MILL POND HABITAT RESTORATION PROJECT

APPENDIX B

HYDROLOGY AND HYDRAULIC REPORT

Photograph of Main Street Dam and Mill Pond, Rippowam River, Stamford, Connecticut

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1) PURPOSE AND SCOPE

The purpose of this study is to evaluate restoration and enhancement alternatives for the Rippowam River (known locally as Mill River), with emphasis on Mill Pond and Mill Pond Park. The scope of the hydrologic and hydraulic study is to determine the hydraulic consequences associated with three basic alternatives: Alternative 1 (no action, without-project), Alternative 2 (removal of Main Street Dam and the retaining walls in Mill Pond Park), and Alternative 4 (removing only the retaining walls). Hydraulic modeling of Alternative 3 (removal of Main Street Dam and the creation of a series of step pools) was not performed since it was determined early on that this alternative was incompatible with ecosystem restoration goals.

This appendix presents information on the hydraulic and sediment transport implications of these alternatives. Hydraulic analyses were performed using the U.S. Army Corps of Engineers HEC-RAS hydraulic model. Analyses include flow, channel velocity, top width, energy gradients, shear stress, and minimum particle size for incipient motion. Hydraulic conditions in the vicinity of Mill Pond Park were analyzed for the 1, 2, 10, 50, 100, and 500-year floods as well as average daily flow, representing a non-flood scenario. Shear stress and particle stability analyses were performed for the three alternatives. While the focus of the restoration efforts is in the vicinity of Mill Pond, hydraulic analyses were extended from 550 feet upstream of Long Island Sound to approximately 2.5 river miles upstream from the Main Street Dam since the study area encompasses this entire reach. Including this entire reach in the model insured that hydraulic parameters were available for all restoration measures considered in addition to the basic alternatives.

2) AUTHORITY

This study was performed by the U.S. Army Corps of Engineers (USACE), New England District, under section 206 of the Water Resources Development Act of 1996 (PL 104-303) entitled "Aquatic Ecosystem Restoration".

3) SITE DESCRIPTION

a. <u>General</u>

The Rippowam River basin (known locally as the Mill River) is an approximately 37.5 square mile watershed located in southeastern New York and southwestern Connecticut. The river originates in Ridgefield, Connecticut and flows 17 miles to the West Branch of Stamford Harbor at Stamford, Connecticut where it empties into Long Island Sound.

The area has experienced considerable urban and residential growth, yet the city of Stamford still maintains a greenbelt. The Main Street Dam is located on the Rippowam River (known locally as Mill River) approximately 3,800 feet from the mouth of the river at Long Island Sound and approximately 3,000 feet north of I-95 (see Figure 1). The dam impounds Mill Pond with a water surface elevation of 12.41 feet above the National Geodetic Vertical Datum of 1929 (NGVD) when measured in April 2002. The dam is the tidal limit with a mean high water line immediately below the dam of 4.3 feet NGVD. Visual inspection leads to the conclusion of backwater influences from the Main Street Dam extending to approximately 300 feet upstream from Broad Street.

The Rippowam River watershed is a mix of residential and urban landscapes. The upper watershed is large lot, single family residential with topography dominated by rolling hills that have average slopes of about 30 feet per mile. The lower half of the watershed, where the study is focused, is primarily flatter topography with an average slope of 15 feet per mile. The lower watershed begins approximately at Cold Spring Road, and is the area most prone to flooding.

The lower reach of the Rippowam River has three principal tributaries: Poor House Brook, Haviland Brook, and Toilsome Brook. All tributary confluences are beyond the extent of the study area with the exception of Toilsome Brook that enters across from Scalzi Park. The lower Rippowam River's discharges are significantly influenced by a series of five water supply reservoirs in the upper watershed. The typical domestic water supply yield of the reservoir system is about 15 million gallons per day (mgd), or about 60% of the estimated average annual runoff in the upper watershed (USACE, 1985).

