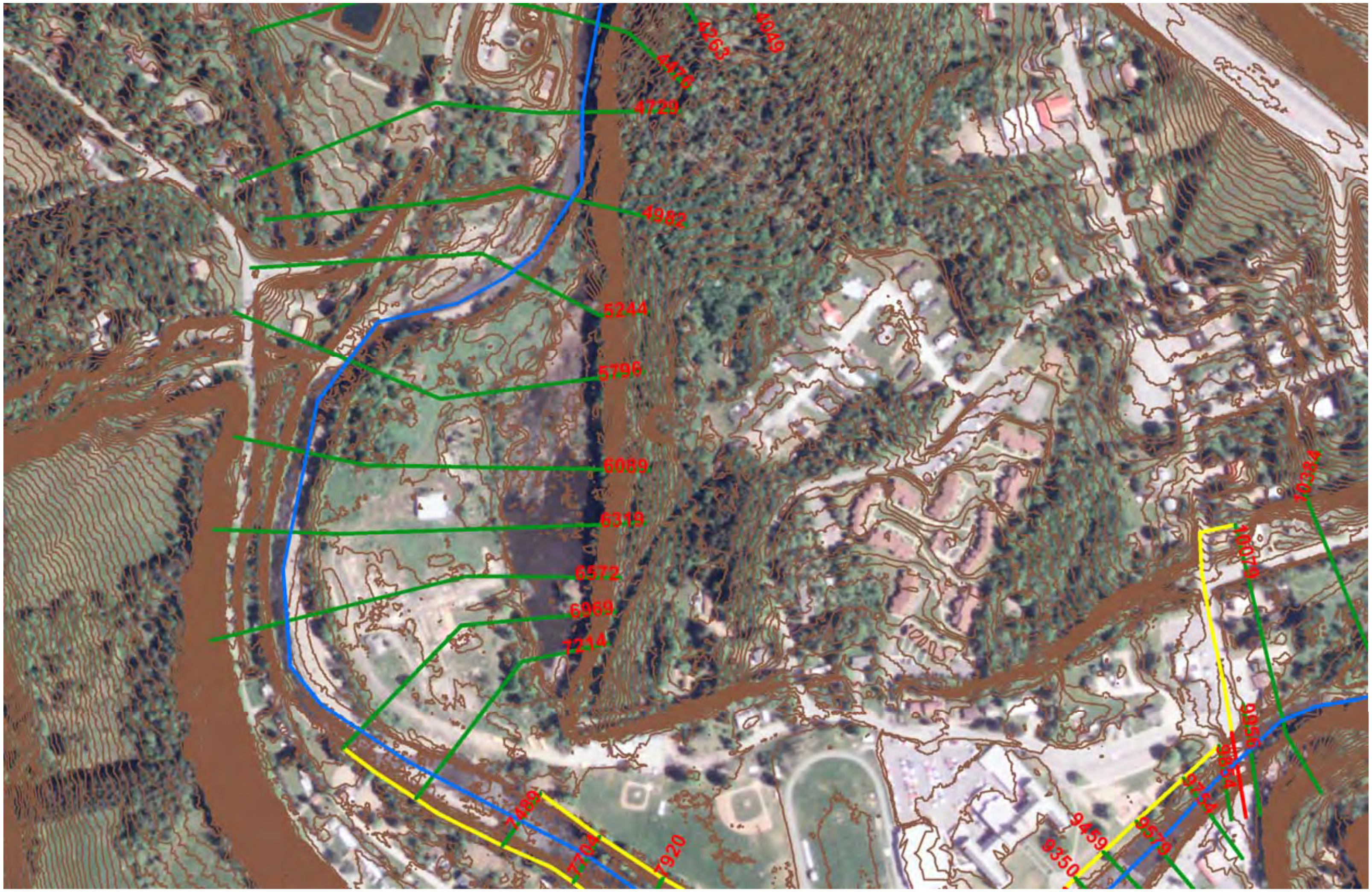


Figure 6.1-Part 3, Willowemoc River – HEC-RAS Features (USDA 2008 Orthographic Image)

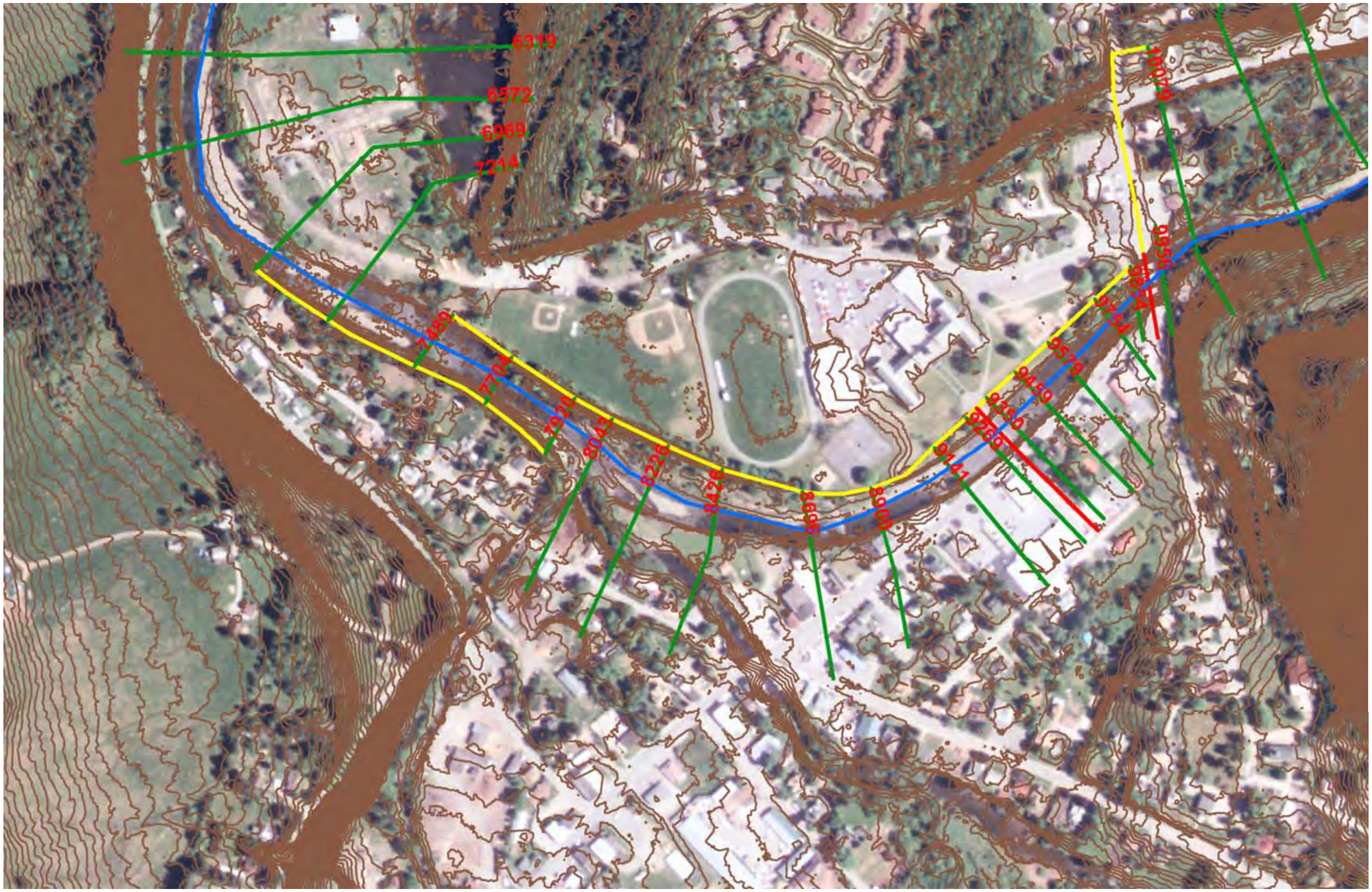


Figure 6.1-Part 4, Willowemoc River – HEC-RAS Features (USDA 2008 Orthographic Image)

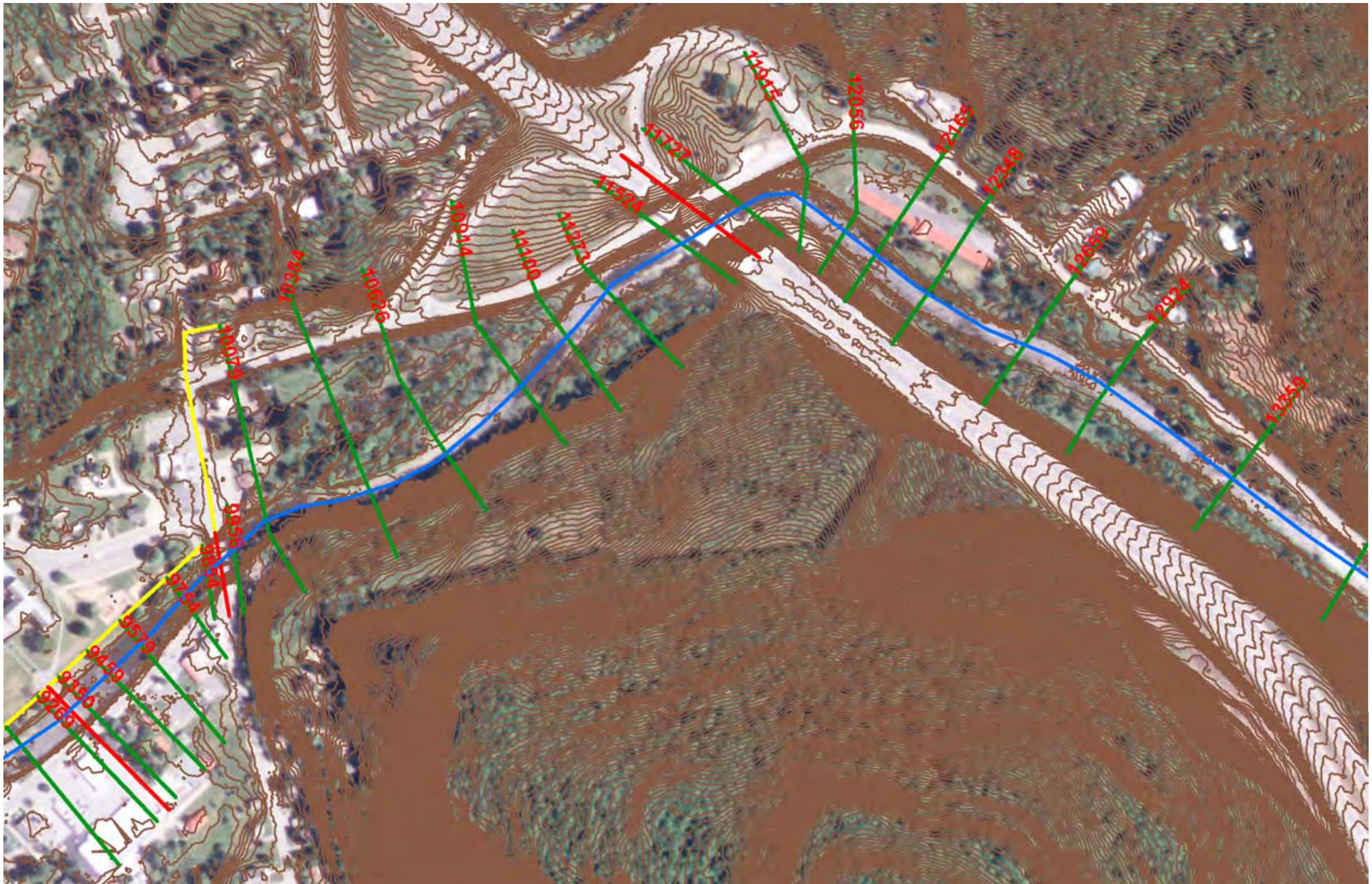


Figure 6.1-Part 5, Willowemoc River – HEC-RAS Features (USDA 2008 Orthographic Image)

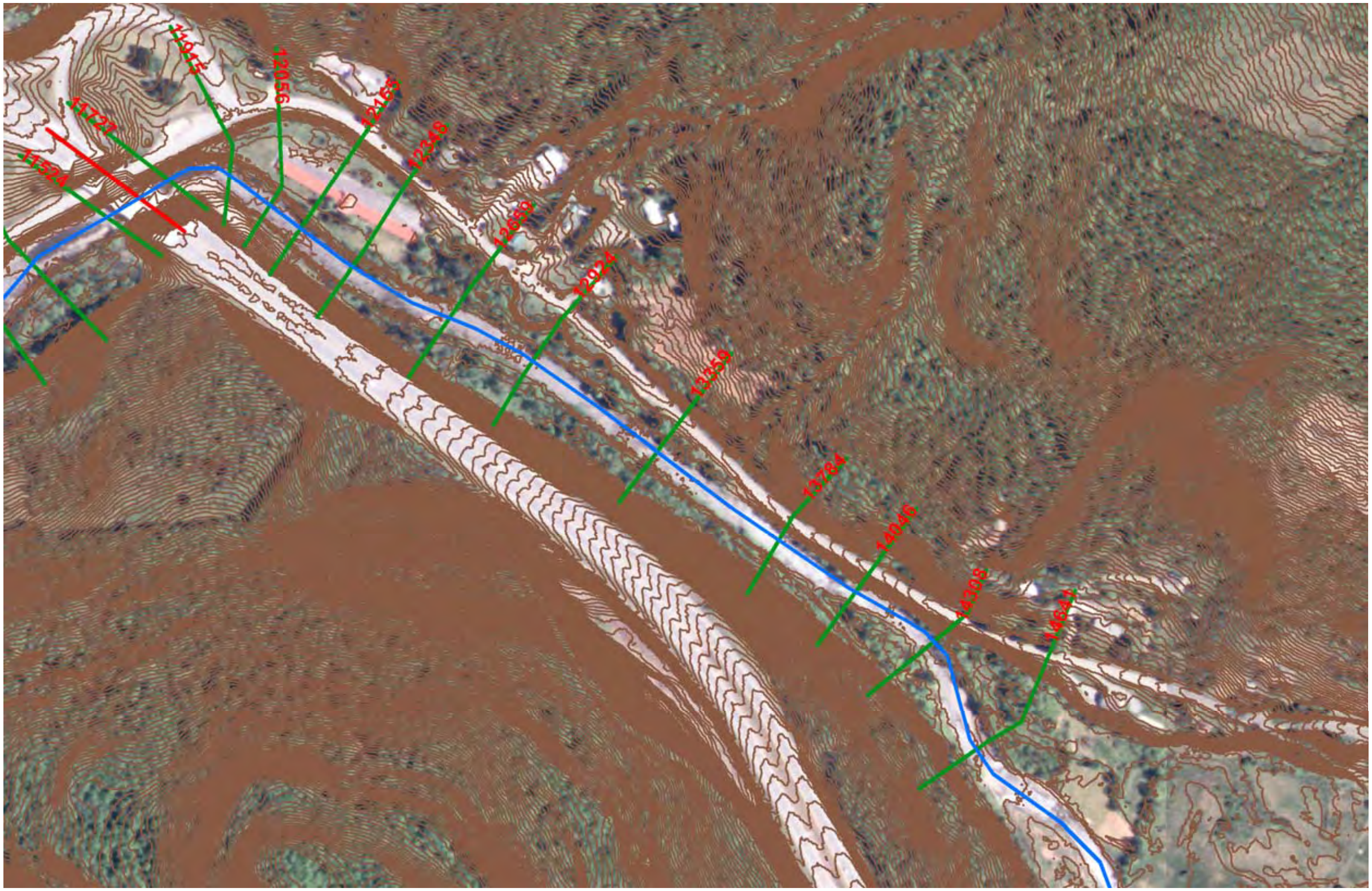


Figure 6.1-Part 6, Willowemoc River – HEC-RAS Features (USDA 2008 Orthographic Image)

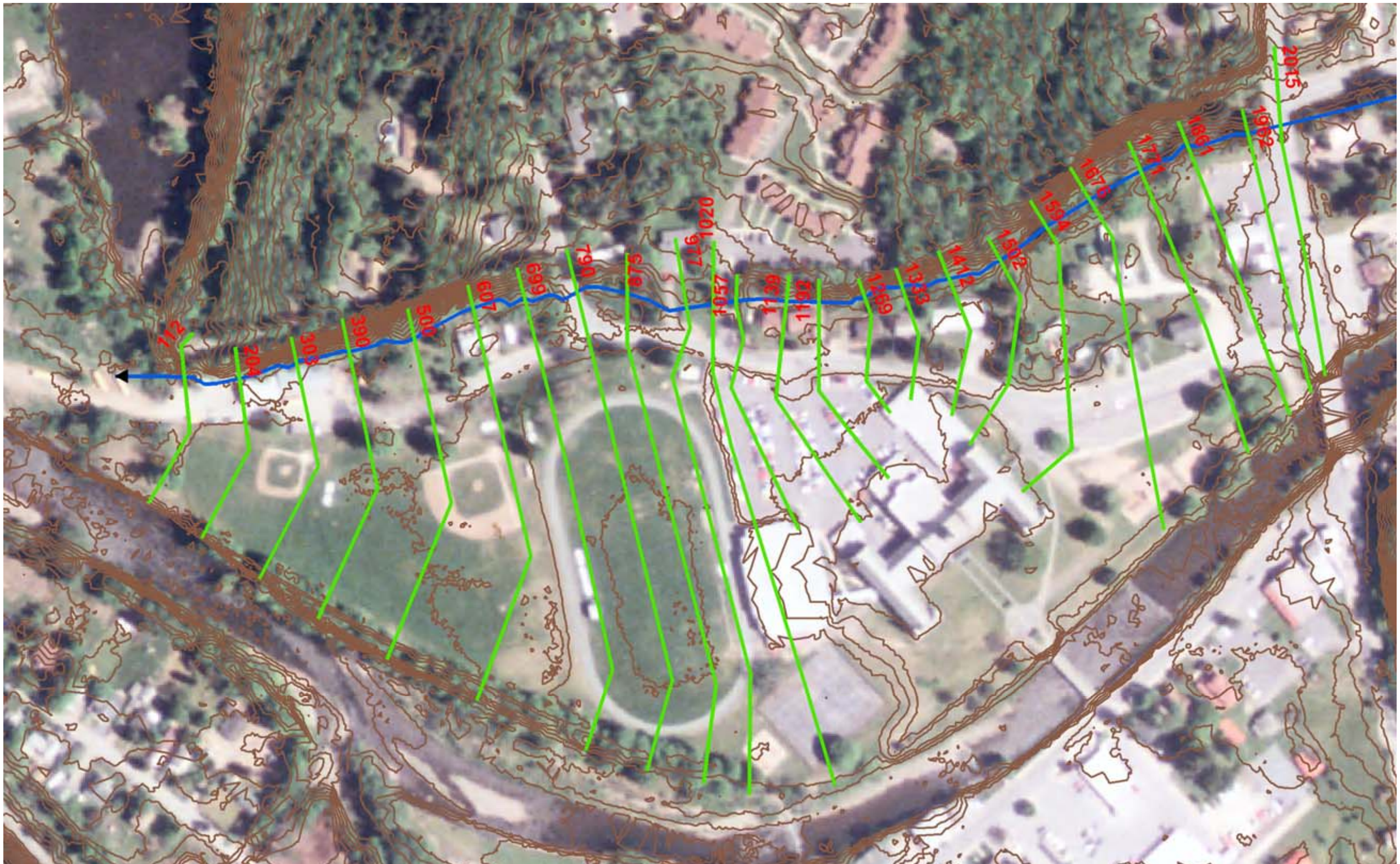


Figure 6.2, Channel Behind School – HEC-RAS Features (USDA 2008 Orthographic Image)

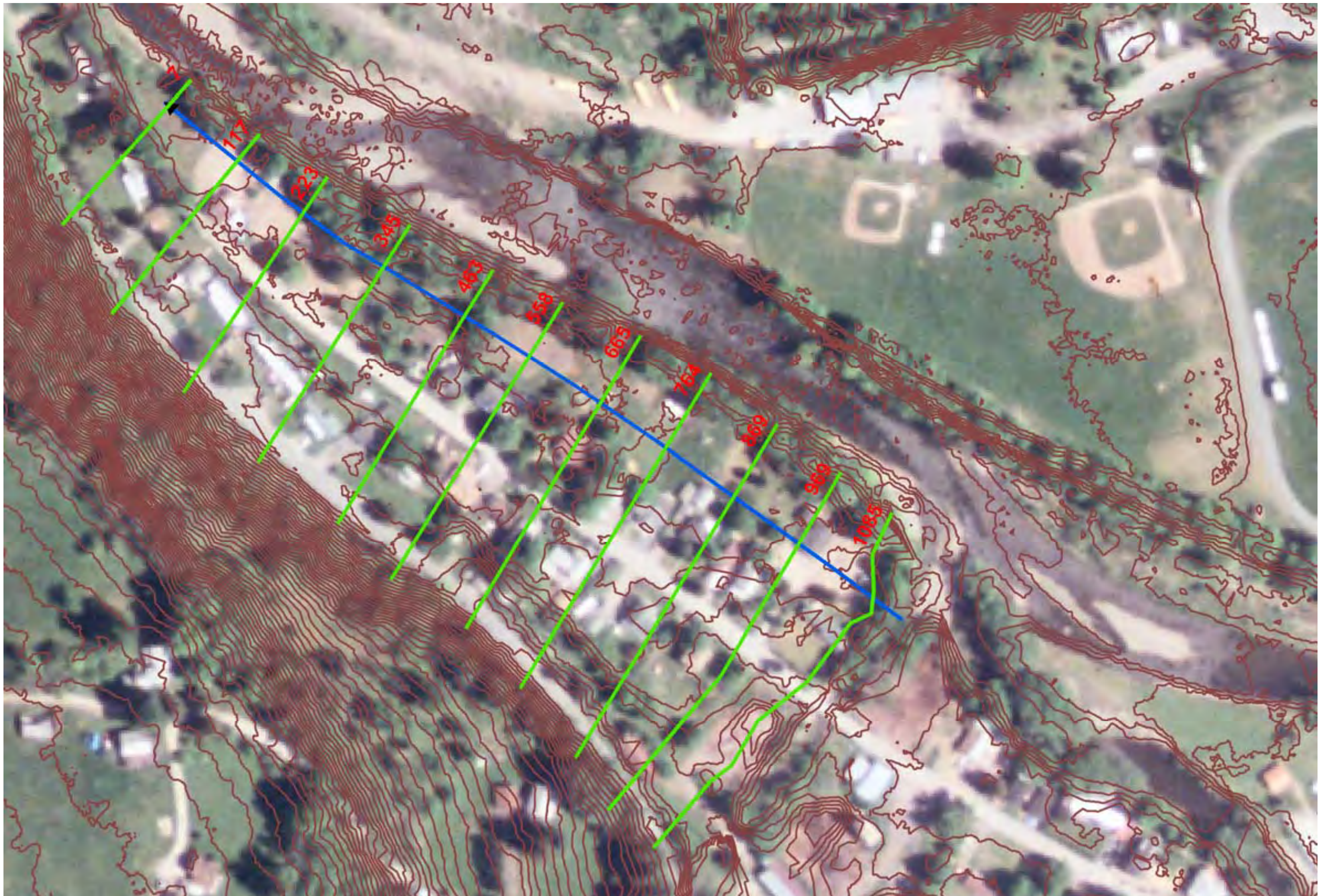


Figure 6.3, Channel Behind Levee on Left Overbank – HEC-RAS Features (USDA 2008 Orthographic Image)

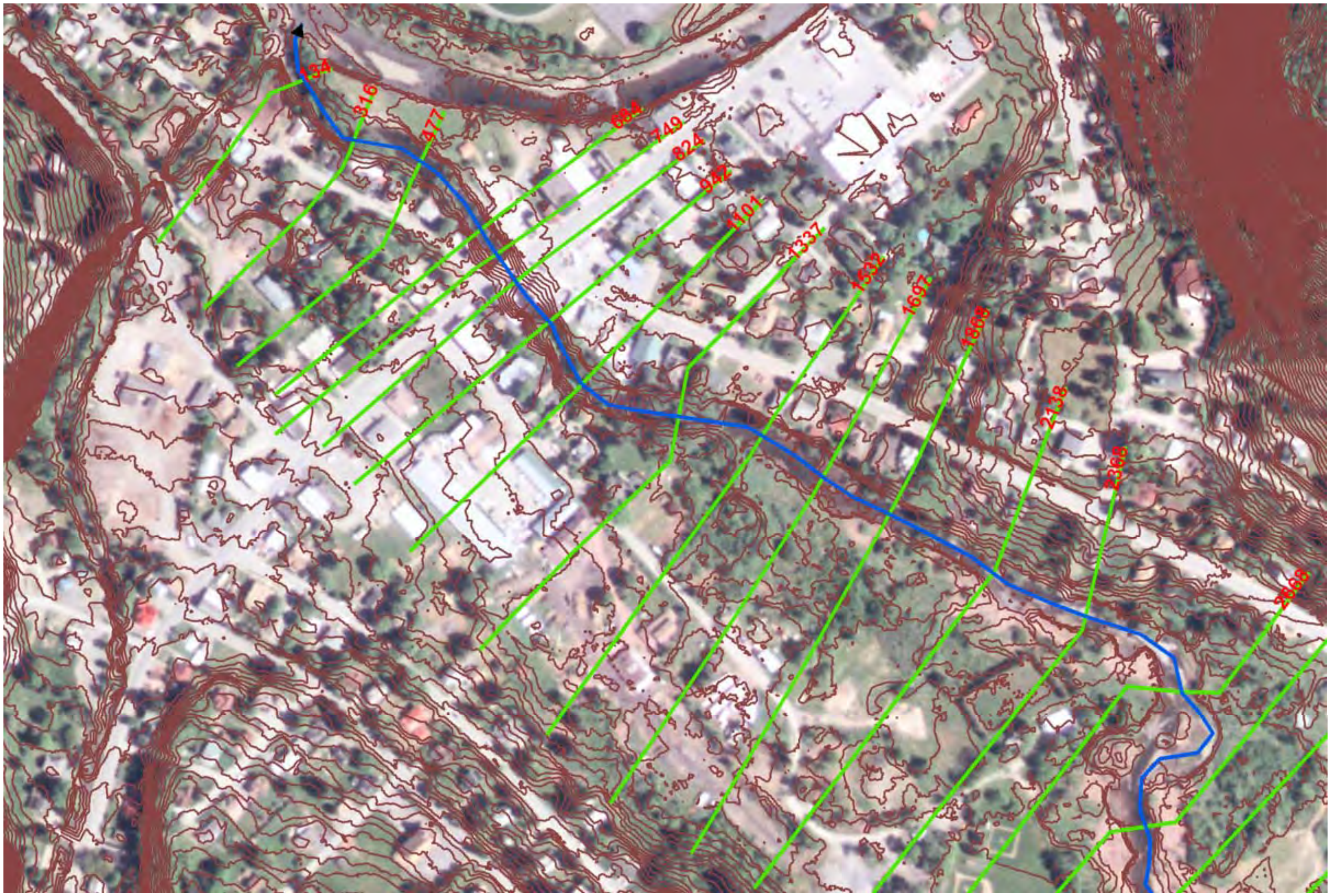


Figure 6.4-Part 1, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

Figure 6.4-Part 2, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

Figure 6.4-Part 3, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

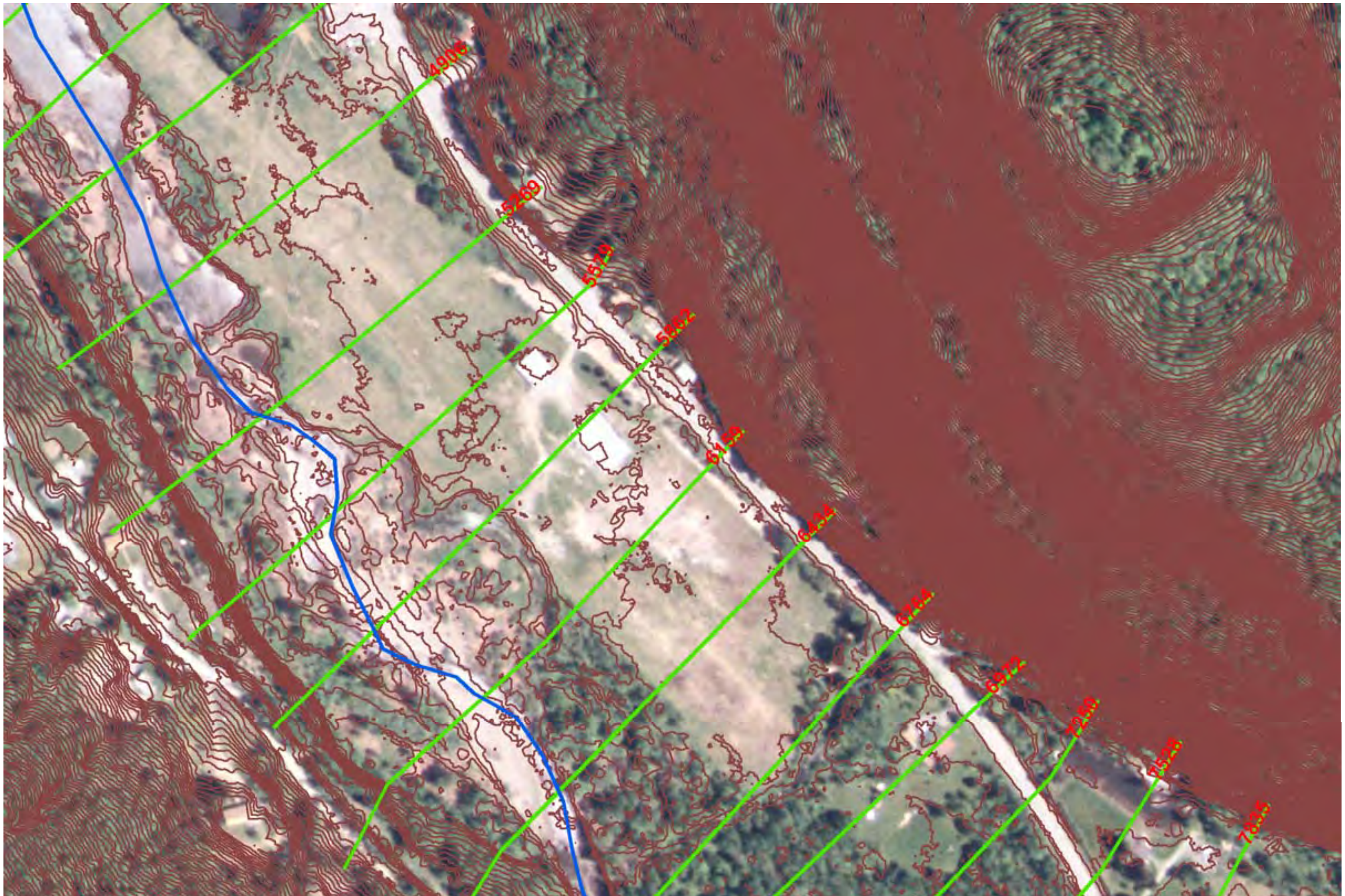


Figure 6.4-Part 4, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

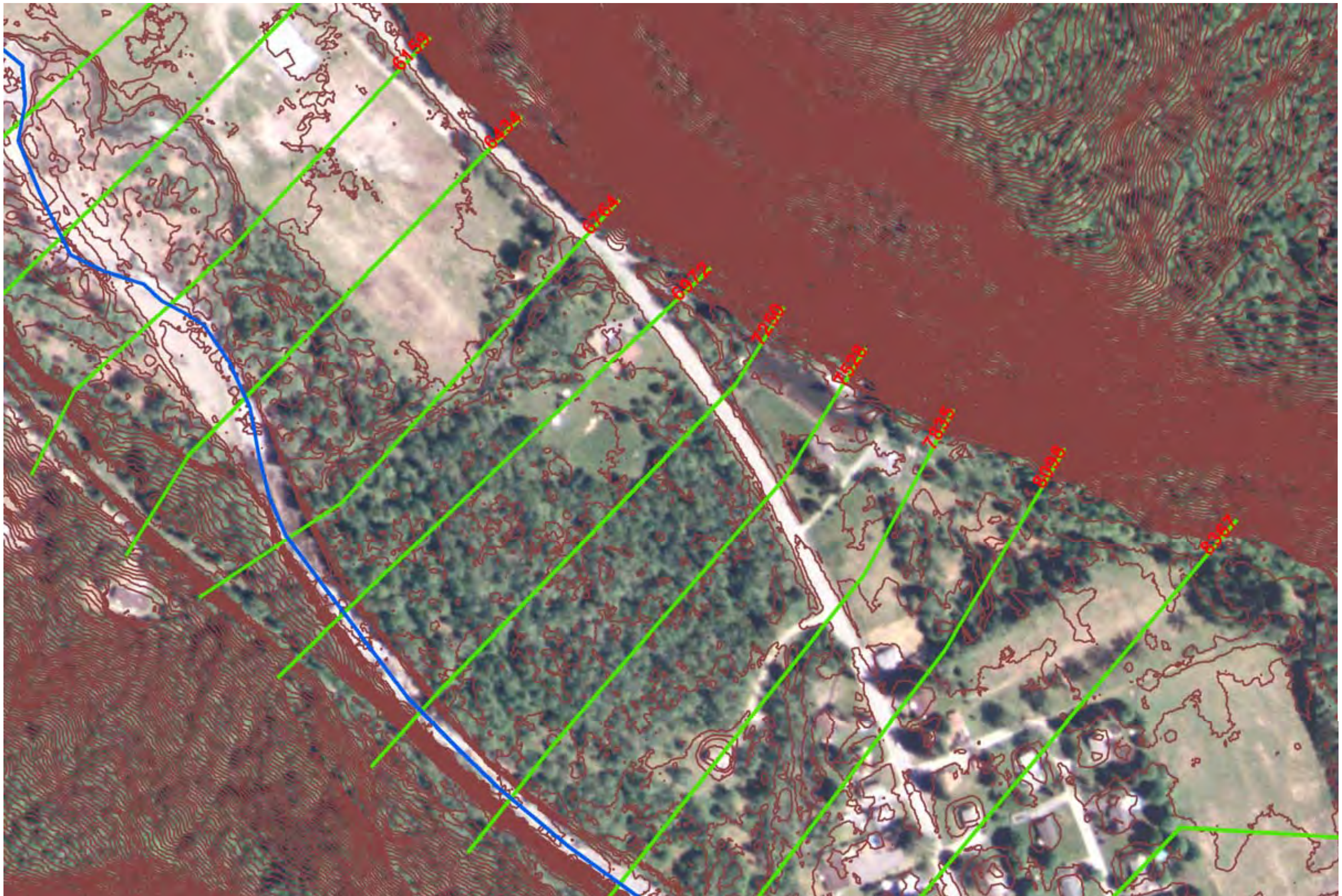


Figure 6.4-Part 5, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

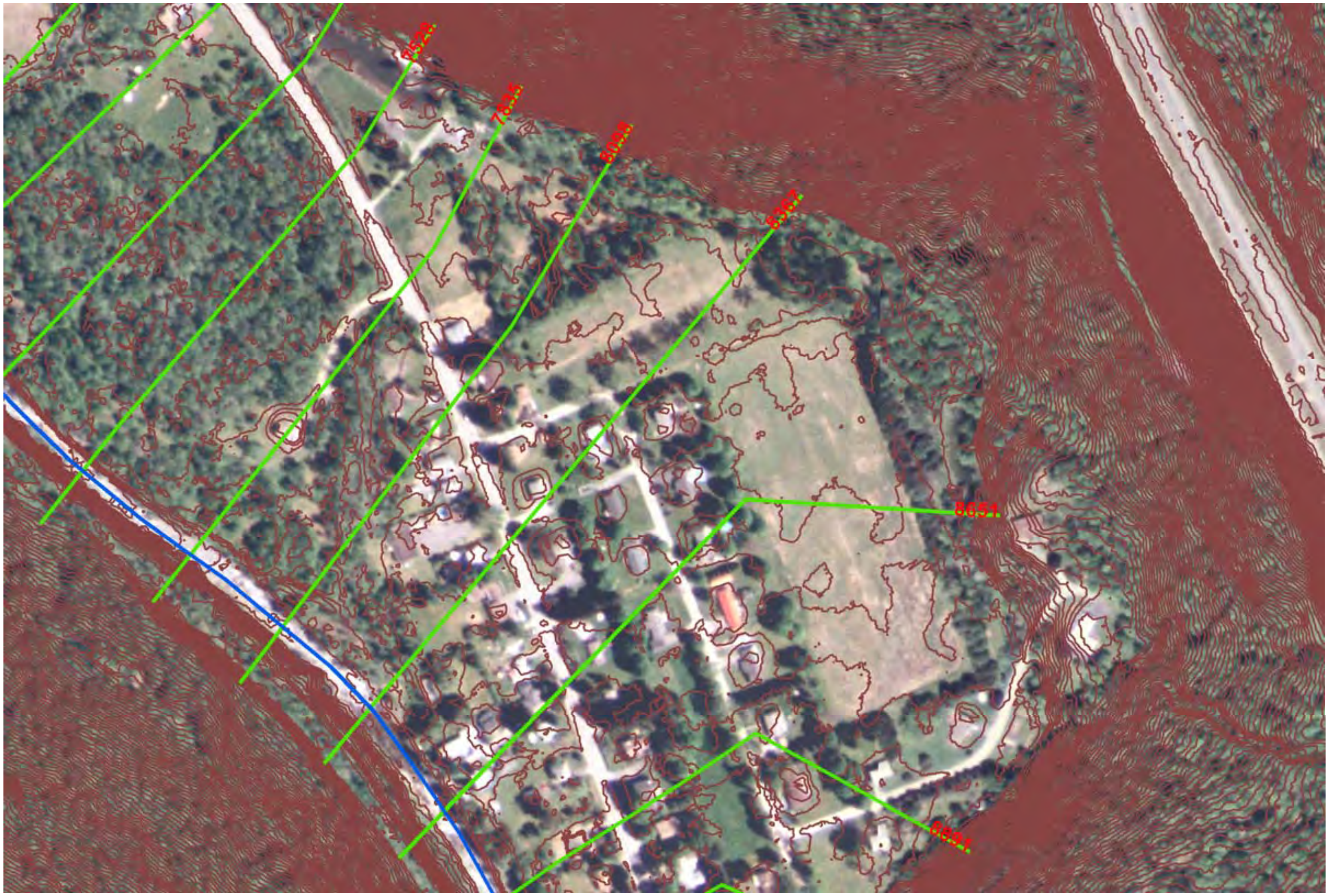


Figure 6.4-Part 6, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

Figure 6.4-Part 7, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

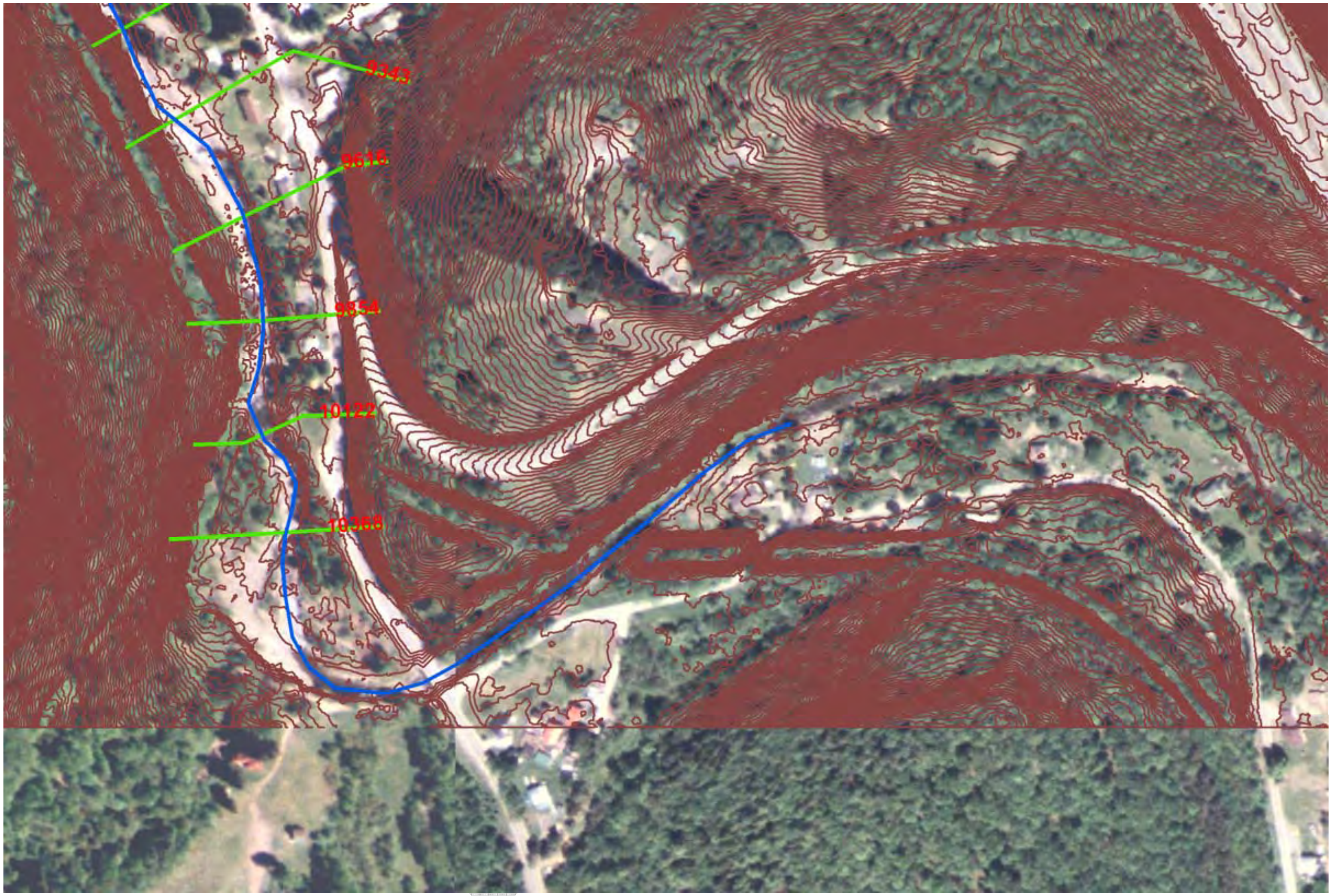


Figure 6.4-Part 8, Little Beaver Kill Creek – HEC-RAS Features (USDA 2008 Orthographic Image)

