Armstrong Dam Removal

FEASIBILITY STUDY

Monatiquot River - Braintree, Massachusetts

Prepared for: *In partnership with:*

Prepared by:

GOMEZ AND SULLIVAN ENGINEERS

FINAL REPORT DECEMBER 2016

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Armstrong Dam Removal

FEASIBILITY STUDY – DRAFT REPORT

EXECUTIVE SUMMARY

Project Purpose

The purpose of this project is to evaluate the feasibility of restoring access for diadromous[1] fish to historically native habitats in the Upper Fore River Basin including its tributaries, the Monatiquot, Farm and Cochato Rivers by removing the Armstrong Dam, also known as Hollingsworth Dam, in Braintree, MA. The ultimate goal is restoring river herring access to the 180-acre Great Pond Reservoir in the Farm River basin headwaters. Great Pond serves as a water supply source for the towns of Braintree, Holbrook and Randolph (Tri-Town Water Board). A recent river herring and spawning nursery habitat assessment (Chase et al. 2015) was conducted that demonstrated Great Pond provides suitable river herring spawning habitat.

Imagery Credit: Duane Raver/USFWS

Species targeted for restoration include both species of river herring (blueback herring and alewife), American eel, and sea lamprey, all of which are diadromous fish that depend upon passage between marine and freshwater habitats to complete their life cycle. Reasons for pursuing fish passage restoration in the Upper Fore River Basin include the importance and historical presence of the target species, the connectivity of and significant potential habitat within the watershed, and active public input and support.

F.X. Messina Enterprises, the Armstrong Dam owner, is interested in removing the dam to eliminate maintenance costs and liability and potentially developing the site. The Town of Braintree (Town) want to improve the lands adjacent to the pond created by the dam and to provide public access to the river. These interests bring in the goals of connectivity and resiliency.

The impact of diadromous fish species extends far beyond the scope of a single restoration project, as they have a broad migratory range along the Atlantic coast and benefit commercial and recreational fisheries of other species.

[1] Diadromous fish include both anadromous and catadromous fish, which are collectively called migratory fish. Anadromous fish, such as river herring, spawn in freshwater and return to the ocean. Alternatively, catadromous fish, such as American eel, spawn in the ocean and migrate to freshwater.

Project Support & Outreach

This project has been led by the Massachusetts Division of Marine Fisheries (*MarineFisheries*) with support from the United States Fish and Wildlife Service (USFWS), F.X. Messina Enterprises (Messina), the Fore River Watershed Association (FRWA) and the Town. Gomez and Sullivan Engineers, DPC (Gomez and Sullivan or GSE) was contracted to conduct the study, which involved developing an existing conditions plan, hydraulic analysis, sediment management, sediment transport and bridge scour analysis, this feasibility report and final public meeting. LEC Environmental Consultants Inc., was subcontracted to conduct a wetlands delineation.

Public involvement is paramount in the process of restoring diadromous fish to the Upper Fore River Watershed and its tributaries. Public input has been, or will be, actively solicited at the following stages in the timeline of the broader restoration effort surrounding this feasibility study (FS): feasibility phase (this study), additional feasibility and consultation phase, design phase, and permitting.

Project History

In 2009, *MarineFisheries* contracted with Gomez and Sullivan to conduct a FS for restoring populations of river herring to the Fore River system (GSE, 2009). In 2009, Gomez and Sullivan Engineers completed a FS that examined three barriers to migratory fish passage near the Armstrong Dam[2] including, in downstream to upstream order: a natural falls called "Rock Falls", Ames Pond Dam[3] and Armstrong Dam—the total distance between the lower Rock Falls and Armstrong Dam is approximately 780 feet.

The Fore River Basin is located south of Boston and primarily includes the towns of Braintree, Randolph, Holbrook, Quincy, and Weymouth. The main river draining into the Fore River Bay is the Monatiquot River. The Monatiquot River is formed by two primary tributaries, the Farm and Cochato Rivers.

The Monatiquot River historically contained a large run of alewife that spawned in Great Pond (Belding, 1921; and Franklin, 2003) (now a water supply reservoir) located in the headwaters; However, successful spawning runs ceased after the construction of dams during the industrial revolution. Although river herring were believed to be absent from the river system, *MarineFisheries* and the FRWA observed river herring at the base of Rock Falls below the Armstrong and Ames Pond Dams in the 1990s and 2000s. *MarineFisheries* suspects that river herring may be responding to a reduction in industrial water quality impairment and are presently spawning in marginal habitat in the main stem of the Monatiquot River near Route 93. Given these observations and the amount of potential spawning habitat further upstream of three physical barriers to fish passage on the Monatiquot River- Rock Falls, Ames Pond Dam and Armstrong Dam- a study was conducted to

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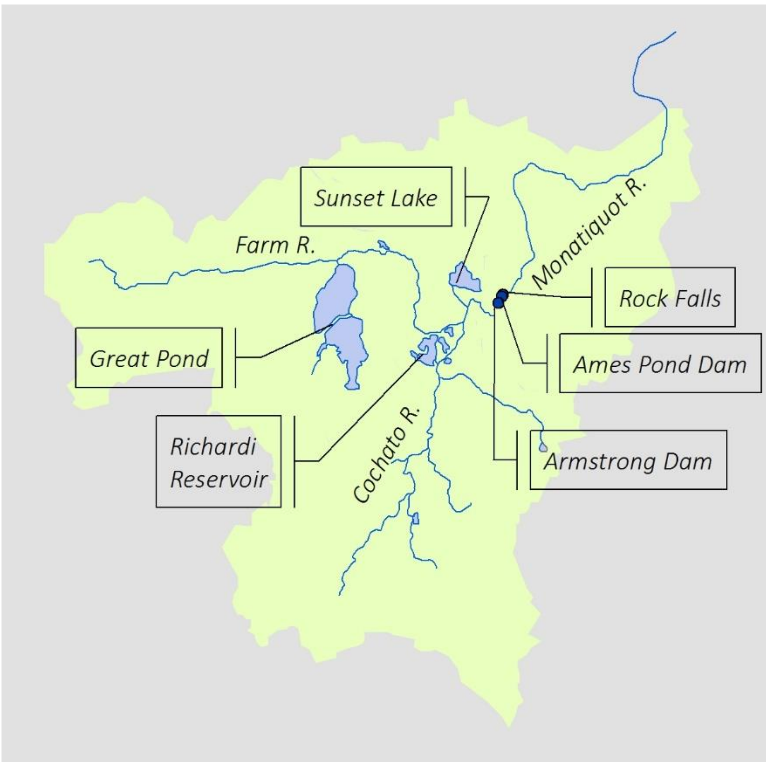
[2] In the 2009 report, the Armstrong Dam is also referred to as the Hollingsworth Dam. For purposes of this study, it was considered the Armstrong Dam.

[3] The Ames Pond Dam is also owned by F.X. Messina Enterprises.

evaluate the feasibility of restoring river herring to the headwater reservoirs[4]. The 2009 FS concluded that with assistance from water supply operations to support Great Pond outflow during the migration seasons and fish passage improvements at the three barriers (Rock Falls, Ames Pond Dam and Armstrong Dam), river herring restoration to the Great Pond Reservoir was feasible.

The 2009 FS did not include a detailed assessment of fish passage options at the 2-3 ft high Ames Pond Dam, located approximately 100 ft upstream of the Rock Falls. The FS did evaluate a fish ladder alternative at the Armstrong Dam located approximately 560 ft upstream of the Ames Pond Dam. The FS did not evaluate the feasibility of removing the 12 ft high Armstrong Dam, but included recommended additional steps needed to proceed with a dam removal option.



Following the 2009 FS, *MarineFisheries* contracted a hydraulic engineer to produce scoping designs for passage improvements at the Rock Falls, Armstrong Dam and Great Pond Reservoir. Fish ladder designs were prepared for the latter two locations during 2011-2012. Relative to Rock Falls, the FS concluded that a previously filled channel around Rock Falls could be restored to facilitate a natural passage around the barrier. Rock Falls designs for a bypass and in-stream weirs were evaluated by *MarineFisheries'* contracted engineer; however, both had fish passage limitations over the expected range of flows. *MarineFisheries* has recently conducted additional measurements of Rock Falls elevations during the very low summer flows of 2015 and 2016 and is now evaluating minor adjustments to the Rock Falls notch and crest as a potential solution for fish passage.

Beginning in 2012, some of the recommended steps outlined in the 2009 FS relative to removal of the Armstrong Dam were initiated including a structural assessment of the dam (Root Engineering, 2014), analytical sediment testing at four locations in the Armstrong Dam pond, and the preparation of a bathymetric map of the pond (Loureiro, 2012). The sediment testing was conducted to evaluate the sediment quality and potential liabilities associated with the exposure and mobilization of sediment

[4] The study examined restoring river herring to Great Pond and Sunset Lake Pond. Fish passage into Sunset Lake would require herring to migrate from the Farm River, up Sunset Lake "canal", through the Pond Street culvert and then the dam. The largest challenge for fish passage to Sunset Lake is the lack of a sufficient flow from the small drainage river. Given this, emphasis was placed on restoring herring to Great Pond.

relative to the Massachusetts Contingency Plan (MCP; 30 CMR 40.0000). The post-2009 FS tasks provided additional incentives for Messina and Project Partners to pursue removal of the Armstrong Dam as the preferred option to fish passage.

Feasibility Study Overview

This FS included:

- Developing an Existing Conditions Plan: the plan is needed for any future engineering drawings and permitting requirements. Additional field survey was required to develop the existing conditions plan.

- Hydraulic Analysis: a hydraulic model of the reach from just below Rock Falls to just upstream of the Cochato and Farm River confluences was developed as part of the 2009 FS. The hydraulic model was supplemented as part of this study in the area through the Armstrong Pond and between the upstream head of the pond and the confluence of the Cochato and Farm Rivers. Additional transect data were obtained as were drawings of the four bridges located between Armstrong Dam and the confluence with the Cochato and Farm Rivers to verify the geometry of the bridges included in the hydraulic model.

- A Wetlands Delineation, which is needed if engineering design and permitting is pursued.

- Sediment Management: this included a) sediment probing to compute the total sediment volume in the Armstrong Dam Pond, b) development of a sediment sampling plan as additional sediment sampling was required, and c) further testing and analysis of sediment.

- Sediment Transport and Bridge Scour analysis: this included a) estimating the mobile sediment volume likely to be transported with the dam removed, and b) a scour analysis at the bridges between the pond and Cochato and Farm Rivers to determine if removal of Armstrong Dam could present a risk to bridge abutments and/or piers.

- Draft and Final Report.

- Public Meeting: a public meeting was held on November 7, 2016 at the Metropolitan Yacht Club to review the study findings.

Site Background

This FS focused on the removal of the Armstrong Dam and the section of the Monatiquot River from the stone arch Massachusetts Bay Transportation Authority (MBTA) Railroad Bridge located immediately downstream of Rock Falls upstream to the confluence of the Farm and Cochato Rivers.

Rock Falls

Just upstream of the stone arch MBTA Railroad Bridge is Rock Falls, which represents the current upstream extent of river herring migration. It consists of a vertical drop of 4 feet over approximately

10 feet of ledge. Below the water falls is a large plunge pool just upstream of the MBTA Railroad Bridge. *MarineFisheries* is presently investigating lowering the crest elevation of the falls as a fish passage option.

Ames Pond Dam

Upstream of Rock Falls is the Ames Pond Dam, which is approximately 2-3 feet high and approximately 50 feet long. It consists of seven bays; the three center bays have lower sill elevations than the two bays flanking each side of the center bays. The three center bays convey most of the flow and only under high flows is water conveyed through all of the bays. Directly below the Ames Pond Dam is a plunge pool formed by a series of rocks in a near semicircle. Only minor changes were deemed necessary to achieve river herring passage at Ames Pond Dam (GSE, 2009); however, the project now seeks full removal as part of site improvements integrated with the Armstrong Dam removal construction.