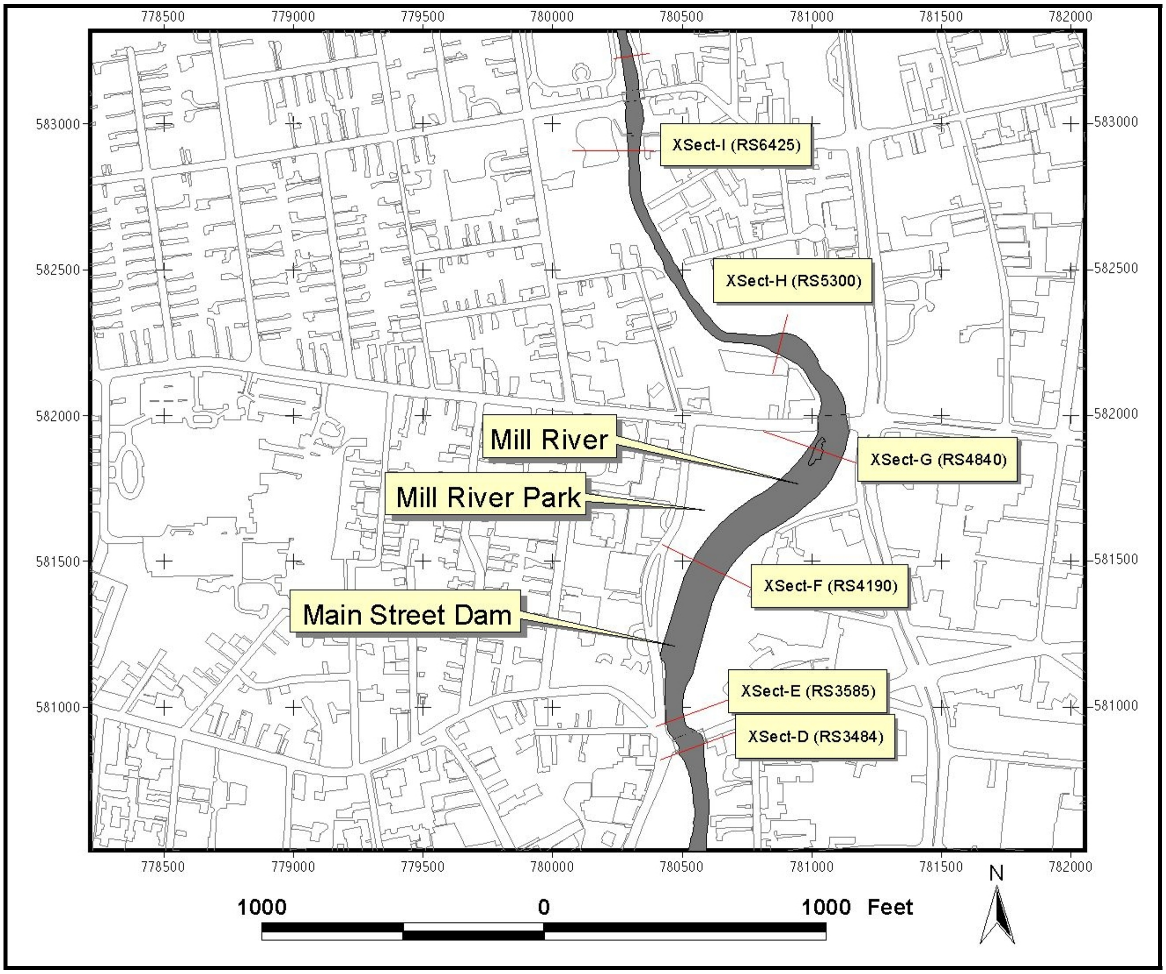


Figure 1. Map of Study Area

b. <u>Mill Pond</u>

Mill Pond is the backwater portion of the Rippowam River within Mill Pond Park, and is formed by Main Street Dam. The park, located entirely downstream of Broad Street, is approximately 9 acres, and the pond within the park is about 3.5 acres (140 feet wide by 1100 feet long), with depths ranging from 1 to 5.5 feet. The pond is constrained the full length of the park within concrete walls approximately 15 feet high. The pond appears to be experiencing accelerated eutrophication, as evidenced by abundant algae and aquatic vegetation.

c. Main Street Dam

Main Street Dam is a concrete dam approximately 138 feet across, about 2 feet wide at the top, and 9.3 feet high. The crest elevation is approximately 12.5 feet NGVD. The dam has a notch on the left side (looking downstream) that is about 0.5 feet deep by 1.5 feet wide. The dam is the tidal limit with a mean high water line immediately below the dam at 4.26 feet NGVD. There is a 4-foot steel intake gate on the right side that is inoperable. The dam is in need of repairs, many of which were documented with the State of Connecticut in 1988.

The Main Street Dam was constructed in 1922 by the Ambursen Construction company, in a style named after the company. The first dam was built on the site in 1641 for a grist mill. The dam is flat-buttressed with an inclined, vertical slab on the upstream side (USACE 1985).

d. Climatology

Average annual precipitation for the region is 45.9 inches, established from 87 years of record through 1979. The greatest recorded monthly precipitation was 17.2 inches in October 1955 resulting largely from a tropical storm. The area commonly experiences intense tropical storms. Mean annual temperature is approximately 50°F with a range from –22° to 104° F recorded for the period from 1892-1979. Average annual snowfall is approximately 34 inches, generally occurring from December through March.

4) STUDY PROCEDURES AND RESULTS

a. Restoration Alternatives

(1) Alternative 1 - No Action

This scenario is evaluated primarily to provide a baseline that the other restoration alternatives could be compared to. Under this scenario, the pond needs to be dredged periodically to prevent it from being filled with sediments.

(2) Alternative 2 – Dam and Wall Removal

This alternative includes removal of the 140 feet long and 9.3-foot high concrete Mill River Dam and removal of the concrete retaining walls within Mill Pond Park. The river channel is regraded under this alternative, with sediments dredged from beneath Mill Pond prior to the dam's removal. An appropriate channel geometry is established, and the river channel "blended" with the overbank (floodplain) areas to enable recreational access to the river, and to restore a more-natural riverine setting.

(3) Alternative 4 – Wall Removal Only

This alternative includes the removal of the cement retaining walls (excepting approximately 75 feet of wall immediately upstream of the Main Street Dam) within Mill Pond Park. Under this scenario, the pond needs to be dredged periodically to prevent it from being filled with sediments. The banks are regraded to slopes that provide recreational access to the impoundment and geotechnical stability.

b. Discharge Frequencies

Modeled floodplain conditions were based upon the 1993 FEMA analyses that provided discharges for the 10, 50, 100, and 500-year recurrence intervals. As limited stream flow data are available for the study area, a comparative statistical analysis was performed for six US Geological Survey gaging stations in the region to establish peak discharge frequencies for the Rippowam River using a log Pearson Type III distribution. Average parameters were developed for the comparative systems and adjusted based on discharge-drainage area relationships. Design flows (Q average daily, 1-year frequency discharge) were determined from gaging data for the Rippowam River available for 1977 through 1982 from a U.S. Geological Survey gage (01209901) located near the Bridge Street bridge (drainage area = 34.0 square miles). Flood frequency data for the 2-year event was used as a design event to determine bankfull channel geometry and for sediment transport analyses (City of Stamford 2001).

Table 1: Rippowam River Discharge Frequencies (cfs)

$Q_{\text{av. daily*}}$	Q1*	Q2+	Q10	Q50	Q100	Q500
45.2	1100	1800	2900	5800	7400	9300
* from 1977-1982 gauge data, + data from WMC Report 2001, all other from FEMA 1993						

c. HEC-RAS Hydraulic Analyses and Water Surface Elevations

The HEC-2 hydraulic model used in the preparation of FEMA's 1993 Flood Insurance Study (FIS) for the City of Stamford, Connecticut, was converted to HEC-RAS format for use in evaluating the hydraulic aspects of the identified alternatives. Comparison of channel bottom elevations from the HEC-RAS model to recently-obtained (2002) sub-sediment and pond bottom elevations leads us to conclude that the <u>HEC-RAS model reflects a limited level of sediment deposition on the pond bottom</u>, although not nearly as much as with present conditions. Therefore, while it is assumed that the HEC-RAS model represents a "without-project" scenario that includes periodic dredging, it is assumed that this scenario, averaged over time, is one with some sediment buildup, as is believed the case when cross-sections were first surveyed for use in preparation of the FEMA Flood Insurance Study for the City of Stamford.

The HEC-RAS model calculated flood elevations somewhat different than those published in FEMA's FIS (using the HEC-2 model) despite use of the same cross-sections, roughness coefficients, flows, and starting water surface elevations (starting elevations were the stillwater tidal flood elevations associated with the recurrence interval being examined). In the 1500-foot reach impacted by Main Street Dam, the HEC-RAS model calculated 100-year peak water surface elevations 0.5 to 0.9 feet lower than those published. Investigation as to the reason for this discrepancy indicated the primary cause to be an incorrect HEC-2 model coding of the Main Street Bridge, located just downstream of the Main Street Dam, resulting in the calculation of erroneously-high flood levels along Mill Pond. This error was corrected in the HEC-RAS model. Additional small differences in flood elevations may be due to the different computational methodologies used by HEC-RAS and HEC-2.

