COPPERMINE BROOK DRAINAGE EVALUATION

BRISTOL, CONNECTICUT

August 14, 2008

MMI #2235-19

Prepared for:

City of Bristol
Department of Public Works
111 North Main Street
Bristol, CT 06010

***Prepared by*:**

MILONE & MACBROOM, INC.
99 Realty Drive
Cheshire, Connecticut 06410
(203) 271-1773
www.miloneandmacbroom.com

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1.0 INTRODUCTION

1.1 Purpose and Scope

The City of Bristol contracted Milone & MacBroom, Inc. (MMI) to prepare a detailed evaluation of the Coppermine Brook watershed and stream channel. In recent years, residents along the Coppermine Brook channel have experienced repeated flooding of yards and residential structures. Many residents have expressed concern that the problem is becoming more severe as the frequency of events increases.

The purpose of this study is to evaluate current conditions in the watershed and along the channel corridor and identify potential strategies to alleviate the flooding problems. This report presents the result of MMI's work effort by detailing the following information:

- Historic flooding patterns along the channel;
- Current and future development patterns within the watershed;
- Existing stream channel characteristics;
- Predicted peak rates of stream flow based on rainfall amounts;
- Peak water surface elevations in the channel based on various rainfall amounts; and
- Potential solutions to alleviate flood elevations in flood prone areas.

It must be noted that a variety of factors often conspire to generate modern drainage and flood-related problems. As such, the solution to drainage problems can be complicated, with improvements in one location potentially contributing to worsening conditions in other areas. Therefore, in developing an overall plan for drainage and flood management within a watershed, recommendations often seek to balance the needs of all community members rather than focusing solely on one neighborhood. This was a particularly important consideration in evaluating Coppermine Brook. As will be documented in this report, the causes and potential remedies to flooding along this channel vary in each neighborhood, yet they are interconnected.

1.2 Summary of Flooding Concerns

Through information provided by the City of Bristol as well as from residents, MMI compiled a summary of flood-related problems in the Coppermine Brook watershed. Flooding complaints have occurred predominately in three specific areas: 1) Richards Court/Stevens Street; 2) Farmington Avenue; and 3) Frederick Street. Each of these areas is described briefly in the following text. Analysis of each area and evaluation of alternative solutions are presented in subsequent report sections. **Note that all orientations relating to left and right bank in this report are presented as facing downstream.**

1.2.1 Richards Court/Stevens Street

The Richards Court and Stevens Street area is subject to extensive flooding. In addition to the testimony from residents, this fact was identified by the Federal Emergency Management Agency (FEMA) in developing its Flood Insurance Study (FIS) of Bristol and was confirmed by MMI's analysis for this report.

Coppermine Brook enters this area after flowing beneath Jerome Avenue. The Jerome Avenue bridge is a large, modern concrete structure with a waterway opening measuring 10 feet high by 81 feet wide that appears to be adequately sized to handle the predicted flow rates.

The channel runs between Stevens Street and Richards Court before making a 90-degree turn to flow under Stevens Street. Observation of homes on Richards Court indicates that basement elevations are similar to the elevation of the channel. In some instances, lawn areas appear lower than the channel. An earthen berm system was constructed at some time in the past, presumably to protect these homes. Field observation indicates that there are two discontinuous berms at this location. A storm drain system also discharges from Richards Court to Coppermine Brook. This system has an open inlet at the rear of the home on the south side of the cul-de-sac. This is important as it allows water to backflow from the channel into the lawn during high flow periods, contributing to flooding.

Downstream of the Richards Court area, the channel passes through the bridge at Stevens Street. The profile of Stevens Street through this area is flat and is generally at the elevation of the adjacent properties. Once passing under Stevens Street, the channel is confined on the right bank by an earthen berm that has been overtopped and is eroded in some locations. The channel bed slope in this reach is generally flat.

1.2.2 Farmington Avenue

Farmington Avenue is near the downstream limit of a broad flat floodplain. A large volume of water is stored in that floodplain area, which is the confluence of Negro Hill Brook, Polkville Brook, and Coppermine Brook. Upstream of Farmington Avenue, the Coppermine Brook channel narrows, constricting flow and contributing to the flooding problems at Farmington Avenue. During large flow events, water flows down Mix Street toward Farmington Avenue in addition to flowing within the Coppermine Brook channel. Downstream of Farmington Avenue, the channel profile remains relatively flat to Louisiana Avenue, which limits the ability of this reach to move water downstream efficiently.

1.2.3 Frederick Street

The Frederick Street area is prone to significant flooding that has resulted in water entering basements and businesses in the area. The Frederick Street bridge is located some 500 feet upstream of the confluence of Coppermine Brook and the Pequabuck River. Based on review of the city's FIS, it appears that flood elevations in the Pequabuck River at this location are elevated due to the downstream railroad bridge being undersized. The profile of the Pequabuck River at this location is presented in Appendix A.

The Frederick Street bridge has existed in its present location since the early 1900s. This bridge is a stone masonry structure downstream of a meander in the Coppermine Brook channel. Upstream of the bridge, a berm has been constructed on the right bank to protect adjacent properties. A vegetated sediment bar has formed on the left side of the channel. Near the upstream end of the right bank berm, it appears that fill was placed in the floodplain of the left bank.

2.0 EXISTING CONDITIONS

2.1 Study Area

This evaluation is intended to identify the causes of and evaluate potential solutions to reduce flooding along the main stem of Coppermine Brook in the City of Bristol. Hydrologic analyses consider the full watershed area including those areas within Burlington, while hydraulic modeling of the channel extends from the Pequabuck River upstream to the corporate boundary of Bristol and Burlington. Coppermine Brook has a significant watershed area, measuring over 18 square miles (11,520 acres). For comparison, the watershed of the Pequabuck River is approximately 45 square miles in area. Figure 2-1 is a watershed map of Coppermine Brook.

2.2 Watershed Characteristics

Flooding along streams and rivers is a normal, natural phenomenon that occurs due to excess surface runoff from precipitation or snowmelt. Human activities and climate change can modify natural flooding. Watershed topography, geology, and vegetation influence runoff rates, which in turn influence the shape, size, and slope of stream channels and floodplains. These factors then influence the presence, depth, and velocity of flood waters which may damage public and private property.

As mentioned previously, the Coppermine Brook watershed is quite large, measuring over 18 square miles. The watershed limits extend from the Pequabuck River north into the town of Burlington. The main stem of Coppermine Brook begins just upstream of Bristol's corporate boundary with Burlington where Wildcat Brook and Whigville Brook join. Negro Hill Brook and Polkville Brook are the other major tributaries of Coppermine Brook.

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(203) 271-1773 Fax: (203) 272-9733
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COPPERMINE BROOK ANALYSIS		LOCATION: Burlington & Bristol, CT	
MMI#: 2235-19-04 MXD: H:SURFICAL_GEO.mxd SOURCE: http://magic.lib.uconn.edu/	N WATERSHED MAP	DATE: 07/07/08 SCALE: 1" = 3000'	SHEET: Figure 2-1

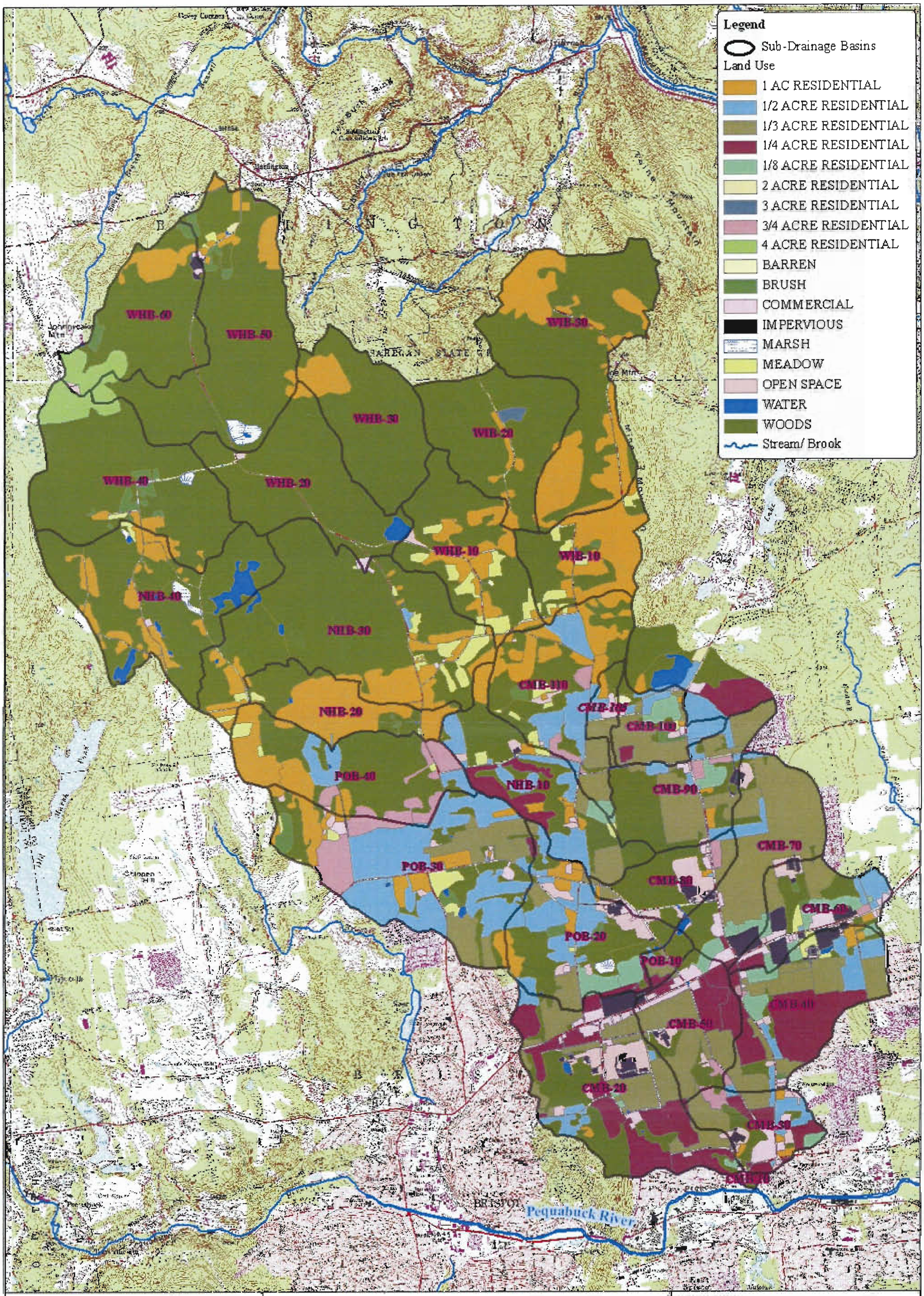
Erosion and deposition of sediments along alluvial channels often create large, nearly level areas of land called floodplains. Floodplains naturally overtop and help convey floodwaters to supplement the channel's capacity. Many floodplains have level, stone-free surfaces that are attractive locations for farms, roads, and communities. However, they remain prone to inundation, and flood damages occur. Coppermine Brook has extensive floodplains that are now flood prone developed areas.

2.2.1 Land Use

In the upper watershed areas of Wildcat and Whigville Brooks, the watershed is steep and undeveloped, although development pressure has been increasing in recent years. Land use practices and policies can have a profound impact on the rate of runoff generated during storm events, which in turn impacts downstream flooding. (See Section 5 for more information about land use practices and regulations in the Coppermine Brook watershed.) In the Wildcat Brook watershed, Nassahegan State Forest comprises a portion of the undeveloped land. The Wildcat Brook subwatershed and some portions of Whigville are vulnerable to development pressures as residential subdivisions continue to be built. Figure 2-2 depicts the current general land use within the Coppermine Brook watershed.

2.2.2 Surficial Geology

The volume and runoff rate within a watershed is attributed in part to the underlying soil characteristics of the area. Sand and gravel materials allow more rainfall to infiltrate into the ground than silts and clays. This infiltration ability reduces the volume of runoff generated. In contrast, shallow bedrock, glacial till soils, and saturated wetlands limit infiltration.



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WATERSHED LANDUSE MAP

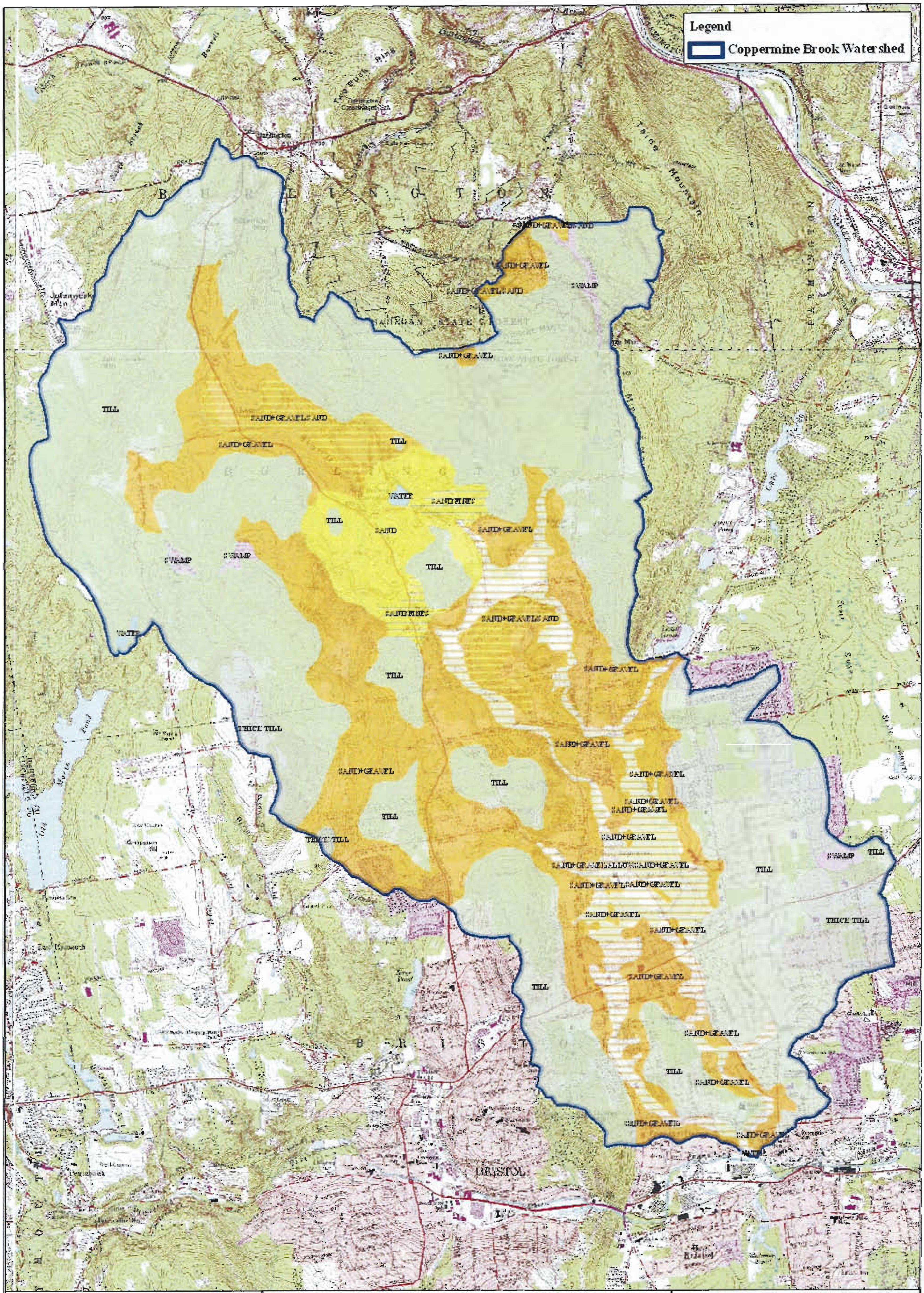
DATE: 07/7/08
SCALE: 1" = 3000'
SHEET: Figure 2-2

Figure 2-3 depicts the soil materials in the Coppermine Brook watershed. From this, we can see that the areas along the main stem of the channel and its tributaries consist of sand and sand and gravel materials. The northern end of the watershed in Burlington and the higher ground along the east and west watershed boundaries are glacial till (in green), which produces high runoff.

The U.S. Geologic Surveys Geologic Map of the Bristol Quadrangle dated 1961 delineates extensive glacial meltwater deposits of sand and gravel in the Coppermine Brook Valley. The area upstream of Louisiana Avenue and extending past Maltby Street to Stevens Street has a broad deposit averaging about one mile wide. This creates the low, wide, nearly level floodplain. A narrower deposit then follows Coppermine Brook to the Pequabuck River.

A portion of this glacial meltwater deposit has surficial layers of alluvium, which are modern floodplain deposits placed by floodwaters. The alluvium deposits correspond well with hydraulic floodplain delineations and include several of the flood prone districts such as Frederick Street, Farmington Avenue near Staples, Maltby Street, and Richards Court. It takes tens to hundreds of years for alluvial soils to form, so these areas have had a long history of flooding.

The Hartford County Soil Survey, published by the U.S. Department of Agriculture in 1958, presents another view of conditions along Coppermine Brook prior to some of today's development. The Richards Court neighborhood is identified as alluvial land (subject to flooding), while the lower valley at Frederick Street has Rumney soils. These are described as "poorly drained moderately coarse textured soils on floodplains. Most areas are flooded frequently."



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Cheshire, Connecticut 06410
(203) 271-1773 Fax: (203) 272-9733
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SURFICIAL MATERIALS MAP

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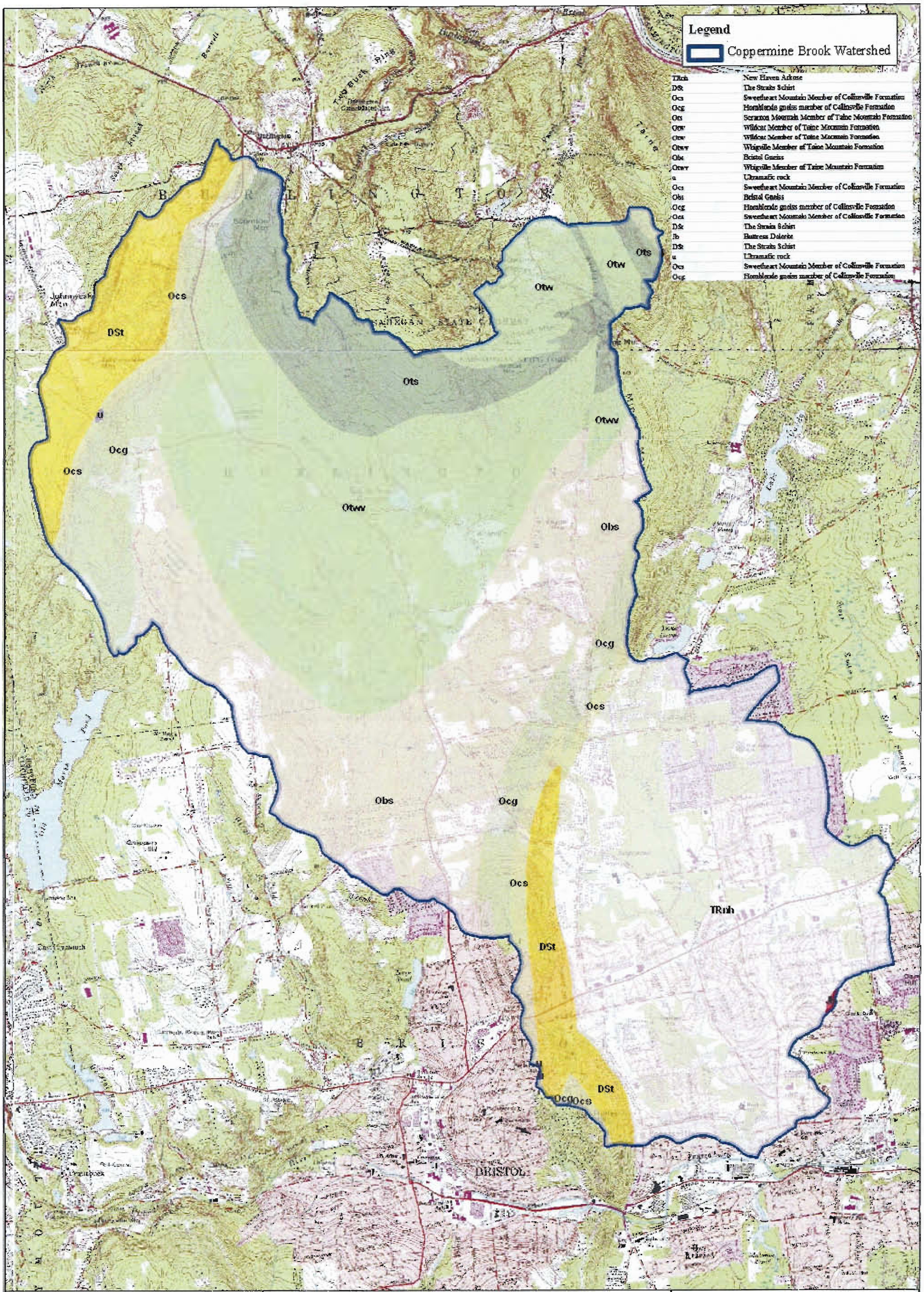
2.2.3 Bedrock Geology

Bedrock geology is shown in Figure 2-4. In general, the higher north and western portion of the watershed is underlain by tough gneiss and schist bedrock that resists erosion. In contrast, the lower, flatter eastern and southern part of the watershed is underlain by sedimentary sand and silt stone from the Triassic era.

2.2.4 Rare and Endangered Species

During the public information meeting held on April 10, 2008 for this project, members of the public commented that previous flood management or drainage projects in this watershed had been halted by the identification of endangered species. The Connecticut Department of Environmental Protection (DEP) maintains what is known as the Natural Diversity Database, which catalogs locations where state and/or federally endangered or threatened species have been identified. Figure 2-5 depicts the current locations in the Coppermine Brook watershed that are listed in this database. Interestingly, each of these areas is located in the upper watershed within the town of Burlington rather than in Bristol where it has been perceived that projects were adversely impacted. It should be noted that inclusion in this database does not necessarily render a site unbuildable. In some instances, the species may no longer exist, and in others the DEP may simply request that the project be modified to consider and protect the species.



