Attachment 9

Hydraulic Report

Blackstone River
Fish Passage Restoration Project

Hydrologic and Hydraulic Modeling of the Blackstone River, Rhode Island

Prepared for

Natural Resources Conservation Service

NRCS

Prepared by

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March 2009
Version: Revision 10
EA Project No.: 62130.12

CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ACRONYMS

EA	EA Engineering, Science, and Technology, Inc.
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FIS	Flood Insurance Study
HEC-RAS	Hydraulic Engineering Center-River Analysis System
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
PLS	Professional Licensed Surveyor
RIDEM	Rhode Island Department of Environmental Management
USACE	U.S. Army Corps of Engineers
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
VA	Voest-Alpine

EXECUTIVE SUMMARY

Fish passage facilities are being proposed for the first four dams on the Blackstone River in Pawtucket and Central Falls, Rhode Island. The dams, starting from the most downstream location, are Main Street Dam, Slater Mill Dam, Elizabeth Webbing Dam, and Valley Falls Dam. A hydrologic and hydraulic analysis of the lower reach of the Blackstone River was conducted to determine the impacts of installation of fish passage facilities at the first four dams on flood levels. Denil fish ladders are being proposed for Main Street, Slater Mill, and Valley Falls Dams. Dam removal or a Denil fish ladder may be proposed for Elizabeth Webbing Dam.

A computer model using the Hydraulic Engineering Center-River Analysis System (HEC-RAS) program was developed and used to simulate flood levels for the 2-, 10-, 25-, and 100-year storm events. The 10- and 100-year flood flows identified in the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) were used for the HEC-RAS simulations, as well as to define the 2- and 25-year flood flow values. Simulation results with the fish passage facilities were compared to simulation results for the existing dams to evaluate impacts that could be expected after installation of the fish passage facilities. Dam removal and installation of a Denil fish ladder at Elizabeth Webbing Dam have been included in the analysis to bracket the range of potential impacts on flood levels.

The HEC-RAS simulations indicate that the proposed fish ladder at Main Street Dam will increase flood levels between Main Street Dam and Slater Mills Dam by as much as 0.7 ft above the flood levels for the existing conditions for all of the storm events. However, downstream of Slater Mill Dam, all of the simulated flood levels with the Main Street Dam fishway would be contained in the existing river channel or are lower than the FEMA flood levels. The proposed fish passage facilities at Slater Mill Dam and Valley Falls Dam would not increase flood levels above the levels for the existing conditions.

The Main Street Dam fishway would increase river water depths and velocities under the bridge. Water depths could increase by as much as 1.8 ft and velocities by as much as 2.6 ft/sec. However, an analysis of the forces on the bridge pier indicates that increased water depths and velocities would not reduce the sliding and overturning factors of safety of the bridge pier to unacceptable values. Expected shear stress with the higher velocity would be less than the permissible shear stress for the bridge pier stone blocks and bedrock foundation.

If Elizabeth Webbing Dam is removed, water levels at the dam location would be 5.1-8.2 ft lower than the existing conditions for all of the storms. Channel velocities at the location about 600 ft upstream of the dam would require channel armoring and bank stabilization to prevent channel bed undercutting after removal of the dam. A shear stress analysis of the channel bed and side bank conditions indicate that the fish passage facilities would not cause erosion at other river reaches downstream from Valley Falls Dam.

1. INTRODUCTION

Fish passage in the lower Blackstone River is currently obstructed by the first four dams and natural falls created by bedrock falls at the first dam on the river. Preliminary surveys by state and federal fisheries biologists have found suitable habitat and conditions for river herring (blueback herring and alewife), American shad, and Atlantic salmon in the Blackstone River. The Natural Resources Conservation Service (NRCS) has entered into contracts with three (3) of the dam owners to install fish passage through or adjacent to their dams. The three facilities are: Main Street Dam (Pawtucket Hydro Power), Slater Mill Dam (Historic Slater Mill), and Valley Falls Dam (Blackstone Hydro Power). The third dam on the river, Elizabeth Webbing Dam, which is located between Slater Mill Dam and Valley Falls Dam, is not under NRCS contract at this time, but is included in the hydrology and hydraulic analysis of the river.

This report, prepared by EA Engineering, Science, and Technology, Inc. (EA), presents the development and analysis results from the hydrology and hydraulic models that were used to determine the design parameters for the three fish passage projects at Main Street Dam, Slater Mill Dam, and Valley Falls Dam. The results presented in this report were also used to determine any impacts of the fish passage facilities on power generation at the existing Main Street and Valley Falls Dams hydroelectric plants, and at future hydropower generation facilities that may be installed at Slater Mill. The analysis of the fishway impacts on the hydroelectric projects is presented in separate reports.

2. EXISTING PROJECT FEATURES

A brief summary of the pertinent features associated with each of the first four dams on the Blackstone River is presented in the following sections. All elevations referenced in this report are based on National Geodetic Vertical Datum 1929 (NGVD 1929) datum.

2.1 MAIN STREET DAM

Pawtucket Hydropower, LLC, originally constructed as the Pawtucket No. 2 Hydroelectric Project, is located at Main Street Dam in Pawtucket, Rhode Island and has a Federal Energy Regulatory Commission (FERC) exemption from license. The project, which is FERC Project No. 3689-RI, has periodically operated since 1896 and is currently owned and operated by Putnam Hydropower, Inc. Main Street Dam is located at the mouth of the Blackstone River in downtown Pawtucket, Rhode Island. The Seekonk River is downstream of the dam and is tidal with normal tides ranges of about 4.5 ft. The Blackstone River drainage area at the dam is 474 sq. miles.

The Pawtucket Hydropower project consists of: (1) a 180-ft long mortared brick, gravity dam, (2) a 17.5 ft diameter, 130-ft long, brick lined intake conduit, (3) a concrete intake structure with a mechanical trash rake, (4) two 9.5-ft diameter, 150-ft long, steel penstocks, (5) two hydraulic turbines, and (6) a 55-ft long concrete lined tailrace. The power facilities are located within the refurbished mill as shown on Figure 1.

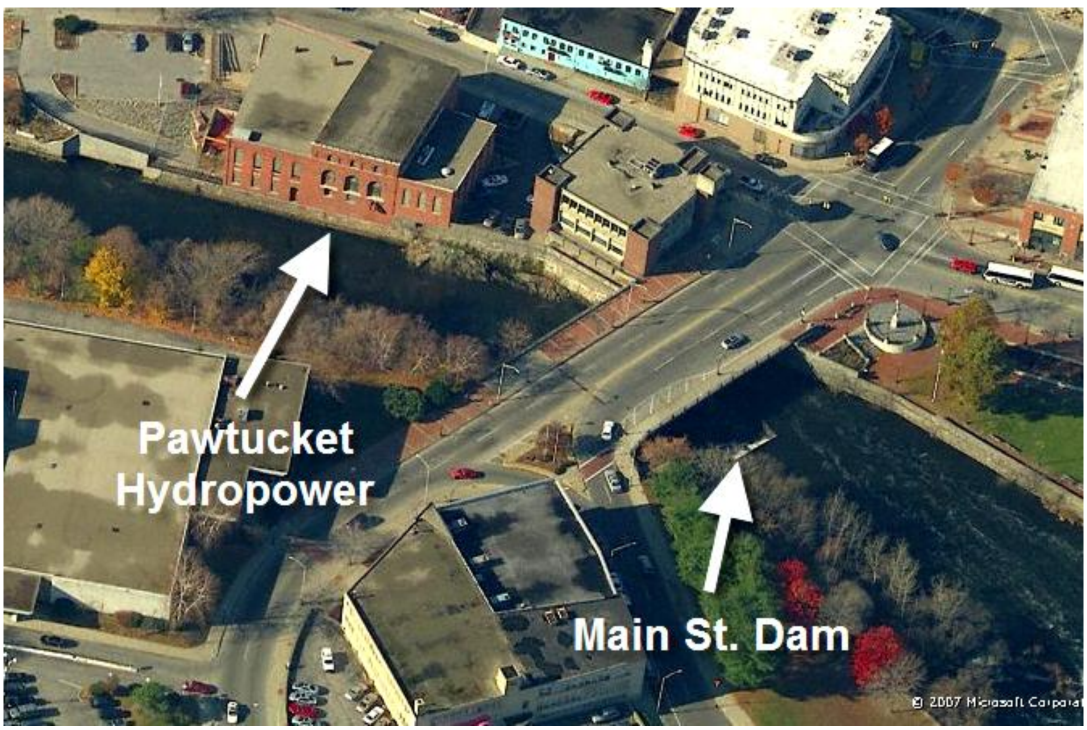

Figure 1 - Aerial Photograph of Main Street Dam
(Note: River flow is from right to left)

A general arrangement of the dam and power facilities is presented on Figure 2 and a centerline profile through Unit 2 is shown on Figure 3. The dam is located at the Main Street Bridge with the power intake located on the right abutment under the bridge. The dam is an overflow spillway with a wood timber batter board facing on the upstream side and a timber top sill with the crest at El. 17.1 ft (NGVD 1929), as shown on Figure 4. The dam is approximately 7 ft high, but sits on natural bedrock ledge which creates a total drop of about 16.5 ft from the top of spillway to the river bottom downstream of the dam. The impoundment has a surface area of 1.1 acres.

The left abutment of the dam ties directly into the granite block abutment of the bridge. A 30 ft long non-overflow masonry wall extends from the right abutment of the dam to the right abutment of the bridge. The 17.5 ft diameter intake conduit for the hydroelectric project is formed in this masonry wall. The intake opening originally had a trash rack which was abandoned due to the inability to clean debris. Access to the intake opening is limited through a 3 ft square access opening in the wall. The intake conduit inlet has a steel plate curtain wall over the opening above El. 16.0 ft.

The intake conduit extends about 150 ft downstream from the dam to the powerhouse forebay. The conduit is located under Main Street and the building adjacent to the river. The forebay is approximately 50 ft long by about 65 ft wide and is located between the building at the corner of Main Street and the hydroelectric project powerhouse. There are 5 bays, about 10 ft wide, located at the downstream end of the forebay. Bays 3-5 have trash racks with a total width of about 30 ft. The trash rack flow channel transitions into Bays 4 and 5 which each have headgates located at the inlets to two 9.5 ft diameter penstocks, one for each unit. Bays 1-3 have been plugged with concrete. The penstocks convey water to two hydraulic turbines, as shown on Figure 3.

The turbines are Kaplan type units (double regulated with wicket gates and adjustable blades), standardized tube type turbine manufactured by Voest-Alpine AG (VA). The turbine hydraulic capacity ranges from about 130 cfs with one unit operating at minimum load up to 1,400 cfs with both units operating at maximum load under low tide and high river flow conditions. The net head range is approximately 11.9-16.5 ft depending on the river flow and tide level. Each unit operates at 200 rpm and has a 4,160 v generator. Both turbines have a 1.9 m diameter runner with a capacity of 750 kw at 600 cfs flow and 16.5 ft head. The combined rated capacity of the units is 1,500 kw. The turbines are currently operated as an instantaneous run-of-river project with a minimum discharge over the dam of 50 cfs or river inflow, whichever is less. When the turbines were licensed, the expected long term average annual generation by the two units was approximately 6,200,000 kwh.

The draft tubes for the units discharge into concrete tailrace chambers, one for each unit. These chambers are pressurized under all tide conditions and are each about 11 ft wide. These tailrace chambers extend about 25 ft downstream from the draft tubes gates where the tailrace becomes single concrete conduit that is about 23 ft wide extending 40 ft downstream to the tailrace exit at the river. The top of the tailrace conduit is at El. 3.74 ft, which allows the tailrace to be a free surface, open channel over all tide levels except the top 1 ft of each high tide.