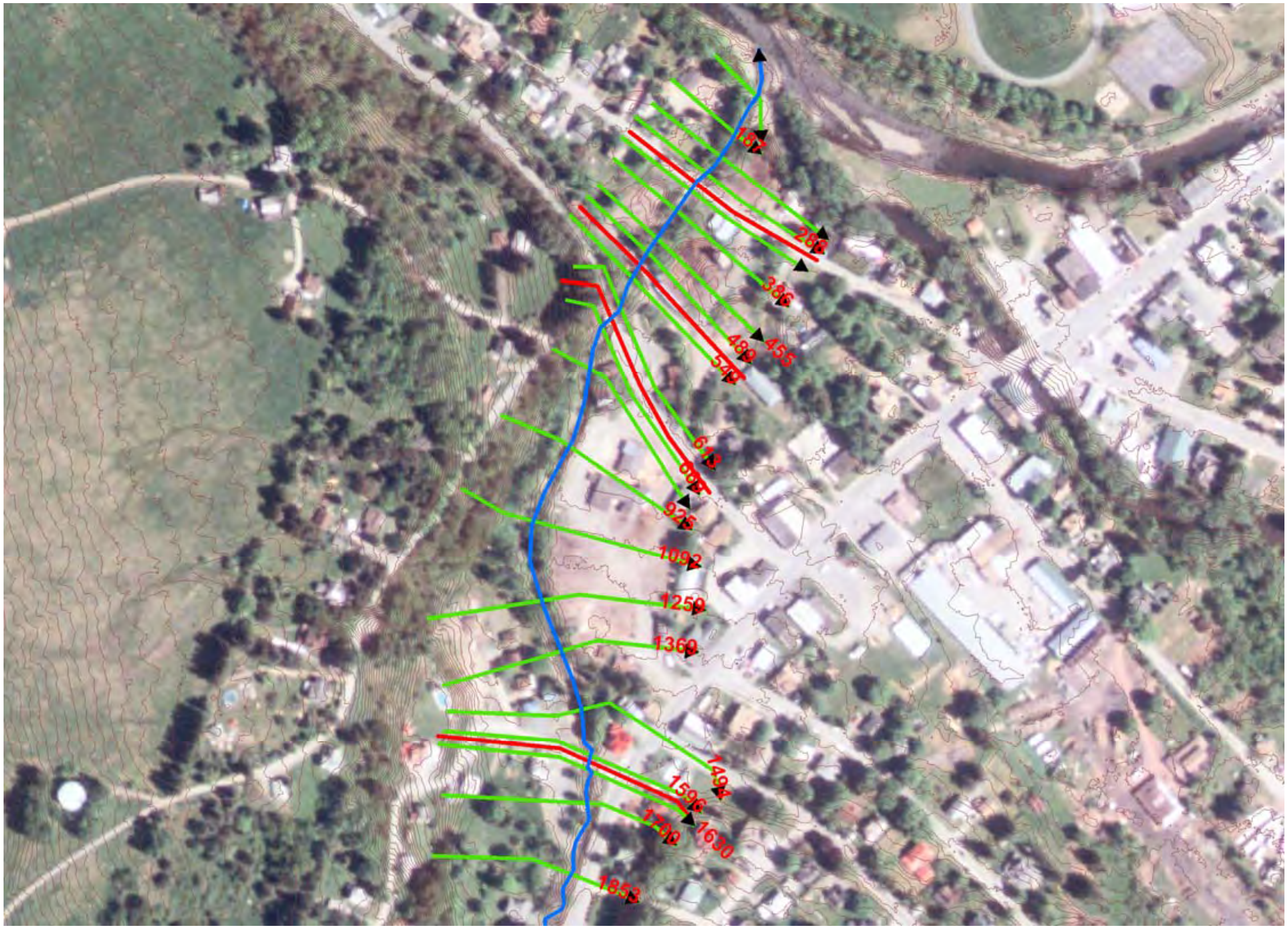


Figure 6.5-Part 1, Cattail Brook – HEC-RAS Features (USDA 2008 Orthographic Image)

Figure 6.5-Part 2, Cattail Brook – HEC-RAS Features (USDA 2008 Orthographic Image)

Figure 6.5-Part 3, Cattail Brook – HEC-RAS Features (USDA 2008 Orthographic Image)

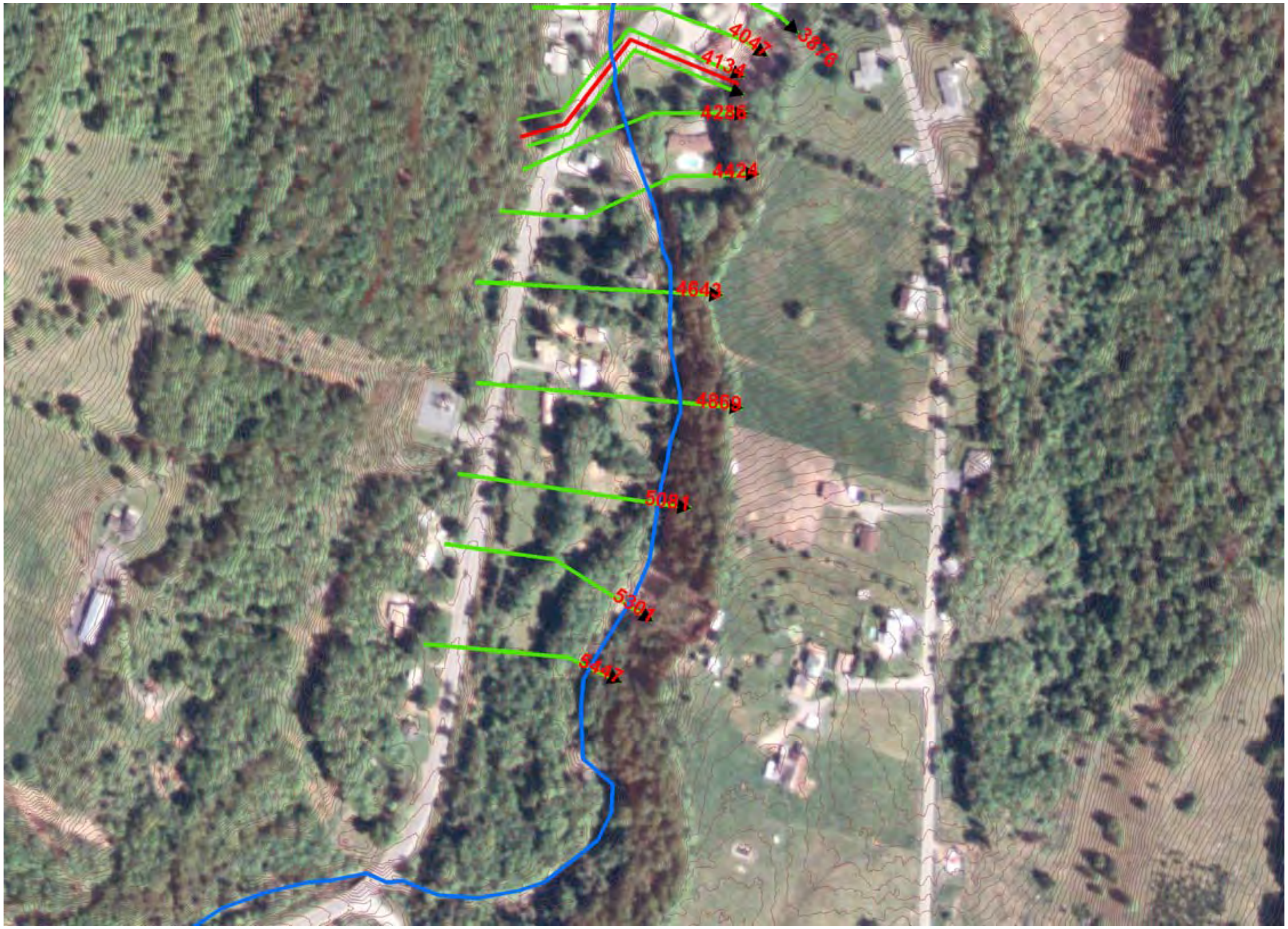


Figure 6.5-Part 4, Cattail Brook – HEC-RAS Features (USDA 2008 Orthographic Image)

Manning’s n roughness factors for both the channel and floodplains of all models were initially assigned using a 2008 USDA aerial photograph. Factors that went into consideration when estimating roughness included channel bed material, stream geometry, vegetation heights, and structures existing in the flood plain. Manning’s n value ranges for the five channels are provided in Table 6.2 below. These values were estimated based on tables in “Open Channel Hydraulics” (Chow, 1959).

Table 6.2
Manning n Values

Model	Channel n	Overbank n
Main Stem Willowemoc	0.035	0.02 – 0.12
Channel behind Levee on LOB	0.05	0.05
Channel behind School	0.02	0.02 – 0.08
Little Beaver Kill	0.04	0.03 – 0.12
Cattail Brook	0.045	0.025 – 0.12

Bridge transition sections upstream and downstream of bridges were applied at 1:1 and 4:1 length to width ratios, respectively. Cross sections upstream and downstream of bridges were assigned contraction and expansion losses of 0.3 and 0.5, respectively. Other areas that necessitated increased contraction and expansion losses included large structures existing in the floodplain and within the vicinity of the Willowemoc / Little Beaver Kill / Cattail Brook confluence. Ineffective flow areas were selected to ensure reasonable flow transitions based on topographic controls and man-made structures such as large buildings. Bridges were modeled assuming no debris blockage.

The main stem Willowemoc, the two Willowemoc back channels, Little Beaver Kill, and Cattail Brook were analyzed as sub-critical flow. (While Cattail Brook is very steep at certain locations and supercritical flow may exist, mixed flow was not considered.) The downstream boundary condition for all models is provided in Table 6.3 below.

Table 6.3
Downstream Boundary Conditions of HEC-RAS Models

Model	Type of Boundary	Value
Main Stem Willowemoc	Normal depth	Energy slope: 0.0038 ft/ft
Channel behind the Levee on the LOB	Known WSEL	From XS-6969 of the main stem Willowemoc model
Channel behind the School	Known WSEL	From XS-7489 of the main stem Willowemoc model
Little Beaver Kill	Known * WSEL	From XS-8043 of the main stem Willowemoc model
Cattail Brook	Normal depth	Energy slope: 0.027 ft/ft

*Note: Hydrographs from the HEC-HMS model at the Willowemoc / Little Beaver Kill confluence showed that their peaks were less than one hour apart for all modeled historic precipitation events. Therefore a peak-on-peak condition was assumed for Little Beaver Kill.

Flow change locations for the main stem Willowemoc, Little Beaver Kill and Cattail Brook were set at locations of noticeable drainage area increase. The flow change locations for the two back channel models were set at locations of large flow increases due to the overtopping of the levees at the low spots. The flow change locations are presented in Table 6.4. The locations are shown on Figures 6.1 thru 6.5.

Table 6.4
Flow Change Locations
for All RAS Models

Model	Flow Change Cross-section
Main Stem Willowemoc	X-14641
	X-12659
	X-8043
	X-2983
Channel behind Levee on LOB	X-1085
	X-764
	X-558
	X-345
	X-117
Channel behind School	X-2015
	X-1771
	X-977
	X-607
	X-390
Little Beaver Kill	X-10368
	X-5862
Cattail Brook	X-5447
	X-3319
	X-2444

B. Calibration - RAS

The historic discharges estimated using the HEC-HMS hydrologic model were transformed into water surface elevations with the USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 4.1.

Published data that was available for calibrating the Willowemoc and Little Beaver Kill HEC-RAS models included:

- Surveyed high water marks contained within the USGS publication entitled "Flood of September 18 – 19, 2004 in the Upper Delaware River Basin, New York", OFR2005-1166
- Surveyed high water marks contained within the USGS publication entitled "Flood of June 26 – 29, 2006, Mohawk, Delaware, and Susquehanna River Basins, New York", OFR2009-1063

In addition to published data, news articles, communication with locals and websites containing descriptions and/or photos of flood conditions were used for calibration. Since no published high water marks were available for Cattail Brook, these articles and descriptions were the foremost means of calibration for the Cattail Brook HEC-RAS model.

The locations of these available high water marks are shown in Figure 6.6.

Figure 6.6 High Water Marks Available for Calibration

Parameters that were adjusted in order to best match the available calibration information include:

- Ineffective flow areas
- Contraction / Expansion losses
    - These, along with the locations for ineffective flow areas were adjusted according to the HEC publication entitled “Flow Transitions in Bridge Backwater Analysis”.
- Manning’s roughness values

1. Results – RAS

High water marks for the 2004 and 2006 events were surveyed along the Willowemoc Creek and Little Beaver Kill Creek by the USGS in NGVD29. The surveyed water surface elevations were converted to NAVD88 by subtracting 0.49 ft from the NGVD29 elevations. The high water marks were spatially located and assigned stream stationing. A comparison between the high water marks and the calculated water surface elevations are provided for the 2004 and 2006 events for the Willowemoc and the Little Beaver Kill in Tables 6.5, 6.6 and 6.7, 6.8 respectively.

Table 6.5
Willowemoc High Water Mark Comparison – September 2004 Event

HWM Station	RAS W.S. (ft-NAVD)	HWM (ft-NAVD)	Delta (feet)	RAS EG (ft-NAVD)	Source of HWM	Remarks
3487	1399.8	1401.93	-2.13	1401.0	#37.1 OFR2005-1166	Per USGS a poor debris line
3748	1402.6	1403.65	-1.05	1403.5	#37.4 OFR2005-1166	Per USGS a poor debris line
8426	1418.21	1419.96	-1.75	1419.16	#34.1	Per USGS a poor debris line
8900	1420.24	1423.04	-2.8	1421.86	#34.2 OFR2005-1166	Per USGS a leaf line
9350	1422.94	1423.1 (approx)	-0.16	1423.75	Photo OFR2005-1166	Wsel in photo assumed to be at peak. W.S estimated to be 1ft less than low chord of school bridge
9729	1424.36	1422.71	1.65	1425.84	#35.3 OFR2005-1166	Per USGS a leaf line
10018	1426.1	1425.85	0.25	1427.41	#35.1 OFR2005-1166	Per USGS a poor debris line
13784	1443.66	1443.85	-0.19	1445.21	USGS	Flood mark surveyed at old gage site.

Table 6.6
Willowemoc High Water Mark Comparison – June 2006 Event

HWM Station	RAS W.S. (ft-NAVD)	HWM (ft-NAVD)	Delta (feet)	RAS EG (ft-NAVD)	Source of HWM	Remarks
1793	1396.39	1398.01	-1.62	1397.04	#28.1 OFR2009-1063	Per USGS good mud line
4476	1405.22	1408 (approx)	-2.78	1406.00	Local Photo	Sewer levee was overtopped, but interior was only partially filled. Added 0.6ft to low spot on levee, 1407.4 ft-NAVD
8900	1420.75	1421 (approx)	-0.25	1422.31	Township Supervisor	School auditorium was flooded approx 1ft deep. Elev. 1421 is 1ft added to 1420 ground contour at auditorium
9674	1424.09	1426.39	-2.3	1425.74	#26.1 OFR2009-1063	Per USGS HWM reported by owner

Table 6.7 Little Beaver Kill High Water Mark Comparison – September 2004 Event						
HWM Station	RAS W.S. (ft-NAVD)	HWM (ft-NAVD)	Delta (feet)	RAS EG (ft-NAVD)	Source of HWM	Remarks
517	1418.19	1419.96	-1.77	1418.59	#34.1 OFR2005-1166	Per USGS poor debris line
1527	1422.60	1424.11	-1.51	1421.48	#36.1 OFR2005-1166	Per USGS a good mud line

Table 6.8 Little Beaver Kill High Water Mark Comparison – June 2006 Event						
HWM Station	RAS W.S. (ft-NAVD)	HWM (ft-NAVD)	Delta (feet)	RAS EG (ft-NAVD)	Source of HWM	Remarks
749	1420.47	1425.17	-4.70	1421.41	#27.1 OFR2009-1063	Per USGS HWM reported by owner
1337	1423.41	1425.43	-2.02	1423.43	#25.2 OFR2009-1063	Per USGS HWM reported by owner
1697	1423.46	1425.65	-2.19	1423.48	#25.1 OFR2009-1063	Per USGS fair seed line
3293	1423.70	1426.75	-3.05	1423.75	#24.1 OFR2009-1063	Per USGS good seed line

Agreement between the published high water marks and the computed HEC-RAS water surface elevations was varied. There were several locations, on both the Willowemoc and Little Beaver Kill, where the computed water surface elevations were within one foot of the published high water marks.

However, there were also several locations where two feet or more separated the computed and observed water surface elevations. Several observed high water marks on both the Willowemoc and Little Beaver Kill are problematic because they reflect poorly defined debris lines. Also, there are other high water marks in locations where the inclusion of a velocity head could explain the difference in water surface elevations. To this end, the tables above include the energy elevation which is the velocity head added to the water surface elevation. The energy elevation is to be used for comparisons at stagnation points such as on the upstream side of a structure. Velocity head could have an important role in determining the water surface elevation at the following locations:

- Willowemoc – 9674, 8900, and 1793
    - These high water marks were located adjacent to buildings.
- Little Beaver Kill – 3293 and 1697
    - These high water marks were located on a porch on the landward side of a house and inside a garage, respectively.

2 Sources of Error - RAS

There were several limitations for calibrating the HEC-RAS models to the chosen historic precipitation events. These include:

- Observed high water marks
  - Several of the surveyed high water marks within and around Livingston Manor were located in hydraulically “unfavorable” positions.
  - Errors could permeate from the accuracy of the surveyed elevations and/or locations themselves.
- Debris blockage
  - While obstructions could play a large role in stream flows and their corresponding water surface elevations, it is standard practice to not model debris blockage.
- HEC-RAS limitations
  - While the one-dimensional flow capabilities of HEC-RAS allow for rapid model development, they might not be adequate to model all hydraulic processes. At locations where flow profiles change slope rapidly (bridges, obstructions, abrupt changes in channel geometries, etc.) two- or three-dimensional flow models may be necessary to accurately represent the processes involved.

The HEC-RAS model is acceptable for the purposes of this technical effort.

C. Frequency Water Surface Profiles

The five calibrated HEC-RAS models were run with downstream starting conditions noted above. The frequency discharges inputted to the models are provided in Table 6.9.

The hydraulic performance of the lateral structures in diverting water out of the main Willowemoc channel to the two back channels is assessed with the information found in Table 6.10. The information in Table 6.10 allows intelligent plan formulation because of the complex interaction between the main stem Willowemoc and the two back channels as mediated by the lateral structures. For example, raising a levee would reduce flow into a back channel but it would also increase the flow in the Willowemoc downstream of what was once a diversion point.

The existing condition frequency water surface profiles for the five hydraulic models are provided as Figures 6.7 to 6.11.

Table 6.9 **Existing Condition Frequency Discharges at Flow Change Locations**								
X-section Location	Discharge (cfs)							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Willowemoc								
X-14641	3360	5850	7880	10900	13500	16400	20800	24500
X-12659	3395	5910	7960	11010	13640	16560	21010	24750
X-8043	4970	8660	11660	16130	19980	24270	30780	36260
X-2983	5040	8780	11820	16350	20250	24600	31200	36750
Little Beaver Kill								
X-10368	1863	3022	3919	5175	6196	7286	8873	10171
X-5862	1890	3066	3976	5250	6286	7392	9002	10318
Behind Left Levee								
X-1085	1	1	27	462	1120	1701	2427	2896
X-764	1	1	27	462	1120	1701	2427	2896
X-558	1	1	27	462	1120	1702	2450	3017
X-117	115	459	684	1359	2225	3058	4521	6339
Behind School Levee								
X-2015	1	1	1	1	1	522	2117	3990
X-1771	1	1	1	100	390	1275	3446	5859
X-977	1	1	1	101	834	2734	6517	10146
X-607	1	1	1	192	1233	3560	7924	11996
X-390	1	1	1	192	1233	3560	7925	12009
Cattail Brook								
X-5447	497	811	1058	1402	1681	1978	2414	2767
X-3319	530	865	1130	1496	1795	2112	2576	2954
X-2444	558	911	1189	1575	1889	2223	2712	3109

Table 6.10 Hydraulic Performance of Lateral Structures for Frequency Events																	
Xsect	Lat Struc	2year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
		Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls
Old Rt17		3395		5910		7960		11010		13640		16560		21010		24750	
	10078		0		0		0		0		1		526		2108		3954
9956		3395		5910		7960		11010		13639		16033		18902		20796	
	9957		0		0		0		0		0		0		0		0
9854		3395		5910		7960		11010		13639		16033		18902		20796	
	9853		0		0		0		108		431		829		1347		1695
9754		3395		5910		7960		10902		13208		15203		17554		19102	
	9753		0		0		0		0		0		4		193		417
9579		3395		5910		7960		10902		13208		15200		17361		18685	
	9578		0		0		0		0		0		0		7		120
9459		3395		5910		7960		10902		13208		15200		17354		18565	
	9458		0		0		0		0		0		0		89		248
9350		3395		5910		7960		10902		13208		15200		17265		18317	
	9351		0		0		0		0		0		0		0		0
9269		3395		5910		7960		10902		13208		15200		17265		18317	

Table 6.10 (continued)
Hydraulic Performance of Lateral Structures for Frequency Events

Xsect	Lat Struc	2year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
		Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls
9269		3395		5910		7960		10902		13208		15200		17265		18317	3395
	9268		0		0		0		0		0		0		0		0
9141		3395		5910		7960		10902		13208		15200		17265		18317	
	9140		0		0		0		0		0		37		138		194
8900		3395		5910		7960		10902		13208		15162		17127		18123	
	8899		0		0		0		0		2		74		381		601
8696		3395		5910		7960		10902		13206		15087		16748		17532	
	8695		0		0		0		0		167		679		1350		1836
8426		3395		5910		7960		10902		13039		14406		15392		15714	
	8425		0		0		0		0		55		324		766		1100
8226		3395		5910		7960		10902		12984		14082		14631		14624	
	8225		0		0		0		30		265		628		1112		1454
8043		4970		8660		11660		15992		19059		21164		23273		24682	
	8042		0		0		0		0		0		14		70		135
7920		4970		8660		11624		15333		17682		19154		20534		21346	

Table 6.10 (continued)
Hydraulic Performance of Lateral Structures for Frequency Events

Xsect	Lat Struc	2year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
		Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls	Qxs	Qls
7920		4970		8660		11624		15333		17682		19154		20534		21346	
	7919		0		0		0		0		0		0		0		0
7704		4970		8660		11624		15333		17682		19154		20534		21346	
	7703		0		0		0		0		0		0		0		0
7489		4970		8660		11624		15333		17682		19154		20534		21346	
Start of Lateral Structures on the Left Overbank																	
	8033		0		0		36		659		1375		1996		2677		3203
7920		4970		8660		11624		15333		17682		19154		20534		21346	
	7910		0		0		0		0		0		0		0		0
7704		4970		8660		11624		15333		17682		19154		20534		21346	
	7694		0		0		0		0		0		0		0		0
7489		4970		8660		11624		15333		17682		19154		20534		21346	
	7479		0		0		0		0		1		7		39		113
7214		4970		8660		11624		15333		17681		19147		20496		21233	
	7204		115		455		654		903		1175		1471		2126		3134
6969		4857		8211		10979		14431		16506		17675		18370		18099	
All diverted flow recombines at X-6572																	
6572		4970		8660		11660		16130		19980		24270		30780		36260	

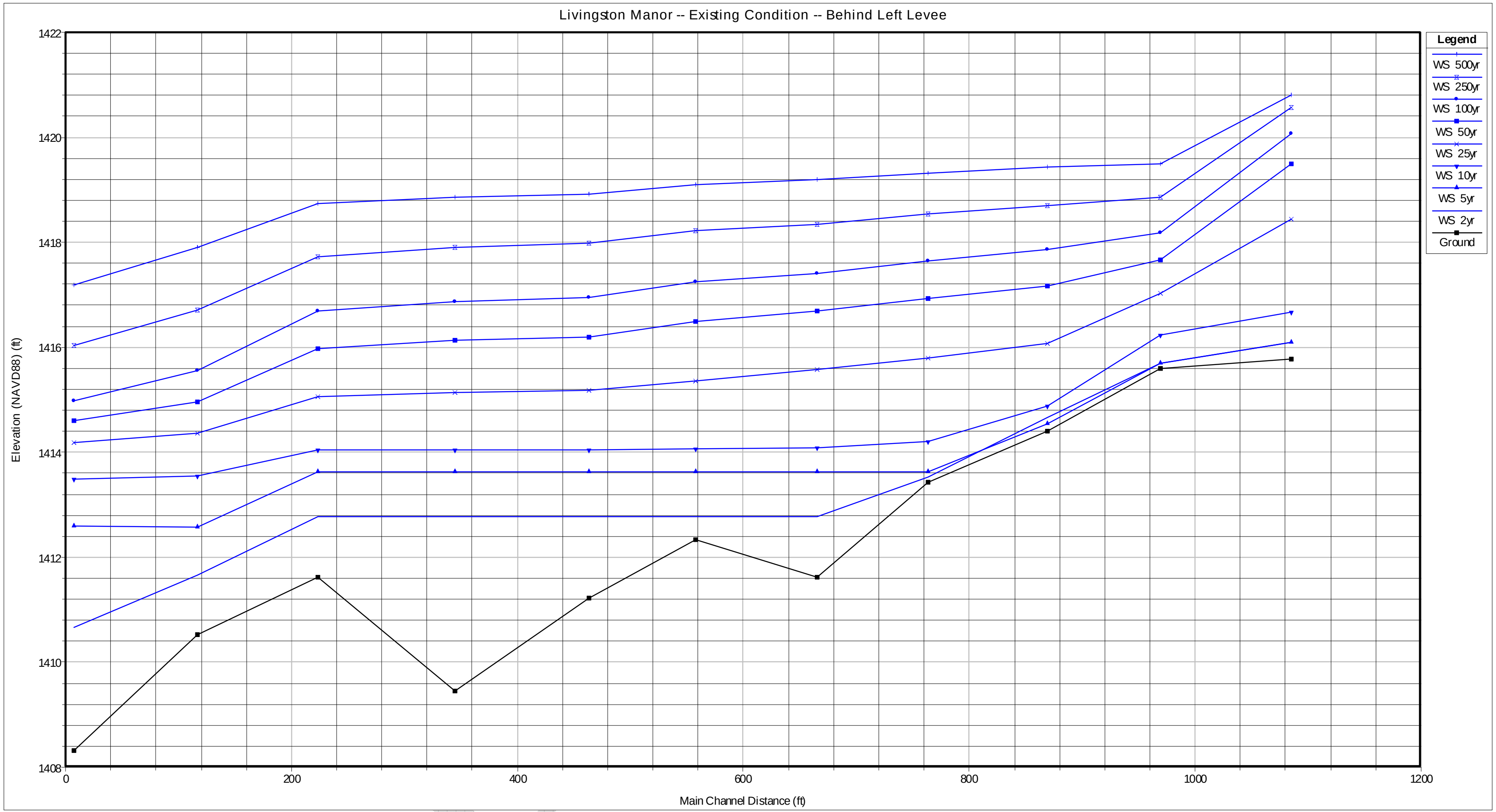


Figure 6.7, Channel Behind Left Levee – Existing Condition Frequency Water Surface Profiles

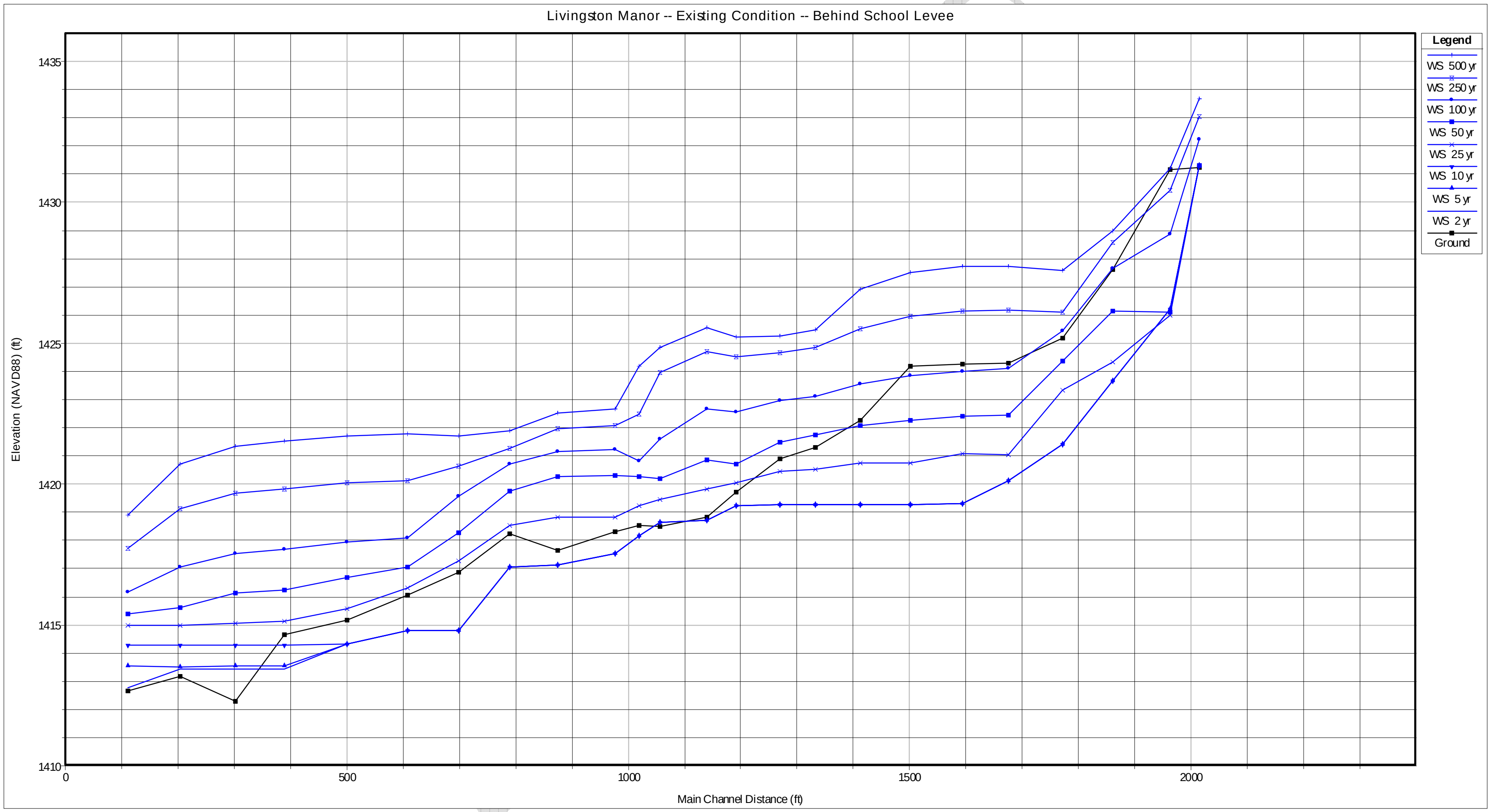


Figure 6.8, Channel Behind School Levee – Existing Condition Frequency Water Surface Profiles

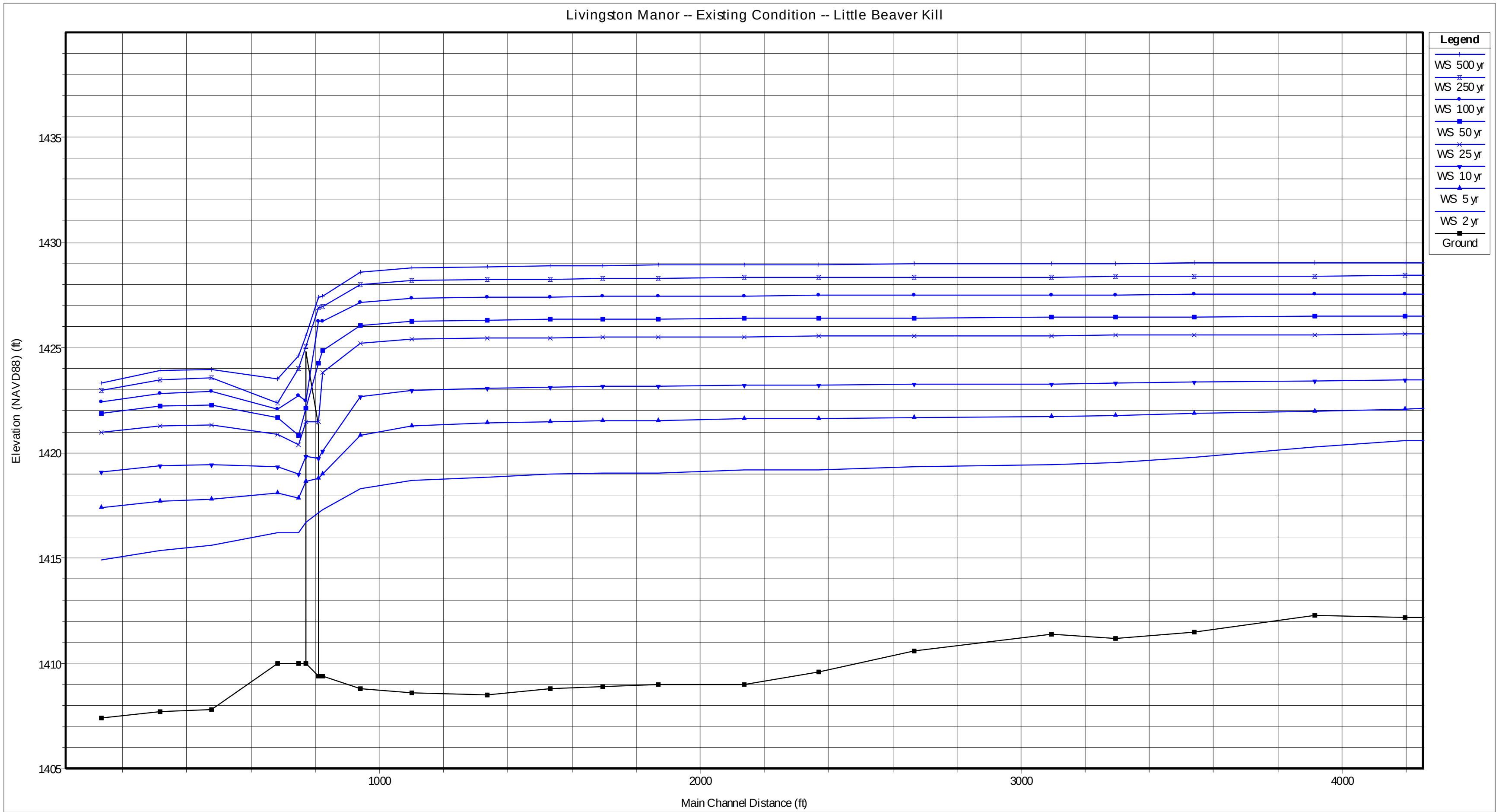


Figure 6.9 - Part 1, Little Beaver Kill – Existing Condition Frequency Water Surface Profiles

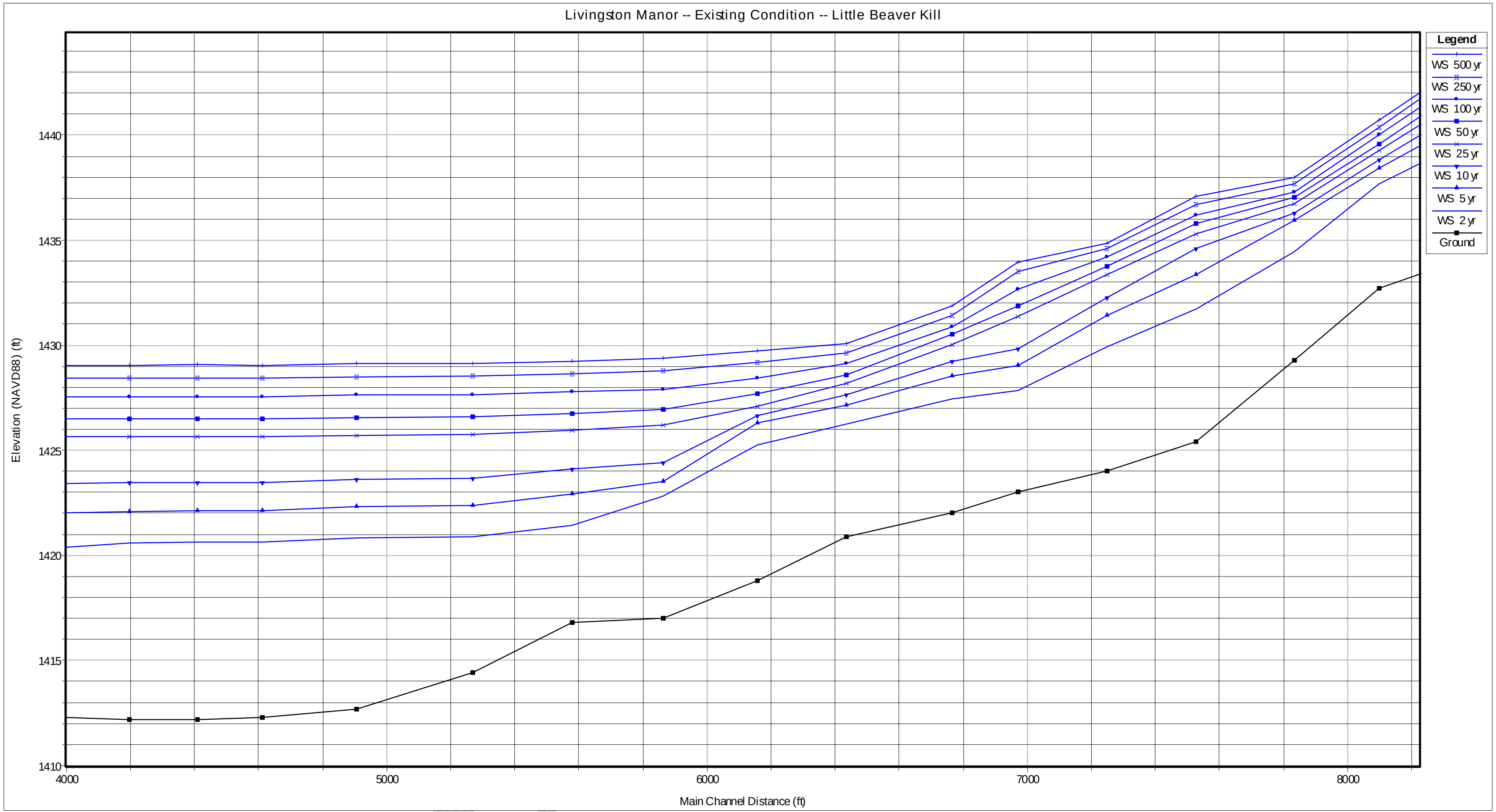


Figure 6.9 - Part 2, Little Beaver Kill – Existing Condition Frequency Water Surface Profiles