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	MMI#: 2235-19-04 MXD: Hbed_geo.mxd SOURCE: http://magic.lib.uconn.edu/	BEDROCK GEOLOGY MAP
	DATE: 07/07/08 SCALE: 1" = 3000'	SHEET: Figure 2-4

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COPPERMINE BROOK ANALYSIS	LOCATION: Burlington & Bristol, CT	
MMI#: 2235-19-04 MXD: HNDDB.mxd SOURCE: http://magic.lib.uconn.edu/	NATURAL DIVERSITY DATABASE AREA MAP	DATE: 07/07/08 SCALE: 1" = 3000'

SHEET: Figure 2-5

2.3 Channel Morphology

Geomorphology is the study of the earth's surface forms and the processes that shape those forms. In the study of rivers and streams (fluvial geomorphology), the primary geomorphologic processes are erosion, sediment transport, and sediment deposition (USDA, 1998). The geomorphologic characteristics of river channels vary depending on the topography, geology, level of watershed urbanization, and rainfall patterns. One of the most common methods of geomorphologic classification is called the Rosgen Method. While there are many classification systems for rivers, this is among the most common, providing common definitions for analysis. In its simplest terms, Rosgen classifies streams based on a combination of plan form, bed slope, and width to depth, assigning channels a letter designation from Aa to G based on these characteristics. Channels are further classified using numerals 1 through 6 based on the substrate material of the channel.

Data on the Coppermine Brook watershed has been reviewed and the watershed inspected to assess runoff and flow conditions. The following text describes Coppermine Brook in a series of discrete segments that have fairly consistent geomorphic characteristics. Coppermine Brook has a variety of channel types ranging from narrow, steep, and rocky channels upstream of Jerome Avenue to broad, low gradient channels with wide floodplains. Basic data of each channel reach as well as the Rosgen classifications are presented in Table 2-1 with more detailed descriptions following.

TABLE 2-1
Channel Characteristics of Coppermine Brook

Reach	From	To	Length (feet)	Slope (%)	Type	Comments
1	Town Line	Jerome Avenue	2,840	1.38	B3	Steep Channel
2	Jerome Avenue	Stevens Street	1,100	1.27	B3	Dike on left bank
3	Stevens Street	Maltby Street	8,625	0.18	C5/6	Broad Floodplain
4	Maltby Street	Farmington Avenue	3,025	0.16	C5/6	Broad Floodplain
5	Farmington Avenue	Footbridge	5,800	0.22	C5	Contraction
6	Footbridge	West Washington Street	2,275	0.64	E2	Knick Point, Fast Run
7	West Washington Street	Frederick Street	1,160	0.55	E4	Fast Run
8	Frederick Street	Pequabuck Confluence	440	0.32	E3	Confluence

2.3.1 Reach 1 – Burlington to Jerome Avenue

The most upstream channel segment in Bristol extends from the municipal boundary downstream to Jerome Avenue. This segment is classified as a Rosgen type B3 channel because it is relatively straight, has little floodplain, and has a cobble streambed. The channel width is typically 20 feet wide and three feet deep with a series of pools, riffles, and runs. Surprisingly, three small in-channel dams were found plus the remains of a fourth former dam. Each dam is approximately three feet high and has little impact on flooding due to well defined high banks. They do not store or detain peak flows.

The river channel splits into three parallel braches just upstream of Jerome Avenue, possibly due to historic activity. A stone masonry wall on the left bank isolates the floodplain from the channel. A narrow overbank floodplain was present on the right bank.

The Jerome Avenue bridge is a large modern structure, about 40 feet wide and 10 feet high with five steel beams, a concrete deck, and abutments. It is in good condition.

2.3.2 Reach 2 – Jerome Avenue to Stevens Street

This reach of channel is critical due to the flood prone nature of the surrounding neighborhoods. This short channel segment also has a type B3 channel that extends parallel to Richards Court, which is a neighborhood of approximately 15 homes on a cul-de-sac. The channel has a moderate gradient with pools, riffles, and fast runs on a gravel and cobble bed. Bank erosion is occurring on the right side. The slightly sinuous channel is confined on the right side by the Stevens Street embankment and on the left side by two low, discontinuous earth berms. The two left bank earth dikes support trees and brush and are not connected or aligned. Overbank flows can go through a broad gap, bypassing the downstream dike. The left floodplain is developed with residential properties on the south side of Richards Court. The neighborhood is subject to shallow, rapid flow at the upstream (west) end and deeper, ponded floodwaters near the cul-de-sac.

A culvert crosses from the rear of the house at #72 Richards Court under the earth dike to the brook, discharging approximately 50 feet upstream of the Stevens Street bridge. Presumably, this was installed to drain water from the rear yards at Richards Court but is certainly capable of having the opposite effect, allowing water from the channel to discharge back into the yard area during high flow events.

The Stevens Street bridge has a sharp approach and exit angles with aggradation (accumulation of sediment) occurring at the waterway opening.

2.3.3 Reach 3 – Stevens Street to Maltby Street

This long, low gradient channel segment has a sand and silty sand bed and low banks with direct floodplain connection. It is a Rosgen type C5/6 channel with no lateral confinement. The broad low forested floodplain is subject to deep inundation and is an excellent floodwater storage and recharge area. It also supplies woody debris to the channel, which has little impact on water levels.

Numerous foot paths pass through this area, which is obviously locally used as unofficial passive open space and for recreation. Fortunately, the floodplain is not developed, although the gentle but higher valley sides have extensive residential neighborhoods. The floodplain includes extensive wetlands.

The Maltby Street bridge is a large, twin span structure. However, it is subject to tailwater due to downstream constrictions, so floodwaters go over the road, which is barely above the floodplain surface.

Trout Brook, a tributary to Coppermine Brook, discharges into this reach near Stevens Street. The Coppermine Brook floodwater levels back up into the Trout Brook area, flooding several houses on the north side of the road. A larger tributary, Negro Hill Brook, also enters Coppermine Brook in this reach.

2.3.4 Reach 4 – Maltby Street to Farmington Avenue

The central part of Reach 4 is a very low gradient sinuous channel with a low level floodplain similar to Reach 3 and is therefore classified as C5/6. However, the upstream reach within this segment has been channelized in the New Britain Water Department public water supply wellfield. Polkville Brook enters Coppermine Brook in this reach. A man-made channel across the floodplain from Mix Avenue is used to infiltrate water into the ground adjacent to New Britain's public water supply wells. The downstream reach approaching Farmington Avenue has also been channelized and both lateral floodplains filled, leaving just a narrow waterway.

On approach to Farmington Avenue, the left floodplain is developed with The Family Medical Group offices, Frank's Transmission Shop, and Family Bank. The right floodplain is blocked by a Staples retail store and Dunkin' Donuts. All of these areas flood, as one would expect. A small, single span bridge between Staples and the medical center confines flow.

The new Farmington Avenue bridge is a low, narrow single span structure built in 2003 with a riprap exit channel. It has a narrow approach and exit channel which, as a coupled system, causes an energy loss. There is no undeveloped floodplain to help convey runoff, so it goes overland. Once water leaves the channel upstream of Staples and the concrete wall at Staples parking lot (which blocks flow), water flows down the street.

2.3.5 Reach 5 – Farmington Avenue to Footbridge

The reach consists of the approximately 1.1-mile long channel from Farmington Avenue to the footbridge at the rear of Hubbell School. This reach has a fairly straight channel with typical widths of 25 feet to 30 feet and a very mild slope of 0.2 percent, which means the river drops only 10 feet per mile. The Rosgen classification of this reach would be C5 at the upstream end, transitioning to a type E4 at the footbridge. Virtually the full channel segment has adjacent developed properties but, fortunately, many of them are wisely set back from the banks. Nevertheless, the low gradient means water gets quite deep, and some homes are in the floodplain. The channel at Louisiana Avenue has been realigned at some point in the past.

The river becomes increasingly incised on approach to Artisan Street, and there is floodplain narrowing near the footbridge. Incised channels are characterized by bed lowering, usually to meet either the limiting geologic regime or a downstream control point. The channel's sandy bed shows evidence of occasional sediment transport.

The water elevations along this reach are influenced by the narrow channel and very limited floodplain approaching the footbridge. The new Artisan Street bridge appears to pass channel flows well, but the older twin span Louisiana Avenue bridge appeared aged.

2.3.6 Reach 6 – Footbridge to West Washington Street

This short channel section wraps around the Hubbell School property and is dramatically different than the adjacent reaches. This channel reach has a steeper bed slope and is incised below the surrounding terrain. This incision has cut the channel off from any associated floodplain. The brook is downcutting a deeper channel that is a transition between the Pequabuck River valley and the uplands. As a result, this narrow transition backs up floodwaters on the flat plain that extends toward Farmington Avenue. The Rosgen classification of this reach would be E3.

The narrow channel has a cobble bed, even some boulders that are absent from the rest of Coppermine Brook. Their source appears to be the glacial knoll upon which the school sits. The channel approaches the new West Washington Street bridge, which spans the bottom of the depressed channel. The increased slope leads to higher velocities, and several areas of bank erosion were observed on the left bank. The higher right bank on the school property is heavily wooded and in good condition. Previous improvements include a supplemental channel under and adjacent to the footbridge, a very helpful step.

2.3.7 Reach 7 – West Washington Street to Frederick Street

This short approximately 1,200-foot long channel segment has a moderate gradient similar to reach 6, but it has developed a wider and incised floodplain similar to a Simon and Schum type IV classification type, where channel and floodplain widening are occurring. This is reflected in the steep high banks, bank erosion, and development of a sinuous alignment. Portions of the geologically new floodplain downstream of West Washington Street are already developed. For example, the Black Bear Auto site is within this area. The Rosgen classification of this reach would be E4.

This segment ends at the Pequabuck River floodplain which flows parallel to Frederick Street. The sinuous channel approaching Frederick Street is eroding on the outside of its bend, which is a natural process. This erosion is extending into the earth dikes that were constructed many years ago. This dike system was constructed immediately adjacent to the channel edge and so has eliminated the floodplain of Coppermine Brook. This reach of channel is inherently unstable as a result of the years of attempts at anthropogenic manipulation. All river systems seek to find equilibrium between flow rate, bed slope, sediment load, and the size of the sediment carried (as defined by Lane in 1955), and this unstable system is no exception. The channel reach is attempting to widen its channel and regain its floodplain while receiving sediment bed load and debris from upstream reaches.

2.3.8 Reach 8 – Frederick Street to the Pequabuck River

This short reach is completely channelized, having been excavated into a rectangular channel at some point in the past, possibly to construct the parking lot of the manufacturing facility. The channel crosses the Pequabuck River floodplain to the river. Review of historic United States Geological Survey mapping indicates that this reach has been generally in this location since between 1906 and 1946, although it should be noted that prior to 1906 the channel discharged in a more direct southern route and the Pequabuck River was north of its present location, flowing west to east adjacent to Frederick Street.

Coppermine Brook has a low gradient gravel bed channel with a few cobbles. The low but steep left bank is generally eroding.

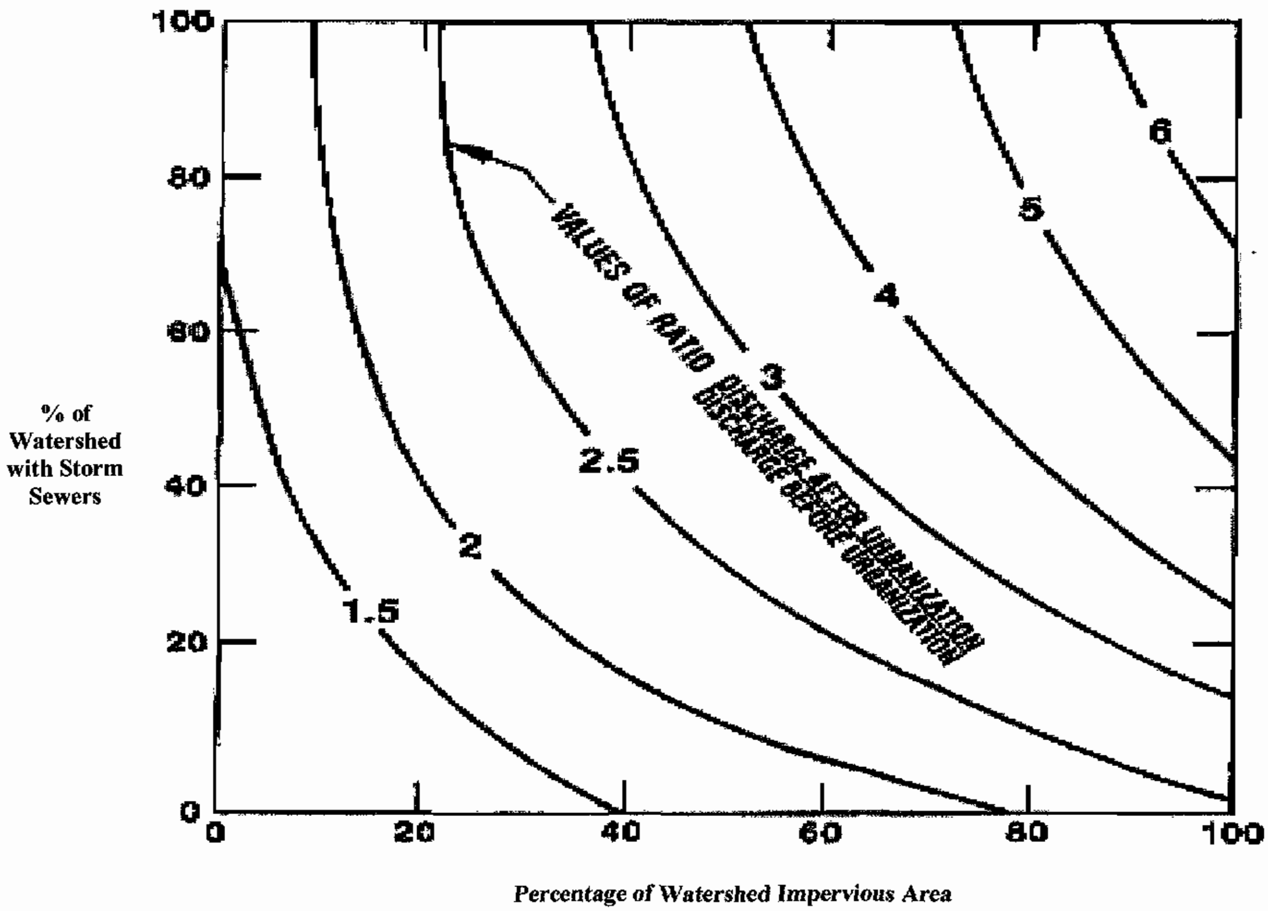
2.4 Connecticut Rainfall and Streamflow Patterns

In evaluating and understanding drainage and flooding, it is imperative to understand rainfall trends and how these relate to changes in streamflow. While Section 3 of this report directly addresses the hydrologic investigations completed by MMI, this section is intended to document how actual data supports changing streamflows throughout Connecticut.

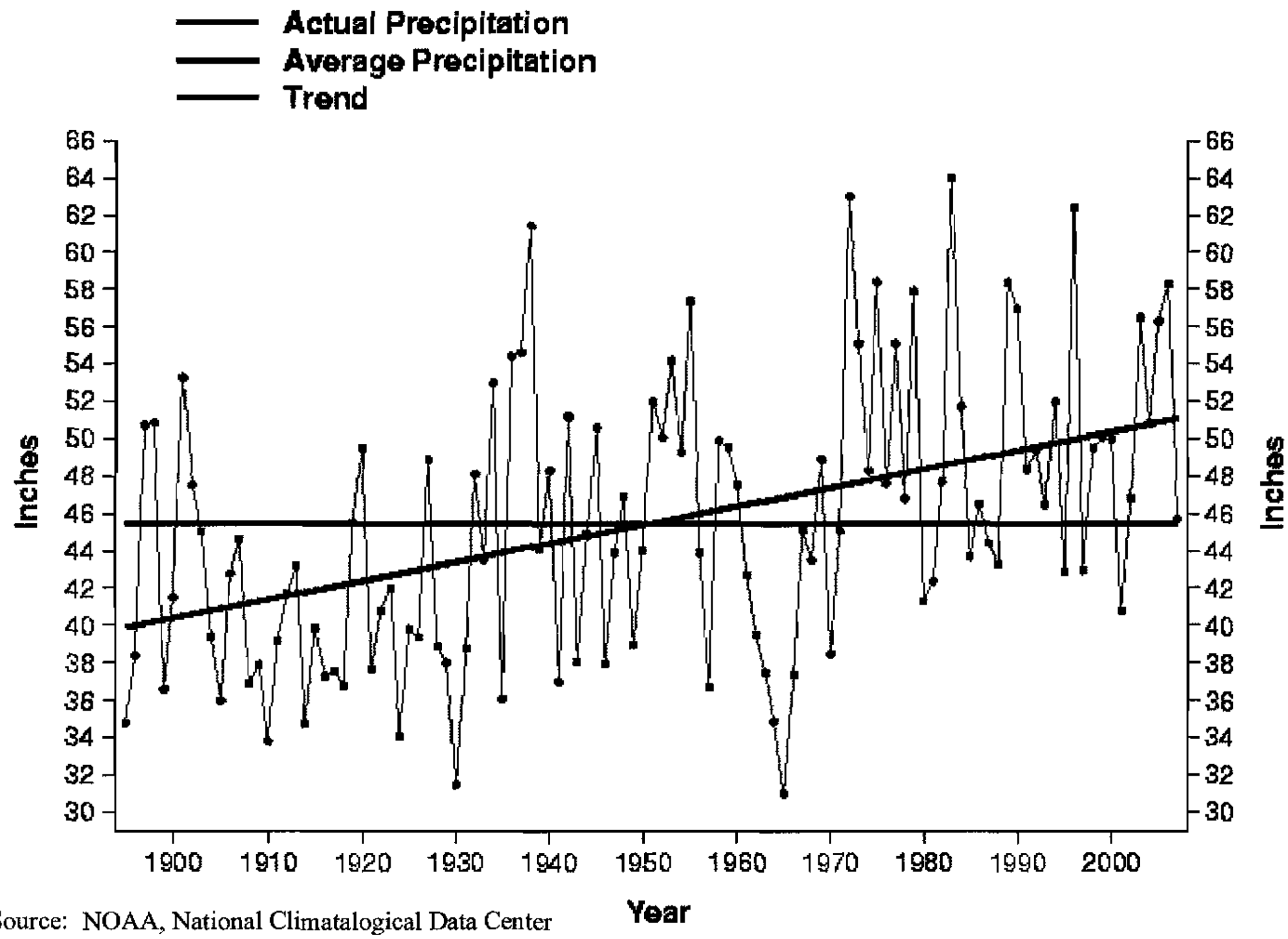
It is well documented that changes to the land's surface associated with land development and other activities can alter hydrologic conditions by modifying the way water moves over, through, and from the land. Watershed deforestation for lumber, firewood, charcoal, and farming was the first impact of colonial settlers. Subsequent drainage of wetlands, channelization, and gravel mining resulted in modification of watershed runoff through the 19th century. During the 20th century, watershed modifications included increased impervious cover (often the result of construction of residential subdivisions, roads, retail stores, and the like) and storm drains with direct discharges to surface water bodies.

In broad classification, typical impacts to wetlands and water resources due to the alteration of hydrologic conditions associated with land development and other activities include (1) degraded water quality; (2) unnatural stream channel geomorphic changes; and (3) increased frequency and severity of flooding. Figure 2-6 graphically depicts how changes in impervious area can alter the rate of runoff from a watershed.

FIGURE 2-6. Impact of Development on Flood Flow Rates



In New England, the effects of urbanization are exacerbated by changes in rainfall patterns that have been observed. Connecticut's annual mean precipitation has consistently increased through the last century, with the increase generally measuring 0.96 inches per decade. This trend is depicted graphically in Figure 2-7.



