US ARMY CORPS
OF ENGINEERS
NEW YORK DISTRICT

NEW JERSEY
DEPARTMENT OF
ENVIRONMENTAL
PROTECTION

UPPER PASSAIC RIVER AND TRIBUTARIES

AT LONG HILL TOWNSHIP, NJ

FLOOD DAMAGE REDUCTION
AND
ECOSYSTEM RESTORATION

DRAFT DETAILED PROJECT REPORT
APPENDIX A – DRAFT HYDROLOGY & HYDRAULICS

HYDROLOGY & HYDRAULICS MODELING OF EXISTING AND FUTURE NO-PROJECT CONDITIONS

HYDROLOGY & HYDRAULICS MODELING OF ALTERNATIVES

Volume 2

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UPPER PASSAIC RIVER FLOOD CONTROL
LONG HILL TOWNSHIP, NEW JERSEY
N.Y. DISTRICT, U.S. ARMY CORPS OF ENGINEERS

DETAILED PROJECT REPORT

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Section 2 Exhibit 3. Bulletin 17b Frequency Analysis For The Passaic River At Little Falls, New Jersey (Station 01389500) 2-18

Section 7. Exhibit 1 7-8

- Spreadsheet-based incipient motion calculations for the identification of potential deposition and erosion zones for 2-year return period discharges
- Spreadsheet-based incipient motion calculations for the identification of potential deposition and erosion zones for 10-year return period discharges
- Spreadsheet-based incipient motion calculations for the identification of potential deposition and erosion zones for 100-year return period discharges

CD Attachment Table of Contents

HEC-RAS Project Files:
- Existing Conditions
- Future Conditions No Project

HEC-HMS Project Files:
- Existing Conditions
- Future Conditions No Project

HEC-IFH Project Files:
- Existing Conditions
- Future Conditions No Project

UPPER PASSAIC RIVER AND TRIBUTARIES AT LONG HILL TOWNSHIP, NEW JERSEY

FLOOD DAMAGE REDUCTION AND ECOSYSTEM RESTORATION

DETAILED PROJECT REPORT APPENDIX A – DRAFT HYDROLOGY & HYDRAULICS

1.0 Introduction

The study area for this project is the Long Hill Township, located in Morris County, New Jersey, along the upper reach of the Passaic River, as shown in Figure 1-1.

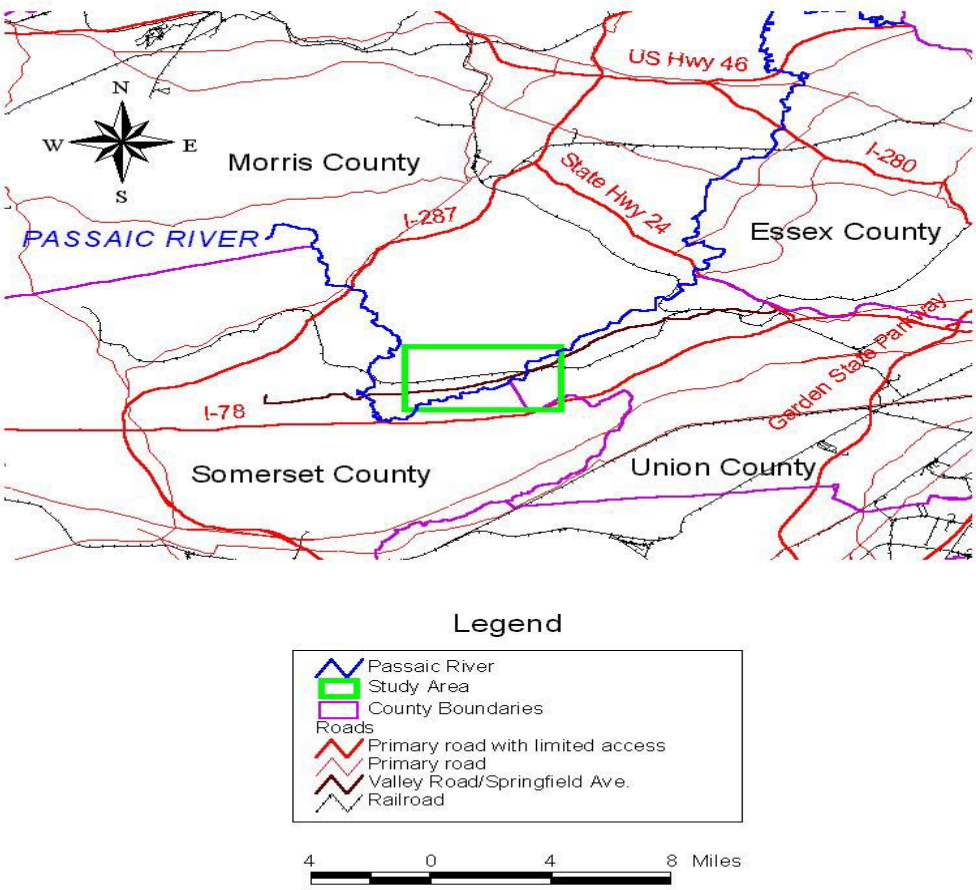

Figure 1-1. Upper Passaic River Flood Control Feasibility Study site location map

The township has experienced flooding as the result of heavy rainfall on many occasions during the 135 years of record, but, at present, there is no flood control system in place.

In 1995 the New York District completed a study for the Passaic River Damage Reduction Project. That study did not include a detailed evaluation of flood effects at Long Hill Township because it's focus was on the lower reaches of the river system.

A preliminary evaluation of flooding damage within the study area was presented in the Reconnaissance Study for the Upper Passaic River Flood Control & Environmental Restoration Study (Section 905-b, WRDA 86) Preliminary Analysis (USACE, N.Y. District, July 1998).

Additional project definition was presented in the Project Study Plan for the Upper Passaic River Flood Control & Environmental Restoration Study (USACE, N.Y. District, June 1999).

In October 2001, the New York District of the Corps of Engineers authorized Michael Baker Jr. Inc. to conduct hydrologic and hydraulic studies in support of an economic evaluation of alternative flood control projects to reduce the annual estimated flood damage within the township.

The study area for hydrologic analysis for the present study was limited to the approximately 100 square mile watershed upstream of the USGS stream gage at Chatham, New Jersey. The study area for hydraulic analysis was limited to the approximately 26,000-foot reach of the Passaic River upstream of the Snyder Avenue crossing near Berkeley Heights.

The items identified for the completion of existing conditions hydrology (Task 1) in the Scope of Work are:

- Statistical analysis of currently existing stream gage records
- Development and calibration of existing conditions hydrologic model
- Development and calibration of existing conditions hydraulic model
- Uncertainty analysis of hydraulic data
- Development of existing conditions HEC-IFH models
- Assessment of stability of existing channel

Future Conditions modeling (Task 3) included all hydrologic, hydraulic, and statistical analysis required to develop stage-frequency curves and associated uncertainty for future conditions with no project. The economic analysis of expected annual damages in a risk-based context was performed under a separate task.

The items identified for the completion of Task 3 in the Scope of Work are:

- Development of future conditions no-project hydrologic model
- Development of future conditions no-project hydraulic model
- Development of future conditions no-project HEC-IFH models

Two structural and one non-structural alternative were proposed for mitigating flood damage in the Long Hill Township in the Reconnaissance Study for the Upper Passaic River Flood Control & Environmental Restoration Study (Section 905-b, WRDA 86) Preliminary Analysis (USACE, N.Y. District, July 1998). These alternatives were:

- Provision of closure structures (sluice gates) and "minimum interior drainage facilities" for the culverts draining the four tributaries to the Passaic River within the study area along Valley Road and Morristown Road
- Provision of sluice gates and interior improvements as above plus the construction of a floodwall south of Valley Road to the 100-year flood elevation. Both alternatives will reduce the frequency of flooding of Long Hill Township from the Passaic River.
- Provision of a flood warning system for the Long Hill Township

Appendix A presents a summary of the findings for all the above enumerated study tasks and documents the work done to complete each of these tasks. All HEC models run for this study are included on the attached CD.

2.0 STATISTICAL ANALYSIS OF CURRENTLY EXISTING STREAM GAGE RECORDS

The U.S. Army Corps of Engineers (USACE), New York District, is undertaking a flood control feasibility study for the Passaic River in Long Hill Township, Morris County, New Jersey. As part of this study, flood frequency analyses were updated for three gaging stations on the Passaic River near Millington (drainage area 55.4 square miles), near Chatham (100 square miles), and at Little Falls (drainage area 762 square miles). The gaging station at Little Falls is not in the immediate study area but was analyzed to check for consistency with the two upstream gages. The USACE developed frequency curves for Chatham and Little Falls using data through 1994 and these analyses are given in General Design Memorandum, Passaic River Flood Damage Reduction Project, Appendix C – Hydrology & Hydraulics, dated September 1995 (USACE, 1995). Frequency curves for the three gaging stations were updated using data through the 2000 water year with annual peak data prior to 1965 adjusted for urbanization following the same approach taken in USACE (1995).

2.1 Analyses

Flood records on the Passaic River at Little Falls are available since 1882 and since the early 1900's for Millington and Chatham. During this time, urbanization has increased significantly in the Passaic River watershed. The USACE (1995) determined there were significant trends in the annual peak data for Chatham and Little Falls and, through the use of statistical tests and knowledge of the watershed, determined that flood records prior to 1965 should be adjusted to be consistent with the more current data since 1965. As described by the USACE (1995), "While we recognize that urbanization is not generally an episodic event, neither is it a linear progression. It is admittedly an approximation to adjust the discharge for urbanization uniformly over a given time period, however it appears to be no greater an approximation than other methodologies available. The adjustment that we performed had the greatest impact on lower peak discharges and little or no impact on larger peak discharges".

For Chatham and Little Falls, the adjusted peak discharges prior to 1965 were taken from Table C-8 of USACE (1995). The adjusted peak discharges for Millington for the period 1904 to 1964 were estimated in this analysis following the method given in USACE (1995). Frequency curves were computed for the two periods 1904 to 1964 and 1965 to 2000 and the following quadratic equation was developed:

$$Y = 36.0618 + 1.2521 * X - 0.00007886 * X^2 \qquad (1)$$

where Y is the frequency estimate, in cubic feet per second (cfs), for selected percent chance exceedances for the period 1965 to 2000, and X is the frequency estimate, in cfs, for selected percent chance exceedances for the period 1904 to 1964. Equation 1 represents the relation in frequency estimates for the two periods. Adjusted annual peak discharges for the period 1904 to 1964 were estimated by applying Equation 1 to the observed peak discharges. The data used in developing Equation 1 are listed in Table 2.1.1 and plotted in Figure 2.1.1.

Table 2.1.1. Supporting data for Annual peak trend analysis for the Passaic River near Millington, New Jersey

Exceedance probability	*Return periods (years)*	*Discharge (1904-64) (cfs)*	*Discharge (1965-2000) (cfs)*
0.95	*1.05*	*422.4*	*536.9*
0.90	*1.11*	*469.7*	*599.5*
0.80	*1.25*	*540.5*	*690.4*
0.50	*2.0*	*734.5*	*925.6*
0.20	*5*	*1052*	*1281*
0.10	*10*	*1298*	*1539*
0.04	*25*	*1652*	*1889*
0.02	*50*	*1949*	*2169*
0.01	*100*	*2277*	*2466*
0.005	*200*	*2639*	*2782*
0.002	*500*	*3180*	*3232*

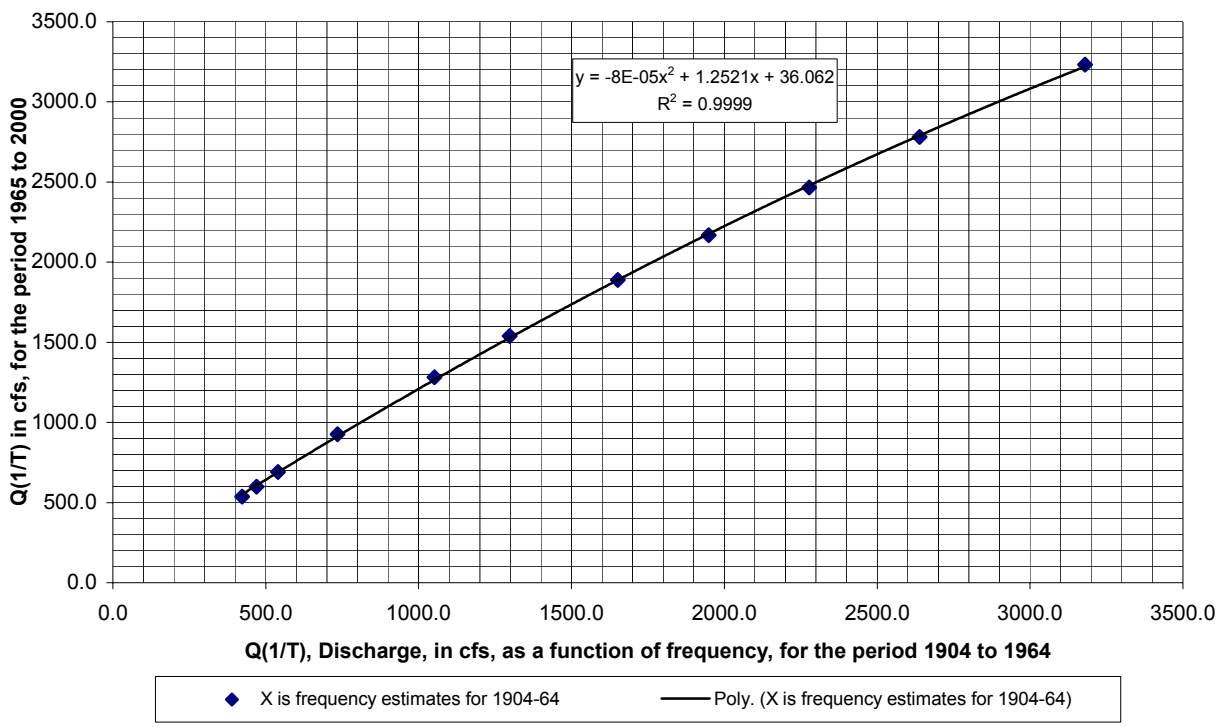

Figure 2.1.1 Comparison of frequency estimates for two time periods for the Passaic River near Millington, New Jersey.

Frequency analyses were performed for the Passaic River gaging stations using station skew rather than weighted (station and generalized) skew because:

(1) the peak discharges are effected by urbanization and the available skew maps are for unregulated (rural) stations,

(2) the storage in the Passaic River watershed is greater than nearby watersheds, and

(3) the record lengths for the Passaic River gaging stations exceed 70 years and weighting with a generalized skew would have little impact on the frequency estimates.

The frequency analyses were performed using the U.S. Geological Survey (USGS) program PEAKFQ (Thomas and others, 1998) that implements Bulletin 17B guidelines (Interagency Advisory Committee on Water Data (IACWD), 1982). The USGS PEAKFQ program is equivalent to the USACE HEC-FFA program (USACE, 1992). The frequency analyses for Millington, Chatham and Little Falls are provided as Exhibits 1, 2 and 3, respectively, at the end of this section.

The Bulletin 17B frequency curve, the annual peak data and the 5- and 95-percent confidence limits for the Passaic River near Millington are given in Figure 2.1.2, below. There is a 90-percent chance that the population frequency estimates will be between the 5- and 95-percent limits.

The 5- and 95-percent confidence limits for Millington and for the other Passaic River stations are quite narrow due to the low variability of the annual peak data (standard deviations ranging from 0.15 to 0.19 log units), long record lengths (ranging from 71 to 105 years), and the fact that the Bulletin 17B guidelines do not include the uncertainty in the skew coefficient in the computation of the confidence limits (IACWD, 1982). For example, the 5- and 95-percent confidence limits for Millington for the 1-percent chance flood discharge (2,490 cfs) are 2,920 and 2,190 cfs, respectively (Exhibit 1).

Using procedures in Kite (1988) that consider the uncertainty in the skew coefficient, the 5- and 95-percent confidence limits become 3,160 and 1,960 cfs, respectively. The standard error of the 1-percent chance flood discharge for Millington is 14.6 percent. The annual peak data shown in Figure 2.1.2 for Millington exhibit a S-shaped pattern that is typical of regulated watersheds. Presumably, this S-shape pattern is due to the significant surface storage upstream of Millington.

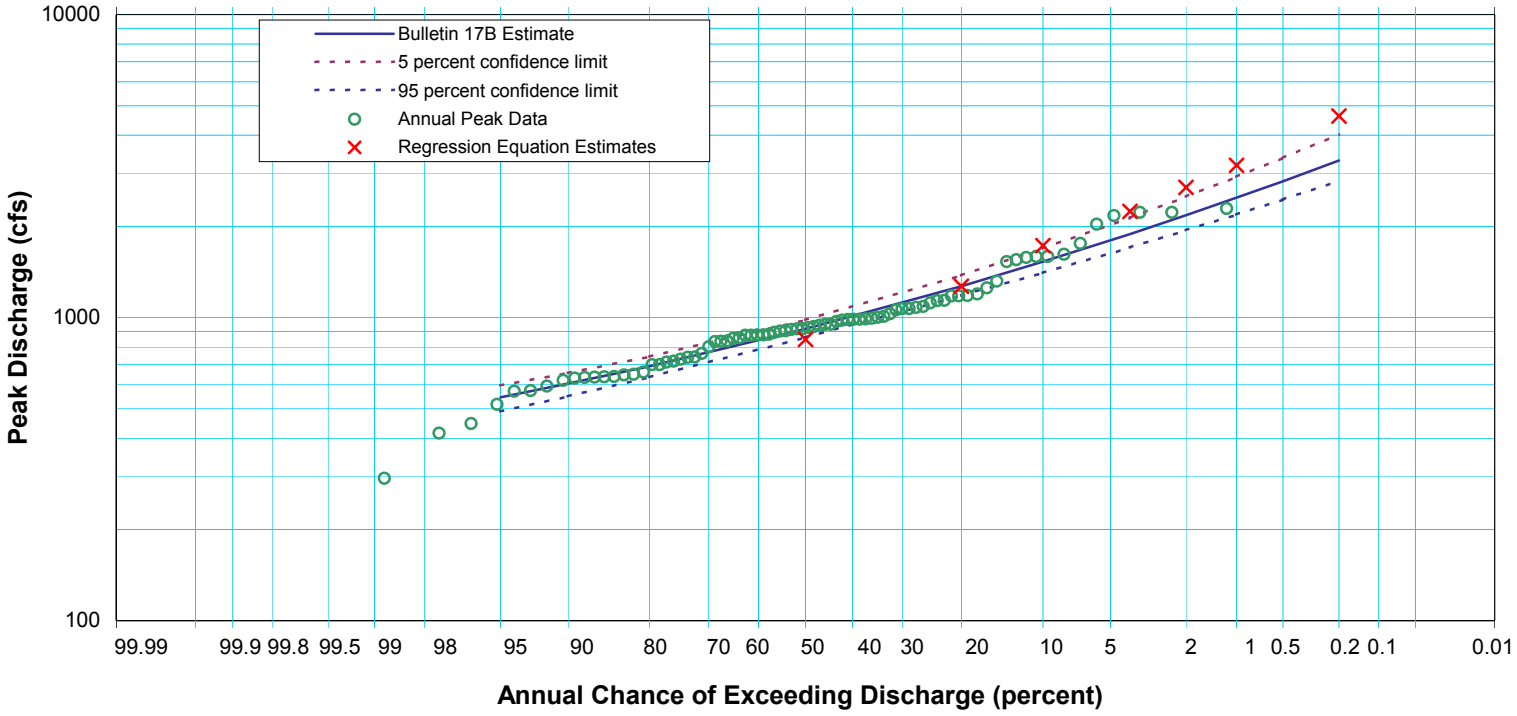

Figure 2.1.2. Bulletin 17B frequency estimates and regression analysis estimates for the Passaic River near Millington, NJ (USGS Station 01379000)

The Bulletin 17B frequency curve, the annual peak data and the 5- and 95-percent confidence limits for the Passaic River near Chatham are given in Figure 2.1.3. The same observations about the narrow 5 and 95 percent confidence limits are also applicable to Chatham. As shown in Exhibit 2, the 5 and 95 percent confidence limits for the 1-percent chance discharge (3,670 cfs) are 4,330 and 3,230 cfs, respectively.

Using procedures in Kite (1988), the 5- and 95-percent confidence limits are 4,690 and 2,870 cfs, respectively. The standard error of the 1-percent chance discharge for Chatham is 14.9 percent. Although not as extreme as Millington, the annual peak data for Chatham also exhibit a S-shaped pattern presumably due to upstream storage.

The mean, standard deviation and skew coefficient for the frequency curves shown in Figures 2.1.2 and 2.1.3 for Millington and Chatham are summarized in Table 2.1.2 and given in Exhibits 1 and 2.

Table 2.1.2. Summary of the mean, standard deviation and skew coefficient for Millington (01379000) and Chatham (01379500)

Station	Mean (log units)	Standard deviation (log units)	Skew coefficient
Millington (01379000)	2.9754	0.1582	0.457
Chatham (01379500)	3.1661	0.1499	0.463

The skew coefficients for Millington and Chatham are 0.46. The generalized skew coefficients in Special Projects Memo 480 of the USACE Hydrologic Engineering Center, "Generalized Skew Study for the State of New Jersey", December 1977, range from 0.3 to 0.4. Therefore, if the station skew coefficients for Millington and Chatham were weighted with the generalized skew from Special Projects Memo 480, the skew coefficient would decrease only slightly. Due to the urbanization and storage characteristics of the Passaic River watershed, the station skew is the most reasonable to use.

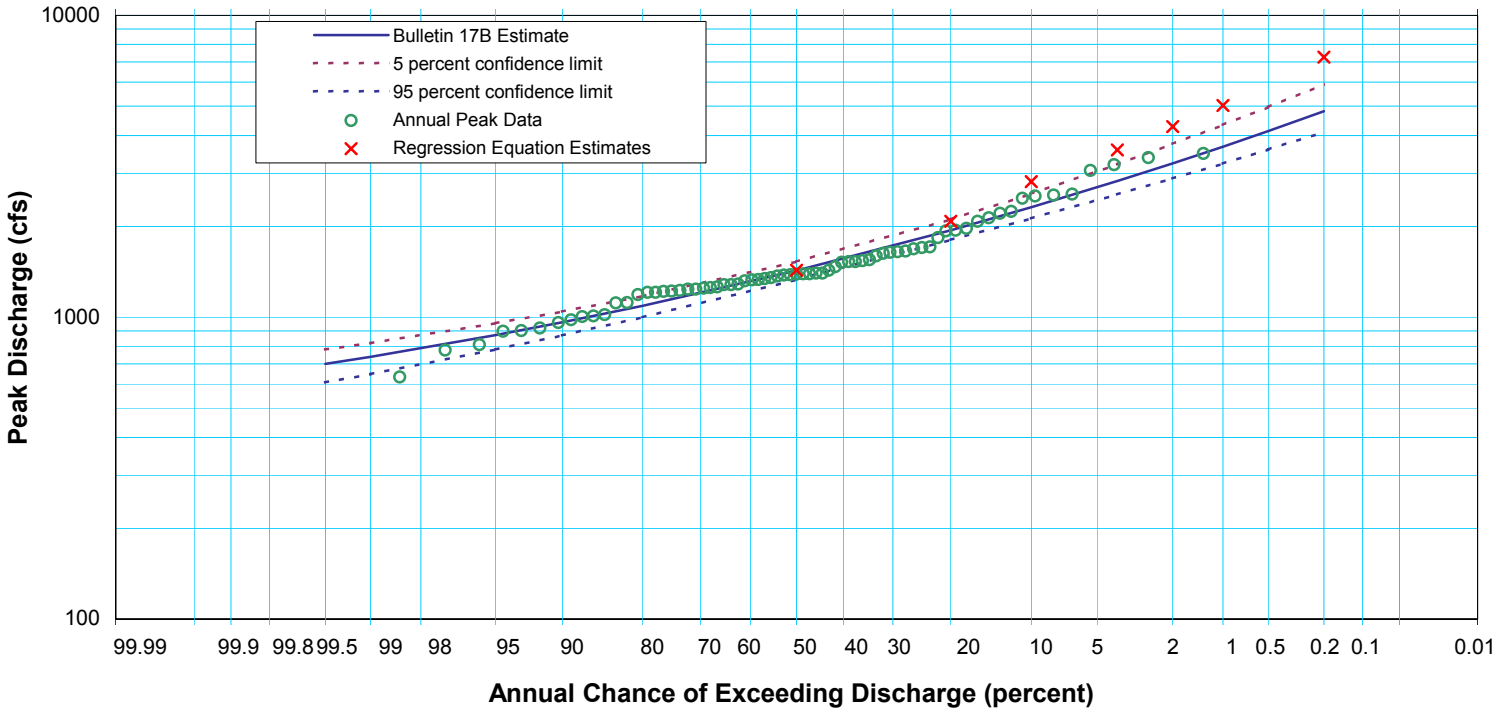

Figure 2.1.3. Bulletin 17B frequency estimates and regression estimates for the Passaic River near Chatham, NJ (USGS Station No. 01379500)

Also shown in Figures 2.1.2 and 2.1.3 are regression estimates from Special Report 38 published jointly the State of New Jersey and the USGS (Stankowski, 1974). The regression equations relate flood discharges for selected percent chance exceedances to watershed characteristics. The watershed characteristics include drainage area, channel slope, surface storage index, and impervious cover. The regression equation for the 1-percent chance discharge is:

$$Q_{1\%} = 136\ A^{0.84}\ S^{0.26}\ St^{-0.51}\ I^{0.14} \qquad (2)$$

where $Q_{1\%}$ is the 1-percent chance discharge, in cfs, A is the drainage area, in square miles, St is the surface storage index, in percentage of the drainage area, and I is the impervious cover, in percentage of the drainage area. The standard error of Equation 2 is 54 percent.

The regression estimates given in Figures 2.1.2 and 2.1.3 are based on 1990 impervious cover as determined by the USACE (1995). The flood discharges and watershed characteristics for Millington and Chatham are given in Table 2.1.3. The 50-, 20-, and 10-percent chance discharges from the regression equations generally are within the 5- and 95-confidence limits of the station frequency curves for Millington and Chatham shown in Figures 2.1.2 and 2.1.3, respectively. However, the more extreme flood discharges from the regression equations plot above the 5-percent confidence limit.

It is likely that the regression equations are not accurately accounting for the storage effects in the Passaic River watershed. The surface storage in the Passaic River watershed – especially in the Great Swamp upstream of the Millington gage - is among the highest of all gaging stations used in the regression analysis. Given there are 82 and 71 years of record for Millington and Chatham, the flood discharges based on the station data are considered most reasonable. For example, the standard error of the 1-percent chance discharge based on gaging station data is about 15 percent for Millington and Chatham while the standard error of the regression estimates is 54 percent.

Table 2.1.3. Summary of watershed characteristics and regression estimates for the Passaic River near Millington and Chatham, New Jersey.

Variable	**Millington**	**Chatham**
Drainage area, mi^2	55.4	100
Channel slope, ft/mi	30.3	11.4
Surface storage index, %	19.1	12.2
Impervious cover, %	19.37	19.93
50-percent chance discharge, cfs	847	1,430
20-percent chance discharge, cfs	1,270	2,080
10-percent chance discharge, cfs	1,730	2,810
4-percent chance discharge, cfs	2,240	3,580
2-percent chance discharge, cfs	2,690	4,280
1-percent chance discharge, cfs	3,180	5,030
0.2-percent chance discharge, cfs	4,630	7,260

A frequency analysis was also performed for the Passaic River at Little Falls as a check on the reasonableness of the 1-percent chance flood discharges for Millington and Chatham. The gaging station at Little Falls has 105 years of flood records. Figure 2.1.4 is a plot of the 1-percent chance discharges for all three stations compared to the USGS regression equations and 1-percent chance discharges provided in USACE (1995).

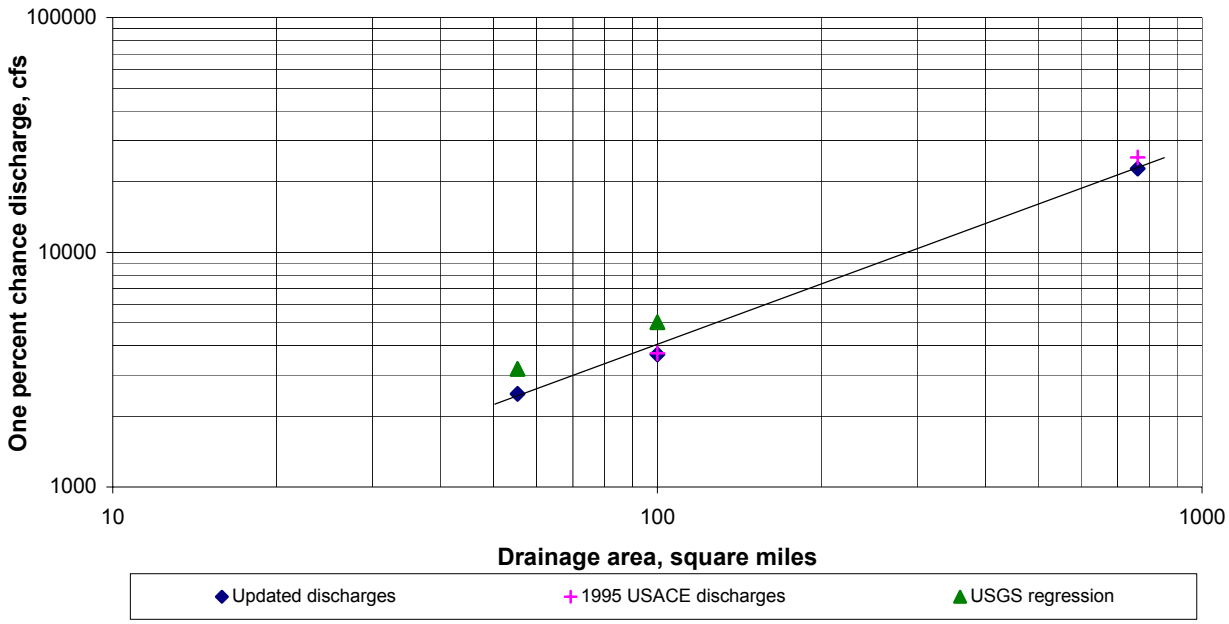

Figure 2.1.4. One percent chance flood discharges for the Passaic River in New Jersey (line drawn through updated discharges)

The slope of the line in Figure 2.1.4 is 0.86 and is consistent with the exponent on drainage area in Equation 2. The line in Figure 2.1.4 was drawn through the updated station data and indicates that the 1-percent chance discharges for Millington, Chatham and Little Falls are consistent with each other.

Summary

The flood discharges for Millington and Chatham based on station skew are considered reasonable for use in the flood control feasibility project. These discharges are considered more reasonable than those based on regression equations because of the long record of flood data. The standard error of the 1-percent chance discharge based on station data is about 15 percent while the regression equation standard error is 54 percent.

References

Interagency Advisory Committee on Water Data, 1982, *Guidelines For Determining Flood Flow Frequency*, Bulletin 17B of the Hydrology Subcommittee, Office of Water Data Coordination, U.S. Geological Survey, Reston, Virginia, 183 p.

Kite, G.W., 1988, *Frequency and Risk Analyses in Hydrology*, Water Resources Publications, Littleton, Colorado, 257 p.

Stankowski, S.J., 1974, *Magnitude and Frequency of Floods in New Jersey with Effects of Urbanization*: Special Report 38, State of New Jersey, Department of Environmental Protection, Division of Water Resources, 45 p.

Thomas, W.O., Jr., Lumb, A.M., Flynn, K.M., and Kirby, W.H., 1998, *User's Manual for Program PEAKFQ, Annual Flood Frequency Using Bulletin 17B Guidelines*: USGS Water Resources Investigations Report, Reston, Virginia, January 1998.

U.S. Army Corps of Engineers, 1992, *HEC-FFA Flood Frequency Analysis*: Hydrologic Engineering Center, Davis, California, May 1992.

U.S. Army Corps of Engineers, 1995, *General Design Memorandum, Passaic River Flood Damage Reduction Project, Appendix C – Hydrology & Hydraulics*: Volume I of II, New York District, September 1995.

Exhibit 1. Bulletin 17B frequency analysis for Passaic River near Millington, New Jersey (station 01379000).

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                    U. S. GEOLOGICAL SURVEY
                ANNUAL PEAK FLOW FREQUENCY ANALYSIS
                Following Bulletin 17-B Guidelines
                          Program peakfq
                     (Version 2.4, Apr, 1998)


          Station - 01379000  PASSAIC RIVER NEAR MILLINGTON NJ
                        1902 JAN 7 15:50:36

                I N P U T   D A T A   S U M M A R Y

            Number of peaks in record           =       82
            Peaks not used in analysis          =        0
            Systematic peaks in analysis        =       82
            Historic peaks in analysis          =        0
            Years of historic record            =        0
            Generalized skew                    =    0.650
            Standard error of generalized skew  =    0.550
            Skew option                         = STATION SKEW
            Gage base discharge                 =      0.0
            User supplied high outlier threshold =   --
            User supplied low outlier criterion  =   --
            Plotting position parameter         =     0.00

 *********  NOTICE  --  Preliminary machine computations.        *********
 *********  User responsible for assessment and interpretation.  *********

   WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                   0.0
   WCF198I-LOW OUTLIERS BELOW FLOOD BASE WERE DROPPED.          1    303.7
   WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS EXCEEDED HHBASE.      2886.7
  *WCF151I-WRC WEIGHTED SKEW REPLACED BY USER OPTION.     0.501     0.457  -1


          Station - 01379000  PASSAIC RIVER NEAR MILLINGTON NJ
                        1902 JAN 7 15:50:36


       ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

                      FLOOD BASE                  LOGARITHMIC
                 ----------------------  -------------------------------
                           EXCEEDANCE                STANDARD
                 DISCHARGE PROBABILITY     MEAN     DEVIATION     SKEW
                 -------------------------------------------------------
SYSTEMATIC RECORD       0.0     1.0000     2.9718      0.1659      0.144
BULL.17B ESTIMATE     303.7     0.9878     2.9754      0.1582      0.457
```

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	95-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	--	368.8	--	--	--
0.9900	--	401.3	--	--	--
0.9500	546.1	507.9	541.7	490.4	596.9
0.9000	605.3	577.9	602.1	549.7	656.5
0.8000	691.8	677.8	689.8	636.3	744.2
0.5000	919.1	928.5	919.1	859.3	982.2
0.2000	1270.0	1289.0	1275.0	1182.0	1379.0
0.1000	1529.0	1537.0	1541.0	1406.0	1689.0
0.0400	1886.0	1863.0	1916.0	1705.0	2136.0
0.0200	2176.0	2114.0	2226.0	1941.0	2509.0
0.0100	2486.0	2373.0	2565.0	2189.0	2917.0
0.0050	2820.0	2640.0	2938.0	2452.0	3366.0
0.0020	3302.0	3009.0	3491.0	2824.0	4028.0

Station - 01379000 PASSAIC RIVER NEAR MILLINGTON NJ
1902 JAN 7 15:50:36

I N P U T D A T A L I S T I N G

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
1904	2225.0		1960	853.0	
1905	2225.0		1961	1198.0	
1906	1551.0		1962	1179.0	
1922	699.0		1963	951.0	
1923	874.0		1964	638.0	
1924	1318.0		1965	636.0	
1925	894.0		1966	650.0	
1926	837.0		1967	801.0	
1927	631.0		1968	1070.0	
1928	874.0		1969	714.0	
1929	914.0		1970	980.0	
1930	718.0		1971	2170.0	
1931	595.0		1972	1070.0	
1932	834.0		1973	1090.0	
1933	1137.0		1974	990.0	
1934	1179.0		1975	1580.0	
1935	647.0		1976	858.0	
1936	2034.0		1977	950.0	
1937	761.0		1978	1010.0	
1938	1253.0		1979	1530.0	
1939	742.0		1980	740.0	
1940	1141.0		1981	621.0	
1941	634.0		1982	882.0	
1942	1593.0		1983	1030.0	
1943	1117.0		1984	1620.0	
1944	729.0		1985	575.0	
1945	928.0		1986	1080.0	
1946	987.0		1987	996.0	
1947	700.0		1988	640.0	
1948	944.0		1989	989.0	
1949	1757.0		1990	972.0	

1950	518.0	1991	661.0
1951	919.0	1992	1060.0
1952	1002.0	1993	935.0
1953	906.0	1994	918.0
1954	447.0	1995	416.0
1955	1184.0	1996	985.0
1956	878.0	1997	2290.0
1957	878.0	1998	836.0
1958	902.0	1999	1590.0
1959	571.0	2000	295.0

Explanation of peak discharge qualification codes

PEAKFQ CODE	WATSTORE CODE	DEFINITION
D	3	Dam failure, non-recurrent flow anomaly
G	8	Discharge greater than stated value
X	3+8	Both of the above
L	4	Discharge less than stated value
K	6 OR C	Known effect of regulation or urbanization
H	7	Historic peak

Station - 01379000 PASSAIC RIVER NEAR MILLINGTON NJ
1902 JAN 7 15:50:36

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	BULL.17B ESTIMATE
1997	2290.0	0.0120	0.0120
1904	2225.0	0.0241	0.0241
1905	2225.0	0.0361	0.0361
1971	2170.0	0.0482	0.0482
1936	2034.0	0.0602	0.0602
1949	1757.0	0.0723	0.0723
1984	1620.0	0.0843	0.0843
1942	1593.0	0.0964	0.0964
1999	1590.0	0.1084	0.1084
1975	1580.0	0.1205	0.1205
1906	1551.0	0.1325	0.1325
1979	1530.0	0.1446	0.1446
1924	1318.0	0.1566	0.1566
1938	1253.0	0.1687	0.1687
1961	1198.0	0.1807	0.1807
1955	1184.0	0.1928	0.1928
1934	1179.0	0.2048	0.2048
1962	1179.0	0.2169	0.2169
1940	1141.0	0.2289	0.2289
1933	1137.0	0.2410	0.2410
1943	1117.0	0.2530	0.2530
1973	1090.0	0.2651	0.2651
1986	1080.0	0.2771	0.2771
1968	1070.0	0.2892	0.2892
1972	1070.0	0.3012	0.3012
1992	1060.0	0.3133	0.3133

1983	1030.0	0.3253	0.3253
1978	1010.0	0.3373	0.3373
1952	1002.0	0.3494	0.3494
1987	996.0	0.3614	0.3614
1974	990.0	0.3735	0.3735
1989	989.0	0.3855	0.3855
1946	987.0	0.3976	0.3976
1996	985.0	0.4096	0.4096
1970	980.0	0.4217	0.4217
1990	972.0	0.4337	0.4337
1963	951.0	0.4458	0.4458
1977	950.0	0.4578	0.4578
1948	944.0	0.4699	0.4699
1993	935.0	0.4819	0.4819
1945	928.0	0.4940	0.4940
1951	919.0	0.5060	0.5060
1994	918.0	0.5181	0.5181
1929	914.0	0.5301	0.5301
1953	906.0	0.5422	0.5422
1958	902.0	0.5542	0.5542
1925	894.0	0.5663	0.5663
1982	882.0	0.5783	0.5783
1956	878.0	0.5904	0.5904
1957	878.0	0.6024	0.6024
1923	874.0	0.6145	0.6145
1928	874.0	0.6265	0.6265
1976	858.0	0.6386	0.6386
1960	853.0	0.6506	0.6506
1926	837.0	0.6627	0.6627
1998	836.0	0.6747	0.6747
1932	834.0	0.6867	0.6867
1967	801.0	0.6988	0.6988
1937	761.0	0.7108	0.7108
1939	742.0	0.7229	0.7229
1980	740.0	0.7349	0.7349
1944	729.0	0.7470	0.7470
1930	718.0	0.7590	0.7590
1969	714.0	0.7711	0.7711
1947	700.0	0.7831	0.7831
1922	699.0	0.7952	0.7952
1991	661.0	0.8072	0.8072
1966	650.0	0.8193	0.8193
1935	647.0	0.8313	0.8313
1988	640.0	0.8434	0.8434
1964	638.0	0.8554	0.8554
1965	636.0	0.8675	0.8675
1941	634.0	0.8795	0.8795
1927	631.0	0.8916	0.8916
1981	621.0	0.9036	0.9036
1931	595.0	0.9157	0.9157
1985	575.0	0.9277	0.9277
1959	571.0	0.9398	0.9398
1950	518.0	0.9518	0.9518
1954	447.0	0.9639	0.9639
1995	416.0	0.9759	0.9759
2000	295.0	0.9880	0.9880

Exhibit 2. Bulletin 17B frequency analysis for the Passaic River near Chatham, New Jersey (station 01379500).

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                    U. S. GEOLOGICAL SURVEY
                ANNUAL PEAK FLOW FREQUENCY ANALYSIS
                Following Bulletin 17-B Guidelines
                          Program peakfq
                     (Version 2.4, Apr, 1998)


            Station - 01379500  PASSAIC RIVER NEAR CHATHAM NJ
                         1902 JAN 7 16:13:17


                 I N P U T   D A T A   S U M M A R Y

             Number of peaks in record           =       71
             Peaks not used in analysis          =        0
             Systematic peaks in analysis        =       71
             Historic peaks in analysis          =        0
             Years of historic record            =        0
             Generalized skew                    =    0.660
             Standard error of generalized skew  =    0.550
             Skew option                         = STATION SKEW
             Gage base discharge                 =      0.0
             User supplied high outlier threshold =   --
             User supplied low outlier criterion  =   --
             Plotting position parameter         =     0.00


  *********  NOTICE  --  Preliminary machine computations.        *********
  *********  User responsible for assessment and interpretation.  *********

    WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                   0.0
    WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS EXCEEDED HHBASE.      3984.9
    WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.            539.3
   *WCF151I-WRC WEIGHTED SKEW REPLACED BY USER OPTION.     0.513     0.463  -1
1


            Station - 01379500  PASSAIC RIVER NEAR CHATHAM NJ
                         1902 JAN 7 16:13:17


         ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

                       FLOOD BASE                  LOGARITHMIC
                  ----------------------  ------------------------------
                             EXCEEDANCE                STANDARD
                  DISCHARGE PROBABILITY     MEAN     DEVIATION     SKEW
                  ---------------------------------------------------------
 SYSTEMATIC RECORD       0.0     1.0000     3.1661      0.1499      0.463
 BULL.17B ESTIMATE       0.0     1.0000     3.1661      0.1499      0.463
```

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	95-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	699.9	699.9	686.6	608.7	781.6
0.9900	739.6	739.6	727.9	648.2	821.5
0.9500	872.6	872.6	864.9	781.4	954.8
0.9000	961.6	961.6	956.0	871.0	1044.0
0.8000	1091.0	1091.0	1088.0	1001.0	1175.0
0.5000	1428.0	1428.0	1428.0	1333.0	1527.0
0.2000	1940.0	1940.0	1948.0	1803.0	2110.0
0.1000	2313.0	2313.0	2334.0	2125.0	2563.0
0.0400	2824.0	2824.0	2874.0	2550.0	3210.0
0.0200	3235.0	3235.0	3318.0	2882.0	3747.0
0.0100	3672.0	3672.0	3801.0	3228.0	4331.0
0.0050	4140.0	4140.0	4332.0	3593.0	4969.0
0.0020	4811.0	4811.0	5117.0	4107.0	5907.0

