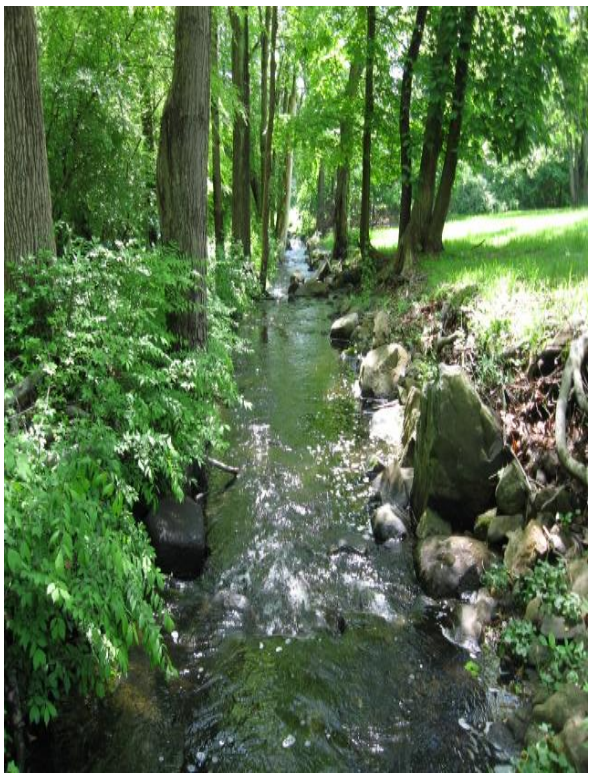
GOODWIVES RIVER
FEMA LOMR APPLICATION



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1.0 INTRODUCTION

The Town of Darien selected Milone & MacBroom, Inc. (MMI) in September 2008 to evaluate flooding conditions along several watercourses within the community. Residents along the Goodwives River have had increased incidence of flooding in recent years. There is a detailed Federal Emergency Management Agency (FEMA) study of the Goodwives River, defining a floodway and 1% and 0.2% floodplains effective June 18, 2010. The study completed by MMI in 2009 re-evaluated flood risks with a revised detailed study of current hydrology and hydraulics.

This report contains information to support a submission to FEMA for a Letter of Map Revision (LOMR) application for acceptance of the existing conditions modeling with current data as the new Effective Model for the length of the current detailed study of the Goodwives River in Darien, Connecticut.

An updated hydrology analysis has been performed to determine current peak flows in the watershed using the rainfall-runoff model Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS). Flows are higher than Effective FEMA flows. Increases in runoff may be due to increases in impervious area and development in the watershed.

The hydraulic modeling originally completed by FEMA in HEC-2 has been converted to use the model Hydrologic Engineering Center – River Analysis System (HEC-RAS) and updated to current conditions. The modeling was completed using the updated flow rates determined from the HEC-HMS model. Recent topography, structure plans, survey information, and field observations were used to update the Effective Model.

A new floodway, 1% annual chance floodplains, and 0.2% annual chance floodplain have been delineated. The revised 1% annual chance floodplain extends beyond the FEMA floodplain in multiple areas due to larger discharges modeled and is more detailed as it is

based on more accurate topography. The revised floodway analysis results in a wider floodway with water surface elevations that are higher than the Effective Model. Four structures are located within the revised floodway.

2.0 PROJECT BACKGROUND

2.1 Project Area

This study includes the mainstem of the Goodwives River in the town of Darien. Hydrologic analyses consider the full watershed area including the northern portion of the watershed in New Canaan while hydraulic modeling of the channel extends from Gorham's Pond upstream to just upstream of Buttonwood Lane. Gorham's Pond at Rings End Road contains flow contributions from Stony Brook and from Goodwives River. The drainage area at this point is 6.2 mi^2. Goodwives River Road crosses Goodwives River upstream of its confluence with Stony Brook. The Goodwives River drainage area at this point is 2.0 mi^2. Figure 2-1 depicts the Goodwives River watershed.

In the current Darien FEMA Flood Insurance Study (FIS), FEMA defines a floodplain along the mainstem Goodwives River that extends from Gorham's Pond to upstream of Buttonwood Lane just south of the Darien Country Club. Upstream of this point, FEMA has an approximate floodplain boundary because the hydraulic analysis did not continue upstream of Buttonwood Lane. A floodway was defined south of Buttonwood Lane downstream to Gorham's Pond. The work presented here has extended the detailed hydraulic modeling approximately 175 feet upstream to include Buttonwood Lane bridge and a small segment of stream upstream.

References in this report to left and right bank are made facing downstream.

2.2 Project Basemapping

This project used recent basemapping of the Goodwives River watershed obtained from the Town of Darien. The data provided was titled "Topographic Map of Darien, CT" developed by James W. Sewall Company of Old Town, Maine under contract to the town. The data was compiled to meet National Map Accuracy Standards for 1"=40' maps on July 20, 2008 by photogrammetric methods from color aerial photographs dated April 3, 2008 at a scale of 1"=300'. The information provided included one-foot contour topography, roadways, and buildings. High resolution aerial photography was taken at the time of topographic mapping and used in conjunction with field observations to identify land uses and delineate the river channel and other watershed features. Significant supplemental ground survey was completed by MMI in 2009 including wet river cross sections and survey of many bridges, culverts, and dams. The horizontal datum of the basemapping is Connecticut State Plane NAD83 feet. The vertical datum of the basemapping is NAVD 1988.

Part of the Goodwives River watershed lies within the town of New Canaan. Its topography was needed for the hydrology analysis. Topographic contours at a five-foot contour interval were generated by MMI based on 2000 Light Detection and Ranging (LiDAR) mapping compiled by the University of Connecticut's Center for Land Use Education and Research (CLEAR).

2.3 Project Data Collection

The Goodwives River mainstem was evaluated during a river assessment walk performed by MMI staff on May 14 and 15, 2009. The channel evaluation started downstream at Rings End Road near Gorham's Pond and continued upstream to Salisbury Road, upstream of the Country Club of Darien where the channel dissipated. During this visit, the overall condition of the brook was evaluated, and observations were made with regard

to bank height, condition, vegetation, and channel bed material. Bridges, culverts, and dams were evaluated for damage and compatibility with river process. Structures were measured for comparison to the FEMA Effective hydraulic model to guide necessary survey updates. During our investigation, storm drainage discharge locations, bank armoring locations, footbridges, and private dams were identified. Overall watershed drainage and land use were also observed.

3.0 EXISTING HYDROLOGIC CONDITIONS

3.1 Flood Flow Estimation

3.1.1 FEMA FIS Discharges

The 2010 FIS prepared by FEMA for the Town of Darien provides discharge values in cubic feet per second (cfs) at two locations within the Goodwives River watershed. The flow rates for the 10-year (10%), 50-year (2%), 100-year (1%), and 500-year (0.2%) floods were estimated in the FIS based on regional regression equations for stream flow in Connecticut that were current at the time of the study. Table 3-1 presents the flow rates published in the FIS.

TABLE 3-1
FEMA FIS Peak Flows

Watershed Location	10%	2%	1%	0.2%
Upstream of Boston Post Road	210	300	360	565
Watershed Outlet (upstream of Stony Brook)	290	410	495	780

3.2 Existing Conditions Hydrology Model Development

The hydrologic model selected to simulate the existing hydrologic conditions in the Goodwives River watershed is the *Hydrologic Modeling System* HEC-HMS (version 3.3) computer modeling program created by the U.S. Army Corps of Engineers (USACE). This modeling was completed to verify and update the flood flow data in the FIS.