Source: NOAA, National Climatalogical Data Center

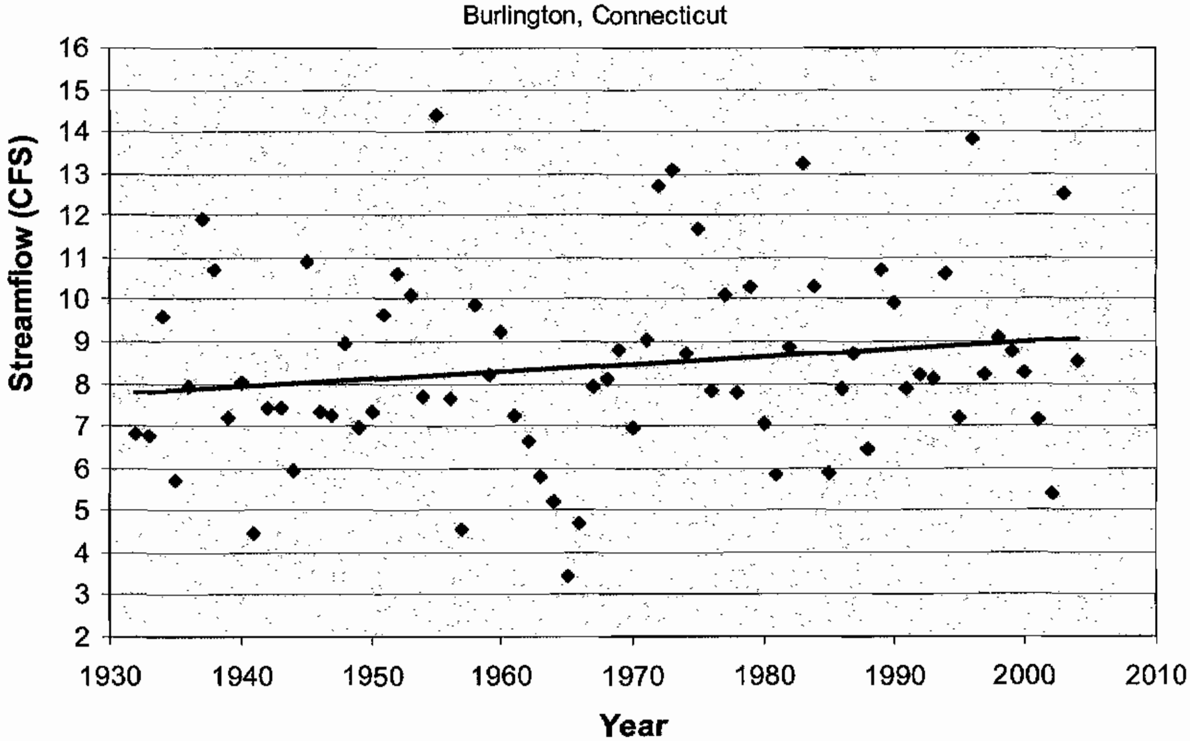
FIGURE 2-7: Precipitation Trends in Connecticut 1895-2003

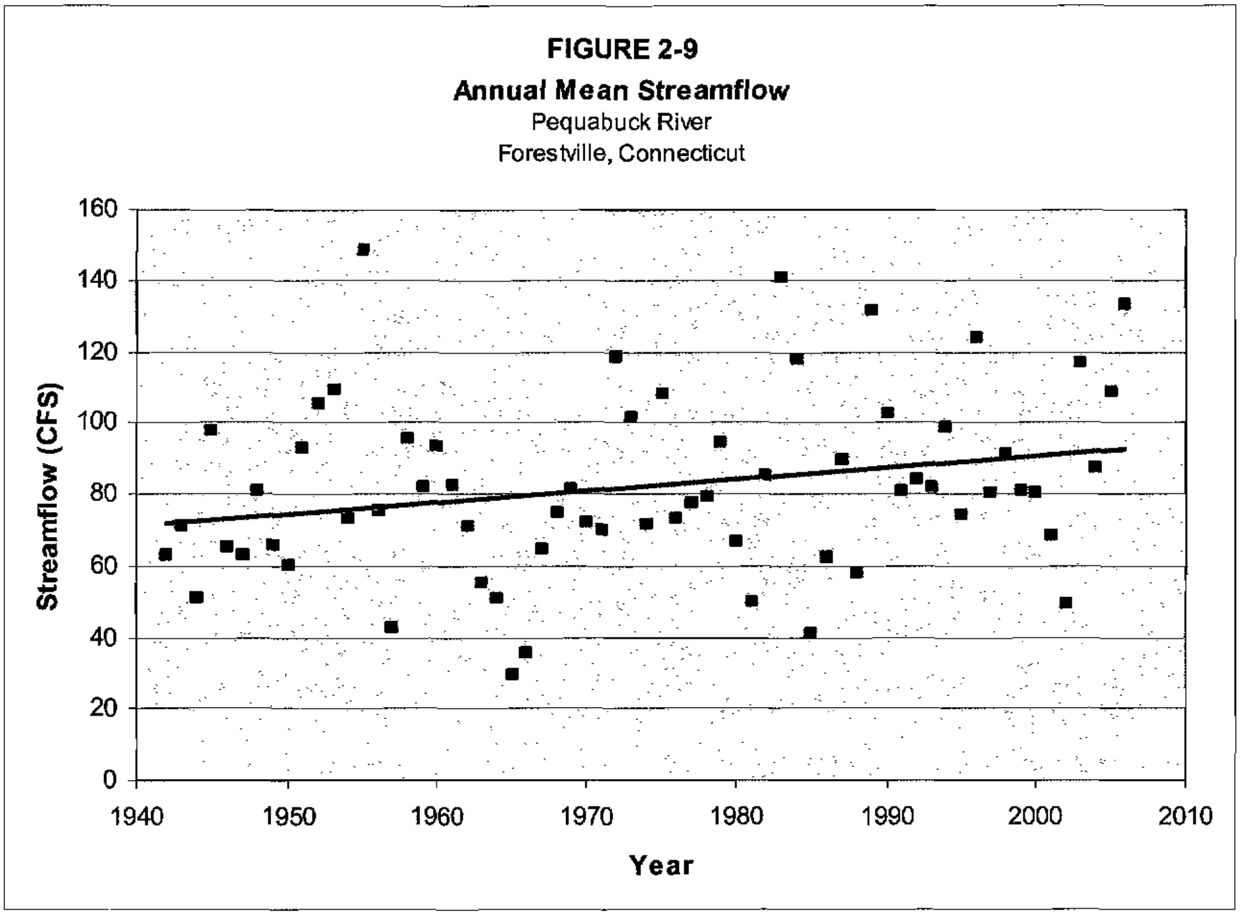
Despite the controversy surrounding the causes of global warming, scientists generally agree that the earth is undergoing a warming trend. Scientific models suggest that the rate of evaporation will increase as the climate warms, which will increase global precipitation. Precipitation is projected to show little change in spring, increase by 10% in summer and fall, and increase by 30% in winter. The amount of precipitation on extremely wet or snowy days in winter is likely to also increase. Since the precipitation increases are not projected to be linear over the course of the year, it is expected that when storm events occur larger rainfall amounts will be recorded and at higher intensities than in the past.

This combination of increased rainfall intensity and increased runoff rates (which can likely be attributed to a combination of increased rainfall and increased development) will invariably result in increases in annual streamflows. This trend is already evident when evaluating streamflow data in Connecticut. Figure 2-8 depicts the mean annual flow rates in Burlington Brook in Burlington, Connecticut from 1932 through 2004. Figure 2-9 depicts the same data for the Pequabuck River from 1942 through 2006. Unfortunately, similar data does not exist for Coppermine Brook, but Burlington Brook and the Pequabuck River were selected for comparison due to their geographic proximity to Bristol. As can be seen from these graphs, annual stream flow has increased in both river systems. In the 4.1 square mile watershed of Burlington Brook, this increase has been slightly more than one cubic foot per second (cfs) over that 72-year period. In the 45.8 square mile watershed of the Pequabuck River, the increase has been more dramatic, increasing some 20 cfs over the 64-year period of record.

FIGURE 2-8

Annual Mean Streamflow
Burlington Brook
Burlington, Connecticut





2.5 Recent Flood Events

Central Connecticut has suffered from an unusual series of flood events in recent years, and these have impacted the Bristol area. Residents have referred to severe flooding in 1999, 2005, and 2006 leading to questions about what caused the floods. There are no active U.S. Geological Survey gauges on Coppermine Brook, or in Bristol, but nearby gauges confirm major regional floods. The damages and problems along Coppermine Brook are not unique.

The USGS stream flow gauge on Bunnel (Burlington) Brook in the town of Burlington is the nearest recording point to Coppermine Brook's headwaters. As discussed in the previous section, records of streamflow exist at this location from 1932 to the present. A flood on September 16, 1999 was the largest in 45 years and second largest up to that point. It is likely that Coppermine Brook could also have had a major flood that day.

Another major flood was measured at Burlington Brook on October 7-10, 2005 from the remnants of Hurricane Tammy. Then a second 2005 flood occurred on October 14, causing record breaking flows throughout the state. Burlington Brook had a flow of 1,850 cubic feet per second, which equates to an average return frequency of over 100 years.

Other significant regional flood events occurred in April 2004; May 15, 2006; and April 17-18, 2007 when the Farmington River had the highest flood since 1955.

3.0 HYDROLOGIC ANALYSIS

As previously discussed, flow rates in a river channel are a function of the watershed size, land use characteristics, soil characteristics, and vegetation as well as rainfall patterns. Hydrology is the science of using this information to determine streamflow rates. This streamflow data can then be used in conjunction with information on the river channel characteristics to predict the depth of water flow during various flood events. This section describes the hydrology of Coppermine Brook, while the following section will describe water depths and flooding limits.

Coppermine Brook originates at the confluence of Whigville Brook and Wildcat Brook located approximately 3,500 feet upstream of the corporate limits of the city of Bristol and town of Burlington. Many tributaries join Coppermine Brook before it flows into the Pequabuck River. Negro Hill Brook and Polkville Brook discharge to Coppermine Brook in the area between Farmington Avenue and Stevens Street. Two other unnamed streams, named Tributary A and Tributary B, also discharge to Coppermine Brook.

3.1 FEMA Flows

The Flood Insurance Study (FIS) prepared by the Federal Emergency Management Agency (FEMA) for the City of Bristol dated May 18, 1981 provides flow rates at selected locations in the Coppermine Brook watershed. The flow rates were estimated based on regional equations for streamflow in Connecticut. The regional equations were based on regression analysis of streamflow records for 105 gauging stations and rainfall data from 23 precipitation gauges in Connecticut. Using the regression equations, flow rates can be predicted based on watershed size, rainfall, channel length, channel slope, and amount of watershed area served by storm sewer. The drainage area and the flows given in the FEMA FIS are shown in Table 3-1.

TABLE 3-1
Coppermine Brook Discharge Rates
Presented in FEMA FIS

Location	Drainage Area (square miles)	FEMA Flows (cfs)			
		10-Year	50-Year	100-Year	500-Year
Upstream of Negro Hill Brook	8.8	1,100	2,000	2,490	4,150
Upstream of Polkville Brook	13.0	1,460	2,610	3,215	5,190
Upstream of Tributary B	15.9	1,890	3,210	3,885	6,200
Upstream of Tributary A	17.3	2,035	3,475	4,175	6,740
Confluence with Pequabuck River	18.6	2,140	3,630	4,340	7,000

The use of regression equations for estimating flows is an accepted practice. Since the regression is developed using field data collected from within Connecticut, the resulting equation is considered to be a moderately accurate predictor of flows. That being said, other methods such as development of hydrologic models using computer programs and real time streamflow measurement can yield more accurate results. In the case of the 1981 FIS, the regression equations used to develop the data in Table 3-1 were published in 1976. Given the changes in streamflow rates that have been observed in Connecticut in recent years (as documented in Section 2), we would expect the flow rates in the FIS to be lower than actual streamflows, which would underestimate the extents of flooding under various storm events.

3.2 Existing Conditions Hydrology

As part of this study, the computer modeling program known as the Hydrologic Modeling System HEC-HMS 3.2 was used to determine the flow rates for the various storm events. Created by the U.S. Army Corps of Engineers, the HEC-HMS program forecasts the rate of surface water runoff and river flow rates based upon several factors. The model input data includes information about the contributing watershed area, the runoff curve number (CN), the lag time of the watershed, the available storage volume of the reservoir, the channel routing, and rainfall data for the area. Each of these elements is described in the ensuing text.

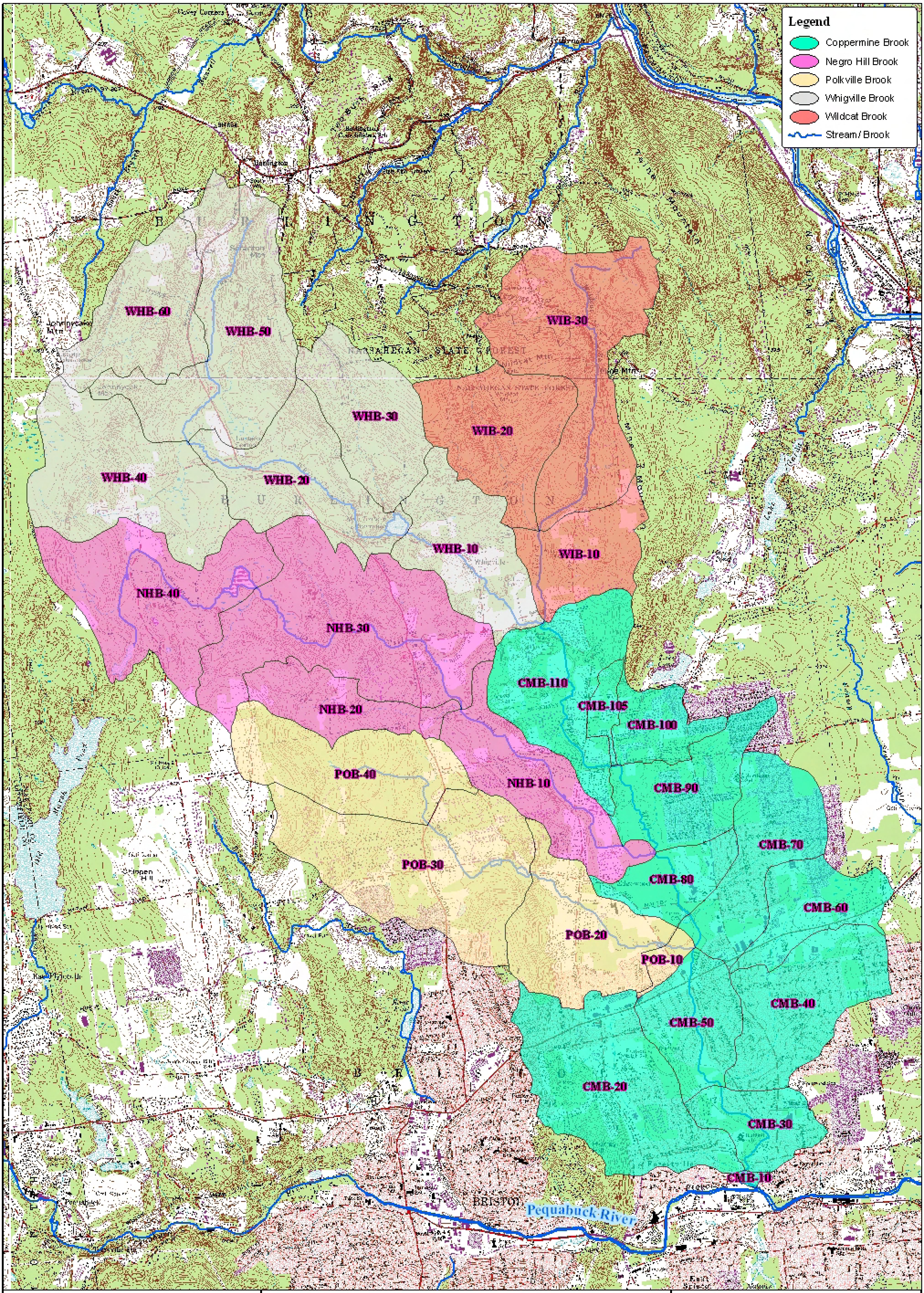
3.2.1 Subwatershed Delineations

Mapping of the Coppermine Brook watershed was obtained from publicly available Geographic Information System (GIS) databases. Information collected includes drainage basin delineation, topography, roadways, and buildings. The overall Coppermine Brook watershed was divided into subwatersheds based on topographic information presented in this mapping and supplemented by two-foot contour and storm drainage system mapping provided by the City of Bristol. Watershed and subwatershed boundaries were then field verified. The Coppermine Brook watershed was divided into 29 subwatersheds for this analysis. Figure 3-1 presents the watershed area and subwatershed boundaries.

3.2.2 Runoff Curve Number

Each subwatershed is defined in the hydrologic model by its size, a runoff curve number, and the time of concentration. The runoff curve number system was developed by the Natural Resources Conservation Service (NRCS, formerly the Soil Conservation Service). Using this system, each subwatershed is assigned a number between 30 and 98. The number used is specific to the subwatershed and is determined based on soil type and land use.

Land use in each subwatershed was determined from the GIS mapping and aerial photography. Land use in the watershed was classified as forested, open space, barren land, residential separated by lot size, commercial, and impervious (paved) cover. A figure depicting land use in the watershed was presented in Section 2.



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COPPERMINE BROOK ANALYSIS

LOCATION: Burlington & Bristol, CT

MMI#: 2235-19-04
MXD: Hdrainage.mxd
SOURCE: http://magic.lib.uconn.edu/

SUBWATERSHED AREAS MAP

DATE: 07/07/08
SCALE: 1" = 3000'

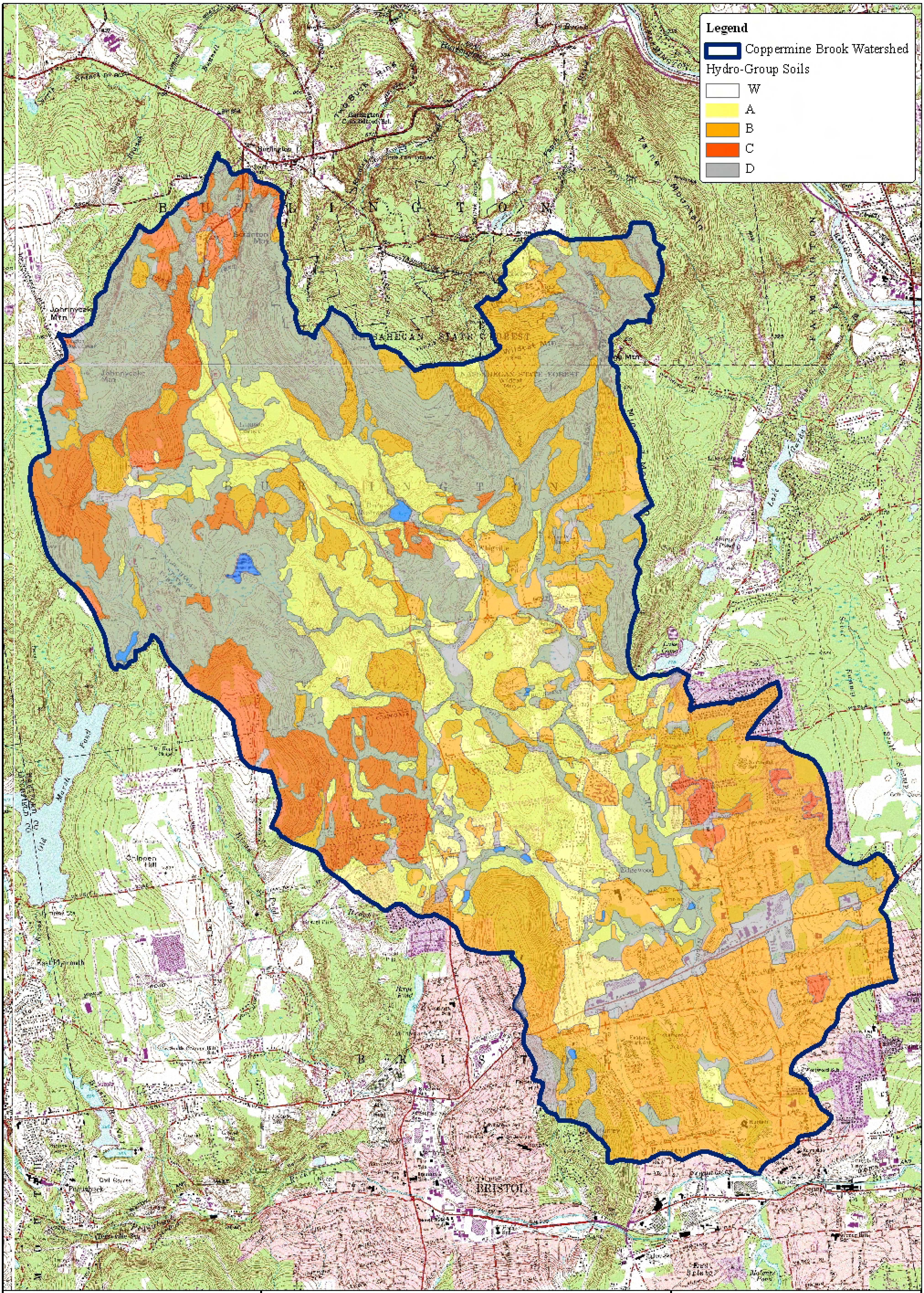
SHEET: Figure 3-1

Soil types in the watershed were determined from the Connecticut Department of Environmental Protection GIS database of the NRCS soil survey for Hartford County, Connecticut. The NRCS divides soils into four groups: A, B, C, or D, depending on their infiltration capacity and ability to absorb water. The NRCS has identified hydrologic groups A, B, C, and D soils in the Coppermine Brook watershed. Hydrologic group A soils have high infiltration capacity and consist of well drained soils. Group D soils have the lowest infiltration capacity and hence generate the highest runoff rates. Recall from Section 2 that many areas within this watershed are mapped as sand. Sandy soils would generally be considered hydrologic soil group A or B because of their high potential infiltration capacity. Soil types for the Coppermine Brook watershed are presented in Figure 3-2.

Based on the soil types and land use, weighted curve numbers were developed for each subwatershed. Areas of imperviousness such as parking lots and buildings were assigned a Curve Number (CN) of 98. The curve numbers used in the model were based on curve numbers for Connecticut developed by MMI to reflect conditions in Connecticut rather than the Midwestern conditions that were used to develop the NRCS's published curve numbers. These numbers have been accepted for use by the NRCS. A memo documenting these numbers and a letter from NRCS authorizing their use are presented in Appendix B. Curve number calculations are presented in Appendix C.

3.2.3 Time of Concentration

The time of concentration is defined as the time it takes a drop of water to travel from the most hydrologically distant point in the watershed (or subwatershed) to the watershed (or subwatershed) outlet. This value generally defines how quickly after the start of a rainfall event that peak flows will be observed in the stream channel. Calculations of the time of concentration for each subwatershed are presented in Appendix D.



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(203) 271-1773 Fax: (203) 272-9733
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COPPERMINE BROOK ANALYSIS

LOCATION: Burlington & Bristol, CT

MMI#: 2235-19-04
MXD: Hhydro_soils.mxd
SOURCE: http://magic.lib.uconn.edu/

HYDROLOGIC SOIL GROUP MAP

DATE: 07/07/08
SCALE: 1" = 3000'

SHEET: Figure 3-2

3.2.4 Summary of Subwatershed Input Data

The area, curve number, and time of concentration of each subwatershed from upstream to downstream are given in Table 3-2.

3.2.5 Precipitation Data

Rainfall data for the analysis was taken from the United States Weather Bureau's Technical Paper 40 published in 1961. According to TP-40, 24-hour rainfall rates for the two-, 10-, 25-, 50-, 100-, and 500-year return frequency storm events of 3.2, 4.7, 5.5, 6.2, 6.9, and 8.9 inches, respectively, were used.

3.2.6 Reservoir and Reach Storage

In most watersheds, water can be stored during storm events in ponds and wetland areas. This storage serves to attenuate flood flows, allowing water to be released slowly over a long period of time. When natural storage areas are lost due to filling or other activity, then peak flows downstream increase. Such storage can be classified as reservoir storage or reach storage. Reservoir storage consists of ponds or large wetland areas. The amount of storage available is defined based on the outflow capacity from the area, and the volume of water that can be stored, both of which vary by water elevation (often called stage). The resulting stage-discharge-storage relationship defines each reservoir. Reach storage occurs within the channel and reflects the time it takes for water in the stream to flow from the upper watershed to the outlet. In areas with a wide floodplain, reach storage can provide significant attenuation. Three storage areas within the Coppermine Brook watershed were identified by MMI and included in the hydrologic model.