Located approximately 560 feet upstream of the Ames Pond Dam is Armstrong Dam, the major obstacle for restoring river herring to the Monatiquot River system. Armstrong Dam sits below a large abandoned brick building, known as the Armstrong Cork Building which is owned by Messina. The dam is classified as an Intermediate sized, High hazard potential structure. According to the most recent dam safety inspection, the dam was found to be in “fair” condition (Root Engineering, 2014).

The dam is approximately 12 feet high and 92 feet long. It consists of nine bays separated by concrete piers extending from the spillway crest to the low chord of the building. It is assumed the piers act as structural supports for the building, although no structural analysis was conducted as part of this FS. Based on survey along the upstream face, the dam appears to angle downward into the pond at a slope of approximately 0.3 horizontal to 1 vertical (0.3H:1V). The spillway is comprised of concrete and it likely sits on bedrock based on observations/pictures below the dam and based on sediment depth probing conducted immediately upstream of the dam.

The left[5] and right abutments are formed by the Armstrong Cork Building. As mentioned above, the concrete piers form the rectangular bays at the top of the spillway and contain slots for stoplogs[6]. The

[5] All directions are given facing in the downstream direction.

[6] Stoplogs are long rectangular timber beams or boards placed on top of each other and dropped into slots inside a weir, gate, or channel causing the upstream water elevation to rise.

piers also support a metal catwalk spanning the length of the dam. The opening of the bays (and the piers) vary in dimensions, but the inside to inside dimensions are on average 8 feet. There are eight main bays on the spillway and one auxiliary bay at a lower elevation.

Armstrong Dam

Five feet in front of the dam, the auxiliary bay acts as a low level outlet. The outlet is a concrete box, which water spills into, leading to a four-foot diameter culvert pipe running through the dam. The low level outlet has stoplogs at its entrance (6.2 feet wide). Downstream of the dam and beneath the building, structural vertical support columns extend to the riverbed.

The building was initially a saw and grist mill in the 1700s. The mill was then bought and operated by the Boston and Braintree Copper and Brass Manufactory in 1823 before being sold to Hollingsworth Interests in 1832 for paper manufacturing. Old ropes were used to make manila paper starting in 1841. The site was sold to Monatiquot Rubber Works in 1901, later becoming the Stedman Rubber Flooring Company. The Armstrong Cork Company took the site over in 1936, later to be known as the Armstrong World Industries (AWI) in the 1950s. AWI used the river water solely for industrial cooling, but manufactured linoleum flooring at the mill until shutting down in the 1990s (Frazier, 1985).

Based on Geographic Information System (GIS) mapping, Armstrong Dam creates an approximate 3.8-acre pond. Based on bathymetric mapping (described later), the gross storage capacity is approximately 19 acre-feet as measured at elevation 94.1 feet or at the spillway crest. The water level impounded by the Armstrong Dam backwaters to just upstream of the Plain Street Bridge. The pond is hemmed in by some narrow wetland areas, but is mostly surrounded by developed areas. The abandoned Armstrong Cork Building and the Armstrong Dam are to the north, a Massachusetts Registry of Motor Vehicles (RMV), the Bayshore Athletic Club and parking lot are to the east, and the MBTA Old Colony Line Middleboro Branch runs to the west.

The dam is operated in a run-of-river mode, meaning that inflow equals outflow on a near instantaneous basis. The dam does not provide any flood protection; in fact, under flooding conditions the dam acts to increase the water level and area of inundation upstream of the dam.

Feasibility Study Findings

The no action alternative assumes that the Armstrong Dam would remain in place. The dam serves as a physical barrier to the free movement of fish and other aquatic resources, and specifically the movement of migratory fish such as river herring and American eel. The dam prevents migratory fish from accessing historic spawning, foraging, and nursery areas within the Upper Monatiquot River and its tributaries (Franklin, 2003). Resident freshwater fish that move up and down a river to find suitable spawning, rearing, and foraging habitat are also affected. In addition to serving as physical barriers to fish passage, the dam creates a pond that inundates riverine fish habitat. Lastly, accumulated sediment may potentially degrade fish spawning and nursery habitat.

Providing fish passage at the Armstrong Dam would open up approximately 5,200 feet of free-flowing habitat on the Monatiquot River plus miles of potential habitat on the tributaries.

Removal of the Armstrong Dam will presumably eliminate a barrier to upstream and downstream fish passage and would open up approximately 5,200 feet of free-flowing habitat on the Monatiquot River. What remains unclear at this juncture is whether the bedrock profile located directly beneath the dam results in a "natural" vertical barrier to fish passage. Many dams are purposely positioned on natural falls, and until further assessment is conducted it is unclear if a natural barrier exists[7]. If the dam removal alternative progresses to the next level of feasibility study, ground-penetrating radar (GPR) is recommended for the dam. GPR could be conducted above the primary spillway to attempt to map the upper surface of bedrock beneath the dam. This information could then be added to the hydraulic model to more accurately predict whether the falls will impede fish passage under a range of flows and lead to scour at the upstream bridges.

Other issues relative to fish passage, and flow management need to be resolved both downstream and upstream of the dam. Downstream of the Armstrong Dam, the Rock Falls represent a vertical barrier to river herring restoration. This barrier would need to be resolved such that river herring can ascend to the Armstrong Dam area. Ames Pond Dam represents a velocity barrier under high flows and its removal is included as a project goal to be developed concurrently with the Armstrong Dam removal.

Above the Armstrong Dam pond, are two water supply reservoirs. A diversion dam is located on the Farm River that diverts flow into Richardi Reservoir (see inset above). Water contained in Richardi Reservoir is subsequently pumped to Great Pond for water supply. Great Pond has suitable conditions for adult river herring to permit spawning and growth of juvenile river herring (Chase et al. 2015). Upstream and downstream fish passage structures at Great Pond are undergoing permitting for construction in 2017. The fish passage facilities at the Great Pond Dam will need coordinated operations with water supply practices to account for the fluctuation in water levels and the seasonal migratory needs of river herring. The most critical migration period will be the fall emigration of juvenile river herring. Juvenile herring will rear and grow during the summer within Great Pond, and in the fall migrate from freshwater to the ocean to grow as adults. This timing can coincide with seasonal low flows.

Results of hydraulic modeling indicate that under average to high flows during the migratory fish passage season and with the dam removed, there appears to be sufficient depths and velocities to permit upstream fish passage through the river reach that would convert from a pond to free-flowing with the dam removed.

During low flow conditions in September, when juvenile river herring are migrating back to the ocean, there are a few locations where water depth (per the hydraulic model) was less than 0.5 foot. Therefore,

[7] The historical record indicates that herring were once able to spawn at Great Pond, this would seem to indicate that fish were able to pass upstream of the dam's location, however the course of the river may have shifted due to the construction of the dam and building, or sediment loading. Additionally, a shifting baseline in hydrological conditions could alter the hydraulics at the site.

the timing of flow releases to assist juvenile emigration will require coordinated operations with the water supply. No minimum flow requirements exist for Great Pond Dam. Instead, *MarineFisheries* and the Tri-Town Board will coordinate to facilitate downstream passage. It is highly recommended that discharges designed to move juvenile river herring downstream be coordinated with high flow precipitation events. Thus, the concern of having adequate river depths will be reduced if the release is coordinated with targeted river flow conditions.

Next Steps

The project will advance to future phases of securing funding, additional feasibility work, consultation with interested parties, sediment management, engineering design, preparation and submittal of permit packages, and construction to ultimately restore diadromous fish passage to the Fore River Watershed.

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LIST OF ABBREVIATIONS

AUL	activity and site use limitation
AWI	Armstrong World Industries
BLSF	bordering land subject to flooding
BVW	bordering vegetated wetlands
CY	cubic yards
CFR	Code of Federal Regulations
CMR	Code of Massachusetts Regulations
cfs	cubic feet per second
CVP	certified vernal pool
CWA	federal Clean Water Act of 1973
DDT	Dichloro-diphenyl-trichloroethane
DS	downstream
DFW	Massachusetts Division of Fisheries and Wildlife
DO	Dissolved Oxygen
DPW	Department of Public Works
DTC	Diversified Technology Consultants
El	elevation
EPA	Environmental Protection Agency
EPH	extractable petroleum hydrocarbons
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FRWA	Fore River Watershed Association
FS	feasibility study
ft	feet
ft/s	feet per second
GIS	Geographic Information System
Gomez and Sullivan or GSE	Gomez and Sullivan Engineers, DPC
GPR	ground penetrating radar
GPS	global positioning system
HEC-RAS	Hydraulic Engineering Center River Analysis System
LEC	LEC Environmental Consultants, Inc.
LiDAR	Light Detection and Ranging
LUW	land under water
MAHW	mean annual high water
MarineFisheries	Massachusetts Division of Marine Fisheries
MassDEP	Massachusetts Department of Environmental Protection
MassDOT	Massachusetts Department of Transportation
MassGIS	Massachusetts Office of Geographic Information
MassODS	Massachusetts Office of Dam Safety
MBTA	Massachusetts Bay Transportation Authority
Messina	F.X. Messina Enterprises
MCP	Massachusetts Contingency Plan
MESA	Massachusetts Endangered Species Act
MGD	million gallons per day

MHC	Massachusetts Historical Commission
mi^2	square miles
NAVD 88	North American Vertical Datum of 1988 (datum used in this report)
NGVD 29	National Geodetic Vertical Datum of 1929
NHESP	Natural Heritage & Endangered Species Program
NID	National Inventory of Dams
ODS	Office of Dam Safety
OHW	ordinary high water
PAH	polycyclic aromatic hydrocarbons
PCB	polychlorinated biphenyl
PEC	probable effects concentration
PMF	probable maximum flood
psi	pounds per square inch
PVC	polyvinyl chloride
PVP	potential vernal pool
RMV	Massachusetts Registry of Motor Vehicles
RTK	real-time kinematic
TEC	threshold effects concentration
Town	Town of Braintree, Massachusetts
TRI	Toxic Release Inventory
US	upstream
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
VOC	volatile organic carbon
WLL	water level logger
WPA	Massachusetts Wetland Protection Act
WSEL	water surface elevation level
WSP	Water Surface Profiles

1. Introduction

1.1 Project History

In 2009, the Massachusetts Division of Marine Fisheries (*MarineFisheries*) contracted with Gomez and Sullivan Engineers, DPC (Gomez and Sullivan or GSE) to conduct a feasibility study for restoring populations of river herring to the Fore River system (GSE, 2009). The Fore River Basin is located south of Boston and primarily includes the towns of Braintree, Randolph, Holbrook, Quincy, and Weymouth. The main river draining into the Fore River Bay is the Monatiquot River. The Monatiquot River is formed by two primary tributaries, the Farm and Cochato Rivers. Shown in Figure 1.1-1 is a layout of the watershed and the proposed migration route for river herring. Shown in Figure 1.1-2 are the Farm River, Cochato River and Monatiquot River drainage areas.

The Monatiquot River historically contained a large run of alewife (*Alosa pseudoharengus*) that spawned in Great Pond (Belding, 1921; and Franklin, 2003) (now a water supply reservoir) located in the headwaters. However, successful spawning runs ceased after the construction of dams during the industrial revolution. Although river herring were believed to be absent from the river system, *MarineFisheries* and the Fore River Watershed Association (FRWA) observed river herring at the natural falls[8] below the Armstrong[9] and Ames Pond Dams in the 1990s and 2000s (see Figure 1.1-3). *MarineFisheries* believes that river herring are spawning in marginal habitat in the main stem of the Monatiquot River near Route 3. Given these observations and the amount of potential spawning habitat further upstream of three physical barriers to fish passage on the Monatiquot River- Rock Falls, Ames Pond Dam and Armstrong Dam- a study was conducted to evaluate the feasibility of restoring river herring to the headwater reservoirs[10]. A final feasibility report and public meeting were held in 2009. In addition to the feasibility study (FS), a river herring spawning and nursery habitat assessment (Chase et al., 2015) was conducted demonstrating suitable herring habitat in the 180-acre Great Pond Reservoir.