3.2.1 Subwatershed Delineations

The electronic basemapping was utilized to first delineate the overall area of the Goodwives River watershed at the one-foot or five-foot contour interval. The overall Goodwives River watershed was then divided into 43 subwatersheds based on the confluence of tributaries, ponds, roadway crossings, or town boundaries. Drainage system mapping was utilized to adjust the effective subwatershed boundaries based on drainage system routing. Figure 3-1 presents the delineated subwatershed boundaries.

3.2.2 Runoff Curve Numbers

Cover type and hydrologic condition in each subwatershed were determined from a zoning map provided by the Town of Darien in Geographic Information System (GIS) format and 2008 aerial photography. Using the cover type and hydrologic conditions listed in Table 2-2a of the TR-55 user's manual, parcel data was classified as open space, impervious (paved or unpaved), urban (commercial or industrial), residential separated by lot size, and wooded (fair or good) (USDA, 1986). Figure 3-2 presents a map of the existing land cover.

Soil types in the watershed were determined from the Connecticut Department of Energy & Environmental Protection GIS database of the Natural Resources Conservation Service (NRCS) soil survey for Fairfield County, Connecticut, which includes Hydrologic Soil Group (HSG) classifications of all soils. Figure 3-3 depicts the soil group classifications for this watershed.

Based on the HSG types and land cover type, weighted or composite curve numbers (CNs) were developed for each subwatershed. Areas of imperviousness such as parking lots and buildings were assigned a CN of 98. The CNs used in the model were based on CNs for Connecticut developed by MMI to reflect conditions in Connecticut rather than the Midwestern conditions that were used to develop the NRCS's published CNs. These numbers have been accepted for use by the NRCS. A memo documenting these numbers and a letter from the NRCS authorizing their use are presented in Appendix A. CN calculations for each subwatershed in the Goodwives River watershed are presented in Appendix A. A summary of the CNs used in the HEC-HMS model is presented in Table 3-2.

TABLE 3-2
Composite CN Values for Existing Conditions HEC-HMS Model

Subbasin	Drainage Area (mi^2)	Composite CN
Mainstem Goodwives River		
GR MS-010	0.084	75
GR MS-020	0.045	76
GR MS-030	0.032	75
GR MS-040	0.046	75
GR MS-050	0.027	76
GR MS-060	0.007	80
GR MS-070	0.027	78
GR MS-080	0.022	76
GR MS-090	0.024	75
GR MS-100	0.047	76
GR MS-110	0.038	76
GR MS-120	0.081	77
GR MS-130	0.061	79
GR MS-140	0.037	87
GR MS-150	0.076	75
GR MS-160	0.073	80
GR MS-170	0.018	82
GR MS-180	0.087	76
GR MS-190	0.038	72
GR MS-200	0.030	67

TABLE 3-2 (continued)
Composite CN Values for Existing Conditions HEC-HMS Model

Subbasin	Drainage Area (mi^2)	Composite CN
Goodwives River Tributary 0		
GR T0-010	0.021	77
Goodwives River Tributary 1		
GR T1-010	0.073	75
GR T1-020	0.089	75
GR T1-030	0.078	76
GR T1-040	0.034	78
GR T1-050	0.006	71
GR T1-060	0.017	74
GR T1-070	0.089	75
GR T1-080	0.023	76
GR T1-TA	0.045	75
GR T1-TB	0.052	75
Goodwives River Tributary 2		
GR T2-010	0.03	80
Goodwives River Tributary 3		
GR T3-010	0.044	77
GR T3-020	0.067	79
Goodwives River Tributary 4		
GR T4-010	0.064	75
GR T4-020	0.027	79
GR T4-TA	0.019	77
Goodwives River Tributary 5		
GR T5-010	0.053	75
GR T5-020	0.073	69
GR T5-030	0.016	72
GR T5-040	0.087	80
Goodwives River Tributary 6		
GR T6-010	0.026	76
Goodwives River Tributary 7		
GR T7-010	0.061	78

3.2.3 Time of Concentration

Calculations of the time of concentration for each subwatershed are presented in Appendix A. Table 3-3 presents the lag time for each subwatershed that was used as input data to the HMS model.

TABLE 3-3
Lag Time Values Used in the Existing Conditions HEC-HMS Model

Subbasin	SCS* Unit Hydrograph - Lag Time (min)
Mainstem Goodwives River	
GR MS-010	52
GR MS-020	34
GR MS-030	32
GR MS-040	41
GR MS-050	27
GR MS-060	44
GR MS-070	24
GR MS-080	32
GR MS-090	31
GR MS-100	39
GR MS-110	43
GR MS-120	44
GR MS-130	34
GR MS-140	34
GR MS-150	31
GR MS-160	17
GR MS-170	15
GR MS-180	56
GR MS-190	39
GR MS-200	39
Goodwives River Tributary 0	
GR T0-010	42
Goodwives River Tributary 1	
GR T1-010	62
GR T1-020	61
GR T1-030	58
GR T1-040	28
GR T1-050	18
GR T1-060	47
GR T1-070	51
GR T1-080	34
GR T1-TA	49
GR T1-TB	41
Goodwives River Tributary 2	
GR T2-010	23
Goodwives River Tributary 3	
GR T3-010	42
GR T3-020	50

TABLE 3-3 (continued)
Lag Time Values Used in the Existing Conditions HEC-HMS Model

Subbasin	SCS* Unit Hydrograph - Lag Time (min)
Goodwives River Tributary 4	
GR T4-010	62
GR T4-020	37
GR T4-TA	33
Goodwives River Tributary 5	
GR T5-010	60
GR T5-020	58
GR T5-030	51
GR T5-040	28
Goodwives River Tributary 6	
GR T6-010	23
Goodwives River Tributary 7	
GR T7-010	44

*SCS = Soil Conservation Service

3.2.4 Precipitation

Estimated total rainfall depth from a storm event and the intensity of the rainfall dictate the overall runoff, and thus peak stream flows, from a watershed. The standard of practice for design engineers in Connecticut is to use rainfall data published in Technical Paper 40 (TP-40) by the United States Weather Bureau in 1961. TP-40 provides rainfall depths over a 24-hour duration for various storm frequencies. TP-40 predicts rainfall depths based on storm data from the first half of the 20th century (Table 3-4).

TABLE 3-4
Rainfall Depth Over 24-Hour Period

	Rainfall Storm Event - Total Rainfall (inches)			
Source	**50%**	**10%**	**2%**	**1%**
TP 40 (1961)	3.3	5.0	6.4	7.2

3.3 Results of Existing Conditions Analysis

Table 3-5 presents the predicted peak flow rates at select areas within the watershed. HEC-HMS input and output files are presented in Appendix B. Rainfall information for a 500-year 24-hour storm is not available. For modeling the 500-year recurrence interval, flows were extrapolated from the flows modeled with HEC-HMS (Table 3-5). Recurrence interval and modeled discharge were plotted for each flow change location (Figure 3-4).