TABLE 3-2
Hydrologic Input Data – Existing Conditions

Subwatershed	Area (acres)	Curve Number (CN)	Time of Concentration (T_C in hours)
WHB-60	399.34	69	1.68
WHB-50	715.41	64	1.60
WHB-40	618.16	64	2.26
WHB-30	373.89	59	1.36
WHB-20	457.40	46	1.47
WHB-10	399.66	57	1.23
WIB-30	740.27	62	2.26
WIB-20	422.51	58	1.48
WIB-10	277.51	64	1.36
NHB-40	622.91	69	1.91
NHB-30	903.79	51	2.03
NHB-20	240.09	58	1.63
NHB-10	404.40	50	1.65
POB-40	460.97	68	1.82
POB-30	757.87	62	1.17
POB-20	444.78	55	1.61
POB-10	42.05	62	0.71
CMB-110	457.31	57	1.45
CMB-105	54.24	61	0.89
CMB-100	165.39	63	1.02
CMB-90	337.79	61	1.21
CMB-80	187.64	64	0.98
CMB-70	388.08	70	1.03
CMB-60	248.35	73	2.54
CMB-50	274.94	69	1.16
CMB-40	438.88	69	1.46
CMB-30	205.29	68	0.73
CMB-20	658.89	68	1.89
CMB-10	9.38	69	0.37

Whigville Reservoir – This reservoir, which is owned by the New Britain Water Department (NBWD) and located in Burlington, is located in model subwatershed WHB 20. This facility was inspected by MMI staff on June 9, 2008 in response to several comments at the April 10, 2008 public meeting that residents suspected dam releases caused prior flooding.

The reservoir has an earth dam about 30 feet high with a ±30-foot long ogee crest concrete spillway. There are no flashboards or emergency spillways. The actual dam is about 150 feet long. A 12-inch diameter raw water main and a ±18-inch diameter low level outlet were observed at the base of the dam. There was no unusual channel erosion or degradation, and the downstream receiving channel is armored for approximately 500 feet.

The dam does not include any physical facilities that could be operated to release large quantities of water. Once the reservoir is full, the spillway discharge rate is no greater than the inflow rate. There is no evidence or mechanism that would indicate the dam increased downstream flooding.

Negro Hill Brook wetland – The existing storage area created by the wetlands upstream of Negro Hill Brook was designated as Coppermine 1 Reservoir in the hydrologic model. The upstream extent of this storage area is the downstream face of the Stevens Street bridge. The stage-discharge-storage relationship for this area was obtained from the HEC-RAS model (see Section 4.0 of this report).

Farmington Avenue wetland – The wetland area between Farmington Avenue and the wetland area at Negro Hill Brook (described above) serves to store large volumes of floodwater and was defined as Coppermine 2 Reservoir in the HEC-HMS model. As with the Negro Hill Brook wetland, the stage-discharge-storage relationship in this area was defined using output from HEC-RAS.

3.2.7 Results of Existing Conditions Analysis

Table 3-3 presents the predicted channel flow rates at select areas within the watershed. HEC-HMS input and output files are presented in Appendix E.

TABLE 3-3
Results of Existing Conditions Analysis

Description	Predicted Peak Flows (cfs)					
	Two-Year	**10-Year**	**25-Year**	**50-Year**	**100-Year**	**500-Year**
Confluence of Whigville Brook & Wildcat Brook	418	1,263	1,841	2,430	3,038	4,975
Downstream of Stevens Street	454	1,382	2,033	2,687	3,388	5,579
Downstream of confluence with Negro Hill Brook	458	1,606	2,411	3,396	4,381	7,220
Downstream of confluence with Polkville Brook	551	1,499	2,329	2,888	4,071	8,096
Upstream of Artisan Street	609	1,632	2,523	3,135	4,384	8,699
Upstream of Frederick Street	656	1,736	2,678	3,360	4,619	9,189
Confluence with Pequabuck River	656	1,737	2,679	3,362	4,606	9,181

Table 3-4 shows a comparison of flow rates used in the FEMA study versus those predicted in MMI's analysis. As presented in that table, the flows computed from the HEC-HMS model are greater than the FEMA flows upstream of Negro Hill Brook. This is due to different computing methods used by FEMA and MMI. The flows computed by MMI at the confluence of the Pequabuck River are lower during the 10- and 50-year storm events and slightly higher during the 100-year storm event when compared to the FEMA flows. The reduction in flows is due to MMI's use of storage area from Stevens Street to Farmington Avenue, which is not accounted for in a detailed manner when using the Regression Equations.

TABLE 3-4
Comparison of FEMA and MMI Drainage Areas and Flows

	Upstream of Negro Hill Brook	At Confluence with Pequabuck River
Drainage Area (square miles)		
FEMA	8.80	18.60
MMI HEC-HMS	8.47	18.95
Difference	-0.33	0.35
Flow Rates (cfs)		
FEMA 10-Year	1,100	2,140
MMI HEC-HMS 10-Year	1,459	1,737
Difference	+359	-403
FEMA 50-Year	2,000	3,630
MMI HEC-HMS 50-Year	2,841	3,362
Difference	+841	-268
FEMA 100-Year	2,490	4,340
MMI HEC-HMS 100-Year	3,575	4,606
Difference	+1,085	+266
FEMA 500-Year	4,150	7,000
MMI HEC-HMS 500-Year	5,920	9,181
Difference	+1,770	+2,181

As presented in the above table, the flows computed by MMI for this study are greater than the FEMA flows upstream of Negro Hill Brook for all storm events. This is due to different computational methods used by FEMA and MMI. The flows computed by MMI at the confluence of the Pequabuck River are lower during the 10- and 50-year storm events and slightly higher during the 100-year storm event when compared to the FEMA flows. The reduction in flows is due to the use of storage area from Stevens Street to Farmington Avenue, which is not accounted for in a detailed manner when using the Regression Equations. Flood storage in wetlands and waterbodies serves to attenuate flood flows, allowing for a more controlled release of water downstream. This has less effect during larger rainfall events because once the volume of runoff exceeds the storage capacity of the wetland all flow generated in the watershed passes through the wetland without being stored.

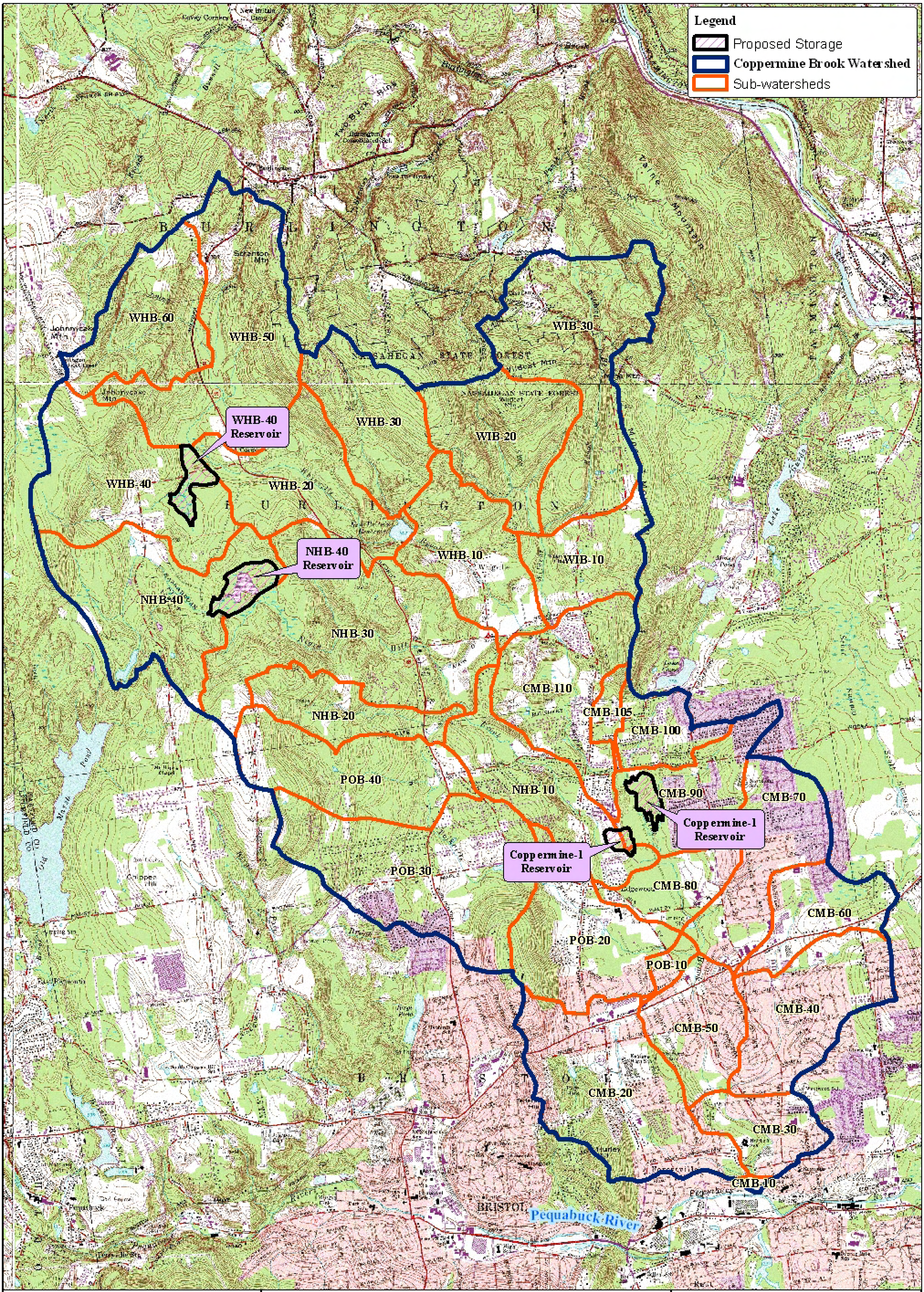
The flow rates estimated by MMI's existing conditions analysis were used in the hydraulic model described in Section 4.

3.3 Potential Future Storage

As has been previously discussed, lake, pond, and wetland storage can play a significant role in reducing flooding. While channel and bridge modifications are a standard way of minimizing flood damage, increasing upstream storage can also reduce damage by decreasing the peak flow rate to downstream areas. Three areas were evaluated for potential proposed conditions storage. These locations are depicted on Figure 3-3. It must be noted that if the city were to pursue any of these alternatives additional analysis will be required. For example, coordination with the Department of Environmental Protection and U.S. Army Corps of Engineers is needed to determine potential permit requirements.

3.3.1 Potential Storage Areas

Reservoir NHB-40: A wetland area in subwatershed 40 of the Negro Hill Brook watershed was modeled as Reservoir NHB-40 by assuming detention created by a spillway. This wetland is located within the Nassahegan State Forest in Burlington. A spillway approximately five feet high and 20 feet wide was assumed to detain water in the wetland.



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COPPERMINE BROOK ANALYSIS

LOCATION: Burlington & Bristol, CT

MMI#: 2235-19-04
MXD: H:Fig 3-3.mxd
SOURCE: http://magic.lib.uconn.edu/

PROPOSED STORAGE LOCATIONS MAP

DATE: 07/07/08
SCALE: 1" = 3000'

SHEET: Figure 3-3

Reservoir WHB-40: The wetland in subwatershed 40 of the Whigville Brook watershed was modeled as Reservoir WHB-40 by assuming detention created by spillway. This wetland is located approximately 1.5 miles upstream of the New Britain Reservoir located in Burlington. A spillway approximately five feet high and 20 feet wide was assumed to detain water in the wetland.

Proposed Coppermine-1 Reservoir: Consideration was given to increasing the total storage available in the wetland area near the confluence of Negro Hill Brook through the excavation of two existing upland areas. The first storage area is located on the east bank, and an estimated 69 acre-feet (one acre of area [43,560 square feet] with water one foot deep) of storage area will be created. The second storage area appears capable of providing an additional 60 acre-feet of storage area.

3.3.2 Results of Potential Future Storage Analysis

The HMS model was run with all three of these reservoirs individually and then also run with all three in place to determine the combined benefit. The results of each of these are presented in the following tables.

TABLE 3-5
Comparison of Existing and Proposed Conditions Peak Flows – Negro Hill Brook Watershed Storage

Storage Frequency	Downstream Negro Hill Brook			Confluence of Pequabuck River		
	Existing	Proposed	% Change	Existing	Proposed	% Change
10	1,606	1,379	-14.1	1,737	1,616	-7.0
25	2,411	2,117	-12.2	2,679	2,487	-7.2
50	3,396	2,944	-13.3	3,362	3,228	-4.0
100	4,380	3,864	-11.8	4,606	3,796	-17.6

TABLE 3-6
Comparison of Existing and Proposed Conditions Peak Flows – Whigville Brook Watershed Storage

Storm Frequency	Downstream Negro Hill Brook			Confluence of Pequabuck River		
	Existing	Proposed	% Change	Existing	Proposed	% Change
10	1,606	1,482	-7.7	1,737	1,669	-3.9
25	2,411	2,248	-6.8	2,679	2,571	-4.0
50	3,396	3,134	-7.7	3,362	3,288	-2.2
100	4,380	4,092	-6.6	4,606	4,151	-9.9

TABLE 3-7
Comparison of Existing and Proposed Conditions Peak Flows – Increased Storage At Negro Hill Brook Confluence[1]

Storm Frequency	Downstream Negro Hill Brook			Confluence of Pequabuck River		
	Existing	Proposed	% Change	Existing	Proposed	% Change
10	1,606	1,606	0.0	1,737	1,737	0.0
25	2,411	2,411	0.0	2,679	2,679	0.0
50	3,396	2,976	-12.4	3,362	3,287	-2.2
100	4,380	3,619	-17.4	4,606	4,191	-9.9

Note: 1. Includes both Coppermine 1 and Coppermine 2 reservoir areas.

TABLE 3-8
Comparison of Existing and Proposed Conditions Peak Flows – Combined Storage at Three Locations

Storm Frequency	Downstream Negro Hill Brook			Confluence of Pequabuck River		
	Existing	Proposed	% Change	Existing	Proposed	% Change
10	1,606	1,253	-22.0	1,737	1,553	-10.6
25	2,411	1,954	-19.0	2,679	2,376	-11.3
50	3,396	2,562	-24.6	3,362	3,139	-6.6
100	4,380	3,083	-29.6	4,606	3,597	-21.9

The result of the hydrologic analysis for potential storage indicates that increasing storage in the upper watershed would decrease flow rates in the lower reaches. These decreased flow rates would lead to decreased flood elevations in some areas.

3.4 Future Flows at Watershed Buildout

In the interest of understanding the potential changes in flow that may occur in Coppermine Brook as development continues in the watershed, the city requested that MMI run the hydrologic model assuming that all property in the watershed is developed to its maximum capacity given the current zoning regulations of Bristol and Burlington. This model was developed by modifying the curve number and time of concentrations for each subwatershed to reflect the additional impervious area that is anticipated. Land in existing state forest and the property owned by the New Britain Water Department was assumed to not be developed in the future. Table 3-9 presents these modified curve numbers and time of concentration for each subwatershed. Calculations are provided in Appendix F.

TABLE 3-9
Hydrologic Input Data – Future Conditions

Subwatershed	Curve Number (CN)		Time of Concentration (T_C in hours)	
	Existing	Future	Existing	Future
WHB-60	69	69	1.68	1.68
WHB-50	64	64	1.60	1.60
WHB-40	64	64	2.26	2.26
WHB-30	59	59	1.36	1.36
WHB-20	46	46	1.47	1.47
WHB-10	57	62	1.23	1.23
WIB-30	62	64	2.26	2.26
WIB-20	58	66	1.48	1.06
WIB-10	64	69	1.36	0.94
NHB-40	69	75	1.91	1.54
NHB-30	51	53	2.03	1.66
NHB-20	58	60	1.63	1.61
NHB-10	50	54	1.65	1.33
POB-40	68	71	1.82	1.34
POB-30	62	63	1.17	0.91
POB-20	55	55	1.61	1.30
POB-10	62	62	0.71	0.71
CMB-110	57	60	1.45	1.03
CMB-105	61	66	0.89	0.89
CMB-100	63	64	1.02	0.74
CMB-90	61	64	1.21	1.06
CMB-80	64	66	0.98	0.98
CMB-70	70	70	1.03	1.03
CMB-60	73	73	2.54	2.54
CMB-50	69	69	1.16	1.16
CMB-40	69	69	1.46	1.46
CMB-30	68	68	0.73	0.73
CMB-20	68	70	1.89	1.89
CMB-10	69	69	0.37	0.37

Table 3-10 presents the results of this analysis. Not surprisingly, future development within this watershed has the potential to increase peak flows significantly.

TABLE 3-10
Comparison of Existing and Future Conditions Peak Flows

Storm Frequency	Downstream Negro Hill Brook			Confluence of Pequabuck River		
	Existing	Future	% Change	Existing	Future	% Change
10	1,606	1,897	+18.1	1,737	2,050	+18.0
25	2,411	2,860	+18.6	2,679	3,032	+13.2
50	3,396	3,875	+14.1	3,362	3,639	+8.2
100	4,380	4,865	+11.1	4,606	5,589	+21.3

3.5 Future Flows at Watershed Buildout with Proposed Storage

The future buildout model was modified to reflect the impact of providing additional storage in the watershed as described in Section 3.3. Given the hypothetical nature of this evaluation, this modeling effort assumed that all three of the storage areas were in place rather than modeling each individually. The effect of each individual storage area is expected to be of a similar order of magnitude as under existing conditions. Table 3-11 compares future flows with and without increased storage.

TABLE 3-11
Comparison of Future Conditions Peak Flows
With and Without Proposed Storage

Storm Frequency	Downstream Negro Hill Brook			Confluence of Pequabuck River		
	Future	Future with Storage	% Change	Future	Future with Storage	% Change
10	1,897	1,482	-22	2,050	1,761	-14
25	2,860	2,213	-23	3,032	2,679	-12
50	3,875	2,778	-28	3,639	3,351	-8
100	4,865	3,355	-31	5,589	3,988	-29

4.0 HYDRAULIC ANALYSIS

The term "hydraulic analysis" refers to the computational prediction of the river's water elevations, depths, and velocities for specified water discharge rates. This analysis is used to predict the elevation that floodwaters will reach given different river flows.

4.1 Hydraulic Modeling Methods

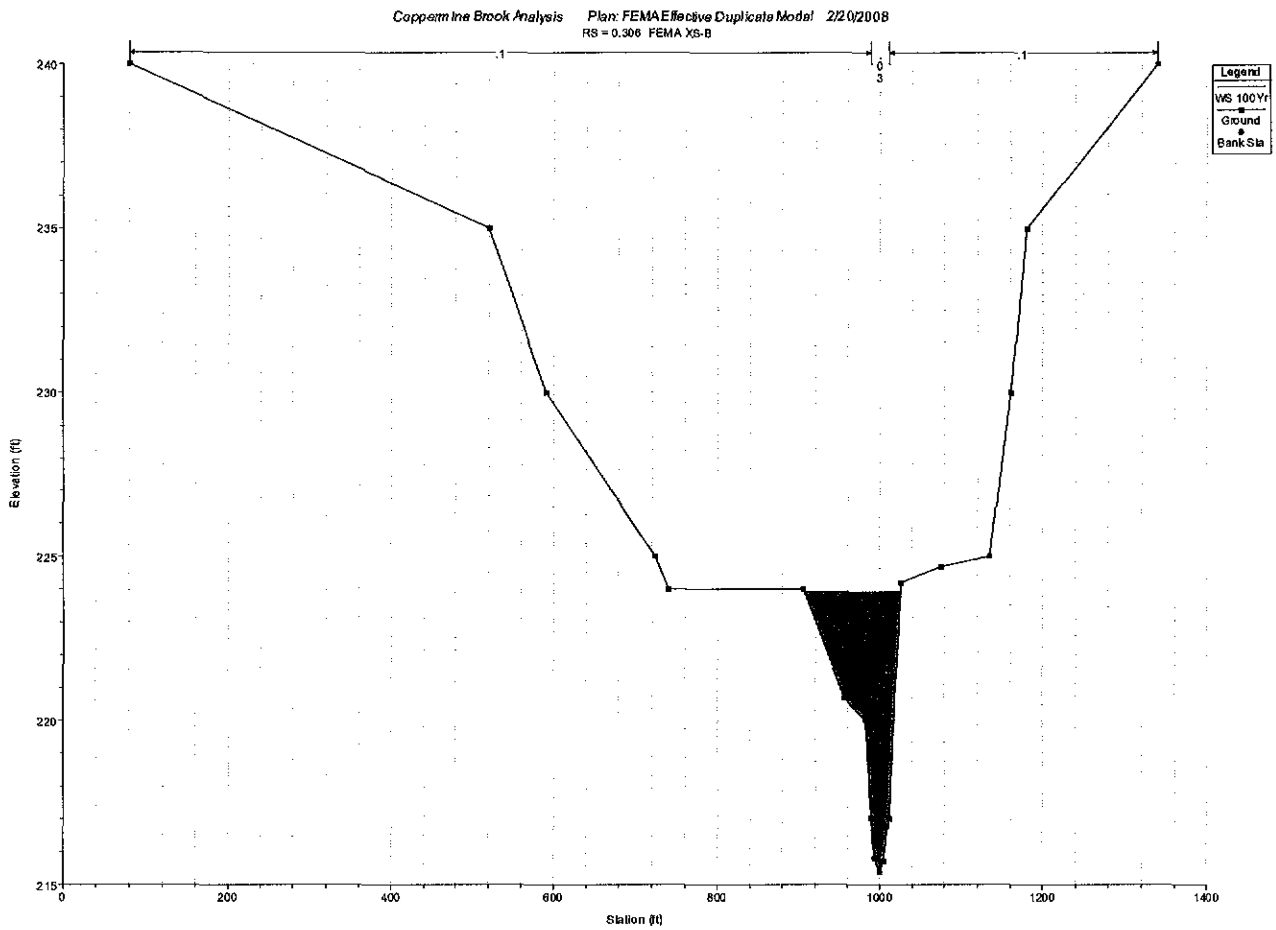
The evaluation of flood conditions requires an understanding of both the amount of flow that is being managed and the conditions within the channel that dictate the elevations that water will reach under a variety of flows. This determination of flood elevations is made through hydraulic analysis as described in this section.

Hydraulic analysis is commonly performed using the U.S. Army Corps of Engineers (ACOE) HEC-RAS (River Analysis System) software. The model is used to compute water surface profiles for one-dimensional, steady state, and gradually varied flow. This model can accommodate a full network of channels, a dendritic system, or a single river reach. HEC-RAS is capable of modeling water surface profiles under subcritical, supercritical, and mixed flow conditions.