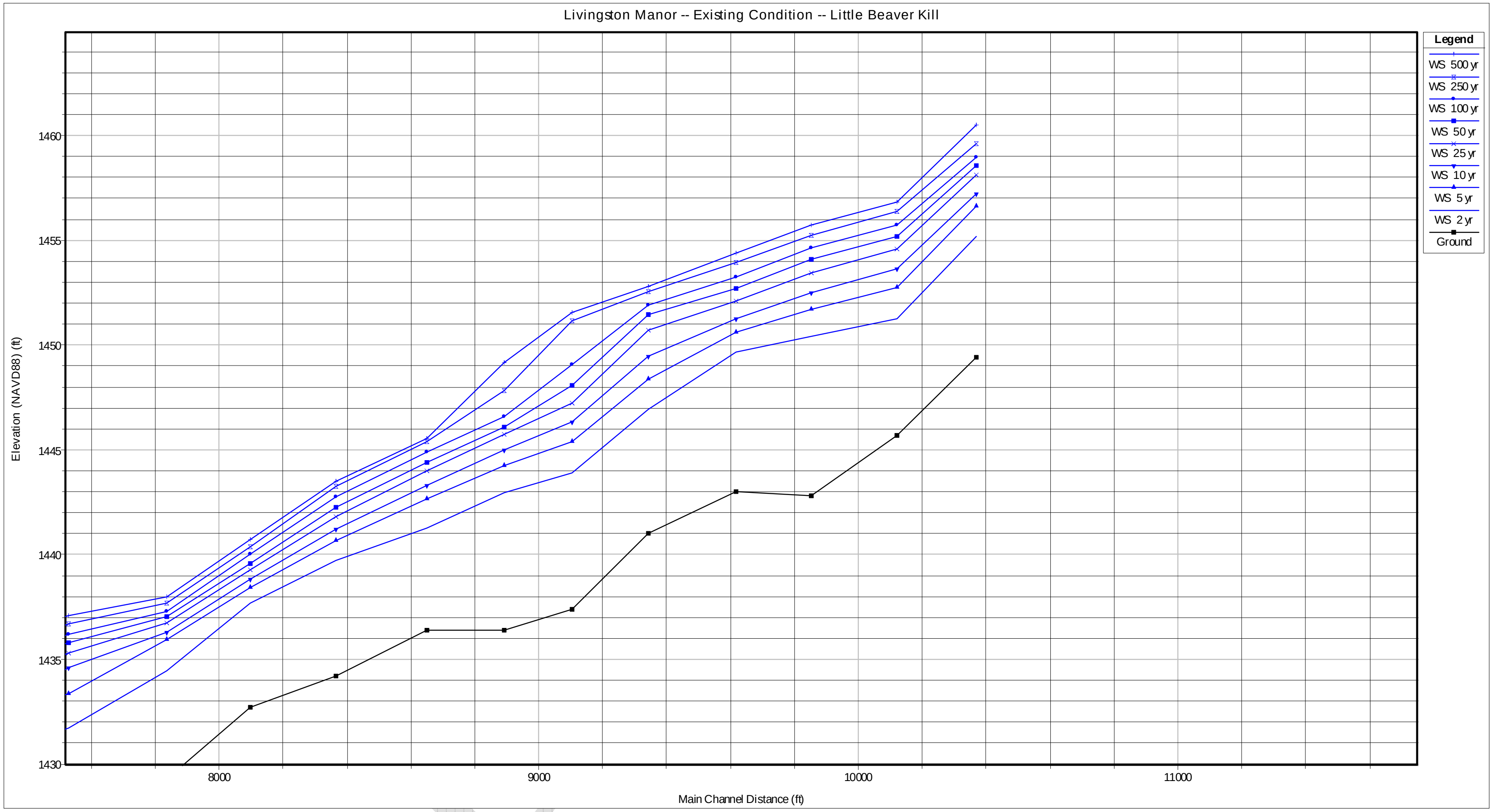


Figure 6.9 - Part 3, Little Beaver Kill – Existing Condition Frequency Water Surface Profiles

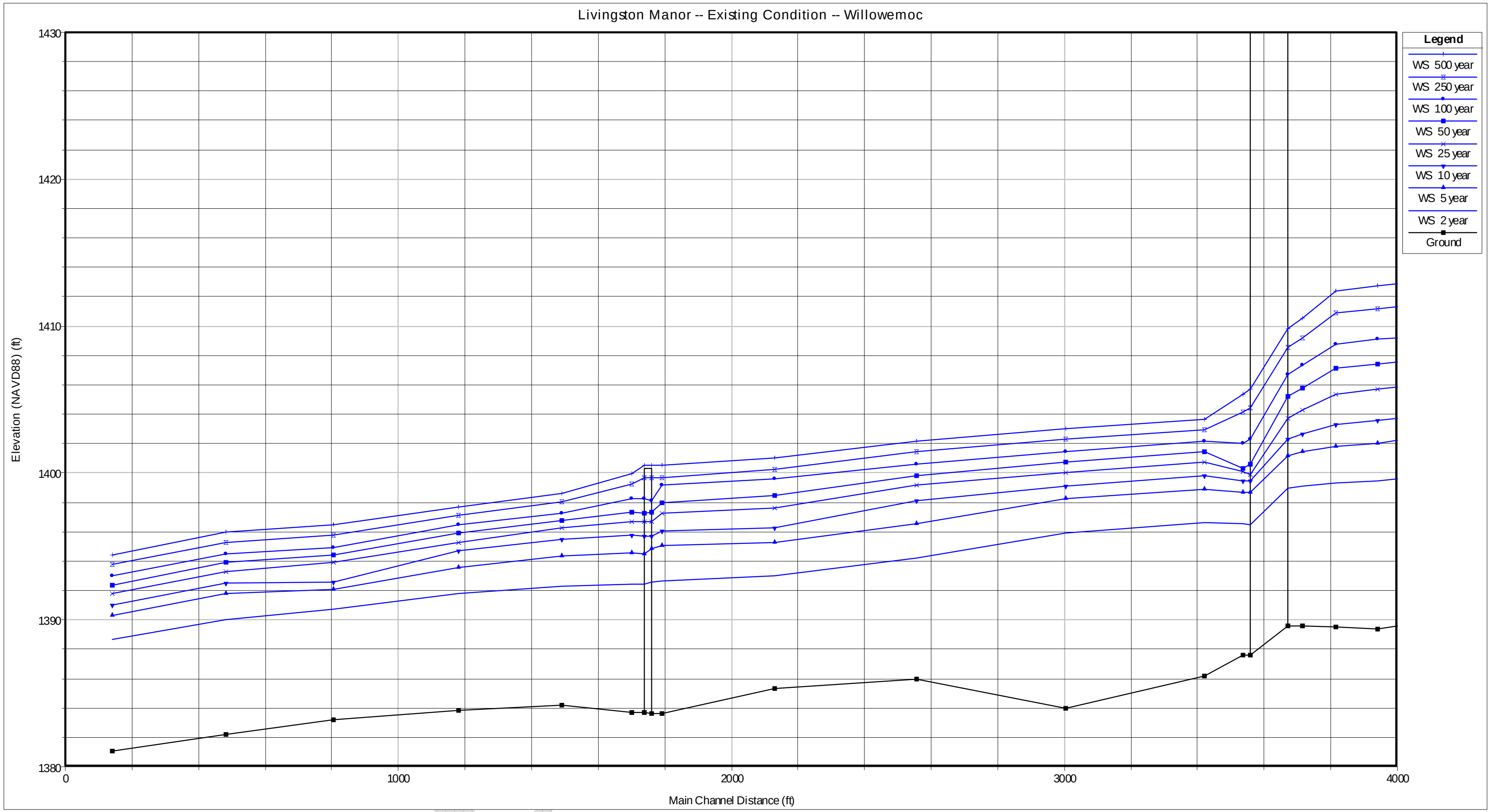


Figure 6.10 - Part 1, Willowemoc – Existing Condition Frequency Water Surface Profiles

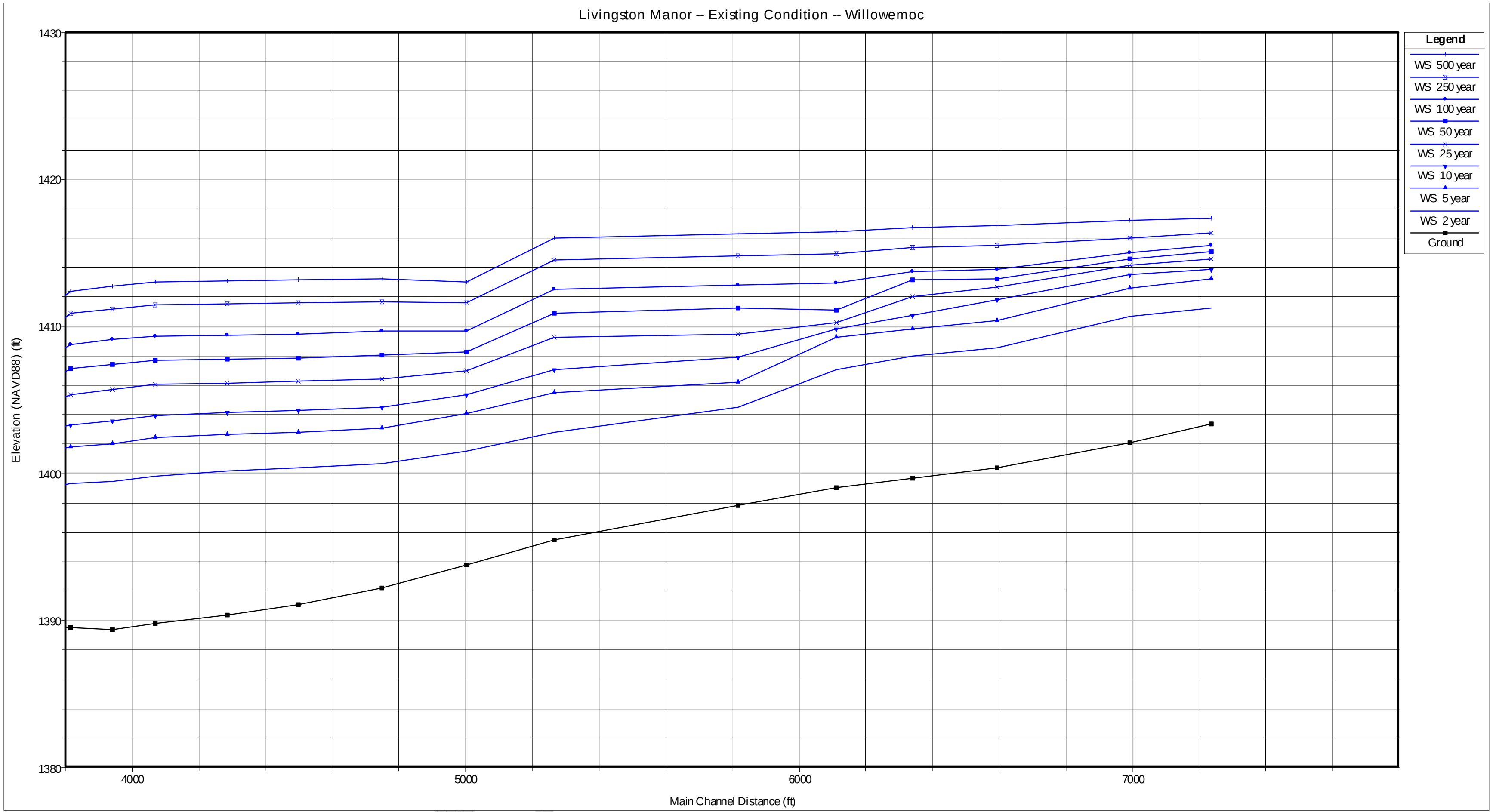


Figure 6.10 - Part 2, Willowemoc – Existing Condition Frequency Water Surface Profiles

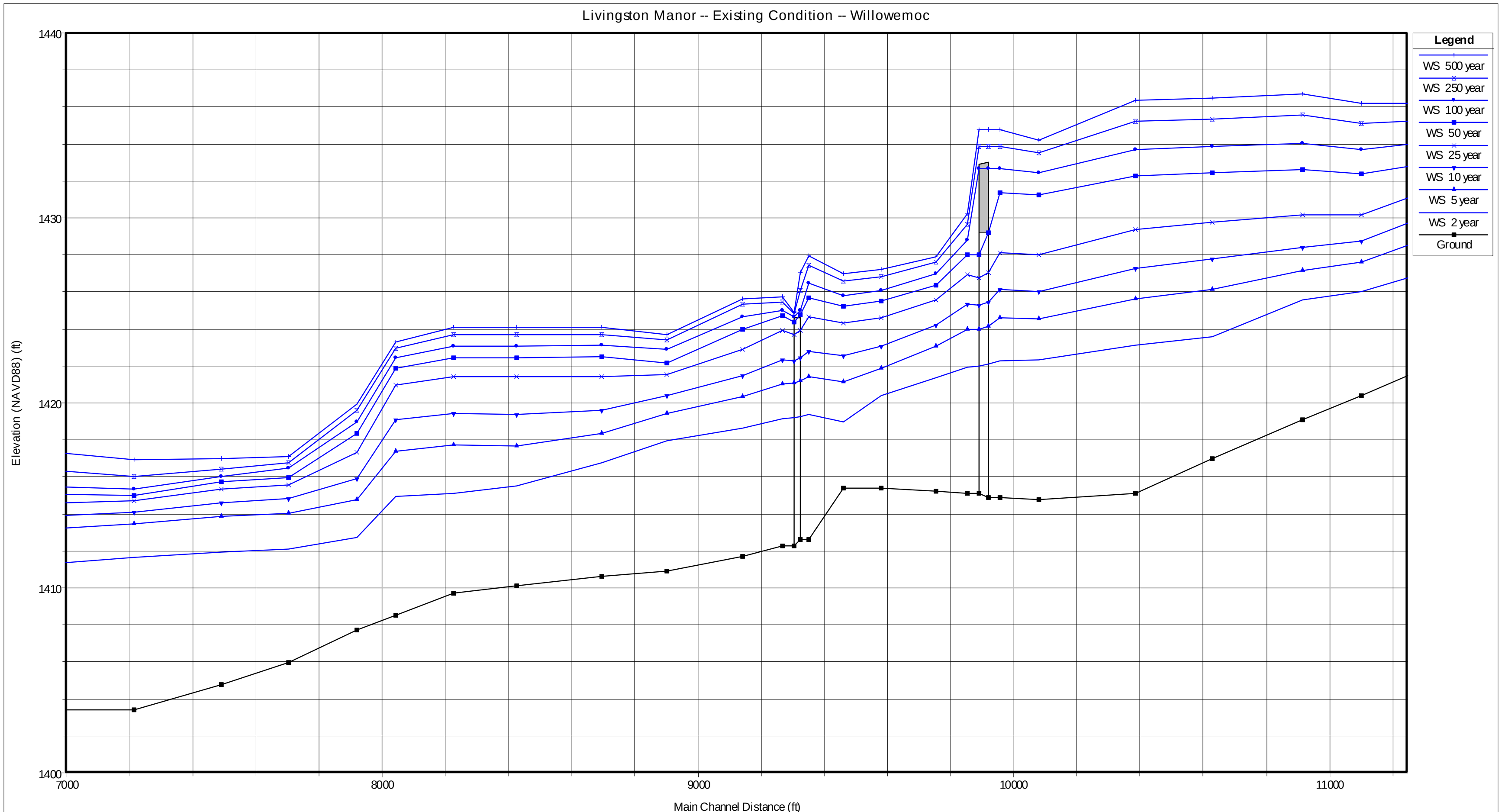


Figure 6.10 - Part 3, Willowemoc – Existing Condition Frequency Water Surface Profiles

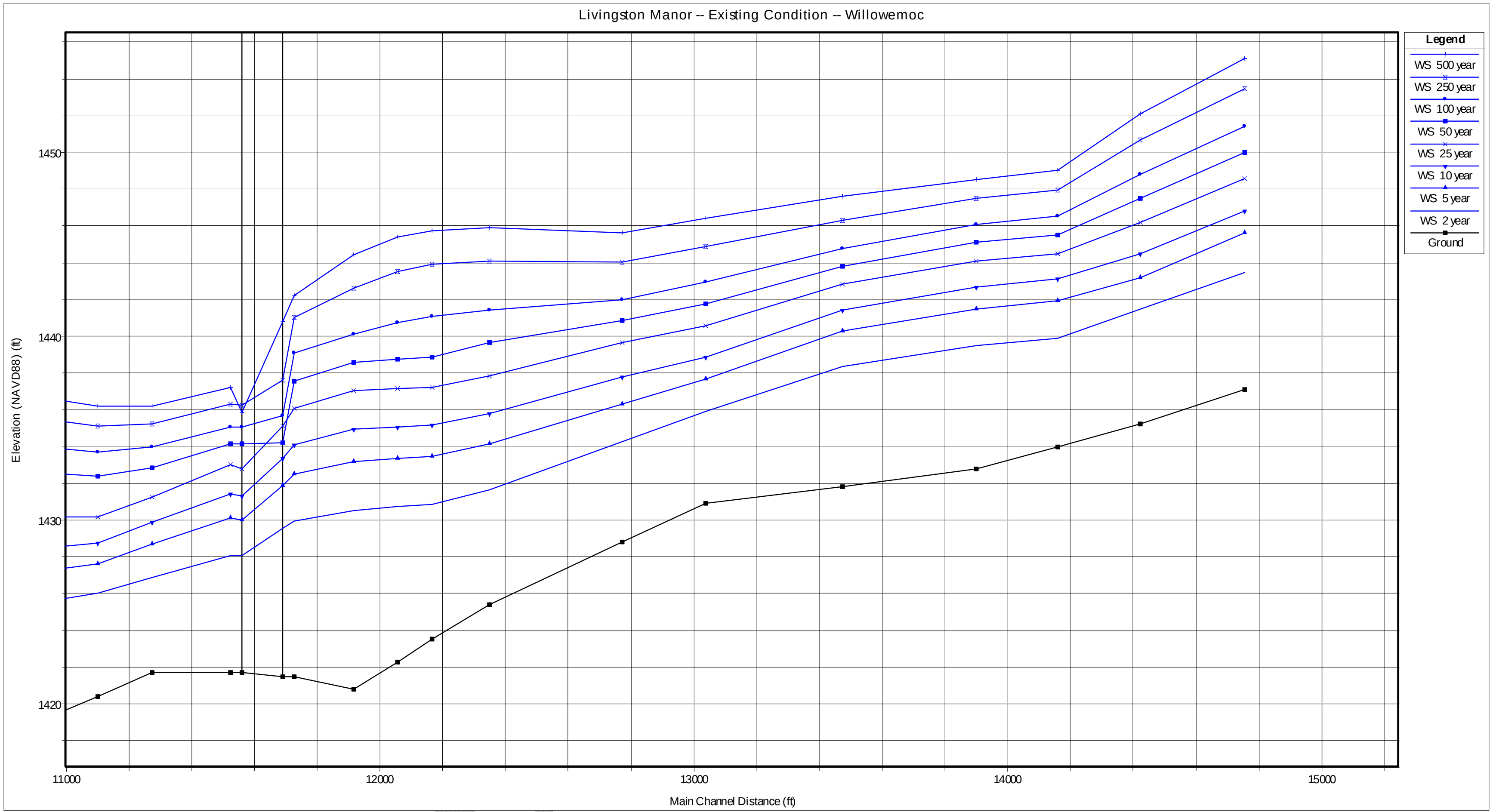


Figure 6.10 - Part 4, Willowemoc – Existing Condition Frequency Water Surface Profiles

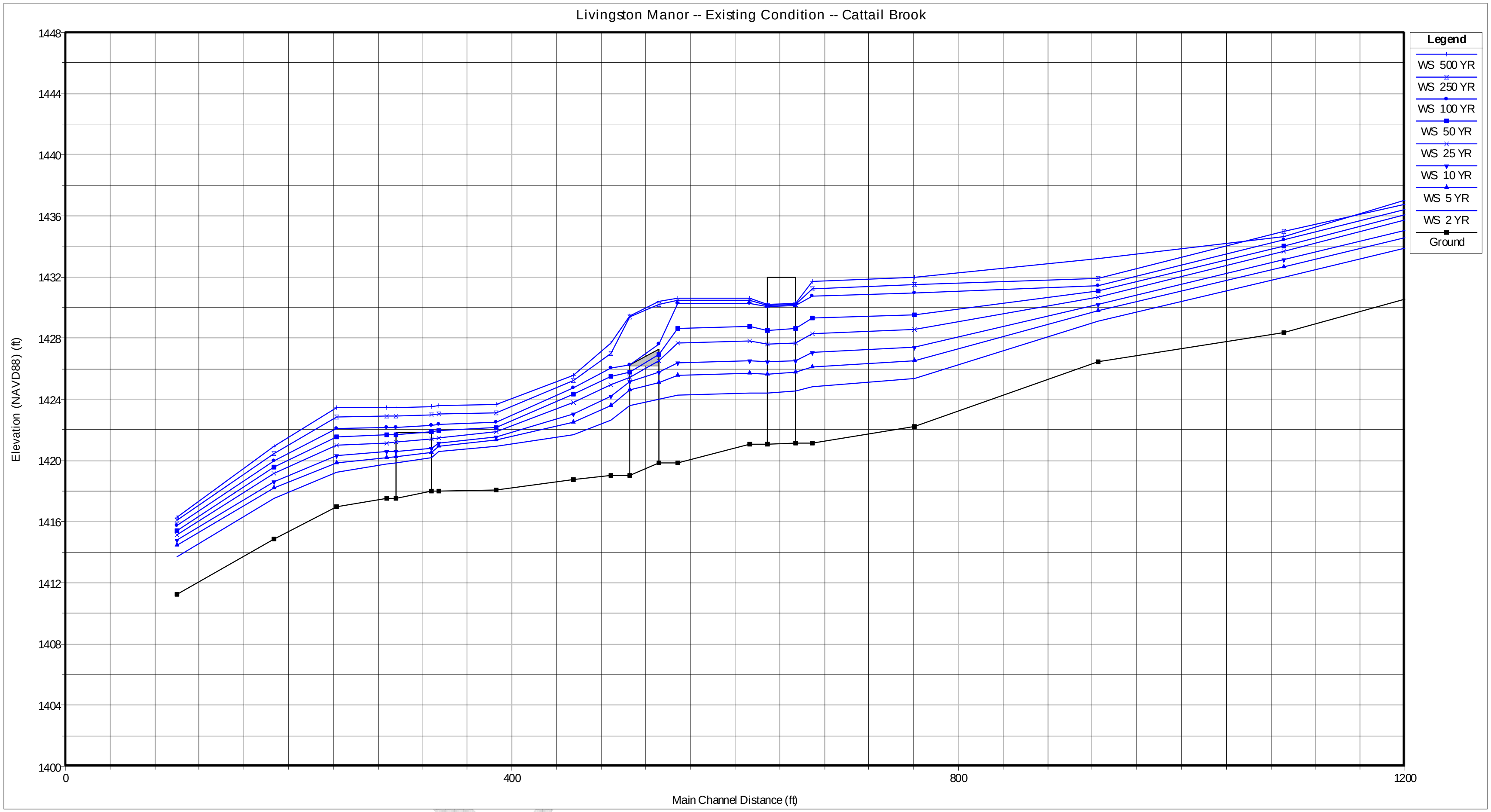


Figure 6.11 - Part 1, Cattail Brook – Existing Condition Frequency Water Surface Profiles

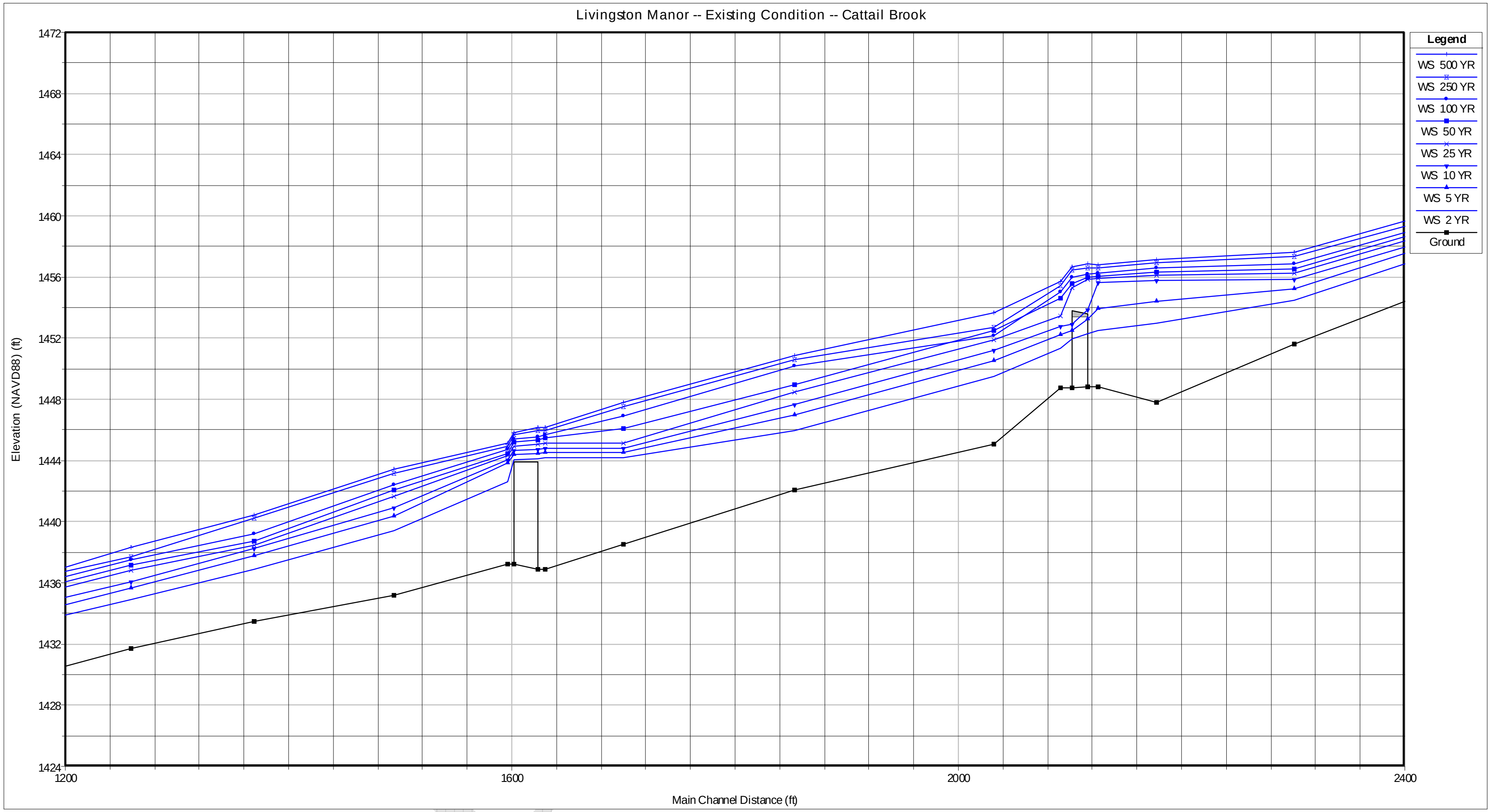


Figure 6.11 - Part 2, Cattail Brook – Existing Condition Frequency Water Surface Profiles

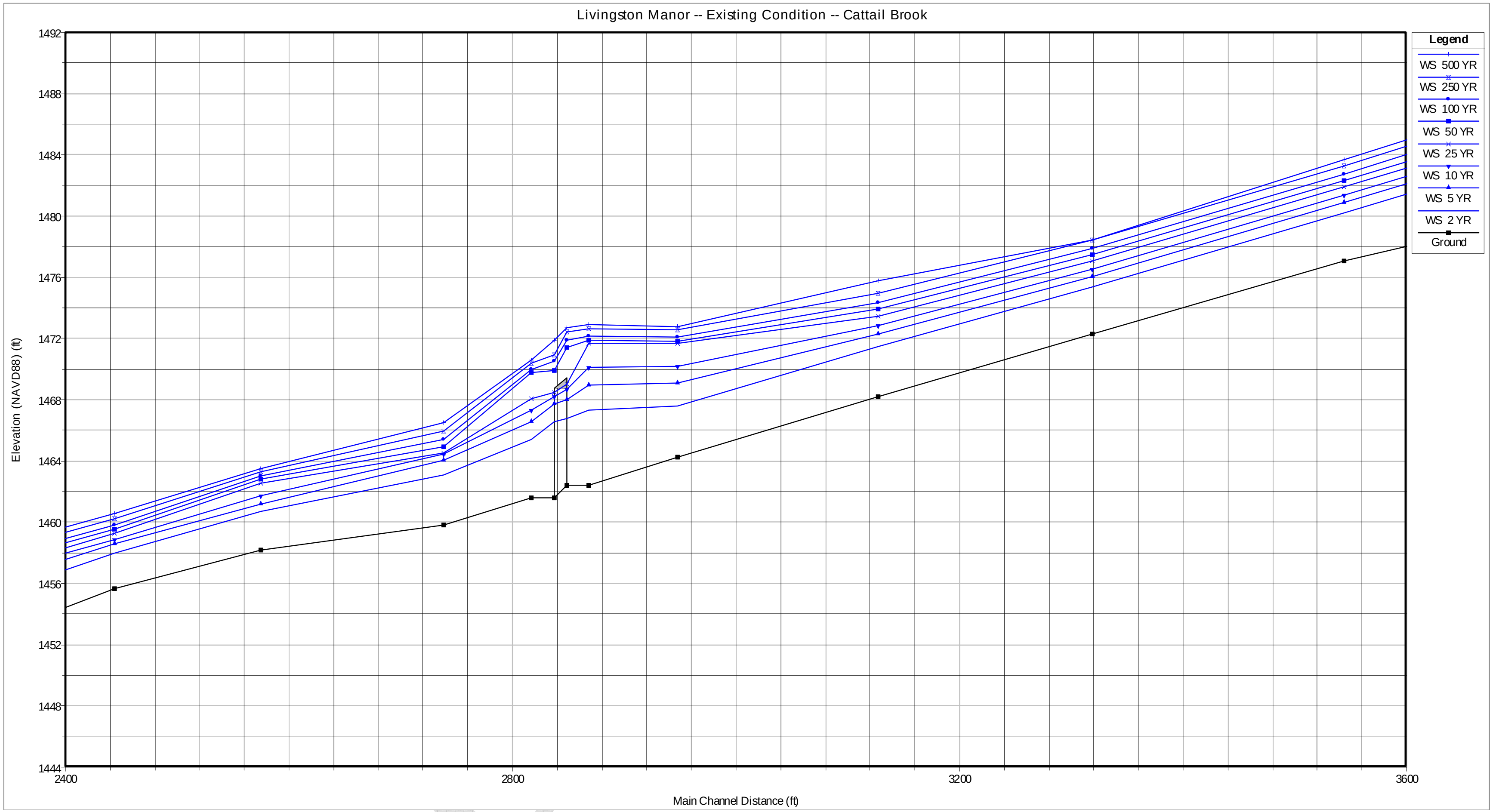


Figure 6.11 - Part 3, Cattail Brook – Existing Condition Frequency Water Surface Profiles

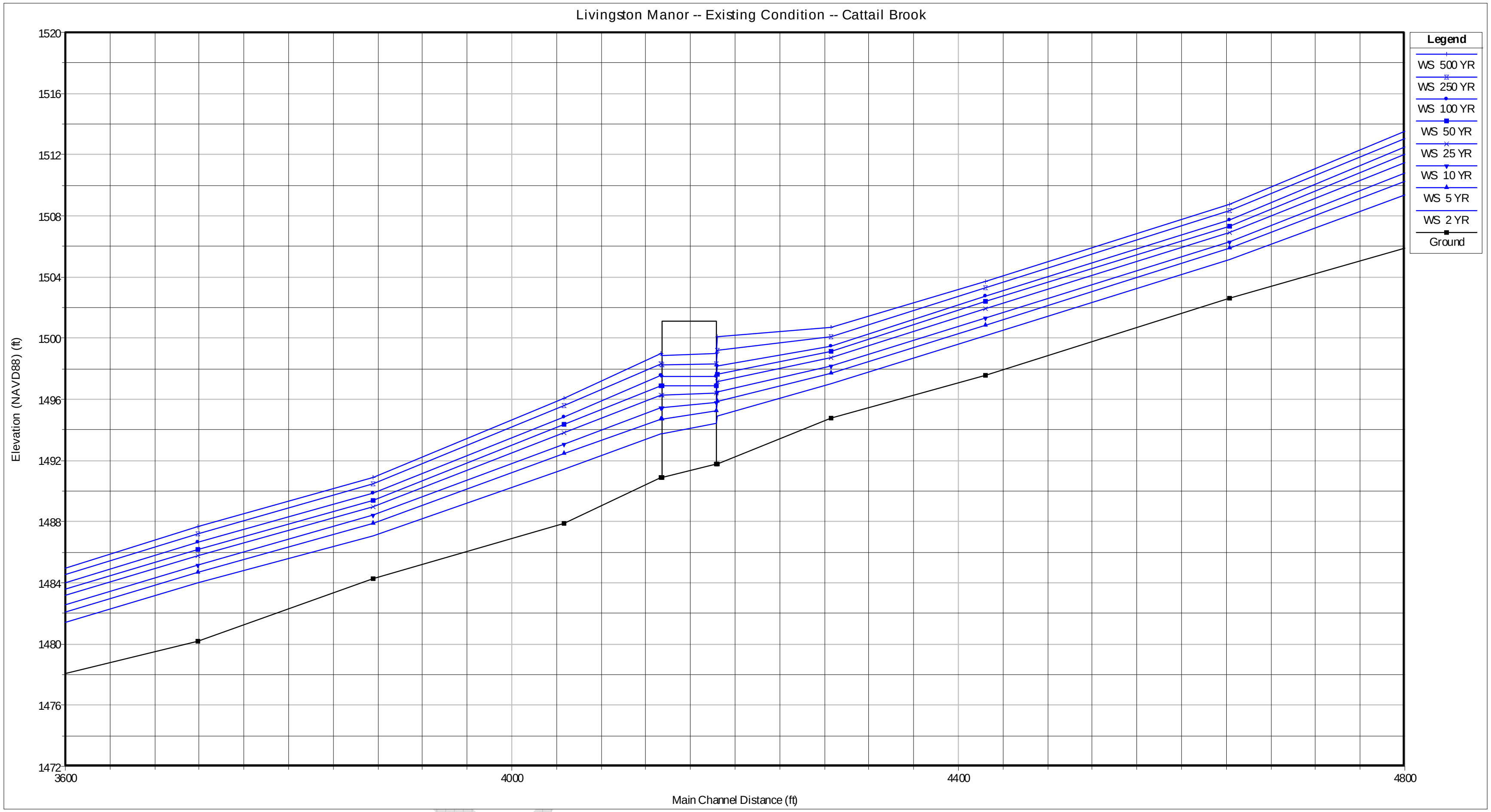


Figure 6.11 – Part 4, Cattail Brook – Existing Condition Frequency Water Surface Profiles

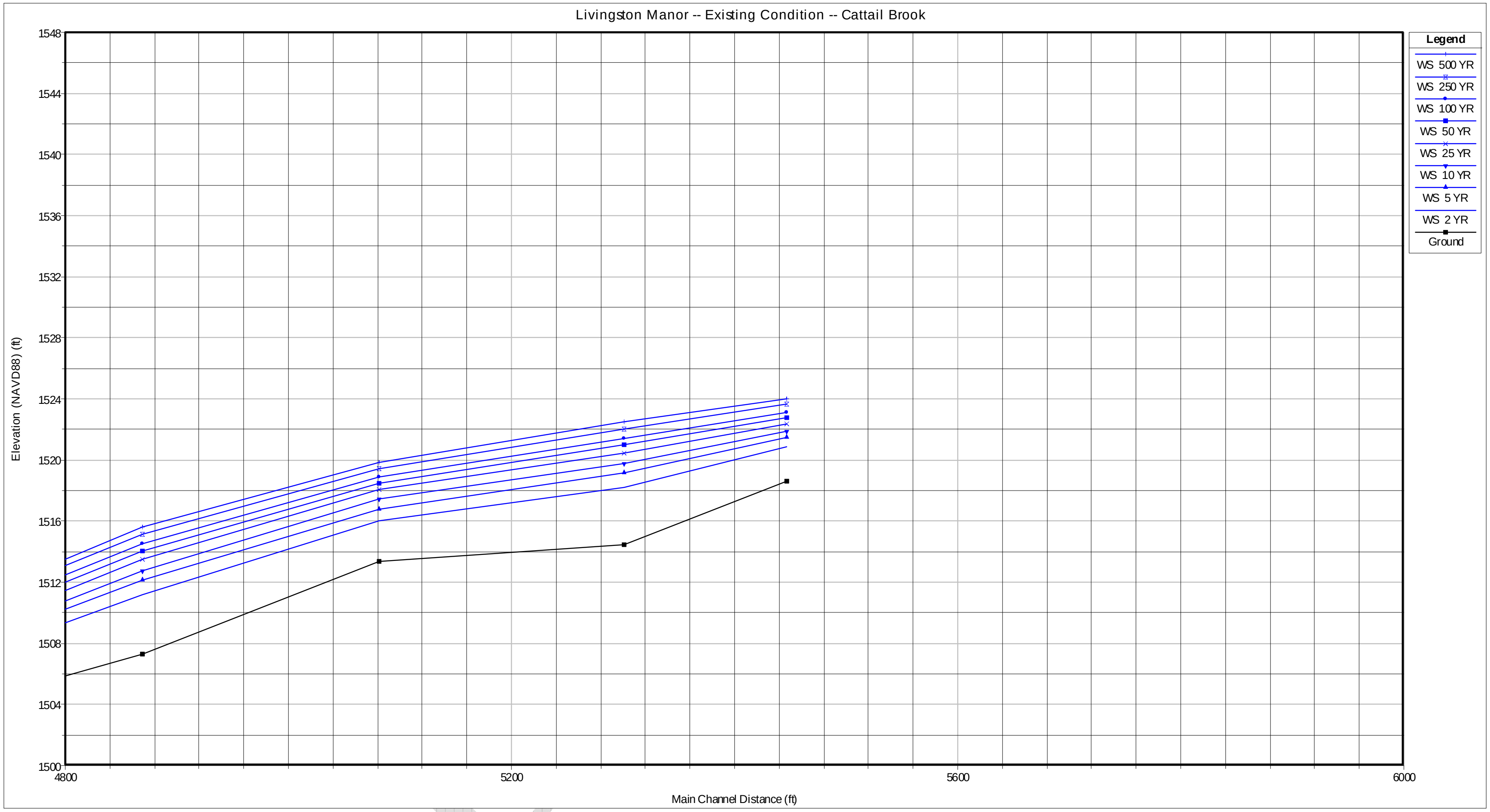


Figure 6.11 – Part 5, Cattail Brook – Existing Condition Frequency Water Surface Profiles

7. ECONOMIC MODEL

Structures within the area of interest were surveyed and valuations assigned to each structure. Structures were aggregated by creek and then further aggregated by reaches. The economic reaches for all of the creeks are shown in Figure 7.1.   Many reaches were specified to ensure an accurate spatial distribution of the damages. For each economic reach a hydraulic cross-section was assigned as an index station. The index stations and their assigned reaches are provided in Tables 7.1, 7.2, 7.3 and 7.4. Water surface elevation-frequency results, shown on Table 7.5, were provided for every index station and average annual damage was calculated for each reach. The majority of the damage is located along Little Beaver Kill Creek.