Based on the study findings, it was concluded that with assistance from water supply operations to support Great Pond outflow during the during the migration seasons and fish passage improvements at the three barriers (Rock Falls, Ames Pond Dam and Armstrong Dam), river herring restoration to the Great Pond Reservoir was feasible. As part of the 2009 FS, it was concluded that with some stream restoration measures to facilitate herring passage around Rock Falls, and removal and/or installation of fish passage facilities at the Ames Pond and Armstrong Dams, upstream fish passage was possible. The Ames Pond and Armstrong Dams are owned by F.X. Messina Enterprises (Messina), a real estate firm located in Braintree. The 2009 FS did not include a detailed assessment of fish passage options at the 2-3 ft high Ames Pond Dam, located approximately 100 ft upstream of the Rock Falls. The FS did evaluate a fish ladder alternative at the Armstrong Dam located approximately 560 ft upstream of the Ames Pond Dam. The FS did not evaluate the feasibility of removing the 12 ft high Armstrong Dam, but included recommended additional steps needed to proceed with a dam removal option.

[8] For purposes of this proposal we have referred to the natural falls as “Rock Falls”.

[9] In the 2009 report, the Armstrong Dam is also referred to as the Hollingsworth Dam. For purposes of this study, it was considered the Armstrong Dam.

[10] The study examined restoring river herring to Great Pond and Sunset Lake Pond. Fish passage into Sunset Lake would require herring to migrate from the Farm River, up Sunset Lake “canal”, through the Pond Street culvert and then the dam. The largest challenge for fish passage to Sunset Lake is the lack of a sufficient flow from the small drainage river. Given this, emphasis was placed on restoring herring to Great Pond.

Following the 2009 FS, *MarineFisheries* contracted a hydraulic engineer to produce scoping designs for passage improvements at the Rock Falls, Armstrong Dam and Great Pond Reservoir. Fish ladder designs were prepared for the latter two locations during 2011-2012. Relative to Rock Falls, the FS concluded that a previously filled channel around Rock Falls could be restored to facilitate a natural passage around the barrier. Rock Falls designs for a bypass and in-stream weirs were evaluated by *MarineFisheries'* contracted engineer; however, both had fish passage limitations over the expected range of flows. *MarineFisheries* has recently conducted additional measurements of Rock Falls elevations during the very low summer flows of 2015 and 2016 and is now evaluating minor adjustments to the Rock Falls notch and crest as a solution for fish passage.

Beginning in 2012, some of the recommended steps outlined in the 2009 FS relative to removal of the Armstrong Dam were initiated including a structural assessment of the dam (Root Engineering, 2014), analytical sediment testing at four locations in the Armstrong Dam pond, and the preparation of a bathymetric map of the pond (Loureiro, 2012). The sediment testing was conducted to evaluate the sediment quality and potential liabilities associated with the exposure and mobilization of sediment relative to the Massachusetts Contingency Plan (MCP; 30 CMR 40.0000). The structural analysis found that future maintenance was needed to meet the Massachusetts Office of Dam Safety (MassODS) standards. Both of these post-2009 FS tasks provided additional incentives for Messina and Project Partners to pursue removal of the Armstrong Dam as the preferred option to fish passage.

Given the above, in 2015, *MarineFisheries* contracted with Gomez and Sullivan to conduct a partial FS for removing the Armstrong Dam. The study was conducted in collaboration with the following Project Partners:

- *MarineFisheries*
- United States Fish and Wildlife Service (USFWS)
- FRWA
- Messina
- Town of Braintree, MA (Town)
- MassBays Healthy Estuaries Grant Program

1.2 Current Project Overview

As part of the Armstrong Dam removal FS, the following steps were conducted as described later in this document:

- Develop Existing Conditions Plan: the plan is needed for any future engineering drawings and permit. Additional field survey was required to develop the existing conditions plan.

- Hydraulic Analysis: a hydraulic model of the reach from just below Rock Falls to just upstream of the Cochato and Farm River confluences was developed as part of the 2009 FS. The hydraulic model was supplemented as part of this study in the area through the Armstrong Pond and between the upstream head of the pond and the confluence of the Cochato and Farm Rivers. Additional transect data were obtained as were drawings of the four bridges located between Armstrong Dam and the confluence with the Cochato and Farm Rivers to verify the geometry of the bridges included in the hydraulic model.

- A wetlands delineation, which is needed if engineering design and permitting is pursued.

- Sediment Management: this included a) sediment probing to compute the total sediment volume in the Armstrong Dam Impoundment, b) development of a sediment sampling plan as additional sediment sampling was required, and c) further testing and analysis of sediment.

- Sediment Transport and Bridge Scour analysis: this included a) estimating the mobile sediment volume likely to be transported with the dam removed, and b) a scour analysis at the bridges between the pond and Cochato and Farm Rivers to determine if removal of Armstrong Dam could present a risk to bridge abutments and/or piers.

- Draft and Final Report.

- Public Meeting: a public meeting was held at the Metropolitan Yacht Club on November 7, 2016 to present the study findings. The meeting was followed by a three-week-long public comment period during which the draft feasibility report was made available digitally and in hardcopy at the Thayer Public Library. Two comments were received on the draft report and are compiled in Appendix L.

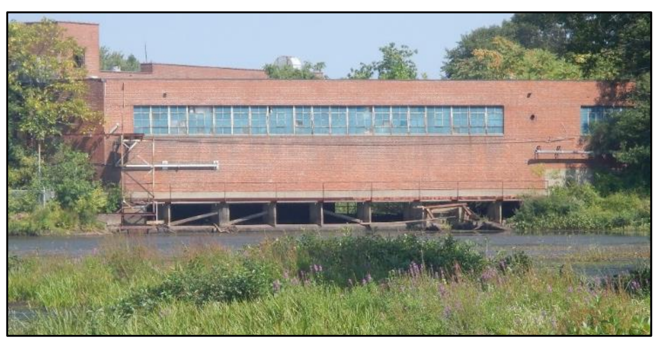
Note that above tasks do not include other tasks required to complete this FS. The additional feasibility related tasks are listed below and would be completed when additional funding becomes available.

- Additional sediment testing may be required. A final sediment management plan would be needed and would be informed by the review of an ecological risk assessor and in consultation with MassDEP.

In addition to the above, should the project proceed to removal, additional work would be necessary including engineering design, permitting, preparation of bid documents, basis-of-design memo, and potential construction oversight.

1.3 Assumptions and Limitations

In conducting this feasibility assessment, Gomez and Sullivan made the following assumptions:



- As shown in the photo inset, there is a building owned by Messina sitting atop the dam. In fact, between each bay opening of the dam, there is vertical concrete column extending from the top of the spillway crest to the low chord of the building. Based on discussions with Messina, the future use of this site will continue to contain a river crossing, with modifications or removal of the existing building. Messina only has a limited easement to enter the property from Hancock Street over the adjacent railroad property. Consequently, the current building provides the only means for accessing the northwestern side of their property. It was assumed that three new vertical columns would span the building and extend from the low chord of the building to the bedrock channel. Gomez and Sullivan did not conduct any structural analysis to determine if three vertical columns were structurally sufficient to carry the building load to the channel bed; the columns were included solely for hydraulic modeling purposes. It is assumed that if the dam removal were to proceed an independent structural assessment of the building would be conducted. In addition, no geotechnical investigation was conducted where the columns would be tied into the ground.

- This FS focused solely on the Armstrong Dam. Existing vertical barriers located below Armstrong Dam—the Ames Pond Dam and Rock Falls – would need modifications to permit upstream fish passage.

- While the project objectives currently intend to remove the Ames Pond Dam, for the hydraulic modeling of this FS, it was assumed that the Ames Pond Dam remained in place. We make note of this because the hydraulics to the base of the Armstrong Dam can be influenced by the backwater caused by the Ames Pond Dam under high flows. If the Ames Pond Dam were removed, further hydraulic modeling is needed.

- For the hydraulic modeling, it was assumed that the hydrological analysis from the 2009 FS is still accurate and flows from the Farm and Cochato Rivers remain the same. The Cochato River once fed Richardi Reservoir prior to the Baird & McQuire chemical manufacturing facility being declared a superfund site. Earthen dykes have since been constructed around Richardi to keep Cochato River water from entering. If the Baird & McQuire site is sufficiently remediated, it is not known if the water supply will seek to use Cochato River and once again divert the flow to the Richardi Reservoir.

1.4 Report Format

The report consists of text and smaller figures and tables. Larger tables, maps and aerial images are included at the end of each section; smaller are included in the text. The report includes the following Appendices:

- Appendix A contains representative photographs of the site.
- Appendix B includes the existing conditions plan.
- Appendix C includes historic drawings of the Armstrong Dam obtained from Messina.
- Appendix D includes drawings and inspection reports obtained from the Massachusetts Department of Transportation (MassDOT) and Massachusetts Bay Transportation Authority (MBTA) relevant to bridges in the area of interest.
- Appendix E contains water and sewer main drawings and maps obtained from the Town
- Appendix F contains a Wetland Report by LEC Environmental Consultants, Inc.
- Appendix G contains Environmental Investigation by Loureiro Engineering Associates, Inc.
- Appendix H contains the Bathymetric Map of the Hollingsworth pond[11] conducted by Alpha Surveying and Engineering, Inc.
- Appendix I contains the Approved Sediment Sampling Plan.
- Appendix J contains the results from the sediment samples processed by Alpha Analytical, Inc.
- Appendix K contains the Conceptual Rendering of Dam Removal provided by Messina.
- Appendix L includes comment letters filed on the Draft Feasibility Report

[11] Note that the Hollingsworth Pond and the Armstrong Dam Impoundment are both the same water body.

1.5 Survey Datum

As described below, a survey of the dam and other infrastructure was conducted as part of this study. The vertical control of the survey is based on the North American Vertical Datum of 1988 (NAVD88). All elevations reported herein are based on NAVD88, unless otherwise noted. The horizontal datum is North American Datum 1983 (NAD83), registered to the Massachusetts State Plane Coordinate System, Mainland Zone (Federal Information Processing Standard Zone 2001). Units are feet (ft).