Station - 01379500 PASSAIC RIVER NEAR CHATHAM NJ
1902 JAN 7 16:13:17

I N P U T D A T A L I S T I N G

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
1904	3490.0		1966	1010.0	
1905	3198.0		1967	1320.0	
1906	1432.0		1968	2560.0	
1907	3062.0		1969	1640.0	
1908	1834.0		1970	1470.0	
1909	1598.0		1971	2540.0	
1910	2478.0		1972	1520.0	
1911	778.0		1973	3380.0	
1938	2137.0		1974	1400.0	
1939	1218.0		1975	2520.0	
1940	1334.0		1976	1240.0	
1941	1005.0		1977	1390.0	
1942	2244.0		1978	1540.0	
1943	1657.0		1979	1710.0	
1944	1210.0		1980	1370.0	
1945	1284.0		1981	899.0	
1946	1284.0		1982	1240.0	
1947	1116.0		1983	1380.0	
1948	1647.0		1984	1930.0	
1949	1941.0		1985	960.0	
1950	921.0		1986	1550.0	
1951	1255.0		1987	1210.0	
1952	1530.0		1988	1220.0	
1953	1392.0		1989	1330.0	
1954	902.0		1990	1190.0	
1955	1628.0		1991	1250.0	
1956	1392.0		1992	1700.0	
1957	1343.0		1993	1260.0	
1958	1383.0		1994	1970.0	

1959	1019.0	1995	633.0
1960	1353.0	1996	1400.0
1961	1530.0	1997	2080.0
1962	1687.0	1998	1290.0
1963	1226.0	1999	2210.0
1964	1119.0	2000	811.0
1965	980.0		

Explanation of peak discharge qualification codes

PEAKFQ CODE	WATSTORE CODE	DEFINITION
D	3	Dam failure, non-recurrent flow anomaly
G	8	Discharge greater than stated value
X	3+8	Both of the above
L	4	Discharge less than stated value
K	6 OR C	Known effect of regulation or urbanization
H	7	Historic peak

Station - 01379500 PASSAIC RIVER NEAR CHATHAM NJ
1902 JAN 7 16:13:17

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	BULL.17B ESTIMATE
1904	3490.0	0.0139	0.0139
1973	3380.0	0.0278	0.0278
1905	3198.0	0.0417	0.0417
1907	3062.0	0.0556	0.0556
1968	2560.0	0.0694	0.0694
1971	2540.0	0.0833	0.0833
1975	2520.0	0.0972	0.0972
1910	2478.0	0.1111	0.1111
1942	2244.0	0.1250	0.1250
1999	2210.0	0.1389	0.1389
1938	2137.0	0.1528	0.1528
1997	2080.0	0.1667	0.1667
1994	1970.0	0.1806	0.1806
1949	1941.0	0.1944	0.1944
1984	1930.0	0.2083	0.2083
1908	1834.0	0.2222	0.2222
1979	1710.0	0.2361	0.2361
1992	1700.0	0.2500	0.2500
1962	1687.0	0.2639	0.2639
1943	1657.0	0.2778	0.2778
1948	1647.0	0.2917	0.2917
1969	1640.0	0.3056	0.3056
1955	1628.0	0.3194	0.3194
1909	1598.0	0.3333	0.3333
1986	1550.0	0.3472	0.3472
1978	1540.0	0.3611	0.3611
1952	1530.0	0.3750	0.3750

1961	1530.0	0.3889	0.3889
1972	1520.0	0.4028	0.4028
1970	1470.0	0.4167	0.4167
1906	1432.0	0.4306	0.4306
1974	1400.0	0.4444	0.4444
1996	1400.0	0.4583	0.4583
1953	1392.0	0.4722	0.4722
1956	1392.0	0.4861	0.4861
1977	1390.0	0.5000	0.5000
1958	1383.0	0.5139	0.5139
1983	1380.0	0.5278	0.5278
1980	1370.0	0.5417	0.5417
1960	1353.0	0.5556	0.5556
1957	1343.0	0.5694	0.5694
1940	1334.0	0.5833	0.5833
1989	1330.0	0.5972	0.5972
1967	1320.0	0.6111	0.6111
1998	1290.0	0.6250	0.6250
1945	1284.0	0.6389	0.6389
1946	1284.0	0.6528	0.6528
1993	1260.0	0.6667	0.6667
1951	1255.0	0.6806	0.6806
1991	1250.0	0.6944	0.6944
1976	1240.0	0.7083	0.7083
1982	1240.0	0.7222	0.7222
1963	1226.0	0.7361	0.7361
1988	1220.0	0.7500	0.7500
1939	1218.0	0.7639	0.7639
1944	1210.0	0.7778	0.7778
1987	1210.0	0.7917	0.7917
1990	1190.0	0.8056	0.8056
1964	1119.0	0.8194	0.8194
1947	1116.0	0.8333	0.8333
1959	1019.0	0.8472	0.8472
1966	1010.0	0.8611	0.8611
1941	1005.0	0.8750	0.8750
1965	980.0	0.8889	0.8889
1985	960.0	0.9028	0.9028
1950	921.0	0.9167	0.9167
1954	902.0	0.9306	0.9306
1981	899.0	0.9444	0.9444
2000	811.0	0.9583	0.9583
1911	778.0	0.9722	0.9722
1995	633.0	0.9861	0.9861

Exhibit 3. Bulletin 17B frequency analysis for the Passaic River at Little Falls, New Jersey (station 01389500).

```
                    U. S. GEOLOGICAL SURVEY
                ANNUAL PEAK FLOW FREQUENCY ANALYSIS
                Following Bulletin 17-B Guidelines
                          Program peakfq
                     (Version 2.4, Apr, 1998)


          Station - 01389500  PASSAIC RIVER AT LITTLE FALLS NJ
                        1902 JAN 19 15:13:03

                I N P U T   D A T A   S U M M A R Y

            Number of peaks in record           =      106
            Peaks not used in analysis          =        1
            Systematic peaks in analysis        =      104
            Historic peaks in analysis          =        1
            Years of historic record            =      120
            Generalized skew                    =    0.672
            Standard error of generalized skew  =    0.550
            Skew option                         = STATION SKEW
            Gage base discharge                 =      0.0
            User supplied high outlier threshold =    18999.0
            User supplied low outlier criterion  =   --
            Plotting position parameter         =     0.00


 *********  NOTICE  --  Preliminary machine computations.        *********
 *********  User responsible for assessment and interpretation.  *********

 **WCF109W-PEAKS WITH MINUS-FLAGGED DISCHARGES WERE BYPASSED.       1
 **WCF113W-NUMBER OF SYSTEMATIC PEAKS HAS BEEN REDUCED TO NSYS =  104
   WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                  0.0
   WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.           2094.9
  *WCF161I-USER HIGH OUTLIER CRITERION REPLACES WRC.    18999.0    28376.1
   WCF165I-HIGH OUTLIERS AND HISTORIC PEAKS ABOVE HHBASE.  4  1    18999.0
  *WCF151I-WRC WEIGHTED SKEW REPLACED BY USER OPTION.     0.314     0.248  -1
   WCF002J-CALCS COMPLETED.  RETURN CODE =  2

          Station - 01389500  PASSAIC RIVER AT LITTLE FALLS NJ
                        1902 JAN 19 15:13:03


       ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

                     FLOOD BASE                  LOGARITHMIC
              ----------------------  ------------------------------
                         EXCEEDANCE                STANDARD
               DISCHARGE PROBABILITY     MEAN     DEVIATION     SKEW
              -------------------------------------------------------
SYSTEMATIC RECORD       0.0     1.0000     3.8871      0.1868      0.279
BULL.17B ESTIMATE       0.0     1.0000     3.8879      0.1867      0.248
```

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	95-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	2822.0	2850.0	2764.0	2430.0	3191.0
0.9900	3076.0	3099.0	3024.0	2674.0	3452.0
0.9500	3932.0	3938.0	3898.0	3508.0	4328.0
0.9000	4511.0	4508.0	4485.0	4078.0	4918.0
0.8000	5358.0	5344.0	5343.0	4914.0	5786.0
0.5000	7590.0	7557.0	7590.0	7077.0	8137.0
0.2000	11030.0	11000.0	11060.0	10210.0	12010.0
0.1000	13540.0	13530.0	13640.0	12400.0	15010.0
0.0400	16990.0	17040.0	17210.0	15300.0	19260.0
0.0200	19750.0	19870.0	20130.0	17570.0	22770.0
0.0100	22690.0	22890.0	23270.0	19950.0	26570.0
0.0050	25830.0	26130.0	26690.0	22440.0	30690.0
0.0020	30310.0	30790.0	31660.0	25950.0	36710.0

Station - 01389500 PASSAIC RIVER AT LITTLE FALLS NJ
1902 JAN 19 15:13:03

I N P U T D A T A L I S T I N G

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
-1882	19000.0	H	1949	9575.0	
1896	16700.0		1950	5086.0	
1898	10085.0		1951	12781.0	
1899	8499.0		1952	12414.0	
1900	10019.0		1953	8116.0	
1901	9868.0		1954	3744.0	
1902	23400.0		1955	10415.0	
1903	11489.0		1956	12045.0	
1904	32300.0		1957	6020.0	
1905	9386.0		1958	7054.0	
1906	6840.0		1959	4372.0	
1907	8757.0		1960	7837.0	
1908	9793.0		1961	8289.0	
1909	7209.0		1962	5716.0	
1910	9594.0		1963	4023.0	
1911	4143.0		1964	4193.0	
1912	10930.0		1965	4630.0	
1913	8135.0		1966	3630.0	
1914	8643.0		1967	5720.0	
1915	8471.0		1968	13500.0	
1916	6859.0		1969	5210.0	
1917	5293.0		1970	9110.0	
1918	7209.0		1971	8010.0	
1919	9262.0		1972	10300.0	
1920	12045.0		1973	10600.0	
1921	6970.0		1974	8910.0	
1922	7280.0		1975	9640.0	
1923	8351.0		1976	6390.0	
1924	11291.0		1977	8440.0	
1925	6768.0		1978	10100.0	

1926	6264.0		1979	10200.0
1927	9335.0		1980	9870.0
1928	6699.0		1981	4370.0
1929	5869.0		1982	4230.0
1930	-4386.0	K	1983	9740.0
1931	4722.0		1984	18400.0
1932	5411.0		1985	3090.0
1933	9120.0		1986	6150.0
1934	6402.0		1987	10300.0
1935	4720.0		1988	3670.0
1936	19200.0		1989	10700.0
1937	6148.0		1990	6020.0
1938	9944.0		1991	4160.0
1939	6411.0		1992	6410.0
1940	6363.0		1993	8700.0
1941	5824.0		1994	5960.0
1942	7750.0		1995	2900.0
1943	8173.0		1996	9440.0
1944	5834.0		1997	8180.0
1945	19500.0		1998	8950.0
1946	7789.0		1999	11600.0
1947	6236.0		1999	11600.0
1948	5971.0		2000	3210.0

Explanation of peak discharge qualification codes

PEAKFQ CODE	WATSTORE CODE	DEFINITION
D	3	Dam failure, non-recurrent flow anomaly
G	8	Discharge greater than stated value
X	3+8	Both of the above
L	4	Discharge less than stated value
K	6 OR C	Known effect of regulation or urbanization
H	7	Historic peak

Station - 01389500 PASSAIC RIVER AT LITTLE FALLS NJ
1902 JAN 19 15:13:03

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	BULL.17B ESTIMATE
1904	32300.0	0.0095	0.0083
1902	23400.0	0.0190	0.0165
1945	19500.0	0.0286	0.0248
1936	19200.0	0.0381	0.0331
-1882	19000.0	--	0.0413
1984	18400.0	0.0476	0.0502
1896	16700.0	0.0571	0.0597
1968	13500.0	0.0667	0.0692
1951	12781.0	0.0762	0.0787
1952	12414.0	0.0857	0.0882
1920	12045.0	0.0952	0.0977

1956	12045.0	0.1048	0.1072
1999	11600.0	0.1143	0.1167
1999	11600.0	0.1238	0.1262
1903	11489.0	0.1333	0.1357
1924	11291.0	0.1429	0.1452
1912	10930.0	0.1524	0.1548
1989	10700.0	0.1619	0.1643
1973	10600.0	0.1714	0.1738
1955	10415.0	0.1810	0.1833
1972	10300.0	0.1905	0.1928
1987	10300.0	0.2000	0.2023
1979	10200.0	0.2095	0.2118
1978	10100.0	0.2190	0.2213
1898	10085.0	0.2286	0.2308
1900	10019.0	0.2381	0.2403
1938	9944.0	0.2476	0.2498
1980	9870.0	0.2571	0.2593
1901	9868.0	0.2667	0.2688
1908	9793.0	0.2762	0.2783
1983	9740.0	0.2857	0.2878
1975	9640.0	0.2952	0.2973
1910	9594.0	0.3048	0.3068
1949	9575.0	0.3143	0.3163
1996	9440.0	0.3238	0.3258
1905	9386.0	0.3333	0.3353
1927	9335.0	0.3429	0.3448
1919	9262.0	0.3524	0.3543
1933	9120.0	0.3619	0.3638
1970	9110.0	0.3714	0.3733
1998	8950.0	0.3810	0.3829
1974	8910.0	0.3905	0.3924
1907	8757.0	0.4000	0.4019
1993	8700.0	0.4095	0.4114
1914	8643.0	0.4190	0.4209
1899	8499.0	0.4286	0.4304
1915	8471.0	0.4381	0.4399
1977	8440.0	0.4476	0.4494
1923	8351.0	0.4571	0.4589
1961	8289.0	0.4667	0.4684
1997	8180.0	0.4762	0.4779
1943	8173.0	0.4857	0.4874
1913	8135.0	0.4952	0.4969
1953	8116.0	0.5048	0.5064
1971	8010.0	0.5143	0.5159
1960	7837.0	0.5238	0.5254
1946	7789.0	0.5333	0.5349
1942	7750.0	0.5429	0.5444
1922	7280.0	0.5524	0.5539
1909	7209.0	0.5619	0.5634
1918	7209.0	0.5714	0.5729
1958	7054.0	0.5810	0.5824
1921	6970.0	0.5905	0.5919
1916	6859.0	0.6000	0.6014
1906	6840.0	0.6095	0.6110
1925	6768.0	0.6190	0.6205
1928	6699.0	0.6286	0.6300
1939	6411.0	0.6381	0.6395

1992	6410.0	0.6476	0.6490
1934	6402.0	0.6571	0.6585
1976	6390.0	0.6667	0.6680
1940	6363.0	0.6762	0.6775
1926	6264.0	0.6857	0.6870
1947	6236.0	0.6952	0.6965
1986	6150.0	0.7048	0.7060
1937	6148.0	0.7143	0.7155
1957	6020.0	0.7238	0.7250
1990	6020.0	0.7333	0.7345
1948	5971.0	0.7429	0.7440
1994	5960.0	0.7524	0.7535
1929	5869.0	0.7619	0.7630
1944	5834.0	0.7714	0.7725
1941	5824.0	0.7810	0.7820
1967	5720.0	0.7905	0.7915
1962	5716.0	0.8000	0.8010
1932	5411.0	0.8095	0.8105
1917	5293.0	0.8190	0.8200
1969	5210.0	0.8286	0.8295
1950	5086.0	0.8381	0.8390
1931	4722.0	0.8476	0.8486
1935	4720.0	0.8571	0.8581
1965	4630.0	0.8667	0.8676
1959	4372.0	0.8762	0.8771
1981	4370.0	0.8857	0.8866
1982	4230.0	0.8952	0.8961
1964	4193.0	0.9048	0.9056
1991	4160.0	0.9143	0.9151
1911	4143.0	0.9238	0.9246
1963	4023.0	0.9333	0.9341
1954	3744.0	0.9429	0.9436
1988	3670.0	0.9524	0.9531
1966	3630.0	0.9619	0.9626
2000	3210.0	0.9714	0.9721
1985	3090.0	0.9810	0.9816
1995	2900.0	0.9905	0.9911
1930	-4386.0	--	--

3.0 DEVELOPMENT AND CALIBRATION OF EXISTING CONDITIONS HYDROLOGIC MODEL

The existing conditions hydrologic model for the current study was based on the modeling of the Passaic River above Chatham, NJ, carried out by the USACE to establish baseline conditions for a study of the basin wide impacts of the Passaic River Flood Damage Reduction Project. That study focussed on problems in the lower watershed and the modeling of the Upper Passaic River was carried out only to provide boundary conditions for the dynamic modeling used for the evaluation of alternatives contemplated for the protection of downstream damage centers. The basin-wide modeling effort is described in the Hydrologic Modeling Appendix of the 1995 General Design Memorandum.

3.1 Evaluation of the 1995 Hydrologic Model

In October, 2001, engineers from Michael Baker Jr. Inc. obtained the HEC-1 models of the Upper Passaic River developed for the 1995 USACE Study of the entire Passaic River watershed and converted them to the HEC-HMS format.

Output was verified and the link-node structure of the model was reviewed to see if it provided an adequate representation of flood processes in the upper watershed.

The USACE model included 13 subsheds upstream of the USGS gage at Chatham, New Jersey. The base map used for the study was not recovered, but the subsheds are shown on a schematic map reproduced as Figure C-29 in the Appendix of the 1995 study. These boundaries were compared with the Hydrologic Unit Code watersheds delineated in NJDEP shape files (DEPHUC14), which were delineated from 1:24,000 scale 7.5 minute USGS quadrangles developed by the NJ District USGS.

Since the earlier Corps study was not done in a GIS framework, there are a number of points where the boundaries do not coincide. But while there is a significant variation between the areas of some NJDEP and COE subbasins, there is a very good level of agreement between the total watershed areas and the areas upstream of the Millington stream gage for the two sets of data, as shown in Table 3.2.1, in the following section.

A concern prior to starting work was that the Corp's 1995 model might not include storage effects in the area upstream of Millington (i.e. the Great Swamp). However, a detailed review of the original hydrologic model indicated that a storage node was included to account for the attenuation of peak runoff in the upper catchment. The storage-discharge values associated with this node compared well to ArcView-based calculations of storage volumes in the watershed upstream of the Millington gage made as part of this study.

Peak flow attenuation in the watershed upstream of Millington is modeled as a lumped parameter process, using one discharge-volume relationship at node 2104 to represent the entire basin. While a more spatially distributed representation of storage would be better

in a model for management of the upper watershed, this is an appropriate representation for a study of downstream flooding.

Because the area of the watershed is correct, and an adequate parameterization of the hydrologic processes within the upper catchment was incorporated, it was confirmed that the original model would be a valid starting point for the present effort. Modifications to the model developed from the previous study are described in the following sections.

3.2 Development of the existing conditions HEC-HMS model

The HEC-HMS model was modified to reflect changes in the study area since 1995, and to better permit the evaluation of the proposed alternatives. The new existing conditions model utilizes 2000 as the base year. The key inputs and parameters of the model are described in the following sections.

3.2.1. Subwatershed delineation

The boundaries of the 13 subwatersheds in the 1995 HEC-1 model were never fixed in a geo-referenced system. Boundaries for the 14 subbasins developed for the present study were fixed in ArcView with reference to subareas shown in DEPHUC14 shapefiles obtained from the NJDEP web site. A comparison of the areas measured for the NJDEP, previous study HEC-1, and present study HEC-HMS watersheds is shown in Table 3.2.1

Table 3.2.1 Comparison of 1995 HEC-1 model and NJDEP-delineated subwatersheds

Basin ID	Subwatershed areas, sq. mi.			Agreement of HEC 1 and HEC-HMS models
	NJ DEP USGS H.U.C.basins (1)	USACE HEC-1 model basins (2)	New HEC-HMS model basins (1)	
Watershed upstream of Millington				
10	4.02	3.27	3.44	105.05%
15	3.87	5.56	5.23	94.04%
20	4.85	3.97	4.35	109.57%
30	23.37	25.8	23.06	89.39%
40	13.32	12.6	13.50	107.15%
50	2.05	1.33	1.66	124.95%
60	2.87	2.87	2.87	100.15%
Sub-total	54.37	55.40	54.11	97.68%
Watershed downstream of Millington				
70	2.84	2.5	2.84	113.45%
80	19.05	20.5	19.30	94.14%
90	13.25	11.8	12.75	108.04%
100	3.27	3.4	3.78	111.07%
110	4.45	4.6	4.45	96.71%
(3) 111	3.04	--	--	--
120	1.67	1.8	1.67	92.98%
Total	101.94	100.00	98.90	98.90%

(1) Areas determined in ArcView.

(2) Areas from 1995 H&H Design Memorandum - Fig. C29 could not be geo-referenced

(3) Area downstream of Chatham gage was not included in HEC-HMS model.

The area represented by subshed 90 in the 1995 HEC-1 model was subdivided into subsheds 91 and 92 for this study to permit the allow comparison of existing and alternative proposed conditions in the study area. Subsheds 91 and 92 are further subdivided for interior flood hydrograph modeling to permit the identification of their area that is separated from the Passaic River floodplain by Mountain Avenue and Passaic Valley Roads. In the HEC-HMS model, direct runoff from this 1,024 acre area is accounted for in the runoff from the larger basins in which they are located. Output from the HEC-HMS model is used for boundary conditions in the HEC-IFH modeling of the area, as described in Section 6.

3.2.2. Hydrologic model structure

The model topology or linkage is shown in Figure 3.2.1; the subwatershed boundaries are shown in Figure 3.2.2.

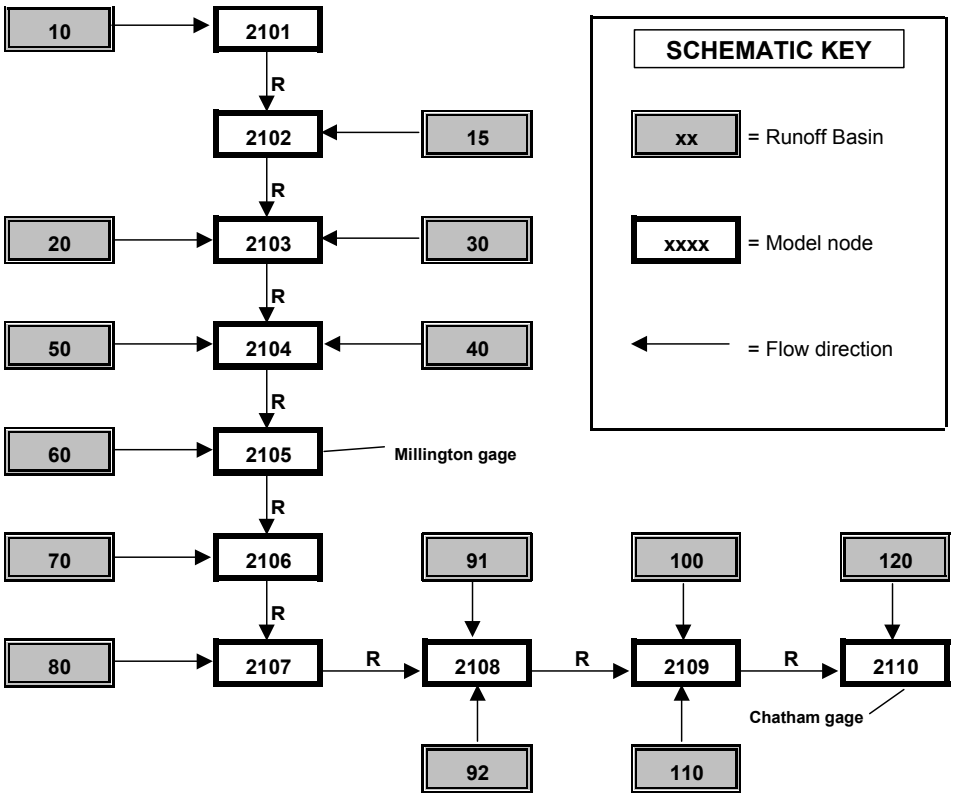


Figure 3.2.1. HEC-HMS subcatchment and channel linkage topology

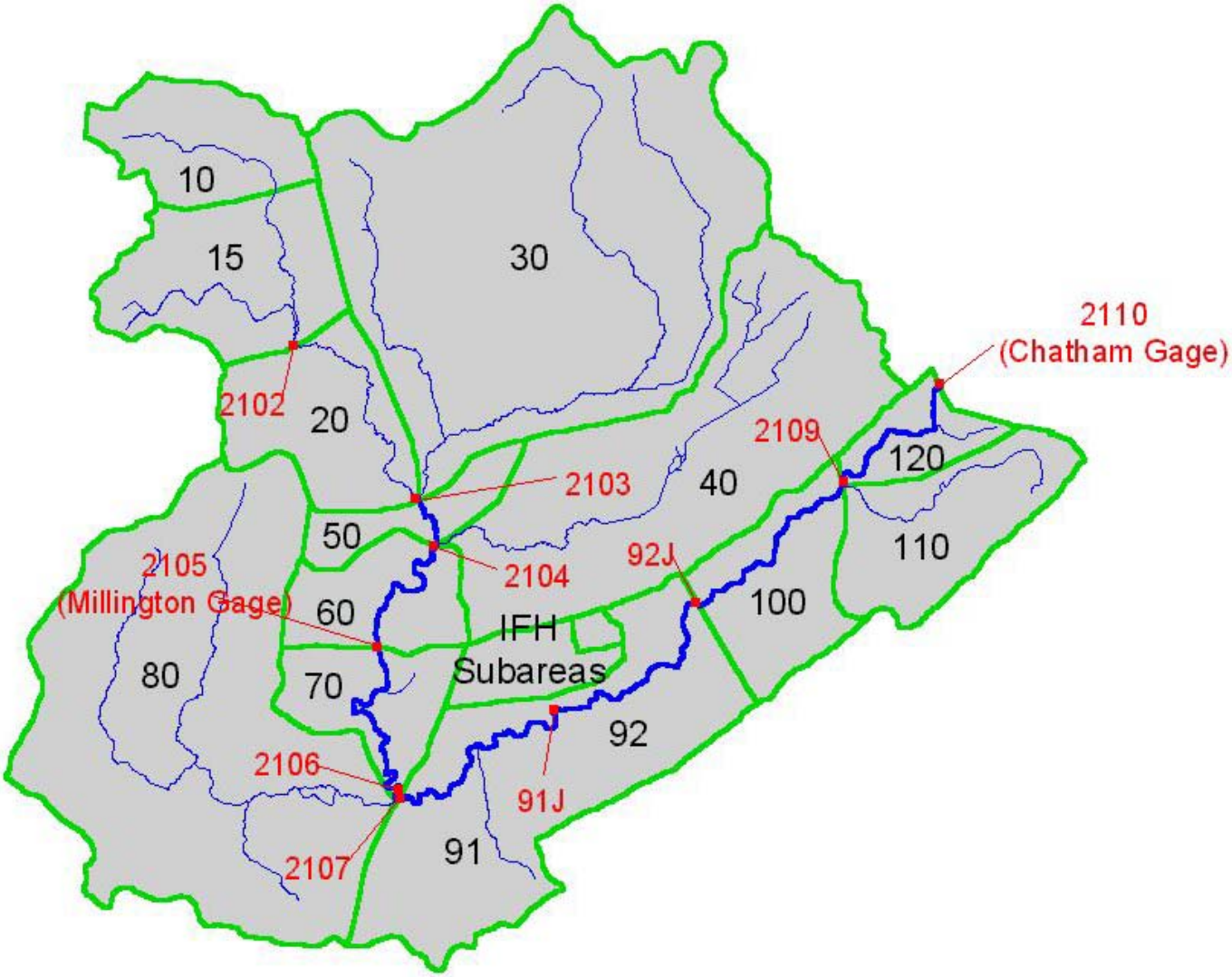


Figure 3.2.2. HEC-HMS nodes for existing conditions model

3.2.3. Direct Runoff Parameters

The key parameters used for simulating direct runoff from each subwatershed in the HEC-HMS model are shown in Table 3.2.2, below. The initial parameter shown for each basin is the parameter as determined from considering its significance in the context of physical hydrology. The second parameter is the value after adjustment for calibration, as described in further detail in section 3.3. Each of these parameters is discussed in further detail below.

Table 3.2.2. Direct Runoff Modeling Parameters (before and after calibration)

HEC-HMS Subbasin	Drainage area, mi2	Time of concentration, in hours		Clark Storage Coefficient, in hours		SCS Curve Number or "RCN" Dimensionless	
		Original estimate (1)	Calibration Value	Original estimate (1)	Calibration Value	HYDPAR Estimate	Calibration Value
0010	3.44	1.50	1.24	3.90	6.15	77	69
0015	5.23	1.90	1.63	4.40	7.02	70	63
0020	4.35	1.70	1.53	6.00	5.00	80	72
0030	23.06	6.00	3.00	17.50	17.71	76	68
0040	13.50	6.00	7.00	15.00	23.80	79	71
0050	1.66	1.40	0.91	8.00	8.20	81	75
0060	2.87	1.50	0.78	11.00	17.40	76	68
0070	2.84	1.50	1.61	9.00	8.70	79	71
0080	19.30	6.00	1.80	17.00	33.40	75	70
0091	7.71	2.50	3.36	9.00	18.80	78	72
0092	5.04	1.80	2.07	7.00	18.30	78	70
0100	3.78	1.50	1.74	6.00	19.20	76	70
0110	4.45	1.90	1.21	5.00	17.00	77	71
0120	1.67	1.40	1.11	3.80	22.99	65	64

1) Tc and R parameters from 1995 COE study - or estimated from regression based on area as required

3.2.3.1. Clark Unit Hydrograph parameters

Following the 1995 HEC-1 model, the 2-parameter Clark unit hydrograph was used to estimate direct runoff from each subcatchment. The Clark unit hydrograph is described by two parameters: the time of concentration, T_c, and the Storage Coefficient, R, both measured in hours. As described in Section 3.3, values of these two parameters were adjusted during the calibration process. For subbasins with no change in area, the 1995 HEC-1 values were used; for the new basins, and those with significant changes in area, the precalibration values were estimated from a curves relating T_c and R with catchment area, developed by the COE for the 1995 study. These values are listed in the third and fifth column of Table 3.2.2 for all 14 subbasins in current model.

The values of T_c selected for these parameters in the Upper Passaic River HEC-1 model are plotted against values measured for USGS gages with in the entire Passaic River watershed in Figure 3.2.3; values of R selected for these watersheds are compared to values measured for the entire watershed in Figure 3.2.4.

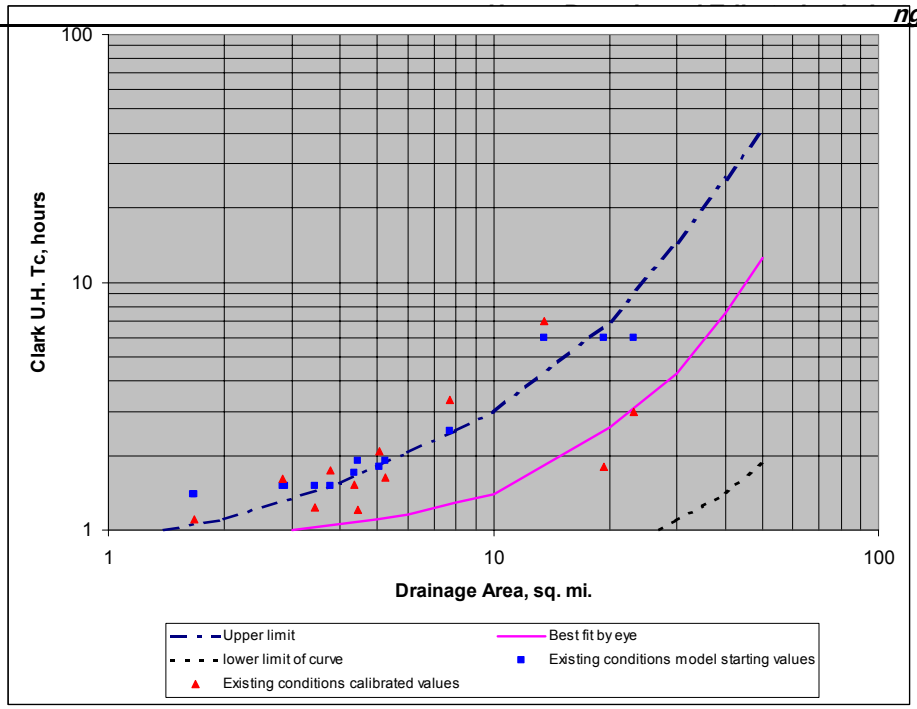


Figure 3.2.3. Tc values for the Passaic River watershed (total watershed curves based on Fig. C-43 of Appendix C, Passaic River Flood Control Feasibility Study, COE, 1995)

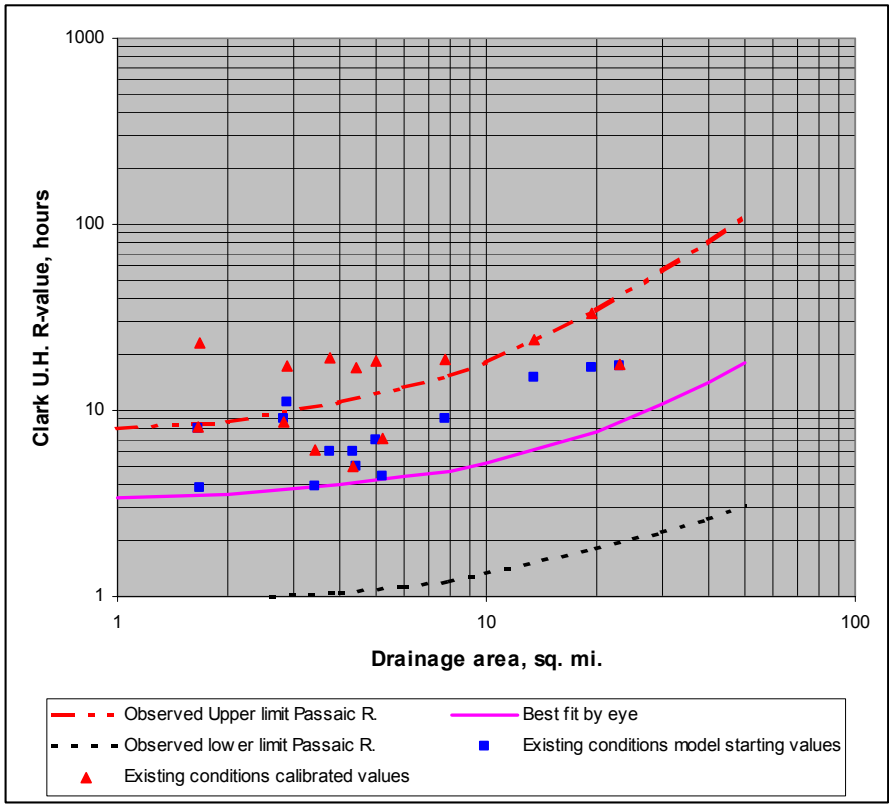


Figure 3.2.4. R-values for the Passaic River watershed (total watershed curves based on Fig. C-44 of Appendix C, Passaic River Flood Control Feasibility Study, COE, 1995)

3.2.3.2. SCS Runoff Curve Numbers

As in the previous study, the effect of landuse on direct runoff was parameterized in terms of the SCS Runoff Curve Number (RCN) in HEC-HMS, with RCN ranging from 0 for completely porous land surfaces to 100 for impervious surfaces. Current landuse in the watershed was characterized from the inspection of 1995 1:40,000-scale color aerial photography from the NJDEP, master plans for communities within the watershed, and referencing the previous two Corps of Engineers studies.

Figure 3.2.5 shows the boundaries of the communities impacting the study watershed. Table 3.23 summarizes key areal statistics for each of these communities that can be used to characterize their impact on the watershed.

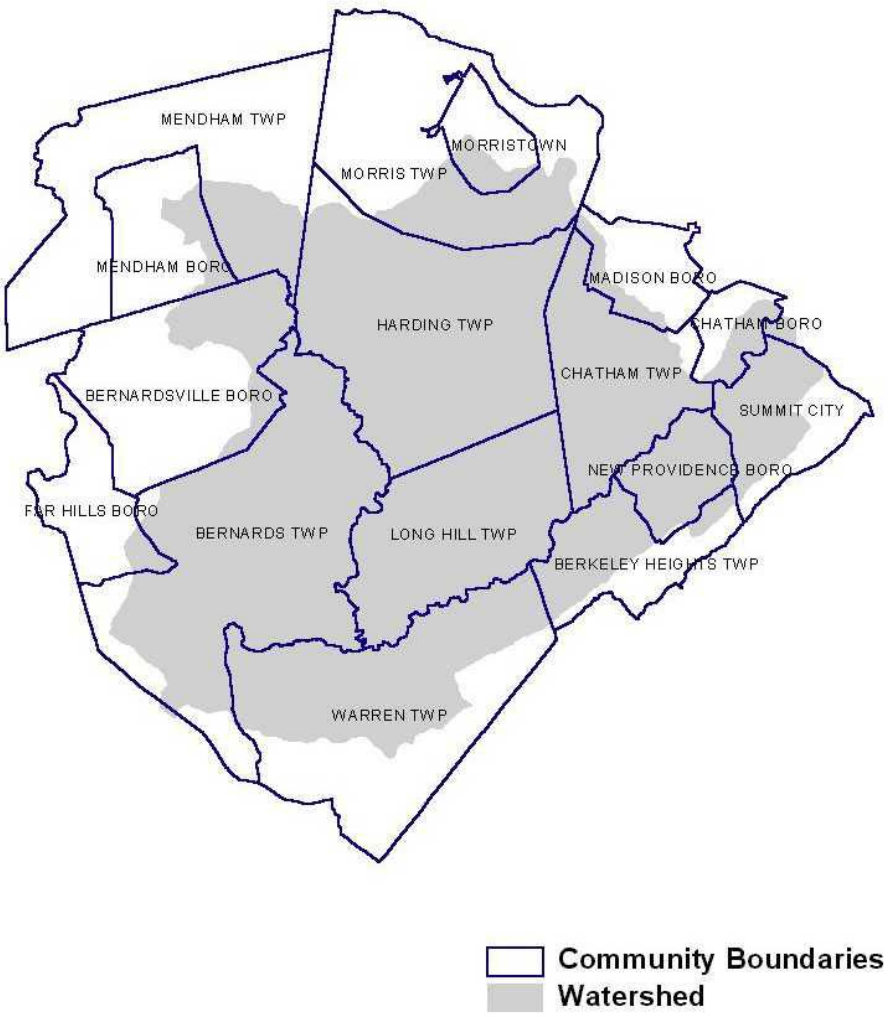


Figure 3.2.5. Communities impacting the Upper Passaic River watershed

Table 3.2.3. Communities partially or wholly within the Upper Passaic River watershed

Municipality	County	Total acreage of community	1990 Population	Acreage in watershed	Percent in Watershed	Percent of watershed	Plan coverage
MORRIS TWP	MORRIS	10,088	19,952	2,854	28.3%	4.5%	1
MENDHAM TWP	MORRIS	11,462	4,537	1,952	17.0%	3.1%	1
MORRISTOWN TOWN	MORRIS	1,941	16,189	600	30.9%	0.9%	1
MENDHAM BORO	MORRIS	3,858	4,890	797	20.7%	1.2%	1
HARDING TWP	MORRIS	13,109	3,640	12,794	97.6%	20.0%	1, 2
MADISON BORO	MORRIS	2,673	15,850	640	23.9%	1.0%	1
CHATHAM TWP	MORRIS	5,988	9,361	5,731	95.7%	9.0%	1, 4
BERNARDSVILLE BORO	SOMERSET	8,309	6,597	2,552	30.7%	4.0%	2,6,7,8
CHATHAM BORO	MORRIS	1,556	8,007	703	45.2%	1.1%	1
BERNARDS TWP	SOMERSET	15,483	17,199	13,689	88.4%	21.4%	1,3,6,7,8
SUMMIT CITY	UNION	3,867	19,757	2,477	64.0%	3.9%	1
FAR HILLS BORO	SOMERSET	3,121	657	348	11.2%	0.5%	1,2,6,7,8
NEW PROVIDENCE BORO	UNION	2,357	11,439	2,325	98.6%	3.6%	1
LONG HILL TWP	MORRIS	7,767	7,826	7,767	100.0%	12.1%	1, 5
BERKELEY HEIGHTS TWP	UNION	4,002	11,980	2,454	61.3%	3.8%	1
WARREN TWP	SOMERSET	12,524	10,830	7,349	58.7%	11.5%	2,6,7,8

1) USDA Great Swamp Hydrologic Unit Area Project, Final Report. Executive Summary and Recommendations, April 1996
2) Reexamination of the Harding Township Master Plan & Redevelopment Regulations
Harding Township Development Board, Adopted May 22, 2000.
3) Master Plan Reexamination Report, Bernards Township, Somerset County, New Jersey. December 1996
4) The Chatham Township Master Plan. Adopted October 18, 1999.
5) 1995 Master Plan. Long Hill Township. Morris County, New Jersey. 1995. Prepared by the Long Hill Township Planning Board.
6) Somerset County Parks, Recreation and Open Space Master Plan. Greenway to the Future. June 1994.
7) Somerset County Parks, Recreation and Open Space Master Plan Update. The Somerset County Park Service System for the 21st Century. December, 2000.
8) Somerset County Master Plan prepared by the Somerset County Planning Board. 1997.

Use was made of work done by the Corps of Engineers in previous studies. The RCN values in their 1987 General Design Memorandum to characterize land use in the Upper Passaic River watershed were developed in HYDPAR from aerial photography dated July 1978. At that time, development in the Great Swamp area was checked against 1940 aerial photography.

The RCN values used to characterize existing land use conditions (1990) in the 1995 model were determined by updating HYDPAR grid cells with reference to then-current master plans of the municipalities within the watershed and then-current aerial photography. The RCN values used to characterize land use in the watershed in 2050 were based on planned full basin development.

The 1978 aerial photography mosaic used by the Corps of Engineers to define 1990 existing conditions in the previous study was compared with 1995 color aerial photography to evaluate changes in the watershed in the intervening years.

As was expected, no unanticipated growth was noted since a great deal of the study watershed is either near full development density or is in parkland protected from any further development by the Federal Government Great Swamp National Wildlife Refuge.

Since no changes in planned growth rates were noted, the initial values of RCN parameters for the year 2000 base conditions model were developed for this study by interpolating between the previously determined values for existing and ultimate development (2050) values for each subbasin.

This was done by straight line interpolation between the 1978-2050 and 1990-2050 intervals. Table 3.2.4 compares the values of RCN for current, year 2000 conditions obtained from both of these methods. Inspection shows that there is not a significant difference between the two approaches.

The RCN values determined from the 1990-2050 interpolation method were used in the pre-calibration model for the study site. These values were allowed to vary by up to 10% during the calibration procedure, as described in Section 3.3.