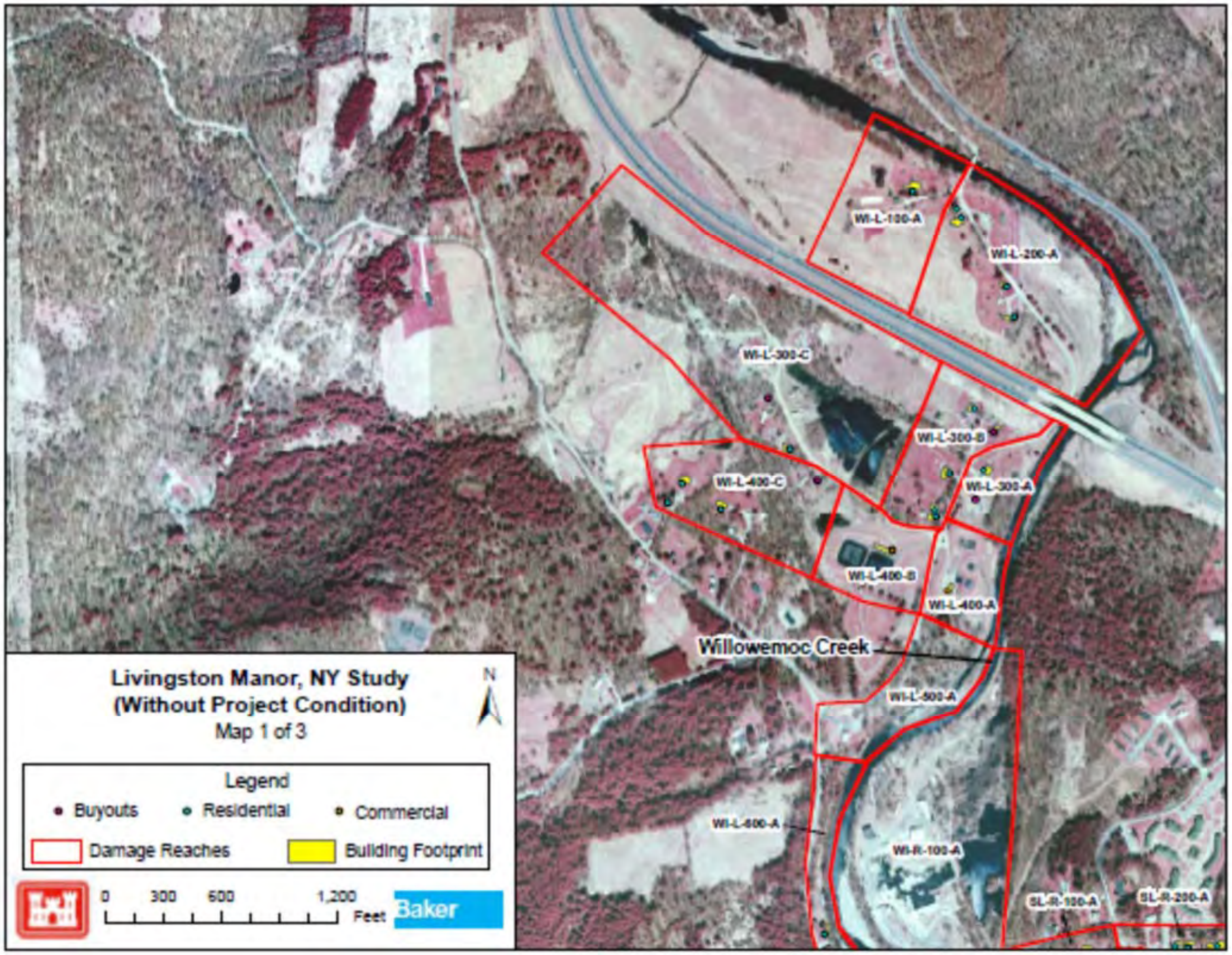


Figure 7.1 - Part 1 - Economic Damage Reaches

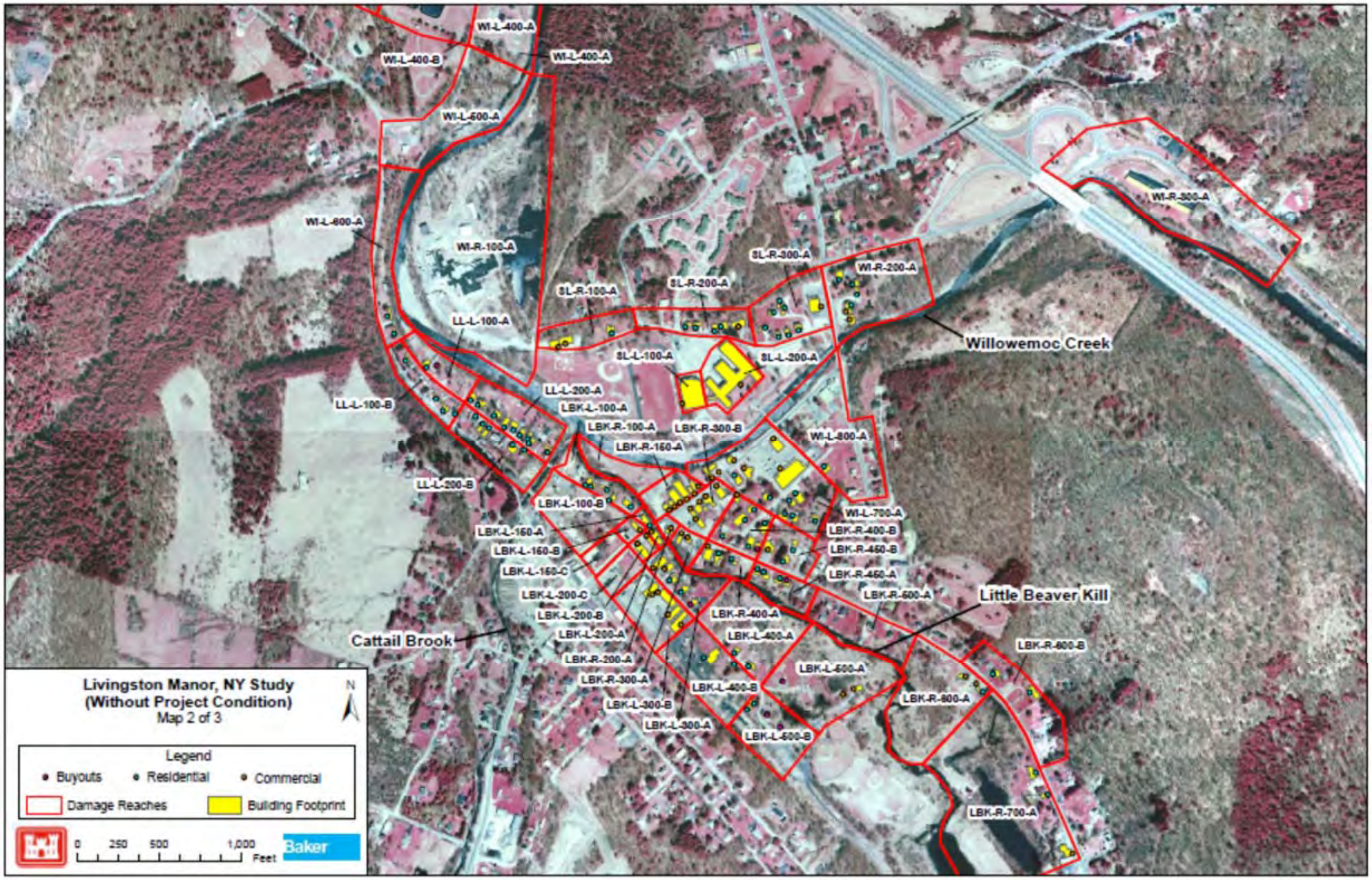


Figure 7.1 - Part 2 - Economic Damage Reaches

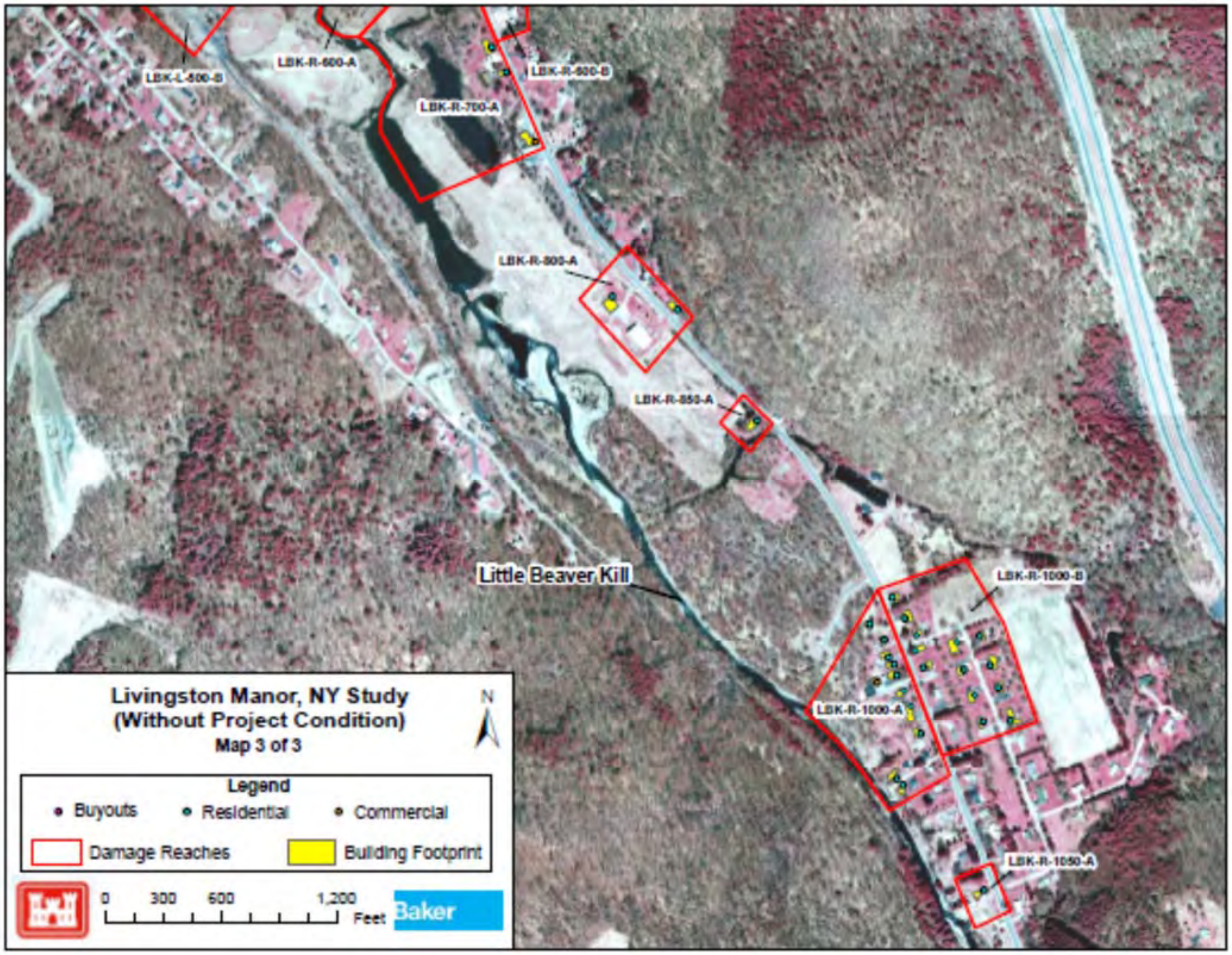


Figure 7.1 - Part 3 - Economic Damage Reaches

Table 7.1 Willowemoc Index Stations	
Economic Reach	X-section to be Used as Index Station
LEFT	
WI-L-100-A	1492
WI-L-200-A	2109
WI-L-300-A	4049
WI-L-300-B	4049
WI-L-300-C	4049
WI-L-400-A	4476
WI-L-400-B	4476
WI-L-500-A	5244
WI-L-600-A	6319
WI-L-700-A	9141
WI-L-800-A	9579
RIGHT	
WI-R-100-A	6319
WI-R-200-A	10079
WI-R-300-A	12348

Notes: - Left and Right are defined looking downstream.
- Numeric labels e.g. 100, 200 proceed from downstream to upstream.
- Alphabetic labels e.g. A, B, C start closest to the creek and proceed perpendicular to the creek towards high ground.

Table 7.2
Little Beaver Kill Index Stations

Economic Reach	X-section to be Used as Index Station
LEFT	
LBK-L-100-A	316
LBK-L-100-B	316
LBK-L-150-A	824 (use WSELs on US face of Bridge for structures on DS side of Bridge)
LBK-L-150-B	
LBK-L-150-C	
LBK-L-200-A	824
LBK-L-200-B	824
LBK-L-200-C	824
LBK-L-300-A	1101
LBK-L-300-B	1101
LBK-L-400-A	1697
LBK-L-400-B	1697
LBK-L-500-A	2138
LBK-L-500-B	2138
RIGHT	
LBK-R-100-A	316
LBK-R-150-A	824 (use WSELs on US face of Bridge for structures on DS side of Bridge)
LBK-R-200-A	824
LBK-R-300-A	942
LBK-R-300-B	942
LBK-R-400-A	1337
LBK-R-400-B	1337
LBK-R-450-A	1697
LBK-R-450-B	1697
LBK-R-500-A	2138
LBK-R-600-A	3293
LBK-R-600-B	3293
LBK-R-700-A	3917
LBK-R-800-A	5862

Table 7.3 Behind Left Levee Index Stations	
Economic Reach	X-section to be Used as Index Station
LEFT	
LL-L-100-A	223
LL-L-100-B	223
LL-L-200-A	764
LL-L-200-B	764
RIGHT	
None	

Table 7.4 Behind School Levee Index Stations	
Economic Reach	X-section to be Used as Index Station
LEFT	
SL-L-100-A	1020
SL-L-200-A	1594
RIGHT	
SL-R-100-A	500
SL-R-200-A	1192
SL-R-300-A	1771

Table 7.5
Existing Condition Frequency Water Surface Elevations
at Economic Index Stations

Economic Index Station	WSEL (ft-NAVD88)							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Willowemoc								
X-1492	1392.26	1394.36	1395.49	1396.26	1396.73	1397.29	1398.03	1398.62
X-2109	1392.98	1395.27	1396.28	1397.63	1398.43	1399.58	1400.23	1401.01
X-4049	1399.82	1402.44	1403.95	1406.04	1407.71	1409.35	1411.46	1413.04
X-4476	1400.35	1402.82	1404.3	1406.27	1407.87	1409.45	1411.6	1413.19
X-5244	1402.82	1405.49	1407.08	1409.24	1410.91	1412.5	1414.5	1416
X-6319	1407.98	1409.81	1410.73	1412.05	1413.19	1413.75	1415.36	1416.73
X-9141	1418.65	1420.36	1421.49	1422.89	1424	1424.66	1425.32	1425.65
X-9579	1420.38	1421.85	1423.07	1424.63	1425.51	1426.07	1426.81	1427.23
X-10079	1422.31	1424.52	1426.04	1427.99	1431.23	1432.47	1433.5	1434.21
X-12348	1431.63	1434.13	1435.77	1437.87	1439.65	1441.44	1444.08	1445.89
Little Beaver Kill								
X-316	1415.38	1417.72	1419.37	1421.28	1422.22	1422.82	1423.46	1423.9
X-824	1417.29	1419.02	1420.07	1423.81	1424.85	1426.25	1426.94	1427.44
X-942	1418.3	1420.81	1422.65	1425.22	1426.07	1427.16	1427.98	1428.57
X-1101	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
X-1337	1418.86	1421.42	1423.07	1425.45	1426.3	1427.38	1428.23	1428.84
X-1697	1419.03	1421.53	1423.15	1425.49	1426.35	1427.42	1428.28	1428.9
X-2138	1419.18	1421.62	1423.21	1425.53	1426.39	1427.46	1428.32	1428.94
X-3293	1419.53	1421.77	1423.31	1425.58	1426.44	1427.51	1428.37	1429
X-3917	1420.3	1421.97	1423.4	1425.62	1426.48	1427.54	1428.41	1429.04
X-5862	1422.83	1423.51	1424.42	1426.18	1426.96	1427.91	1428.77	1429.39
Behind Left Levee								
X-223	1412.77	1413.63	1414.05	1415.07	1415.98	1416.69	1417.72	1418.74
X-764	1413.52	1413.63	1414.2	1415.8	1416.92	1417.64	1418.54	1419.32
Behind School Levee								
X-500	1414.31	1414.31	1414.31	1415.57	1416.68	1417.93	1420.03	1421.72
X-1020	1418.16	1418.16	1418.16	1419.23	1420.27	1420.81	1422.49	1424.19
X-1192	1419.25	1419.25	1419.25	1420.06	1420.7	1422.54	1424.5	1425.21
X-1594	1419.32	1419.32	1419.32	1421.08	1422.42	1424.01	1426.13	1427.72
X-1771	1421.43	1421.43	1421.43	1423.33	1424.38	1425.42	1426.12	1427.57

WITH PROJECT CONDITION

The with project analysis was concentrated on Little Beaver Kill Creek. The aim was to reduce the frequency wsels along Pearl Street. Hydraulic, hydrologic and combined hydraulic and hydrologic solutions were considered. The hydraulic solutions involve floodplain modifications to the Willowemoc and Little Beaver Kill Creek. The hydrologic solutions considered construction of a new reservoir at the Airport Ponds and modification of six existing reservoirs within the Little Beaver Kill watershed.
The two hydrologic solutions were also combined with the best hydraulic solution.

8. HYDRAULIC SOLUTIONS

The area of focus for the hydraulic solutions is shown on Figure 8.1. There are three components to the stage reductions:

- modification of the ball field levees along the Willowemoc to lower the wsels at the mouth of Little Beaver Kill. The ball field modifications involved moving the levee landward and lowering the floodplain on the river side of the relocated levee.

- replacement of Main St bridge over the Little Beaver Kill with a wider bridge.

- lowering of the right overbank of the Little Beaver Kill downstream of Main St.

Figure 8.1 – Overview of Pearl Street Area

Modifying the ball field levees landward has the effect of lowering the Willowemoc water surface elevations at the mouth of the Little Beaver Kill Creek. There are two types of modification: moving the levee landward and moving the levee landward and then lowering the created floodplain approximately 2feet. The floodplain was lowered to the elevation of the existing 2year water surface elevation of the Willowemoc. This was done to maintain the sediment transport capacity of the Willowemoc. Three shifts of the levee were analyzed: 300, 100 and 50ft.

The Main Street Bridge over the Little Beaver Kill is constrictive and causes a jump in the water surface across the bridge. This jump occurs even when the water surface does not touch the steel girder. A new wider bridge was considered. It was assumed that the two buildings, upstream and downstream of the bridge on the left side of the creek will be purchased and demolished allowing the bridge's width to be increased by 20 feet. A plan view of the proposed work is shown on Figure 8.2. Initially the new bridge was analyzed assuming a pier, but the majority of bridge runs assumed that a pier would not be required. In order to protect the fish habitat and to maintain sediment transport capacity a channel bench approximately 5feet above the existing channel was placed under the new portion of the bridge. The new bridge was also analyzed with a 1year bench, approximately 3feet above the existing channel. (Subsequent to the completion of the hydraulic analysis, the building upstream of Main Street Bridge on the right side was destroyed by fire. Additional analysis may show that it is possible to increase the capacity of a new bridge, beyond the increase considered within, by utilizing the newly available space on the right side.)

Figure 8.2 – Plan View of Proposed Widening of Main Street Bridge

Lowering the water surface energy at the downstream face of the existing Main Street Bridge can lower the water surface elevations on the upstream side of the bridge. One way to lower the energy at the downstream face of the bridge is to lower the channel bank height. The right side of the creek downstream from Main Street was excavated creating a bench and providing more flow area. Two elevations were analyzed for the bench: a 2year bench approximately 6ft above the existing channel and a 1year bench, approximately 3ft above the existing channel. For both options the excavation daylight line in the Park is the same. Approximately 10feet of the parking lot downstream of Main Street will need to be taken.

Figure 8.3 is a plan view of the Park with the proposed 1 year bench contours shown. Select 1ft contours have been labeled. Of interest is the highlighted “Limit of Excavation” which shows the extent of the park which must be sacrificed to implement this option. The width of the bench is approximately 25 feet. Trees may be planted at the top of the newer lower banks but the majority of the bench should be planted with grass to provide hydraulic efficiency.

Figure 8.3 – Plan View of 1 Year Bench along Little Beaver Kill

Various scales of modification and various combinations of components were considered. In all 26 separate hydraulic runs were made. Stage reductions were tabulated at various locations as shown on Figure 8.4.

Table 8.1 lists all hydraulic runs and stage reductions for the 5, 25 and 100 year events. The stage reductions are indicative but not determinative of damage reduction. A plan with a large drop in water surface elevation may still result in flood water remaining out of bank. Tables 8.2 and 8.3 provide complete elevation-frequency results for cross-sections 1101 and 1532 respectively. Cross-section 1101 is approximately 250 feet upstream of Main Street Bridge and cross-section 1532 is approximately 680 feet upstream of the bridge at the low spot of Pearl Street, which is elevation 1419 ft-NAVD88. The with project frequency water surface elevations can be compared to ground or structural elevations as an indication of flood protection.

The various runs of Table 8.1, reflecting both options and various scales are summarized below.

Modify ball field levees only.
(Plans: 2, 2A, 2B, 2C, 2D, 2E)

Modify the bridge only
(Plans: 3A, 3B)

Modify bridge and ball field levee.
(Plans: 4A, 4B-1, 4B-2)

Modify floodplain downstream of Main Street only.
(Plans: 5, 5X)

Modify floodplain downstream of Main Street and ball field levee.
(Plans: 5-2, 5-4, 5X-2, 5X-4)

Modify floodplain downstream of Main Street and Main Street Bridge.
(Plans: 6B, 6B-X)

Modify floodplain downstream of Main Street, Main Street Bridge and ball field levee.
(Plans: 6B-2, 6B-4, 6B-6, 6BX-2, 6BX-4, 6CX-4)

Figure 8.4 – Locations of Tabulated Results

Table 8.1
Stage Reductions for Hydraulic Plans

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Existing	No Plan, Flooding unchanged	NA	NA	NA	NA	NA	NA	NA	NA	NA
Plan 1	Rt 17 bridge widened. (D/S of Sewer Plant)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Plan 2	Move ball field levees along Willowemoc 300 ft landward.	0.09	-1.50	-1.67	0.06	-0.65	0.00	0.01	0.02	0.00
Plan 2A	Move ball field levee 300ft + lower floodplain approx 2ftx300ft	-0.99	-2.82	-2.96	-0.5	-0.65	0.00	-0.02	0.02	0.00
Plan 2B	Ball field levee relocation 50 ft landward	0.03	-0.65	-0.51	0.02	-0.65	0.00	0.00	0.02	0.00
Plan 2C	Ball field levee relocation 50ft + lower floodplain 2ftx50ft	-0.36	-1.00	-0.70	-0.21	-0.65	0.00	-0.01	0.02	0.00
Plan 2D	Ball field levee relocation 100 ft landward	0.05	-0.97	-0.87	0.03	-0.65	0.00	0.00	0.02	0.00

Table 8.1 (continued)
Stage Reductions for Hydraulic Plans

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Plan 2E	Ball field levee relocation 100ft + lower floodplain 2ftx100ft	-0.60	-1.74	-1.49	-0.33	-0.65	0.00	-0.02	0.02	0.00
Plan 3A	Main Street Bridge widened WITH pier (Br bench=1414.8)	0.00	0.00	0.00	0.72	1.08	0.00	-0.74	-1.08	-0.96
Plan 3B	Main Street Bridge widened without pier (Br bench=1414.8)	0.00	0.00	0.00	0.72	1.08	0.00	-0.85	-1.28	-1.10
Plan 4A	Main Street Bridge widened WITH pier (3A) (Br bench=1414.8); + move ball field levee 300ft (2)	0.09	-1.5	-1.67	0.76	0.43	0.52	-0.72	-1.61	-0.84
Plan 4B-1	Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + move ball field levee 300ft (2)	0.09	-1.50	-1.67	0.76	0.43	0.52	-0.83	-1.80	-0.99
Plan 4B-2	Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + move ball field levee 300ft and floodplain lowering (2A)	-0.99	-2.82	-2.97	0.42	0.31	0.78	-0.96	-1.84	-0.90

Table 8.1 (continued)
Stage Reductions for Hydraulic Plans

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Plan 4B-3	Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 50ft relocation (2B)	**Not Analyzed since stage reductions will be less than Plan 4B-2.**								
Plan-4B-4	Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 50ft relocation and floodplain lowering (2C)	**Not Analyzed since stage reductions will be less than Plan 4B-2.**								
Plan 4B-5	Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 100ft relocation 2D	**Not Analyzed since stage reductions will be less than Plan 4B-2.**								
Plan 4B-6	Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 100ft relocation and floodplain lowering (2E)	**Not Analyzed since stage reductions will be less than Plan 4B-2.**								
Plan 5	Widen LBK Floodplain below existing Main Street Bridge (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03	0.00	0.00	0.00	0.14	0.59	-0.22	-0.56	-0.32	-0.18

Table 8.1 (continued) Stage Reductions for Hydraulic Plans										
Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Plan 5-2	Widen LBK Floodplain below existing Main Street Bridge (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03) + move ball field levee 300ft and floodplain lowering (2A)	-0.99	-2.82	-2.97	-0.36	-1.17	-1.94	-0.56	-0.31	-0.18
Plan 5-4	Widen LBK Floodplain below existing Main Street Bridge (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03) + ball field levee 50ft relocation and floodplain lowering (2C)	-0.36	-1.00	-0.70	-0.07	-0.29	-0.99	-0.56	-0.31	-0.18
Plan 6B	Widen LBK Floodplain below Main Street Bridge (P5) (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8)	0.00	0.00	0.00	0.61	1.02	0.21	-1.00	-1.65	-1.12
Plan 6B-2	Widen LBK Floodplain below Main Street Bridge (P5) (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + move ball field levee 300ft and floodplain lowering (2A)	-0.99	-2.82	-2.97	0.35	-0.33	-1.17	-1.08	-2.18	-1.48

Table 8.1 (continued) Stage Reductions for Hydraulic Plans										
Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25y	100yr	5yr	25yr	100yr
Plan 6B-4	Widen LBK Floodplain below Main Street Bridge (P5) (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 50ft relocation and floodplain lowering (2C)	-0.36	-1.00	-0.70	0.49	0.28	-0.34	-1.04	-2.01	-1.13
Plan 6B-6	Widen LBK Floodplain below Main Street Bridge (P5) (Bench=1416 (2yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 100ft relocation and floodplain lowering (2E)	-0.60	-1.74	-1.49	0.43	-0.06	-0.92	-1.06	-2.11	-1.18
Plan 5X	Widen LBK Floodplain below existing Main Street Bridge(Bench=1413 (1yr); SS-1V-2H; Bench n=0.03)	0.00	0.00	0.00	-0.01	0.58	-0.22	-0.56	-0.32	-0.18
Plan 5X-2	Widen LBK Floodplain below existing Main Street Bridge (Bench=1413 (1yr); SS-1V-2H; Bench n=0.03) + move ball field levee 300ft and floodplain lowering (2A)	-0.99	-2.82	-2.97	-0.58	-1.64	-3.12	-0.56	-0.31	-0.18

Table 8.1 (continued) **Stage Reductions for Hydraulic Plans**										
Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge(X-1101)		
		5 yr	25yr	100yr	5yr	25yrr	100yr	5yr	25yr	100yr
Plan 5X-4	Widen LBK Floodplain below existing Main Street Bridge (Bench=1413 (1yr); SS-1V-2H; Bench n=0.03) + ball field levee 50ft relocation and floodplain lowering (2C)	-0.36	-1.00	-0.70	-0.19	-0.26	-0.88	-0.56	-0.32	-0.18
Plan 6B-X	Widen LBK Floodplain below Main Street Bridge (P5X) (Bench=1413 (1yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8)	0.00	0.00	0.00	0.19	0.87	0.08	-1.17	-1.84	-1.12
Pln 6BX-2	Widen LBK Floodplain below Main Street Bridge (P5X) (Bench=1413 (1yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + move ball field levee 300ft and floodplain lowering (2A)	-0.99	-2.82	-2.97	-0.32	-1.04	-2.10	-1.22	-2.22	-1.56

Table 8.1 (continued) **Stage Reductions for Hydraulic Plans**										
Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Pln 6BX-4	Widen LBK Floodplain below Main Street Bridge (P5X) (Bench=1413 (1yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1414.8) + ball field levee 50ft relocation and floodplain lowering (2C)	-0.36	-1.00	-0.70	-0.02	0.01	-0.53	-1.21	-2.15	-1.16
Pln 6CX-4	Widen LBK Floodplain below Main Street Bridge (P5X) (Bench=1413 (1yr); SS-1V-2H; Bench n=0.03) Main Street Bridge widened withOUT pier (3B) (Br bench=1413.0) + ball field levee 50ft relocation and floodplain lowering (2C)	-0.36	-1.00	-0.70	0.08	0.11	-0.51	-1.49	-2.39	-1.47

Table 8.2
Frequency Water Surface Elevations (ft-NAVD88)
250ft Upstream of Main Street (X-1101)

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
Plan 1	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
Plan 2	1418.68	1421.3	1422.98	1425.43	1426.27	1427.34	1428.19	1428.8
Plan 2A	1418.68	1421.28	1423.00	1425.43	1426.27	1427.34	1428.19	1428.8
Plan 2B	1418.68	1421.29	1422.99	1425.43	1426.27	1427.34	1428.19	1428.8
Plan 2C	1418.68	1421.28	1422.99	1425.43	1426.27	1427.34	1428.19	1428.8
Plan 2D	1418.68	1421.29	1422.98	1425.43	1426.27	1427.34	1428.19	1428.8
Plan 2E	1418.68	1421.27	1423.00	1425.43	1426.27	1427.34	1428.19	1428.8
Plan 3A	1418.33	1420.55	1422.14	1424.33	1425.43	1426.38	1427.73	1428.42
Plan 3B	1418.25	1420.44	1422.00	1424.13	1425.3	1426.24	1427.58	1428.34
Plan 4A	1418.33	1420.57	1422.05	1423.8	1425.39	1426.5	1427.73	1428.42
Plan 4B-1	1418.25	1420.46	1421.9	1423.61	1425.21	1426.35	1427.56	1428.34
Plan 4B-2	1418.25	1420.33	1421.79	1423.57	1425.24	1426.44	1427.56	1428.34
Plan 4B-3	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 4B-4	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 4B-5	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 4B-6	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 5	1418.68	1420.73	1422.78	1425.09	1426.27	1427.16	1428.19	1428.8
Plan 5-2	1418.68	1420.73	1422.76	1425.1	1426.27	1427.16	1428.19	1428.8
Plan 5-4	1418.68	1420.73	1422.76	1425.1	1426.27	1427.16	1428.19	1428.8
Plan 6B	1418.21	1420.29	1421.8	1423.76	1425.22	1426.22	1427.54	1428.34
Plan 6B-2	1418.21	1420.21	1421.54	1423.23	1424.52	1425.86	1427.36	1428.34
Plan 6B-4	1418.21	1420.25	1421.65	1423.4	1424.99	1426.21	1427.48	1428.34

Table 8.2 (continued) Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream Of Main Street (X-1101)								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Plan 6B-6	1418.21	1420.23	1421.59	1423.3	1424.57	1426.16	1427.39	1428.34
Plan 5X	1418.52	1420.73	1422.76	1425.09	1426.27	1427.16	1428.19	1428.8
Plan 5X-2	1418.52	1420.73	1422.76	1425.1	1426.27	1427.16	1428.19	1428.8
Plan 5X-4	1418.52	1420.73	1422.76	1425.09	1426.27	1427.16	1428.19	1428.8
Plan 6BX	1418.04	1420.12	1421.62	1423.57	1425.14	1426.22	1427.49	1428.34
Plan 6BX-2	1418.04	1420.07	1421.14	1423.19	1424.44	1425.78	1427.27	1428.34
Plan 6BX-4	1418.04	1420.08	1421.47	1423.26	1424.61	1426.18	1427.38	1428.34
Plan 6CX-4	1417.73	1419.8	1421.2	1423.02	1424.32	1425.87	1427.37	1428.23

Table 8.3
Frequency Water Surface Elevations (ft-NAVD88)
680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532)

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
Plan 1	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
Plan 2	1418.97	1421.5	1423.11	1425.49	1426.34	1427.41	1428.26	1428.88
Plan 2A	1418.97	1421.48	1423.12	1425.49	1426.34	1427.41	1428.26	1428.88
Plan 2B	1418.97	1421.49	1423.11	1425.49	1426.34	1427.41	1428.26	1428.88
Plan 2C	1418.97	1421.48	1423.11	1425.49	1426.34	1427.41	1428.26	1428.88
Plan 2D	1418.97	1421.49	1423.11	1425.49	1426.34	1427.41	1428.26	1428.88
Plan 2E	1418.97	1421.48	1423.12	1425.49	1426.34	1427.41	1428.26	1428.88
Plan 3A	1418.67	1420.86	1422.34	1424.44	1425.53	1426.47	1427.82	1428.51
Plan 3B	1418.6	1420.77	1422.21	1424.24	1425.4	1426.34	1427.67	1428.43
Plan 4A	1418.67	1420.88	1422.25	1423.94	1425.49	1426.58	1427.81	1428.51
Plan 4B-1	1418.6	1420.78	1422.13	1423.77	1425.31	1426.44	1427.66	1428.43
Plan 4B-2	1418.6	1420.68	1422.03	1423.73	1425.34	1426.53	1427.66	1428.43
Plan 4B-3	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 4B-4	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 4B-5	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 4B-6	**Not Analyzed since stage reductions will be less than Plan 4B-2.**							
Plan 5	1418.97	1421.01	1422.91	1425.17	1426.34	1427.23	1428.26	1428.88
Plan 5-2	1418.97	1421.01	1422.9	1425.17	1426.34	1427.23	1428.26	1428.88
Plan 5-4	1418.97	1421.01	1422.9	1425.17	1426.34	1427.23	1428.26	1428.88
Plan 6B	1418.56	1420.65	1422.04	1423.9	1425.33	1426.31	1427.63	1428.43
Plan 6B-2	1418.56	1420.59	1421.83	1423.42	1424.66	1425.98	1427.46	1428.43
Plan 6B-4	1418.56	1420.62	1421.92	1423.57	1425.1	1426.31	1427.57	1428.43

Table 8.3 (continued) Frequency Water Surface Elevations (ft-NAVD88) 680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532)								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Plan 6B-6	1418.56	1420.6	1421.87	1423.48	1424.71	1426.26	1427.48	1428.43
Plan 5X	1418.83	1421.01	1422.9	1425.17	1426.34	1427.23	1428.26	1428.88
Plan 5X-2	1418.83	1421.01	1422.9	1425.17	1426.34	1427.23	1428.26	1428.88
Plan 5X-4	1418.83	1421.01	1422.9	1425.17	1426.34	1427.23	1428.26	1428.88
Plan 6BX	1418.41	1420.51	1421.89	1423.73	1425.25	1426.31	1427.59	1428.43
Plan 6BX-2	1418.41	1420.47	1421.5	1423.38	1424.59	1425.9	1427.37	1428.43
Plan 6BX-4	1418.41	1420.48	1421.77	1423.44	1424.74	1426.28	1427.47	1428.43
Plan 6CX-4	1418.16	1420.25	1421.55	1423.23	1424.48	1425.98	1427.46	1428.33

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.

9. HYDROLOGIC SOLUTIONS

The hydrologic solutions consist of a dry dam just upstream of Livingston Manor at the Airport Ponds. This solution is called the Fulton Plan named after a local citizen who suggested it. In addition six existing upstream dams were considered for modification to fully control the watershed above them.

A. Fulton Plan

A possible realization of the Fulton Plan is shown on Figure 9.1. The ground surrounding the airport ponds is raised to elevation 1428 ft-NAVD88. There is limited storage at the site so the embankment design allows for safe overtopping. This is accomplished with a 5% vegetated exit slope. The embankment across the channel is provided with sufficient freeboard to prevent overtopping. Three variations of the channel outlet were analyzed:

A – a gated structure that releases inflow up to 1600 cfs. Inflow greater than 1600cfs is throttled so channel outflow is not greater than 1600 cfs. (1600cfs is the flow that produces a wsel 1ft lower than Pearl Street under existing channel geometry.)

B – a constrictive open channel with a bottom width of 12 ft and side slopes of 1V – 2H

C – a constrictive open channel with a bottom width of 5 ft and side slopes of 1V – 2H

Because of the limited storage of the site, the effectiveness of the design depends on the ability of the throttle to pass, without ponding, the non-damaging flows while storing the higher flows.

The effectiveness of the Fulton Plan was assessed with a reservoir routing analysis. The elevation-capacity curve was determined by measuring the surface areas of the 2 ft contours upstream of the embankment and calculating the volumes with the average end

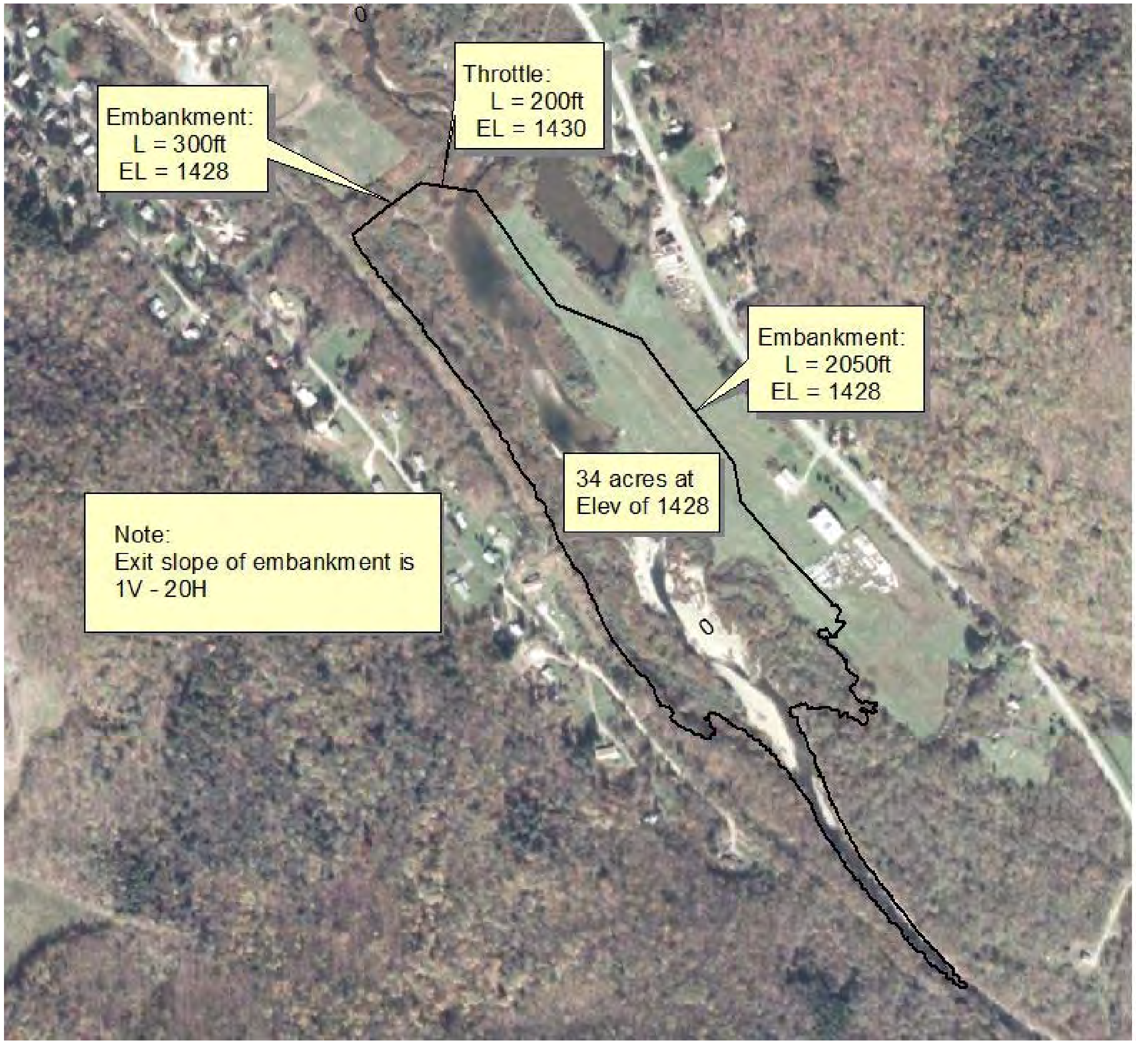


Figure 9.1 - Fulton Plan

method. Table 9.1 provides the elevation-volume results. The elevation-capacity curve is the same for all three low level outlet options.