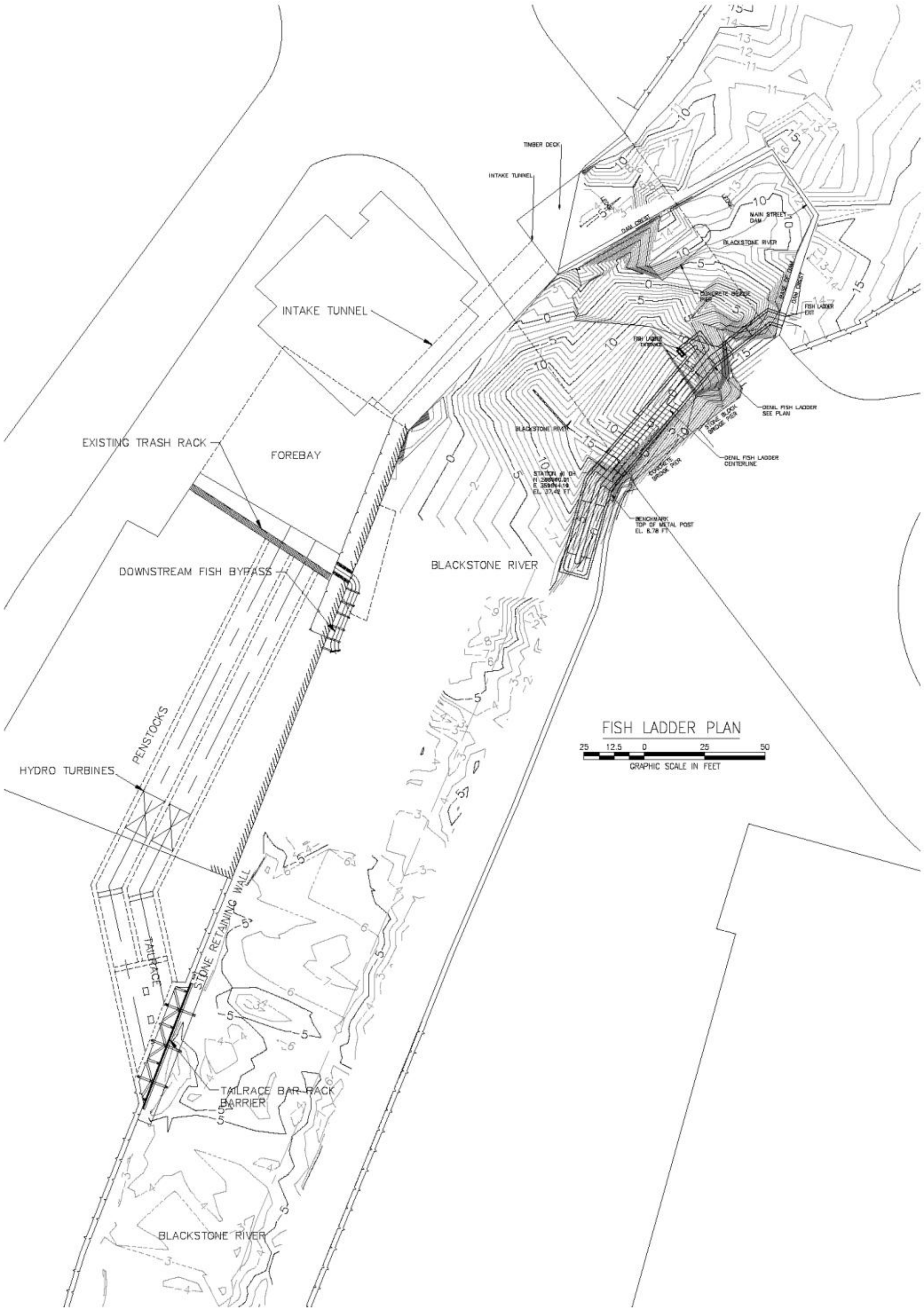

Figure 2 - General Arrangement of Pawtucket No. 2 Hydroelectric Project
(Flow from top right to bottom left)

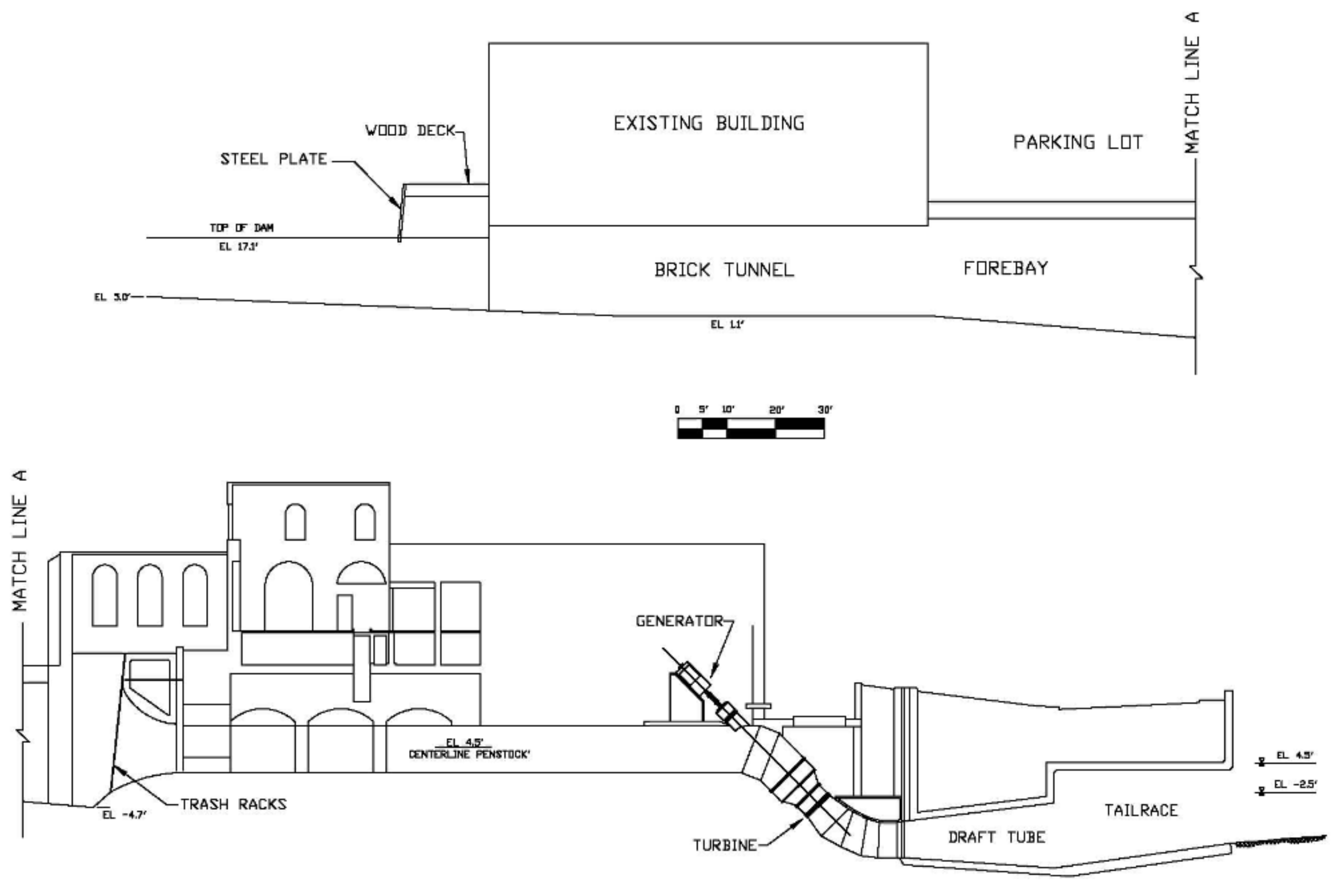

Figure 3 - Profile Through Pawtucket No. 2 Hydroelectric Project Turbines

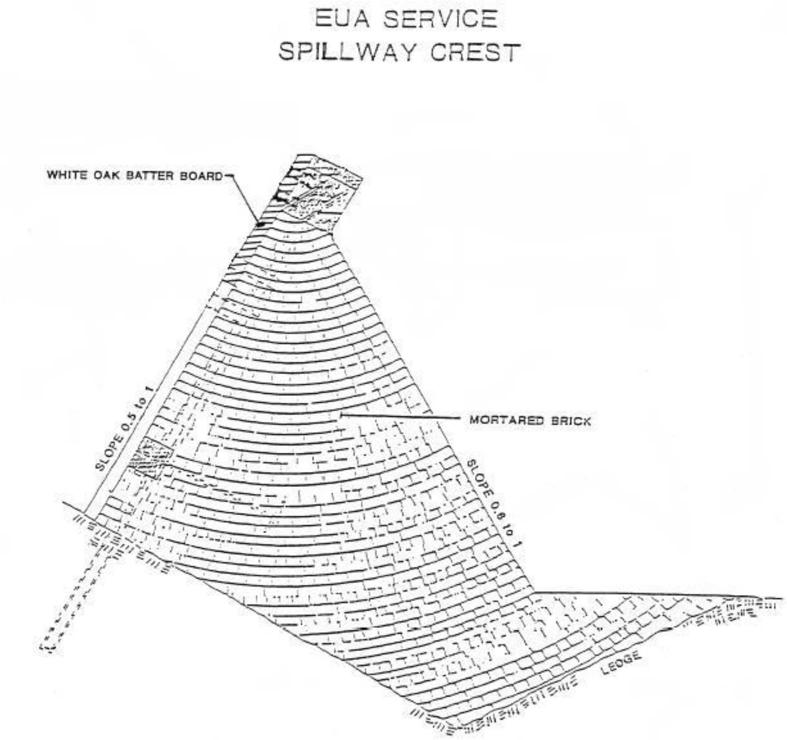

Figure 4 – Typical Cross Section of Main Street Dam
(Flow is from left to right)

2.2 SLATER MILL DAM

The Slater Mill Dam is located approximately 300 feet upstream of the Main Street Dam (see Figure 5). Slater Mill was built in 1793 and served as the first commercially viable cotton-spinning mill in the United States used to convert raw cotton into cloth. Slater Mill is generally cited as the birthplace of the Industrial Revolution in America, thus having a significant historical value. Therefore, since maintaining the integrity of Slater Mill Dam is imperative from a historic perspective, the Blackstone River Fish Passage Restoration Project is considering a fishway located near the southeast abutment on the opposite site of the dam from Slater Mill.

The dam is currently jointly owned by Blackstone Valley Electric Company and Slater Mill Association. The Blackstone River drainage area at Slater Mill Dam is 474 sq. miles and the impoundment surface area is 11 acres. The dam is a 192-ft long overflow gravity spillway with a timber batter board facing on the upstream side with timber truss supports. The dam is about 7 ft high with the crest varying in from El. 23.47 ft at the left abutment (looking downstream) to El. 22.78 ft at the right abutment. The dam crest average level is at El. 23.1 ft. The abutments are cut stone and mortared walls with the mill located on the northwest side of the dam (right abutment looking downstream). A typical cross section of the dam is presented on Figure 6.

