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Consequences of changing climate and land use to 100-year flooding in the Lamprey River Watershed of New Hampshire

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CONSEQUENCES OF CHANGING CLIMATE AND LAND USE TO 100-YEAR FLOODING

in the

LAMPREY RIVER WATERSHED OF NEW HAMPSHIRE

BY

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THESIS

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This thesis has been examined and approved.

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List of Acronyms

CN – Curve Number
DEM – Digital Elevation Model
EGL – Energy Grade Line
EIC – Effective Impervious Cover
FEMA – Federal Emergency Management Agency
FIS – Flood Insurance Study
GIS – Geographic Information System
HEC-GeoHMS – Hydraulic Engineering Center Geographic Hydrologic Modeling System
HEC-GeoRAS - Hydraulic Engineering Center Geographic River Analysis System
HEC-HMS - Hydraulic Engineering Center Hydrologic Modeling
HEC-RAS - Hydraulic Engineering Center River Analysis System
HSG – Hydrological Soil Group
IC – Impervious Cover
IDA – Instantaneous Data Archive
LID – Low Impact Development
NH GRANIT – New Hampshire Geographically Referenced Analysis and Information Transfer System
NRCC - Northeast Regional Climate Center
NWS – National Weather Station
TOC – Time of Concentration
TP-40 – Technical Paper 40
WATSTORE - Water Data Storage and Retrieval System
WQV – Water Quality Volume
WSE – Water Surface Elevation

Abstract

Consequences of Changing Climate and Land Use to 100-Year Flooding

By

Ann M. Scholz

University of New Hampshire, December 2011

Communities are confronting the effects of rapid development and associated land use transformation, while also dealing with the serious impacts of a changing climate. Both factors influence the frequency and magnitude of flood events. This project presents a method used to assess the flood risk associated with current and projected changes in land use and climate for a 213 square mile coastal New Hampshire watershed. The evaluation includes the use of Low Impact Development (LID) as an adaptation planning tool, and, in particular, as a means for building community resiliency in managing water resources.

The hydrologic and hydraulic modeling methods used include the Army Corp of Engineers Hydraulic Engineering Center software Hydrologic Modeling System and River Analysis System, specialty tool kits, in combination with GIS.

The rainfall-runoff analysis was consistent with guidance for Federal Emergency Management Agency floodplain analysis. The land use conditions were modeled for historic, current and a future climate change scenarios. Revised precipitation data from the Northeast Regional Climate Center was used with 8.5 inches for the 100-year, 24-hr design rainfall depth, a 26% increase along the seacoast area of New Hampshire as of 2011. LID strategies, including

infiltration, pervious pavements, bioretention systems, and undisturbed cover, were modeled as a runoff reduction method using revised curve numbers for the distributed storage.

Results of the hydrologic rainfall-runoff analysis, using increased rainfall depth, indicate a 45% increase in the 100-year flood flow at a USGS gaged location on the Lamprey River near Newmarket, NH. The increased flood flows raise the base flood elevations by an average of 2.7 feet along the 36 mile study reach. The conventional build-out scenario indicated an additional 0.3 feet increase in base flood elevation with a 4.3% flood flow increase of 11,109 cfs up from the 2005 flows of 10,649 cfs, and a 2.8% increase with the LID scenario of 10,952 cfs. Differences between conventional and LID build-out scenarios were minimal at the watershed scale because total impervious cover was low (<7.5%); whereas differences were substantial in developed subwatersheds with high impervious cover. Analysis of results from three smaller developed sub basins in urban settings demonstrated that LID had substantial runoff reductions for build-out scenarios and in one instance actually reduced beyond current conditions. Conventional build-out had increases in runoff ranging from 29-36% whereas LID build-out had a range of -2-7%. This last finding is substantial in that it illustrates that LID in a redevelopment scenario can serve to reduce runoff from current conditions.

The long-term watershed management implications of LID zoning as a redevelopment strategy are tremendous. It is important to note that the degree of benefit appears to increase with increasing degree of impervious cover.

Chapter 1

Introduction

1.1 Background

The Lamprey River has flowed in overbank conditions during several recent major flood events, May 2006, April 2007 (CFMSC 2008), and March 2010. These events expose the problems associated with the hydrology that was used by engineers more than 30 years ago to map flood prone areas. The flood insurance studies for the Lamprey River are now outdated and no longer reflect current conditions and therefore do not provide accurate information on flooding conditions for the respective communities.

1.2 Climate Change

Notable change to our climate that have occurred in recent decades (e.g., increases in global average temperature, increases in the amount of water vapor, increase in annual precipitation in mid-latitudes) are very likely caused by human activity as opposed to natural variability (Karl and Trenberth 2003). For example, review of trends in earth's surface temperature in the Fourth Assessment Report (IPCC 2007) show an upward trend globally with an increase of 0.18 ± 0.05°C/decade since 1990. The increase in the observed mean temperature is a straightforward indicator of climate change. Temperature increase is characteristically consistent with increases in atmospheric moisture holding capacity at a rate of about 7% per °C (IPCC 2007). This rising amount of water vapor has generated a widespread increase in heavy precipitation events and intensification of the global water cycle. Middle and high latitude areas of the Northern Hemisphere are experiencing more intense (more than two inches in 48 hours) precipitation and an increase of these events (Wake et al. 2006). Global warming causes water to evaporate and the atmosphere to hold more water vapor making more clouds rich with moisture (Madson 2007). Global increases in intense precipitation could be natural variability in some locations but the effect of anthropogenic forcing cannot be discounted (Kunkel 2003). Projections of a warmer, greenhouse-enriched atmosphere indicate a continuing probability of intense precipitation events (Groisman, Knight et al. 2005; Madson 2007). Rainfall intensity, frequency, and duration are parameters seldom included in forecasts or simulations (Trenberth, Dai et al. 2003). Although some projection studies are contradictory, most of the evidence is consistent with intensification of the water cycle (Huntington 2006).

According to the National Oceanic and Atmospheric Administration's (NOAA's) National Weather Service (NWS 2011), the mean annual precipitation for the project area is 44 to 48 inches. In the northeast US, detailed analysis of meteorological records show a consistent long-term trend in annual precipitation of $+9.5 \pm 2$ mm/decade ($+0.37\pm0.8$ inches/decade) over the last century (Hayhoe 2007). Extreme precipitation events have also increased across the Northeast US(Wake et al. 2006; Spierre and Wake 2010). Most countries are experiencing either significant increases or decreases in seasonal precipitation. In some cases there was no change to the seasonal total; however, an increase in the frequency of heavy precipitation (Easterling, Evans et al. 2000).

Coastal areas in the northeast are clearly showing higher annual mean precipitation (50 to 55 inches) compared to inland areas (25 to 40 inches) and an increase in the annual number of one inch rainfalls, greater than fifteen (15) along the coast to less than nine (9) inland (Spierre and Wake 2010). Extreme precipitation events (two inches and greater) have the potential to affect the region's rivers and streams.

The National Weather Service (NWS) has provided national standards for rainfall depth at specified frequencies and durations since 1953 (Bonnin 2002). The eastern United States uses Weather Bureau Technical Paper 40 (TP-40) (Hershfield 1961). TP-40 has been an efficient estimate of rainfall intensities for particular durations and locations for the design of a wide range of infrastructure: stormwater drainage systems, detention, bridges, dams, and spillways. The atlas is a reference design standard from the local to federal level of engineering agencies. It was generated from recording-gage data and nonrecording-gage data with rainfall observations made once daily at Weather Bureau stations for eighteen (18) and nineteen (19) years respectively. The recorded periods from 1938 through 1957 do not provide an accurate measure of rainfall frequency for current weather conditions. A recent joint effort between the NOAA Northeast Regional Climate Center (NRCC) (operated by the Department of Earth and Atmospheric Sciences at Cornell University) and the National Resource Conservation Service (NRCS) used 50 additional years (1938 through 2010) of data to generate a new rainfall frequency atlas for the New England states and New York (called the NRCC Atlas for the remainder of this thesis). The revised rainfall depths for the seacoast of New Hampshire in comparison to the original TP-40 rainfall rates is provide in Figure 1.

For the hydrological model, two different approaches are used to provide rainfall data. For historical and current scenarios, the hydrological model uses the TP-40 (2005 TP-40 for the remainder of this thesis) and NRCC (2005 NRCC for remainder of this thesis) rainfall frequency atlases respectively to determine peak runoff.

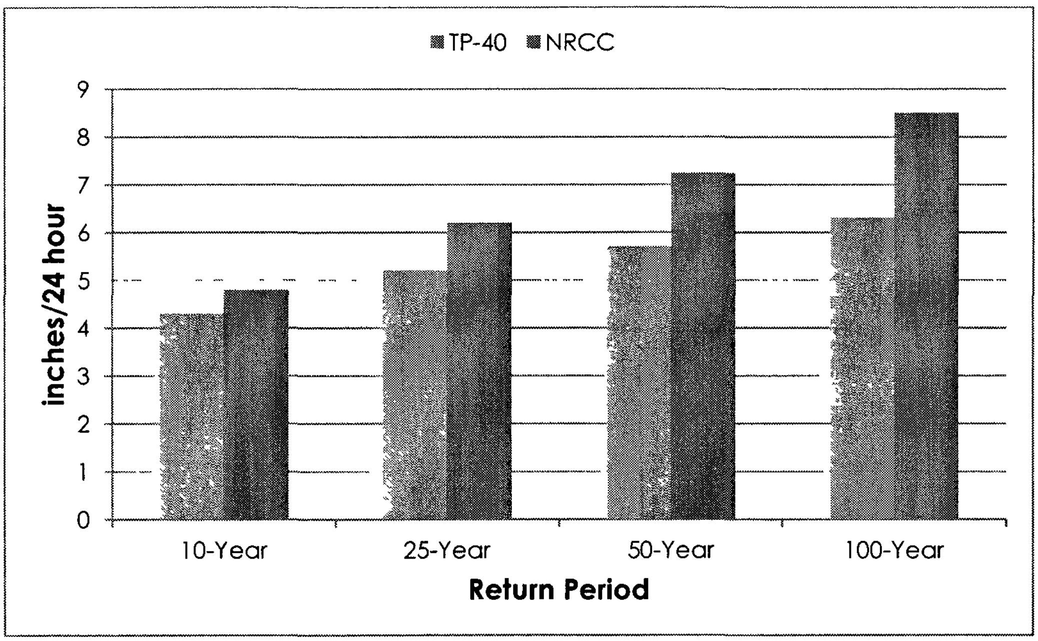

Figure 1: Twenty-four-hour design rainfall depths for project area as quantified by the TP-40 (1938-1957) and the Northeast Regional Climate Center (1938 - 2010)

To evaluate possible future changes in climate, scientists use general circulation model (a.k.a global climate model) simulations driven by future emission scenarios. An emissions scenario incorporates assumptions about population, energy use, and technology to build pictures of how the future might look. Each scenario is associated with a unique "signature" of greenhouse gases emissions. Here, we use the high (A1fi) and a low (B1) emissions scenarios from the Intergovernmental Panel on Climate Change's (IPCC) Special Report on Emissions Scenarios (SRES) (Nakicenvoic et al. 2000).

Under the *A1fi higher-emissions scenario*, SRES assumes a world with fossil fuel-intensive economic growth and a global population that peaks mid-century and then declines. New and more efficient technologies are introduced toward the end of the century. In this scenario, atmospheric carbon dioxide (CO_2) concentrations reach 940 parts per million (ppm) by 2100—more than triple pre-industrial levels. The *B1 lower-emissions scenario* also represents a world with high economic growth and a global population that peaks mid-century and then declines. However, this scenario includes a shift to less fossil fuel-intensive industries and the introduction of clean and resource-efficient technologies. Emissions of greenhouse gases peak around mid-century and then decline. Atmospheric carbon dioxide concentrations reach 550 ppm by 2100—about double pre-industrial levels (green line in Figure 14). As diverse as they are, the SRES scenarios still do not cover the entire range of possible futures. By choosing a high CO_2 and a low CO_2 scenario, we hope to create an envelope of future climate change that the Great Bay may fall within by the end of the 21st century.

For the future climate scenarios used in this study, the maximum daily precipitation amount is projected using downscaled model output from four atmospheric-ocean general circulation models (AOGCMs)(Table 1)(Hayhoe, Wake et al. 2007). The methods used to downscale AOGCM output to particular meteorological stations is described in detail in Wake et al., 2011.

Downscaled projections of the maximum daily precipitation from 2000 to 2100 under the high emissions scenario for Durham NH and Lawrence MA are listed in Table 1. There is a considerable range in results from 6.3 inches (for Durham using the CCSM AOGCM) to 11.4 (for Lawrence using the CCSM AOGCM). The large range in results from these projects suggest that using the existing 24-hour, 100-year design storm depth of 8.5 inches provided by the NRCC Atlas represents a reasonable value for the time period from 2035 to 2069.

Table 1: Downscaled global projections to regional level

Global Climate Model	Maximum Daily Precipitation – A1F1	
	Durham, NH (in/24 hour)	Lawrence, MA (in/24 hour)
CCSM	6.3	11.4
GFDL	6.5	6.7
HADCM3	7.8	9.0
PCM	7.5	10.0

CCSM – National Center for Atmospheric Research Community Climate System Model
GFDL - Atmospheric Administration/Geophysical Fluid Dynamics Laboratory
HADCM3 - United Kingdom Meteorological Office Hadley Centre Climate Model v3
PCM - National Center for Atmospheric Research Parallel Climate Model

1.3 Streamflow

One of the important aspects of climate change is the effect of these extreme and frequent precipitation events on the morphology of the river. A river's dimension, pattern and profile are the fundamental components. These components not only reflect the events of the past but also the streamflow determined by the climate and landform (Rosgen 1996). Increased precipitation due to climate change can swell the rate and direction of channel adjustment.

The annual mean flow in the Lamprey River is steadily increasing. Figure 2 demonstrates this increasing rate through the entire record of flow rates. It also indicates that prior to 1970; the Lamprey River's average flow rate was declining but nothing substantial. Post 1970, the average flow rate has been steadily increasing. The average flow rate from 1935 to 1970 was 273 cfs. The average flow rate from 1971 and 2010 has increased by 13.5% to 310 cfs.

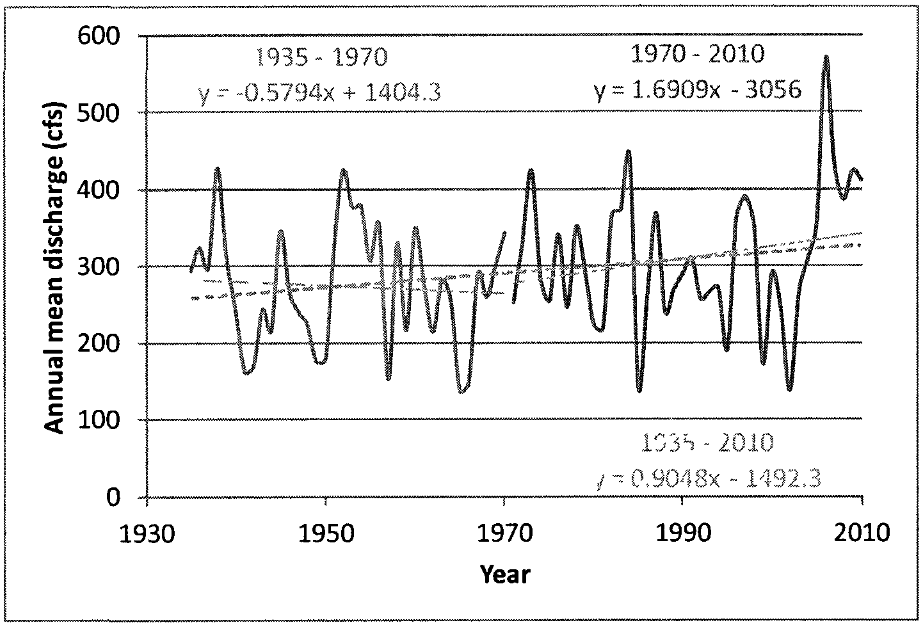

Figure 2: USGS surface-water annual statistics for Lamprey River near Newmarket, NH

1.4 Flooding

Historically, the Lamprey River has seen an increased frequency of flood flow events. Of the fifteen (15) largest events since 1934; eight (8) have occurred in last 25 years, five (5) have occurred in last 15 years, and three (3) have occurred in last five (5) years[1]. These events range from a recorded 4,270 cfs in April 1960 to 8,400 cfs in May 2006 (Figure 3).

Milly examined the frequency of flooding and found substantial increases during the twentieth century and modeling that suggest a global continuance (Milly, Wetherald et al. 2002).

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[1] Peak streamflow USGS gage 01073500, Lamprey River, Newmarket, NH http://waterdata.usgs.gov/nh/nwis/uv?site_no=01073500

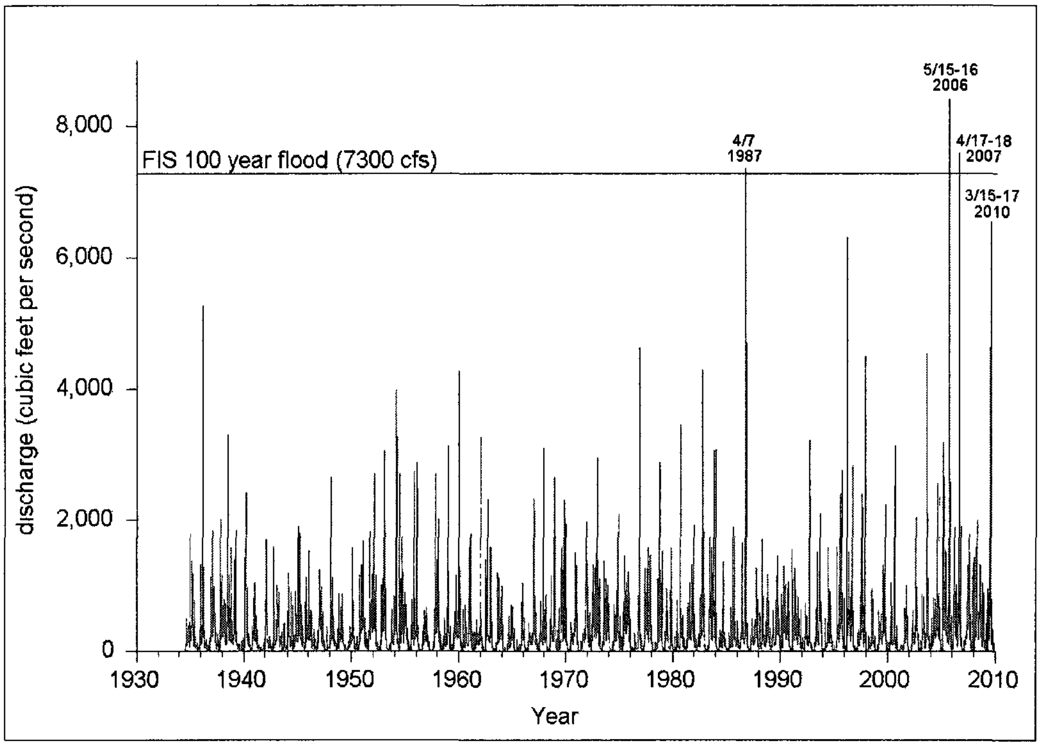

Figure 3: Daily discharges for the Lamprey River near Newmarket - July 1934 - July 2010

According to the Federal Emergency Management Agency (FEMA), flooding is one of the most common hazards in the United States (FEMA 2011). The National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973 began the federal administration process of mapping flood prone areas. *Although flooding can happen anywhere, certain areas are more prone to* serious flooding. These flood hazard locations include the low-lying areas near water or in proximity to a dam. Flood insurance studies (FIS) provide flood risk data that is used to establish actuarial flood insurance rates and to institute community floodplain regulations for sound land use and floodplain development.

A FIS contains information on the 10-, 50-, 100-, and 500-year storms, which have annual probabilities of 10-, 2-, 1-, and 0.2-percent respectively. Each FIS includes a written report about the area studied and the engineering methods used to calculate flood frequency. Associated maps illustrate floodplain boundaries for the 1- and 0.2-percent storms, base flood elevations (BFE) for the 1-percent event, and floodway boundaries based on the 1-percent storm. The BFE is the computed elevation to which floodwater is anticipated to rise during a base flood.

The FIS for the Lamprey River is included in the Rockingham and Strafford County studies(FEMA 2005). In May 2005, the community studies were compiled into county studies. Those community studies available for the Lamprey River included Raymond, Epping, Durham and Newmarket. The community of Lee, between the Epping and Durham reaches, did not have a published study available for inclusion.

The FIS used an annual peak flow frequency analysis to determine the 100-year flood flow. This type of analysis follows Bulletin 17B which is the recommended procedure for flood-frequency analysis in gaged systems. It utilizes the available peak annual stream flow data and weighted coefficient of skewness (U.S. Interagency Advisory Committee on Water Data 1982). The FIS flood flow was calculated with peak annual stream flow from 1935 through 1987. Results of that analysis applied 10-, 50-, 100-, and 500-year flood flows at the gaged location and then used a regional equation and drainage area ratios for the ungaged locations along the Lamprey River. A summary of the Town studies are provided in Table 2.

Table 2: Summary of Town Flood Insurance Studies

Community	County	Town FIS Date	Study performed by	Completed
Epping	Rockingham	October 15, 1981	Soil Conservation Service (SCS)	September 1979
Newmarket	Rockingham	May 2, 1991	USGS	August 1989
Raymond	Rockingham	October 15, 1981	SCS	September 1979
		April 15, 1992	Rivers Engineering Corp.	October 1989
		May 2, 1995	Roald Haestad, Inc.	March 1993
Durham	Strafford	May 3, 1990	SCS	September 1987
		August 23, 2001	USGS	April 1998
Lee	Strafford	No published study available		

FEMA provides guidelines for revaluating current studies based on the significance of the changes to the effective FIS flood flows. The most recent approved or revised National Flood Insurance Program (NFIP) data and maps (NFIP 2005) are considered effective and will hereafter be referred to as the FIS flood flows. The guideline bases the revaluation on the 68-percent confidence interval of the most recent analysis of peak 100-year discharge. If the new estimate is within the 68-percent confidence interval, the FIS flow remains in effect. If the new estimate falls outside the interval, the estimate is considered significant and a new study is recommended (FEMA 2009).

For the USGS gage (01073500) at Packers Falls Road near Newmarket , the pre-1987 FIS 100-year discharge for the 183 square mile watershed upstream of the gage is 7,300 cfs (FEMA 2005). Using the methodology found in Bulletin 17B(U.S. Interagency Advisory Committee of Water Data, 1982), the lower (L) and upper (U) limit of the 68-percent confidence interval for the FIS flood discharge of 7,300 cfs is:

$$L_{0\,01\,0\,68} = 6{,}886 \text{ cfs} \quad H_{0\,01\,0\,68} = 7{,}834 \text{ cfs}$$

To develop a new estimate, an updated data set of annual peak flows for the years 1935 through 2009 was collected for input into the Peak flow FreQuency analysis program (PKFQWin). This analysis program implements Bulletin 17B. The new 100-year flow estimate is 9,411 cfs. As is evident from Figure 4, due to the higher floods since 1987, there is a significant increase *(outside the upper 68-percent confidence interval for the FIS flood flow)* to the FIS model flood flow at the gage and a need for revaluation.

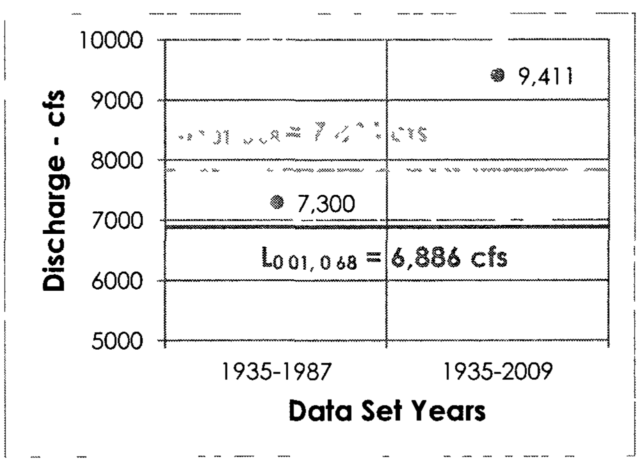

Figure 4: LP3 analysis on peak discharge at Packers Falls

1.5 Land use change

Coastal communities of New Hampshire and other states have been experiencing increased development over the past three decades. The Lamprey River is a sub basin within the Great Bay coastal watershed, which covers 1,086 square miles and includes 52 towns in Maine in New Hampshire.

Between 1990 and 2005, 21,641 acres of impervious surface was generated in the coastal region (PREP 2009). Since 1990, the coastal area has experienced a 75% increase in impervious surface (Table 3). This summarizes a steady land conversion rate of nearly 1,500 acres per year or 0.2-percent of the land area.

Table 3: Percent of total impervious area for New Hampshire Coastal watershed (1,086 sq.mi.)

Year	Impervious Area Cover (%)	Impervious Area Cover (Acres)
1990	4.30	28,710
2000	6.30	42,618
2005	7.50	50,351

Source: http://www.granit.und.edu/

Piscataqua Region Estuaries Partnership (PREP) is a joint local/state/federal program created under the Clean Water Act that has established a goal of keeping the coverage of impervious surfaces in coastal watersheds to less than 10%. In 2005, approximately 6,707 acres, or 4.9%, of the Lamprey River watershed was covered with an impervious surface. This is an 87% increase since 1990 when approximately 3,587 acres, or 2.6%, of the watershed had impervious land cover. Lamprey River watershed communities already greater than 10% impervious surface include Newmarket and Exeter (PREP 2009). The Lamprey River watershed communities experiencing more than 90% increase of impervious surface since 1990 include Newfields, Deerfield, Fremont, Epping, and Brentwood. This information is based on NOAA's coastal impervious surface survey data. A table giving the watershed community's impervious cover change is provided in Appendix A.

State wide, the population of New Hampshire has increased by 6.53% since 2000 (Census 2010). The coastal communities are experiencing a faster growth rate and associated development. Hillsborough, Merrimack and Rockingham counties account for almost 65% of the state's population. The fastest growing county in the last decade was Strafford (NHOEP 2011). A table of the population growth within the watershed communities during the past five (5) decades is provided in Appendix A. The population growth and increase in land development are displayed on Figure 5.

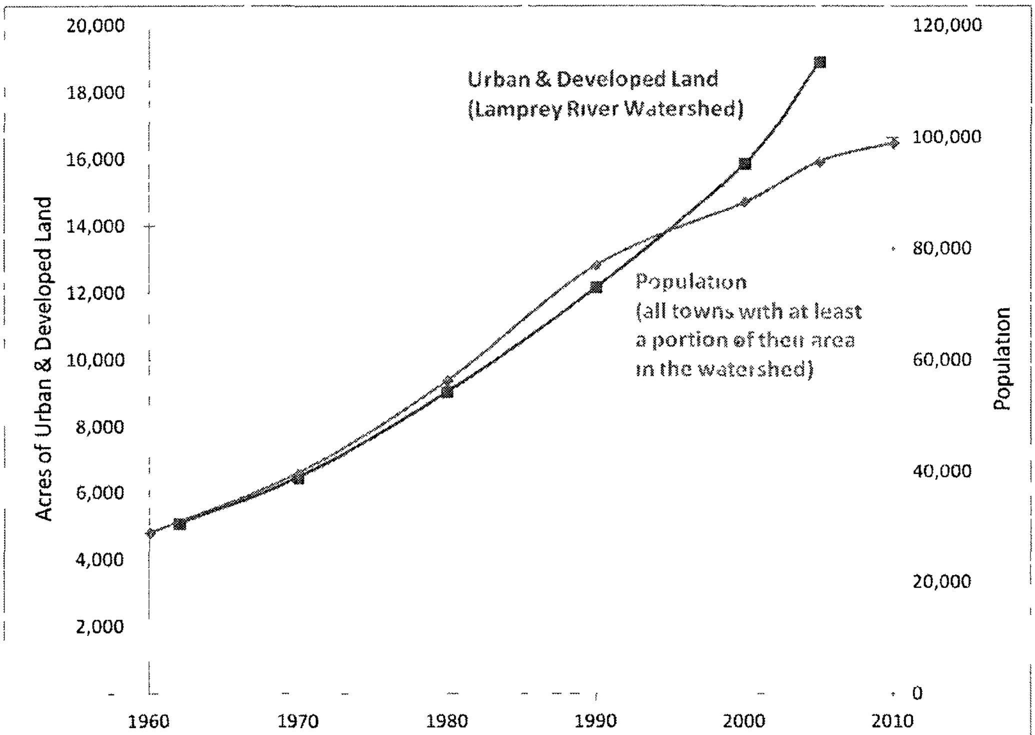

Figure 5: Population growth and land development for municipalities in the Lamprey River watershed

1.6 Development alternatives

Because business-as-usual development usually increases impervious surfaces, this decreases the available area of water infiltration and increases in runoff. Changes in anthropogenic impervious surfaces from the 1950s to the 1990s and the coincident historical mean daily streamflow have been analyzed in the upper Accotink Creek subwatershed near Annandala, VA (Jennings and Jarnagin 2002). Results of the study indicate that the amount of precipitation needed to generate bankfull discharge dropped from 80 mm (3.15 inches) in the first decade to approximately 55 mm (2.16 inches) in the final decade.

Communities have the responsibility to plan for the occurrence of potential damage of flood events and yet still allow economic growth. This research provides a demonstration of how municipal planning and low impact development strategies can be used to reduce runoff volume. By mitigating potential flood hazards, a community's resilience is increased in the wake of a flooding disaster.

Applying low impact development (LID) and redevelopment designs can reduce effective impervious cover (EIC) to future development. EIC reduction would be accomplished by hydraulically disconnecting an impervious surface from a channel drainage system (Holman-Dodds, Bradley et al. 2003) through the wide spread use of filtration and infiltration systems in a decentralized manner. LID planning can reduce the development footprint by promoting land preservation and the inclusion of infiltration based stormwater management systems (Williams and Wise 2006).

The UNH Stormwater Center (UNHSC) studies a wide range of LID designs put into practice: rain gardens and bioretention, vegetated swales, buffers and filter strips, tree filters, rain barrels, porous pavement, and other impervious surface replacements. LID approaches in instances where curbing, storm structures and pipe are eliminated are less expensive than conventional stormwater management systems (UNHSC 2009). In a highly urbanized setting, the use of many scattered LID treatment areas helps minimize frequency and size of runoff events (Zhen, Shoemaker et al. 2006). The Maryland Stormwater

Design Manual categorizes these treatment practices with a combination of planning as environmental site design (MDDES 2000).

1.7 Research objectives

The purpose of this research is to reassess the hydrology and hydraulics of the Lamprey River watershed and the spatial extent and elevation of the 100-year food event in the watercourse resulting from current and projected future land cover and rainfall depths.

There are three main goals for this study. The first goal is to quantify the change in the spatial extent of the 100-year floodplain based on current land use and revised rainfall depths. It is hypothesized that increased development and an increase in rainfall depths are responsible for an increase in the flood flow discharge, raising the flooding water surface elevation and widening the spatial extent of flooding.

The second goal is to apply build-out conditions to 2050 within the watershed based on past rates of residential and commercial/industrial development. Land development will be based on conventional stormwater design implementation which seeks to immediately convey runoff as quickly as possible, directing it with curbing to low spots for catchment and piping to detention as a means for peak reduction (Holman-Dodds, Bradley et al. 2003). The impact to the hydrologic model of the watershed is analyzed for the 2035 - 2069 time period using the NRCC rainfall depth. It is hypothesized that the build-out condition will result in increased flood levels and additional flood inundation.

Finally, the third goal is to apply low impact development (LID) and redevelopment designs that reduce EIC to the build-out scenario. It is hypothesized that a reduction in the hydrologic and hydraulic models for runoff, *peak discharge, and changes to the floodplain water surface elevation will* result.

Chapter 2

Watershed Description

The Lamprey River watershed is the largest sub-watershed of the Great Bay drainage in southeastern New Hampshire. Because of the four important falls within a short distance from the Great Bay, mills became an early central piece of the river valley (LRWA 2011). The lower falls, recognized as Macallen Dam in Newmarket was originally harnessed for power sometime around 1650.

2.1 **Existing conditions**

The majority of the Lamprey River Watershed lies within Rockingham County. The Towns of Northwood, Nottingham, and Deerfield are located in the northwestern section of the watershed. In the central section of the watershed are the Towns of Brentwood and Fremont. The Towns of Epping, Newmarket and Newfields occupy the eastern portion of the watershed. Exeter is located in the south-eastern section. The Towns of Raymond and Candia occupy the western portion of the watershed. A portion of the Strafford County towns of Strafford, Barrington, Lee, and Durham occupy the northeast portion of the watershed (Figure 6).

The Lamprey River watershed upstream of Macallen Dam is 213 square miles and is located in the Saco River coastal basin. It originates in the hilly Saddleback Mountains in Northwood, flowing through the gently rolling hills of Raymond and Fremont, to the flat coastal plains of Newmarket. Its total trek is 47 miles to the Great Bay estuary. Within the watershed are significant tributaries: Bean River, Little River, North Branch River, Pawtuckaway River, North River, and the Piscassic River. Pawtuckaway State Park and Pawtuckaway Pond are located in Nottingham and are dominant features in the upper watershed.

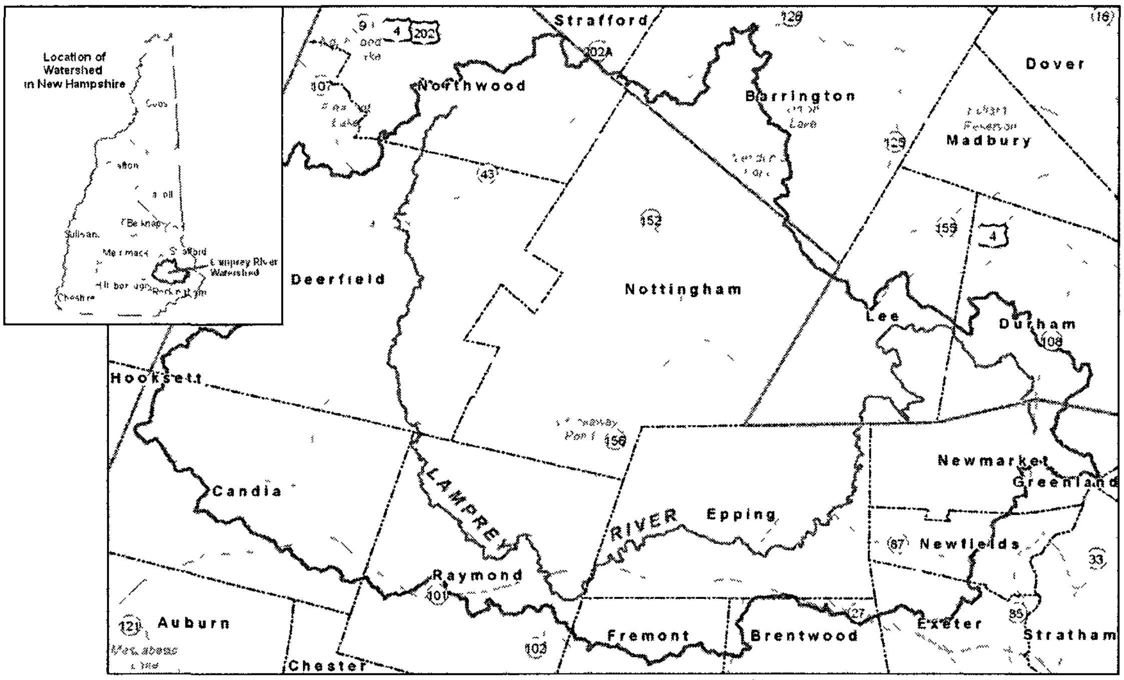

Figure 6: Primary towns and bordering communities within the Lamprey River watershed.

Land cover in the headwaters of Northwood and Deerfield is mostly undeveloped and forested (Figure 8). Most of the river's corridor upstream of Raymond is relatively undisturbed. Residential development is a common form of land use along the river's corridor and accounts for 13,646 acres of land cover within the watershed. Based on the zoning districts established in the communities, there are 125,072 acres of residential zoned land and approximately 10.5% of those acres are developed. Other than Durham, Exeter, Newfields, Newmarket, and Raymond, minimum lot size is two acres.

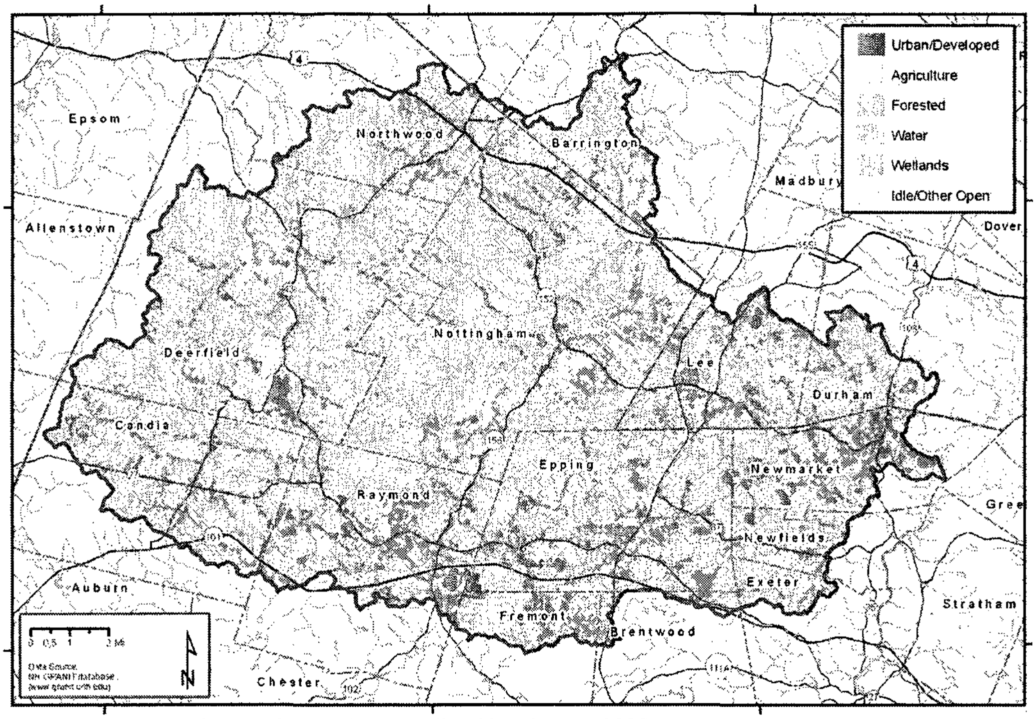

Figure 7: Lamprey River watershed generalized land use – 2005

Nottingham includes Pawtuckaway State Park and a substantial amount of conservation land. The impervious cover in this community is one of the lowest listed in Table 4. Based on the 2005 land use, 73% of the land cover within the watershed is forested, agricultural or other open space use, 13% is residential, 3% industrial/commercial and the remaining 11% is water/wetlands.

There are 1,243 acres of industrial/commercial land use, with a significant stretch adjacent to the Lamprey River in the Town of Raymond. Raymond occupies 9% of the watershed area and roughly 7.5 miles of the Lamprey River. Approximately 386 acres of the community is already established with business and industrial/commercial property. The proximity of the development is directly adjacent to the Lamprey River (Figure 8).

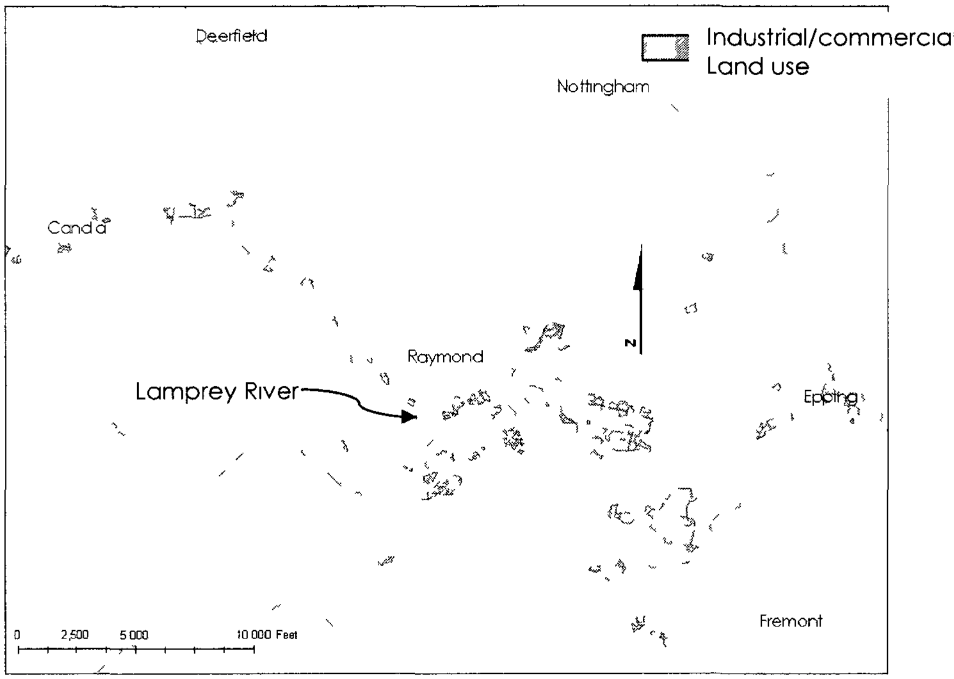

Figure 8: Current business, industrial, commercial development in Raymond

An additional 9,000 acres in Raymond is zoned for future development of business/industrial/commercial use (Figure 9). The Town has included residential development in the C2 – Residential/Commercial zoning district. This is an example of the potential development for one of the watershed communities.

Table 4 provides a breakdown of the current land cover and 2010 population in each of the fifteen (15) communities. The percent impervious cover is for the entire community, not the area within the watershed. This information is based on NOAA's coastal impervious surface survey data (PREP 2009). The impervious cover includes developed hard surfaces such as pavement, roof, and concrete. Even though open water and wetlands can be considered impervious, since there are no infiltration losses, these areas are not included in the impervious percent coverage in the communities.

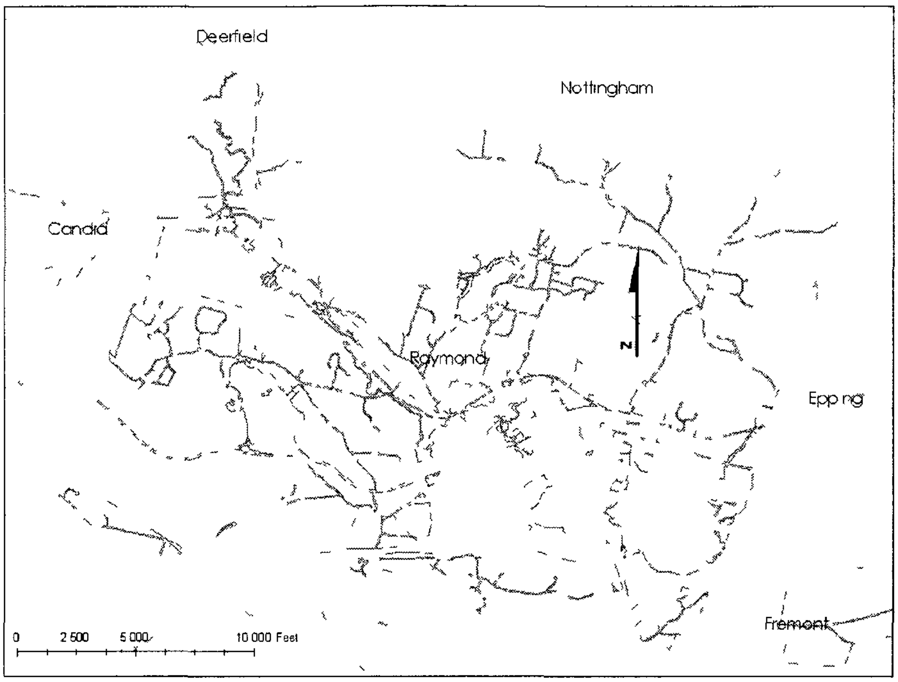

Figure 9: Map of Raymond highlighting approved zoning districts for business, industrial, commercial development

One of the goals of PREP is to keep the coverage of impervious surfaces in coastal watersheds to less than 10 percent. Impervious surfaces such as paved parking lots, roadways, and building roofs increase the pollutant load, sediment load, volume, and velocity of stormwater. Studies conducted in other regions of the country have demonstrated water quality deterioration where impervious surfaces cover greater than 10 percent of the watershed area (CWP 2003).

Table 4: Existing conditions of watershed communities

Land cover description	Acres of land cover in watershed communities							
	Barrington	Brentwood	Candia	Deerfield	Durham	Epping	Exeter	Fremont
Residential	356	35	1,184	1,576	746	1,836	115	557
Industrial/Commercial and Business	-	72	93	100	28	268	4	13
Rail/Gravel Road w/ROW					7.9			
Paved Road w/ROW	34	50	220	208	75	444	13	58
Open Space	26	78	534	1,535	547	1,386	53	268
Pasture, grassland or range	13	12	12	1,303	468	1,093	44	207
Farmsteads		4	65	158	13	123	1	24
Brush	8	41	117	113	91	657	21	80
Woods	3,316	595	9,564	24,613	3,427	11,342	1,412	2,136
Open Water/Wetlands	863	156	767	1,685	646	2,576	508	653
Natural Desert (Beaches)			4					
Newly graded (Disturbed land)	5	9	89	140	20	257	34	63
Fallow Bare Soil	1			8.8				
Community Statistics								
2010 Population	8,576	4,486	3,909	4,280	4,638	6,411	14,306	4,283
% Impervious Cover	4.7	9.5	4.8	3	7.7	7.8	12.4	5.9

Table 4: Existing conditions of watershed communities (cont'd)

Land cover description (Acres)	Lee	Newfields	Newmarket	Northwood	Nottingham	Raymond	Strafford
Residential	1,292	355	1,074	582	2,077	1,854	9
Industrial/Commercial and Business	62	1	89	54	74	386	-
Rail/Gravel Road w/ROW			1				
Paved Road w/ROW	124	9	88	62	250	487	<1
Open Space	758	151	469	271	837	368	
Pasture, grassland or range	945	146	404	175	497	188	
Farmsteads	47	19	19	4	15	66	
Brush	107	90	66	3	5	199	
Woods	4,981	2,250	3,383	7,602	27,339	8,463	87
Open Water/Wetlands	997	593	1,021	744	5,252	1,296	4
Natural Desert (Beaches)					1		
Newly graded (Disturbed land)	169	11	28	18	168	112	
Fallow Bare Soil				27	67	1	
Community Statistics							
2010 Population	4,330	1,680	8,936	4,241	4,785	10,138	3,991
% Impervious Cover	6.6	6.8	10.1	4	2.8	9.3	2.3

2.2 **History of floods**

Past history within the watershed indicates that most major flooding occurs during the spring, fall, and winter seasons. The most severe flooding occurs in the spring (March – May) resulting from a combination of snow melt, high soil moisture, and heavy rains. The floodplain areas in Raymond, Epping, Durham, and Newmarket are subject to periodic inundation caused by overflows of the Lamprey River. Real-time data records for peak annual flows recorded since 1934 at the USGS gage number 01073500 are listed in Table 5. Of the fifteen (15) largest events since 1934; eight (8) have occurred in last 25 years, five (5) have occurred in last 15 years, and three (3) have occurred in last five (5) years.

Table 5: 15 highest peak annual flows

Rank	Date	Discharge (cfs)	Return Period [1]
1	16-May-06	8,970	76-Year
2	18-Apr-07	8,450	38-Year
3	7-Apr-87	7,570	25.3-Year
4	22-Oct-96	7,080	19-Year
5	15-Mar-10	6,760	15.2-Year
6	20-Mar-36	5,490	12.7-Year
7	15-Mar-77	5,000	10.9-Year
8	15-June-98	4,720	9.5-Year
9	3-Apr-04	4,690	8.4-Year
10	30-Mar-83	4,570	7.6-Year
11	6-Apr-60	4,470	6.9-Year
12	11-May-54	4,070	6.3-Year
13	2-Feb-81	3,670	5.8-Year
14	31-July-38	3,530	5.4-Year
15	1-Apr-93	3,400	5.1-Year

[1]Return period based on order statistics and Wiebull plotting position of peak annual events

2.3 **Subwatersheds**

The subwatershed delineation is displayed on Figure 10. The entire Lamprey River watershed consists of eleven (11) sub basins. These were created by developing a catchment location along the river's path that coincides with a change of flow regime from the FIS. The catchment location is the downstream site that delineates a sub basin for every stream segment. These sub basins range in size from 0.9 to 58.3 square miles. The sub basin numbers are labeled automatically during the process of delineation with the Hydraulic Engineering Center Geographic Hydrologic Modeling System (HEC-GeoHMS)(USACE 2009) tools in ArcMap. Table 6 provides a description of the boundary condition for the individual sub basins and delineated drainage area. Figure 10 is a graphic presentation of the HEC-GeoHMS of delineating the sub basins.

Table 6: Lamprey River watershed sub basin data

Sub basin No.	Boundary Condition	Area (mi^2)	Cumulative Area (mi^2)
W6510	(RT27) Alt. RT 101, Raymond	32.2	32.2
W8600	Langford Road, Raymond	19.0	51.2
W11900	Downstream corporate limit, Town of Raymond	16.0	67.2
W10910	Western corporate limits, Town of Epping	6.5	73.7
W8380	Blake Road, Epping	12.3	86.0
W11020	RT 101, Epping	6.1	92.1
W6730	Northern corporate limits, Town of Epping	58.3	150.4
W7060	USGS Gage No. 01073500	33.9	184.3
W7920	Durham/Newmarket corporate limits	4.5	188.8
W10250	Confluence of Pisscassic River	21.7	210.5
W8590	Macallen Dam, Newmarket	0.9	211.4

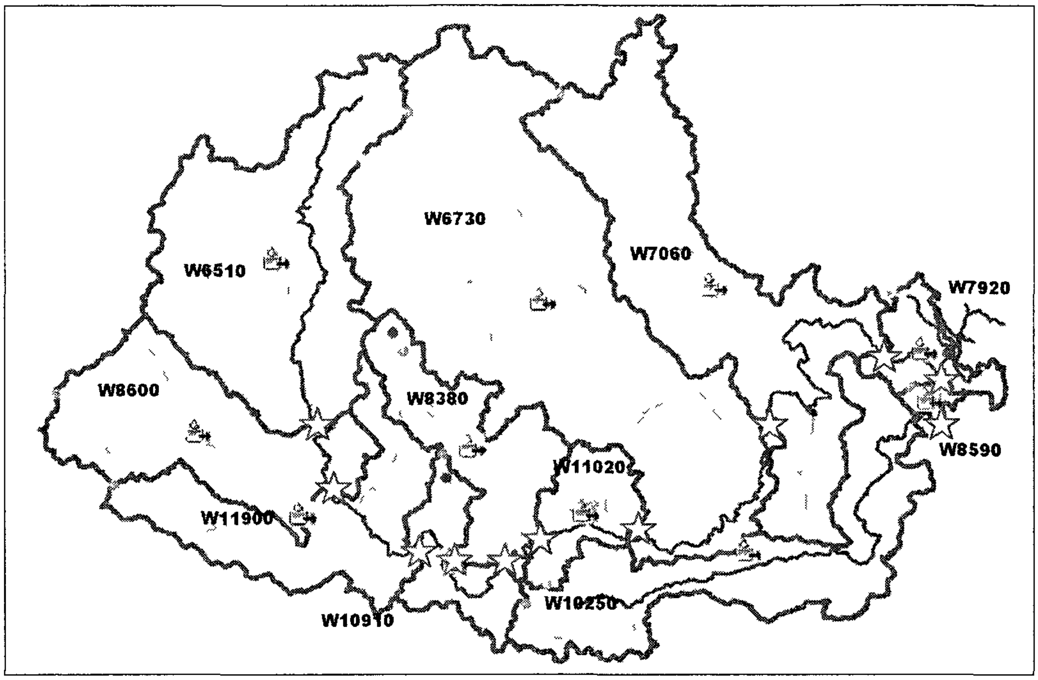

Figure 10: Lamprey River watershed showing the delineation of the eleven sub basins. Catchment locations are noted with yellow stars along the Lamprey River.

Chapter 3

Materials and Methods

3.1 Methodology overview

The proceeding sections describe in detail the steps taken to develop a hydrologic and hydraulic model for the Lamprey River. These following flow charts are a condensed version of the overall project:

- Project overview (Figure 11)
- Future build-out condition assessment (Figure 12)
- Process using the ArcMap tool HEC-GeoHMS (Figure 13)
- Hydrologic analysis and calibration in HEC-HMS (Figure 14)
- ArcMap tool HEC-GeoRAS (Figure 30)
- Hydraulic analysis and calibration in HEC-RAS (Figure 32)

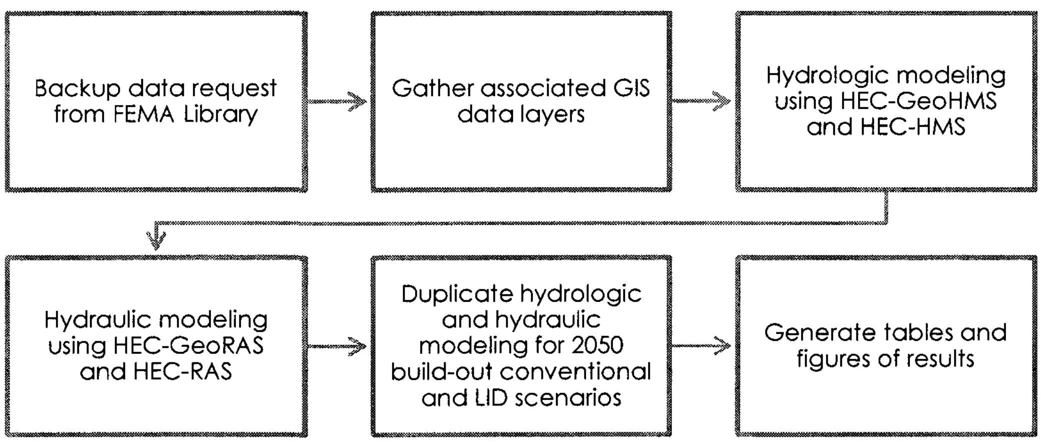

Figure 11: Overview of project

3.2 Hydrologic Modeling

3.2.1 *Flood Insurance Study (FIS) Model*

The FIS used an annual peak flows (1935 – 1987) and a frequency analysis to determine the 100-year flood flow. This type of analysis follows Bulletin 17B which is the recommended procedure for flood-frequency analysis in gaged systems (U.S. Interagency Advisory Committee of Water Data, 1982). Table 4 in the Rockingham County FIS, provided in Appendix F, lists the summary of peak discharges and affiliated drainage areas (FEMA 2005).

The United States Geological Survey (USGS) operates a streamflow gage upstream of Packer's Falls Road near Newmarket. In order to duplicate the FIS, the *annual maximum instantaneous peak discharges were collected for the* years 1935 through 1987 and saved in a standard Water Data Storage and Retrieval System (WATSTORE) text format. This input file was used in the USGS Office of Surface Water software program, Peak flow FreQuency analysis program (PKFQWin). This analysis program implements Bulletin 17B using a log Pearson Type IIII (LPIII) distribution analysis. The program provides an estimate of flood exceedance probabilities including the 100-year event.

For this research the complete data set of the peak annual discharges for years 1935 through 2009 was downloaded, saved in the same text format, and analyzed with PKFQWin. Based on the recent extreme flood events, a data set for the past 30 years of record (1980 to 2009) was evaluated in regards to the impact of climate change on annual peak discharges. Appendix B provides the PKFQWIN reports for these three analyses.

The FIS, Table 2, for the Lamprey River was the template used to establish sub basins within the 213 square mile watershed. In the FIS, sub basins are identified by land markings such as road crossings or corporate boundaries. These were the initial outlet points used along the watercourse to establish downstream boundaries. The original FIS backup data files provided by the FEMA library provided specific information on the cross section where a flow change occurs. This relationship between the cross section and assigned flood flow revised the outlet point locations for the sub basins. The sub basin boundaries were regenerated in HEC-GeoHMS using the georeferenced cross section locations.

3.2.2 ***Rainfall-Runoff Model***

Because this thesis is focused on the land use within the watershed, a rainfall-runoff model was developed to simulate current conditions, future build-out, and future build-out with LID. This is an acceptable approach according to FEMA's guidelines (FEMA 2009). In this research, hydrology was generated for the 213 square mile watershed upstream of Macallen Dam for the Lamprey River using using Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS)(USACE 2008) and Geographic Hydrologic Modeling System (HEC-GeoHMS)(USACE 2009).