TABLE 3-5
Predicted Peak Flows From HMS Modeling Results

Location	Drainage Area (mi^2)	HMS 50%	HMS 10%	HMS 2%	HMS 1%	Extrapolated 0.2%	HMS Junction
At Buttonwood Lane	0.87	196.1	440.5	658.3	788.4	1,016.3	J-090-100
Confluence of Tributary 3 (Downstream of Mansfield Place)	1.03	241.8	542.9	811.9	970.8	1,252.1	J-100-110
Upstream of Overbrook Lane	1.18	276.9	620.1	930.3	1,111.5	1,434.0	J-110-120
Upstream of Boston Post Road	1.32	309.9	693.5	1,046.2	1,250.2	1,613.5	J-130-140
MMI Datalogger Location	1.51	339.5	756.3	1,144.4	1,371.8	1,768.3	J-160-170
At Interstate-95	1.78	377.4	852.5	1,297.7	1,559.5	2,012.6	J-170-180
Upstream of Andrews Drive	1.86	394.9	895.9	1,365.3	1,642	2,119.6	J-180-190
Upstream of Confluence with Stony Brook	1.99	413.6	943.8	1,444.3	1,742.6	2,249.3	J-200

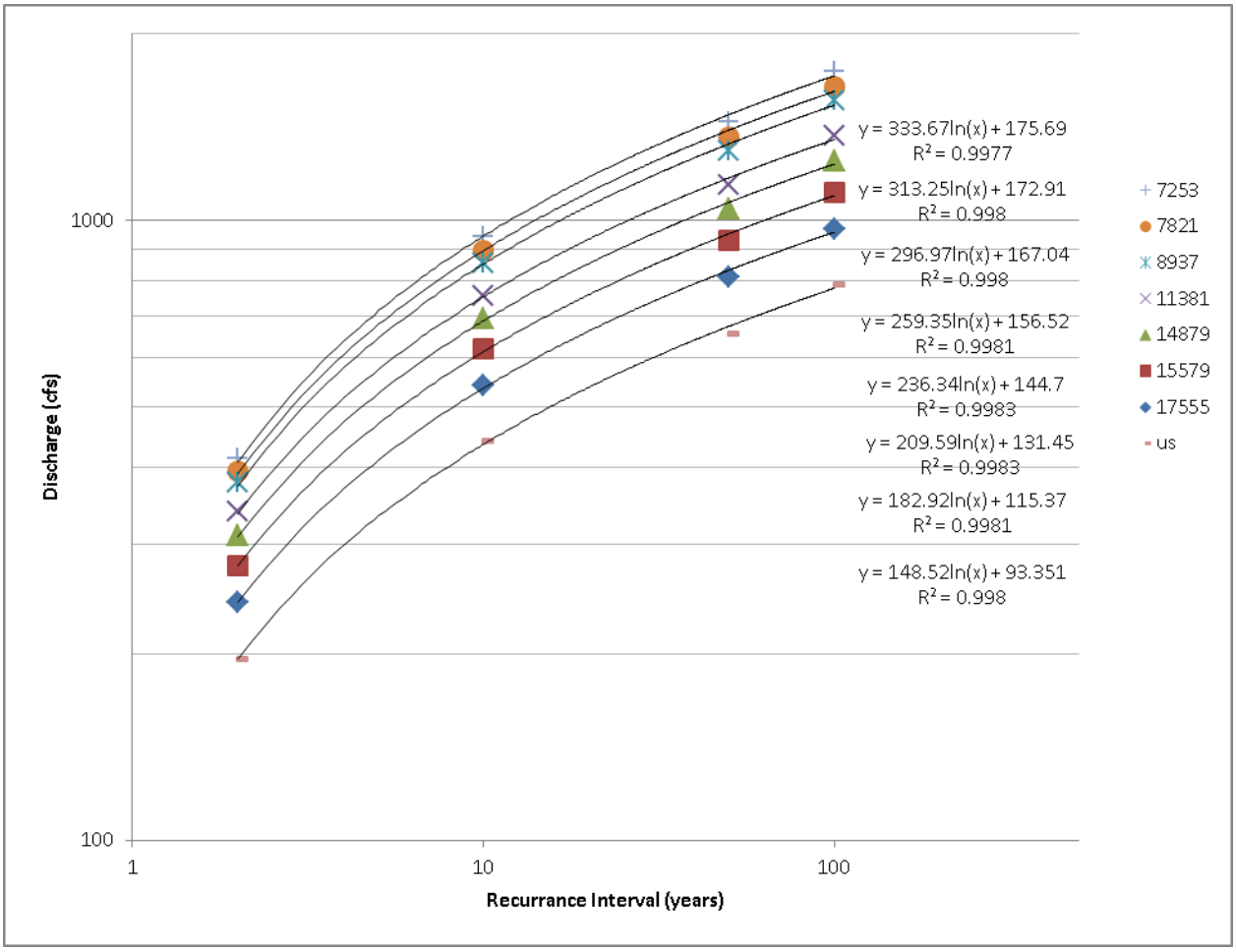

Figure 3-4: Relationship between modeled predicted discharge values for each point of interest (shown with River Stations) used to extrapolate to the 500-year recurrence interval.

Table 3-6 compares the data in Table 3-1 with that of Table 3-5. The results of the HMS analysis are higher than the FEMA Effective.

TABLE 3-6
Comparison of Hydrologic Analysis Results at Watershed Outlet

Methodology	10%	2%	1%	0.2%
FEMA FIS	290	410	495	780
HMS Model	414	1,444	1,743	2,249

4.0 EXISTING CONDITIONS HYDRAULIC ANALYSIS

4.1 FEMA Effective Model

An initial FIS of Darien with Flood Insurance Rate Maps (FIRMs) was published in 1978. The FIS included a detailed study of Goodwives River between Buttonwood Lane and Rings End Road at Gorham's Pond. The 2010 FIS indicates that Goodwives River was updated to include tidal backwatering, and the original modeling was maintained. MMI received the FEMA Effective Model in hard copy from FEMA contractors on March 30, 2009. This model uses the HEC-2 step backwater model as described in the current FIS. HEC-2 is the precursor modeling software to HEC-RAS. Model data provided to MMI from FEMA is presented in Appendix C.

Water surface elevations at each cross section in the HEC-2 model output, provided as part of the model, were compared to those presented in the FIS and were found to match. This confirms that the HEC-2 model received was the one used to produce the base flood data presented in the FIS and is the FEMA Effective Model. FEMA lettered cross sections are used as points of reference through the remainder of this document. The FEMA Effective HEC-2 Model serves as the starting point for the hydraulics analysis presented in the following sections.

4.2 Duplicate Effective Model

The Effective Duplicate Model was created by inputting the FEMA Effective Model into the USACE HEC-RAS 4.0 program (USACE, 2005). Flow rates from the FIS were used for this model run. The downstream boundary condition starting water surface elevation was retained. The model was executed in the subcritical flow regime. An upstream boundary condition of normal depth was used; unlike HEC-2, HEC-RAS requires an upstream boundary condition.

HEC-2 and HEC-RAS have some differences in modeling approach including conveyance calculations; bridge, culvert, and dam modeling approaches; and critical depth calculations. Each of these model differences can cause some differences in results, especially at structures. HEC-2 has two bridge modeling approaches, the normal bridge and special bridge. Normal bridges are similar to HEC-RAS bridges and provide most necessary information. Special bridges do not provide bridge opening geometry but rely on the user input of opening cross-sectional area for calculations. Special bridge openings were estimated based on provided maximum low chord elevation, channel invert, and opening width. In some cases, if a bottom elevation was specified that did not match the channel invert, a culvert was input to provide the correct bridge opening invert. HEC-2 does not have a specific inline structure feature, so three significant dams were included as bridge structures, and smaller dams were included as normal cross sections. These were input to HEC-RAS as they were modeled in HEC-2.