The HEC-RAS model is nevertheless believed calibrated. The HEC-RAS model of Alternative 1 (the without-project scenario) calculated a peak 100-year elevation just upstream of Main Street Dam of 19.4 feet NGVD. This is in exact agreement with the high watermark of 19.4 feet NGVD, measured just upstream of the Main Street Dam during the October 1955 flood, an event attributed to have a 100-year recurrence interval (U.S. Army Corps of Engineers, June 1985). Calculated water surface elevations for Mill Pond under an average daily flow scenario provided a pond elevation of 12.75 feet NGVD29, which closely matched the pond elevation of 12.41 feet NGVD29 measured in April 2002. Modeling of non-flood flows (the average daily flow) also indicated backwater impacts from the Main Street Dam extending 1500 feet upstream of the dam, which corresponds well with observed conditions. It is therefore believed that the HEC-RAS model may be used with confidence for purposes of comparing the hydraulic impacts of the various alternatives.

Hydraulic modeling of other alternatives was accomplished by changing the geometry of the converted HEC-2 model (of the without-project scenario). Hydraulic modeling of Alternative 2 was accomplished by removing Main Street Dam and retaining walls, changing the river bottom between the location of the (removed) dam and the Broad Street Bridge to reflect dredging in that reach, and making additional minor changes to the geometry to allow a maximum slope of 3H on 1 V from the river to the floodplain. Hydraulic modeling of Alternative 4 involved only removal of the retaining walls along Mill Pond Park and other minor changes to the geometry. The river bottom was considered to be the same as with Alternative 1, i.e. although the pond may be periodically dredged, the scenario, averaged over time, is one with some sediment buildup.

Table 2 provides peak water surface elevations at various cross-sections for Alternatives 1 (the without-project scenario) and 2 (the dam/wall removal scenario), and lists the differences in the peak water surface elevations.

Table 2. Peak Water Surface Elevations at Cross-Sections in the Vicinity of Mill Pond for the With- and Without-Dam Scenarios

River Station	**Flow**	**Alternative 1 – No Action**	**Alternative 2 – Remove Dam and Walls**	**Difference in water surface elevation**
	(cfs)	**(feet, NGVD)**	**(feet, NGVD)**	**(feet)**
6425 (x-sec I)	45 (av. daily)	13.22	13.18	0.04
	2900 (Q10)	20.42	20.44	-0.02
	5800 (Q50)	23.20	23.16	0.04
	7400 (Q100)	**24.46**	**24.27**	**0.19**
	9300 (Q500)	25.91	25.64	0.27
5300 (x-sec H) located	45 (av. daily)	12.76	10.97	1.79
	2900 (Q10)	17.47	15.86	1.61
	5800 (Q50)	20.62	18.85	1.77

upstream of Broad St	**7400 (Q100)**	**22.25**	**20.65**	**1.60**
	9300 (Q500)	24.04	23.14	0.90
4840 (x-sec G) located downstream of Broad St	45 (av. daily)	12.76	8.71	4.05
	2900 (Q10)	17.08	14.03	3.05
	5800 (Q50)	19.27	16.78	2.49
	7400 (Q100)	**20.26**	**18.01**	**2.25**
	9300 (Q500)	21.12	19.72	1.40
4470	45 (av. daily)	12.75	7.64	5.11
	2900 (Q10)	16.71	12.79	3.92
	5800 (Q50)	18.63	16.12	2.51
	7400 (Q100)	**19.47**	**17.48**	**1.99**
	9300 (Q500)	20.15	19.33	0.82
4190 (x-sec F)	45 (av. daily)	12.75	6.57	6.18
	2900 (Q10)	16.68	12.13	4.55
	5800 (Q50)	18.59	15.68	2.91
	7400 (Q100)	**19.44**	**17.05**	**2.39**
	9300 (Q500)	20.14	18.89	1.25
3890 (50 feet upstream of damsite)	45 (av. daily)	12.75	5.21	7.54
	2900 (Q10)	16.57	11.55	5.02
	5800 (Q50)	18.35	15.21	3.14
	7400 (Q100)	**19.13**	**16.53**	**2.60**
	9300 (Q500)	19.69	18.5	1.19
3585 (x-sec E)	45 (av. daily)	4.96	4.96	0.00
	2900 (Q10)	11.18	11.18	0.00
	5800 (Q50)	14.88	14.88	0.00
	7400 (Q100)	**16.26**	**16.26**	**0.00**
	9300 (Q500)	18.39	18.39	0.00

An examination of the data provided in Table 2 indicates that peak water surface elevations associated with all recurrence intervals of floods will be reduced upstream of Main Street Dam if the dam and the walls along Mill Pond Park are removed. For example, at the 100-year recurrence interval, peak water surface elevations will be lowered by between approximately 2.0 and 2.6 feet between the location of the (removed) dam and Broad Street, located approximately 1100 feet upstream. At FEMA cross-section H, located approximately 330 feet upstream of Broad Street, the reduction in the 100-year flood level would be approximately 1.6 feet, with the reduction in water level dwindling to 0.2 feet at cross-section I, approximately 1500 feet upstream of the Broad Street Bridge. Water levels associated with normal flows, as indicated by the modeling of average daily flows, will be reduced by several feet, especially in the reach extending from the damsite to Broad Street.

While it is recognized that removing the dam will, at times, enable saltwater to travel further upstream than at present, overbank flooding from tidal flood events is not expected to occur. The overbanks along the Rippowam River in the vicinity of Mill Pond are at an elevation of approximately 17-18 feet NGVD with flood tides much lower in elevation (stillwater tide levels are 9.3 feet NGVD, 10.9 feet NGVD, 11.6 feet NGVD for the 10-year, 50-year, and 100-year recurrence intervals, respectively). The reach of river currently submerged by Mill Pond will not experience any tidal fluctuations during normal tides, since the restored river channel will range from approximately 5 – 8 feet NGVD (as compared to a mean spring high water elevation of 4.9 feet NGVD, for example).

The capacity of Mill Pond to reduce flooding, in the short reach downstream of Mill Pond, was evaluated. With a maximum surcharge storage capacity of only approximately 20 acre-feet (equivalent to 0.01 inches of runoff from the 37.5 square-mile watershed), the flood storage capability of Mill Pond is deemed negligible.