Table 3.2.4. Interpolation of RCN values for the year 2000.

sub-basins	Estimated RCN values			Interpolated 2000 from		difference
	1978 (1)	1990 (2)	2050 (3)	1978-2050	1990-2050	(4)
0010	76.7	77.1	78	77	77	0.0%
0015	67.6	69.0	73	69	70	1.4%
0020	79.4	80.1	80	80	80	0.0%
0030	75.4	75.8	77	76	76	0.0%
0040	78.9	79.1	79	79	79	0.0%
0050	79.8	81.0	81	80	81	1.2%
0060	74.7	75.9	78	76	76	0.0%
0070	78.5	78.9	80	79	79	0.0%
0080	73.3	74.3	78	75	75	0.0%
0090	76.4	77.2	80	78	78	0.0%
0100	75.8	76.1	77	76	76	0.0%
0110	76.4	76.7	77	77	77	0.0%
0120	63.0	63.8	67	64	64	0.0%

(1) Archived data - Routing slip 19 June 1991: "Determination of Curve Numbers for Passaic Sub-basins"
(2) Archived data - Routing slip 19 June 1991: "Determination of Curve Numbers for Passaic Sub-basins", rounded-off values coincide with existing conditions HEC-1 model input files.
(3) Values from input files for HEC-1 model of future conditions, provided by the COE.
(4) Comparison of RCN values interpolted between 1978-2050 and 1990-2050 after round-off

3.2.4. Flood Routing parameters.

Watershed 90 was subdivided and an additional storage node was added along the reach of the Passaic River adjacent to the study site. The routing parameters for these nodes account for floodplain storage as determined from field surveys of the channel and floodplain conducted in 2000. These parameters will be changed to reflect the impact of any levee alternatives considered for the study reach.

The storage-discharge relations (SV/SQ) for the channel routing were developed from the new HEC-RAS model of the Upper Passaic River developed for this study. Storage

volumes between the Stirling-Plainfield Road and the Passaic Valley Road were calculated by subtracting the cumulative volumes at the respective sections, output in HEC-RAS tables for a range of flows between 500 and 5,000 cfs. These calculations are summarized in the following table.

Table 3.2.5. Storage-discharge calculations for channel routing on the Passaic River

Q, in cfs, for storage calculation	W.S. Elev from HEC-RAS	Flow Area from HEC-RAS, sq. ft.	Top Width from HEC-RAS, ft.	Cumulative Channel Length, ft.	Cumulative Channel Volume (ac ft)	Volume in routing reach (ac-ft)
Routing between Sections 188 & 223						
500	207.74	447	131	115,214	16,767	413
1000	209.67	620	221	115,214	17,604	738
1500	211.08	746	261	115,214	18,277	1,048
2000	212.06	835	283	115,214	19,050	1,315
2500	213.01	920	304	115,214	19,993	1,607
3000	213.82	993	323	115,214	20,927	1,889
3500	214.54	1,057	346	115,214	21,842	2,157
4000	215.14	1,111	370	115,214	22,824	2,412
4500	215.78	1,168	467	115,214	23,908	2,700
5000	216.49	3,182	782	115,214	24,964	2,999
Routing between Section 223 & 245						
500	209.47	288	72	132,821	17,110	342
1000	210.85	413	146	132,821	18,681	1,077
1500	211.99	1,079	745	132,821	20,159	1,882
2000	212.85	3,117	3,263	132,821	21,670	2,621
2500	213.64	5,695	3,275	132,821	23,425	3,432
3000	214.39	8,151	3,290	132,821	25,138	4,211
3500	215.09	10,449	3,307	132,821	26,821	4,978
4000	215.69	12,464	3,322	132,821	28,491	5,667
4500	216.36	14,689	3,335	132,821	30,325	6,416
5000	217.04	16,970	3,346	132,821	32,113	7,148

3.3 Model Calibration

Preliminary values for the key hydrologic parameters of the existing-conditions HEC-RAS model was chosen as described in Section 3.2. These values were adjusted by calibration in a manner consistent with normal engineering practice, as described in the following sections.

3.3.1 Database analysis

The NOAA precipitation gage and USGS stream gage databases were searched to identify a set of coincident rainfall and runoff records that could be divided into subsets for calibration and subsequent output verification. Table 3.3.1.1, below, summarizes the data available for each of the gages in the study area for which NOAA maintains records.

The last four columns summarize data availability for the October 1996 and September 1999 extreme storm events referenced in the Scope of Work.

Table 3.3.1.1 Data available for precipitation gage sites in the study area.

Station & Map ID		Period of Record		COOPID	Latitude/Longitude	County	State Plain Coords.		ppt for 9/17/99		ppt for 10/19/96	
							North (y)	East (x)	Avail?	tot. in.	Avail?	tot. in.
1	Boonton 1 SE	01 Aug 1948	02 Oct 1998	280907	40°54'N / 74°24'W	Morris	752781	519767	No	No	No	No
2	**Bound Brook 2 W**	**06 Sep 1956**	**Present**	**280927**	**40°34'N / 74°35'W**	**Somerset**	**631344**	**468974**	**Yes**	**7.5**	**Yes**	**5.9**
3	Rahway	01 Aug 1948	Present	287393	40°36'N / 74°15'W	Union	643575	561542	No	No	No	No
4	Springfield	01 Aug 1948	Present	288423	40°42'N / 74°20'W	Union	679949	538334	No	No	No	No
5	Watchung	01 Aug 1948	Present	289271	40°40'N / 74°25'W	Union	667773	515241	No	No	No	No
6	Chatham	01 Aug 1948	19 Nov 1965	281590	40°45'N / 74°22'W	Morris	698147	528141	No	No	No	No
7	Chatham 2 W	01 Apr 2000	Present	281592	40°45'N / 74°25'W	Morris	698131	515212	No	No	No	No
8	**Long Valley**	**01 Aug 1948**	**Present**	**285003**	**40°47'N / 74°47'W**	**Morris**	**710390**	**413668**	**Yes**	**4.1**	No	No
9	Milton	01 Aug 1948	31 Mar 1972	285597	41°01'N / 74°32'W	Morris	795270	482927	No	No	No	No
10	Morris Plains 1 W	01 Aug 1948	01 Jan 1992	285769	40°50'N / 74°30'W	Morris	728474	492125	No	No	No	No
34	Morristown	01 Mar 1962	Present	54738*	40°48'N / 74°25'W	Morris	716346	515195	No	No	No	No
11	**Oak Ridge Reservoir**	**01 Aug 1948**	**Present**	**286460**	**41°02'N / 74°30'W**	**Morris**	**801340**	**492125**	**Yes**	**5.6**	No	No
12	Pompton Plains	30 Oct 1956	Present	287204	40°58'N / 74°17'W	Morris	777127	551956	No	No	No	No
13	**Pottersville 2 NNW**	**01 Mar 1968**	**Present**	**287301**	**40°44'N / 74°44'W**	**Morris**	**692135**	**427465**	**Yes**	**7.1**	No	No
14	Split Rock Pond	01 Aug 1948	16 Sep 1998	288402	40°58'N / 74°28'W	Morris	777054	501330	No	No	No	No
15	West Wharton	01 Apr 1959	26 Nov 1990	289608	40°54'N / 74°36'W	Morris	752781	464483	No	No	No	No
16	Bernardsville 2 E	01 Apr 1959	31 Mar 1979	280797	40°43'N / 74°32'W	Somerset	685979	482886	No	No	No	No
17	Blackwells Mills	01 Oct 1956	Present	280847	40°28'N / 74°35'W	Somerset	594916	468940	No	No	No	No
18	Far Hills 2 N	01 Nov 1979	Present	282900	40°42'N / 74°38'W	Somerset	679933	455158	No	No	No	No
19	ManVille	01 Aug 1948	21 Mar 1969	285197	40°33'N / 74°34'W	Somerset	625269	473600	No	No	No	No
20	ManVille 1 E	01 Jan 1983	Present	285202	40°33'N / 74°35'W	Somerset	625273	468969	No	No	No	No
21	Raritan	01 Jan 1967	Present	287427	40°34'N / 74°41'W	Somerset	631386	441194	No	No	No	No
22	Readington	01 Nov 1979	Present	287460	40°33'N / 74°44'W	Somerset	625348	427288	No	No	No	No
23	Seeley Mills	01 Nov 1979	Present	288000	40°40'N / 74°24'W	Somerset	667778	519864	No	No	No	No
24	Somerville 4 NW	01 Aug 1948	Present	288194	40°37'N / 74°39'W	Somerset	649538	450485	No	No	No	No
25	Clarksville	01 Mar 1973	01 Nov 1981	281703	40°15'N / 74°44'W	Union	516065	426999	No	No	No	No
26	**Cranford**	**01 Dec 1968**	**Present**	**282023**	**40°39'N / 74°18'W**	**Union**	**661754**	**547617**	**Yes**	**9.76**	No	No
27	Elizabeth	01 Aug 1948	13 Feb 1970	282644	40°40'N / 74°14'W	Union	667874	566096	No	No	No	No
28	Elizabethport	01 Aug 1948	31 May	282652	40°38'N / 74°12'W	Union	655761	575384	No	No	No	No
29	**Plainfield**	**01 Aug 1948**	**Present**	**287079**	**40°36'N / 74°24'W**	**Union**	**643492**	**519892**	**Yes**	**9.77**	No	No
30	Summit	18 Apr 1958	01 Jan 1988	288625	40°43'N / 74°23'W	Union	685998	524463	No	No	No	No
31	Westfield	01 Aug 1948	30 Jun 1961	289455	40°39'N / 74°21'W	Union	661726	533744	No	No	No	No
32	Clinton 2 N	01 Nov 1967	Present	281754	40°40'N / 74°55'W	Hunterdon	668036	376545	No	No	No	No
33	Columbia 2 N	01 Aug 1971	Present	281858	40°58'N / 75°07'W	Warren	777654	321835	No	No	No	No

Figure 3.3.1.1, below, shows the location of the precipitation gages listed in Table 3.3.1.1. The catchment area upstream of the Chatham gage is shown on the map for reference.

While NOAA reports data for over 30 precipitation gages in the area, the availability of complete year 15-minute precipitation data sets is highly problematic, as can be seen in Table 3.3.1.1. Most of the sites with 15-minute data are not in the immediate area of the study, reducing their suitability for calibration.

The Bound Brook precipitation gage is located south of the watershed, but its cumulative precipitation values correlate quite well with those of four other 15-minute recording gages surrounding the site, as shown in Table 3.3.1.2. This makes it possible to use observed hyetographs from Bound Brook, scaled to an appropriate depth, to represent precipitation at the site. Figure 3.3.1.2 shows the cumulative rainfall data for the September 1989 rainfall event for the gages summarized in Table 3.3.1.2.

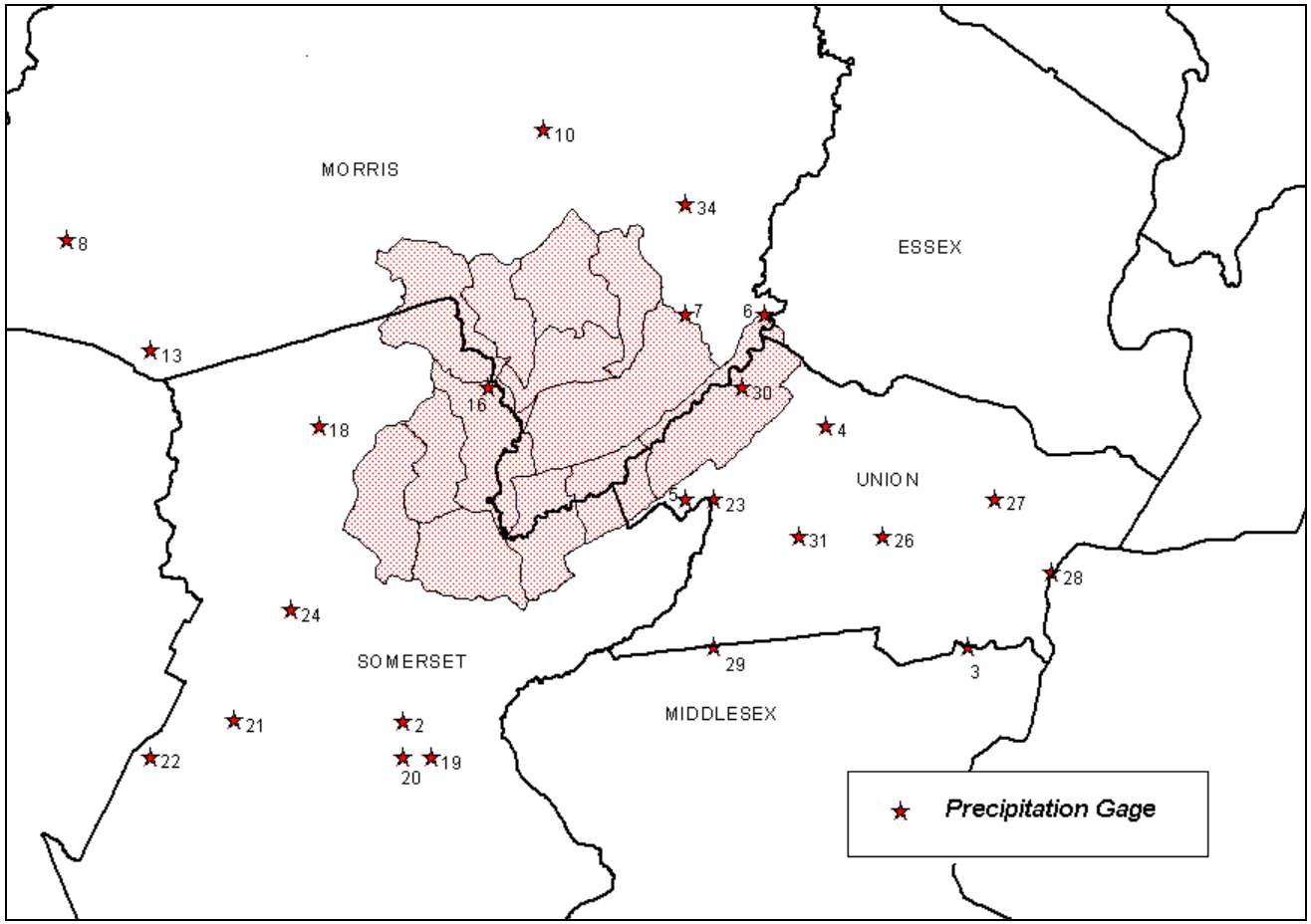


Figure 3.3.1.1. Precipitation gage locations in the study area with records maintained by NOAA

Table 3.3.1.2. Results of correlation analysis for cumulative rainfall series

Gage site	Statistic [1]	Observed storm event		
		1989	1992	1994
Springfield precipitation gage	m	1.327111	2.846817	0.945205
	b	-0.151128	-0.536162	-0.214127
	R^2	0.925141	0.984163	0.963714
Wachtung precipitation gage	m	1.285968	2.275668	0.860689
	b	-0.075404	-0.068884	-0.125871
	R^2	0.953601	0.982125	0.988031
Rahway precipitation gage	m	1.354262	1.496598	0.785443
	b	0.00681	-0.00123	-0.153912
	R^2	0.922118	0.980826	0.851526
Boonton precipitation gage	m	no data	1.793368	1.211143
	b		-0.513024	-0.373658
	R^2		0.948166	0.859793

1) ppt(site, t) = m*ppt(Bound Brook, t) + b

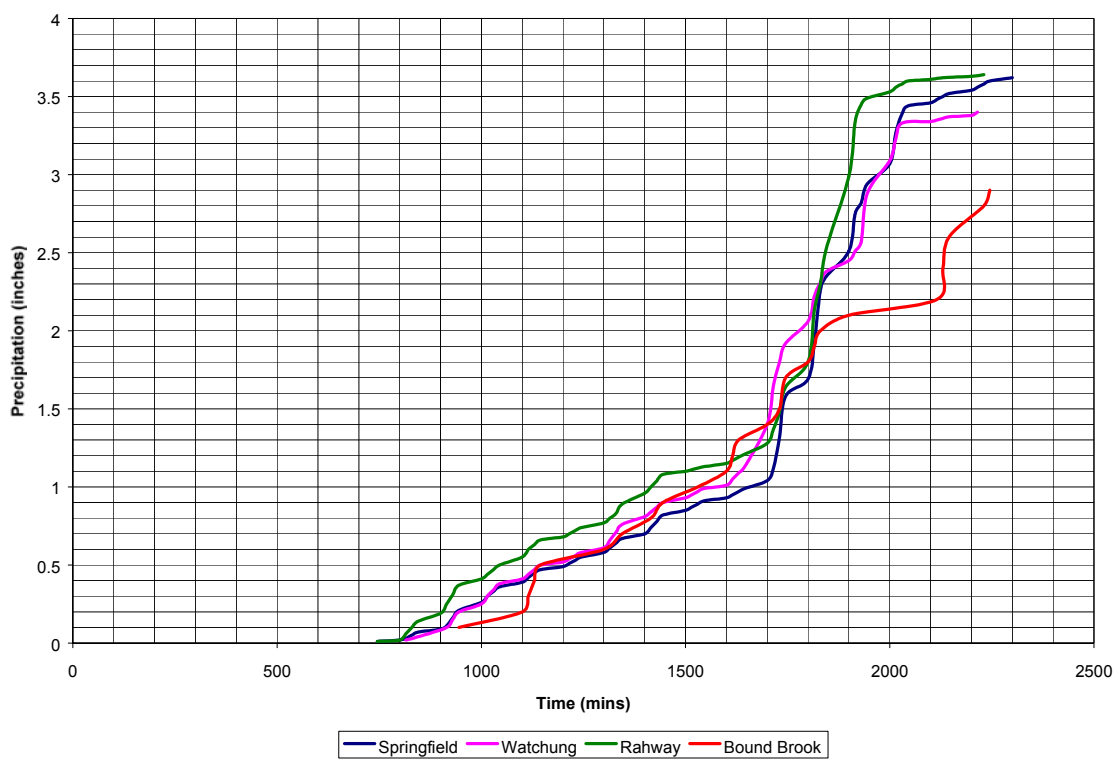


Figure 3.3.1.2. Hyetographs recorded by area gages for September 1999 rainfall event

After a review of available rainfall and stream gage records, it was determined that the storm of September 1999 (Floyd) could be used for calibration of the HEC-HMS model for the Upper Passaic River watershed. The cumulative and instantaneous hyetographs for the Hurricane Floyd rainfall event are shown in Figure 3.3.1.3 below.

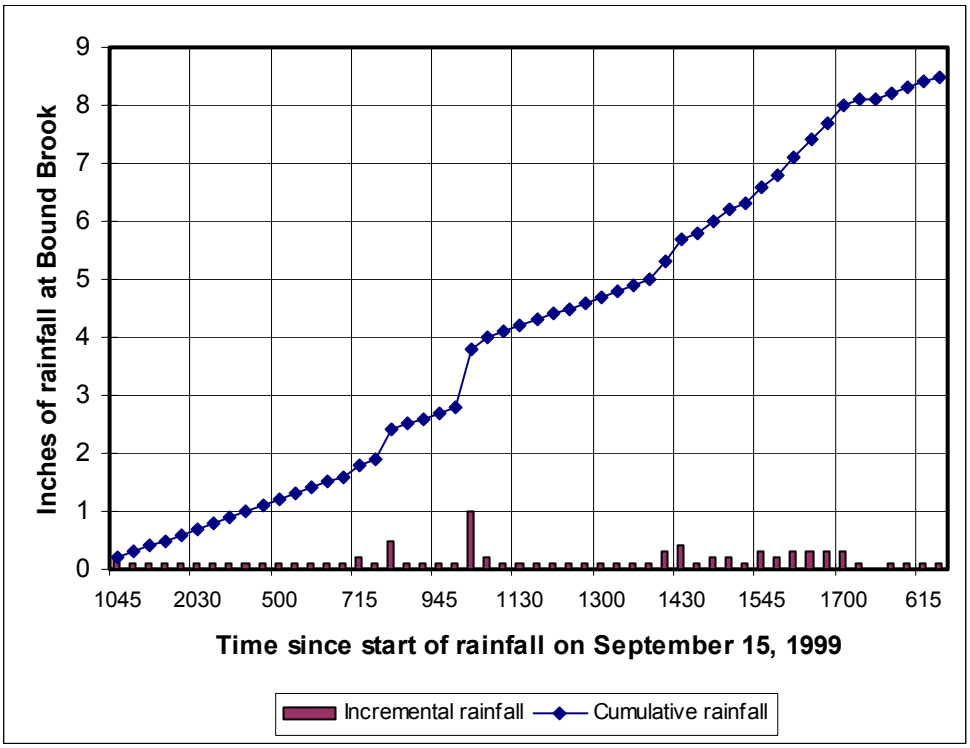


Figure 3.3.1.3. Hurricane Floyd rainfall hyetograph (Bound Brook, NJ ppt. Gage)

3.3.2 Calibration procedures

The existing conditions model was divided into an upper watershed model for calibration against observed stream flow at the Millington stream gage, and a lower watershed model for calibration against observed stream flow at the Chatham gage. The optimization option in HEC-HMS was used to systematically vary T_c, R, and RCN for each subbasin and stream routing parameters for each reach in the model, with the object of minimizing the weighted root mean square error between the observed and simulated hydrographs at the Millington and Chatham gages. The range for calibrated values of RCN was constrained between 90 and 110% of the values determined in HYDPAR.

The results of each automated optimization run were evaluated for reasonableness using good engineering judgement to reconcile conflicting requirements for matching the timing and magnitude of peak discharges against the total volume of runoff. When each

subwatershed model was calibrated, the two models were rejoined. The initial and final calibrated values of T_c, R, and RCN are listed in Tables 3.2.2, above, in Section 3.2.3. Key statistics for the upper and lower watershed calibration models are summarized in Tables 3.3.2.1 and 3.3.2.2, respectively.

Table 3.3.2.1. Summary of calibration results for the Hurricane Floyd event at Millington

Search Method:	Univariate Gradient
Objective Function:	Peak-Weighted RMS Error = 154.66
Start Time:	11 September 1999, 00:15
End Time:	22 September 1999, 24:00
Simulated Peak Flow:	1573.8 cfs
Observed Peak Flow:	1590 cfs
Difference:	-16.2 cfs
Simulated Time of Peak:	17 September 1999, 15:45
Observed Time of Peak:	17 September 1999, 05:00
Difference:	645 minutes

Table 3.3.2.2. Summary of calibration results for the Hurricane Floyd event at Chatham

Search Method:	Univariate Gradient
Objective Function:	Peak-Weighted RMS Error = 270.26
Start Time:	11 September 1999, 00:15
End Time:	22 September 1999, 24:00
Simulated Peak Flow:	2257.2 cfs
Observed Peak Flow:	2210 cfs
Difference:	47.2 cfs
Simulated Time of Peak:	18 September 1999, 05:30
Observed Time of Peak:	17 September 1999, 17:00
Difference:	750 minutes

As can be seen from Tables 3.3.2.1 and 3.3.2.2, it was possible to match the peak discharge value quite well in the calibration model, but there is a significant error in the timing of the peak discharges.

Figure 3.3.1.4 shows the observed and calibrated model hydrographs for the Millington stream gage for the Hurricane Floyd calibration event. Figure 3.3.1.5 shows the observed and calibrated model hydrographs for the Chatham stream gage for the Hurricane Floyd calibration event. It was noted above there is a significant error in the timing of peak discharges. The plots of both the observed and simulated hydrographs show rather flat peaks, however, which tends to reduce the impact of the missed timing, making the model suitable for its intended use for a flood control feasibility study.

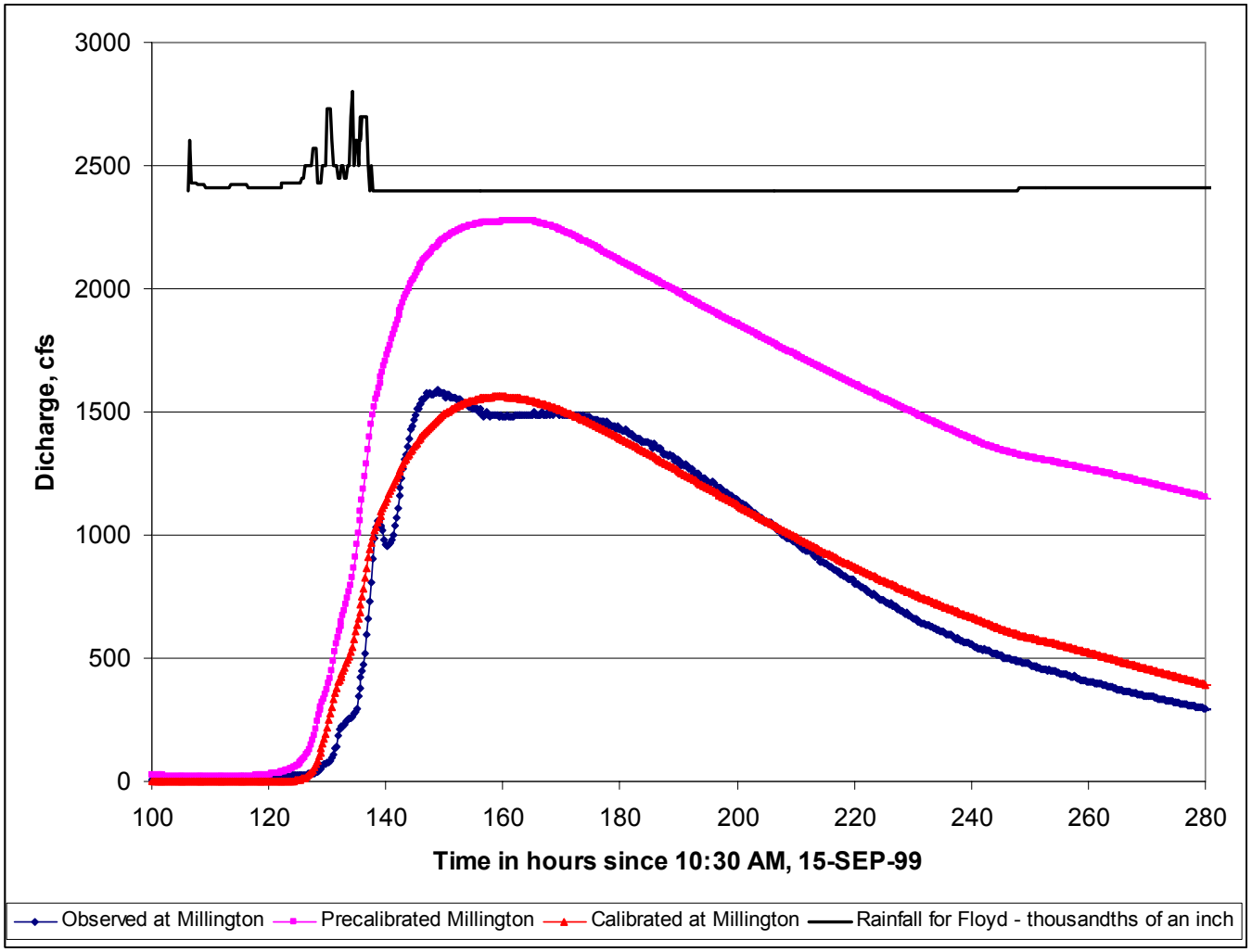


Figure 3.3.1.4. Observed and calibrated hydrographs for Hurricane Floyd at the Millington, NJ stream gage

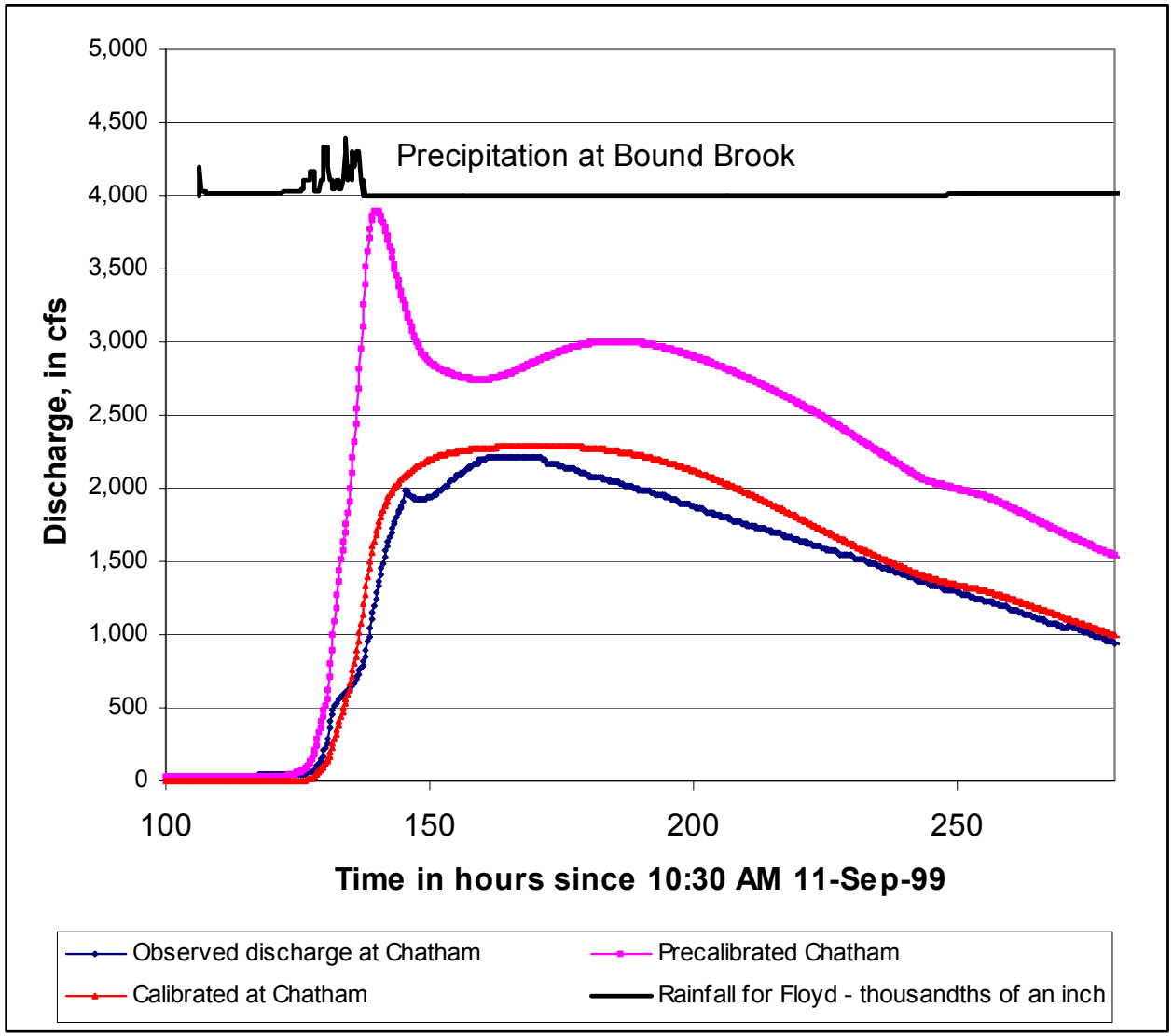


Figure 3.3.1.5. Observed and calibrated model hydrographs for Hurricane Floyd at the Chatham, NJ stream gage

3.3.3. Calibrated model verification

The October 15, 1996 rainfall event was used for verification of the existing conditions model. The model was run with observed rainfall for this event as recorded at the Bound Brook precipitation gage. Figure 3.3.3.1, on the following page, shows the observed and simulated hydrographs for the verification storm event at the Millington and Chatham gages. The general shape and timing of the observed storm event is modeled adequately, but the model does not adequately reproduce peak discharge. This failure is attributed to failing to capture the antecedent soil moisture content within the watershed and the use of rainfall records that do not adequately represent rainfall that fell in the basin.

The models were also run for the design rainfall events and the peak discharges were compared with the values determined from Bulletin 17-B-based discharge-frequency statistics developed for Chatham and Millington stream gages in subtask 1-a. Increasing design rainfall values by 10% resulted in matching the peak discharges quite well. This work is described in Section 3.4.

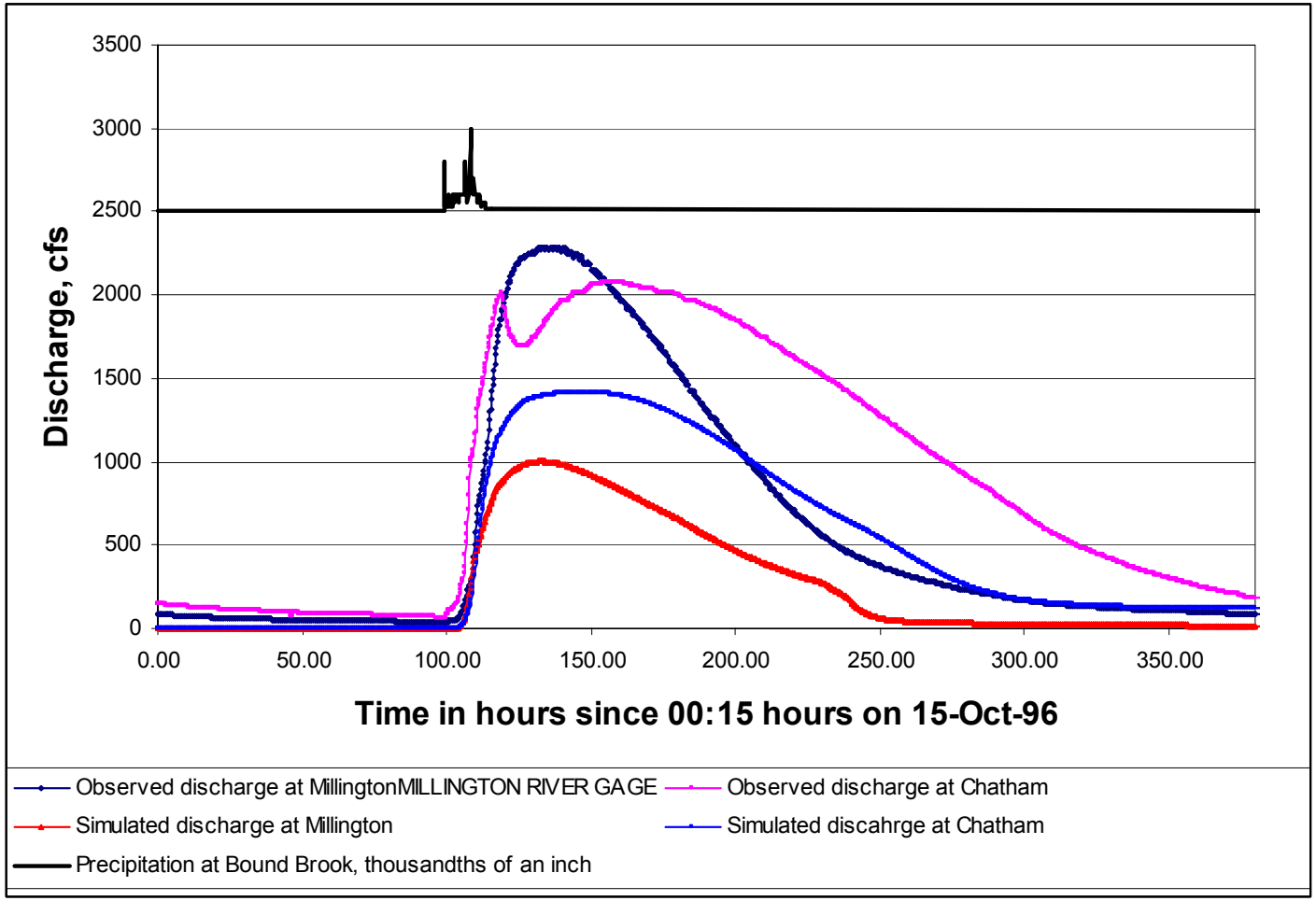


Figure 3.3.3.1. Verification storm modeled and observed hydrographs

3.4 Design storm precipitation input structure of the HEC-HMS model.

Following the 1995 HEC-1 model, rainfall inputs for the current HEC-HMS model were determined with the Hypothetical Storm option, defining consistent depth duration and depth/area relationships for each return period. Table 3.4.1 summarizes the point data entered for the design rainfall events for each return period storm (these values were increased by 10% during calibration). This data is shown graphically in Figure 3.4.1

Table 3.4.1. Depth-duration data utilized for design storm modeling in HEC-HMS

duration, hours	Recurrence Interval, 1/f							
	1 year	2 years	5 years	10 years	25 years	50 years	100 years	500 years
0.08	1.25	1.50	1.65	1.95	2.35	2.80	3.35	4.00
0.25	1.45	1.75	1.95	2.35	2.80	3.35	4.00	4.80
1.00	1.75	2.20	2.50	3.05	3.60	4.30	5.20	6.40
2.00	2.10	2.60	2.95	3.70	4.30	5.25	6.40	7.75
3.00	2.30	2.90	3.40	4.10	4.80	5.75	7.00	8.45
6.00	2.70	3.35	3.75	4.55	5.45	6.50	7.80	9.40
12.00	3.00	3.75	4.25	5.20	6.25	7.45	9.00	10.70
24.00	3.65	4.55	5.15	6.30	7.60	9.00	10.70	12.70

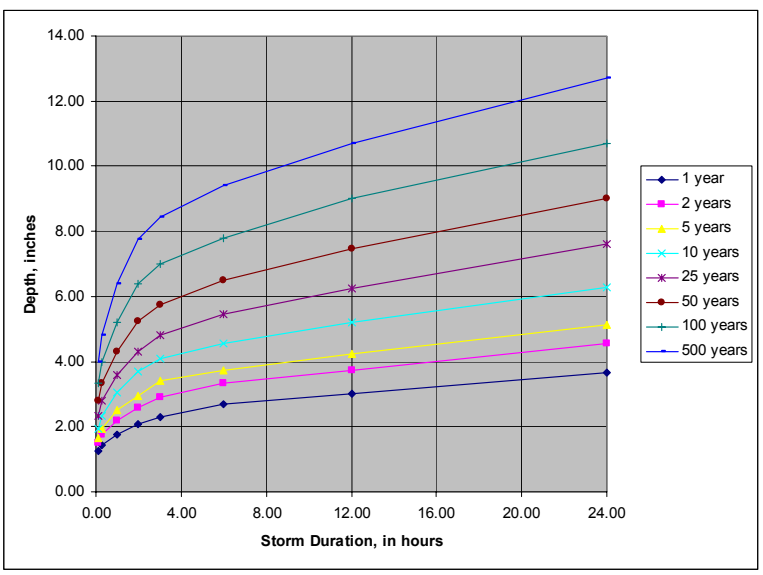


Figure 3.4.1. Design Storm Depth-duration curves used for 1995 HEC-1 modeling of the Upper Passaic River watershed

Following common modeling practice, the hypothetical storm was computed over each subbasin with the total rainfall distributed in accordance with the specified depth-duration data. A triangular precipitation distribution was constructed such that the depth specified for any duration occurs during the central part of the storm. The first non-zero depth

specified is the most intense portion of the storm. The design rainfall hyetograph developed in this manner for the 100-year event over one of the subbasins in the model is shown in Figure 3.4.2, below.

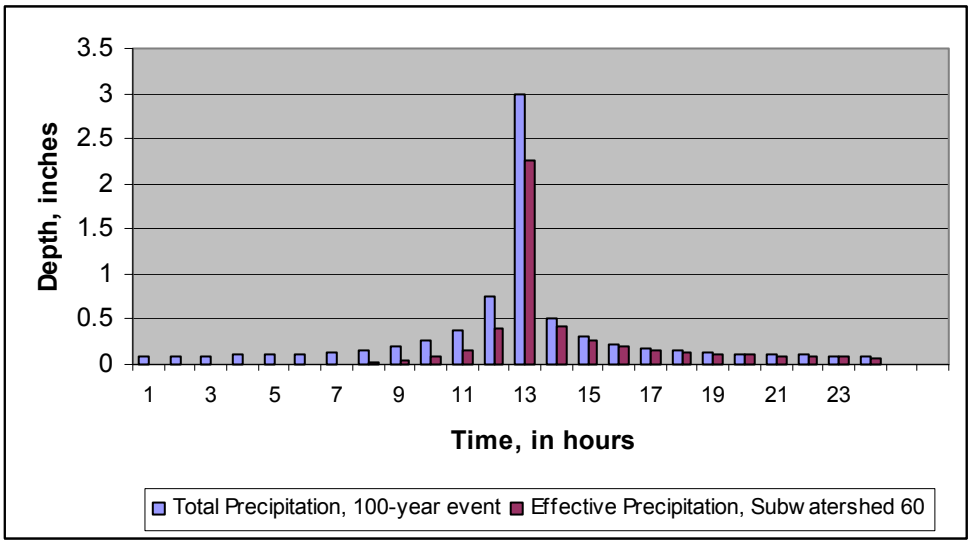


Figure 3.4.2. Rainfall hyetograph for 100-year rainfall event

Common modeling practice is to develop average precipitation depths over variable areas by the extension of point rainfall depth records. Basin area was used to compute the reduction of point rainfall depths using the depth-area option. The factor, F, for adjusting point rainfall is:

Factor = 1. – BV * (1. – EXP(-.015 * AREA))

where BV is the maximum reduction of the point rainfall, as given in Table 3.4.2, below.

Table 3.4.2. Point-to-Areal Rainfall Conversion Factors (reproduced from Table 3.4 of HEC-1 Flood Hydrograph Users Manual; USACE, September 1990)

Duration (hours)	**BV, Maximum reduction of point rainfall**
0.5	0.48
1	0.35
3	0.22
6	0.17
24	0.09
48	0.068
96	0.055
168	0.049
240	0.044

Point rainfall depths for each duration were adjusted for the area of the watershed utilizing five JD records with areas of 0.2, 20, 50, 100, and 1000 square miles. Table 3.4.3 shows the point rainfall reduction factors adopted for present use from the 1995 HEC-1 model.

Table 3.4.3. Point rainfall reduction factors calculated for design rainfall modeling

Area, sq. mi.	Duration of rainfall, in hours								
	0.5	1	3	6	24	48	96	168	240
0.2	0.9986	0.9990	0.9993	0.9995	0.9997	0.9998	0.9998	0.9999	0.9999
20	0.8756	0.9093	0.9430	0.9559	0.9767	0.9824	0.9857	0.9873	0.9886
50	0.7467	0.8153	0.8839	0.9103	0.9525	0.9641	0.9710	0.9741	0.9768
100	0.6271	0.7281	0.8291	0.8679	0.9301	0.9472	0.9573	0.9619	0.9658
1000	0.5200	0.6500	0.7800	0.8300	0.9100	0.9320	0.9450	0.9510	0.9560

Figure 3.4.3, below, shows the point precipitation reduction factor as a function of area for storm durations of 0.5, 1,6, and 24 hours.