The bridge modeling approach was specified by the HEC-2 modeler in choosing either normal or special bridge calculations. The normal bridge method uses only the energy equations. The special bridge method calculates losses for low flow, weir flow, and pressure flow. Even if the special bridge method is selected, if there are no piers in the cross section, the model will revert to the energy only calculation. The HEC-2 model was carefully set up to include narrow artificial piers to ensure that the weir flow and pressure flow were calculated when appropriate. Weir flow calculations are specified for the dam structures (modeled as bridges) by including an artificial tiny opening with a pier. Bridge model approach was selected in the HEC-RAS model to match the method used in the HEC-2 model.

All HEC-2 bridges have channel cross sections co-located with the bridge opening, which is not allowed in HEC-RAS. The approach to providing distance between the face cross sections and the bridge varied with bridge type and information provided in HEC-2. Most normal bridges have six cross sections, three upstream and three downstream, with the inner four exact copies. If the bridge edge cross section matched the next adjacent

cross section, it was removed from the model, and channel distances were adjusted. In cases where geometry was different between the next adjacent cross section and the bridge opening cross section, an internal cross section was used to input the correct geometry at the opening. Special bridges typically include only four cross sections, all necessary in HEC-RAS. In this case, the HEC-RAS bridge width table was used to slightly narrow the bridge deck, without moving the bridge opening cross sections, to provide a minimal amount of space. In all cases, cross sections were not moved to affect overall channel distance or slope.

The Duplicate Effective Model approximately represents the FEMA Effective Model although in multiple locations the predicted water surface elevations varied. Table 4-1 is a comparison of published water surface elevations and the results of MMI's Duplicate Effective Model. Most cross sections duplicate the FEMA Effective water surface elevation within ±0.3 feet. A few cross sections have significantly different results, generally attributed to differences in computations between the two models. At FEMA E, the difference in water surface elevation of -1.2 feet is attributed to differences in conveyance calculations at the very steep (35.5% slope) drop representing a seven-foot tall dam directly upstream. At FEMA N, the cross section is at critical depth in the duplicate model but was not calculated to be critical in the FEMA Effective Model, causing a difference in water surface elevation of -1.2 feet. This difference appears to be due to an adverse channel slope upstream forcing the cross section to critical although the Froude number is only 0.89. Additional differences of 0.5 feet and -2.1 feet at FEMA Q and Z are attributed to differences in bridge modeling calculations at Tokeneke Road and Prospect Avenue structures. HEC-RAS report of input and output is included in Appendix D.

A floodway analysis was completed as part of the Duplicate Effective Model. The Effective Model received in hard copy format included output from the floodway analysis. Encroachment stations from the HEC-2 output were used as input to the Duplicate Effective Model Encroachment Table using Method 1. The encroached results

duplicate the Effective Model well but have a few differences (Table 4-1). Differences between the Effective Model and Duplicate Effective Model encroached results are at locations described above where the unencroached model showed differences (FEMA sections E, N, Q, Z). Other insignificant changes to the water surface elevation are seen. The exact width of the floodway and location of encroachment stations were maintained to not misrepresent the Effective floodway configuration. Differences in hydraulic calculations between the two models result in slightly different surcharge values.

TABLE 4-1
Comparison of Water Surface Elevation Between
Published 2010 FIS and Effective FEMA Duplicate Models
1% Recurrence Discharge

FEMA Cross-Section Location		Water Surface Elevation (feet NAVD88)					
Letter	Description	2010 FEMA FIS	Effective Duplicate Model	Difference (Effective Duplicate - FIS)	2010 FEMA FIS	Effective Duplicate Model	Difference (Effective Duplicate - FIS)
		Unencroached	Unencroached	Unencroached	Encroached	Encroached	Encroached
A	Approximately 2,000' US of Rings End Road; 2,020' DS of Goodwives River Road	5.4	5.4	-0.1	5.4	5.4	0.0
B	920' DS of Goodwives River Road	5.4	5.4	-0.1	5.4	5.4	0.0
C	100' DS of Goodwives River Road	5.7	5.5	-0.2	5.7	5.6	-0.2
D	50' US of Dam #2 (Upstream of Goodwives River Road)	12.5	12.4	-0.1	12.5	12.4	-0.1
E	610' US of Dam #2 (Constriction at Dam #3)	13.2	12.0	-1.2	13.2	12.1	-1.2
F	805' DS of Andrews Drive (Dam #6 crest)	24.5	24.5	0.0	24.5	24.6	0.1
G	100' DS of Andrews Drive	29.8	29.8	0.0	30.3	30.3	0.0
H	65' US of Andrews Drive	32.5	32.4	-0.1	32.5	32.3	-0.2
I	117' DS of Private Driveway	32.9	32.8	-0.1	33.1	33.0	-0.1

J	87' DS of Private Driveway (Constriction at Abutments)	34.1	34.2	0.1	35.1	35.5	0.4
K	40' US of Private Driveway	35.8	35.6	-0.2	35.8	35.7	-0.1
L	50' DS of Locust Hill Road	36.3	36.2	-0.1	36.4	36.4	0.0
M	40' US of Locust Hill Road	37.1	36.8	-0.3	37.1	37.4	0.3
N	350' US of Locust Hill Road (Under I-95)	37.9	36.8	-1.2	38.1	37.4	-0.7
O	595' US of Locust Hill Road	39.5	39.7	0.2	39.9	40.0	0.1
P	45' DS of Tokeneke Road	40.1	40.2	0.0	40.7	40.7	0.0
Q	20' US of Tokeneke Road	42.5	43.0	0.5	42.5	43.4	0.9
R	170' US of Tokeneke Road (US of Railroad)	43.3	43.6	0.3	43.3	43.8	0.5
S	50' DS of Parking Lot Access	43.5	43.7	0.2	43.5	44.1	0.6
T	70' US of Parking Lot Access	44.1	44.2	0.1	44.2	44.6	0.4
U	45' DS of Old Kings Highway North	44.4	44.2	-0.2	44.5	44.8	0.3
V	50.5' US of Old Kings Highway North	45.0	44.9	-0.1	45.0	45.1	0.1
W	55' DS of Boston Post Road	50.8	50.7	-0.1	51.4	50.8	-0.6
X	75' US of Boston Post Road	53.4	53.3	-0.1	53.4	53.1	-0.3
Y	25' DS of Prospect Avenue	60.1	60.2	0.1	60.1	60.3	0.2
Z	40' US of Prospect Avenue	65.1	63.1	-2.0	65.1	63.3	-1.8
AA	230' US of Prospect Avenue (Dam #7 Crest)	66.2	66.2	0.0	66.2	66.2	0.0
AB	350' US of Prospect Avenue (Dam #8 Crest)	67.8	67.7	-0.1	67.8	67.7	-0.1
AC	50' DS of Granaston Lane	80.1	79.8	-0.3	80.5	79.9	-0.6
AD	55' US of Granaston Lane	84.7	84.9	0.2	84.7	84.7	0.0
AE	42' DS of Private Driveway	87.5	87.5	0.0	87.5	87.4	-0.1
AF	32' US of Private Driveway	89.7	89.8	0.1	89.8	89.9	0.1