The hydroelectric turbines in the mill have not been in operation for more than 20 years. The entrance to a power canal was originally located about 70 feet upstream of the spillway, but has been filled in with only a 4-ft diameter pipe connecting the canal to the river. The power canal extends under the mill building adjacent to the dam and originally extended downstream of the Main Street Dam. The canal originally supplied water to a number of turbines in mills located along the canal from the dam to below the Main Street Dam. All of the mills have been demolished except for the two building at the dam which are owned and maintained by the Slater Mill Historic Site. The power canal currently terminates about 100 ft downstream of the building over the canal (Figure 5). A park has been created from the end of the canal to Main Street.

Only two turbines remain at the Slater Mill Historic Site in the building over the canal and one mid-breast water wheel in the building at the current end of power canal. These turbines have been abandoned and the water wheel is only operated at minimal flow generally less than about 450 gpm (1 cfs) for demonstration purposes. The tailrace from the mill is located about 45 feet downstream from the spillway and conveys the water wheel demonstration flow and leakage from the power canal back to the river.

Slater Mill Historic Site is in the process of completing a feasibility study that will assess alternatives for restoring hydroelectric power and preserving the two hydraulic turbines in the mill building adjacent to the dam. The assessment will identify repairs needed to preserve, restore, and/or replace the existing Jonval and Francis turbines in this building only. However, the Slater Mill Historic Site has not decided on a course of action for the Turbine Rehabilitation.

Project changes to reflect future hydroelectric development at Slater Mill are not included in this report.

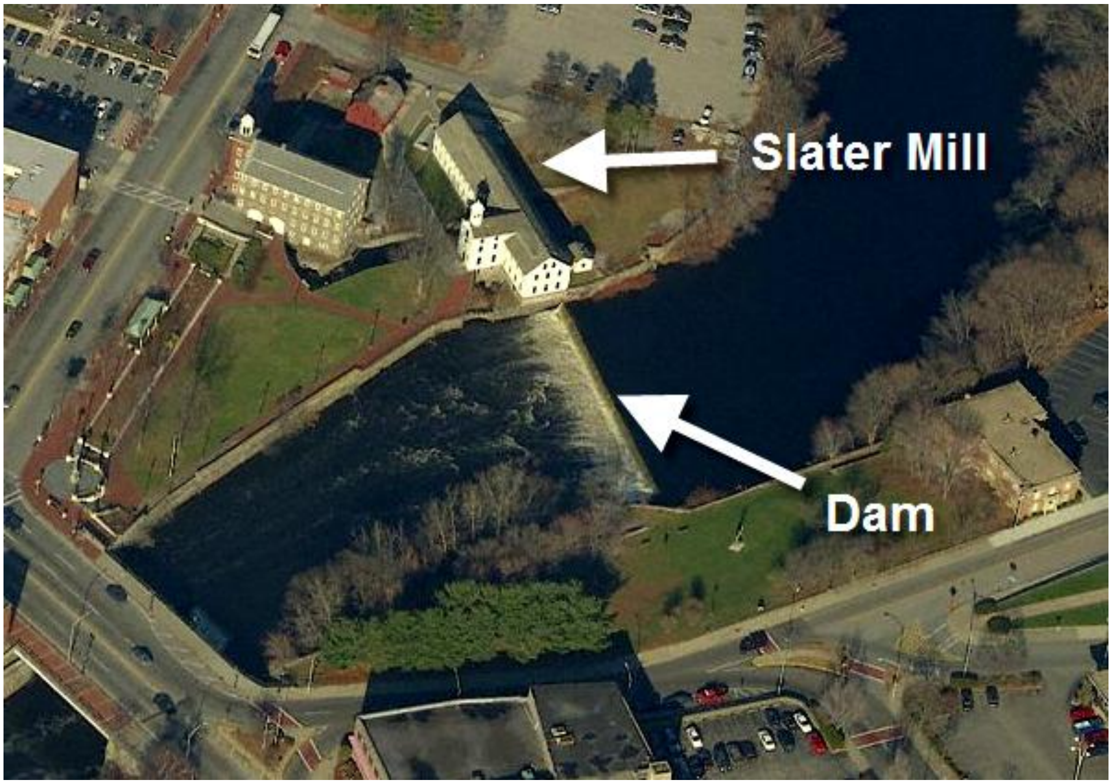

Figure 5 - Aerial Photograph of Slater Mill Dam
(Flow is from right to left)

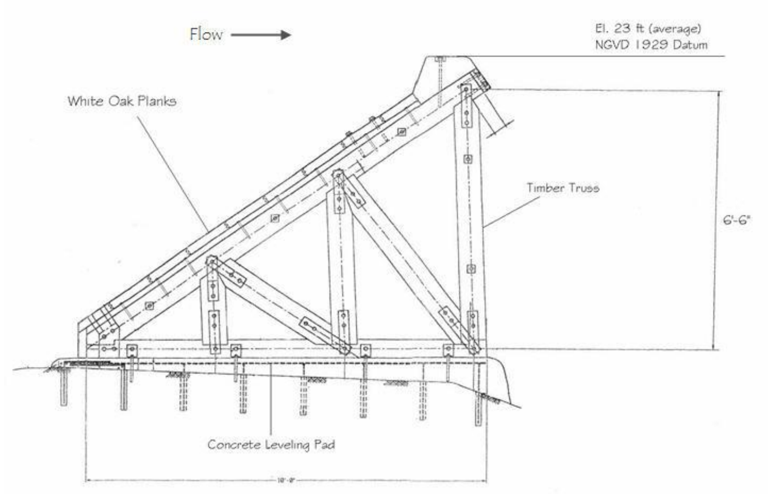

Figure 6 - Typical Cross Section of Slater Mill Dam

2.3 ELIZABETH WEBBING MILLS DAM

The Roosevelt Hydroelectric Project is located at Elizabeth Webbing Mills Dam, but has not operated since 2001. The new mill property owner (Tai-O Associates, L.P.) does not intend to operate the plant and is in the process of turning the hydroelectric project property over to the Rhode Island Department of Environmental Management (RIDEM). Tai-O Associates and RIDEM would like to improve the aesthetics of the site by removing the hydroelectric project powerhouse. RIDEM is partnering with NRCS to restore fish passage in the Blackstone River. Demolition of the hydroelectric project structures could be an integral part of the river restoration, especially if removal of Elizabeth Webbing Mills Dam is the most-cost effective fish passage alternative.

The Roosevelt Hydroelectric Project at Elizabeth Webbing Mills Dam consisted of: (1) a 220-ft long rock fill, gravity, earth dam, (2) a concrete intake structure with a mechanical trash rake, and (3) a 65-ft long concrete intake canal, a concrete powerhouse, and a 45 ft long concrete tailrace. The dam crest is at El. 34.9 ft. The hydroplant was licensed for flashboards with a top level at El. 35.53 ft. Figure 7 shows an aerial view of the dam and powerhouse. The powerhouse has been locked, the turbine has not operated for the last 6 years, and the unit has been disconnected from the power grid.

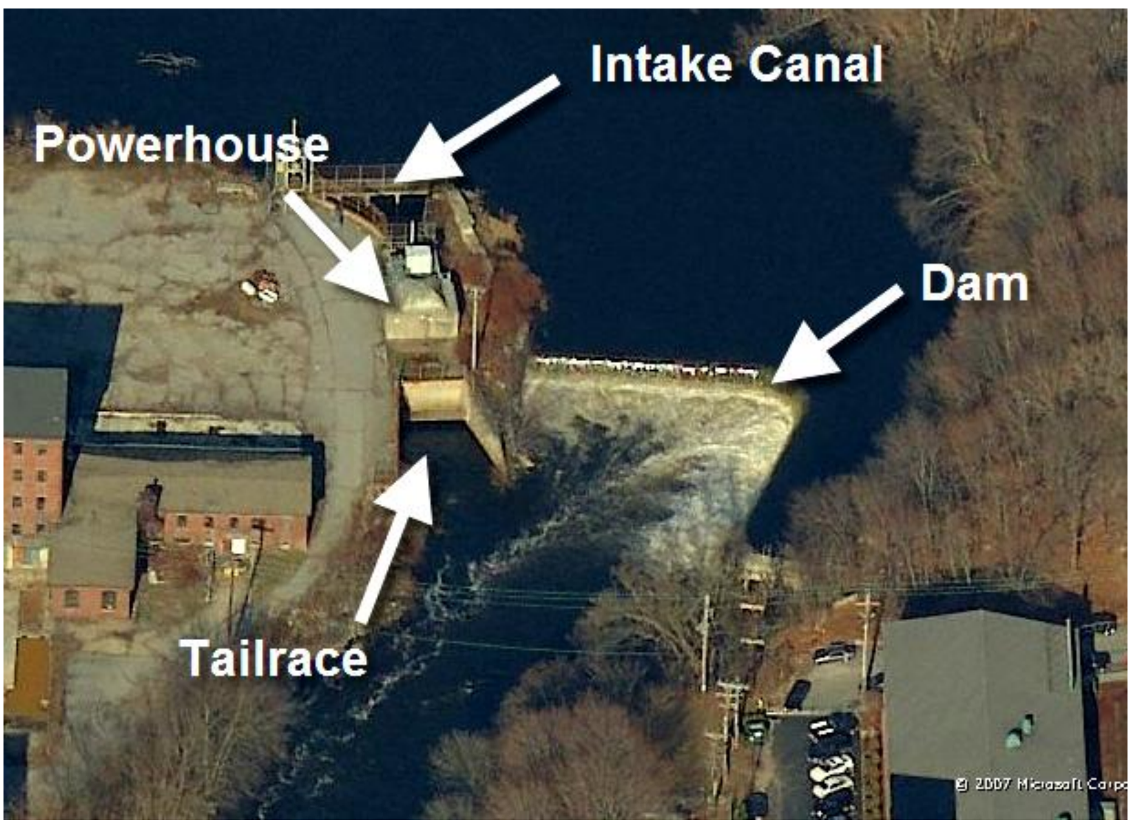

Figure 7 - Aerial Photograph of Elizabeth Webbing Mills Dam
(Flow from top to bottom)

The Blackstone River drainage area at Elizabeth Webbing Mills Dam is 473 sq. miles and the impoundment has a surface area of 26 acres. The dam is approximately 14 ft high with the powerhouse located at the right abutment and an abandoned structure supporting two wooden gates on the left abutment of the dam. Normal tailwater is approximately 10 ft below the dam crest.

The Slater Mill impoundment extends upstream almost to the Elizabeth Webbing Dam and tailwater levels at Elizabeth Webbing Dam could be affected by Slater Mill Dam. Implementation of fish passage facilities at Elizabeth Webbing Dam would not affect river conditions at the downstream dams (Slater Mill and Main Street), but could impact the tailwater conditions and fish passage facilities at Valley Falls Dam. Therefore, this report addresses the range of potential fish passage options at Elizabeth Webbing Dam, which are represented by: (1) complete dam removal, and (2) installation of a fish ladder with no modifications to the dam.

Abbott Run is a tributary which enters the Blackstone River between the Elizabeth Webbing and Valley Falls Dams. Pawtucket Water Supply Board (PWSB) manages Abbott Run to provide drinking water to the City of Pawtucket and neighboring communities (Narragansett Bay Estuary Program 2002). According to the Restoration Plan for the Blackstone River, there are six dams on Abbott Run that block upstream fish migration. The first four dams upstream from the Blackstone River could easily accommodate fish passage facilities. The two uppermost dams at Arnold Mills and Diamond Hill Reservoirs present considerable obstacles to fish passage because of the configuration of the dam facilities and fluctuating reservoir water levels. Due to the small size of the tributary, Abbot Run would probably not provide any useable habitat for American shad.