1.6 Prior Reports

Prior to this study, the following evaluations have been conducted at the site:

- Feasibility Analysis for Restoring River Herring to the Fore River (GSE, 2009)
- Hazard Classification of the Armstrong Dam (Fuss & O'Neill, 2006)
- Structural Assessment of the Armstrong Dam (Root Engineering, 2014)
- Environmental Investigation of Hollingsworth Pond (Loureiro, 2012)
- Bathymetric Map of the Impoundment (Alpha Surveying and Engineering Inc., 2006)

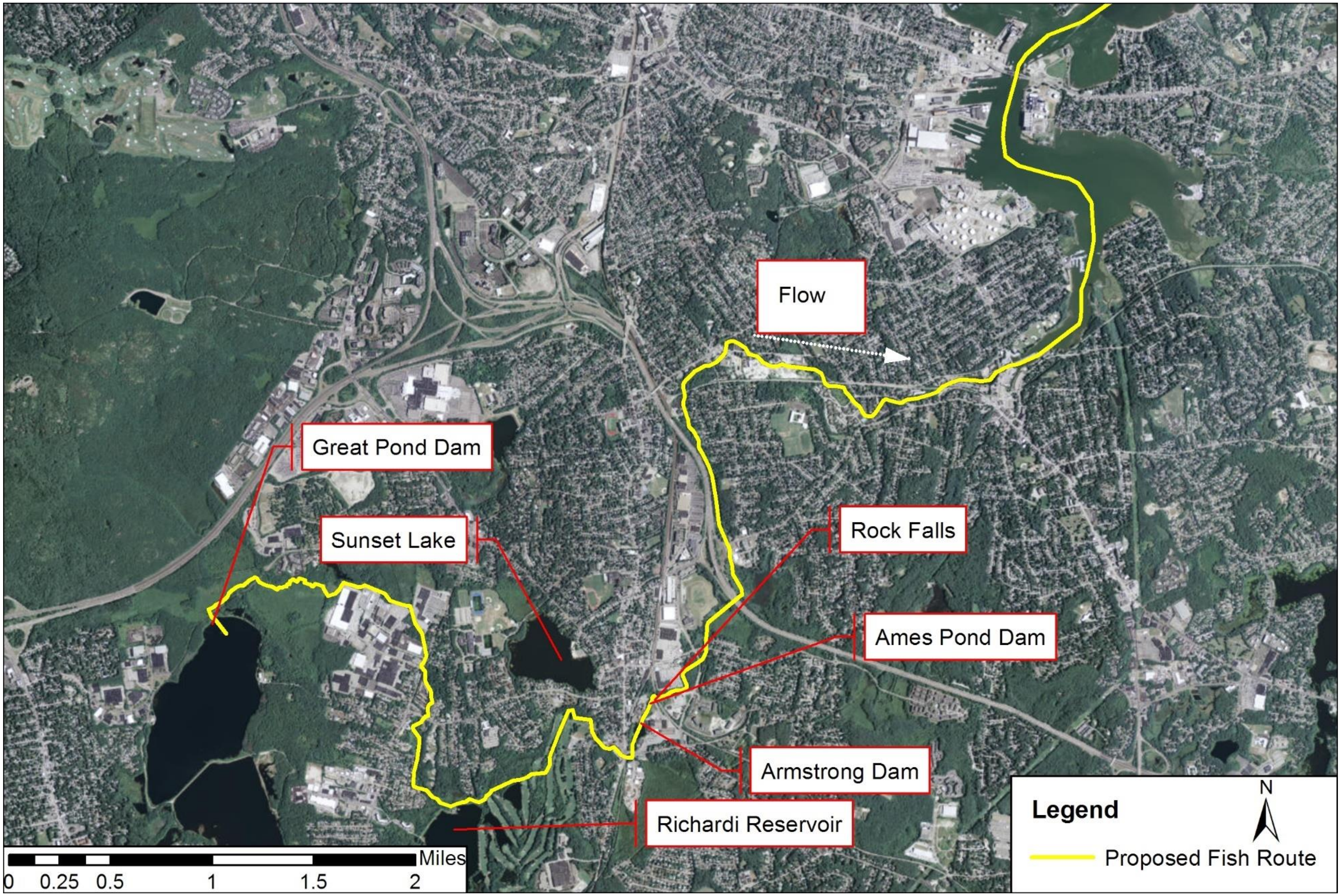


Figure 1.1-1: Proposed Fish Migration Route

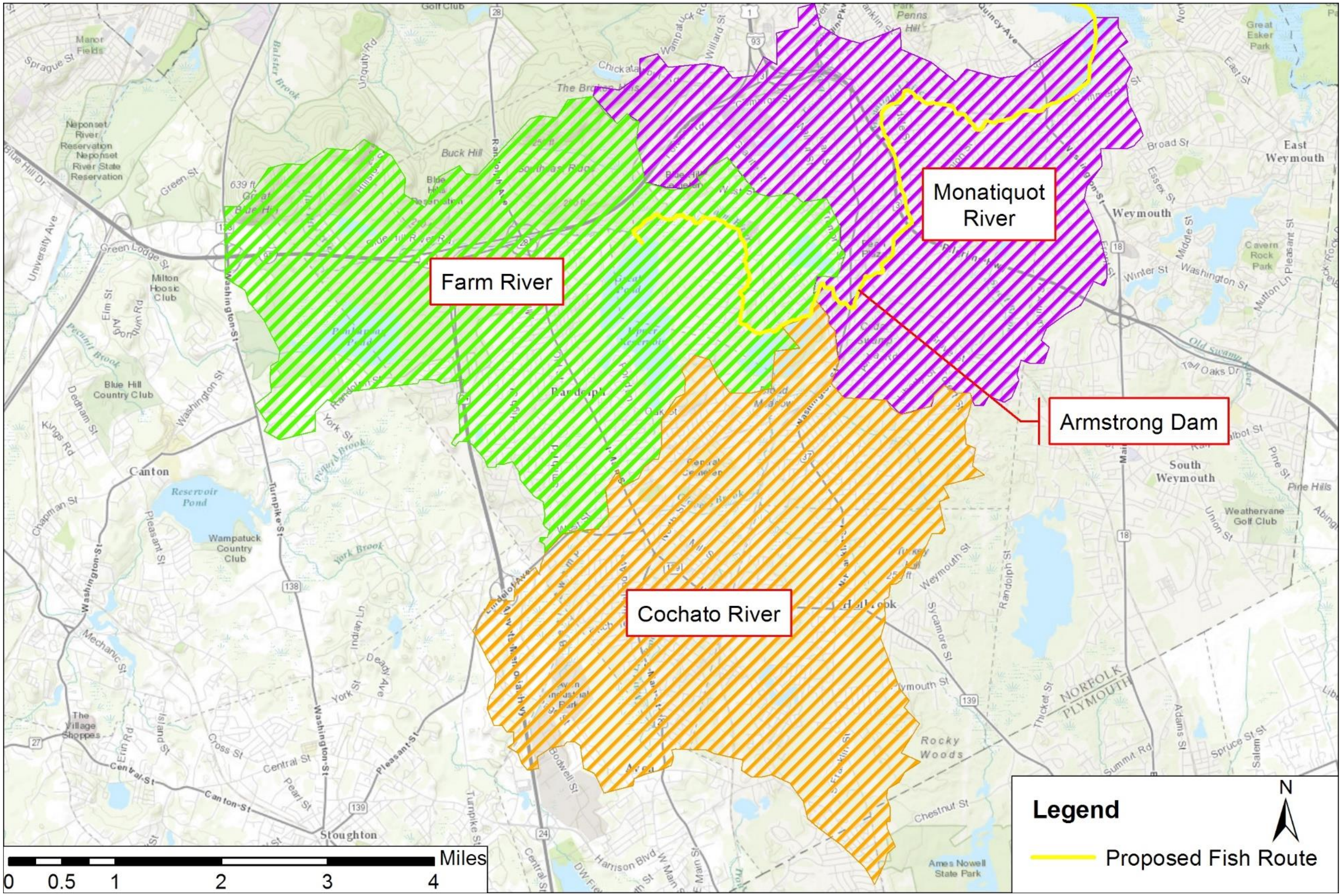


Figure 1.1-2: Drainage Areas of the Cochato, Farm and Monatiquot Rivers

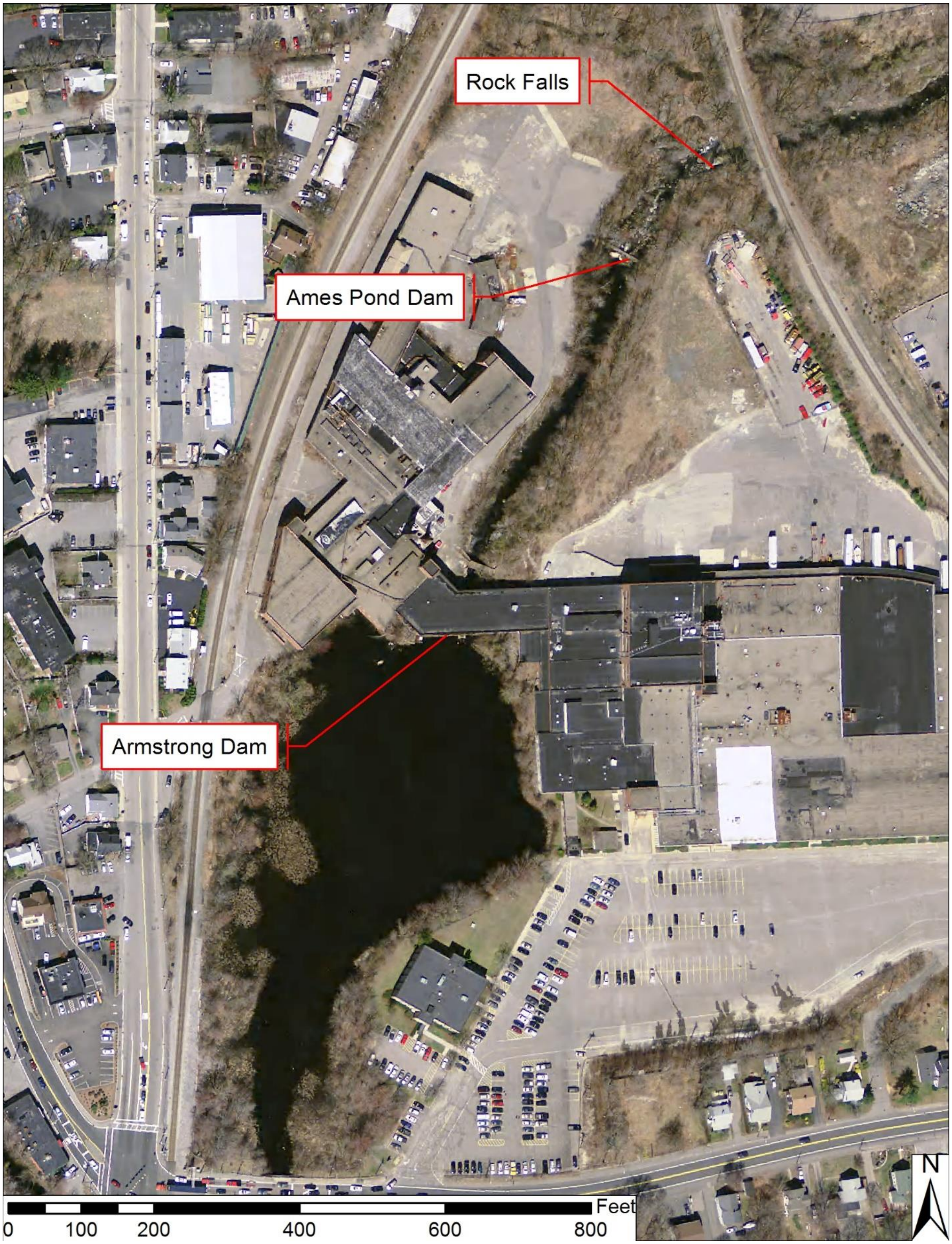


Figure 1.1-3: River Features

2. Project Settings

2.1 Project Area

This section includes a description of the main features in the Project area from the Massachusetts Bay Transportation Authority (MBTA) Railroad Bridge located immediately downstream of Rock Falls upstream to the confluence of the Farm and Cochato Rivers. Project area maps and aerial imagery denoting the key features described below are shown in Figures 1.1-2 to 2.1-3. Appendix A contains several numbered photographs, which are referenced throughout this section.

2.2 Monatiquot River Features - Downstream of Armstrong Dam

The hydraulic modeling study reach extends from just upstream MBTA Railroad Bridge[12] (Photos 1 and 2) to the lower reaches of the Cochato and Farm Rivers. Just upstream of the MBTA Railroad Bridge is Rock Falls, which represents the current upstream extent of river herring migration. It consists of a near vertical drop of approximately 4 feet through ledge (Photo 3). Below the water falls is a large plunge pool just upstream of the MBTA Railroad Bridge. As noted earlier, the 2009 FS evaluated options for passing river herring upstream of Rock Falls by revitalizing the nearby bypass channel (Photo 3 – right side) and *MarineFisheries* has been investigating the option of lowering the crest elevation.