AG	40' DS of Dam #11	93.6	93.6	0.0	93.6	93.6	0.0
AH	30' US of Dam #11	98.2	98.2	0.0	98.2	98.2	0.0
AI	420' US of Dam #11 (Dam #12 Toe)	98.9	98.6	-0.3	99.1	98.7	-0.4
AJ	430' US of Dam #11 (Dam #12 Crest)	101.1	101.1	0.0	101.3	101.3	0.0
AK	70' DS of Overbrook Lane	103.6	103.7	0.1	104.2	104.3	0.1
AL	20' US of Dam #15 (US of Overbrook Lane)	107.9	107.9	0.0	107.9	107.9	0.0
AM	705' US of Dam #15 (Constriction at Footbridge)	112.7	112.8	0.1	112.7	112.7	0.0
AN	1,190' DS of Buttonwood Lane	116.6	116.4	-0.2	117.1	116.4	-0.7
AO	650' DS of Buttonwood Lane	124.2	124.2	0.0	124.3	124.3	0.0
AP	190' DS of Buttonwood Lane	129.1	129.2	0.1	129.3	129.4	0.1
AQ	DS of Buttonwood Lane	132.7	132.7	0.0	133.3	132.9	-0.5

Notes: The Effective Model has a vertical datum of NGVD29. The Duplicate Effective and Revised Duplicate Effective Models retain the vertical datum, but output values listed were converted for comparison using the conversion factor of -1.0 used in the 2010 FIS update.

US = upstream

DW = downstream

4.3 Revised Duplicate Effective Model

Modeling techniques were updated to current standards, and errors and obvious typos were corrected in the Duplicate Effective Model to create the Revised Duplicate Effective Model. Flow rates from the FIS were retained for this modeling effort. The following changes and corrections were incorporated:

- Changed method of calculating conveyance from "between every coordinate point (HEC2 Style)" to "At breaks in n values only." This is the standard calculation method for use in HEC-RAS.

- Corrected ineffective flow areas upstream and downstream of structures to reflect 1:1 expansion and contraction ratios (1:1.5 downstream of culverts). Also corrected elevations to be minimum road elevation on upstream side and an average of minimum road elevation and maximum low chord on the downstream side of structure.

- Corrected location of bank stations (in HEC-2 bank stations were used to set the ineffective flow areas so it brought them off the bank into the channel) at model bridge cross sections where inside they were specified as in the channel.

- Corrected obvious typos in river distance data at model cross section 36.3. This shifted the revised duplicate 4.5 feet upstream above this location.

- The Dam #2, #11, and #15 structures were input as inline structures, with information provided in the HEC-2 model.

The base flood elevation profile generated from the Revised Duplicate Effective Model matches the Duplicate Effective Model at the majority of the cross sections as shown in Table 4-2. The significant change from the Duplicate Effective Model is at FEMA J and the immediate upstream sections. In the Effective Model, the bridge opening was partially blocked with ineffective flow areas. These were corrected and have changed hydraulics at the bridge, causing critical depth to occur differently. Smaller differences of 0.1 and -0.2 feet are seen at FEMA V and Z and may be attributed to the corrected ineffective flow areas. HEC-RAS report of input and output is included in Appendix E.

TABLE 4-2
Comparison of Water Surface Elevation Between
Effective FEMA Duplicate and Revised Effective Duplicate Models
1% Recurrence Discharge

FEMA Cross-Section Location	Water Surface Elevation (feet NAVD88)					
Letter	Effective Duplicate Model	Revised Effective Duplicate Model	Difference (Revised Effective Duplicate - Effective Duplicate)	Effective Duplicate Model	Revised Effective Duplicate Model	Difference (Revised Effective Duplicate - Effective Duplicate)
	Unencroached	Unencroached	Unencroached	Encroached	Encroached	Encroached
A	6.4	6.4	0.0	6.4	6.4	0.0
B	6.4	6.4	0.0	6.4	6.4	0.0
C	6.5	6.6	0.0	6.6	6.6	0.0
D	13.4	13.4	0.0	13.4	13.4	0.0
E	13.0	13.0	0.0	13.1	13.0	0.0
F	25.5	25.5	0.0	25.6	25.6	0.0
G	30.8	30.9	0.0	31.3	31.3	0.0
H	33.4	33.4	0.0	33.3	33.3	0.0
I	33.8	33.8	0.0	34.0	34.0	0.0
J	35.2	34.2	-1.0	36.5	36.6	0.0
K	36.6	37.6	1.0	36.7	36.7	0.0
L	37.2	37.9	0.7	37.4	37.4	0.0
M	37.8	38.2	0.4	38.4	38.4	0.0
N	37.8	37.8	0.0	38.4	38.4	0.0
O	40.7	40.7	0.1	41.0	41.1	0.1
P	41.2	41.2	0.1	41.7	41.9	0.2
Q	44.0	44.0	0.0	44.4	44.5	0.1
R	44.6	44.7	0.0	44.8	45.0	0.1
S	44.7	44.8	0.1	45.1	45.2	0.1
T	45.2	45.2	0.0	45.6	45.7	0.1
U	45.2	45.3	0.0	45.8	45.9	0.0
V	45.9	46.0	0.1	46.1	46.2	0.0
W	51.7	51.7	0.0	51.8	51.8	0.0
X	54.3	54.3	0.0	54.1	54.1	0.0

FEMA Cross-Section Location	Water Surface Elevation (feet NAVD88)					
Letter	Effective Duplicate Model	Revised Effective Duplicate Model	Difference (Revised Effective Duplicate - Effective Duplicate)	Effective Duplicate Model	Revised Effective Duplicate Model	Difference (Revised Effective Duplicate - Effective Duplicate)
	Unencroached	Unencroached	Unencroached	Encroached	Encroached	Encroached
Y	61.2	61.2	0.0	61.3	61.3	0.0
Z	64.1	64.0	-0.2	64.3	64.3	0.0
AA	67.2	67.2	0.0	67.2	67.2	0.0
AB	68.7	68.8	0.1	68.7	68.8	0.1
AC	80.8	80.8	0.0	80.9	80.9	0.0
AD	85.9	85.6	-0.3	85.7	85.5	-0.3
AE	88.5	88.5	0.0	88.4	88.4	0.0
AF	90.8	90.8	0.0	90.9	90.9	0.0
AG	94.6	94.6	0.0	94.6	94.6	0.0
AH	99.2	99.2	0.0	99.2	99.2	0.0
AI	99.6	99.7	0.1	99.7	99.7	0.0
AJ	102.1	102.1	0.0	102.3	102.3	0.0
AK	104.7	104.6	0.0	105.3	105.4	0.0
AL	108.9	108.9	0.0	108.9	108.9	0.0
AM	113.8	113.8	0.0	113.7	113.7	0.0
AN	117.4	117.5	0.1	117.4	117.5	0.1
AO	125.2	125.2	0.0	125.3	125.3	0.0
AP	130.2	130.2	0.0	130.4	130.4	0.0
AQ	133.7	133.7	0.0	133.9	133.9	0.0

Note: The Effective Model has a vertical datum of NGVD29. The Duplicate Effective and Revised Duplicate Effective Models retain the vertical datum, but output values listed were converted for comparison using the conversion factor of -1.0 used in the 2010 FIS update.