Restoration of fish passage to Abbott Run may be implemented after fish passage is restored at the first four dams on the Blackstone River. Therefore, the hydraulic analysis of Abbot Run has not been included in this study. This study treats Abbot Run like all the other small tributaries in the watershed that convey runoff to the lower Blackstone River.

2.4 VALLEY FALLS DAM

Blackstone Hydro Associates owns and operates the Blackstone Falls Hydroelectric Project at Valley Falls Dam in Central Falls and Cumberland, Rhode Island. The project, which is FERC Project No. 3063, has operated since 1985 and was licensed in 1981. The Valley Falls Dam is located adjacent to the Broad Street Bridge on the border of Central Falls and Cumberland, Rhode Island. The Blackstone Falls Hydroelectric Project consists of: (1) a 200-ft long curved granite masonry structure about 10 ft in height, (2) a wooden gate house containing 5 timber gates, each 7 ft 10 in. wide by 9 ft 3 in. high, (3) a 300-ft long open channel head race varying from 26-60 ft wide, (4) concrete intake structure with a trash rack and two hydraulic turbines, and (5) a 1200-ft long by 25-ft wide tailrace.

The dam is located just downstream of the bridge on Broad St. that connects Central Falls and Cumberland, as shown on Figure 8. The dam, which was originally built in 1853, is an overflow spillway with the crest at El. 49.3 ft (NGVD 1929). The Blackstone River drainage area at

Valley Falls Dam is 445 sq. miles. The dam is approximately 10 ft high and the impoundment has a surface area of 60 acres.

The gate house is located at the river bank immediately upstream of the dam. The gate house is a masonry arch structure with 5 gate openings. The timber gates are constructed in two segments with each segment operated by a rack and pinion drive system. Both gate sections are about 4 ft 9 in. high.

The intake canal extends about 300 ft downstream from the gate house. The invert of the canal is almost flat at El. 39.0 ft from the dam to the end of the canal. The powerhouse intake structure is located about 150 ft downstream of the gatehouse. Trash racks sloped at 1H:1V are installed at the intake opening. The trash racks have a 3 inches clear opening and extend from the canal bottom at El. 39.0 ft to the operating deck at El. 50.8 ft.

Figure 8 - Aerial Photograph of Valley Falls Dam
(Flow from bottom right to top left)

3. RIVER FLOW DATA

U.S. Geological Survey (USGS) flow gage 01112500 is located on the Blackstone River at Woonsocket, Rhode Island. The drainage area of the Blackstone River at the gage is 416 square miles with a period of record of 1929 to present. River flow at Main Street, Slater Mill, Elizabeth Webbing, and Valley Falls Dams has been estimated as the ratio of the watershed drainage area at the site and the USGS gage. The drainage areas at Main Street, Slater Mill, Elizabeth Webbing, and Valley Falls Dams are 474, 474, 473, and 445 square miles, respectively. Flow duration data for the four dams are provided on Tables 1 through 3.

TABLE 1 MAIN STREET AND SLATER MILL DAMS FLOW DURATION DATA

Percent (%) Time Exceeded	Flow (cfs) for Percent Time Exceeded (Main St/Slater)			
	Annual Jan. 1 – Dec. 31	Upstream Migration April 1 - June 30	Downstream (Juvenile) Sept 1 - Nov 15	Downstream (Adult) June 1 - July 15
Maximum	29,511	12,420	15,382	12,420
1	4,398	4,694	3,190	3,863
5	2552	2,883	1,515	1,846
10	1,937	2,279	975	1196
20	1,345	1,641	629	769
30	1,020	1,265	458	593
40	790	1,044	354	489
50	615	868	295	406
60	465	831	250	345
70	343	720	212	292
80	253	464	181	244
90	185	335	146	194
100	24	50	33	50

TABLE 2 ELIZABETH WEBBING DAM FLOW DURATION DATA

Percent (%) Time Exceeded	Flow (cfs) for Percent Time Exceeded (Elizabeth Webbing)			
	Annual Jan. 1 – Dec. 31	Upstream Migration April 1 - June 30	Downstream (Juvenile) Sept 1 - Nov 15	Downstream (Adult) June 1 - July 15
Maximum	29,449	12,394	15,350	12,394
1	4,389	4,685	3,184	3,854
5	2547	2,877	1,512	1,842
10	1,933	2,274	973	1,194
20	1,342	1,637	628	767
30	1,018	1,262	457	591
40	788	1,042	354	488
50	614	866	294	405
60	464	829	249	345
70	342	719	211	291
80	252	463	181	243
90	184	334	146	193
100	24	50	33	50

TABLE 3 VALLEY FALLS DAM FLOW DURATION DATA

Percent (%) Time Exceeded	Flow (cfs) for Percent Time Exceeded (Valley Falls)			
	Annual Jan. 1 – Dec. 31	Upstream Migration April 1 - June 30	Downstream (Juvenile) Sept 1 - Nov 15	Downstream (Adult) June 1 - July 15
Maximum	27,706	11,660	14,441	11,660
1	4,129	4,407	2,995	3,626
5	2,396	2,706	1,423	1,733
10	1,819	2,139	916	1,123
20	1,262	1,540	590	722
30	957	1,187	430	556
40	741	980	333	459
50	578	815	277	381
60	436	780	234	324
70	322	676	199	274
80	237	435	170	229
90	173	314	137	182
100	22	47	31	47

4. FEMA FLOOD STUDY

Data from the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) of the Blackstone River for the City of Central Falls (July 6, 1981) and the City of Pawtucket (January 3, 1986) were used as the basis of the flood data for this report. In these studies, HEC2 modeling was performed along the Blackstone River beginning from just upstream (north) of the Main St. Dam in Pawtucket to the beginning of the Valley Falls Pond in Cumberland. Peak discharges for the 10-year, 50-year, and 100-year flood events on the Blackstone River, as reported in the FEMA FIS for Central Falls and Pawtucket, are presented in Table 4. The FEMA flood studies include the flood of record which occurred on August 20, 1955.

TABLE 4 FEMA PEAK FLOOD DISCHARGES ON THE BLACKSTONE RIVER

	Flood Frequency Peak Discharge (cfs)			
Location	10-year	50-year	100-year	500-year
At Main Street Dam	10,780	18,380	23,700	43,000
At Slater Mill Dam	10,780	18,380	23,700	43,000
At Pantex Dam Dam [1)]	10,720	18,220	23,580	42,870
At Sayles Dam [2)]	10,350	17,600	22,770	41,405

1) Elizabeth Webbing Dam was formerly known as Pantex Dam.
2) Valley Falls Dam was formerly known as Sayles Dam.

Flood levels in the Blackstone River between Main Street and Valley Falls Dam, as reported in FEMA FISs for Pawtucket and Central Falls, are summarized in Table 5.

TABLE 5 FEMA PEAK FLOOD ELEVATIONS ON THE BLACKSTONE RIVER

	Flood Frequency Water Level (NGVD 1929)			
Location	10-year	50-year	100-year	500-year
Upstream of Main Street Dam	El. 27.8 ft	El. 29.9 ft	El. 31.4 ft	El. 39.1 ft
Downstream of Slater Mill Dam	El. 28.0 ft	El. 30.5 ft	El. 32.2 ft	El. 40.0 ft
Upstream of Slater Mill Dam	El. 30.2 ft	El. 31.5 ft	El. 32.7 ft	El. 40.0 ft
Downstream of Pantex Dam [1)]	El. 32.7 ft	El. 35.8 ft	El. 39.0 ft	El. 50.5 ft
Upstream of Pantex Dam [1)]	El. 40.9 ft	El. 43.2 ft	El. 45.0 ft	El. 50.5 ft
Downstream of Sayles Dam [2)]	El. 48.0 ft	El. 51.2 ft	El. 53.3 ft	El. 58.7 ft
Upstream of Sayles Dam [2)]	El. 55.6 ft	El. 58.8 ft	El. 60.2 ft	El. 68.1 ft

1) Elizabeth Webbing Dam was formerly known as Pantex Dam.
2) Valley Falls Dam was formerly known as Sayles Dam.

The current FEMA Flood Study for Pawtucket was based on an original analyses performed by the U.S. Army Corps of Engineers (USACE). The updated version was prepared by PRC Harris for FEMA under Contract No. H-4776. The starting water-surface elevations upstream of Main Street Dam for the FEMA FIS were based on a study prepared by C. E. Maguire, Inc. in 1976 entitled "Water Resources Study, Blackstone River Basin, Massachusetts and Rhode Island, Hydrology and Hydraulic Studies."

5. PROPOSED FISH PASSAGE FACILITIES

The Blackstone River Fish Passage Restoration Project is considering Denil fish ladders at Main Street, Slater Mill, and Valley Falls Dam to pass alewives, blueback herring, and American shad. Eel ladders are being incorporated into the fish passage facilities at these three dams. Fish passage at Elizabeth Webbing Mills Dam is also being considered, and the owner and RIDEM are interested in removing the Elizabeth Webbing Mills Dam. However, the fate of the dam is not known at this time. In addition, the Slater Mill Historic Site has not made a decision about the Slater Mill Turbine Rehabilitation Project. Therefore, this study addresses changes in river conditions resulting from installation of Denil fishways at all four dams, and removal of Elizabeth Webbing Dam in conjunction with Denil fishways at the other three dams on the lower Blackstone River.

The Denil fish ladder designs are based on the configurations that were developed by US Fish and Wildlife Service (USFWS) for the Blackstone River Watershed Reconnaissance Investigation, which was prepared by the USACE, New England District and presented in a report dated August 1997. The USFWS configuration for the Valley Falls has been relocated to the abandoned mill raceway adjacent to the left dam abutment. All Denil fishways proposed for the Blackstone River are 4 ft wide with 8H:1V bottom slopes. Flow through the fishways is controlled by the top baffle and is dependent on the water levels at the fishway exit into the headpond of each dam. Effective operation of the fishways is also dependent on proper design of the fishway entrance and exit channels for the range of headwater and tailwater levels, respectively. At dams where hydropower turbines or the project configuration creates a competing flow for the fishway entrance, additional attraction water at the fishway entrance and tailrace bar rack barriers are required to ensure that fish are not attracted up the tailrace channel to the turbine draft tube exits. Additional fishway attraction flow over the dam and a tailrace barrier have been proposed by USFWS for Main Street and Valley Falls, and may be necessary for Slater Mill depending on future decisions by the project owners relative to power generation.

In addition to the upstream fish passage facilities, each dam will have provisions for downstream passage. At Main Street Dam, USFWS originally envisioned an angled trash rack at the powerhouse intake under the Main Street bridge at the right abutment of the spillway. However, this option was conditional on the hydraulic conditions at the intake and acceptability of the conditions for effective downstream passage. In order to develop a cost-effective solution for downstream passage, EA conducted a preliminary evaluation of three downstream passage options. These options included a surface bypass at the existing trash rack, an angled trash rack, and a louver fish diversion system. The angled bar rack and louver system configurations were located upstream of the Main Street bridge because of the access and limited space under the bridge at the hydroplant intake. Therefore, the surface bypass at the existing trash racks was considered to be the most cost-effective option and was selected for detailed design.

A downstream bypass chute adjacent to the Denil fishway exit at Slater Mill Dam and a surface bypass adjacent to the hydroplant intake at Valley Falls Dam were identified by USFWS as the preferred options for downstream fish passage at these dams. The conceptual design of the Denil fishway prepared by USFWS for Elizabeth Webbing Dam also included a surface bypass adjacent to the hydroplant intake. All of the USFWS conceptual designs for downstream passage are incorporated in this study for the Denil fishway installations. For the Elizabeth Webbing Dam removal option, EA assumed a full breach of the dam and that resulting channel would be acceptable for fish passage.