The basic computational procedure of HEC-RAS is the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's Equation) and the contraction/expansion coefficient multiplied by the change in velocity head. The momentum equation is used in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations, hydraulics of dams and bridges, and evaluating profiles at a river confluence.

In developing a hydraulic model, cross section data is input to the model to define the channel at selected locations. A sample cross section is presented as Figure 4-1. This may include areas of broad floodplain, channel constrictions caused by erosion in the channel, and/or bridge crossings. At each location, the channel is defined by elevation as well as the type of vegetation present on the channel banks and the make-up of the channel bed. All of these influence water elevations during flood events.

FIGURE 4-1
Sample HEC-RAS Cross Section



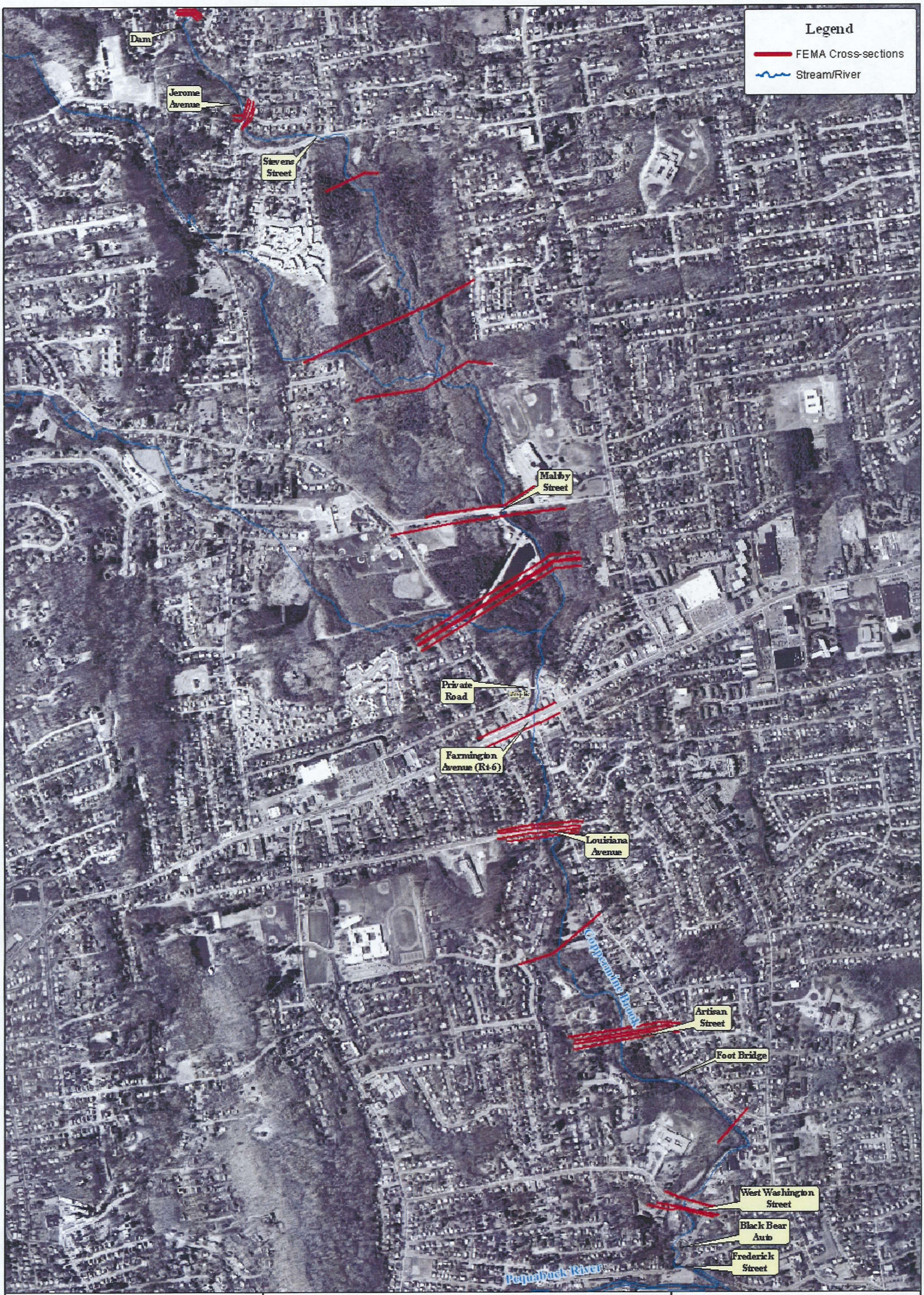
Another important consideration for hydraulic modeling is the elevation of water in the downstream end of the channel at the start of the storm event. This initial "tailwater" can influence flood elevations in the study channel, although the length of this influence varies based on the slope of the channel bed and the height of the water. The initial water levels are based upon those of the river into which a stream discharges.

4.2 FEMA Published Analysis

In all hydraulic analyses, there is a standardized procedure to be followed to ensure consistent results. This procedure starts with the Federal Emergency Management Agency (FEMA) study of the channel. Despite the inaccuracies that are sometimes identified in these studies, they do define the limit of the floodplain for regulatory and legal purposes. If future work will change the limits of the floodplain, the changes must be understood as they relate to the FEMA published information. Therefore, the procedure for analysis starts with the FEMA data as the baseline of comparison to future work.

In developing the City of Bristol FIS dated May 18, 1981, FEMA predicted water surface elevations for Coppermine Brook from the Pequabuck River upstream to the corporate boundary between the city of Bristol and the town of Burlington. A 100-year floodplain and floodway were delineated for Coppermine Brook, as was a 500-year floodplain.

MMI obtained the hydraulic model data (often called the effective model) that was developed by FEMA. Using that information, MMI created a duplicate effective model in HEC-RAS by manually inputting all data from the HEC-2 model into RAS. Figure 4-2 depicts the location of cross sections that were used by FEMA to model Coppermine Brook. The duplicate effective model directly incorporates the information used by FEMA without modifying the data to reflect current field conditions. The purpose of this is to ensure that the model can be accurately duplicated on current computers using



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COPPERMINE BROOK ANALYSIS

LOCATION: Burlington & Bristol, CT

MMI#: 2235-19-6
MXD: H:Fig4-2.mxd
SOURCE: http://magic.lib.uconn.edu/

N

FEMA CROSS-SECTIONS LOCATION MAP

DATE: 07/07/08
SCALE: 1:1,000

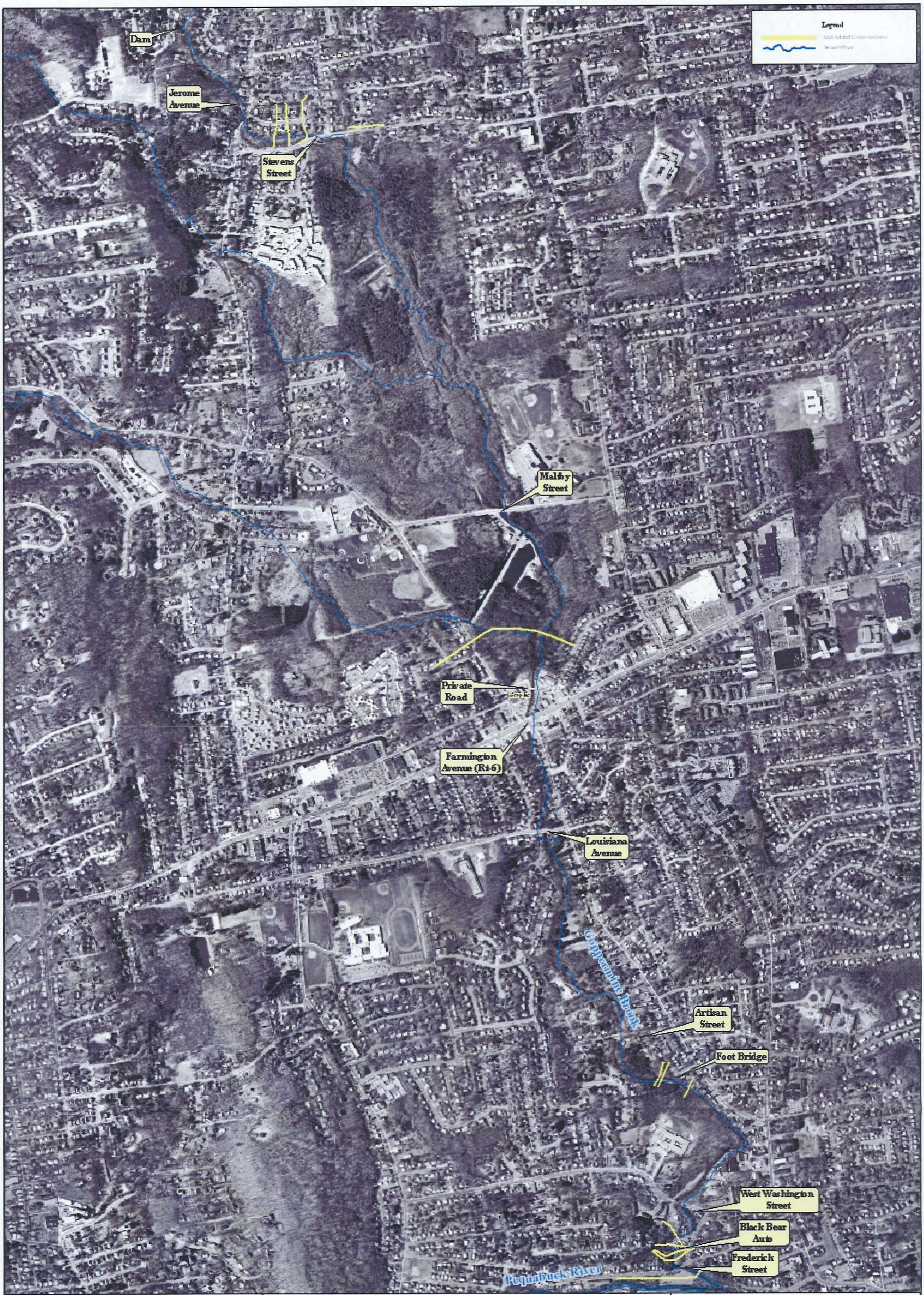
SHEET: Figure 4-2

current software. A copy of the FEMA model data as well as output from MMI's duplicate model are presented in Appendix G.

In preparing the FEMA duplicate model (to test our program), MMI used the same boundary conditions and model computations as represented in the FEMA HEC-2 model. A slope of 0.0055 was used as the downstream boundary condition, and the subcritical method was used as the model computation method. The use of slope as the downstream boundary rather than a known water surface elevation or normal depth is an error, in our opinion, and this was corrected in MMI's existing conditions model.

4.3 Existing Conditions Analysis

Using the FEMA duplication model as a base, MMI developed a more detailed model of Coppermine Brook for this study. In developing this model, MMI modified some FEMA cross sections, and additional cross sections were added using information from topographic mapping provided by the city and field survey by MMI. Bridge dimensions were also verified and corrected as necessary to reflect replacements and modifications that had occurred since the 1981 FEMA analysis. A total of 25 cross sections were modified, and 13 cross sections were added to the FEMA model. These are shown in Figure 4-3. As part of this effort, MMI evaluated each bridge crossing on Coppermine Brook to determine the available waterway opening. Table 4-1 is a summary of these structures and their available hydraulic capacity.



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COPPERMINE BROOK ANALYSIS

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N

MMI CROSS-SECTIONS LOCATION MAP

DATE: 07/07/08
SCALE: 1:1,000

SHEET: Figure 4-3

TABLE 4-1
Summary of Bridge Dimensions

Road	Opening Size (square feet)	Average Depth (feet)	Width (feet)
Jerome Avenue	770	9.5	80
Stevens Street	176	5.5	36
Maltby Street	572	6.8	110
Private Bridge	214	7.0	30
Farmington Avenue	279	7.5	38
Louisiana Avenue	241	6.3	42
Artisan Street	190	6.2	33
Foot Bridge	740	10.5	94
West Washington Street	264	8.0	33
Frederick Street	206	6.0	33

Note that in Table 4-1 the dimensions provided are average, while the opening was determined based on the most conservative cross section, which would reflect the hydraulic capacity of the structure. Regardless of the computational method, the important element of Table 4-1 is the variation in bridge opening through the watershed. Generally, as we move downstream in the watershed we expect flow rates to increase. As a result, bridge waterway openings generally should increase (there are exceptions to this generalization for bridges that are designed to overtop, but the general principle applies). Along Coppermine Brook, we see a wide variation in bridge capacity through the channel length.

During the course of this project and at the public information meeting, much discussion was had about the influence of the Pequabuck River on flooding at Coppermine Brook. It is our opinion that the FEMA model did not properly account for this; therefore, in our existing conditions model we sought to correct this element. Joint probability analysis using the methodology defined in the Connecticut Department of Transportation (ConnDOT) Drainage Manual was completed to estimate the impact of Pequabuck River tailwater on the downstream boundary condition of the model. The drainage areas of the Pequabuck River and Coppermine Brook at the confluence of the two waterbodies are approximately 44.6 and 18.6 square miles, respectively. Therefore, the ratio of the drainage areas of the Pequabuck River to Coppermine Brook is 2.4 to 1. Based on Table 8-3 of the ConnDOT Drainage Manual, the 2.4:1 ratio would require a coincidental occurrence frequency of 1:1 between the two waterbodies. This means that if one of the river basins was receiving precipitation in flood conditions it is likely the other adjacent basin is in flood stage as well. Therefore, it would be appropriate to use a 10-year water surface elevation of the Pequabuck River as the starting water surface elevation for evaluation of the 10-year flood in Coppermine Brook and a 100-year elevation in the Pequabuck River to analyze the 100-year flood on Coppermine Brook.

Water surface elevations in the Pequabuck River as determined from the FIS were used as the downstream boundary conditions for our analysis of Coppermine Brook. Water surface elevations and velocity distribution were computed for the 2-, 10-, 25-, 50-, 100-, and 500-year storm event using the discharge rates predicted by the existing conditions HEC-HMS analysis for Coppermine Brook. Recall from Section 3 that for the 100-year event MMI predicted flows that are higher than those used by FEMA. Table 4-2 presents the results of the existing conditions modeling in comparison to the elevations predicted in the FEMA study at selected locations along the channel. Model output is presented in Appendix H.

TABLE 4-2
Comparison of Predicted Water Surface Elevations at 100-Year Flows - FEMA Published Data Versus MMI Existing Conditions

FEMA Cross Section Description	FEMA Cross Section Location	FEMA Published 1981	MMI Existing Conditions 2008	Difference (feet)
A	Upstream of Frederick St.	219.9	218.4	-1.5
B	Downstream of West Washington St.	225.7	224.4	-1.3
C	Between West Washington St. & Foot Bridge	231.0	231.4	0.4
D	Upstream of Foot Bridge	242.9	237.5	-5.4[1]
E	Upstream of Artisan St.	246.3	246.6	0.3
F	Between Louisiana Ave. & Artisan St.	246.7	247.3	0.6
G	Upstream of Louisiana Ave.	247.8	248.0	0.2
H	Upstream of Farmington Ave.	254.8	254.6	-0.2
I	Between Dam at Treatment Plant & Private Driveway	256.5	255.6	-0.9
J	Upstream of Maltby St.	256.7	256.1	-0.4
K	Between Maltby St. & Stevens St.	257.4	258.5	1.1
L	Between Maltby St. & Stevens St.	262.1	262.0	-0.1
M	Upstream of Stevens St.	272.7	273.1	0.4
N	Upstream of Jerome Ave.	287.5	285.7	-1.8

Note: All elevations are in feet based on the National Geodetic Vertical Datum of 1929.

1. MMI revised the bridge opening to reflect the existing compound channel, which is not reflected in the FEMA model.

The difference in water surface elevation between MMI's existing conditions model and FEMA published data can be attributed to the difference in the computation methods between the two models, new survey data, and bridge improvements. The addition of cross sections added by MMI resulted in a more detailed definition of the floodplain geometry and more current bridge cross section data. Therefore, despite the increased flow rates predicted by MMI, the predicted water surface elevations dropped. The existing conditions modeling results indicate several crossings overtopping during a 100-year storm event. The crossings include Frederick Street, West Washington Street, Artisan Avenue, Louisiana Avenue, Farmington Avenue, private driveway near Staples, Maltby Street, and Stevens Street.

Using the elevations developed in this existing conditions model and topographic mapping available from the City of Bristol, MMI mapped the floodplain of Coppermine Brook and compared that mapping to the floodplain limit published by FEMA. The mapping is presented in Appendix I.

4.4 Mitigation Analysis

As part of this study, the city requested that MMI evaluate potential strategies for alleviating flooding along Coppermine Brook. As previously presented, three distinct flood prone areas were identified: Richards Court/Stevens Street, Farmington Avenue, and Frederick Street. Strategies for managing flooding at each of these locations were evaluated as described in the following sections.

4.4.1 Stevens Street/Richards Court

Results of the existing conditions analysis indicated that the 10-year flood stays within the channel and berm except at the location where the berm has failed. During the 25-, 50-, and 100-year storms water overtops the berm and flows laterally and downstream on the developed floodplain, flooding houses at Richard Court and Candy Lane before it overtops Stevens Road at the low point. While the hydraulic capacity of the Stevens Street bridge is limited, the elevation of adjacent homes and property control the elevation of Stevens Street and therefore limit the ability to increase the capacity here in a meaningful way. As a result of our analysis, we also noted that the Stevens Street bridge is subject to tailwater due to limited capacity in the channel downstream. During this alternatives analysis, we evaluated correcting that downstream condition in an effort to increase the efficiency of the available waterway opening at Stevens Street.

The following potential improvements were identified and evaluated at this area:

- Removal of sediment from beneath the Stevens Street bridge.
- Repair of the berm upstream of Stevens Street at its current elevation.
- Replacement of the berm upstream of Stevens Street at an elevation that fully contains the 100-year flood.
- Removal of the berm upstream of Stevens Street and creation of a compound channel.
- Relocation of the berm farther from the channel.
- A combination of modifying the channel downstream of Stevens Street, lowering the channel at the bridge, and relocating the berm further from the channel.

Each of these alternatives was evaluated using the existing conditions HEC-RAS model. Results of each of these are presented as follows. Model output for all alternatives related to Stevens Street is presented in Appendix J. For all alternatives, sealing the existing drain pipe under the earth berm at #72 Richards Court is essential. A flap gate could be used, but a pump station may be the ultimate solution.

Removal of Sediment from Stevens Street Bridge

A sediment bar consisting of gravel and cobble has formed at the upstream side of this bridge. The bar is believed to have formed here because of several conditions, including a sharp channel bend, a channel slope reduction, and the periodic bridge headwater pool that encourages settling. During field observations in spring 2008, sediment has accumulated as deep as two feet over half of the channel. Removal of this sediment was modeled to evaluate if that material is impacting flooding in this area. Results of this analysis are presented in Table 4-3.

TABLE 4-3
Comparison of Existing and Proposed Water Surface Elevations at 10-Year Flows – Removal of Sediment at Stevens Street Bridge

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
Downstream face of Stevens Street	269.87	269.87	0.00
Upstream face of Stevens Street	272.92	272.25	-0.67
50 feet upstream of Stevens Street	272.91	271.99	-0.92
100 feet upstream of Stevens Street	273.05	272.14	-0.91
375 feet upstream of Stevens Street	273.71	273.35	-0.36
540 feet upstream of Stevens Street	275.18	275.11	-0.07
50 feet downstream of Jerome Avenue	282.40	282.40	0.00

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

The above table shows a decrease in the water surface elevation during a 10-year flood, although this decrease is localized, affecting the area immediately upstream of the bridge most significantly. No substantial decrease in water surface elevation was observed during the 25-, 50-, and 100-year storm events. Removal of sediment from this area will provide some localized flood benefit during smaller, more frequent storms; however, the benefit to the homes on Richards Court is not expected to be large. Most importantly, this sediment bar could increase over time and needs to be monitored and periodically removed.

Repair of the Berm at Current Elevation

Under this alternative, the existing berm(s) would be modified to eliminate the low point that allows water to flow toward Richards Court. This would require work on property that is not within the city's control. During our field evaluation, it appeared that two parallel berms are located in this area, and an existing low point allows water to flow between them, ending up behind the rear yards of homes at the end of Richards Court. Table 4-4 shows the comparison of existing and proposed water surface elevations during the 10-year flood.

TABLE 4-4
Comparison of Existing and Proposed Water Surface Elevations at 10-Year Flows
Repair Berm at Current Elevation

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
Downstream face of Stevens Street	269.87	269.87	0.00
Upstream face of Stevens Street	272.92	272.92	0.00
50 feet upstream of Stevens Street	272.91	272.91	0.00
100 feet upstream of Stevens Street	273.05	273.05	0.00
375 feet upstream of Stevens Street	273.71	273.71	0.00
540 feet upstream of Stevens Street	275.18	275.14	0.04
50 feet downstream of Jerome Avenue	282.40	282.40	0.00

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

At first glance, modification of the berm provides no apparent benefit for managing flooding at the properties on Richards Court. During larger storms, the berm overtops; therefore, no flood management benefit would be realized during these storms. However, the HEC-RAS computer model has limited ability to represent this flow scenario. It is our opinion that closing the gap between the berms is warranted.

Replace Berm at Higher Elevation

Under this alternative, the existing berm would be raised two feet along its entire length to prevent overtopping. Table 4-5 shows the comparison of existing and proposed water surface elevations during the 25-year flood. This larger flood was selected (as opposed to the 10-year event of the previous alternatives) because model results show that raising the berm would successfully prevent overtopping during the 10-year event. However, during the 25-year event the undersized Stevens Street bridge would force overtopping of the channel approximately 50 feet upstream of the bridge. Therefore, replacing this berm at a higher elevation would provide some benefit, but it needs to be more than two feet higher, and the drainage pipe from #72 Richards Court must be removed or sealed.

TABLE 4-5
Comparison of Existing and Proposed Water Surface Elevations at 25-Year Flows
Replace Berm at Higher Elevation

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
Downstream face of Stevens Street	271.30	271.30	0.00
Upstream face of Stevens Street	272.17	274.72	+2.55
50 feet upstream of Stevens Street	272.40	274.52	+2.12
100 feet upstream of Stevens Street	271.81	274.61	+2.80
375 feet upstream of Stevens Street	274.85	275.16	+0.31
540 feet upstream of Stevens Street	276.55	276.74	+0.19
50 feet downstream of Jerome Avenue	283.32	283.31	-0.01

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

It is important to understand that these results predict an increase in water surface elevation because the increased berm height constricts the channel flow area. When the flow area is reduced while the peak flow rates remain the same, the water surface elevation must become higher.