A rainfall-runoff hydrologic analysis is commonly used to assess the changes to land use within the watershed and develop flood flows. The process presented includes the NRCS (former Soil Conservation Service (SCS)) Curve Number (CN) method. Interception, depression storage, evaporation, and infiltration are all accounted for in the loss calculations of the SCS runoff curve number method (Akan 2003).

The CN values are generated by analysis of land use and hydrological soil groups (HSGs). Soils are classified into HSGs to indicate the rate at which water enters the soil at the surface. The four HSGs defined by the NRCS include: group A, low runoff potential and high infiltration; group B, moderate infiltration; group C, low infiltration; and group D, very low infiltration and high runoff potential. By defining soils with this identification, characterization of land parcels that present a high potential for infiltration of surface waters can be identified (Brito, Costa et al. 2006).

The simulation for direct runoff of excess precipitation is achieved with a transform method. The selected transform method used the SCS unit hydrograph (UH) empirical model to convert excess precipitation into a hydrograph. This method was also selected as it permitted the procedure of calibrating the UH to observed events.

Subsurface processes interact with the infiltration and surface runoff. During the calibration of the UH to the observed event, the baseflow of the Lamprey River was subtracted from the observed flow so that only a direct runoff hydrograph was used for comparison.

The Muskinghum-Cunge routing method for the river segments was selected because it uses channel properties and works in reaches with mildl slopes. This physical-based routing method uses Manning's equation and Manning's roughness coefficients (USACE 2008).

The final hydrologic parameter for each sub basin remaining is the time of concentration (TOC). The TOC is estimated in accordance to the NRCS TR-55 methodology (NRCS 1986).

3.2.3 ***2050 Build-out Model***

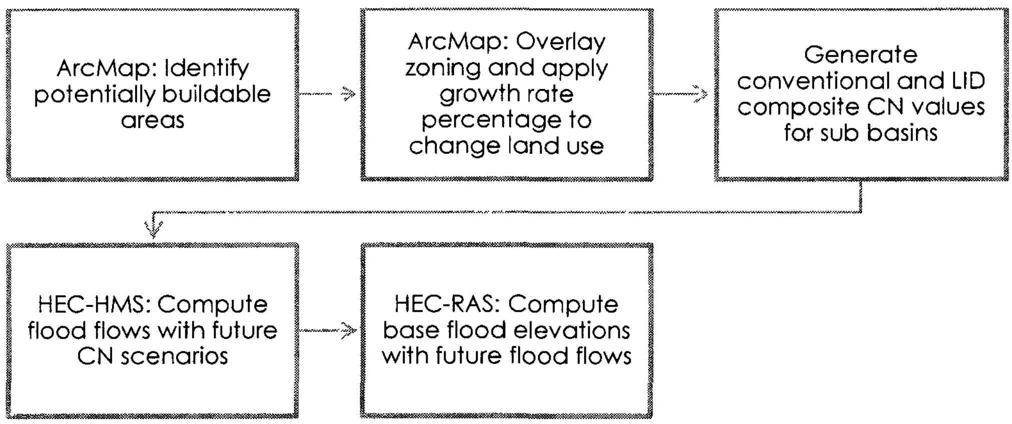

Figure 12: Build-out scenario overview

As presented in Table 2, the Lamprey River flows through five communities although its watershed includes the entire or portions of 15 communities. Each one of these communities has zoning standards for development (i.e. minimum lot size, frontage, allowable building footprint). Geographic Information System (GIS) layers of the watershed were acquired from the New Hampshire Geographically Referenced Analysis and Information Transfer System (NH GRANIT).

Projections of growth and development are required to evaluate the future potential increase to the floodplain elevation and spatial extent. Expected changes in land use due to population growth and associated development, as well as increased precipitation resulting from climate change, will affect the floodplain. The methodology used for the build-out identifies potentially buildable areas by first eliminating the following:

- Developed land (from 2005 land use layer) including residential, commercial, industrial, transportation, and utilities
- Wetlands listed under the National Wetland Inventory (NWI) /surface water
- Steep slopes, based on soil slope categories of D (moderately steep) or E (steep), which eliminates all slopes in excess of 15%
- Conservation lands
- Public water supply protection areas

By overlaying the zoning for residential, followed by commercial/industrial, and then applying the percentage of growth rate (Table 7), newly developed lands are distinguished in the watershed.

Table 7: Percentages used to determine build-out for the Lamprey River watershed

Zoned Use	Estimated Build-out and Growth Projections[1]		
	2006 - 2030	2031 - 2050	Total % increase by 2050
Residential	1.2%/Year	0.6%/Year	51.87%
Commercial/Industrial	0.85%/Year	0.55%/Year	37.9%

[1]Rockingham Planning Commission and Southern New Hampshire Planning Commission growth data projections

3.2.4 ***Geospatial Hydrology in ArcMap***

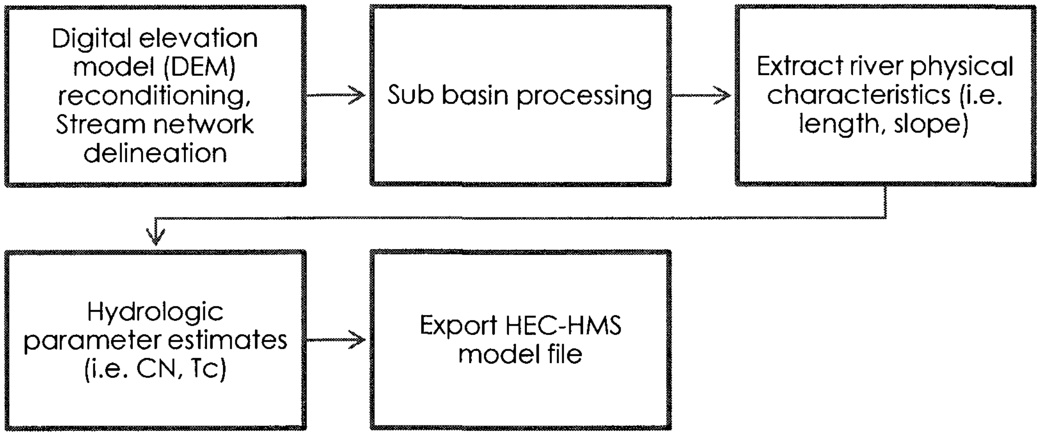

Figure 13: HEC-GeoHMS overview

(a) Data Management

The USGS 7.5-minute digital elevation model (DEM) contains a grid of terrain cells of surface elevations at a spacing of 30 meters in both the x and y direction. This DEM data was used to generate the stream network and sub basin areas within the watershed. All data layers were projected to the NAD83 New Hampshire State Plane coordinate system. A list of the GIS layers appears in Appendix C.

(b) Terrain Preprocessing

Before hydrologic modeling with HEC-HMS was possible, the terrain model is used to derive eight datasets described in proceeding sections. These processes were performed using HEC-GeoHMS in ArcMap. The steps involved delineation of the Lamprey River, its tributaries, watershed, and watershed properties (i.e. runoff curve number, time of concentration). With the DEM and GIS tools, the watershed properties were extracted using automated procedures (USACE 2009).

The spacing of elevations in the DEM was not sufficient for the stream centerline; therefore the DEM was modified to be consistent with the input vector stream network. This DEM reconditioning increases the degree of agreement between stream networks delineated from the DEM and input vector stream network shape file. If the reconditioning process was not applied, the path of the Lamprey River would be too crude meaning that the reach lengths generated in ArcMap would not represent site conditions. This is mainly due to the precision of elevation data along the stream in the initial DEM. By manually detailing, or burning in the stream network, a distinct stream profile was created and a new DEM was created. Application of this process is followed by the filling in of any sinks that have been created in the grid; if a cell is surrounded by higher elevations, the water is trapped and cannot flow out of the sink.

After the terrain preprocessing, the reconditioned DEM is the starting point for delineating sub basins and river reaches. The first five of the eight datasets are grid layers that represent:

- Flow Direction, defines the direction of steepest descent for each terrain cell
- Flow Accumulation, determines the number of cells upstream draining to a given cell (upstream drainage area can be calculated by multiplying the flow accumulation value (number of cells) by the grid cell area (30m x 30m)
- Stream Definition, the generation of a stream defined by the number of Flow Accumulation cells
- Stream Segmentation, divides the grid of streams into segments, these are sections of the stream that connect two joining streams (junction), junction and an outlet, or junction and the drainage divide

- Catchment Delineation, creates a grid layer that delineates a sub basin for each stream segment

When establishing the stream definition for the Lamprey River, the number of flow accumulation cells belonging to one stream network was defined at 1% of the largest drainage area in the entire DEM. This minimized the number of streams defined so the catchment delineation process generated the larger sub basins for the Lamprey River. The flow accumulation was set to 5.2 square kilometers (2 square miles).

A separate terrain processing was performed on a portion of the Oyster River watershed along the RT108 corridor. Flood flows from the Lamprey bypass into Hamil Brook which is a tributary to the Oyster River. Since the Oyster River bypass area was less than two (2) square miles, the small tributaries did not get defined at the same flow accumulation scale used for the Lamprey River watershed. In order to generate a stream network and eventual sub basins, the flow accumulation was defined at an area of 0.4 square kilometers (0.017 square miles).

These five functions created datasets that are digital images referred to as rasters. The next two functions convert the raster data developed into vector format (geometrical shapes). This included:

- Catchment Polygons, processing the catchment delineation grid into a polygons with assigned perimeter length and area attributes
- Drainage Line, converts the stream definition grid into a drainage feature that identifies in which catchment polygon it belongs

Finally the last function is:

- Watershed Aggregation, this accumulates the upstream sub basins at every stream confluence

(c) Basin Processing

After the terrain processing is completed, the first watershed project was defined by identifying the downstream outlet area. For the Lamprey River, it was the Macallen Dam in Newmarket.

Since the FIS drainage areas are to be duplicated in this research, the associated flow regime changes described in FIS data were chosen as the drainage points to batch the sub basins within the watershed. The catchment polygons were either merged or subdivided until the area of the eleven sub basins were comparable to the drainage area size noted in the FIS.

Next, it was possible to extract physical characteristics of the streams and sub basins. These characteristics included computed length of the river segments, upstream and downstream elevations of the reach and the slope of the river segments. Basin slope uses the slope grid to determine the average slope for the sub basin. Longest flow path creates a polyline that stores the upstream and downstream elevations and slope between endpoints.

The Basin Centroid, Centroid Elevation, and Centroidal Flow Path are hydrologic elements easily performed in GIS. Some techniques for estimating flood-peak discharges require this data. Although these characteristics were generated, the rainfall-runoff methodology employed in this research did not require this attribute information.

A second project file was created for the RT108 crossing over Hamil Brook in Durham. This crossing is approximately 1.2 miles north of where the RT108 crosses over the Lamprey River in Newmarket. Hamil Brook is a tributary in the southern portion of the Oyster River watershed. The basin processing was performed for the watershed and saved as a separate map file.

(d) Hydrologic Parameter Estimation

After extraction of the physical characteristics of the streams and sub basins, a number of hydrologic parameters are estimated. These are the model input parameters used in HEC-HMS. HEC-GeoHMS has the tools to estimate and assign a number of watershed and stream parameters (i.e. CN, loss rates, reach routing, time of concentration). In order to simulate the process of direct runoff of excess precipitation on the watershed, the specifications for this project included a loss and transform method.

The SCS CN loss method was selected to determine the loss of total precipitation for the watershed during rainfall events. This loss method equates the sum of infiltration and precipitation left on the surface equal to the total incoming precipitation.

A precipitation transform method (converting rainfall to runoff) is selected to generate actual surface runoff. Several HMS options are available and this research used the SCS unit hydrograph (UH) method. The basic concept of the SCS UH is a dimensionless, single-peaked UH that when watershed lag time is specified, an entire hydrograph can be generated from precipitation. Lag is the time separation between the centroid of the rainfall excess hyetograph and the peak of the hydrograph. Lag is empirically related to time of concentration by: Lag = 0.6(TOC).

TOC, in accordance to the NRCS TR-55 methodology: sheet flow, shallow concentrated flow, and channel flow, was used to estimate travel time for the flow paths of the individual sub basins. The estimated flow regimes populated an external spreadsheet that was evaluated to overwrite the GIS derived times. The spreadsheet required additional inputs such as Manning's roughness coefficient, channel cross section area, and wetted perimeter. The geometry (cross section area and wetted perimeter) needed to generate time of channel flow were produced by using the New Hampshire 2005 Regional Hydraulic Geometry Curves (Schiff 2006). Regional hydraulic geometry curves describe the relationship between drainage area of a channel and the bankfull hydraulic characteristics. Use of these curves have allowed the hydrologic community alternatives to traditional point-based gauging methods that require survey and remote sensing (Smith and Pavelsky 2008). The TOC for the individual sub basins would also be a selected parameter during optimization trials for calibrating the UH (Section 3.2.6).

Channel characteristics for the reach routing is an estimated parameter in GIS and performed in the same way the NRCS channel flow regimes; however, the parameter needs to be entered manually into the HEC-HMS model. The Muskinghum-Cunge method was selected for reach routing as a placeholder until the simulated parameter in the channel reaches are established through HEC-HMS optimization trials.

(e) HEC-HMS Model File

Upon completion of the previous steps, HEC-GeoHMS verifies all the data for consistency. The two data project files were checked for unique names used for river reaches and sub basins in order to keep data separated and not risk any overwriting or loss of information. Additionally it confirms that river reaches and centroids are contained within each sub basin and that there is connectivity between the stream segments, sub basins and the outlet point. Once confirmed, the project schematic of the hydrologic system was generated to show sub basin nodes and reach links/junctions. Geographic coordinates are tabulated for each hydrologic feature to maintain the geospatial information after export. Finally a background map to capture the geographic information of the sub basin boundaries and stream reaches is prepared for export. A HEC-HMS basin file was generated containing all the hydrologic elements, their connectivity, and related geographic information.

3.2.5 *HEC-HMS Model Components*

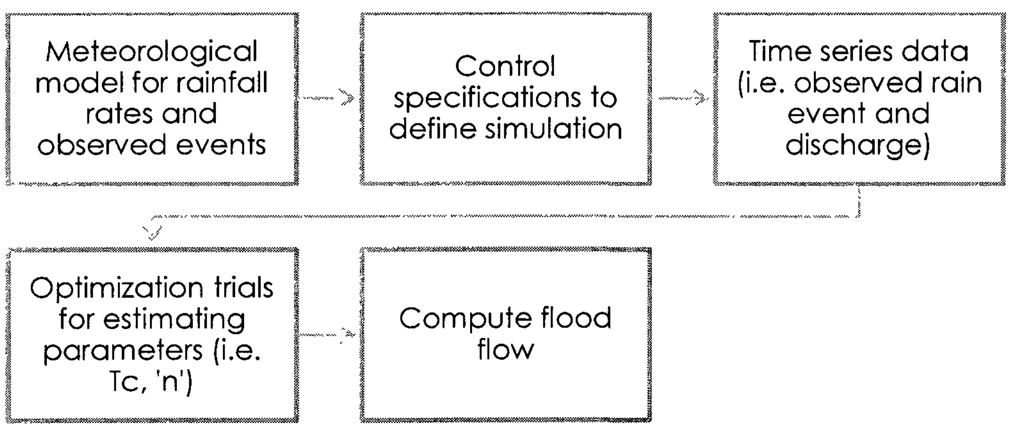

Figure 14: HEC-HMS overview

The set of files generated in GIS with HEC-GeoHMS made it possible to represent the sub basins in the watershed with several hydrologic elements.

Once opened in HEC-HMS the physical characteristics and estimated hydrologic parameters are accessible for setting up optimization trials and simulation runs.

(a) Basin Geometric and Hydrologic Model

The basin model contains the physical representation of the watershed. Hydrologic elements (i.e. sub basins, reaches, junctions) are connected into a network to simulate runoff processes. Each element contains all the parameters associated with the methodology chosen in HEC-GeoHMS. For instance, the sub basins parameter includes the composite CN value for abstractions and UH for runoff calculations. These values are now independent of GIS values extracted with HEC-GeoHMS.

(b) Meteorologic Model Manager

The meteorologic model manager is one of the main input components of the hydrologic analysis. Several meteorologic models were added such as the 24-hour, 100-year design storms based on the TP-40[2] and the NRCC[3] rainfall atlases. Additional calibration events included the measured rainfall hyetographs for the May 2006[4], April 2007[5] and March 2010[6] observed events.

(c) Model Control Specifications

The model control specifications do not contain much parameter data. The specifications define the simulation window (starting and stopping) and the computational time interval. A control specification was established for each of the simulations. For the synthetic precipitation design storms, the storm must be

---

[2] Technical Paper 40 (based on records from 1938 – 1958)
[3] Northeast Regional Climate Center (based on records from 1938 – 2010)
[4] 13.26 inches precipitation over thirteen days http://www.weather.unh edu/multiple mp
[5] 7.65 inches precipitation over five days http://www weather.unh edu/multiple mp
[6] 7.02 inches precipitation over four days http://www weathei unh.edu/rr ultiple.mp

sufficiently long so that the entire watershed is contributing to runoff at the downstream concentration point (USACE 2000). Therefore, the end and start time was three times the longest sub basin time of concentration. For the calibration events, the start and end times matched the observed event data.

(d) Time Series Data

Time series data is often called observed flow or observed discharge. This data was helpful in calibrating the model and used for the optimization trials. The raw data for three precipitation events and three gage hydrographs were used as the recorded 15 minute rainfall and river discharge during the May 2006, April 2007, and March 2010 events.

3.2.6 ***Model Calibration***

The goal of calibration is to identify parameter value adjustments so that the simulated results match the observed hydrographs. The mathematical search is a trial and error analysis (optimization trials) that iterates until the simulated measurements: runoff volume, peak flow, time of peak, and time of center of mass, is within an acceptable error range (less than 5%) of the observed hydrograph. By comparing measured discharge from a significant event to the model, the reliability of the model is improved (FEMA 2009).

(a) Optimization Trials

A test is set up by creating a trial name and selecting a simulation run that contains one element in the simulation where there is observed flow. This element was the USGS gage at Packer's Falls Road near Newmarket and defined as J1271 in the HEC-HMS basin model.

The USGS gage is located 380 feet upstream of Packer's Falls Road crossing near Newmarket. It has been in operation since 1934. The upstream drainage area is 183 square miles. Real-time discharge and raw precipitation data were obtained from the USGS Instantaneous Data Archive (IDA) for gage site 01073500, Lamprey River near Newmarket, NH and the University of New Hampshire Weather Station (UNHWS) respectively. Three major events considered included: May 12 – 16, 2006; April 15 – 18, 2007; and March 12 – 16, 2010.

In considering these three events, another important comparison is the distribution of rainfall. The NRCS has four (4) synthetic 24-hour rainfall distributions: type I, IA, II, and III. These rainfall distributions are fundamental to the SCS UH. Southern New Hampshire is located within the type III region. The total precipitation for each of these rainfall events was applied to a type III distribution to construct a hyetograph (Figure 15, Figure 16, and Figure 17).

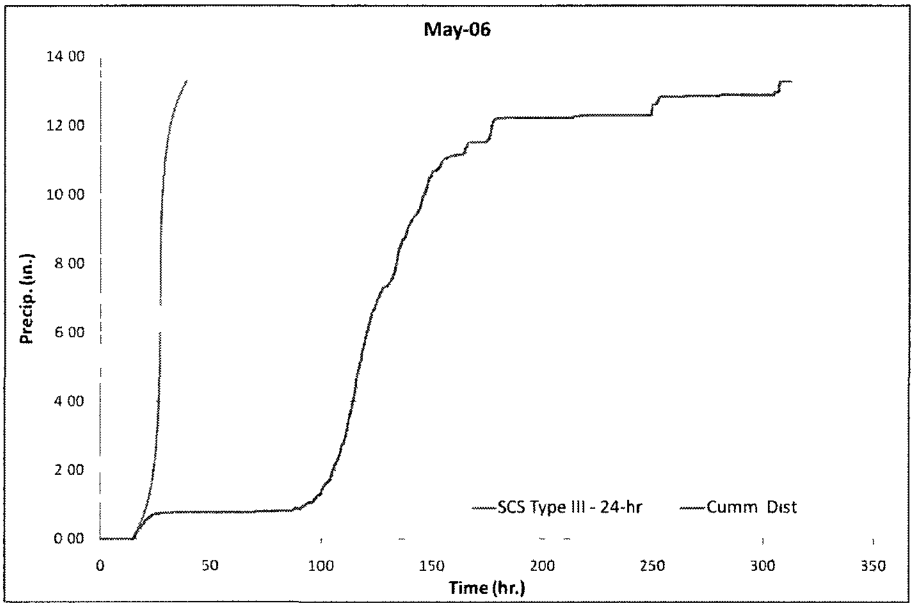

Figure 15: Type III rainfall distribution for May 2006 event

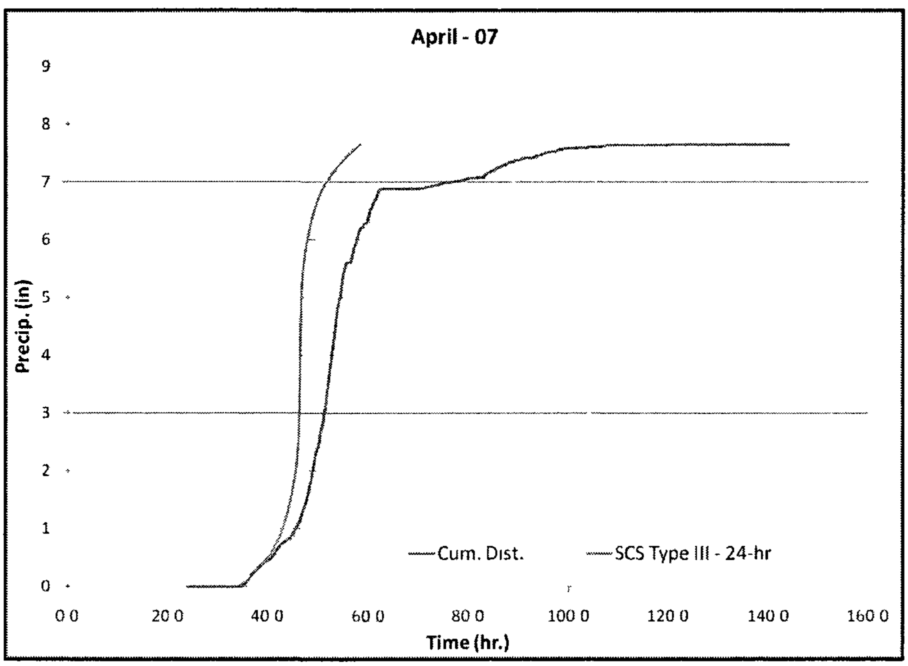

Figure 16: Type III rainfall distribution for April 2007 event

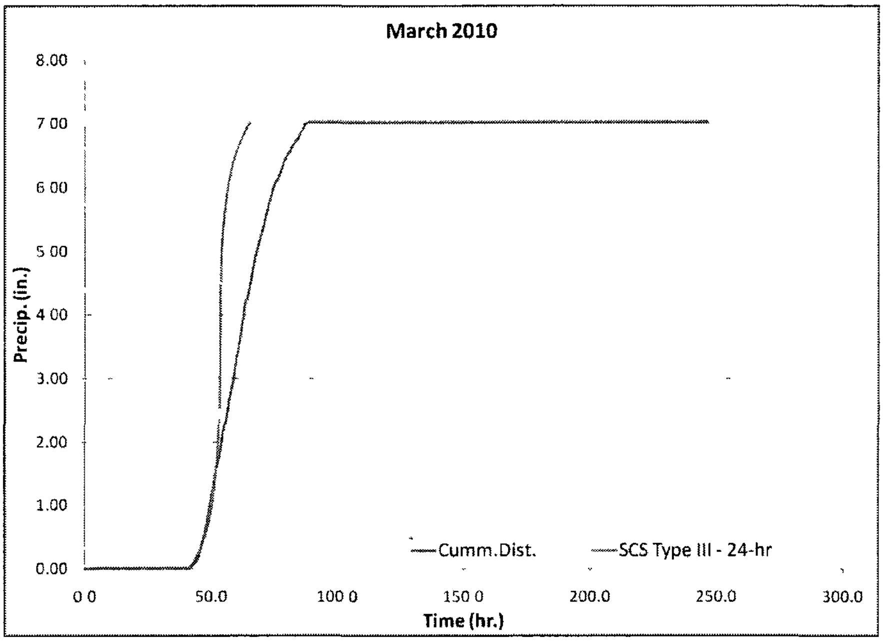

Figure 17: Type III rainfall distribution for March 2010 event

The UNHWS is located on the UNH campus in Durham, NH. In order to validate its use for the watershed, recording National Weather Stations (NWS) in Durham, Epping, and Greenland were used for comparison. A graphic representation of the three events is provided in Figure 18, Figure 19, and Figure 20. The figures indicate that the rainfall intensity and cumulative amount was similar throughout the watershed. During the March 2010, the NWS in Durham did not record any data and the Epping NWS station recorded less rainfall intensity on day three compared to the University's and Greenland's station, 0.09-, 0.21-, 0.19-in/hr respectively. The raw precipitation data from the UNH weather station provided a measured precipitation that was entered as time series data into HEC-HMS.

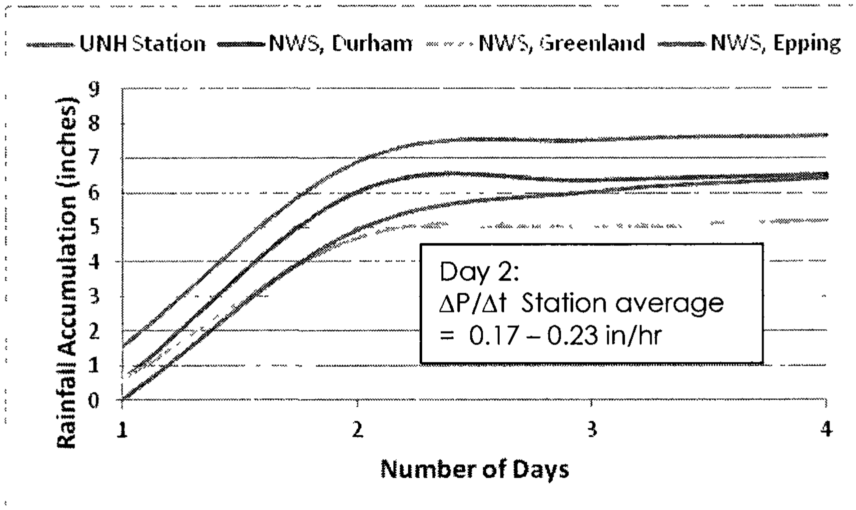

Figure 18: April 15 - 18, 2007 Record of climatological observations

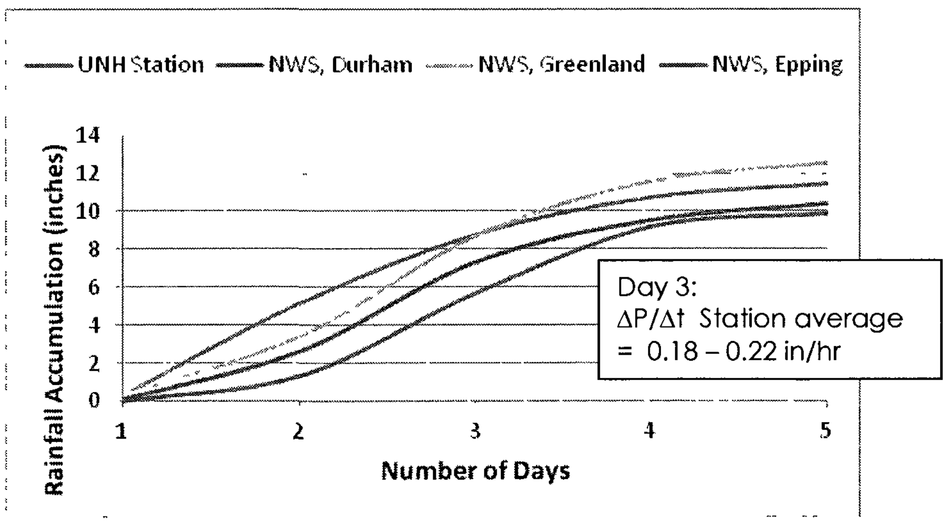

Figure 19: May 12 – 16, 2006 Record of climatological observations

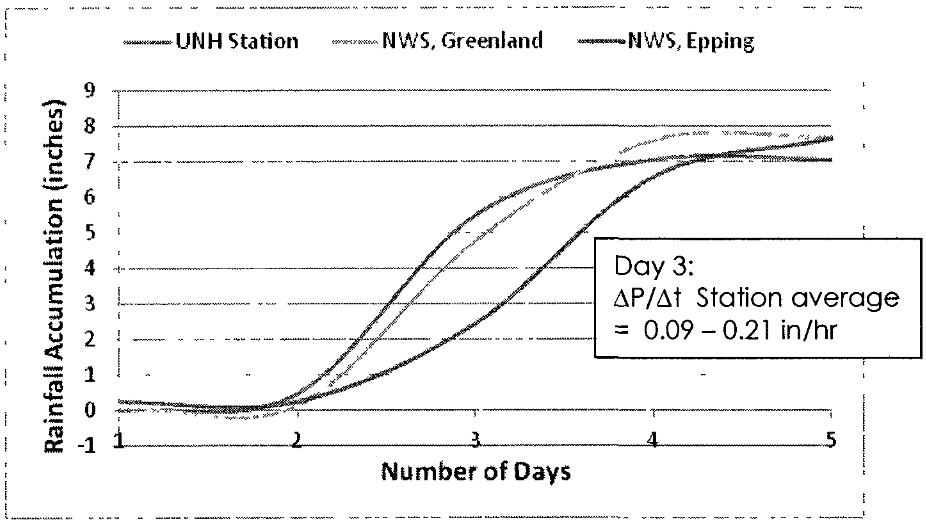

Figure 20: March 12 - 16, 2010 Record of climatological observations (Durham not available)

Gage discharge data was downloaded from the USGS IDA for the three major events and loaded into individual spreadsheets. The data was retrieved for an additional day prior to and after the precipitation event. This discharge data was then used to generate total runoff hydrographs. The total runoff hydrograph consists of two parts, direct runoff and baseflow. Three methods were evaluated to separate the direct runoff and baseflow: Constant-Discharge; Constant-Slope; and Concave (Figure 21). The resulting direct runoff hydrographs used constant and the concave baseflow separation processes for simplistic and realistic reasons respectively. The direct runoff hydrograph was entered as the discharge gage time-series data in HEC-HMS.

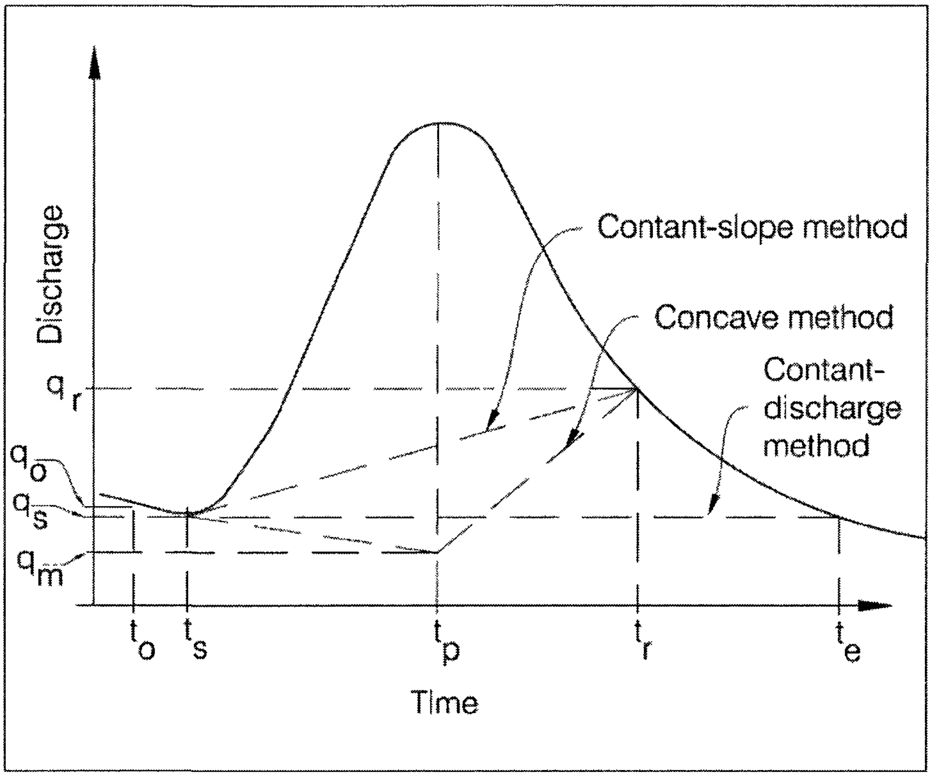

Figure 21: Baseflow separation methods (McCuen 2004)

Constant discharge baseflow separation is the simplest method to use and is set at the lowest discharge rate. The constant flow is subtracted from the observed flow to generate a direct runoff hydrograph.

The baseflow function is:

$$q_b = q \; for \; t < t_s \quad (1a)$$

$$q_b = q_s \; for \; t_s \le t \le t_e \quad (1b)$$

$$q_b = q \; for \; t_e < t \quad (1c)$$

Where: q_b = baseflow (cfs)

q = observed flow (cfs)

q_s = lowest discharge rate (cfs)

t = time at observed flow

t_s = time at lowest discharge rate

t_e = time where end discharge equals q_s

For the concave method, baseflow continues to decline pending the time of peak discharge. After time of peak, it increases meeting the inflection point on the recession limb of the discharge hydrograph.

The baseflow function is:

$$q_b = q \ for \ t < \ t_s \quad (2a)$$

$$q_b = q_s + (t - t_s)\left[\frac{q_o - q_s}{t_o - t_s}\right] \ for \ t_s \le t \le t_p \quad (2b)$$

$$q_b = q_m + (t - t_p)\left[\frac{q_r - q_m}{t_r - t_p}\right] \ for \ t_p \le t \le t_r \quad (2c)$$

$$q_b = q \ for \, t_r < t \quad (2d)$$

Where: q_b = baseflow (cfs)

q = observed flow (cfs)

q_o = discharge rate directly before lowest (cfs)

q_s = lowest discharge rate (cfs)

q_m = discharge rate at peak time (cfs)

t = time at observed flow

t_o = time at q_o

t_s = time at q_s

t_r = time at inflection point

t_e = time where end discharge equals q_s

Of the three baseflow separation procedures, the concave method is the more realistic representation of the physical processes that control flow (Chow 1959; McCuen 2004).

(b) Estimated Parameters and Analyzing Simulations

In order to match the simulated results to the observed event, the parameter estimation process called optimization is used to adjust the initial HEC-GeoHMS estimates for the sub basin transform parameter (lag time) and reach routing parameter (Mannning's n for the Muskinghum-Cunge method and Muskinghum X, Y, and number of subreaches for the Muskinghum method). The optimization uses search algorithms to provide the best value of an index, also known as the objective function. This index is a goodness of fit between the simulated and observed hydrograph.

The sub basin's transform parameter, and reach routing parameters were the focus for estimation to calibrate the simulated model. The sub basin loss rate parameter, CN, was estimated in order to verify the sensitivity of this parameter. The research would use the initial composite CNs generated by a land use analysis In ArcMap and be adjusted for future build-out scenarios.

Three streamflows, May 2006, April 2007 and March 2010, were used as the observed hydrographs to estimate selected parameters during the optimization trials in HEC-HMS. Prior to beginning the optimization trials, a hydrologic analysis was performed using the initial ArcMap input values (i.e. sub basin area, CN, reach routing, lag time). The initial value for SCS Lag, provided in Table 24, is the result of the basin processing performed in ArcMap with the HEC-GeoHMS tools. Results indicated that the modeled watershed was draining too fast. In reviewing the comparison between the center of the rainfall event and the time to peak at the USGS gage, the three rainfall events: May 2006, April 2007, and March 2010, had lag times of 48, 58, and 43 hours respectively. The time of concentration for the Lamprey River watershed was more than 70 hours demonstrated by the graphic presentation of the rainfall hyetograph and discharge runoff for the March 2010 event (Figure 22).

Because the total travel time initially generated in ArcMap was less than 12 hours, the selected parameters for estimation included the sub basin's transform parameter, lag time, and reach routing parameters: Mannning's n for the Muskinghum-Cunge method and Muskinghum X, Y, and number of subreaches for the Muskinghum method.

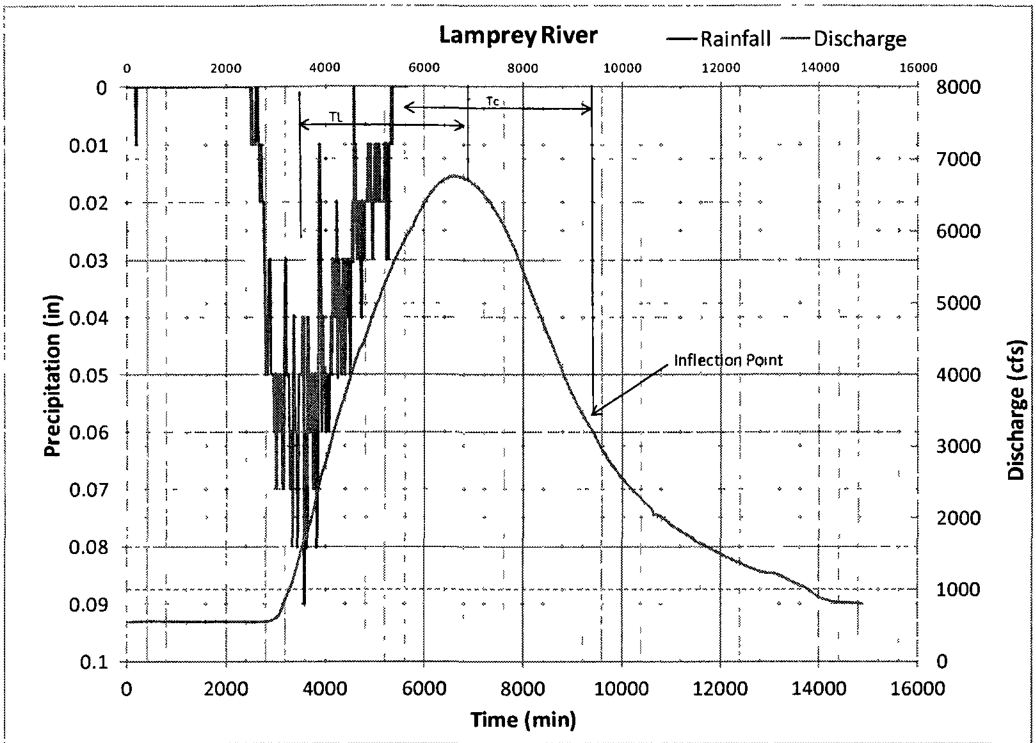

Figure 22: Time of concentration and lag time for March 2010 event

Initial optimization trials began with comparison to the entire time frame and volume of the discharge hydrograph. The process progressed to applying a goodness of fit between the time span of the rising and lowering limb of the discharge hydrograph. The next set of trials used direct runoff hydrographs developed with the constant-discharge baseflow separation process.

The previously mentioned parameters were estimated individually for each trial. As noted, the Muskinghum-Cunge routing method was selected for the river reaches. The Manning's roughness coefficient 'n' is the only reach routing parameter that can be estimated for this method selection. Since accurate geometry could not be gathered from the DEM surface, the channel geometry used in HEC-HMS was the standard trapezoidal cross section configuration. The trapezoidal width and side slope were based on an average value observed from the FIS cross sections located within that channel reach.

As an alternative, channel flow was modeled using the Muskinghum method. This method assumes a linear relationship exists between the volume of water stored in a reach, the upstream flow rate, and the downstream flow rate. Three parameters can be estimated using this reach routing method: Muskinghum K; Muskinghum X; and number of steps (subreaches). Using the drainage area to each reach and the regional hydraulic geometry curves, the channel flow area and discharge were generated. These values were used to generate the channel velocity and then the number of subreaches (reach length/velocity times the time step) and the Muskinghum K value (channel length/velocity times 3600) (Wanielista 1997). This routing alternative was eventually discontinued since the optimization trials did not significantly alter the estimated values. Once it was determined to use Muskinghum-Cunge, the Manning's n parameter be would be the only parameter estimated for the reach routing.

The initial optimization trials had a common occurrence of simulating high peak flow. The trials displayed on Figure 23 differed by 9.8 to 38.7% between the simulated and observed value. The trial setups differed by selecting the May 2006 (blue) or April 2007 (green) hydrographs to match. At each progression, estimated parameters, such as SCS Lag, from a previous trial were applied and then optimized again for the same or different parameter (SCS Lag, n, CN).

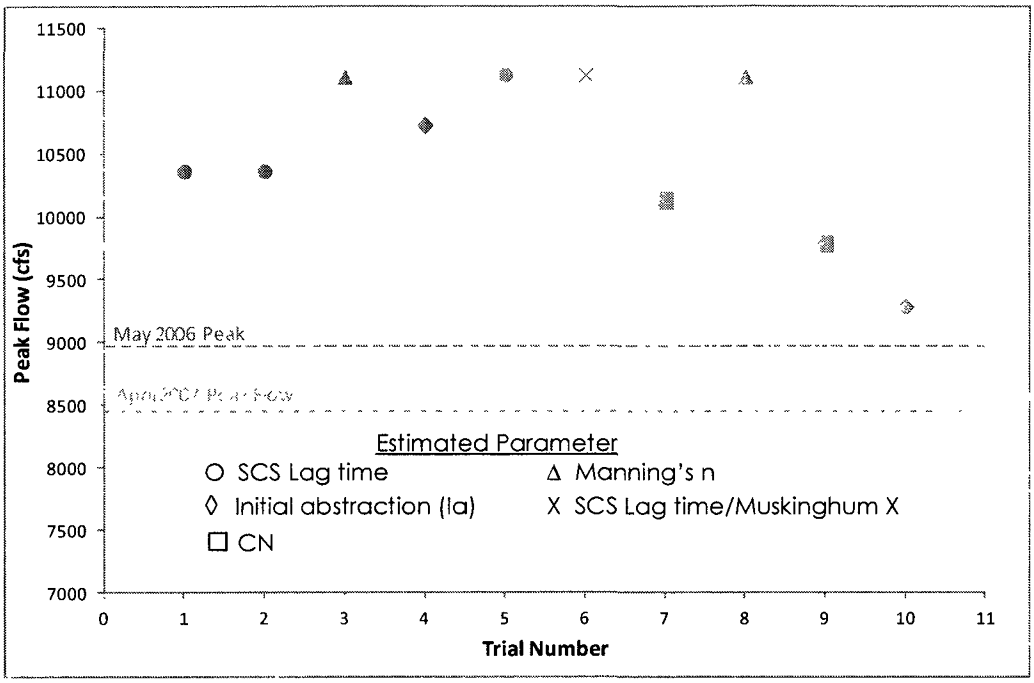

Figure 23: Simulated peak flows from optimization trials

The simulated and observed hydrographs for trial number seven is shown on Figure 24.

The objective function start and end labeled on Figure 24 is the defined time frame for a goodness-of-fit between the observed direct runoff and simulated hydrograph of the parameter being optimized. The start and end times were established at the time of lowest baseflow to the time where end discharge nearly equals baseflow again. Differences at the start and end are due to the use of direct runoff for the observed hydrograph. In removing baseflow, the transition isn't as smooth as the simulated hydrograph in the leading and ending limbs.

The objective function selected for calibration was the peak-weighted root mean square error (RMSE).

$$Z = \left\{\frac{1}{NQ}\left[\sum_{i=1}^{NQ}\left(q_o(i) - q_s(i)\right)^2 \left(\frac{q_o(i)+q_o(mean)}{2q_o(mean)}\right)\right]\right\}^{\frac{1}{2}} \quad (3)$$

Where: Z = objective function

i = index varying from 1 to NQ

NQ = number of computed hydrograph ordinates

$q_o(i)$ = the ith ordinate of the observed hydrograph

$q_o(mean)$ = mean of ordinates of observed hydrograph

$q_s(i)$ = the ith ordinate of the simulated hydrograph

The RMSE indicates how close the observed data points are to the models predicted values. Lower values indicate a better fit.

An examination of the simulated hydrograph and associated file data showed that the lag time for sub basin W7060 (Figure 10) had been considerably decreased from the initial value and was falling outside the rising limb of the watershed hydrograph. To override the optimization, the lag time for this sub basin was adjusted to fit within the rising limb of the watershed hydrograph and used in the successive trials for other parameter estimations.

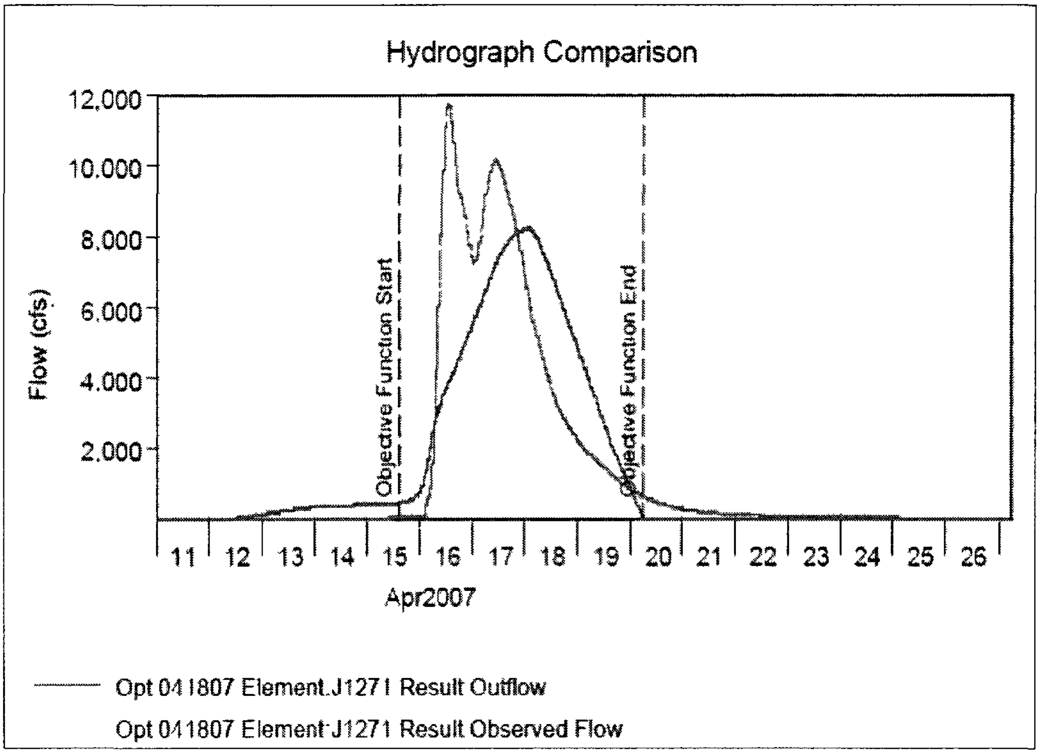

Figure 24: First simulated hydrograph

Similar to the Muskinghum values, the optimization trials were not adjusting the Manning's n value significantly. The initial channel roughness coefficients were the values entered in the FIS WSP2 model. Along the entire reach the values ranged from 0.03 to 0.10. As previously stated, it was obvious that flood flow attenuation was occurring along the flow path. During several field investigations in the upper reaches through Raymond and Epping, it was notable that the channel was blocked with woody debris from downed trees and deep pools impeding flood flow. For these known segments, the Manning's n value was increased to 0.12. The presence of log jams and their size, shape, number and distribution tends to increase the value of n (Chow 1959).

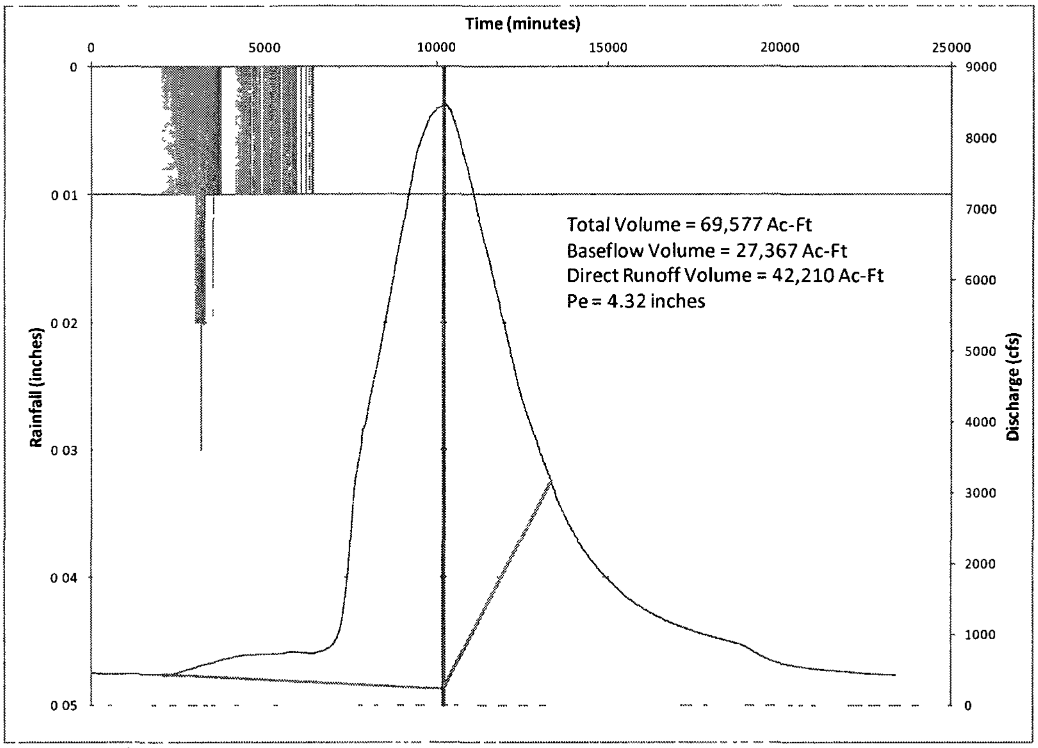

Figure 25: April 2007 direct runoff hydrograph

Using this adjusted Manning's n value and the direct runoff hydrograph for the April 2007 event developed with the concave-discharge baseflow separation (Figure 25), optimization trials for a similar selection of parameters was estimated to simulate observed flows. The comparison between simulated and observed runoff differed by 0.5 to 2.7%. Percent peak flow differences ranged from 1.3 to over 300% (Figure 26).

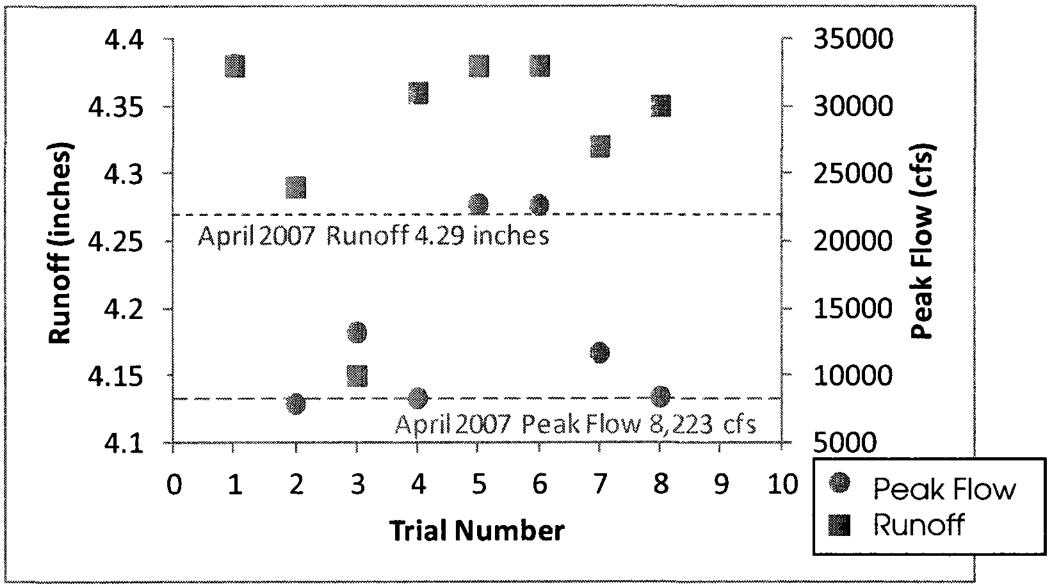

Figure 26: Simulated runoff and peak flows from optimization trials

3.2.7 *Curve Number (CN) and Land Use Analysis*

(a) Conventional development

The CN value represents the loss rate parameter for the hydrologic model. The sub basins in the watershed consist of several different land uses and respective hydrological soil group (HSG). The area of each sub basin land use and respective HSG are weighted to establish a composite CN for the sub basin. A case study performed by Knebl, Yang et al. (Knebl, Yang et al. 2005) on the San Antonio River used the composite CN as it is the one technique that enables spatially distributed infiltration calculations.

With this type of rainfall loss modeling, the precipitation excess is estimated by the following equation:

$$P_e = \frac{(P-I_a)^2}{P-I_a+S} \quad (4)$$

Where: P_e = accumulated precipitation excess at time t

P = accumulated rainfall depth at time t

I_a = the initial abstraction (initial losses)

S = potential maximum retention, storage

The maximum retention, S, is related to the CN watershed characteristic by:

$$S = \frac{1000 - 10\,CN}{CN} \quad \textbf{(5)}$$

This is the form of the equation in the foot-pound system. The sub basins in this project consisted of several soil types and land uses, a composite CN for the sub basins was calculated with:

$$CN_{composite} = \frac{\sum A_i CN_i}{\sum A_i} \quad \textbf{(6)}$$

Where: $CN_{composite}$ = the composite CN used for runoff volume computations

$_i$ = an index of watersheds subdivisions of uniform land use and soil type

CN_i = the CN for the subdivision i

A_i = the drainage area of subdivision i

The runoff CN for commercial, business, industrial and residential land use have been applied an average percent impervious cover (IC) (Table 8). Figure 27 demonstrates how the effect of impervious cover increases the CN value for the four HSGs defined in Section 3.2.2.

Table 8: Runoff Curve numbers for conventional development from TR-55 (NRCS 1986)

Land Use	% IC	HSG-A	HSG-B	HSG-C	HSG-D
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential 1/8 acre	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
5 acres*	7%	43	62	75	80
10 acres*	3%	40	59	73	79
25 acres*	1.3%	39	57	71	78
50 acres*	0.7%	38	56	71	77
Undeveloped	0%	38	55	70	77

*Values for 3 acres density on up are calculated by a best fit trendline using a high order polynomial with curve number as the dependent variable, impervious cover as the independent variable, and the intercept equivalent to redevelopment conditions (Appendix)

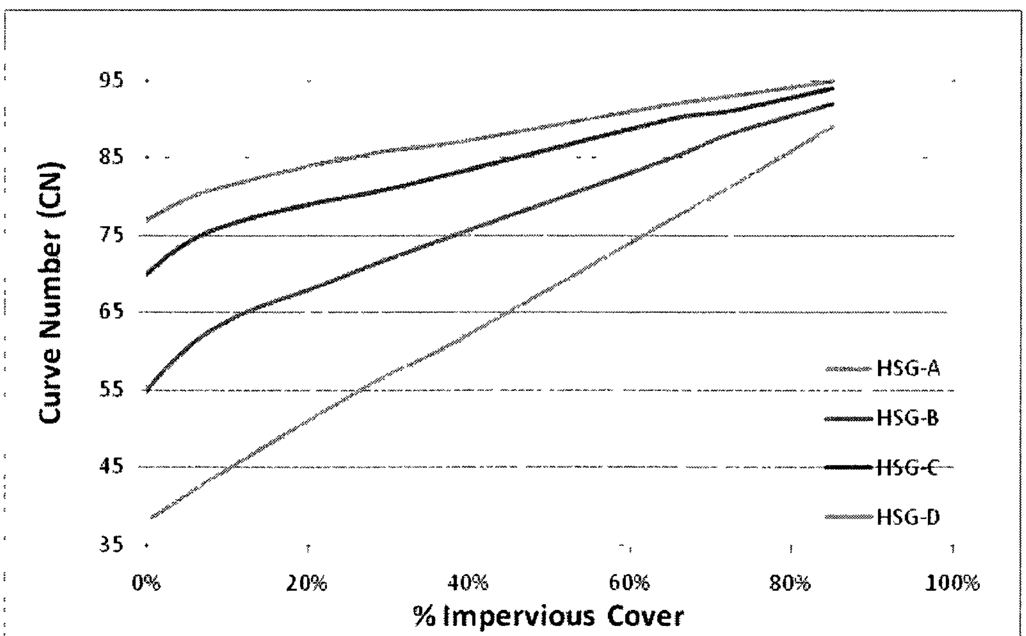

Figure 27: Coventional runoff curve number vs. impervious cover based on NRCS (1986) Runoff Method

GRANIT's GIS 2005 land use dataset (Appendix C) has detailed land use data from high resolution, remotely sensed data sources. The GRANIT analysis includes 58 land use categories. The land use category for residential built up land did not extend to a level of identifying lot sizes as does the cover type identified as residential districts by lot size in TR-55. As a separate task completed by others[7], the multi-residential communities (Zoning) of Exeter (R1, RU), Newfields (RA), Raymond (B, C2), Newmarket (M2, M3, M4, R1, R2, R3, R4), and Durham (R, RB) were evaluated to determine the average percentage of impervious cover for the residential zoned areas. The residential land use code of 113 was extended to a fourth digit, based on the cover percentage (Table 9).