Design of a fish passage facilities at the Main Street, Slater Mill, and Valley Falls Dams are based on criteria relative to species migration, water levels, flow rates, and other site characteristics. General design criteria for the proposed fish passage facilities as defined by USFWS are discussed in the following sections.

5.1 MIGRATION PERIOD

Alewives, blueback herring, and American shad are the primary target species for the Blackstone River Fish Passage Restoration Project. These species are expected to move upstream in the Blackstone River between April 1 and June 30. Downstream migration of these species is expected during the June 1-July 15 period for adults and September 1-November 15 period for juveniles.

American eel is also a primary target species on the Blackstone River. Upstream eel migration is expected to be April 1-July 15 and downstream migration from September 1-December 15.

5.2 RIVER CONDITIONS FOR FISHWAY OPERATION

The fish passage facilities should be designed to operate at river conditions ranging from minimum flow during the upstream migration period up to a maximum flow of about 3 to 4 times the average flow during the migration periods. The river flow range selected for design of the fish passage facilities is summarized in Table 6.

Data presented in Table 6 are based on USGS flow data adjusted to each of the dams by the ratio of the drainage area for the dam to the drainage area of the gage. Daily flow records were reviewed to identify a maximum design flow that was not exceeded for four consecutive days during the upstream fish migration period. The maximum design flows were selected because there would only be two periods during the last 15 years when the maximum design flow would have been exceeded on 4 or more consecutive days. During the 78 year period of record there would have been only 7 occurrences when the design flow was exceeded on 4 or more consecutive days

5.3 FISH PASSAGE FACILITY FLOWS

The discharge rating curve for a 4-ft wide Denil fish ladder is provided in Table 7. At hydroelectric projects similar to the plants at Main Street and Valley Falls Dams, USFWS typically requires an attraction flow at the fishway entrance equal to 3% of the hydroplant flow.

TABLE 6 FISH PASSAGE DESIGN FLOWS FOR LOWER BLACKSTONE RIVER DAMS

Location	River Flow (cfs)		
	Minimum	Average	Maximum
Main Street and Slater Mill Dams			
Upstream Fishways	50 [1)]	868 [2)]	4,000 [3)]
Downstream Bypass	33 [4)]	337 [5)]	1,011 [6)]
Valley Falls Dam			
Upstream Fishways	47 [1)]	815 [2)]	3,755 [3)]
Downstream Bypass	31 [4)]	316 [5)]	948 [6)]

1) Minimum flow for period of record during April 1 – June 30 upstream migration period.
2) 50% exceedance value for the April 1 – June 30 upstream migration period.
3) Approximately 4 times the average flow during the upstream migration period.
4) Minimum flow for period of record during June 1 – November 15 downstream migration period.
5) 50% exceedance value for the June 1 – July 15 and September 1 - November 15 downstream migration periods.
6) Approximately 3 times the average flow during the downstream migration period.

TABLE 7 DENIL FISHWAY (4-FT WIDE) FLOW RATING CURVE

Depth of Water in Exit Channel (ft)	Discharge (cfs)
1.5	1.3
2.0	5.5
2.5	11.5
3.0	17.5
3.5	25.0
4.0	32.5

This attraction flow would be about 42 cfs maximum at Main Street Dam and 24 cfs at Valley Falls Dam. Bottom or sidewall diffusers are typically used to convey the attraction flow to the Denil ladder entrance. Floor (bottom) diffusers should be designed for 1 ft^2/cfs, which is a 1 ft/sec exit velocity. Wall diffusers should be designed for 0.5 ft/sec exit velocity. The fishway entrance velocity should be about 5 ft/sec, as recommended by USFWS.

Because of the river configuration and the hydraulic capacity of the hydroelectric plant at Main Street and Valley Falls, additional attraction flow from the fishway entrance is not necessary. For this analysis, the minimum attraction flow has been assumed to be a combination of the fishway flow and additional flow over the dam. The minimum fishway attraction flow has been assumed to be in addition to the 50 cfs minimum flow release at Main Street Dam and 108 cfs at Valley Falls Dam.

The total downstream fish bypass flow typically required by USFWS at large hydroelectric project is 2-3% of the hydroplant flow for angle trash racks. Two percent (2%) of the hydroplant flow amounts to about 28 cfs at Main Street Dam and 16 cfs at Valley Falls Dam. For trash racks perpendicular to the flow, up to five percent (5%) of the hydroplant flow is required for downstream passage, which would be about 60 cfs at Main Street. At smaller hydroelectric intakes like the one at Main Street and Valley Falls Dams, the minimum size surface bypass is 36 inches wide with a minimum water depth of 30 inches, which would pass about 45 cfs and is the basis of design for the downstream passage facilities at these two projects.

5.4 WATER LEVELS

The Denil fishway at the entrance and at the exit must have a 2 ft minimum water depth at minimum river design flow, and a 30-in. water depth at normal flows. At Main Street Dam, minimum water level at the fishway entrance must take into account tidal fluctuations expected during the fish migration period. The Slater Mill Dam fishway design must account for water level changes that may result from rehabilitation of the hydraulic turbines. The Valley Falls Dam fishway design must account for the potential water level changes that would occur with either removal of the dam or installation of a Denil fishway.

5.5 DOWNSTREAM BYPASSES

Downstream fish diversion facilities for the hydroelectric projects should be designed for a 2 ft/sec or less approach velocity at bar racks that are perpendicular or angled to the flow in accordance with USFWS design criteria. Angled bar racks should be up to 45^{o} off the direction of flow. Surface bypasses with bar racks that are perpendicular to the approach flow will be used at the Main Street and Valley Falls Dam power intakes.

Small notches in dams where there is no power generation, similar to Slater Mill Dam, are typically used to pass downstream migrants during low flow periods. At Slater Mill Dam, the notch required by USFWS is 3 ft wide and approximately 2.0 ft below the crest of the dam.

5.6 DENIL FISHWAY CONFIGURATION

Fishway entrances should be located as close to the spillway as possible at the most upstream point of the dam. This requirement dictates a bend of almost 180^{o} from the entrance to the first ascending baffle section and a second 180^{o} bend back toward the dam to the fishway exit channel. At Main Street and Valley Falls, the fishway will be located on the left abutment of the dam (looking downstream) to minimize impacts on the existing historical and hydroelectric project structures. The fishway exit at the Valley Falls Dam will located near the left abutment through the exiting timber gate opening such that the dam crest length will not change to minimize impacts on the existing water level in addition to reducing impacts on the hydroelectric project.

6. HYDRAULIC ANALYSIS

6.1 MODEL DEVELOPMENT

EA prepared a HEC-RAS model to represent localized flow under existing and proposed conditions of the Blackstone River from downstream of Main Street Dam and hydroelectric project to approximately 2,500 ft along the centerline of the river upstream of the Valley Falls Dam in Cumberland. The HEC-RAS model uses Manning's formula and input parameters include channel cross sections, channel roughness coefficients, and river flow. Input data for the HEC-RAS model is provided in Appendix A.

The HEC-RAS model incorporated cross sections obtained from the FEMA FIS for Pawtucket and Central Falls. Detailed survey data was obtained by EA at Main Street Dam, Slater Mill Dam, and Valley Falls Dam to develop fishway designs at these dams. This survey data was obtained by a Rhode Island Professional Licensed Surveyor (PLS) on 20-21 August 2007. Cross sections from the survey data were incorporated into the HEC-RAS model in the vicinity of the four dams and FEMA cross sections were used in the river reaches between the dams. The approximate locations of the cross sections used in the EA HEC-RAS model of the Blackstone River are shown on Figure 9. The river station for each of the dams is defined in Table 8.

In order to study proposed conditions, the model cross sections were adjusted to reflect the fish passage facilities at each of the dams. Cross sections used in the model for the existing river channel and dams and with the proposed fish passage facilities are presented in Appendix C.

The FEMA FIS for Central Falls used a channel roughness coefficient (e.g. Manning's n) of 0.03 and overbank coefficient of 0.05 for the hydraulic analysis. The FEMA FIS for Pawtucket used coefficients ranging from 0.025 to 0.05 for the center channel and 0.050 to 0.110 for the overbank regions. For EA's HEC-RAS study, a coefficient of 0.04 for the center channel and 0.07 for the overbank were selected as conservative values based on the observed conditions for the Blackstone River reaches at the dams.

Spillway length and geometry of the dams for EA's study were based on field survey measurements. The overall length of existing spillway at Main Street is 180 ft as measured at the centerline of the spillway and 177 ft as measured at the downstream side of the crest. For the HEC-RAS model, an effective length of 170 ft was used for the existing spillway to account for the sharp angles in the dam crest. The HEC-RAS model reflected the 192.5 ft spillway length for existing Slater Mill Dam and 185.0 ft spillway length for existing Valley Falls Dam based on the field surveys. The effective length of the spillways at these two dams was equal to the overall measured length because of the relatively straight spillway geometry.

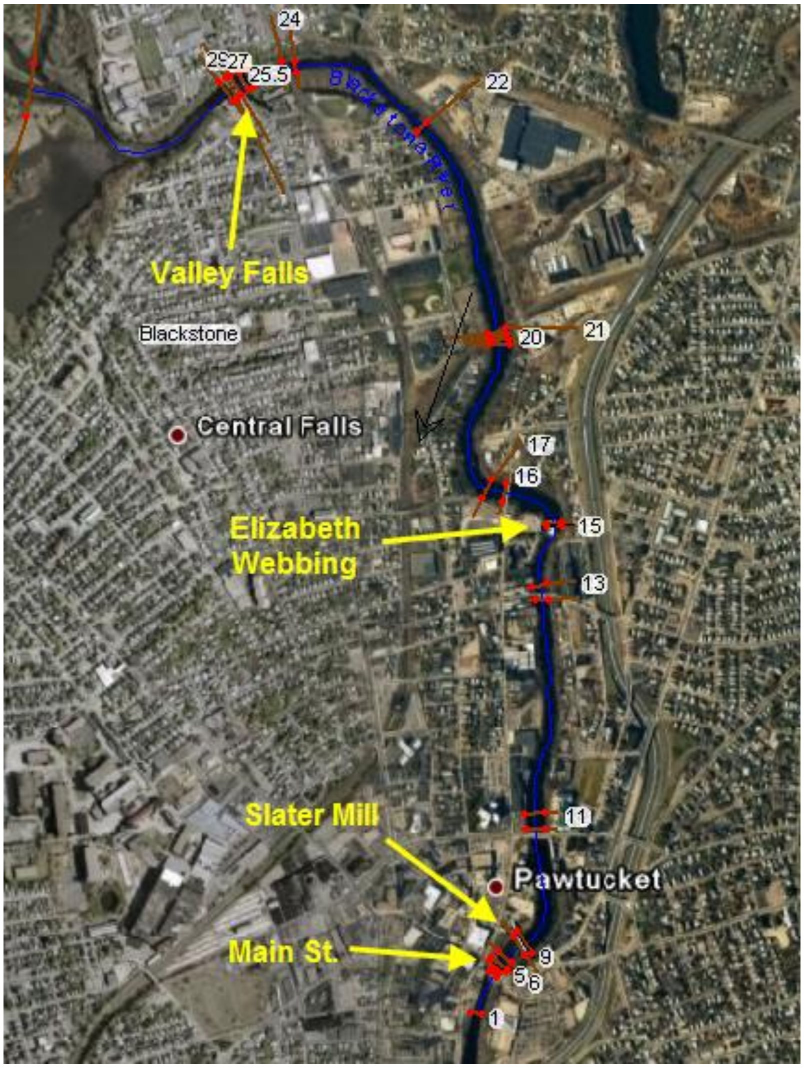

Figure 9 – Approximate location of cross sections

TABLE 8 BLACKSTONE RIVER DAM RIVER STATIONS USED FOR HEC-RAS MODEL

Location	Station[1)] (ft)	Section Number
Main Street Dam	495	5.5
Slater Mill Dam	819	8.5
Elizabeth Webbing Mills Dam	4,947	15.5
Valley Falls Dam	11,362	26.5
1) Distance upstream from Section 1.		