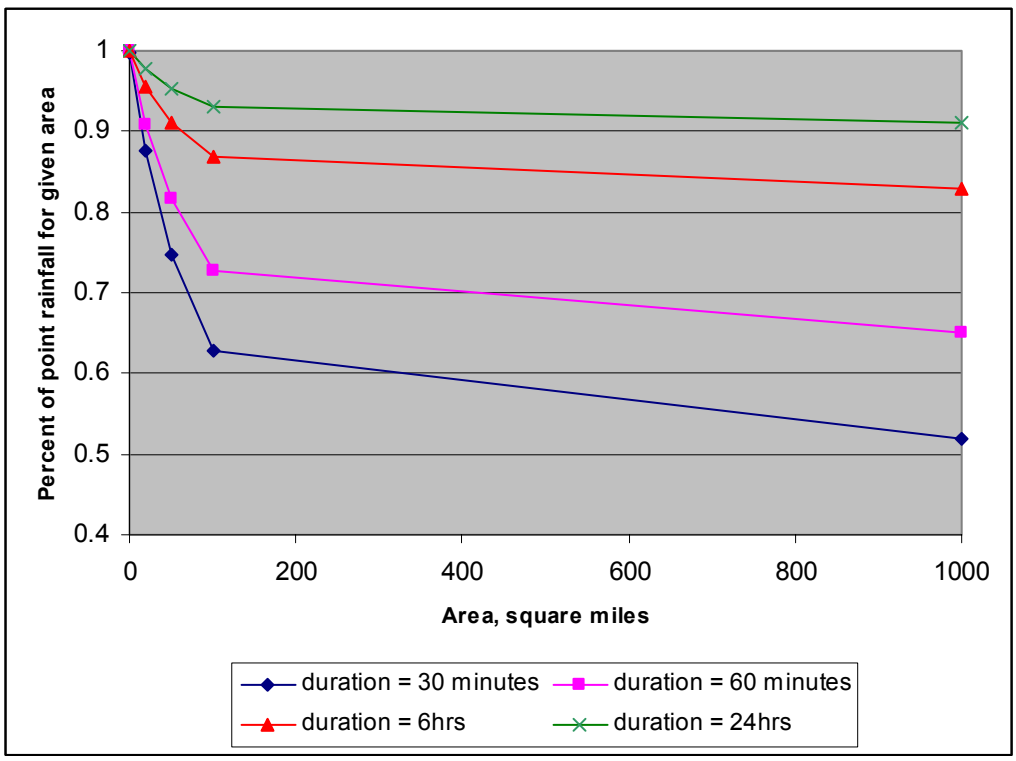


Figure 3.4.3. Point rainfall reduction factors utilized in design storm modeling

3.5 HEC-HMS model Output

After the existing conditions model was calibrated, it was used for the synthesis of discharge-frequency relations for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year return period storms. Design rainfall depths from the 1995 study were increased by 10% to obtain values of peak rainfall within the confidence intervals identified during the statistical analysis of gage data undertaken in Task 1a.

Table 3.5.1 below summarizes the peak discharges at key points within the channel network system for the design rainfall events and for the Hurricane Floyd calibration event. The 10-, 50-, 100-, and 500-year event discharges presented in the 1999 Preliminary Flood Insurance Study for the Township of Long Hill, NJ are shown on the last line for purposes of comparison.

Table 3.5.1 Peak discharge, in c.f.s, along channel network (HEC-HMS output for calibrated model)

Node	Peak annual discharge, in cfs							Hurricane Floyd
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr	
2105 (Millington)	732	1,127	1,471	1,869	2,138	2,529	3,145	1,562
2,106	664	1,036	1,354	1,721	1,969	2,331	2,900	1,441
2107 (Dead R.)	990	1,608	2,130	2,724	3,124	3,713	4,636	2,273
91J (Plainfield)	936	1,485	1,971	2,502	2,865	3,405	4,286	2,108
92J (Snyder Rd.)	947	1,509	2,010	2,549	2,918	3,472	4,360	2,149
2109 (Passaic/Long Hill)	990	1,582	2,105	2,671	3,053	3,630	4,555	2,256
2110 (Chatham)	1,000	1,605	2,134	2,719	3,108	3,685	4,622	2,290
FEMA Restudy, 91J	--	--	1,850	--	2,680	3,099	4,120	--

The summary output tables for each design rainfall event obtained from HEC-HMS are reproduced in Tables 3.6a and 3.6b at the end of this section. All model files have been submitted to the New York District in electronic formats.

Figures 3.5.1 and 3.5.2 show a comparison of simulated peak discharges with the annual peak values obtained from a statistical analysis of gage data, presented in Section 2. As can be seen from an inspection of these figures, there is a good fit for the lower frequency storms. The simulated two-year event falls below the lower confidence limit established by statistical analysis. It is hypothesized that the dampening effect of the large area of wetlands in the watershed, which is responsible for reducing peak discharges of the lower frequency events below the per-square-mile yields commonly expected, does not come into effect for the high frequency events with a two-year or lower return period.

Because there is no damage associated with the storm, no correction to rainfall was attempted to make the two-year peak discharge plot within limits. The design rainfall could be reduced for the 2-year event, but it was felt that a global adjustment to design rainfall intensity was more consistent with previous modeling practice for the basin.

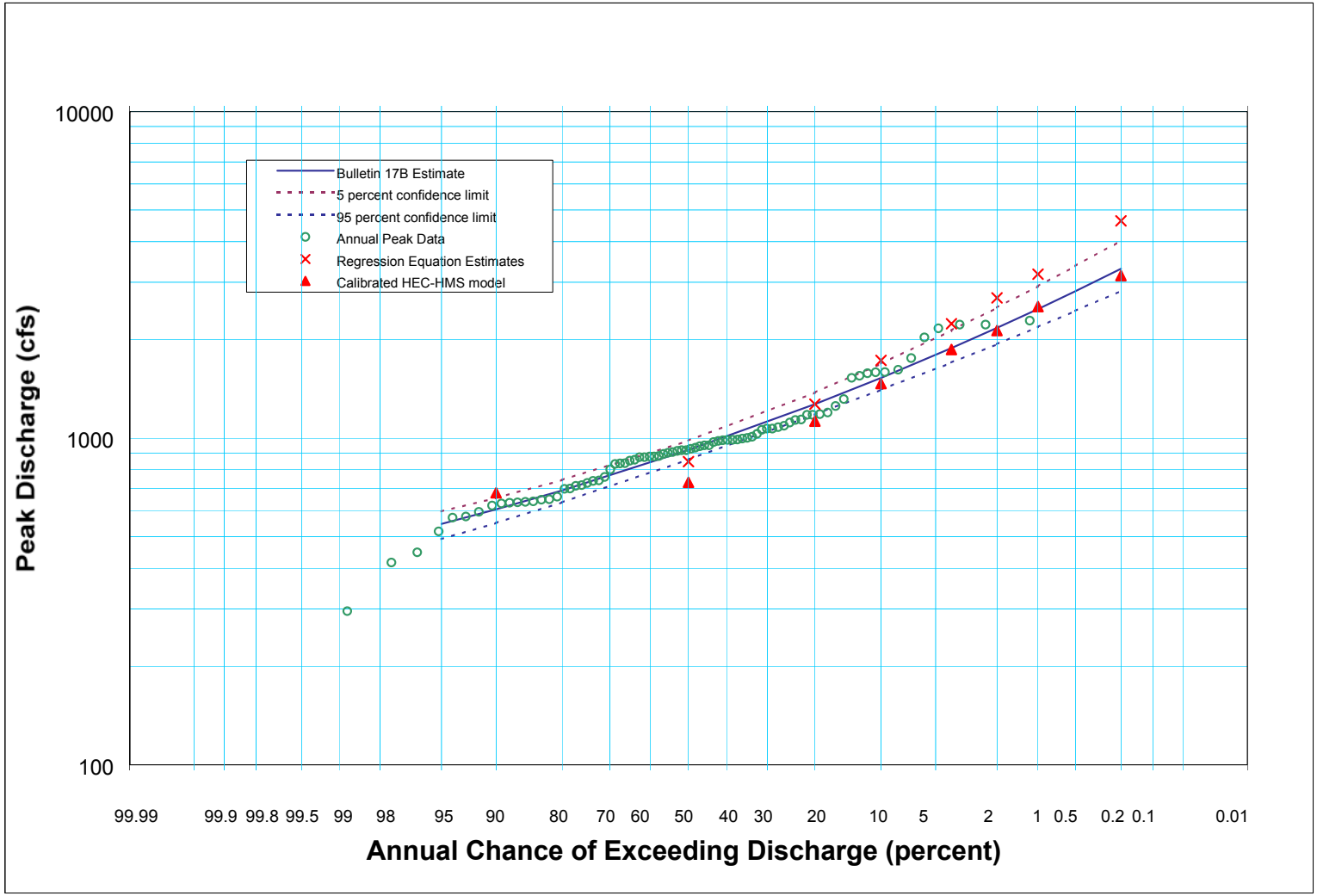


Figure 3.5.1 Comparison of alternative annual peak frequency curves for the USGS gage at Millington, NJ.

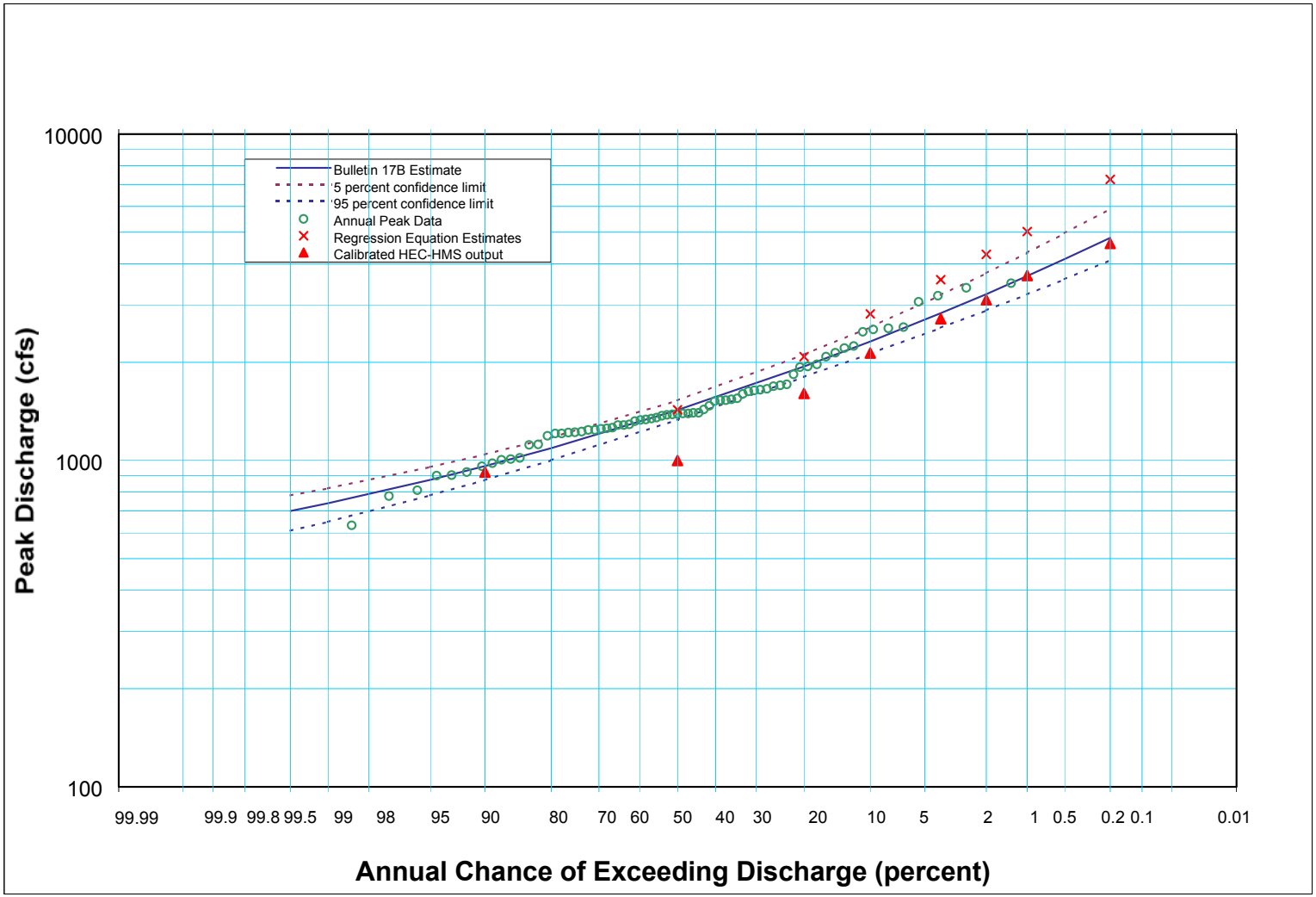


Figure 3.5.2 Comparison of alternative annual peak frequency curves for the USGS gage at Chatham, NJ.

4.0 DEVELOPMENT AND CALIBRATION OF EXISTING CONDITIONS HYDRAULIC MODEL

A HEC-RAS model was developed for the upper Passaic River along the study reach to develop standard project flood water surface profiles for the 2-, 5-, 10-, 25- 50-, 100-, 250-, and 500-year storm events. The work included:

- The incorporation of cross-section geometry for the floodplains and the main channel of the study reach utilizing recent survey data and existing model data where available and as appropriate
- The incorporation of geometric data and hydraulic parameters for all bridge and culvert crossings from recent survey data
- The determination of the variation in discharge along the study reach for each storm event to best represent design conditions with respect to the comparative times-to-peak for the main stem of the Passaic River and the interior drainage areas
- The determination of downstream boundary conditions for each storm event

The existing conditions HEC-RAS model was calibrated in terms of observed high water marks recovered from Hurricane Floyd, using discharges observed the Millington and Chatham, NJ stream gages for that storm event. The variation in discharge between those two gages was estimated from the in-channel peak discharge values calculated in HEC-HMS. The downstream starting water surface elevation for that event was determined from a rating curve for the Chatham gage obtained from the USGS.

The details of existing conditions hydraulic work are described in the following sections.

4.1 Description of Study Reach

Channel and overbank conditions along the Passaic River vary significantly between the Millington and Chatham gages. The Millington gage is located in a constricted section with a narrow channel and steep floodplain valley walls with a stream gradient of 0.22%.

The floodplain widens considerably near Stirling, resulting in significant overbank storage. At this point, the stream bed gradient becomes essentially flat for approximately 6 miles, varying in elevation between 200 and 203 (NGVD) with adverse stream gradients in the Stirling area. The flat gradient combined with the significant overbank storage leads to local flooding as flow in the Passaic River spills out of the main channel and is stored in the overbank areas. This flood storage leads to reduced flood discharges downstream.

From Berkley Heights downstream to the Chatham gage, there are many small tributary inflows from highly developed areas. These inflows lead to a two-peaked hydrograph at Chatham with the local tributaries creating the first peak before the floodwave comes downstream from the upper portions of the Passaic River basin.

In the HEMP it was noted that local emergency management officials have reported flow reversals (floodwaters traveling upstream) along the Passaic River channel during the onset of flooding events. Flat slopes, exacerbated by sedimentation and insufficient downstream channel capacity in combination with the rapid delivery of large quantities of water from upstream areas could cause this phenomenon. Flow reversal would tend to exacerbate both flood levels as well as extend the duration of flooding. Subsequent discussions revealed that the flow reversal may well be restricted to the small tributaries of the Passaic River draining the interior areas of the Long Hill Township.

4.2 Selection of Hydraulic Model

The use of existing hydraulic models of the Passaic River for the current study was considered. A UNET model was used for the hydraulic analysis of the Passaic River in the 1995 General Design Memorandum; however, the upstream end of the UNET model is the Chatham gage that is approximately 9 miles downstream of Stirling. A HEC-2 model was developed for the Long Hill Township FEMA Flood Insurance Rate Map based on channel survey data from 1977. Neither of these models was considered usable for this feasibliliy study for the Stirling area.

Both UNET and HEC-RAS modeling options were considered for the feasibility study. UNET modeling was considered for the hydraulic analysis because of its capacity for modeling reverse flow because, as noted above, reports of reverse flow during storm events had been noted during elaboration of the HEMP report. UNET is a dynamic model, which simulates the entire flooding event, while HEC-RAS is less complex and simulates only the peak flood condition. UNET requires significantly more data than HEC-RAS.

Although flooding in Stirling is relatively frequent, the flooding is not very deep even for the most severe floods; expected annual damages were estimated for the reconnaissance report interior drainage areas as $100,650. Weighing the level of uncertainty, expected annual damages, and the complexity of the flooding conditions, it was determined that the additional effort required to develop the more complex UNET model would not be justified and it was decided to use HEC-RAS modeling for the feasibility study.

4.3 Development of Existing Conditions Hydraulic Model

The flood study reach is approximately 28,000 feet (5.3 miles) in length and begins 1000 feet downstream of Snyder Avenue in Berkeley Springs (about 4.5 miles downstream of the center of Stirling). It ends approximately 4000 feet upstream of the center of Stirling where Main Street extends to the Passaic River.

Topographic data was obtained in a field survey in the Summer of 2000 for 59 channel cross sections along the study reach of the Passaic River, spaced approximately 500 feet apart. Cross sections were also obtained at the up- and down-stream faces of all bridges. Bridge geometry was also surveyed. Fifty-eight (58) channel cross sections were taken in the interior drainage areas with two long cross sections taken through the main low-lying areas. Aerial photography of the study area was flown in March of 2000 and 2-foot

contour maps were developed for most of the study area. For the right (south) Passaic River overbank, a stereoscopic model was developed from the aerial photographs but no contour mapping was produced. The stereoscopic model was used to obtain overbank ground elevations for the HEC-RAS cross sections.

Approximately 77,000 feet (14.4 miles) of the effective FEMA model for the Upper Passaic River from the Chatham gage to the Millington gage, based on cross sections surveyed in 1977, was directly imported to HEC-RAS from HEC-2 format. Minor modifications to the HEC-RAS model were necessary to correct minor discrepancies resulting from the data import procedure. The new cross sectional and bridge data within the study reach was spliced into the FEMA model at the closest cross section located downstream of the Long Hill Township study reach at River Station 177 (338,775).

The year 2000 survey geometry (including overbank and channel points) was imported and added to the FEMA model beginning with station 178 (M1) and ending with station 244 (M59). The reach lengths at these sections were corrected to reflect these changes. In the course of adding the newly surveyed cross sections, we found that these sections contained greater than 500 individual points.

Because HEC-RAS can only accommodate a maximum of 500 cross section points, the number of points in the surveyed cross sections were reduced by removing duplicate points and non-essential intermediate points. The geometry for the remaining upstream cross sections, River Station 369310 through River Station 431760 of the effective FEMA model were imported and added to the combined model geometry at River Station 244.1 and 324 respectively. The reach lengths at these sections were corrected to reflect these changes.

Since 1977, several bridges along the Upper Passaic River have been replaced, including Snyder Avenue, Mountain Avenue, and Stirling/Plainfield Road. One bridge, Main Street extended, has been removed. This bridge apparently has not been in place for quite some time.

The Snyder Avenue Bridge is under construction while the study was being conducted. Therefore, the proposed bridge geometry (low chord, top-of-road, and opening width) was taken from the Replacement Design Plans and incorporated in the combined model.

Bank stations were set at each cross section at an approximate elevation of 204 ft or greater based on the assumption that normal water surface elevation is approximately at elevation 204 ft through the study reach. At bridges, bank stations were set at the inside wall of the abutments. Bank station locations were “spot-checked” with channel widths measured from aerial photographs at a scale of 1inch equals 1000 feet. Ineffective flow areas were added to the bridges at the inside abutment wall and where appropriate along the cross sections. Ineffective flow areas for upstream and downstream areas at the bridges were set to account for expansion and contraction of flow.

An additional field visit to the Passaic River study area was made in January 28, 2002 to assess the roughness coefficients in the overbanks. Photographs taken were compared to those in the USDOT FHA Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains and an n-value was assigned for each photograph. Aerial photography was reviewed to delineate the approximate limits of the n-values assigned to each photograph based on vegetation patterns and land use visible in the aerial photography.

The final hydraulic model was executed in HEC-RAS (version 3.0) using the discharge values calculated for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year storm events. The variation in discharge along the study reach was determined for each storm event to best represent design conditions. Additionally, boundary conditions for each storm event were determined based upon the rating curve for the Chatham gage. The hydraulic model input and summary output data are attached.

4.4 Calibration of Existing Conditions Model

The combined updated model was calibrated to observed high water marks (HWMs) for the Hurricane Floyd event in September of 1999 in Stirling, as proposed in the Scope of Work. The variation in peak discharge along the study reach for the Hurricane Floyd event was obtained from the HEC-HMS output developed during model calibration.

The peak discharge and stage during Hurricane Floyd on September 17, 1999, at the Chatham gage was recorded at 2210 cfs and elevation 201.11 ft. At Millington, the peak discharge during Hurricane Floyd on September 17, 1999 was recorded at 1590 cfs with a water surface elevation of 224.51 ft. These peak discharges correspond to an event between the 10- and 25-year storm.

Because the old Snyder Avenue Bridge was in place during Hurricane Floyd, the original geometry was used in the calibration model based upon survey data from the year 2000. Water surface elevations from results of the HEC-RAS calibration run were compared with five observed water surface elevations determined from High Water Marks in Stirling and Long Hill Township.

Following standard engineering practice, calibration was achieved by adjusting n-values along the stream for the left and right over banks and the main channel separately to minimize the unexplained difference in the simulated Hurricane Floyd water surface profile and observed High Water Marks. Final n-values are consistent with the upper range of roughness reported in the literature. The main channel n-values vary between 0.030 and 0.040; n-values for the overbank areas vary between 0.080 and 0.125, with ineffective flow areas set to 0.200.

The high water marks (HWM) used for calibration of the HEC-RAS model are listed in Table 4.4.1, along with a comparison of observed and simulated water surface elevations after calibration. The HWM locations are shown in Figure 4.4.1, superimposed on the simulated water surface profile for the Hurricane Floyd calibration event.

Table 4.4.1. Calibration of Existing Conditions Model

HWM No.	Distance U/S of Chatham	Location of HWM	Elevation		Difference in feet
			Observed HWM	Floyd WSP *	
1	0	Chatham gage reading	201.11	201.11	0.00
2	24,616	Berkeley Heights WWTP near Snyder Avenue, mark on footing of Main Builiding	212.31	211.93	0.38
3	33,790	Average sitings for House Nos. 223, 233, and 237 River Bend Road	212.35	212.69	-0.34
4	46,399	Wooden fence at the Calabria II Restaurant, 10 feet from the northeast corner of the building along the fence that runs parallel to Valley Road.	214.91	213.605	1.30
5	46,631	Long Hill Township WWTP, steel door at southernmost door of the pump house.	214.81	213.62	1.19
6	46,920	Fire Hydrant on Valley Road	215.35	213.64	1.71

* HEC-RAS calibration model

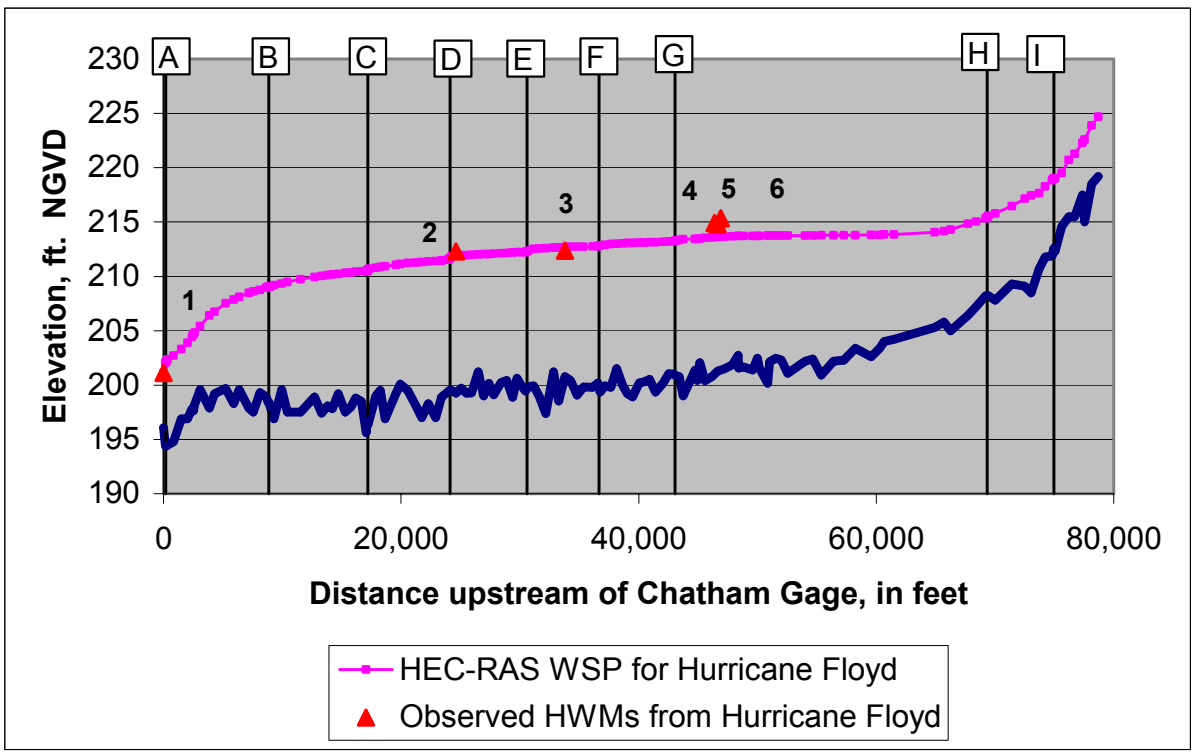


Figure 4.4.1 Calibration Profile and High Water Mark locations (Bridges: A - Stanley Avenue Bridge; B - Passaic/South Bridge; C - Central Avenue Bridge; D - Snyder Avenue Bridge; E - Conrail Bridge; F - Mountain Avenue Bridge; G – Stirling-Plainfield Avenue Bridge; H - Passaic Valley Road Bridge; I - Haas Road Bridge)

As can be seen from Figure 4.1.1, a very good level of agreement was obtained between the simulated profile and the elevations estimated for HWM Nos. 2 and 3. The HWM elevations estimated in Long Hill Township are more than one foot above the simulated water surface profile at that section.

All of the observations within Long Hill Township (HWM Nos. 4, 5, and 6) are located near Cross Sections 231 (M49) and 232 (M50), at varying distances from the main channel. Of these, the HWM No. 5, at the Long Hill Township WWTP, is considered the most representative for calibration of the model because it was located closest to the Upper Passaic River. The HWM No. 6, at the Police Station, is located on the northern side of Passaic Valley Road, so that the water surface at that location includes head required to create lateral flow towards the main channel, and was, therefore, given less consideration in the calibration.

In order to reduce the difference between the HWM and simulated water surface elevations at that section to less than 1 foot would require raising n-values along the reach above the accepted upper limit. The desired rise in the water surface profile could also be obtained by increasing the ineffective flow areas, but there is no documented blockage of any of the bridges, and such blockage should not be included in the design hydraulic model.

4.5 Water surface profiles

After the calibration model was complete, the multiple profile hydraulic model was updated with the adjusted n-value parameters and water surface profiles were calculated for all design storm events, using discharge values determined by with the HEC-HMS model developed for Task 1C.

Design water surface profiles for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year water surface profiles are presented in Table 4.5.1, attached as an appendix to this section. Stationing is reported as distance upstream of the Chatham Gage, by HEC-RAS cross-section number, and by Surveyor's cross section number, as appearing on topographic maps prepared for preliminary design studies. Full input and output files for the HEC-RAS model used in the development of these profiles are also printed in the exhibits attached to this section.

There is a significant difference in the 100-year water surface profile developed for this study and the 100-year water surface profile presented in the FEMA study, as shown schematically in Figure 4.5.1, presented on the following page.

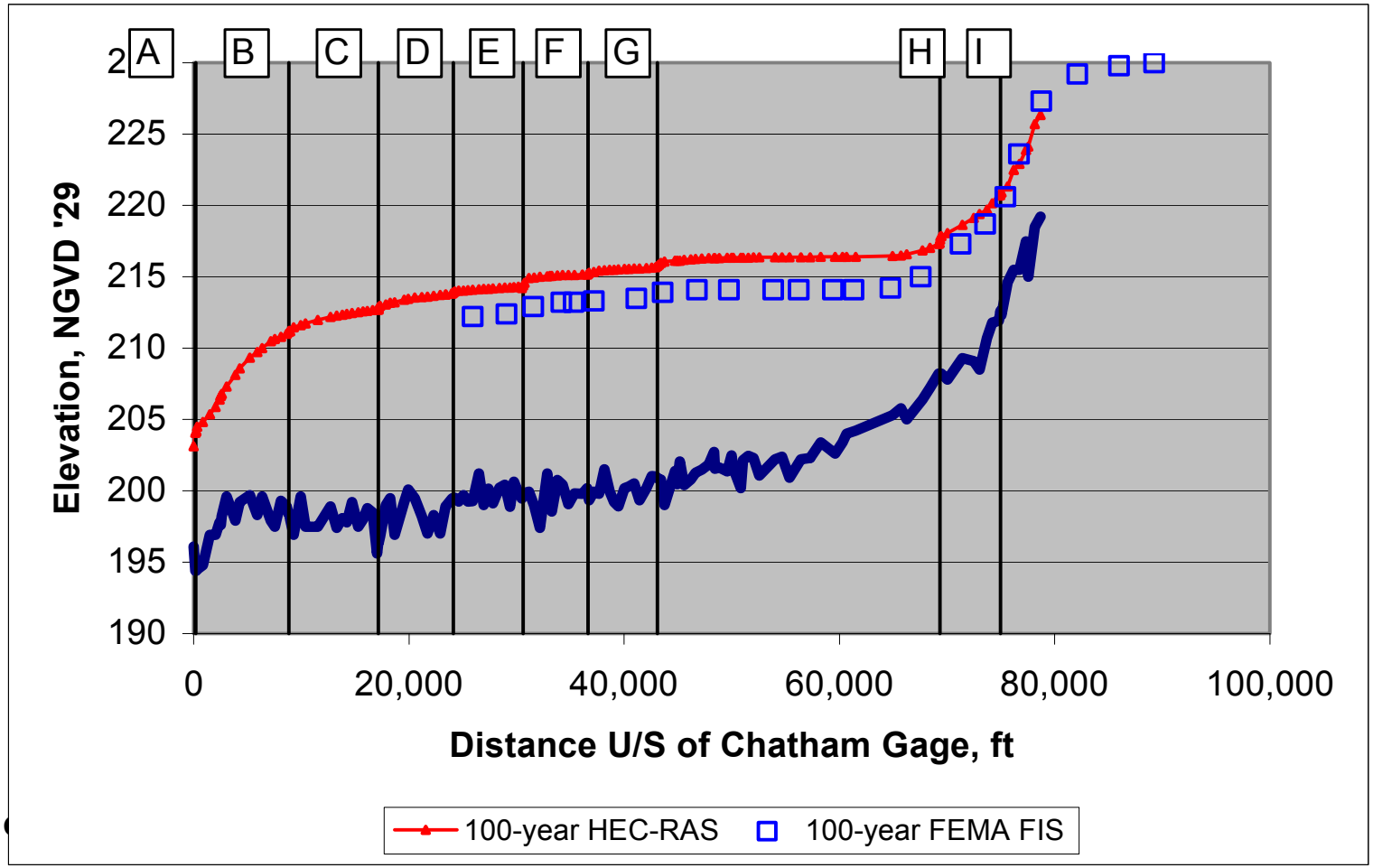


Figure 4.5.1 Comparison of FEMA FIS and Current Study 100-year Water Surface Profiles. (Bridges: A - Stanley Avenue Bridge; B - Passaic/South Bridge; C - Central Avenue Bridge; D - Snyder Avenue Bridge; E - Conrail Bridge; F - Mountain Avenue Bridge; G – Stirling-Plainfield Avenue Bridge; H - Passaic Valley Road Bridge; I - Haas Road Bridge)

4.6 Floodplain limits

Floodplain limits for the main channel were delineated in GEO-RAS by intersecting the TIN developed from site topography with the water surface profiles exported from HEC-RAS. The extent of flooding for each design period was also determined in HEC-IFH, as described in Section 6. Shape files of the interior flood area were combined with the GEO-RAS data for exterior flooding to show the footprint of generalized flooding at the site. Figure 4.6.1 shows the limits of the 100-year floodplain as determined in this study. Figure 4.6.2 shows the limits of the 10-, 25-, 50-, 100-, and 500-year floodplains.

4.7 Existing Conditions Hydraulic Modeling Exhibits

Table 4.5.1. Water surface profiles for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year return period design storm events.

The following files and graphics are presented in the CD attached to the H&H Appendix.

FEMA HEC-2 modeling imported to HEC-RAS
- Input data
- Output summary table

HEC-RAS calibration model
- Calibration water surface profile (HEC-RAS output graphic)
- Input data
- Output summary table

HEC-RAS Upper Passaic River multiple profile model
- Design Storm water surface profiles (HEC-RAS output graphic)
- Model cross-sections and design depths
- Input data
- Output summary table

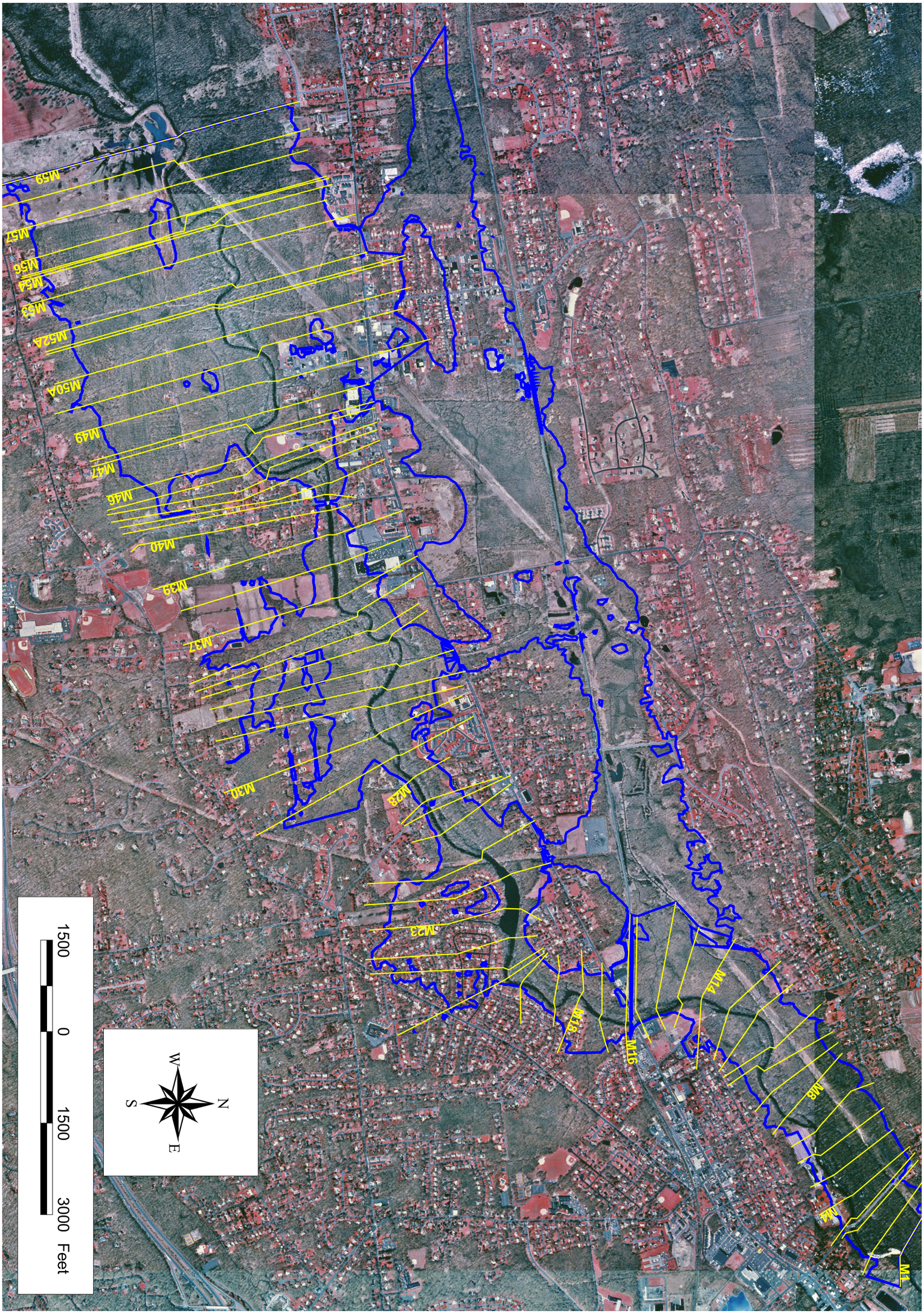


Figure 4.6.1
Floodplain Limits for the 100-yr Storm

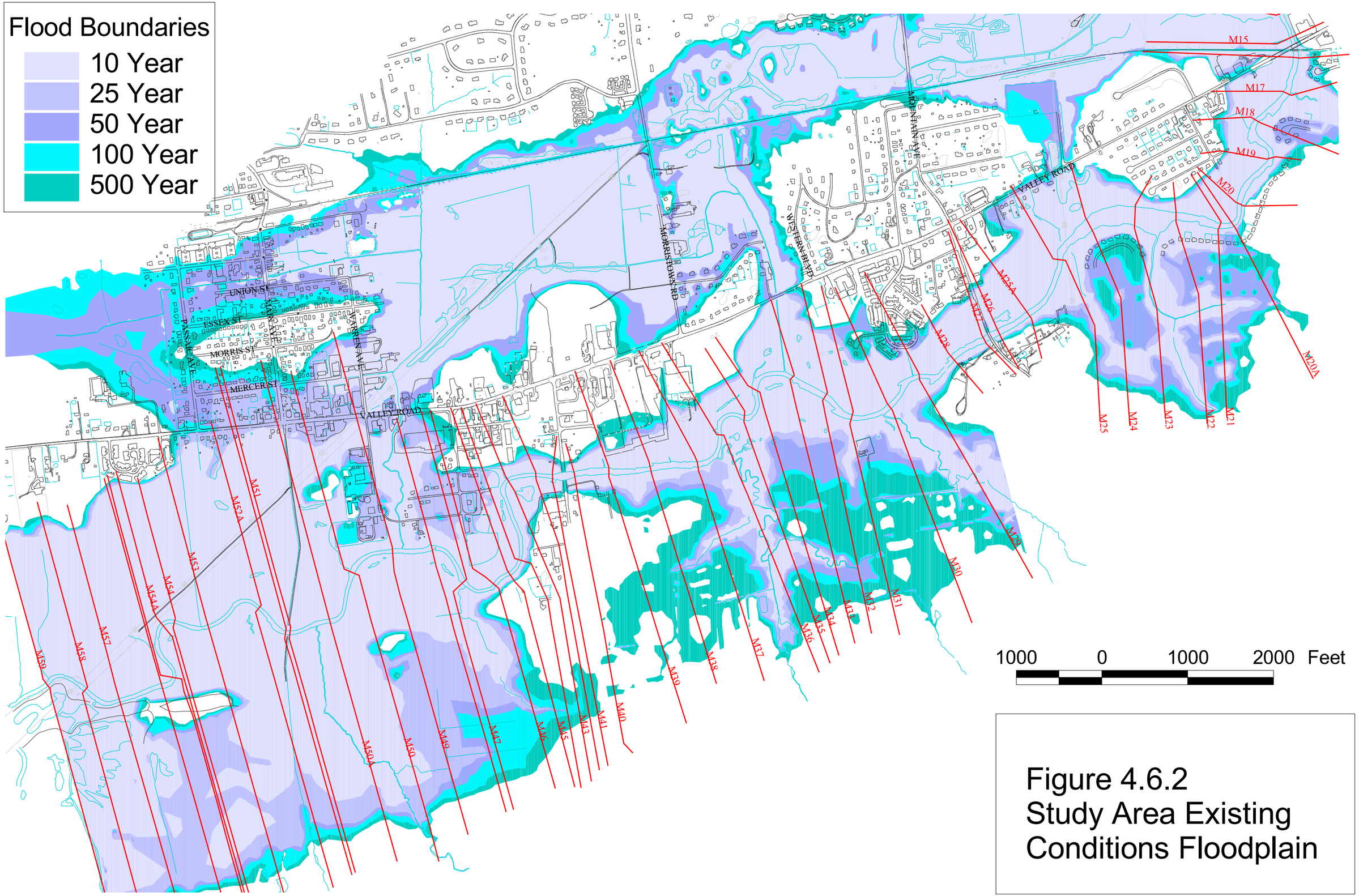


Figure 4.6.2
Study Area Existing Conditions Floodplain

5.0 UNCERTAINTY ANALYSIS OF HYDRAULIC DATA

The Corps of Engineers requires a risk-based analysis for the evaluation of flood control project feasibility. This entails the quantification of uncertainty in discharge-frequency, stage-discharge, and stage-damage functions for use in the risk-based flood damage modeling conducted as part of this project.

The quantification of the uncertainty in stage-discharge relations along the upper Passaic River developed for the current study is presented in this section. The quantification of uncertainty in the discharge-*frequency* relations developed for this study was discussed in Section 1A; uncertainty in stage-damage functions was not studied for this contract.

Stage uncertainty can be accounted for in a risk-based analysis by using the standard deviation statistic, estimated from the range of reasonable upper- and lower-bound water stages for each design discharge. Water stages can be impacted by a number of factors including:

- Variation in cross sections due to sedimentation or scour
- Timing of flooding of the Passaic River and interior areas
- Errors associated with cross-section surveys
- Variation of n-values for the overbank and channel

Calculations presented in Section 7 of this report indicate the potential for sediment deposition along the study reach of the Passaic River. This would have the effect of increasing stages for each return period. However, as noted in Section 7, comparisons of cross-sections were inconclusive in determining whether there had been any significant deposition during the 23 years between surveys. For that reason, no adjustment for deposition was included in the quantification of stage uncertainty.

Concurrent flooding of the main channel of the Passaic River and the interior area of the Long Hill Township would have a significant impact on interior stage hydrographs. As discussed in Section 6 of this report, it was determined that the interior flood will be receding before the flood wave from the main channel arrives for all reasonable scenarios. For this reason, timing issues were not considered in the quantification of stage uncertainty.

Errors associated with cross-section surveys can have an impact of the stage-discharge relations for a given channel reach. Systematic errors are controlled for by the surveyor and have assumed to be eliminated. Random errors can have the effect of increasing or decreasing stages for each discharge. The impact of random errors in the measurement of channel geometry was considered to be small when compared with errors in roughness.

In consideration of the previous discussion, the reasonable bounds for the stage were established for this study strictly in terms of potential variations in channel roughness, following the procedures outlined in the USACE Engineering & Design Manual EM1110-2-1619, Risk-based Analysis for Flood Damage Reduction Studies.

Reasonable bounds were estimated from HEC-RAS model runs utilizing n-values varied above and below the calibrated values by one standard deviation. The standard deviation of the n-value estimate was obtained from a relation with reach mean n-value estimates presented in the above referenced manual, reproduced below as Figure 5.1.