Table 9: Land use code assignment in residential multi-zoned community

NRCS Table 2-2a	GRANIT Land Use Code 113		
Average Lot Size	% Cover <	% Cover >	Extended LU code
1/8 acre or less	65	38	1136
1/4 acre	38	30	1135
1/3 acre	30	25	1134
1/2 acre	25	20	1133
1 acre	20	12	1132
2 acre	12	0	1131

The land use and HSG features are merged into one data set so that a query can identify both features. In instances of dually assigned HSGs, C/D for instance, type D was assigned to indicate low infiltration and high runoff potential. Unassigned HSGs included rock outcrops, urban land, water, and Udorthents which is where the native soils have been removed. These HSGs were assigned D, C, D, and C respectively.

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[7] Fay Rubin, GRANIT Project Director

Using the associated and extended land use categories (GRANIT 2008) and the NRCS runoff curve number Tables 2-2a through 2-2d (NRCS 1986), a CNLookUp table is created (USACE 2009). For the 2005 land use conditions and future build-out scenario (conventional developed and redeveloped land) used the assigned value of CN noted in Table 10. Merwade's demonstration of creating a CN grid in GIS can be used to determine the composite curve number for any sized sub basin within the project watershed (Merwade 2009).

Table 10: Conventional CNLookup Table Correlating land use (LU) Code and Curve Number (CN)

LUValue	A	B	C	D	LUValue	A	B	C	D
112	46	65	77	82	151	81	88	91	93
113	46	65	77	82	152	81	88	91	93
1131	46	65	77	82	153	89	92	94	95
1132	51	68	79	84	158	81	88	91	93
1135	57	72	81	86	159	89	92	94	95
1136	61	75	83	87	161	89	92	94	95
114	51	68	79	84	169	89	92	94	95
115	54	70	80	85	171	39	61	74	80
119	46	65	77	82	172	39	61	74	80
121	89	92	94	95	173	39	61	74	80
122	89	92	94	95	174	39	61	74	80
123	89	92	94	95	178	39	61	74	80
124	89	92	94	95	180	39	61	74	80
125	89	92	94	95	200	39	61	74	80
126	89	92	94	95	290	59	74	82	86
127	89	92	94	95	300	30	48	65	73
128	89	92	94	95	400	30	55	70	77
129	89	92	94	95	500	98	98	98	98
130	81	88	91	93	600	98	98	98	98
137	81	88	91	93	720	63	77	85	88
142	76	85	89	91	730	63	77	85	88
144	98	98	98	98	740	98	98	98	98
1445	98	98	98	98	750	98	98	98	98
1446	98	98	98	98	760	77	86	91	94
148	76	85	89	91	790	74	83	88	90

(b) Low Impact Development (LID)

The LID Curve Number Analysis was applied using a method developed by McCuen (McCuen 2004) and formalized in practice by the Maryland Department of the Environment (MDDES 2000). It is a volume based approach developed by storing the increased runoff depth on the developed site by implementing LID. The revised LID CN was determined by the following equation:

$$CN^* = \frac{200}{\left[(P+2Q+2)-\sqrt{5PQ+4Q^2}\right]} \qquad (7)$$

Where: CN^* = the reduced CN used to reflect runoff volume stored by the infiltration practices

P = the design rainfall depth in inches

Q = the after development runoff depth minus the runoff depth retained by the infiltration practice (ΔQ) in inches

Figure 28 demonstrates how the effect of implementing LID lowers the CN value for the four soil type groups as impervious cover increases. The analysis showed that LID practices, by this methodology, began to show no effect residential density greater than two acre lot size. It also showed that the greatest benefit, in terms of CN reduction, is obtained for poor quality soils in high density development (impervious cover greater than 70%).

Because there are a limitless variety of applications of LID systems in a design context, the CN analysis performed here is based on providing a 1" water quality volume (WQV) for all impervious surfaces. CN values would be adjusted for less or more WQV designs. For the CN analysis, the practice type (i.e. bioretention, sandfilter, infiltration trench, etc) is unimportant, but rather the volume reduction.

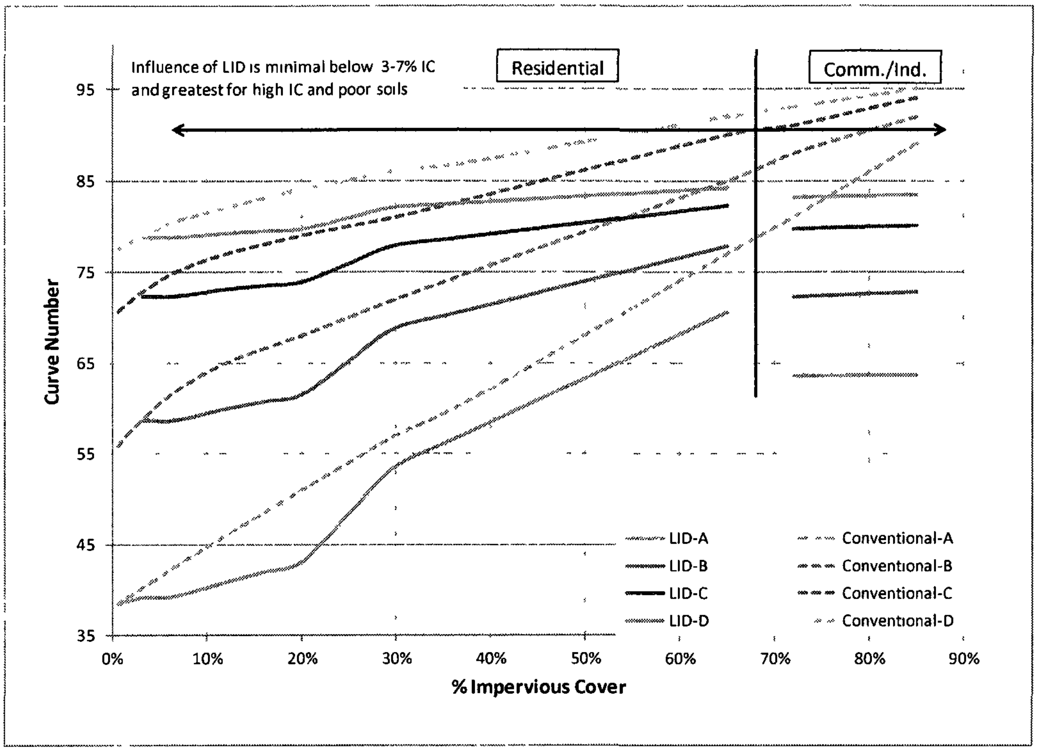

Figure 28: Comparison of Runoff Curve Numbers for conventional and LID 1" WQV vs. impervious cover for Hydrologic Soil Groups A, B, C, D

This analysis applied the use of LID for all developed and redeveloped sites. One (1) acre lot sizes and above incorporated the use of porous pavement which adds substantial additional volume reduction. Commercial and industrial site designs included parking (porous asphalt) and roads (standard asphalt and bioretention), and rooftop infiltration. The common practice of limiting porous pavement usage to parking areas was applied. The figures used for determining the CN for an LID build-out condition are provided in Appendix D. Table 11 provides the LID adjustment CN applied using this method.

Table 11: LID adjusted Runoff Curve Numbers for a 1" WQV

Land Use	% IC	HSG-A	HSG-B	HSG-C	HSG-D
Commercial and business	85%	64	73	80	83
Industrial	72%	64	72	80	83
Residential 1/8 acre	65%	71	78	82	84
1/4 acre	38%	57	71	79	83
1/3 acre	30%	54	69	78	82
1/2 acre	25%	48	65	76	81
1 acre	20%	43	62	74	80
1.5 acres	16%	42	61	74	79
2 acres	12%	41	60	73	79
5 acres*	7%	39	59	72	79
10 acres*	3.3%	39	59	72	79
25 acres*	1.3%	39	59	72	79
50 acres*	0.7%	38	59	72	79
Undeveloped	0%	38	55	70	77

*Values for 3 acres density on up are calculated by a best fit trendline using a high order polynomial with curve number as the dependent variable, impervious cover as the independent variable, and the intercept equivalent to redevelopment conditions (Appendix)

For the future LID development and redevelopment the CNLookUp table values provided in Table 12 was applied. LUValue codes are defined in Appendix C.

Table 12: LID CNLookup Table Correlating land use (LU) Code and Curve Number (CN)

LUValue	A	B	C	D	LUValue	A	B	C	D
1100	41	60	73	79	1450	72	82	87	89
1101	41	60	73	79	1460	72	82	87	89
1102	43	62	74	80	1470	72	82	87	89
1105	57	71	79	83	1480	76	85	89	91
1106	71	78	82	84	1710	39	61	74	80
1120	46	65	77	82	1720	39	61	74	80
1130	46	65	77	82	1730	39	61	74	80
1131	46	65	77	82	1740	39	61	74	80
1132	51	68	79	84	1780	39	61	74	80
1135	61	75	83	87	1800	39	61	74	80
1136	77	85	90	92	2000	39	61	74	80
1140	51	68	79	84	2900	59	74	82	86
1150	54	70	80	85	3000	30	48	65	73
1190	46	65	77	82	5000	98	98	98	98
1200	64	73	80	83	6000	98	98	98	98
1299	64	73	80	83	7200	63	77	85	88
1420	76	85	89	91	7300	63	77	85	88
1441	98	98	98	98	7400	98	98	98	98
1442	98	98	98	98	7500	98	98	98	98
1446	98	98	98	98	7600	77	86	91	94
1447	98	98	98	98	7900	74	83	88	90
1449	98	98	98	98					

3.2.8 *Hydrologic Flood Flows*

With the final optimized runoff parameters, hydrologic analyses were performed to calculate peak flow discharges for the various scenarios.

The historic model used the TP-40 24-hour, 100-year design storm depth of 6.3 inches. The current and future models will use NRCC 24-hour, 100-year design storm depth of 8.5 inches. Table 13 provides the project model scenarios.

Table 13: Project scenarios evaluated for various modeling conditions

Land Use Conditions	Rainfall Depths and Global Change Model Scenario		
	Rainfall Atlas		Climate Period 2035-2069
	TP-40	NRCC	Regional Climate Model (RCM)
	6.3 in.	8.5 in.	8.5 in.
2005 Current	X	X	
2050 Build-out			X
2050 LID/Build-out			X
Flood Insurance Study[1]			

NRCC – Northeast Regional Climate Center (based on records from 1938 – 2010)
TP-40 – Technical Paper 40 (based on records from 1938 – 1958)
RCM - http://climateprediction.net/content/regional climate-models
[1]100-year peak discharges established for Flood Insurance Study

3.3 Hydraulic Modeling

In this research, steady flow was simulated along the 36 mile reach of the Lamprey River and floodplain elevations and extents were developed using Hydrologic Engineering Center River Analysis System (HEC-RAS)(USACE 2001) and Geographic River Analysis System (HEC-GeoRAS)(USACE 1999).

3.3.1 *Historical Model*

(a) Flood Insurance Study (FIS) Hydraulic Analysis

The FIS was performed with a computer program called Water Surface Profile 2 (WSP2) that was developed in the early 1970's (Merkel H. 1992). WSP2 was used by the former SCS and others in floodplain management studies. This program computed water surface profiles (Figure 29) and estimated head loss at restricted sections such as bridges and culverts using a ratio of conveyances (USDA 1993). Several computer program upgrades have been developed and

WSP2 is now been removed from FEMA's list of acceptable modeling programs. This analysis used HEC-RAS which is an approved software program (FEMA 2009).

The basic energy balance equation between successive cross sections used in WSP2 is:

$$z_2 + d_2 + \propto_2 V_2^2 / 2g = z_1 + d_1 + \propto_1 V_1^2 / 2g + \textit{Energy losses} \quad \textbf{(8)}$$

Where: z = elevation or datum of channel bottom (L)

d = depth of water at the cross section (L)

V = average velocity at the cross section (L/T)

g = gravitational acceleration (L/T^2)

$\propto$ = velocity head correction factor

Subscript 1 and 2 refer to the downstream and upstream cross sections respectively. Energy losses equal the sum of friction loss, expansion, and contraction losses.

The program repeats until the energy equation is solved. This is when the up and downstream energy elevation is within the tolerance of 0.1 foot (USDA 1993).

In comparison to the software used to generate FIS studies, there is much improvement in the accuracy and visualization for representing flood flows on land surfaces (Yang, Townsend et al. 2006). There is a lot of flexibility to create geometric data for use in hydraulic modeling giving engineers a cost-effective approach to sizeable watersheds. Several modelers (Solaimani 2009) use the HEC-GeoRAS extension for interpolation of the digital terrain because of its advantage to generate a visualization of flooding.

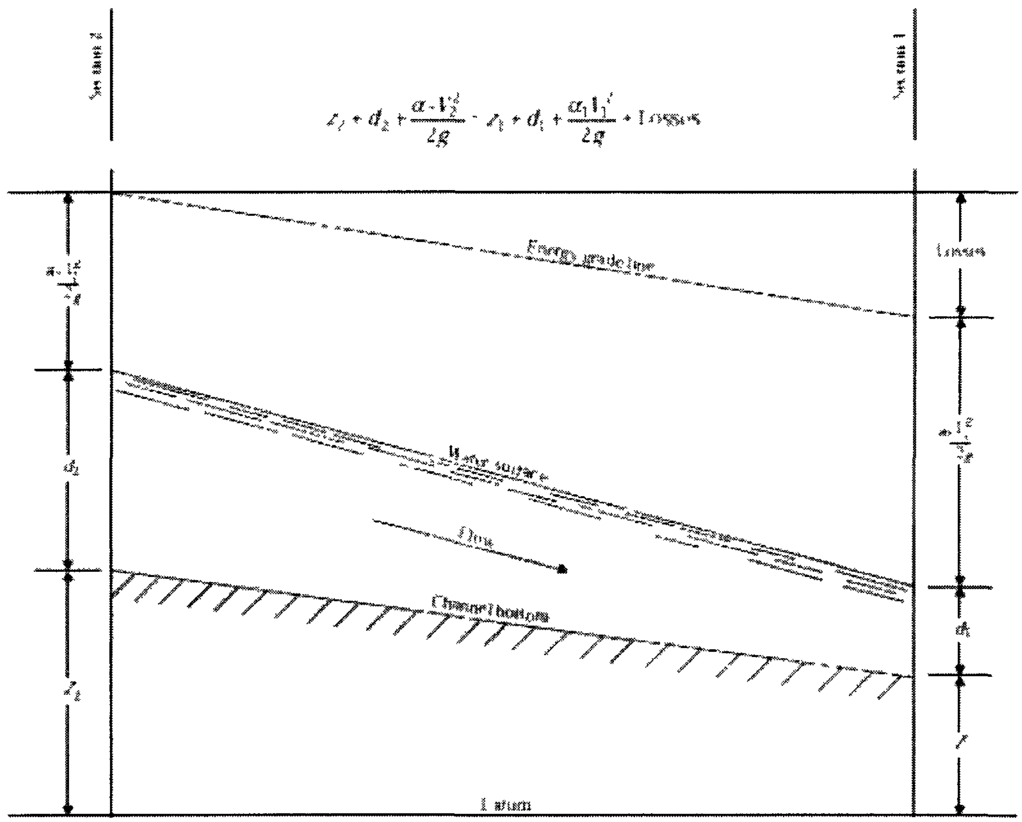

Figure 29: Energy Balance Profile (Chow 1959)

(b) FEMA Backup Data

Because the FIS is the foundation of the model, it necessitated acquisition of existing data. A FIS data request form for all records on the FIS was applied for from the FEMA project library.

Information on the WSP2 card printouts provided the placement (ordering) of cross sections, cross section station and elevation data, loss coefficients, roadway deck elevations, and bridge/culvert opening dimensions. Card data was complete for Strafford County, but unfortunately all structural data for Rockingham County was absent.

Duplicating the FIS was not practicable. The inclusion of the reach through Lee, in this improved model, provided a more accurate boundary condition for the remaining analysis through Rockingham County. The changes in hydraulic software used for the FIS has considerable differences in modeling flows at bridges and culverts that cannot be duplicated with HEC-RAS. The FIS modeled bypass flows to the Oyster River using iterative hydraulic analyses. Final values resulted when the downstream flow of the Lamprey River, plus the diverted flow to the Oyster River equaled the upstream inflow to the watershed divide. No drainage area hydrology was computed from the watersheds in the bypass region.

3.3.2 ***Geospatial Hydraulics in ArcMap***

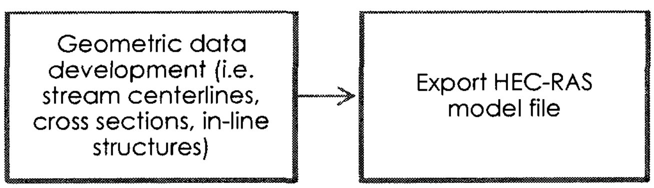

Figure 30: HEC-GeoRAS overview

HEC-GeoRAS is a GIS extension tool that can be used with the DEM. The file created in HEC-GeoRAS provided the georeferenced stream network and stationed cross section locations. With these tools, a HEC-RAS input file was created containing the river channel, tributaries, junctions, cross section stations and geometric data.

(a) Geometric Data Development

The DEM conditioned during the hydrologic processing is used again for the hydraulic data development.

The required RAS layers created in GIS include the stream centerline and the cross section cut lines. The hydraulic structures such as bridges and dams were also created in GIS to keep them in alignment with the other geometric data.

(b) RAS Layers

Stream Centerline – This represents the river and reach network and is displayed as the schematic in the HEC-RAS geometric editor. The New Hampshire GIS hydrography stream network vectors used to define the hydraulic model included: Lamprey River, Piscassic River, Beaudette Brook, Bedford Brook, Ellison Brook, Hamil Brook, LaRoche Brook, and Longmarsh Brook. All of these represent one continuous flow path but some consisted of more than one reach. The reaches were connected with junctions defining the intersection of two or more upstream or downstream endpoints. The stream centerline topographic characteristics are completed with HEC-GeoHMS menu tools that populate the length, slope, and stationing. Figure 31 is a schematic in the vicinity of the Lamprey and Oyster River watershed divide. These water courses were included in the hydraulic analysis in order to improve the bypass modeling to the Oyster River watershed.

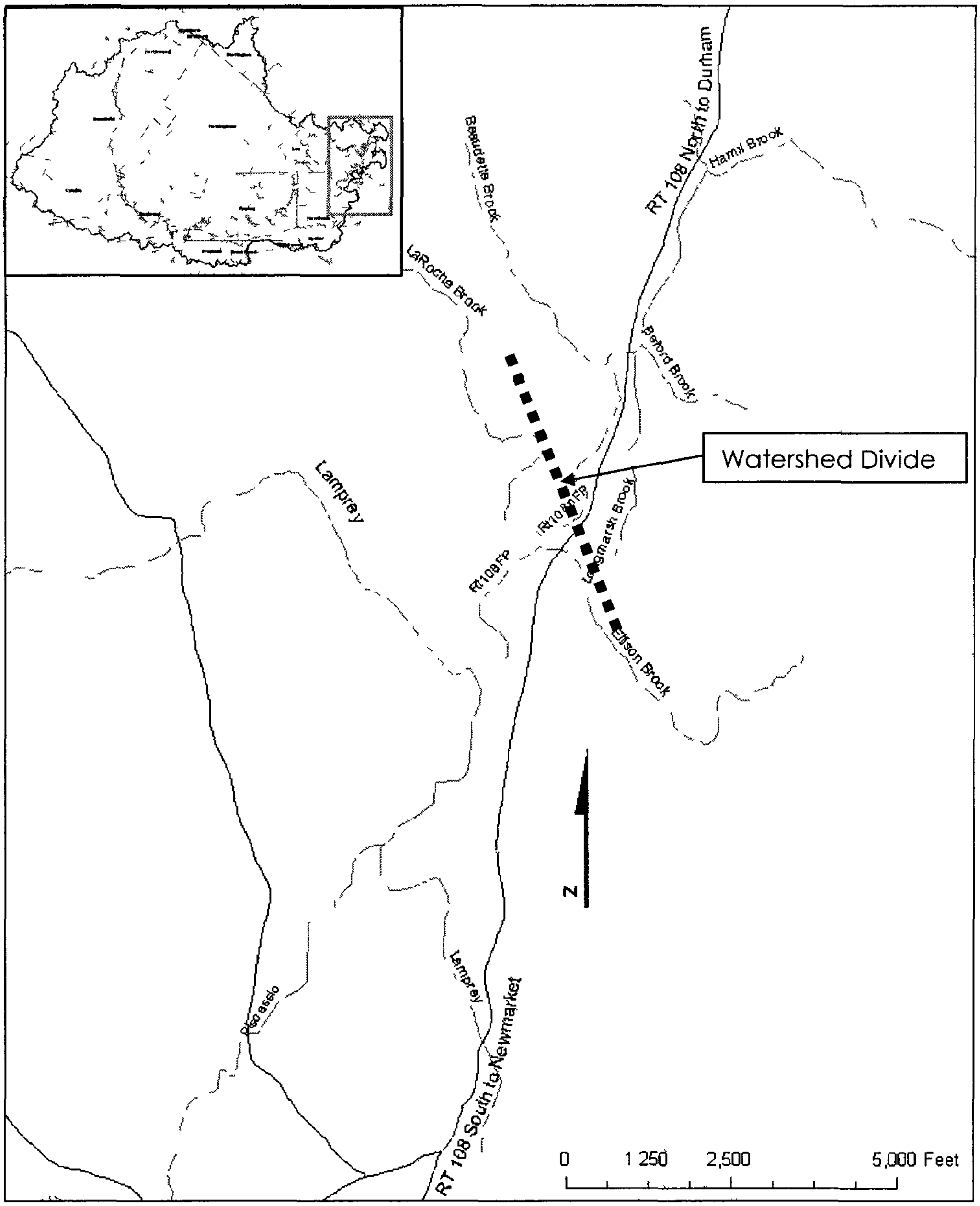

Figure 31: Stream network in bypass area along the RT108 corridor

Cross Sectional Cut Lines – This represents the location, position, and extent of cross sections. The 2D cross section vector lines consisted of the FIS lettered sections in Strafford and Rockingham Counties and new sections at the stream junctions and along the tributaries. The station-elevation data are extracted from the DEM along these cut lines. The cross section attributes are completed with menu tools that populate the associated river and reach, river station based on the intersection with the stream centerline, and downstream reach lengths.

Bridges/Culverts/Dams – These represent the structure locations and are treated much the same as for cross sections. The cut lines were used to identify the correct river station for the inline structure. Other attributes were applied in the HEC-RAS program.

There are several optional layers that can be created in GIS for the RAS import file. However the channel banks, ineffective areas, and Manning's n, were generated in the HEC-RAS program. HEC-GeoRAS manages the data layers used for extracting the attribute information needed in the RAS GIS import file.

(c) HEC-RAS Model File

Importing the data generated from the GIS layers does not create a complete HEC-RAS river hydraulic model. HEC-RAS reads the geometry from a text file that includes the river network, cross sections, bridge/culvert, and inline structures. In HEC-RAS each cross section requires review for assigning the stations where changes in roughness coefficient (Manning's n) occur and the left and right bank reach lengths. The bridge/culvert and inline structures all required editing to reflect WSP2 card data, as-built plans, or other analysis file data.

3.3.3 ***HEC-RAS Model Components***

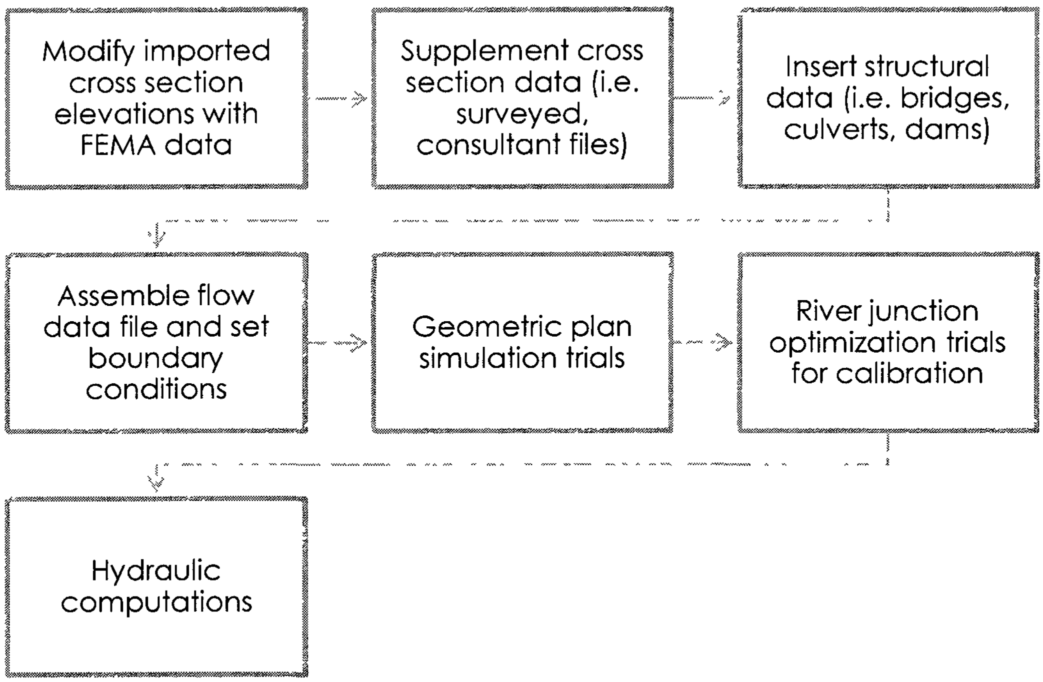

Figure 32: HEC-RAS overview

(a) Geometic Data

The imported FIS cross section station and elevations were modified with the FEMA back up data. The WSP2 program consisted of data entry cards identified with a control word and then several fields of data. Table 14 includes a list of all the types of cards used in the FIS model and deciphered for input into HEC-RAS.

Table 14: WSP2 Input data (USDA 1993)

Control Word	Data Field				
	11-20	21-30	31-40	51-60	61-70
TITLE					
DISCHARGE	Total D.A	CSM	CSM		
STARTE	XSEC Name	Starting Elevation for 1st listed CSM	Starting Elevation for 2nd listed CSM		
OUTPUT	10 Options	S-Segment Table (conveyance, discharge, velocity) K-Conveyance Table (Top width and conveyance and segment conveyance)			
TRIB	The xsec names where data are to be held for use as starting data on later profiles				
REACH	XSEC Name	D.A. (sq.mi)	Hydraulic channel length to next DS section	Hydraulic main flood plain length to next DS section	
REACH2	Transposed XSEC	Elevation displacement			
ROAD	XSEC Name	Weir Coef.	Reach lengths		
SECTION	XSEC Name	Height Instr.	TR-20 Rating	Left Encr.	Right Encr.
	X, Y, data records to describe shape of section				
ENDTABLE	Indicates end of section table				
SEGMENT	XSEC Name	No. of segments 1-6	Type C, D, N	Last station marks end	Last elevation
NVALUE	'n' value				

Table 14: WSP2 Input data (USDA 1993) cont'd

BPR –data for computing bridges	XSEC Name	Skew Type (A or B) Fig.31-2	Base Curve (1-3) Fig.31-3	Pier Curve (1-8) Fig.31-4	
GIRDER – items pertaining to opening	Elev Full Elevation where orifice flow begins	Elev Grdr Bot elev where girders reduce channel flow	Skew angle angle of flow per'd to road	Orif Coef for orifice flow formula	Weir coef for flow over the deck
	X, Y, data records to describe shape of bridge girder				
CULV1	XSEC Name	No. of Pipe	Culv. Code from table page 31A-26		
CULV2 – continuation of CULV1	Dia. or Height of circular, box or arch culvert	Width of box or piper arch (blank for cir.)	Total length of pipe	Upstream invert elev.	Downstream Invert elev.

Information on the WSP2 card printouts provided the placement (ordering) of cross sections, cross section station and elevation data, loss coefficients, roadway deck elevations, and bridge/culvert opening dimensions. Card data was complete for Strafford County, but unfortunately all structural data for Rockingham County was absent. Examples of the WSP2 records are provided in Appendix E.

The FEMA library data provided station and elevation records for over 100 cross sections. These records were used to replace the geometric attributes of the georeferenced cross sections generated in GIS. The following steps were taken to perform this replacement:

1. Thalweg stations for the matching GIS and FIS section are compared to generate a difference.

2. This difference was added to each station of that individual FIS cross section record to get an equivalent thalweg station.
3. In HEC-RAS, the cross section coordinates for the GIS section were replaced with the FIS records as the DEM generated elevations were crude compared to the FIS. Field validation of elevations was not included in this research.
4. The stations were adjusted by adding the previously determined difference.
5. After applying the changes, the locations were all georeferenced.

In order to build a complete hydraulic model, additional sources beyond the FEMA data was needed. As previously noted, the community of Lee did not have a published FIS. The eight miles of the Lampey River through Lee and supplemental sections in Newmarket were brought into the model by surveying twelve river cross sections and duplicating them as needed to model this reach. During a review of aerial photography, the survey sites were selected wherever a significant change in conveyance occurred. These sites were field verified and flagged. Prior to additional field work, Assessor maps were reviewed to find property owner name and addresses for site access. Approximately 17 property owners were sent letters requesting access to the Lamprey River through their property. None of the contacted owners denied right of access. UNH Facilities provided personnel and a Trimble Real Time Kinematic (RTK) Global Positioning System (GPS) unit to identify the sites coordinates and set up a bench mark. The GPS is set to read horizontal coordinates based on New Hampshire State Plane NAD 1983 and the vertical datum is NAVD 1988. Once sited, the locations were brought into a map file set up in GIS keeping the location georeferenced with the remaining river. Several field days with a survey level and rod generated the cross section stations and elevation data. Since the vertical datum of the FIS hydraulic model is North American Vertical Datum of 1929 (NAVD29), the cross section elevations were adjusted by +0.722 feet from the recorded NAVD88 elevations.

Additional sections added to the model came from previous hydraulic analysis performed by private consultants[8], the New Hampshire Department of Transportation (NHDOT), and the New Hampshire Department of Environmental Services (NHDES). Recent bridge replacements, dam removal and assessment analyses provided river elevation data as well as in-line structural data. Completing the list (Table 15) are the sections developed in ArcMap. The GIS sections were predominantly located at the confluences of perennial and intermittent streams and their immediate reach to the Lamprey River. FEMA sections up and downstream of the bridges were duplicated and stationed closer to the structures in order to model the ineffective flow areas generated by the crossings. These cross section elevations were revised based on the distance and slope of the channel. The channel slope was determined by the difference between the up and downstream cross section thalweg elevations divided by the length between them.

[8] Data source listed in Table 16

Table 15: Summary of cross section data source

Source	Number
FEMA FIS Backup Data	111
FEMA Duplicates	19
Surveyed	12
NHDOT/Consultants	34
GIS	44
Total	220

The missing structural data for Rockingham County was graciously provided by the NHDOT, NHDES, and several private consulting firms.

Table 16 provides a list of the sources used for generating the structures in the HEC-RAS model. The vertical datum of any additional resource was verified and converted into NAVD29 as needed.

At each bridge crossing the ineffective flow areas were established by determining the contraction and expansion distances upstream and downstream of the bridge respectively. The upstream condition assumed a 1:1 contraction rate and the flow elevation set to the low point of the top-of-road. The downstream condition assumed a 2:1 expansion rate and the flow elevation set at the average elevation between the low chord and minimum top-of-road (USACE 2010).

A recent NHDOT survey of the RT108 corridor in Durham was used for elevation data for this highway section. This one mile stretch was entered into HEC-RAS as a bridge with multiple openings. There are seven crossings ranging from 12" diameter culverts to four (4) foot by five (5) foot box culverts. Setting up a combination of openings establishes blocked ineffective flow areas where no conveyance occurs until the water surface reaches an elevation to flow through the next lowest culvert. The seven openings were configured for the RT108 (Newmarket Road) corridor (Figure 33) and two openings for the RT87 (Hedding Road) bridge in Epping (Figure 34). The RT87 bridge had a Conspan® arch in the floodplain in addition to the single span bridge.

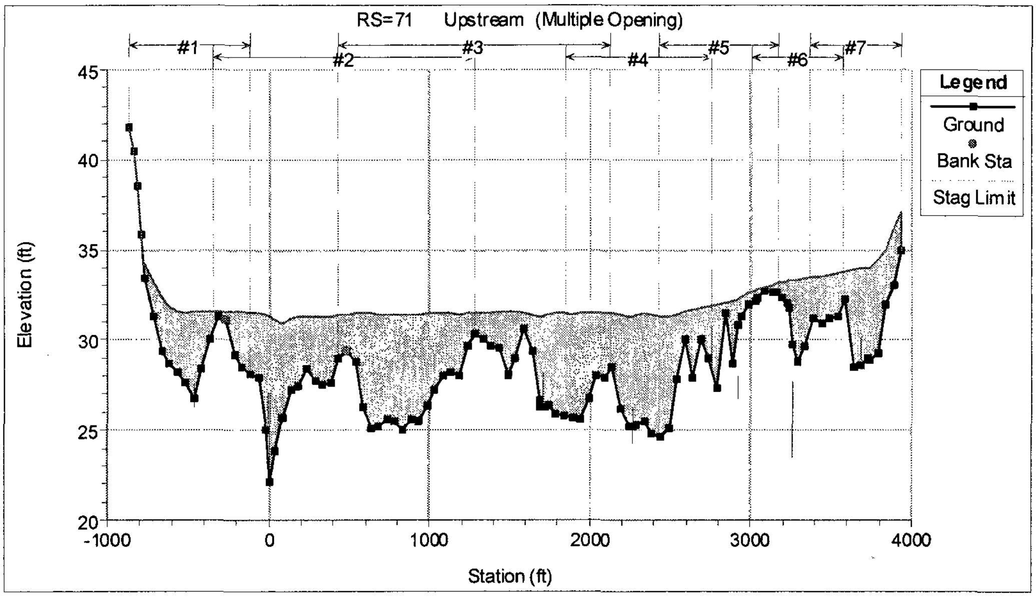

Figure 33: RT108 modeling for multiple openings

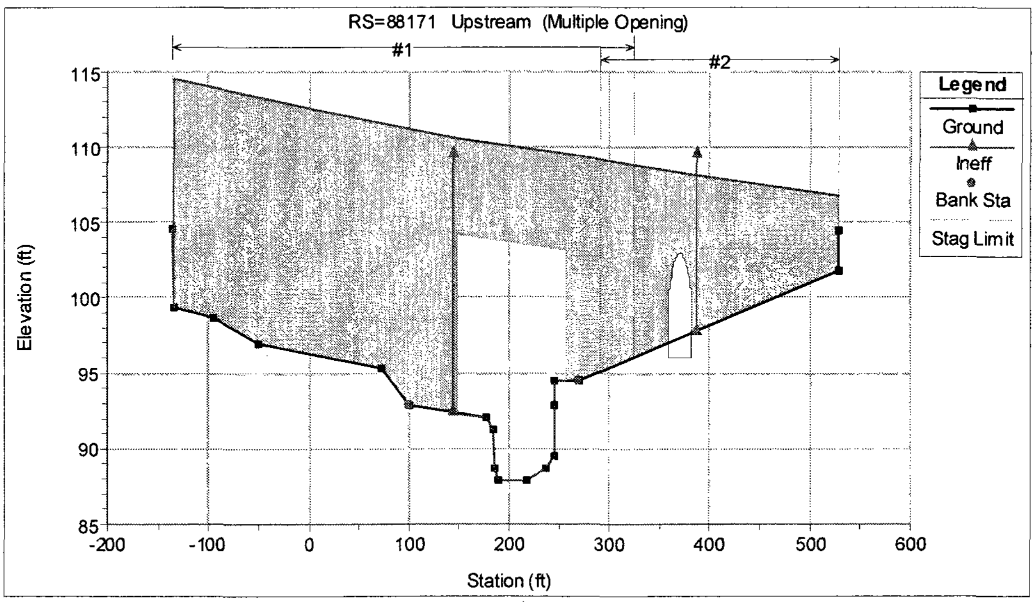

Figure 34: RT87 modeling for multiple openings

Table 16: Summary of bridge and in-line structure source for the Lamprey River

Community	Station	Road/Structure Name	Data Source
Raymond	181300	Dudley Road	Electronic WSP2 files from Roald Haestad, Inc.
	180964	Raymond Road (RT 27)	Electronic WSP2 files from Roald Haestad, Inc.
	167900	Langford Road	Electronic WSP2 files from Roald Haestad, Inc.
	160746	Main Street	Electronic WSP2 files from Roald Haestad, Inc.
	155060	Epping Street	Electronic WSP2 files from Roald Haestad, Inc.
	154106	B&M Railroad	Electronic WSP2 files from Roald Haestad, Inc.
	147643	Freetown Road (RT 107)	Electronic WSP2 files from Roald Haestad, Inc.
	141372	Prescott Road	Electronic WSP2 files from Roald Haestad, Inc.
Epping	136759	State Route 101	Electronic WSP2 files from Roald Haestad, Inc.
	127937	Epping Road (RT 27)	Electronic HEC-RAS files from NHDOT
	127265	Bunker Pond Dam	Electronic HEC-RAS files from NHDOT
	123964	Blake Road	1952 As-builts from NHDOT
	107459	Main Street (Plummer)	1936 As-builts from NHDOT
	106269	Mill Street	Electronic HEC-RAS files from NHDOT
	105560	Calef Hwy (RT 125)	Electronic HEC-RAS files from NHDOT
	88171	Hedding Road (RT 87)	WSPRO print out and As-built from NHDOT
Lee	61457	Wadleigh Falls Road	1933 As-builts from NHDOT
	61266	Wadleigh Falls Dam	Land Records
	35683	Lee Hook Road	1923 As-builts from NHDOT
Durham	20082	Wiswall Road	Electronic HEC-RAS files from CLD Consulting
	19859	Wiswall Dam	Electronic HEC-RAS files from CLD Consulting
	16028	Packer's Falls Road	FEMA FIS Backup Data
Newmarket	1602	RT 108	FEMA FIS Backup Data
	1286	Coffee Sluice	Electronic HEC-RAS files from Wright-Pierce
	1164	Macallen Dam	Electronic HEC-RAS files from Wright-Pierce

(b) Flow Data and Boundary Conditions

The FIS established starting water surface elevations (WSE) by computing critical depths at the Macallen Dam. The gates were assumed to be closed (FEMA 2005). The 100-year flood elevation was based upon high water elevation data for the April 1987 flood.

The downstream boundary condition for this project is Macallen Dam in Newmarket. In order to determine WSE for the current, predicted, and observed flows, a rating curve for the Macallen Dam was developed. The dam has a 70 ft long spillway (weir) at elevation 22.9, a second 38 ft long spillway at elevation 30.7, and three 7 ft x 7 ft gates with inverts at elevation 16.7 feet (Figure 35). For all the analysis, the gates are open completely as advised by the Town's public works director.

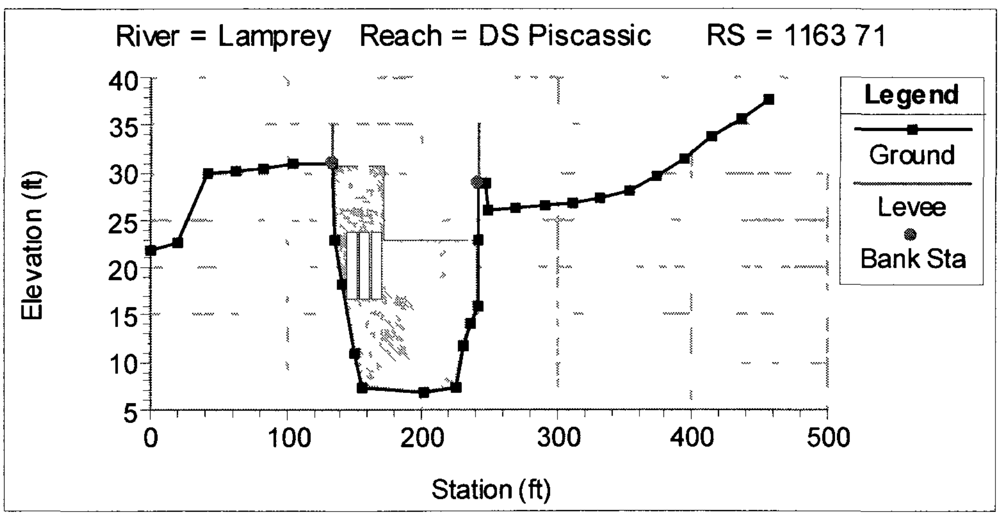

Figure 35: Macallen Dam Structure Data

A stage and discharge curve was calculated using the following orifice and weir discharge equations.

$$Q_{orifice} = CA(2gh)^{0.5} \quad \textbf{(9)}$$

Where: C = orifice discharge coefficient (0.6)

A = orifice area (L^2)

g = gravitational acceleration

h = depth of water above orifice (L)

$$Q_{weir} = C_w L H^{\frac{3}{2}} \quad \textbf{(10)}$$

Where: C_w = weir discharge coefficient (2.69-3.1)

L = effective crest length (L)

H = depth of water above crest (L)

The gates were modeled as weir discharge until they were submerged at elevation 23.7. The levees shown in Figure 35 do not exist in the field. They were used to confine the flood flows over the dam as is typically performed by the Town of Newmarket as means to keep flood waters from adjacent properties. Orifice and weir discharges were totaled to develop the associated WSE. The stage and discharge for these calculations and the information from FEMA backup data is shown on Figure 36. Upstream boundary conditions for the model were set at normal depth.

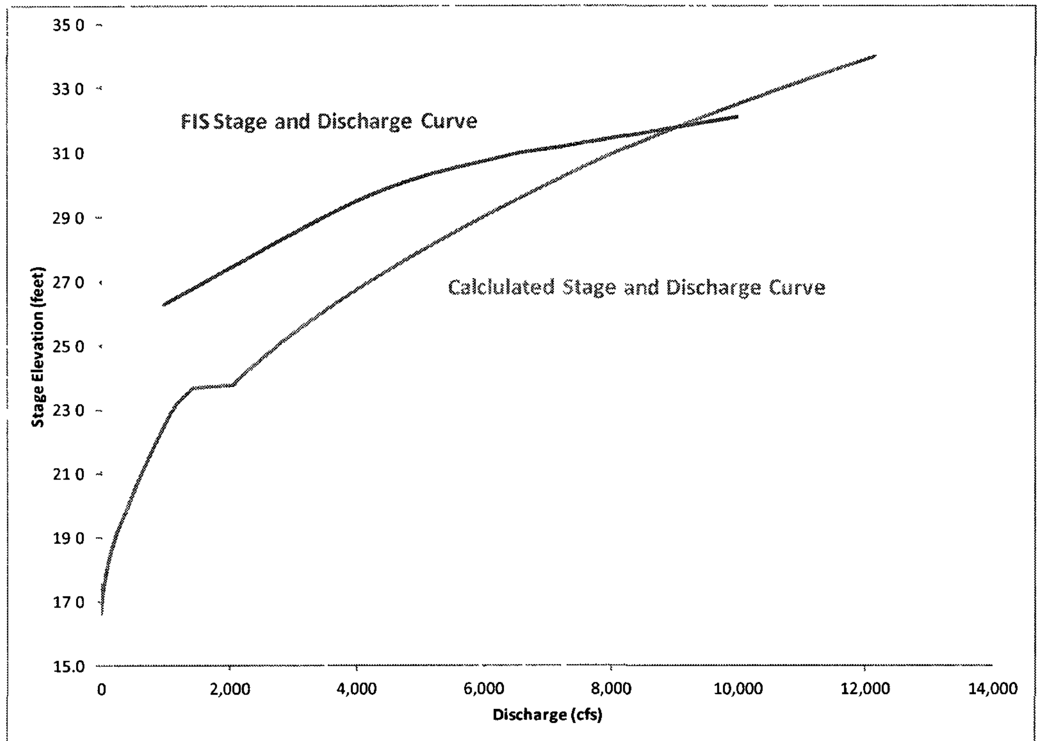

Figure 36: Modeled stage and discharge for Macallen Dam

(c) Modeled Channel Geometry for RT108

RT108 is a state highway between Newmarket and Durham. During major events, flows from the Lamprey River bypass under and over a one mile stretch of RT108 to Hamil Brook which is within the Oyster River watershed. The RT108 area was challenging and multiple channel geometries were modeled to examine the bypass.

The first geometry channel modeled for the RT108 crossing (Figure 37) divided the stretch of highway into two (2) bridge structures to mimic the tributaries that flow southwesterly toward the Lamprey floodplain and those that flow northeasterly toward Hamil Brook. Along with the two bridge sections, a lateral weir, based on elevations from the DEM, was generated along the divide of the two watersheds. The HEC-RAS analysis failed to converge after the allotted iterations. Water surface elevations (WSEs) over the bridge at the divided location and on either side of the lateral weir did not equal as should be expected.

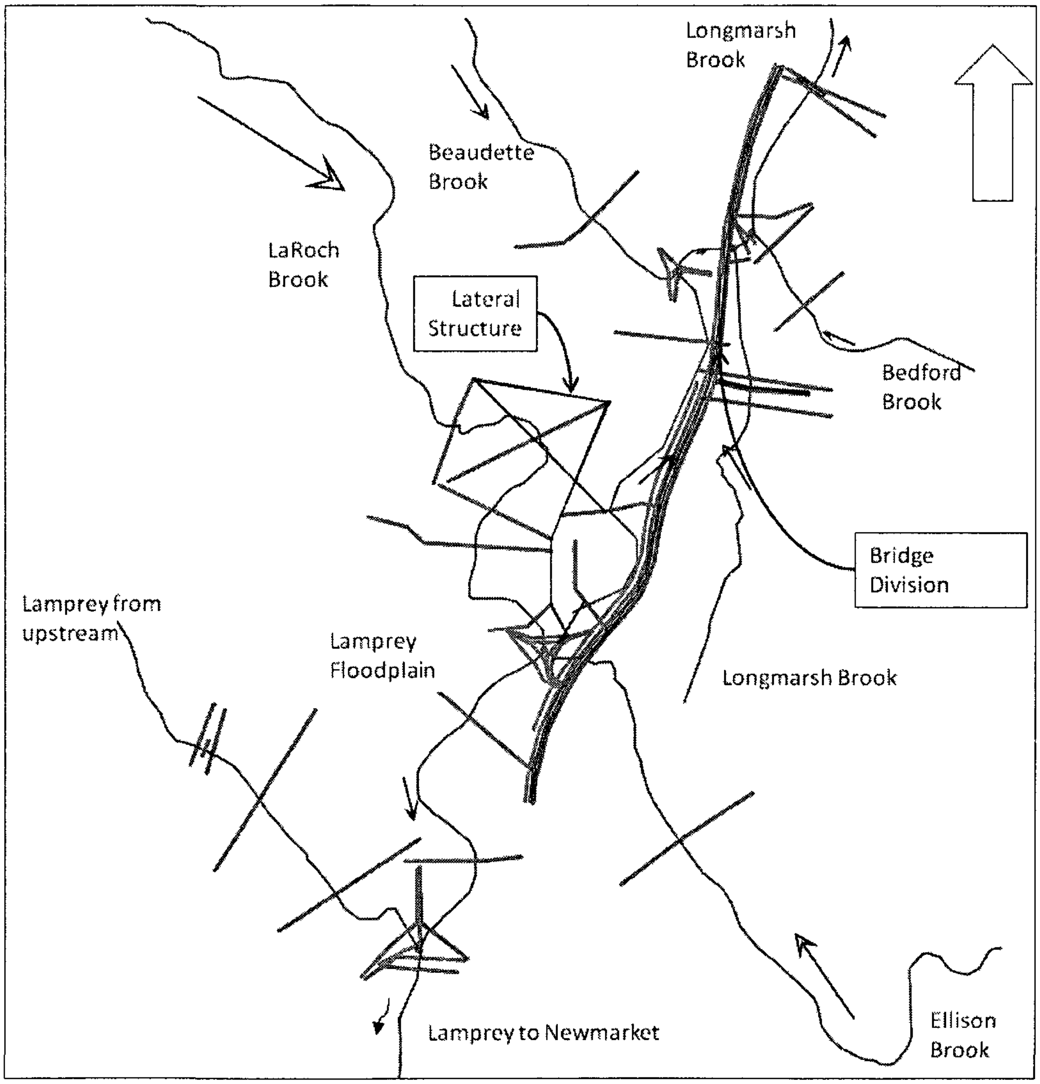

Figure 37: HEC-RAS V1 geometric data view of RT108 corridor with stream network (blue), cross sections (green) and structures (black)

The second geometry for RT108 (Figure 38) eliminated the divided bridge. It was speculated that during flood stage, the flow direction of the tributaries is a minor concern. Once again the analysis failed to converge with the lateral weir used as regulating the bypass flow.

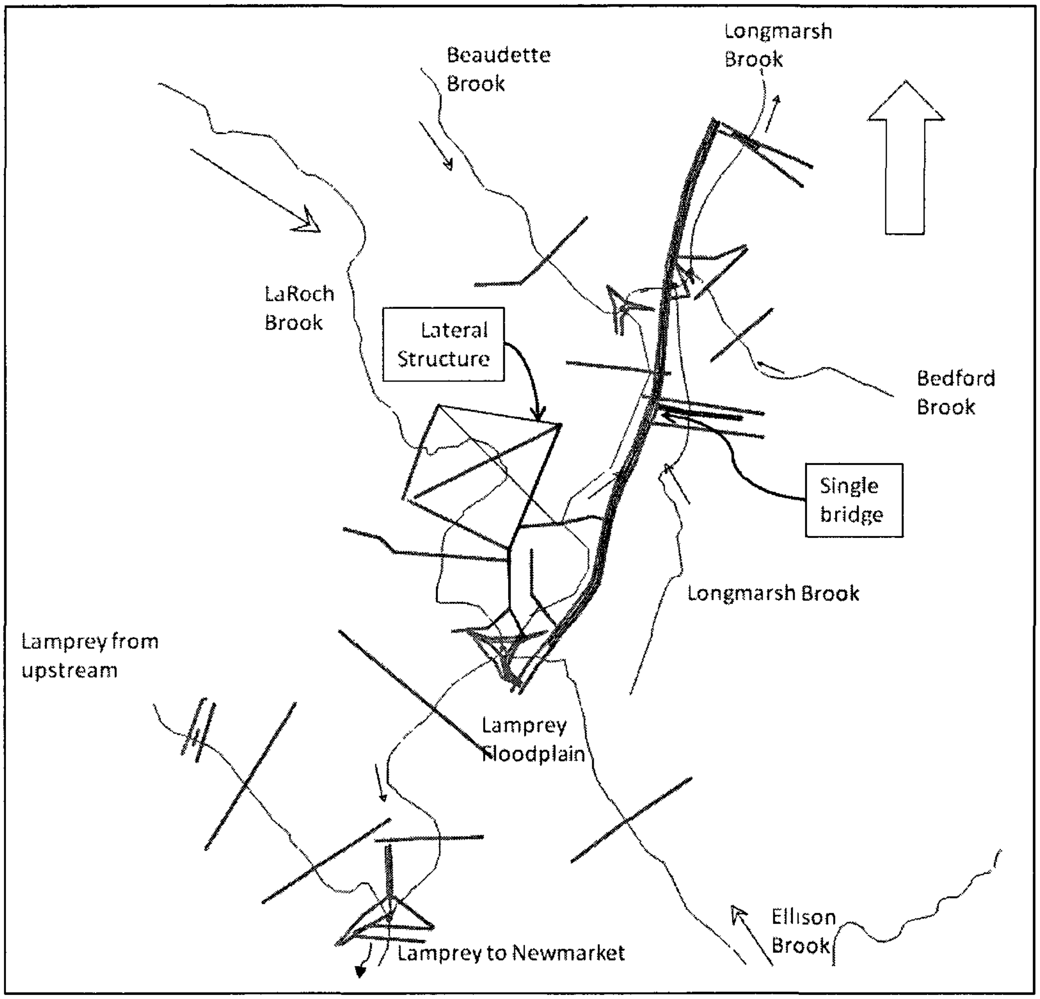

Figure 38: HEC-RAS V2 geometric data view of RT108 corridor with stream network (blue), cross sections (green) and structures (black)

The third geometry (Figure 39) reversed the flow direction of the perennial floodplain confluence with the Lamprey River, connected the perennial floodplain to the tributary of Hamil Brook, and eliminated the lateral weir. This established a split junction at the perennial floodplain confluence and the model convergence was successful.

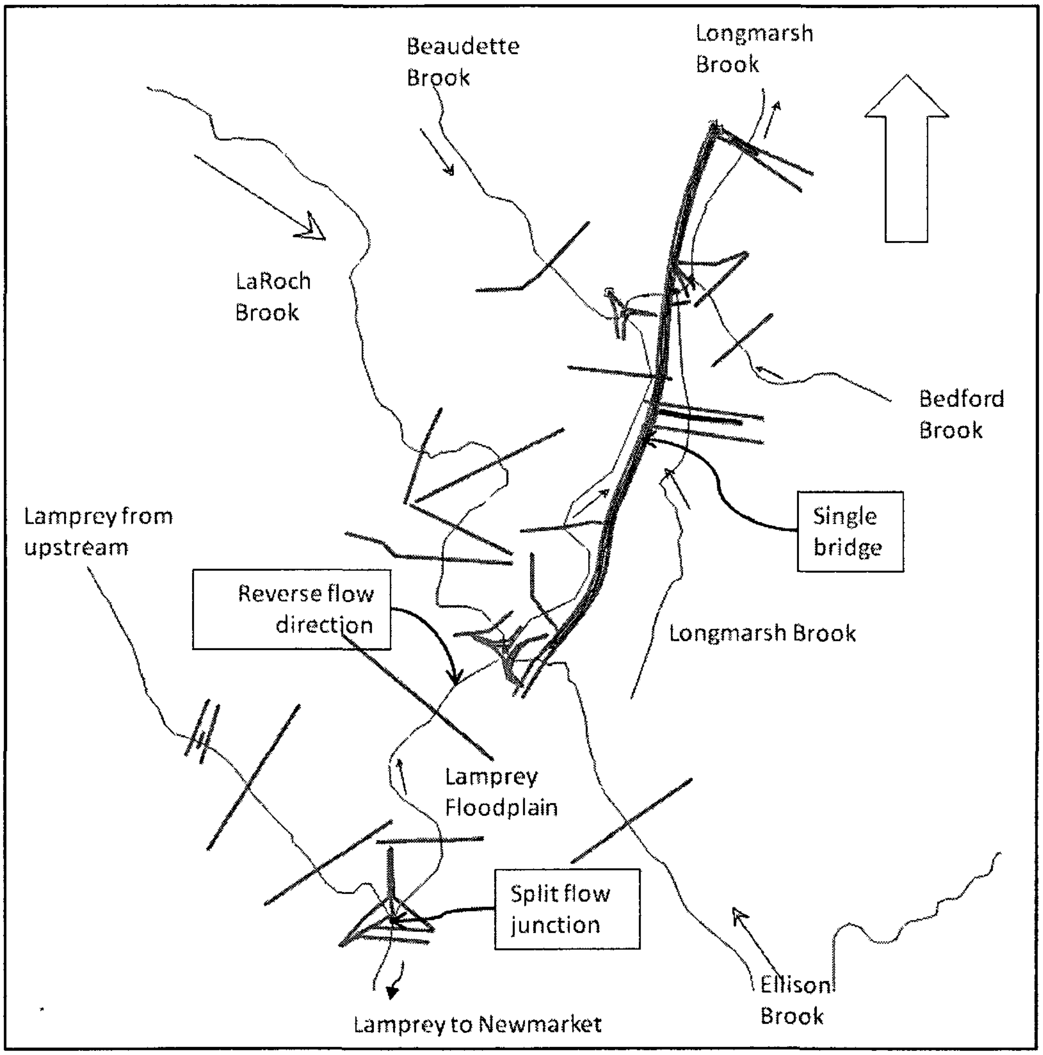

Figure 39: HEC-RAS V3 geometric data view of RT108 corridor with stream network (blue), cross sections (green) and structures (black)

(d) Junction Optimization Trials

This third version required split flow optimization calculations. Split flow optimization calculations in HEC-RAS continues to attempt to balance flow splitting from one reach into two until the energy gradelines (EGL) of the receiving streams are within the specified tolerance (USACE 2001). This process is iterated to determine the point where the EGL differential is the smallest. For this analysis, the downstream EGL tolerance was 0.02 feet. In Figure 40, Reach 1 symbolizes the USRT108 FP (Lamprey River), Reach 2 symbolizes the DSRT108 FP (Lamprey River) and Reach 3 symbolizes OR Bypass (perennial floodplain). An initial estimate of the flow that is leaving the main river is entered in the flow profiles. Table 17 provides the optimization results at the split flow junction.