Remove Berm and Construct Compound Channel

Under this scenario, the berm would be removed and the overbank would be regraded to provide a flood shelf and additional conveyance capacity during high flows. The channel location and geometry would remain the same, but the left bank area would be modified to remove the berms. This channel cross section would be more hydraulically efficient than the current configuration, possibly providing some reduction in flooding. It is important to understand that implementation of this alternative will require extensive work on private property. Results of this modeling effort are presented in Table 4-6.

TABLE 4-6
Comparison of Existing and Proposed Water Surface Elevations at 25-Year Flows
Remove Berm and Construct Compound Channel

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
Downstream face of Stevens Street	271.30	271.3	0.00
Upstream face of Stevens Street	272.17	272.17	0.00
50 feet upstream of Stevens Street	272.40	272.40	0.00
100 feet upstream of Stevens Street	271.81	271.81	0.00
375 feet upstream of Stevens Street	274.85	274.98	0.13
540 feet upstream of Stevens Street	276.55	276.05	-0.50
50 feet downstream of Jerome Avenue	283.32	283.32	0.00

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

This alternative resulted in minimal changes in water surface elevation during a 25-year storm event as seen in Table 4-6. While there is some benefit, it is localized and does not resolve the major flooding issues in this area.

Relocate Berm Further From Channel

For this alternative, the existing berms would be removed and reconstructed closer to the homes on Richards Court at an elevation that fully contains the 100-year flow. The result would be some encroachment into the lawn areas in the hope of reducing flood damage. The berm would be raised by approximately two feet at the existing break to prevent overtopping. Table 4-7 presents the results of this alternative.

TABLE 4-7
Comparison of Existing and Proposed Water Surface Elevations at 25 -Year Flows
Relocate Berm Further From Channel

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
Downstream face of Stevens Street	271.30	271.30	0.00
Upstream face of Stevens Street	272.17	274.96	+2.79
50 feet upstream of Stevens Street	272.40	274.77	+2.37
100 feet upstream of Stevens Street	271.81	274.83	+3.02
375 feet upstream of Stevens Street	274.85	275.55	+0.70
540 feet upstream of Stevens Street	276.55	276.50	-0.05
50 feet downstream of Jerome Avenue	283.32	283.31	-0.01

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

While this alternative contains the 25-year flood within the channel, this is achieved by increasing the water surface elevation at Stevens Street. The projected three-foot increase in water surface elevation would increase flow velocities and be detrimental to the existing stream banks. In other words, this alternative is a short-term unsustainable solution. Relocating the berm creates a wider flow area, resulting in lower velocity. A lower velocity is associated with deeper water.

Combination of Modifying Channel Downstream of Stevens Street, Lowering the Channel at the Bridge, and Relocating the Berm Further From the Channel

The goal of this alternative is to maximize use of the existing bridge. Site inspections revealed that an old gravel road parallels Coppermine Brook on the right bank downstream of the Stevens Street bridge, and surveys indicate this short channel reach has some gradient.

Modeling was completed in a series of incremental steps to evaluate the potential impacts (or benefits) of increasing the downstream conveyance, lowering the channel at the bridge, and relocating the berm further from the channel. Flow through the Stevens Street bridge is adversely impacted by high water surface elevations downstream of the bridge. The undersized downstream channel creates a backwater condition that impacts this undersized structure and encourages sediment deposition.

The first step was to model lowering and widening of the downstream channel. If this work were completed, the 10-year flood event would be lowered by approximately five feet 50 feet downstream of bridge, but the water surface elevation upstream of the bridge would not be affected.

The next step involved lowering the channel at the bridge as well. The channel at the downstream face was lowered by three feet, and the sediment at the upstream face of the bridge was removed. (It should be noted that bed lowering at the bridge may adversely impact the footings of this structure. The elevation of these footings must be evaluated if this alternative is to be pursued further.) The model results indicate that the crossing can now convey a 25-year storm without overtopping. Relocating the upstream berm further from the channel further enhanced the channel conveying capacity upstream of the crossing. A combination of these proposed changes resulted in Stevens Street conveying a 50-year flood event without overtopping at the crossing and the berm; however, it must be noted that this alternative proposes work on private property. Table 4-8 presents the results of this model run.

TABLE 4-8
Comparison of Existing and Proposed Water Surface Elevations at 50-Year Flows
Combination of Potential Changes at Stevens Street

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
Downstream face of Stevens Street	271.95	268.34	-3.61
Upstream face of Stevens Street	272.45	267.84	-4.61
50 feet upstream of Stevens Street	272.76	272.21	-0.55
100 feet upstream of Stevens Street	273.45	272.71	-0.74
375 feet upstream of Stevens Street	276.09	275.94	-0.15
540 feet upstream of Stevens Street	277.47	277.18	-0.29
50 feet downstream of Jerome Avenue	284.06	284.06	0.00

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

Candy Lane

The testimony at the public meeting and computer model of existing flow conditions both indicate that the Candy Lane neighborhood is subject to flooding. The low lying area is on the north side of Stevens Street and is designated on early maps as alluvial soil floodplains.

Field inspections indicate the brook from Lake Como passes through this area and under Stevens Lane via pipe culverts. The floodwater from Coppermine Brook backs up into and above these pipes, restricting outflow. The long, low gradient channel from Stevens Street to Maltby Street cannot easily or practically be modified to prevent this. Flood hazard control methods are limited to protective measures such as flood proofing, raising buildings, and use of fill to create barriers.

4.4.2 Farmington Avenue

Based on input from residents and city staff, the area around Farmington Avenue was identified for evaluation. Farmington Avenue is a state highway with densely developed commercial properties surrounding the bridge. This structure was replaced by the Connecticut Department of Transportation in 2005. The existing conditions hydraulic analysis indicates that Farmington Avenue passes up to a 10-year storm event without overtopping. Under existing conditions, the model predicts that Farmington Avenue overtops by two to four feet for the storm events ranging from a 25-year to 100-year event. This cannot be corrected just by using a longer bridge.

Also affecting flooding in this area is a narrow private bridge that connects the Staples parking lot with the commercial property on the east bank of the river. This structure appears quite old and is narrow when compared to the upstream channel. Compounding the problem in this area is the fact that the channel is narrowing downstream of the large wetland floodplain area that exists at Maltby Road.

Further creating hydraulic restrictions here is the downstream channel. Recall from Section 2.3 that the channel downstream of this bridge is incising, which separates the channel from its floodplain. While incision is part of the natural progression of channel evolution, it does restrict the capacity of the channel.

The following alternatives were evaluated for this area:

- Removal of the Farmington Avenue bridge.
- Removal of the undersized private bridge.
- Removal of the Farmington Avenue bridge and undersized private bridge.

Each of these alternatives was evaluated using the existing conditions HEC-RAS model. Results are presented as follows. Model output for all alternatives related to Farmington Avenue is presented in Appendix K.

It should be noted that numerous reports were provided by residents and city staff that flooding at Farmington Avenue is not solely the result of these structures. Many reports were provided that indicated water enters Mix Street north of Staples and then flows down Mix Street to the intersection at Farmington Avenue. This condition may occur because the channel constricts from the broad floodplain at the confluence of Polkville Brook to the narrow channel that is observed near Farmington Avenue. This constriction limits the amount of flow in the channel. The elevation of Mix Street is only slightly higher than the floodplain wetland in this area, allowing water to readily enter the street. Correcting this condition cannot be easily accomplished due to floodplain development.

Removal of Farmington Avenue Bridge

This structure is within a state highway and is not the city's responsibility to maintain. However, there have been numerous resident complaints suggesting that the flooding of

Farmington Avenue is caused by this structure being undersized. This alternative is intended to evaluate whether this is indeed a correct assumption. To evaluate this, the existing conditions model was modified and the bridge was removed entirely, including both the superstructure and the walls. Some minor grading was assumed necessary to reflect sloped banks rather than the vertical ones that currently exist for the bridge. Table 4-9 presents the results of this analysis.

TABLE 4-9
Comparison of Existing and Proposed Water Surface Elevations at 100-Year Flows
Removal of Farmington Avenue Bridge

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
50 feet upstream of Louisiana Avenue	248.02	248.02	0.00
50 feet downstream of Farmington Avenue	253.18	251.65	-1.53
Downstream face of Farmington Avenue	254.45	252.17	-2.28
Upstream face of Farmington Avenue	254.55	252.38	-2.17
50 feet upstream of Farmington Avenue	254.55	252.53	-2.02
50 feet downstream of Private Driveway	254.81	253.92	-0.89
Downstream face of Private Driveway	254.78	253.86	-0.92
Upstream face of Private Driveway	255.00	254.57	-0.43
50 feet upstream of Private Driveway	255.08	254.80	-0.28
720 feet upstream of Private Driveway	255.52	255.34	-0.18

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

The existing bridge overtops during a 25-year storm event, and removal of the bridge resulted in a decrease in water surface elevation. However, the predicted elevations are still above the adjacent sections of Farmington Avenue. In other words, Farmington Avenue would still overtop and the surrounding properties would flood even if the bridge did not exist.

Removal of Undersized Private Bridge

In evaluating the existing conditions model, it was noted that this structure is undersized and cannot adequately convey the flood flows. To evaluate the potential benefits of removing this structure, the existing conditions model was modified and the bridge was removed, including both the superstructure and the walls. Some minor grading was assumed needed to reflect sloped banks rather than the vertical ones that currently exist for the bridge. Table 4-10 presents the results of this analysis.

TABLE 4-10
Comparison of Existing and Proposed Water Surface Elevations at 100-Year Flows
Removal of Undersized Private Bridge

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
50 feet downstream of Private Driveway	254.81	254.81	0.00
Downstream face of Private Driveway	254.78	254.74	-0.04
Upstream face of Private Driveway	255.00	254.71	-0.29
50 feet upstream of Private Driveway	255.08	254.82	-0.26
720 feet upstream of Private Driveway	255.52	255.35	-0.17
Upstream of Maltby Street	256.05	255.98	-0.07

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

The existing private bridge overtops during a 25-year storm event. Although this bridge is undersized, its removal produces minimal changes in water surface elevation, even during a large 100-year storm event.

Removal of Farmington Avenue Bridge and Undersized Private Bridge

This alternative evaluates removal of both the Farmington Avenue bridge and the undersized private bridge along with widening the channel from downstream of the Farmington Avenue bridge to upstream of the private bridge. The existing conditions model was modified, and the bridges were removed entirely, including both the superstructure and the walls. The existing 25- to 30-foot wide channel between the two bridges was widened to 40 feet. It was observed that under this scenario the proposed channel can handle up to a 50-year storm event without overtopping.

TABLE 4-11
Comparison of Existing and Proposed Water Surface Elevations at 50-Year Flows
Removal of Farmington Avenue and Undersized Bridge

Cross Section Description	Existing Conditions WSEL	Proposed Conditions WSEL	Difference (feet)
50 feet downstream of Farmington Avenue	250.15	249.94	-0.21
Downstream face of Farmington Avenue	254.94	249.94	-5.00
Upstream face of Farmington Avenue	255.02	251.54	-3.48
50 feet upstream of Farmington Avenue	255.03	251.36	-3.67
50 feet downstream of Private Driveway	255.12	251.85	-3.27
Downstream face of Private Driveway	255.11	251.95	-3.16
Upstream face of Private Driveway	255.19	252.26	-2.93
50 feet upstream of Private Driveway	255.21	252.90	-2.31
720 feet upstream of Private Driveway	255.42	253.64	-1.78

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

For the sake of completeness of the evaluation, another run was performed assuming that the upstream channel was widened as described above and that the Farmington Avenue bridge was raised by one foot when compared to the existing conditions. It was observed that a 50-year storm flood passes through the Farmington Avenue crossing with the water staying within the proposed channel and barely overtopping upstream of Farmington Avenue.

4.4.3 Frederick Street

Frederick Street is located approximately 450 feet upstream of the confluence of Coppermine Brook with the Pequabuck River. The existing bridge has a waterway opening width of 33 feet. Concrete parapet walls approximately 3.25 feet higher than the roadway elevation are located on the upstream and downstream face of the crossing.

There is a maximum of nine feet of clearance between the channel bed and the low chord of the structure at its upstream face, but part of waterway is filled with sediment.

Upstream of the bridge, Coppermine Brook is contained within an earthen berm on the right bank. On the left bank, a vegetated sediment bar has developed, and the channel makes two approximately 90 degree bends immediately before entry to the bridge. The earth channel below the Frederick Street crossing is trapezoidal in shape and appears to have been significantly manipulated over time.

Critical elevations in the vicinity of this crossing are as follows:

- Low point of Frederick Street: 215.31 feet (NDVD 29)
- Roadway elevation at bridge: 217.98 feet (NGVD 29)
- Finished floor elevation at the house located on the right bank upstream of Frederick Street: 219.95 feet (NGVD29)
- Bottom of bridge beam: 214.48 feet (NGVD29)
- Dike elevation: 217.00 to 217.98 feet (NGVD29)
- Pequabuck River 10-year: 212.20 feet (NGVD29)
- Pequabuck River 50-year: 216.00 feet (NGVD29)
- Pequabuck River 100-year: 216.40 feet (NGVD29)

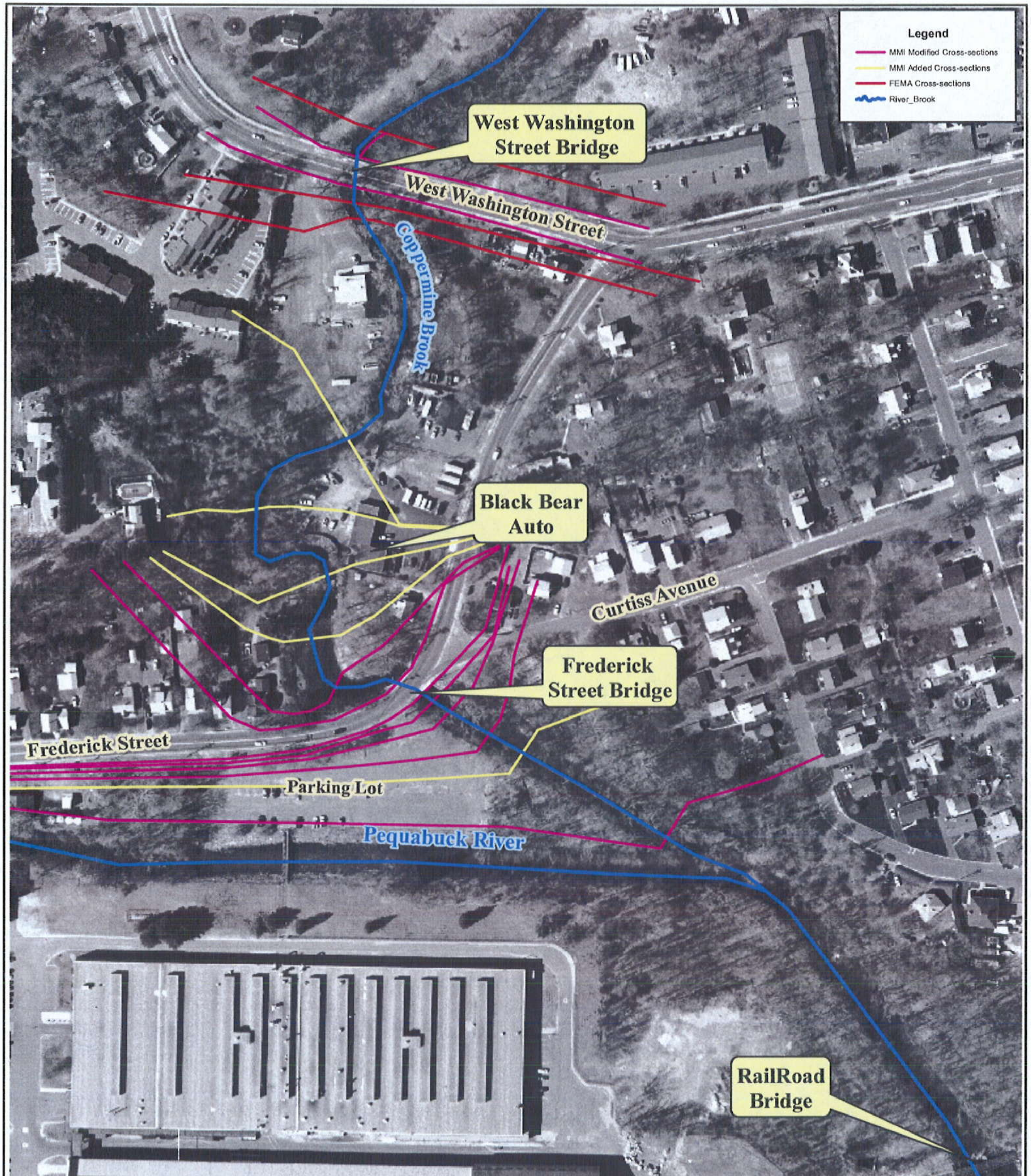
The key elevations indicate that the yard at the house located on the right bank upstream of Frederick Street will be flooded from the Pequabuck River if Coppermine Brook was not present. It has been reported that the Frederick Street crossing overtops during flood events due to inadequate hydraulic capacity of the bridge. This is the result of both tailwater flooding from the Pequabuck River as well as Coppermine Brook flows. Nuisance flooding occurs with water flowing across Frederick Street at its low point. Flooding has also occurred upstream of the bridge. This occurs when the channel overtops behind Black Bear Auto and flows through the parking lot of that property into Frederick Street, reentering Coppermine Brook downstream of Frederick Street. The houses located on the right bank upstream of the crossing also experience flooding due to water overtopping the berm that bounds the brook. Figure 4-4 depicts this area and also presents the cross sections referenced specifically in this analysis.

Frederick Street is subject to three types of flooding:

1. River flooding and erosion along the dike that can be resolved with a larger channel and higher dike.
2. Bridge backwater that can be partially resolved with a larger or supplemental bridge.
3. Pequabuck River backwater, which cannot be resolved.

The following alternatives were evaluated to relieve flooding at this location:

- Removal of the Frederick Street crossing.
- Replacement of the Frederick Street crossing with a structure capable of passing the 25-, 50-, and 100-year storm events.
- Construction of a high overflow culvert on the right bank through the existing parking lot.
- Construction of a formal compound channel upstream of Frederick Street behind Black Bear Auto.
- Relocation of the channel behind Black Bear Auto to eliminate the meander.



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COPPERMINE BROOK ANALYSIS

LOCATION: Burlington & Bristol, CT

MMI#: 2235-19-06

MXD: H:Fig 4-5.mxd

SOURCE: http://magic.lib.uconn.edu/

N

DETAIL OF FREDERICK STREET MAP

DATE: 07/07/08

SCALE: 1" = 200'

SHEET: Figure 4-4

Each of these alternatives was evaluated by modifying the existing conditions HEC-RAS model to reflect proposed changes discussed above. There has been much public comment about the influence of the Pequabuck River on flooding in this reach of Coppermine Brook. In order to more fully understand that influence, MMI performed one model run assuming the Pequabuck River had no influence on Coppermine Brook and compared it to runs that assumed the Pequabuck River had some influence. Table 4-12 presents these results and shows that the Pequabuck River has some influence up the Frederick Street crossing but limited influence upstream of it.

TABLE 4-12
Comparison of Water Surface Elevations With Pequabuck River Backwater and Without – 25-Year Event

Cross Section Description	Existing WSEL With Pequabuck River Tailwater	Existing WSEL With Normal Depth	Difference (feet)
Confluence of Pequabuck River	212.20	210.99	-1.21
175 ft. downstream of Frederick Street	212.91	212.60	-0.31
120 ft. downstream of Frederick Street	213.03	212.84	-0.19
50 ft. downstream of Frederick Street	213.91	213.92	0.01
20 ft. downstream of Frederick Street	214.21	214.22	0.01
Downstream face of Frederick Street	214.06	214.07	0.01
Upstream face of Frederick Street	215.22	215.22	0.00
100 ft. upstream of Frederick Street	215.77	215.77	0.00

The alternatives analysis uses the known water surface elevation of the Pequabuck River as the downstream boundary condition. Modeling results of each alternative are presented in Appendix L.

Removal of Frederick Street Crossing

Under this alternative, the Frederick Street bridge would be removed entirely with two dead-end streets resulting. It should be noted that we do not believe this alternative is feasible for reasons of public safety. Emergency response is severely hindered by the creation of dead-end roads, and most communities (including Bristol – see section 5.03(5) of the Subdivision Regulations) limit the length of dead-end roads as well as the number of houses that can be served by a dead end. Given the land uses surrounding the Frederick Street bridge, we would not recommend such conversion of this road. However, it was modeled in an effort to demonstrate the hydraulics of this section of channel and more fully understand the influence of the bridge structure.

In modeling this alternative, the channel was widened on the right bank by approximately 30 feet. The channel bed was lowered by up to one foot to 175 feet downstream of the current bridge location to maintain a uniform slope of the channel. The left bank approximately 100 to 250 feet upstream of the crossing was modified to widen the bend. A berm was included from the Black Bear Auto property downstream to force water to stay within the channel. Tables 4-13 and 4-14 compare existing and proposed water surface elevations during the 10- and 25-year floods, respectively.