Figure 2 shows the peak water surface elevations for the 100-year recurrence interval flood (only) for Alternatives 1 and 2. In addition, the figure shows the change in river bottom elevation between the location of the removed Main Street Dam and Broad Street, a reach that would be dredged prior to the dam's removal.

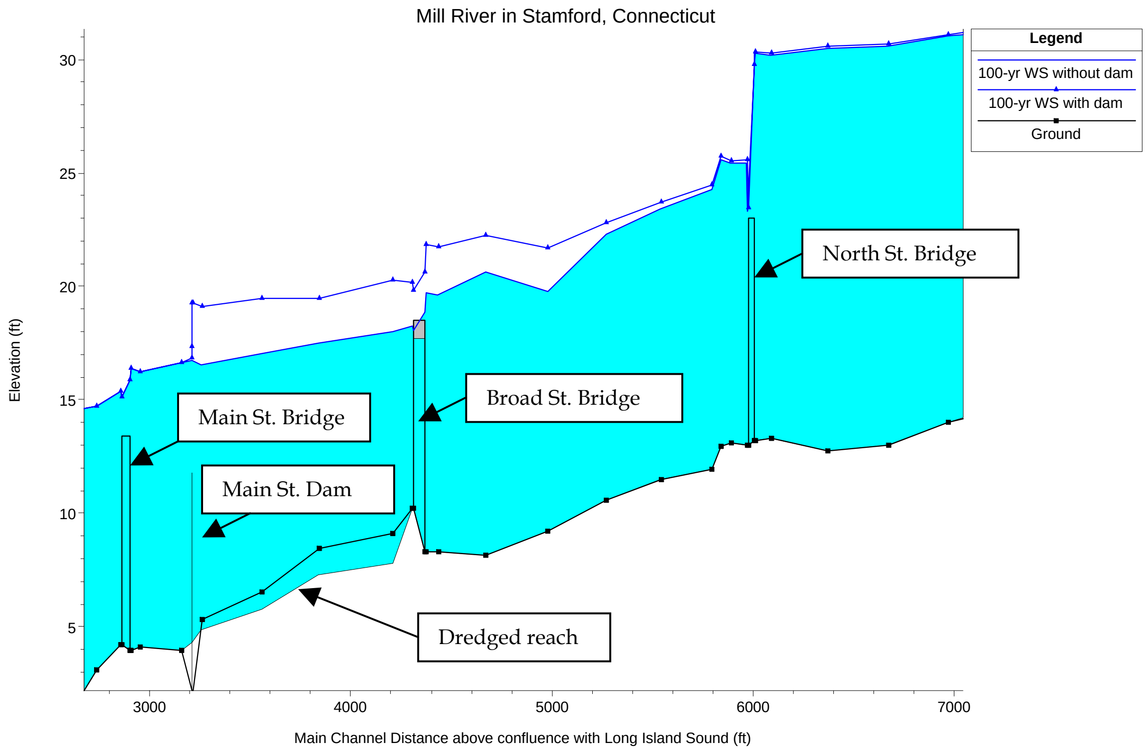


Figure 2. 100-year Flood Profiles for the With and Without Dam Scenarios

Figure 3 shows 100-year floodplain boundaries with Alternative 1 (with Main Street Dam and the retaining walls) and with Alternative 2 (without Main Street Dam and the retaining walls).

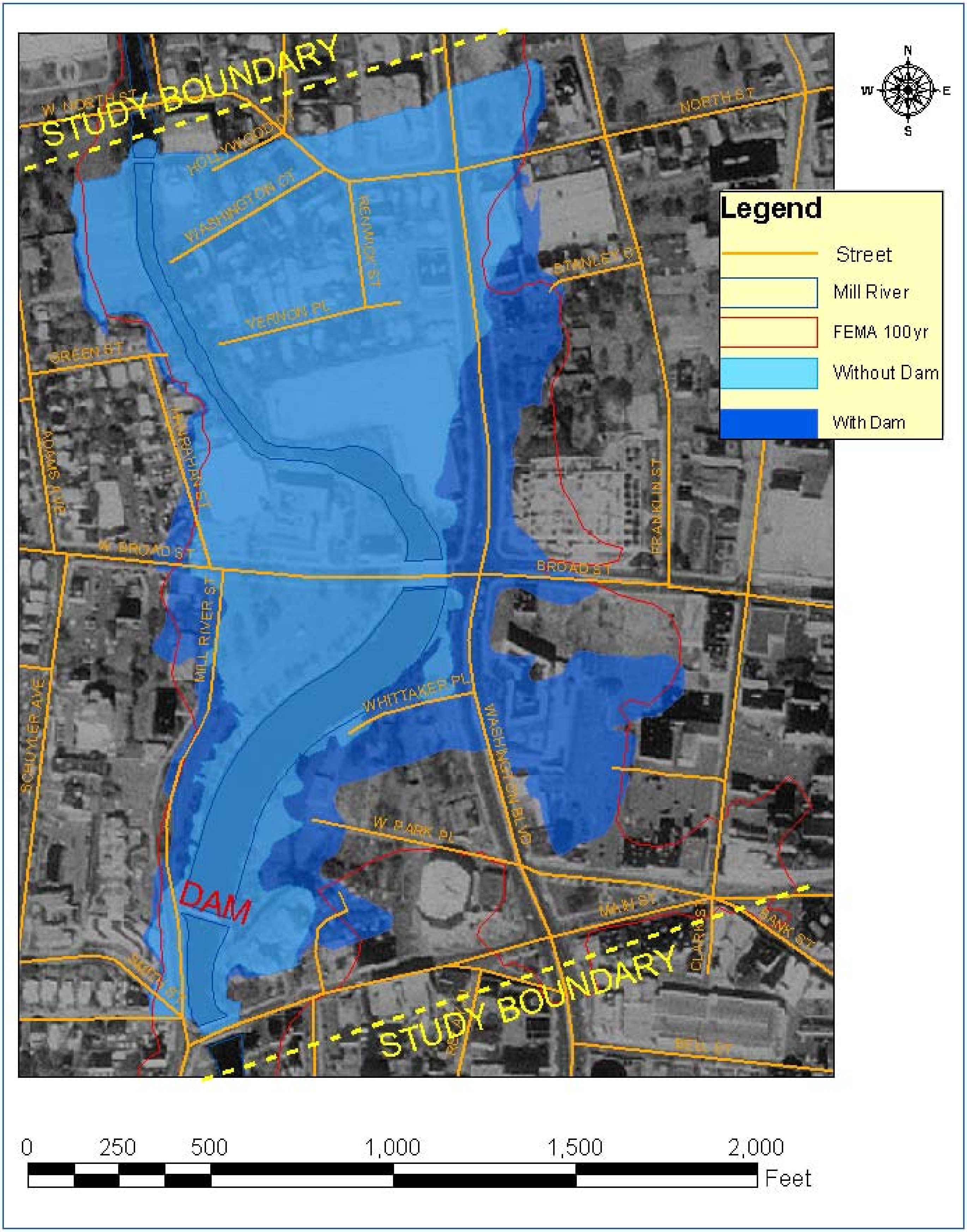


Figure 3. 100-Year Floodplain Boundaries with and without Main Street Dam

Table 3 provides peak water surface elevations at various cross-sections for Alternatives 1 (the without-project scenario) and 4 (the wall removal scenario), and lists the differences in the peak elevations.