4.4 Existing Conditions Model

4.4.1 Model Development

Changes to the watercourse have occurred since the FEMA Effective Model was created. The Town of Darien has high resolution aerial photography and topographic mapping with one-foot contour data created in 2008, which served as a basis for the model updates. HEC-GeoRAS 4.1.1, an extension for *ArcGIS* (ESRI 2006), was used to extract stream system geometry from terrain data for automated input to HEC-RAS. HEC-GeoRAS is an interactive platform for setting up all geometry components necessary for HEC-RAS modeling and viewing results. Topography from the town was processed using *ArcGIS* to create a triangulated irregular network (TIN) representing ground elevation for use in modeling.

The stream centerline and overbank distances were delineated based on 2008 mapping, updating the distance between cross sections and length of river channel from the FEMA model. FEMA Effective Model cross section locations were maintained, and additional new cross sections were added where necessary. Floodplain topography was extracted from the 2008 topographic mapping with HEC-GeoRAS for all model cross sections. Field survey of the wet channel cross sections completed by MMI was then substituted into the model for all new cross sections and to update FEMA Effective Model cross sections. Figure 4-1 depicts the location of cross sections surveyed by MMI for this study. Of the 104 total cross sections in the model, 38 FEMA sections were retained, 44 sections were resurveyed as part of this project, and 22 were newly surveyed and added to the model.

Bridge and culvert geometry was updated with survey, field measurements, and existing bridge plans. Bridge plans reviewed for this study are listed in the References section of

this report and included in electronic format on a CD in Appendix H. Fourteen of the 15 total bridge structures were updated with survey information as part of this study.

The model was extended upstream to include the Buttonwood Lane bridge. Multiple other structures were included in MMI's model that were not represented in the FEMA Effective Model. Four dam structures were added to the model including two of the larger structures between Goodwives River Road and Andrews Drive (Dam #4 and Dam #6), upstream of Granaston Lane (Dam #10), and the dam backwatering the downstream side of Overbrook Lane (Dam #14). The structures for I-95 and the railroad crossing were added to existing conditions. These were not included in the FEMA Effective Model as bridges but as a single cross section representing the channel under the bridge deck (FEMA M and Q).

Manning's n values used in the FEMA Effective Model were verified and updated based on field observations, digital photographs, and high resolution aerial photography. Manning's n values were varied horizontally in HEC-RAS to allow for accurate representation of changes in roughness in each cross section. These varied between 0.03 and 0.05 in the channel and between 0.015 and 0.12 in the overbank.

Expansion and contraction coefficients were verified and largely maintained from the Duplicate Effective Model. The Duplicate Effective Model typically specified 0.1 and 0.3 for a normal cross section and 0.3 and 0.5 at bridge cross sections. These were verified and corrected where necessary.

The vertical datum of the HEC-RAS model was updated from NGVD 1929, which is the datum of the Effective Model, to the current standard of NAVD 1988. A conversion factor of -1.1 was used as calculated using the National Geodetic Survey *VERTCON* online software. The conversion factor reported in the 2010 countywide FIS is -1.0. The watershed specific conversion factor of -1.1 is more accurate for this specific area.

Flow data developed by MMI and documented in Section 3.0 was used in the hydraulic model. The FEMA Effective Model included only one flow change location, upstream of Boston Post Road. Five additional flow change locations were added in MMI's model at tributaries and significant changes in watershed area to describe the hydrology as accurately as possible (Table 4-3). As described in detail in Section 3.0, the existing conditions flows are higher than the FEMA effective flows.

TABLE 4-3
Estimated Existing Peak Flows Used for the Hydraulic Model

Flow Change Location		1% Chance Discharge (cfs*)	
Description	**HEC-RAS River Station**	**FEMA Effective**	**MMI Updated**
Upstream of Buttonwood Lane	17,555	360	971
Confluence of Tributary 3	15,579	--	1,112
Upstream of Overbrook Lane	14,879	--	1,250
Upstream of Boston Post Road	11,381	495	1,372
MMI Datalogger Location	8,937	--	1,560
Upstream of Interstate 95	7,821	--	1,642
Upstream of Andrews Drive	7,253	--	1,743

*cfs = cubic feet per second

The upstream model boundary condition was maintained as normal depth (S = 0.012). The downstream boundary condition is normal depth (S = .00035). An additional trial was run to determine the influence of the tide in Gorham's Pond, using known water surface elevations associated with the tidal storm surge, as indicated on the 2010 FIS Profile. The high water surface levels of the storm surge converge with the normal water surface elevation at Dam #2 (just upstream of Goodwives River Road) and do not affect conditions farther upstream.

The Existing Conditions Model covers 3.2 miles of the Goodwives River. The river stationing indicates a net increase in total stream length of 0.12 miles due to higher resolution data defining a more sinuous channel and extension of the model approximately 175 feet upstream to include the Buttonwood Lane bridge instead of

ending at the downstream face of the structure. This bridge was included to evaluate potential mitigation for overtopping of Buttonwood Lane during floods.

4.4.2 Existing Conditions Unencroached Results

Peak 1% annual chance existing conditions water surface elevations are generally higher than the FEMA Effective base flood elevations. Higher values are in part a result of higher flow values used in the model. Changes in channel configurations identified by MMI also increase predicted water surface elevations. For example, a dip in bed elevation near FEMA O in the Darien Land Trust land was resurveyed and found to be 2.5 feet higher, removing the negative bed slope and significantly changing the resulting water surface elevation.

Also, multiple structures were added that were not included in the FEMA Effective Model. Inclusion of constricting valley walls shows backwatering downstream of Andrews Drive, affecting hydraulics at the crossing. Inclusion of the railroad bridge appears to have captured backwatering upstream not accounted for in the FEMA Effective Model. Increased accuracy of bridge configuration also showed large changes from the FEMA Effective Model. The definition of the arch at Prospect Avenue showed increased water surface elevation of 8.1 feet higher than the Revised Duplicate Model.

The downstream boundary condition of normal depth caused decreases in water surface elevation through Gorham's Pond. As noted above, this boundary condition does not take into consideration the tidal influence of the pond, which is combined with the hydraulic study when mapping the floodplains.

Table 4-4 presents water surface elevations for the Existing Conditions Model and Revised Duplicate Effective Model. Appendix F contains the HEC-RAS summary report generated for the Existing Conditions Model.

TABLE 4-4
Comparison of Water Surface Elevations
Revised Duplicate Effective and Existing Conditions Unencroached Model Results
100-Year (1%) Recurrence Discharge