TABLE 4-13
Comparison of Existing and Proposed Water Surface Elevations at 10-Year Flows – Removal of Frederick Street Bridge

Cross Section Description	Existing WSEL	Proposed WSEL	Difference (feet)
Confluence of Pequabuck River	212.20	212.20	0.00
175 ft. downstream of Frederick Street	212.91	212.91	0.00
120 ft. downstream of Frederick Street	213.03	212.91	-0.12
50 ft. downstream of Frederick Street	213.91	213.26	-0.65
20 ft. downstream of Frederick Street	214.21	213.45	-0.76
Downstream face of Frederick Street	214.06	213.41	-0.65
Upstream face of Frederick Street	215.22	213.86	-1.36
100 ft. upstream of Frederick Street	215.77	214.91	-0.86
At the bend upstream of Frederick Street	215.89	214.95	-0.94
Downstream of Black Bear Auto	215.25	214.17	-1.08
Adjacent to Black Bear Auto	216.98	216.95	-0.03
Upstream of Black Bear Auto	216.26	216.48	0.22
FEMA cross section B	219.15	219.09	-0.06
40 ft. downstream of West Washington Street	219.12	219.06	-0.06
Downstream face of West Washington Street	221.66	221.68	0.02
Upstream face of West Washington Street	221.67	221.69	0.02

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

TABLE 4-14
Comparison of Existing and Proposed Water Surface Elevations at 25-Year Flows - Removal of Frederick Street Bridge

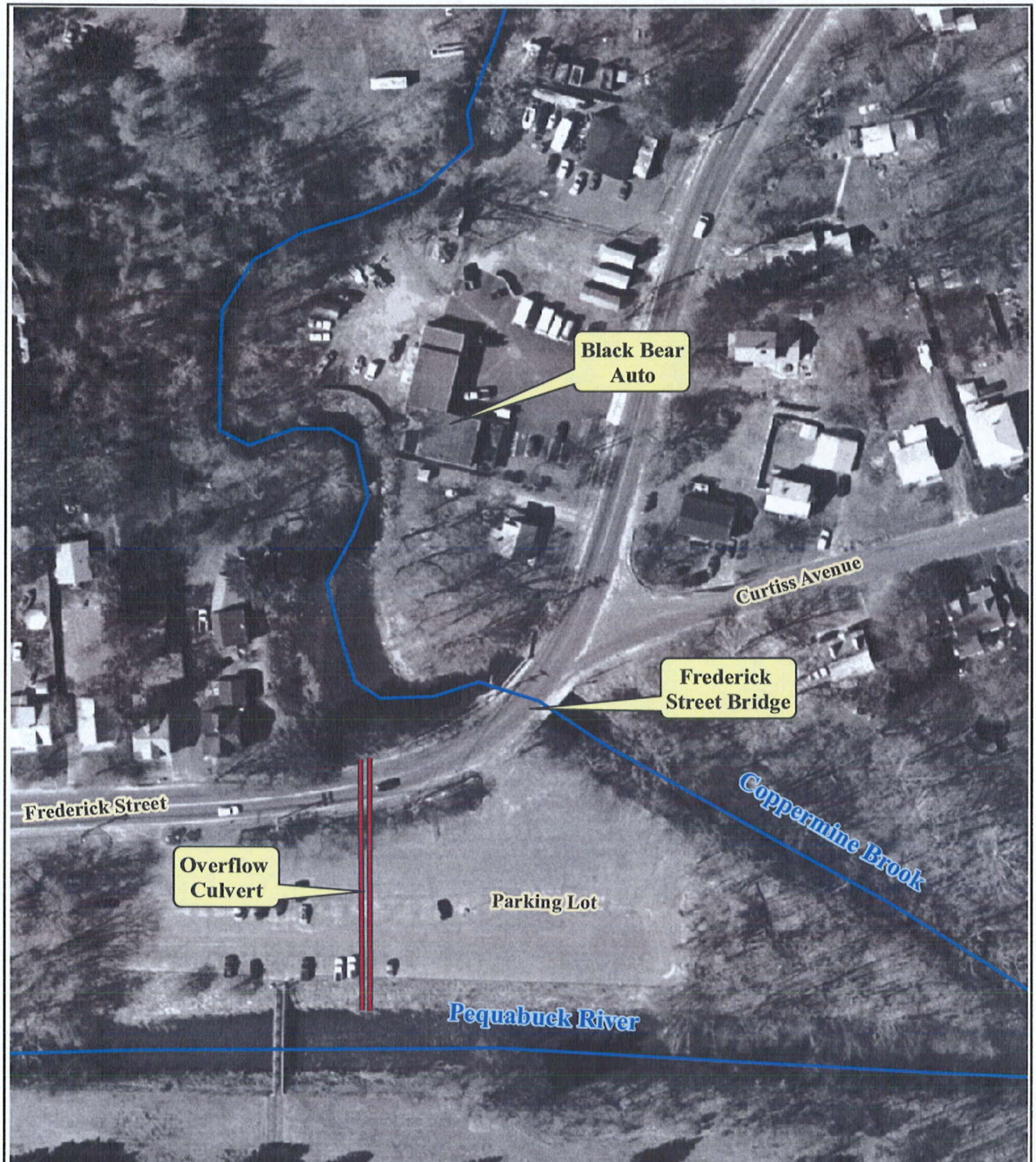
Cross Section Description	Existing WSEL	Proposed WSEL	Difference (feet)
Confluence of Pequabuck River	214.50	214.50	0.00
175 ft. downstream of Frederick Street	214.75	214.84	0.09
120 ft. downstream of Frederick Street	214.67	214.75	0.08
50 ft. downstream of Frederick Street	215.52	215.08	-0.44
20 ft. downstream of Frederick Street	215.70	215.18	-0.52
Downstream face of Frederick Street	215.29	214.97	-0.32
Upstream face of Frederick Street	217.24	215.43	-1.81
100 ft. upstream of Frederick Street	217.40	216.92	-0.48
At the bend upstream of Frederick Street	217.50	216.95	-0.55
Downstream of Black Bear Auto	217.39	215.66	-1.73
Adjacent to Black Bear Auto	217.96	218.84	0.88
Upstream of Black Bear Auto	217.29	217.29	0.00
FEMA cross section B	221.04	221.04	0.00
40 ft. downstream of West Washington Street	221.46	221.46	0.00
Downstream face of West Washington Street	223.74	223.74	0.00
Upstream face of West Washington Street	224.88	224.88	0.00

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

Based on this information, it is clear that the complete removal of the Frederick Street structure and channel improvements would have some limited influence on water surface elevations upstream of the structure.

Construction of a High Overflow Culvert Beneath Theis Steel Parking Lot

The intention of this alternative is to increase the capacity of the Frederick Street crossing without completing a full bridge replacement. Twin concrete box culverts with dimensions six feet wide by eight feet high are proposed to be installed at the low point of Frederick Street, extending across the parking lot of Theis Steel and discharging in the Pequabuck River upstream of Coppermine Brook. The existing Frederick Street crossing and the channel downstream will be retained. Figure 4-5 is a conceptual sketch of this alternative.



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COPPERMINE BROOK ANALYSIS			LOCATION: Burlington & Bristol, CT	
MMI#: 2235-19-06 MXD: H:Fig 4-6.mxd SOURCE: http://magic.lib.uconn.edu/	N	CONCEPT SKETCH-OVERFLOW CULVERT	DATE: 07/07/08 SCALE: 1" = 100'	SHEET: Figure 4-5

The intent of this culvert is to convey flows during flood events only and, as such, the elevation of these culverts was set 1.15 feet above the current channel bed. During periods of mean flow in the channel, all of the water would continue on its current course.

This alternative was represented as split flow analysis in the hydraulic model. A separate reach was established for the overflow culvert, and the amount of flow through the culvert was controlled by the flow optimization option in the model. This allows the model to split the flow automatically. Table 4-15 presents these results.

TABLE 4-15
Comparison of Existing and Proposed Water Surface Elevations at 25-Year Flows – High Overflow Culvert

Cross Section Description	Existing WSEL	Proposed WSEL 2-6'x8'	Proposed WSEL 2-6'x24'
Confluence of Pequabuck River	214.50	214.50	214.50
175 ft. downstream of Frederick Street	214.75	215.03	215.11
120 ft. downstream of Frederick Street	214.67	215.00	215.09
50 ft. downstream of Frederick Street	215.52	215.16	215.16
20 ft. downstream of Frederick Street	215.70	215.22	215.19
Downstream face of Frederick Street	215.29	215.10	215.14
Upstream face of Frederick Street	217.24	215.86	215.55
100 ft. upstream of Frederick Street	217.40	216.22	215.54
At the bend upstream of Frederick Street	217.50	216.34	215.71
Downstream of Black Bear Auto	217.39	215.66	215.66
Adjacent to Black Bear Auto	217.96	218.84	218.84
Upstream of Black Bear Auto	217.29	217.29	217.29
FEMA cross section B	221.04	221.04	221.04
40 ft. downstream of West Washington Street	221.46	221.46	221.46
Downstream face of West Washington Street	223.74	223.74	223.74
Upstream face of West Washington Street	224.88	224.88	224.88

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

This alternative indicates that overtopping of Frederick Street would continue during a 25-year storm. The proposed culvert is not hydraulically adequate to convey additional flow during high storm events. The size of the overflow culvert was limited by constructability issues – the culverts cannot be higher because of the current elevation of Frederick Street and the parking lot. Wider box culverts up to 24 feet were evaluated.

The results indicate that although the water surface elevation drops the Fredrick Street bridge continues to overtop during a 25-year storm event.

Replacement of Frederick Street Bridge with Larger Structure

Under this alternative, the Frederick Street bridge would be fully replaced and the upstream and downstream channels modified to maximize hydraulic efficiency. Increasing the capacity of a bridge structure can be accomplished by providing a wider structure and sometimes also by raising the structure to provide more clearance between the channel bed and the low chord (i.e, bottom) of the bridge. Because of the proximity of Curtiss Avenue on the east and homes to the west, significantly raising this bridge is not possible. However, the modeling did assume that the low chord of the structure was raised by six inches.

Several iterations were modeled by increasing the width of the crossing and the channel downstream. In evaluating an appropriately sized structure, one foot of freeboard was assumed necessary as this is consistent with standard engineering practice. In all alternatives evaluated, the channel downstream of the Frederick Street crossing was lowered by up to one foot. Increasing the bridge and downstream channel width by 40 feet allows the 25-year storm to pass but does not provide one foot of freeboard at the low point of Frederick Street. As a result, if this is to be implemented we would recommend some modification of Frederick Street to provide one foot of freeboard. A further increase in the channel and bridge width did not result in reducing flooding during a 50-year storm event. Therefore, proposing a structure capable of passing 50-year and 100-year storm events is not possible. Results of this evaluation are presented in Table 4-16.

TABLE 4-16
Comparison of Existing and Proposed Water Surface Elevations at 25-Year Flows – Replacement of Frederick Street Bridge

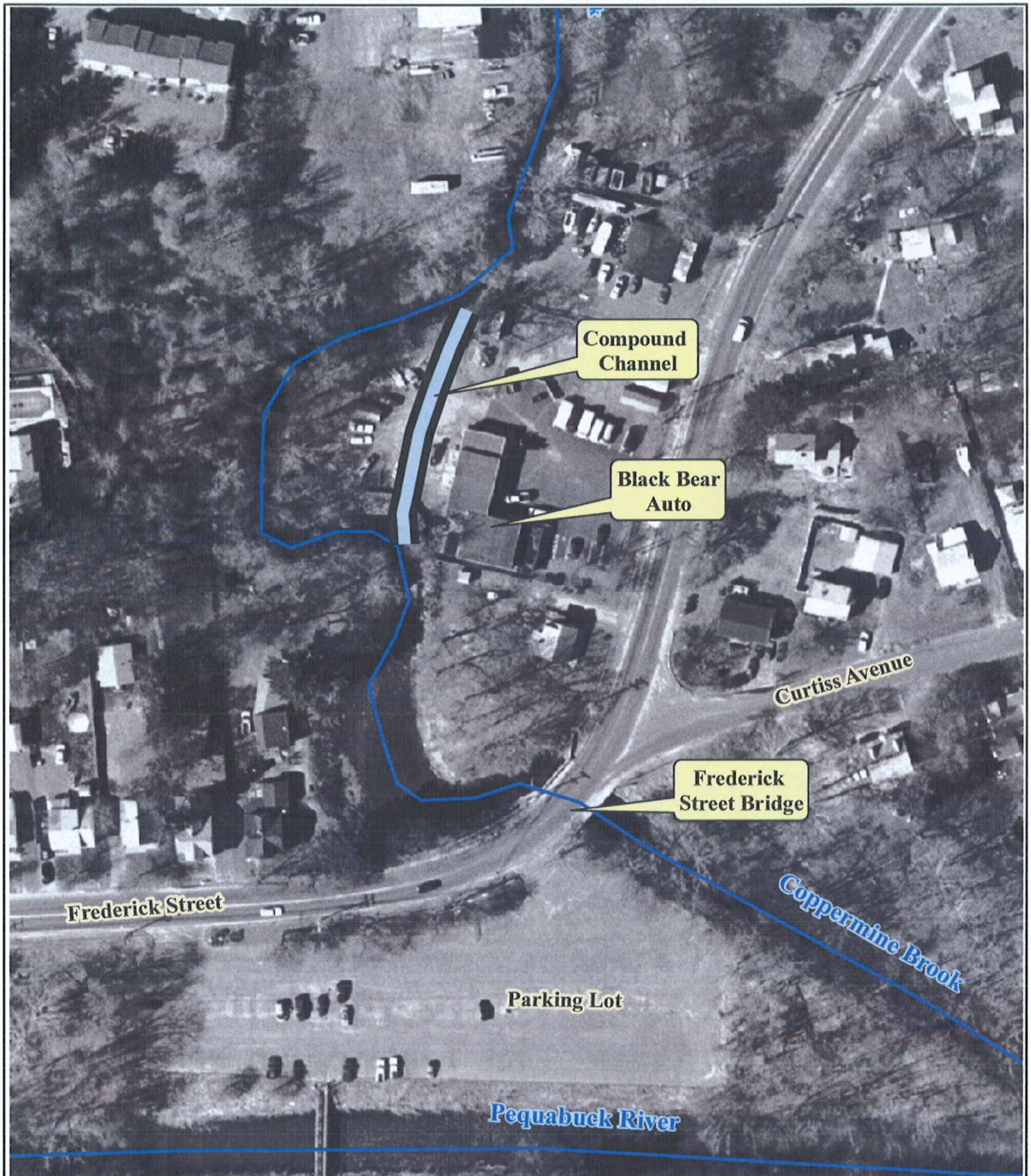
Cross Section Description	Existing WSEL	Proposed WSEL	Difference (feet)
Confluence of Pequabuck River	214.50	214.50	0.00
175 ft. downstream of Frederick Street	214.75	214.55	-0.20
120 ft. downstream of Frederick Street	214.67	214.58	-0.09
50 ft. downstream of Frederick Street	215.52	215.01	-0.51
20 ft. downstream of Frederick Street	215.70	215.04	-0.66
Downstream face of Frederick Street	215.29	215.02	-0.27
Upstream face of Frederick Street	217.24	215.24	-2.00
100 ft. upstream of Frederick Street	217.40	215.28	-2.12
At the bend upstream of Frederick Street	217.50	215.59	-1.91
Downstream of Black Bear Auto	217.39	215.66	-1.73
Adjacent to Black Bear Auto	217.96	218.84	0.88
Upstream of Black Bear Auto	217.29	217.29	0.00
FEMA *cross section* B	221.04	221.04	0.00
40 ft. downstream of West Washington Street	221.46	221.46	0.00
Downstream face of West Washington Street	223.74	223.74	0.00
Upstream face of West Washington Street	224.88	224.88	0.00

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

Replacement of the bridge with a larger structure and widening the channel downstream would reduce flood elevations upstream of the crossing. Although a structure cannot be provided that would pass the 100-year event, increasing the bridge size would reduce the frequency with which Frederick Street overtops and would potentially benefit the adjacent property owners.

Construction of Compound Channel Behind Black Bear Auto

Information from residents indicates that the upland area behind Black Bear Auto floods on some regular basis. During such events, the flow of Coppermine Brook overtops the channel bank at the north end of the property and flows in a straight path to the south end of the property, circumventing the meander in the channel at this location. In this alternative, creation of a compound channel through this area was evaluated, and Figure 4-6 depicts this concept. This would require excavation of material from the rear of this



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MMI#: 2235-19-06 MXD: H:Fig 4-5.mxd SOURCE: http://magic.lib.uconn.edu/	N CONCEPT SKETCH - COMPOUND CHANNEL	DATE: 07/07/08 SCALE: 1" = 100'
		SHEET: Figure 4-6

building and would provide for floodplain storage. Table 4-17 compares existing and proposed water surface elevations during the 10-year flood.

TABLE 4-17
Comparison of Existing and Proposed Water Surface Elevations at 10-Year Flows – Compound Channel at Black Bear Auto

Cross Section Description	Existing WSEL	Proposed WSEL	Difference (feet)
Upstream face of Frederick Street	215.22	215.22	0.00
100 ft. upstream of Frederick Street	215.77	215.82	0.05
At the bend upstream of Frederick Street	215.89	215.90	0.01
Downstream of Black Bear Auto	215.25	215.52	0.27
Adjacent to Black Bear Auto	216.98	216.61	-0.37
Upstream of Black Bear Auto	216.26	216.39	0.13
FEMA cross section B	219.15	218.59	-0.56
40 ft. downstream of West Washington Street	219.12	219.00	-0.12
Downstream face of West Washington Street	221.66	221.70	0.04

Notes: WSEL = water surface elevation.
All elevations are presented in feet based on the National Geodetic Vertical Datum of 1929.

Model results indicate that the water surface elevation through this reach would vary, with little benefit realized at Frederick Street. However, providing the compound channel at this location would restore some of the floodplain of Coppermine Brook. As indicated in Section 2.3, this floodplain has been cut off from the channel over time, which contributes to flooding in this area.

Construction of High Overflow Culvert at Theis Steel and Black Bear Auto

At the request of city staff, we evaluated construction of two high overflow culverts: one through the Theis Steel parking lot as previously discussed and a second one from Black Bear Auto down Frederick Street and discharging into the channel downstream of the Frederick Street bridge. Tables 4-18 and 4-19 present the results of this analysis and compare them to the results of the single overflow culvert previously discussed.

TABLE 4-18
Comparison of Proposed Water Surface Elevations at 25-Year Flows – Twin 6' by 8' Culverts at Black Bear Auto and Theis Steel (Four Total)

Cross Section Description	WSEL for Overflow Proposed at Theis Steel	WSEL for Overflow Proposed at Theis and Black Bear	Difference
Confluence of Pequabuck River	214.50	214.50	0.00
175 ft. downstream of Frederick Street	215.03	215.04	+0.01
120 ft. downstream of Frederick Street	215.00	215.01	+0.01
50 ft. downstream of Frederick Street	215.16	215.16	0.00
20 ft. downstream of Frederick Street	215.22	215.46	+0.24
Downstream face of Frederick Street	215.10	215.41	+0.31
Upstream face of Frederick Street	215.86	215.91	+0.05
100 ft. upstream of Frederick Street	216.22	216.12	-0.10
At the bend upstream of Frederick Street	216.34	216.14	-0.20
Downstream of Black Bear Auto	215.66	215.28	-0.38
Adjacent to Black Bear Auto	218.84	217.29	-1.55
Upstream of Black Bear Auto	217.29	217.29	0.00
FEMA cross section B	221.04	221.04	0.00
40 ft. downstream of West Washington Street	221.46	221.46	0.00
Downstream face of West Washington Street	223.74	223.74	0.00
Upstream face of West Washington Street	224.88	224.88	0.00

TABLE 4-19
Comparison of Proposed Water Surface Elevations at 25-Year Flows – Twin 6' by 24' Culverts at Black Bear Auto and Theis Steel (Four Total)

Cross Section Description	WSEL for Overflow Proposed at Theis Steel	WSEL for Overflow Proposed at Theis and Black Bear	Difference
Confluence of Pequabuck River	214.50	214.50	0.00
175 ft. downstream of Frederick Street	215.11	215.01	-0.10
120 ft. downstream of Frederick Street	215.09	214.97	-0.12
50 ft. downstream of Frederick Street	215.16	215.16	0.00
20 ft. downstream of Frederick Street	215.19	215.79	+0.60
Downstream face of Frederick Street	215.14	215.78	+0.64
Upstream face of Frederick Street	215.55	215.92	+0.37
100 ft. upstream of Frederick Street	215.54	215.96	+0.42
At the bend upstream of Frederick Street	215.71	215.96	+0.25
Downstream of Black Bear Auto	215.66	215.74	+0.08
Adjacent to Black Bear Auto	218.84	216.31	-2.53
Upstream of Black Bear Auto	217.29	217.29	0.00
FEMA cross section B	221.04	221.04	0.00
40 ft. downstream of West Washington Street	221.46	221.46	0.00
Downstream face of West Washington Street	223.74	223.74	0.00
Upstream face of West Washington Street	224.88	224.88	0.00

Results of this analysis indicate that there is limited benefit to providing overflow culverts in two locations. With 24-foot wide culverts, there is a significant drop in water surface elevation at Black Bear Auto, but this benefit is localized and does not influence the water surface elevations near Frederick Street in any meaningful way. Therefore, this alternative is not recommended for further evaluation.

5.0 WATERSHED MANAGEMENT AND LAND USE REGULATIONS

As part of this study, the city requested that the current land use regulations of both Bristol and Burlington be evaluated with respect to stormwater management. This section presents background information on stormwater management mechanisms that can reduce the impact of development on increasing stream flows as well as some analysis of the existing land use regulations in each community.

5.1 Principles of Watershed Management

Many factors require that riverine and watershed management efforts extend beyond the banks that contain flowing water. In the Coppermine Brook watershed, management issues result from watershed land use patterns as well as floodplain encroachments. It is essential to recognize that besides conveying storm runoff, streams serve many ecological, economic, and social functions, and the planning and design of management systems must consider these.

The increased industrialization and urban growth after the Civil War was followed by the rapid growth of 1950s suburbs dependent on automobile transportation. Urban and suburban development both increase the area of impervious surfaces and use artificial drainage systems to collect runoff. The prevailing stormwater management philosophy for 100 years or more was to convey the runoff to rivers as rapidly as possible. This reduces infiltration and evapotranspiration, increasing the volume of runoff and raising peak flow rates in rivers. In recent years, that philosophy has changed, with more attention being given to detention and infiltration of stormwater.

In addition to raising peak flows, urbanization reduces riverine base flows that are necessary to sustain aquatic life, recreation, and water supply in dry weather. The percentage of a watershed that is covered with impervious surfaces is one of the key parameters affecting urban runoff. Increased runoff into a river's channel and floodplain affects the river's hydraulics, altering its flow depth, velocities, flood frequency, scour, and sediments. Channel and floodplain encroachments, such as fill material, buildings, bridges, and culverts, can also reduce flow capacity of a river and increase peak flow rates and velocities.

Aside from impacts to watershed hydrology and riverine hydraulics, land use can have a marked impact on stream water quality, temperatures, and sedimentation and erosion. With increased impervious surfaces come higher peak rates of stormwater runoff, greater transport of contaminants, higher stream velocities, and often degraded water quality due to increased temperatures and an influx of pollutants.