Table 3. Peak Water Surface Elevations at Cross-Sections in the Vicinity of Mill Pond for Alternatives 1 and 4

River Station	Flow	Alternative 1 – No Action	Alternative 4 – Remove Walls (only)	Difference in water surface elevation
	(cfs)	(feet, NGVD)	(feet, NGVD)	(feet)
6425 (x-sec I)	45 (av. daily)	13.22	13.22	0.00
	2900 (Q10)	20.42	20.42	0.00
	5800 (Q50)	23.20	23.16	0.04
	7400 (Q100)	**24.46**	**24.37**	**0.09**
	9300 (Q500)	25.91	25.80	0.11
5300 (x-sec H) located upstream of Broad St	45 (av. daily)	12.76	12.76	0.00
	2900 (Q10)	17.47	17.44	0.03
	5800 (Q50)	20.62	20.38	0.24
	7400 (Q100)	**22.25**	**21.86**	**0.39**
	9300 (Q500)	24.04	23.72	0.32
4840 (x-sec G) located downstream of Broad St	45 (av. daily)	12.76	12.76	0.00
	2900 (Q10)	17.08	17.04	0.04
	5800 (Q50)	19.27	19.00	0.27
	7400 (Q100)	**20.26**	**19.80**	**0.46**
	9300 (Q500)	21.12	20.70	0.42
4470	45 (av. daily)	12.75	12.75	0.00
	2900 (Q10)	16.71	16.63	0.08
	5800 (Q50)	18.63	18.21	0.42
	7400 (Q100)	**19.47**	**18.81**	**0.66**
	9300 (Q500)	20.15	19.51	0.64

4190 (x-sec F)	45 (av. daily)	12.75	12.75	0.00
	2900 (Q10)	16.68	16.63	0.05
	5800 (Q50)	18.59	18.21	0.38
	7400 (Q100)	**19.44**	**18.81**	**0.63**
	9300 (Q500)	20.14	19.51	0.63
3890 (50 feet upstream of damsite)	45 (av. daily)	12.75	12.75	0.00
	2900 (Q10)	16.57	16.53	0.04
	5800 (Q50)	18.35	17.97	0.38
	7400 (Q100)	**19.13**	**18.50**	**0.63**
	9300 (Q500)	19.69	19.14	0.55
3585 (x-sec E)	45 (av. daily)	4.96	4.96	0.00
	2900 (Q10)	11.18	11.18	0.00
	5800 (Q50)	14.88	14.88	0.00
	7400 (Q100)	**16.26**	**16.26**	**0.00**
	9300 (Q500)	18.39	18.39	0.00

Examination of the data provided in Table 3 indicates that peak water surface elevations associated with all major floods will be slightly reduced upstream of Main Street Dam if the walls (only) along the Mill River Park are removed. For example, at the 100-year recurrence interval, peak water surface elevations will be lowered by approximately 0.5 feet between dam and Broad Street, located 1100 feet upstream of the dam. At FEMA cross-section H, located approximately 330 feet upstream of Broad Street, the reduction in the 100-year flood level would be approximately 0.4 feet, with the reduction dwindling to 0.1 feet at cross-section I, located nearly 1500 feet upstream of the Broad Street Bridge. Water surface elevations of normal flows, as modeled using average daily flows, would be unchanged by removal of the walls.

Figure 4 shows the peak water surface elevations for the 100-year recurrence interval flood (only) for Alternatives 1 and 4.