Upstream of Rock Falls is the Ames Pond Dam (Photo 4). The dam is approximately 2-3 feet high and approximately 50 feet long. It consists of seven bays; the three center bays have lower sill elevations than the two bays flanking each side of the center bays. The three center bays convey most of the flow and only under high flows is water conveyed through all of the bays. Directly below the Ames Pond Dam is a plunge pool formed by a series of rocks in a near semicircle. The 2009 FS indicated that only minor changes are necessary to achieve river herring passage at Ames Pond Dam. The dam is operated in a run-of-river mode, meaning inflow to the dam instantaneously equals outflow; no regulation of flow exists, and the “impoundment” created by the dam is minimal.

2.3 Armstrong Dam, Impoundment and Area

Located approximately 560 feet upstream of the Ames Pond Dam is Armstrong Dam, the major obstacle for restoring river herring to the Monatiquot River system. Armstrong Dam sits below a large abandoned brick building, known as the Armstrong Cork Building (Figure 2.1-3 and Photo 5), which is owned by Messina. The dam is approximately 12 feet high and 92 feet long. It consists of nine bays separated by concrete piers extending from the spillway crest to the low chord of the building. It is assumed the piers act as structural supports for the building, although no structural analysis was conducted as part of this FS. The spillway crest elevation is 94.1 feet NAVD88, though it is uneven due to settling. A current plan and profile drawing of the dam is shown in Drawing 4 of Appendix B; historical drawings[13] of the dam are shown in Appendix C. Based on survey along the upstream face, the dam appears to angle downward into the pond at a slope of approximately 0.3 horizontal to 1 vertical (0.3H:1V). The spillway

[12] There are two MBTA railroad bridges in this study. This is the downstream one (Bridge No. B-21-041).

[13] The historical drawing is circa 1945 by the Armstrong Cork Co. Engineering Department (Lancaster, PA). The vertical datum is unknown; however, by choosing a common point such as the crest of the dam elevation 100.83 ft on the historical plans and 94.1 ft NAVD88 via survey, a correction factor to bring the historical drawings into the project datum can be found by using the difference in elevations.

is comprised of concrete[14] and it likely sits on bedrock based on pictures below the dam (Photos 6 and 7) and based on sediment depth probing conducted immediately upstream of the dam.

The left[15] and right abutments are formed by the Armstrong Cork Building. As mentioned above, the concrete piers form the rectangular bays at the top of the spillway (Photo 8) and contain slots for stoplogs[16]. Stoplogs were not installed in the bays during the field work. The piers also support a metal catwalk (Photo 9) spanning the length of the dam. The opening of the bays (and the piers) vary in dimensions, but the inside to inside dimensions are on average 8 feet. There are eight main bays on the spillway and one auxiliary bay at a lower elevation.

About five feet in front of the dam, the auxiliary bay acts as a low level outlet (Photos 10 and 11). The outlet is a concrete box, which water spills into, leading to a four-foot diameter culvert pipe running through the dam. The low level outlet has stoplogs at its entrance (about 6.2 feet wide), which were set to elevation 93.1 feet during the field survey. The invert elevation of the culvert pipe at its discharge point is 87.1 feet (Photo 11). The low level outlet appears to be the only operational structure for releasing water from the pond other than the spillway crest. Due to safety constraints obtaining the invert elevation of the entrance to the low level outlet (if all of the stop logs were removed) was not possible. However, the adjustable invert ranges approximately 2 – 6 feet below the primary spillway crest elevation, which provides some drawdown capacity. If dam removal were to occur in the future, it is recommended that all of the stop logs be removed prior to removal to further dewater the pond and allow sufficient time for vegetation to grow and help stabilize exposed sediment.

Downstream of the dam and beneath the building, structural vertical support columns extend to the riverbed (Photo 12 and Drawing 2). After the Monatiquot River passes under the building it crosses a pedestrian foot bridge (Photo 13).

Based on Geographic Information System (GIS) mapping, Armstrong Dam creates an approximate 3.8-acre pond. Based on bathymetric mapping conducted (described later), the gross storage capacity is approximately 19 acre-feet as measured at elevation 94.1 feet or at the spillway crest. The water level impounded by the Armstrong Dam backwaters to just upstream of the Plain Street Bridge. The pond is hemmed in by some narrow wetland areas, but is mostly surrounded by developed areas. The abandoned Armstrong Cork Building and the Armstrong Dam are to the north, a Massachusetts Registry of Motor Vehicles (RMV), the Bayshore Athletic Club and parking lot are to the east, and the MBTA Old Colony Line Middleboro Branch runs to the west.

Armstrong Dam is operated as a run-of-river facility, whereby inflow equals outflow on a near continuous basis. This means that water levels behind the dam are controlled by the low level outlet, but when inflows exceed the hydraulic capacity of the low level outlet, water is passed via the spillway crest. Armstrong Dam does not provide any flood protection as it has negligible storage. Removal of the Armstrong Dam would have no impact on the timing and magnitude river flows; however, the river width and depth upstream of the dam would decrease.

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[14] The interior composition of the dam is unknown and should be investigated as part of any potential final design.

[15] All directions are given facing in the downstream direction.

[16] Stoplogs are long rectangular timber beams or boards placed on top of each other and dropped into slots inside a weir, gate, or channel causing the upstream water elevation to rise.

2.4 Monatiquot River Features – Upstream of Dam

As described later, WLLs were installed at various locations in the Monatiquot, Farm and Cochato River to determine the approximate upstream extent of the Armstong Dam Pond based on the current configuration (stoplogs use) at the dam and under high flow conditions. By determining the upstream extent of the pond under high flows, the area of impact could be determine if the dam were removed. Based on WLL monitoring, it appears that the Armstrong Dam pond terminates at the upstream face of the Plain Street Bridge, which acts as a hydraulic control under high flow conditions. The Monatiquot River then extends upstream through another MBTA Railroad bridge, Washington Street Bridge, Jefferson Street Bridge and upstream through the Braintree Golf Course. A discussion of each bridge is provided in the next section.

Figure 2.1-1: Project Location Topographic Map

Figure 2.1-2: Project Location Map

Figure 2.1-3: Aerial Image of Site Looking West

3. Infrastructure and Resources

Whenever a dam is considered for removal, impacts to infrastructure in the impact area need to be evaluated as the river hydraulics (depth, width and velocity) in the former impounded reach will change upstream or downstream of the dam due to lower water levels, higher water velocities, and/or scouring of sediments. This section presents a summary of available existing information collected from background research and field work associated with this FS on the various instream infrastructure, including bridges, utilities and stormwater outfalls.

3.1 Bridges

Five bridges in the vicinity of the Armstrong Dam are discussed in this section. In downstream to upstream order they include:

- MBTA Railroad Bridge (below Rock Falls),
- Plain Street Bridge,
- MBTA Railroad Bridge
- Washington/Hancock Street Bridge and,
- Jefferson Street Bridge.

The MassDOT and MBTA were contacted to request bridge plans and inspection reports, which are included in Appendix D.

Downstream MBTA Railroad Bridge

The downstream MBTA Railroad Bridge (Bridge No. B-21-041, Photos 1 and 2) is located approximately 860 feet downstream of the Armstrong Dam. It is a stone masonry closed spandrel deck arch bridge that the Old Colony Plymouth Branch line tracks run over. According to a 2014 MBTA inspection report prepared by Diversified Technology Consultants (DTC), it was constructed around 1900 and underwent structural modifications to widen the arch by an additional 15 feet in 1994. (DTC, 2014) (Appendix D).

The bridge has a 21.5-foot clear distance at the spring line, or point where the arch starts to come together, and the width of the deck is about 44 feet. It has stone masonry walls and is supported by sloped and exposed ledge footing on the south (DTC, 2014). An underwater inspection report, also found in Appendix D, noted that there was some undermining along the northeast wingwall and minor scour was found along the toe of the north breastwall (Bourne Consulting Engineering, 2013). The main interest in this bridge is, if the preferred alternative for sediment management is to allow for the natural release of impounded sediment, could the released sediment become “clogged” at the bridge opening. Based on the bridge span width across the Monatiquot River and overall large bridge opening, clogging of sediment is not expected to impact the hydraulic capacity of the bridge opening.

Plain Street Bridge

The Plain Street Bridge (Bridge No. B-21-014, Photos 14 and 15), also known as the Corporal G.W. Reardon Memorial Bridge, is located approximately 960 feet upstream of the Armstrong Dam. Originally built in 1929, it was rebuilt in 1975. It is a pre-stressed concrete slab structure. It has a span of approximately 19.5 feet and its deck is about 58 feet wide. The bridge is cantilevered (no piers) across the Monatiquot River and is supported by a right and left abutment. The main interest of the bridges located upstream of the Armstrong Dam are the potential for scouring the bridge abutments (and/or piers) and creating structural issues as water velocities will increase and the channel bed could lower due to sediment transport. This potential impact is assessed later in this document.

Upstream MBTA Railroad Bridge

The Upstream MBTA Railroad Bridge (Bridge number B-21-042, Photos 16 and 17) is located approximately 1,260 feet upstream of the Armstrong Dam and approximately 300 feet upstream of the Plain Street Bridge. It carries the MBTA Old Colony Middleboro Branch line. Originally built in 1895, it was modified in 1994. It is a simple span bridge having two granite piers and two granite abutments; its deck is about 14.5 feet wide. In 1994, the piers on the upstream ends were capped with concrete.

Washington/Hancock Street Bridge

The Washington Street Bridge (Bridge No. B-21-16, Photos 18 and 19), also known as the George W Anastos Bridge, is located immediately upstream of the Upstream MBTA Railroad Bridge and approximately 1,330 feet upstream of the Armstrong Dam. It is a pre-stressed concrete slab bridge. Originally built in 1924, it was rebuilt in 1982. The bridge is cantilevered (no piers) across the Monatiquot River and is supported by a right and left abutment.

Jefferson Street Bridge

The Jefferson Street Bridge (Bridge No. B-21-54, Photo 20), also known as the Sergeant H. MacArthur Bridge, is located approximately 2,400 feet upstream of the Armstrong Dam and immediately downstream of the Braintree Golf Course. It was built in 1955 and is a single span structure with concrete abutments and no piers. The Jefferson Street Bridge spans 19.5 feet and the deck width is about 38 feet.

3.2 Utilities

The office of the Braintree Department of Public Works (DPW) Engineering Division was visited to obtain information on any utility lines upstream or near the Armstrong Dam that could be impacted by its removal. DPW provided Gomez and Sullivan with a CAD drawing of the project area, which included plans of the water and sewer mains in the Project Area (between the Cochato/Farm River confluences to the lower MBTA Railroad Bridge). The CAD drawing included planimetrics, including utility line information. If the Armstrong Dam were removed water velocities in the pond will increase, potentially leading to additional scour of the channel and potential exposure of buried water or sewer mains. As part of this FS, the potential for scouring any identified utility lines was evaluated later in this report.

The plans and maps obtained from the DPW are included in Appendix E. Some utility line information has been included in the Existing Conditions Plans in Drawing 2 of Appendix B. Note that private gas and electrical companies were not contacted regarding lines that may be located beneath the river channel and in the potential dam removal area. Should dam removal proceed, it is recommended that Dig Safe®[17] be contacted so that private utility lines are identified early on. Many of the utility lines shown on the Braintree CAD drawing appear to be affixed directly to the bridges crossing the Monatiquot River and not buried.

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[17] Dig Safe® is a not-for-profit clearinghouse that notifies participating utility companies of your plans to dig. In turn, these utilities (or their contract locating companies) respond to mark out the location of their underground facilities.