The outlet rating curves for the three low level options are shown on Table 9.2.   The curves were calculated by adding the embankment/spillway rating curve (constant for any low level outlet) to each low level rating curve. The low level rating curves for the various throttles were determined with appropriately modified existing condition HEC-RAS models.   The embankment rating curve was calculated with a backwater analysis starting at normal depth at the downstream toe of the embankment.

Frequency inflow hydrographs to the Fulton impoundment were taken from the existing condition HEC-HMS model. 24 hour frequency precipitation was applied to the calibrated existing condition HMS model to produce estimates of the frequency flow hydrographs. However, since the peak flows of the HMS calculated hydrographs were low relative to the 17B estimates of frequency flow, the HMS generated hydrographs were adjusted upward. The frequency ratios of the17B to HMS peak flows were used to adjust all of the flows of the hydrographs.

The 5, 10, 25, 50 and 100 year events were routed through the proposed reservoirs and the flow reductions are shown in Table 9.3.

Table 9.1
Elevation Capacity for Three Throttle Options

Elevation (ft-NAVD88)	Cumulative Storage Volume (acre-feet)
1416	0
1418	10.46
1420	27.45
1422	53.43
1428	201.11
1430	268.32

Table 9.2
Outlet Rating Curves for Three Throttle Options

Option A (max release 1600 cfs)				Option B (BW=12ft)				Option C (BW=5ft)			
Elevation (NAVD)	Low Level (cfs)	Spillway (cfs)	Total Outflow (cfs)	Elevation (NAVD)	Low Level (cfs)	Spillway (cfs)	Total Outflow (cfs)	Elevation (NAVD)	Low Level (cfs)	Spillway (cfs)	Total Outflow (cfs)
1416.11	50	0	50	1416.38	50	0	50	1416.84	50	0	50
1416.61	100	0	100	1416.96	100	0	100	1417.63	100	0	100
1417.65	250	0	250	1418.17	250	0	250	1419.18	250	0	250
1418.41	400	0	400	1419.13	400	0	400	1420.23	400	0	400
1418.85	510	0	510	1419.71	510	0	510	1420.87	510	0	510
1419.56	750	0	750	1420.79	750	0	750	1422.01	750	0	750
1420.20	1000	0	1000	1421.72	1000	0	1000	1422.99	1000	0	1000
1420.98	1300	0	1300	1422.68	1300	0	1300	1423.99	1300	0	1300
1421.69	1600	0	1600	1423.51	1600	0	1600	1424.86	1600	0	1600
1428.00	1600	0	1600	1424.24	1890	0	1890	1425.61	1890	0	1890
1428.08	1600	10	1610	1426.67	3066	0	3066	1428.00	3014	0	3014
1428.16	1600	50	1650	1428.00	3852	0	3852	1428.08	3052	10	3062
1428.31	1600	200	1800	1428.08	3899	10	3909	1428.16	3095	50	3145
1428.48	1600	500	2100	1428.16	3946	50	3996	1428.31	3183	200	3383
1428.68	1600	1000	2600	1428.31	4046	200	4246	1428.48	3282	500	3782
1428.84	1600	1500	3100	1428.48	4166	500	4666	1428.68	3399	1000	4399
1428.97	1600	2000	3600	1428.68	4307	1000	5307	1428.84	3492	1500	4992
1429.20	1600	3000	4600	1428.84	4419	1500	5919	1428.97	3568	2000	5568
1429.40	1600	4000	5600	1428.97	4511	2000	6511	1429.20	3702	3000	6702
1429.57	1600	5000	6600	1429.20	4673	3000	7673	1429.40	3819	4000	7819
1429.74	1600	6000	7600	1429.40	4814	4000	8414				

Table 9.3
Reduced Flows (cfs) from the Fulton Plan

Condition	Throttle	5 year	10 year	25 year	50 year	100 year
Existing		3017	3921	5172	6218	7292
Fulton Plan -A	Gates; Max Release 1600cfs	2215	3512	5028	6200	7283
Fulton Plan -B	Bottom width 12 ft; SS:1V-2H	2594	3448	4909	6161	7277
Fulton Plan -C	Bottom width 5ft; SS:1V-2H	2535	3512	5044	6197	7282

The flows in Table 9.3 apply downstream from the Airport Ponds to the mouth of Little Beaver Kill Creek. The water surface elevations corresponding to the frequency flows of Fulton Plans A and B were calculated with the existing condition hydraulic model and the results are found in Tables 9.4, 9.5 and 9.6. (Plan C was not run because the flows are similar to Plan B.) The frequency water surface elevations for Fulton Plans A and B were also calculated with the geometry of the best of the hydraulic plans, Plan 6CX-4. The results are found in Tables 9.7, 9.8 and 9.9.

Table 9.4 **Stage Reductions** **Fulton Plan with Existing Channel Geometry**										
Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Fulton Plan -A	Gates; Max Release 1600cfs	-1.80	-0.33	-0.05	-1.22	-0.25	-0.14	-1.85	-0.59	-0.11
Fulton Plan -B	Bottom width 12 ft; SS:1V-2H	-1.00	-0.51	-0.06	-0.68	-0.40	-0.14	-1.02	-0.79	-0.11

Table 9.5
Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream of Main Street (X-1101)
Fulton Plan with Existing Channel Geometry

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
Fulton Plan -A	1418.68	1419.44	1422.15	1424.82	1426.21	1427.23	1428.19	1428.80
Fulton Plan -B	1418.68	1420.27	1422.03	1424.62	1426.19	1427.23	1428.19	1428.80

Table 9.6
Frequency Water Surface Elevations (ft-NAVD88)
680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532)
Fulton Plan with Existing Channel Geometry

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
Fulton Plan -A	1418.97	1419.73	1422.30	1424.90	1426.28	1427.30	1428.26	1428.88
Fulton Plan -B	1418.97	1420.53	1422.19	1424.70	1426.26	1427.30	1428.26	1428.88

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.

Table 9.7
Stage Reductions
Fulton Plan with Plan 6CX-4 Channel Geometry

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Fulton Plan -A	Gates; Max Release 1600cfs	-1.90	-1.28	-0.77	-1.18	-0.13	-0.59	-2.95	-2.69	-1.59
Fulton Plan -B	Bottom width 12 ft; SS:1V-2H	-1.21	-1.43	-0.77	-0.61	-0.26	-0.59	-2.28	-2.85	-1.59

Table 9.8 Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream of Main Street (X-1101) Fulton Plan with Plan 6CX-4 Channel Geometry								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
Fulton Plan -A	1417.73	1418.34	1420.49	1422.72	1424.21	1425.75	1427.37	1428.23
Fulton Plan -B	1417.73	1419.01	1420.39	1422.56	1424.16	1425.75	1427.37	1428.23

Note: Existing water surface elevations are based on existing flows and existing channel geometry.

Table 9.9 Frequency Water Surface Elevations (ft-NAVD88) 680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532) Fulton Plan with Plan 6CX-4 Channel Geometry								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
Fulton Plan -A	1418.16	1418.79	1420.91	1422.95	1424.37	1425.86	1427.46	1428.33
Fulton Plan -B	1418.16	1419.48	1420.82	1422.79	1424.32	1425.86	1427.46	1428.33

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.
Existing water surface elevations are based on existing flows and existing channel geometry.

The water surface elevations corresponding to the frequency flows of the Fulton Plan – Option B outlet were calculated with the Plan 5 hydraulic model (2 year bench downstream of Main Street) and the results are found in Tables 9.10, 9.11 and 9.12. The frequency water surface elevations for the Fulton Plan – Option B were also calculated with the geometry of the Plan 5X hydraulic model (1 year bench downstream of Main Street) and the results are found in Tables 9.13, 9.14 and 9.15.

Table 9.10
Stage Reductions
Fulton Plan with Plan 5 Channel Geometry

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Fulton Plan -B	Bottom width 12 ft; SS:1V-2H	-1.00	-0.51	-0.06	-0.60	0.14	-0.28	-1.14	-0.95	-0.42

Table 9.11
Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream of Main Street (X-1101)
Fulton Plan with Plan 5 Channel Geometry

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
Fulton Plan -B	1418.68	1420.15	1421.83	1424.46	1426.19	1426.92	1428.19	1428.80

Note: Existing water surface elevations are based on existing flows and existing channel geometry.

Table 9.12
Frequency Water Surface Elevations (ft-NAVD88)
680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532)
Fulton Plan with Plan 5 Channel Geometry

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
Fulton Plan -B	1418.97	1420.43	1422.01	1424.55	1426.26	1426.99	1428.26	1428.88

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.
Existing water surface elevations are based on existing flows and existing channel geometry.

Table 9.13
Stage Reductions
Fulton Plan with Plan 5X Channel Geometry

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
Fulton Plan -B	Bottom width 12 ft; SS:1V-2H	-1.00	-0.51	-0.06	-0.79	0.11	-0.28	-1.6	-0.95	-0.42

Table 9.14 **Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream of Main Street (X-1101)** **Fulton Plan with Plan 5X Channel Geometry**								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
Fulton Plan -B	1418.52	1419.69	1421.85	1424.46	1426.19	1426.92	1428.19	1428.80

Note: Existing water surface elevations are based on existing flows and existing channel geometry.

Table 9.15 **Frequency Water Surface Elevations (ft-NAVD88)** **680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532)** **Fulton Plan with Plan 5X Channel Geometry**								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
Fulton Plan -B	1418.83	1420.03	1422.03	1424.55	1426.26	1426.99	1428.26	1428.88

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.
Existing water surface elevations are based on existing flows and existing channel geometry.

B. Modification to Upstream Impoundments

Only dams in the Little Beaver Kill watershed were considered for modification because the majority of the economic damages are along the Little Beaver Kill. Dams in the Willowemoc watershed were not considered because their modification was judged to have a minor effect on damage reduction. This is due to the small drainage areas controlled by those dams relative to the large drainage area of the Willowemoc at Livingston Manor.

Six existing impoundments upstream of Livingston Manor in the Little Beaver Kill watershed were modified and assessed for flow reduction along Pearl Street. The dams were selected based on issues of ownership and the relatively large size of the drainage areas controlled by the dams. The six dams are shown in Figure 9.2. The drainage areas upstream of the dams are noted.

The modification of the hydrologic model consisted of removing the watershed upstream of the dam to show the maximum possible benefit. This is equivalent to raising the dam to contain all potential runoff events. It is highly unlikely hence the calculated flow reductions are for analytical purposes only. As shown on Table 9.16 the six dams were assigned an order then each dam modification was added to the one previously to present cumulative effects. The effect of each dam modification was assessed with a range of 24 hour rainfalls with a Type I SCS distribution. The information in Table 9.16 can be used to construct flow reduction curves at Pearl Street for each of the alternatives. However, a flow reduction curve was calculated only for Plan D6 because it reflects appreciable flow reductions relative to existing condition. The flow reduction curve was used to transform the existing discharge frequency curve to the D6 with project discharge frequency curve and the result is shown on Table 9.17. All six dams must be modified to obtain the with project discharge frequency shown on Table 9.17.

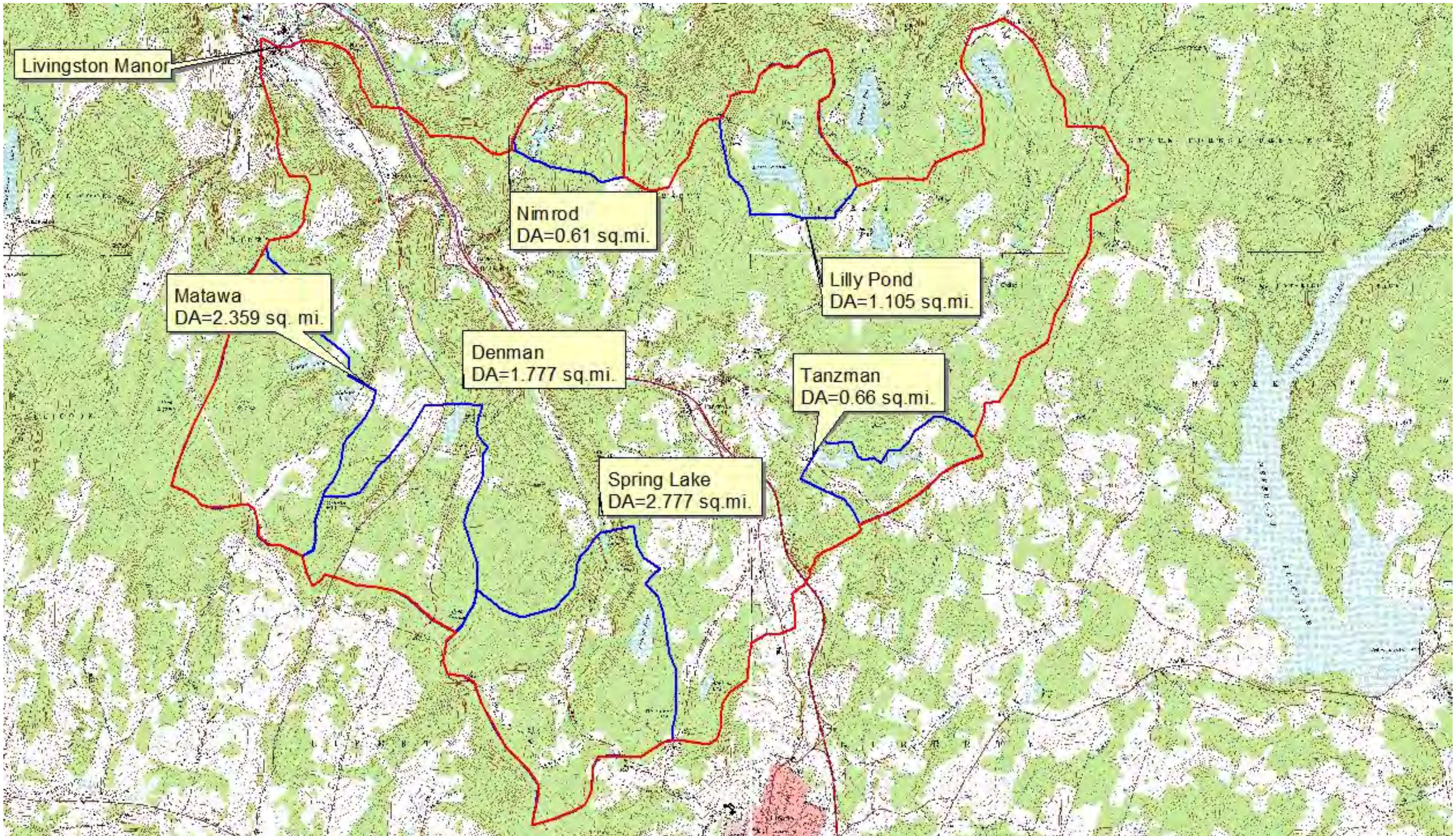


Figure 9.2 – Six Impoundments in the Little Beaver Kill Watershed

Table 9.16
Flow Reductions at Pearl Street from Removing the Watershed Upstream of Existing Dams

Condition	Description	Drainage Area Removed (sq. mi)	Discharge at Pearl Street (cfs)				
			1 inch 24hr Storm	2 inch 24hr Storm	3inch 24hr Storm	5inch 24hr Storm	8 inch 24hr Storm
Existing		NA	85	606	1459	3751	9613
D1	Remove watershed Upstream of: Matawa Dam	2.359	81	598	1448	3729	9155
D2	Remove watershed Upstream of: Matawa, Denman dams	4.136	77	591	1439	3709	8616
D3	Remove watershed Upstream of: Matawa, Denman, Tanzman dams	4.796	76	588	1431	3698	8607
D4	Remove watershed Upstream of: Matawa, Denman, Tanzman, Nimrod dams	5.406	75	585	1427	3685	8324
D5	Remove watershed Upstream of: Matawa, Denman, Tanzman, Nimrod, Lilly Pond dams	6.511	73	583	1425	3682	8318
D6	Remove watershed Upstream of: Matawa, Denman, Tanzman, Nimrod, Lilly Pond dams and, Spring Lake	9.288	67	531	1305	3353	7387

Note: Removing the watershed upstream of the dam is equivalent to modifying the dam such that it captures all runoff from the smallest to the largest storm.

Table 9.17 Discharge-Frequency for Impoundment Plan D6 at Pearl St			
Exceedance Frequency	Event (year)	Discharge (cfs) at Pearl Street	
		Existing	D6
99	1.01	510	446
50	2	1890	1690
20	5	3066	2741
10	10	3976	3508
4	25	5250	4385
2	50	6286	5097
1	100	7392	5859
0.4	250	9002	6967
0.2	500	10318	7929

Note: Drainage Area at Pearl Street is 30.2 sq. mi.

The flows in Table 9.17 apply downstream from the Airport Ponds to the mouth of Little Beaver Kill Creek. The water surface elevations corresponding to the frequency flows of Plan D6 were calculated with the existing condition hydraulic model and the results are found in Tables 9.18, 9.19 and 9.20. The frequency water surface elevations for Plan D6 were also calculated with the geometry of the best of the hydraulic plans, Plan 6CX-4. The results are found in Tables 9.21, 9.22 and 9.23.

Table 9.18 **Stage Reductions** **Upstream Impoundment D6 with Existing Channel Geometry**										
Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
D6	Remove watershed Upstream of: Matawa, Denman, Tanzman, Nimrod, Lilly Pond dams and Spring Lake	-0.69	-1.28	-0.93	-0.47	-0.97	-2.07	-0.69	-1.70	-0.94

Table 9.19 **Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream of Main Street (X-1101)** **D6 Modification with Existing Channel Geometry**								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
D6	1418.18	1420.6	1422.14	1423.71	1425.14	1426.4	1426.93	1427.87

Table 9.20 **Frequency Water Surface Elevations (ft-NAVD88)** **680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532)** **D6 Modification with Existing Channel Geometry**								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
D6	1418.47	1420.84	1422.3	1423.81	1425.21	1426.45	1427	1427.93

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.

Table 9.21
Stage Reductions
Upstream Impoundment D6 with Plan 6CX-4 Channel Geometry

Plan	Description	Plan WSEL – Existing WSEL								
		Mouth of LBK (X-134)			DS face of Main St Bridge (X-749)			250 ft US of Main St Bridge (X-1101)		
		5 yr	25yr	100yr	5yr	25yr	100yr	5yr	25yr	100yr
D6	Remove watershed Upstream of: Matawa, Denman, Tanzman, Nimrod, Lilly Pond dams and Spring Lake	-0.95	-2.08	-1.84	-0.39	-0.84	-1.71	-2.03	-3.60	-3.56

Table 9.22
Frequency Water Surface Elevations (ft-NAVD88) 250ft Upstream of Main Street (X-1101)
D6 Modification with Plan 6CX-4 Channel Geometry

Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.68	1421.29	1422.99	1425.41	1426.26	1427.34	1428.19	1428.8
D6	1417.32	1419.26	1420.48	1421.81	1422.81	1423.78	1425.38	1426.41

Note: Existing water surface elevations are based on existing flows and existing channel geometry.

Table 9.23 Frequency Water Surface Elevations (ft-NAVD88) 680ft Upstream of Main Street at Low Spot of Pearl Street (X-1532) D6 Modification with Plan 6CX-4 Channel Geometry								
Plan	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Existing	1418.97	1421.49	1423.12	1425.47	1426.33	1427.41	1428.26	1428.88
D6	1417.73	1419.73	1420.90	1422.11	1423.03	1423.96	1425.50	1426.51

Note: Elevation of Pearl Street at X-1532 is 1419 ft-NAVD88.
Existing water surface elevations are based on existing flows and existing channel geometry.

C. Realistic Modification to Matawa Dam

The impoundment modification analysis above considers the watershed above Matawa dam removed from the hydrologic model. This corresponds to the theoretical maximum flow reduction. The sponsor requested an assessment of Matawa Dam under a more realistic set of assumptions since the Town of Rockland owns the dam and real estate is not an issue. A practical modification of Matawa Dam is to drain the normal pool and convert it to a dry dam. That is, base and moderate flows are released thru a low level outlet and larger flows are impounded and released gradually after the flows on Little Beaver Kill drop back to normal.

Matawa Dam is a masonry structure constructed in 1949 for water supply. It no longer serves as a water supply and has become a run of river dam with inflow passing uncontrolled over its concrete spillway.

Pertinent data can be found in Table 9.24.

Table 9.24
Matawa Dam – Pertinent Data
(from NYS Inventory of Dams)

Item	Value
Length	120 ft
Height	22 ft
Reservoir Surface Area	26 acres
Normal Storage	240 acre-ft
Maximum Storage	275 acre-ft
Maximum Discharge	215 cfs
Spillway Width	18 ft
Hazard Potential	Low

A low level outlet was not apparent during a site visit. However, a working low level outlet is necessary for this modification to be effective. A means is required to pass base flow for environmental reasons and to quickly drain down the pool after a storm event to make storage available for the next storm event. This analysis assumes an empty reservoir for each storm analyzed.

The drainage area upstream of the dam is 2.359 sq. mi. with 1.009 sq. mi. controlled by Lenape Dam. The drainage area of the Matawa tributary at its confluence with Little Beaver Kill is 3.22 sq. mi. The drainage area of Little Beaver Kill just downstream of the junction is 28.5 sq. mi.

Figure 9.3 shows the location where the discharges are tabulated to assess the effect of the pool lowering.

This option converts Matawa Dam from a run of river dam to a dry dam. An estimate of the storage available with the normal pool drain down is required. Figure 9.4 shows assumed contours beneath the water surface. The contours reflect the assumption that there has been no shoaling since dam construction. If this assumption is not true then the predicted flow reductions will be conservative. If there has been shoaling and one wishes to obtain the predicted benefits then the reservoir will need to be dredged as part of the modification.

Table 9.25 shows the assumed storage volume available if the pool is drained down. The "Elevations" are measured from 0.0 which is the normal pool level.

Table 9.25 Matawa Dam Elevation-Capacity Curve				
Elevation*	Area (acres)	Average Area (acres)	Incremental Volume (acre-ft)	Cumulative Volume (acre-ft)
-20.5	0.185			0.000
		0.804	0.402	
-20.0	1.422			0.402
		3.163	15.815	
-15.0	4.903			16.215
		8.230	41.150	
-10.0	11.556			57.363
		14.845	74.225	
-5.0	18.133			131.586
		20.330	101.650	
0.0	22.526			238.069
		23.862	35.793	
1.5	25.198			273.862
		28.104	140.520	
6.5	31.009			414.378
		32.903	164.515	
11.5	34.797			543.100

*Relative to Normal Pool

Table 9.26 is the outlet rating curve. The outlet considers only the spillway. The low level outlet is unknown but its release is assumed to be small.

Table 9.26 Matawa Outlet Rating Curve	
Elevation*	Outflow (cfs)
-20.5	0
-20.0	0
-15.0	0
-10.0	0
-5.0	0
0.0	5.7
0.5	16.6
1	47
1.5	86
1.6	111
4.0	2877

*Relative to Normal Pool

Note: In reality there will be no zero outflows. The low level outlet will always be passing base flow which for this drainage area is 2 to 4 cfs.

Figure 9.3 - Matawa Dam - Discharge Tabulation Locations

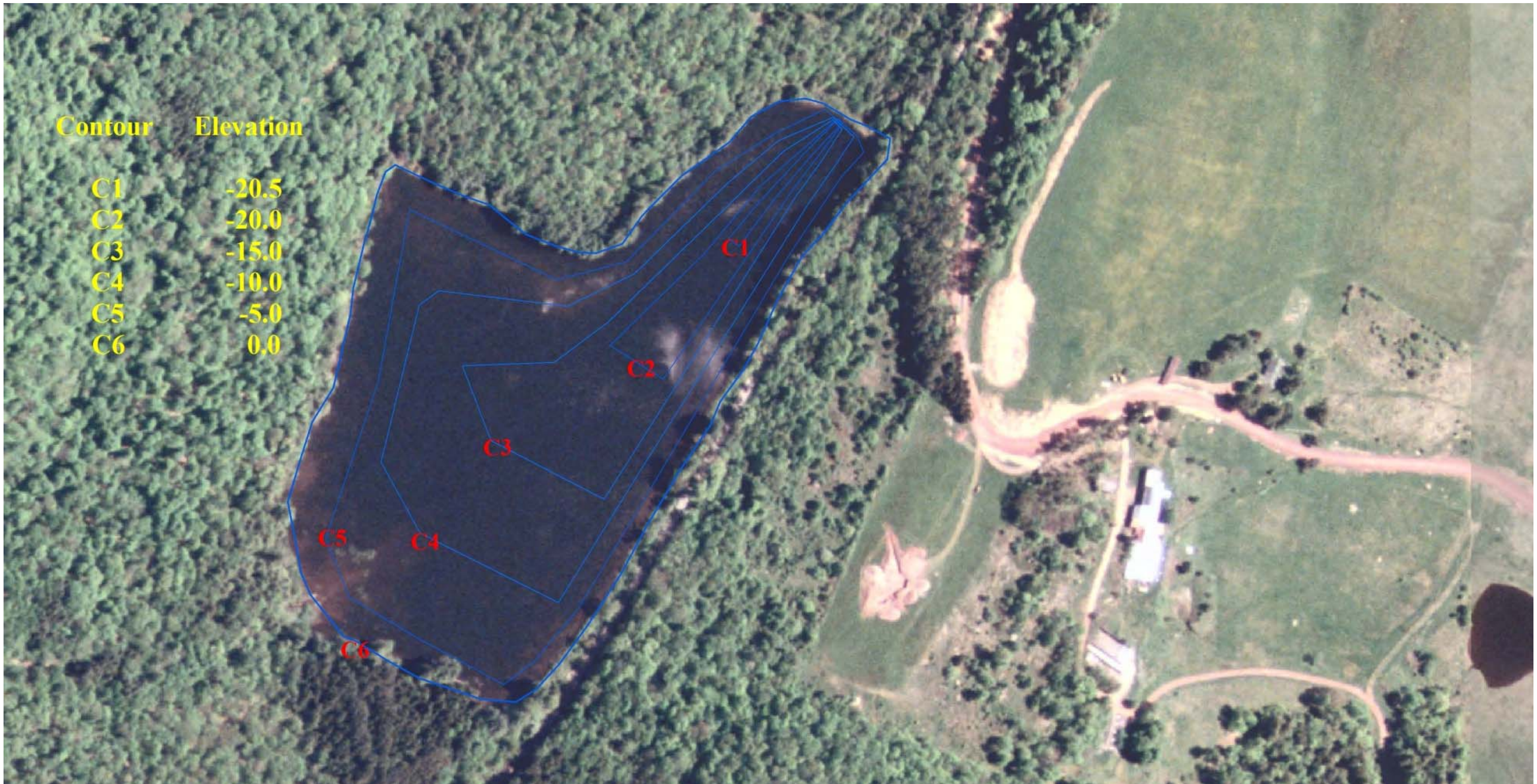


Figure 9.4 - Matawa Dam – Assumed Sub-Aqueous Contours

The existing condition HEC-HMS model for Matawa Dam was edited to reflect with project condition. The starting pool was set 20.5 ft lower than existing water surface elevation and the elevation capacity curve was adjusted to reflect assumed storage below the normal water level. Various rainfalls were applied to the model and the results were tabulated at 3 locations: just downstream of Matawa Dam, on the Little Beaver Kill just downstream of the confluence with the Little Beaver Kill and at Pearl Street. The results are in Table 9.27.

If one compares the flows at Pearl Street of Table 9.27 to the Pearl Street flows of Table 9.16 for the D1 option one finds that the flows are the same. Hence one can conclude that the practical modification of Mataw Dam of lowering the pool level has the same effect of theoretically removing the watershed.

Table 9.27
Discharge Effects of Matawa as Dry Dam

Location	Discharge (cfs)									
	1 inch 24hr Storm		2 inch 24hr Storm		3inch 24hr Storm		5inch 24hr Storm		8 inch 24hr Storm	
	Existing	W/ Project	Existing	W/ Project	Existing	W/ Project	Existing	W/ Project	Existing	W/ Project
Just DS of Dam	4	0	12	0	33	0	143	0	670	94
LBK just DS of Confluence	83	78	588	579	1402	1391	3582	3555	9306	8841
Pearl St	85	81	606	598	1459	1448	3751	3729	9613	9155

10. CATTAIL BROOK ANALYSIS POST SEPTEMBER 2012 FLOOD

Initially, only the “Existing” condition was analyzed for Cattail Brook. That analysis is documented in Section 6 of this report. The initial analysis reflected the channel conditions post the June 2006 flood. The June 2006 caused unprecedented damage along Cattail Brook and was caused by an abnormally intense rain storm coupled with massive tree debris that blocked multiple bridges. Because of the debris at Finch Street Bridge the water jumped out of bank onto County Route 149 and flowed towards the center of Livingston Manor as a 2 ft deep torrent causing much erosion damage.

Without bridge blockage Cattail Brook infrequently exceeds its channel capacity. Because of the abnormal nature of the June 2006 event and the limited average annual damage potential, a With Project analysis for Cattail Brook was not initially performed.

However, on September 18, 2012 Cattail Brook experienced another abnormally rare event - very heavy rain (6 inches of rain in a 2hour period) with tree debris that blocked bridges. The flow patterns and the erosion damage of the September 2012 event was very similar to the June 2006 event.

In response to the September 2012 event the non-Federal sponsor (NYSDEC) and Rockland Township requested an abbreviated With Project analysis.

The original HEC-RAS model (reflecting post 2006 conditions) was modified to reflect post September 2012 without project conditions. The September 2012 destroyed two bridges (Hoos and a Private Bridge) and caused channel erosion. The Private Bridge was returned to the status quo ante and Hoos Bridge (a 20ft width) was replaced with a new bridge with a 40ft width. (The bank downstream of Hoos Bridge had the riprap replaced, stepping back the stone to allow expansion of high water.) The sponsor indicated that the majority of the channel erosion was repaired such that the post June 2006 channel model is a reasonable representation of the post September 2012 condition. Therefore the post September 2012 without project model is the post June 2006 existing condition model with Hoos Bridge modeled as a 40ft width span.

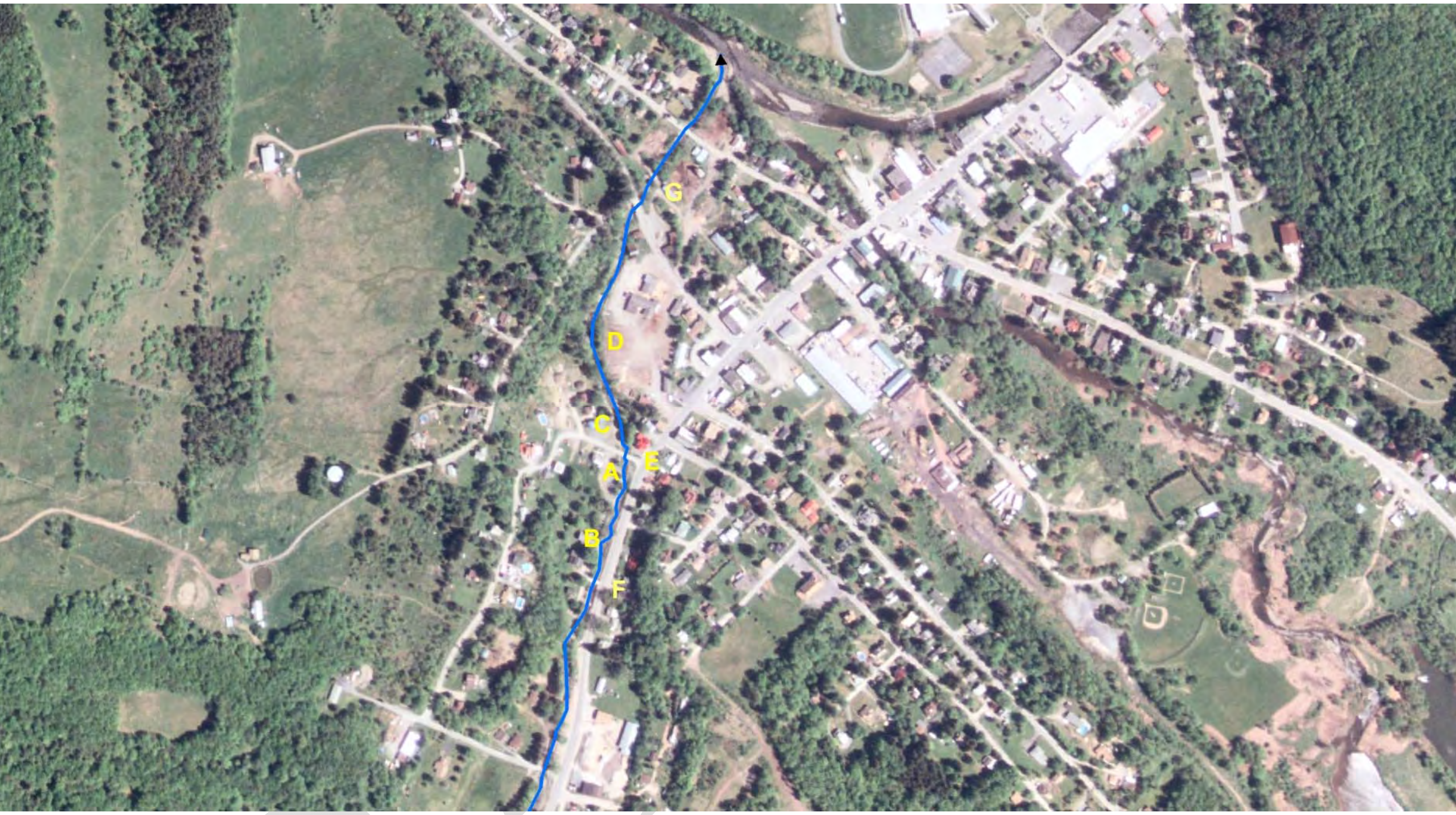
A. With Project Analysis

The following solutions were considered:

- -- divert flow onto the left overbank upstream of Finch Bridge
  - a) diversion point approx 50 ft upstream of bridge
  - b) diversion point approximately 300 ft upstream of bridge
- -- increase the capacity of Finch Bridge by excavating a bench on the left downstream bank
- -- increase the capacity of Finch Bridge by excavating a bench on the right downstream Bank
- -- replace Finch Bridge with a 40ft width span
- -- remove the Private bridge (downstream of Hoos Bridge)
- -- remove old Railroad Bridge (between River and Creamery Roads)

Figure 10.1 provides an overview of Cattail Brook. The various bridges are labeled. Figure 10.2 identifies the various With Project options and the descriptions of the With Project options are provided in Table 10.1.

ure 10.1 – Overview of Cattail Brook



ure 10.2 – With Project Options for Cattail Brook

Table 10.1
Description of Cattail Brook With Project Options

Option	Description	Status
A	Excavate an Overflow Path approximately 50ft upstream of Finch Bridge to Willoughby Street	Not Analyzed with a hydraulic model. The left bank could be graded to deliver more flow to the Willoughby Street, but it was not considered further because it would increase flooding.
B	Excavate an Overflow Path approximately 300ft upstream of Finch Bridge to Willoughby Street	Not Analyzed with a hydraulic model. The left bank could be graded to divert flow from the Cattail to the landward side of a knoll which would flow to Willoughby Street. It was not considered further because it would increase flooding.
C	Excavate Bench on LOB just downstream Finch Bridge	Analyzed with a hydraulic model
D	Excavate Bench on ROB for 400 ft downstream of Finch Bridge	Analyzed with a hydraulic model
E	Replace Finch Bridge with a new 40ft wide Bridge	Analyzed with a hydraulic model
F	Demolish Private Road Bridge	Analyzed with a hydraulic model
G	Demolish Old RR bridge	Analyzed with a hydraulic model

Options A and B have the potential to reduce the flow diversion onto Route 49, but at the cost of increased flow and possible increased damage to the houses along Willoughby Street. It was because of this trade-off that these options were not pursued further. Figures 10.3 and 10.4 provide a conceptual plan view of options A and B respectively.

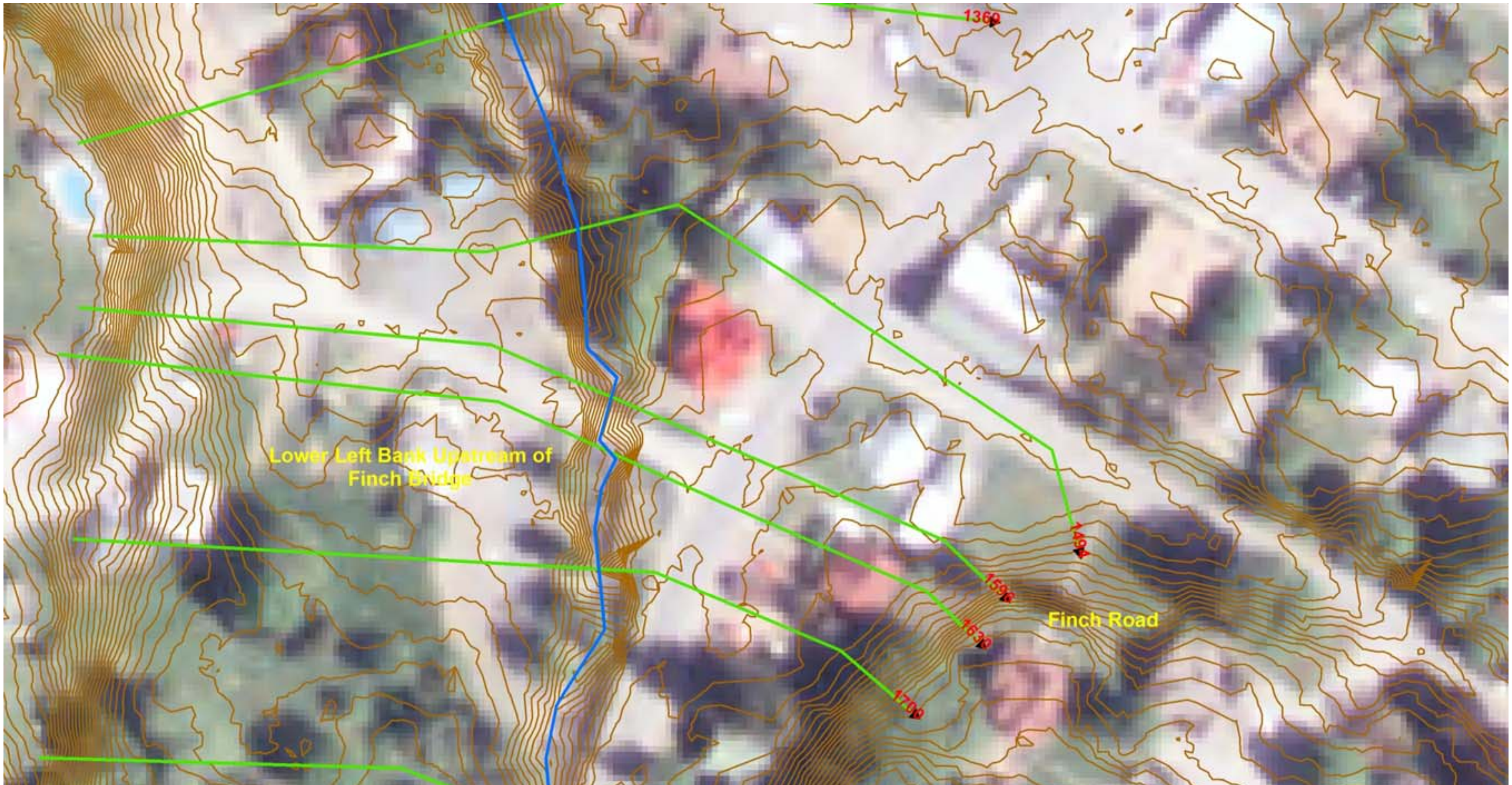


Figure 10.3 – Conceptual Plan View of Option A

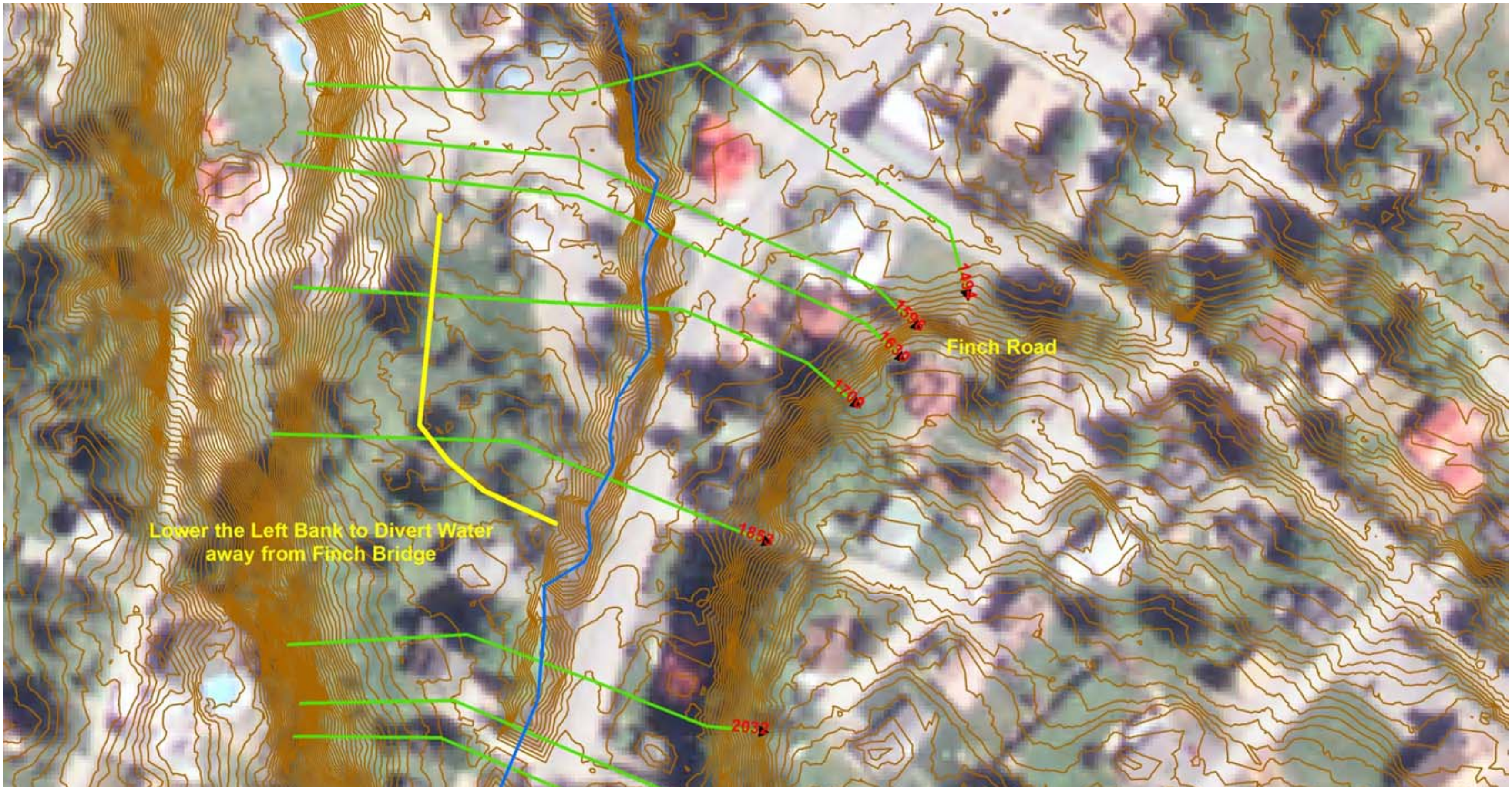


Figure 10.4 – Conceptual Plan View of Option B

Finch Bridge is the critical location for analysis. It determines the flow split between Cattail Brook and Route 149. The flow split is a function of debris blockage on the upstream face of the bridge. The greater the blockage, the greater the flow to Rt. 149 and the lesser of the flow to Cattail Brook. The performance of the bridge was analyzed under two conditions: unblocked and blocked. Blocked condition corresponds to 85% of the channel obstructed and 100% of the railing area obstructed. This level of blockage was assumed to apply to all frequency events that are subject to blockage. The 25, 50, 100, 250 and 500 year events were assumed subject to blockage. That is, the rainfall intensity is assumed great enough to dislodge trees along the bank. The calculation of the flow splits involves a trial and error procedure to balance the flow splits of two separate hydraulic models at a common water surface elevation. Two models are involved: main stem Cattail Brook model and a normal depth model for Rt. 149. The common cross-section for both models is the cross-section labeled 1630 in the Cattail Model.