For the proposed conditions, the Denil fishways at Main Street and Slater Mill Dams reduced the spillway flow area by 45 ft^2 (15 ft long by 3 ft high rectangular area) at all of the simulated flood conditions. Stoplogs will be placed in the fishway exit channel during non-migration periods and in anticipation of flood events to minimize potential damage to the Denil baffles. Flood level estimates would be conservatively high with the stoplogs installed because the fishway flow would be limited to only that flow over the stoplogs. At the Slater Mill Dam, a low level notch with a flash board, which is designed to fail, has been incorporated into the fish passage facilities to assure no increase in existing upstream flood levels. At Valley Falls Dam, the fishway does not impact the spillway crest length and flood levels would be unchanged upstream of the dam.

Field survey data was not obtained for Elizabeth Webbing Mills Dam and existing data for the project was used for the HEC-RAS model. An effective length of 200 ft, compared to the 210 ft overall length, was used to account for the two 90° angles in the spillway geometry. Two alternatives for fish passage at Elizabeth Webbing Mills Dam were evaluated. The fish ladder alternative assumed that the effective length of the Elizabeth Webbing Mills Dam spillway was 15 ft shorter to account for the fishway exit and non-overflow section adjacent to the fish ladder. The dam removal alternative assumed that the entire Elizabeth Webbing Dam was removed.

The HEC-RAS program default discharge coefficient (C) for spillways and weirs is 2.6. This coefficient is representative of a broad-crested weir with an extremely shallow upstream reservoir depth relative to the depth of water over the weir. Sharp crested weirs with weir heights greater than one-fifth of water depth over the weir have relatively constant discharge coefficients of 3.3 (United States Department of Interior 1977). The HEC-RAS default coefficient provides conservatively high water level predictions.

Ogee shaped spillways are designed for optimum flow and have discharge coefficients ranging from 3.1 to as high as 3.9, depending on the depth of water over the crest and the depth of the reservoir. A 3.1 coefficient is representative of a very shallow reservoir depth sharp relative to the depth of water over the crest. A 3.9 coefficient value is representative of very deep reservoirs relative to the depth of water over the ogee shaped weir.

The spillways at the four dams on the lower Blackstone River all have geometries that are between a sharp crested weir and an ogee shaped weir. Spillway discharge coefficients selected for the HEC-RAS model were selected to represent the discharge coefficient at an average spillway flow of about 600 cfs, which is comparable to the average annual river flow in the lower Blackstone River

Water depths over the spillway crests at all four dams would be about 1 ft using the weir formula, 600 cfs spillway flow, and C = 3.3. The weir formula is:

$$Q = CLH^{3/2}$$

Where: Q = discharge (cfs)
C = discharge coefficient
L = Length of weir (ft)

H = vertical distance between the weir crest and the water level upstream of the weir (ft).

The discharge coefficient was computed using the formula:

$$C = 3.22 + (0.4(H/P))$$

Where, P = vertical distance from the base to the crest of the weir (ft)
H = vertical distance between the weir crest and the water level upstream of the weir (ft).

The P values used in this formula were 4 ft for Main Street and Slater Mill, and 6 ft for Main Street Dam. Assuming H = 1.0 ft, C = 3.3 for all four dams. This formula indicates that higher water depths (H) result in higher discharge coefficients for a given crest geometry, resulting in conservatively high water level predictions for flows greater than 600 cfs using a constant a constant C = 3.3 .

Higher values of C would result in lower water levels upstream of the dams and would underestimate flood impacts resulting from implementation of the fish passage facilities. Therefore, in order to conservatively estimate water levels in the river, a discharge coefficient of 3.3 was selected for the HEC-RAS analysis of all four dams.

6.2 RIVER CONDITIONS

The HEC-RAS model study evaluated flow conditions for the 2-year, 10-year, 25-year, and 100-year storm events. These events were evaluated to investigate the impacts of the proposed fishways on river conditions as required for the RIDEM and USACE permit applications. The 10-year and 100-year flow values were taken from the FEMA FIS for Pawtucket and Central Falls at each of the four dams. The FEMA data was plotted on a log scale to interpolate the 2-year and 25-year flow values. The river flow at each dam for each flood event that was used for the simulations is presented in Table 9. For this analysis, the hydropower plants at the dams were assumed to be shutdown and all river flow was over the spillways. This assumption results in the highest flood levels through the lower reach of the river.

TABLE 9 BLACKSTONE RIVER FLOWS USED FOR HEC-RAS SIMULATIONS

	Flow (cfs)			
Location	2-year	10-year	25-year	100-year
Main Street.	6,000	10,780	14,762	23,700
Slater Mill	6,000	10,780	14,762	23,700
Elizabeth Webb.	5,965	10,720	14,679	23,580
Valley Falls	5,760	10,350	14,172	22,770

The downstream boundary conditions used in the HEC-RAS model were obtained from the Seekonk River in the FEMA City of East Providence FIS and are presented in Table 10. The flood levels represent the stillwater elevations for coastal storms and do not necessarily coincide with flood events in the Blackstone River Watershed. The values for the 10-year and 100-year events were plotted for interpolation of the 25-year and 2-year events. The downstream boundary of the HEC-RAS model is just past the Pawtucket Hydropower tailrace exit to the river which is about 500 ft river feet downstream from Main Street Dam.

TABLE 10 DOWNSTREAM BOUNDARY WATER ELEVATIONS
USED FOR HEC-RAS SIMULATIONS

Flood Event	Elevation (ft NGVD)
100-year	16.0
25-year	12.3
10-year	10.2
2-year	7.9

6.3 SIMULATION RESULTS COMPARED TO FEMA FLOOD LEVELS

The 10-year and 100-year flood events were simulated with the existing dam and spillway structures for comparison to the FEMA FIS water levels. Table 11 presents water levels along the lower Blackstone River predicted with the HEC-RAS model and the FEMA water levels. Water profiles along the river centerline for the HEC-RAS simulations are presented in Appendix D. Model results are provided in Appendix B.

As shown in Table 11, the HEC-RAS starting levels downstream of Main Street Dam were the same as the Seekonk River stillwater levels reported in the FEMA FIS for the City of East Providence. Between Main Street Dam and the Slater Mill headpond, the HEC-RAS simulated water levels are lower than the FEMA levels because of differences in the spillway geometries and discharge coefficients. The C.E. Maguire report, which defined the basis of the FEMA FIS for the Pawtucket, does not define spillway length and discharge coefficient assumptions for Main Street Dam used to develop the FEMA flood levels. Therefore, resolution of the discrepancy between the HEC-RAS and FEMA FIS was not possible.

The discharge rating curves for spillways and overflow dam are typically developed using the weir formula (see page 23). The weir formula indicates that the differences between the HEC-RAS and FEMA water levels upstream of Main Street, Slater Mill, and Valley Falls Dams could be only be attributed to differences in flow (Q), the spillway lengths (L), and discharge coefficients values (C). The HEC-RAS model used the same flood flows reported by FEMA. Therefore, flow is not a source of the difference between the two studies. However, the difference could be attributed to shorter spillway lengths or lower discharge coefficients in the FEMA FIS.

TABLE 11 COMPARISON OF EA HEC-RAS RESULTS TO FEMA WATER LEVELS

Location	10-year Water Level (NGVD 1929)			100-year Water Level (NGVD 1929)		
	FEMA	EA	Difference	FEMA	EA	Difference
Downstream of Main Street Dam[1)]	El. 10.2 ft	El. 10.2 ft	0.0 ft	El. 16.0 ft	El. 16.0 ft	0.0 ft
Upstream of Main Street Dam	El. 27.8 ft[2)]	El. 23.9 ft	- 3.9 ft	El. 31.4 ft[2)]	El. 28.1 ft	- 3.3 ft
Downstream of Slater Mill Dam	El. 28.0 ft	El. 24.5 ft	- 3.5 ft	El. 32.2 ft	El. 29.4 ft	- 2.8 ft
Upstream of Slater Mill Dam	El. 30.2 ft	El. 29.6 ft	- 0.6 ft	El. 32.7 ft	El. 32.4 ft	-0.3 ft
Downstream of Elizabeth Webbing Mills Dam	El. 32.7 ft	El. 33.9 ft	+ 1.2 ft	El. 39.1 ft	El. 39.9 ft	+ 0.8 ft
Upstream of Elizabeth Webbing Mills Dam	El. 40.9 ft	El. 40.9 ft	0.0 ft	El. 45.0 ft	El. 45.0 ft	0.0 ft
Downstream of Valley Falls Dam	El. 48.0 ft	El. 49.2 ft	+ 1.2 ft	El. 53.3 ft	El. 54.4 ft	+1.1 ft
Upstream of Valley Falls Dam	El. 55.6 ft	El. 56.2 ft	+ 0.6 ft	El. 60.2 ft	El. 59.7 ft	- 0.5 ft

1) Stillwater elevations for Seekonk River downstream of Main Street Dam presented in FEMA FIS for City of East Providence.
2) FEMA FIS for City of Pawtucket referenced the basis for these staring water elevations from Water Resources Study, Blackstone River Basin, 1976, by C.E. Maguire.

In order to investigate the sensitivity in water level predictions relative to the Main Street Dam spillway length and discharge coefficient, EA used the HEC-RAS model with various combinations of C and L. A spillway length of 171 ft with a C value of 2.6 would replicate the FEMA 100-year flood level. However, to replicate the 10-year FEMA flood upstream of Main Street Dam, the spillway length would be approximately 118 ft with C = 2.6. With the 100 year flood flow and a spillway length of 118 ft, the discharge coefficient would have to be 3.7. A comparison of these numbers indicates that the FEMA flood levels were based on a different combination of spillway length and discharge coefficient assumptions for the FIS than measured for the HEC-RAS analysis used for this study.

Differences in water levels predicted with the HEC-RAS model and reported by FEMA for the river reaches between Slater Mill and Valley Falls Dams can be attributed to differences in Manning's roughness coefficients (n) used in the two studies. Between Slater Mill and Valley

Falls Dams, the HEC-RAS channel roughness coefficient assumptions (see Section 6.1) produce a greater slope for the energy and hydraulic grade lines which explain the differences from the FEMA water levels. The FEMA flood levels developed with the lower channel roughness coefficient are approximately 1.1-1.2 ft lower than the HEC-RAS predicted water levels downstream of Valley Falls Dam for the 10-year and 100-year flood events (see Table 11). Regardless of the channel and overbank roughness assumptions selected for the HEC-RAS analysis, the impacts of the fish passage facilities on the existing river conditions would be similar because the same assumptions have been applied to the simulation of the existing and proposed conditions.