FEMA Cross-Section Location	Description	Water Surface Elevation (Feet NAVD)			
		Revised Duplicate Effective	Revised Duplicate Effective (NAVD)	Existing Conditions (NAVD)	Difference
A	Approximately 2,000' US of Rings End Road; 2,020' DS of Goodwives River Road	6.4	5.3	4.9	-0.4
B	920' DS of Goodwives River Road	6.4	5.3	5.2	0.0
C	100' DS of Goodwives River Road	6.6	5.5	6.8	1.4
D	50' US of Dam #2 (Upstream of Goodwives River Road)	13.4	12.3	14.4	2.1
E	610' US of Dam #2 (Constriction at Dam #3)	13.0	11.9	14.5	2.5
F	805' DS of Andrews Drive (Dam #6 Crest)	25.5	24.4	27.1	2.7
G	100' DS of Andrews Drive	30.9	29.8	35.6	5.9
H	65' US of Andrews Drive	33.4	32.3	36.2	3.9
I	117' DS of Private Driveway	33.8	32.7	36.4	3.7
J	87' DS of Private Driveway (Constriction at Abutments)	34.2	33.1	36.8	3.6
K	40' US of Private Driveway	37.6	36.5	38.4	1.9
L	50' DS of Locust Hill Road	37.9	36.8	39.3	2.5
M	40' US of Locust Hill Road	38.2	37.1	40.4	3.3
N	350' US of Locust Hill Road (Under I-95)	37.8	36.7	41.8	5.2
O	595' US of Locust Hill Road	40.7	39.6	42.7	3.1
P	45' DS of Tokeneke Road	41.2	40.1	43.6	3.5
Q	20' US of Tokeneke Road	44.0	42.9	45.8	2.9
R	170' US of Tokeneke Road (US of Railroad)	44.7	43.6	48.5	4.9
S	50' DS of Parking Lot Access	44.8	43.7	48.5	4.8
T	70' US of Parking Lot Access	45.2	44.1	48.5	4.4
U	45' DS of Old Kings Highway North	45.3	44.2	48.5	4.3
V	50.5' US of Old Kings Highway North	46.0	44.9	48.6	3.6
W	55' DS of Boston Post Road	51.7	50.6	51.7	1.1
X	75' US of Boston Post Road	54.3	53.2	56.2	2.9
Y	25' DS of Prospect Avenue	61.2	60.1	61.7	1.6
Z	40' US of Prospect Avenue	64.0	62.9	71.0	8.1
AA	230' US of Prospect Avenue (Dam #7 Crest)	67.2	66.1	71.0	4.8

FEMA Cross-Section Location	Description	Water Surface Elevation (Feet NAVD)			
		Revised Duplicate Effective	Revised Duplicate Effective (NAVD)	Existing Conditions (NAVD)	Difference
AB	350' US of Prospect Avenue (Dam #8 Crest)	68.8	67.7	70.9	3.3
AC	50' DS of Granaston Lane	80.8	79.7	83.4	3.6
AD	55' US of Granaston Lane	85.6	84.5	91.8	7.3
AE	42' DS of Private Driveway	88.5	87.4	91.8	4.4
AF	32' US of Private Driveway	90.8	89.7	94.0	4.3
AG	40' DS of Dam #11	94.6	93.5	96.3	2.8
AH	30' US of Dam #11	99.2	98.1	99.3	1.2
AI	420' US of Dam #11 (Dam #12 Toe)	99.7	98.6	101.7	3.1
AJ	430' US of Dam #11 (Dam #12 Crest)	102.1	101.0	102.8	1.8
AK	70' DS of Overbrook Lane	104.6	103.5	105.7	2.2
AL	20' US of Dam #15 (US of Overbrook Lane)	108.9	107.8	109.2	1.4
AM	705' US of Dam #15 (Constriction at Footbridge)	113.8	112.7	114.8	2.1
AN	1,190' DS of Buttonwood Lane	117.5	116.4	117.0	0.6
AO	650' DS of Buttonwood Lane	125.2	124.1	123.1	-1.0
AP	190' DS of Buttonwood Lane	130.2	129.1	130.7	1.7
AQ	DS of Buttonwood Lane	133.7	132.6	133.0	0.4

US = upstream
DS = downstream

Floodplain mapping was developed by exporting HEC-RAS results back to *ArcGIS* using HEC-GeoRAS for each flood profile. Floodplain mapping was corrected to remove errors in topography and correctly represent conditions at structures. The tidal influence of Long Island Sound was included in the revised mapping based on data in the 2010 FIS. The FEMA Effective floodplain and Existing Conditions Model results were compared on an annotated FIRM, and this mapping is presented in Appendix G. The shape of the existing conditions floodplain is also much more detailed as it is based on more accurate topography, also presented in Appendix G.

4.4.3 Existing Conditions Floodway Analysis (Encroached)

An encroachment analysis was completed for the Revised Existing Conditions Model to establish a more current floodway reflecting updated existing conditions. The existing

conditions floodway is compared to the Revised Duplicate Effective Encroached Model results (Table 4-5), showing increases in water surface elevation in almost all locations. Like the floodplains, the floodway widths were generally increased to include a wider area (Table 4-6). Four buildings have been identified in the revised floodplain on properties at 668 and 676 Boston Post Road and 5 and 10 Overbrook Lane.

TABLE 4-5
Comparison of Water Surface Elevations
Revised Duplicate Effective and Existing Conditions Encroached Model Results
100-Year (1%) Recurrence Discharge

FEMA Cross-Section Location	Description	Water Surface Elevation (Feet NAVD)				
		Existing Conditions	Existing Conditions	Existing Conditions Surcharge	FEMA Published 2010 FIS	Difference (Existing Conditions - 2010 FIS)
		Unencroached	Encroached	Difference	Encroached	Encroached
A	Approximately 2,000' US of Rings End Road; 2,020' DS of Goodwives River Road	4.9	4.9	0.0	5.4	-0.5
B	920' DS of Goodwives River Road	5.2	5.2	0.0	5.4	-0.2
C	100' DS of Goodwives River Road	6.8	6.9	0.1	5.7	1.2
D	50' US of Dam #2 (Upstream of Goodwives River Road)	14.4	14.5	0.1	12.5	2.0
E	610' US of Dam #2 (Constriction at Dam #3)	14.5	14.5	0.0	13.2	1.3
F	805' DS of Andrews Drive (Dam #6 Crest)	27.1	27.4	0.3	24.5	2.9
G	100' DS of Andrews Drive	35.6	35.6	0.0	30.3	5.3
H	65' US of Andrews Drive	36.2	37.1	0.9	32.5	4.6
I	117' DS of Private Driveway	36.4	37.3	0.9	33.1	4.2
J	87' DS of Private Driveway (Constriction at Abutments)	36.8	37.7	0.9	35.1	2.6
K	40' US of Private Driveway	38.4	38.8	0.4	35.8	3.0
L	50' DS of Locust Hill Road	39.3	39.9	0.6	36.4	3.5
M	40' US of Locust Hill Road	40.4	41.1	0.6	37.1	4.0
N	350' US of Locust Hill Road (Under I-95)	41.8	42.0	0.2	38.1	3.9
O	595' US of Locust Hill Road	42.7	43.6	0.9	39.9	3.7
P	45' DS of Tokeneke Road	43.6	44.5	0.9	40.7	3.8
Q	20' US of Tokeneke Road	45.8	46.4	0.6	42.5	3.9