Figure 10.5 is a plot of cross-section 1630 (X-1630) from the Cattail Model. The vertical green lines confine the active flowing water. Any flow which does not flow on Rt. 149 flows between the green lines. Figure 10.6 is the right hand side of X-1630 and it serves as the normal depth cross-section for the Rt. 149 model. When a flow split occurs, the water surface elevation of the two cross-sections will be the same and the sum of the separate flows will equal to the total flow into Finch Bridge.

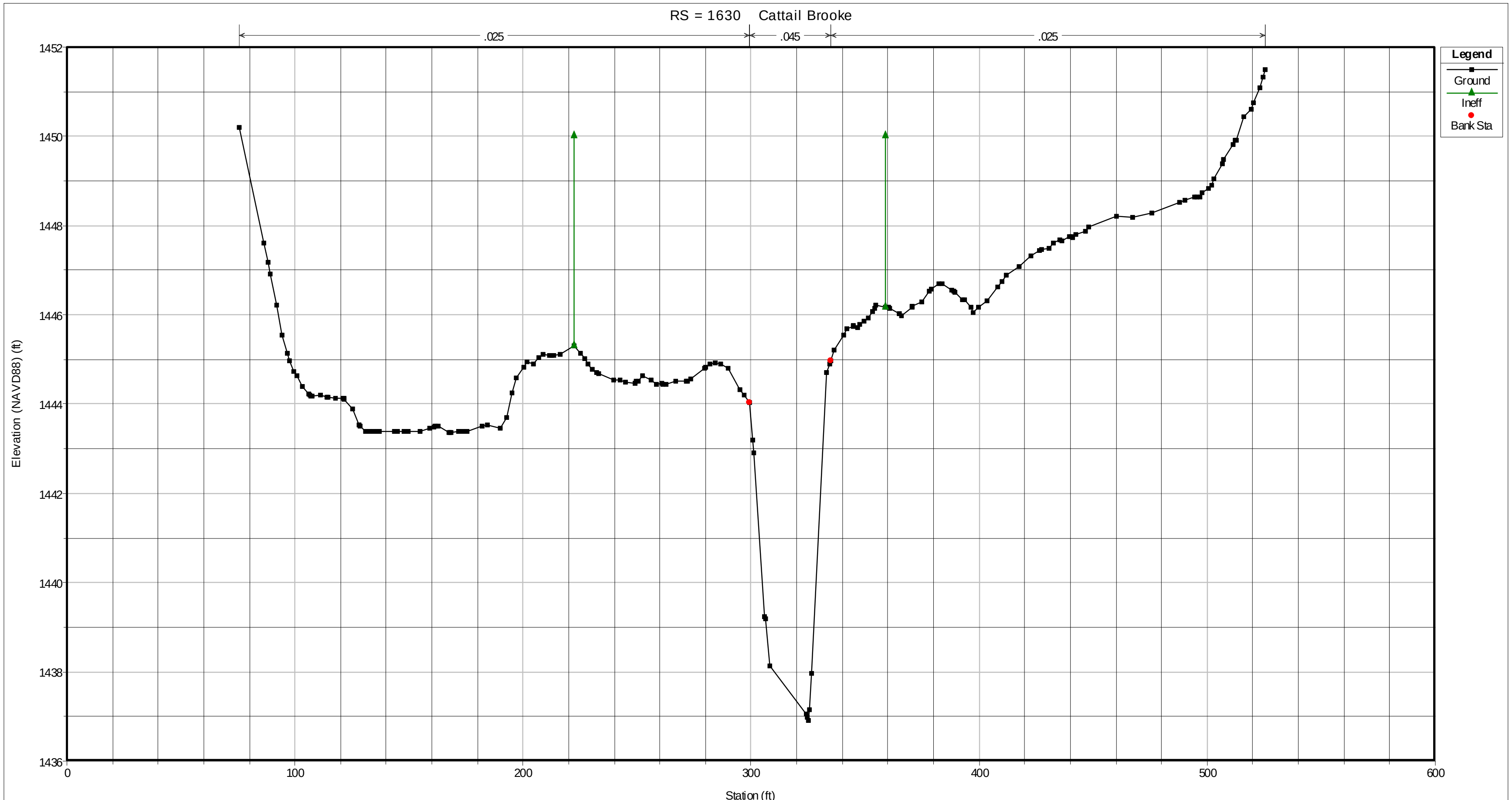


Figure 10.5 - Cross-section 1630 for the Cattail Brook Hydraulic Model

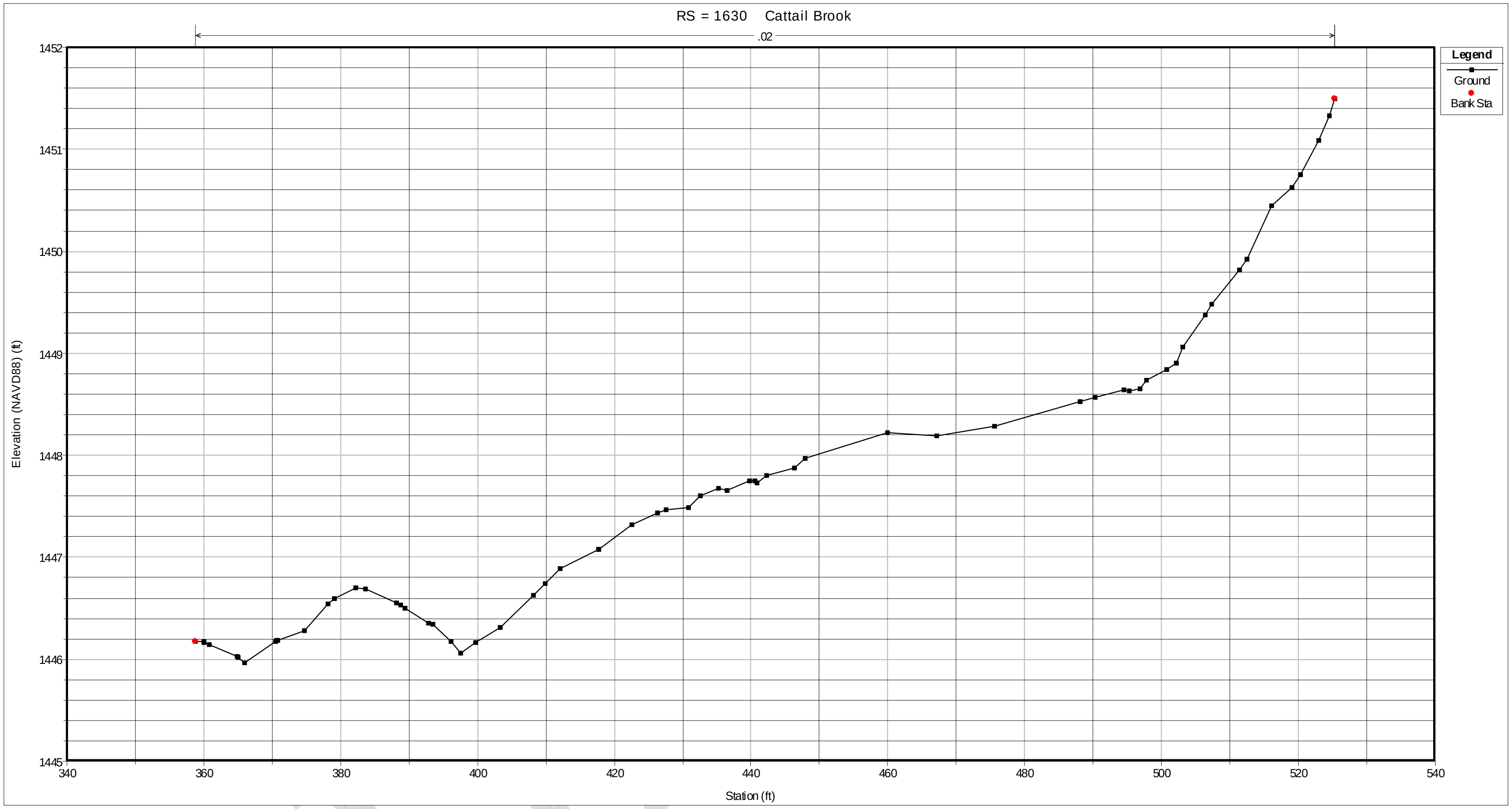


Figure 10.6 - Cross-section “1630” for the Route 149 Hydraulic Model

The flow splits for Finch Bridge under the no debris assumption were determined in a two step process. First, rating curves (a relationship between flow and water surface elevation) were developed for the two models. The rating curves are found in Table 10.2.

Table 10.2
Rating Curves for Common Cross-section 1630 at Finch Bridge
No Debris Blockage at Finch Bridge

Cattail Brook		Route 149 (Main Street)	
Cattail Discharge (cfs)	WSEL (ft-NAVD88)	Rt. 149 Discharge (cfs)	Critical Depth WSEL (ft-NAVD88)
94	1438.92	5.00	1446.25
558	1441.28	10.00	1446.32
911	1442.44	20.00	1446.42
1189	1443.27	40.00	1446.57
1575	1445.79	60.00	1446.69
1889	1447.09	80.00	1446.77
2223	1447.33	100.00	1446.83
2712	1447.95	150.00	1446.99
3109	1448.33	200.00	1447.13
		300.00	1447.36
		400.00	1447.57
		500.00	1447.76
		600.00	1447.91
		700.00	1448.06
		800.00	1448.25
		900.00	1448.36
		1000.00	1448.46
		1100.00	1448.57
		1200.00	1448.65
		1300.00	1448.73
		1400.00	1448.81
		1500.00	1448.88

Note: Finch Bridge's dimensions post 2012 are the same as post 2006.

Then for each frequency event a trial and error process of guessing flow splits and determining the water surface elevation at the common cross-section of each model was performed until a flow split combination was found that produced the same water surface elevation at the common cross-section. The results of that process are found on Table 10.3.

Table 10.3
Flow Splits at Common Cross-section 1630
No Debris Blockage at Finch Bridge

Event	Total Flow on Cattail Upstream Of Finch Street (cfs)	Assumed Cattail Flow into Finch Bridge (cfs)	Cattail WSEL at X-1630 (ft-NAVD88)	Assumed Diversion to Route 149	Critical Depth WSEL on Rt. 149 (ft-NAVD88)
1 year	94	94	1438.92	0	NA
2 year	558	558	1441.28	0	NA
5 year	911	911	1442.44	0	NA
10 year	1189	1189	1443.27	0	NA
25 year	1575	1575	1445.79	0	NA
50 year	1889	1809	1446.76	80	1446.77
100 year	2223	2003	1447.17	220	1447.18
250 year	2712	2352	1447.49	360	1447.49
500 year	3109	2589	1447.79	520	1447.79

Note: Finch Bridge's dimensions post 2012 are the same as post 2006.

The same process was repeated for the with debris assumption. The Cattail Brook model was modified to reflect 85% blockage of the channel and 100% blockage of the railing. The same level of blockage was assumed for all magnitude of events which results in blockage. In addition, only the 25 year event and larger were assumed to cause blockage. The rating curves at the common cross-section of 1630 are found in Table 10.4 and the results of the trial and error flow splits are found in Table 10.5.

Table 10.4
Rating Curves for Common Cross-section 1630 at Finch bridge
With Debris Blockage at Finch Bridge

Cattail Brook		Route 149 (Main Street)	
Cattail Discharge (cfs)	WSEL (ft-NAVD88)	Rt. 149 Discharge (cfs)	Critical Depth WSEL (ft-NAVD88)
94	1440.48	5.00	1446.25
558	1446.65	10.00	1446.32
911	1447.29	20.00	1446.42
1189	1447.71	40.00	1446.57
1575	1448.24	60.00	1446.69
1889	1448.64	80.00	1446.77
2223	1449.04	100.00	1446.83
2712	1449.59	150.00	1446.99
3109	1450	200.00	1447.13
		300.00	1447.36
		400.00	1447.57
		500.00	1447.76
		600.00	1447.91
		700.00	1448.06
		800.00	1448.25
		900.00	1448.36
		1000.00	1448.46
		1100.00	1448.57
		1200.00	1448.65
		1300.00	1448.73
		1400.00	1448.81
		1500.00	1448.88

Note: Finch Bridge’s dimensions post 2012 are the same as post 2006.

Table 10.5 **Flow Splits at Common Cross-section 1630** **With Debris Blockage at Finch Bridge**					
Event	Total Flow on Cattail Upstream Of Finch Street (cfs)	Assumed Cattail Flow into Finch Bridge (cfs)	Cattail WSEL at X-1630 (ft-NAVD88)	Assumed Diversion to Route 149	Critical Depth WSEL on Rt. 149 (ft-NAVD88)
1 year	94	94	1438.92	0	NA
2 year	558	558	1441.28	0	NA
5 year	911	911	1442.44	0	NA
10 year	1189	1189	1443.27	0	NA
25 year	1575	1145	1447.64	430	1447.63
50 year	1889	1319	1447.89	570	1447.87
100 year	2223	1493	1448.13	730	1448.12
250 year	2712	1732	1448.44	980	1448.44
500 year	3109	1909	1448.66	1200	1448.65

Note: Finch Bridge's dimensions post 2012 are the same as post 2006.
All events less than the 25 year event are assumed not to block.

The frequency flow splits and water surface elevation with and without debris blockage at Finch Bridge are summarized in Table 10.6.

Table 10.6 **Flow Splits for Finch Bridge With and With Out Debris Blockage at Finch Bridge**							
Event	Total Discharge (cfs)	No Debris			With Debris		
		Flow to Cattail (cfs)	Flow to Rt. 149 (cfs)	WSEL at X-1630 (ft-NAVD88)	Flow to Cattail (cfs)	Flow to Rt. 149 (cfs)	WSEL at X-1630 (ft-NAVD88)
25 yr	1575	1575	0	1445.79	1145	430	1447.64
50 yr	1889	1809	80	1446.76	1319	570	1447.89
100 yr	2223	2003	220	1447.17	1493	730	1448.13
250 yr	2712	2352	360	1447.49	1732	980	1448.44
500 yr	3109	2589	520	1447.79	1909	1200	1448.66

Note: Finch Bridge's dimensions post 2012 are the same as post 2006.
All events less than the 25 year event are assumed not to block.

For the Post Sep 2012 RAS model, the frequency flows and hence the frequency water surface elevations on Cattail Brook downstream of Finch Bridge are a function of the debris blockage at Finch Bridge. Table 10.7 shows a comparison of frequency water surface elevations for minimum flow on Cattail Brook (that is, debris blockage and maximum flow diverted to Rt. 149) versus total flow on Cattail Brook (that is, various solutions are assumed to eliminate all flow diversion onto Rt. 149). Table 10.7 can be considered a minimum and maximum numerical profile plots. The locations of the cross-sections are shown in Figure 10.7.

Table 10.7
Cattail Brook Frequency Water Surface Elevations Downstream of Finch Bridge for Various Debris Assumptions

Event	Water Surface Elevations Downstream of Finch Bridge (Minimum Cattail Flow / Total Cattail Flow)																		
	Cross-sections																		
	100	187	243	288	335	386	455	489	549	613	669	761	925	1092	1259	1369	1494	1596	1630
500 yr	1415.47	1419.57	1421.59	1421.90	1423.43	1423.60	1424.17	1425.55	1428.71	1428.82	1429.35	1429.62	1431.11	1434.06	1437.19	1438.74	1441.30	1443.88	1448.67
	1416.32	1420.92	1423.42	1423.47	1424.87	1424.90	1425.55	1427.67	1431.18	1431.21	1432.19	1432.40	1432.42	1435.33	1438.16	1440.46	1443.22	1445.51	1448.33
250 yr	1415.30	1419.35	1421.30	1421.60	1423.25	1423.43	1423.95	1425.23	1428.17	1428.28	1428.81	1429.09	1430.92	1433.84	1437.04	1438.59	1441.02	1442.92	1448.45
	1416.06	1420.48	1422.84	1422.90	1424.52	1424.55	1425.26	1427.01	1430.93	1430.96	1431.66	1431.87	1431.91	1434.99	1437.82	1440.17	1442.59	1445.21	1447.95
100 yr	1415.06	1419.04	1420.91	1421.34	1423.08	1423.25	1423.51	1424.83	1427.41	1427.54	1428.04	1428.36	1430.64	1433.53	1436.73	1438.41	1440.59	1442.54	1448.14
	1415.73	1419.95	1422.07	1422.28	1423.66	1423.83	1424.55	1426.03	1430.22	1430.26	1430.75	1430.95	1431.46	1434.43	1437.50	1439.12	1441.75	1444.36	1447.33
50 yr	1414.90	1418.78	1420.64	1421.12	1422.89	1423.07	1423.30	1424.47	1426.76	1426.92	1427.43	1427.79	1430.39	1433.32	1436.46	1438.28	1440.22	1442.22	1447.90
	1415.44	1419.54	1421.56	1421.88	1423.42	1423.59	1424.14	1425.52	1428.65	1428.76	1429.29	1429.56	1431.09	1434.04	1437.19	1438.72	1441.28	1443.82	1447.09
25 yr	1414.71	1418.54	1420.34	1420.93	1422.70	1422.88	1423.13	1424.11	1426.27	1426.43	1426.90	1427.29	1430.15	1433.07	1436.05	1438.15	1439.75	1441.84	1447.65
	1415.15	1419.13	1421.05	1421.41	1423.15	1423.32	1423.67	1424.98	1427.66	1427.79	1428.30	1428.60	1430.74	1433.64	1436.86	1438.46	1440.74	1442.68	1445.79
10 yr	1414.77	1418.60	1420.42	1420.98	1422.76	1422.94	1423.18	1424.20	1426.40	1426.55	1427.03	1427.42	1430.22	1433.13	1436.11	1438.22	1439.88	1441.94	1443.27
	1414.77	1418.60	1420.42	1420.98	1422.76	1422.94	1423.18	1424.20	1426.40	1426.55	1427.03	1427.42	1430.22	1433.13	1436.11	1438.22	1439.88	1441.94	1443.27
5 yr	1414.44	1418.17	1419.96	1420.67	1422.47	1422.64	1422.86	1423.58	1425.54	1425.71	1426.14	1426.52	1429.79	1432.67	1435.65	1437.74	1439.19	1441.36	1442.44
	1414.44	1418.17	1419.96	1420.67	1422.47	1422.64	1422.86	1423.58	1425.54	1425.71	1426.14	1426.52	1429.79	1432.67	1435.65	1437.74	1439.19	1441.36	1442.44
2 yr	1413.70	1417.51	1419.44	1420.20	1421.84	1422.04	1422.21	1422.64	1424.26	1424.43	1424.84	1425.37	1429.09	1432.00	1434.91	1436.90	1438.21	1440.48	1441.28
	1413.70	1417.51	1319.44	1420.20	1421.84	1422.04	1422.21	1422.64	1424.26	1424.43	1424.84	1425.37	1429.09	1432.00	1434.91	1436.90	1438.21	1440.48	1441.28
1 yr	1412.42	1416.09	1418.18	1418.98	1419.60	1419.96	1420.13	1420.68	1421.58	1422.08	1422.68	1423.47	1427.48	1430.03	1433.19	1435.06	1436.40	1438.33	1438.92
	1412.42	1416.09	1418.18	1418.98	1419.60	1419.96	1420.13	1420.68	1421.58	1422.08	1422.68	1423.47	1427.48	1430.03	1433.19	1435.06	1436.40	1438.33	1438.92

Figure 10.7 – Plan View of Cross-sections Downstream of Finch Street Bridge

In order to reduce the flow diversion to Route 149 at Finch Bridge three options were considered: excavate a bench on left bank downstream of the bridge, excavate a bench on the right bank downstream of the bridge and replace the bridge with a 40 ft wide span. (The low steel elevation of the proposed 40ft span is assumed the same as the existing span.)

A conceptual plan for the left bench is shown on Figure 10.8. A With and Without project water surface elevation comparison is shown on Table 10.8. The water surface elevations in Table 10.8 reflect the diversion of flows to Route 149 for the no debris blockage condition.

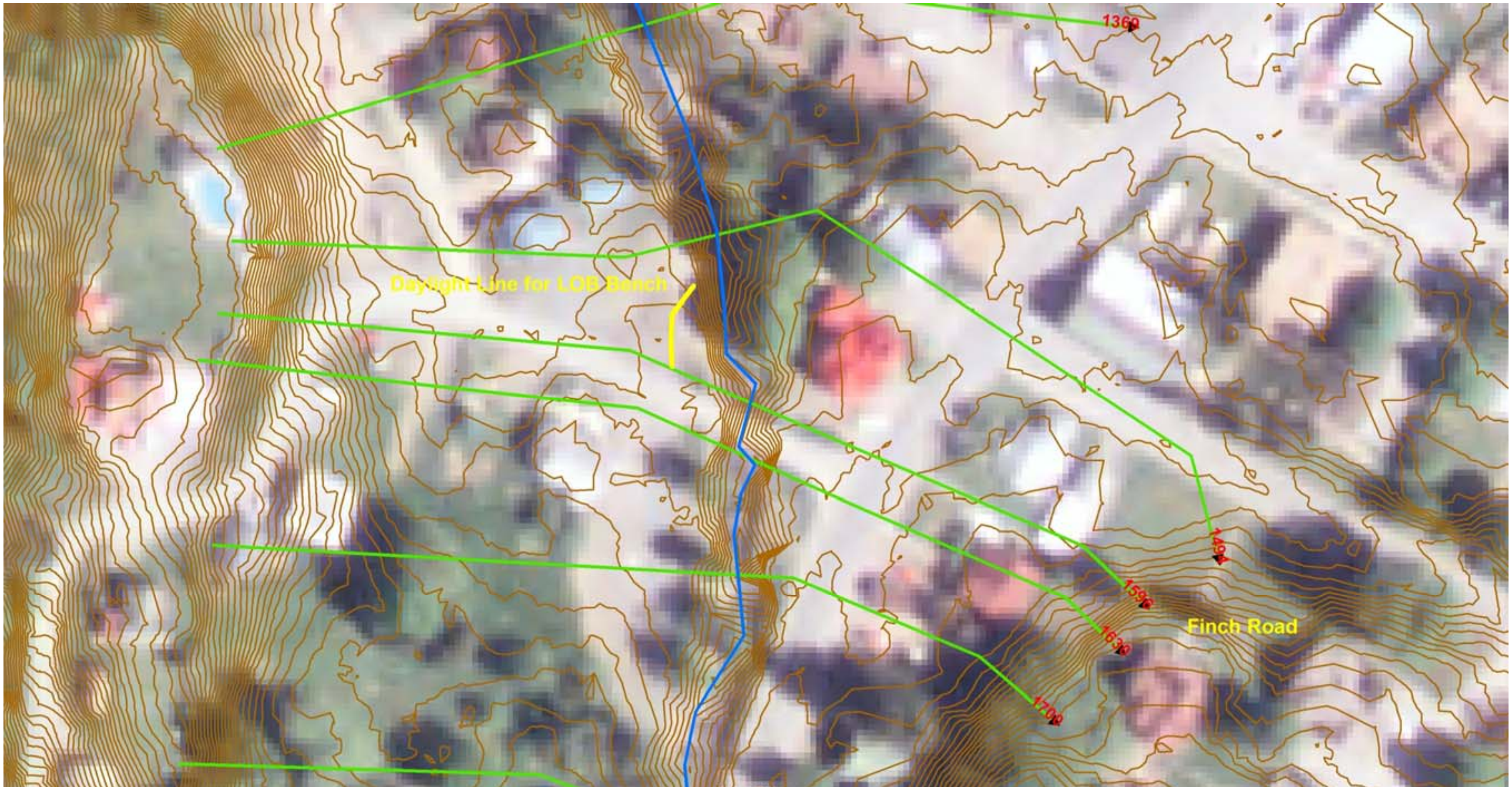


Figure 10.8 – Conceptual Plan for Left Bench Downstream of Finch Bridge

Table 10.8
With and Without Project Cattail Brook Water Surface Elevation Comparison for Left Bench Downstream of Finch Bridge
No Debris Blockage at Finch Bridge

X-Section	Event																	
	1 year		2 year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench
1494	1436.4	1436.4	1438.21	1438.21	1439.19	1439.19	1439.88	1439.88	1440.74	1440.74	1441.15	1441.15	1441.44	1441.44	1441.91	1441.91	1442.36	1442.39
1596	1438.33	1438.33	1440.48	1440.48	1441.36	1441.49	1441.94	1442.17	1442.68	1442.87	1443.05	1443.19	1444.07	1443.46	1444.75	1443.87	1445.13	1443.88
1630	1438.92	1438.92	1441.28	1441.28	1442.44	1442.45	1443.27	1443.37	1445.79	1444.5	1447.04	1446.17	1447.02	1447.02	1447.56	1447.56	1448.01	1448.03
1700	1440.59	1440.59	1442.97	1442.97	1443.93	1443.93	1444.51	1444.51	1445.73	1445.24	1446.85	1446.08	1446.67	1446.67	1447.21	1447.21	1447.58	1447.58

Note: Finch Bridge's dimensions post 2012 are the same as post 2006.
The Cattail Brook Flows reflect diversion to Rt. 149.
X-1494 is approximately 100 ft downstream of Finch Bridge
X-1596 and X-1630 are the downstream and upstream faces of the Finch Bridge.
X-1700 is approximately 70ft upstream of Finch Bridge.

The effect of a LOB bench downstream of Finch Bridge attenuates to zero at cross-section X-1853.

A conceptual plan for the right bench is shown on Figure 10.9.   A With and Without project water surface elevation comparison is shown on Table 10.9. The water surface elevations in Table 10.9 reflect the diversion of flows to Route 149 for the no debris blockage condition.

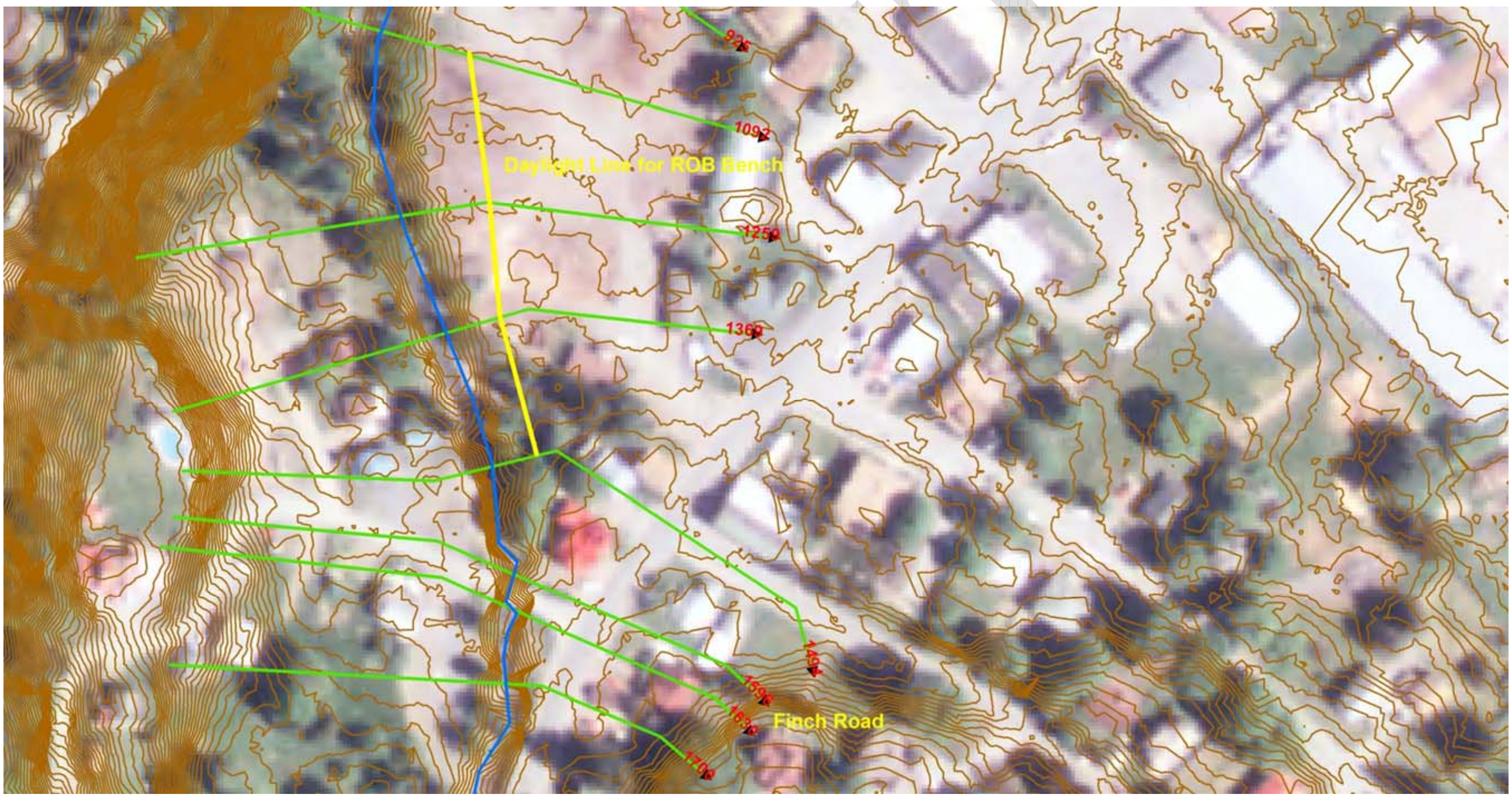


Figure 10.9 – Conceptual Plan for Right Bench Downstream of Finch Bridge

	Table 10.9 With and Without Project Cattail Brook Water Surface Elevation Comparison for Right Bench Downstream of Finch Bridge No Debris Blockage at Finch Bridge																	
X-Section	Event																	
	1 year		2 year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench	Exist	DS Bench
1494	1436.4	1436.4	1438.21	1438.34	1439.19	1439.25	1439.88	1439.69	1440.74	1440.23	1441.15	1440.54	1441.44	1440.76	1441.91	1441.15	1442.36	1441.71
1596	1438.33	1438.33	1440.48	1440.33	1441.36	1441.11	1441.94	1441.79	1442.68	1442.62	1443.05	1443.06	1444.07	1444.07	1444.75	1444.82	1445.13	1444.98
1630	1438.92	1438.92	1441.28	1441.28	1442.44	1442.51	1443.27	1443.30	1445.79	1445.84	1447.04	1447.04	1447.02	1447.02	1447.56	1447.56	1448.01	1448.01
1700	1440.59	1440.59	1442.97	1442.97	1443.93	1443.93	1444.51	1444.51	1445.73	1445.76	1446.85	1446.85	1446.67	1446.67	1447.21	1447.21	1447.58	1447.58

Note: Finch Bridge's dimensions post 2012 are the same as post 2006.
The Cattail Brook Flows reflect diversion to Rt. 149.
X-1494 is approximately 100 ft downstream of Finch Bridge.
X-1596 and X-1630 are the downstream and upstream faces of the Finch Bridge.
X-1700 is approximately 70ft upstream of Finch Bridge.

The effect of a ROB bench downstream of Finch Bridge attenuates to zero at cross-section X-1700.

A conceptual plan for a new 40ft wide Finch Bridge is shown on Figure 10.10. The low steel elevation of the proposed 40ft span is assumed the same as the existing bridge. A With and Without project water surface elevation comparison is shown on Table 10.10. The water surface elevations in Table 10.10 reflect, for existing condition, the diversion of flows to Route 149 for the no debris blockage condition. However, for the With Project condition of a 40 ft wide span there is flow diversion to Route 149 for only the 250 and 500 year events. (A 40ft wide span is assumed to experience no debris blockage. In reality such a wide bridge will likely pass a tree debris load without clogging similar to the 2006 and 2012 performance of the Shandelee Road Bridge upstream of Hoos Bridge.) For events 50 year and greater, the With Project flows into Finch Street are greater than without project flows.

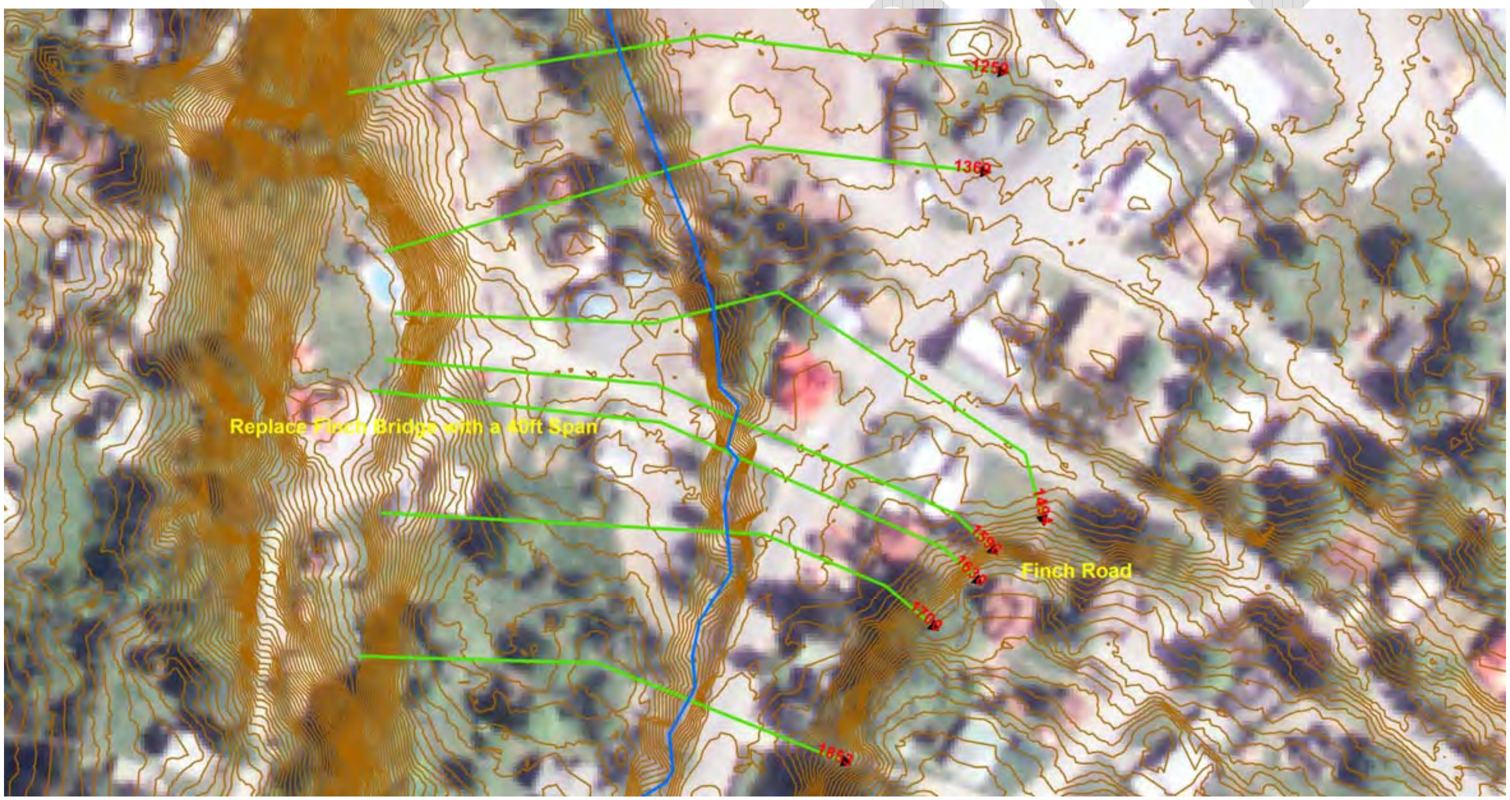


Figure 10.10 – Conceptual Plan for New 40ft Wide Finch Bridge

Table 10.10
With and Without Project Cattail Brook Water Surface Elevation Comparison for New 40 ft Wide Finch Bridge
No Debris Blockage at Finch Bridge

X-Section	Event																	
	1 year		2 year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
	Exist	40 ft	Exist	40 ft	Exist	40 ft	Exist	40 ft	Exist	40 ft	Exist	40 ft	Exist	40 ft	Exist	40 ft	Exist	40 ft
1494	1436.4	1436.4	1438.21	1438.21	1439.19	1439.19	1439.88	1439.88	1440.74	1440.74	1441.15	1441.28	1441.44	1441.75	1441.91	1442.59	1442.36	1443.22
1596	1438.33	1437.99	1440.48	1440.16	1441.36	1441.34	1441.94	1442.1	1442.68	1442.9	1443.05	1443.41	1444.07	1443.89	1444.75	1444.30	1445.13	1444.38
1630	1438.92	1438.26	1441.28	1440.44	1442.44	1441.64	1443.27	1442.43	1445.79	1443.31	1447.04	1443.92	1447.02	1444.57	1447.56	1445.50	1448.01	1446.18
1700	1440.59	1440.59	1442.97	1442.97	1443.93	1443.93	1444.51	1444.51	1445.73	1445.24	1446.85	1446.08	1446.67	1446.56	1447.21	1447.21	1447.58	1447.58

Note: Finch Bridge's dimensions are proposed to be 40ft wide. At a 40ft width, the new Finch Bridge is assumed not to block from debris.
Cattail Brook flows are total flows, including the 250 and 500 year events even though there is diversion to Rt. 149 for these two events.
X-1494 is approximately 100 ft downstream of Finch Bridge.
X-1596 and X-1630 are the downstream and upstream faces of the Finch Bridge.
X-1700 is approximately 70ft upstream of Finch Bridge.