Water Main Crossing

There are three water main crossings upstream of Armstrong Dam. A 12-inch-diameter cast iron water main crosses the Monatiquot River at the upstream end of the pond. The main crosses near the downstream side of the Plain Street Bridge. It is unclear if the water main is buried beneath the Monatiquot River or affixed to the bridge. When contacted, the Town indicated that they do not have profile drawings of any of the water main pipes (as noted later, should dam removal proceed, a profile of the water main is absolutely needed to determine the potential for exposure). Sheet 46 of The Town of Braintree Water System Atlas (Appendix E) shows a plan view of the water main.

A 12-inch-diameter cast iron water main crosses the Monatiquot River immediately upstream of the Washington Street Bridge. The water main is affixed to the upstream face of the Washington Street Bridge (Photo 21); it is not buried. Sheet 46 of The Town of Braintree Water System Atlas (Appendix E) shows a plan view of the water main.

A 10-inch-diameter ductile iron water main pipe crosses the Monatiquot River at the Jefferson Street Bridge. It is unclear if this water main is buried beneath the Monatiquot River or is affixed to the bridge. Again, the Town has no profile drawing of this water main and again a profile of the water mains is needed to determine the potential for exposure. Sheet 41 of The Town of Braintree Water System Atlas (Appendix E) shows a plan view of the water main.

Note that there are no Town water main crossings between Armstrong Dam and lower MBTA Railroad Bridge.

Sewer Main Crossing

There are also three sewer mains crossing the Monatiquot River upstream of the Armstrong Dam. A 24-inch-by-38-inch reinforced concrete elliptical sewer main crosses the Monatiquot River just upstream of the Armstrong Dam pond at a point approximately 25 ft upstream of the upstream face of the Plain Street Bridge. A plan and profile drawing of the sewer main is shown in Sewer Assessment Plan 6638 of Appendix E. According to the profile, the sewer main is encased in six (6) inches of concrete around the pipe, and the top of the concrete is buried about approximately six (6) inches below the channel bed. The sewer main has a slope of 0.006 ft/ft beneath the river and an invert elevation at the thalweg of about 94.9 feet. Note that there is no date shown on sewer main drawings--the channel bed elevation shown on the drawings may have shifted over time. Also note that the vertical datum of the drawings is not shown. According to the Town, (Personal Communication, Kelly Phalen, 9/19/2016) the vertical datum is likely the Braintree datum used by the Water and Sewer Department.

A 12-inch-diameter cast iron sewer main crosses the Monatiquot River immediately upstream of the upstream face of the Washington/Hancock Street Bridge. A plan and profile drawing of the sewer main is shown in Sewer Assessment Plan 4807 of Appendix E. The 1948 plan shows the sewer main "encased" in concrete on three sides with 2.5 inches of concrete above the sewer main, 12 inches on the sides and 12 inches of 1-inch crushed stone below. According to the profile, the sewer main was buried about one foot below the channel bottom. According to the Town (Personal communication, John Morse, 2/25/16) flow may no longer be conveyed through this main.

An 8-inch-diameter cast iron sewer pipe crosses the Monatiquot River less than 100 feet downstream of the Jefferson Street Bridge. A plan and profile drawing of the sewer main is shown in Sewer Assessment Plan 5602 of Appendix E. The 1956 plan shows the pipe is encased in concrete with the top elevation of the concrete just below the channel bottom.

3.3 Stormwater Outfalls

Stormwater outfalls on the Monatiquot River, from the Armstrong Dam to the confluence with the Farm and Cochato Rivers, were mapped and documented[18]. If the dam were removed and water levels lowered, stormwater discharges may not empty directly into the river. In some cases, the stormwater may discharge onto upland areas and flow on the ground before emptying into the river. To avoid erosion of these upland areas, it is typically recommended to place some type of erosion protection between the point of stormwater discharge and the river. The culvert location, size, material and invert elevation are mapped in Figure 3.3-1. The figures also reference Photos 22 – 30, which can be found in Appendix A.

[18] Some outfalls may have been underwater or surrounded by dense vegetation and may have been missed during field investigations.

Outfall Number	Shape	Material	Invert El. (ft, NGVD29)	Diameter (in.)	Photo
1	Circular	Clay	94.1	10	22
2	Circular	Concrete	95.6	24	23
3	Oval	Concrete	96.3	16x24	24
4	Circular	Concrete	94.7	24	25
5	Circular	Concrete	94.6	24	25
6	Circular	PVC lined, set in concrete	96.6	12	26
7	Circular	Corrugated Pipe	95.3	24	27
8	Circular	Concrete	97.5	12	27
9	Circular	Concrete	96.0	15	28
10	Circular	Concrete	97.2	11	28
11	Circular	Concrete	96.0	11.5	29
12	Circular	Concrete	96.7	12	30

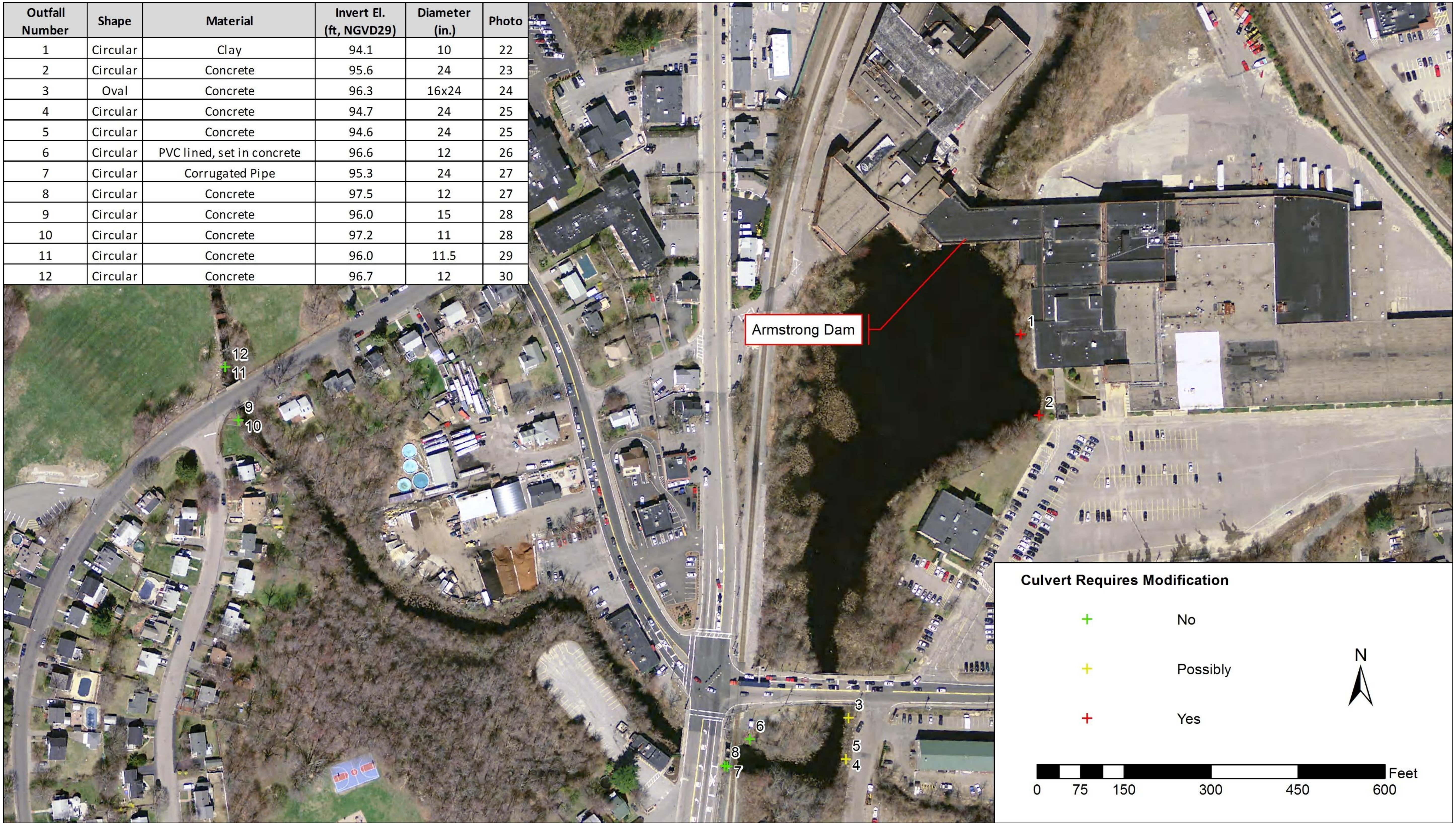


Figure 3.3-1: Stormwater Outfalls

4. Diadromous Fishery Resources

4.1 Target Species

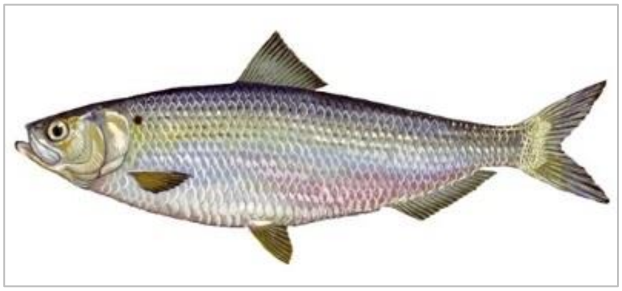
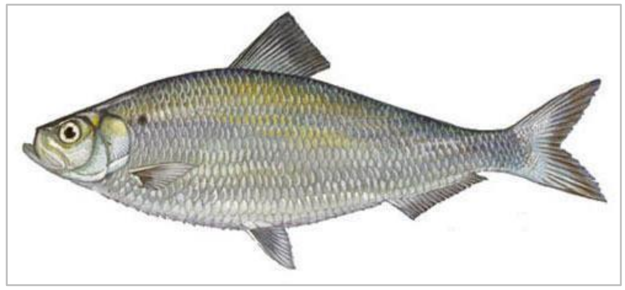
A primary goal of this project is to provide upstream and downstream fish passage for diadromous and resident fish species. The term "diadromous" refers to fish that migrate between freshwater and marine environments, and includes anadromous and catadromous fish. Anadromous fish (such as river herring) hatch from eggs deposited in freshwater habitats, migrate as juveniles to salt water where they remain until maturity, then return to natal rivers to complete their reproductive cycle. Catadromous fish (such as American eel) spawn in the ocean and migrate to fresh water to grow to adult size.

Diadromous fish species targeted for restoration in the Monatiquot River and its tributaries include both species of river herring (blueback herring and alewife), and American eel. The restoration of diadromous species is important to the greater Fore River watershed as they provide forage to many species of fish and wildlife (e.g., striped bass (Morone *saxatilis*), trout (*Salmo sp.*), cod (*Gadus morhua*), bluefish (*Pomatomus saltatrix*), tuna (*Thunnus sp.*), ospreys (*Pandion haliaetus*), herons (*Ardea sp.*), cormorants (*Phalacrocorax sp.*), otters (family Lutrinae), seals (family Phocidae), whales (family Cetacea), etc.) and facilitate the transport of nutrients between marine and freshwater environments. Because of their status as forage species, diadromous fish are important for commercial and recreational fisheries of other species. Their impacts extend far beyond the site of a single restoration project, as the target species are distributed along the entire Atlantic coast from Newfoundland (alewife) to Florida (blueback herring), and from Greenland to South America (American eel). Diadromous fish also provide cultural benefits to citizens who value fish runs for food, bait, and as a sign of a healthy river.