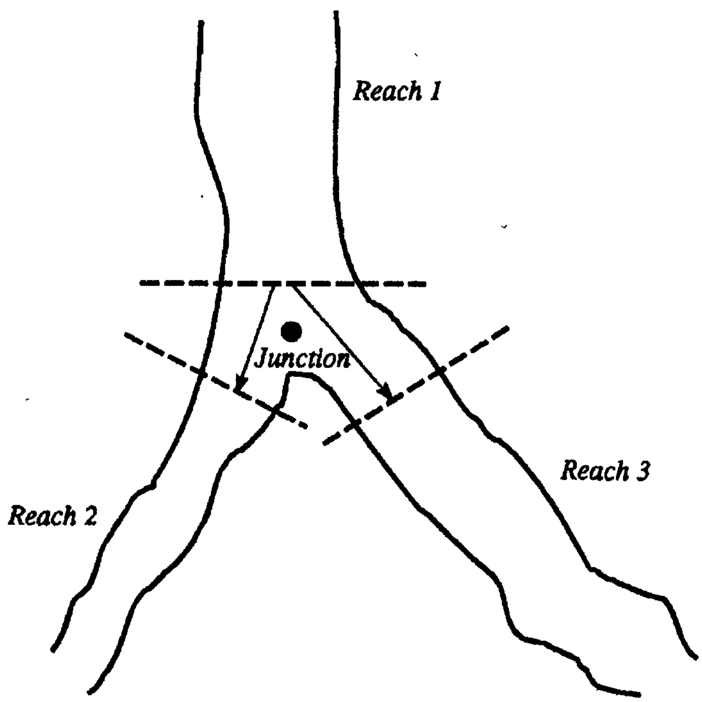

Figure 40: Flow split at junction (USACE 2010)

Table 17: Final results of split flow optimization

Reach	HEC-RAS River Sta	Profile	W.S. Elev (ft)	E.G. Elev (ft)	Q Total (cfs)	Downstream EGL Diff.
USRT108 FP	8998	NRCC 100-YR	36.98	37.14	10,649	0.02
USRT108 FP	8998	7-Apr	35.06	35.2	8,332	0.01
USRT108 FP	8998	10-Mar	34.88	35	7,481	0.00
USRT108 FP	8998	2050 Trad	37.24	37.41	11,109	0.00
USRT108 FP	8998	20502 LID	37.2	37.36	10,952	0.00
Junction:	RT108 FP					
% Lamprey River Main Channel						
DS RT108 FP	8890	NRCC 100-YR	36.8	37.1	9,943	93.4%
DS RT108 FP	8890	7-Apr	34.93	35.16	7,573	90.9%
DS RT108 FP	8890	10-Mar	34.81	34.96	6,158	82.3%
DS RT108 FP	8890	2050 Trad	37.02	37.35	10,630	95.7%
DS RT108 FP	8890	20502 LID	36.98	37.3	10,493	95.8%
% Bypass to Oyster River Watershed						
OR_Bypass	6377	NRCC 100-YR	37.08	37.08	706	6.6%
OR_Bypass	6377	7-Apr	35.15	35.15	759	9.1%
OR_Bypass	6377	10-Mar	34.95	34.96	1,323	17.7%
OR_Bypass	6377	2050 Trad	37.35	37.35	812	7.3%
OR_Bypass	6377	20502 LID	37.3	37.3	788	7.2%

(e) Hydraulic Computations

Flow for the Lamprey River was modeled as quasi steady using flow changes dictated by the HEC-HMS model.

Similar to the WSP2 program, the water surface profiles are calculated from one cross section to the next by solving the Energy Equation using the standard step method.

$$\boldsymbol{Z_2 + Y_2 + \frac{a_2 V^2{}_2}{2g} = Z_1 + Y_1 + \frac{a_2 V^2{}_1}{2g} + h_e} \qquad \textbf{(11)}$$

Where: Z_1, Z_2 = elevation of main channel invert

Y_1, Y_2 = depth of water at cross sections (L)

V_1, V_2 = average velocity

(total discharge/total flow area) (L/T)

a_1, a_2 = velocity weighting coefficients

g = gravitational acceleration (L/T^2)

h_e = energy loss (L)

Friction losses and contraction or expansion losses make up the energy head loss between cross sections. The equation for the energy loss is:

$$h_e = L\overline{S}_f + C\left|\frac{a_2V_2^2}{2g} - \frac{a_1V_1^2}{2g}\right| \qquad \textbf{(12)}$$

Where: L = discharge weighted reach length (L)

S_f = friction slope between two sections based on average conveyance

C = expansion or contraction loss coefficient

The weighted reach length is calculated using the sum of cross section reach lengths in left and right overbanks and main channel multiplied times their respective flow in each section and then divided by the total cross section discharge.

The Manning Equation is used to determine conveyance within each subdivision of the cross section. Using the input cross section n-value points of change, the cross section is subdivided into units for which the velocity is uniformly distributed. Conveyance is determined by equation:

$$Q = K\overline{S}_f^{\frac{1}{2}} \qquad \textbf{(13)}$$

$$K = \frac{1.486}{n} AR^{\frac{2}{3}} \qquad \textbf{(14)}$$

Where: K = conveyance for subdivision

n = Manning's roughness coefficient for subdivision

A = flow area for subdivision (L^2)

R = hydraulic radius for subdivions (area/wetted perimeter) (L^2/L)

3.3.4 ***Hydraulic Model Calibration***

(a) Field Verification

Following completion of the hydraulic model, calibration options were examined. The public works directors and/or public officials for the communities along the Lamprey River were e-mailed asking for any flood flow elevations noted on bridges and buildings during recent observed events. The USGS website provided field measurements to develop rating curves of the river section nearest to the gage location. The USGS provided additional information regarding noted high water marks at the Langford Lane bridge in Raymond following one observed flood flow. The on-call Town Engineer for Epping, Chris Albert, provided photos and high water elevations at the Mill Street bridge crossing. Along the RT108 corridor, the Durham Boat Club, 220 Newmarket Road and the resident at 216 Newmarket Road provided elevations for high water marks during the two modeled observed events (Table 18).

(b) USGS Gage Discharge Curves

USGS Gage No. 01073500 is located 380 feet upstream of the Packer's Falls Road bridge near Newmarket (HEC-RAS Sta. 16,077). The USGS stream flow measurements were entered under observed rating curves in the Options menu of the HEC-RAS Steady Flow Data file. In order to get a complete rating curve modeled in HEC-RAS, additional low flow profiles were included in the Steady Flow Data file. Figure 41 provides a comparison between the observed and modeled water surface elevations. Minor differences for flows less than 1,000 cfs may be attributed to the precision of the FIS geometry used for the cross section. During flows greater than 4,000 cfs more than 12% of the discharge begins to spread into the right overbank.

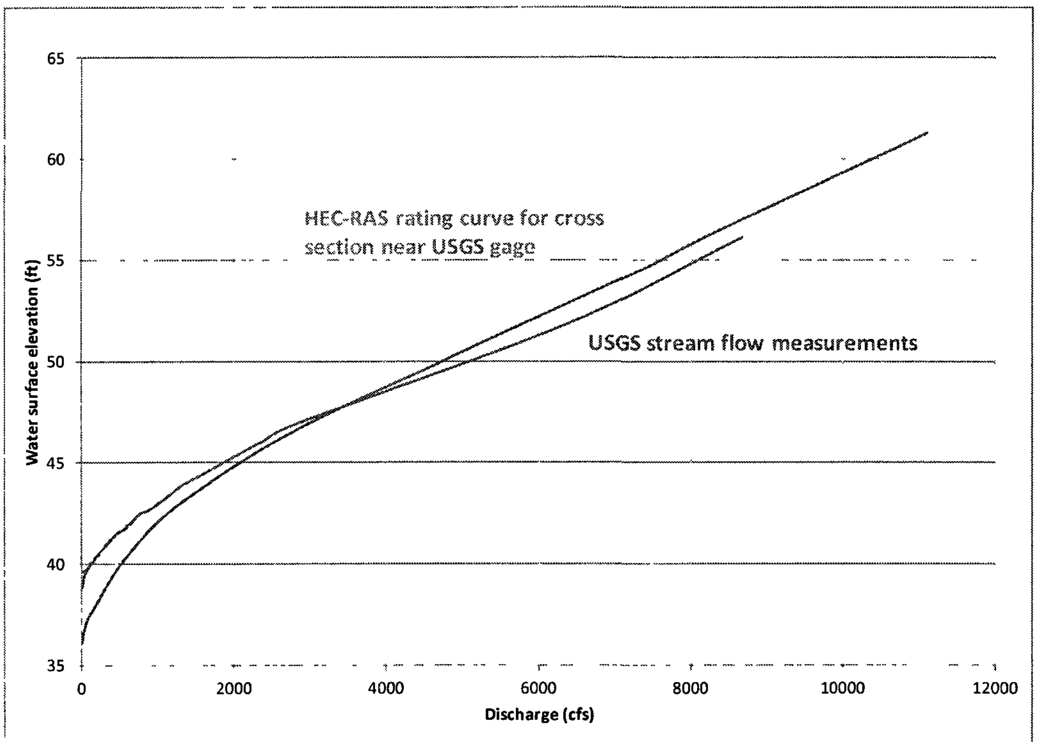

Figure 41: Rating curve at USGS gage in Newmarket

USGS Gage No. 01073319 is located 100 feet downstream of the Langford Road bridge in Raymond (HEC-RAS Sta. 167,810). There is only a short section provided for comparison. This gage station has only been in operation since July of 2008. Differences again are related to the precision of geometry and the questionable location of the gage in relation to the location of the modeled cross section. The USGS coordinates placed the gage in the far left overbank of the Lamprey River.

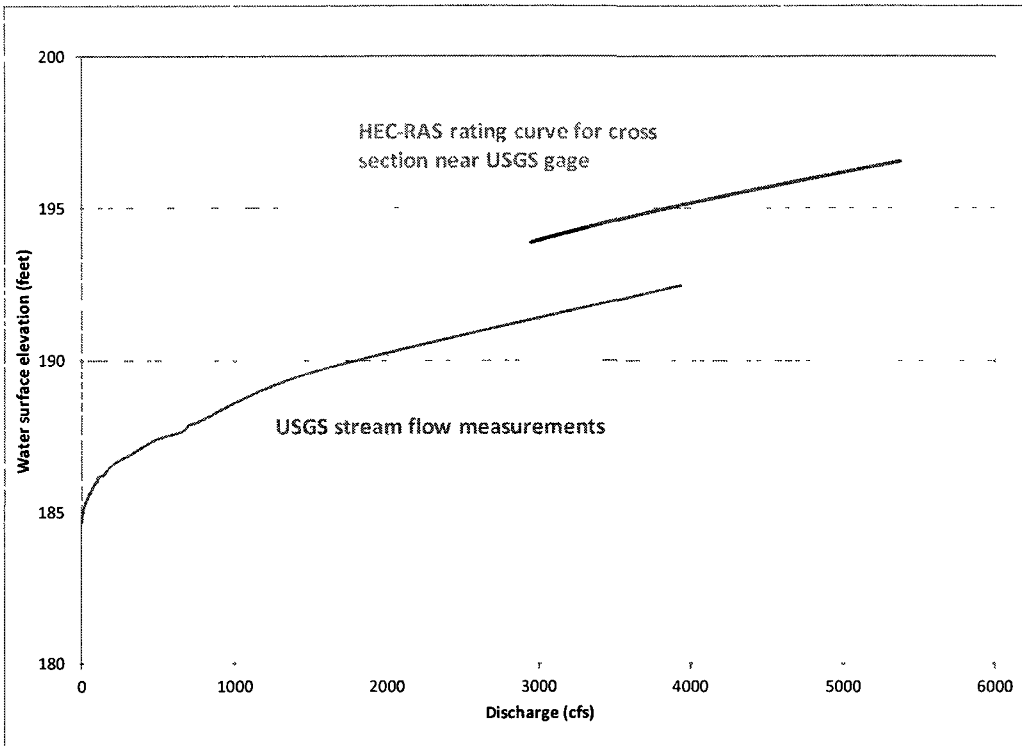

Figure 42: Rating curve at USGS gage in Raymond

(c) Observed high water marks

The April 2007 and March 2010 events provided historic high water and flooding. Following the event the USGS surveyed high water indicators (HWI) at the Langford Road bridge in Raymond. High water elevation at the Mill Street bridge in Epping was photographed and referenced to adjacent landmarks. The Durham Boat Club's interior walls provided the indication of high water marks and personnel have labeled and recorded most major flooding events. In using the recent NHDOT survey of RT108, the markings were converted to approximate elevations. Table 18 provides the obtained information and reference to the HEC-RAS model location.

Table 18: Observed high water elevations

Location	River HEC-RAS Sta.	Event	WSE (NAVD29)	Indication
Langford Road, Raymond	Lamprey 167,900	April 2007	197.155	Wash line
			197.205	Wash line
			194.88	Seed line
Mill Street, Epping	Lamprey 106,389	April 2007	113.00	Observed HWI
Durham Boat Club, Newmarket	Beaudette 71	April 2007	34.1	HWI in building
		March 2010	33.3	HWI in building

Chapter 4

Results and Discussion

The accuracy of the hydrologic and hydraulic modeling was successfully calibrated through more than 30 optimization trials to mimic the observed flood flows and water surface elevations recorded during historic events. Three historic flood events in May 2006, April 2007, and March 2010 were used for observed discharges. The May 2006 event was eliminated because the event was spread out over 13 days; the largest precipitation fell after antecedent moisture conditions were saturated, and consequently did not represent a type III rainfall distribution.

The subbasin loss rate parameters estimated included initial abstraction and curve number. Lag time was the sub basin transform parameters estimated. The reach routing parameters estimated included Manning's n for the Muskinghum-Cunge method and Muskinghum X, K, and number of steps for the Muskinghum method.

Optimization trials started by comparing the model to observed discharges without baseflow losses. This resulted with simulated peak flows much higher than the observed discharge (Figure 23). Three methods were evaluated to separate the direct runoff and baseflow: Constant-Discharge; Constant-Slope; and Concave. Results of the separation for the three events (May 2006, April 2007, and March 2010) and excess precipitation are shown in Figure 43 through Figure 45. The Constant-Discharge method resulted with less than 11.2-, 19.8-

and 20.2% of the total volume being separated as baseflow for the May 2006, April 2007, and March 2010 events respectively. In all three instances, the baseflow was greater than the direct runoff for the Constant-Slope method. The Concave method provided improved consistency between the events with a range of difference of 36.4 to 41% of the total runoff volume being separated as baseflow.

The next round of trials compared the simulated model to observed discharges with a constant-discharge baseflow. This round of optimizations provided results outside the 5% range of accuracy for the simulated runoff volume (in), peak flow (cfs), time of peak, and time of center of mass in matching the observed event.

The final round of trials compared the model to observed discharges with a concave baseflow separation applied. After a series of trials, the Manning's n coefficient was set to 0.12 in the reaches upstream of Lee, due to the presence of log jams.

Simulations were closer to what was acceptable following this adjustment. The best fitting hydrologic model was established using the April 2007 event.

The simulated parameters were used for the hydrologic scenarios: TP-40 2005, NRCC 2005, NRCC 2050 Conventional, NRCC 2050 LID. In order to evaluate the implementation of LID, the modeling results were examined at both watershed scale and urban subwatershed scale because greater resolution can be observed at the smaller scale.

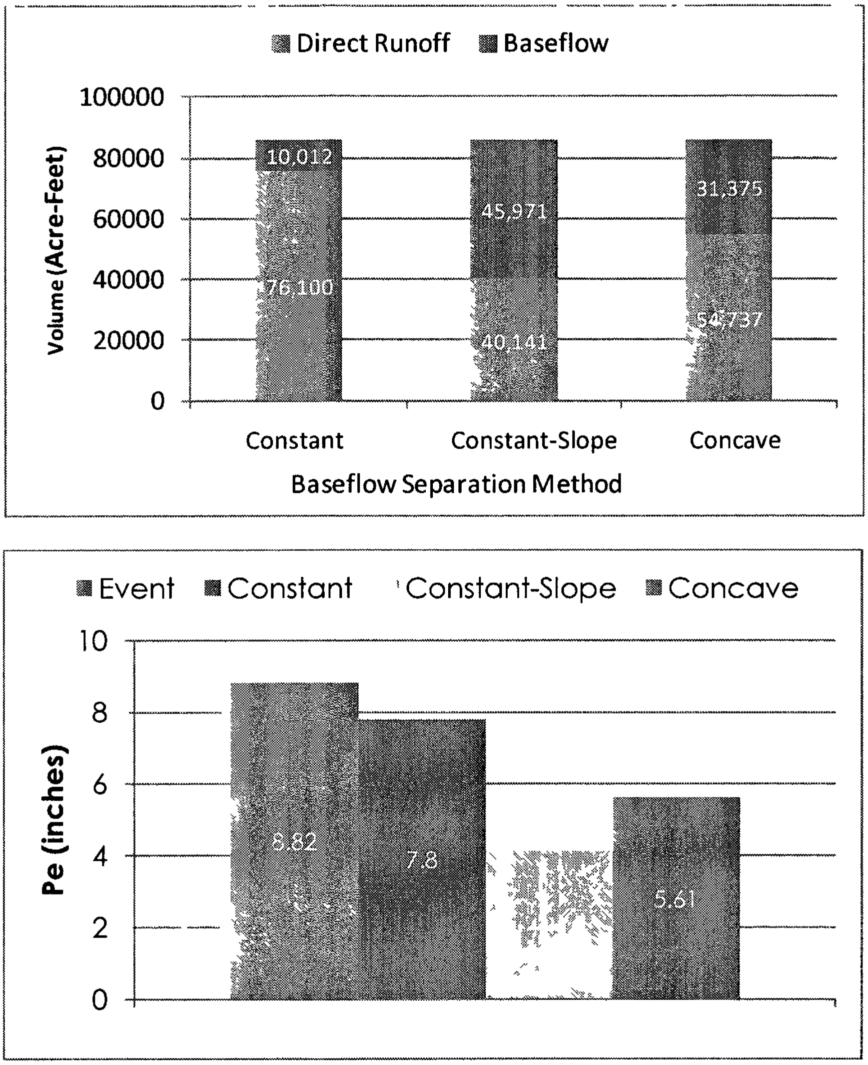

Figure 43: May 2006 baseflow separation and excess precipitation

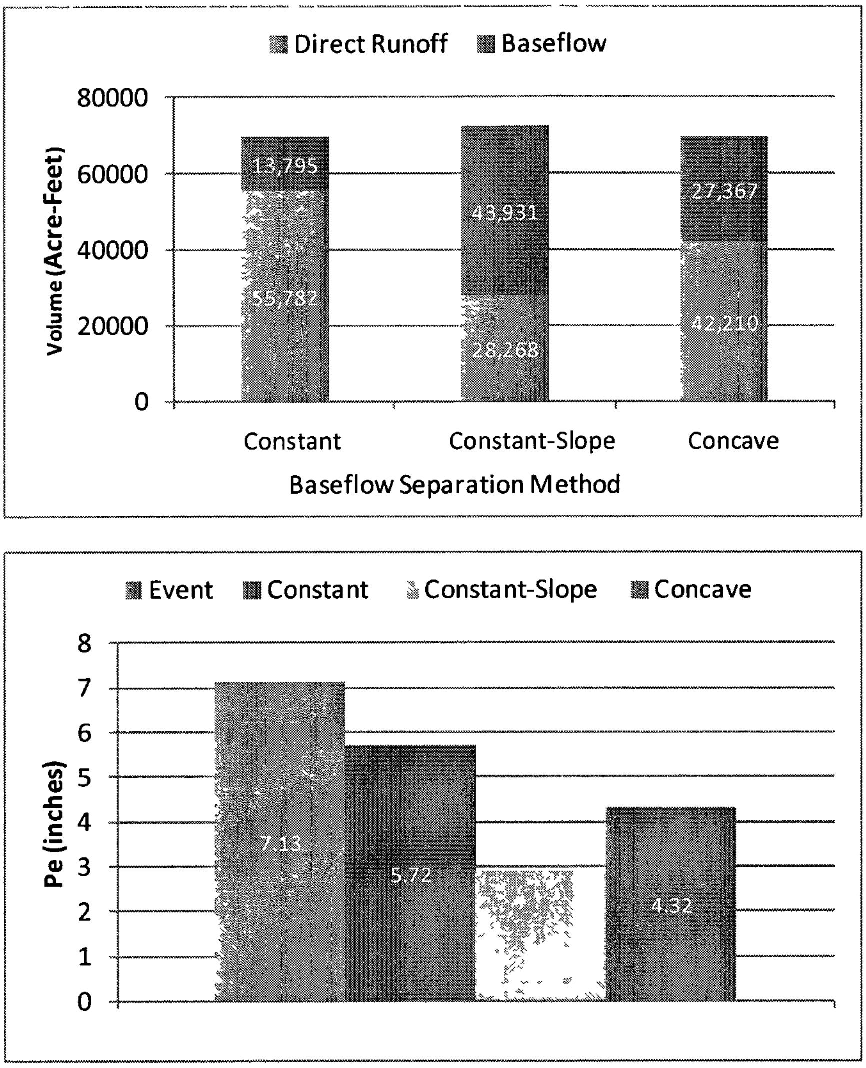

Figure 44: April 2007 baseflow separation and excess precipitation

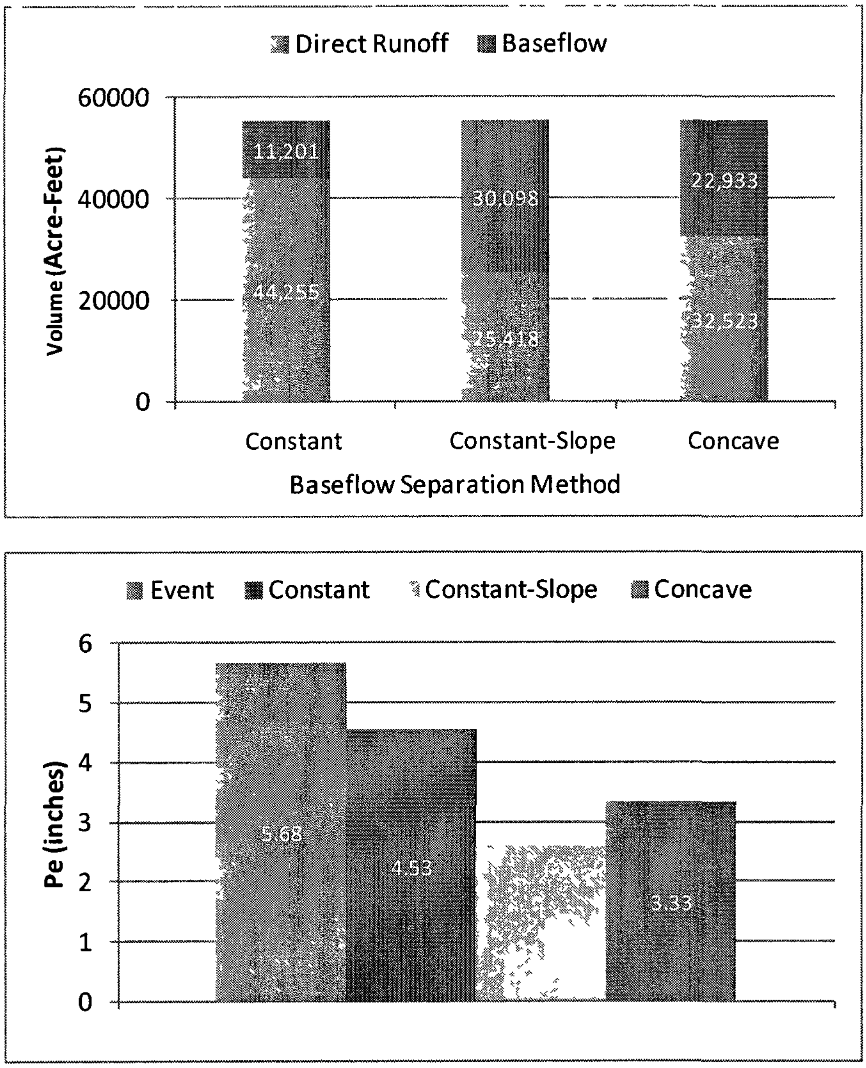

Figure 45: March 2010 baseflow separation and excess precipitation

4.1 Flood Insurance Study (FIS) Model

At the USGS gage location near Newmarket, the FIS 100-year discharge for the 183 square mile watershed upstream is 7,300 cfs (FEMA 2005). This flood flow was verified using the annual peak discharges for the years 1935 through 1987 as the input file for the USGS Office of Surface Water software program, Peak flow FreQuency analysis program (PKFQWin). FEMA's Map Modernization criterion for revaluation was applied using annual peak discharges for the years 1935 through 2009. The updated analysis performed with PKFQWIN resulted with a significant change to the FIS flood flows (Figure 4) since 9,411 cfs falls outside of the upper 68-percent confidence interval for the FIS flood flow of 7,300 cfs;

$$L_{0.01,0.68} = 6{,}886 \text{ and } H_{0.01,0.68} = 7{,}834.$$

The third record set of annual peak discharges included the last 30 years (1980 through 2009) in order to evaluate the impact of climate change on predicted discharge at a 100-year recurrence interval. This data set resulted with a 100-year flood flow of 13,770 cfs. The details for the PKFQWin calculations are provided in Appendix B.

The PKGQWIN program follows the Bulletin 17B recommendation of fitting the complete annual peak discharge data to a log-Pearson Type III (LPIII) probability distribution (U.S. Interagency Advisory Committe on Water Data, 1982). The station skew is a measure of the symmetry for the flow distributions. The closer this value is to zero, the more the values are evenly distributed. The full data set (1935 – 2009) adds eight more extreme events (flows > 3,400 cfs) to the flow distributions. Extreme flood events often affect the skew as they adjust the symmetry in the probability distribution of values. Estimates of flood frequency discharges are sensitive to the skew coefficient (McCuen 2004). This can be observed by the 66% increase in the station skew by comparing the full record to the FIS data set.

In utilizing the most recent 30 years of data, the increase in the positivity of station skew indicates that the bulk of the values lie to the left of the mean. Bulletin 17B provides procedures for weighting the station skew with the generalized skew to improve the accuracy for the watershed's estimated flood flow. The generalized skew coefficient used for developing a weighted skew is taken from a generalized skew map provided in Bulletin 17B (Appendix B). This map was prepared in 1976 and was generated from then current information about skew for sites within a standardized region. Table 19 summarizes the resulting weighted skew coefficient based on the station skew and a generalized map skew value of 0.554.

Table 19: LPII Discharges and skew coefficients using generalized map skew of 0.554

Years	100-year Q (cfs)	Station Skew	Weighted skew
1935 – 1987	7,300	0.052	0.178
1935 - 2009	9,411	0.398	0.435
1980 - 2009	13,770	0.589	0.574

A new method for determining generalized skews in New Hampshire has been completed to replace the outdated mapping (Olson 2009). The new generalized skew map for New Hampshire is provided in Appendix B and the watershed is located within the 0.30 contour. The weighted skew for the two data sets, inclusive of the recent extreme events: 1935 – 2009 and 1980 – 2009, are 0.375 and 0.466 respectively. Both of these provide a 16- to 23% decrease than previously calculated with the former generalized skew. The adjustment estimates flood frequency discharges provided in Table 20.

Table 20: LPIII Discharges and skew coefficients using generalized map skew of 0.30

Years	100-year Q (cfs)	Station Skew	Weighted skew
1935 - 2009	9,196	0.398	0.375
1980 - 2009	13,145	0.589	0.466

Olson evaluated several gaged locations in New Hampshire in regards to the increased annual peak discharges experienced in the recent decades and their effect to flood-discharge frequency estimates. The results indicated no definite pattern that would suggest limiting the use of the entire period of record available for the stream gage (Olson 2009).

4.2 **Modeling**

4.2.1 ***Hydrologic Model Results***

(a) Sensitivity of parameters

The final optimized runoff parameters provided the watershed characteristics. These parameters along with the rainfall depths developed the historic, current and future build-out hydrologic models. For a watershed of this size (213 sq. mi.), the sensitivity of the rainfall input is decreased (McCuen 2004) and this research evaluated the sensitivity of the watershed characteristics. The initial runoff curve number (CN) determined in ArcMap with the HEC-GeoHMS tools was utilized as the current sub basin loss rate parameter. The CN was included in the optimization trials; however, the estimated values did not provide a result of acceptable accuracy. A plot of a percent change in CN verses the percent change in peak flow is given in Figure 46. This indicates that an increase or decrease in CN would cause an equal percent increase or decrease in peak flow.

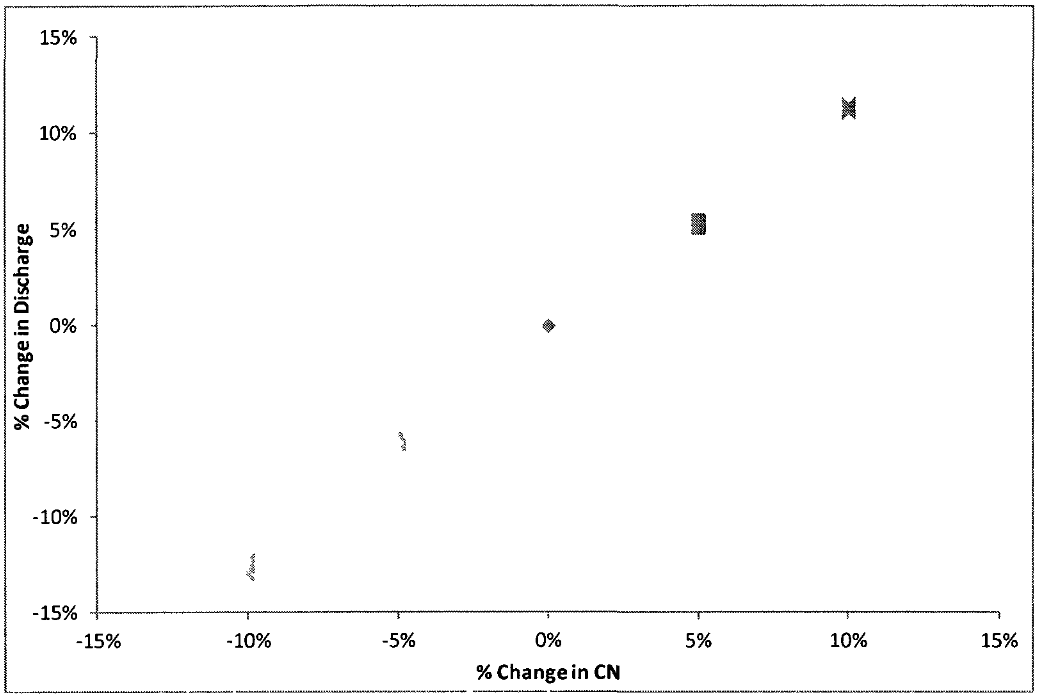

Figure 46: Sensitivity of curve number (CN) adjustments

(b) Watershed scale Curve Number (CN)

The entire Lamprey River watershed consists of eleven (11) sub basins. These were created by developing a catchment location along the river's path that coincides with a change of flow regime from the FIS. The catchment is the downstream location that delineates the sub basin for every stream segment. These sub basins range in size from 0.9 to 33.9 square miles. At this watershed scale, composite CNs where determined for each sub basin based on the area, land use and hydrologic soil group for current and the 2050 conventional and 2050 LID build-out conditions (Figure 10). Table 21 provides the comparison of the CNs for the three scenarios and the 2050 build-out ΔR. ΔR is the same terminology as ΔQ in the SCS graphic method. R (Q) is the depth of runoff in inches and is a function of the depth of rainfall and the runoff CN. ΔR (ΔQ) is the difference in runoff between the 2050 conventional and LID build-out.

There is a limited variation in CN values at the watershed scale. In comparing the current 2005 CN values to the 2050 conventional build-out, the range of overall CN values increased by the least amount in sub basin W8380 by 1.9 and by the greatest amount in sub basin W10910 by 6.1. This increase in sub basin W10910 is reasonable. This sub basin includes the previously discussed business, industrial, and commercial zoning districts available for development in the Town of Raymond. The small change in sub basin W8380 is realistic as this sub basin includes a large portion of Pawtuckaway State Park and land areas protected from development.

In comparing the 2050 conventional and LID build-outs, the overall CN values were adjusted by the least amount in sub basin W8380 by 0.5 and by the greatest in sub basin W10910 by 2.0. The same sub basins have the least and greatest adjustments. Positive results can be anticipated by implementing LID for any potential development in Raymond. The LID scenario translates to a reduction in sub basin runoff from a conventional by 0.06 to 0.20 inches. The total overall decrease equates to approximately 945 ac-ft less runoff from the entire watershed.

The future CN values for the eleven sub basins are compared to the current 2005 values in Figure 47. Both types of future development do increase the current CN value as they plot to the left of the non-effect line. The slight difference seen in comparing the conventional and LID CN is noted by the close proximity of the plotted data to the non-effect line.

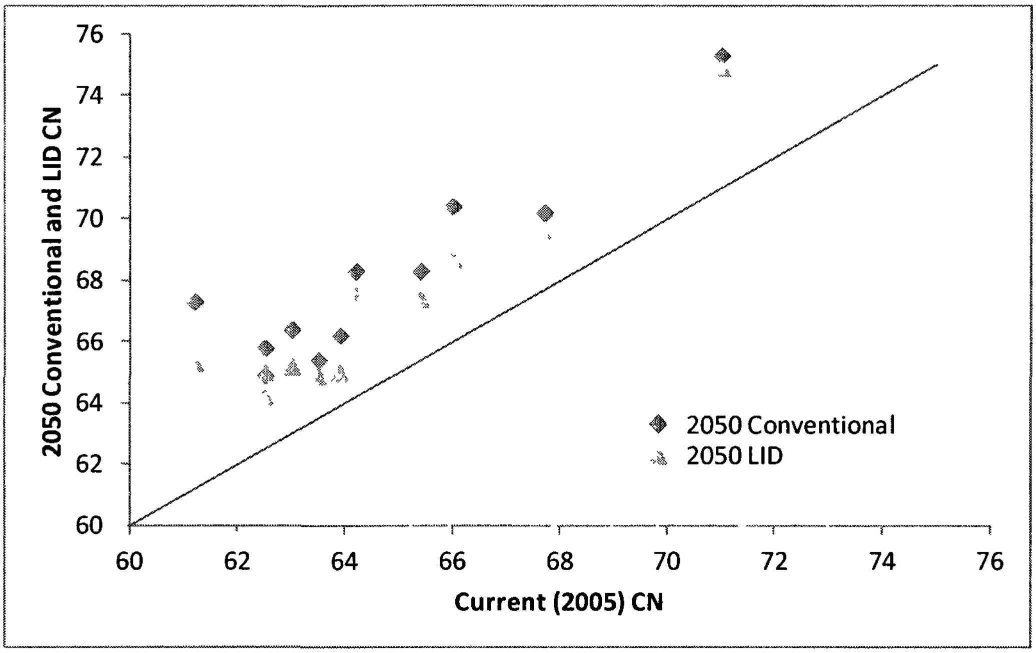

Figure 47: Watershed Scale comparison of CN values for eleven sub basins

Additional examination of the limited variation in watershed average CN values is related to the fact that about 73% or 155 of the 213 square mile watershed is forested, agricultural or other open space. These cover types have CN values ranging from 30 for forested type A soils to 80 for open space on type D soils. The build-out scenario decreased this land cover by 60 square miles or 45%. As a result, the forested area of the watershed is still able to maintain a relatively low CN. In building out the watershed, most of the 15 communities require at least a two (2) acre lot for a single residential use. Only Durham, Newmarket, and Raymond have residential zone districts with minimum lot sizes less than one (1) acre. In reference to Figure 28, influence of LID is minimal below 3 – 7% impervious cover. Additionally, these same communities have 5-15% of land area protected from future development. Approximately 13% of the watershed is covered with wetlands and open water. In rainfall runoff models, this type of land use has the highest CN value applied; therefore there are no losses and the direct runoff equals the precipitation (Figure 48). The LID implementation cannot adjust the CN applied for this land use and as a result the inclusion of these 27.7 square miles contributes to the minor difference observed.

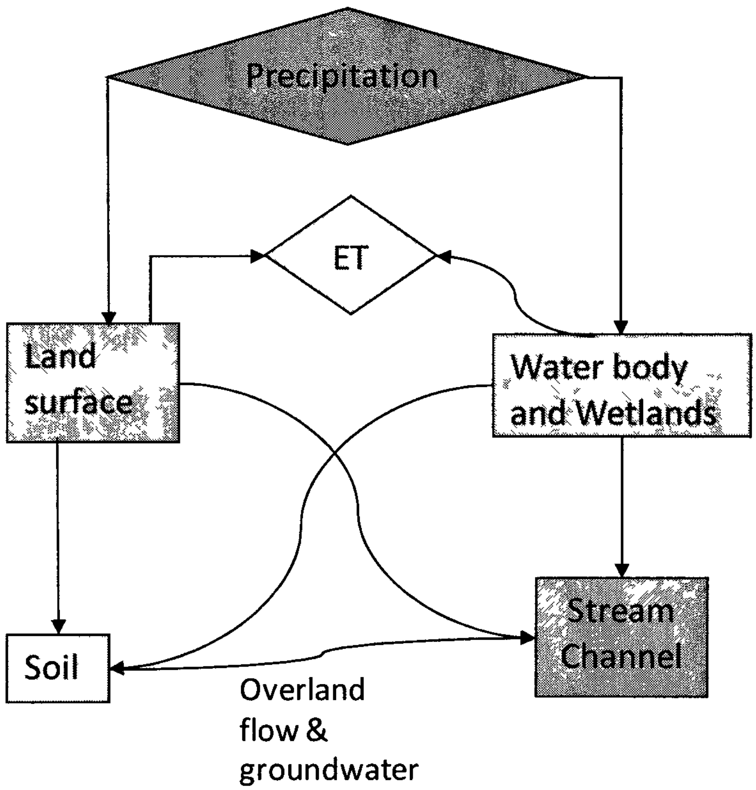

Figure 48: Representation of watershed runoff on land and water

Table 21: Watershed scale CN values and runoff difference of future scenarios

Sub basin	Boundary Condition (Catchment)	Sub basin Area (sq.mi.)	Current (2005)	2050 Conventional Build-out	2050 LID Build-out	Build-out ΔR (in)
W6510	(RT27) Alt. RT 101, Raymond	32.2	62.5	64.9	64.2	-0.08
W8600	Langford Road, Raymond	19.0	63	66.4	65.2	-0.14
W11900	Downstream corporate limit, Town of Raymond	16.0	64.2	68.3	67.5	-0.10
W10910	Western corporate limits, Town of Epping	6.5	61.2	67.3	65.3	-0.24
W8380	Blake Road, Epping	12.3	63.5	65.4	64.9	-0.06
W11020	RT 101, Epping	6.1	62.5	65.8	65.0	-0.09
W6730	Northern corporate limits, Town of Epping	58.3	65.4	68.3	67.4	-0.11
W7060	USGS Gage No. 01073500	33.9	63.9	66.2	65.0	-0.14
W7920	Durham/Newmarket corporate limits	4.5	67.7	70.2	69.5	-0.08
W10250	Confluence of Pisscassic River	21.7	66	70.4	68.7	-0.20
W8590	Macallen Dam, Newmarket	0.9	71	75.3	74.9	-0.05

Net gains and (losses) from the existing land use (Table 4) to build-out conditions of the watershed communities are provided in Table 22. The change in residential growth was more than 200% in Deerfield, Exeter, and Newfields. Industrial and commercial growth saw the greatest gain in Candia, Northwood, and Nottingham. The effect of this build-out resulted in net losses of developable land. The largest watershed loss was more than 35,000 acres of wooded land. General open space decreased by approximately 2,300 acres while the other losses combined to less than 1,200 acres within the watershed.

(c) Urban sub watershed scale CN

Three smaller sub watersheds in urban settings were examined for differences between build-out with LID versus conventional. Sub watersheds were selected based on their urban setting and that they are mapped as Zone A on the Flood Insurance Rate Map (FIRM). Zone A is a special flood hazard area where the water surface elevation of the 1% annual chance flood was determined in the FIS by approximate methods. No BFEs are provided in these areas because of the lack of a detailed hydraulic analysis.

The subwatersheds are: 1) Moonlight Brook in Newmarket, 2) unnamed tributary in Epping, 3) unnamed tributary in Raymond. Moonlight Brook flooded and caused extensive damage in Newmarket during the May 2006 event. The subwatersheds in Epping and Raymond drain into the Lamprey River. The calculated CN, and the runoff and discharge were examined following a 24-hour, 100-year design storm of 8.5 inches for existing conditions and the two

Table 22: Gains and (losses) of conditions in watershed communities from current to the 2050 build-out condition

Land cover description	Acres of land cover in watershed communities							
	Barrington	Brentwood	Candia	Deerfield	Durham	Epping	Exeter	Fremont
Residential	304	36	2,085	3,515	396	3,508	234	580
Industrial/Commercial and Business	<1	71	261	2	8	229	1	22
Open Space	(8)	(42)	(230)	(491)	(307)	(471)	(37)	(151)
Pasture, grassland or range	-	-	-	-	-	-	-	-
Farmsteads	-	(4)	(41)	(68)	(9)	(55)	(0)	(16)
Brush	-	(3)	(37)	(13)	(36)	(290)	(9)	(42)
Woods	(451)	(289)	(2,694)	(6,527)	(1,276)	(4,397)	(664)	(1,006)
Open Water/Wetlands	-	-	-	-	-	-	-	-
Natural Desert (Beaches)	-	-	-	-	-	-	-	-
Newly graded (Disturbed land)	(3)	(8)	(20)	(37)	(8)	(92)	(15)	(25)
Fallow Bare Soil	(1)	-		-	-	-	-	-

Table 22: Gains and (losses) of conditions in watershed communities from current to the 2050 build-out condition (cont')

Land cover description (Acres)	Acres of land cover in watershed communities						
	Lee	Newfields	New-market	Northwood	Nottingham	Raymond	Strafford
Residential	2,287	732	1,990	602	2,998	1,097	4
Industrial/Commercial and Business	-	-	57	533	235	135	-
Open Space	-	(80)	(140)	(82)	(191)	(107)	-
Pasture, grassland or range	-	-	-	-	-	-	-
Farmsteads	(7)	(6)	(5)	(1)	(10)	(34)	-
Brush	(67)	(48)	(13)	-	(2)	(66)	-
Woods	(2,476)	(1,180)	(1,654)	(2,781)	(7,665)	(1,943)	(74)
Open Water/Wetlands	-	-	-	-	-	-	-
Natural Desert (Beaches)	-	-	-	-	-	-	-
Newly graded (Disturbed land)	(50)	(6)	1	(5)	(39)	(12)	-
Fallow Bare Soil	-	-	-	(10)	(5)	-	-

build-out scenarios. In these urbanized settings, where commercial and industrial land use is predominant, large increases in CN were observed for future conventional development.

Table 23 illustrates that with implementation of LID, the 8.5 inch rainfall depth over 2005 existing conditions can be practically maintained in the future 2050 build-out scenarios. R is the same terminology as Q in the SCS graphic method. R (Q) is the depth of runoff in inches and is a function of the depth of rainfall and the runoff CN. Q is the same terminology as q_p in the SCS graphic method. Q (q_p) is the peak discharge in cubic feet per second and is a function of lag time, precipitation, initial abstraction, drainage area, and runoff.

The future CN values for the urban sub basins are compared to the current 2005 values in Figure 49. The future conventional development does increase the current CN value in the three urban sub basins as they plot to the left of the non-effect line. There is only a slight difference between the LID CN and current CN noted by the close proximity of the plotted data and non-effect line.

Table 23: Urban scale composite CNs, runoff, and discharge based on 100-year 24-hour rainfall depth of 8.5 inches

Subwatershed	Moonlight Brook, Newmarket			Intermittent Stream, Epping			Intermittent Stream, Raymond		
Area (sq. mi.)	0.88			1.2			0.86		
Subwatershed values	CN	R (in)	Q (cfs)	CN	R (in)	Q (cfs)	CN	R (in)	Q (cfs)
Current 2005	66.8	4.5	655	70	4.9	1,031	65.8	4.4	508
2050 conventional build-out	78	5.9	852	81.7	6.3	1,320	79.0	6.0	696
2050 LID build-out	69.5	4.8	704	69.4	4.8	1,016	66.6	4.5	520

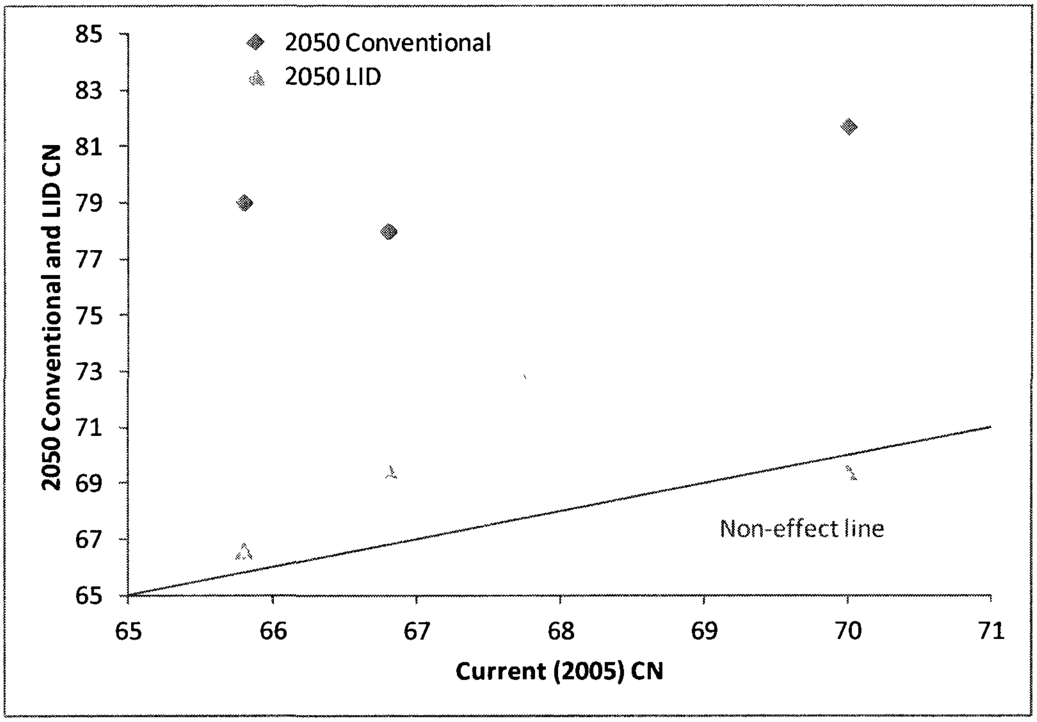

Figure 49: Urban Scale comparison of CN values for sub basins in Raymond, Newmarket, and Epping (left to right)

As the conversion of land use increases the impervious cover and resulting CN, the LID application indicates that redevelopment can have a positive effect and actually decrease the current CN coefficient. This is verified by the future LID CN value for Epping.

More than one inch of additional direct runoff was observed for the conventional build-out. Conversely, the LID build-out provided less than 0.30 inches of additional runoff. In some cases, by implementing LID in the redevelopment of commercial and industrial properties, the build-out conditions have less runoff than the current conditions. The urban setting selected in Epping includes a highly developed portion along RT125. Approximately 27 acres are currently developed and the weighted CN for this use is 90. By 2050, it is estimated that 80 acres will be developed. If the land is developed conventionally, the CN maintains a high value of 91. However, if the development and redevelopment implements LID, the CN decrease to 71. This substantial decrease reflects the CN adjustments between conventional and LID development. For example, conventional industrial development on type A soils has a CN value of 89. In that same footprint, by implementing LID, the CN is lowered to 64. This can also be observed in Figure 50 through Figure 52 when a response can be seen to occur at higher CNs and on poor soils.

The two callouts on Figure 50 reference undeveloped forested land with HSG soil types C and D. Because of the anticipated growth rate in the watershed, these parcels are likely developed as residential and commercial/industrial property by 2050. If conventional development is applied the CN value increases by 7 for the residential use and 24 for the commercial/industrial use. By implementing LID, the CN number adjustment is lowered and only increases by 3 for the residential use and 10 for commercial/industrial use. This is because LID adjusts curve numbers based on the designed reduction of runoff volume. Using LID practices, one inch of rainfall on an impervious surface is infiltrated into the surrounding terrain. Thereby it is effectively disconnected from traditional catchment and conveyance and site runoff is reduced.

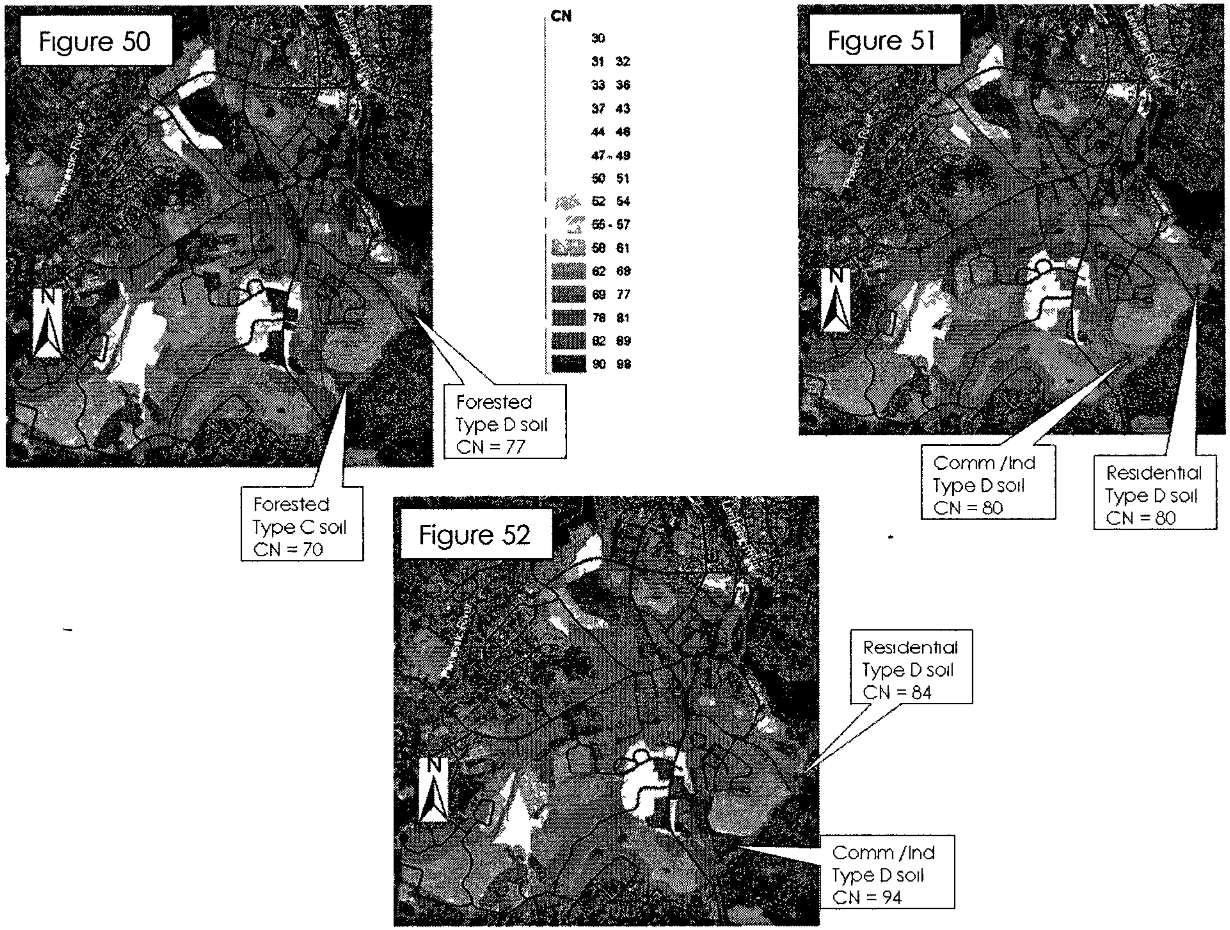

Figure 50 Current (2005) CN conditions for Moonlight Brook
Figure 51: Conventional 2050 build-out CN conditions for Moonlight Brook
Figure 52: LID 2050 build-out CN conditions for Moonlight Brook

4.2.2 ***HMS optimization trial results***

The initial sub basin parameters were developed in HEC-GeoHMS. This included the sub basin area, composite CN, lag time, and reach routing. The model was calibrated to an observed rainfall and associated stream flow discharge event.

Calibrating the hydrologic model to the April 2007 historic flood event produced the closest comparison. The rainfall distribution of the April 2007event was similarly graphed in respects to a SCS synthetic 24-hour type III storm (Figure 16).

The objective function selected for calibration was the peak-weighted *root mean square error* (RMSE) *which indicates how close the observed data* points of the observed hydrograph are to the simulated hydrograph predicted values. Lower values indicate a better fit. Calibration was eventually achieved by running optimization trials until results were in the acceptable range of accuracy for the model (5% difference between observed and simulated hydrographs).

The objective function graph (Figure 53) provides the value of the objective function after the iteration during the search method in addition to an indication of how fast the model was able to converge to the best possible parameter values. The maximum iterations allowed for the trial was 50 and the objective function was achieved with less than 35 iterations. When the function value is within a 5% difference between simulated and observed hydrographs, iteration ends for that parameter.

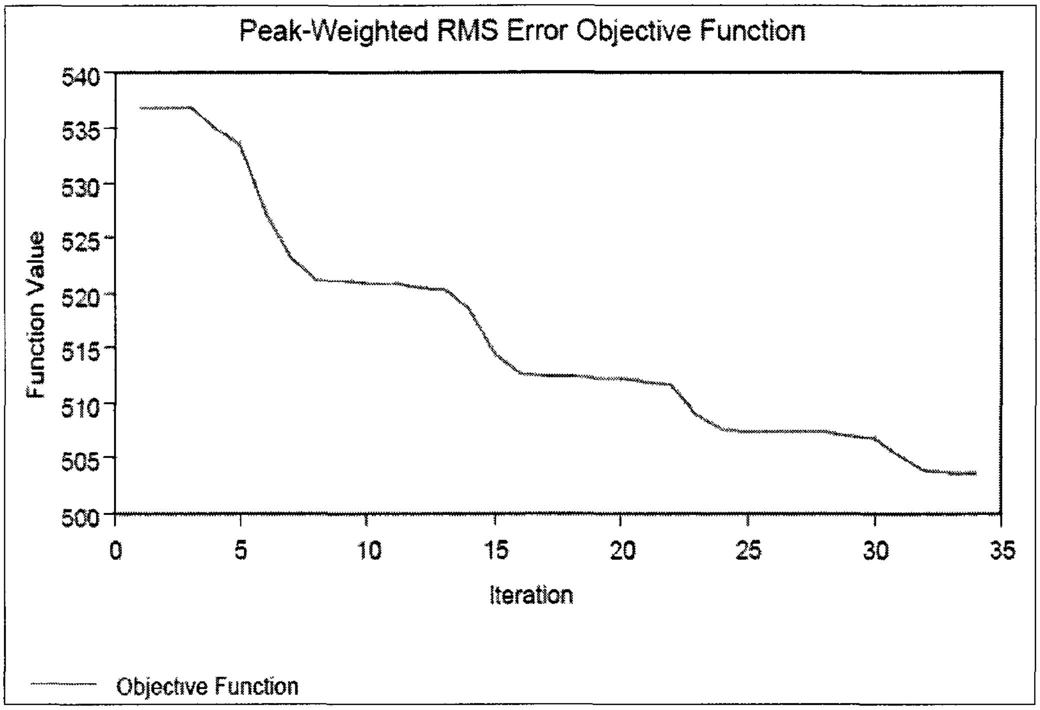

Figure 53: Objective Function Graph indicating the peak-weighted root mean square error after the iteration between the observed and simulated hydrographs for the estimated parameter

The flow comparison chart provides a graphic presentation of the simulated flow plotted against the observed flow (Figure 54). If equal, it should plot at a straight 45° line. The amount of scatter helps indicate the quality of the parameter estimation. At flows higher than 6,500 cfs the simulated and observed flows are practically equal. When the plotting falls in line, from 6,800± cfs to 7,500± cfs, the selected parameter in the model has been predicted exactly the same as the observed ordinate. Red and blue data points are comparisons before and after the peak respectively. Data points above the 45° line represent ordinates that are over predicted by the model and likewise those plotted below are under predicted. Scatter in the hydrograph comparisons before and after the flow rate of 3,500± cfs is caused by the rise and fall of the simulated hydrograph. The start and end of the objective function did not provide a matched flow rate between the models because of the time period selected and the removal of baseflow from the observed hydrograph. Removal of the baseflow from the observed hydrograph established flow rates of 0.0 cfs at the end of the simulation period.