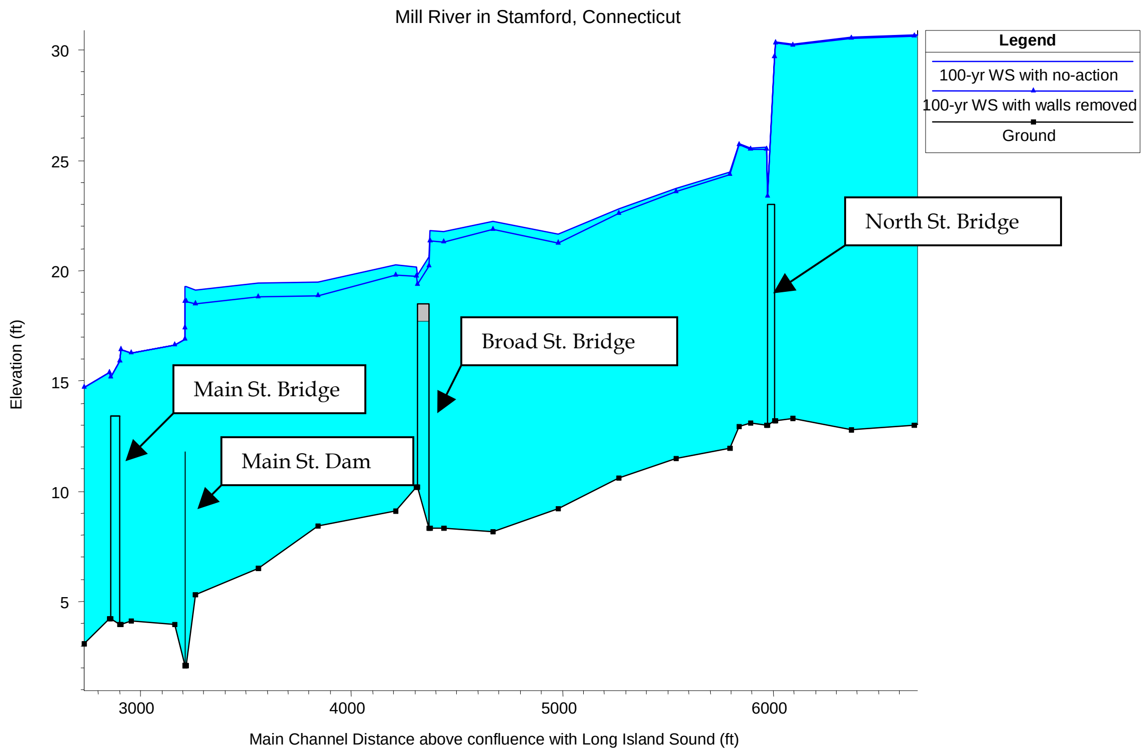


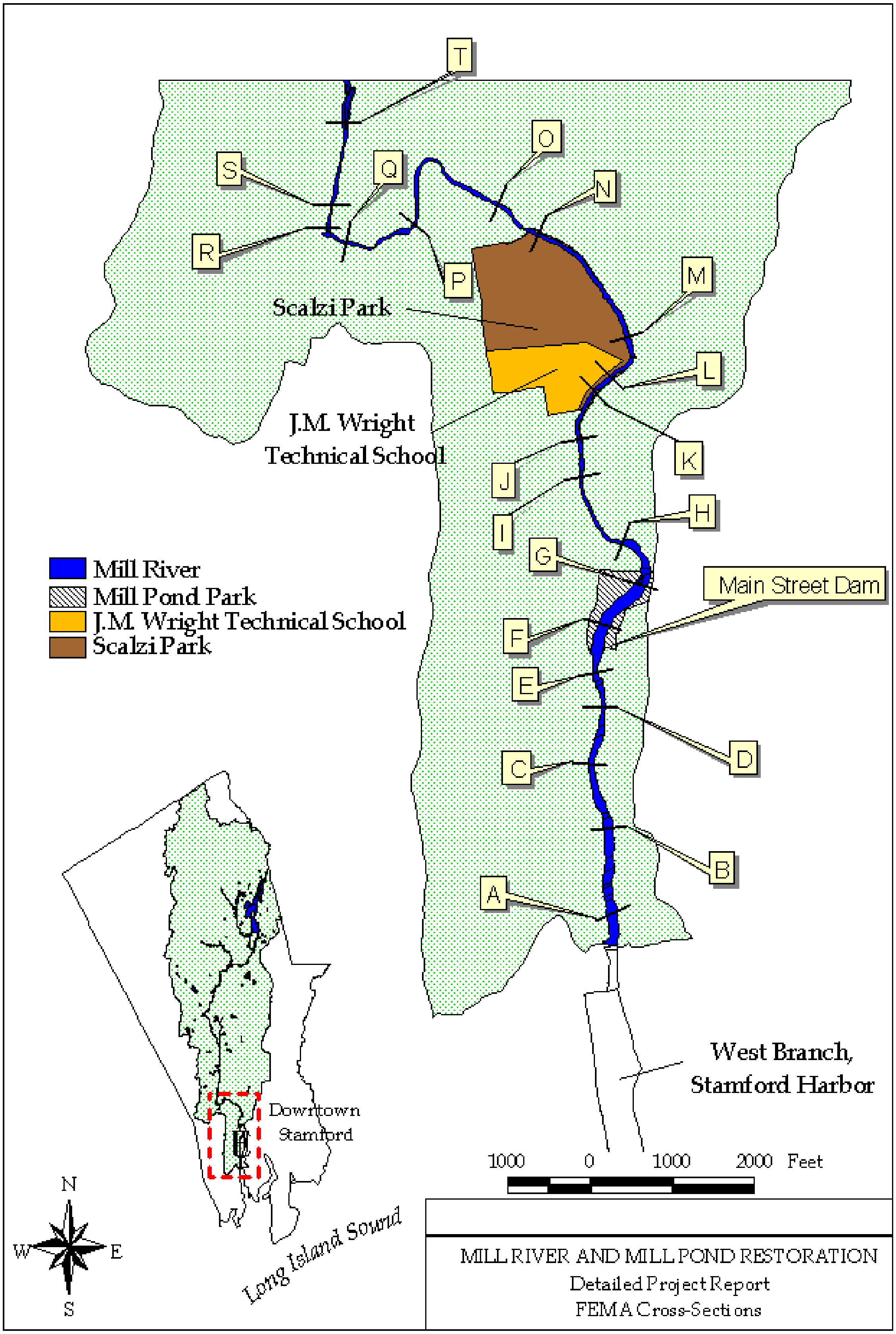
Figure 4. 100-year Flood Profiles for the With and Without Walls Scenarios

An examination of Table 3 and Figure 4 indicates that the role of the retaining walls on peak flood elevations is minimal at the 100-year recurrence interval flood. The walls tend to (slightly) increase peak flood levels within the channel (pond) itself, since the hydraulic conveyance of flood flow is primarily confined to the channel. Under the "with-walls" scenario, floodwaters get into the overbank areas at several locations including at the Broad Street Bridge, the access openings (through the walls) located along the pond, and even over the walls since virtually no freeboard (safety factor) is provided, however, this overbank area conveys little flow. With the retaining walls removed, the conveyance capacity of the overbanks is utilized, thereby lowering peak flood levels slightly. The flood-protective capability of the retaining walls can be considered to be minor.

d. Stream Conditions

Visual observations indicate that siltation is occurring as a result of the backwater influence behind the dam. Fine sediment deposition of as much as 5.5 feet has occurred in the impoundment since it was last dredged to sub-surface sediments. The total estimated volume of fine sediment behind the dam is 18,600 cubic yards based on a comparison of pond bottom elevations to sub-sediment bed elevations (Appendix J).

The particle size distributions determined by pebble counts at the locations of FEMA cross-sections (see Figure 5) are presented in Table 3.



FEMA Cross-Section locations within the Project Area.

Figure 5. FEMA Cross-Section Locations within Project Area

Table 3. Particle Size Distribution at FEMA-Surveyed Cross-Sections

X-sec	D15 (mm)	D50 (mm)	D85 (mm)
13595(S)	0.70	9.5	55
6720(J)	0.90	14	100
6425 (I)	0.30	25	75
5300 (H)	0.005	0.048	0.52
4840 (G)	< 0.05	< 0.05	< 0.05
4190 (F)	< 0.05	< 0.05	< 0.05
3841 (DAM)	< 0.05	< 0.05	< 0.05
3585 (E)	0.25	16	45
3484 (D)	0.20	5.7	50