River Herring

River herring are actually two closely related members of the *Clupeidae* family including the alewife (*Alosa pseudoharengus*) and blueback herring (*Alosa aestivalis*). Both are anadromous species, spending most of their life in the ocean, and migrating to spawn in freshwater during spring. Their appearances (images below) and life histories are so similar that they are often grouped together and managed as one species. Alewives begin to spawn in late March to mid-May when water temperatures reach about 51°F, but can arrive earlier following mild winters. Blueback herring begin to spawn about three to four weeks later in the spring (late April through June) when water temperatures reach about 57°F. While both species are capable of spawning in a variety of freshwater environments, blueback herring generally spawn in more riverine areas, whereas alewives tend to spawn in more lacustrine (ponds and lakes) areas. Adult herring that survive spawning and predation return to the sea. After utilizing freshwater nursery habitat for most of the summer, juvenile herring begin their migration to the ocean in July. Migration peaks usually occur in late summer and early fall, but are variable and can continue into December. After maturing in the marine environment until about three to five years of age, the fish return to freshwater, many to their natal streams (Nelson et al., 2011).

Adult alewife average 10 to 14 inches in length and weigh less than a pound. Blueback herring are generally smaller than alewife, averaging around 9.5 to 12 inches in length. Adult river herring swim at a cruising speed of about 3 feet per second (ft/s), a sustained speed up to 5 ft/s, and can reach burst speeds of 7 ft/s (Bell, 1991).

Blueback herring *(Alosa aestivalis)* Alewife *(Alosa pseudoharengus)*
Imagery Credit: Duane Raver/USFWS

Alewife are found from northeastern Newfoundland to South Carolina, but are most abundant in the Mid-Atlantic and the Northeast. Blueback herring are found from Nova Scotia to northern Florida, but are most numerous in waters from Chesapeake Bay south (ASMFC, 2016).

Historically, river herring were one of the most valuable anadromous fishes harvested commercially in Massachusetts and were sold as food or for commercial bait (Belding, 1921). More than 100 coastal Massachusetts rivers and streams are home to the two species. Presently, river herring are valued for their ecologically contributions as forage for many fish and wildlife predators, the dramatic spring spawning runs, and role in the transfer of nutrients between freshwater and marine systems.

In recent years, however, river herring abundance throughout Massachusetts has declined to historically low levels. In 2005, the declines prompted *MarineFisheries* to establish a three-year moratorium on the sale and harvest of river herring throughout the state, which continues presently. In addition, the National Marine Fisheries Service has listed blueback herring and alewife as Species of Special Concern under their Endangered Species Act review process. A 2013 petition to list the species as threatened under the Act was reviewed, but resulted in a negative finding for Threatened or Endangered Status.

American Eel

The American eel (*Anguilla rostrata*) is the only catadromous species in North America, meaning that it spends most of its lifetime in ponds and rivers and migrates to the ocean to spawn. All adult American eels spawn in the Sargasso Sea located in the western Atlantic Ocean. The larvae drift into the Gulf Stream and mature into clear "glass eels" as they approach the coast, and develop into elvers soon after. In the Gulf of Maine, migration of glass eels occurs mainly from April to June, and elvers can continue upstream into early fall. It can take several years for eels to migrate up rivers, during which time they may travel hundreds of miles. As elvers grow, they become known as yellow eels. Yellow eels may spend six to 30 or more years in freshwater before they metamorphose into mature silver eels. On dark, rainy nights during September to December, most silver eels descend rivers and begin their journey to the Sargasso Sea. Eels spawn only once, so their spawning migration also represents the last stage of their life before dying (Gulf of Maine Council on the Marine Environment, 2007).

Imagery Credit: Duane Raver/USFWS

Eel swimming performance is dependent upon age and size. As they make their way upstream in freshwater rivers, juveniles gradually increase in size as they grow from glass eels (2 to 3 inches long) to elvers (2 ½ to 4 inches long) to yellow eels (generally considered to be greater than 6 inches long). Adult females may attain lengths of nearly 50 inches, while males may only reach about 16 inches. At the Armstrong Dam, eels are expected to be in the elver or yellow eel stage. Sustained swimming speeds are estimated to be about 0.25 ft/s for 2-inch-long glass eels, 0.5 ft/s for 4-inch-long glass eels, and 2 to 7 ft/s for adult eels ranging in length from 2 to 8 feet, respectively (Bell, 1991). Elvers that are 3 to 4 inches long can swim at burst speeds of approximately 2 to 3 ft/s over distances of less than 5 feet. However, at water velocities of 1 ft/s, elvers generally cannot swim further than 10 feet. Older juveniles can swim 5 ft/s but cannot swim far against fast water. Water velocities in excess of swimming speed, long distances, lack of refuges from currents, strong turbulence, and complex flows will all reduce swimming performance and hinder migration. Eels are good climbers and can ascend vertical surfaces if there is a wet, rough substrate for them to climb. However, a large proportion of eels will not attempt to climb (Gulf of Maine Council on the Marine Environment, 2007). Additionally, eels may suffer from increased predation as they aggregate at the base of structures seeking passage routes.

American eels range from Greenland to northeastern South America, occurring in all major streams along the coastline. They represent a single breeding population, meaning that eels from South America, Greenland, and anywhere in between may breed with each other. Thus, there are no distinct watershed or regional "stocks" as there are for anadromous species (Gulf of Maine Council on the Marine Environment, 2007).

4.2 Target Fish Passage Thresholds

In order for diadromous fish to readily pass to and from their spawning habitat, certain physiological and behavioral needs and physical river conditions must be met, including seasonal flow magnitudes, depths and velocities. As described later, a hydraulic model of the study reach was developed to simulate depths, and velocities during the fish passage season, low flow conditions, and flood flows. During the fish passage season, the hydraulic model was used to simulate dam-out (Armstrong Dam only) conditions to determine if water depths and velocities were sufficient to create passage for the target species. Important considerations for restoration activities are described below.

Flow Timing

It is important to understand when diadromous fish are typically moving up and downstream to be able to evaluate whether hydraulic parameters such as water depth and velocity will be appropriate during those times. For the upstream migration period of adult river herring, water depth and velocity were evaluated for peak migration month of May. For the downstream migration period of juvenile river herring, water depth and velocity were evaluated for the period September through October to account for expected flow limitations in the Fore River watershed during July and August.

Flow Depth

Water depth in the river channel and through obstacles such as bridges and culverts must be sufficient to accommodate the physical dimensions of fish navigating upstream. In order for fish to swim normally, the minimum depth of flow should generally be 1.5 to 2 times the body thickness of the largest target species[19]. Since the alewife is the largest of the target species in terms of body thickness, its dimensions serve as a conservative surrogate for all of the target species. Assuming an average body thickness to total body length ratio of 25% and an adult body length of 15 inches, body thickness would be about 4 inches, and the minimum depth required for passage would be about 6 to 8 inches.

The number and length of obstacles fish must negotiate should be considered, and this guideline should be adjusted as necessary. If fish encounter few passage barriers, they can likely negotiate fairly shallow water. However, the same species moving up a stream with many obstacles may arrive at the spawning area in poor condition if passage depths are minimal (Bovee, 1992).

Flow Width

The width over which water flows may impact a fish's ability to pass upstream or downstream. A narrow opening will concentrate flow and accelerate velocity, which may elicit an avoidance response, inhibit swimming ability or even injure fish. The minimum pool and channel width recommended for American eel and river herring within a nature like step-pool fishway is three and five feet, respectively (USFWS, 2016). However, these species commonly pass through narrower widths of a foot or less at fishways in Massachusetts with limited discharge.

Flow Velocity

Diadromous and other migratory riverine species often encounter zones of high velocity flow, such as where flow is restricted going through a road crossing or a narrow, rocky section of channel, that impede their migrations. Generally, fish swimming performance is characterized by the following levels of swimming speeds (Bell, 1991):

- **Cruising speed** – A speed that can be maintained for hours without causing any major physiological changes; employed for general movement and migration
- **Prolonged speed** – A speed that can be maintained for minutes; employed for passage through difficult areas
- **Burst speed** – A speed that can be maintained for seconds. A single effort that is not sustainable; employed for feeding or escape purposes

Table 4.2-1 provides a summary of the various swimming speeds for the target fish species, as discussed in the previous section. The most important swimming speed for fish passage considerations is prolonged speed. Where flows exceed maximum sustained swim speed, successful passage may still be possible, provided that fish can accomplish the needed burst speed without additional impedance such as high water temperatures and/or low dissolved oxygen (Bell, 1991).

American eel generally has lower sustained swimming speeds than those of the herring family target species, but they exhibit climbing behaviors that may help them navigate obstructions that are impassable to herring. Of the two alosine target species, alewife appear to be the weakest swimmers, and thus can be used as a conservative threshold for the others. Considering this information in conjunction with the swimming speeds in Table 4.2-1 it was determined that a maximum water velocity of 5 ft/s to 7 ft/s would be an appropriate target to ensure that the target species should be able to navigate barriers using either prolonged or burst speeds.

[19] The USFWS recommends 2 times body thickness (USFWS, 2016).

Table 4.2-1: Summary of Swimming Speeds for Target Species

Species*[20]	Swimming Speed (ft/s)		
	Cruising	*Sustained*	*Burst*
Alewife	0 - 3	3 - 5	5 - 7
Blueback herring			
American eel (glass eels & elvers)**[21]	-	0.25 - 0.50	1 - 5

**Swimming speeds are reported for the upstream migrant life stage.*
***Climbing and/or attachment behaviors may help eel pass through difficult obstacles.*
Sources: All swimming speeds estimated from table in Bell, 1991 except American eel burst speeds, which are from Gulf of Maine Council on the Marine Environment, 2007

[20] Swimming speeds are reported for the upstream migrant life stage.
[21] Climbing and/or attachment behaviors may help eel pass through difficult obstacles.

5. Wetland Resources

As part of this FS, wetlands in the vicinity of the Project Area were delineated by LEC Environmental Consultants, Inc. (LEC) in July, 2016. The delineation is depicted on the site plan in Sheets 2 and 3 of Appendix B. Aerial views of the delineated wetlands are also available within the Wetland Report in Appendix F. As part of this FS, anticipated changes to wetland resources in the Project Area were not calculated.

On July 12, 14 and 15, 2016 LEC located the boundaries of local, state and federally protected wetlands and jurisdictional boundaries associated with the Monatiquot River and Hollingsworth Pond. Wetland delineations were along the Field Survey Area from approximately 2,200 feet upstream to approximately 800 feet downstream of the Armstrong Dam. Bordering Vegetated Wetlands (BVW), Ordinary High Water (OHW) and Mean Annual High Water (MAHW) boundaries were established, marked with flagging tape and their coordinates obtained via a Trimble Handheld GEO XH-6000 GPS unit.

The wetland evaluation was conducted in accordance with the *Massachusetts Wetlands Protection Act* (WPA; M.G.L. c. 131, s.40), its implementing *Regulations* (310 CMR 10.00), the federal *Clean Water Act* (CWA; 33 U.S.C. 1344, s.404) and its *Regulations* (33 CFR and 40 CFR) and the *Town of Braintree's Wetland Protection Bylaw* (Chapter 12.20) and its implementing *Regulations.*

The Wetland Resource Areas protected under the state WPA and the local *Bylaw* include BVW, Bank, Land Under Water (LUW), Riverfront Area and Bordering Land Subject to Flooding (BLSF). The boundaries of BVW and the OHW mark establish the extent of jurisdiction under the federal *CWA (*33 U.S.C. 1344, s.404) and its *Regulations* (33 CFR and 40 CFR).

According to the Wetland Report, vegetation throughout the forested upland portions of the Field Survey Area consists of a canopy layer of northern catalpa (*Catalpa speciosa*), Norway maple (*Acer platanoides*), northern red oak (*Quercus rubra*), white oak (*Quercus alba*), and honey locust (*Gleditsia triacanthos*). The understory includes saplings from the canopy layer and a shrub layer of staghorn sumac (*Rhus typhina*), smooth sumac (*Rhus glabra*), multiflora rose (*Rosa multiflora*) and common buckthorn (*Rhamnus cathartica*). The groundcover layer contains expansive patches of poison ivy (*Toxicodendron radicans*) (LEC, 2016).