Based on the Impervious Cover Model (ICM) developed by Schueler, water quality degradation begins to occur when 10 percent of the watershed is covered in impervious surfaces. Degradation at this stage consists of a loss of the most sensitive aquatic organisms. Streams with 10 to 25 percent impervious cover usually are impacted with erosion, channel deterioration, unstable banks, reduced habitat, reduced biodiversity, and declining water quality. Watersheds with over 25 percent impervious cover tend to be flood prone and highly unstable, with poor water quality and limited aquatic life.

5.2 Low Impact Development

Since the mid 1990s, there has been a movement to minimize the hydrologic impact that development activities have on riverine systems. These so-called "low impact development" techniques can also serve to improve water quality. Most development of the Coppermine Brook watershed occurred long before low impact development was a consideration for most communities. In fact, some of the development, particularly in the Bristol portion of the watershed, likely occurred before stormwater detention was required for controlling peak flow. As a result, watershed development followed the standard paradigm of moving water away from homes and streets as quickly as possible.

The general goal of low impact development is to combine "hydrologically functional" site design with stormwater quality practices to minimize development-related impacts to the extent practical. This does not mean preventing full development of property as may be allowed under local land use regulations. However, it does mean incorporating, to the extent practical, the following goals:

- protect existing vegetation;
- minimize changes in surface water drainage patterns;
- avoid excessive site grading;
- reduce the area of impervious and managed surface coverage;
- disconnect impervious surfaces;
- promote temporary storage of stormwater runoff;
- promote infiltration of stormwater runoff; and
- reduce or mitigate increases in the volume of stormwater runoff as well as changes in magnitude, frequency, and duration of stormwater discharges to receiving waters.

The use of these planning and design tools can oftentimes reduce or even eliminate the requirement for more costly and sometimes obtrusive storage, infiltration, or end-of-pipe structural practices for the management of stormwater runoff. It can also result in development proposals that better fit the existing characteristics of a site, are aesthetically pleasing, and protect the environment.

The State of Connecticut, in publishing the 2004 Connecticut Stormwater Quality Manual (SQM), provided guidance on low impact development site design and measures to protect water quality from potential development impacts. Reference to this manual should be incorporated into the town's land use regulations.

5.2.1 Areas Suitable for Low Impact Development

Water quality and decreases in stormwater runoff can be addressed through Low Impact Development (LID) techniques during development and redevelopment. Many of the areas within the Coppermine Brook watershed, particularly along the main stem of the channel, may be suitable for this type of approach, particularly since the underlying soils are classified as sand and gravel materials, which would have high infiltration rates.

5.2.2 Site Planning for LID

LID site planning techniques attempt to combine hydrologically functional site design with pollution prevention measures while allowing full development of the property. LID site planning begins by understanding the essential hydrologic functions of the site, including the streams, wetlands, buffer areas, floodplains, steep slopes, high permeability soils, and conservation zones. The remaining site area is the "development zone," the area where development activities will have the least impact on hydrologic function.

Successful LID requires the micromanagement of site watersheds and hydrology. This means addressing stormwater control on a lot-by-lot basis. "On-lot" stormwater management may include "microstorage," functional landscaping, open swale drainage systems, reduction in impervious cover, increased runoff travel time and depression storage (Prince George's County Maryland, 1999).

Since the watershed areas and stormwater runoff volumes being managed using LID are much smaller, the range of available management techniques increases. For instance, use of a rain garden would not be possible to control runoff from an 11-lot subdivision and associated roadway. However, placement of rain gardens on a lot-by-lot basis may be feasible.

Often the first step in LID planning is to assess local land use regulations to determine the site development requirements. In many Connecticut communities, zoning and subdivision regulations effectively preclude the use of LID techniques because hard and fast design requirements are mandated.

The following site design elements incorporate LID:

1. Reduce paved areas to the extent possible. This may include reducing the width of paved roadways and cul-de-sac diameters, eliminating on-street parking, promoting use of common driveways, or using narrower driveway widths (perhaps nine or 10 feet).

2. Use permeable pavement materials, such as grass pavers, whenever possible.

3. Avoid compaction of high permeability soils.

4. Minimize the area dedicated for construction easements and stockpile areas.

5. To the extent possible, plan site activities to limit the removal of trees and vegetation.

6. Disconnect impervious areas. Do not connect roof drains and footing drains into a piped drainage system (consider drywells or other infiltration devices). Provide curbless roads to allow sheet flow.

7. Maintain existing topography to the extent possible. The intent is to maintain runoff travel distances, slopes, roughness, and channel shapes whenever possible.

8. Maximize the use of open drainage systems, such as grass swales.

9. Alter front yard setbacks to move houses forward on a lot to reduce driveway lengths.

In addition to incorporating these elements, storage and treatment technologies can be considered to address any increases in peak flow or water quality concerns that remain.

The selection of specific Best Management Practices (BMPs) varies from site to site. Some applications, such as infiltration systems, may not be appropriate for all land uses or all sites. For instance, the use of infiltration basins or trenches may not be appropriate in parking areas in the Coppermine Brook watershed that are in close proximity to the City of New Britain's Maltby Wells.

5.2.3 LID Techniques for Development

In assessing site designs that incorporate LID techniques, hydrologic analysis is a key consideration. Land use regulations often stipulate the design storm to be analyzed. Although the city's Subdivision Regulations do specify design storms for culvert and bridge designs, no design storm is specified for detention and on-site drainage features. Instead, Section 5.08 of those regulations requires that plans make adequate provision of stormwater runoff control, stipulating that the City Engineer must approve the design. This requirement is flexible and allows development of sites to occur in a manner appropriate for the area instead of forcing a "one size fits all" approach to stormwater management. This is important since detention can sometimes lead to increased flooding depending on the watershed location where it is provided. Despite this fact, many communities require that detention be provided on all sites.

For sites using LID, a design storm must be selected to evaluate hydrologic conditions at the site. Typically, analysis of the first inch of runoff is used for stormwater quality evaluations, and the two-year storm is analyzed to evaluate erosion and sediment related impacts to receiving streams. Per the city's Subdivision Regulations, a design storm is specified for bridges and culverts, and the required storm depends on the upstream watershed area.

Once a site plan has been developed that incorporates as many LID principles as possible, management practices such as the following can be used to further reduce hydrologic impact and provide water quality control:

- Bioretention and Rain Gardens
- Drywells
- Filter Strips
- Vegetated Buffers
- Level Spreaders
- Grassed Swales
- Rain Barrels
- Infiltration Trenches
- Rain Gardens

For Coppermine Brook, the appropriateness and viability of each application will need to be decided on a case-by-case basis.

5.3 Evaluation of Land Use Regulations

LID practices, if designed and constructed properly, allow development to mimic natural hydrologic conditions, reducing potential downstream impacts. If natural hydrologic conditions are mimicked, then adverse impacts to downstream hydraulic structures should not occur.

New development and redevelopment projects in the watershed would need to comply with the current land use regulations of the watershed communities of Bristol and Burlington. One general observation is that the regulations of both communities should be updated to reference the 2004 Connecticut Stormwater Quality Manual. While this manual provides design guidance for water quality measures, it also describes the LID design process and measures that can be taken to reduce the overall impact of development activities. All land use applications should incorporate the information in this manual.

- Section 5.03 of Bristol's Subdivision Regulations requires a minimum 50-foot right-of-way for new streets with the possibility of requesting a larger right-of-way for minor arterials or collectors. Larger right-of-way widths allow additional space for surface stormwater management measures such as grass swales. A 60-foot wide right-of-way should be considered for all proposed streets.

- Section 5.04 of Bristol's Subdivision Regulations requires sidewalks to be installed on both sides of all subdivision streets regardless of the size of the development unless the Planning Commission defers the need for sidewalks. While sidewalks are a vital part of communities and provide for public safety, they do represent impervious surfaces that increase the rate of postdevelopment runoff. To that end, consideration should be given to modifying this standard to allow for no sidewalks on permanent dead ends serving less than 10 homes and for sidewalks on only one side of other streets. The exception to this might be in the urban center of the community where sidewalks on both sides of the street may be warranted. None of the existing deferment criteria in Section 5.04(2) involve the size of the population served by the roadway.

- Section 5.08 of the city's Subdivision Regulations sets requirements for stormwater management facilities. Generally, this regulation provides a great deal of flexibility to the designer by not setting strict standards that must be followed for every development. Instead, design of drainage systems is deferred to the design engineer with approval from the City Engineer. This flexibility is a good thing and should be maintained. If the city were to incorporate reference to the 2004 Stormwater Quality Manual into the Subdivision Regulations, this section would be the logical location for such reference.

- Both the Zoning and Subdivision Regulations of the city include provisions for Open Space Developments (OSD). The concept of OSD is to allow a reduction in requirements for front yard and side yard setbacks and the like in exchange for more open space dedicated to the city or other management entity. This can be an effective way of minimizing sprawl. In some communities, OSD includes a provision for a "density bonus," which allows the developer to gain one or more lots or units than would be allowed under a standard subdivision layout. The inclusion of such a provision in Bristol's regulations may entice more developers to use this regulation.

- The land use regulations do not include any reference to the Federal Emergency Management Agency's floodplain regulations. Typically, a floodplain overlay district would be implemented for all areas of the community designated as a floodplain by FEMA. We strongly encourage the city to adopt a floodplain overlay to control and manage development within flood prone areas. Appendix M includes recent sample floodplain regulations developed by the State of Connecticut for municipal use.

- Section VIII.B of the city's Zoning Regulations specifies parking standards for specific land uses within the community. This section of the regulations then goes on to allow for shared parking among adjacent land uses and also for deferral of construction of some portion of the parking. While this does not eliminate parking entirely, it does delay the construction of potentially unnecessary impervious surfaces. It would be helpful if the regulations also allowed for the use of permeable pavement materials in select locations.

- The town of Burlington's Zoning Regulations incorporate similar standards for deferral of parking and shared parking. Burlington also allows for alternative surface treatments of parking areas if approved by the commission.

- Neither the City of Bristol nor the Town of Burlington's land use regulations reference the 2004 Stormwater Quality Manual at this time. We recommend that such reference be incorporated.

5.4 LID in the Coppermine Brook Watershed

There are a number of BMPs that are suitable for use in the Coppermine Brook watershed. Some of these are BMPs the town would implement, while others would be implemented during the development or redevelopment process.

LID techniques should be considered for any new development or redevelopment project undertaken in Bristol. Examples of viable BMPs include:

- Allowing sheet flow from parking lots (no curbs);
- Providing infiltration trenches or drywells for rooftop runoff;
- Using grass swales to transfer water to discharge locations; and
- Using created wetlands for stormwater treatment.

The type of method used will vary from site to site depending on the proposed land use as well as site geology and topography. Other factors, such as depth to water and depth to bedrock, will also need to be a consideration. Table 5-1 presents a summary of stormwater management techniques and consideration for their use.

Generally, those portions of the watershed are underlain by sand, and sand and gravel (see Figure 2-3). Infiltration practices may be appropriate in some Type C soils, depending on the site geology and layout. Regardless of mapped soil type, it is imperative that site specific permeability testing be completed before infiltration practices are proposed.

Other LID techniques such as promoting sheet flow and using grass swales and created wetlands for water quality renovation can be used anywhere in the watershed without regard for soil type.

TABLE 5-1
Considerations of Use of LID BMPs

BMP Type	***Watershed Size***	***Space Requirements***	***Site Considerations***	***Maintenance***
Rain Barrels	Limited to roof area. Provide multiple barrels to accommodate larger roof areas.	Limited	None	Low
Infiltration Basins or Trenches	Trenches: 5 acres maximum; 2 acres recommended. Basins: 25 acres maximum; 10 acres recommended.	Varies with watershed size. Minimum 20 square feet.	Do not use at properties with high potential for sediment load. Keep minimum of 50' from slopes 15% or greater; bottom of unit >3' to water; 75' min. from wells and septic.	Moderate to high
Dry Wells	< one acre	Varies with watershed size. Minimum 20 square feet.	Not for use where rooftop may contribute pollutants. Bottom of unit 3' above water, 4' above bedrock; 75' min .from wells and septic.	Low
Pervious Pavement	Traffic volume <500 ADT	Not applicable	Min. infiltration of underlying soils 0.3 in/hr, but less than 5.0 in/hr; no use in aquifer recharge areas except in approved "clean" applications; no use on slopes greater than 15%; depth to water – 3' min. depth to bedrock – 4' min. 75' min. from wells.	Moderate
Green Roof Storage	Generally limited to roof area	Varies with size of roof	Depending on materials used, structural considerations may be needed.	Low

TABLE 5-1
Considerations of Use of LID BMPs

BMP Type	***Watershed Size***	***Space Requirements***	***Site Considerations***	***Maintenance***
Bioretention/Rain Gardens	5-10 acres; rooftop area for rain gardens	200 sq. ft. min; 25 sq. ft. rain garden	Slopes 6% or less; 3' from bottom of structure to water	Low
Grass Swales	As space permits for swale construction	2' min. bottom width	Avoid steep slopes to prevent erosion	Low
Oil/Water or Hydrodynamic Separators	<1 acre impervious cover	None. Below grade structure	None	Low
Created Wetlands	25 acre max.	Proportional to watershed size	Must intersect ground water if unlined; not appropriate for land uses generating large amounts of contamination; must have base flow into system; steep slopes not appropriate	Moderate to High
Detention Basins	1 acre min.	Proportional to watershed size	Must intersect ground water if unlined and wet basin; not appropriate for land uses generating large amounts of contamination; must have base flow into system; steep slopes not appropriate	Moderate

6.0 CONCLUSIONS AND RECOMMENDATIONS

The study completed by MMI has included a comprehensive evaluation of watershed and stream corridor conditions along Coppermine Brook. The result of the analyses is recommendations that may reduce the severity of flooding in some locations; however, even if these improvements are made the fact remains that a number of issues contribute to the flooding problems that residents have been experiencing. These have been described in detail through this report but are summarized here:

1. Rainfall patterns in the northeast are changing, resulting in increasing streamflows. There has been widespread flooding in central Connecticut in recent years, including 1999, 2005, and 2006. These events were not unique to Coppermine Brook. Federal records also confirm a long-term increase in stream flow throughout Connecticut.

2. Historic development has resulted in floodplain encroachment that cannot be easily mitigated. Much of this development predates FEMA's Flood Insurance Program and certainly predates the increasing rainfall patterns and stream flows discussed above.

3. The FEMA study is outdated and, based on our analysis, some properties should be identified within the floodplain that are currently not. These properties will not be eligible for federal flood insurance unless FEMA approves a floodplain modification.

4. Future land use buildout could theoretically increase peak flows by 10 to 20 percent if unmitigated.

5. New Britain's Whigville Reservoir does not have any facilities that could be operated so as to suddenly cause a significant increase in stream flow rates. The

source of the flood flow that was reported by residents could not be identified with certainty, but it is possible that the failure of weir boards in one or more of the three dams located upstream of Jerome Avenue contributed to this.

6. Some bridges along the channel corridor are undersized resulting in overtopping during some storm events. In some instances, this is due to floodplain encroachment as much as it is undersized structures. For example, even if the Farmington Avenue bridge were removed, the roadway would still be flooded. The only solution evaluated that could correct this problem is increasing the size of this structure slightly in conjunction with widening the channel upstream. Such widening would impact the existing land uses in the floodplain such as Staples.

It is absolutely critical that residents and town officials alike recognize that it will not be possible to stop all flooding of structures along Coppermine Brook. The recommendations herein will, however, decrease the severity and frequency of flooding.

Based on the work completed, we recommend the following:

1. **Pursue the construction of watershed storage areas**. The hydrologic analysis presented in Section 3 indicated that upstream storage could be very effective at reducing downstream flow rates. We recommend that the area identified as Coppermine 1 be pursued first. This is because the area appears to generally be upland, and the state and federal regulators frown on the use of existing wetlands for flood storage. In other words, we think this will be the easiest area to obtain permits for construction. The following tasks will need to be completed to pursue this recommendation:

a. Delineate the limit of state and federally regulated inland wetlands in the potential storage area.
b. Obtain detailed topographic mapping (40-scale, two-foot contours are recommended) of the area.
c. Identify property owners in the area and secure easements as necessary. This will require A-2 survey of selected properties. The exact number will be determined during the design process.
d. Design an outlet structure or weir as necessary to manage discharge from the impoundment.
e. Develop a grading plan for the excavation of the area for storage. The grading plan will need to include provisions for postconstruction planting and stabilization of the area.
f. Prepare and submit local, state, and federal permit applications as necessary. The exact scope of permitting is difficult to determine without detailed base mapping and wetland delineation. The hope would be to keep impacts below 5,000 square feet to avoid state and federal wetlands permitting.
g. Develop final design plans and construction documents.

Design and permitting of this basin is expected to be on the order of $50,000 to $75,000 depending on the level of permitting required.

2. **Manage flooding at Richards Court through dike improvements, sealing the existing storm drain through the dike, and channel improvements downstream of the Stevens Street bridge.** The problems at Richards Court are caused by a number of issues. Regardless of the improvements that are made as a result of this study, the fact remains that this neighborhood sits atop what was once mapped as floodplain soils. The issues here are compounded by the fact that much of the improvements suggested are on private property. The exception is the downstream channel improvements, which would occur on property we

believe to be owned by the City of New Britain. It is not clear what obligation the city has to repair the former dike, which is located on private property. Regardless, if the city were to pursue this work, the following scope of services is suggested:

a. Delineate the limit of state and federally regulated inland wetlands from Jerome Avenue to approximately 300 feet downstream of Stevens Street.
b. Obtain detailed topographic mapping (40-scale, two-foot contours are recommended) of the area.
c. Identify property owners in the area and secure easements as necessary. This will require A-2 survey of selected properties. The exact number will be determined during the design process.
d. Develop and execute a boring program along the existing dike. One issue of concern here is the lack of understanding of how the current dike was constructed. If the city were to pursue improvements to the dike, it would be important to understand the existing construction and to correct deficiencies if necessary. The overarching concern is that if the city were to execute improvements it may become liable if the structure were to fail in the future.
e. Evaluate the existing drainage system in Richards Court and identify potential alternatives for relocating, sealing, or abandoning the existing drainage pipe behind #72. This may require survey of nearby drainage structures and may include the evaluation of a small stormwater pump station. The pump station would be the alternative of last resort as they can be expensive to install and operate and often are forgotten during times when flooding does not occur, only to be found in a state of disrepair when they are needed.
f. Develop a grading and restoration plan for channel improvements downstream of Stevens Street. This will need to include measures, preferably nonstructural (i.e., vegetation is preferred), to stabilize the new bank. Such measures should be based on the predicted velocities developed from hydraulic modeling.

g. Evaluate configurations for and design improvements to the existing dike on the left bank of the channel between Jerome Avenue and Stevens Street. This may include replacing the dike in kind, raising it, and/or extending it upstream toward Jerome Avenue some distance.

h. Evaluate the potential impact of these improvements on downstream flow rates and flooding.

i. Prepare and submit local, state, and federal permit applications as necessary. The exact scope of permitting is difficult to determine without detailed base mapping and wetland delineation, but it is not clear that state and federal permitting can be avoided for these improvements.

Design and permitting of this work is expected to be on the order of $70,000 to $100,000 depending on the level of permitting required and the final solution selected for the drainage pipe at 72 Richards Court.

3. **Make improvements near Farmington Avenue.** Flooding at and upstream of Farmington Avenue is occurring because of floodplain construction and development and high tailwater along the low gradient channel. Bridge improvements alone cannot solve flood hazards, but the combination of removing the private driveway bridge supplemented by channel improvements may provide some benefit. As with the improvements at Richards Court, both of these recommendations involve work on private property. Modification of the Farmington Avenue bridge is not suggested at this time as this is clearly not the responsibility of the city. That being said, once the upstream channel improvements suggested herein are completed, the city may choose to discuss Farmington Avenue with the DOT. The following scope of work is suggested:

    a. Delineate the limit of state and federally regulated inland wetlands from Farmington Avenue to approximately 100 feet upstream of the private driveway bridge.

b. Obtain detailed topographic mapping (40-scale, two-foot contours are recommended) of the area.

c. Identify property owners in the area and secure easements as necessary. This will require A-2 survey of selected properties. The exact number will be determined during the design process.

d. Develop a grading and restoration plan for channel improvements downstream from approximately 100 feet upstream of the bridge to Farmington Avenue. This will need to include measures, preferably nonstructural (i.e., vegetation is preferred), to stabilize the new bank. Such measures should be based on the predicted velocities developed from hydraulic modeling. Plans should also include provisions for water handling and sediment and erosion control.

e. Evaluate the potential impact of these improvements on downstream flow rates and flooding.

f. Prepare and submit local, state, and federal permit applications as necessary. The exact scope of permitting is difficult to determine without detailed base mapping and wetland delineation, but it is not clear that state and federal permitting can be avoided for these improvements.

g. Develop final design plans and construction documents.

Design and permitting of this work is expected to be on the order of $35,000 to $45,000 depending on the level of permitting required.

4. **Make improvements at Frederick Street.** The Frederick Street area is subject to flooding and erosion due to riverine sources, bridge construction, and Pequabuck River backwater. Bridge and channel improvements could reduce the frequency of flooding, but long-term hazards remain. At this point, given the age of this structure the most prudent alternative would be replacement of this bridge. It needs to be clear that this will not fully alleviate flooding at Frederick Street as

the nearby residences are within the floodplain. The following task items are suggested for this work:

a. Delineate the limit of state and federally regulated inland wetlands from the Pequabuck River to approximately 200 feet upstream of Frederick Street.
b. Obtain detailed topographic mapping (40-scale, two-foot contours are recommended) of the area.
c. Identify property owners in the area and secure easements as necessary. This will require A-2 survey of selected properties. The exact number will be determined during the design process.
d. Develop and execute a boring plan to evaluate materials for bridge footings.
e. Develop plans for the replacement of the structure as well as channel modifications upstream and downstream. This will include structural, grading, restoration, water handling, and sediment and erosion control plans.
f. Evaluate the potential impact of these improvements on downstream flow rates and flooding.
g. Prepare and submit local, state, and federal permit applications as necessary. It is our current opinion that this project will be regulated by the CTDEP through the Diversion Act in addition to the 401 Water Quality Certificate program.
h. Develop final design plans and construction documents.

Design and permitting of this work is expected to be on the order of $80,000 to $90,000 depending on the level of permitting required.

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