Inspection of particle sizes from Table 3 indicates that the D50 size of particles for the reach upstream and downstream of Mill Pond is gravel (gravel is defined as particles with diameters between 2 and 64 millimeters). A large and dramatic change in the size of particles occurs between FEMA cross-section I, located approximately 1000 feet upstream of the tailwater of Mill Pond, and cross-section H, located at the approximate tail of Mill Pond (approximately 300 feet upstream of the Broad Street Bridge). At cross-section H, the D50 of sediment particles drops to that of silt (silt is defined as particles with diameters of less than 0.062 millimeters). Sediment samples indicate that the sediments between Main Street Dam and the Broad Street Bridge (between FEMA cross-sections G and H) consist of only silt/clay-size particles. This is consistent with the knowledge that smaller diameter suspended particles do not settle until water velocities drop suddenly, as found in the reach where flow conditions change from that of a relatively fast-moving river to that of a slow-moving pond.

e. Particle Stability and Sediment Transport Analyses

Particle stability was determined by shear stress assessment per American Society of Civil Engineers (ASCE) Manual 54 and U.S. Army Corps of Engineers Engineering Manual "Engineering and Design – Channel Stability Assessment for flood Control Projects" (EM 1110-2-1418). The Shield's parameter was used to determine the particle size that will experience incipient motion (i.e. will begin to move) at the shear forces experienced (Simons et al, 1982). Particles larger than this size are expected to remain settled, while those smaller are expected to be in motion.

The formula employed to determine the maximum particle size for incipient motion is given as follows:

$$D_s = \frac{\tau_c}{0.047(\gamma_s - \gamma_w)}$$

D_s=particle size
τ_c= critical shear stress
γ_s=specific weight of sediment
γ_w=specific weight of water

Table 4 provides the results of use of this equation for the with-dam scenario (Alternative 1). The critical shear stress value used in the equation was that channel shear stress with the dam in place, as calculated by the HEC-RAS hydraulic computer program. The specific weight of sediment is assumed to be 165.4 pounds per cubic foot, with that of water assumed to be 62.4 pounds per cubic foot.

Table 4. Maximum Particle Size for Incipient Motion At Various Flows for Cross-Sections With Main Street Dam In Place (Alternative 1)

X-sec	$Q_{av.\ daily}$	Q1	Q2	Q10	Q100
	Ds (mm)	Ds (mm)	Ds (mm)	Ds (mm)	Ds (mm)
13595 (S)	12.0	41.6	37.8	15.1	90.7
6720 (J)	8.2	39.0	46.0	50.4	32.1
6425 (I)	3.1	18.9	26.4	39.0	87.5
5300 (H)	<1.0	6.3	10.1	15.1	23.9
4840 (G)	<1.0	3.1	5.7	9.4	20.1
4470	<1.0	7.6	13.2	22.0	54.1
4190 (F)	<1.0	1.9	3.8	7.6	21.4
3890	<1.0	1.9	4.4	8.8	29.6
3843 (DAM)	<1.0	<1.0	1.3	3.1	12.0
3585 (E)	3.1	28.3	32.7	36.5	49.7
3484 (D)	1.9	11.3	13.2	14.5	25.8

Examination of the data shown in Table 4 indicates that sand, silt and clay (all particles with diameters less than 2.0 mm) should not settle at any of the cross-sections examined upstream of Mill Pond, since their diameters are smaller than the calculated diameters of incipient motion for all flow scenarios examined, including average daily flow. However, at the tailwater of Mill Pond (at approximately cross-section H), the situation changes dramatically. At average daily flows, it is apparent that sediment transport continuity is disrupted, with virtually all suspended sediments settling in the stilled waters of the impoundment. Stability analyses are consistent with field observations that indicate sedimentation of fines is occurring within the backwater. During a 1-year flood, clay, silt, sand, and even fine gravel begins moving, depending on its location in the pond. During a 100-year flood, the sediment transport of coarse gravel is re-established.

It should be noted that the calculations of diameters of incipient motion for

Alternative 1 (without-project scenario) were made using Mill Pond bathymetry surveyed during the preparation of Stamford's FIS (1993). The pond bathymetry survey conducted in April 2002 indicates that significant sedimentation has occurred since then, with channel bottoms roughly 2 feet higher than those of the FIS. The pond may accordingly have lost much of its capacity to store settled sediments. The impact of this lost capacity is likely to be the passing of more sediment than that indicated by calculations for this alternative 1.

Channel water velocities and shear stresses associated with Alternative 4 (removal of walls only) were found (in the HEC-RAS model) to be virtually the same as those of Alternative 1. Therefore, the statements pertaining to Alternative 1 concerning sediment transport also apply to Alternative 4.

Table 5 provides the results of use of this equation for the without-dam scenario (Alternative 2). The critical shear stress value used in the equation was the channel shear stress without the dam in place, as calculated by HEC-RAS.

Table 5. Maximum Particle Size for Incipient Motion At Various Flows for Cross-Sections With Main Street Dam Removed (Alternative 2)

X-sec	$Q_{av.\ daily}$	Q1	Q2	Q10	Q100
	Ds (mm)	Ds (mm)	Ds (mm)	Ds (mm)	Ds (mm)
13595 (S)	12.0	41.6	37.8	15.1	90.7
6720 (J)	8.2	39.7	45.3	49.7	32.1
6425 (I)	3.8	19.5	25.8	39.0	91.3
5300 (H)	<1.0	15.7	23.9	30.9	35.3
4840 (G)	2.5	27.7	37.8	49.1	78.7
4470	29.0	51.0	61.7	68.4	64.2
4190 (F)	3.8	37.1	44.7	47.9	58.6
3890	34.0	36.5	40.9	42.8	60.4
3843 (DAM)	----	----	----	----	----
3585 (E)	3.1	28.3	32.7	36.5	49.1
3484 (D)	1.9	11.2	13.2	14.5	25.8

Examination of the data shown in Table 5 indicates that sand, silt and clay (all with diameters less than 2.0 mm) should not settle at any of the cross-sections with Main Street Dam removed even with average daily flow, since their diameters are smaller than the calculated diameters of incipient motion. (A single exception occurs with average daily flows at cross-section H, however, this is believed due to an elevated portion of the channel at the downstream side of Bridge Street as can be seen on the flood profiles (see Figure 2 or 3). During a 100-year flood scenario, even coarse gravels are moving at all cross-sections. Comparison of particle sizes between the various cross-sections indicates that virtually all sand, silt and clay particles will be flushed through the entire reach with Main Street Dam removed.

Table 6 provides a comparison of the maximum particle sizes for incipient motion for Alternative 1 (with-dam) and Alternative 2 (without-dam) under the average daily flow scenario, and for the 1-year flood at various cross-sections.