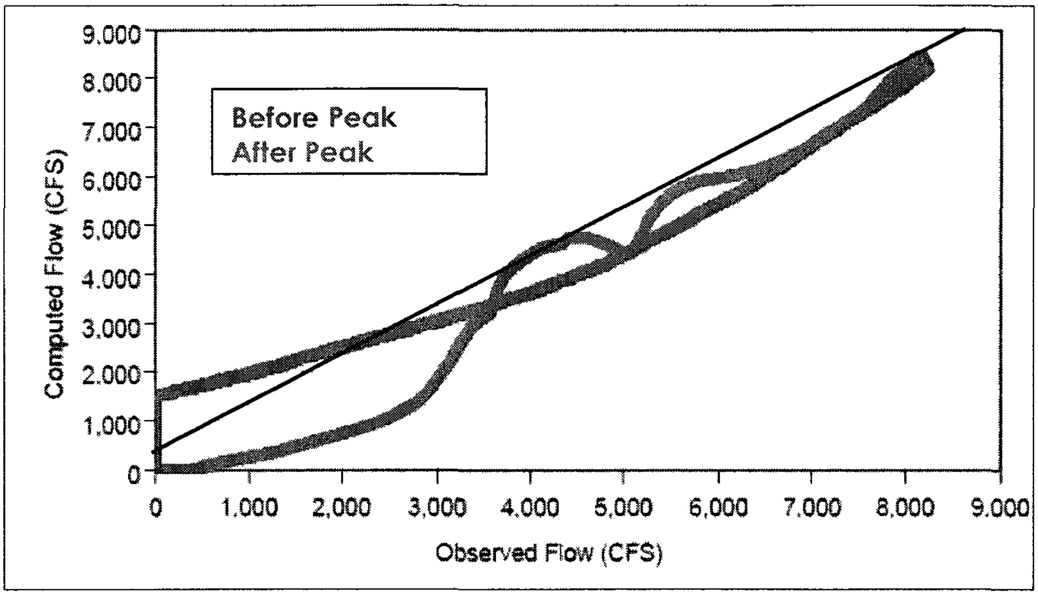

Figure 54: Flow Comparison Graph plotting the simulated flow against the observed (April 2007) flow

The differences between the simulated and observed hydrograph for each time step is provided on the flow residuals graph (Figure 55). The magnitude of the residuals indicates the quality of the parameter estimation and also where there are biases in agreement between the two hydrographs. The objective function start and end labeled on the figure is the defined time frame for a goodness-of-fit between the observed streamflow and computed hydrograph of the parameter being optimized. The bias at the beginning, near -1,500 cfs, is due to the slight difference in the location of the rising limb of the hydrograph (Figure 56). During the center time period between the beginning and end of the objective function, the differences vary by 500 cfs and cross between the simulated being greater or less than the observed. At the end the bias is again due to the fact that the observed hydrograph has a flow rate of 0.0 and the simulated is generating a declining limb for a UH.

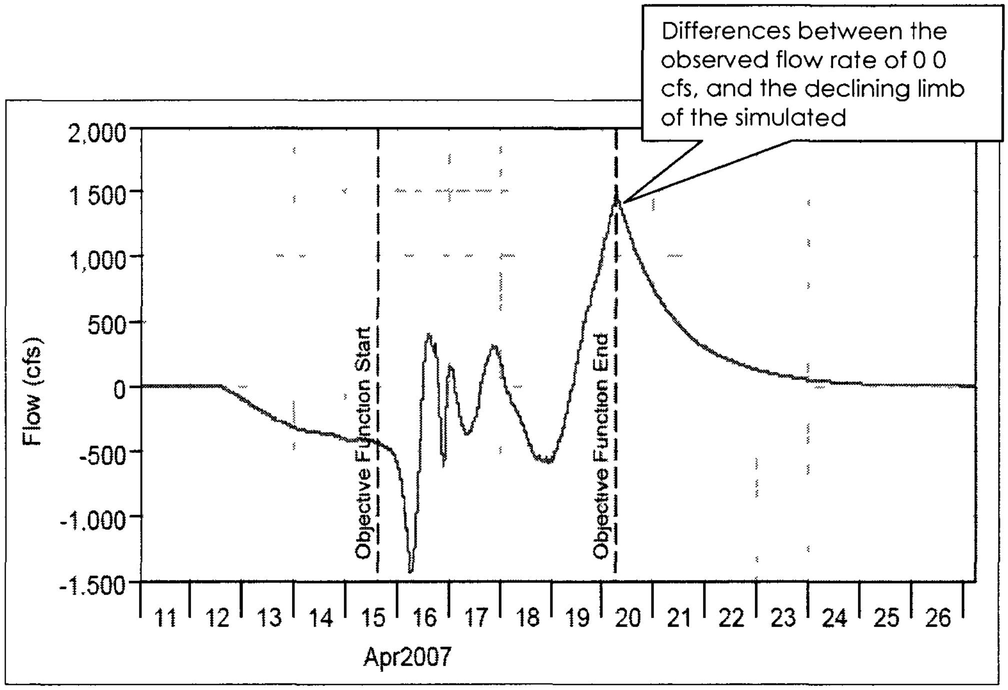

Figure 55: Flow Residuals

After assigning a Manning's n of 0.12 in the upstream reaches through Epping and Raymond, because of the numerous log jams, the optimization trial resulted with sub basin lag times within the range of accuracy accepted by this research. The final optimized lag times simulated for the eight sub basins, upstream of the USGS gage, resulted with a discharge hydrograph (Figure 56) comparable to the observed runoff volume, peak flow, time to peak discharge, and time to center of mass observed at the USGS gage near Newmarket (HMS junction J1271) for the April 2007 event. The resulting trial provided a simulated runoff volume of 4.35 inches and peak flow of 8,439 cfs compared to the observed of 4.29 inches and 8,223 cfs respectively (Table 24).

Table 24: Optimization results of simulation to determine suitable parameter value

April 2007 gage discharge, less baseflow from concave method				
Measure	Simulated	Observed	Difference	% Difference
Runoff volume (IN)	4.36	4.29	0.07	1.63
Peak flow (CFS)	8332	8223	109	1.32
Time of Peak	17Apr2007, 23:02	18Apr2007, 01:45	2 hr 43 min	0.11
Time of Center of Mass	18Apr2007, 08:16	17Apr2007, 18:59	13 hr 15 min	0.52
Optimized parameter results				
Sub basin	Parameter	Unit	Initial Value	Final Value
W6510	SCS Lag	MIN	277	1,396
W8600	SCS Lag	MIN	443	1,380
W11900	SCS Lag	MIN	542	1,873
W10910	SCS Lag	MIN	204	450
W8380	SCS Lag	MIN	193	830
W11020	SCS Lag	MIN	386	446
W6730	SCS Lag	MIN	489	2,205
W7060	SCS Lag	MIN	660	666

These results followed more than two dozen trials that included estimations for sub basin loss parameters (CN, initial abstraction), sub basin transform parameters (lag time), and reach routing parameters (Muskingum X, Y, subreaches and Manning's n) that were set to match one of the observed events (May 2006, April 2007, March 2010).

The accepted results of the optimization trials do not provide the only solution but it is unique in the sense that all the estimated and assigned parameters produce on specific hydrograph.

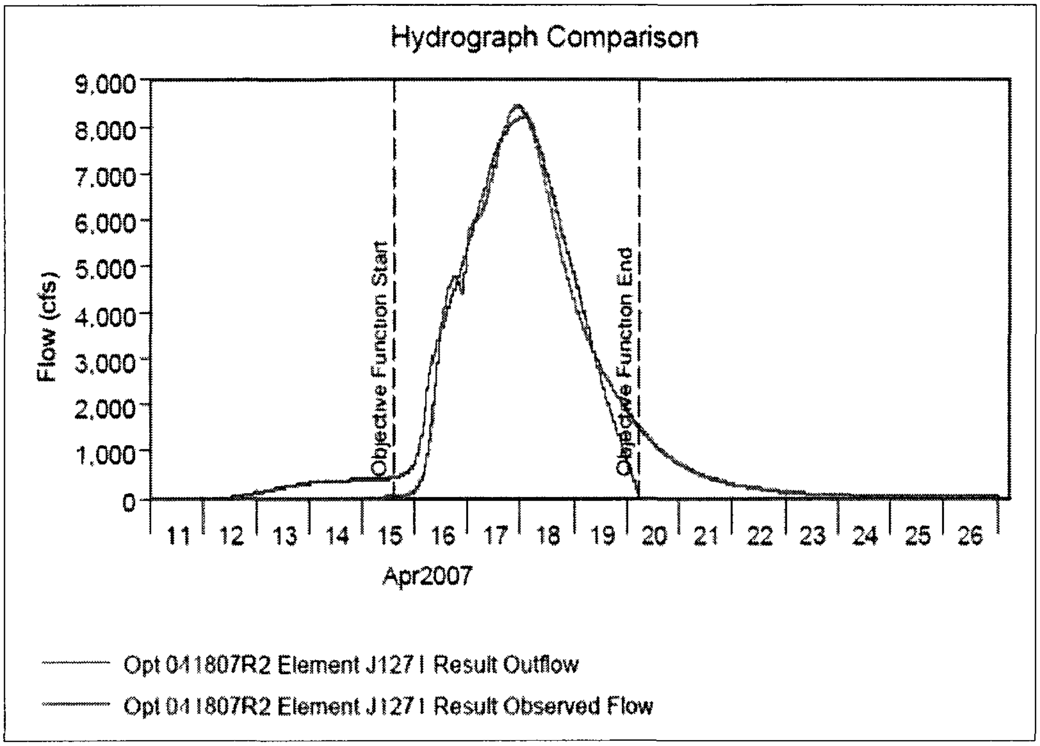

Figure 56: Second simulated hydrograph with sub basin W7060 lag time adjustment

4.2.3 *Steady flow data*

The calibrated model generated the 100-yr flows for each sub basin using a meteorologic model from the two rainfall atlases (TP-40 and NRCC). Figure 57 and Figure 58 provide the delineation of the defined sub basins in the Lamprey River and Oyster River bypass respectively. Running the calibrated HEC-HMS model with 24-hour, 100-year design storm of 6.3 inches and 8.5 inches provided flows for a historic (TP-40 atlas) and current (NRCC atlas), respectively.

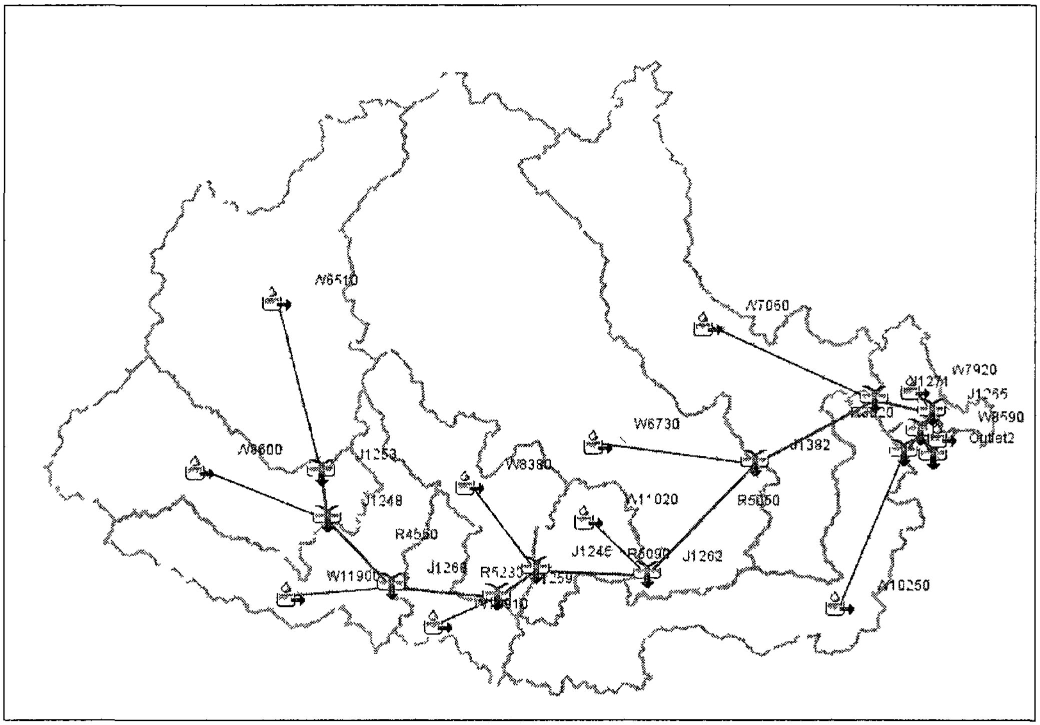

Figure 57: Lamprey River sub basin schematic of the hydrologic elements: watershed catchment, stream reach, and confluence.

To complete the list of project scenarios evaluated (Table 13), the model's CN values were adjusted to the 2050 conventional and 2050 LID build-out conditions and ran with the 24-hour, 100-year design storm depth of 8.5 inches for future conditions. Table 25 provides the hydrology used to model steady flow flood flows in the hydraulic program for the river reaches. Beaudette Brook, Bedford Brook, Hamil Brook, and Longmarsh Brook are in the Oyster River watershed.

The FIS column is duplicated from the summary discharges (Table 4 in both the Rockingham and Strafford County FIS). Recall these discharges were developed from an annual peak flow frequency analysis to determine the 100-year flood flow. There were not any discharges listed in the FIS for the tributaries into Hamil Brook (Oyster River watershed). To run the hydraulic model, hydrology from the 2005 TP-40 analysis were applied at these reach stations. A rainfall runoff analysis generated the remaining columns of steady flows. In comparing the FIS and 2005 TP-40 flow rates, the results of the 6.3 in/24 hour design storm provides similar flow rates. If the FIS 100-year discharge rates were developed with a rainfall runoff analysis, then the 6.3 in/24 hour design storm would have been applied at that time.

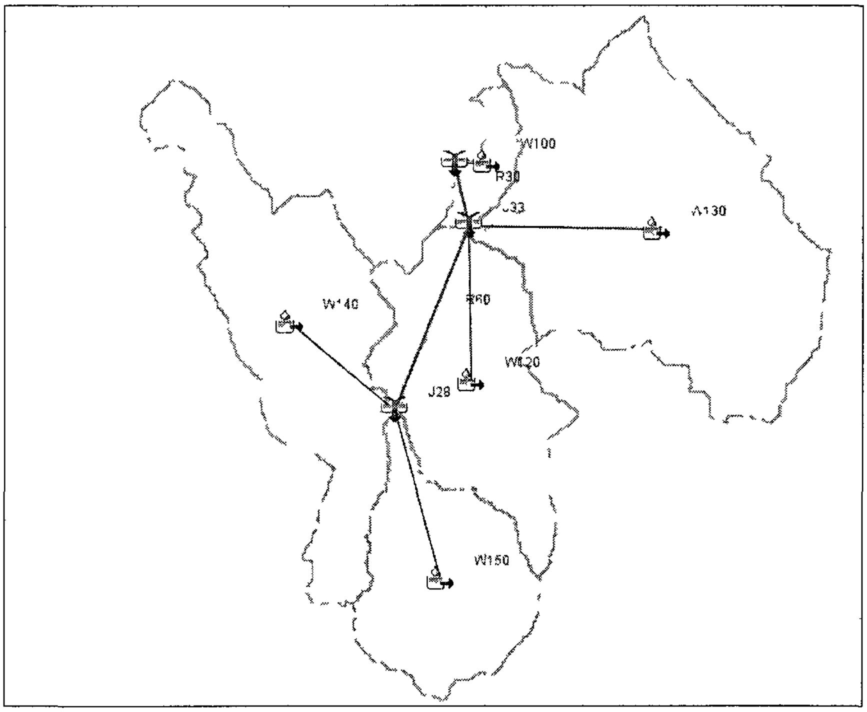

Figure 58: Oyster River bypass sub basin schematic of the hydrologic elements: watershed catchment, stream reach, and confluence.

The difference between the 2005 TP-40 and 2005 NRCC is the applied rainfall. The physical characteristics and other parameters of the sub basins are the same for each model. The average increase in flood flow between the two models is 47.9% with the largest increase of 69.5% for the sub basin at the Pisscassic River confluence (HEC-RAS station 5568) (Figure 59). The smallest increase in flood flow happens at HEC-RAS station 6377. This is where the flow splits between the main reach and Oyster River bypass and is set at an initial estimated percent bypass therefore not dependent on the increased difference.

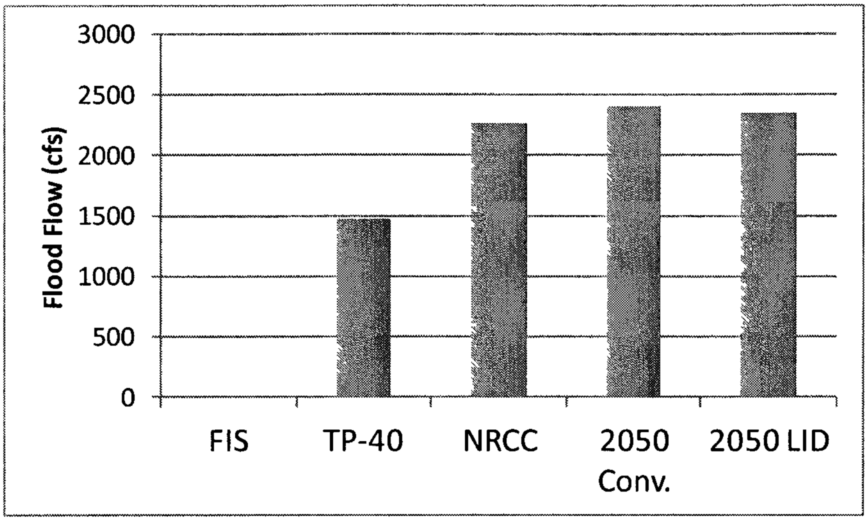

Figure 59: Change in flood flows for the Piscassic River (not included in FIS)

The last two columns are for the future build-out scenarios with conventional and LID development applications. The watersheds for the tributaries in the Oyster River watershed were not included in the build-out. Hydrology is the same as the 2005 NRCC event. Changes only occur in flow rates because of the estimated bypass into this watershed. In comparing the 2005 NRCC values to the 2050 conventional build-out for the Lamprey River, the average increase in flow ranges from 4.0 to 4.7%. The variation in the CN value is the only parameter adjustment in the analysis since the 24-hour, 100-year design storm of 8.5 inches was used for future conditions. There is a limited variation in hydrology at the watershed scale.

In comparing the hydrology values for the 2050 conventional and LID build-outs, the decrease in flood flow is an average of 1.4%. Figure 60 provides changes seen in the vicinity of the USGS gage near Newmarket. The increase between two rainfall depths (TP-40, NRCC) for the current (2005) scenario is much more significant than the difference between the current NRCC flow and the two future development scenarios. At the watershed scale, the LID implementation cannot significantly adjust the hydrology for flood events. This is similar to the adjustments seen in the CN values at the watershed scale and attributed to the same reasoning. At the urban scale, where the type of development application is more influential, the decrease in peak discharge (Q) is an average of 28.3% (Table 23).

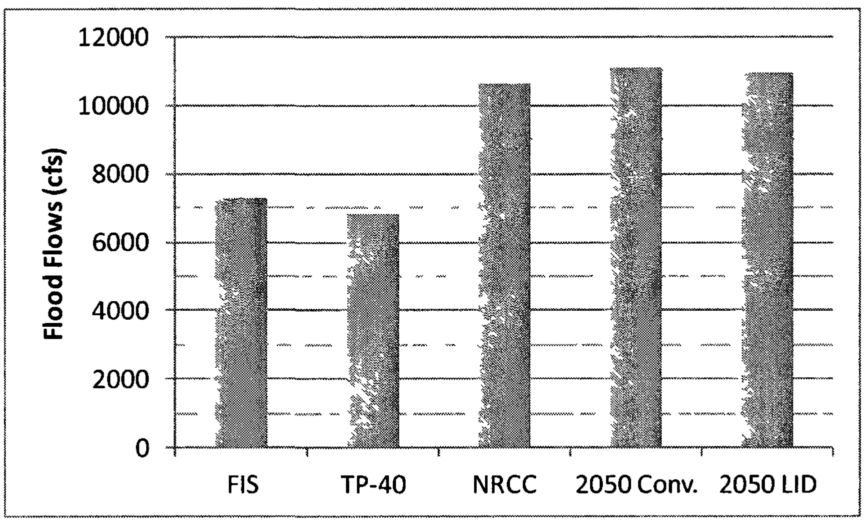

Figure 60: Change in flood flows at USGS gage near Newmarket

Table 25: Hydrology data for respective sub basins input into the HEC-RAS steady flow file

River Name	Reach Name	HEC-RAS River Sta. No.	FIS (cfs)	2005 TP-40 (cfs)	2005 NRCC (cfs)	2050 Conv. (cfs)	2050 LID (cfs)
Beaudette Brook	Headwaters	1319	NA	163	249	249	249
Beaudette Brook	DS RT108 FP	465	NA	2,848*	3,747*	2,669*	2,645*
Beford Brook	Headwaters	660	NA	138	207	207	207
Ellison Brook	Headwaters	1960	NA	126	187	187	187
Hamil Brook	Headwaters	2157	NA	350	526	526	526
Hamil Brook	DS Longmarsh	1263	1,300	3,461*	4,667*	3,589*	3,565*
Lamprey	RT101, Raymond	190667	3,300	2,082	3,214	3,342	3,304
Lamprey	Langford Road, Raymond	178854	4,370	3,328	5,138	5,370	5,296
Lamprey	Raymond Corp. limits	160646	5,290	4,074	6,278	6,568	6,479
Lamprey	Epping west corp. limits	136639	4,180	4,000	6,172	6,459	6,371
Lamprey	Blake Road, Epping	126647	4,720	4,316	6,708	7,017	6,923
Lamprey	RT125, Epping	111088	4,930	4,312	6,708	7,018	6,923
Lamprey	Epping north corp. limits	67214	5,600	6,730	10,437	10,882	10,713
Lamprey	USGS gage	19842	7,300	6,829	10,649	11,109	10,952
Lamprey	Newmarket north corp. limits	8890	6,000	5,463	9,158	11,109	10,952
Lamprey	Piscassic confluence	5568	6,000	6,239	10,576	12,643	12,438
LaRoche Brook	Headwaters	2967	NA	1,193	1,820	1,899	1,881
Longmarsh Brook	Headwaters	4232	NA	126	187	187	187
Longmarsh Brook	DS Beaudette	3012	NA	2,974*	3,933*	2,856*	2,832*
Longmarsh Brook	DS Bedford	2785	NA	3,112*	4,141*	3,063*	3,040*
Piscassic	Headwaters	3490	NA	1,479	2,257	2,406	2,344
RT108_FLDPLN	DS Lamprey Junc	6377	NA	1,366*	1,491*	334*	329*
RT108_FLDPLN	DS Elison_LaRoche	3008	NA	2,685*	3,498*	2,420*	2,396*

NA: Not available *% Includes initial estimate of flow leaving the main river

4.3 **Hydraulic Models**

4.3.1 ***FIS***

The FIS model could not be duplicated by reason of: missing FEMA file data for Rockingham County inline structures, lack of inclusion of the reach through Lee, changes in hydraulic software, FIS modeling of bypass flows to the Oyster River watershed, deficient steady flow data for tributaries. Hydraulic information regarding the BFEs recorded in the FIS and displayed on the FIRMs was used for comparison to the calculated water surface elevations of the current and future scenarios. The FIS flood profiles are provided in Appendix F. Discrepancies between the FIS and revised model were not unexpected for the reasons previously listed. This is not uncommon with model revisions.

4.3.2 ***2005 TP-40***

While the models were not duplicates, the modeled water surface profile for the 2005 TP-40 condition was similar to the FIS. A portion of the FIS and 2005 TP-40 energy grade lines (EGLs) through Epping is plotted as a longitudinal profile (Figure 61). There are six bridges and one in-line structure, Bunker Pond dam, along this 13± mile stretch. The EGL switches between the two models in several locations. One of those areas is near the bridges between station 106,000 and 108,000 (Figure 62). In this section of the River the FIS flood flow is 4,720 cfs and the calculated TP-40 flood flow is 4,316 cfs.

The EGL for Longmarsh Brook to Hamil Brook is along the RT108 corridor (Figure 63). Any significant change to the slope of the line or increase/decrease in elevation is likely due to the different modeling techniques of the FIS (WSP2) and HEC-RAS software. The EGL and the water surface elevation (WSE) for all the profiles are equal due to the slow velocity and resulting minimal velocity head between HEC-RAS cross sections. There is a greater increase in the EGL for Hamil Brook because HEC-RAS was able to accurately model the bypass into the Oyster River watershed.

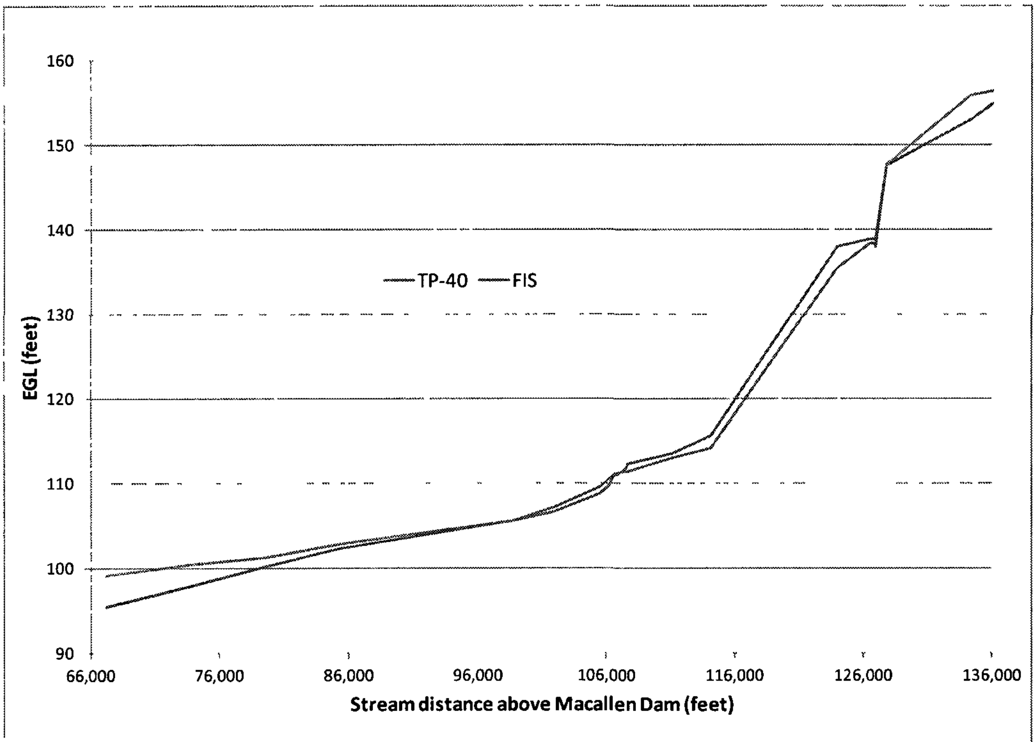

Figure 61: Lamprey River EGL longitudinal profile through Epping for FIS and 2005 TP-40

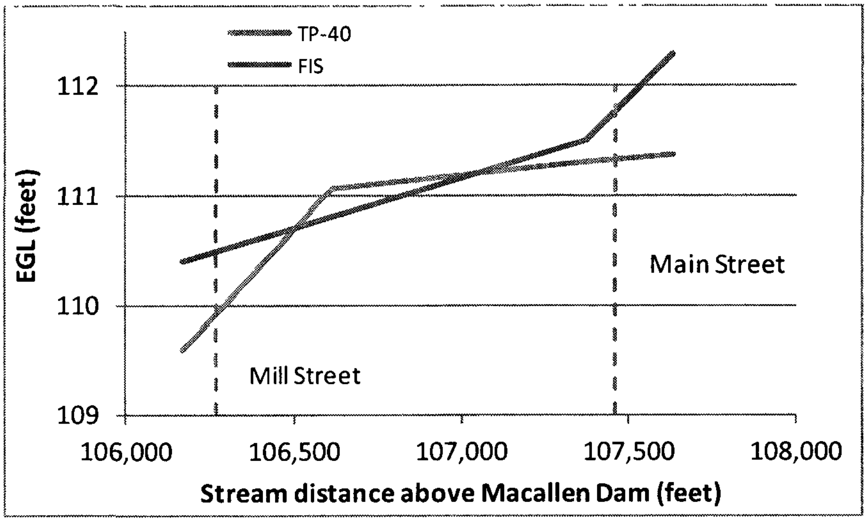

Figure 62: Lamprey River EGL profile between Main Street and Mill Street bridges for FIS and 2005 TP-40

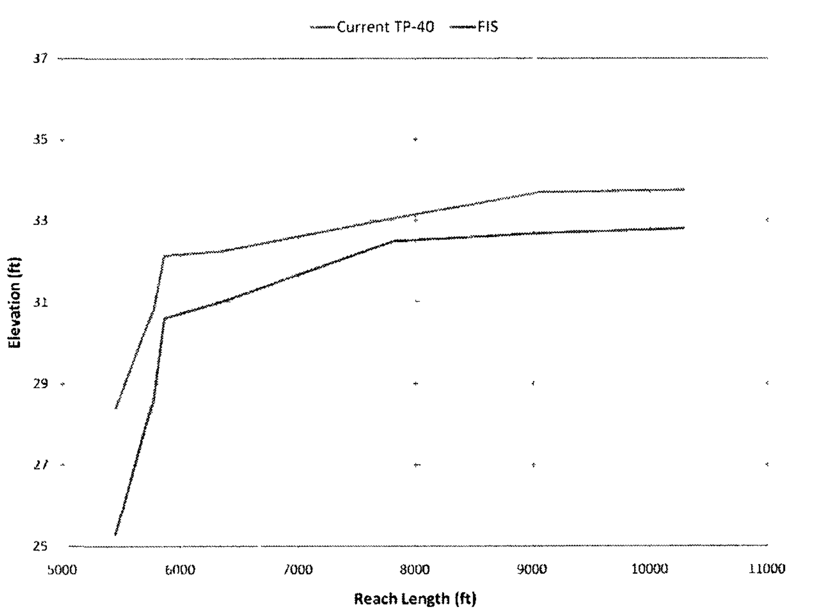

Figure 63: Longmarsh Brook to Hamil Brook energy grade line longitudinal profile for FIS and 2005 TP-40

4.3.3 ***NRCC Current***

Running the 2005 TP-40 model in contrast with the 2005 NRCC model indicates that the current water surface profile increases an average of 2.7 feet along the length of the study. The additional rainfall depth of 2.1 inches (8.5 in –

6.3 in) results with this significant water surface elevation increase. This has serious implications on the bridge structures. These were likely designed to pass the 100-year flood flow based on the historic TP-40 rainfall depth or the FIS study with one-foot of freeboard. A 2.7 foot increase now results with backwater, pressure flow, or topping of the structures.

The slope of the energy grade line is relatively consistent between the FIS and NRCC models. In most sections the flow is uniform and occasionally gradually varied. The channel slope is very mild: normal depth is greater than critical depth; and the friction slope is equal to the channel slope. This is an important comparison as the cross section geometry and structures are replicated from the FIS backup data. Any significant difference may indicate data entry errors in the HEC-RAS program.

A notable increase in the water surface elevation arises where bridges cause a restriction and backwater occurs as well as overtopping (Figure 64 and Figure 66). The bridge stationing provided in Table 16 correlates to the stationing along the horizontal axis in the following figures. In Epping, there is a four (4) to five (5) foot increase in the base flood elevation upstream of Blake Road (station 123964) and Figure 65 shows the three (3) foot increase upstream of Mill Street (station 106269). The base flood elevation increases by more than six feet upstream of the Lee corporate limits (station 67214). Since the community of Lee did not have a published FIS, the FIS EGL profile was not generated for an eight mile stretch of the river. This break in the FIS flood profile eliminated Macallen Dam as the downstream boundary condition for the Rockingham County FIS. The 2005 NRCC analysis provides increased accuracy because of the inclusion of the reach through Lee. Upstream of Packer's Falls Road in Durham (16028), the base flood elevation increases by more than seven (7) feet. At this location the flood flow is increased from 7,300 to 10,649 cfs between the FIS and 2005 NRCC hydrologic models. The base flood elevation, along the impounded still water reach upstream of Macallen Dam, increases by approximately four (4) feet.

The updated flood flow and hydraulic model for the Oyster River bypass generated a base flood elevation increase of three (3) to six (6) feet along the Longmarsh Brook to Hamil Brook reach (Figure 66).

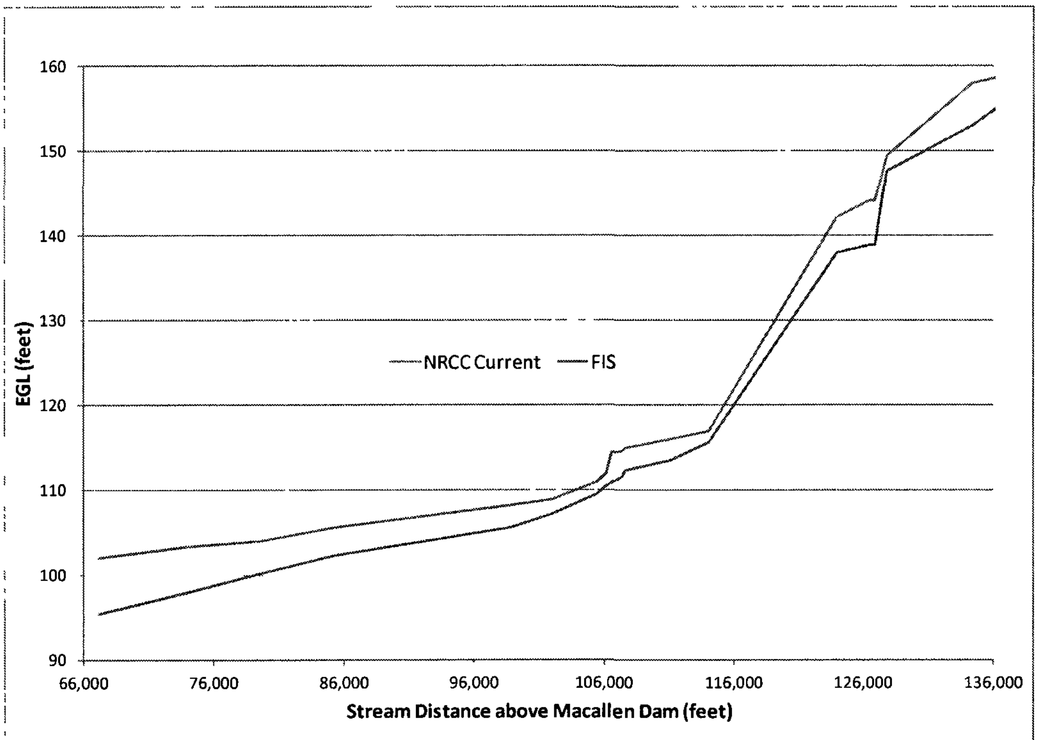

Figure 64: Lamprey River EGL longitudinal profile through Epping for FIS and 2005 NRCC

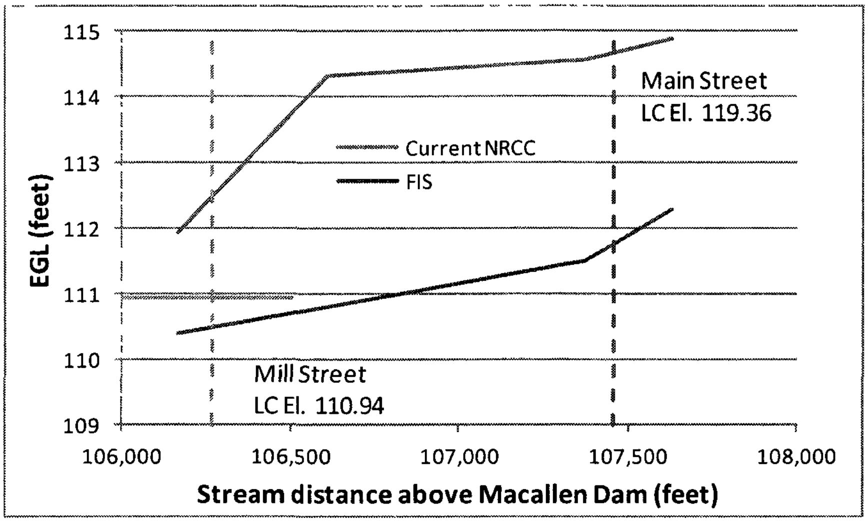

Figure 65: Lamprey River EGL profile between Main Street and Mill Street bridges for FIS and 2005 NRCC. Low chord (LC) elevations of bridges are from as-built drawings.

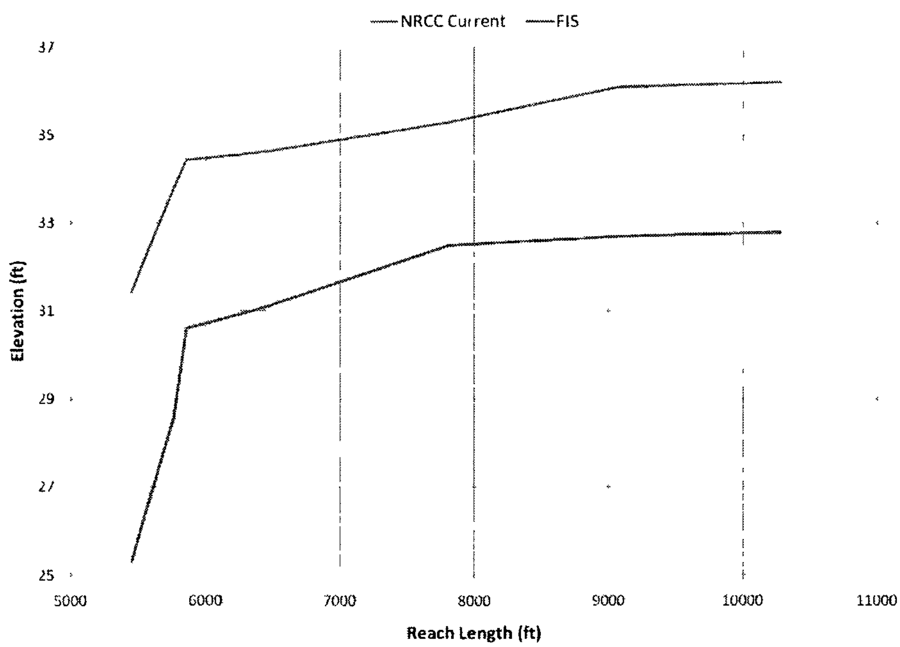

Figure 66: Hamil Brook energy grade line longitudinal profile created for FIS and 2005 NRCC

4.3.4 ***2050 NRCC Conventional Build-out***

Except for the headwater reach upstream of station 181300, the conventional build-out condition increases the WSEs in comparison with the FIS flood profile for the Lamprey River. The average WSE difference between the 2050 conventional build-out and FIS cross sections is 3.0 feet (8.0 feet maximum, -0.4 feet minimum, and 2.24 feet σ).

The 2050 NRCC conventional build-out and the 2005 NRCC models shows an increase in the WSE an average of 0.30 feet (0.31 feet maximum, -0.19 feet minimum, and 0.23 feet σ) along the study length. Figure 67 shows a portion of the reach through Epping from upstream of Blake Road (station 123,964) to downstream of Mill Street (station 106629). The flood flows have increased from 6,708 cfs to 7,017 cfs between the compared scenarios. Even though the conventional build-out scenario does not significantly increase the water surface elevation, a 0.3 foot increase is significant in accordance to the design elements that must be met for bridge/structure replacements on regulated watercourses. FEMA requires that the existing base flood elevations must not be raised any greater than 0.01 feet when a new structure is proposed. This minor change in their hydraulic design requirements cannot be met if the increase in flood flows is not held to a minimum.

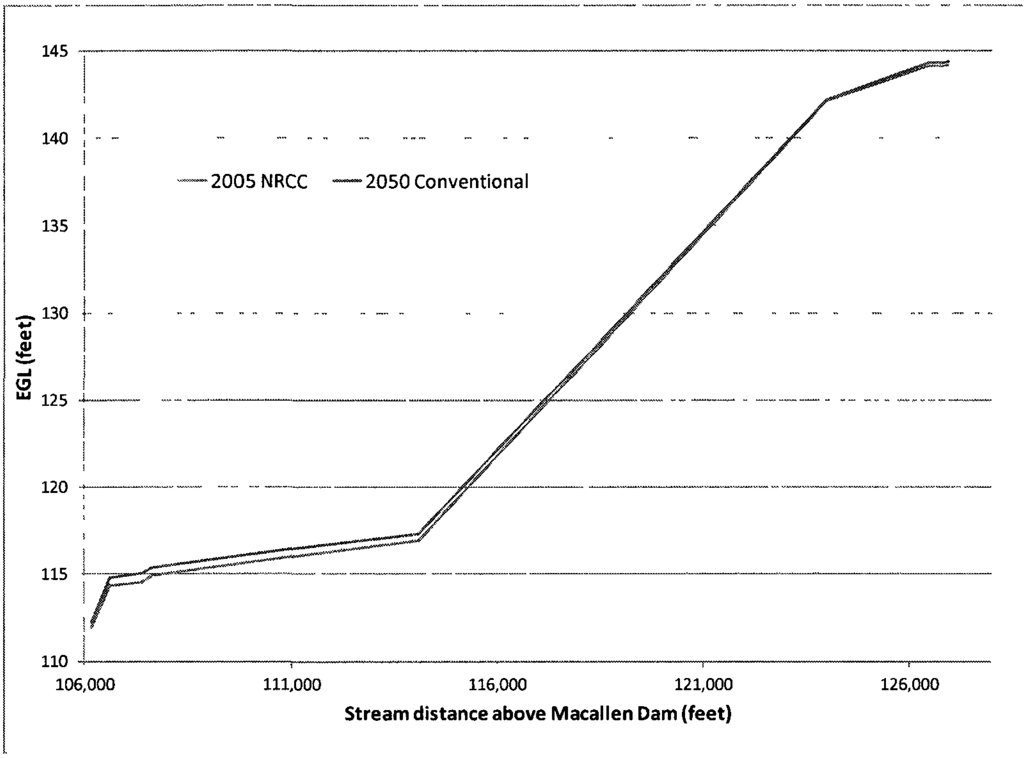

Figure 67: Lamprey River energy grade line longitudinal profile for FIS and 2050 NRCC Conventional

4.3.5 ***2050 NRCC LID***

The flood profile for the 2050 NRCC LID model decreases the WSE minimally in comparison to the 2050 NRCC conventional model. The average difference in elevations between the conventional and LID build-out is 0.11feet (0.29 feet maximum, 0.02 feet minimum, and 0.06 feet σ).

Figure 68 shows this change from upstream of Main Street (station 107459) to downstream of Mill Street (station 106269) in Epping. Appendix F – FIS Flood Profilesincludes summary tables of the entire longitudinal profile.

Although this seems like a minor improvement at a full watershed scale, the changes are resolved clearly at the smaller sub-catchments scale. The three urban sub basins presented in 4.2.1(c) illustrate a decrease in flood flows by implementing LID. Mapping the spatial extent of all the flood flow scenarios was not possible due to the accuracy of the digital elevation model (DEM) used in ArcMap. Future research will make use of a DEM that will provide the accuracy needed to import the georeferenced HEC-RAS data back into ArcMap.

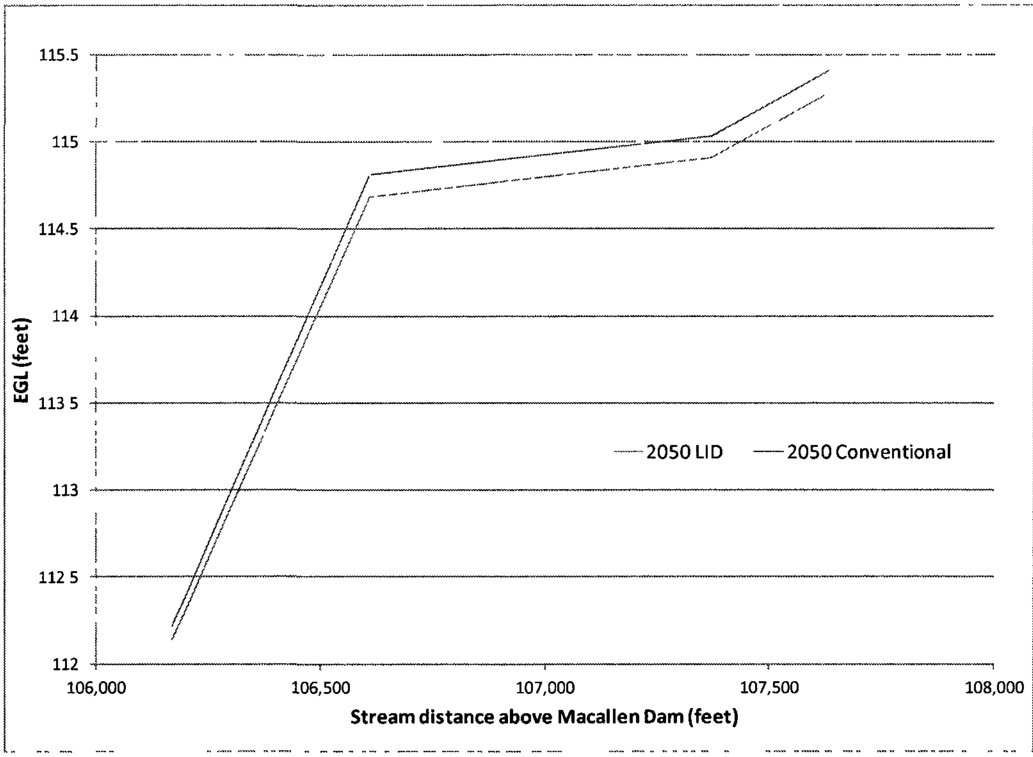

Figure 68: Lamprey River energy grade line longitudinal profile for 2050 NRCC Conventional and 2050 NRCC LID

WSEs at the bridge structures are an important element in planning weir and low chord elevations for new structures. There was an average increase in the WSE from 2005 to 2050 conventional of 0.7% with the highest increase at Packer's Falls Road of 2.23 feet. Implementing LID lowered the increase to 0.6%.

Table 26 illustrates WSEs for the 2005 NRCC, 2050 NRCC Conventional, and 2050 NRCC LID for a list of the bridges affected by the increase in flow. The FIS elevation is for the nearest FIS cross section located upstream of bridge structures and is provided as a reference but not necessarily for comparison. Of the twenty (20) structures modeled along the Lamprey River for the NRCC 2005 condition, thirteen (13) bridges create a constriction and backwater occurs and Main Street in Raymond is overtopped. The future build-out generates flows that overtop Mill Street in Epping as well as Main Street in Raymond.

Table 26: Upstream WSE at bridge structures

Bridge	WSE Upstream (NAVD27)				Deck Elevations	
	FIS	2005	2050 Conv.	2050 LID	High Chord	Low Chord
Raymond Road (RT 27)	211.7	210.33	211.33	211.68	213.5	207.7
Langford Road	195.8	198.51	198.97	198.82	200.9	194.8
Main Street	190.8	192.84	193.01	192.93	191.6	185.5
Epping Street	190.1	191.73	191.97	191.74	192.3	185.3
Prescott Road	166.3	167.99	168.39	168.27	170.5	166
Epping Road (RT 27)	153	150.48	150.48	150.51	154.51	148.68
Blake Road	138	142.14	142.18	142.16	143	138.7
Main Street (Plummer)	112.3	114.51	114.99	114.87	127.63	122.3
Mill Street	110.8	114.12	114.62	114.5	114.32	111.03
Hedding Road (RT 87)	105.6	106.6	107.15	107.04	110.64	104.25
Wadleigh Falls Road		98.9	99.27	99.13	100.5	97.73
Lee Hook Road		80.74	81.18	81.01	82.4	78.7
Wiswall Road	62.7	63.44	63.65	63.56	66.5	63.38
Packer's Falls Road	53.4	58.38	60.61	60.07	58	54.4

Another important element in planning is the velocity at these structures. The reaction to increased velocity is scour. Table 27 provides a comparison between the cross section velocities recorded in the FIS to those calculated by HEC-RAS at the same sections. A majority of the cross sections result with decreased velocity except for FIS section P near Blake Road in Epping. Since structural data was missing from the FEMA backup, it is impossible to verify the structure geometry used in the FIS. The HEC-RAS model includes in-stream remains of an historic structure that decreases the flow area at this section. The former comparison of WSEs can also provide an indication of the increase in flow area at these sections. Additional flow area would decrease the rate of velocity. The HEC-RAS detailed bridge output tables are provided in Appendix G.

Table 27: Cross section velocity comparisons

FIS (HEC-RAS)	Nearest bridge	Average velocity of flow in total cross section			
		FIS	2005	2050 Conv.	2050 LID
AR (181380)	Dudley Road	2.10	1.52	1.50	1.51
AQ (181044)	Raymond Road (RT 27)	1.45	0.97	0.95	0.95
AK (168069)	Langford Road	3.10	2.07	2.07	2.07
AH (160646)	Main Street	1.98	1.47	1.50	1.51
AG (155206)	Epping Street	1.21	0.93	0.95	0.96
AE (154211)	B&M Railroad	4.14	3.25	3.30	3,28
Z (147373)	Freetown Road (RT 107)	2.50	1.60	1.59	1,59
X (141318)	Prescott Road	4.00	3.98	4.03	4.02
V (136639)	State Route 101	3.30	2.30	2.34	2,33
T (127810)	Epping Road (RT 27)	2.30	2.70	2.78	2.76
P (123979)	Blake Road	3.40	8.75	9.12	9.01
M (107630)	Main Street (Plummer)	4.30	2.95	2.85	2.87
K (106610)	Mill Street	3.40	2.68	2.63	2.63
I (105450)	Calef Hwy (RT 125)	4.60	2.61	2.58	2.58
F (88195)	Hedding Road (RT 87)	1.30	4.75	4.97	4.90
61492	Wadleigh Falls Road	Not available	5.30	5.36	5.34
35780	Lee Hook Road	Not available	3.96	3.92	3.94
M (20163)	Wiswall Road	2.50	2.60	2.62	2.62
G (16117)	Packer's Falls Road	4.70	2.84	2.80	2.81
1602	Newmarket Road (RT 108)	Not available	10.59	11.44	11.23

Chapter 5

Summary, Conclusions, and Recommendations

5.1 Land use conditions

5.1.1 *Current*

The Lamprey River watershed is the largest watershed that drains into the Great Bay estuary. There has been considerable focus on maintaining and in some places improving the water quantity and quality discharged into the Lamprey River. Analysis if GIS data shows that in 1962, 5,098 acres, or 3.7%, of the land had been converted into residential, commercial, industrial, transportation, or developed land use. In 2005, 18,910 acres, or 13.9%, of the land has been converted which represents an increase of 270% since1962.

5.1.2 *Future build-out condition of the watershed*

In 1960, the population of the 15 communities completely or partially within the watershed was 28,915. The latest 2010 census recorded 98,990. This is a 242% increase in population. To build out the watershed for 2050 conditions, a fixed rate of change, based on the Rockingham and Southern New Hampshire Planning Commission's growth data projection for two time periods, was applied to ideal land available for development and redevelopment (Table 7). By 2050 this build-out resulted in 26,752 acres, or 19.6%, of the land will be converted from to residential and commercial/industrial uses.

5.2 Comparison of results to previous studies

5.2.1 *Hydrology*

The purpose for this research was to reassess the hydrology of the Lamprey River watershed in response to changes in land use and climate. This reassessment provides a more accurate reflection of existing conditions compared to the results presented in the Flood Insurance Study.

At the USGS gage location near Newmarket, the FIS 100-year discharge for the 183 square mile watershed upstream is 7,300 cfs (FEMA 2005). This flood flow was verified using the annual peak discharges for the years 1935 through 1987 as the input file for the USGS Office of Surface Water software program, *Peak flow Frequency analysis program* (PKFQWin). FEMA's *Map Modernization* criterion for revaluation was applied using annual peak discharges for the years 1935 through 2009. The updated analysis performed with PKFQWIN resulted with a significant change to the FIS flood flows (Figure 4) since the computed 9,411 cfs falls outside of the upper 68-percent confidence interval for the FIS flood flow of 7,300 cfs;

$$L_{0.01,0.68} = 6{,}886 \text{ and } H_{0.01,0.68} = 7{,}834.$$

New analyses of annual peak discharges included the last 30 years (1980 through 2009) in order to evaluate the impact of changes in the magnitude and frequency of extreme precipitation events and land use on predicted discharge at a 100-year recurrence interval. Olson's evaluation of stream gages was based on the most recent 20 years (Olson 2009). A thirty year data set includes ten (10) of the largest fifteen (15) annual peak events that have been recorded since 1934. The analysis concluded that the new 100-year flood flow is 13,770 cfs. The details for the PKFQWin calculations are provided in Appendix B.

The hydrologic model performs a rainfall-runoff analysis in which an element within a watershed generates infiltration and surface runoff. Rainfall-runoff is a loss method which equates the sum of infiltration, initial abstraction, and excess precipitation left on the surface equal to the total precipitation. Parameters from the watershed including sub basin area, lag time and composite CN (losses), and reach routing are used along with a depth of precipitation to generate a peak discharge from a dimensionless generic unit hydrograph.

For the hydrological model, two different approaches are used to provide rainfall data. For historical and current scenarios, the hydrological model uses the TP-40 and NRCC rainfall frequency atlases respectively to determine peak runoff. All references to 2005 TP-40 means 2005 land use conditions and 6.3 inches 24-hour, 100-year rainfall depth. All references to 2005 NRCC means 2005 land use and 8.5 inches for the 24-hour, 100-year rainfall depth.

The TP-40 rainfall depth was used to duplicate the FIS hydrology. The 24-hour, 100-year design storm depth of 8.5 inches from the NRCC atlas was used to determine current conditions. FEMA requires current depth-duration-frequency data for updating studies. *Rainfall depths for the middle of the century (2035-*2064) were projected by downscaling output from four different Atmosphere-Ocean General Circulation Models (AOGCMs) (Wake et al. 2011) using two different emission scenarios (B1 scenario based on stabilizing atmospheric CO2 concentrations; A1f1 scenario based on increasing higher CO2 concentrations).

Comparison of the downscaled model simulations with the NRCC Atlas results indicated that existing Atlas results of a rainfall depth of 8.5 inches is a reasonable approximation of future rainfall depths.

The analysis of the 2005 TP-40 model resulted with a 100-year flood flow of 6,829 cfs at the gaged location. There is a 6.9% difference compared with the FIS peak annual flood frequency analysis of 7,300 cfs.

The entire Lamprey River watershed consists of eleven (11) sub basins (Figure 10). Composite CNs were determined for each sub basin based on the area, land use and hydrologic soil group for current and the 2050 conventional and LID build-out conditions. Limited variations of CN values were observed between the condition scenarios at the watershed scale. In comparing the current 2005 CN values to the 2050 conventional build-out, the range of overall CN values increased by the least amount in sub basin W8380 by 1.9 and by the greatest amount in sub basin W10910 by 6.1. In comparing the 2050 conventional and LID build-outs, the overall CN values were adjusted by the least amount in sub basin W8380 by 0.5 and by the greatest in sub basin W10910 by 2.0. Table 21 provides the comparison of these results. Table 25 lists the resulting changes in flood flows for these scenarios. At the watershed scale, the LID implementation cannot significantly adjust the hydrology for flood events. This is similar to the adjustments seen in the CN values at the watershed scale and attributed to the fact that about 45% or 60 of the 213 square mile watershed remains forested, agricultural or other open space after the build-out. Because influence of LID is minimal below 3 – 7% impervious cover, most of the 15 community's requirement for at least a two (2) acre lot for a single residential use generates an insubstantial increase in runoff. Additionally, these same communities have 5-15% of land area protected from future development and approximately 13% of the watershed is covered with wetlands and open water.