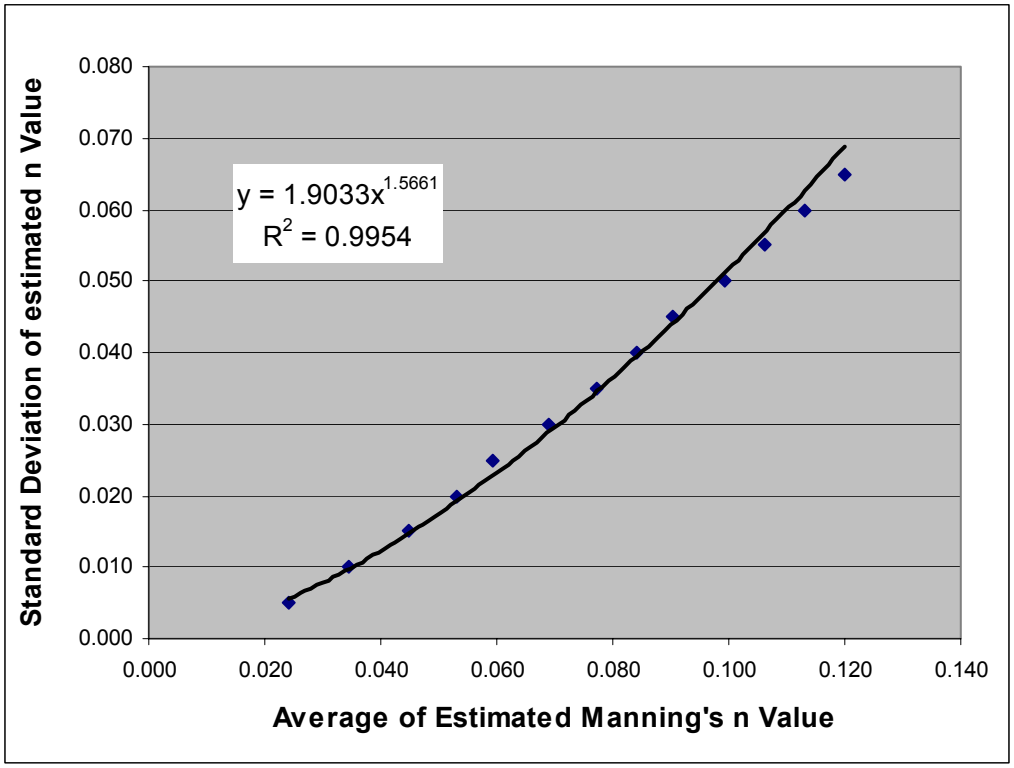


Figure 5.1. Uncertainty of Manning's n-value estimates based on experimental mean values (after Figure 5-4 of EM 1110-2-1619)

Table 5.1 summarizes the adjustments made to the calibrated upper Passaic River model to obtain upper and lower bound estimates of the water surface profile for this analysis. The factors obtained for correcting n-values were applied to every section of the model utilizing the n-value table adjustment option in HEC-RAS.

Table 5.1. Adjustment of n-values for upper- and lower-bound HEC-RAS runs

Cross Section Feature	Average n-value, MV	Upper Limit = MV + SD	Lower Limit = MV - SD	Increase in n	Decrease in n
channel	0.04	0.052	0.028	130.77%	69.23%
overbank	0.13	0.208	0.052	159.97%	40.03%
Use average for HEC-RAS modeling:				**145.37%**	**54.63%**
Factor used for multiplying n-value table:				**1.45**	**0.54**

Figure 5-2, below, shows the upper- and lower-bound water surface profiles for the 100-year design discharge obtained from the variation of n-values by one standard deviation above and below the calibrated values.

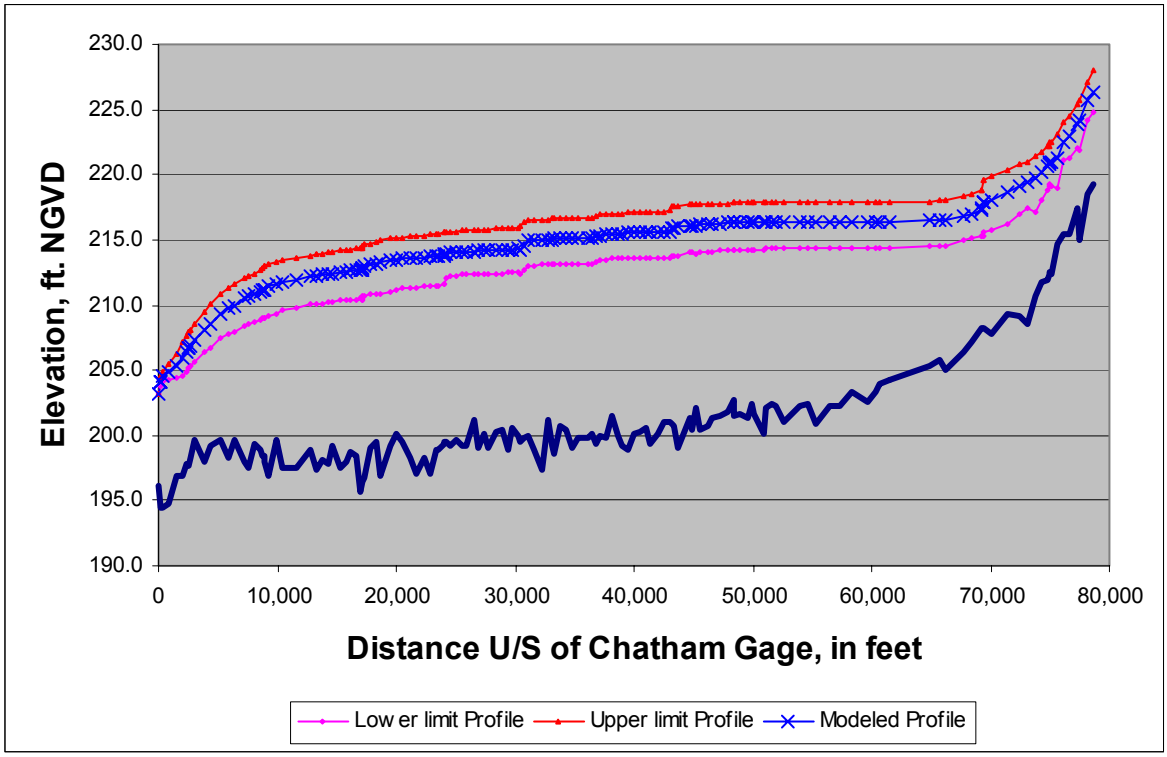


Figure 5.2. Modeled 100-year water surface profile along the upper Passaic River accounting for Standard errors of n-value estimation

The standard deviation of stage uncertainty for each design discharge was estimated as half of the mean stage difference between the upper and lower water surface profiles along the study reach.

For the purposes of this study, the stage uncertainty relationship calculated for HEC-RAS Section #230, at Valley Road near Warren Avenue (approximately 46,170 feet upstream of the Chatham gage) was considered as representative of the entire study reach. Uncertainty relationships may be developed for any other sites within the study area, if required.

Statistics for the representative study reach cross-section are summarized in Table 5.2. The spreadsheet calculation of these statistics is reproduced as Table 5.4, in the Exhibits at the end of this section.

Table 5.2. Mean and standard deviation of stage difference for all design storms

Return Period, years	Mean Stage difference, ft	Standard Deviation, ft	River Stage, in feet *
0	0.00	0.00	204.10
2	2.46	1.23	210.44
5	2.89	1.44	212.19
10	3.13	1.56	213.41
25	3.28	1.64	214.53
50	3.42	1.71	215.22
100	3.50	1.75	216.21
250	3.55	1.78	216.90
500	3.71	1.86	217.57

* Stage for HEC-RAS Station 230, Valley Road near Warren Avenue

The graph of the relation between standard deviation and stage as listed for Section #230 in Table 5.2 is presented in Figure 5.3, below.

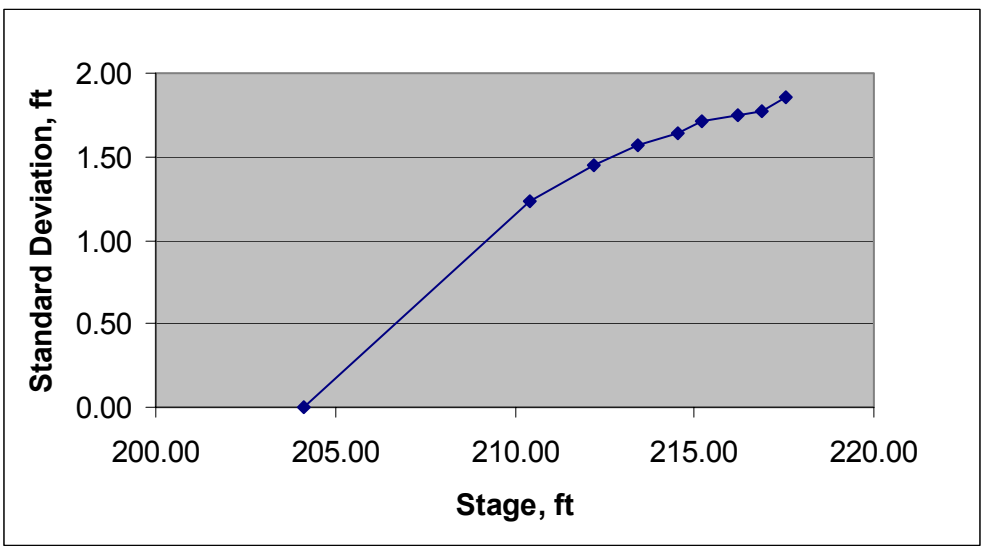


Figure 5.3 Estimated variation of standard deviation with stage for Section 230.

Stage-discharge uncertainty was characterized for the interior flood area using maximum interior water surface elevations determined from the HEC-IFH model, which accounts for back flow from the Passaic River. Because of the tailwater effect of the river on interior stages, the standard deviations associated with the river stages were used for the interior areas as well.

Table 5.3 summarizes the calculation of stage uncertainty for the interior area.

Table 5.3. Stage uncertainty analysis for interior flood areas

Return Period, years	Interior flood Stage, ft	Standard Deviation, ft
0 *	209.00	0.00
2	210.83	1.23
5	212.40	1.44
10	213.32	1.56
25	214.14	1.64
50	214.62	1.71
100	215.25	1.75
250	215.66	1.78
500	216.06	1.86

* Dry interior stage

The tabulated stage-discharge-uncertainty functions, along with the eight flood profiles (2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year return periods) and discharge frequency statistics determined in Task 1a, are provided in HEC-FDA format for the economics task (done by others) to support the analysis of expected annual damage.

It is recommended that the error distribution of the stage uncertainty to be used in the HEC-FDA analysis be Gaussian, following guidance provided in EM 1110-2-1619.

5.1 Exhibits

- Table 5.4. Calculation of mean stage difference on the Upper Passaic River for all design storm return periods
- Table 5.5. Upper Passaic River Profiles for input into HEC-FDA

6.0 DEVELOPMENT OF EXISTING CONDITIONS HEC-IFH MODELS

The Long Hill Township is bounded by the Long Hill Ridge to the north, and the Passaic River to the south. Runoff from the hill slope ponds in the low-lying areas to the east and west of the Stirling town center. Existing channels interconnect the low-lying areas, causing floodwater to "mix" from several of the tributaries during a heavy rainfall event. Three culverts pass the local floodwaters underneath Valley Road and into small natural channels that take the water a short distance (1000-2000 feet) to the Passaic River.

The principal damage centers in the study area are north of Valley Road, which runs parallel to the river and prevents floodwater from high frequency events from entering the town from the riverside, while preventing runoff from the interior area from freely draining to the river.

It is generally agreed that local runoff is not the cause of serious flooding in the study area. Instead, flooding occurs under two conditions:

- The Passaic River is at a high stage and interior floodwaters cannot flow to the river (the less severe condition), or
- Floodwaters from the Passaic River back flood over Valley Road whether or not a local event is occurring (the more severe condition)

The Hydrologic Engineering Center's Interior Flood Hydrology (HEC-IFH) computer program was used to model the interior area to provide data for the development of existing conditions stage-frequency curves and flooding durations for the interior area, to use as a baseline for evaluating alternative flood control strategies.

HEC-IFH includes a runoff model that is essentially the same as HEC-1. It includes a pond routing model that requires the definition of a stage-volume relation for the interior area and the definition of outlet hydraulics functions for the culverts and weirs draining the interior area for headwater and tailwater control.

Accounting for variable tailwater conditions makes it necessary to consider the timing of interior peak runoff with that of peak flow in the Passaic River. Coincident peaking is theoretically possible. However, the Passaic River at Long Hill Township has a much larger drainage area (84 mi^2) than the local drainage (1-2 mi^2), leading to a much longer time to peaking. The large storage volume in the Great Swamp attenuates and further delays the arrival of runoff from the Upper Basin. For these reasons, coincident peaking is extremely unlikely to occur.

The outlet module of HEC-IFH does not account for backflow from the riverside of the interior flood area. Backflow, which is a significant flood process in the study area, was accounted for using the seepage option of the model.

Details of the HEC-IFH model are presented in the following sections.

6.1 Delineation of interior area for the IFH model

The interior area to be modeled is bounded by Valley Road on the south, Mountain Avenue on the east, and high ground on the north and west. Fifty-eight (58) channel cross-sections were taken in the interior drainage areas with two long cross sections taken through the main low-lying areas. Aerial photography of the study area flown in March of 2000 and 2-foot contour maps were developed for most of the study area.

Figure 6.1.1a is an aerial photograph of the study site showing the limits of the roughly rectangular 1.9 square mile interior area, bounded by Valley Road to the south, high ground to the west, Long Hill ridge to the north, and Morristown Road to the West.

Figure 6.1.1b is a line drawing at the same scale showing the location of drainage ditches and culverts surveyed for this study. Elevation contours from the photogrammetric survey are shown for reference. The contour for elevation 214, the approximate 100-year flood elevation in the project area, is shown in yellow. Contours for elevation 220 and above are shown at 20-foot intervals; contours for elevation 212 and below are shown in green.

The topography indicates that the area west of Morristown Road and north of the Erie and Lackawanna Railroad embankment is connected with the interior flood area, but field inspection showed that the connection is a partially filled, small diameter cast iron equalizing culvert with the outboard end largely isolated from the rest of the catchment, which drains east to the Passaic River through culverts under Mountain Avenue. Since this area does not include any development that can be damaged by flooding, and has very little hydraulic connection with the remainder of the study area, it was not included in the IFH model.

Figures 6.1.1a and b also show the location of the three major drainage outlets from the interior area. These include three culverts passing flow under Valley Road towards the Passaic River:

- Crossing #1, 240 feet west of Passaic Avenue,
- Crossing #2, 400 feet east of Warren Avenue, and
- Crossing #3, 500 feet west of Western Boulevard,

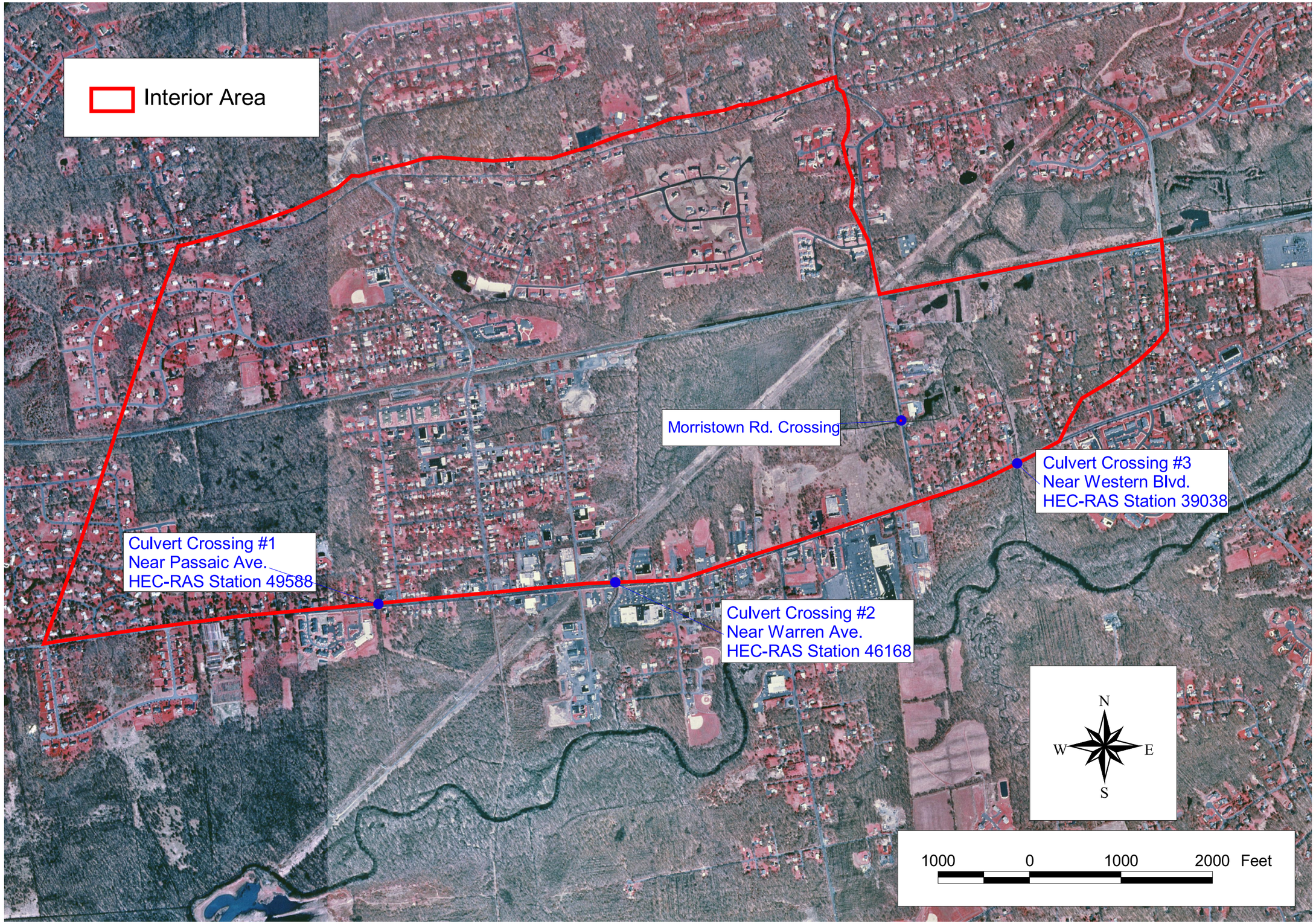


Figure 6.1.1a
Aerial Photograph of the Interior Area

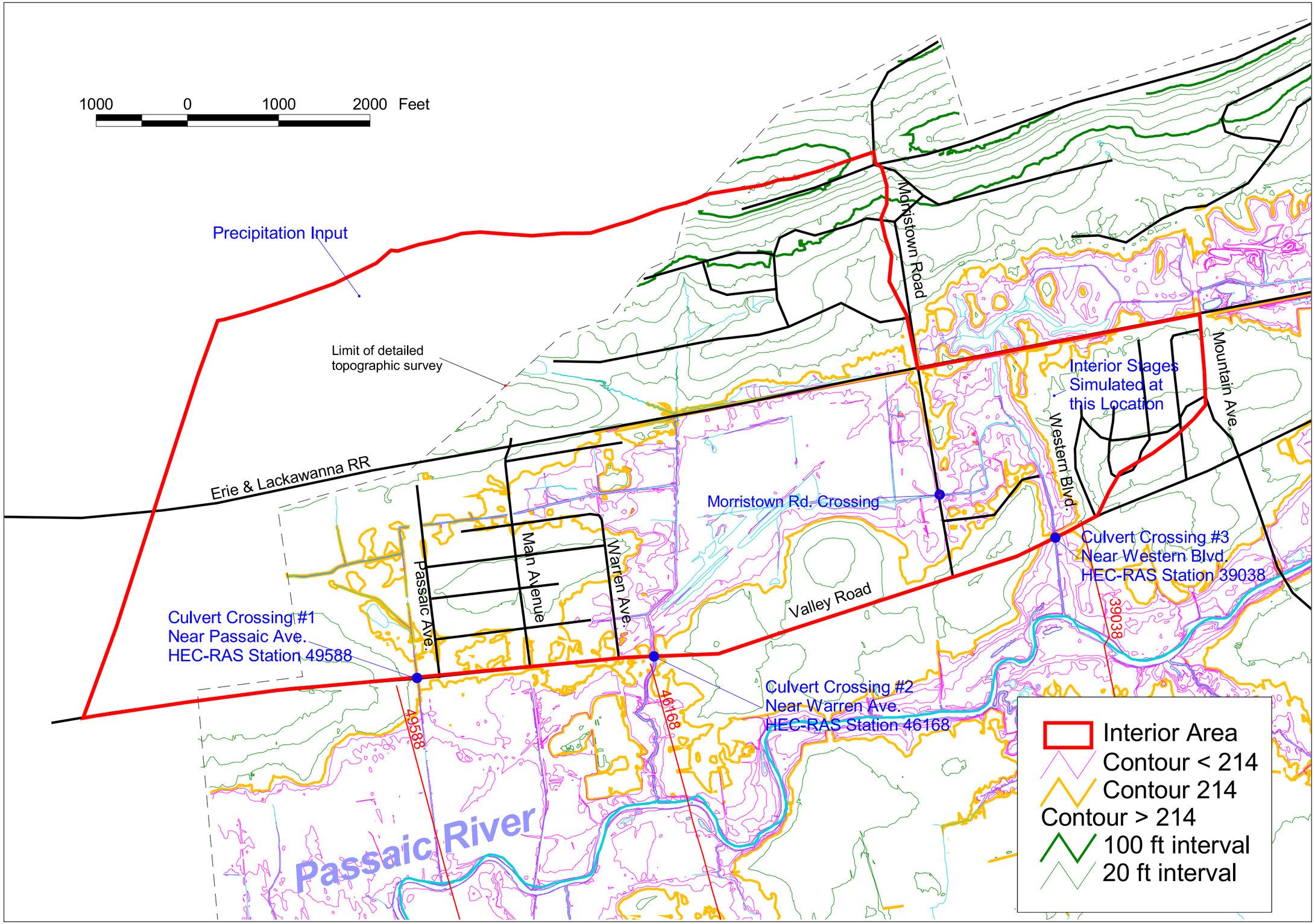


Figure 6.1.1b.
Location diagram for interior area

6.2 **Model development**

Consideration was given to the creation of a separate IFH model for each of the culverts draining the interior area. However, the channels draining to the Passaic and Warren Avenue culverts are connected, and there is a clear drainage pattern towards the Western Boulevard culvert immediately east of the channel draining to the Warren culvert. For any rainfall event that generates a flow greater than the capacity of the interior drainage channels, there is no clear drainage divide between the flooding to these three culverts.

Morristown Road, which runs north and south, is a potential divide between the area that drains directly to the crossing near Western Boulevard, east of Morristown Road, and the rest of the area, west of Morristown Road, which are connected by a double 48-inch concrete culvert (25.2 sq. ft. opening) under Morristown Road (see Table 6.3.1).

In order to determine the effect of Morristown Road on flows within the interior area, two HEC-IFH models were developed with variable tailwater rating curves: one (Model A) for the area east of Morristown Road, and one (Model B) for the area on the west (the backup for these models and analyses are included in the exhibit at the end of Section 6).

The east-area model (A) was run with only the Western culvert outlet, with the Passaic River stage hydrograph for the tailwater, ignoring inflow from the west side. The resulting interior peak elevations were then set as the tailwater on the Morristown culvert outlet for the west-area model (west of Morristown Road/B), and the Passaic River peaks as the tailwater on the Passaic and Warren culvert outlets.

The flow through the Morristown culverts and weir flow over Morristown Road from Model B was then set as auxiliary inflow for Model A. Model A was run, resulting in new interior peak elevations that were assigned to Model B as the tailwater on the Morristown culverts. This iterative process was repeated until the interior peak elevations for the east model, Model A, converged.

Based on these results, it was seen that the head loss for flow in either direction through the culverts and over Morristown road is small, and does not exert a significant hydraulic control on flood stages inside the interior area. Because there is so little difference in the stages calculated for the east- and west-area models, and because the square foot opening of the Morristown Road culvert is almost the same as the pipe under Valley Road at Western Boulevard (meaning that under steady state conditions, the Morristown Road culvert can convey approximately the same amount of water as the Western Boulevard pipe can pass), **it was determined that a single IFH model would be sufficient to provide stage hydrographs for annual effective damage calculations**.

<u>Determination of interior stage-storage relations</u>

Two-foot contours were produced during the photogrammetric survey for this study. These contours were converted to a DEM-grid with ten-foot cells and ArcView GIS was utilized to compute the storage volume within the interior area delineated as described

above. Each cell was assigned a value based on the contour to which it was closest. The area contained by a given contour was determined by summing the amount of cells with an elevation equal to, or less than, the contour and multiplying by 100 square feet, the area of each cell.

Because the elevation value of the cells changes halfway between two contour lines, the area computed was determined to be the area at or below the average elevation of the two contours. For instance, the cells with a value of 212 extend halfway to the 214 contour. Therefore, the area of cells with a value of 212 or lower was assigned to elevation 213. In this way, an area-elevation table was created, and values from it were put into the interior pond input module, which HEC-IFH uses to compute a volume-elevation table from the area-elevation values, as shown in Table 6.2.1.

Table 6.3.1. Volume-area-elevation data for interior flood area

elevation (ft)	area (ac)	volume (ac-ft)
205.9	0.0	0.0
207.0	0.6	0.3
209.0	48.7	49.6
211.0	154.4	252.6
213.0	208.0	615.0
214.0	251.9	845.0
215.0	322.7	1,132.3
217.0	392.4	1,847.3
219.0	433.6	2,673.3
221.0	530.3	3,637.1
223.0	587.7	4,755.1
225.0	610.1	5,953.0
227.0	629.8	7,192.9
229.0	647.1	8,469.9
231.0	659.6	9,776.5
233.0	673.3	11,109.4
235.0	682.8	12,465.5
237.0	705.0	13,853.3
239.0	714.4	15,272.7
241.0	723.9	16,710.9

6.3 Culvert analysis

HEC-IFH requires rating tables for each gravity outlet, consisting of the headwater depth required for a range of tailwater depths and flow rates. As described above, the interior area modeled in IFH is drained by three culverts (and two culverts connect the areas east and west of Morristown Road). Table 6.3.1 summarizes the key hydraulic characteristics of each of these drainage sites.

Table 6.3.1 Culvert outlets & characteristics for Long Hill Township HEC-IFH models

#	Crossing Location	Conveyance	Opening (Sq. Ft.)	Invert Elev.	Minimum T.O.R. Elev
1	Passaic Avenue	48-inch RCP	12.5	209.99	214.0
2	Warren Avenue	9.9x4.4-ft opening	43.6	207.20	213.8
3	Western Boulevard	72-inch RCP	28.3	205.95	215.0
4	Morristown Road	Twin 48-inch RCP	25.2	206.70	211.2

Headwater elevations were computed for each of these culvert for flows between 0 and 500 cubic feet per second for a range of tailwater elevations on the Upper Passaic River associated with return periods between one and five hundred years. The hydraulics of flow over the road when culvert capacity is exceeded was accounted for using weir equations.

Tables of stage-discharge values were compiled in a spreadsheet from the results of multiple runs with Haestad's ***CulvertMaster***; these tables were included on the CD submitted with the IFH models (see list of Interior Flood Hydrograph exhibits at the end of Section 6.

6.4 Consideration of the timing of interior and exterior flood hydrographs

The time to peaking for the interior and the Passaic River hydrographs was compared to determine the factors that produce flooding within the interior area north of Valley Road. Two scenarios were considered:

- The interior runoff hydrograph peaks well before the Passaic River peaks so that interior floodwater drains through the culverts under Valley Road before the Passaic River rises significantly. In this case, either the interior runoff or backwater from the Passaic River would cause the maximum interior flood stages north of Valley Road, but not a combination of the two.

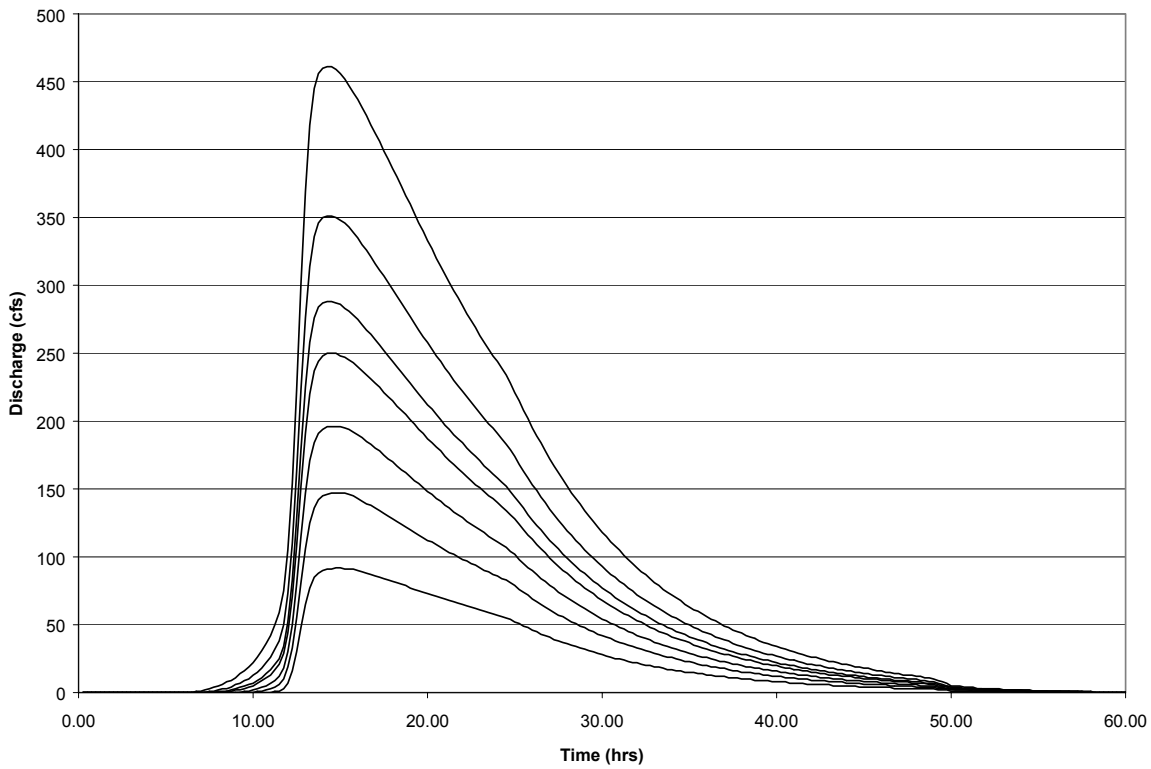
- The Passaic River peaks before the interior floodwater rises causing back-flow through the culverts so that the interior storage volume is reduced by backwater and an initial tailwater condition exists at the culverts. In this case interior flooding is increased over what would occur due to flooding from the Passaic River alone.

The interior runoff discharge hydrograph and interior stage hydrographs calculated without any initial water in the pond and no tailwater effects are shown in Figures 6.4.1 and 6.4.2, respectively, plotted with the same time scale. As expected, the peak of the interior stage hydrograph lags behind the discharge hydrograph peak as headwater surcharge increases at the outlet.

An inspection of Figure 6.4.2 shows that, in the absence of tailwater from the Passaic River, the runoff from the interior does not cause any overtopping of Valley Road.

Discharge hydrographs from the Passaic River near Warren Avenue are shown in Figure 6.4.3, below. These hydrographs were determined in HEC-HMS as described in Task 1B. Subarea 90 was divided into two subareas, and the discharges in Figure 6.4.3 were taken at the junction between these two subareas.

Figure 6.4.1. Interior runoff hydrograph (2-, 5-, 10-, 25-, 50-, 100-, and 500-year events)



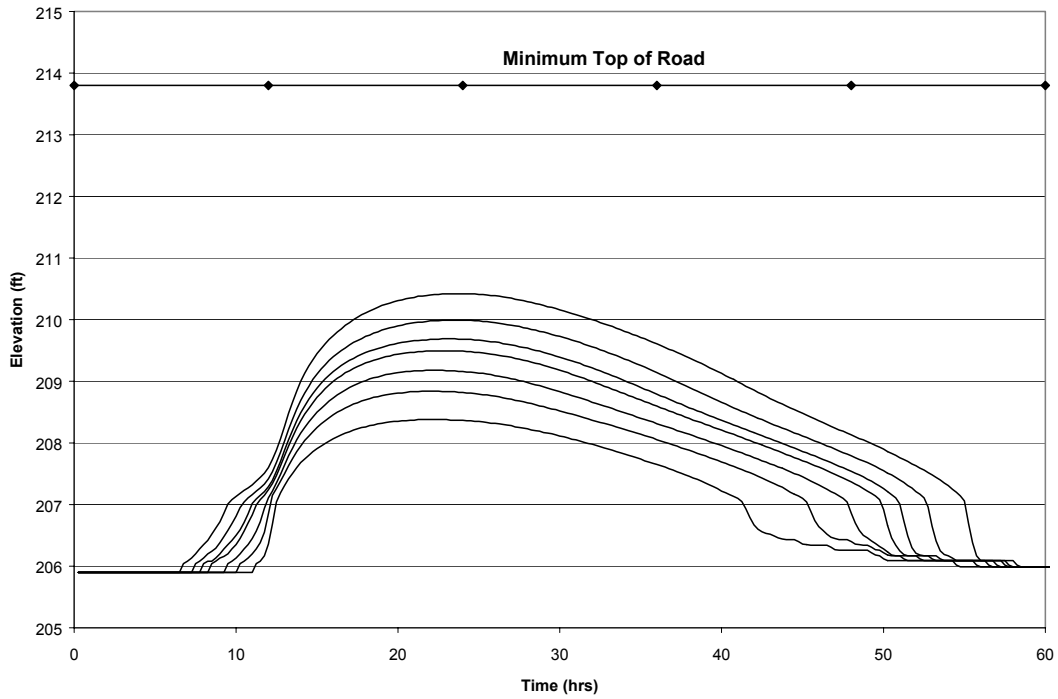


Figure 6.4.2. Interior pond stage hydrograph for 2-, 5-, 10-, 25-, 50-, 100-, and 500-year events, ignoring tailwater effects from the Passaic River.

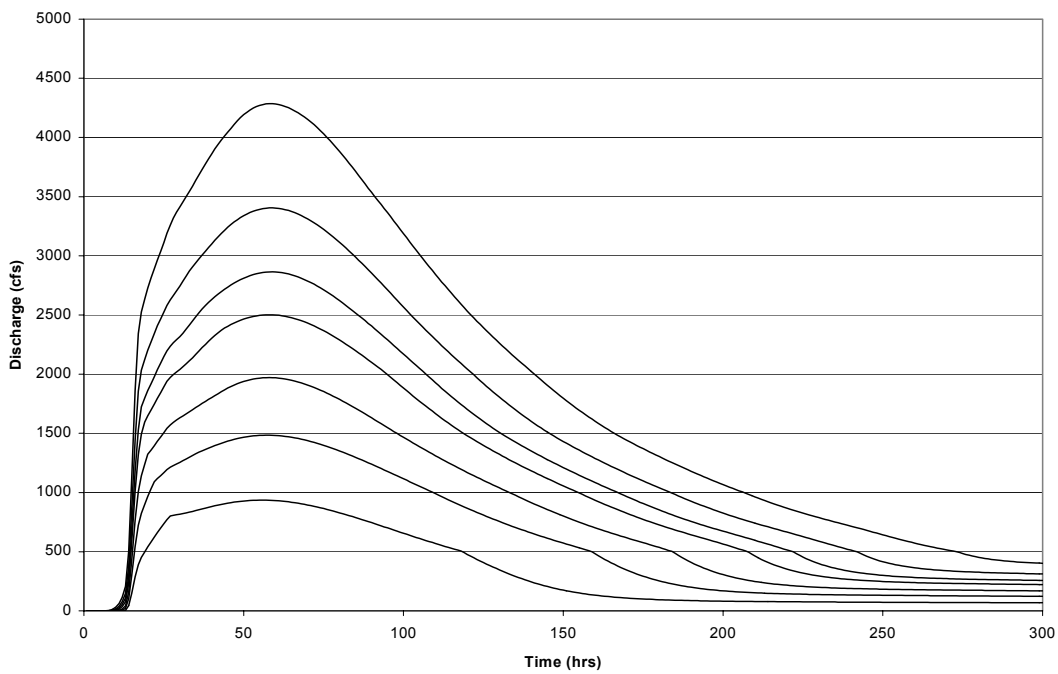


Figure 6.4.3. Passaic River discharge hydrograph at Western Avenue for 2-, 5-, 10-, 25-, 50-, 100-, and 500-year events.

As seen in Figure 6.4.3, the peak flow in Passaic River at the project site occurs about 24 hours after the start of rainfall, with an ultimate peak after about 60 hours. The interior elevation peaks at about 24 hours, and takes up to 55 hours to recede for the condition of no tailwater - the interior flood will be receding by the time the Passaic River peaks.

HEC-IFH models were produced for various initial storage and tailwater (Passaic River) elevations. Stage hydrographs for the Passaic River at each culvert crossing were developed using discharge-elevation relationships, shown in Table 6.4.1 below, of the Passaic River cross section nearest each culvert crossing.

Table 6.4.1. Discharge-Elevation Rating Table for the Passaic River

Discharge (cfs)	Water Surface Elevations (ft)		
	Culvert near Western	Culvert Near Warren	Culvert Near Passaic
0	203.50	204.10	204.50
500	207.89	208.54	208.93
1000	210.06	210.62	210.87
1500	211.67	212.19	212.35
2000	212.87	213.41	213.53
2500	213.86	214.42	214.52
3000	214.73	215.35	215.47
3500	215.56	216.27	216.35
4000	216.31	217.03	217.10
4500	216.99	217.79	217.87
5000	217.52	218.41	218.48

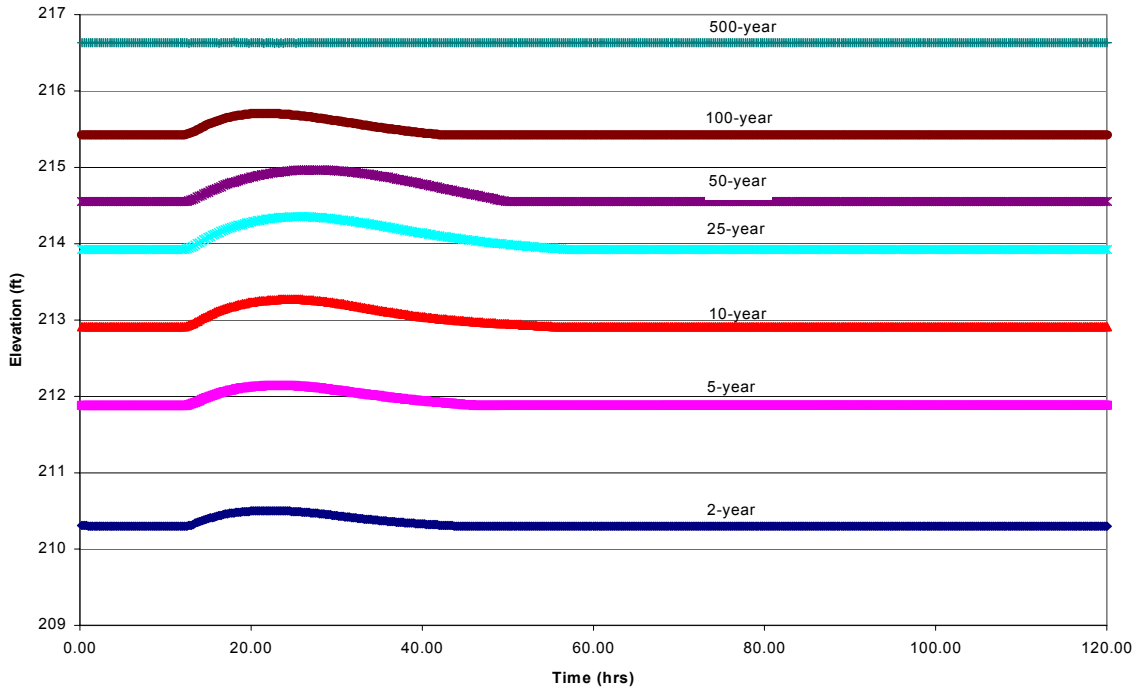


Figure 6.4.4. Interior stage hydrographs for tailwater and pond elevations equal to Passaic River 36-hour peak.

One group of HEC-IFH models was run assuming the elevation of the Passaic River and the initial pond elevation were equal to the peak elevation of the Passaic River that occurs in the first 36 hours. The stage hydrographs for this condition are shown in Figure 6.4.4.

6.5 Final Model Analysis

Setting the interior and tailwater elevations to the 36-hour peak of the Passaic River is conservative, and not a likely scenario. For it to occur, a rainfall event would have to fall on the upper area of the watershed approximately 36 hours before the storm begins on the interior area. To model the more probable scenario, a HEC-IFH model was developed with no initial interior storage, and a variable tailwater elevation at the culvert crossings corresponding to the stage hydrographs of the Passaic River. The stage hydrographs for the Passaic River near Western Boulevard are shown in Figure 6.5.1, and the maximum Passaic River stages at each culvert for each design rainfall are shown in Table 6.5.1. The maximum elevations from the 36-hour scenario are also included in Table 6.5.1.

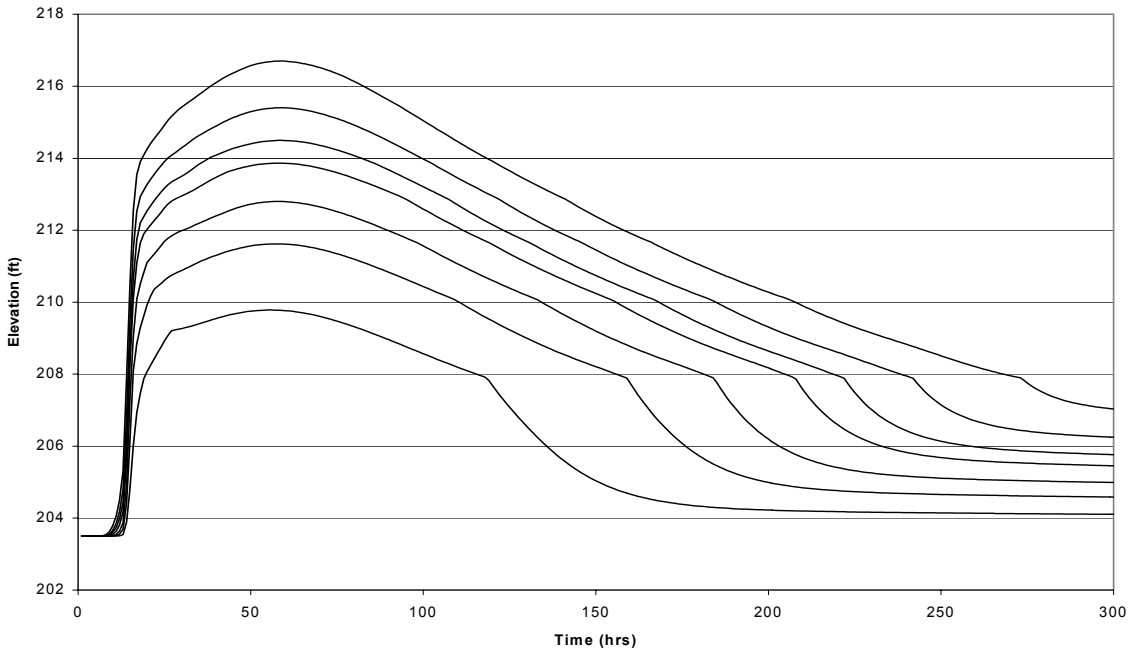


Figure 6.5.1. Passaic River stage hydrographs near Western Boulevard.

Table 6.5.1. Maximum stages of Passaic River at each culvert crossing (with backflow)

Recurrence Interval	Maximum Stage (ft)			
	Culvert near Western	Culvert Near Warren	Culvert Near Passaic	Interior peaks for 36-hour scenario
2-YR	209.78	210.35	210.62	210.5
5-YR	211.62	212.14	212.30	212.14
10-YR	212.80	213.34	213.46	213.27
25-YR	213.86	214.42	214.52	214.35
50-YR	214.49	215.10	215.21	214.96
100-YR	215.40	216.10	216.18	215.7
500-YR	216.70	217.47	217.54	216.64

As seen in Table 6.5.1, the conservative 36-hour scenario produces interior stages less than the Passaic River stage. Therefore, backflow from the Passaic River is an issue. HEC-IFH does not include an option for backflow, but it has a seepage module, which enables the user to specify seepage into the interior area as a function of differential head. So backflow through the outlets under Valley Road was simulated with the seepage module, using culvert rating tables to provide the differential head-discharge relation.