The effect of a 40 ft width span for Finch Bridge attenuates to zero at cross-section X-1853.

The diversion of Cattail Brook flows to Route 149 occurs at cross-section X-1700 for the 250 and 500 year events. However, farther upstream water spills on to Rt. 149 and may bypass Finch Street and flow towards the center of town. To mitigate this possibility one may be able to re-grade the road to create a "dip and a bump" which has the effect of channeling road flow back to Cattail Brook. Two locations that may be appropriate for such re-grading are the intersection of Finch Street and Rt. 149 and the intersection of Rt. 149 and Brown Street. Such re-grading is necessary because of the low right overbank elevations in the vicinity of the Private Road Bridge.

A conceptual plan for removing Private Road Bridge is shown on Figure 10.11. A With and Without project water surface elevation comparison is shown on Table 10.11.

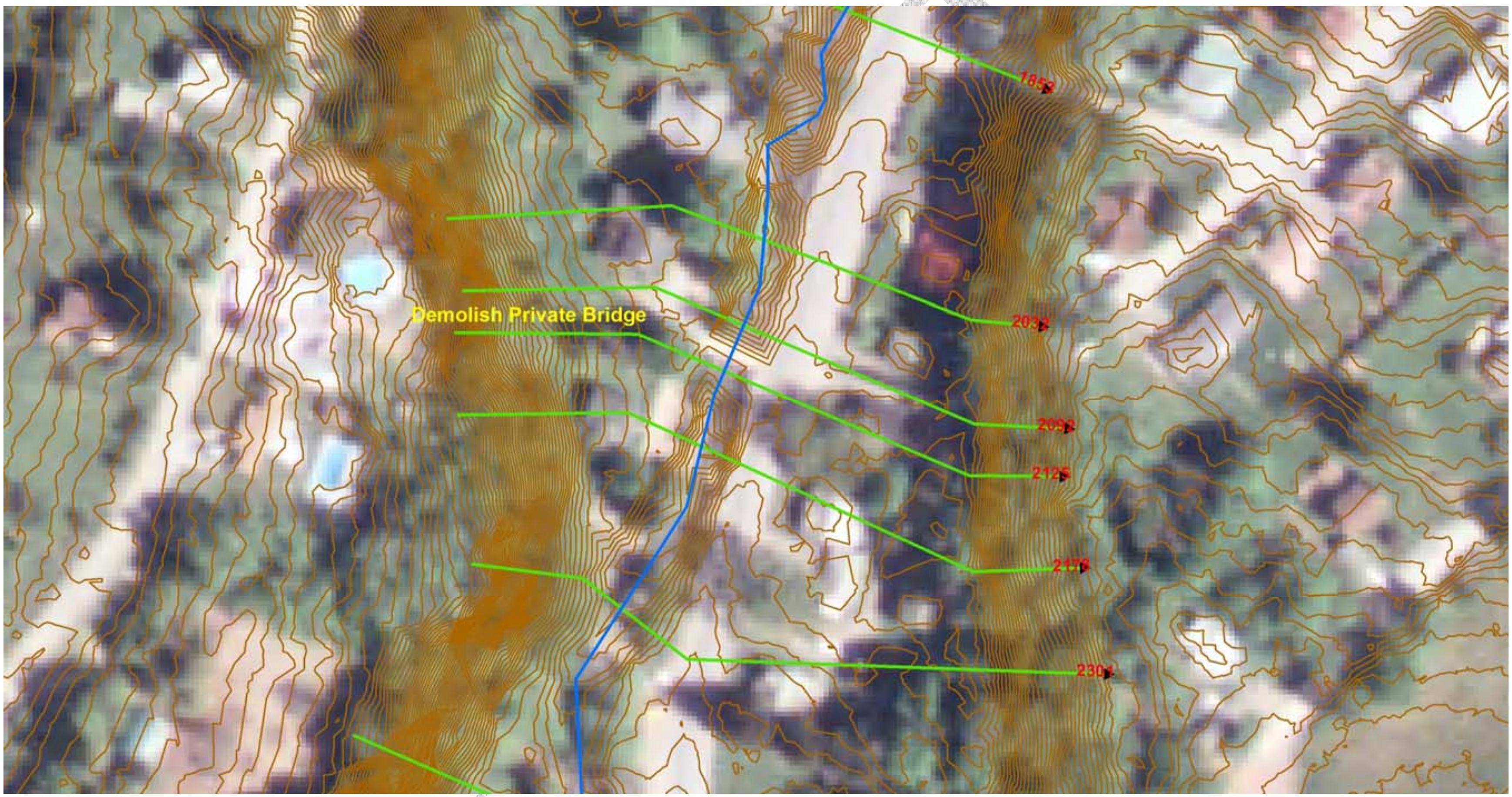


Figure 10.11 – Conceptual Plan for Removing Private Road Bridge

Table 10.11
With and Without Project Cattail Brook Water Surface Elevation Comparison for Removing Private Road Bridge
No Debris Blockage at Finch Bridge

X-Section	Event																	
	1 year		2 year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
	Exist	Pvt Br	Exist	Pvt Br	Exist	Pvt Br	Exist	Pvt Br	Exist	Pvt Br	Exist	Pvt Br	Exist	Pvt Br	Exist	rvt Br	Exist	rvt Br
2092	1449.69	1449.69	1451.36	1451.36	1452.21	1452.21	1452.76	1452.76	1453.46	1453.46	1454.61	1454.61	1455.03	1455.03	1455.43	1455.43	1455.70	1455.70
2125	1450.32	1450.32	1452.50	1452.37	1453.90	1453.30	1455.63	1453.89	1455.91	1454.73	1456.02	1454.89	1456.24	1455.23	1456.59	1455.65	1456.81	1456.17
2178	1450.52	1450.50	1452.98	1452.87	1454.38	1453.96	1455.79	1454.70	1456.14	1455.53	1456.35	1455.97	1456.61	1456.35	1456.94	1456.80	1457.16	1457.06

Note Cattail Brook flows are total flows.
X-2092 and X-2125 are the downstream and upstream faces of the Private Bridge.
X-2178 is approximately 50ft upstream of Private Road Bridge.

The effect of removing the Private Bridge attenuates to zero at cross-section X-2301.

Figure 10.12 is a cross-section plot of the upstream face of Private Bridge after its removal. Note that the 50, 100, 250 and 500 year events are on the right overbank and may bypass Finch's Bridge irrespective of the modification made to Finch's bridge. Without the removal of Private Road Bridge flow in the right overbank starts at the 10 year event. In addition removal of Private Road Bridge may deliver a heavier debris load to Finch Bridge.

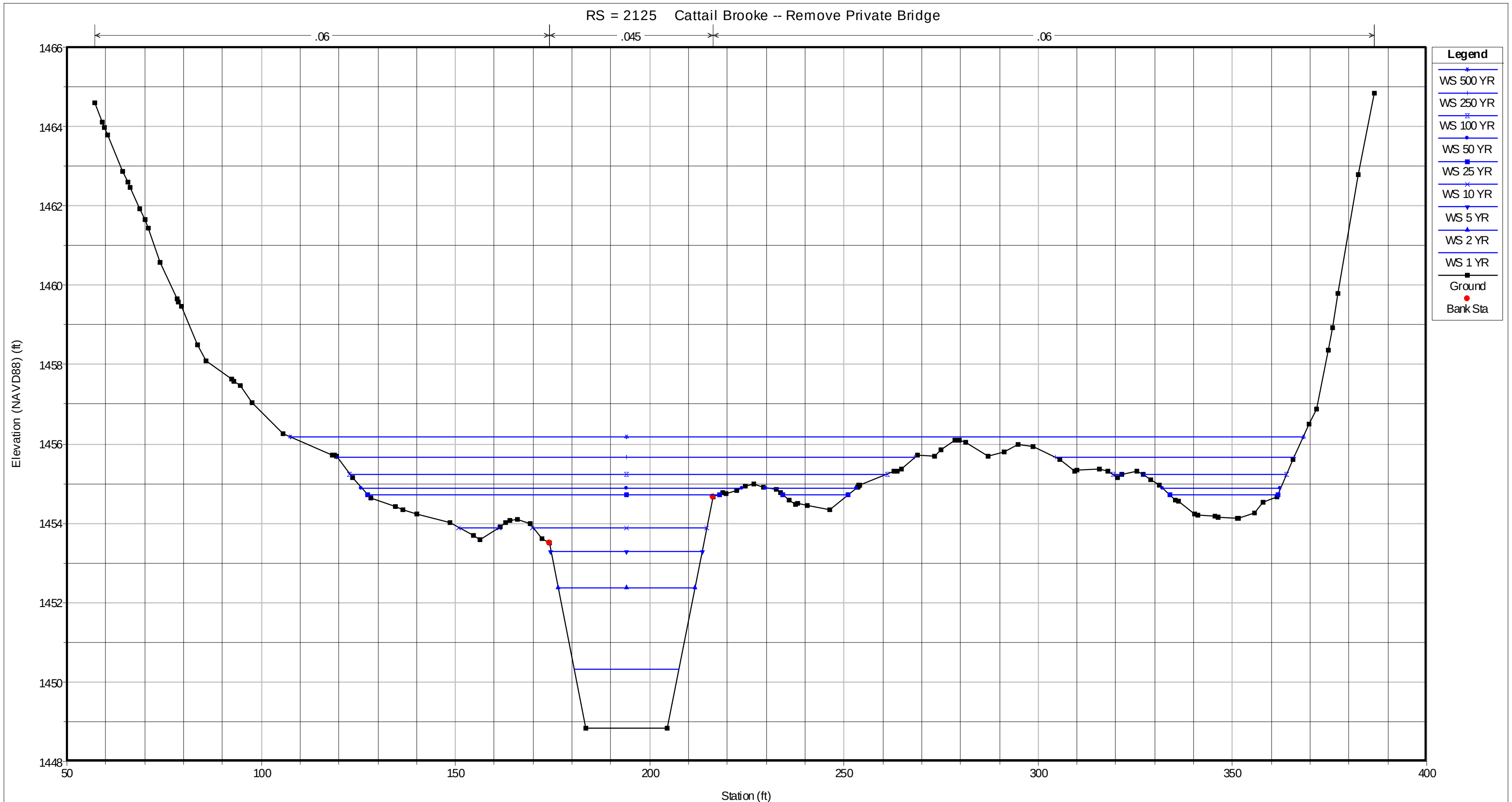


Figure 10.12 – Upstream Face of Private Bridge after Removal

Proposed solutions such as removing the Private Road Bridge and replacing Finch Bridge with a 40 ft span has the effect of eliminating debris blockage at these locations. If this occurs the debris may block at downstream bridges such as Creamery Road Bridge. Further assessment of the downstream bridges and their susceptibility to blockage is required if Finch Street is to be enlarged.

An old constrictive Railroad bridge is downstream of Creamery Road. Its removal will have stage reduction benefits. A conceptual plan for removing the Old Railroad Bridge is shown on Figure 10.13. A With and Without project water surface elevation comparison based on total un-diverted Cattail flow is shown on Table 10.12.

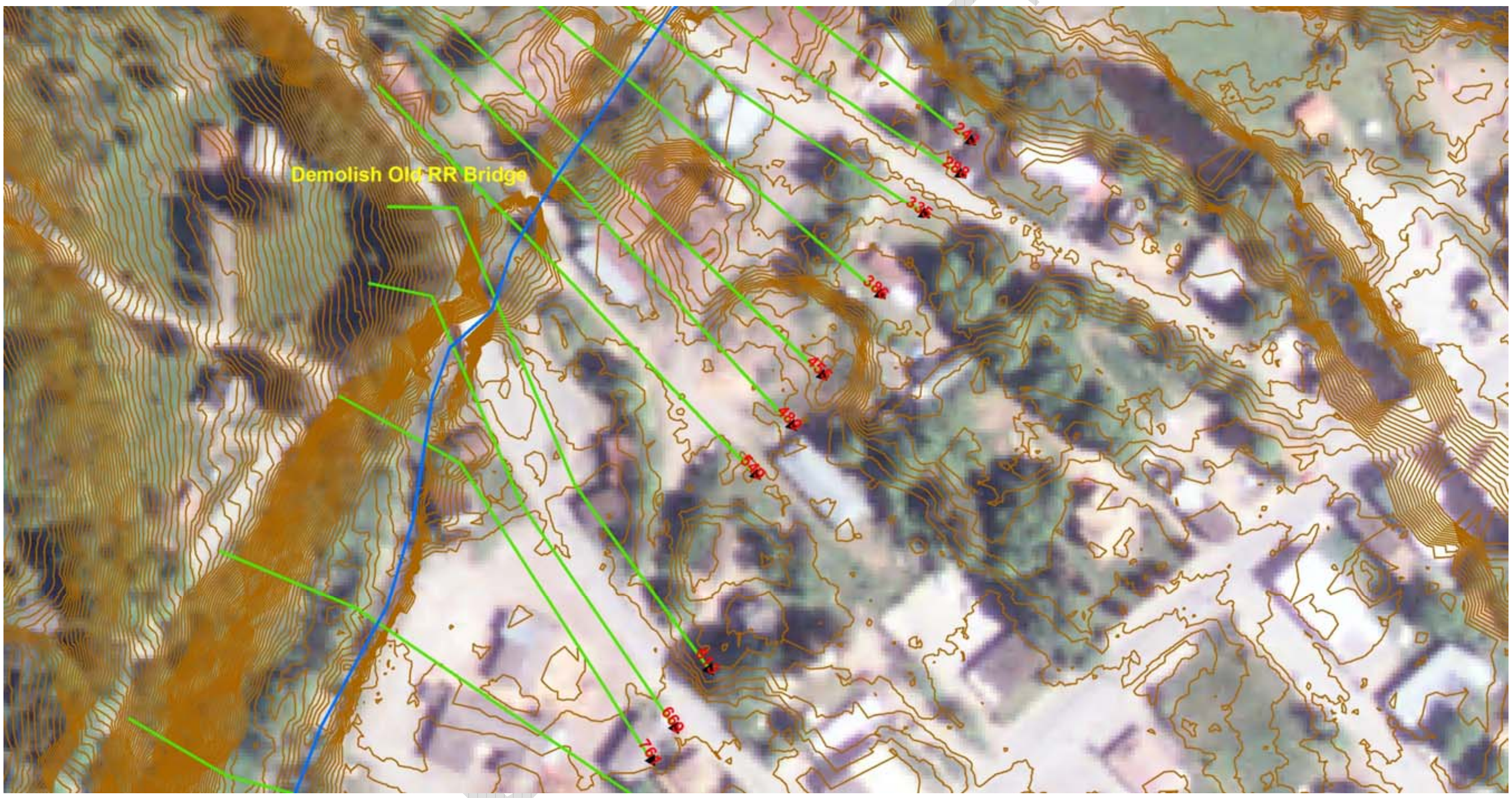


Figure 10.13 – Conceptual Plan for Removing of Old Railroad Bridge

Table 10.12
With and Without Project Cattail Brook Water Surface Elevation Comparison for Removing Old Railroad Bridge
No Debris Blockage at Finch Bridge

X-Section	Event																	
	1 year		2 year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR
489	1420.68	1420.57	1422.64	1422.70	1423.58	1423.54	1424.20	1424.09	1424.98	1424.79	1425.52	1425.20	1426.03	1425.57	1427.01	1426.04	1427.67	1426.35
549	1421.58	1421.09	1424.26	1423.12	1425.54	1424.02	1426.40	1424.63	1427.66	1425.40	1428.65	1425.91	1430.22	1426.39	1430.93	1427.02	1431.18	1427.50
761	1423.47	1423.47	1425.37	1425.30	1426.52	1426.28	1427.42	1426.94	1428.60	1427.86	1429.56	1428.50	1430.95	1429.16	1431.87	1430.66	1432.40	1431.55

Note Cattail Brook flows are total flows.
X-489 and X-549 are the downstream and upstream faces of the RR bridge.
X-761 is upstream of Creamery Road.

The effect of removing the RR Bridge attenuates to zero at cross-section X-925.

A With and Without Project water surface elevation comparison based on reduced Cattail flow because of diversion at Finch Street is shown on Table 10.13.

Table 10.13
With and Without Project Cattail Brook Water Surface Elevation Comparison for Removing Old Railroad Bridge
With Debris Blockage at Finch Bridge

X-Section	Event																	
	1 year		2 year		5 year		10 year		25 year		50 year		100 year		250 year		500 year	
	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR	Exist	RR
489	1420.68	1420.57	1422.64	1422.70	1423.58	1423.54	1424.20	1424.09	1424.11	1424.01	1424.47	1424.34	1424.83	1424.66	1425.23	1425.01	1425.55	1425.22
549	1421.58	1421.09	1424.26	1423.12	1425.54	1424.02	1426.40	1424.63	1426.27	1424.54	1426.76	1424.90	1427.41	1425.26	1428.17	1425.67	1428.71	1425.93
761	1423.47	1423.47	1425.37	1425.30	1426.52	1426.28	1427.42	1426.94	1427.29	1426.83	1427.79	1427.31	1428.36	1427.68	1429.09	1428.18	1429.62	1428.54

Note Cattail Brook flows are split flows.
X-489 and X-549 are the downstream and upstream faces of the RR Bridge.
X-761 is upstream of Creamery Road.

The effect of removing the RR Bridge attenuates to zero at cross-section X-925.

B. Conclusions

After analyzing various solutions, it appears the most immediate and effective solution for Cattail Brook will be a combination of the following measures:

1. Replace the existing Finch Street Bridge with a 40 ft span.

2. Demolish the Old Railroad Bridge.

3. Encourage and partner with local residents to replant the stream banks of Cattail Brook with native vegetation and create a riparian buffer around the brook.
This practice will encourage the stability of the banks and potential reduce future erosion and loss of mature trees. Various native small trees, shrubs and grasses can be planted along the stream bank for erosion control and will enhance the property value. In addition, these planting should also provide important riparian habitat for local wildlife (e.g., birds).

11. POST INTERIM SELECTED PLAN

A. Overview

The Interim selected plan (Plan G of the Main Report) focused solely on Little Beaver Kill Creek. The plan consisted of an excavated bench on the downstream right overbank (ROB – defined looking downstream) of Main Street Bridge and a detention basin upstream in the Airport ponds called the Fulton Plan. Analysis completed since the Interim Report resulted in the Fulton Plan being dropped from further consideration. Plan J has been selected as the Post Interim final plan.

Plan J consists of a modification to the Interim bench on the right overbank downstream of Main St Bridge, a 10ft wide by 4ft high box culvert placed on the right side of Main St Bridge and a stable channel restoration starting just upstream of Main Street and extending 5700 feet to the upstream end of the Airport Ponds. The addition of the box culvert was made economical by a fire that destroyed the building on the ROB just upstream of Main Street. Figures 11.1 and 11.2 show the building pre and post fire, respectively.

The channel restoration design was contracted out to the United States Fish and Wildlife Service (USFWS). Their 30% design analysis is documented in the report, Little Beaver Kill Creek Stream Restoration, Livingston Manor, NY: Project Assessment and 30% Design Report, CBFO-S15-07. As part of their efforts, the USFWS surveyed a channel profile, dated May 2015, from downstream of Main Street Bridge to the upstream limits of the project. Before Plan J was hydraulically modeled, the without project RAS model was edited to incorporate the most recent channel elevations and to remodel the Main Street Bridge to reflect the remnants of the burned out building on the ROB.

Figure 11.1 - Building on ROB Upstream of Main Street Bridge

Figure 11.2 - Remnant of Building on ROB Upstream of Main Street Bridge after Fire

B. Post Interim Without Project

Channel changes since completion of the Interim Without Project hydraulic analysis required an update to the Little Beaver Kill Creek hydraulic model. The without project HEC-RAS model was edited to incorporate the most recent channel elevations surveyed by the USFWS and to remodel the Main Street Bridge to reflect the remnants of the burned out building on the upstream right over bank.

The frequency starting water surface elevations (SWSEL) for the Interim and Post Interim without project condition are the same; they are the Willowemoc River frequency water surface elevations (WSEL) at the mouth of Little Beaver Kill Creek under a peak on peak assumption. They are documented in Table 6.3. The final Post Interim without project water surface elevations at the economic index stations are provided in Table 11.1 and the frequency profile plots are shown on Figure 11.3.

Table 11.1
Little Beaver Kill Creek
Post Interim Without Project Frequency Water Surface Elevations at Economic Index Stations

Economic Index Station	WSEL (ft-NAVD88)							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
X-316	1415.48	1417.92	1419.6	1421.55	1422.53	1423.19	1423.93	1424.45
X-824	1418.24	1420.69	1423.27	1424.97	1425.47	1425.63	1427.05	1427.56
X-942	1419.27	1422.14	1424.01	1425.36	1425.88	1426.18	1427.45	1428.01
X-1101	1419.61	1422.53	1424.27	1425.62	1426.2	1426.58	1427.82	1428.40
X-1337	1419.79	1422.63	1424.34	1425.69	1426.27	1426.67	1427.90	1428.49
X-1697	1419.91	1422.69	1424.38	1425.73	1426.32	1426.73	1427.97	1428.57
X-2138	1420.03	1422.76	1424.44	1425.79	1426.38	1426.81	1428.04	1428.65
X-3293	1419.98	1422.79	1424.47	1425.82	1426.43	1426.86	1428.09	1428.70
X-3917	1420.10	1422.84	1424.50	1425.85	1426.46	1426.89	1428.12	1428.74
X-5862	1423.88	1424.50	1425.38	1426.48	1427.07	1427.53	1428.61	1429.20

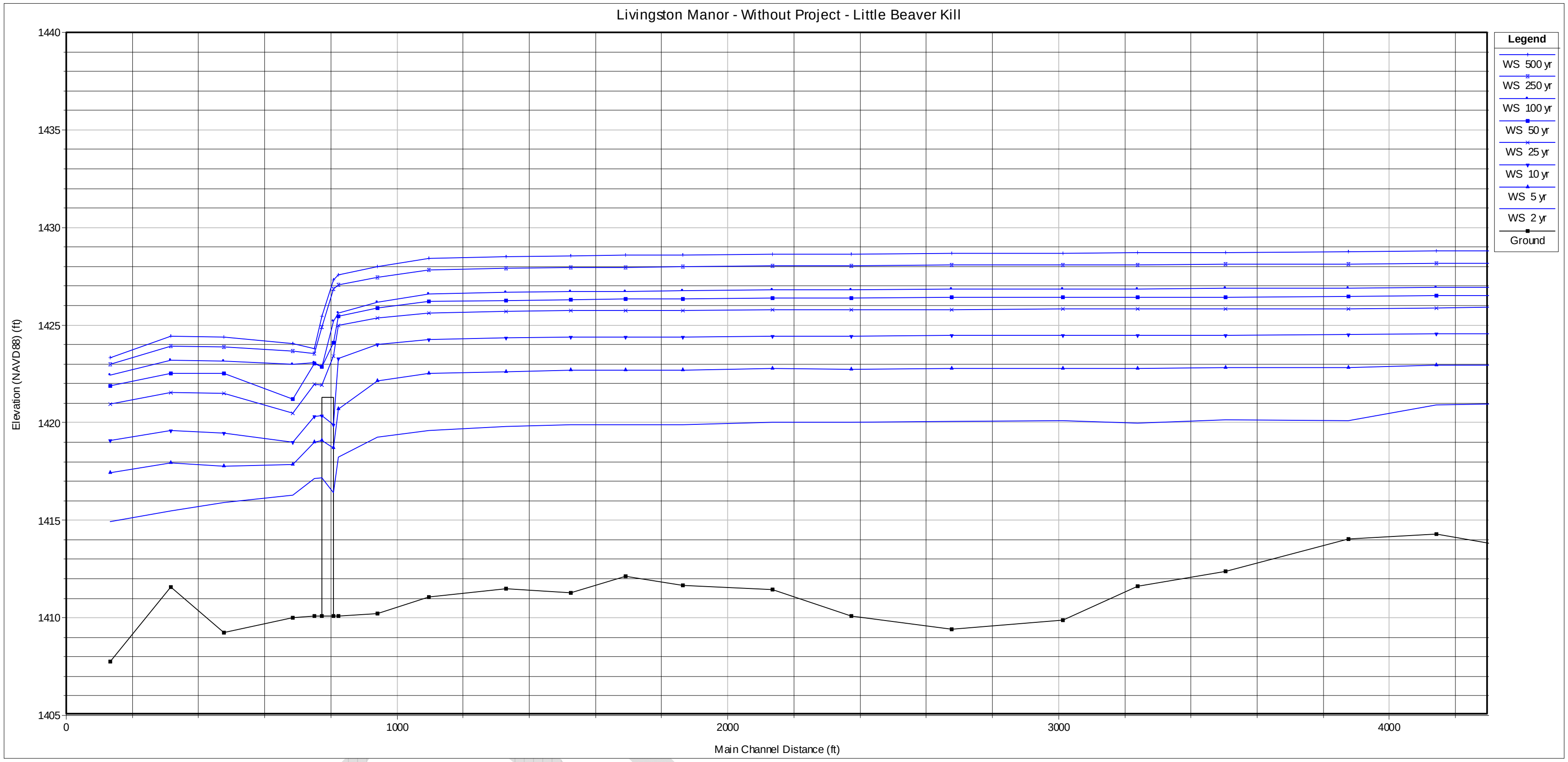


Figure 11.3 - Part 1 - Little Beaver Kill Creek Post Interim Without Project Frequency Water Surface Elevation Profiles

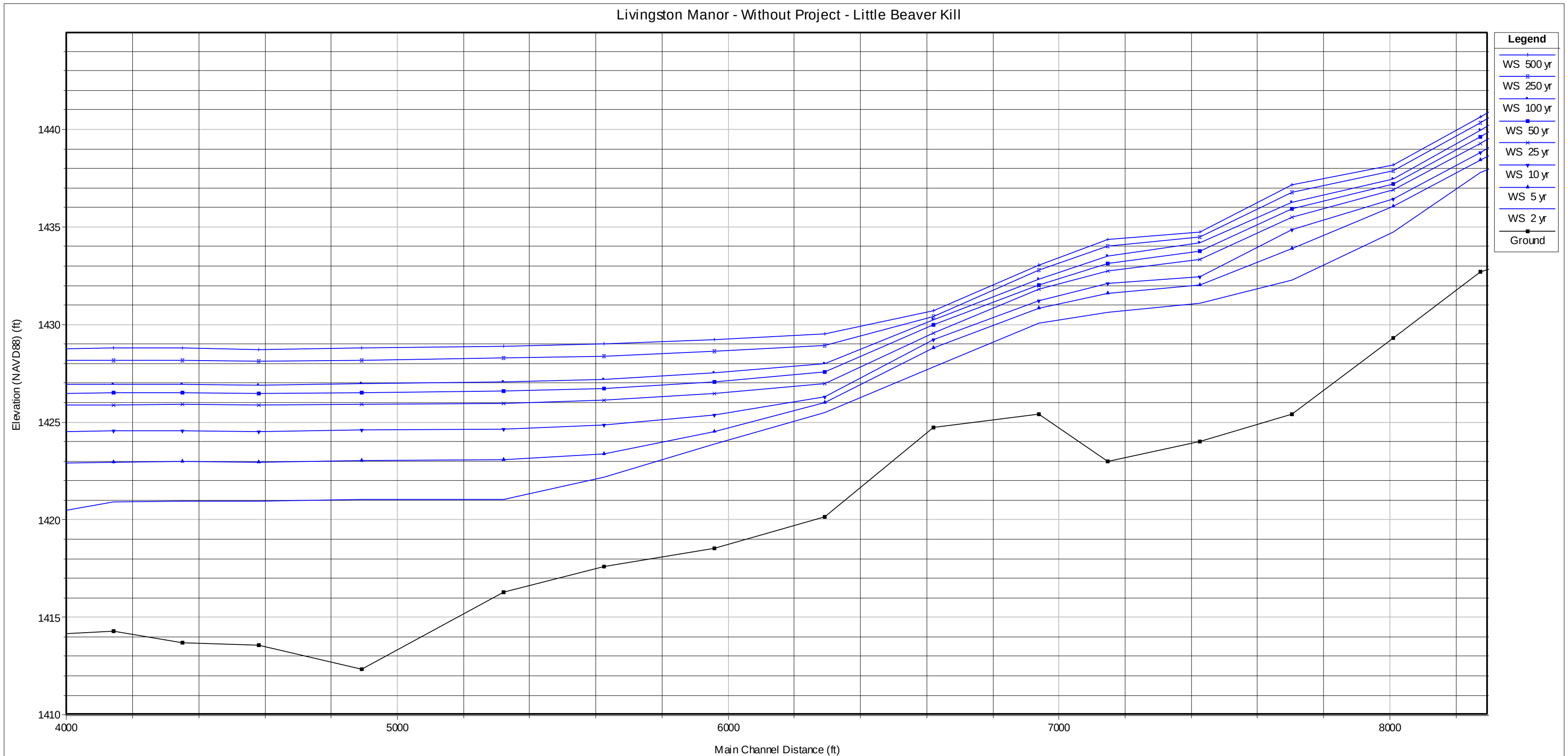


Figure 11.3 - Part 2 - Little Beaver Kill Creek Post Interim Without Project Frequency Water Surface Elevation Profiles

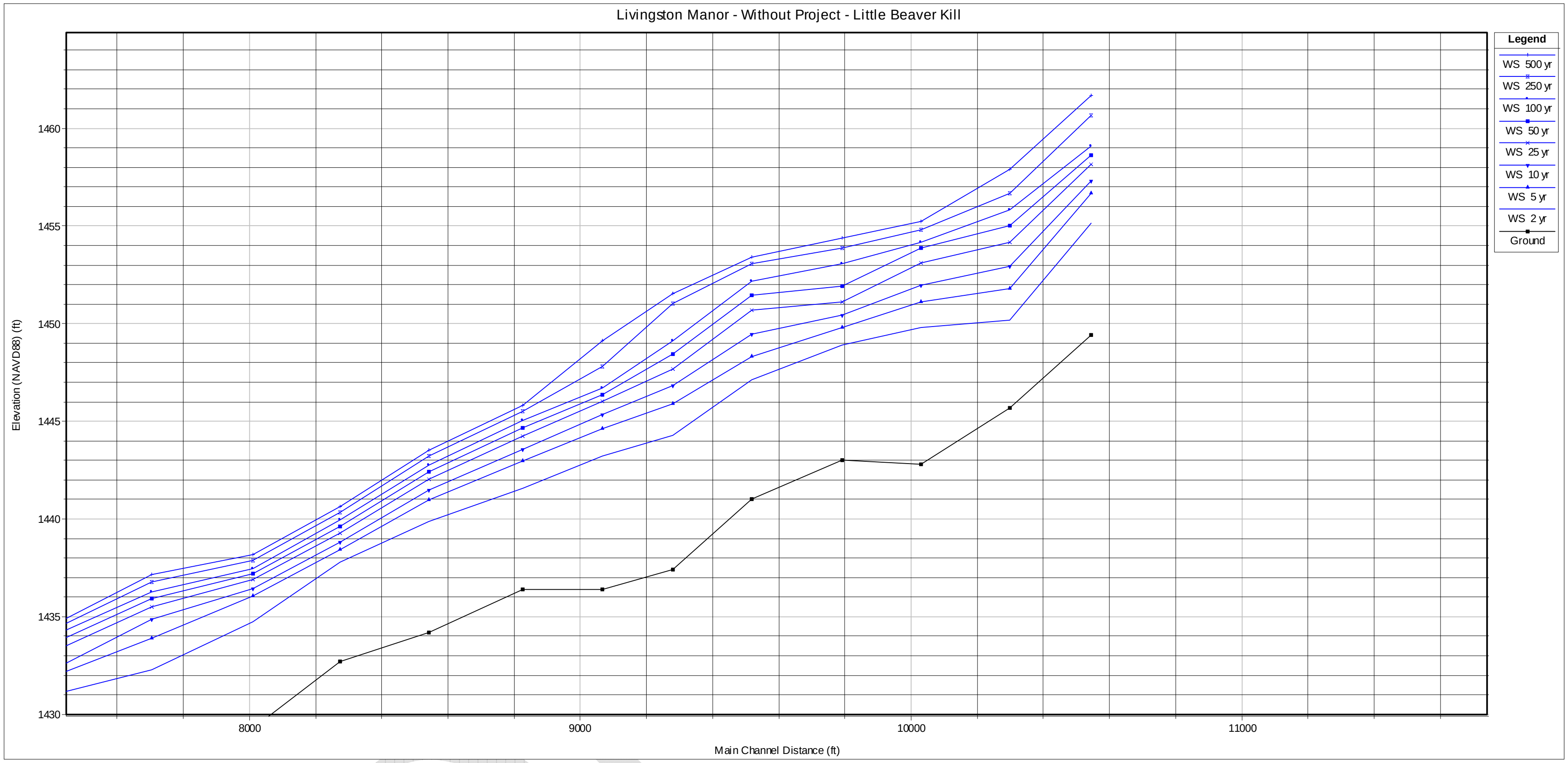


Figure 11.3 - Part 3 - Little Beaver Kill Creek Post Interim Without Project Frequency Water Surface Elevation Profiles

C. Post Interim With Project

Plan J is the Post Interim Selected Plan. It consists of an excavated bench on the right overbank downstream of Main Street, demolition of a concrete pad and building foundation upstream of Main Street on the ROB, placement of a 10ft wide by 4ft tall concrete box culvert adjacent to the right abutment and stream stability work from Main Street to the upstream end of the Airport Ponds.

The plan view of the Post Interim excavated bench is shown on Figure 11.4. The top of bank / bench elevation of 1415 ft-NAVD88 was selected to match the existing bank full elevation under the Main Street Bridge. The Post Interim bench elevation is 1 foot less than the Interim bench elevation of 1416 ft-NAVD88. The width of the bench varies from 20ft to 35ft. The bench and the side slope (to the daylight line) will be planted in grass to minimize hydraulic losses. The 10ft wide, 4ft high box culvert is the largest that can be placed given the vertical and horizontal constraints.

Figure 11.2 shows that the building foundation wall sits on a concrete pad. The elevation of this pad is 1417.8 ft-NAVD88. As part of the culvert placement, the concrete pad (and the foundation wall) will be demolished and a vegetated bench at elevation 1415 ft-NAVD88 will be placed. The box culvert will see water only for flows greater than bank full which is estimated by the USFWS to be 800 cfs.

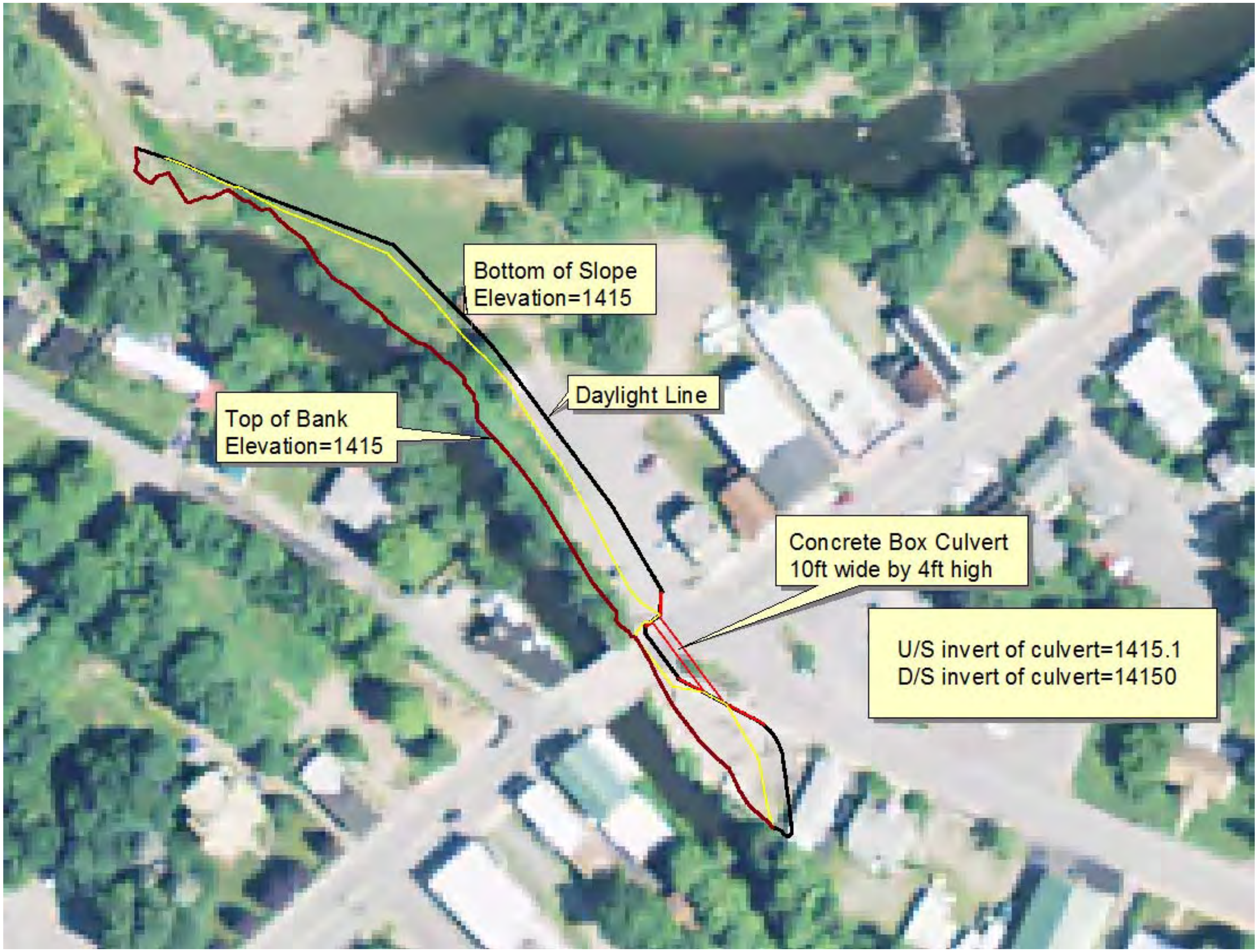


Figure 11.4 - Plan View of Post Interim Excavated Bench Downstream of Main Street Bridge

The plan view of the Post Interim stable channel design is shown on Figure 11.5. Details of the design can be found in the USFWS 30% Design Report referenced above. Typical riffle and pool cross-sections are provided on Figure 11.6. The outer bends at the pools will be armored with toe wood. The upstream and downstream ends of the project will be stabilized with cross vanes. The design requires both excavation and fill. The width of disturbance varies but the maximum width is approximately 525 feet. Figure 11.7 is plot of existing cross-section 2668 with an over plot of the proposed channel and floodplain. An insert of the cross-section location is provided.