At the urban subwatershed scale, there was a clear indication that implementing LID can minimize the impact of development. The three small subwatersheds analyzed are tributaries to the Lamprey River and in Zone A on the Flood Insurance Rate Map (FIRM) for Rockingham County. The data provided in Table 23 indicates that if future development employs LID, the CN number and the resulting hydrology will not significantly change in these smaller urban watersheds. The results also show that redevelopment of commercial and industrial properties with LID can provide an even stronger advantage by decreasing curve numbers to less than currently experienced. In comparing future LID to from the current (2005) conditions, there was a decrease in the CN of 0.6 (0.9%), runoff of 0.1 in. (2%), and peak discharge of 15 cfs (1.4%) for the intermittent stream in Epping. Moonlight Brook in Newmarket caused extensive flooding damage during May 2006 flooding event. The 2005 NRCC peak discharge from the subwatershed increases by 30.1% if developed conventionally in our build-out scenario. LID implementation only increases the peak discharge by 7.5%. Additionally, LID decreases the runoff by 1.1 inches which means more infiltration, improved water quality, recharging of the groundwater, higher baseflows and therefore cooler summer temperatures. This finding is important in that it illustrates that LID in a redevelopment scenario can serve to reduce runoff from current conditions. The long-term watershed management implications of LID zoning as a redevelopment strategy are tremendous.

5.2.2 ***Hydraulics***

The Lamprey River is predominantly steady gradually varied flow. The FIS hydraulic model was created with a standard step method to generate water surface profiles. The computer program WSP2 used for the original FIS analysis is no longer an appropriate tool to delineate floodplains because it is no longer accepted by FEMA for the National Flood Insurance Program. The HEC-RAS program was used to update most of the methodologies initiated in WSP2. Duplicating the FIS with HEC-RAS was not possible because: a majority of the original FIS files were missing, the reach through Lee was added and adjusted the boundary condition for the remaining analysis through Rockingham County, the changes in hydraulic software have considerable differences in modeling flows at bridges and culverts, and the FIS modeled bypass flows to the Oyster River but did not include the hydrology of flood flows from the watersheds in the bypass region.

A refined hydraulic model was developed with a split flow junction at the confluence of the RT108 corridor floodplain. The cross section and structural data was developed from several sources: FEMA backup data, field survey, NHDOT, Engineering Consultants, and through GIS. Supplementing the FEMA data with the other sources resulted with a complete hydraulic model including inline structures and uninterrupted reach from the Macallen Dam in Newmarket to the headwaters in Raymond. This reassessment is an improvement from the FIS that dates back to the 1980s because the analysis of the eight mile stretch of the Lamprey River through Lee has been included and current land use and rainfall depths are applied.

The 2050 NRCC flood flow discharge for the 100-year event raised the water surface elevation (WSE) an average of 2.7 feet along the length of the study. The 2050 conventional build-out scenario increased the WSE an additional 0.3 feet to an average of three (3) feet higher than the FIS. At the watershed scale, a slight positive effect was observed by implementing low impact development versus conventional development design in the hydrologic and hydraulic models for runoff, peak discharge, and changes to the floodplain water surface elevation. Implementation of LID decreased the overall impact of development on the WSE by an average of 0.11 feet. This difference could be significant for FEMA where minor changes in flood elevations will require map revisions.

5.3 **Resiliency planning with Low Impact Development (LID)**

If not already enforced, watershed communities need to implement stormwater management tools to mitigate increased runoff. Stormwater programs are required to address the effects of development and increased frequency of high precipitation events.

There are several sources available for developing a stormwater policy beginning with the Center for Watershed Protection[9]. Each resource follows the same premise that site planning and design techniques need to promote the

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[9] http://www.cwp.org/your-watershed-101/stormwater-management.html

concept of minimizing directly connected impervious areas in order to decrease the volume and velocity of stormwater runoff. Based on this research, conventional and low impact development practices at the urban scale generate substantial difference in stormwater runoff. Implementation of a LID policy is a practical way for municipalities to mitigate the increase in stormwater runoff that is generated by additional impervious surface and the increase in precipitation due to climate change.

Zoning regulations that focus on site design elements such as parking, sidewalks, roadways, landscaping, open space, roofs, and stormwater can be established to protect the surface and groundwater resources. Development standards should include LID thereby improving the site's appearance, intercept and manage stormwater runoff and optimize natural infiltration of rainwater.

By using LID for future development and redevelopment projects, not only can cost savings be achieved but permitting issues regarding volume and pollutant reduction are addressed. Sites that include LID will capture and retain stormwater runoff close to its source thereby reducing the amount entering adjacent storm sewer systems or streams. Individual practices benefit the entire community's resiliency to flooding. There are multiple environmental benefits including the filtering of pollutants.

5.4 **Call for stormwater utility**

Changes in climate and land use have stressed stormwater infrastructure. Culverts and bridges that have been in place for several decades are no longer sized for the current climate and landform. Traditional federal funding sources and grants are not sufficient to replace old infrastructure. A community operated stormwater program can provide immediate financial resources for repair and replacement of stormwater infrastructure. Unlike other utilities such as power or drinking water, the public does not see an immediate benefit from paying a stormwater utility fee. For this reason it is difficult for municipalities to create them (EPA 2008). Community leaders often find it difficult to divert funds from their general budgets for stormwater pollution control. An EPA study identified three major advantages of stormwater utilities over funds generated through property tax revenues (NRDC 1999):

- Increased stability and predictability
- Greater equity
- Opportunity for incorporating incentives for implementation of on-site stormwater management

The Lamprey River watershed has growing communities where changes to the landscape and hydrology are occurring. There would be a benefit to the ratepayer on improving their site's stormwater management by linking a fee to the contributing area generating untreated stormwater runoff. Fees could be collected for inspections and land development permits at varying rates based on extent of directly connected impervious surfaces.

5.5 Spatial extent of the Lamprey River floodplain

The final step of projecting the spatial extent of the floodplain could not be completed at this date. The project was georeferenced in order to generate an inundation area for the current and future build-out conditions. The currently available digital elevation model (DEM) does not have sufficient accuracy to generate reliable maps. Mapping the spatial extent requires generating HEC-RAS cross sections from a more accurate DEM in order to transpose the resulting floodplain extents onto an aerial or topographic view of the watercourse. This will be accomplished with the recently acquired coastal New Hampshire LiDAR data.

5.6 **Conclusions**

The project's hydrologic and hydraulic models provide updated conditions for the Lamprey River. Hydrology was modeled with a rainfall runoff analysis in order to generate direct runoff from the current (2005) and future land use conditions. Optimization trials in HEC-HMS generated a calibrated model that matched the observed conditions during the April 2007 event. TP-40 and NRCC rainfall atlases provided the historic and current, and rainfall rates for the scenarios. An average of the projections of future rainfall rates under a "high emissions" (A1Fi) and "low emissions" (B1) emissions scenario were similar to the NRCC rainfall atlas values. As a result, the Atlas values were retained as a reasonable estimate of future rainfall rates. This updated model provides the necessary means to develop hydrology for small to large rainfall events, current to future land use conditions, and conventional to LID development applications.

The analysis of the 2005 TP-40 modeled a 100-year flood flow of 6,829 cfs at Packers Falls gauging station. There is a 6.9% difference compared with the FIS peak annual flood frequency analysis of 7,300 cfs. The rainfall-runoff analysis for the 2005 NRCC modeled a 100-year flood flow of 10,649 cfs compared the log Pearson type III results of 9,411 cfs at the gaged location.

FIS back up data from the FEMA library provided the initial elements for the hydraulic model. Additional cross section elevations and inline structures were integrated from other sources. This established a complete hydraulic model from Macallen Dam in Newmarket to the headwaters in Raymond. This reassessment is an improvement from the FIS that dates back to the 1980s because the analysis of the eight mile stretch of the Lamprey River through Lee has been included and current land use and rainfall depths are applied.

This research offers updated information based on current land use and adopted rainfall depths hence property owners can be aware of changes not reflected on official FIRMs. Community officials can use the information for developing master plans and flood zone regulations not only for current planning but in anticipation of population growth and development. The planning and zoning officials can refer to this study to support development and redevelopment regulations that include LID. This is especially important when considering development and redevelopment in small urban scale watersheds. Urban development along the river's corridor in Raymond, Epping, Lee, Durham and Newmarket has an immediate effect. Future development with LID has the potential to keep runoff volumes and peak discharges at today's level and in some instances lower them (Table 23).

Other watershed communities such as Deerfield, Candia, Nottingham, Exeter, and Newfields have shared responsibility in curtailing the impact that development has on generating an increase in direct runoff. Land use planning is an important way to adapt to our changing climate. This information could be used to limit development in current and future flood risk areas and to guide development practices in the usage of LID to protect water quality and contribute to community resiliency.

The methodology developed through this research can be applied to conduct similar analyses in watersheds beyond the Lamprey River. A standardized procedure can be of relevant use to other coastal or interior regions experiencing increased precipitation or just land use change and development pressures. The technical application may differ based on the available sources of data for the watershed but would likely follow the following steps.

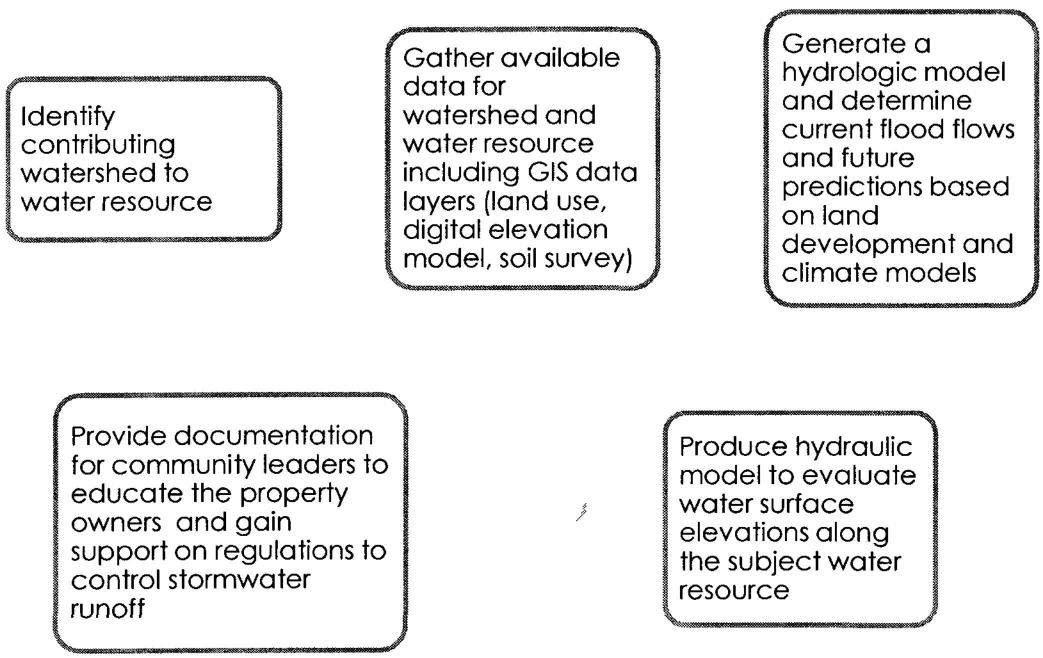

Figure 69: Standard methodology for similar analyses

5.7 **Future Recommendations**

5.7.1 ***Hydrology***

Six years have passed since the most recent land cover assessment. Several of the hydrologic modeling parameters are based on land cover and its affect upon rainfall losses and routing. What is being referred to as current conditions may not be accurate for 2011 anticipated flood flows. If the land use data set is updated, future work should include a new generation of CNs. Changes in this parameter may not greatly change associated runoff from the sub basins at watershed scale but could be more evident in the urban scale sub basins.

Antecedent moisture condition can affect the resultant peak discharge from a watershed. If a watershed is in a saturated condition, the initial abstraction (I_a) will approach zero. A dry condition increases I_a to represent the maximum precipitation depth that will fall without producing runoff. The hydrologic model used a default value 0.2 times the potential retention, which is calculated from the curve number, Ia = 0.2 S (a). Although not standard engineering practice, future work may include modeling based not only on dry or saturated conditions but on frozen or snow covered conditions. Additional work should also explore a more accurate estimation of Ia.

5.7.2 ***Hydraulics***

The increase in flows has likely caused aggradation and degradation of the channel. Cross section elevations have likely adjusted since the FIS. The accuracy of the model would be improved with implementation of current elevation data. The project budget limited the amounts of effort needed to field verify the structure condition and survey cross sections up and downstream for modeling. Several assumptions were required to fill the gaps in available information. Again, accurate field or a digital elevation model with greater precision would be an improvement to the project.

Although the bypass into the Oyster River watershed was included in this analysis, it would be beneficial to also include any bypass lost to the Pisscassic River watershed in the vicinity of river station 58,147 in Lee. During recent extreme flow events, RT152 was inundated with flood waters leaving the Lamprey River and discharging into the headwaters of the Pisscassic River.

Additionally, the tributaries that join the Lamprey; Bean River; Little River; North Branch River; Pawtuckaway River; North River; and the Piscassic River should be hydraulically modeled for resultant backwater flood elevations caused by the increased flood elevation on the Lamprey River. The flood controls provided by Pawtuckaway Lake and Mendums Pond and diversion for municipal water supply in Durham was not included in this model. The impact of the directive is apparent at lower flows and does not affect the peak flows.

Since the Lamprey River is a gaged watercourse, maintained records for extreme historical events are retrievable. This data could be beneficial to the public in evaluating hazards posed by extreme floods that have a higher flood elevation than the 100-year FIS floodplain BFE. This type of information provides prospective property owners and others, tasked with protecting the public interest, to determine a buildable finish elevation needed to remain above the maximum historical flood event recorded in the area.

The Town of Newmarket is considering alternatives for the maintenance or removal of Macallen Dam. Recent flooding from springtime events have led to questions whether it might be in the best interest to remove the structure. Macallen Dam creates an impoundment that reaches upstream approximately 9,000 feet. Downstream of Packer's Falls Road there is a fifteen (15) foot natural grade change in the stream bed. The river and floodplain upstream of this location would not likely be affected by removal of the dam. The area that could benefit from removal or lowering of the dam is the RT108 corridor. The highest spillway on the dam is elevation 30.7 and the 2005 NRCC flood flow generates a water surface elevation of 33.5 at the dam. The lowest centerline elevation of RT108 is 30.9 and the response to the impoundment is flooding at elevation 36.1 along this floodplain corridor.

5.8 **LiDAR**

The sea coast area of New Hampshire has been included in a contract arranged by the USGS to collect 1-foot, 4-band aerial imagery (Figure 70). Collection of the raw LiDAR data has been completed and is now being processed. Use of the data can replace the DEM generated from USGS maps which are far less accurate.

The LiDAR collection process is not water penetrating. This results with a flat plane at water surfaces. Cross sections can be cut from the LiDAR's DEM using HEC-GeoRAS tools in ArcMap at increments that will clarify floodplain limits in the areas between those in the current hydraulic model. There are two options for developing the channel conveyance area where the LiDAR DEM is limited.

Option 1:

- Use the closest FIS cross section for channel stationing and elevations
- adjust the elevations to NAVD88
- determine the slope of the channel between consecutive FIS sections
- raise or lower the cross section elevations based on the channel slope and the distance between the FIS and LiDAR section

Option 2:

- Utilize the New Hampshire Regional Hydraulic Geometry Curves to determine the cross section area based on the upstream drainage area
- Compare elevation of flat plane between sections
- Create this area as a trapezoid or rectangular section using the flat plane as the top width
- Verify that the slope of the thalweg mimics the slope of the channel edge

The conveyance of the channel verses the floodplain varies along the 36 miles of the Lamprey River studied for this research. In comparing a baseflow of 500 cfs to a flood flow of 10,500 cfs from the western corporate limits of Lee to Macallen Dam, the channel conveyance is 100% for the baseflow and ranges between 40- to 100-percent during the flood flow (x-bar 89%, σ 14.5). The percent difference in the depth of flow ranges between 42- and 2,256% (x-bar 236%, σ 327%). The percent difference in the width of flow ranges from 0- to 2,030% (x-bar 261%, σ 337%). These comparisons for the Lamprey River reach between Wiswall Road and Packer's Falls Road in Durham in provided in Appendix . The LiDAR will provide more accurate elevations along the floodplain but until the cross sections created with the LiDAR DEM is examined; it is unknown whether the lack of channel elevations influences the results.

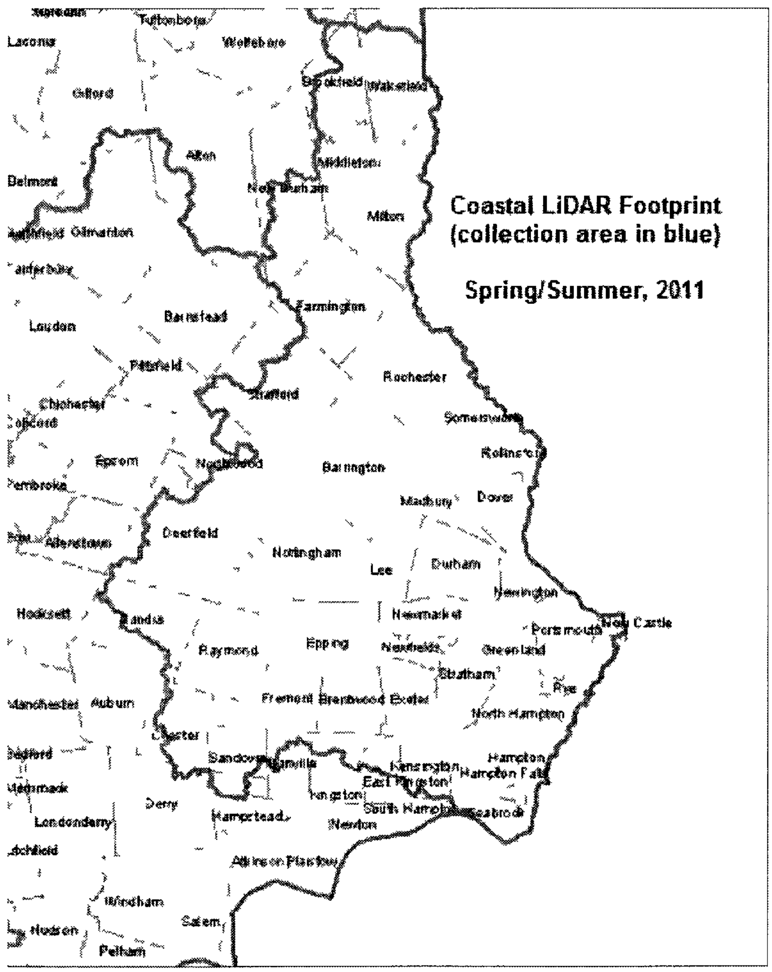

Figure 70: Coastal LiDAR collection area

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Appendices

Appendix A - Tables

Table 28: Percent of total impervious area for Watershed

Watershed Community	% increase since 1990	% Impervious		
		1990	2000	2005
Barrington	80.8%	2.6	4	4.7
Brentwood	90.0%	5	7.7	9.5
Candia	77.8%	2.7	4.1	4.8
Deerfield	100.0%	1.5	2.4	3
Durham	63.8%	4.7	7.2	7.7
Epping	95.0%	4	6.5	7.8
Exeter	65.3%	7.5	11	12.4
Fremont	96.7%	3	4.9	5.9
Lee	78.4%	3.7	5.8	6.6
Newfields	119.4%	3.1	5.5	6.8
Newmarket	71.2%	5.9	8.8	10.1
Northwood	66.7%	2.4	3.4	4
Nottingham	86.7%	1.5	2.3	2.8
Raymond	75.5%	5.3	8	9.3
Strafford	64.3%	1.4	2	2.3

Table 29: Population growth in watershed communities

Watershed Community	% increase since 1960	Census Year					
		1960	1970	1980	1990	2000	2010
Barrington	728%	1,036	1,865	4,404	6,164	7,475	8,145
Brentwood	318%	1,072	1,468	2,004	2,590	3,197	3,692
Candia	162%	1,490	1,997	2,989	3,557	3,911	4,154
Deerfield	499%	714	1,178	1,979	3,124	3,678	4,103
Durham	166%	5,504	8,869	10,652	11,818	12,664	13,276
Epping	220%	2,006	2,356	3,460	5,162	5,476	6,072
Exeter	98%	7,243	8,892	11,024	12,481	14,058	14,665
Fremont	447%	783	993	1,333	2,576	3,510	3,975
Lee	365%	931	1,481	2,111	3,729	4,145	4,405
Newfields	128%	737	843	817	888	1,551	1,584
Newmarket	183%	3,153	3,361	4,290	7,157	8,027	9,153
Northwood	310%	1,034	1,526	2,175	3,124	3,640	3,969
Nottingham	668%	623	952	1,952	2,939	3,701	4,360
Raymond	443%	1,867	3,003	5,453	8,713	9,674	10,096
Strafford	453%	722	965	1,663	2,965	3,626	3,971

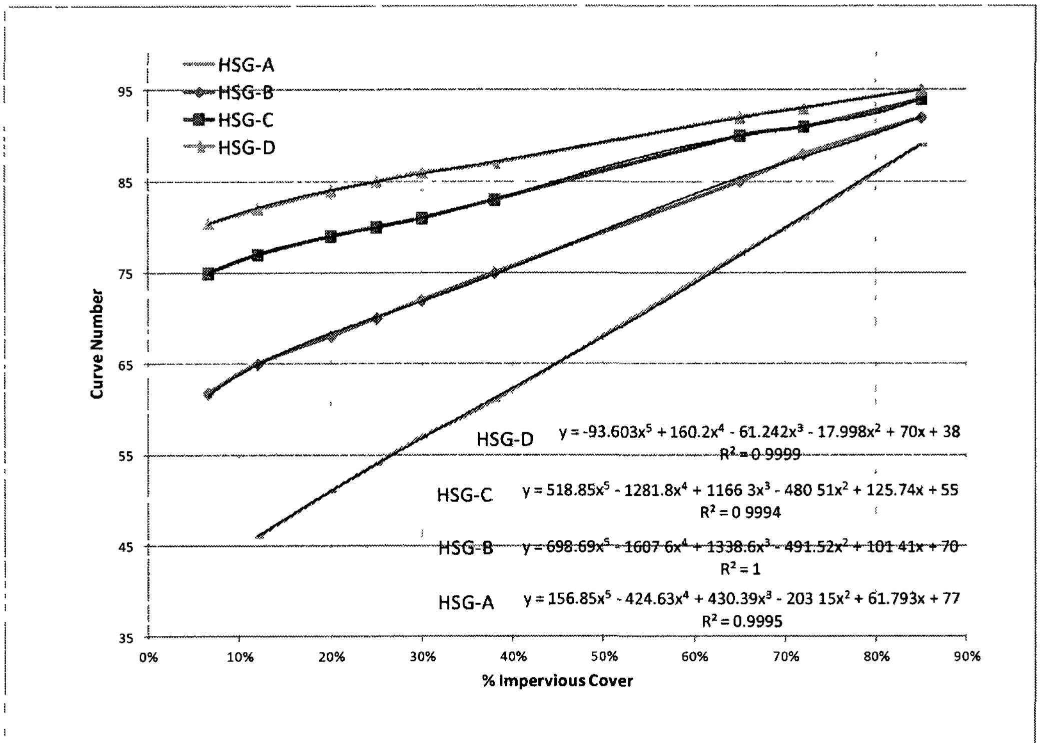


Figure 71: Curve Numbers based on function of Impervious Cover using 5th order polynomial trendline with intercept set at predevelopment conditions

Table 30: Channel and Floodplain conveyance comparison between baseflow and flood flow

River Sta	Q Total	Min Ch El	W.S. Elev	Depth	Depth % Difference	Top Width	Width % Difference	Flow Area Ch	Flow Area L	Flow Area R	Q Perc Chan	Q Perc L	Q Perc R
20082	Wiswall Road, Durham												
20073	500	44.78	57.35	12.57		275.98		846.14			100		
20073	10500	44.78	63.87	19.09	52%	538.75	95%	1533.31			100		
19934	500	42.1	57.35	15.25		235.05		2337.8	0.96		100	0	
19934	10500	42.1	64.16	22.06	45%	647.06	175%	3953.92	611.1	486.46	92.29	4.16	3.55
19908	500	44.18	57.35	13.17		194.5		2205.46			100		
19908	10500	44.18	64.09	19.91	51%	194.5	0%	3515.76			100		
19907.5	Wiswall Dam, Durham												
19863	500	42.1	57.16	15.06		174.73		2027.76	5.14		99.96	0.04	
19863	10500	42.1	63.56	21.46	42%	616.22	253%	3099.43	508.12	337.68	93.88	3.83	2.29
19859.55	500	56.1	56.8	0.7		172.67		108.21	2.45		99.69	0.31	
19859.55	10500	56.1	61.12	5.02	617%	252.52	46%	830.16	99.44	37.05	98.29	1.42	0.29
19842.98	500	42.1	42.89	0.79		37.1		17.54			100		
19842.98	10500	42.1	60.71	18.61	2256%	226	509%	3100.39		190.34	98.52		1.48
19367.12	500	38.6	40.83	2.23		98.7		106.88			100		
19367.12	10500	38.6	60.6	22	887%	238.75	142%	2453.02	738.97	818.48	84.94	6.6	8.46
17730.92	500	35.4	40.72	5.32		125.27		497.95		0.22	100		0
17730.92	10500	35.4	60.52	25.12	372%	841	571%	3007.93	3818.76	1106.42	82.85	14.01	3.14
16215.78	500	31.2	40.65	9.45		97.09		553.22			100		
16215.78	10500	31.2	60.42	29.22	209%	325	235%	2759.12	1667.37	770.32	81.36	11.64	6.99
16117.94	500	36.9	40.55	3.65		84.4		207.05			100		
16117.94	10500	36.9	60.34	23.44	542%	276.1	227%	2160.22	931.79	584.88	77.55	13.31	9.14
16077.05	500	35.7	39.92	4.22		37.42		102.31			100		
16077.05	10500	35.7	60.2	24.5	481%	373.44	898%	1013.59	507.09	1879.51	52.4	9.48	38.12
16047	500	33.2	39.76	6.56		37.8		156.33			100		
16047	10500	33.2	57.81	24.61	275%	363.44	861%	908.5	49.48	41.43	99.33	0.38	0.29
16028	Packer's Falls Road, Durham												

Appendix B – PKFQWIN Calculations

Historical (FIS) – 1934 through 1987

Full Record – 1934 through 2009

30-Yr Record – 1980 - 2009

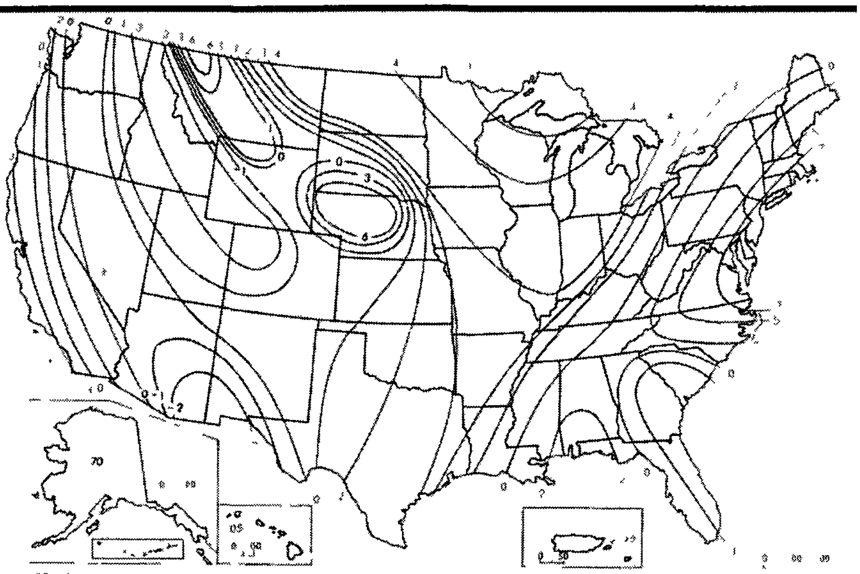
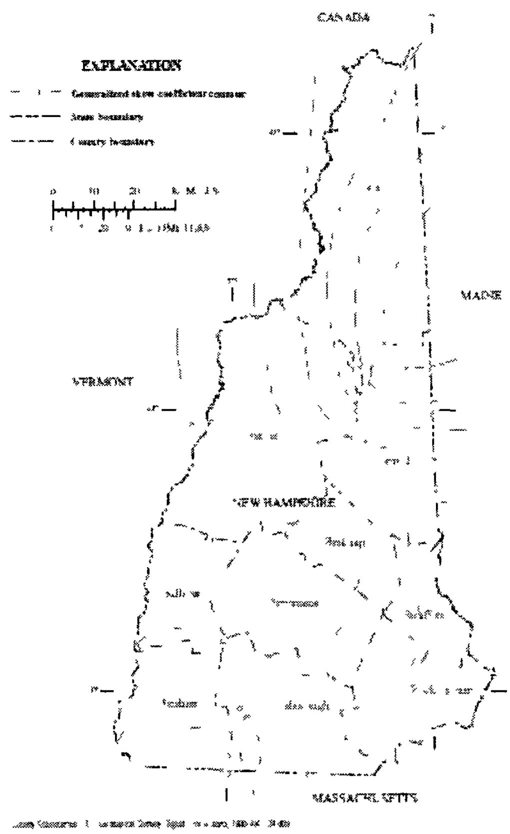


FIGURE 3.9

Generalized skew coefficients of logarithms of annual maximum streamflow (From Interagency Advisory Committee on Water Data, 1982.)



Historical (FIS) – 1934 through 1987

```
                                   1935_1987.PRT
68861
  Program PeakFq          U. S. GEOLOGICAL SURVEY              Seq.000.000
  Ver. 5.2           Annual peak flow frequency analysis       Run Date / Time
  11/01/2007         following Bulletin 17-B Guidelines        01/18/2011 10:56

                        --- PROCESSING OPTIONS ---

                     Plot option          = None
                     Basin char output    = None
                     Print option         = Yes
                     Debug print          = No
                     Input peaks listing  = Long
                     Input peaks format   = WATSTORE peak file

                     Input files used:
                        peaks (ascii)  - C:\DOCUMENTS AND SETTINGS\AMJ387\MY
DOCUMENTS\
                        ANN DOCS 2\FLOODPLAIN MAPPING\HYDRspecifications -
PKFQWPSF.TMP
                     Output file(s):
                        main - C:\DOCUMENTS AND SETTINGS\AMJ387\MY DOCUMENTS\ANN
DOCS 2\
                               FLOODPLAIN MAPPING\HYDR
1

  Program PeakFq          U. S. GEOLOGICAL SURVEY              Seq.001.001
  Ver. 5.2           Annual peak flow frequency analysis       Run Date / Time
  11/01/2007         following Bulletin 17-B Guidelines        01/18/2011 10:56

              Station - 01073500  LAMPREY RIVER NEAR NEWMARKET, NH

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

               Number of peaks in record            =       53
               Peaks not used in analysis           =        0
               Systematic peaks in analysis         =       53
               Historic peaks in analysis           =        0
               Years of historic record             =        0
               Generalized skew                     =    0.554
                    Standard error                  =    0.550
                    Mean Square error               =    0.303
               Skew option                          =   WEIGHTED
               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
    WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.             541.2
    WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS EXCEEDED HHBASE.       8647.6
1

  Program PeakFq          U. S. GEOLOGICAL SURVEY              Seq.001.002
  Ver. 5.2           Annual peak flow frequency analysis       Run Date / Time
  11/01/2007         following Bulletin 17-B Guidelines        01/18/2011 10:56
```

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

	FLOOD BASE		LOGARITHMIC		
	DISCHARGE	EXCEEDANCE PROBABILITY	MEAN	STANDARD DEVIATION	SKEW
SYSTEMATIC RECORD	0.0	1.0000	3.3351	0.2157	0.052
BULL.17B ESTIMATE	0.0	1.0000	3.3351	0.2157	0.178

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	68-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	653.9	616.8	620.8	612.1	695.0
0.9900	727.2	694.4	697.7	683.7	769.9
0.9500	980.7	962.9	960.8	932.9	1028.0
0.9000	1156.0	1148.0	1141.0	1106.0	1206.0
0.8000	1419.0	1423.0	1409.0	1366.0	1472.0
0.6667	1726.0	1741.0	1721.0	1669.0	1784.0
0.5000	2132.0	2154.0	2132.0	2065.0	2201.0
0.4292	2330.0	2354.0	2333.0	2257.0	2406.0
0.2000	3270.0	3282.0	3295.0	3154.0	3397.0
0 1000	4124.0	4099.0	4188.0	3954.0	4313.0
0.0400	5316.0	5207.0	5470.0	5055.0	5611.0
0.0200	6286.0	6083.0	6548.0	5942.0	6679.0
0.0100	7326.0	7001.0	7740.0	6886.0	7834.0
0.0050	8445.0	7967.0	9067.0	7895.0	9085.0
0.0020	10060.0	9324.0	11060.0	9339.0	10900.0

1

Program PeakFq U. S. GEOLOGICAL SURVEY Seq.001.003
Ver. 5.2 Annual peak flow frequency analysis Run Date / Time
11/01/2007 following Bulletin 17-B Guidelines 01/18/2011 10:56

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

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

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
1935	1820.0	K	1962	3320.0	K
1936	5490.0	K	1963	2390.0	K
1937	1940.0	K	1964	1240.0	K
1938	3530.0	K	1965	788.0	K
1939	1890.0	K	1966	1070.0	K
1940	2520.0	K	1967	2480.0	K
1941	1080.0	K	1968	3220.0	K
1942	1700.0	K	1969	2710.0	K
1943	1620.0	K	1970	2300.0	K
1944	1300.0	K	1971	1510.0	K
1945	1950.0	K	1972	2030.0	K

1946	1570.0	K	1973	3030.0	K
1947	1280.0	K	1974	1430.0	K
1948	2800.0	K	1975	2240.0	K
1949	884.0	K	1976	1540.0	K
1950	1720.0	K	1977	5000.0	K
1951	1700.0	K	1978	1610.0	K
1952	2780.0	K	1979	2970.0	K
1953	3120.0	K	1980	1600.0	K
1954	4070.0	K	1981	3670.0	K
1955	1880.0	K	1982	1970.0	K
1956	2960.0	K	1983	4570.0	K
1957	676.0	K	1984	3200.0	K
1958	2790.0	K	1985	1420.0	K
1959	3300.0	K	1986	1930.0	K
1960	4470.0	K	1987	7570.0	K
1961	1860.0	K			

Explanation of peak discharge qualification codes

PeakFQ CODE	NWIS 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

- \- Minus-flagged discharge -- Not used in computation
  -8888.0 -- No discharge value given
- \- Minus-flagged water year -- Historic peak used in computation

1

Program PeakFq U. S. GEOLOGICAL SURVEY Seq.001.004
Ver. 5.2 Annual peak flow frequency analysis Run Date / Time
11/01/2007 following Bulletin 17-B Guidelines 01/18/2011 10:56

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	BULL.17B ESTIMATE
1987	7570.0	0.0185	0.0185
1936	5490.0	0.0370	0.0370
1977	5000.0	0.0556	0.0556
1983	4570.0	0.0741	0.0741
1960	4470.0	0.0926	0.0926
1954	4070.0	0.1111	0.1111
1981	3670.0	0.1296	0.1296
1938	3530.0	0.1481	0.1481
1962	3320.0	0.1667	0.1667
1959	3300.0	0.1852	0.1852
1968	3220.0	0.2037	0.2037
1984	3200.0	0.2222	0.2222

1953	3120.0	0.2407	0.2407
1973	3030.0	0.2593	0.2593
1979	2970.0	0.2778	0.2778
1956	2960.0	0.2963	0.2963
1948	2800.0	0.3148	0.3148
1958	2790.0	0.3333	0.3333
1952	2780.0	0.3519	0.3519
1969	2710.0	0.3704	0.3704
1940	2520.0	0.3889	0.3889
1967	2480.0	0.4074	0.4074
1963	2390.0	0.4259	0.4259
1970	2300.0	0.4444	0.4444
1975	2240.0	0.4630	0.4630
1972	2030.0	0.4815	0.4815
1982	1970.0	0.5000	0.5000
1945	1950.0	0.5185	0.5185
1937	1940.0	0.5370	0.5370
1986	1930.0	0.5556	0.5556
1939	1890.0	0.5741	0.5741
1955	1880.0	0.5926	0.5926
1961	1860.0	0.6111	0.6111
1935	1820.0	0.6296	0.6296
1950	1720.0	0.6481	0.6481
1942	1700.0	0.6667	0.6667
1951	1700.0	0.6852	0.6852
1943	1620.0	0.7037	0.7037
1978	1610.0	0.7222	0.7222
1980	1600.0	0.7407	0.7407
1946	1570.0	0.7593	0.7593
1976	1540.0	0.7778	0.7778
1971	1510.0	0.7963	0.7963
1974	1430.0	0.8148	0.8148
1985	1420.0	0.8333	0.8333
1944	1300.0	0.8519	0.8519
1947	1280.0	0.8704	0.8704
1964	1240.0	0.8889	0.8889
1941	1080.0	0.9074	0.9074
1966	1070.0	0.9259	0.9259
1949	884.0	0.9444	0.9444
1965	788.0	0.9630	0.9630
1957	676.0	0.9815	0.9815

1

```
 End PeakFQ analysis.
   Stations processed :        1
   Number of errors   :        0
   Stations skipped   :        0
   Station years      :       53

Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4,  or *.)
(2, 4, and * records are ignored.)

 For the station below, the following records were ignored:

 FINISHED PROCESSING STATION:  01073500       USGS LAMPREY RIVER NEAR NEWMARKET,
```

Most recent – 1934 through 2009

```
                                   1935_2009.PRT
1
  Program PeakFq          U  S. GEOLOGICAL SURVEY              Seq.000.000
  Ver  5.2              Annual peak flow frequency analysis     Run Date / Time
  11/01/2007            following Bulletin 17-B Guidelines      01/18/2011 10:35

                           --- PROCESSING OPTIONS ---

                        Plot option         = None
                        Basin char output   = None
                        Print option        = Yes
                        Debug print         = No
                        Input peaks listing = Long
                        Input peaks format  = WATSTORE peak file

                        Input files used:
                           peaks (ascii)  - C.\DOCUMENTS AND SETTINGS\AMJ387\MY
DOCUMENTS\ANN DOCS 2\FLOODPLAIN MAPPING\HYDR
                           specifications - PKFQWPSF.TMP

                        Output file(s):
                           main - C:\DOCUMENTS AND SETTINGS\AMJ387\MY DOCUMENTS\ANN
DOCS 2\FLOODPLAIN MAPPING\HYDR

1

  Program PeakFq          U. S. GEOLOGICAL SURVEY              Seq.001.001
  Ver. 5.2              Annual peak flow frequency analysis     Run Date / Time
  11/01/2007            following Bulletin 17-B Guidelines      01/18/2011 10:35

            Station - 01073500  LAMPREY RIVER NEAR NEWMARKET, NH

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

                Number of peaks in record            =       75
                Peaks not used in analysis           =        0
                Systematic peaks in analysis         =       75
                Historic peaks in analysis           =        0
                Years of historic record             =        0
                Generalized skew                     =    0.554
                     Standard error                  =    0.550
                     Mean Square error               =    0.303
                Skew option                          =   WEIGHTED
                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
   WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.              466.6
   WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS EXCEEDED HHBASE.       10929.9
1

  Program PeakFq          U. S. GEOLOGICAL SURVEY              Seq.001.002
  Ver. 5.2              Annual peak flow frequency analysis     Run Date / Time
  11/01/2007            following Bulletin 17-B Guidelines      01/18/2011 10.35
```

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

	FLOOD BASE		LOGARITHMIC		
	DISCHARGE	EXCEEDANCE PROBABILITY	MEAN	STANDARD DEVIATION	SKEW
SYSTEMATIC RECORD	0.0	1.0000	3.3538	0.2348	0.398
BULL.17B ESTIMATE	0.0	1.0000	3.3538	0.2348	0.435

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	68-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	699.3	686.6	678.7	661.6	736.6
0.9900	764.6	753.4	746.0	725.6	803.3
0.9500	997.0	990.7	983.9	954.0	1040.0
0.9000	1164.0	1161.0	1154.0	1119.0	1209.0
0.8000	1422.0	1422.0	1415.0	1373.0	1470.0
0.6667	1737.0	1741.0	1733.0	1683.0	1790.0
0.5000	2172.0	2179.0	2172.0	2109.0	2236.0
0.4292	2392.0	2400.0	2394.0	2324.0	2464.0
0.2000	3506.0	3511.0	3528.0	3392.0	3628.0
0.1000	4607.0	4601.0	4668.0	4431.0	4801.0
0.0400	6277.0	6239.0	6435.0	5986.0	6602.0
0.0200	7741.0	7665.0	8026.0	7335.0	8199.0
0.0100	9411.0	9280.0	9888.0	8862.0	10040.0
0.0050	11320.0	11110.0	12080.0	10590.0	12150.0
0.0020	14260.0	13920.0	15570.0	13240.0	15430.0

1

Program PeakFq	U. S. GEOLOGICAL SURVEY	Seq.001.003
Ver. 5.2	Annual peak flow frequency analysis	Run Date / Time
11/01/2007	following Bulletin 17-B Guidelines	01/18/2011 10:35

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

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

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
1935	1820.0	K	1973	3030.0	K
1936	5490.0	K	1974	1430.0	K
1937	1940.0	K	1975	2240.0	K
1938	3530.0	K	1976	1540.0	K
1939	1890.0	K	1977	5000.0	K
1940	2520.0	K	1978	1610.0	K
1941	1080.0	K	1979	2970.0	K
1942	1700.0	K	1980	1600.0	K
1943	1620.0	K	1981	3670.0	K
1944	1300.0	K	1982	1970.0	K
1945	1950.0	K	1983	4570.0	K

1946	1570.0	K	1984	3200.0	K
1947	1280.0	K	1985	1420.0	K
1948	2800.0	K	1986	1930.0	K
1949	884.0	K	1987	7570.0	K
1950	1720.0	K	1988	1390.0	K
1951	1700.0	K	1989	1740.0	K
1952	2780.0	K	1990	1500.0	K
1953	3120.0	K	1991	1720.0	K
1954	4070.0	K	1992	1310.0	K
1955	1880.0	K	1993	3400.0	K
1956	2960.0	K	1994	2210.0	K
1957	676.0	K	1995	1670.0	K
1958	2790.0	K	1996	2850.0	K
1959	3300.0	K	1997	7080.0	K
1960	4470.0	K	1998	4720.0	K
1961	1860.0	K	1999	976.0	K
1962	3320.0	K	2000	2310.0	K
1963	2390.0	K	2001	3290.0	K
1964	1240.0	K	2002	1040.0	K
1965	788.0	K	2003	2130.0	K
1966	1070.0	K	2004	4690.0	K
1967	2480.0	K	2005	2650.0	K
1968	3220.0	K	2006	8970.0	K
1969	2710.0	K	2007	8450.0	K
1970	2300.0	K	2008	1850.0	K
1971	1510.0	K	2009	2110.0	K
1972	2030.0	K			

Explanation of peak discharge qualification codes

PeakFQ CODE	NWIS 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

- Minus-flagged discharge -- Not used in computation
  -8888.0 -- No discharge value given
- Minus-flagged water year -- Historic peak used in computation

1

```
Program PeakFq          U. S. GEOLOGICAL SURVEY           Seq.001.004
Ver. 5.2             Annual peak flow frequency analysis    Run Date / Time
11/01/2007           following Bulletin 17-B Guidelines     01/18/2011 10:35

         Station - 01073500  LAMPREY RIVER NEAR NEWMARKET, NH
```

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	BULL.17B ESTIMATE
2006	8970.0	0.0132	0.0132

1935_2009.PRT

2007	8450.0	0.0263	0.0263
1987	7570.0	0.0395	0.0395
1997	7080.0	0.0526	0.0526
1936	5490.0	0.0658	0.0658
1977	5000.0	0.0789	0.0789
1998	4720.0	0.0921	0.0921
2004	4690.0	0.1053	0.1053
1983	4570.0	0.1184	0.1184
1960	4470.0	0.1316	0.1316
1954	4070.0	0.1447	0.1447
1981	3670.0	0.1579	0.1579
1938	3530.0	0.1711	0.1711
1993	3400.0	0.1842	0.1842
1962	3320.0	0.1974	0.1974
1959	3300.0	0.2105	0.2105
2001	3290.0	0.2237	0.2237
1968	3220.0	0.2368	0.2368
1984	3200.0	0.2500	0.2500
1953	3120.0	0.2632	0.2632
1973	3030.0	0.2763	0.2763
1979	2970.0	0.2895	0.2895
1956	2960.0	0.3026	0.3026
1996	2850.0	0.3158	0.3158
1948	2800.0	0.3289	0.3289
1958	2790.0	0.3421	0.3421
1952	2780.0	0.3553	0.3553
1969	2710.0	0.3684	0.3684
2005	2650.0	0.3816	0.3816
1940	2520.0	0.3947	0.3947
1967	2480.0	0.4079	0.4079
1963	2390.0	0.4211	0.4211
2000	2310.0	0.4342	0.4342
1970	2300.0	0.4474	0.4474
1975	2240.0	0.4605	0.4605
1994	2210.0	0.4737	0.4737
2003	2130.0	0.4868	0.4868
2009	2110.0	0.5000	0.5000
1972	2030.0	0.5132	0.5132
1982	1970.0	0.5263	0.5263
1945	1950.0	0.5395	0.5395
1937	1940.0	0.5526	0.5526
1986	1930.0	0.5658	0.5658
1939	1890.0	0.5789	0.5789
1955	1880.0	0.5921	0.5921
1961	1860.0	0.6053	0.6053
2008	1850.0	0.6184	0.6184
1935	1820.0	0.6316	0.6316
1989	1740.0	0.6447	0.6447
1950	1720.0	0.6579	0.6579
1991	1720.0	0.6711	0.6711
1942	1700.0	0.6842	0.6842
1951	1700.0	0.6974	0.6974
1995	1670.0	0.7105	0.7105
1943	1620.0	0.7237	0.7237
1978	1610.0	0.7368	0.7368
1980	1600.0	0.7500	0.7500
1946	1570.0	0.7632	0.7632
1976	1540.0	0.7763	0.7763
1971	1510.0	0.7895	0.7895
1990	1500.0	0.8026	0.8026
1974	1430.0	0.8158	0.8158
1985	1420.0	0.8289	0.8289
1988	1390.0	0.8421	0.8421

1992	1310.0	0.8553	0.8553
1944	1300.0	0.8684	0.8684
1947	1280.0	0.8816	0.8816
1964	1240.0	0.8947	0.8947
1941	1080.0	0.9079	0.9079
1966	1070.0	0.9211	0.9211
2002	1040.0	0.9342	0.9342
1999	976.0	0.9474	0.9474
1949	884.0	0.9605	0.9605
1965	788.0	0.9737	0.9737
1957	676.0	0.9868	0.9868

1

```
 End PeakFQ analysis.
   Stations processed :       1
   Number of errors   :       0
   Stations skipped   :       0
   Station years      :      75

Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4,  or *.)
(2, 4, and * records are ignored.)

 For the station below, the following records were ignored:

 FINISHED PROCESSING STATION:  01073500       USGS LAMPREY RIVER NEAR NEWMARKET,
```

30-Yr Record – 1980 – 2009

```
                                   1980_2009.PRT
1
  Program PeakFq           U. S. GEOLOGICAL SURVEY                Seq.000.000
  Ver. 5.2               Annual peak flow frequency analysis      Run Date / Time
  11/01/2007             following Bulletin 17-B Guidelines       12/01/2011 14:41

                             --- PROCESSING OPTIONS ---

                         Plot option         = None
                         Basin char output   = None
                         Print option        = Yes
                         Debug print         = No
                         Input peaks listing = Long
                         Input peaks format  = WATSTORE peak file

                         Input files used:
                            peaks (ascii)  - C:\DOCUMENTS AND SETTINGS\AMJ387\MY
DOCUMENTS\ANN DOCS 2\FLOODPLAIN MAPPING\HYDR
                            specifications - PKFQWPSF.TMP

                         Output file(s):
                            main - C:\DOCUMENTS AND SETTINGS\AMJ387\MY DOCUMENTS\ANN
DOCS 2\FLOODPLAIN MAPPING\HYDR

1

  Program PeakFq           U. S. GEOLOGICAL SURVEY                Seq.001.001
  Ver. 5.2               Annual peak flow frequency analysis      Run Date / Time
  11/01/2007             following Bulletin 17-B Guidelines       12/01/2011 14:41

            Station - 01073500  LAMPREY RIVER NEAR NEWMARKET, NH

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

                  Number of peaks in record           =        30
                  Peaks not used in analysis          =         0
                  Systematic peaks in analysis        =        30
                  Historic peaks in analysis          =         0
                  Years of historic record            =         0
                  Generalized skew                    =     0.554
                       Standard error                 =     0.550
                       Mean Square error              =     0.303
                  Skew option                         =   WEIGHTED
                  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.       12372.6
    WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.              535.3
1

  Program PeakFq           U. S. GEOLOGICAL SURVEY                Seq.001.002
  Ver. 5.2               Annual peak flow frequency analysis      Run Date / Time
  11/01/2007             following Bulletin 17-B Guidelines       12/01/2011 14:41
```

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

	FLOOD BASE		LOGARITHMIC		
	DISCHARGE	EXCEEDANCE PROBABILITY	MEAN	STANDARD DEVIATION	SKEW
SYSTEMATIC RECORD	0.0	1.0000	3.4105	0.2661	0.589
BULL.17B ESTIMATE	0.0	1.0000	3.4105	0.2661	0.574

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	68-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER
0.9950	737.5	743.5	685.3	668.7	805.6
0.9900	803.8	809.1	755.7	732.3	874.5
0.9500	1048.0	1051.0	1012.0	967.9	1127.0
0.9000	1231.0	1232.0	1202.0	1145.0	1315.0
0.8000	1622.0	1622.0	1602.0	1429.0	1818.0
0.6667	1893.0	1891.0	1881.0	1788.0	1999.0
0.5000	2428.0	2424.0	2428.0	2303.0	2568.0
0.4292	2709.0	2704.0	2716.0	2571.0	2855.0
0.2000	4206.0	4203.0	4286.0	3967.0	4477.0
0.1000	5803.0	5806.0	6037.0	5414.0	6258.0
0.0400	8404.0	8425.0	9082.0	7717.0	9234.0
0.0200	10840.0	10880.0	12160.0	9832.0	12080.0
0.0100	13770.0	13850.0	16160.0	12340.0	15550.0
0.0050	17280.0	17430.0	21410.0	15310.0	19800.0
0.0020	23030.0	23280.0	31020.0	20090.0	26840.0

1

Program PeakFq	U. S. GEOLOGICAL SURVEY	Seq.001.003
Ver. 5.2	Annual peak flow frequency analysis	Run Date / Time
11/01/2007	following Bulletin 17-B Guidelines	12 01 2011 14:41

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

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

WATER YEAR	DISCHARGE	CODES	WATER YEAR	DISCHARGE	CODES
1980	1600.0	K	1995	1670.0	K
1981	3670.0	K	1996	2850.0	K
1982	1970.0	K	1997	7080.0	K
1983	4570.0	K	1998	4720.0	K
1984	3200.0	K	1999	976.0	K
1985	1420.0	K	2000	2310.0	K
1986	1930.0	K	2001	5290.0	K
1987	7570.0	K	2002	1040.0	K
1988	1390.0	K	2003	2130.0	K
1989	1740.0	K	2004	4690.0	K
1990	1500.0	K	2005	2650.0	K

1991	1720.0	K	2006	8970.0	K
1992	1310.0	K	2007	8450.0	K
1993	3400.0	K	2008	1850.0	K
1994	2210.0	K	2009	2110.0	K

Explanation of peak discharge qualification codes

PeakFQ CODE	NWIS 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

- Minus-flagged discharge -- Not used in computation
  -8888.0 -- No discharge value given
- Minus-flagged water year -- Historic peak used in computation

Program PeakFq — U. S. GEOLOGICAL SURVEY — Seq.001.004
Ver. 5.2 — Annual peak flow frequency analysis — Run Date / Time
11/01/2007 — following Bulletin 17-B Guidelines — 12/01/2011 14:41

Station - 01073500 LAMPREY RIVER NEAR NEWMARKET, NH

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	BULL.17B ESTIMATE
2006	8970.0	0.0323	0.0323
2007	8450.0	0.0645	0.0645
1987	7570.0	0.0968	0.0968
1997	7080.0	0.1290	0.1290
1998	4720.0	0.1613	0.1613
2004	4690.0	0.1935	0.1935
1983	4570.0	0.2258	0.2258
1981	3670.0	0.2581	0.2581
1993	3400.0	0.2903	0.2903
2001	3290.0	0.3226	0.3226
1984	3200.0	0.3548	0.3548
1996	2850.0	0.3871	0.3871
2005	2650.0	0.4194	0.4194
2000	2310.0	0.4516	0.4516
1994	2210.0	0.4839	0.4839
2003	2130.0	0.5161	0.5161
2009	2110.0	0.5484	0.5484
1982	1970.0	0.5806	0.5806
1986	1930.0	0.6129	0.6129
2008	1850.0	0.6452	0.6452
1989	1740.0	0.6774	0.6774
1991	1720.0	0.7097	0.7097
1995	1670.0	0.7419	0.7419
1980	1600.0	0.7742	0.7742

```
      1990          1500.0          0.8065          0.8065
      1985          1420.0          0.8387          0.8387
      1988          1390.0          0.8710          0.8710
      1992          1310.0          0.9032          0.9032
      2002          1040.0          0.9355          0.9355
      1999           976.0          0.9677          0.9677
1


 End PeakFQ analysis.
   Stations processed :       1
   Number of errors   :       0
   Stations skipped   :       0
   Station years      :      30

Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4,  or *.)
(2, 4, and * records are ignored.)

 For the station below, the following records were ignored:

 FINISHED PROCESSING STATION:  01073500       USGS LAMPREY RIVER NEAR NEWMARKET,


 For the station below, the following records were ignored:

 FINISHED PROCESSING STATION:
```