Using no initial storage and the Passaic River stage hydrographs as the tailwater on the outlets, and considering backflow through the outlets, interior peak stages were computed at each culvert outlet as shown in the stage-frequency graph shown in Figure 6.5.2.

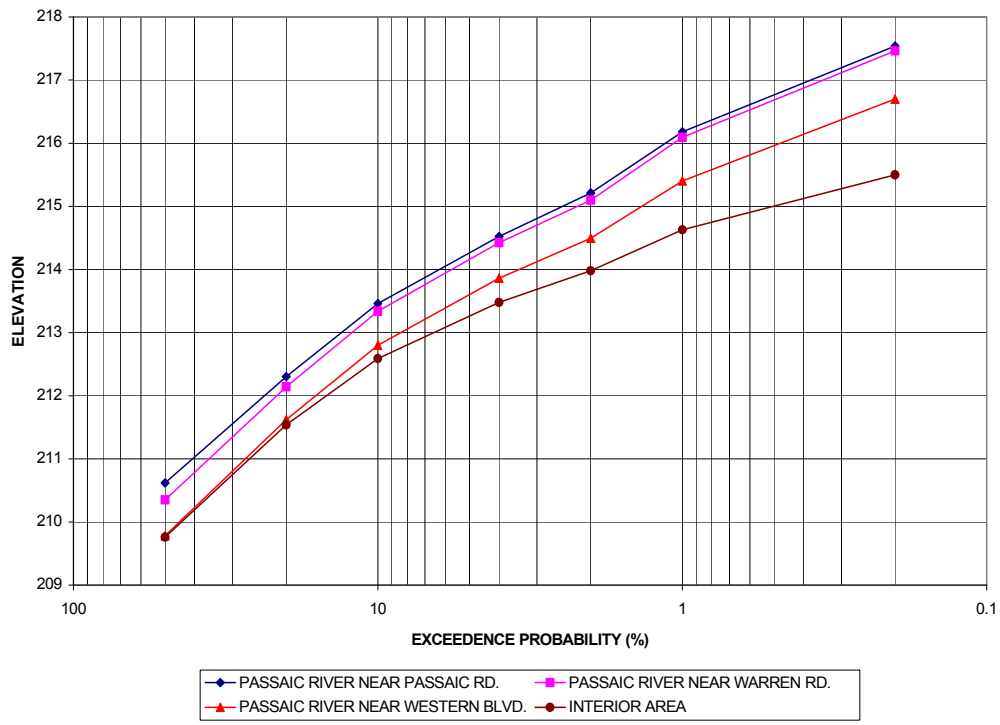


Figure 6.5.2. Stage-Frequency results for the interior area and the Passaic River at each outlet under Valley Road

To determine the extent to which the Passaic River controls flooding within the interior area, a model was run with no backflow through the outlets. Table 6.5.2 lists the peak elevations of the interior area with and without backflow, and the peak stage of the Passaic River at the outlet near Western Boulevard.

Table 6.5.2. Peak stages for Passaic River and interior area with, and without, backflow

Recurrence Interval	Maximum Stage (ft)		
	Passaic River near Western	Interior Area without backflow	Interior Area with backflow
2-YR	209.78	209.71	209.76
5-YR	211.62	210.34	211.54
10-YR	212.80	210.76	212.59
25-YR	213.86	211.16	213.48
50-YR	214.49	211.41	213.98
100-YR	215.40	211.83	214.63
500-YR	216.70	212.47	215.50

The 100-year stage hydrograph is shown in Figure 6.5.3, below. Stage hydrographs for all recurrence intervals are included in the exhibit at the end of this section.

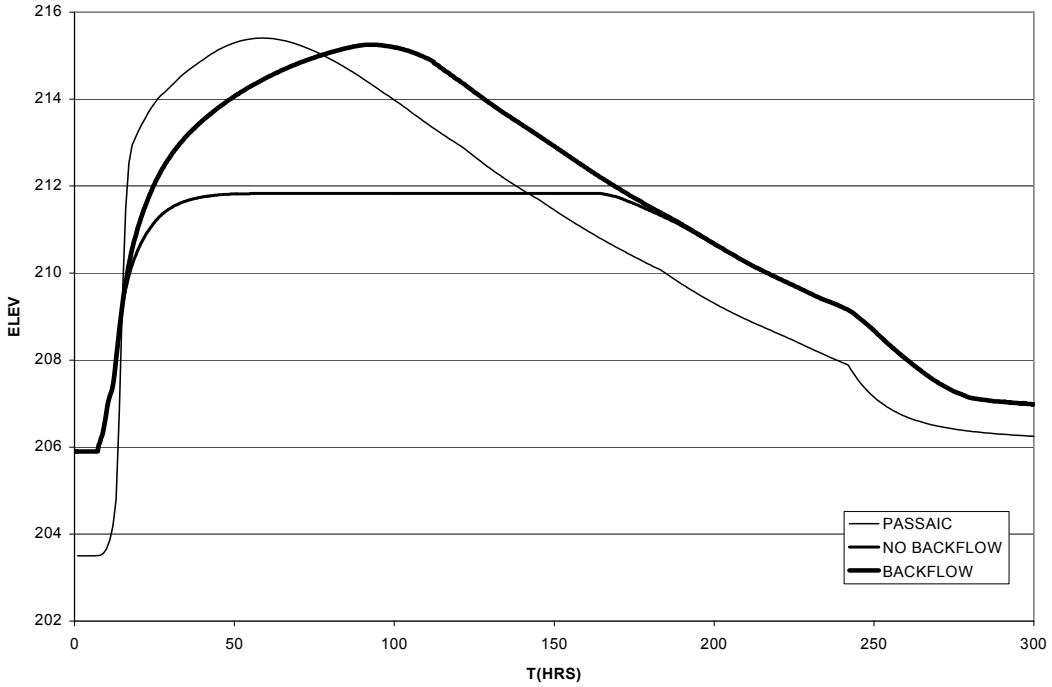


Figure 6.5.3. Stage hydrographs of Passaic River and interior area for with, and without, backflow

6.6 Interior Flood Hydrograph Modeling Exhibits

Interior Flood Stage Hydrographs

- 2 year
- 5 year
- 10 year
- 25 year
- 50 year
- 100 year
- 500 year

Haestad ***CulvertMaster*** Runs for IFH input

All HEC-IFH files run developed for this study are included in the attached CD.

7.0 Task 1f. Assessment of stability of existing channel

An assessment of the relative stream stability was completed through a comparison of the channel cross sections and the profiles of the minimum channel elevation from the 1977 FEMA study and the year 2000 survey (see attached). A comparison of the minimum channel elevation profiles shows obvious differences that seem to indicate a pattern of net erosion and deposition of sediment through time. This comparison seems to exhibit a general pattern of erosion upstream and around bridges with larger areas of deposition downstream of bridges. Additionally, there appears to have been significant channel deepening at the most upstream reach of the cross sections surveyed in the year 2000. However, the general trend in minimum channel elevations seems to be associated more with the position of bridges along the profile.

A comparison of the channel cross-section geometry (Table 7.1) seems to support the patterns evident in the channel profiles. Cross sections from the 1977 FEMA study and the year 2000 survey were compared for visually evident trends in channel elevation changes. Additionally, local areas where there are sharp bends in the channel generally exhibited increased channel deepening as compared to relatively straight areas where increased deposition was apparent. Overall, the comparison of cross sections and stream centerline profiles is inconclusive and does not indicate the type of significant trend for deposition that could cause an aggravation of flooding within the study reach.

Table 7.1. Results of comparing 1977 and 2000 cross sections.

1977 FEMA Study Long Hill Twp FIRM	2000 Survey	Approximate Change in Minimum Channel Elevation	Notes
A (341475)	M7	-0.5 ft	Located upstream of Snyder Bridge and at bend in channel.
B (344625)	M14	None Apparent	
C (347090)	M18	+0.5 ft	Located at upstream approach to Conrail bridge where channel is relatively straight.
D (349715)	M23	+1.0 ft	Located downstream of Mountain Ave bridge midway to Conrail bridge, channel relatively straight.
E (350850)	M25	+1.0 ft	Located just downstream of Mountain Ave, channel relatively straight.
F (352755)	M28	-1.5 ft	Located just upstream of Mountain Ave
G (356690)	M37	None Apparent	
H (359030)	M43	None Apparent	
I (362230)	M49	None Apparent	
J (365220)	M54	+1.0 ft	Located in far upstream reach of study area

The 1977 and 2000 cross-sections referenced in Table 7.1 are shown in Figures 7.1 through 7.10 on the following pages.

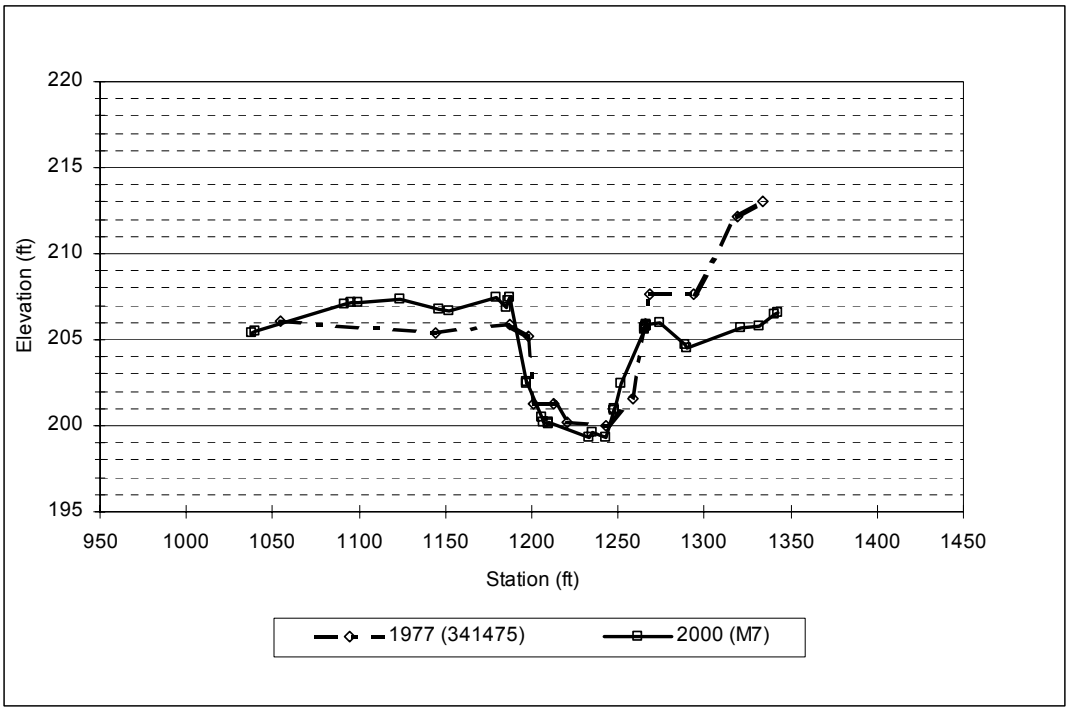


Figure 7.1. Comparison of 1977 and year 2000 cross-sections at station A (3414+75)

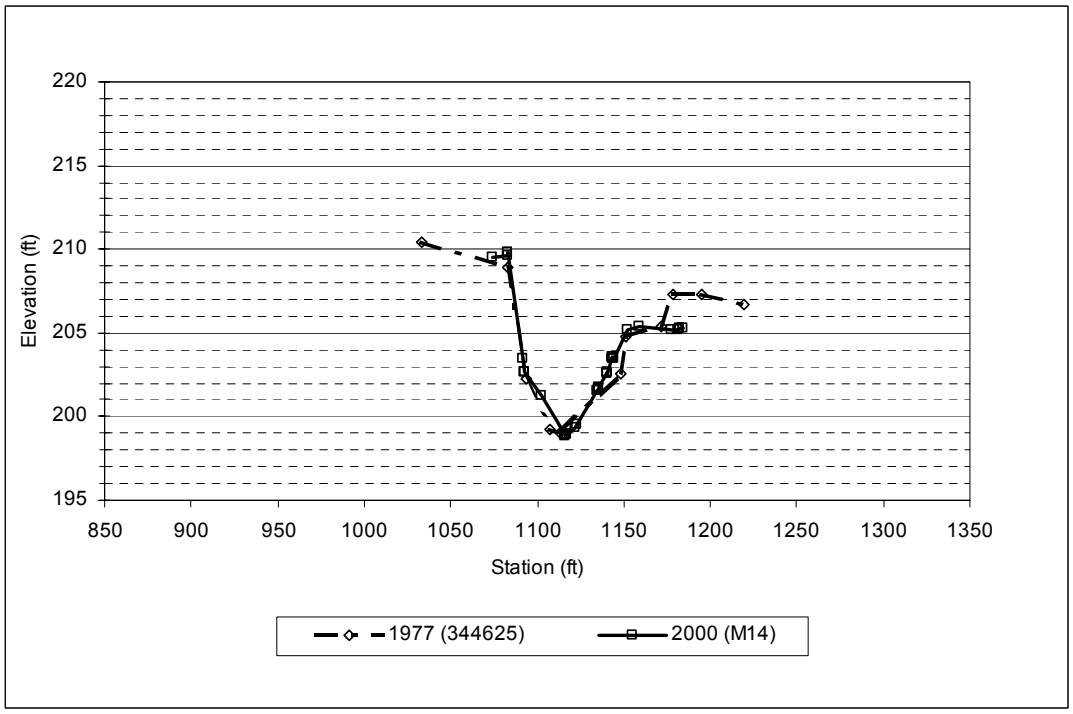


Figure 7.2. Comparison of 1977 and year 2000 cross-sections at station B (3446+25)

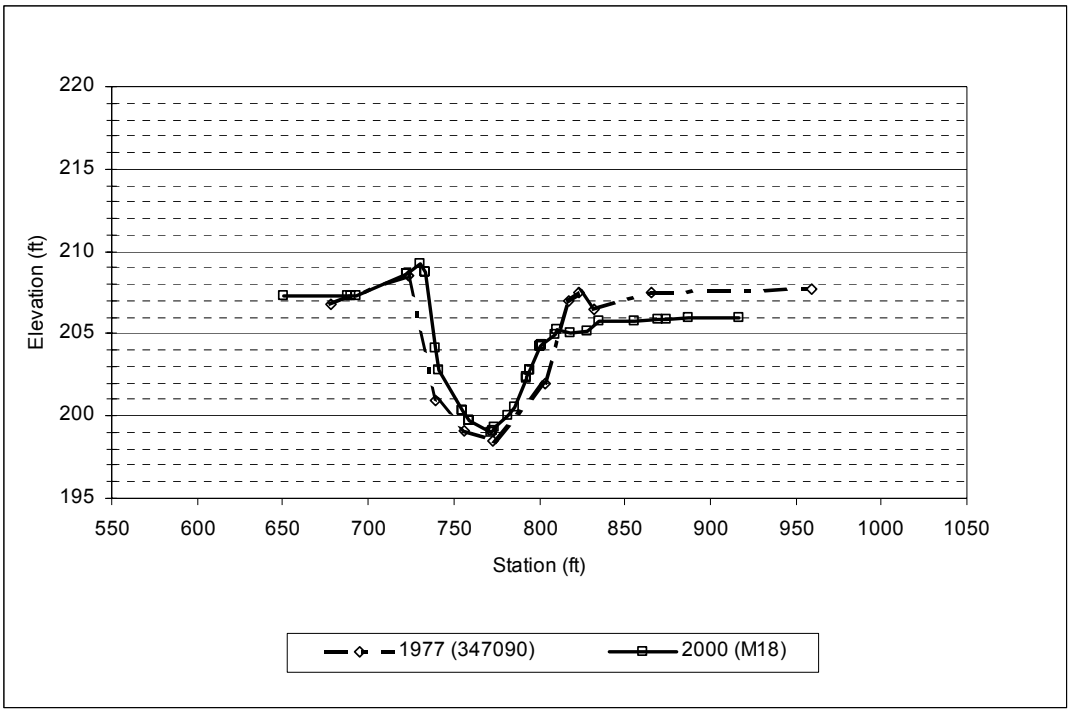


Figure 7.3. Comparison of 1977 and year 2000 cross-sections at station C (3470+90)

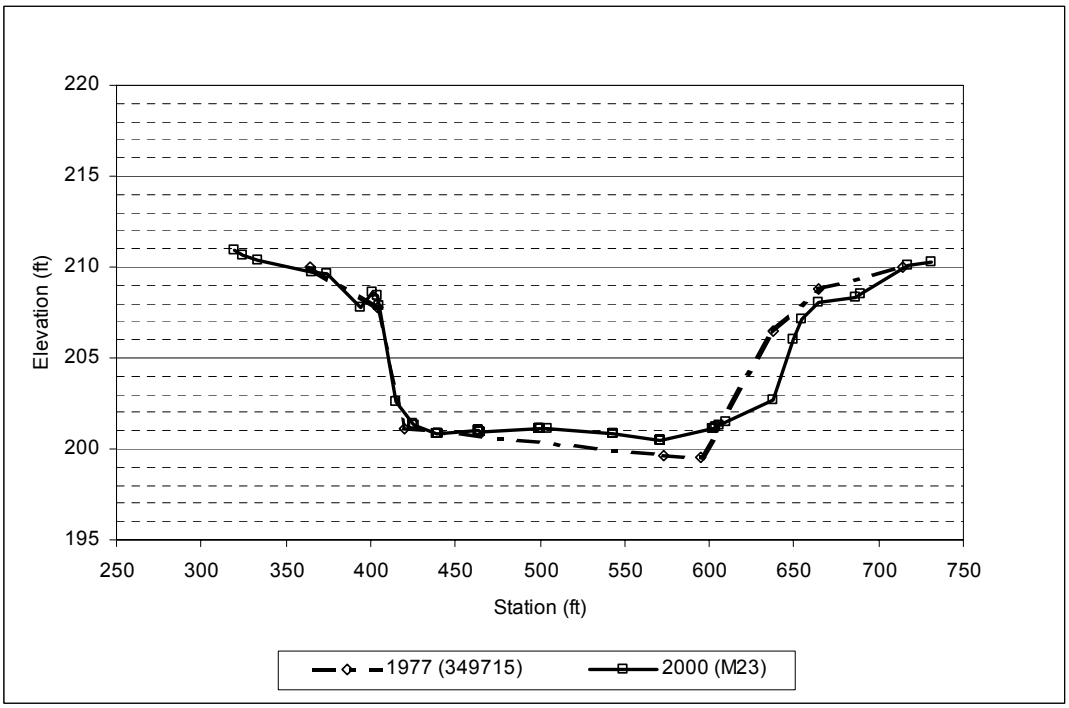


Figure 7.4. Comparison of 1977 and year 2000 cross-sections at station D (3497+15)

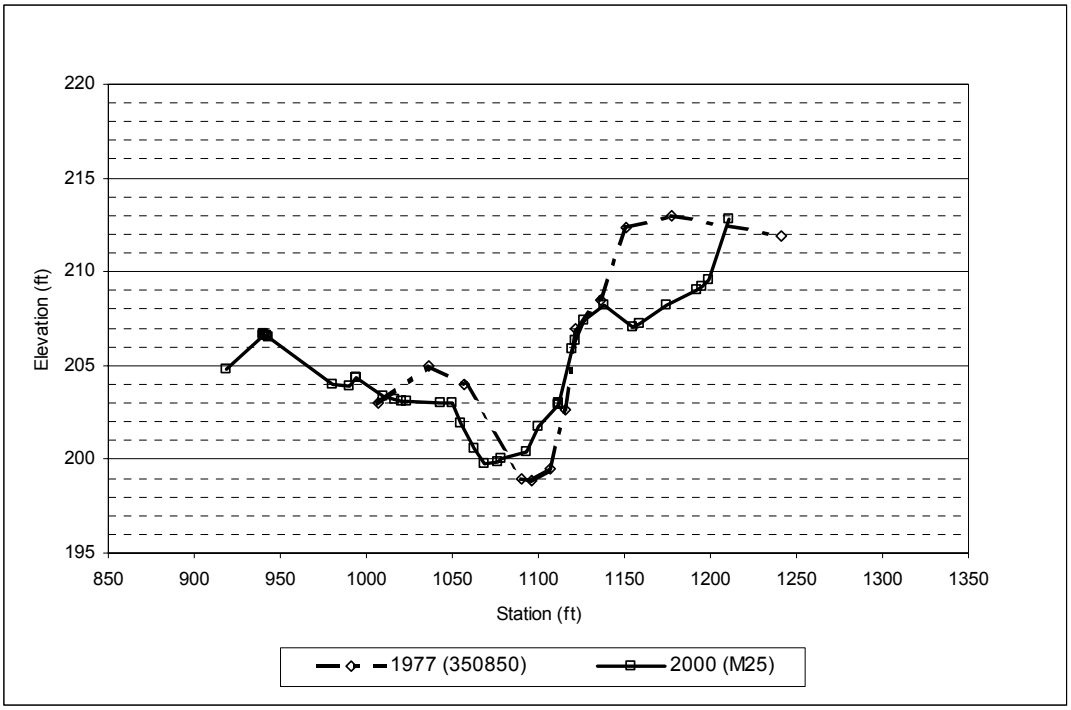


Figure 7.5. Comparison of 1977 and year 2000 cross-sections at station E (3508+50)

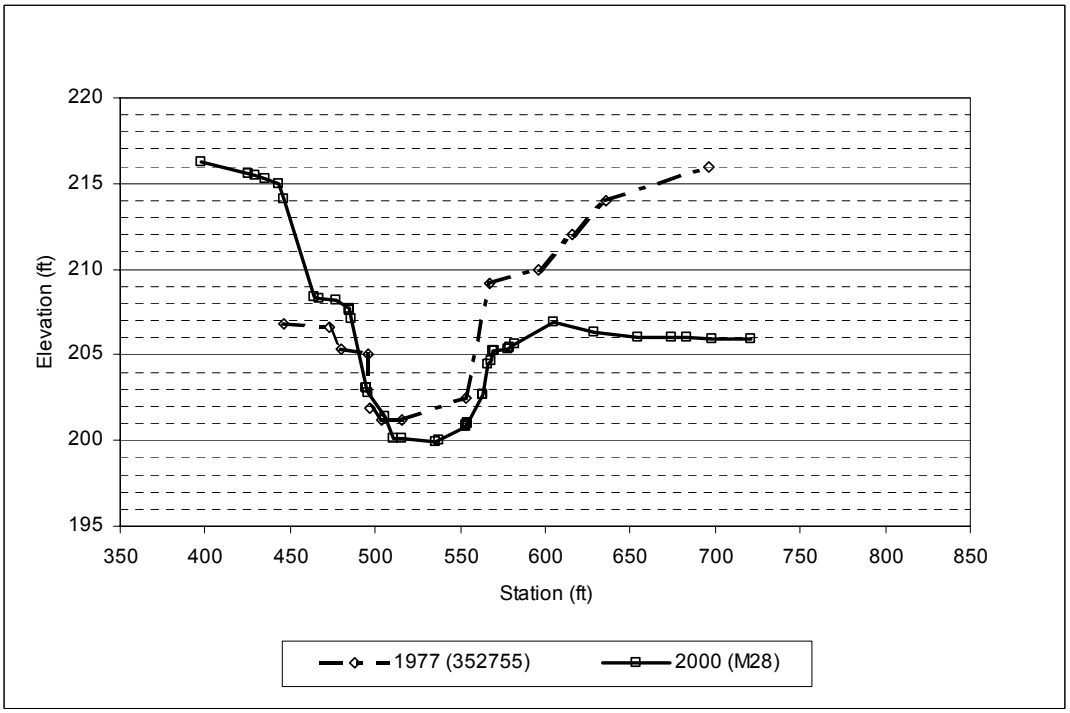


Figure 7.6. Comparison of 1977 and year 2000 cross-sections at station F (3527+55)

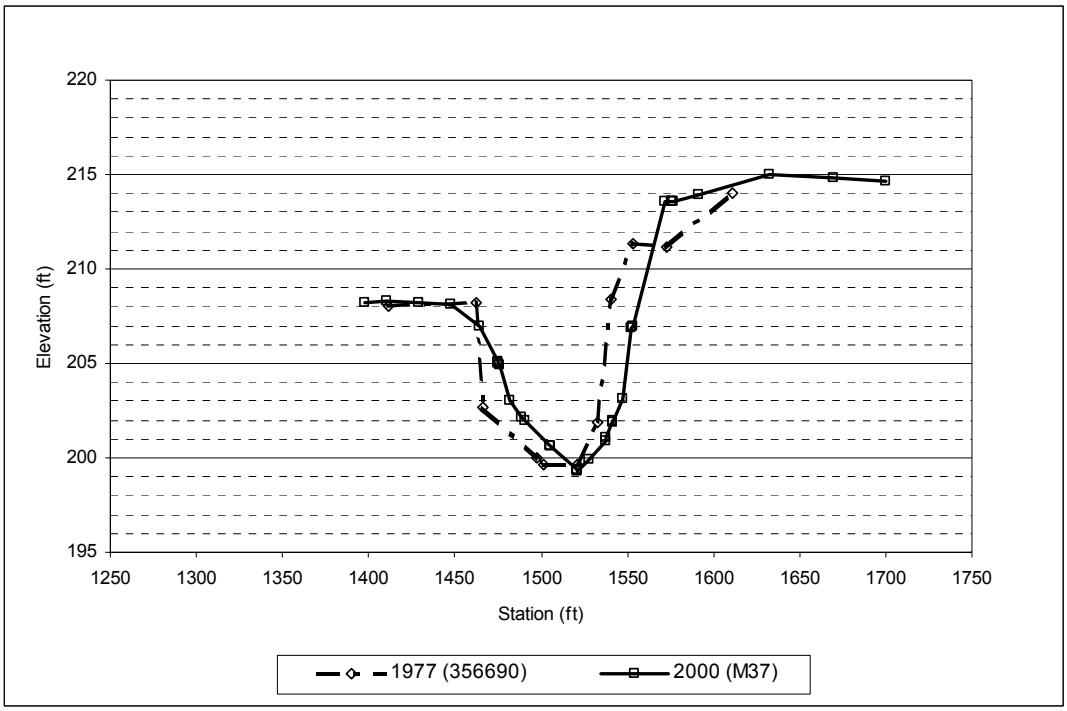


Figure 7.7. Comparison of 1977 and year 2000 cross-sections at station G (3566+90)

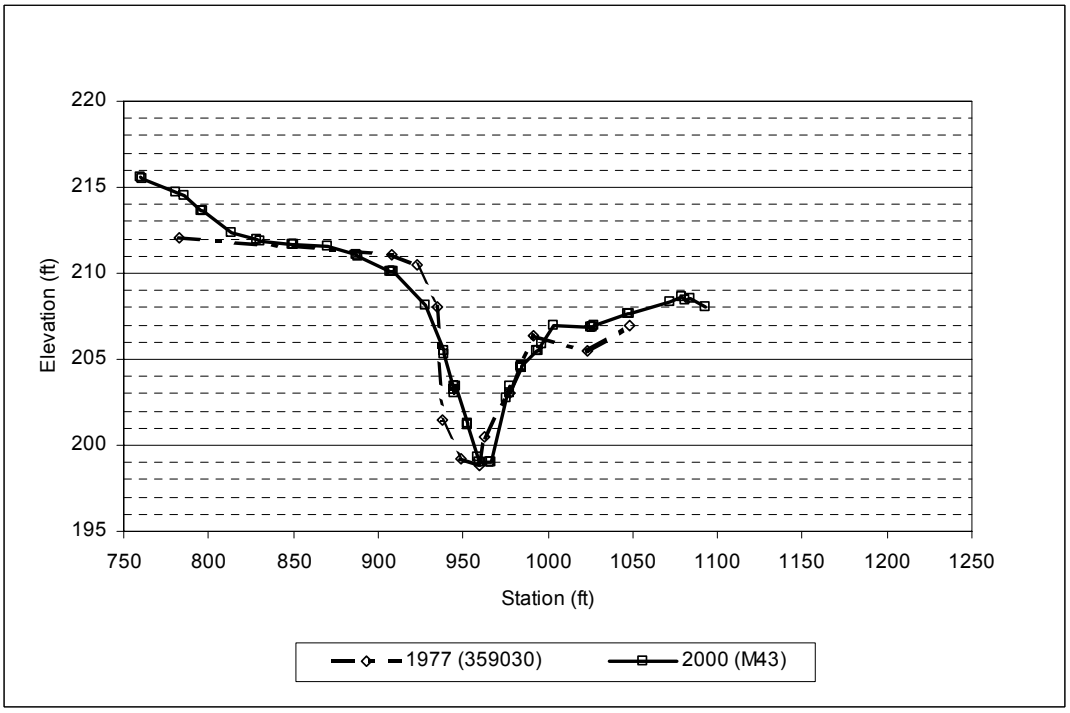


Figure 7.8. Comparison of 1977 and year 2000 cross-sections at station H (3590+30)

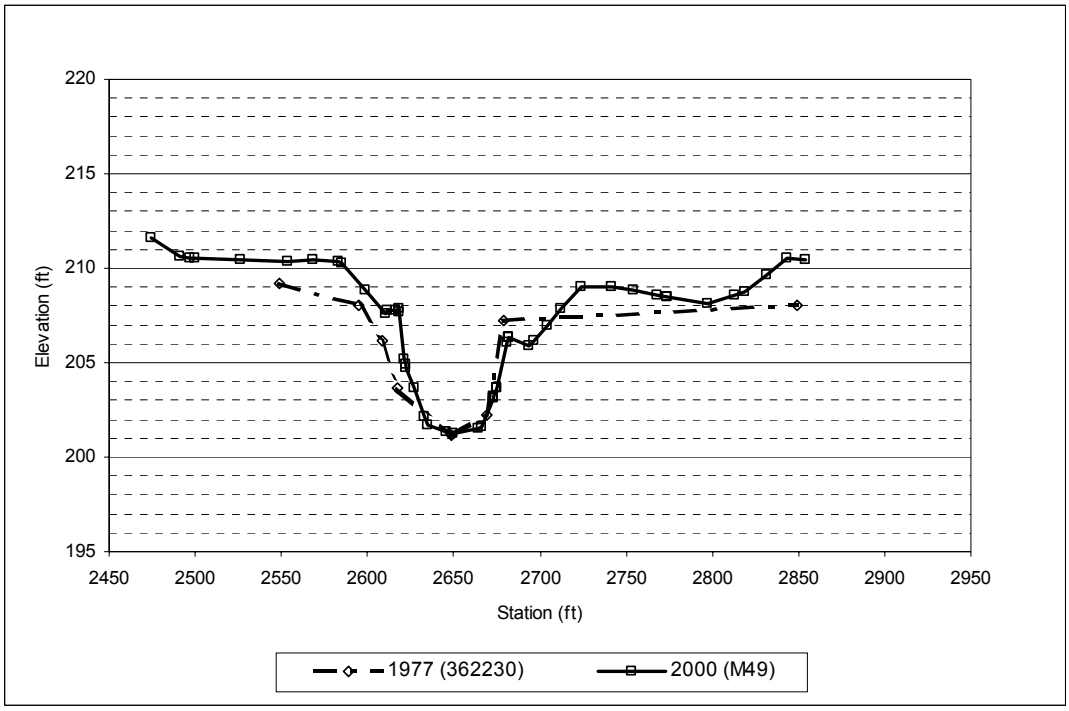


Figure 7.9. Comparison of 1977 and year 2000 cross-sections at station I (3622+30)

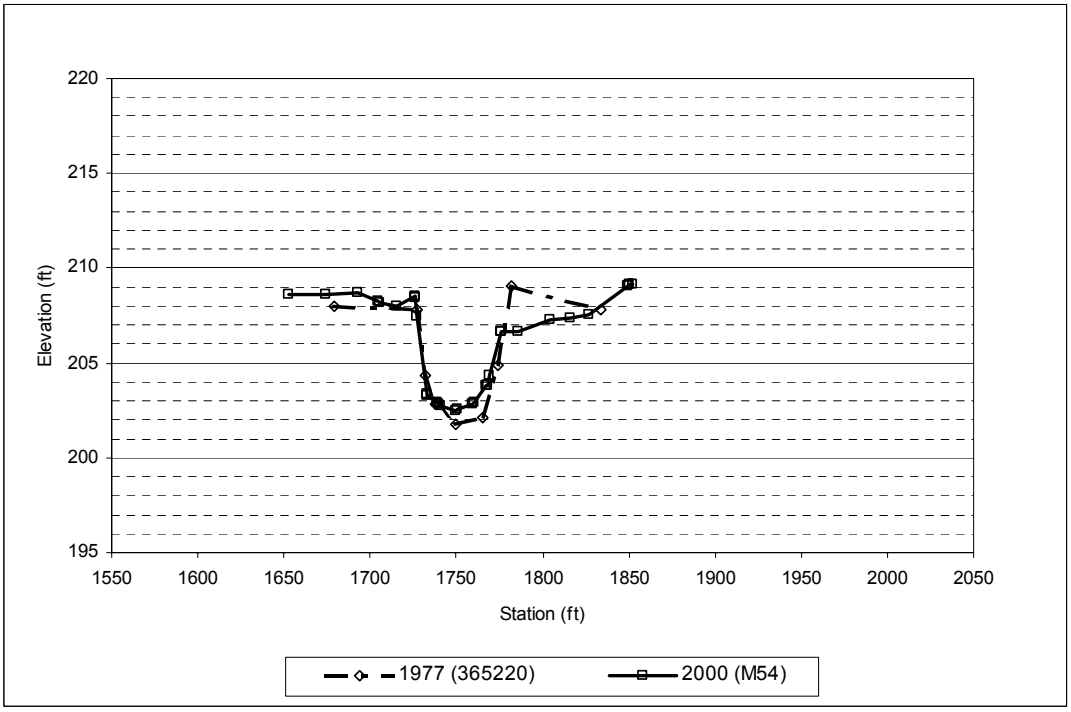


Figure 7.10. Comparison of 1977 and year 2000 cross-sections at station J (3652+20)

Preliminary field investigations of the study site were conducted in December 2001 and January 2002 to assess the current channel stability. Sediment samples were collected at several locations within the study reach and examined for a determination of an approximate range of particle sizes. Spreadsheet calculations for incipient motion were also developed to identify zones of potential deposition and erosion (see Exhibit 1). The reach of the Passaic River from the Chatham gage to the Millington gage was analyzed for the 2-, 10-, and 100-year flood events over a range of sediment particle sizes. Using the hydraulic depth for each cross section from the HEC-RAS output and an approximate D50 particle size, the following formula was used to compute the critical velocity for incipient motion (Transportation Research Board):

Critical Velocity = (11.17) * (Hydr. Depth^1/6) * (D50^1/3)

The calculated critical velocity was compared to the average channel velocity from the HEC-RAS output. If the average velocity in the channel is greater than the critical velocity for incipient motion, then sediment particles of that size will be transported. Conversely, if the average velocity in the channel is less than the critical velocity for incipient motion, then sediment particles of that size will not be transported. The attached calculations identify some obvious characteristics throughout the reach from the Chatham gage to the Millington gage.

For the full range of flood events considered, the reach of the Passaic River within the study area, from just downstream of Snyder Avenue to approximately the Corys Brook confluence, does not have the capacity to transport sediment for any particle size except those within the silt range. 100-year velocities within this reach are generally below 1 foot per second, with some exceptions at the bridges. Both upstream and downstream of the study area, where the profile is significantly more steep, the stream does have the capacity to transport sediment for all the flood events and particle sizes considered.

These calculations suggest that the potential exists for sediment deposition on the reach of the Passaic River within Long Hill Township. An evaluation of the sediment availability within the upstream reaches and the watershed in general has not been conducted. The comparison of the cross sections and stream centerline profile were inconclusive for determining if significant deposition occurred over the period from 1977 to the year 2000. It is possible, however, that some deposition could have been occurring over the years, only to be have been washed away during the 1999 Floyd event.

EXHIBIT 1.

- Spreadsheet-based incipient motion calculations for the identification of potential deposition and erosion zones for 2-year return period discharges
- Spreadsheet-based incipient motion calculations for the identification of potential deposition and erosion zones for 10-year return period discharges
- Spreadsheet-based incipient motion calculations for the identification of potential deposition and erosion zones for 100-year return period discharges

8.0 DEVELOPMENT OF FUTURE CONDITIONS NO PROJECT HYDROLOGIC MODEL

Peak runoff in the upper Passaic River watershed is not expected to increase by a great deal by 2050: most of the urbanized area of the upper watershed is close to ultimate development for existing (2000) conditions; much of the rest of the watershed is protected wetlands for which no future development is anticipated. Table 8.1 summarizes the Corps' previous estimate of changes to peak discharge by 2050.

Table 8.1. Previous COE Estimate of changes to peak discharge at Chatham for future (2050) conditions. Data summarized from Table C-31 of 1995 General Design Memorandum, Appendix C H&H

Return Period	Q peak, cfs		
	Existing (1990)	Future (2050)	Increase
10	2,450	2,570	4.9%
25	3,020	3,150	4.3%
50	3,420	3,550	3.8%
100	4,020	4,170	3.7%
500	4,970	5,120	3.0%

The HEC-HMS model for future conditions was developed by increasing the calibrated existing conditions model Run-off Curve Numbers model by the estimated percentage increase in runoff obtained from the HYDPAR analysis conducted by the COE in 1995. Table 8.2 shows the changes in the RCN estimated for the future conditions model.

Table 8.2. Increases in RCN parameters for Future Conditions HEC-HMS modeling

HEC-HMS Subbasin, present study	Drainage area, mi2	HYDPAR ANALYSIS, 1995 COE Study					HEC-HMS Modeling, present study	
		Basin ID, previous modeling	1990	2000 base year (2)	2050	Ultimate development increase (3)	Calibration Value (4)	Increase to Calibration Value (5)
0010	3.44	0010	77	**77**	78	1.30%	69	70
0015	5.23	0015	69	**70**	73	4.29%	63	66
0020	4.35	0020	80	**80**	80	0.00%	72	72
0030	23.06	0030	76	**76**	77	1.32%	68	69
0040	13.50	0040	79	**79**	79	0.00%	71	71
0050	1.66	0050	81	**81**	81	0.00%	75	75
0060	2.87	0060	76	**76**	78	2.63%	68	70
0070	2.84	0070	79	**79**	80	1.27%	71	72
0080	19.30	0080	74	**75**	78	4.00%	70	73
0091 (1)	7.71	0090	77	**78**	80	2.56%	72	74
0092 (1)	5.04	--	--	--	--	2.56%	70	72
0100	3.78	0100	76	**76**	77	1.32%	70	71
0110	4.45	0110	77	**77**	77	0.00%	71	71
0120	1.67	0120	64	**65**	67	3.08%	64	66

1) Subdivision of 1990 watershed to improve H&H resolution for this study
2) Straight-line interpolation; see Task 1 Report
3) Percent increase, 2000 to 2050.
4) Value of RCN value as a result of calibration; see Task 1 Report
5) Increase to calibration value set equal to ultimate development increase

The future conditions HEC-HMS model utilizes the same subshed and node topology as the existing conditions model, as shown in Figure 3.2.1 and Figure 3.2.2, respectively, earlier in this report.

To estimate future conditions hydrology, the existing conditions HEC-HMS model was rerun with the future conditions runoff curve numbers presented in Table 8.2. The peak discharges simulated for Chatham increased by between 3 and 6 percent, with the greater increases occurring for the higher frequency events. These increases are in line with the values estimated in the 1995 COE study, as referenced in Table 8.1. Increases at other nodes were of a similar order of magnitude, as is shown in Table 8.3.

Table 8.4, at the end of this Section, includes the information from the HEC-HMS summary output tables for all nodes and reaches in the model for all future condition no project design storms runs.

Table 8.3. Comparison of Existing and Future Conditions No-project HEC-HMS model output for selected nodes

		2105, Millington (not in HEC-RAS model)	2106, U/S of Dead River (60,292 ft. U/S of start)	2107, D/S of Dead River (59,587 ft. U/S of start)	91J, Plainfield (44,730 ft. U/S of start)	92J, Snyder Road (25,482 ft. U/S of start)	2109, Passaic Rd./Long Hill (9,952 ft. U/S of start)	2110, Chatham (HEC-RAS Station 0)
2-year Event	Existing (2000)	732	664	990	936	947	990	1,000
	Future (2050)	747	679	1,052	991	1,004	1,050	1,061
	% Increase	2.0%	2.2%	6.2%	5.9%	6.0%	6.0%	6.1%
5-year Event	Existing (2000)	1,127	1,036	1,608	1,485	1,509	1,582	1,605
	Future (2050)	1,148	1,055	1,688	1,555	1,584	1,658	1,683
	% Increase	1.9%	1.8%	5.0%	4.7%	5.0%	4.8%	4.9%
10-year Event	Existing (2000)	1,471	1,354	2,130	1,971	2,010	2,105	2,134
	Future (2050)	1,495	1,377	2,222	2,054	2,095	2,197	2,229
	% Increase	1.6%	1.7%	4.3%	4.2%	4.2%	4.4%	**4.5%**
25-year Event	Existing (2000)	1,869	1,721	2,724	2,502	2,549	2,671	2,719
	Future (2050)	1,897	1,747	2,826	2,590	2,642	2,769	2,818
	% Increase	1.5%	1.5%	3.7%	3.5%	3.6%	3.7%	**3.6%**
50-year Event	Existing (2000)	2,138	1,969	3,124	2,865	2,918	3,053	3,108
	Future (2050)	2,167	1,996	3,232	2,960	3,018	3,158	3,213
	% Increase	1.3%	1.4%	3.4%	3.3%	3.4%	3.4%	**3.4%**
100-year Event	Existing (2000)	2,529	2,331	3,713	3,405	3,472	3,630	3,685
	Future (2050)	2,560	2,360	3,828	3,506	3,579	3,744	3,801
	% Increase	1.2%	1.3%	3.1%	3.0%	3.1%	3.1%	**3.2%**
500-year Event	Existing (2000)	3,145	2,900	4,636	4,286	4,360	4,555	4,622
	Future (2050)	3,179	2,932	4,759	4,394	4,474	4,677	4,748
	% Increase	1.1%	1.1%	2.7%	2.5%	2.6%	2.7%	**2.7%**

91J is at the boundary between subbasins 91 and 92 - see diagram below.
92J is in the location of old node 2108.

9.0 DEVELOPMENT OF FUTURE CONDITIONS NO PROJECT HYDRAULIC MODEL

The HEC-RAS model developed for this subtask was based on the final calibrated Existing Conditions HEC-RAS described in Section 4.0. The discharges estimated with the Future Conditions No-project HEC-HMS model, described in the previous section, were used as input. The downstream boundary condition was set as a known water surface elevation for each flood discharge, using rating curve information for the Chatham gage station, summarized in Table 9.1. This data is graphed in Figure 9.1, below.