Table 6 – Maximum Particle Size (mm) for Incipient Motion For Alternative 1 (With-Dam) and Alternative 2 (Without-Dam) During Av. Daily Flow and 1-Year Flood

X-sec	Av. Daily Flow	Av. Daily Flow	1-yr flood	1-yr flood
	With Dam	Without Dam	With Dam	Without Dam
13595 (S)	12.0	12.0	41.6	41.6
6720 (J)	8.2	8.2	39.0	39.7
6425 (I)	3.1	3.8	18.9	19.5
5300 (H)	<1.0	<1.0	6.3	15.7
4840 (G)	<1.0	2.5	3.1	27.7
4470	<1.0	29.0	7.6	51.0
4190 (F)	<1.0	3.8	1.9	37.1
3890	<1.0	34.0	1.9	36.5
3843 (DAM)	<1.0	----	<1.0	----
3585 (E)	3.1	3.1	28.3	28.3
3484 (D)	1.9	1.9	11.3	11.2

Incipient motion size and particle distribution analyses indicate that much of the Rippowam River is stable and armored, with the exception of the impoundment. Comparison of incipient motion particle size for Alternative 1 (with-dam) and Alternative 2 (without-dam) in Table 6 illustrates a great improvement in sediment transport capacity in the reach currently submerged by Mill Pond when Main Street Dam is removed, for average flow and 1-year flood flow situations. It is evident that the higher water velocities and shear stresses associated with removal of the dam for this reach will mean that dredging of this reach will no longer be required. Average channel water velocity would increase approximately 2.8 feet/second at the 100-year recurrence interval (from approximately 4.1 feet/second with-dam to 6.9 feet/second without-dam) at a cross-section 375 feet upstream from the damsite. Sediments entering this reach will, instead, continue to Stamford Harbor, where they will settle once water velocities are reduced. A similar with- and without-dam comparison of incipient motion particle size for rarer floods (2-year, 10-year, and 100-year floods) can be performed by examining particle sizes for those rarer floods shown in Tables 4 and 5. Sediment transport capacity within the location of the former impoundment is significantly increased with those less-frequent events as well. The decreased "wetted" area of cross-sections within the former Mill Pond under Alternative 2 results in an increase in water velocities and shear stresses along the bottom of the channel under all flow scenarios examined, thereby enhancing sediment transport under all flow scenarios.

f. Establishment of Channel Dimensions

The establishment of appropriate bankfull channel dimensions is believed critical to insure a geomorphologically-stable channel should Alternative 2 (dam and wall removal) be pursued. The shape of the natural (without-dam) Rippowam River may no longer be obvious, even with the dam removed, due to floodplain encroachment behind the retaining walls constructed along the banks of the Rippowam River in Mill Pond Park. Calculation of appropriate channel size is to be done using established procedures based upon observations of stable natural rivers. In general, bankfull channel cross-section geometry has been shown to correspond with a discharge that has a recurrence interval of approximately 1.5 years in the annual flood series (Dunne and Leopold, 1978). Data for the 1-year and 2-year recurrence interval flood discharges of the Rippowam River were used to estimate appropriate bankfull channel cross-section dimensions of the project area design channel.

Sub-sediment elevations, obtained during a April 2002 field survey, were used to establish bed elevations that reasonably approximate the pre-impoundment channel bottom that could be expected should Alternative 2 be implemented. Cross-section geometry was established based on channel capacity needed to pass a 1.5-year flood, and the channel shape was blended into the topography of the area as shown on local topographic maps. The resulting concept design (see Figures 8 and 9 of the Detailed Project Report for this project) for a geomorphologically-stable channel in the reach above the (removed) dam was developed at a feasibility level of detail. Design-level specifications for the dam-removal scenario are likely to require additional survey with special consideration of concerns such as the buried sewer pipe that underlies the park and impoundment just downstream of the Broad Street Bridge, and the determination of the elevation of stable channel bottom directly beneath the dam and under Broad Street Bridge. FEMA is likely to require that someone (the project sponsor?) establish final revised flood profiles with the dam removed in order that its major flood-reduction benefits are reflected in both the delineated floodplain boundaries and flood elevations. Additionally, since the Flood Insurance Rate Maps also show a floodway, floodway runs will need to be revised too. Once definitive channel and adjacent floodplain geometry is established (including the determination of the elevation of final channel bottom) during preparation of Plans and Specifications, hydraulic modeling may be performed to establish flood profiles, and floodplain and floodway boundaries.

5) HYDRAULICS SUMMARY

HEC-RAS hydraulic model results indicate that established flood elevations would be reduced significantly if Main Street Dam and the walls along Mill Pond Park are removed, and the channel bottom dredged (Alternative 2). For example, at the 100-year recurrence interval, peak water surface elevations will be lowered by between approximately 2.0 and 2.6 feet between the location of the (removed) dam and Broad Street, located approximately 1100 feet upstream. The reduction in the 100-year flood level would be approximately 1.6 feet at the upstream end of the current impoundment (approximately 330 feet upstream of Broad Street) with the reduction in water level dwindling to 0.2 feet at a location 1500 feet upstream of the Broad Street Bridge. Water levels associated with normal flows, as indicated by the modeling of average daily flows, will be reduced by several feet, especially in the reach extending from the damsite to Broad Street.

HEC-RAS model results indicate that peak water surface elevations associated with all major floods will be reduced by only a small amount upstream of Main Street Dam if the walls (only) along the Mill River Park are removed (Alternative 4). For example, at the 100-year recurrence interval, peak water surface elevations will be lowered by approximately 0.5 feet between dam and Broad Street, located 1100 feet upstream of the dam. The reduction in the 100-year flood level would be approximately 0.4 feet at the upstream end of the impoundment, with the reduction in water level dwindling to 0.1 feet at a location 1500 feet upstream of the Broad Street Bridge. Water surface elevations of normal flows would be unchanged by removal of the walls.

Hydraulic analysis of Alternative 1 (with-dam) indicates that during average daily flows, sands, clays, and silts should pass through upstream reaches of the Rippowam River, but settle in the stilled waters of Mill Pond impoundment, as confirmed by field observations. Hydraulic analysis of Alternative 2 (removing the Main Street Dam) will enable sediment transport to revert to its natural cycle, with sand, fines, and clay largely passing through the former impoundment without settling. In that case, the channel bottom can be expected to resemble that of the reference reaches upstream of Mill Pond, where sediments consist largely of gravel, and the channel is self-maintaining. Channel water velocities and shear stresses associated with Alternative 4 (removal of walls only) were found (in the HEC-RAS model) to be virtually the same as those of Alternative 1. Therefore, statements pertaining to Alternative 1 concerning sediment transport are also applicable to Alternative 4.

6) REFERENCES

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