Appendix C - GIS

GRANIT GIS Datasets

GRANIT Target Land Use Catagories

GRANIT GIS Datasets

New Hampshire Conservation/Public Lands at 1:24,000 Scale

Digital Elevation Models

Level 6 Hydrologic Unit Boundaries for New Hampshire

New Hampshire Land Cover Assessment – 2005

NH Public Roads

New Hampshire Hydrography Dataset

New Hampshire Political Boundaries at 1:24,000 Scale

Railroads

Soil Survey Geographic (SSURGO) database for New Hampshire

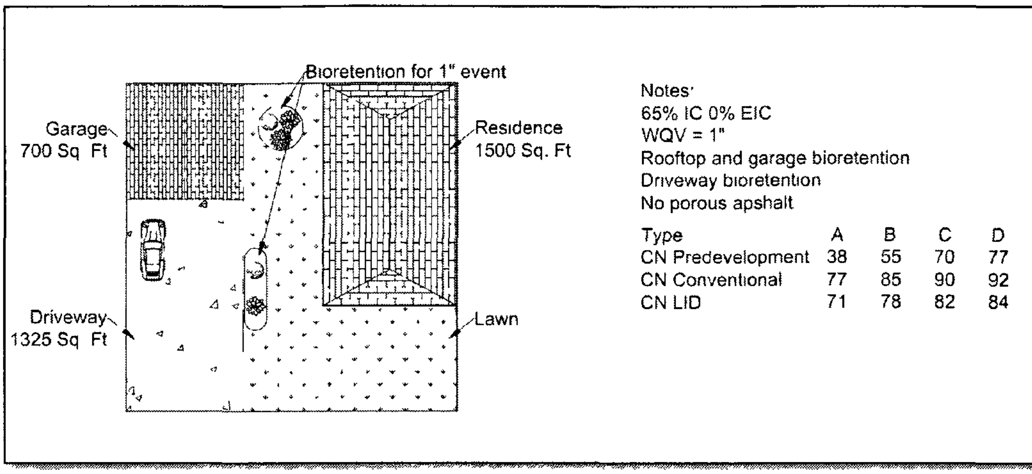
Tax parcel zoning

GRANIT Target Land Use Catagories

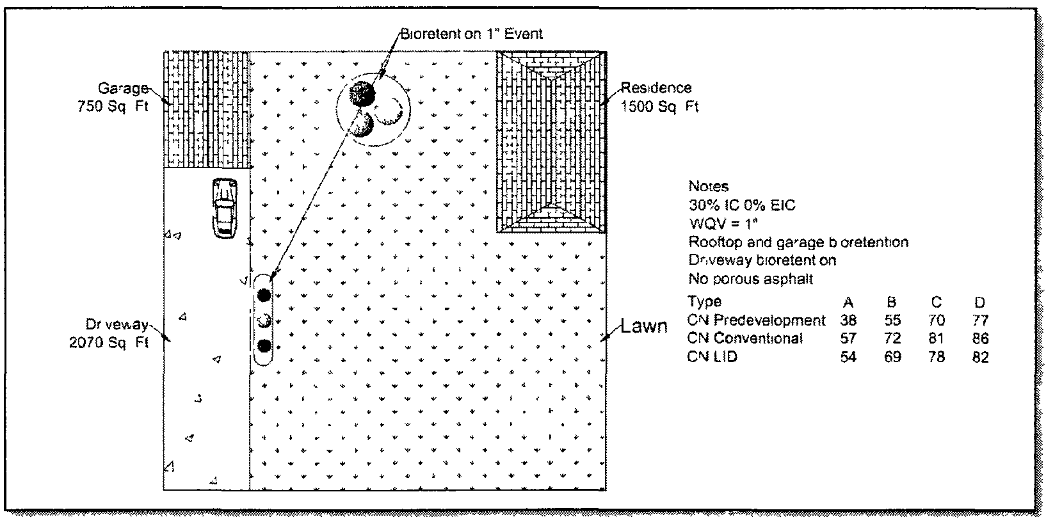
Urban and Built-Up Land (1)		
Residential (11)		
	1110	Multi-family, medium to high rise apartments and condominiums (4 or more stories)
	1120	Multi-family, low rise apartments and townhouses, but not duplexes (1 - 3 stories)
	1130	Single family/duplex
	1140	Mobile home parks
	1150	Group and transient quarters
	1190	Other residential
Commercial, Services, and Institutional (12)		
	1210	Commercial retail
	1220	Commercial wholesale
	1230	Services
	1240	Lodging
	1250	Government
	1260	Institutional
	1270	Educational
	1280	Indoor cultural/public assembly
	1290	Other commercial, services, and institutional
Industrial (13)		
	1300	Industrial
	1370	Mining
Transportation, Communications, and Utilities (14)		
	1410	Air transportation
	1420	Rail transportation
	1430	Water transportation
	1440	Road transportation
	1441	Limited & controlled highway right-of-way
	1442	Road right-of-way
	1445	Park & ride lot
	1446	Parking structure/lot
	1447	Auxiliary transportation
	1449	Other road transportation
	1450	Communication
	1460	Electric, gas and other utilities
	1470	Water and wastewater utilities
	1480	Solid waste utilities
	1490	Other transportation, communications, and utilities
Industrial and Commercial Complexes (15)		
	1510	Industrial park
	1520	Office park
	1530	Shopping mall
	1580	Other industrial complexes
	1590	Other commercial complexes
Mixed Developed Uses (16)		
	1610	Multiple stories, residential in upper stories only
	1690	Other mixed uses
Outoor and Other Urban and Built-Up Land (17)		
	1710	Outdoor cultural
	1720	Outdoor public assembly
	1730	Outdoor recreation
	1740	Cemeteries
	1790	Other outdoor and other urban or built-up land
Vacant (18)		
	1800	Vacant Land
Agriculture (2)		
	2000	Agricultural Land
	2900	Other Agricultural Land
Transitional (3)		
	3000	Brush or Transitional Between Open and Forested
Forest (4)		
	4000	Forest Land
Water (5)		
	5000	Water (see 143 for transportation uses and 233 for agricultural uses)
Wetlands (6)		
	6000	Wetlands
Barren (7)		
	7100	Salt Flats
	7200	Beaches and River Banks
	7300	Sandy Areas (non-beaches)
	7400	Bare/Exposed Rock
	7500	Strip Mine/Quarry or Gravel Pit
	7600	Disturbed Land
	7900	Other Barren Lands
Tundra (8)		
	8000	Tundra

Appendix D – LID Scenarios

1/8 and 1/3 Acre Residential LID Development Scenario

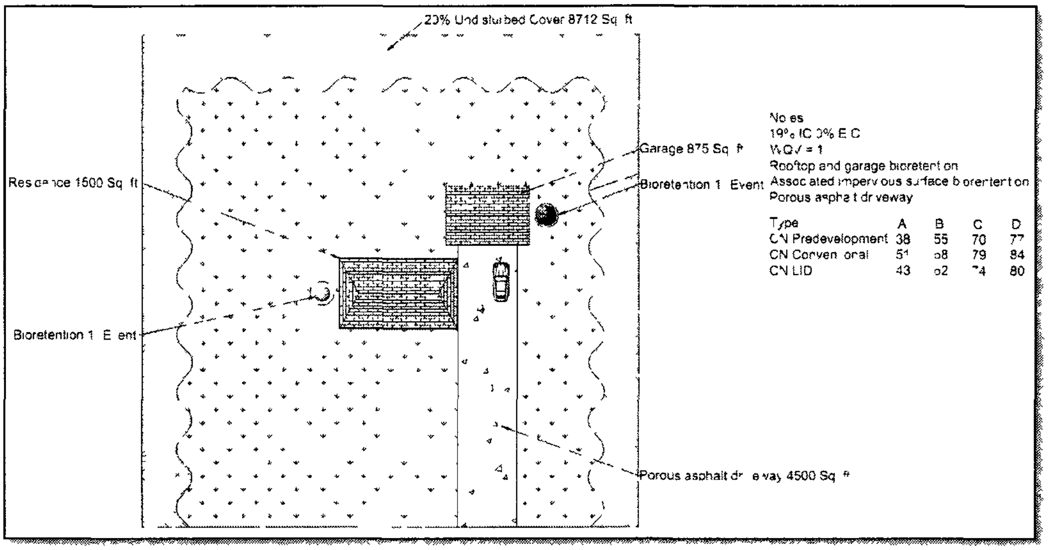


One Eighth Acre Residential LID Development Scenario; EIC=Effective Impervious Cover; CNs listed for predevelopment, post-development conventional and LID

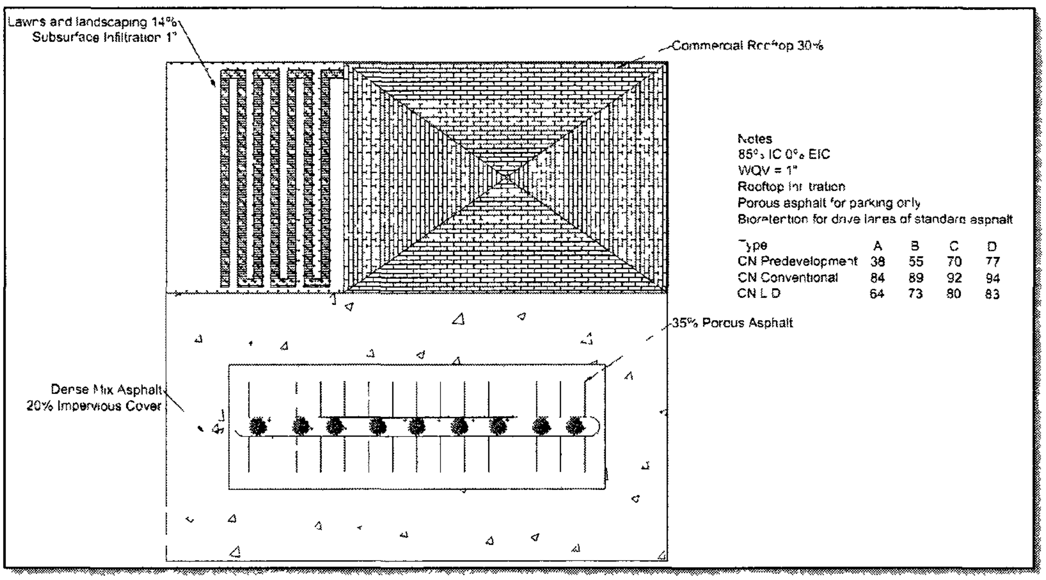


One Third Acre Residential LID Development Scenario; EIC=Effective Impervious Cover; CNs listed for predevelopment, post-development conventional and LID

1Acre Residential and Commercial LID Development Scenario



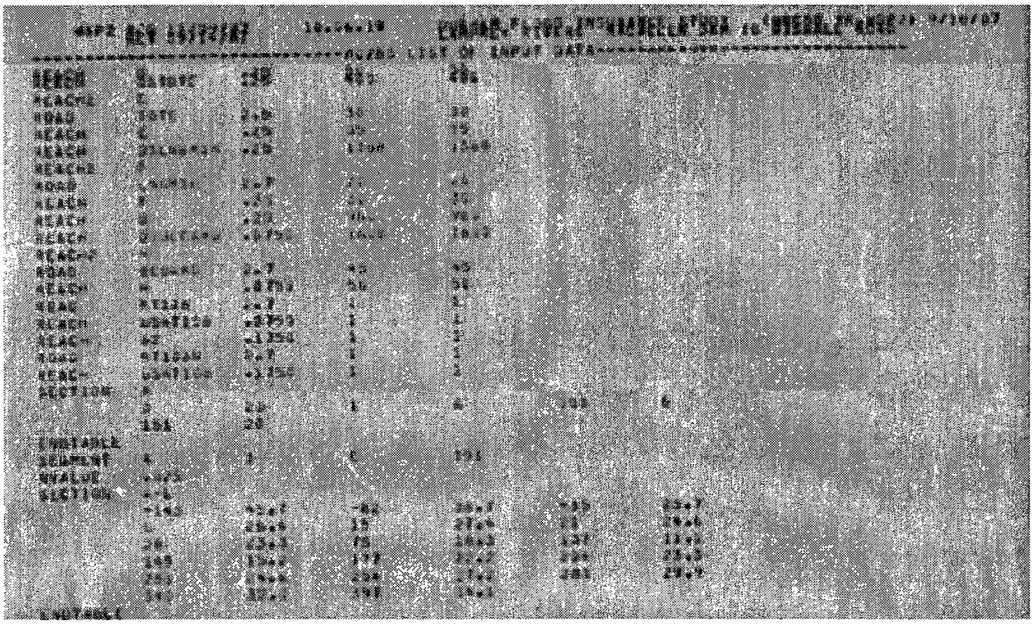
One Acre Residential LID Development Scenario; EIC=Effective Impervious Cover; CNs listed for predevelopment, post-development conventional and LID

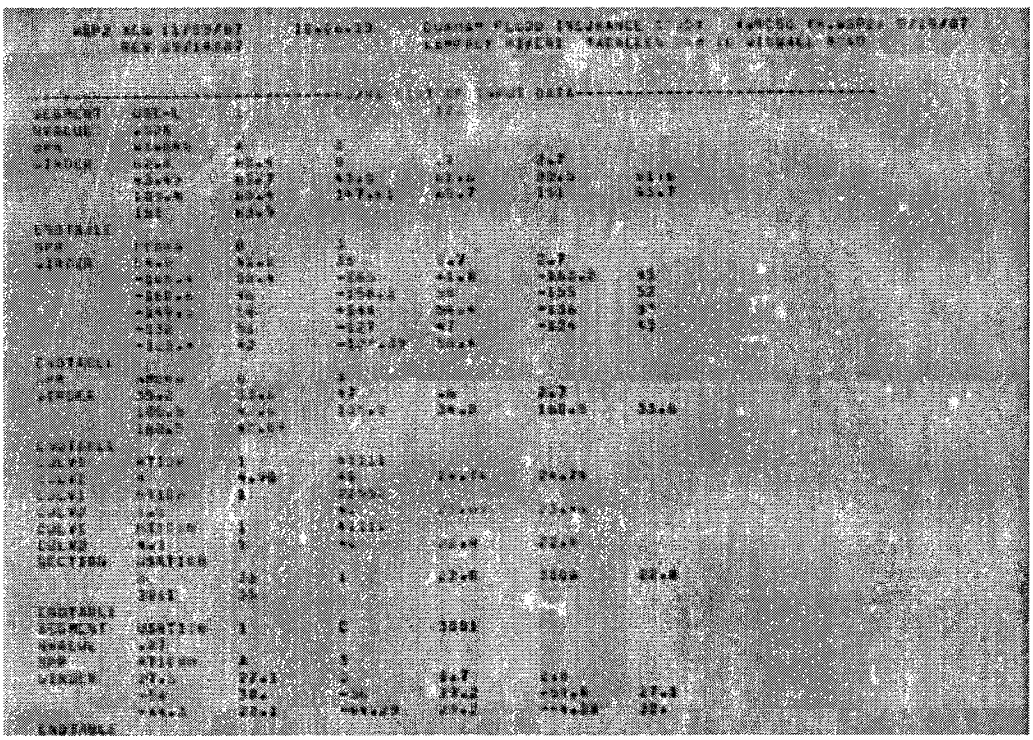


One Acre Commercial LID Development Scenario; EIC=Effective Impervious Cover; CNs listed for predevelopment, post-development conventional and LID

Appendix E – FEMA WSP2 Data

Control Word	Data Field				
	11-20	21-30	31-40	51-60	61-70
TITLE					
DISCHARGE	Total D.A	CSM	CSM		
STARTE	XSEC Name	Starting Elevation for 1st listed CSM	Starting Elevation for 2nd listed CSM		
OUTPUT	10 Options	S-Segment Table (conveyance, discharge, velocity) K-Conveyance Table (Top width and conveyance and segment conveyance)			
TRIB	The xsec names where data are to be held for use as starting data on later profiles				
REACH	XSEC Name	D.A. (sq.mi)	Hydraulic channel length to next DS section	Hydraulic main flood plain length to next DS section	
REACH2	Transposed XSEC	Elevation displacement			
ROAD	XSEC Name	Weir Coef.	Reach lengths		
SECTION	XSEC Name	Height Instr.	TR-20 Rating	Left Encr.	Right Encr.
	X, Y, data records to describe shape of section				
ENDTABLE	Indicates end of section table				
SEGMENT	XSEC Name	No. of segments 1-6	Type C, D, N	Last station marks end	Last elevation
NVALUE	'n' value				
BPR –data for computing bridges	XSEC Name	Skew Type (A or B) Fig.31-2	Base Curve (1-3) Fig.31-3	Pier Curve (1-8) Fig.31-4	
GIRDER – items pertaining to opening	Elev Full Elevation where orifice flow begins	Elev Grdr Bot elev where girders reduce channel flow	Skew Angle angle of flow per'd to road	Orif Coef for orifice flow formula	Weir coef for flow over the deck
	X, Y, data records to describe shape of bridge girder				
CULV1	XSEC Name	No. of Pipe	Culv. Code from table page 31A-26		
CULV2 – continuation of CULV1	Dia. or Height of circular, box or arch culvert	Width of box or piper arch (blank for cir.)	Total length of pipe	Upstream invert elev.	Downstream Invert elev.





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------------------------ LIST OF INPUT DATA WITH LIMITED EDITING --------
TITLE     [illegible] FLOOD INSURANCE STUDY [illegible]
TITLE     [illegible]
DISCHARGE [illegible]
OUTPUT    [illegible]
REACH     [illegible]
ROAD      [illegible]
[illegible]
ENDTABLE
```