Table 9.1. Existing and future conditions HEC-RAS downstream boundary conditions (Chatham Gaging Station)

Return Period, years	Existing Conditions Q, cfs	Existing Condition WS Elev., ft.	Future Conditions Q, cfs	Future Condition WS Elev., ft.
2	1,000	199.20	1,061	199.40
5	1,605	200.19	1,683	200.27
10	2,134	200.99	2,229	201.03
25	2,719	201.84	2,818	201.86
50	3,108	202.37	3,213	202.42
100	3,685	203.13	3,801	203.26
250	4,153	203.77	4,275	203.93
500	4,622	204.41	4,748	204.61

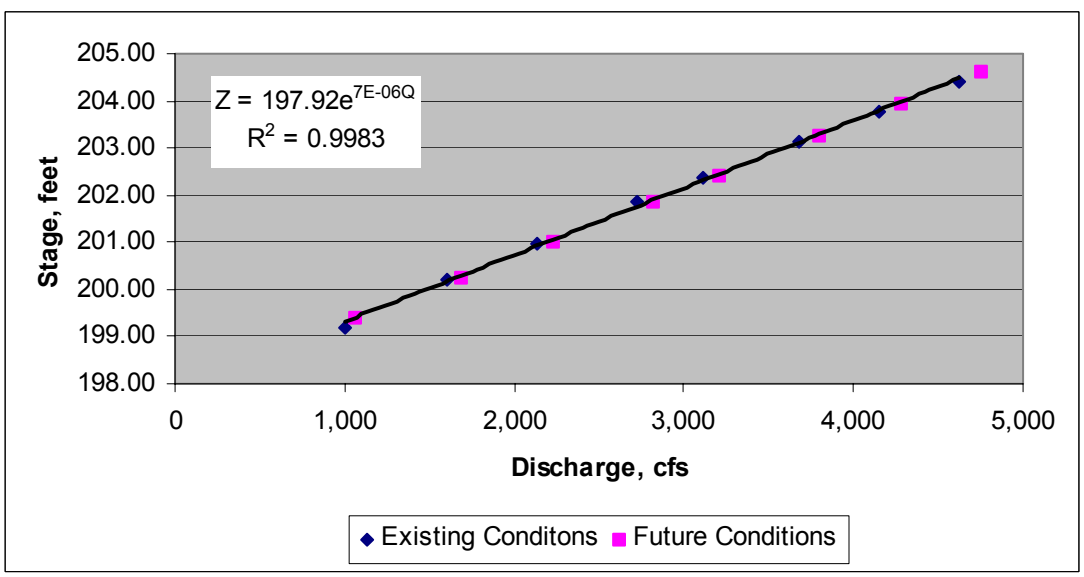


Figure 9.1 Existing and future conditions downstream rating curve (at Chatham)

Because the upper watershed is near to built-out conditions, no significant increases in sedimentation along the study reach due to construction activities are anticipated. For this reason, the channel cross-sections developed for Existing Conditions modeling were used for the future conditions model.

Water surface profiles were developed for the study site from design storms with 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year return periods. These profiles are presented in Table 9.2. All Future Conditions No-Project HEC-RAS project files developed for this study are included on the attached CD.

The increase in water surface elevations for future conditions averages about 0.15 feet for the 100-year event. A comparison of the existing and future conditions 100-year water surface profiles is presented in Table 9.3. A graphical comparison of the two profiles is presented in Figure 9.2, with the approximate limits of study indicated with vertical lines.

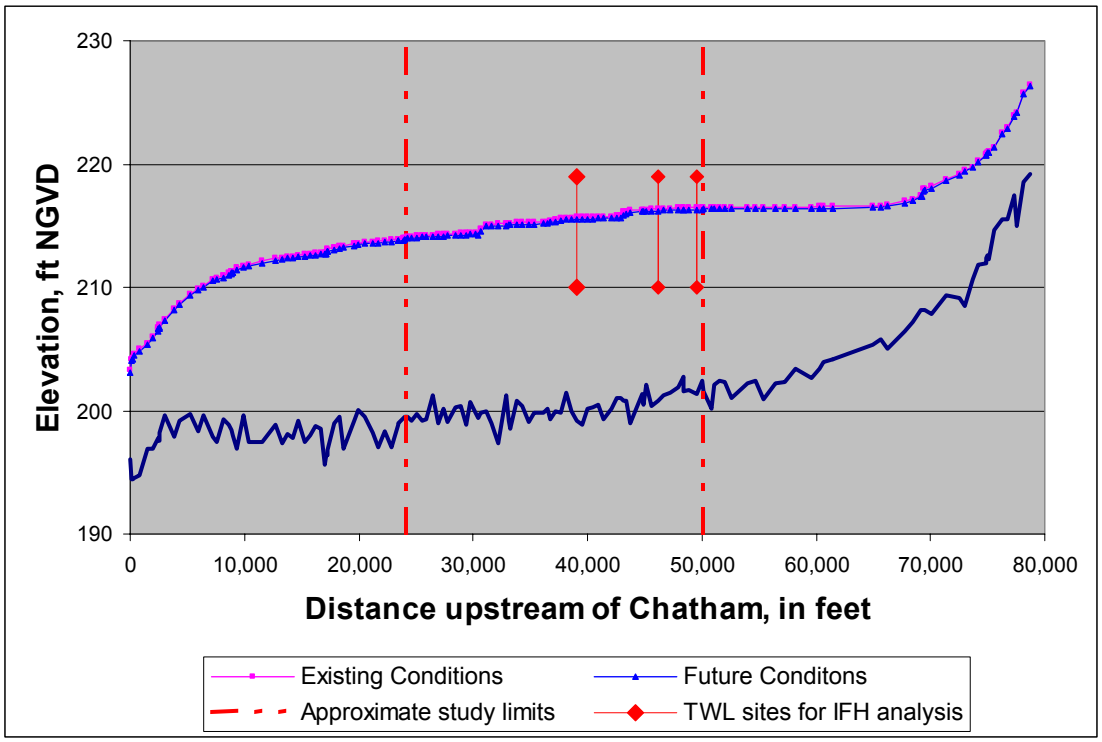


Figure 9.2 Existing and future conditions no-project 100-year water surface profiles simulated for the upper Passaic River

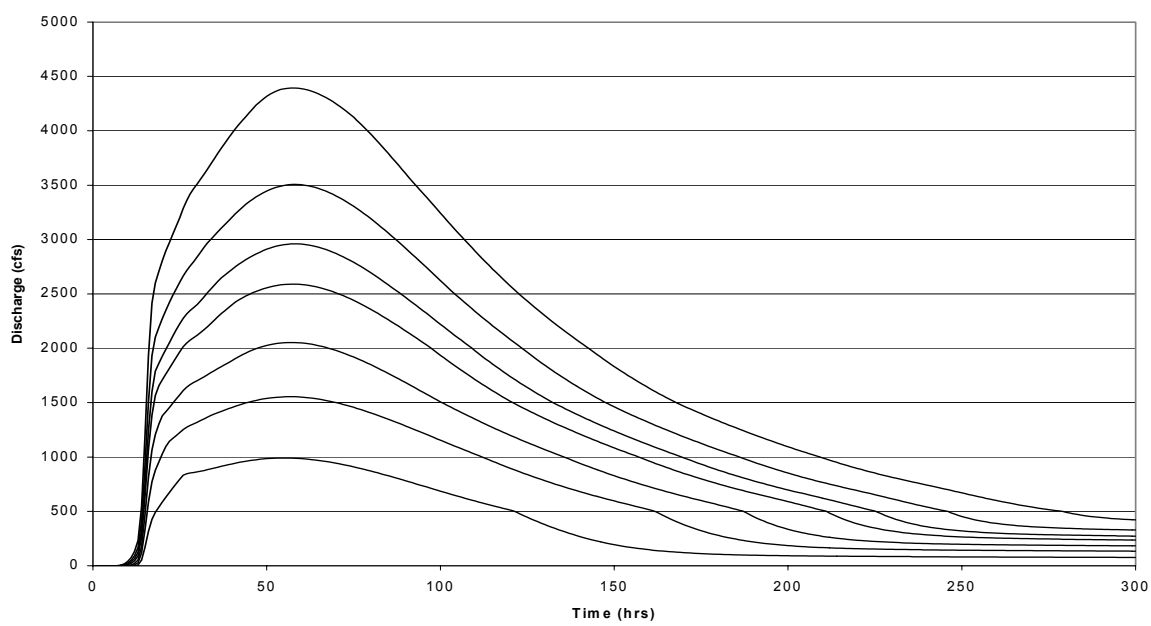
10.0 DEVELOPMENT OF FUTURE CONDITIONS NO PROJECT HEC-IFH MODELS

Adjustments were made to existing conditions HEC-IFH models developed for the future conditions no-project modeling. No changes were made to the runoff hydrographs for the IFH model because the interior area is now completely developed and the contributing area will remain the same. Modeled future increases in runoff from other parts of the watershed were accounted for by increasing the tailwater elevations for the HEC-IFH model. All Future Conditions No-Project HEC-IFH project files developed for this study are included on the attached CD.

Figure 6.1b, (see page 6-4) is a location map of the interior area, bounded by Valley Road on the south, Mountain Avenue on the east, and high ground on the north and west. The map shows the three culvert crossings that pass flow under Valley Road towards the Passaic River: Crossing #1, 240 feet west of Passaic Avenue, Crossing #2, 400 feet east of Warren Avenue, and Crossing #3, 500 feet west of Western Boulevard. There is also a culvert under Mountain Avenue, 250 feet north of the Erie and Lackawanna Railroad.

The future conditions discharge hydrographs computed in HEC-HMS for the 2-, 5-, 10-, 25-, 50- 100-, and 500-year return periods on the Passaic River at Junction 91J are shown in Figure 10.1. These hydrographs give the discharge at Junction 91J in the HEC-HMS model, which is used in the HEC-RAS model along the reach between 25,481 and 44,730 feet upstream of Chatham.

Figure 10.1. Passaic River discharge hydrographs 38,146 feet upstream of Chatham (coincident with Western Boulevard)



The discharge hydrographs were converted to stage hydrographs at a section 38,146 feet above Chatham (coincident with the extended centerline of Western Boulevard using rating tables developed from HEC-RAS output. These stage hydrographs were input to the exterior stage module of HEC-IFH. The exterior stage module of HEC-IFH allows the user to transfer the stage hydrograph from the primary outlet to secondary outlets. The rating tables were used as the transfer relation, with the culvert near Western Boulevard serving as the primary outlet. The rating tables are shown in Table 10.1.

Table 10.1. Discharge-Elevation Rating Table for the Passaic River

Discharge (cfs)	Approx. Return Period *	Water Surface Elevations (ft)		
		Culvert near Western	Culvert Near Warren	Culvert Near Passaic
0	--	203.50	204.10	204.50
500	0.9	207.89	208.54	208.93
1000	1.9	210.06	210.62	210.87
1500	4.0	211.67	212.19	212.35
2000	8.6	212.87	213.41	213.53
2500	18.1	213.86	214.42	214.52
3000	38.4	214.73	215.35	215.47
3500	81.2	215.56	216.27	216.35
4000	172.0	216.31	217.03	217.10
4500	364.1	216.99	217.79	217.87
5000	770.8	217.52	218.41	218.48

* T, in years = 0.4263*exp(.0015Q), based on combined HEC-HMS/HEC-RAS output at node 91J (44,730 ft U/S of Chatham)

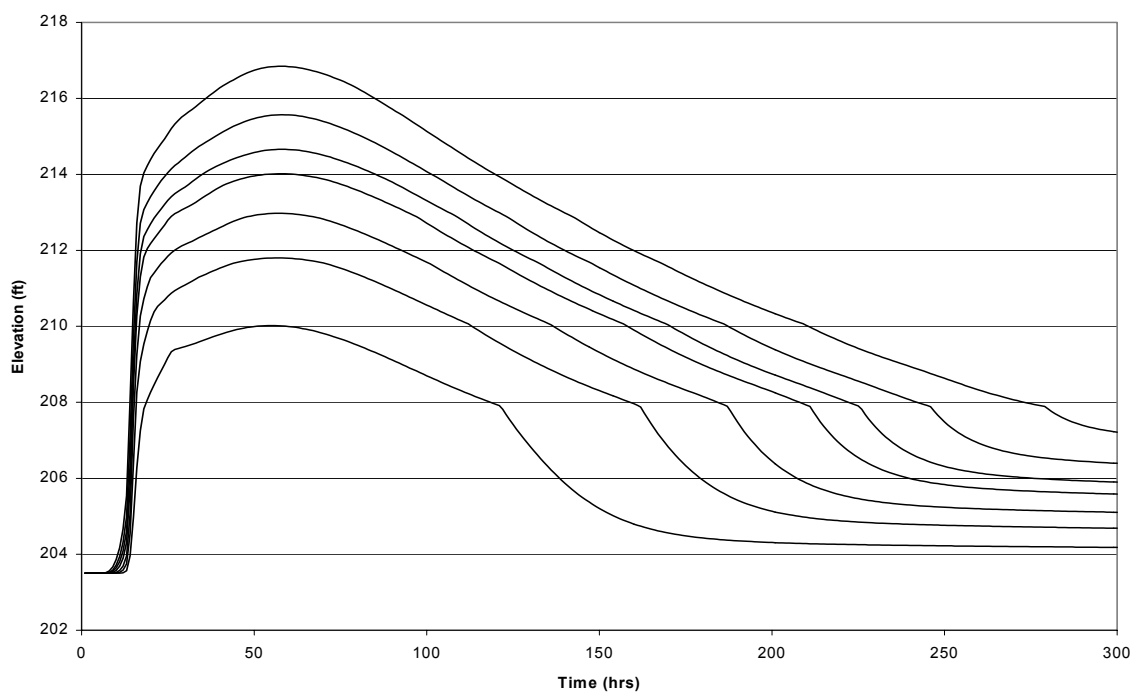


Figure 10.2. Passaic River stage hydrographs near Western Boulevard

All other modules prepared for the existing conditions HEC-IFH model were used without changes for the future conditions. This includes the interior runoff parameters, the interior stage-storage relations, the gravity outlets (existing culverts), and the back-flow from the Passaic River. HEC-IFH does not include an option for backflow, but it does have a seepage option in the auxiliary flow module. The seepage option enables the user to input seepage into the interior area for variable differential head. In order to model backflow through the outlets under Valley Road, the culvert rating tables were used to produce a differential head-discharge table to input into the auxiliary module.

Using no initial storage (reflecting complete desynchronization of interior and exterior runoff) and the Passaic River stage hydrographs as the tailwater on the outlets, and considering backflow through the outlets, interior peak stage frequencies were computed with the model as shown in Figure 10.3. The stage-frequencies for the Passaic River at each outlet are included.

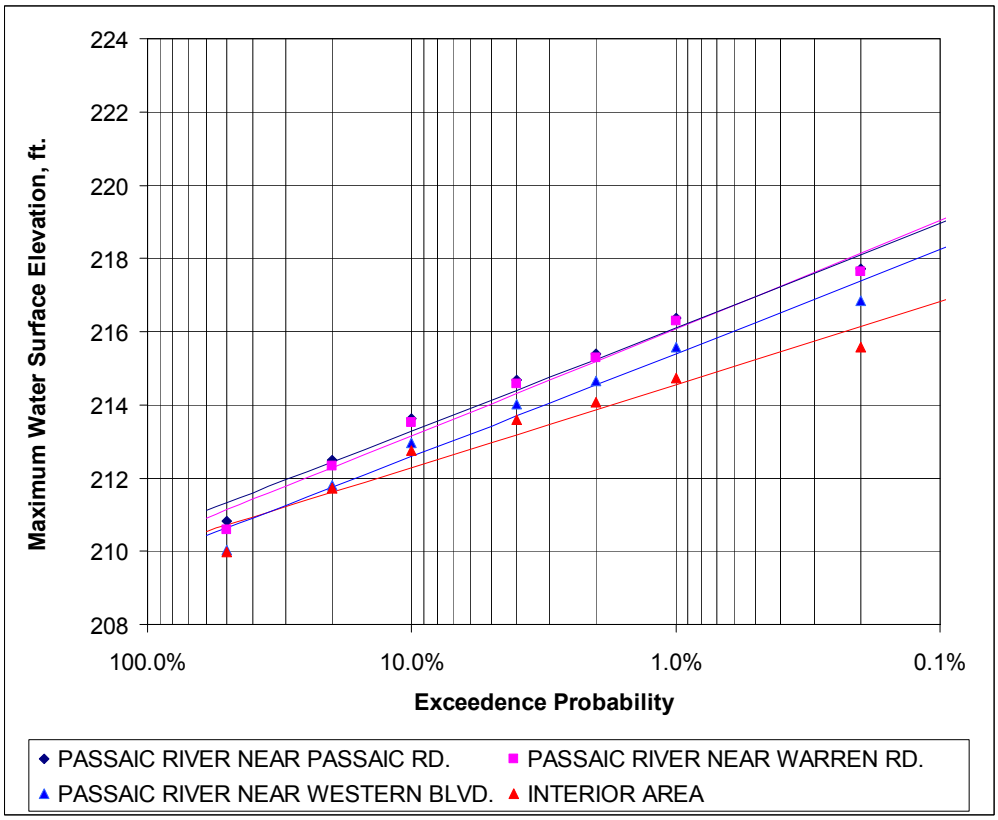


Figure 10.3. Stage-Frequency results for the (initially empty) interior area and the Passaic River at each outlet under Valley Road

A comparison of the existing and future conditions maximum interior stages is provided in Table 10.2.

Table 10.2. Maximum Simulated Stages for Existing and Future Conditions

Recurrence Interval	Maximum Stage (ft)	
	Existing Conditions	Future Conditions
2-YR	209.76	209.99
5-YR	211.54	211.71
10-YR	212.59	212.74
25-YR	213.48	213.60
50-YR	213.98	214.08
100-YR	214.63	214.72
500-YR	215.50	215.59

11.0 H&H ANALYSIS OF STRUCTURAL ALTERNATIVES FOR FLOOD CONTROL

Two structural alternatives were proposed for mitigating flood damage in the Long Hill Township in the Reconnaissance Study for the Upper Passaic River Flood Control & Environmental Restoration Study (Section 905-b, WRDA 86) Preliminary Analysis (USACE, N.Y. District, July 1998). Alternative 2 consists of providing closure structures and "minimum interior drainage facilities" for the culverts draining the four tributaries to the Passaic River within the study area along Valley Road and Morristown Road. Alternative 4 includes the facilities provided with Alternative 2 plus the construction of a floodwall south of Valley Road to the 100-year flood elevation. Both alternatives will reduce the frequency of flooding of Long Hill Township from the Passaic River.

11.1 Alternative 2 – Sluice Gates on Culverts under existing roadway

Valley Road serves as a line of protection between the Passaic River and the majority of Long Hill Township. Until the Passaic River overtops Valley Road, flow from the Passaic River into Long Hill is limited to three culverts under Valley Road. Culvert Crossing Number 1 lies approximately 200 feet west of Passaic Avenue, Culvert Crossing Number 2 is approximately 400 feet east of Warren Avenue, and Culvert Crossing Number 3 is approximately 500 feet west of Western Boulevard. Initially, it was thought that a fourth culvert crossing, under Morristown Road approximately 200 feet north of the Erie and Lackawanna Railroad, allowed flooding from the Passaic River into Long Hill. However, it was found that this flooding is isolated to an area north of the railroad tracks.

The lowest point of Valley Road is at elevation 213.6, near the intersection of Valley Road and Warren Avenue. This corresponds to a Passaic River flood elevation between those related to the 10- and 25-year storm events. Installing sluice gates at the three culvert crossings would prevent flooding from the Passaic River in the interior area for the 10-year and more frequent events. Passaic River floodwaters would be allowed to enter into the interior drainage areas through the opened sluice gates to flood wetland areas up to the 3-year flood level of 211.0 if higher river elevations are not expected.

Closing the sluice gates at the culvert crossings would reduce the storage available for the Passaic River, thereby increasing the peak discharge. Of the design storms evaluated (2-, 5-, 10-, 25-, 50-, 100-, and 500-year), only the 5- and 10-year peak discharges for the Passaic River are affected by the installation of sluice gates. During a 2-year event, the sluice gates would remain open, and for the 25-year and higher events Valley Road would be overtopped.

The installation of sluice gates would have no effect on the hydraulics of the Passaic River beyond the existing impact of Valley Road. For existing and Alternative 2 proposed conditions, the main channel water surface elevations are only impacted for the 5- and 10-year events due to a change in discharge associated with the loss of storage north of Valley Road.

The flooding due to local runoff in the interior area (north of Valley Road) will be greater with the sluice gates closed. It was assumed that there would be no gravity outflow through the culverts before the gates are closed. In reality, the interior runoff usually peaks before the Passaic River peaks, so some drainage occurs before the river rises. Peak interior stages due to interior runoff and Passaic River back flow under existing and Alternative 2 proposed conditions are presented in Table 1. For the interior drainage, it was assumed that future conditions will closely approximate existing conditions.

Table 11.1 Stage-frequency calculations for Existing and Alternative 2 conditions

Return Interval (yr)	Flood Elevation Due to Interior Runoff (ft)	Existing Conditions Passaic River Flood Elevation (ft)	Interior Flood Elevation with Sluice Gates (ft)	Sluice gates status	Condition
2	209.86	210.27	210.27	open	Flooding backed through culverts
5	210.43	212.05	210.43	closed	Sluice gates prevent backflow
10	210.83	213.29	210.83	closed	Sluice gates prevent backflow
25	211.23	214.41	214.41	closed	road overtopped *
50	211.48	215.11	215.11	closed	road overtopped *
100	211.90	216.12	216.12	closed	road overtopped *
500	212.54	217.50	217.50	closed	road overtopped *

* minimum top of road elevation = 213.4

11.2 Alternative 4 – Floodwall and Sluice Gates

The benefits of installing sluice gates are limited due to the low elevation of Valley Road. In order to reduce the flooding of Long Hill Township from higher magnitude floods on the Passaic River, the feasibility of a floodwall was evaluated.

The floodwall will be located south of Valley Road and tie into high ground from approximately 300 feet west of Passaic Avenue to just west of Poplar Drive. The total length is approximately 3,970 feet. The top of the floodwall will be set to elevation 217.0, with three weir sections, totaling approximately 1,470 feet, with inverts at elevation 216.2, just below the 100-year flood elevation of the Passaic River. The floodwall alignment and with-project conditions floodplain is shown in Figure 11.1

The sluice gates at Culvert Crossing Numbers 1 and 2 will be located at the floodwall as opposed to at Valley Road as in Alternative 2. The operation of the sluice gates will be the same as Alternative 2. The 3-year and more frequent storms will be allowed to flood the area north of the floodwall and Valley Road.

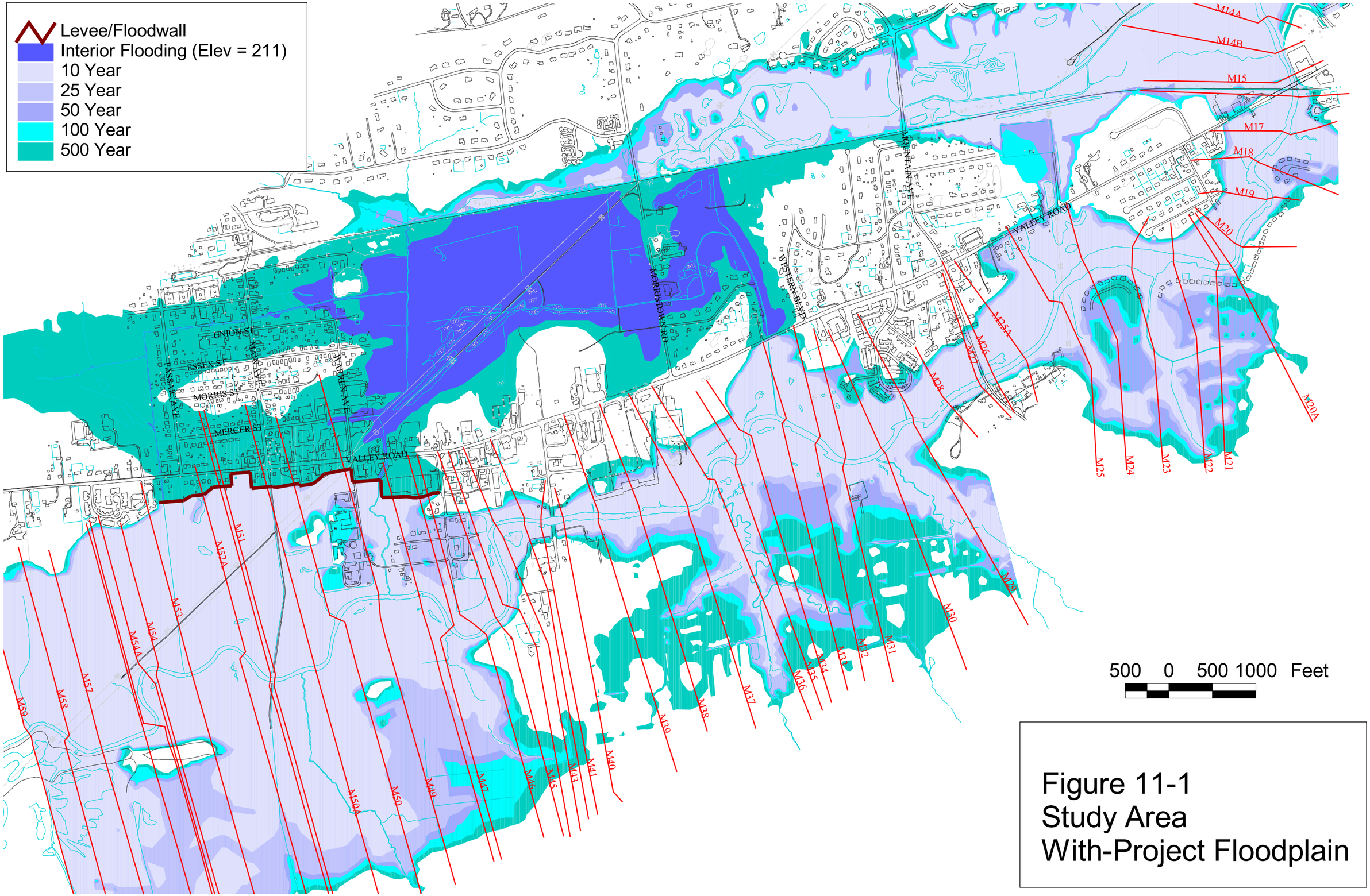


Figure 11-1
Study Area
With-Project Floodplain

As aligned near the edge of the effective flow, the floodwall would have no significant effect on main channel flow. The maximum change in flood elevation due to loss of conveyance area for any flood event is 0.01 feet. The interior flooding elevations for existing hydrologic conditions with and without the floodwall and sluice gates are given in the Table 11.2.

Since the floodwall has no freeboard for the 100-year storm, it is assumed that the floodwall will overtop and the storage behind the floodwall will be available during the 100-year and higher events. Therefore, there is no change in discharge for the 100- and 500-year floods. For more frequent events, but less frequent than the 3-year event, the storage of the interior area will not be available, resulting in a slightly increased discharge on the Passaic River downstream of the study site. This increase in discharge is partially offset by the reduction in watershed area due to detaining the interior runoff north of the floodwall. The overall increase in discharge produces a maximum water surface elevation increase of 0.1 feet.

Table 11.2 Stage-frequency calculations for Existing and Alternative 4 conditions

Return Interval (yr)	Flood Elevation Due to Interior Runoff (ft)	Existing Conditions Passaic River Flood Elevation (ft)	Interior Flood Elevation (ft) with Sluice Gates *	Sluice gates status	Condition
2	209.86	210.27	210.27	open	Flooding backed through culverts
5	210.43	212.05	210.43	closed	Sluice gates prevent backflow
10	210.83	213.29	210.83	closed	Sluice gates prevent backflow
25	211.23	214.41	211.23	closed	Floodwall protecting interior *
50	211.48	215.11	211.48	closed	Floodwall protecting interior *
100	211.90	216.12	211.90	closed	Floodwall protecting interior *
500	212.54	217.50	217.50	closed	Floodwall overtopped *

*** Floodwall overflow weir elevation = 216.2; top of wall elevation = 217.0**

Evaluation of pumping requirements

The sluice gates proposed for Alternatives 2, and the floodwall and sluice gates proposed for Alternative 4 prevent gravity drainage of the interior area when the Passaic River rises, which, under some combinations of storm events might cause some damage not previously experienced. The provision of pumps to eliminate this residual damage was evaluated for both proposed alternatives.

Table 11.3 provides a summary of estimated pumping requirements for each alternative in order to maintain a maximum interior stage of 211, the starting elevation for significant flood impact in the study area. Any flooding below that elevation is entirely local and would be handled by privately maintained sump pumps, which are not accounted for in the project analysis.

Table 11.3 Order-of-magnitude pumping requirements for alternative flood control projects to maintain interior flood elevation of 211.0 (448.8 gpm = 1 cfs)

rval (yr)	Alt. 2 - protection to Elev 312.4			Alt. 4 - protection to Elev 216.2		
	Max. Interior flood stage, no pumps	Flood storage volume above elev. 211, Ac-ft	eighteen-hour required average pumping rate, gpm *	Max. Interior flood stage, no pumps	Flood storage volume above elev. 211, Ac-ft	eighteen-hour required average pumping rate, gpm *
2	210.27	--	--	210.27	--	--
5	210.43	--	--	210.43	--	--
10	210.83	--	--	210.83	--	--
25	214.41	710.16	214,251	211.23	41.68	12,574
50	215.11	918.99	277,254	211.48	86.98	26,241
100	216.12	1280.11	386,200	211.90	163.08	49,201
500	217.50	unlimited	not feasible	217.50	unlimited	not feasible

$$* \quad = \frac{Acft * 43560 \frac{ft^3}{acft} * 7.48 \frac{gal}{ft^3}}{hours * 60 \min / hr}$$

The order of magnitude calculations show that the use of pumping to increase the level of protection for Alternative 2 is clearly not economically feasible. Once the stage of the Passaic River rises above the top of road elevation, the pumping requirements for maintaining an elevation of 211 would be excessive.

The use of pumping to increase flood control benefits for Alternative 4 is technically feasible and could not be rejected out of hand based on the order of magnitude estimates shown in Table 3. However, the combination of high discharge and low head (estimated as less than 10 feet, including lift and losses) indicates the need for turbine or propeller irrigation pumps, which are expensive installations.

A typical pump installation for the combination of head and discharge service required at this site would be a 20-inch single stage pump, operating at about 11,000 gpm against 10 feet of head at about 85% efficiency (see, for instance, the Series 8312 Fairbanks Morris Vertical Turbine). Based on this operating point, 2, 3, and 5 pumps would be required for the 25-, 50-, and 100-year flood events, respectively.

Table 3 presents order-of-magnitude estimates only, because the timing of flooding is not accounted for. As a check on the pumping requirements calculated from simple volume considerations, the HEC-IFH program was used to simulate interior flood stages with pumping provided for during the flooding event associated with rainfall over the interior catchment in conjunction with high stages on the Passaic River.

For this simulation, it was assumed that the pumps would be provided with an appropriate wet well configuration so that pumps would begin to operate at sump water elevation 205.5, shutting off at elevation 205.1.

Figure 11.2 shows the interior stage hydrograph for Alternative 4 augmented by pumps with a total parallel operating capacity of between 10 and 60 cfs, as simulated in IFH for the 50-year interior flood event. The simulation confirms the order of magnitude estimate of pumping requirements based on simple volume considerations.

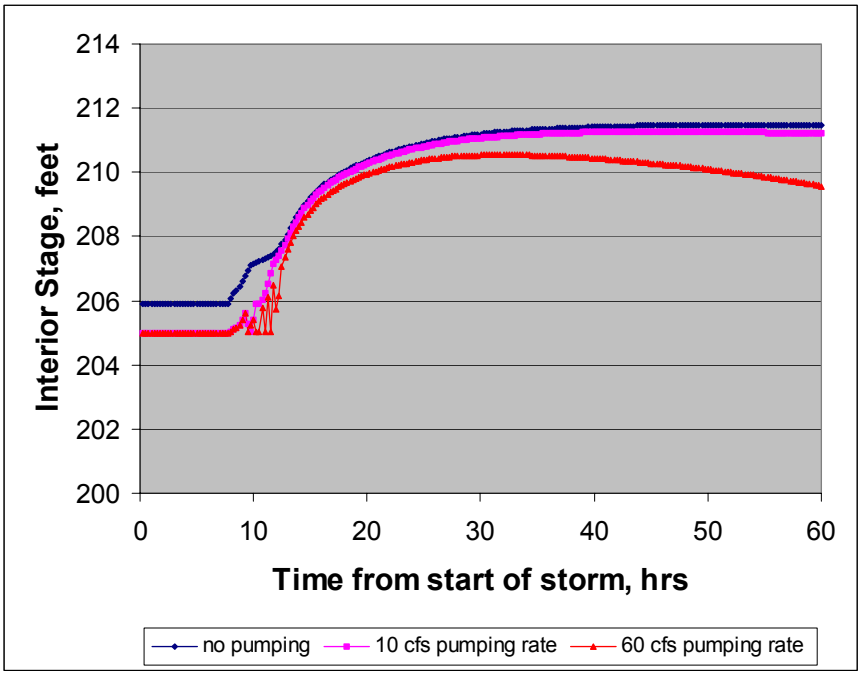


Figure 11.2 Interior stage hydrographs for 50-year interior storm with Alternative 4 flood control works augmented by pumping capacity between 10 (4,480 gpm) and 60 cfs (26,880 gpm)

A detailed cost estimate for a pumping plant with the capacity described above was not prepared, but providing an installation with capacity for the 50- or 100-year event (and the additional required standby pumps) would have a substantial first cost. The estimated installed cost of an 18-inch 10,000-gpm vertical turbine pump is approximately $25,000. Associated project costs in addition to the approximately $150,000 pump purchase and installation costs would include:

- Purchase of land and site engineering
- Construction of pump foundation pads
- Excavation and construction of sumps
- Construction of housing for pumps and electrical gear

Operation and maintenance costs would be additional.

As shown in Table 11.3, the maximum 50- and 100-year interior flood elevations without any pumping or gravity drainage are 211.5 and 211.9, respectively.

With significant damage beginning at elevation 212.0, and considering the high cost of providing pumps to eliminate the small amount of damage associated with flooding

between elevation 211 and 212, it would be more cost effective to flood proof the few buildings that are flooded below elevation 212.0 than to depend on pumps to prevent flood damage. A more detailed analysis will be performed for the selected alternative but it is expected that the conclusions will not change. A small concrete pad could be provided at one or more of the sluice gate locations to accommodate portable pumps, should that be considered desirable.

Impact of floodwall on wetlands

The alignment of the approximately 3,970-foot proposed sheet pile floodwall for the Long Hill Township is shown in Figure 11.3. As located, it will intercept surface flows from an approximately 1,337-acre interior drainage area. This catchment is intersected by an existing network of drainage ditches and, in the lower reaches immediately upstream of the wall, overland flow is interrupted by curbs and gutters associated with the streets of Long Hill Township.

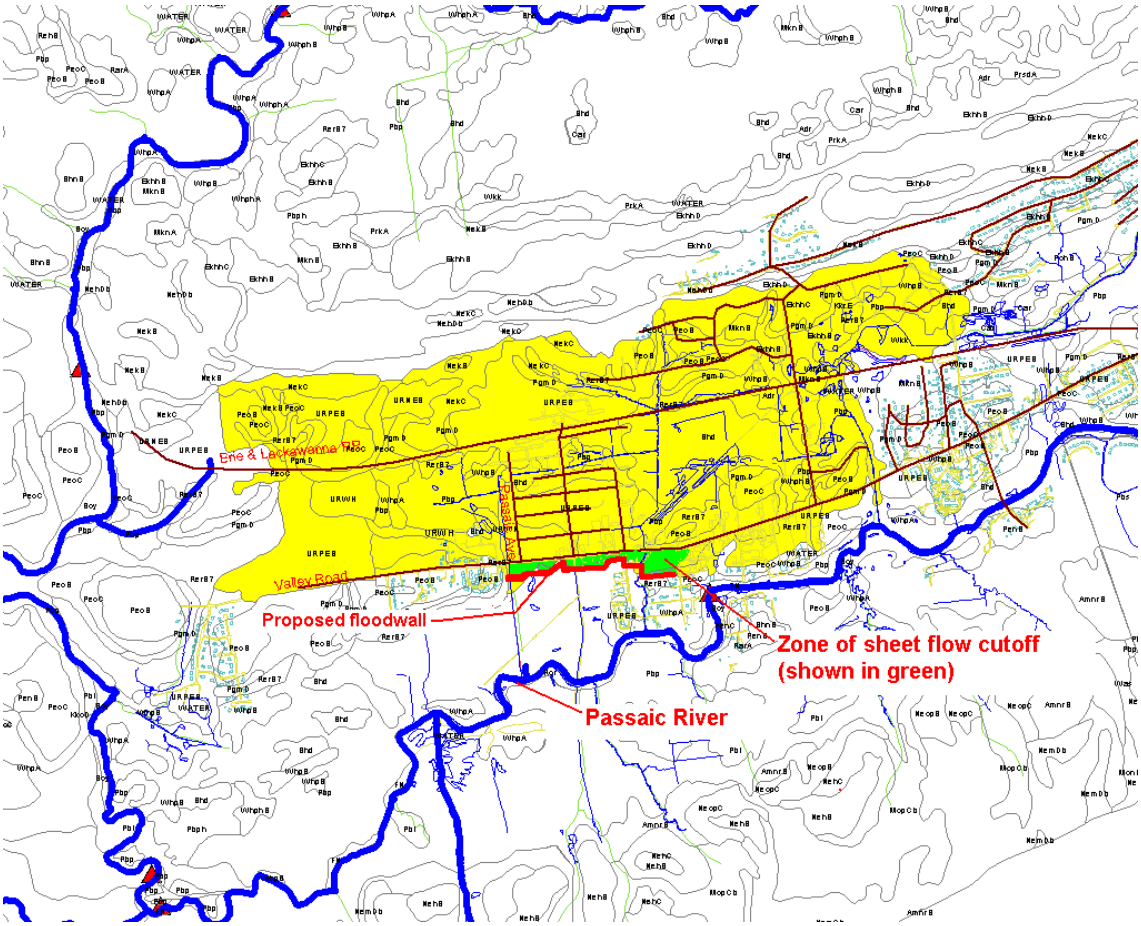


Figure 11.3 Upstream drainage area impacted by proposed floodwall

For existing conditions, sheet flow is limited to the zone between Valley Road and the Passaic River, with flow lengths and areas varying with the changing water level. Approximately 9 acres of the sheet flow area will be impacted by the alignment of the proposed wall, as shown in green in Figure 3.

Based on average climate data for Morris County (Soil Survey of Morris County, NJ, USDA SCS, 1976), the area where the proposed floodwall is to be placed receives an excess of rainfall over estimated potential evaporation for 11 out of 12 months, as shown in Table 11.4.

Table 11.4 Annual water balance for Long Hill Township

Month	Average Temperature		Index term	Potential ET (2)		Ppt in.	Excess	Loss	Net Rain
	Faren (1)	Cent.	[Ta/5]^1.5	cm	inches	(1)	(3)	in.	in.
January	29.0	-1.7	0.00	0.00	0.00	3.1	3.10	0.00	3.10
February	30.0	-1.1	0.00	0.00	0.00	2.9	2.90	0.00	2.90
March	38.0	3.3	0.54	1.07	0.42	4.2	3.78	0.00	3.78
April	49.5	9.7	2.71	3.95	1.56	4.1	2.54	0.00	2.54
May	59.0	15.0	5.20	6.71	2.64	4.5	1.86	0.00	1.86
June	66.0	18.9	7.34	8.89	3.50	3.6	0.10	0.00	0.10
July	73.0	22.8	9.72	11.18	4.40	4.3	-0.10	-0.10	0.00
August	71.0	21.7	9.02	10.52	4.14	5.2	1.06	0.00	0.96
September	64.0	17.8	6.70	8.26	3.25	3.9	0.65	0.00	0.65
October	54.0	12.2	3.82	5.22	2.06	3.4	1.34	0.00	1.34
November	43.0	6.1	1.35	2.24	0.88	4.5	3.62	0.00	3.62
December	31.5	-0.3	0.00	0.00	0.00	4.1	4.10	0.00	4.10
Thornthwaite Temperature Index, I =			46.42			47.8	24.95		24.95

1) data from USDA SCS Soil Survey, Morris County, NJ

2) PET, cm, = 1.6*[10*Tavg/I]^a

a = 0.49 + .0179*I - .0000771*I^2 + .000000675*I^3 = 1.22

$$I = \sum_{i=1}^{12} \left[\frac{T_{ai}}{5} \right]^{1.5}$$

3) Excess = Ppt - PET

In addition to local rainfall, soils at the site are inundated by inflow from the Passaic River at stages in excess of approximately 208 feet NGVD. The average number of days of inundation at the site each year was estimated by developing a stage-duration curve for the Passaic River at the site as follows.

- Discharge-duration curves were developed for the Chatham and Millington USGS gage sites based on rank ordering of over 80 years of average daily discharge records.
- Discharge-duration curves were converted to stage-duration curves utilizing stage-rating curves for each site, supplied by the USGS
- Design storm stages simulated in the calibrated HEC-RAS model at Section M48 were correlated with stages at Chatham to provide an equation for converting the stage-duration curve at that point to a stage-duration curve at Warren Road (see Table 11.5).

Table 11.5 Stage-frequency correlation for Warren Avenue and Chatham gage site

Return Period, years	Chatham		Warren Avenue *		Millington	
	Q, cfs	Z, ft	Q, cfs	Z, ft	Q, cfs	Z, ft
2	1000	199.20	990	210.44	732	223.11
5	1605	200.19	1608	212.19	1127	223.95
10	2134	200.99	2130	213.41	1471	224.61
25	2719	201.84	2724	214.53	1869	225.31
50	3108	202.37	3124	215.22	2138	225.74
100	3685	203.13	3713	216.21	2529	226.34
250	4153	203.77	4175	216.90	2837	226.79
500	4622	204.41	4636	217.57	3145	227.22

* Stage Warren (Sect. M4*) = 1.3545*Stage Chatham - 59.028 (R^2 = 0.9927)

Based on the transfer of data from the Chatham stream gage to the study site, a stage-duration curve was developed to estimate the expected annual percentage of time soils at the site are inundated under existing conditions. This curve is presented as Figure 11.4.