FEMA Cross-Section Location	Description	Water Surface Elevation (Feet NAVD)				
		Existing Conditions	Existing Conditions	Existing Conditions Surcharge	FEMA Published 2010 FIS	Difference (Existing Conditions - 2010 FIS)
		Unencroached	Encroached	Difference	Encroached	Encroached
R	170' US of Tokeneke Road (US of Railroad)	48.5	48.5	0.0	43.3	5.2
S	50' DS of Parking Lot Access	48.5	48.5	0.0	43.5	5.0
T	70' US of Parking Lot Access	48.5	48.9	0.4	44.2	4.7
U	45' DS of Old Kings Highway North	48.5	48.9	0.4	44.5	4.4
V	50.5' US of Old Kings Highway North	48.6	49.3	0.8	45.0	4.3
W	55' DS of Boston Post Road	51.7	52.1	0.3	51.4	0.7
X	75' US of Boston Post Road	56.2	56.8	0.6	53.4	3.4
Y	25' DS of Prospect Avenue	61.7	62.1	0.4	60.1	2.0
Z	40' US of Prospect Avenue	71.0	71.6	0.7	65.1	6.5
AA	230' US of Prospect Avenue (Dam #7 Crest)	71.0	71.5	0.5	66.2	5.3
AB	350' US of Prospect Avenue (Dam #8 Crest)	70.9	71.7	0.7	67.8	3.9
AC	50' DS of Granaston Lane	83.4	83.4	0.0	80.5	2.9
AD	55' US of Granaston Lane	91.8	92.7	0.9	84.7	8.0
AE	42' DS of Private Driveway	91.8	92.8	1.0	87.5	5.3
AF	32' US of Private Driveway	94.0	95.0	1.0	89.8	5.2
AG	40' DS of Dam #11	96.3	96.3	0.0	93.6	2.7
AH	30' US of Dam #11	99.3	99.7	0.4	98.2	1.5
AI	420' US of Dam #11 (Dam #12 Toe)	101.7	101.7	0.0	99.1	2.6
AJ	430' US of Dam #11 (Dam #12 Crest)	102.8	102.8	0.0	101.3	1.5
AK	70' DS of Overbrook Lane	105.7	106.0	0.3	104.2	1.8
AL	20' US of Dam #15 (US of Overbrook Lane)	109.2	109.2	0.0	107.9	1.3
AM	705' US of Dam #15 (Constriction at Footbridge)	114.8	115.3	0.4	112.7	2.6
AN	1,190' DS of Buttonwood Lane	117.0	117.6	0.6	117.1	0.5
AO	650' DS of Buttonwood Lane	123.1	123.7	0.6	124.3	-0.6
AP	190' DS of Buttonwood Lane	130.7	131.2	0.5	129.3	1.9
AQ	DS of Buttonwood Lane	133.0	133.7	0.7	133.3	0.3

US = upstream
DS = downstream

TABLE 4-6

Comparison of Floodway Width
Published 2010 FEMA FIS and Existing Conditions Encroached Model Results
100-Year (1%) Recurrence Discharge

FEMA Cross-Section Location	Description	Floodway Width (Feet)		
		FEMA Published 2010 FIS	Existing Conditions	Difference
A	Approximately 2000' US of Rings End Road; 2,020' DS of Goodwives River Road	220	220	0.0
B	920' DS of Goodwives River Road	90	90	0.0
C	100' DS of Goodwives River Road	75	50	-25.0
D	50' US of Dam #2 (Upstream of Goodwives River Road)	125	90	-35.0
E	610' US of Dam #2 (Constriction at Dam #3)	25	40	15.0
F	805' DS of Andrews Drive (Dam #6 Crest)	40	50	10.0
G	100' DS of Andrews Drive	25	230	205.0
H	65' US of Andrews Drive	40	50	10.0
I	117' DS of Private Driveway	40	60	20.0
J	87' DS of Private Driveway (Constriction at Abutments)	120	60	-60.0
K	40' US of Private Driveway	40	50	10.0
L	50' DS of Locust Hill Road	20	60	40.0
M	40' US of Locust Hill Road	40	60	20.0
N	350' US of Locust Hill Road (Under I-95)	20	40	20.0
O	595' US of Locust Hill Road	20	60	40.0
P	45' DS of Tokeneke Road	20	80	60.0
Q	20' US of Tokeneke Road	20	40	20.0
R	170' US of Tokeneke Road (US of Railroad)	20	120	100.0
S	50' DS of Parking Lot Access	20	80	60.0
T	70' US of Parking Lot Access	40	60	20.0
U	45' DS of Old Kings Highway North	50	50	0.0
V	50.5' US of Old Kings Highway North	30	40	10.0
W	55' DS of Boston Post Road	15	50	35.0
X	75' US of Boston Post Road	15	130	115.0
Y	25' DS of Prospect Avenue	20	30	10.0
Z	40' US of Prospect Avenue	60	60	0.0
AA	230' US of Prospect Avenue (Dam #7 Crest)	70	40	-30.0

FEMA Cross-Section Location	Description	Floodway Width (Feet)		
		FEMA Published 2010 FIS	Existing Conditions	Difference
AB	350' US of Prospect Avenue (Dam #8 Crest)	50	40	-10.0
AC	50' DS of Granaston Lane	15	30	15.0
AD	55' US of Granaston Lane	15	50	35.0
AE	42' DS of Private Driveway	10	30	20.0
AF	32' US of Private Driveway	20	30	10.0
AG	40' DS of Dam #11	40	80	40.0
AH	30' US of Dam #11	65	80	15.0
AI	420' US of Dam #11 (Dam #12 Toe)	20	70	50.0
AJ	430' US of Dam #11 (Dam #12 Crest)	20	80	60.0
AK	70' DS of Overbrook Lane	20	120	100.0
AL	20' US of Dam #15 (US of Overbrook Lane)	160	200	40.0
AM	705' US of Dam #15 (Constriction at Footbridge)	35	40	5.0
AN	1,190' DS of Buttonwood Lane	20	40	20.0
AO	650' DS of Buttonwood Lane	110	100	-10.0
AP	190' DS of Buttonwood Lane	25	60	35.0
AQ	DS of Buttonwood Lane	15	60	45.0

US = upstream
DS = downstream

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Bridge plans were evaluated for updating the hydraulic modeling. Plans reviewed include the following:

Goodwives River Road Bridge Plan – Town of Darien, Proposed Slab Bridge over Goodwives River, Bridge 035-003.

Andrews Drive Bridge Safety Inspection - State Project No. 170-2357. Routine Inspection Report for Bridge No. 04994. Carrying Andrews Drive over Goodwives River. Darien, Connecticut. Concrete Culvert. Inspected September 28, 2005. Prepared by Garg Consulting Services, Inc.

Andrews Drive Bridge Plan - Town of Darien, Connecticut, Bridge Rehabilitation, Bridge Plan and Specifications Sheet. Prepared by Stearns & Wheler. Signed October 1990.

Andrews Drive Capacity Calculations – Capacity Calculations and associated plans and data Prepared by Close, Jensen and Miller, P.C. March 1994.

Tokeneke Road Bridge Plans – Town of Darien, Bridge No. 136-0.90 Widening of Bridge. Prepared by Connecticut State Highway Department. Dated April 1939.

Interstate Route 95 Plans – Rehabilitation of Interstate Route 95, Bridge 00044 over Kings Highway & Goodwives River, General Plan. Prepared by F.G.A. Services, Inc. Dated May 26, 1989.

Old Kings Highway North Bridge Inspection – Bridge Safety Inspection, State Project No. 170-2357, Routine Inspection Report for Bridge No. 04144, Carrying Old Kings Highway North over Goodwives River, Darien, Connecticut. Adjacent Prestressed Concrete Deck Units. Prepared by AI Engineers, Inc. Inspected September 21, 2005.

Boston Post Road Bridge Plans – Bridge No. US1-12.0 over Goodwives River, Town of Darien, Plan Showing Proposed Reconstruction. Prepared by Connecticut State Highway Department. Dated October 26, 1950.

Prospect Avenue Bridge Plans – Prospect Avenue over Goodwives River. Prepared by Goodkind & O'Dea, Inc. Dated July 19, 1991.

Granaston Lane Bridge Plans – Proposed Replacement of the Granaston Lane Bridge over the Goodwives River, Town of Darien, Connecticut. Prepared by Goodkind & O'Dea, Inc. Dated June 18, 2001.

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