```
------------------------ [illegible] LIST OF INPUT DATA ------------------------
---------------- MESSAGE - DATA POINTS ON SECTION [illegible] REORDERED BY PROGRAM ACCORDING TO X VALUES
                 DUPLICATE X POINTS NOT CHANGED -----------------------
SEGMENT   [illegible]
XVALUE    [illegible]
SECTION   [illegible]
[illegible]
ENDTABLE
---------------- MESSAGE - DATA POINTS ON SECTION [illegible] REORDERED BY PROGRAM ACCORDING TO X VALUES
                 DUPLICATE X POINTS NOT CHANGED -----------------------
SEGMENT   [illegible]
XVALUE    [illegible]
SECTION   [illegible]
[illegible]
ENDTABLE
```

Appendix F – FIS Flood Profiles

Rockingham County:

Lamprey River cross sections A through Z and AA through AU

Strafford County:

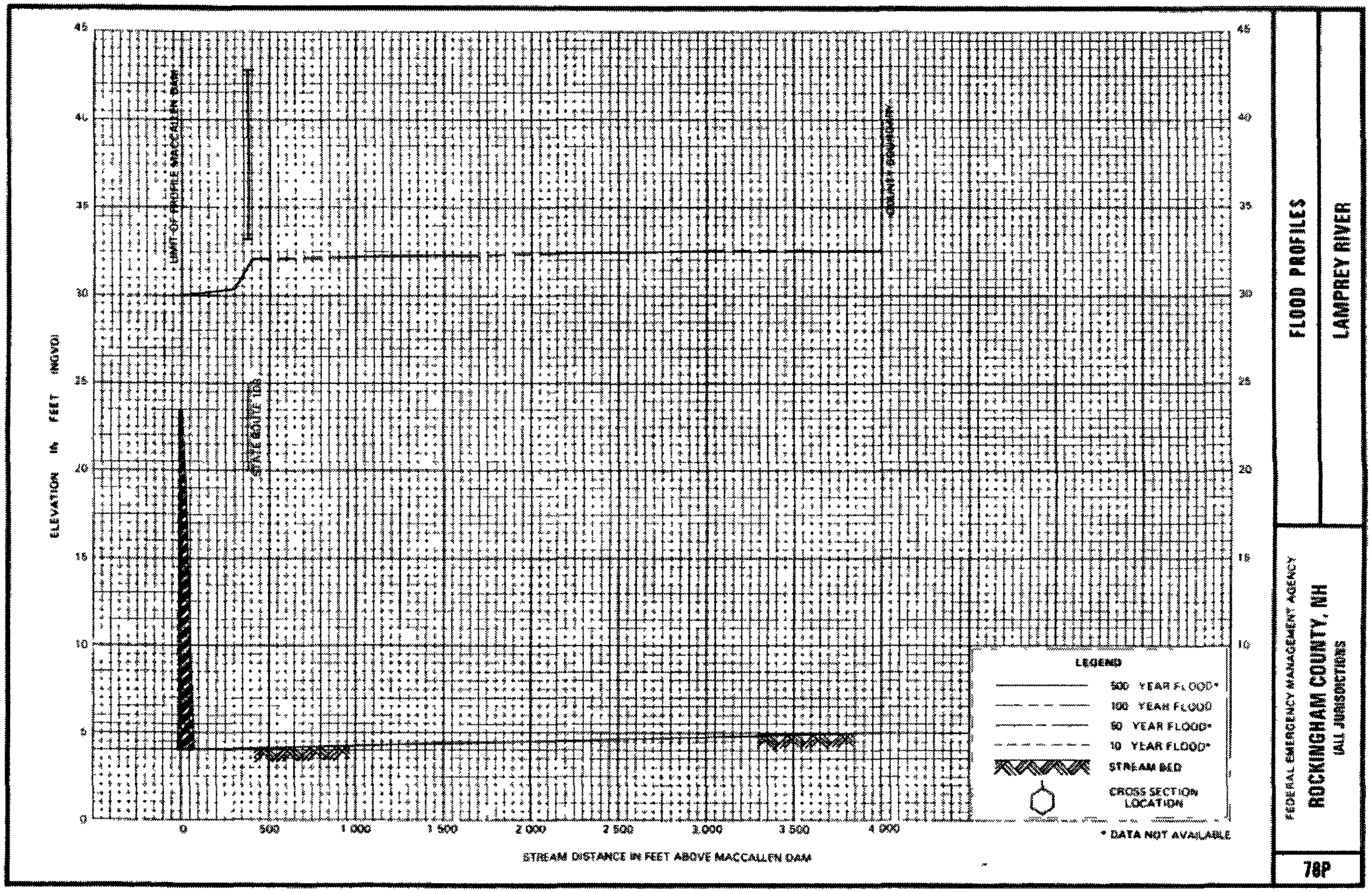
- Lamprey River cross sections A through N
- Hamil Brook cross sections A through E

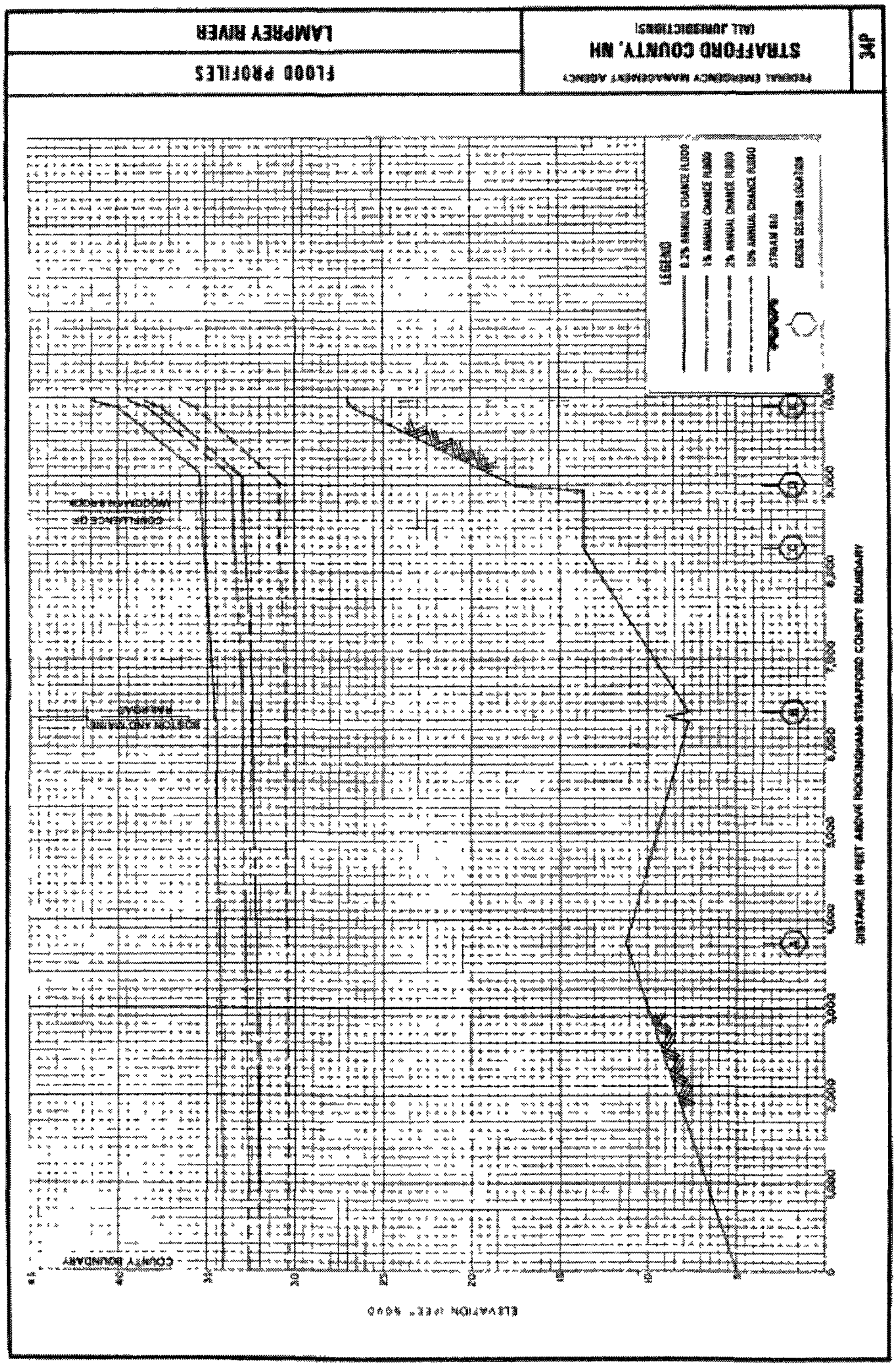
Longitudinal Profile Tables

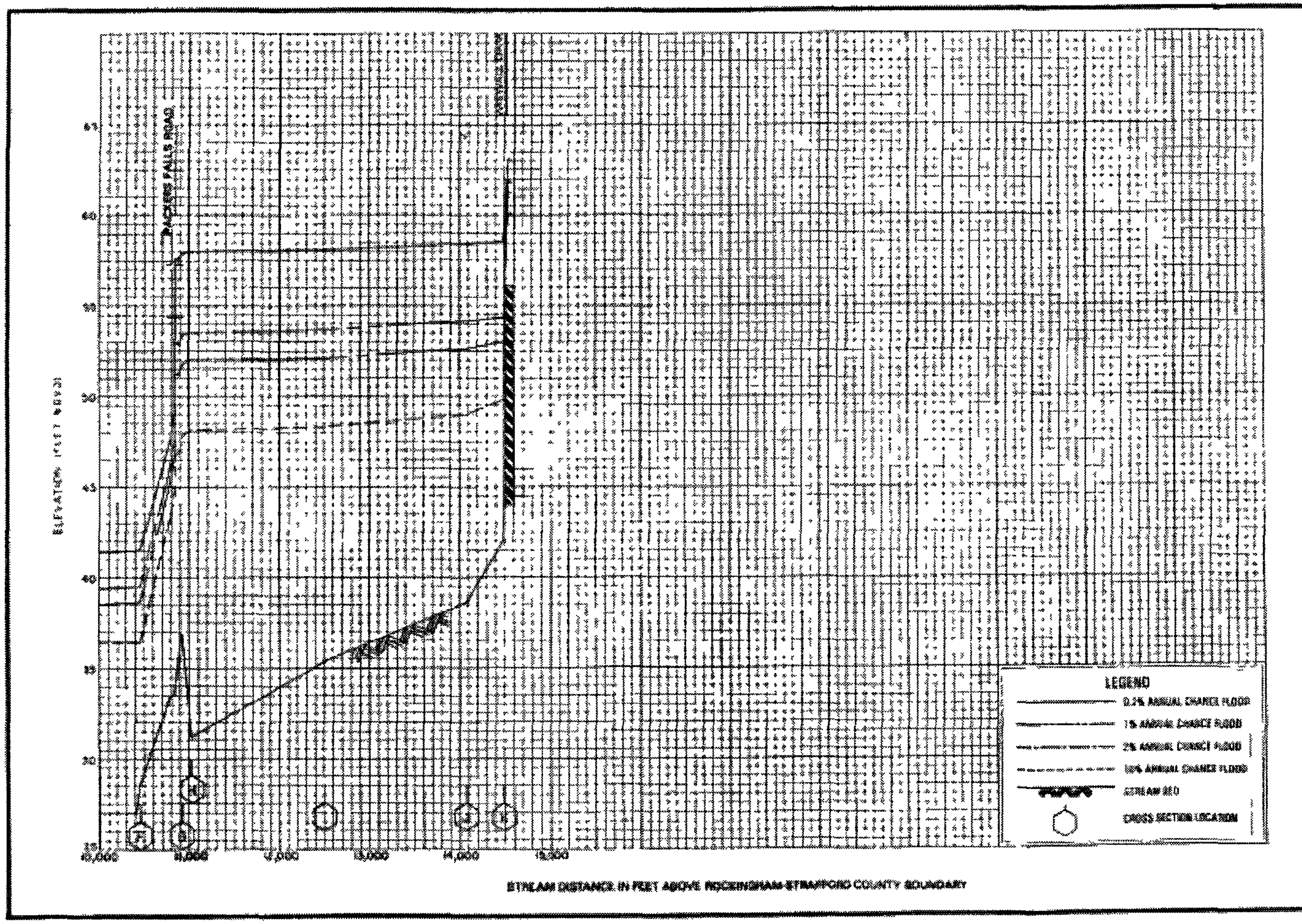
TABLE 4 – SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-YEAR	50-YEAR	100-YEAR	500-YEAR
LAMPREY RIVER					
At Durham-Newmarket corporate limits	188	*	*	6,000	*
At USGS Gage No. 01073500	183	*	*	7,300	*
At the northern corporate limits of Town of Epping	154	3,500	5,000	5,600	6,900
At State Route 101	112	2,960	4,370	4,930	6,270
At Blake Road	102	2,820	4,240	4,720	6,020
At the western corporate limits of Town of Epping	74	2,380	3,740	4,180	5,360
At the downstream corporate limits of Town of Raymond	74	2,760	4,330	5,290	7,470
At Langford Road	52	2,200	3,590	4,370	6,340
At Alternate State Route 101	33	1,600	2,710	3,300	4,880

Lamprey River




FEDERAL EMERGENCY MANAGEMENT AGENCY
STRAFFORD COUNTY, NH
(ALL JURISDICTIONS)
34P
FLOOD PROFILES
LAMPREY RIVER
LEGEND
0.2% ANNUAL CHANCE FLOOD
1% ANNUAL CHANCE FLOOD
2% ANNUAL CHANCE FLOOD
10% ANNUAL CHANCE FLOOD
STREAM BED
CROSS SECTION LOCATION
CONFLUENCE OF WOODMAN BROOK
BOSTON AND MAINE RAILROAD
COUNTY BOUNDARY
DISTANCE IN FEET ABOVE ROCKINGHAM-STRAFFORD COUNTY BOUNDARY
ELEVATION (FEET NGVD)



FLOOD PROFILES

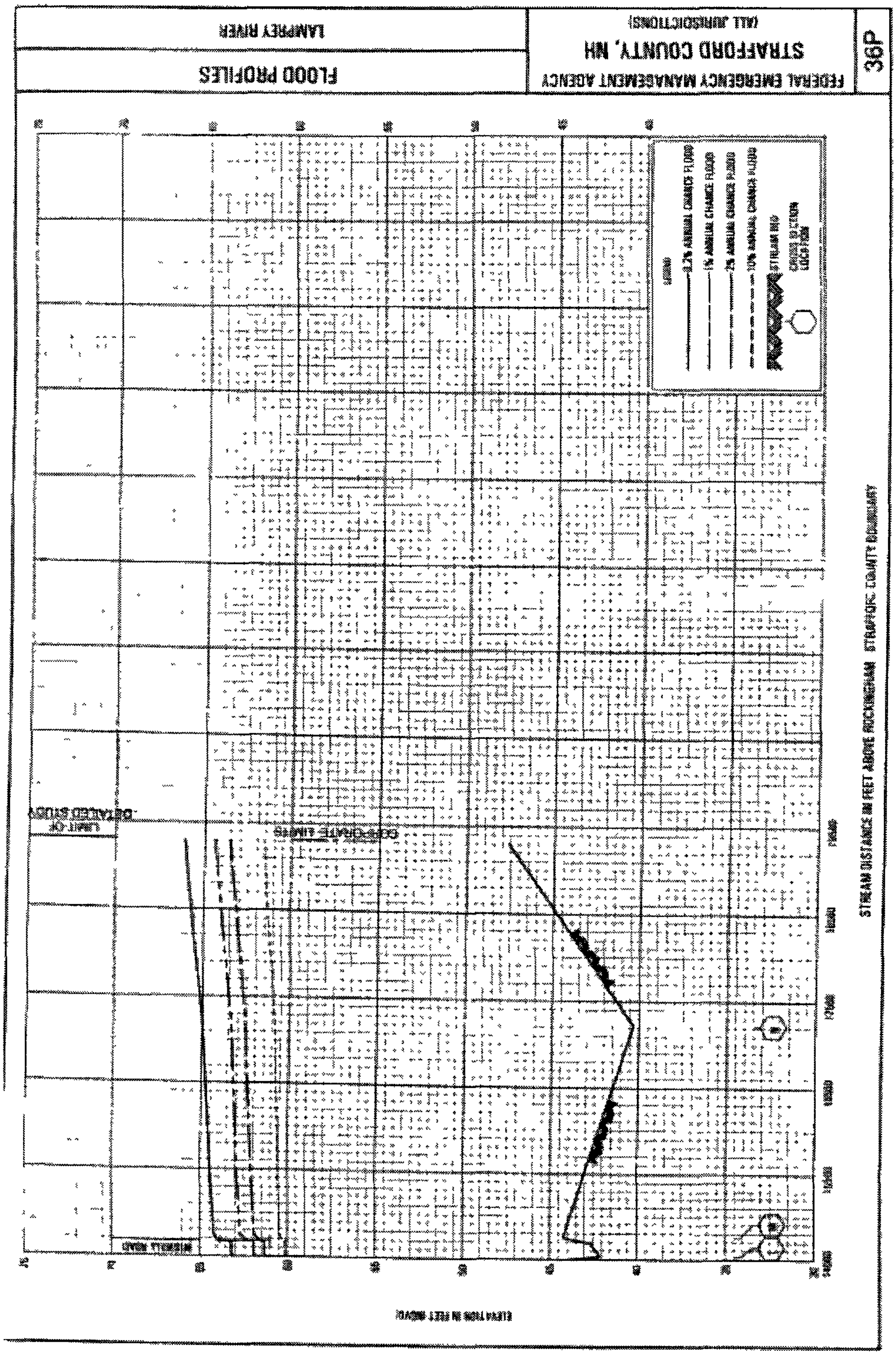
LAMPREY RIVER

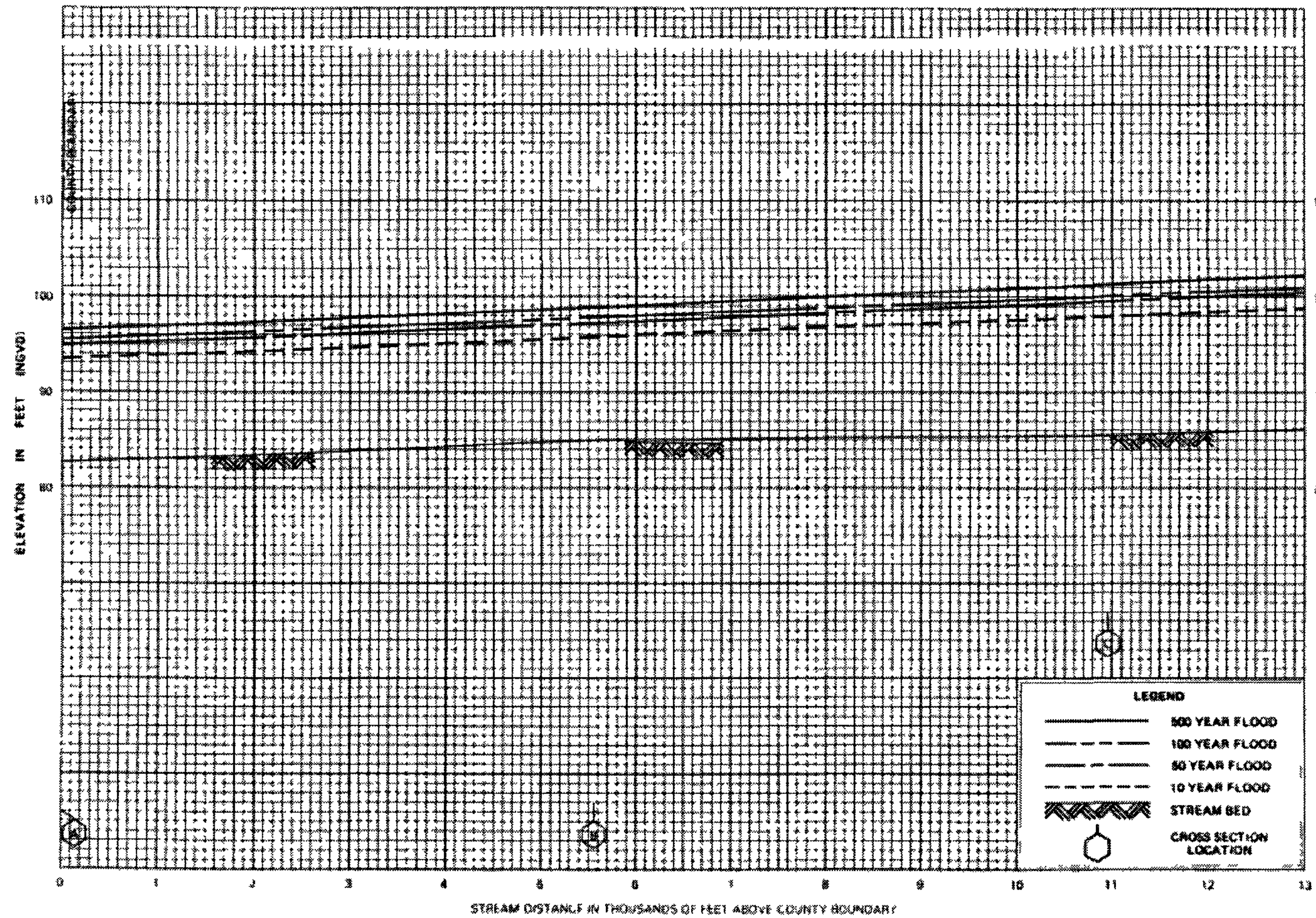
FEDERAL EMERGENCY MANAGEMENT AGENCY

STRAFFORD COUNTY, NH

(ALL JURISDICTIONS)

35P


FEDERAL EMERGENCY MANAGEMENT AGENCY
STRAFFORD COUNTY, NH
(ALL JURISDICTIONS)
36P
FLOOD PROFILES
LAMPREY RIVER
ELEVATION IN FEET (NGVD)
STREAM DISTANCE IN FEET ABOVE ROCKINGHAM STRAFFORD COUNTY BOUNDARY
CORPORATE LIMITS
LIMIT OF DETAILED STUDY



FLOOD PROFILES

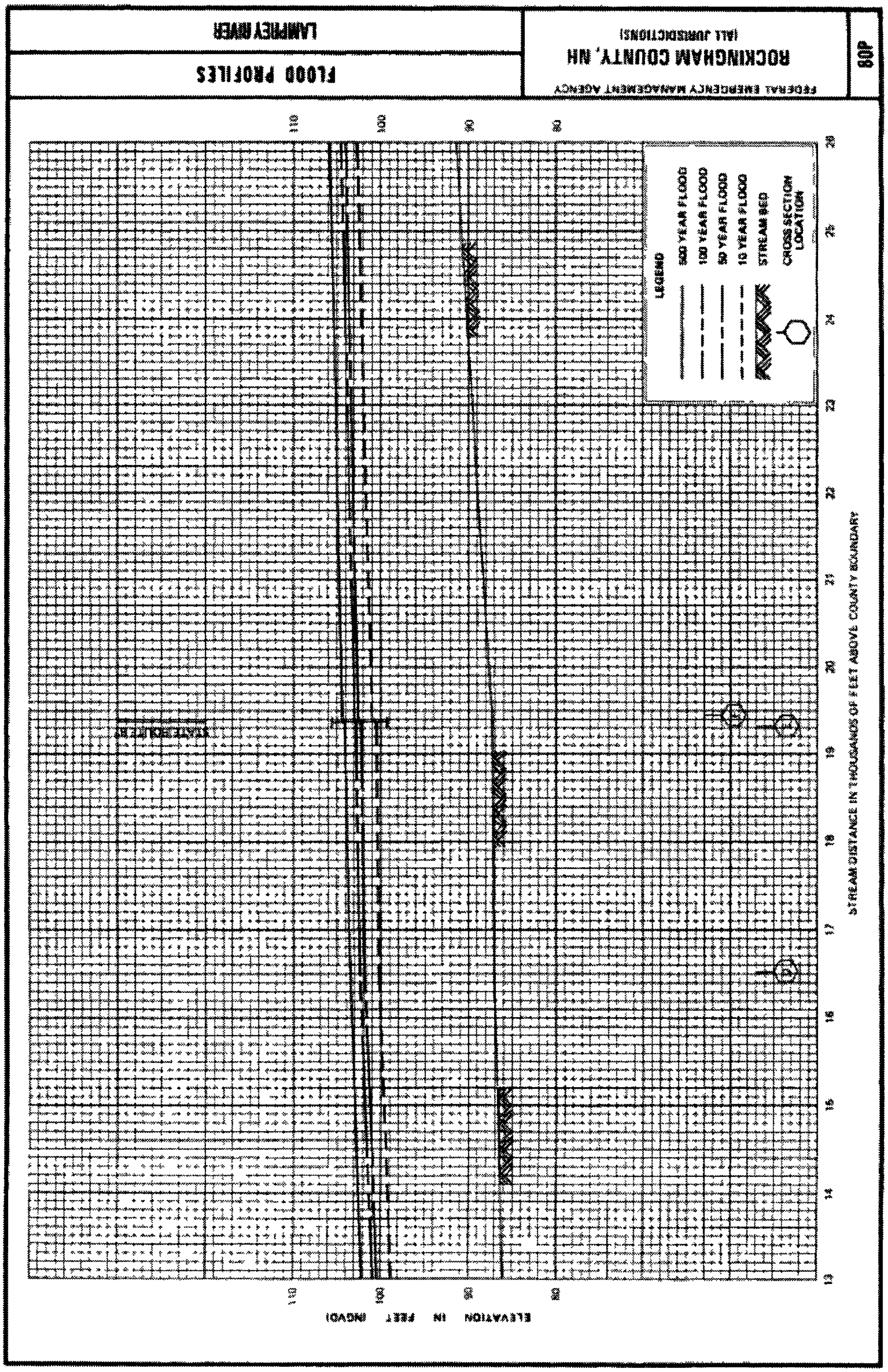
LAMPREY RIVER

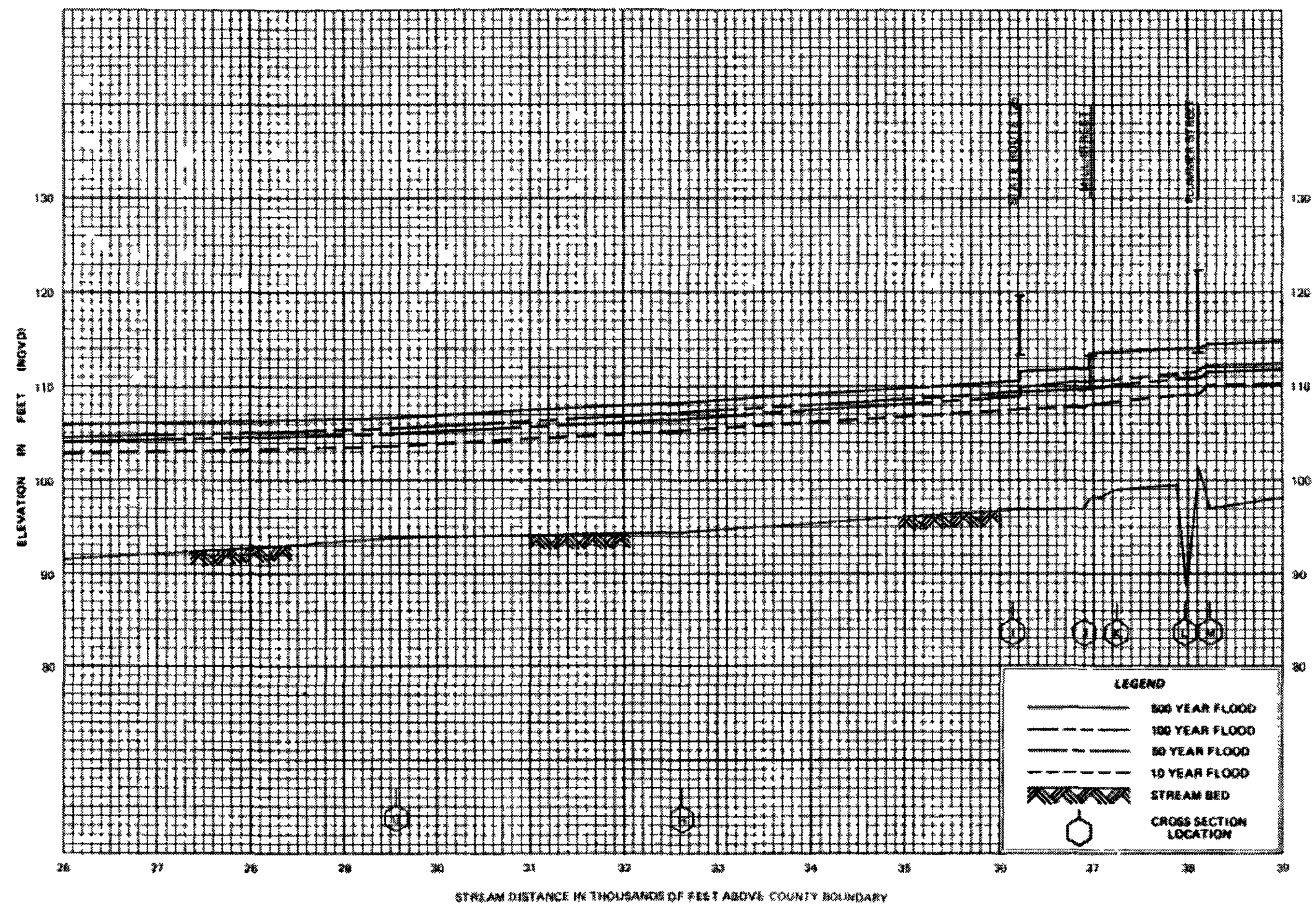
FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

79P


FEDERAL EMERGENCY MANAGEMENT AGENCY
ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)
80P
FLOOD PROFILES
LAMPREY RIVER
LEGEND
500 YEAR FLOOD
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
ELEVATION IN FEET (NGVD)
80
90
100
110
STREAM DISTANCE IN THOUSANDS OF FEET ABOVE COUNTY BOUNDARY
13
14
15
16
17
18
19
20
21
22
23
24
25
26



FEDERAL EMERGENCY MANAGEMENT AGENCY

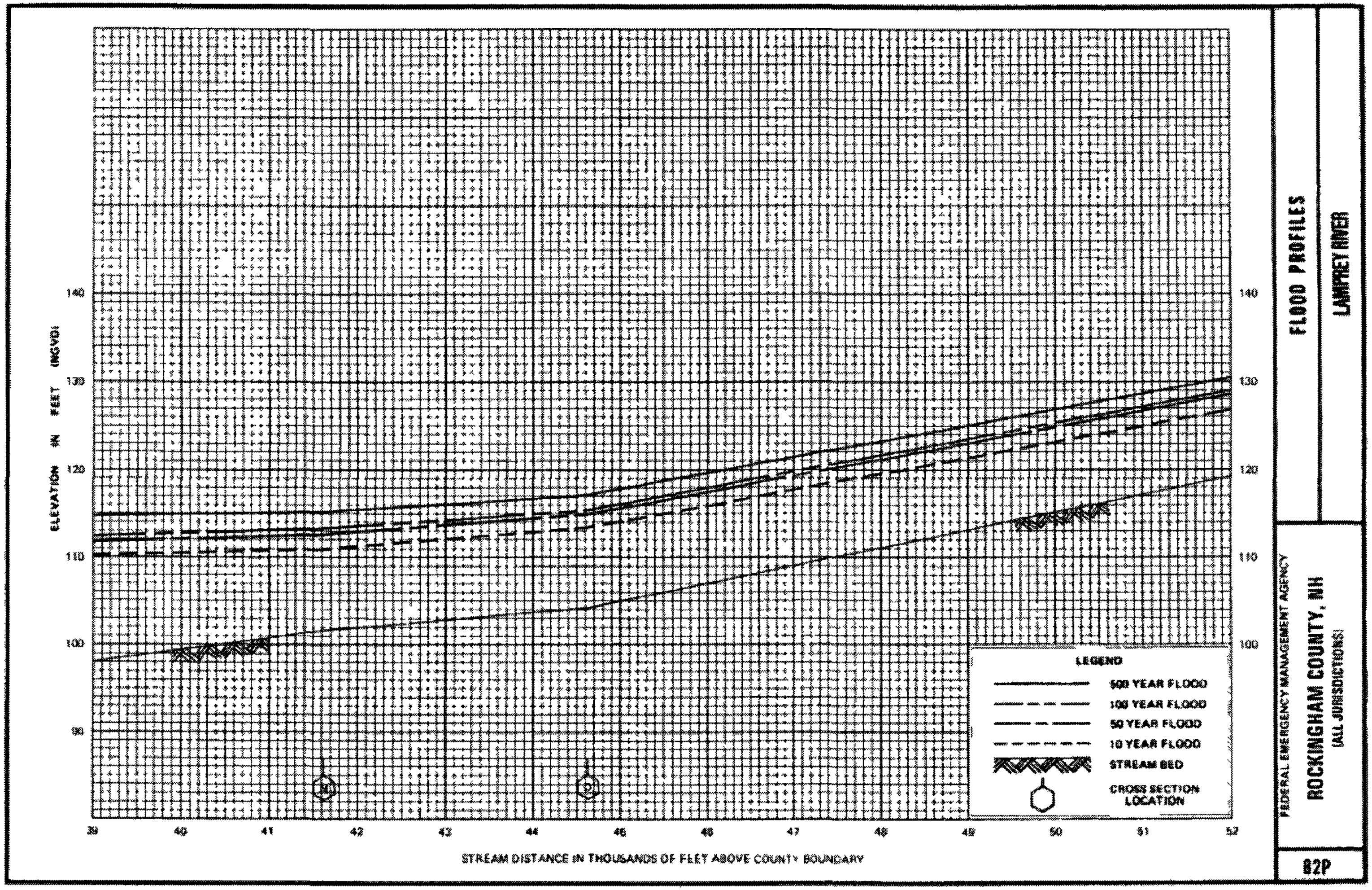
ROCKINGHAM COUNTY, NH

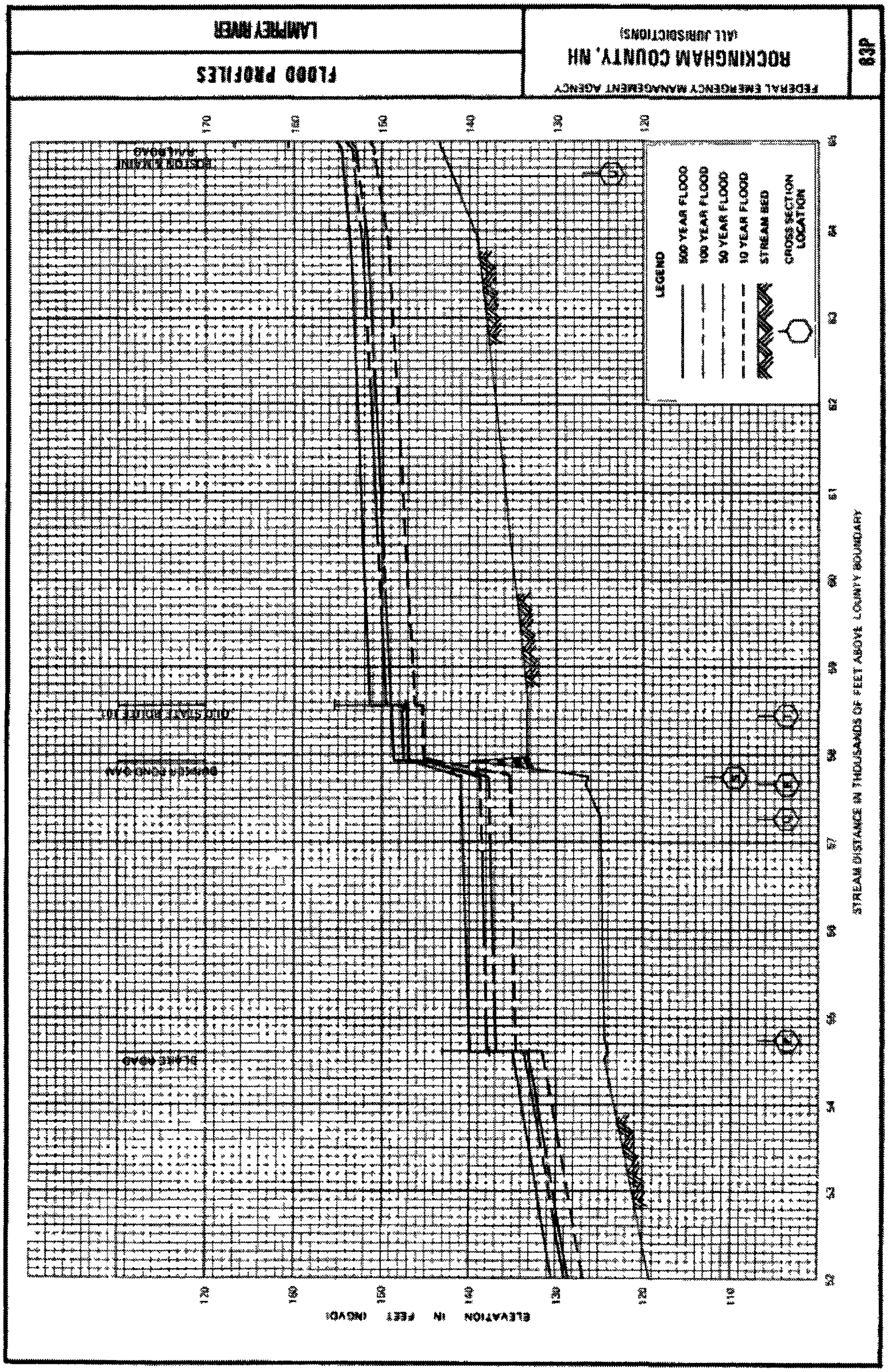
(ALL JURISDICTIONS)

FLOOD PROFILES

LAMPREY RIVER

81P


FLOOD PROFILES
LAMPREY RIVER
FEDERAL EMERGENCY MANAGEMENT AGENCY
ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)
82P
LEGEND
500 YEAR FLOOD
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
ELEVATION IN FEET (NGVD)
STREAM DISTANCE IN THOUSANDS OF FEET ABOVE COUNTY BOUNDARY
90
100
110
120
130
140
39
40
41
42
43
44
45
46
47
48
49
50
51
52


FLOOD PROFILES
LAMPREY RIVER
FEDERAL EMERGENCY MANAGEMENT AGENCY
ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)
63P
LEGEND
500 YEAR FLOOD
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
ELEVATION IN FEET (NGVD)
STREAM DISTANCE IN THOUSANDS OF FEET ABOVE COUNTY BOUNDARY

FEDERAL EMERGENCY MANAGEMENT AGENCY

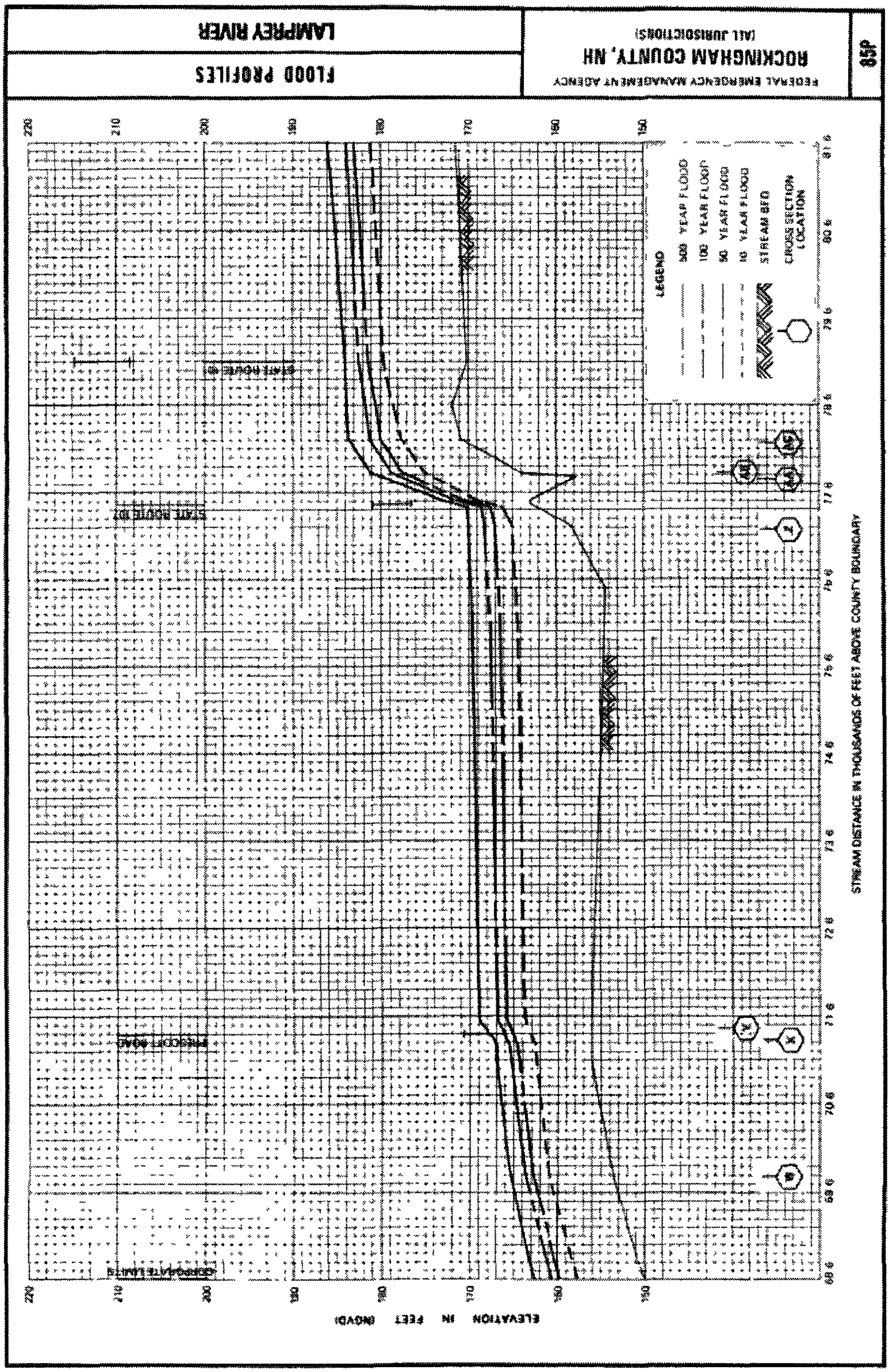
ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

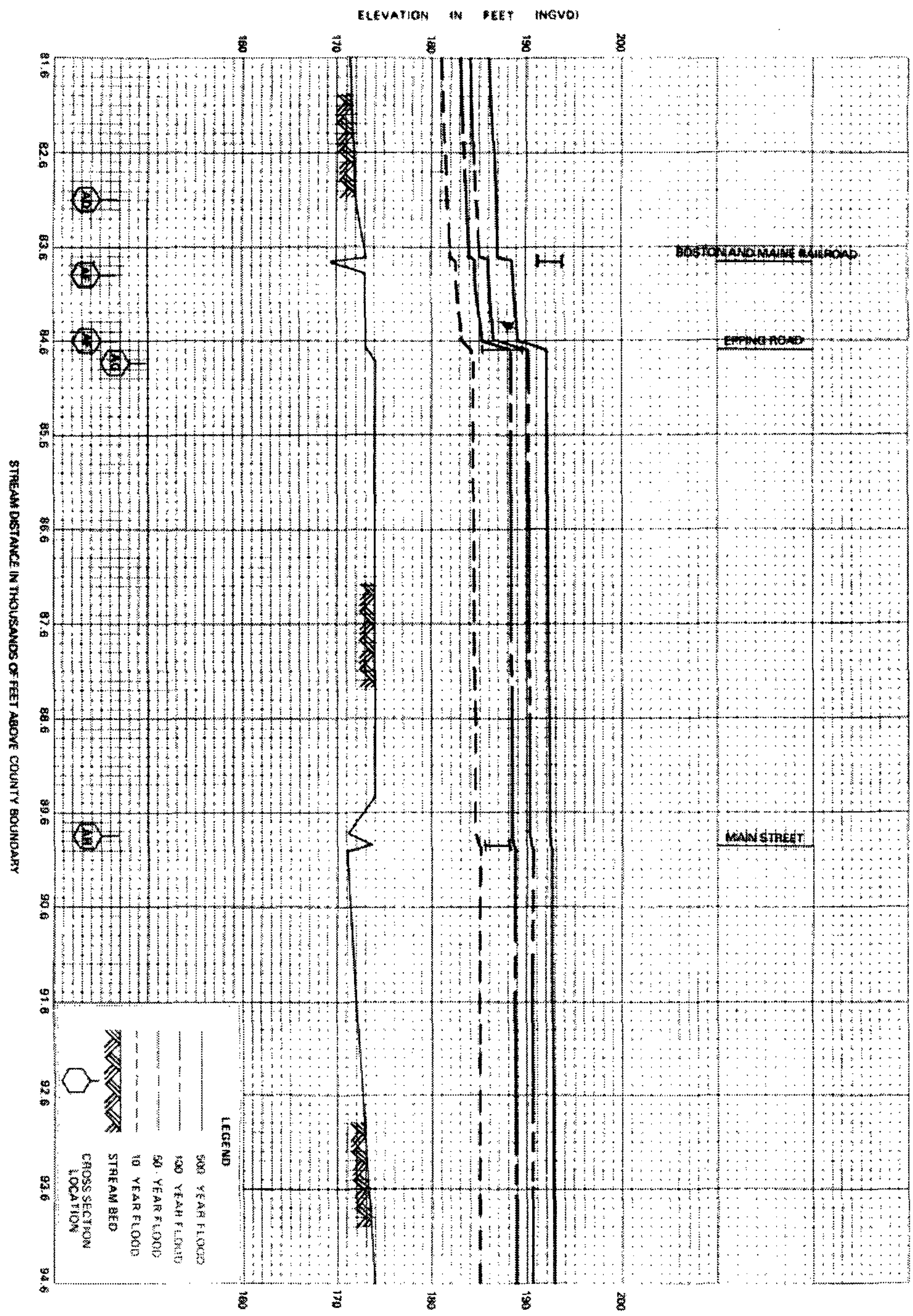
FLOOD PROFILES

LAMPREY RIVER

84P



FEDERAL EMERGENCY MANAGEMENT AGENCY
ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)
FLOOD PROFILES
LAMPREY RIVER
85P
LEGEND
500 YEAR FLOOD
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
STATE ROUTE 107
PRESCOTT ROAD
CORPORATE LIMITS
ELEVATION IN FEET (NGVD)
STREAM DISTANCE IN THOUSANDS OF FEET ABOVE COUNTY BOUNDARY



FEDERAL EMERGENCY MANAGEMENT AGENCY

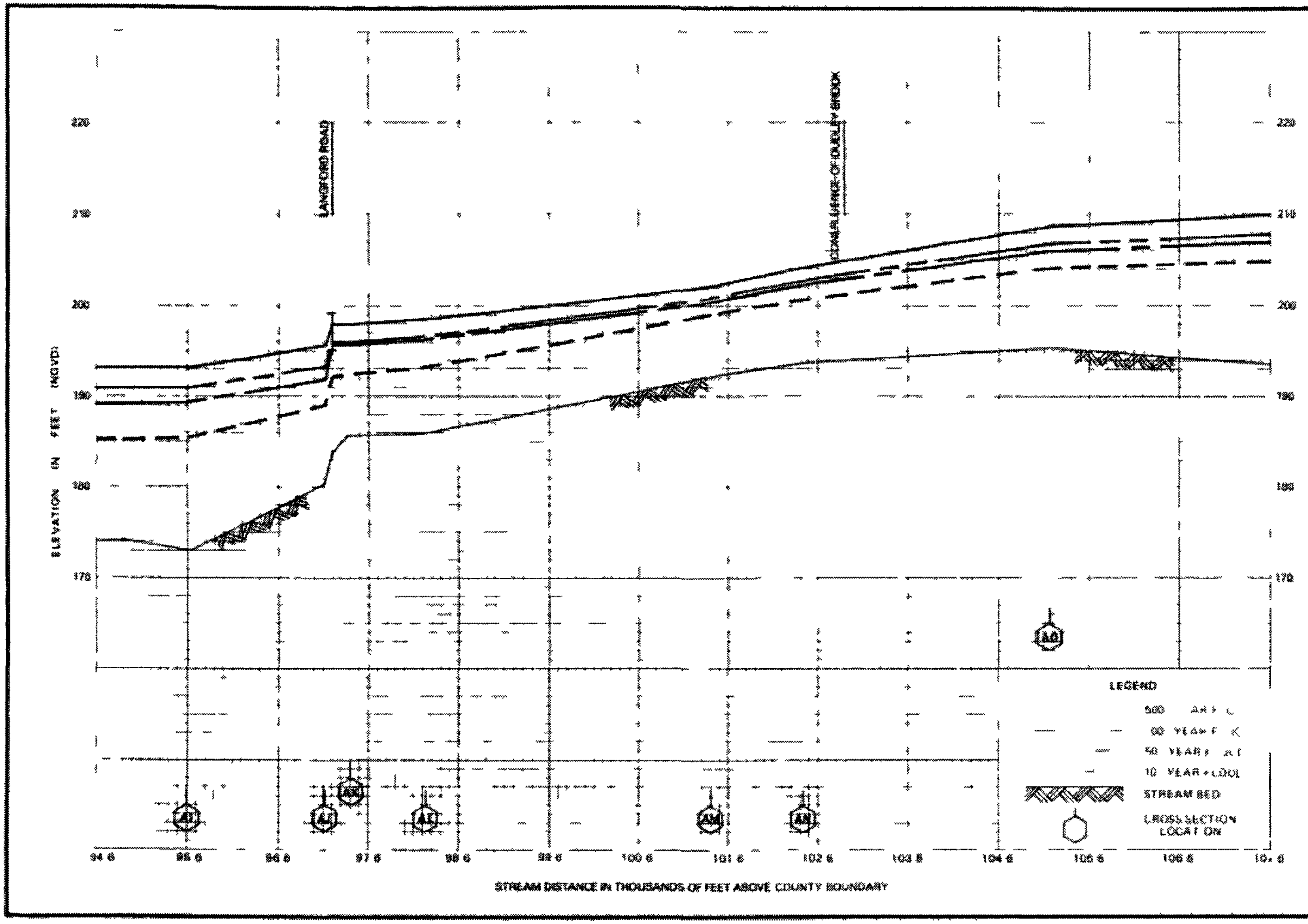
ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOOD PROFILES

LAMPREY RIVER

86P



FLOOD PROFILES

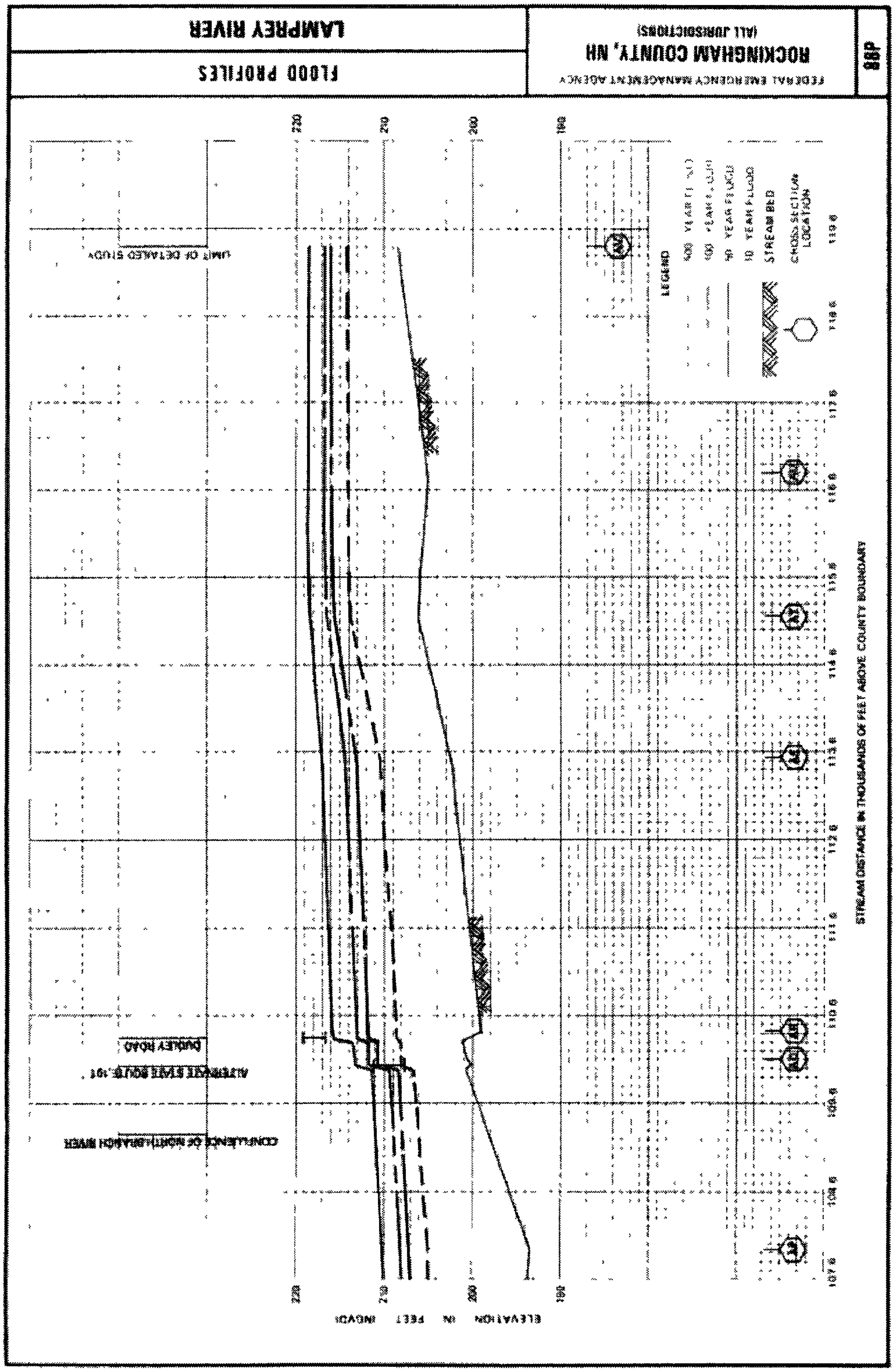
LAMPREY RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

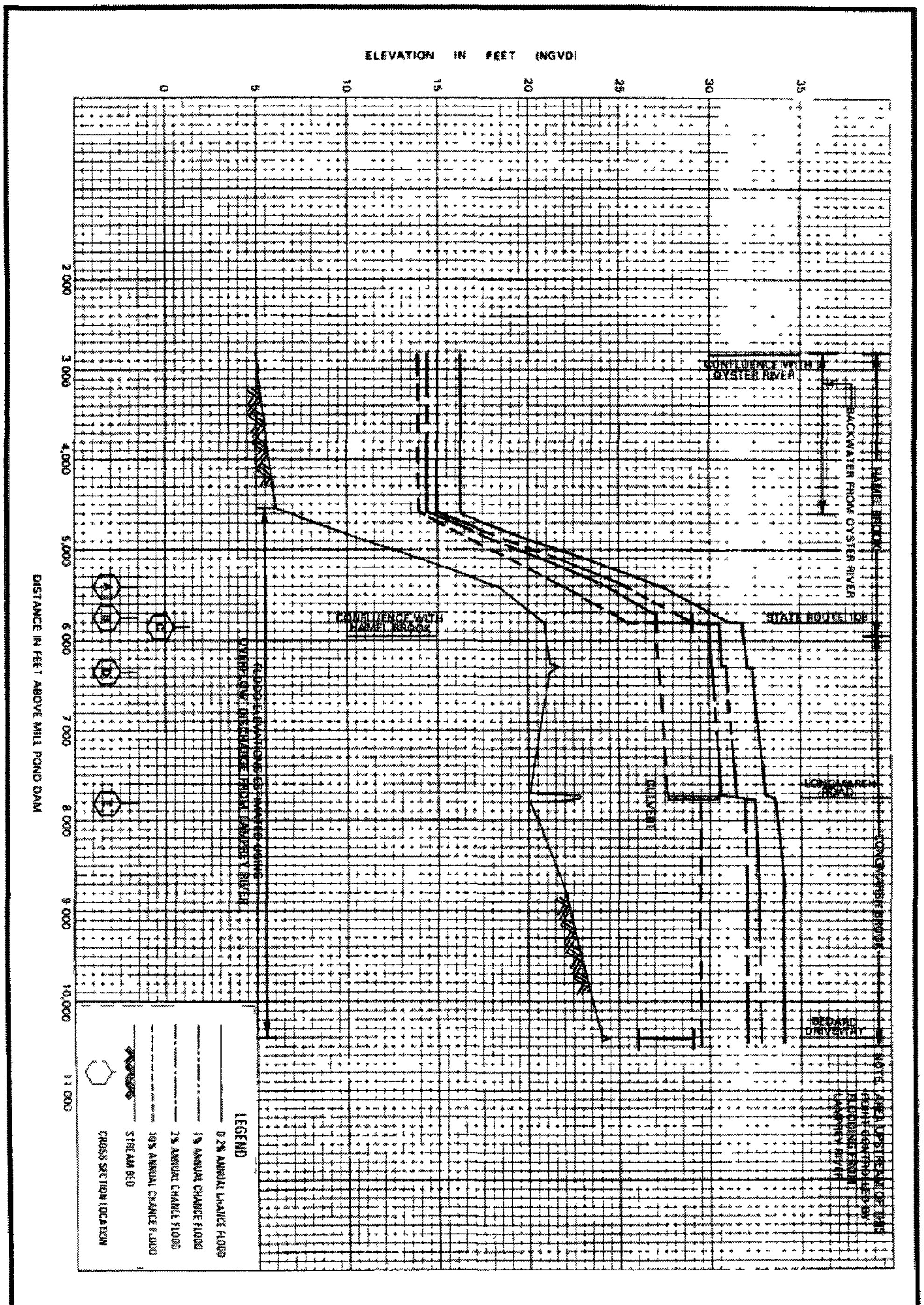
(ALL JURISDICTIONS)

87P



LAMPREY RIVER
FLOOD PROFILES
FEDERAL EMERGENCY MANAGEMENT AGENCY
ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)
88P
LEGEND
500 YEAR FLOOD
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
LIMIT OF DETAILED STUDY
ALTERNATE STATE ROUTE 101
DUDLEY ROAD
CONFLUENCE OF NORTH BRANCH RIVER
ELEVATION IN FEET (NGVD)
STREAM DISTANCE IN THOUSANDS OF FEET ABOVE COUNTY BOUNDARY

Hamil Brook



29P	FEDERAL EMERGENCY MANAGEMENT AGENCY **STRAFFORD COUNTY, NH** (ALL JURISDICTIONS)	**FLOOD PROFILES** **HAMEL BROOK – LONGMARSH BROOK**

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
190667.1	3213.7	216.73	216.95	216.89	216.8	AV	0.15	0.22	0.09	0.06
187998.5	3213.7	216.51	216.74	216.67	216.7	AU	0.04	0.23	-0.03	0.07
186330.5	3213.7	215.72	216	215.91	216.4	AT	-0.4	0.28	-0.49	0.09
184688.1	3213.7	214.36	214.72	214.61	214.4	AS	0.32	0.36	0.21	0.11
181380.6	3213.7	213.04	213.47	213.34	213	AR	0.47	0.43	0.34	0.13
181317	3213.7	212.77	213.19	213.06						
181300	Bridge				Dudley Road, Raymond					
181280	3213.7	212.37	212.8	212.67	211.7	UU-1D		0.43		0.13
181044	3213.7	212.51	212.94	212.81	211.7	AQ	1.24	0.43	1.11	0.13
180981	3213.7	212.31	212.76	212.62				0.45		0.14
180964	Bridge				Route 27, Raymond					
180924	3213.7	210.47	210.74	210.65				0.27		
178854.2	5137.9	209.89	210.16	210.07	207.9	AP	2.26	0.27	2.17	0.09
176106.1	5137.9	208.36	208.6	208.52	206.7	AO	1.9	0.24	1.82	0.08
173299.7	5137.9	202.38	202.49	202.45	202.6	AN	-0.11	0.11	-0.15	0.04
172347.7	5137.9	201.06	201.36	201.26	200.6	AM	0.76	0.3	0.66	0.1
168993	5137.9	199.29	199.75	199.6	196.4	AL	3.35	0.46	3.2	0.15
168069.7	5137.9	198.96	199.43	199.28	195.8	AK	3.63	0.47	3.48	0.15
167917	5137.9	198.51	198.97	198.82				0.46		0.15
167900	Bridge				Langford Road, Raymond					
167883	5137.9	196.34	196.56	196.49				0.22		0.07
167810	5137.9	192.76	192.94	192.89				0.18		0.05
167800.8	5137.9	193.09	193.26	193.18	193.1	AJ	0.16	0.17	0.08	0.08
166302.9	5137.9	193.37	193.55	193.47	190.8	AI	2.75	0.18	2.67	0.08
165726	5137.9	193.36	193.54	193.46				0.18		0.08
160796	5137.9	193.17	193.35	193.27				0.18		0.08
160763	5137.9	192.84	193.01	192.93				0.17		0.08

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
160746	Bridge				Main Street, Raymond					
160729	5137.9	192.21	192.46	192.25				0.25		
160646.2	6277.9	192.25	192.49	192.28	190.3	AH	2.19	0.24	1.98	0.21
160246	6277.9	192.24	192.49	192.27				0.25		0.22
155206.2	6277.9	192.02	192.26	192.04	190.1	AG	2.16	0.24	1.94	0.22
155070	6277.9	191.73	191.97	191.74				0.24		0.23
155060	Bridge				Epping Road, Raymond					
155043	6277.9	187.84	188.18	188.07				0.34		0.11
154959.6	6277.9	187.87	188.22	188.11	186.4	AF	1.82	0.35	1.71	0.11
154211.9	6277.9	187.54	187.84	187.77	185.9	AE	1.94	0.3	1.87	0.07
154123	6277.9	187.19	187.5	187.41				0.31		0.09
154106.3	Bridge				B&M Railroad					
154089	6277.9	187.2	187.51	187.42				0.31		0.09
154056.3	6277.9	187.26	187.58	187.48				0.32		0.1
153416.3	6277.9	187.1	187.43	187.33	184.7	AD	2.73	0.33	2.63	0.1
149219.5	6277.9	184.35	184.67	184.57				0.32		0.1
148775.8	6277.9	183.59	183.94	183.83				0.35		0.11
148747.3	6277.9	183.73	184.08	183.97				0.35		0.11
148397.3	6277.9	183.52	183.88	183.77	181	AC	2.88	0.36	2.77	0.11
148051.9	6277.9	181.43	181.81	181.69	178.6	AB	3.21	0.38	3.09	0.12
147932.1	6277.9	176.62	176.82	176.76	177.1	AA	-0.28	0.2	-0.34	0.06
147683.2	6277.9	172.49	172.73	172.66				0.24		0.07
147660	6277.9	172.38	172.63	172.55				0.25		0.08
147643.2	Bridge				State Route 107, Raymond					
147626	6277.9	168.98	169.4	169.27				0.42		0.13
147603.2	6277.9	169.73	170.13	170.01				0.4		0.12
147373.2	6277.9	169.77	170.17	170.05	167.8	Z	2.37	0.4	2.25	0.12

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
146452.2	6277.9	169.7	170.1	169.98				0.4		0.12
141672.2	6277.9	169	169.42	169.29				0.42		0.13
141522.2	6277.9	168.97	169.39	169.26				0.42		0.13
141442.2	6277.9	168.56	168.99	168.86	166.3	Y	2.69	0.43	2.56	0.13
141387.6	6277.9	167.99	168.39	168.27				0.4		0.12
141372.6	Bridge				Prescott Road, Raymond					
141355	6277.9	165.85	166.05	165.99				0.2		0.06
141318.7	6277.9	166.34	166.57	166.5	165.7	X	0.87	0.23	0.8	0.07
141237.3	6277.9	166.25	166.49	166.41				0.24		0.08
141141	6277.9	166.12	166.36	166.29				0.24		0.07
139946.1	6277.9	163.46	163.79	163.69	163.7	W	0.09	0.33	-0.01	0.1
136946.6	6277.9	162.71	163.02	162.93				0.31		0.09
136820	6277.9	162.11	162.39	162.3				0.28		0.09
136759.6	Bridge				State Route 101, Epping					
136697	6277.9	158.69	158.93	158.86				0.24		0.07
136639.6	6171.9	158.74	158.99	158.91	155.4	V	3.59	0.25	3.51	0.08
134384.7	6171.9	157.9	158.14	158.07	153	U	5.14	0.24	5.07	0.07
133485.2	6171.9	154.73	154.92	154.86				0.19		0.06
130390.2	6171.9	152.28	152.44	152.41				0.16		0.03
129065.2	6171.9	152.17	152.33	152.3				0.16		0.03
128505.2	6171.9	152.16	152.32	152.29				0.16		0.03
127977.2	6171.9	151.89	152.03	152.01				0.14		0.02
127957.4	6171.9	150.48	150.48	150.51				0		-0.03
127937.2	Bridge				Route 27, Epping					
127917	6171.9	147.62	147.59	147.6				-0.03		-0.01
127842.2	6171.9	148.91	149.05	149.01				0.14		0.04
127810.5	6171.9	149.4	149.56	149.51	147.6	T	1.96	0.16	1.91	0.05

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
127552.2	6171.9	149.26	149.41	149.36				0.15		0.05
127334.2	6171.9	149.09	149.23	149.19				0.14		0.04
127265.5	Inl Struct	Bunker Pond Dam								
127067.2	6171.9	144.15	144.35	144.28				0.2		0.07
126921.1	6171.9	144.14	144.34	144.28	138.9	S	5.44	0.2	5.38	0.06
126831	6171.9	144.12	144.32	144.26	138.9	R	5.42	0.2	5.36	0.06
126647.2	6707.8	144.14	144.34	144.28				0.2		0.06
126451.6	6707.8	144.08	144.28	144.22	138.8	Q	5.48	0.2	5.42	0.06
123979.1	6707.8	142.13	142.18	142.16	138	P	4.18	0.05	4.16	0.02
123964	Bridge	Blake Road, Epping								
123949	6707.8	135.77	135.72	135.72				-0.05		0
114102.7	6707.8	116.9	117.29	117.18	115.6	O	1.69	0.39	1.58	0.11
111088.9	6708.1	116	116.45	116.33	113.5	N	2.95	0.45	2.83	0.12
107630.3	6708.1	114.9	115.41	115.28	112.3	M	3.11	0.51	2.98	0.13
107480	6708.1	114.52	115	114.88				0.48		0.12
107459	Bridge	Main Street (Plummer), Epping								
107438	6708.1	114.32	114.79	114.67				0.47		0.12
107370.9	6708.1	114.55	115.03	114.91	111.5	L	3.29	0.48	3.41	0.12
106610.3	6708.1	114.32	114.81	114.68	110.8	K	4.23	0.49	3.88	0.13
106389	6708.1	114.12	114.62	114.5				0.5		0.12
106269	Bridge	Mill Street, Epping								
106249	6708.1	112.28	112.6	112.51				0.32		0.09
106169.7	6708.1	111.93	112.22	112.14	110.4	J	1.82	0.29	1.74	0.08
105755	6708.1	111.69	111.99	111.91				0.3		0.08
105620	6708.1	111.4	111.7	111.62				0.3		0.08
105560	Bridge	State Route 125, Epping								
105530	6708.1	110.77	111.05	110.98				0.28		0.07

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
105450.7	6708.1	110.96	111.26	111.18	109.5	I	1.76	0.3	1.68	0.08
101956.4	6708.1	108.91	109.3	109.2	107.2	H	2.1	0.39	2	0.1
98842.17	6708.1	108.16	108.61	108.51	105.6	G	3.01	0.45	2.91	0.1
88321	6708.1	106.74	107.29	107.18				0.55		0.11
88195	6708.1	106.65	107.19	107.08				0.54		0.11
88171	Mult Open				Route 87, Epping[1]					
88041	6708.1	106.24	106.73	106.64				0.49		0.09
85281.58	6708.1	105.58	105.87	105.77	102.3	D	3.57	0.29	3.47	0.1
79594.89	6708.1	104	104.31	104.2	100.1	C	4.21	0.31	4.1	0.11
73925.28	6708.1	103.31	103.62	103.5	97.9	B	5.72	0.31	5.6	0.12
67214.17	10436.9	101.99	102.31	102.19	95.4	A	6.91	0.32	6.79	0.12
61707.38	10436.9	99.55	99.93	99.79		Srvy starts		0.38		0.14
61492	10436.9	99.29	99.66	99.52				0.37		0.14
61457	Bridge				Wadleigh Falls Road, Lee					
61427	10436.9	95.21	95.45	95.37				0.24		0.08
61266.58	Inl Struct				Wadleigh Falls Dam, Lee					
60189.33	10436.9	88.5	88.77	88.67				0.27		0.1
58147.13	10436.9	87.67	87.94	87.84				0.27		0.1
54529.59	10436.9	86.5	86.78	86.68				0.28		0.1
45448.66	10436.9	84.65	85	84.87				0.35		0.13
35780.01	10436.9	81.73	82.2	82.02				0.47		0.18
35697	10436.9	80.74	81.18	81.01				0.44		0.17
35683	Bridge				Lee Hook Road, Lee					
35669	10436.9	75.51	75.73	75.64				0.22		0.09
35247.9	10436.9	75.59	75.84	75.74				0.25		0.1
33376.17	10436.9	68.84	69.01	68.95		Srvy Ends		0.17		0.06
22681.01	10436.9	64.53	64.79	64.68	63.4	N	1.39	0.26	1.28	0.11

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
20163	10436.9	64.07	64.32	64.21	62.7	M	1.62	0.25	1.51	0.11
20112	10436.9	63.44	63.65	63.56				0.21		0.09
20082	Bridge				Wiswall Road, Durham					
20073	10436.9	63.18	63.35	63.28				0.17		0.07
19934	10436.9	63.5	63.69	63.61	62	L	1.69	0.19	1.61	0.08
19908	10436.9	63.43	63.61	63.54				0.18		0.07
19863	10436.9	63.42	63.61	63.53				0.19		0.08
19859.91	Inl Struct				Wiswall Dam, Durham					
19859.55	10436.9	61.11	61.19	61.15				0.08		0.04
19842.98	10649	60.97	61.75	61.47	54.4	K	7.35	0.78	7.07	0.28
19367.12	10649	60.86	61.64	61.36	54.1	J	7.54	0.78	7.26	0.28
17730.92	10649	60.78	61.58	61.29	53.7	I	7.88	0.8	7.59	0.29
16215.78	10649	60.68	61.47	61.19	53.5	H	7.97	0.79	7.69	0.28
16117.94	10649	60.61	61.4	61.12	53.4	G	8	0.79	7.72	0.28
16077.05	10649	60.48	61.31	61.01				0.83		0.3
16047	10649	58.38	60.61	60.07				2.23		0.54
16028	Bridge				Packer's Falls Road, Durham					
16009	10649	51.88	52.11	52.03				0.23		0.08
15843.92	10649	47.94	48.15	48.08				0.21		0.07
15531.77	10649	41.5	41.83	41.73	39.5	F	2.33	0.33	2.23	0.1
15474.44	10649	41.51	41.84	41.74				0.33		0.1
15046.53	10649	41.39	41.72	41.61				0.33		0.11
14970.24	10649	39	39.28	39.21	38.3	E	0.98	0.28	0.91	0.07
14021.84	10649	37.79	38.08	38.02	32.9	D	5.18	0.29	5.12	0.06
13857.9	10649	38.06	38.35	38.29				0.29		0.06
13223.32	10649	37.96	38.25	38.19	33.4	C	4.85	0.29	4.79	0.06
11347.97	10649	37.62	37.9	37.84	33	B	4.9	0.28	4.84	0.06

River Sta	Q Total	WSE (ft)	WSE (ft)	WSE (ft)	FIS		2050 Conv.	2050 Conv.	2050 LID	2050 Conv.
	(cfs)	2005 NRCC	2050 Conv	2050LID	W.S. Elev.	Station	to FIS	to 2005 NRCC	to FIS	to 2050 LID
11291.77	10649	37.62	37.9	37.84				0.28		0.06
11230.52	10649	37.59	37.87	37.81				0.28		0.06
10739.63	10649	36.89	37.15	37.1				0.26		0.05
9851.749	10649	37.03	37.29	37.25				0.26		0.04
9113	10649	36.96	37.22	37.18				0.26		0.04
8998	10649	36.98	37.24	37.2				0.26		0.04
8890	9945.15	36.8	37.02	36.98				0.22		0.04
8855.391	9945.15	36.95	37.19	37.14	32.8	A	4.39	0.24	4.34	0.05
6238.36	9945.15	36.88	37.11	37.06				0.23		0.05
5865	9945.15	36.52	36.71	36.67				0.19		0.04
5568	11362.95	36.47	36.66	36.62				0.19		0.04
3250.152	11362.95	36.45	36.64	36.6				0.19		0.04
3067.628	11362.95	36.38	36.55	36.52				0.17		0.03
2053.77	11362.95	36.32	36.49	36.46				0.17		0.03
1842.21	11362.95	36.19	36.34	36.32				0.15		0.02
1645	11362.95	35.66	35.74	35.73				0.08		0.01
1619	11362.95	34.27	34.08	34.14				-0.19		-0.06
1602.5	Bridge				RT 108, Newmarket					
1586	11362.95	34.1	33.93	33.98				-0.17		-0.05
1560.457	11362.95	34.12	33.96	34				-0.16		-0.04
1328.75	11362.95	35.05	35.04	35.05				-0.01		-0.01
1287.71	11362.95	34.97	34.95	34.96				-0.02		-0.01
1286.71	Inl Struct				Coffe Sluice - Macallen Dam					
1267.71	11362.95	34.8	34.76	34.77				-0.04		-0.01
1182.71	11362.95	34.73	34.68	34.7				-0.05		-0.02
1163.71	Inl Struct				Macallen Dam					
1147.93	11362.95	33.47	34.01	33.89				0.54		0.12

Appendix G – HEC-RAS Tables

RS: 181300 Profile: NRCC 2005

E.G. US. (ft)	213.07	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	212.77	E.G. Elev (ft)	213.00	212.92
Q Total (cfs)	3213.70	W.S. Elev (ft)	212.48	212.31
Q Bridge (cfs)	3213.70	Crit W.S. (ft)	205.61	206.37
Q Weir (cfs)		Max Chl Dpth (ft)	13.48	11.31
Weir Sta Lft (ft)		Vel Total (ft/s)	5.77	6.24
Weir Sta Rgt (ft)		Flow Area (sq ft)	557.14	514.80
Weir Submerg		Froude # Chl	0.28	0.33
Weir Max Depth (ft)		Specif Force (cu ft)	4000.64	3535.38
Min El Weir Flow (ft)	218.61	Hydr Depth (ft)	12.24	11.31
Min El Prs (ft)	216.70	W.P. Total (ft)	71.36	68.13
Delta EG (ft)	0.22	Conv. Total (cfs)	81455.4	61367.3
Delta WS (ft)	0.40	Top Width (ft)	45.50	45.50
BR Open Area (sq	714.35	Frctn Loss (ft)	0.05	0.01
BR Open Vel (ft/s)	6.24	C & E Loss (ft)	0.03	0.06
Coef of Q		Shear Total (lb/sq ft)	0.76	1.29
Br Sel Method	Energy only	Power Total (lb/ft s)	-62.00	-62.00

RS: 180964 Profile: NRCC 2005

E.G. US. (ft)	212.52	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	212.31	E.G. Elev (ft)	212.51	210.66
Q Total (cfs)	3213.70	W.S. Elev (ft)	207.70	207.70
Q Bridge (cfs)	3213.70	Crit W.S. (ft)	206.29	206.05
Q Weir (cfs)		Max Chl Dpth (ft)	7.60	7.00
Weir Sta Lft (ft)		Vel Total (ft/s)	9.19	8.84
Weir Sta Rgt (ft)		Flow Area (sq ft)	349.76	363.52
Weir Submerg		Froude # Chl	0.59	0.59
Weir Max Depth (ft)		Specif Force (cu ft)	2039.17	2038.62
Min El Weir Flow (ft)	213.30	Hydr Depth (ft)		
Min El Prs (ft)	207.70	W.P. Total (ft)	141.22	149.20
Delta EG (ft)	1.86	Conv. Total (cfs)	23784.3	24451.9
Delta WS (ft)	1.85	Top Width (ft)		
BR Open Area (sq	349.76	Frctn Loss (ft)		
BR Open Vel (ft/s)	9.19	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)	2.82	2.63
Br Sel Method	Press Only	Power Total (lb/ft s)	124.30	124.30

RS: 167900 Profile: NRCC 2005

E.G. US. (ft)	198.98	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	198.51	E.G. Elev (ft)	198.97	198.83
Q Total (cfs)	5137.90	W.S. Elev (ft)	198.51	198.51
Q Bridge (cfs)	4913.12	Crit W.S. (ft)	190.89	193.40
Q Weir (cfs)	3.74	Max Chl Dpth (ft)	14.61	14.44
Weir Sta Lft (ft)	426.00	Vel Total (ft/s)	0.00	10.72
Weir Sta Rgt (ft)	498.00	Flow Area (sq ft)		479.21
Weir Submerg	0.00	Froude # Chl	0.37	0.51
Weir Max Depth (ft)	0.42	Specif Force (cu ft)	6703.20	5430.76
Min El Weir Flow (ft)	198.57	Hydr Depth (ft)		
Min El Prs (ft)	195.10	W.P. Total (ft)	146.54	137.57
Delta EG (ft)	1.31	Conv. Total (cfs)		
Delta WS (ft)	2.17	Top Width (ft)		
BR Open Area (sq	471.05	Frctn Loss (ft)		
BR Open Vel (ft/s)	10.43	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	279.00	355.00

RS: 160746 Profile: NRCC 2005

E.G. US. (ft)	193.17	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	192.84	E.G. Elev (ft)	193.16	192.38
Q Total (cfs)	5137.90	W.S. Elev (ft)	192.84	192.21
Q Bridge (cfs)	2793.73	Crit W.S. (ft)	182.66	181.57
Q Weir (cfs)	2218.46	Max Chl Dpth (ft)	18.94	20.74
Weir Sta Lft (ft)	60.74	Vel Total (ft/s)	4.80	2.89
Weir Sta Rgt (ft)	460.74	Flow Area (sq ft)	1070.27	1776.83
Weir Submerg	0.39	Froude # Chl	0.28	0.17
Weir Max Depth (ft)	1.81	Specif Force (cu ft)	7025.98	8226.78
Min El Weir Flow (ft)	191.37	Hydr Depth (ft)	2.68	4.44
Min El Prs (ft)	185.50	W.P. Total (ft)	534.93	540.90
Delta EG (ft)	0.79	Conv. Total (cfs)		
Delta WS (ft)	0.63	Top Width (ft)	400.00	400.00
BR Open Area (sq	444.60	Frctn Loss (ft)		
BR Open Vel (ft/s)	6.28	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	60.74	60.74

RS: 155060 Profile: NRCC 2005

E.G. US. (ft)	192.02	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	191.73	E.G. Elev (ft)	192.01	191.18
Q Total (cfs)	6277.90	W.S. Elev (ft)	191.73	190.96
Q Bridge (cfs)	6003.24	Crit W.S. (ft)	183.00	183.01
Q Weir (cfs)	348.67	Max Chl Dpth (ft)	18.83	18.06
Weir Sta Lft (ft)	-10.45	Vel Total (ft/s)	11.27	7.88
Weir Sta Rgt (ft)	642.55	Flow Area (sq ft)	557.13	796.88
Weir Submerg	0.00	Froude # Chl	0.49	0.55
Weir Max Depth (ft)	2.91	Specif Force (cu ft)	8008.83	7811.68
Min El Weir Flow (ft)	190.51	Hydr Depth (ft)	11.91	3.61
Min El Prs (ft)	185.30	W.P. Total (ft)	148.44	324.46
Delta EG (ft)	3.74	Conv. Total (cfs)		
Delta WS (ft)	3.89	Top Width (ft)	433.60	220.80
BR Open Area (sq	457.63	Frctn Loss (ft)		
BR Open Vel (ft/s)	13.12	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	-10.45	189.00

RS: 154106.3 Profile: NRCC 2005

E.G. US. (ft)	187.70	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	187.19	E.G. Elev (ft)	187.69	187.61
Q Total (cfs)	6277.90	W.S. Elev (ft)	187.15	187.20
Q Bridge (cfs)	6277.90	Crit W.S. (ft)	179.14	179.42
Q Weir (cfs)		Max Chl Dpth (ft)	17.95	14.20
Weir Sta Lft (ft)		Vel Total (ft/s)	5.49	5.11
Weir Sta Rgt (ft)		Flow Area (sq ft)	1142.63	1227.39
Weir Submerg		Froude # Chl	0.28	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	9214.51	8981.23
Min El Weir Flow (ft)	199.75	Hydr Depth (ft)	13.65	12.92
Min El Prs (ft)	191.20	W.P. Total (ft)	93.57	95.14
Delta EG (ft)	0.10	Conv. Total (cfs)	224401.3	250803.1
Delta WS (ft)	-0.01	Top Width (ft)	83.71	95.00
BR Open Area (sq	1514.11	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	5.49	C & E Loss (ft)	0.07	0.00
Coef of Q		Shear Total (lb/sq ft)	0.60	0.50
Br Sel Method	Energy only	Power Total (lb/ft s)	177.08	177.08

RS: 147643.2 Profile: NRCC 2005

E.G. US. (ft)	174.35	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	172.38	E.G. Elev (ft)	174.23	171.64
Q Total (cfs)	6277.90	W.S. Elev (ft)	171.04	166.98
Q Bridge (cfs)	6277.90	Crit W.S. (ft)	171.04	168.27
Q Weir (cfs)		Max Chl Dpth (ft)	8.34	4.79
Weir Sta Lft (ft)		Vel Total (ft/s)	14.33	17.32
Weir Sta Rgt (ft)		Flow Area (sq ft)	437.97	362.54
Weir Submerg		Froude # Chl	1.00	1.39
Weir Max Depth (ft)		Specif Force (cu ft)	4394.54	4137.07
Min El Weir Flow (ft)	179.72	Hydr Depth (ft)	6.39	4.15
Min El Prs (ft)	177.60	W.P. Total (ft)	88.82	95.96
Delta EG (ft)	2.85	Conv. Total (cfs)	47134.7	32670.3
Delta WS (ft)	5.51	Top Width (ft)	68.57	87.40
BR Open Area (sq	906.90	Frctn Loss (ft)		
BR Open Vel (ft/s)	17.32	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)	5.46	8.71
Br Sel Method	Momentum	Power Total (lb/ft s)	179.54	152.04

RS: 141372.6 Profile: NRCC 2005

E.G. US. (ft)	168.88	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	167.99	E.G. Elev (ft)	168.88	167.35
Q Total (cfs)	6277.90	W.S. Elev (ft)	166.60	165.85
Q Bridge (cfs)	6277.90	Crit W.S. (ft)	162.63	162.82
Q Weir (cfs)		Max Chl Dpth (ft)	10.60	9.85
Weir Sta Lft (ft)		Vel Total (ft/s)	9.22	9.85
Weir Sta Rgt (ft)		Flow Area (sq ft)	680.59	637.62
Weir Submerg		Froude # Chl	0.50	0.55
Weir Max Depth (ft)		Specif Force (cu ft)	5467.04	4965.07
Min El Weir Flow (ft)	170.00	Hydr Depth (ft)		9.52
Min El Prs (ft)	166.60	W.P. Total (ft)	151.38	67.48
Delta EG (ft)	1.53	Conv. Total (cfs)	68872.5	105883.7
Delta WS (ft)	2.14	Top Width (ft)		67.00
BR Open Area (sq	667.90	Frctn Loss (ft)		
BR Open Vel (ft/s)	9.40	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)	2.33	2.07
Br Sel Method	Press Only	Power Total (lb/ft s)	354.08	106.00

RS: 136759.6 Profile: NRCC 2005

E.G. US. (ft)	162.70	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	162.11	E.G. Elev (ft)	162.67	159.01
Q Total (cfs)	6277.90	W.S. Elev (ft)	162.10	158.68
Q Bridge (cfs)	6277.90	Crit W.S. (ft)	153.96	151.27
Q Weir (cfs)		Max Chl Dpth (ft)	16.30	12.88
Weir Sta Lft (ft)		Vel Total (ft/s)	5.07	4.38
Weir Sta Rgt (ft)		Flow Area (sq ft)	1238.07	1432.11
Weir Submerg		Froude # Chl	0.29	0.23
Weir Max Depth (ft)		Specif Force (cu ft)	9673.60	9463.43
Min El Weir Flow (ft)	170.88	Hydr Depth (ft)	11.45	11.55
Min El Prs (ft)	167.10	W.P. Total (ft)	147.61	150.70
Delta EG (ft)	3.69	Conv. Total (cfs)	143144.8	196359.5
Delta WS (ft)	3.42	Top Width (ft)	108.16	124.00
BR Open Area (sq	1731.50	Frctn Loss (ft)		
BR Open Vel (ft/s)	5.07	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)	1.01	0.61
Br Sel Method	Momentum	Power Total (lb/ft s)	198.00	279.54

RS: 127937.2 Profile: NRCC 2005

E.G. US. (ft)	151.93	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	150.48	E.G. Elev (ft)	151.93	151.61
Q Total (cfs)	6171.90	W.S. Elev (ft)	150.48	150.48
Q Bridge (cfs)	4898.95	Crit W.S. (ft)	144.97	146.80
Q Weir (cfs)	1103.52	Max Chl Dpth (ft)	20.58	16.88
Weir Sta Lft (ft)	700.00	Vel Total (ft/s)	6.76	5.54
Weir Sta Rgt (ft)	1300.00	Flow Area (sq ft)	913.46	1114.43
Weir Submerg	0.00	Froude # Chl	0.51	0.65
Weir Max Depth (ft)	0.86	Specif Force (cu ft)	6799.53	6089.98
Min El Weir Flow (ft)	151.08	Hydr Depth (ft)		4.83
Min El Prs (ft)	148.68	W.P. Total (ft)	94.41	321.71
Delta EG (ft)	0.32	Conv. Total (cfs)		
Delta WS (ft)	2.86	Top Width (ft)		230.83
BR Open Area (sq	399.13	Frctn Loss (ft)		
BR Open Vel (ft/s)	12.27	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	700.00	700.00

RS: 123964 Profile: NRCC 2005

E.G. US. (ft)	143.32	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	142.14	E.G. Elev (ft)	143.32	143.32
Q Total (cfs)	6707.80	W.S. Elev (ft)	142.14	142.14
Q Bridge (cfs)	6615.60	Crit W.S. (ft)	134.57	134.57
Q Weir (cfs)	111.79	Max Chl Dpth (ft)	17.74	17.74
Weir Sta Lft (ft)	188.50	Vel Total (ft/s)	10.95	10.95
Weir Sta Rgt (ft)	437.76	Flow Area (sq ft)	612.82	612.82
Weir Submerg	0.00	Froude # Chl	0.53	0.53
Weir Max Depth (ft)	0.32	Specif Force (cu ft)	8192.38	8192.38
Min El Weir Flow (ft)	143.01	Hydr Depth (ft)		
Min El Prs (ft)	138.70	W.P. Total (ft)	103.70	103.70
Delta EG (ft)	3.62	Conv. Total (cfs)		
Delta WS (ft)	6.36	Top Width (ft)		
BR Open Area (sq	532.99	Frctn Loss (ft)		
BR Open Vel (ft/s)	12.41	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	178.36	178.36

RS: 107459 Profile: NRCC 2005

E.G. US. (ft)	115.04	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	114.51	E.G. Elev (ft)	114.99	114.90
Q Total (cfs)	6708.10	W.S. Elev (ft)	114.27	114.32
Q Bridge (cfs)	6708.10	Crit W.S. (ft)	102.73	100.65
Q Weir (cfs)		Max Chl Dpth (ft)	21.96	23.38
Weir Sta Lft (ft)		Vel Total (ft/s)	6.78	6.12
Weir Sta Rgt (ft)		Flow Area (sq ft)	988.87	1095.72
Weir Submerg		Froude # Chl	0.26	0.22
Weir Max Depth (ft)		Specif Force (cu ft)	11240.24	13325.09
Min El Weir Flow (ft)	122.81	Hydr Depth (ft)	19.78	21.91
Min El Prs (ft)	122.30	W.P. Total (ft)	50.49	50.91
Delta EG (ft)	0.14	Conv. Total (cfs)	305053.5	279941.2
Delta WS (ft)	0.19	Top Width (ft)	50.00	50.00
BR Open Area (sq	1351.75	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	6.78	C & E Loss (ft)	0.07	0.00
Coef of Q		Shear Total (lb/sq ft)	0.59	0.77
Br Sel Method	Energy only	Power Total (lb/ft s)	182.98	370.10

RS: 106269 Profile: NRCC 2005

E.G. US. (ft)	114.26	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	114.12	E.G. Elev (ft)	114.26	113.93
Q Total (cfs)	6708.10	W.S. Elev (ft)	114.12	113.62
Q Bridge (cfs)	5869.59	Crit W.S. (ft)	109.40	108.49
Q Weir (cfs)	535.15	Max Chl Dpth (ft)	15.12	14.62
Weir Sta Lft (ft)	188.13	Vel Total (ft/s)	7.92	6.62
Weir Sta Rgt (ft)	464.00	Flow Area (sq ft)	846.77	1013.82
Weir Submerg	0.00	Froude # Chl	0.40	0.41
Weir Max Depth (ft)	1.56	Specif Force (cu ft)	7052.16	7267.15
Min El Weir Flow (ft)	113.00	Hydr Depth (ft)	4.06	5.26
Min El Prs (ft)	111.03	W.P. Total (ft)	442.80	418.30
Delta EG (ft)	1.68	Conv. Total (cfs)		
Delta WS (ft)	1.84	Top Width (ft)	208.79	192.88
BR Open Area (sq	650.11	Frctn Loss (ft)		
BR Open Vel (ft/s)	9.03	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	165.37	0.00

RS: 105560 Profile: NRCC 2005

E.G. US. (ft)	111.91	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	111.40	E.G. Elev (ft)	111.84	111.68
Q Total (cfs)	6708.10	W.S. Elev (ft)	111.23	110.79
Q Bridge (cfs)	6708.10	Crit W.S. (ft)	105.17	105.68
Q Weir (cfs)		Max Chl Dpth (ft)	13.23	13.79
Weir Sta Lft (ft)		Vel Total (ft/s)	6.24	7.57
Weir Sta Rgt (ft)		Flow Area (sq ft)	1074.58	886.45
Weir Submerg		Froude # Chl	0.30	0.43
Weir Max Depth (ft)		Specif Force (cu ft)	7596.93	6742.68
Min El Weir Flow (ft)	117.00	Hydr Depth (ft)	10.54	9.56
Min El Prs (ft)	115.30	W.P. Total (ft)	127.80	120.71
Delta EG (ft)	0.30	Conv. Total (cfs)	165058.5	124411.0
Delta WS (ft)	0.64	Top Width (ft)	102.00	92.73
BR Open Area (sq	1232.63	Frctn Loss (ft)	0.07	0.05
BR Open Vel (ft/s)	7.57	C & E Loss (ft)	0.09	0.02
Coef of Q		Shear Total (lb/sq ft)	0.87	1.33
Br Sel Method	Energy only	Power Total (lb/ft s)	61.00	0.00

RS: 61457 Profile: NRCC 2005

E.G. US. (ft)	99.34	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	98.90	E.G. Elev (ft)	98.17	98.26
Q Total (cfs)	10436.90	W.S. Elev (ft)	94.06	94.16
Q Bridge (cfs)	10436.90	Crit W.S. (ft)	94.06	94.16
Q Weir (cfs)		Max Chl Dpth (ft)	9.13	9.97
Weir Sta Lft (ft)		Vel Total (ft/s)	16.27	16.26
Weir Sta Rgt (ft)		Flow Area (sq ft)	641.34	641.77
Weir Submerg		Froude # Chl	0.95	0.91
Weir Max Depth (ft)		Specif Force (cu ft)	7938.38	7952.29
Min El Weir Flow (ft)	99.70	Hydr Depth (ft)	8.22	8.23
Min El Prs (ft)	97.73	W.P. Total (ft)	91.92	95.07
Delta EG (ft)	2.67	Conv. Total (cfs)	77323.9	75694.9
Delta WS (ft)	3.69	Top Width (ft)	78.00	78.00
BR Open Area (sq	892.71	Frctn Loss (ft)		0.45
BR Open Vel (ft/s)	16.27	C & E Loss (ft)		0.00
Coef of Q		Shear Total (lb/sq ft)	7.94	8.01
Br Sel Method	Energy only	Power Total (lb/ft s)	-130.00	-143.00

RS: 35683 Profile: NRCC 2005

E.G. US. (ft)	81.93	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	80.74	E.G. Elev (ft)	80.73	78.90
Q Total (cfs)	10436.90	W.S. Elev (ft)	75.61	69.73
Q Bridge (cfs)	10436.90	Crit W.S. (ft)	75.61	72.32
Q Weir (cfs)		Max Chl Dpth (ft)	10.78	8.66
Weir Sta Lft (ft)		Vel Total (ft/s)	18.16	24.30
Weir Sta Rgt (ft)		Flow Area (sq ft)	574.57	429.52
Weir Submerg		Froude # Chl	0.97	1.46
Weir Max Depth (ft)		Specif Force (cu ft)	8838.63	9527.93
Min El Weir Flow (ft)	82.41	Hydr Depth (ft)	10.26	7.67
Min El Prs (ft)	78.70	W.P. Total (ft)	76.77	72.44
Delta EG (ft)	4.49	Conv. Total (cfs)	72591.8	46461.9
Delta WS (ft)	5.23	Top Width (ft)	56.00	56.00
BR Open Area (sq	747.85	Frctn Loss (ft)	0.29	0.61
BR Open Vel (ft/s)	24.30	C & E Loss (ft)	0.91	1.21
Coef of Q		Shear Total (lb/sq ft)	9.66	18.68
Br Sel Method	Energy only	Power Total (lb/ft s)	-256.86	-83.25

RS: 20082 Profile: NRCC 2005

E.G. US. (ft)	64.09	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	63.44	E.G. Elev (ft)	64.05	63.99
Q Total (cfs)	10436.90	W.S. Elev (ft)	63.37	63.17
Q Bridge (cfs)	10436.90	Crit W.S. (ft)	54.77	56.09
Q Weir (cfs)		Max Chl Dpth (ft)	18.29	18.39
Weir Sta Lft (ft)		Vel Total (ft/s)	6.61	7.29
Weir Sta Rgt (ft)		Flow Area (sq ft)	1579.78	1432.18
Weir Submerg		Froude # Chl	0.27	0.30
Weir Max Depth (ft)		Specif Force (cu ft)	14540.20	12509.91
Min El Weir Flow (ft)	66.56	Hydr Depth (ft)		13.90
Min El Prs (ft)	63.28	W.P. Total (ft)	236.09	107.42
Delta EG (ft)	0.11	Conv. Total (cfs)	238163.2	341870.9
Delta WS (ft)	0.26	Top Width (ft)		103.00
BR Open Area (sq	1443.98	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	7.29	C & E Loss (ft)	0.04	0.02
Coef of Q		Shear Total (lb/sq ft)	0.80	0.78
Br Sel Method	Energy only	Power Total (lb/ft s)	194.00	90.00

RS: 16028 Profile: NRCC 2005

E.G. US. (ft)	60.18	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	58.38	E.G. Elev (ft)	59.21	56.82
Q Total (cfs)	10649.00	W.S. Elev (ft)	54.52	51.31
Q Bridge (cfs)	10649.00	Crit W.S. (ft)	48.30	48.30
Q Weir (cfs)		Max Chl Dpth (ft)	21.32	18.11
Weir Sta Lft (ft)		Vel Total (ft/s)	17.38	18.84
Weir Sta Rgt (ft)		Flow Area (sq ft)	612.88	565.33
Weir Submerg		Froude # Chl	0.66	0.78
Weir Max Depth (ft)		Specif Force (cu ft)	12402.47	10974.27
Min El Weir Flow (ft)	57.30	Hydr Depth (ft)		24.07
Min El Prs (ft)	54.40	W.P. Total (ft)	106.35	81.78
Delta EG (ft)	4.24	Conv. Total (cfs)	39031.3	40645.7
Delta WS (ft)	6.51	Top Width (ft)		23.49
BR Open Area (sq	612.88	Frctn Loss (ft)	2.14	0.15
BR Open Vel (ft/s)	18.84	C & E Loss (ft)	0.25	0.72
Coef of Q		Shear Total (lb/sq ft)	26.78	29.62
Br Sel Method	Energy only	Power Total (lb/ft s)	-385.00	-385.00

RS: 1602.5 Profile: NRCC 2005

E.G. US. (ft)	36.01	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	34.27	E.G. Elev (ft)	35.98	35.90
Q Total (cfs)	11360.82	W.S. Elev (ft)	34.17	34.09
Q Bridge (cfs)	11360.82	Crit W.S. (ft)	25.73	25.73
Q Weir (cfs)		Max Chl Dpth (ft)	23.77	23.69
Weir Sta Lft (ft)		Vel Total (ft/s)	10.79	10.79
Weir Sta Rgt (ft)		Flow Area (sq ft)	1053.15	1052.64
Weir Submerg		Froude # Chl	0.39	0.39
Weir Max Depth (ft)		Specif Force (cu ft)	14397.38	14316.50
Min El Weir Flow (ft)	42.89	Hydr Depth (ft)	373.57	105.06
Min El Prs (ft)	34.20	W.P. Total (ft)	138.67	131.39
Delta EG (ft)	0.13	Conv. Total (cfs)	201542.5	208752.2
Delta WS (ft)	0.16	Top Width (ft)	2.82	10.02
BR Open Area (sq	1053.19	Frctn Loss (ft)	0.08	0.01
BR Open Vel (ft/s)	10.79	C & E Loss (ft)	0.00	0.02
Coef of Q		Shear Total (lb/sq ft)	1.51	1.48
Br Sel Method	Energy only	Power Total (lb/ft s)	105.50	105.50