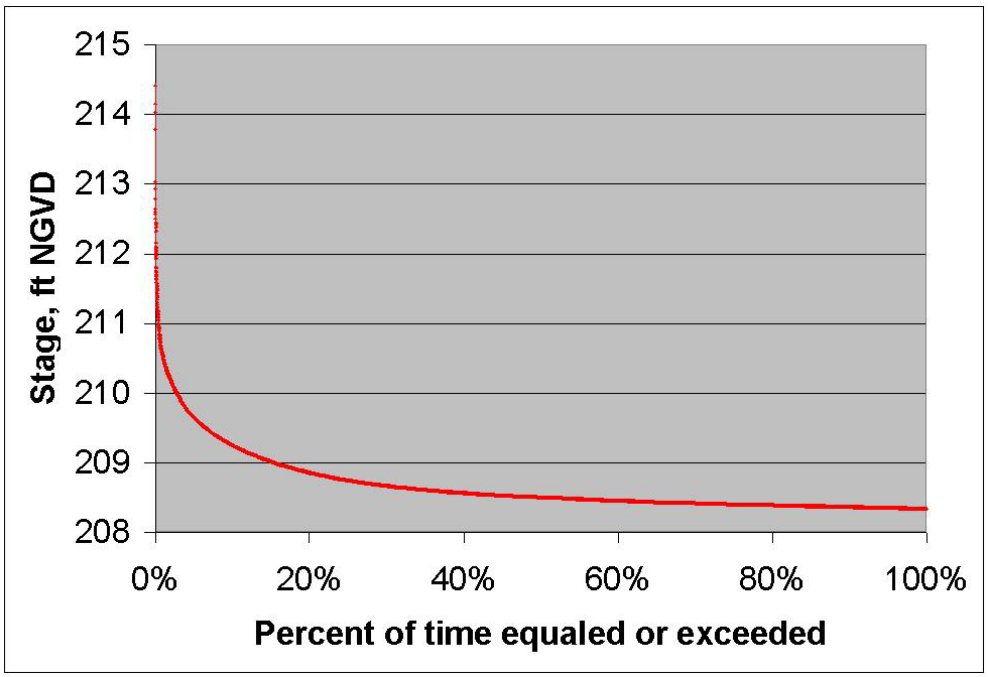


Figure 11.4 Stage-duration curve for the Passaic River at Warren Avenue

The stage-duration data in Figure 11.4, above, is summarized in Table 11.6 for one-foot increments of elevation within the project zone. Percent exceedence for elevations equal to and greater than 215.0 (which has not occurred in the observed record) were estimated from flood frequencies developed from Log Pearson Type III curve fitting procedures carried out in the H&H analysis for the flood control project.

Table 11.6 Summary of stage-duration calculations for the Passaic River at Warren Avenue

River Stage, ft	Stage Exceedence		Method of Estimation
	Percent of days	Days per 100 years	
208.34	98.29%	35,902	Stage-duration analysis
209.00	15.50%	5,662	Stage-duration analysis
210.00	2.81%	1,027	Stage-duration analysis
211.00	0.44%	161	Stage-duration analysis
212.00	0.10%	35	Stage-duration analysis
213.00	0.03%	9	Stage-duration analysis
214.00	0.01%	4	Stage-duration analysis
215.00	0.005%	2	approx. stage 1 days in 50 years
216.00	0.003%	1	approx. stage 1 days in 100 years
217.00	0.001%	0	approx. stage 1 days in 275 years

Based on flow duration relationships for the Passaic River at the site, and a representative cross-section through the floodwall area (see Figures 5 and 6), existing local flow patterns will be impacted to some extent when the Passaic River stage is between elevations 209 and 217 feet, NGVD in the area shown in green in Figure 3. This occurs approximately 15.5 % of the time, approximately 56 days each year, on average.

While water from the Passaic River will be cut off within the transition zone between the river and town during those days, incoming and outgoing flow through the Warren Avenue and Passaic Avenue culverts will not be impeded until the sluice gates are closed when the Passaic River Stage exceeds Elevation 212.0. Based on our analysis, this will occur with an estimated frequency of about one day in three years.

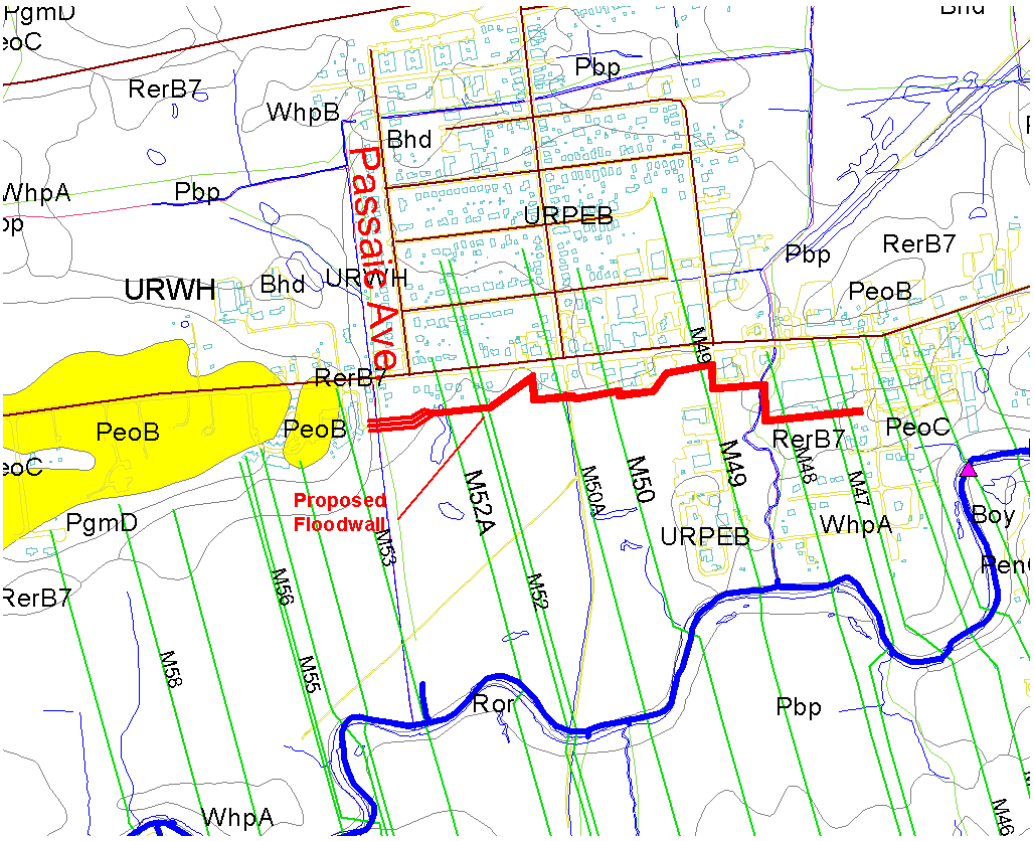


Figure 11.5 Plan view of proposed floodwall for Long Hill Township

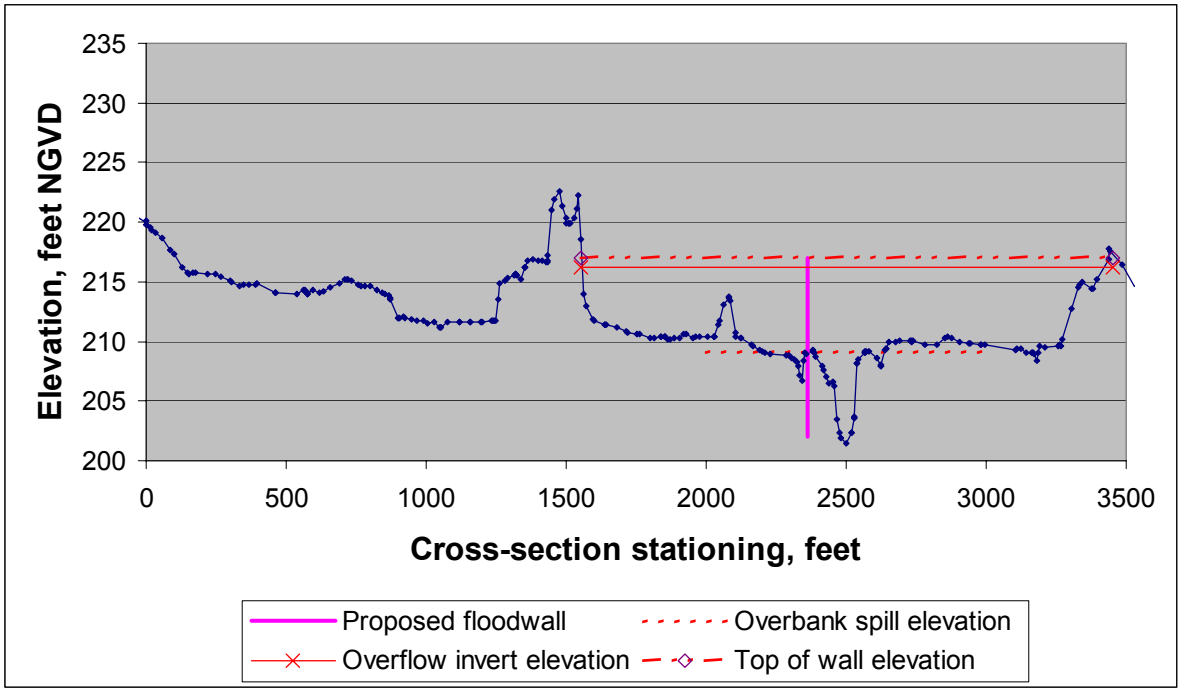


Figure 11.6 Cross-section "M-50" through proposed sheet pile floodwall for Long Hill Township (looking downstream)

12.0 H&H ANALYSIS OF FLOOD WARNING SYSTEM

A Flood Warning System was recommended as a non-structural alternative for mitigating flood damage in the Long Hill Township in the Reconnaissance Study for the Upper Passaic River Flood Control & Environmental Restoration Study (Section 905-b, WRDA 86) Preliminary Analysis (USACE, N.Y. District, July 1998). A B/C ratio of 2.0 was estimated for the alternative in the Project Study Plan for the Upper Passaic River Flood Control & Environmental Restoration Study (USACE, N.Y. District, June 1999).

The benefits of a flood warning system are derived from the reduction of loss of life and property within Long Hill Township by providing additional lead-time to prepare for flooding, allowing such measures as evacuating cars from the area and moving household goods to higher floors or higher ground. Additional benefit specific to this site can be obtained from the FWS by utilizing it to provide guidance for the operation of the sluice gates proposed for the floodwall alternative.

An Automated Flood Warning System (AFWS) is already operating for the lower reaches of the Passaic River. The system includes a network of stream gages and precipitation gages that provide continuous updates to a River Forecasting Model run by the NWS. Daily projections of the river stage are provided at a number of critical locations along the river up to Little Falls, approximately 35 miles downstream of Chatham. The existing system also provides flood warnings for the Chatham and Millington gages, but provides no information for Long Hill Township.

The desired flood warning capacity for the Long Hill Township can be provided by a new, independent system, by utilizing the existing AFWS "as is", or by integrating the township into the existing system and providing additional rainfall and stream gage instrumentation.

The work done for this task has included:

- H&H modeling to provide a preliminary estimate of rainfall thresholds for the flood warning system
- The recommendation of the preferred Concept of Operation, project formulation, and the definition of system hardware requirements at a level of detail sufficient to permit the estimate of annual capital and operation costs
- Estimation of the potential lead-time that can be obtained from the FWS in the study area

The results of this work are summarized in the following sections.

12.1 H&H Modeling for Alternative 5

The existing conditions HEC-HMS model for the study site was developed utilizing a design storm methodology, which is consistent with the needs of estimating existing and proposed annual expected damages. The model was modified for the evaluation of

Alternative 5 to provide lead times for a flooding, starting from different base flow conditions.

Varying base flow conditions were estimated as percentages of the 1-year return period peak discharges calculated for each sub-basin in the calibrated existing-conditions HEC-HMS model, as shown in Table 12-1.

Table 12-1 Base flow estimates for evaluating lead times for flood warning system

HEC-HMS Subbasin	Drainage area, mi^2	Time of concentration, in hours		HEC-HMS 1-year Q peak	Baseflow estimate	
		Original estimate	Calibration Value		Qp(T=1)/2	Qp(T=1)/10
0010	3.44	1.50	1.24	285.00	143	29
0015	5.23	1.90	1.63	278.00	139	28
0020	4.35	1.70	1.53	484.00	242	48
0030	23.06	6.00	3.00	803.00	402	80
0040	13.50	6.00	7.00	148.00	74	15
0050	1.66	1.40	0.91	429.00	215	43
0060	2.87	1.50	0.78	102.00	51	10
0070	2.84	1.50	1.61	199.00	100	20
0080	19.30	6.00	1.80	455.00	228	46
0091	7.71	2.50	3.36	309.00	155	31
0092	5.04	1.80	2.07	189.00	95	19
0100	3.78	1.50	1.74	136.00	68	14
0110	4.45	1.90	1.21	185.00	93	19
0120	1.67	1.40	1.11	39.00	20	4

Constant intensity rainfall thresholds were established for each of these starting, or base flow, conditions for 1-, 3-, 6-, 12-, and 24-hour storms to permit the estimation of lead times and threshold precipitation levels that would cause flooding at the study site. It should be noted that these thresholds are determined for basin-wide average rainfall. For final design, additional study to account for areal variation may be appropriate, as discussed below.

Rainfall thresholds were established systematically for the following levels of alert based on water surface elevations at the Plainfield Bridge in Stirling:

- Level 0 - Negligible Possibility of Flooding (< 211.0)
- Level 1 - Possible Flooding (211.0 - 212.5)
- Level 2 - Probable Flooding (212.5 – 213.5)
- Level 3 - Imminent Flooding (> 213.5)

Table 12-2 shows the range of threshold rainfall rates determined for this evaluation.

Table 12-2 Estimated threshold alert values for flooding at study site (no floodwall)

Duration	Constant Intensity, inches/hour; Baseflow = Qp(T = 1-yr)/2															
s	0.3	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0
1	0	0	0	0	0	0	0	1	1	1	1	2	2	3	3	3
3	0	0	0	1	1	2	2	3	3	3	3	3	3	3	3	3
6	1	1	2	2	3	3	3	3	3	3	3	3	3	3	3	3
12	1	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3
24	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3

Duration	Constant Intensity, inches/hour; Baseflow = Qp(T = 1-yr)/10															
s	0.3	0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0
1	0	0	0	0	0	0	0	0	0	0	1	1	1	2	2	2
3	0	0	0	0	0	1	1	2	2	2	3	3	3	3	3	3
6	0	1	1	1	1	2	3	3	3	3	3	3	3	3	3	3
12	0	1	2	2	3	3	3	3	3	3	3	3	3	3	3	3
24	1	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3

* The grayed-out cells indicate HEC-HMS runs for these values

The threshold values were set for each pair of rainfall intensity and duration values by comparing the peak discharge for node 91J from the HEC-HMS run with the rating curve for HEC-RAS section 44730.

The HEC-RAS profiles are shown for the one inch-per-hour intensity storms of 1-, 3-, 6-, and 10-hour durations in Figure 12.1.

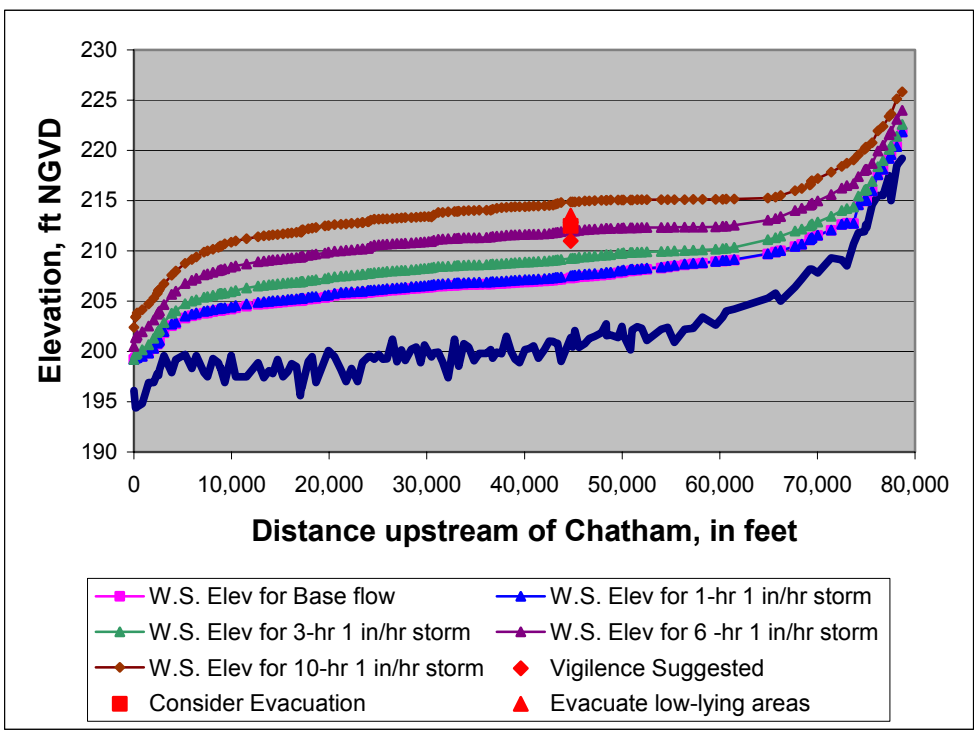


Figure 12-1 Upper Passaic River water surface profiles for 1-inch rainfall of varying duration

The lead-time provided by a flood warning system is used to estimate the benefits of the alternative. A representative lead-time was estimated for the flood warning system for the no-wall scenario based on a one inch per hour, 10-hour storm ($Q_p(T=1)/2$ base flow). The lead-time for the with-wall scenario is presented in the following section.

For the no-wall scenario, flooding at the study site begins when the Passaic River reaches a stage of approximately 212 NGVD. The lead-time provided by the system depends on the timing of the storm event. The lead-time was estimated from critical water surface elevations at the study site determined from the HEC-HMS discharges at node 91-J and the rating curve at determined for that point, station 44730.02, in the HEC-RAS steady state, no-project run. Table 12-3 summarizes the results of lead-time estimation for the 1 inch/hour, 10-hour event. The hydrograph for this event is shown in Figure 12-2.

Table 12-3 Representative lead times for the no-wall flood-warning alternative

Event	Hour	Q at 91J, cfs	Water Surface Elevation	Lead Time, in hours		
				From start of storm	From Mid point of storm	From end of storm
Start of 1"/hr rainfall	0.00	350	<210.30	n/a	n/a	n/a
Midpoint of 10-hr storm	5.00	662	<210.30	n/a	n/a	n/a
Start of flooding	8.60	1469	212.00	8.6	3.6	-1.4
End of storm	10.00	1801	212.56	n/a	n/a	n/a
Flooding of 24 inches	21.50	2306	214.00	21.5	16.5	11.5

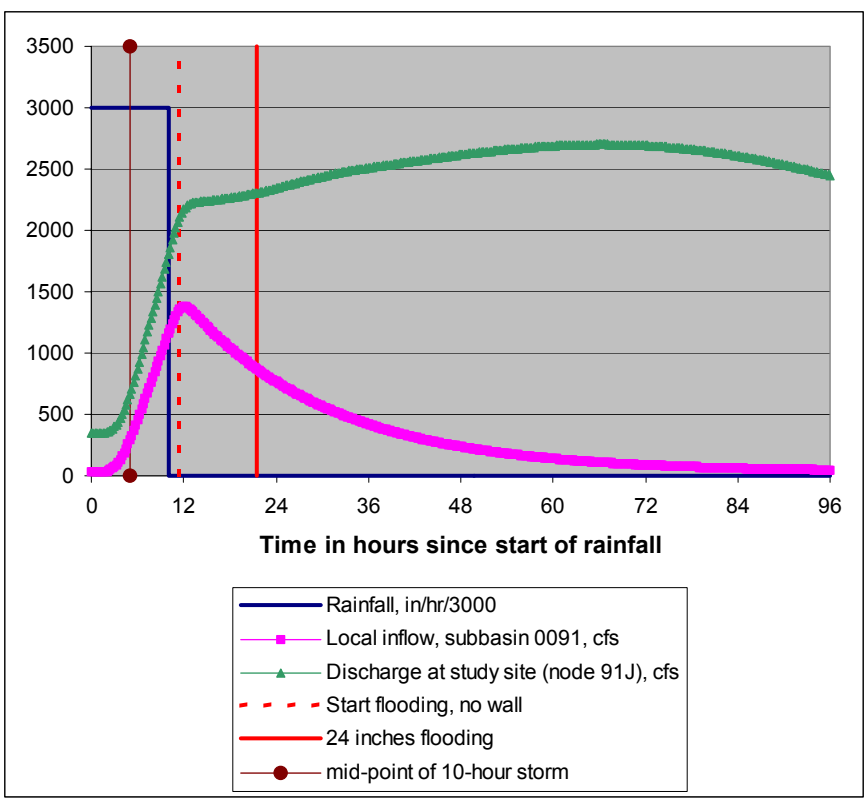


Figure 12-2 Storm hydrograph used for representative lead-time estimation (no wall)

12.2 Concept of Operations Plan for FWS

A Flood Warning System is already in place in the downstream area of the Passaic River basin. Alternative formulations to be considered for the proposed flood-warning alternative to serve the Long Hill Township study site include:

- Subscribing to the existing FWS with limited warning service, with no additional investment,
- Utilizing the existing FWS with the addition of an alternate vendor's software to provide data interpretation and automated alarm procedures
- Utilizing existing and additional remote sensed data sources and providing software to permit enhanced data interpretation and automated alarm procedures.

The existing system is described in the following sub-section and a preferred system of operation of the proposed system is described in subsequent sections.

12.2.1 Existing Passaic River Flood Warning System

The existing Passaic River Flood Warning System was completed in 1988, and consists of 35 rain gages and 21 streamflow gages located throughout the catchment. Data from these gages are transmitted by radio, telephone, or satellite to 12 computer base stations located at Federal, State, and county offices (Passaic Flood Warning System, USGS Fact Sheet FS-092-98). A schematic of the full system is reproduced as Figure 3.1 below.

The Middle Atlantic River Forecasting Center, run by the National Weather Service (NWS) in State College, PA, maintains rainfall-runoff models for all major rivers discharging to the Chesapeake and Delaware Bays, and for the Passaic and Raritan Rivers, discharging to the Atlantic Ocean. Each watershed model accounts for soil moisture, and is updated every 24 hours for rainfall, and snowmelt inputs; for the Passaic River basin this data is provided in part from radio transmissions from rain gages monitored from the Mt. Holly NWS Forecast Office, and in part from satellite data received at other NWS facilities.

The Forecasting center provides daily forecasts and flood warnings, as needed, back to the Mt. Holly office, which notifies the State Police when advised of imminent flooding conditions. The police inform the Morris County Risk Management Office, which, in turn, notifies the Director of Emergency Management for the Long Hill Township (Mr. Ken Fullagher at the date of writing).

Most of the instrumentation for the existing Passaic River warning system is located in the lower basin, where population density and flood damage is greatest. At present, flood watches and warnings are automatically transmitted to 15 high-risk municipalities (HRM's) from the county base-station computers, but no flood forecasting is provided upstream of Little Falls, NJ, and, while flood warnings are provided at Chatham and Millington, the system provides no warnings specific to the Long Hill Township.

12.2.2 Proposed Long Hill Township Flood Warning System

As noted above, the majority of the data measurements for the existing system are located along the lower reaches of the watershed and do not provide a timely indication of conditions at the study site. For a minimum cost, detailed in a following section, Long Hill Township can be integrated into the existing IFLOW-AFWS system. The DIAD software used in this system can be tailored to report precipitation observed at the gage locations with the most hydrologic impact on the study site, as described below.

A list of the IFLOW-AFWS precipitation gages in the three counties containing the Passaic River watershed above Chatham, NJ is provided in Table 12.4.

Table 12.4 Existing 15-min IFLOW-AFWS precipitation gages in 3 Counties of the upper Passaic River Basin

AFWS ecipitation Gage ID	Gage Name	County	Gage Transmitter ID	Latitude	Longitude	NJ State Plain Coordinates		Approximate Gage Elevation
						Northing	Easting	
1610	Basking Ridge	Morris	BKRN4	40:42:48	74:30:22	684,751	490,434	230
1618	Picatinny SG	Morris	PICN4	40:58:00	74:32:00	777,067	482,929	0
1620	West Wharton	Morris	WWNN4	40:54:23	74:35:53	755,112	465,011	690
1628	Pine Brook SG	Morris	PINN4	40:52:00	74:19:00	740,687	542,819	0
1630	Lincoln Park	Morris	LINN4	40:54:52	74:18:09	758,073	546,707	170
1638	Macopin Dam SG	Morris	MCPN4	41:01:00	74:24:00	795,296	519,719	580
1640	Pequannock	Morris	PEQN4	40:58:00	74:17:37	777,132	549,121	200
1648	Morristown SG	Morris	WHIN4	40:48:00	74:27:00	716,339	505,967	0
1650	Morristown	Morris	MSTN4	40:47:57	74:24:55	716,055	515,574	190
1658	Two Bridges SG	Morris	TWBN4	40:53:00	74:16:00	746,768	556,631	0
1660	Morris Plains	Morris	MSPN4	40:50:00	74:30:59	728,467	487,587	660
1670	Lake Hopatcong	Morris	LAKN4	40:54:56	74:39:52	758,491	446,692	950
1680	Oak Ridge Res.	Morris	ORRN4	41:02:19	74:30:06	803,259	491,656	880
1690	Bershire Valley	Morris	BVYN4	40:57:00	74:34:29	770,990	471,492	720
1700	Ironia	Morris	IRON4	40:50:12	74:37:58	729,733	455,381	720
1708	Boonton A SG	Morris	BNTN4	40:54:00	74:25:00	752,776	515,151	364
1713	Boonton	Morris	BONN4	40:53:45	74:23:33	751,254	521,842	230
1717	Boonton Elev SG	Morris	BONN4	40:54:00	74:35:00	752,776	469,099	364
1718	Boonton B SG	Morris	BONN4	40:54:00	74:25:00	752,776	515,151	196
1828	Pompton Plain SG	Morris	PPPN4	40:58:00	74:17:00	777,139	551,966	180
1359	Burnt Mills SG	Somerset	BTMN4	40:38:04	74:41:13	656,063	440,255	80
1369	Neshanic Sta. SG	Somerset	NSNN4	40:31:04	74:43:18	613,610	430,491	75
1379	Watchung SG	Somerset	WTCN4	40:38:12	74:27:06	656,849	505,529	185
1389	Warrenville SG	Somerset		40:36:00	74:31:00	643,477	487,488	240
1399	So. Branch S. SG	Somerset	SBRN4	40:33:24	74:41:19	627,759	439,723	60
1409	Somerville SG	Somerset	PTBN4	40:35:00	74:37:00	637,414	459,713	60
1410	Somerville	Somerset	PTBN4	40:35:00	74:37:00	637,414	459,713	60
1419	Bound Brook SG	Somerset	BDBN4	40:34:00	74:33:00	631,350	478,235	40
1420	Bound Brook	Somerset	BDBN4	40:34:00	74:33:00	631,350	478,235	40
1429	N. Plainfield SG	Somerset	STBN4	40:37:00	74:26:00	649,567	510,641	80
1430	N. Plainfield	Somerset	STBN4	40:37:00	74:26:00	649,567	510,641	80
1449	So. Branch N. SG	Somerset	SBCN4	40:32:48	74:41:48	624,121	437,464	50
1450	Middlebush	Somerset	MIDN4	40:30:00	74:31:00	607,048	487,481	112
1459	Griggstown SG	Somerset	GTNN4	40:26:20	74:37:06	584,813	459,198	40
1460	Pottersville	Somerset	POTN4	40:42:00	74:42:00	679,969	436,674	380
1470	Skillman	Somerset	SKIN4	40:25:00	74:42:00	576,767	436,440	100
1480	Sourland	Somerset	NESN4	40:28:00	74:43:00	594,991	431,835	560
1490	Far Hills	Somerset	FHLN4	40:42:00	74:36:00	679,921	464,400	224
1507	Raritan SG	Somerset	RTTN4	40:36:00	74:40:00	643,520	445,838	50
1509	Far Hills SG	Somerset	FARN4	40:42:30	74:38:11	682,958	454,312	230
1517	Bound Brook#2 SG	Somerset	BDKN4	40:33:05	74:32:54	625,776	478,704	0
1529	Middlesex SG	Somerset	MDSN4	40:35:00	74:30:00	637,393	492,125	37
1530	Middlesex	Somerset	MDSN4	40:35:00	74:30:00	637,393	492,125	37
1533	Bound Brook	Somerset	BBKN4	40:33:00	74:34:00	625,269	473,591	50
1538	Manville SG	Somerset	MNVN4	40:33:00	74:35:00	625,273	468,978	21
1539	Belle Meade SG	Somerset	BMDN4	40:28:00	74:38:00	594,945	455,038	59
1540	Belle Meade	Somerset	BMDN4	40:28:00	74:38:00	594,945	455,038	59
1553	Somerville	Somerset	SMVN4	40:34:00	74:37:00	631,367	459,705	0
1558	Blkwell Mills SG	Somerset	BKWN4	40:28:00	74:35:00	594,928	468,949	40
1559	Belle Meade SG	Somerset	BRBN4	40:29:00	74:39:00	601,000	450,402	67
1560	Belle Meade	Somerset	BRBN4	40:29:00	74:39:00	601,000	450,402	67
1570	Martinsville	Somerset	MRTN4	40:36:00	74:35:00	643,487	468,995	240
1579	Seely Mills SG	Somerset	GBKN4	40:39:00	74:24:00	661,707	519,871	184
1590	Watchung	Somerset	WHGN4	40:38:00	74:27:00	655,611	506,001	194
1599	Weston SG	Somerset	WETN4	40:31:47	74:35:19	617,879	467,498	65
1609	Raritan SG	Somerset	RITN4	40:33:52	74:38:10	630,537	454,314	60
1669	Millsone SG	Somerset	MITN4	40:30:10	74:35:15	608,080	467,794	45
1669	North Branch SG	Somerset	NBAN4	40:36:00	74:40:27	643,524	443,755	55
1918	Millington SG	Somerset	MILN4	40:41:00	74:32:00	673,823	482,890	0
1203	Plainfield	Union	PLFN4	40:36:00	74:24:00	643,492	519,892	0
1218	Rahway SG/Rahway	Union	RWYN4	40:37:05	74:17:00	650,144	552,279	9
1228	Springfield SG	Union	SPGN4	40:43:00	74:18:00	686,052	547,562	66
1238	Rahwy SG/Kenilwr SG	Union	KENN4	40:40:23	74:18:48	670,161	543,909	35
1260	Mountainside	Union	MTSN4	40:41:00	74:22:00	673,850	529,092	400
1278	Robinsn Br SG/Ra	Union	RBRN4	40:36:20	74:07:07	645,746	598,019	50
1439	Green Brk SG/Pla	Union	GRBN4	40:36:00	74:30:00	643,476	492,125	55
1440	Plainfield	Union	GRBN4	40:36:00	74:30:00	643,476	492,125	55
1608	Passaic SG/Chatm	Union	CAMN4	40:43:00	74:23:00	686,011	524,472	193

Figure 12.4, below, shows which of those 15-minute recording gages would be most useful in providing warnings of flood potential at the study site. Point rainfall accumulations can be reported at each of these gages, and a weighted average rainfall can be reported for the entire set of upstream gages based on a Thiesen Polygon approximation.

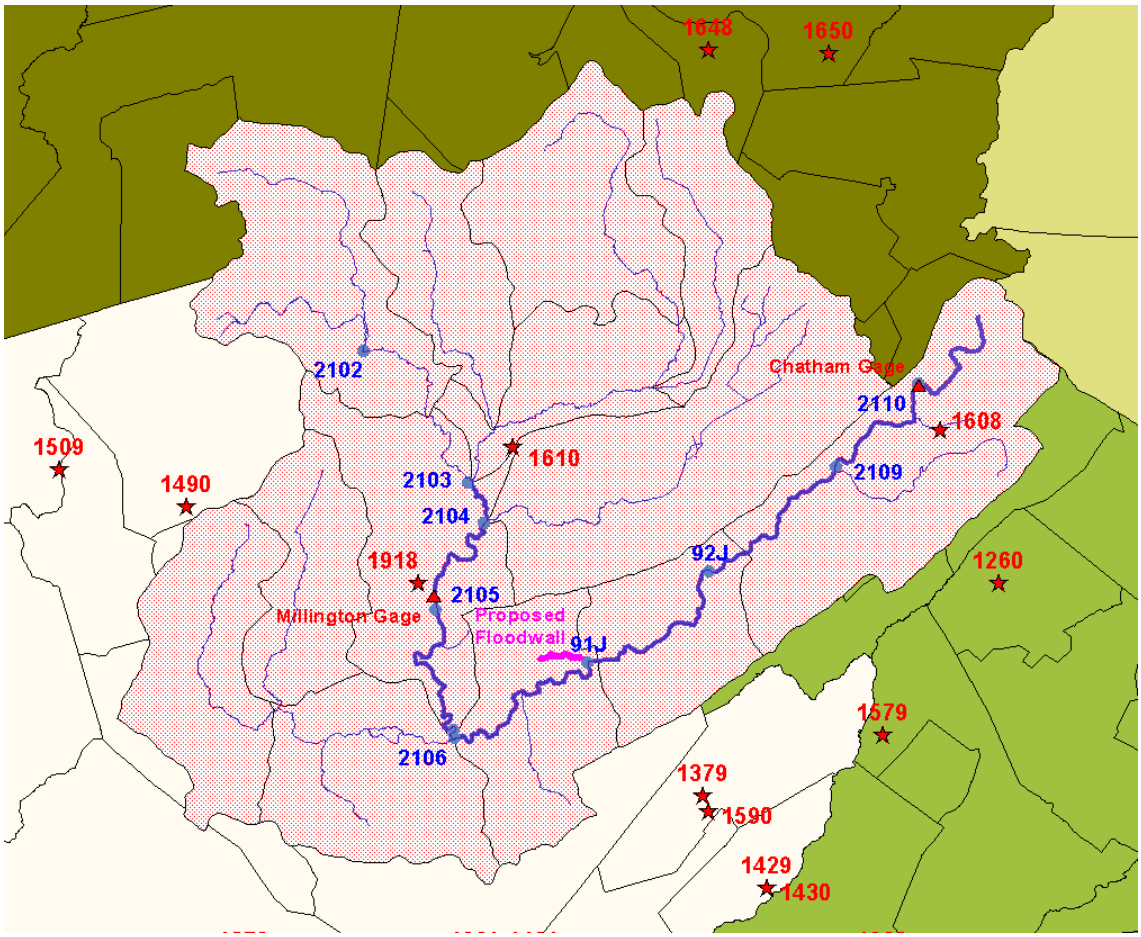


Figure 12.4 Existing IFLOW-AFWS precipitation gages (red stars) and USGS stream gages (red triangles) in relation to proposed site of Long Hill Township floodwall and flood warning system. HEC-HMS model nodes are shown as blue circles.

For the scenario where a floodwall is not provided (the "do nothing" alternative), the lead-time provided by a flood warning system was estimated to be about 4 hours from the midpoint of rainfall. For this alternative, there would be no advantage to obtaining additional stage measurements, and the existing IFLOW-AFWS system would provide the most cost-effective flood warning capability.

For the scenario where the proposed Long Hill Township floodwall is built, the provision of additional Stage monitoring at the Stirling-Plainfield Avenue Bridge can be justified because observations of the river stages at the study site would be useful for coordinating sluice gate operations, and substantial lead-times can be provided between the time that the sluice gates are closed and overtopping of the floodwall begins, as described below.

Suggested warning levels for the flood warning system installed in conjunction with the proposed floodwall are shown in Table 12.5.

Table 12.5 Critical predicted water surface elevations for the Long Hill Township Flood Warning System

Condition		With floodwall		No floodwall	
		WSL, ft NGVD	Action	WSL, ft NGVD	Action
0	No Flooding	< 211.0	Maintain sluice gates open	< 211.0	--
1	Possible Flooding	211.0 - 215.5	Close sluice gates	211.0 - 212.5	Vigilence suggested
2	Probable Flooding	215.5 - 216.7	Consider evacuation	212.5 - 213.5	Consider evacuation
3	Imminent Flooding	> 216.7	Evacuate	> 213.5	Evacuate

The lead-time that can be obtained for overtopping of the floodwall spillway is presented in Table 12.6. This estimate is based on runs for the first set of base flow elevations shown in Table 12.1 with a one inch per hour constant intensity rainfall with a duration of ten hours. It was necessary to model the operation of the floodwall with the higher base flow conditions to avoid HEC-HMS instabilities resulting from storages in excess of tabulated values.

Table 12.6 Representative lead times for the with-wall flood-warning alternative

Event	Hour	Q at 91J, cfs	Water Surface Elevation	Lead Time, in hours		
				From start of storm	From Mid point of storm	From end of storm
t of 1"/hr rainfall	0.00	1750	212.73	n/a	n/a	n/a
e Sluice gates	0.00	1750	212.50	n/a	n/a	n/a
10-hr storm	5.00	3038	215.43	n/a	n/a	n/a
storm	10.00	3067	215.49	n/a	n/a	n/a
topping spillway	65.00	3456	216.20	21.50	60.0	55.0
topping floodwall	*	3966	217.00	21.50	n/a	n/a

oodwall not overtopped for this storm

The hydrograph used for estimating lag times is shown in Figure 12.5.

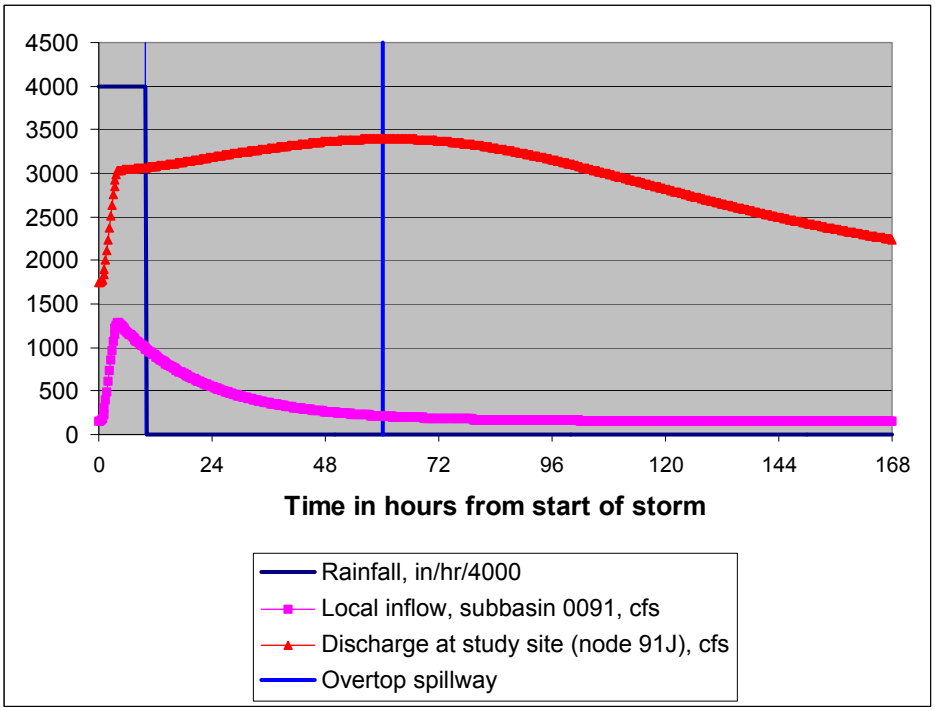


Figure 12.5 Storm hydrograph used for representative lead-time estimation for with-wall scenario

Table 12.7 summarizes the flood warning system project elements for the with-wall and without-wall scenarios.

Table 12.7 Project elements for Long Hill Township Flood Warning System

Flood Warning System Project Element	
Without Floodwall	With floodwall
Institutional and physical integration of Long Hill Township into Passaic River AFWS network	Institutional and physical integration of Long Hill Township into Passaic River AFWS network
Installation of dedicated PC at Long Hill Township Police Station	Installation of dedicated PC at Long Hill Township Police Station
n/a	Installation of stage measuring instrumentation at the Plainfield-Stirling bridge.
Installation of DIAD software and provision of graphical readout for selected upstream precipitation gages	Installation of DIAD software and provision of graphical readout for selected upstream precipitation gages and for real time stage at Plainfield-Stirling bridge

The "physical integration" of the Long Hill Township into the IFLOWS-AFWS system includes hooking up a dedicated PC to the radio network, and installing the DIAD software, which is a Windows application that collects local weather data in real time, displays dynamic data and storm movement on a map background, interfaces with the hydrologic model, and provides alarms and paging functionality.

For the with-wall project alternative, an additional task is the installation of a stage gage at the Stirling-Plainfield Bridge. Contractors specializing in environmental system monitoring routinely install stage gages of this type. The stage is measured on a continuous or periodic basis indirectly from a voltage reading on a pressure transducer installed at the bridge. The probe of the transducer is placed well below the low water elevation of the river at the study site, housed in a steel pipe attached to a bridge pier. A wire cable conducts the voltage signal to a data logger and transmitter. The signal is received by a radio modem installed in the same location as the dedicated PC and DIAD software.

A budget estimate for these items is included in Table 3.5, below.

"Institutional integration" includes arrangements for housing the system and installing the software, for providing scheduled maintenance, and for establishing modes of interaction between the Emergency Management Director of the Township and the NWS.

Mr. Michael Peoples, the Chief of Police of the Long Hill Township has stated that he would welcome the installation of the dedicated computer in the dispatching office of the police station and that, in principal, the space would be rent free.

The Passaic River AFWS is funded by the NY District of the Corps of Engineers, so, in principal, a mechanism is in place to allocate funds for the NWS contractor to install the DIAD software on the Townships PC and provide any customizing required with either the with-wall or without-wall options.

Mr. Steven Francis, a NWS administrator of the IFLOWS-AFWS system, has indicated that there would be no problem with designating the Long Hill Township as a "High Risk Municipality" for inclusion in the system; he also said that it would be useful for the municipality to provide a liaison with the NWS, to attend the quarterly "user group" meetings which are chaired by the NWS alternately in the Morristown and Patterson county offices. Mr. Ken Fullagher, the Emergency Management Director for the township would be the reasonable candidate for carrying out this function as part of normal duties.

The installation and maintenance costs of the proposed system for the with-wall and without-wall options are presented in Table 12.8, on the following page.

Table 12.8 Cost Estimates for With-wall and Without-wall Flood Warning System

em	With-wall Scenario Estimated Cost	Without-wall Estimated Cost
urchase and installation of PC	$1,500	$1,500
munication system upgrade	$800	$800
tware License	$1,000	$1,000
installation and customization	$1,500	$1,500
nstallation of transducer-type stream gage & real-time data ransmitter at Plainfield-Stirling bridge	$12,100	
Transducer - $1000		
Data Logger & transmitter - $5200		
Radio Modem - $1700		
Cable and pipe - $200		
Contractor Installation, $2000		
USGS rating computation and review - $2000		
stimated Total Cost of Installation	$16,900	$4,800
SL communication line monthly charge	$120	$120
rental of office space at Long Hill Township Police Station	$0	$0
puter System Maintenance (estimated cost to COE of esignated contractor)	$83	$83
onthly stage gage maintance (form USGS annual operation & ublication figures)	$100	
ttendance at quarterly AFWS users meeting - monthly proration	$120	$120
stimated Monthly Maintenance	$423	$323

12.3 Summary

The preferred configuration of the Long Hill Township flood warning system depends on whether the proposed floodwall is installed.

For the case where no floodwall is installed, it is recommended to establish a connection to the existing Passaic River Flood Warning System. Physical access is provided by installing a dedicated PC equipped with DIAD software that can be modified by a NWS contractor to maximize the benefit of existing instrumentation. The installation cost for this minimal system is estimated to be $4800; the monthly cost of operation and maintenance is estimated conservatively as $325.

For the case where the proposed floodwall and sluice gates are provided, it is still recommended to establish a connection with the existing NJ AFWS, as described above, but it is also recommended to install additional stage measuring instrumentation at the study site and link it with the existing system. This would provide a substantial lead-time for overtopping of the floodwall and would provide useful information for operating the sluice gates. For this scenario, the cost of installation increases to $16,900, and monthly maintenance costs increase to about $425.