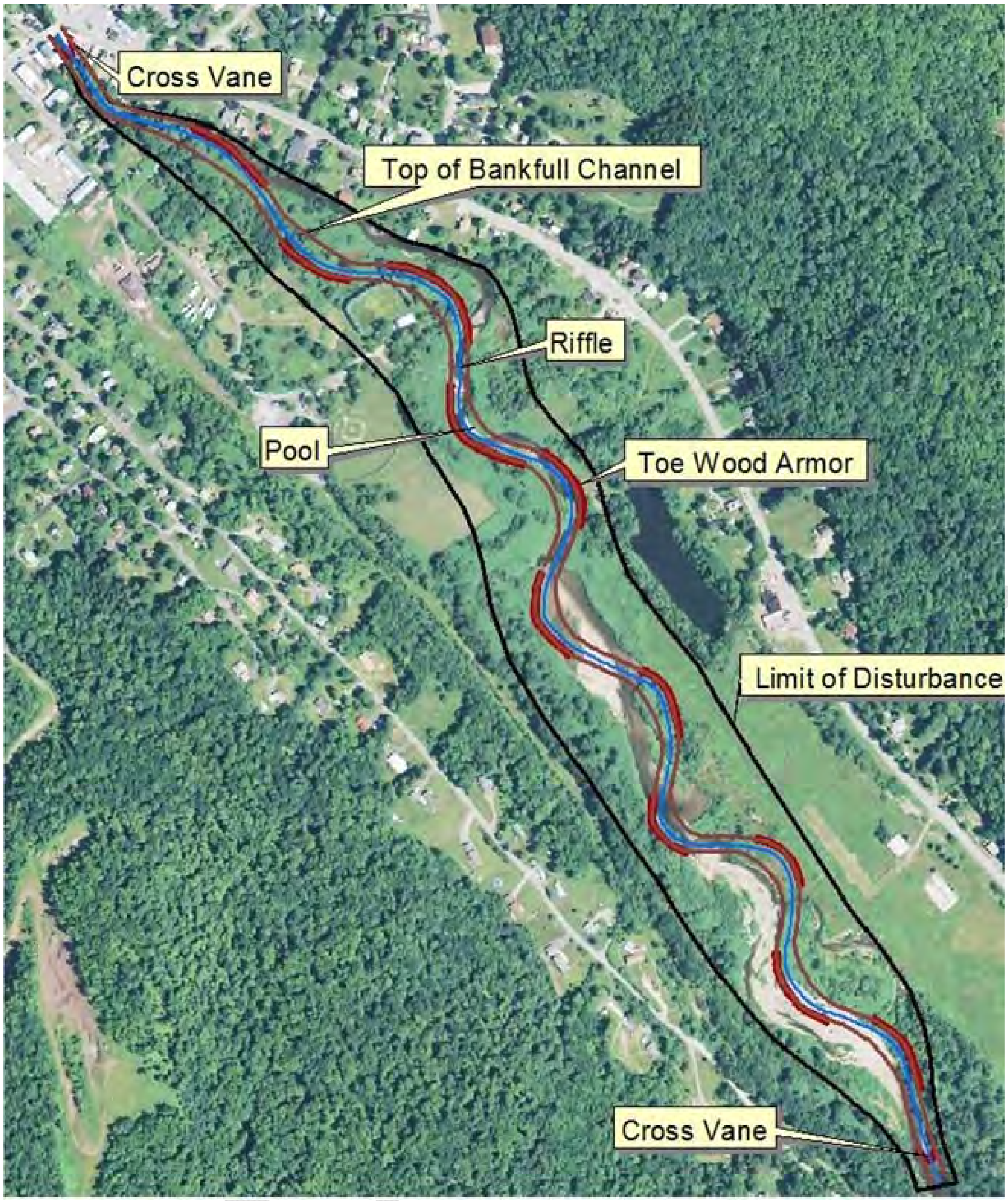


Figure 11.5 – Plan View of Post Interim Stable Channel Design

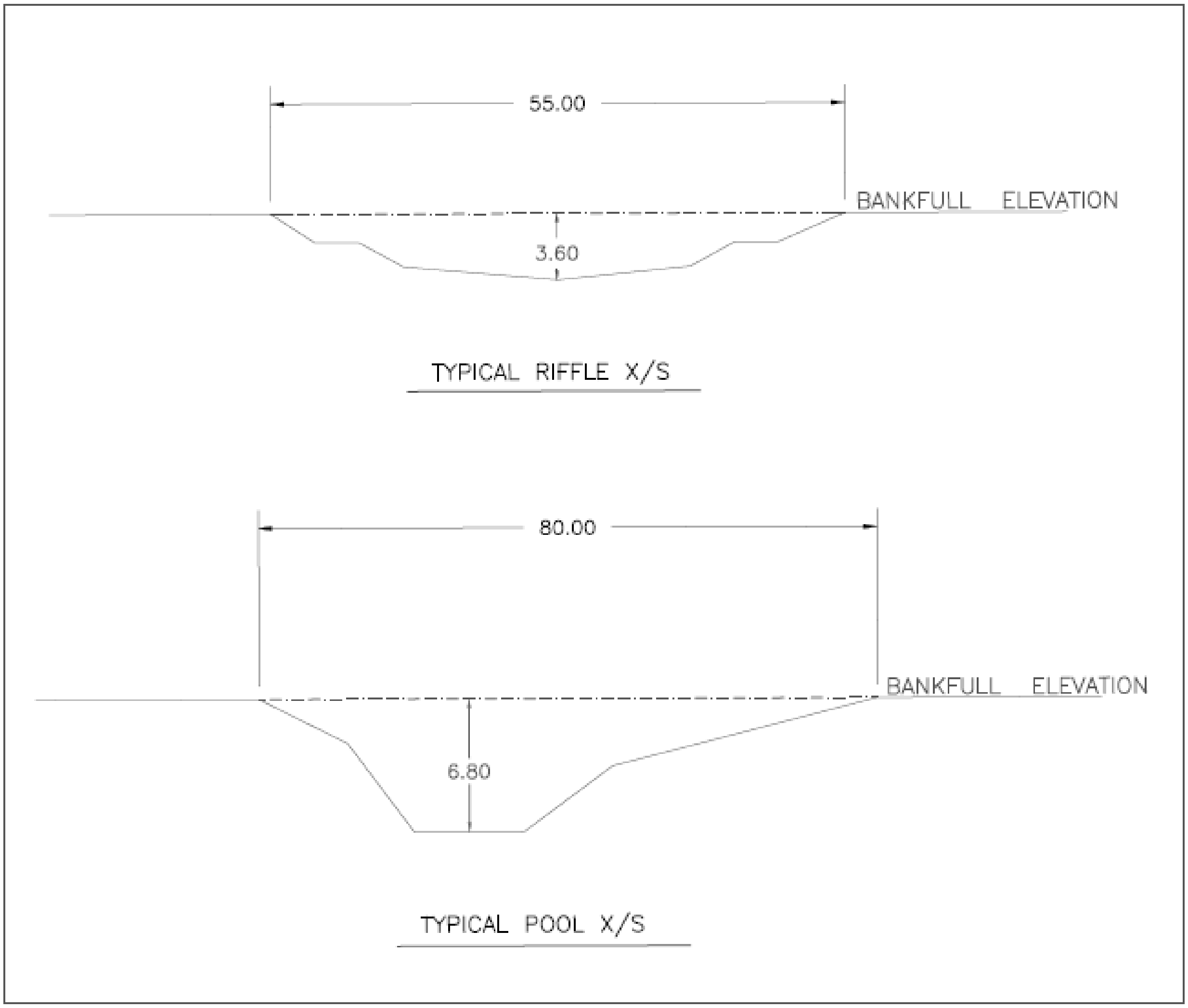


Figure 11.6 – Typical Channel Sections for Post Interim Stable Channel Design

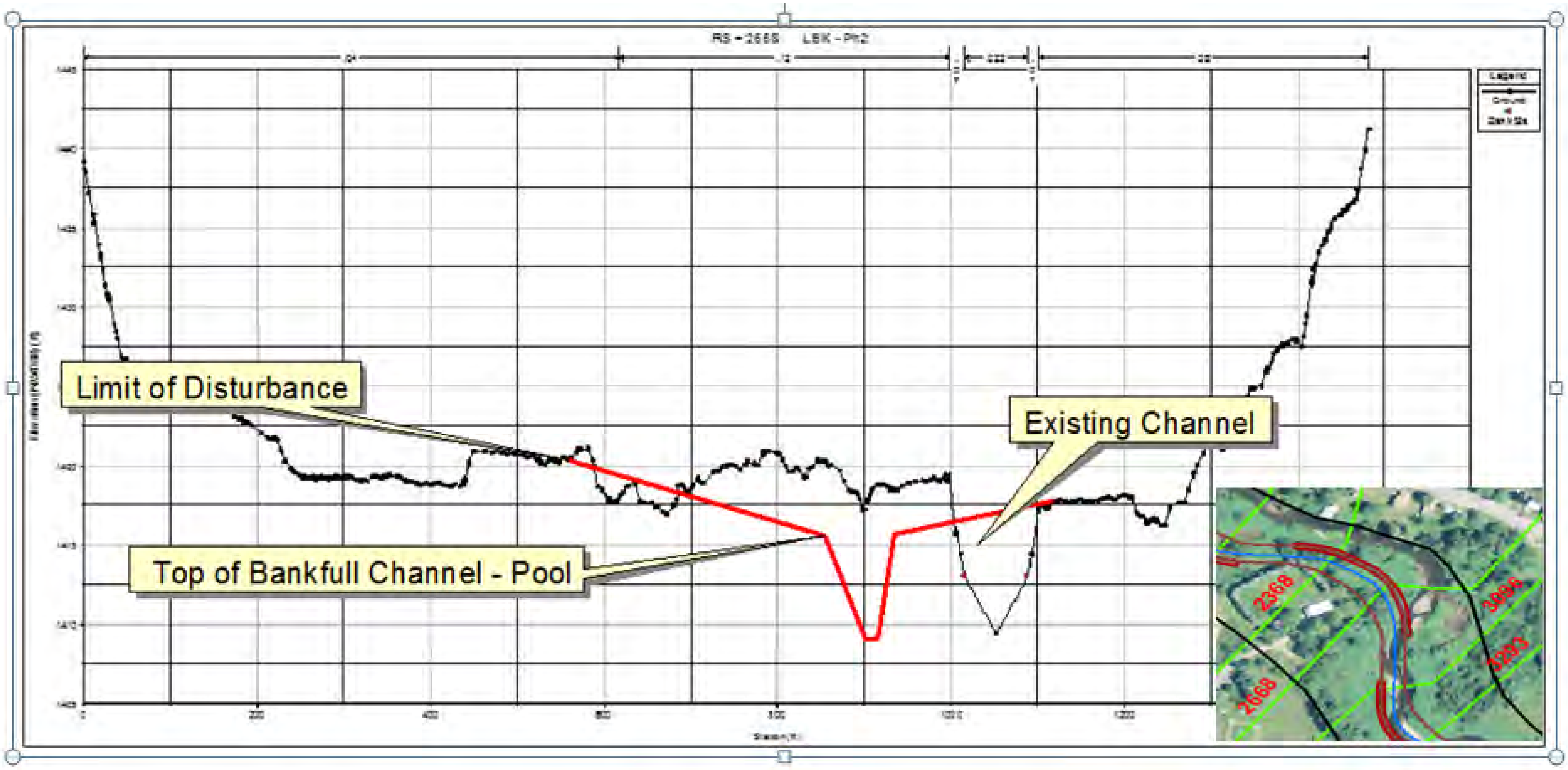


Figure 11.7 – Over Plot of Existing and Post Interim With Plan for X-2668

The Post Interim Without Project HEC-RAS model was modified to reflect with project condition as described above. For the stable channel portion of the model, the without project cross-sections were edited to reflect the channel elevations, dimensions (for riffles and pools) and alignments as defined by the USFWS 30% Plan. The channel Manning n value was specified as 0.033 for cobbles; the overbank n values were kept the same as Post Interim Without Project. The labeling of the cross-sections was unchanged, but the channel reach lengths were adjusted to match the measured reach lengths of the more sinuous channel. The overbank reach lengths were not changed. In order to model all riffles and pools, four cross-sections (X-3541, X-4906, X-5269, X-5862) were duplicated and edited to reflect their moved locations. Figure 11.8 is a plan view of the With Plan cross-sections. Yellow cross-sections are the duplicated cross-sections at their moved locations.

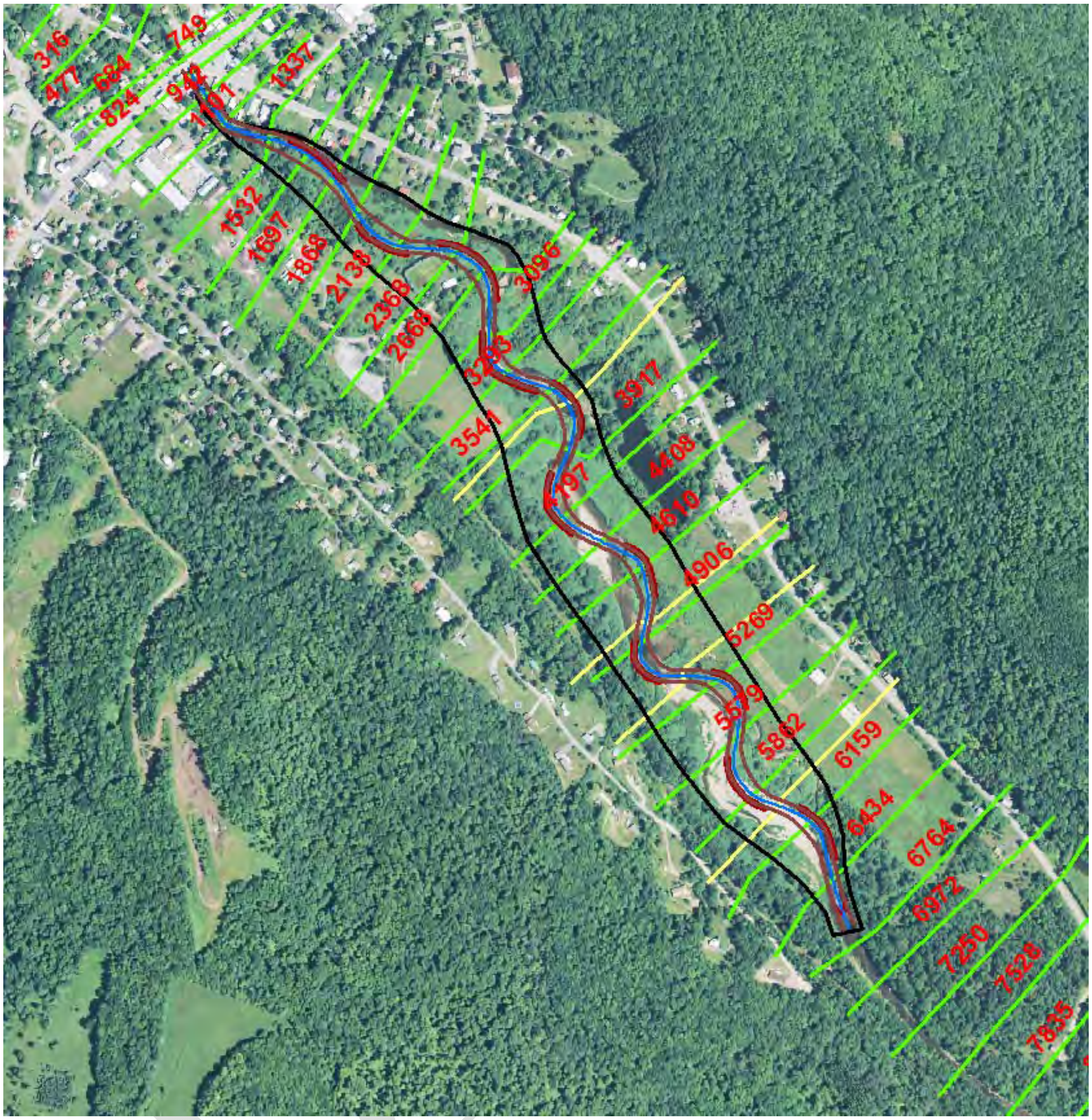


Figure 11.8 – Plan View of Post Interim With Plan Cross-sections

The starting water surface elevations for the Interim With Project (Plan G) and the Post Interim With Project (Plan J) are the same and are found in Table 6.3. The Post Interim With Plan frequency water surface elevations at the economic index stations are provided in Table 11.2 and the frequency water surface elevation profile plots are shown on Figure 11.9.

Table 11.2
Little Beaver Kill Creek
Post Interim With Plan Project Frequency Water Surface Elevations at Economic Index Stations

Economic Index Station	WSEL (ft-NAVD88)							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
X-316	1415.48	1417.92	1419.6	1421.55	1422.53	1423.19	1423.93	1424.45
X-824	1417.23	1418.91	1420.15	1421.75	1423.10	1424.40	1426.07	1427.11
X-942	1417.38	1419.02	1420.25	1422.02	1423.73	1424.98	1426.56	1427.61
X-1101	1418.41	1420.32	1421.76	1423.24	1424.43	1425.54	1427.01	1428.01
X-1337	1418.61	1420.67	1422.02	1423.43	1424.58	1425.66	1427.11	1428.10
X-1697	1418.94	1420.90	1422.18	1423.56	1424.68	1425.75	1427.19	1428.18
X-2138	1419.24	1421.10	1422.33	1423.68	1424.79	1425.84	1427.26	1428.24
X-3293	1419.51	1421.25	1422.46	1423.79	1424.88	1425.92	1427.33	1428.31
X-3917	1420.25	1421.38	1422.59	1423.90	1424.97	1426.00	1427.40	1428.38
X-5862	1426.94	1427.45	1427.84	1428.25	1428.57	1428.9	1429.35	1429.81

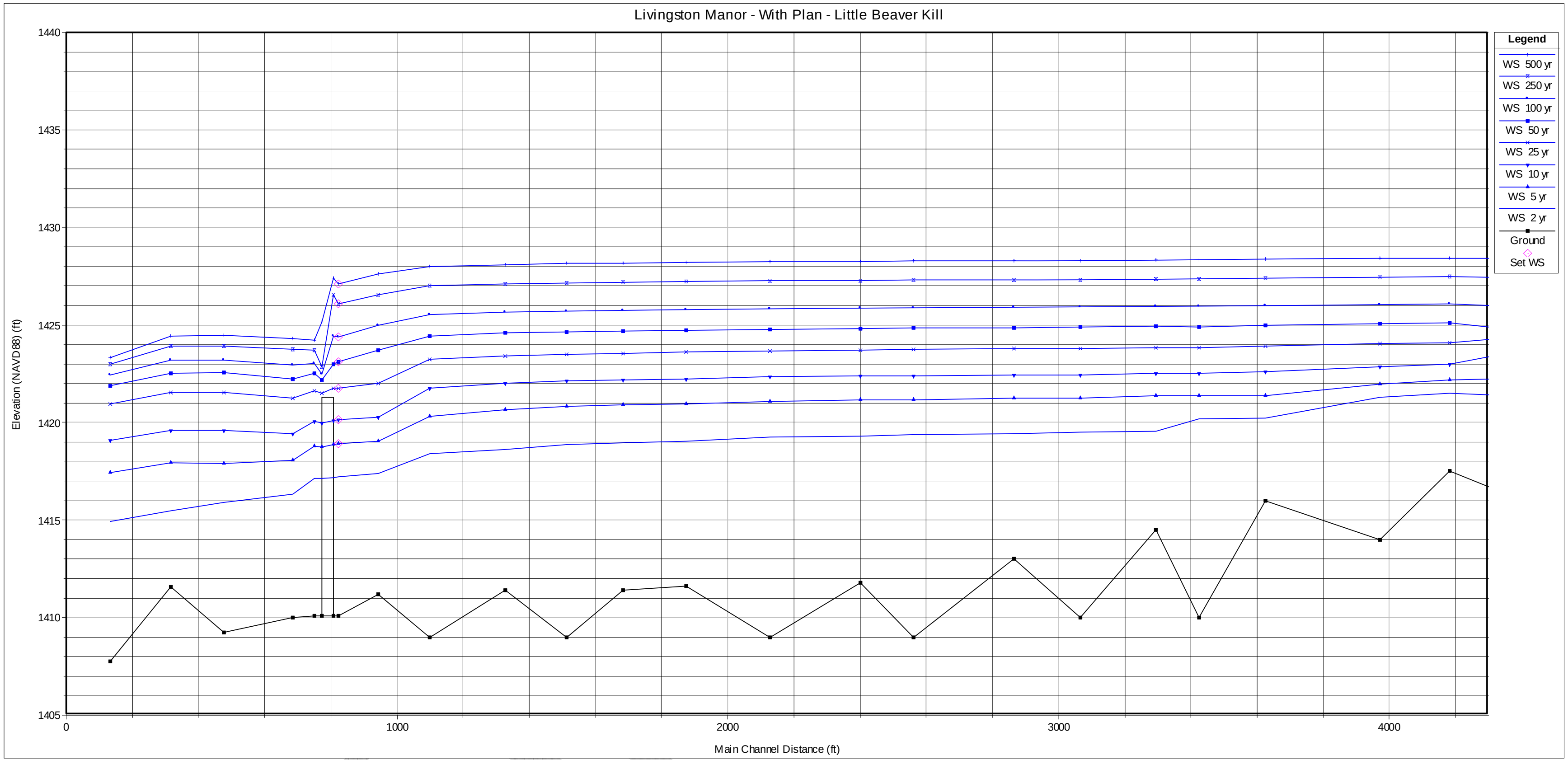


Figure 11.9 - Part 1 - Little Beaver Kill Creek Post Interim Selected Plan J Frequency Water Surface Elevation Profiles

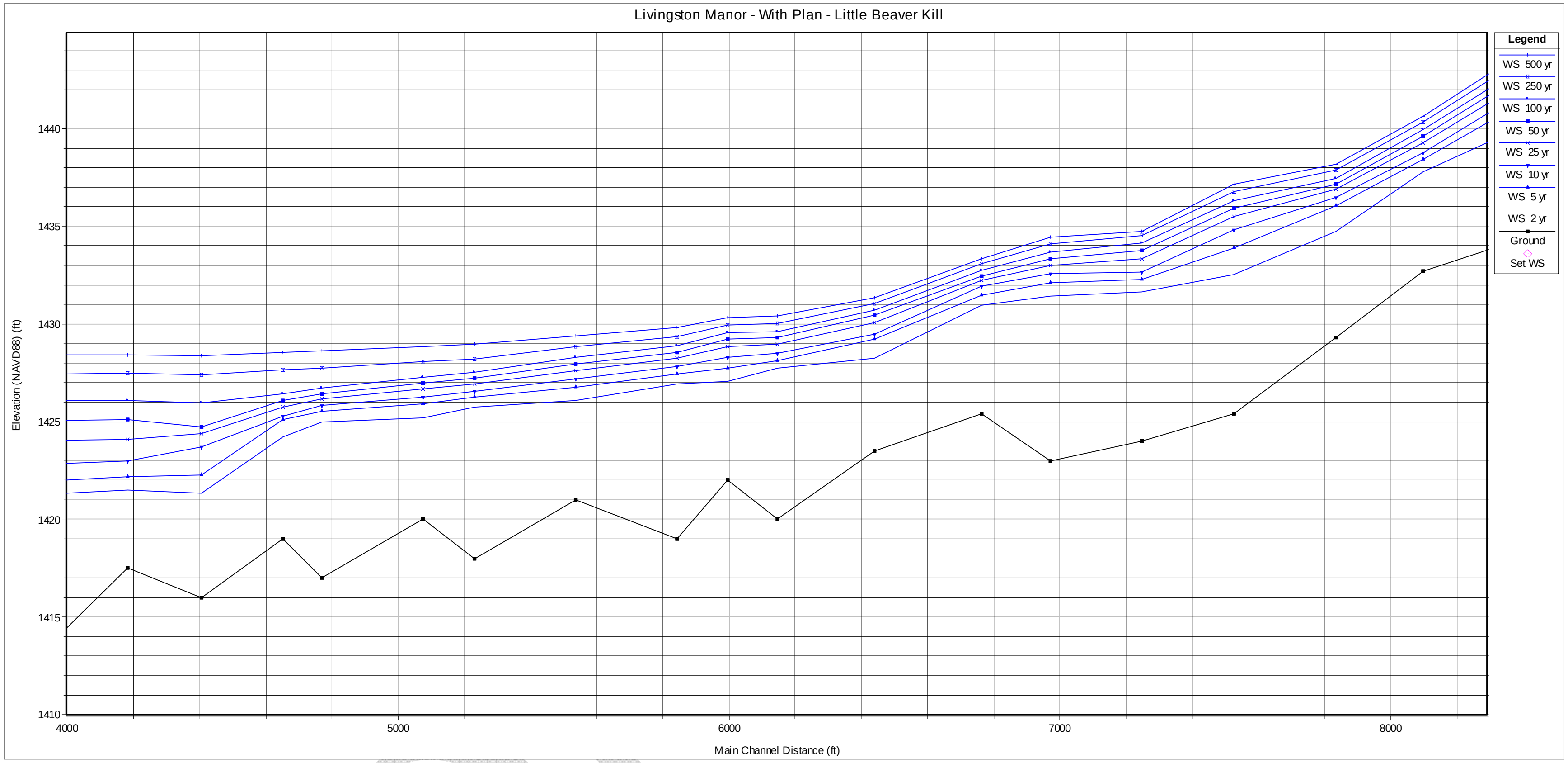


Figure 11.9 - Part 2 - Little Beaver Kill Creek Post Interim Selected Plan J Frequency Water Surface Elevation Profiles

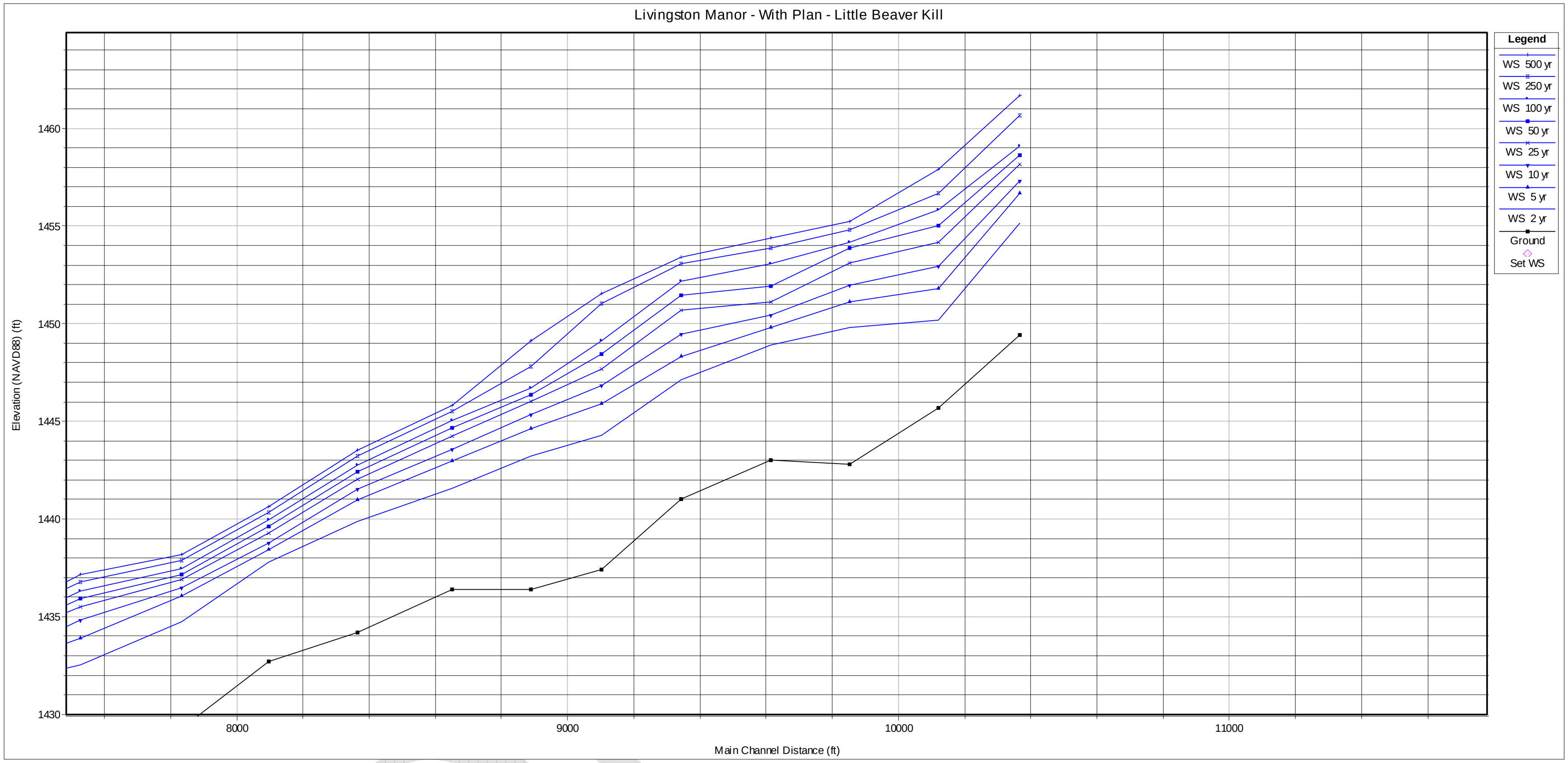


Figure 11.9 - Part 3 - Little Beaver Kill Creek Post Interim Selected Plan J Frequency Water Surface Elevation Profiles

The frequency water surface reductions at the economic index stations are provided in Table 11.3. It is the resulting of subtracting the values in Table 11.2 from the corresponding values in Table 11.1. Stage reductions of over 2ft for the 10yr and 25yr events are predicted upstream of Main Street Bridge along Pearl Street, the major damage center. Even the 100yr event is predicted to have a 1ft reduction.

Table 11.3
Little Beaver Kill Creek
Post Interim With Plan – With OUT Plan Frequency Water Surface Elevations at Economic Index Stations

Economic Index Station	(Feet)							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
X-316	0	0	0	0	0	0	0	0
X-824	-1.01	-1.78	-3.12	-3.22	-2.37	-1.23	-0.98	-0.45
X-942	-1.89	-3.12	-3.76	-3.34	-2.15	-1.20	-0.89	-0.4
X-1101	-1.20	-2.21	-2.51	-2.38	-1.77	-1.04	-0.81	-0.39
X-1337	-1.18	-1.96	-2.32	-2.26	-1.69	-1.01	-0.79	-0.39
X-1697	-0.97	-1.79	-2.20	-2.17	-1.64	-0.98	-0.78	-0.39
X-2138	-0.79	-1.66	-2.11	-2.11	-1.59	-0.97	-0.78	-0.41
X-3293	-0.47	-1.54	-2.01	-2.03	-1.55	-0.94	-0.76	-0.39
X-3917	0.15	-1.46	-1.91	-1.95	-1.49	-0.89	-0.72	-0.36
X-5862	3.06	2.95	2.46	1.77	1.50	1.37	0.74	0.61

Table 11.3 indicates that the water surface elevations at X-5862 are greater than Without Project. This result was investigated by tabulating and comparing without and with project water surface results for the cross-sections at the Airport Ponds. Table 11.4 shows the comparison. The increase in water surface elevations for Post Interim With Project is due to the proposed higher channel elevations relative to Post Interim Without Project channel elevations in the Airport Ponds. The With Plan water surface elevations return to without project water surface elevations at X-7528, which is downstream of the settlement of Morston. Morston is the first cluster of buildings upstream of the project.

Table 11.4
Little Beaver Kill Creek
Post Interim With Plan and Without Frequency Water Surface Elevations at Airport Pond Cross-sections

X-section	Event							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
Without Project Water Surface Elevations (ft-NAVD88)								
X-4610	1420.96	1422.94	1424.52	1425.85	1426.46	1426.89	1428.11	1428.72
X-4906	1421.04	1423.02	1424.59	1425.91	1426.52	1426.96	1428.18	1428.79
X-5269	1421.06	1423.07	1424.63	1425.96	1426.59	1427.05	1428.27	1428.90
X-5579	1422.20	1423.38	1424.84	1426.11	1426.73	1427.19	1428.38	1429.00
X-5862	1423.88	1424.50	1425.38	1426.48	1427.07	1427.53	1428.61	1429.20
X-6159	1425.47	1425.98	1426.28	1426.98	1427.55	1428.01	1428.94	1429.50
X-6434	1427.81	1428.81	1429.24	1429.56	1429.98	1430.23	1430.43	1430.69
X-6764	1430.06	1430.85	1431.23	1431.83	1432.05	1432.31	1432.81	1433.05
X-6972	1430.60	1431.59	1432.13	1432.73	1433.13	1433.50	1434.03	1434.37
X-7250	1431.07	1432.04	1432.45	1433.34	1433.78	1434.18	1434.48	1434.75
X-7528	1432.28	1433.87	1434.88	1435.52	1435.92	1436.28	1436.79	1437.14
With Project Water Surface Elevations (ft-NAVD88)								
X-4610	1421.33	1422.28	1423.73	1424.39	1424.71	1425.95	1427.40	1428.38
X-4906	1425.00	1425.55	1425.84	1426.17	1426.44	1426.74	1427.76	1428.63
X-5269	1425.76	1426.24	1426.54	1426.93	1427.22	1427.51	1428.22	1428.95
X-5579	1426.08	1426.77	1427.17	1427.63	1427.97	1428.29	1428.84	1429.40
X-5862	1426.94	1427.45	1427.84	1428.25	1428.57	1428.90	1429.35	1429.81
X-6159	1427.73	1428.14	1428.51	1428.95	1429.29	1429.62	1430.05	1430.42
X-6434	1428.24	1429.24	1429.48	1430.09	1430.45	1430.70	1431.06	1431.34
X-6764	1430.96	1431.46	1431.95	1432.24	1432.47	1432.75	1433.09	1433.33
X-6972	1431.42	1432.09	1432.56	1433.01	1433.35	1433.70	1434.12	1434.43
X-7250	1431.63	1432.27	1432.66	1433.34	1433.75	1434.16	1434.52	1434.75
X-7528	1432.52	1433.89	1434.82	1435.52	1435.93	1436.29	1436.77	1437.14
Difference in WSEL (With Plan – Without Plan) (feet)								
X-4610	0.37	-0.66	-0.79	-1.46	-1.75	-0.94	-0.71	-0.34
X-4906	3.96	2.53	1.25	0.26	-0.08	-0.22	-0.42	-0.16
X-5269	4.70	3.17	1.91	0.97	0.63	0.46	-0.05	0.05
X-5579	3.88	3.39	2.33	1.52	1.24	1.10	0.46	0.40
X-5862	3.06	2.95	2.46	1.77	1.50	1.37	0.74	0.61
X-6159	2.26	2.16	2.23	1.97	1.74	1.61	1.11	0.92
X-6434	0.43	0.43	0.24	0.53	0.47	0.47	0.63	0.65
X-6764	0.90	0.61	0.72	0.41	0.42	0.44	0.28	0.28
X-6972	0.82	0.50	0.43	0.28	0.22	0.20	0.09	0.06
X-7250	0.56	0.23	0.21	0.00	-0.03	-0.02	0.04	0.00
X-7528	0.24	0.02	-0.06	0.00	0.01	0.01	-0.02	0.00

D. Post Interim With and Without Project Hydraulic Uncertainty

The water surface elevations above for both Without and With Project are calculated using the “best” estimates of hydraulic input parameters. However, to determine a reasonable range of water surface elevation outputs, the without and with project hydraulic models were modified to reflect reasonable but “low” and “high” estimates of input parameters. Table 11.5 summarizes the changes to the models to calculate a reasonable range of final frequency water surface elevations.

Table 11.5
Willowemoc River and Little Beaver Kill Creek
Post Interim Hydraulic Parameters Adjusted for “Low” and “High” WSELs

	“Low”	“High”
Manning’s n Value	n X 0.85	n X 1.15
Bridge Expansion and Contraction Coefficients	0.2 and 0.4	0.4 and 0.6
Bridge Debris (1)	None	Willowemoc: Covered Bridge and Rt. 17 near Sewer Plant
Weir Coefficient for Willowemoc Levees (2)	2.8	2.4

(1) Best estimate of bridge debris was none.
(2) Best estimate Cw=2.6.

All Manning n values for “best” condition were multiplied by 0.85 and 1.15 for “low” and “high” conditions respectively. The aim was to adjust the n values while still maintaining values reasonable for the ground cover.

Little Beaver Kill Creek has only one bridge and it does not have a pier. However, on the Willowemoc River, downstream of the Little Beaver Kill Creek confluence, there are two bridges. These bridges have piers and for the “high” WSEL run, floating pier debris was assumed 8ft wide and 4ft high. The Willowemoc River model was adjusted to quantify hydraulic risk because the Willowemoc River provides the starting water surface elevation for the Little Beaver Kill Creek. Standard deviations were not calculated for the Willowemoc River directly; they are implicitly included in the Little Beaver Kill Creek standard deviations through their effect on the starting water surface elevations.

The starting water surface elevations (SWSELs) for the Little Beaver Kill Creek for best, low and high conditions come from the corresponding Willowemoc River hydraulic models. The range of SWSELs is provided in Table 11.6.

Table 11.6
Little Beaver Kill Creek
Post Interim Range of Starting WSELs*

Event	Low WSEL (ft-NAVD88)	Best WSEL (ft-NAVD88)	High WSEL (ft-NAVD88)
2	1414.73	1414.92	1415.16
5	1417.19	1417.41	1417.70
10	1418.84	1419.08	1419.31
25	1420.77	1420.97	1421.09
50	1421.84	1421.88	1421.97
100	1422.43	1422.43	1422.54
250	1422.97	1422.97	1423.05
500	1423.31	1423.31	1423.38

*Starting water surface elevations are the same for without and with project Little Beaver Kill Creek models.

Without Project frequency water surface elevations profiles were calculated for Little Beaver Kill Creek for both “low” and “high” conditions shown in Table 11.5. The frequency water surface elevation results are summarized by calculating for each frequency, a standard deviation at the Economic Index Stations. The standard deviation is defined as: (“high” WSEL – “low” WSEL) /4. The Without project standard deviations are provided in Table 11.7.

Table 11.7 Little Beaver Kill Creek Post Interim Without Project Frequency Standard Deviations at the Economic Index Stations								
Index Station	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
316	0.1625	0.1375	0.1225	0.0875	0.0425	0.0375	0.0275	0.0375
824	0.0925	0.1375	0.3400	0.0975	0.0925	0.1125	0.1450	0.1475
942	0.1250	0.1900	0.1825	0.1000	0.0950	0.0975	0.1500	0.1525
1101	0.1250	0.1600	0.1650	0.0925	0.0900	0.0850	0.1350	0.1375
1337	0.1250	0.1525	0.1625	0.0950	0.0900	0.0825	0.1350	0.1375
1697	0.1225	0.1525	0.1625	0.0950	0.0925	0.0850	0.1350	0.1400
2138	0.1225	0.1525	0.1625	0.0950	0.0925	0.0825	0.1375	0.1400
3293	0.1475	0.1575	0.1650	0.0975	0.1000	0.0875	0.1425	0.1425
3917	0.1325	0.1625	0.1700	0.1025	0.1025	0.0900	0.1475	0.1500
5862	0.1150	0.1175	0.1925	0.1525	0.1550	0.1450	0.1800	0.1800

Note: Standard deviations apply throughout the project life.

The With Project Plan's best estimate of water surface elevations was also modified for low and high conditions. The downstream boundary conditions (best, low and high) are the same as Without Project condition. The adjustments of the hydraulic parameters for the low and high runs are also the same as Without Project. The frequency standard deviations were calculated at the Economic Index stations and are provided in Table 11.8.

Table 11.8 Little Beaver Kill Creek Post Interim With Project Frequency Standard Deviations at the Economic Index Stations								
Index Station	Events							
	2yr	5yr	10yr	25yr	50yr	100yr	250yr	500yr
316	0.1625	0.1375	0.1225	0.0875	0.0425	0.0375	0.0275	0.0375
824	0.1400	0.1700	0.1800	0.2150	0.3075	0.3250	0.3175	0.1350
942	0.2200	0.2550	0.2700	0.3350	0.3250	0.2900	0.3050	0.1525
1101	0.1175	0.1575	0.1550	0.1325	0.2200	0.2350	0.2625	0.1400
1337	0.1425	0.1550	0.1450	0.1300	0.2125	0.2300	0.2575	0.1400
1697	0.1400	0.1525	0.1450	0.1350	0.2125	0.2300	0.2575	0.1450
2138	0.1375	0.1525	0.1475	0.1375	0.2125	0.2300	0.2575	0.1500
3293	0.1450	0.1650	0.1550	0.1450	0.2150	0.2325	0.2600	0.1525
3917	0.0000	0.2375	0.1700	0.1525	0.2225	0.2350	0.2625	0.1575
5862	0.0600	0.0925	0.1200	0.1375	0.1625	0.1750	0.2050	0.2225

Note: Standard deviations apply throughout the project life.