6.4 FISHWAY IMPACTS ON WATER LEVELS

The 2-year, 10-year, 25-year, and 100-year flood events were simulated using the HEC-RAS model with the existing dam and spillway structures, and with the modifications required for the proposed fishways. The simulations predict changes in water surface elevations and velocities that would result from installation of fish passage facilities at Main Street, Slater Mill, Elizabeth Webbing Mills, and Valley Falls Dams. The simulations included Denil fishways at all four dams. Because the tailwater at Valley Falls could be affected by the fish passage facilities at Elizabeth Webbing Mills Dam, the hydraulic analysis also included full dam removal of Elizabeth Webbing Mills Dam. Comparisons of the simulation results comparing water levels with the existing spillways to the water levels after installation of the fish passage facilities are presented in Table 12-15 for each flood event.

TABLE 12 COMPARISON OF SIMULATED 2-YEAR FLOOD LEVELS WITH AND WITHOUT FISHWAYS

	Water Level (NGVD 1929)				
Location	Existing	Alternative A[1)]	Difference	Alternative B[2)]	Difference
Upstream of Main Street Dam	El. 21.8 ft	El. 22.0 ft	+ 0.2 ft	El. 22.0 ft	+0.2 ft
Downstream of Slater Mill Dam	El. 22.1 ft	El. 22.3 ft	+ 0.2 ft	El. 22.3 ft	+ 0.2 ft
Upstream of Slater Mill Dam	El. 27.4 ft	El. 27.3 ft	- 0.1 ft	El. 27.3 ft	- 0.1 ft
Downstream of Elizabeth Webbing Dam	El. 30.7 ft	El. 30.7 ft	0.0 ft	El. 30.7 ft	0.0 ft
Upstream of Elizabeth Webbing Dam	El. 38.9 ft	El. 30.7 ft	- 8.2 ft	El. 39.1 ft	+ 0.2 ft
Downstream of Valley Falls Dam	El. 46.3 ft	El. 46.3 ft	0.0 ft	El. 46.3 ft	0.0 ft
Upstream of Valley Falls Dam	El. 53.7 ft	El. 53.7 ft	0.0 ft	El. 53.7 ft	0.0 ft

1) Denil fish ladders at the Main Street, Slater Mill, and Valley Falls Dams, and complete removal of the Elizabeth Webbing Mills Dam.
2) Denil fish ladders at all four dams.

TABLE 13 COMPARISON OF SIMULATED 10-YEAR FLOOD LEVELS WITH AND WITHOUT FISHWAYS

	Water Level (NGVD 1929)				
Location	Existing	Alternative A[1)]	Difference	Alternative B[2)]	Difference
Upstream of Main Street Dam	El. 23.9 ft	El. 24.4 ft	+ 0.5 ft	El. 24.4 ft	+ 0.5 ft
Downstream of Slater Mill Dam	El. 24.5 ft	El. 24.9 ft	+ 0.4 ft	El. 24.9 ft	+ 0.4 ft
Upstream of Slater Mill Dam	El. 29.6 ft	El. 29.2 ft	- 0.4 ft	El. 29.2 ft	- 0.4 ft
Downstream of Elizabeth Webbing Dam	El. 33.9 ft	El. 33.8 ft	- 0.1 ft	El. 33.8 ft	- 0.1 ft
Upstream of Elizabeth Webbing Dam	El. 40.9 ft	El. 33.8 ft	- 7.1 ft	El. 41.1 ft	+ 0.2 ft
Downstream of Valley Falls Dam	El. 49.2 ft	El. 49.2 ft	0.0 ft	El. 49.2 ft	0.0 ft
Upstream of Valley Falls Dam	El. 56.2 ft	El. 56.2 ft	0.0 ft	El. 56.2 ft	0.0 ft

1) Denil fish ladders at the Main Street, Slater Mill, and Valley Falls Dams, and complete removal of the Elizabeth Webbing Mills Dam.
2) Denil fish ladders at all four dams.

TABLE 14 COMPARISON OF SIMULATED 25-YEAR FLOOD LEVELS WITH AND WITHOUT FISHWAYS

	Water Level (NGVD 1929)				
Location	Existing	Alternative A[1)]	Difference	Alternative B[2)]	Difference
Upstream of Main Street Dam	El. 25.3 ft	El. 25.6 ft	+ 0.3 ft	El. 25.6 ft	+ 0.3 ft
Downstream of Slater Mill Dam	El. 26.1 ft	El. 26.4 ft	+ 0.3 ft	El. 26.4 ft	+ 0.3 ft
Upstream of Slater Mill Dam	El. 30.7 ft	El. 30.4 ft	- 0.3 ft	El. 30.4 ft	- 0.3 ft
Downstream of Elizabeth Webbing Dam	El. 35.9 ft	El. 35.8 ft	- 0.1 ft	El. 35.8 ft	- 0.1 ft
Upstream of Elizabeth Webbing Dam	El. 42.7 ft	El. 35.8 ft	- 6.9 ft	El. 43.1 ft	+ 0.4 ft
Downstream of Valley Falls Dam	El. 51.1 ft	El. 51.0 ft	- 0.1 ft	El. 51.1 ft	0.0 ft
Upstream of Valley Falls Dam	El. 57.5 ft	El. 57.5 ft	0.0 ft	El. 57.5 ft	0.0 ft

1) Denil fish ladders at the Main Street, Slater Mill, and Valley Falls Dams, and complete removal of the Elizabeth Webbing Mills Dam.
2) Denil fish ladders at all four dams.

TABLE 15 COMPARISON OF SIMULATED 100-YEAR FLOOD LEVELS WITH AND WITHOUT FISHWAYS

Location	Water Level (NGVD 1929)				
	Existing	Alternative A[1)]	Difference	Alternative B[2)]	Difference
Upstream of Main Street Dam	El. 28.1 ft	El. 28.8 ft	+ 0.7 ft	El. 28.8 ft	+ 0.7 ft
Downstream of Slater Mill Dam	El. 29.4 ft	El. 29.8 ft	+ 0.4 ft	El. 29.8 ft	+ 0.4 ft
Upstream of Slater Mill Dam	El. 32.4 ft	El. 32.4 ft	0.0 ft	El. 32.4 ft	0.0 ft
Downstream of Elizabeth Webbing Dam	El. 39.9 ft	El. 39.9 ft	0.0 ft	El. 39.9 ft	0.0 ft
Upstream of Elizabeth Webbing Dam	El. 45.0 ft	El. 39.9 ft	- 5.1 ft	El. 45.4 ft	+ 0.4 ft
Downstream of Valley Falls Dam	El. 54.4 ft	El. 54.3 ft	- 0.1 ft	El. 54.4 ft	0.0 ft
Upstream of Valley Falls Dam	El. 59.7 ft	El. 59.7 ft	0.0 ft	El. 59.7 ft	0.0 ft

1) Denil fish ladders at the Main Street, Slater Mill, and Valley Falls Dams, and complete removal of the Elizabeth Webbing Mills Dam.
2) Denil Denil fish ladders at all four dams.

Flood levels between Main Street and Slater Mill Dams with the Denil fishways are 0.2-0.7 ft higher that the existing conditions for the 2-year, 10-year, 25-year, and 100-year events. The HEC-RAS model results for the 2-year, 10-year, and 25-year floods indicate that water levels between the first two dams are lower than the top of the stone retaining walls (El. 30.0 ft minimum on the right bank) lining both sides of the river channel and are lower than the existing topography of the river overbank adjacent to the retaining walls (El. 27.0 ft minimum on the right bank) for both the existing dam and with the proposed fishways. Flood levels between Main Street and Slater Mill Dams during the 100-year flood are higher than the existing topography along the river overbank, but are below the top of the river channel stone flood walls (El. 30.1 ft). However, a comparison of the 100-year (Table 15) flood levels with the fishways to the FEMA FIS (Table 11) flood levels between Main Street and Slater Mill Dams indicates that the HEC-RAS model results are lower than FEMA flood levels.

Flood levels immediately upstream of Slater Mill Dam with the fish ladder are 0.1-0.4 ft lower than the existing conditions for all flood events. Flood levels with the fish ladder are lower than the existing topography (El. 31.0 ft) immediately upstream of the dam for the 25-year and more frequent storm events. During the 100-year flood, water levels would be higher than the overbank elevation immediately upstream of Slater Mill Dam, with and without the fish ladder, but lower than the top of the right river flood wall (El. 34.3 ft). Immediately downstream of Elizabeth Webbing Mills Dam, change in flood levels for all of the storm events would be 0.1 ft lower or less with the fish ladder at Slater Mill than flood levels with the existing Slater Mill

Dam. However, the simulated flood levels with and without the fish ladder at Slater Mill Dam are lower than the FEMA 10-year and 100-year flood levels (Table 11) between Slater Mill Dam and Elizabeth Webbing Mills Dam.

With removal of Elizabeth Webbing Dam (Alternative A in Tables 12-15), water levels upstream of the dam will be 5.1-8.2 ft lower than the existing conditions depending on the flood event. If a fish ladder is installed at Elizabeth Webbing Dam (Alternative B in Tables 12-15), flood levels would be 0.2-0.4 ft higher than the flood levels with the existing dam for all storm events.

At Valley Falls Dam, downstream flood levels with removal of Elizabeth Webbing Dam would be 0.1 ft lower than the flood levels with the existing Elizabeth Webbing Dam for all of the simulated flood events. If a fish ladder is installed at Elizabeth Webbing Dam, there would be no change in flood levels immediately downstream of Valley Falls Dam from the existing conditions.

With the installation of the fish ladder at Valley Falls Dam, the Valley Falls pond levels remain the same as existing flood levels for all the flood year events. These levels are lower than the FEMA flood levels (Table 11).

6.5 FISHWAY IMPACTS ON FLOW CONDITIONS UNDER MAIN STREET BRIDGE

The HEC-RAS model was used to investigate changes in river conditions under the Main Street bridge. Sections 2-5 in the model reflected surveyed topography in the river channel from the base of the dam to about 30 ft downstream of the bridge pier, as shown on Figure 10. Section 6 in the model is located just upstream of the Main Street Dam and is parallel to the dam and Section 5.

Simulated water levels and velocities in the river channel under the bridge for 100-year flood are summarized in Tables 16 and 17, respectively, with the existing conditions and with the proposed fishway. Water levels under the Main Street bridge with the fishway are 1-2 ft higher than the existing conditions. At the upstream nose of the bridge pier (Section 4), simulated water levels and channel velocity with the fishway are no different than the existing conditions. Upstream of the bridge pier, channel velocities with the fishway are slightly lower than the channel velocities for the existing conditions because of the slightly higher water depths. At the downstream end of the bridge pier (Section 3), water levels and channel velocity with the fishway are higher than the existing conditions because the fishway slightly reduces the river cross sectional area.

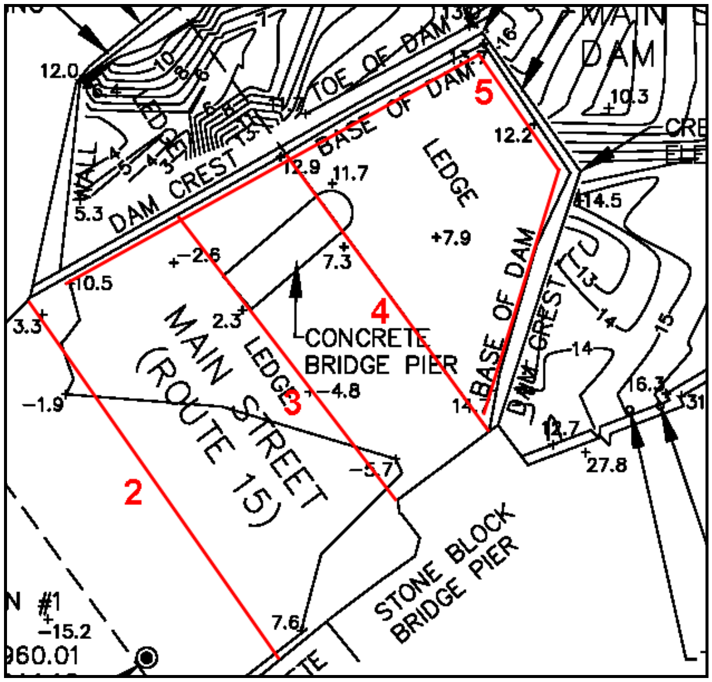

Figure 10 HEC-RAS Model Cross Sections at Main Street

TABLE 16 100-YEAR FLOOD
WATER LEVELS
UNDER MAIN STREET BRIDGE

Water Levels (ft NGVD 1929)			
Section	Existing	Proposed	Increase
6	28.1	28.8	0.7
5	26.3	28.1	1.8
4	19.1	20.6	1.5
3	17.8	18.1	0.3
2	17.9	18.2	0.3

TABLE 17 100-YEAR FLOOD
WATER VELOCITIES
UNDER MAIN STREET BRIDGE

Velocity (ft/sec)			
Section	Existing	Proposed	Increase
6	8.4	8.0	-0.4
5	9.6	8.7	-0.9
4	20.9	20.9	0
3	17.7	20.3	2.6
2	15.1	15.8	0.7

The 100-year event is the design flood for the fish passage facilities and represents the greatest change in hydraulic forces on the bridge pier after installation of fish ladder. More frequent flood events would have forces on the bridge pier with the fish ladder closer to the existing conditions.

Water levels upstream of the bridge pier are higher with the fishway which results in lower channel velocities between the bridge pier and dam. However, the water levels are below the masonry walls lining both sides of the river channel. At the pier upstream nose (Station 4), water levels are higher with the fishway, but the channel velocity is similar to the existing conditions.

At the downstream end of the bridge pier (Station 3), water levels and channel velocities are higher with the fishway than the existing conditions because the fishway does reduce the channels cross sectional area. At Section 2, which is near the downstream end of the fishway, water levels and velocities with the fishway are slightly higher than the existing conditions. At this location, the river is wider and the fishway is a lower percentage of the river cross sectional area resulting in lower increases in water level and velocities than at Section 3. Downstream of Section 2, the fishway does not affect the river channel cross sectional area and flow conditions; channel velocities with the fishway would be the same as the existing conditions.

Differential water levels from the upstream to downstream ends of the pier with the fish ladder would be 1.1 ft greater than the existing conditions. The differential pressure on the pier nose would increase by 8,000 pounds force from about 22,000 pounds with the existing condition water levels to about 30,000 pounds with the water levels after installation of the fishway. Dynamic forces on the pier would increase from about 65,000 pounds force without the fishway to about 76,000 pounds with the fishway. These forces would reduce the factor of safety for the pier to resist sliding and overturning by about 25 percent based on conservative assumptions for the pier foundation. These assumptions are:

- Pier is founded on rock and no additional resistance is provided by the pier being socketed into the rock, shear pins, or rock anchors.

- Coefficient of sliding for rock on rock is 0.12.

- One-half of the weight of each arch is supported by the center pier resulting in the dead weight of 2,400,000 pounds (20,000 ft^3 of stone at 120 pounds per ft^3).

- A 1.6 loading factor on the sliding and overturning forces.

The estimated factor of safety of the pier against sliding is 4.1 with the existing conditions and 3.0 with installation of the fishway. The estimated factor of safety of the pier against overturning is 23.3 with the existing conditions and 18.1 with the fishway. A factor of safety greater than 1.0 indicates that a structure is stable. Typically, minimum factors of safety of 2.0 to 3.0 are used to design these type of structures. Therefore, the increase water level and velocities on the bridge pier resulting from installation of the fishway do not affect the stability of the bridge pier.

Simulated velocities along the bridge pier during the 100-year flood event are 17.7-20.9 ft/sec for the existing conditions and 20.3-20.9 ft/sec for the proposed conditions with the fishway. These velocities correspond to shear stresses of 9.8 lbs/ft^2 for the existing conditions and 10.8 lbs/ft^2 with the proposed fishway as determined with the HEC-RAS model. Permissible shear stress for concrete is 12.5 lbs/ft^2, which would be comparable to the bridge pier stone blocks. Therefore, the higher velocity along the pier resulting from installation of the fishway would not result in more erosion than expected with the existing conditions.

7. CHANNEL STABILITY

Channel stability in the lower Blackstone River has been assessed to determine if installation of the fish passage facilities would change the river conditions to the extent that channel or bank stabilization measures should be implemented as part of the fish passage projects. This analysis has been used to identify river reaches at the Main Street, Slater Mill, and Valley Falls Dams where the Denil fishways proposed for these sites would create higher velocities than the existing conditions and determine the extent of channel and bank protection. In addition, the information provided in this section provides data about the stabilization measures that would be necessary for removal of Elizabeth Webbing Dam after the type of material at the site has been identified.

The channel stability analysis for the channel bed and banks was based on definition of flow conditions necessary to mobilize components of the stream bed (Lorang and Hauer 2003). Estimates of sediment transport rates in modified systems were determined using critical or permissible shear stress values that describe the force directly related to water velocities, necessary to initiate motion of particles of a particular size. The physical size of the channel and bank material is integral to evaluating changes in the rates of particle movement resulting from modification to hydraulics.

Since the actual channel and bank material in the river reach between Main Street and Valley Falls Dam is not precisely known, the channel stability analysis compares computed shear stresses for existing conditions to shear stresses with installation of fish passages facilities, and compares these values to permissible shear stress values for various channel and bank materials. The 100-year flood event was used for the analysis because this flood condition would have the highest velocities and greatest potential for channel and bank erosion velocities.

Permissible shear stress values relating to the channel bed and bank were obtained from empirical data gathered and summarized in "*Stability Thresholds for Stream Restoration Materials"* (Fischenich 2001). Permissible shear stress values for channels and banks having various materials are presented in Tables 18 and 19, respectively.

TABLE 18 CHANNEL BED PERMISSIBLE SHEAR STRESS VALUES

Lining Category	Lining Material	Permissible Shear Stress (lb/ft^2)
Soils	Graded Loam to Cobbles	0.38
Gravel / Cobble	1 in.	0.33
	2 in.	0.67
	6 in.	2.0
	12 in.	4.0
Rip Rap	6 in. d_{50}	2.5
	9 in. d_{50}	3.8
	12 in. d_{50}	5.1
	18 in. d_{50}	7.6

TABLE 19 PERMISSIBLE BANK SHEAR STRESS VALUES

Lining Category	Lining Material	Permissible Shear Stress (lb/ft^2)
Vegetation	Long Native Grasses	1.2 – 1.7
	Hardwood Tree Plantings	0.41 – 2.5
Soil Bioengineering	Coir Roll	3.0 – 5.0
	Vegetated Coir Mat	4.0 – 8.0
	Brush Layering	0.4 – 6.25
	Live Willow Stakes	2.10 – 3.10

The HEC-RAS model was used to calculate the shear stress values along the river channel bed and banks. The simulation results are summarized in Tables 20 and 21 for the channel bed and river banks, respectively, for the existing river conditions and the conditions expected with the Denil fishways installed at Main Street, Slater Mill, and Valley Falls Dams and with Elizabeth Webbing Mills Dam completely removed. As shown in the tables, there are no river reaches that require channel bed or river bank stabilization as a result of the Denil fish ladder installations. Shear stresses in all of these areas are similar with and without the fishways. However, immediately upstream of the existing Elizabeth Webbing Mills Dam (Station 4,957) may need channel armoring and bank stabilization riprap if the dam is removed to the headpond bottom elevation and there is no original channel armor stone.

TABLE 20 CHANNEL BED SHEAR STRESS ANALYSIS

Location	River Station (ft)	Channel Shear Stress Existing lb/ft^2	Channel Shear Stress Proposed lb/ft^2	Stabilization Necessary	Stabilization Reason
	13,853	0.09	0.08	No	Negligible Difference
	11,578	0.95	0.90	No	Negligible Difference
	11,513	0.84	0.80	No	Negligible Difference
	11,427	0.91	0.88	No	Negligible Difference
Valley Falls Dam	11,387	0.82	0.8	No	Negligible Difference
	11,317	1.59	1.91	No	Negligible Difference
	10,997	1.38	1.42	No	Negligible Difference
	10,877	2.49	2.65	No	Negligible Difference
	9,457	1.56	1.92	No	Negligible Difference
	7,357	0.6	1.01	No	Negligible Difference
	7,307	1.01	1.63	No	Negligible Difference
	7,272	1.01	1.66	No	Negligible Difference
	7,222	0.77	1.19	No	Negligible Difference
	5,722	0.78	1.70	No	Negligible Difference
	5,557	0.98	3.09	Yes	Higher Shear Stress[1)]
Elizabeth Webbing Mills Dam	4,947	1.39	1.39	No	Negligible Difference
	4,337	1.19	1.17	No	Negligible Difference
	4,287	1.31	1.30	No	Negligible Difference
	4,187	2.06	2.03	No	Negligible Difference
	2,127	1.06	1.04	No	Negligible Difference
	1,977	1.31	1.27	No	Negligible Difference
	947	1.19	1.14	No	Negligible Difference
Slater Mill Dam	827	1.67	1.58	No	Negligible Difference
	767	1.91	1.76	No	Negligible Difference
	597	2.62	2.34	No	Negligible Difference
	520	1.31	1.20	No	Negligible Difference
Main Street Dam	492	1.83	1.5	No	Negligible Difference
	452	9.83	10.8	No	Bedrock and Concrete
	428	6.63	9.99	No	Bedrock and Concrete
	400	4.22	4.91	No	Negligible Difference
	0	4.12	4.12	No	Negligible Difference

1) Station 4,947 is about 10 ft upstream of the Elizabeth Webbing Mills Dam. If the dam structure is removed to the headpond bottom elevation, channel armoring may be required depending on the bottom material.

TABLE 21 SIDE BANK SHEAR STRESS ANALYSIS

Location	River Station (ft)	Channel Shear Stress		Stabilization	
		Existing lb/ft^2	Proposed lb/ft^2	Necessary	Reason
	13,853	0.03	0.03	No	Negligible Difference
	11,578	0.01	0.02	No	Negligible Difference
	11,513	0.03	0.02	No	Negligible Difference
Valley Falls Dam	11,387	0.02	0.03	No	Negligible Difference
	10,997	0.58	0.59	No	Negligible Difference
	10,877	0.96	1.01	No	Negligible Difference
	9,457	0.97	1.18	No	Negligible Difference
	7,357	0.17	0.26	No	Negligible Difference
	7,307	0.31	0.46	No	Negligible Difference
	7,272	0.31	0.47	No	Negligible Difference
	7,222	0.26	0.38	No	Negligible Difference
	5,722	0.23	0.4	No	Negligible Difference
	5,557	0.44	0.85	No	Negligible Difference
Elizabeth Webbing Mills Dam	4,947	0.51	0.51	No	Negligible Difference
	4,337	0.52	0.52	No	Negligible Difference
	4,287	0.57	0.57	No	Negligible Difference
	4,187	0.97	0.96	No	Negligible Difference
	2,127	0.34	0.33	No	Negligible Difference
	1,977	0.52	0.53	No	Negligible Difference

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Appendix A

HEC-RAS Model Input Data

Appendix B

HEC-RAS Model Output Data

Appendix C

HEC-RAS Model Cross Sections

Appendix D

HEC-RAS Model Result Profiles