According to the Soil Survey of Norfolk and Suffolk Counties, the portion of the Field Survey Area in the vicinity of the industrial buildings contains Urban Land, Udorthents (wet substratum), Udorthents (Loamy), and Water. The wetland system west of Hancock Street is mapped with Whitman fine sandy loam. Field evaluations of the soil conditions were generally consistent with the soil survey mapping.

The Project Area, including Armstrong Dam, is not located within a *Priority Habitat of Rare Species* or *Estimated Habitat of Rare Wildlife*, according to the 13th edition (October 1, 2008) of the *Massachusetts Natural Heritage Atlas* published by the Natural Heritage & Endangered Species Program (NHESP). No Certified Vernal Pools (CVP) or Potential Vernal Pools (PVP) are mapped on or within the immediate vicinity of the site either. Therefore, the Project is not subject to review under the *Massachusetts Endangered Species Act* (MESA, M.G.L. c. 131A) and its implementing *Regulations* (321 CMR 10.00).

6. Existing Conditions Plan

As part of this FS, Existing Conditions Plans (Appendix B) were developed for the Project Area from the downstream MBTA Railroad Bridge to the Washington Street Bridge. Should removal of the Armstrong Dam proceed, the plans will help with design and permitting. The plans were developed from a variety of resources including survey, Braintree-supplied CAD drawings, GIS and historical resources. This section describes the data sources, data collection approach and any assumptions made in developing the Existing Conditions Plans.

6.1 Existing Information

Several sources of data were available for creating the plans including:

- Bathymetric map and AutoCAD drawings of the Armstrong Dam (Alpha Surveying and Engineering Inc., 2011)
- Partial survey of the Armstrong Dam (GSE, 2009)
- Historical drawings of the Armstrong Dam provided by Messina (Armstrong Cork Co., 1954)
- AutoCAD drawings provided by the Town (acquired 09/08/2015)
- GIS datalayers from Massachusetts Office of Geographic Information (MassGIS).

Armstrong Cork Co. Dam Drawings

Historical drawings of the Armstrong Dam from 1945 by the Armstrong Cork Co. Engineering Department (Lancaster, PA) were utilized for some of the structural dimensions. The vertical datum of these drawings is unknown; however, a correction factor to bring the historical drawings into the current project vertical datum was estimated by determining the difference in elevations on common points, such as the spillway crest, elevation 100.83 feet on the historic drawings and 94.1 feet NAVD88 from the survey.

Existing AutoCAD Drawings

Property lines, structures outlines, utility lines, roadways, railroads and other planimetrics were obtained from reference drawings by the Town and by Alpha Surveying and Engineering, Inc. which were provided by Messina. The Town drawings also included 2-foot contours, which were in NGVD29[22].

MassGIS Datalayers

Federal Emergency Management Agency (FEMA) data, including the 100-year flood zone were obtained from MassGIS on December 15, 2015. United States Geological Survey (USGS) topographic maps and high resolution Orthophotos were also obtained from MassGIS on December 31, 2015.

2009 Feasibility Study Data

Hydraulic Engineering Center River Analysis System (HEC-RAS) transects[23] from the 2009 FS were also included in the drawings. As described in Section 8.1 these transects were in NGVD29 and converted to NAVD88.

[22] These contours have been left in NGVD29 on the Existing Conditions Plans to keep the contour intervals as whole numbers on the plans.

6.2 Survey Data

Survey for the Existing Conditions Plans were performed on November 4, 2015 and December 17, 2015. On November 4, 2015 a survey rod and level was used in conjunction with a real-time kinematic global positioning system (RTK-GPS) unit to collect elevation data of Armstrong Dam and other pertinent features for use in the Existing Conditions Plan (and hydraulic model). Supplementary survey data were collected, mostly below Armstrong Dam, on December 17, 2015 using a total station and an RTK-GPS unit.

The survey focused on obtaining the following data:

- key elevations of the dam, building above the dam, low level outlet, spillway and other pertinent hydraulic structures,
- planimetrics near the dam (utilities, infrastructure, hydraulic obstructions),
- river channel dimensions below the dam, and
- water surface elevations (WSELs).

Instrumentation Error

Survey data collected for this study relied on RTK-GPS, total station instruments and a conventional survey rod and level. The RTK-GPS unit was a Leica GS14. The reported accuracy of the Leica GS14 is ±8 mm + 1 ppm horizontal and ±15 mm + 1 ppm vertically. Accuracy for the RTK-GPS unit is dependent upon several factors including number of satellites, obstructions, ephemeris accuracy, ionosphere conditions, and multipath. Data were collected in Mass Mainland NAVD88 using a network base station setup connected to Maine Technical Source Hanover.

The total station utilized for the field survey was the CST/Berger CST-205. Accuracy of a total station is a function of the distance of the rod and prism from the total station times the angular accuracy. The CST-205 has a reported accuracy of 5 arcsec and a distance accuracy of ±5 mm + 3 ppm x D, where D is the distance from the total station. It has a resolution of 5 arcsec and 0.005 ft and measurement range of 1.6 miles, assuming average conditions of slight haze and sunny. The level used for the survey rod and level portion of the survey was a Nikon AX-2S. The reported level of accuracy for this level is ± 0.5 in.

Total station and rod and level surveys are dependent upon a means of horizontal and vertical control to tie them into a common datum. Benchmarks were installed and surveyed with the RTK-GPS unit for this purpose. They were used as reference points to rotate and transpose data from an arbitrary datum to the Project datum. Since the total station and rod and level survey relied on temporary benchmarks, which surveyed in by the RTK-GPS unit, the survey accuracy is a function of the combined error of RTK-GPS and total station accuracy or RTK-GPS and rod and level accuracy. Accumulating the error, it is expected that the combined survey error is approximately ± 0.1 ft vertical and ± 0.05 ft horizontal.

6.3 Drawings

There are four sheets to the Existing Conditions Plans (Appendix B) as follows:

[23] All transects in the drawings are only displayed in the wetted area even though they extend through the 100-year flood zone.

Drawing 1: Cover sheet,

Drawing 2: Plan view of the northern part of the Project area,

Drawing 3: Plan view of the southern part of the Project area, and

Drawing 4: Profile and section view of Armstrong Dam.

As mentioned in Section 6.1 the contour data is in NGVD29; all other elevations called out in the Existing Conditions Plans are in NAVD88, including all survey and FEMA data.

7. Sediment

Rivers provide natural functions of water and sediment transport. Under high flow conditions, increasing water velocity causes sediment to scour in the form of suspended sediment and bedload sediment. When the transported sediment flows into the headpond created by a dam, water velocities drop causing the transported sediment to deposit and accumulate in the impoundment. Over time, the impoundment can become filled in due to years of deposition. Another consideration is that sediments can also carry pollutants, which would also be deposited in the dam's headpond.

When considering dam removal, the quality and quantity of sediment needs to be evaluated in order to inform a sediment management plan. In addition, typically due diligence is conducted to determine the likelihood of having contaminated sediment that becomes exposed and/or mobilized if the dam is removed. Given the historical industrial use of the pond and the Baird & McGuire Superfund site located upstream on the Cochato River, mobilizing sediment by removing Armstrong Dam and changing the hydraulics in the impoundment is of particular concern.

Prior to this FS, Loureiro Engineering Associates, Inc. (Loureiro, 2012) performed sediment testing in the Armstrong Dam pond at the request of Messina. The Loureiro Environmental Investigation is available in Appendix G.

For this investigation, four samples were collected on November 2, 2012 from the Armstrong Dam pond at the locations shown in Figure 7-1 (and Figure 1 of the Loureiro report). The samples were distributed spatially to capture areas within the pond where sediments are likely to be exposed or potentially mobilized if the dam were removed and to gather representative information regarding the sediment conditions. The sampling locations and descriptions are as follows:

- LEA-SS-001 was located along the eastern portion of the pond adjacent to the parking lot and its stormwater outfall.

- LEA-SS-002 was located along a shallow "sand bar" in the southern portion of the Pond mid-channel with the incoming river; upstream of the main portion of the Pond.

- LEA-SS-003 was located in the northwestern portion of the Pond adjacent to the junction of the channel spanning portion of the building and the portion of the building that occupies the western area of the Site.

- LEA-SS-004 was collected in the western portion of the Pond adjacent to a narrow vegetated buffer between the Pond and the commuter rail tracks.

Samples were collected using a 2-inch polyvinyl chloride (PVC) tube with an open end and a capped end with an air vent. The device was manually driven into the sediment until refusal, upon which the air vent was closed and the tube retrieved. At the surface, the air vent was opened, releasing the sediment from the tube.

Samples were analyzed for the following parameters:
- VOCs by Environmental Protection Agency (EPA) Method 8260;
- Polycyclic Aromatic Hydrocarbons (PAHs) by EPA Method 8270;
- Extractable Petroleum Hydrocarbons (EPHs) with target PAHs;

- 14 Metals.

Loureiro compared the results to the MCP Reportable Concentrations (RCS1, RCS2) and MCP Method 1 Standards (S1GW1, S1GW2, S1GW3 and UCLs). The analytes detected above the reporting limit were shown in bold, while the analytes detected above the RC and/or Method 1 clean up standard were highlighted and shown in bold. As Table 7-1 shows, there are several exceedances of VOCs, PAHs, PAHs and metals.

The Loureiro Environmental Investigation did not include the minimum standard suite of chemical tests. Testing requirements for dam removals are described in MA 314 CMR 9.07 (2)(b)6 and are included in Table 7-2. The investigation did not include pesticides, polychlorinated biphenyls (PCBs), nor grain size analysis; in addition, sample LEA-SS-004 was not tested for VOCs.

A 401 Water Quality Certification Dredge Permit Form is required for projects involving sediment removal and disposal. Per 314 CMR 9.07, for projects up to 10,000 cubic yards of sediment, one core is required every 1,000 cubic yards of dredged material. Since the 2012 samples were insufficient for permitting requirements, additional samples were needed for analysis. Note that besides chemical testing, samples needed to be collected for physical properties (grain size) as well.

GSE proposed four additional samples be collected for this FS. Prior to collection, a sediment sampling plan was developed for approval by Project Partners and MassDEP.

7.1 Sediment Sampling Plan

A Draft Sediment Sampling Plan was sent to MassDEP for approval on October 6, 2015. The sampling plan called for the collection of four samples. After reviewing the Draft Sediment Sampling Plan, in addition to the previous sediment sampling results from the work conducted by Loureiro Engineering Associates, Inc. for Messina, MassDEP altered the scope of the associated sediment sampling. MassDEP required a 21E File Review and increased the number of samples for analysis from the proposed four to nine samples. The 21E Site File Review is a due diligence report focusing on potential contaminants from the existing and historic use at the dam and impoundment. It requires a review of federal, state and local databases for possible contaminants that could be located within the Project Area. A 21E Site is a location where there has been a documented release of oil or hazardous waste to MassDEP.

The Draft Sediment Sampling plan was updated based on the additional testing required by MassDEP and the due diligence and was sent to MassDEP again for final approval on June 3, 2016. The Approved Sediment Sampling Plan is attached in Appendix I. The due diligence findings are summarized in Section 7.2, and all findings are available within Appendix J.

7.2 Due Diligence

7.2.1 History of Hollingsworth Pond

The mill site just downstream of Hollingsworth Pond was initially a saw and grist mill in the 1700s. The mill was then bought and operated by the Boston and Braintree Copper and Brass Manufactory in 1823 before being sold to Hollingsworth Interests in 1832 for paper manufacturing. Old ropes were used to make manila paper starting in 1841. The site was sold to Monatiquot Rubber Works in 1901, later becoming the Stedman Rubber Flooring Company. The Armstrong Cork Company took the site over in 1936, later to be known as the Armstrong World Industries (AWI) in the 1950s. AWI used the river water solely for industrial cooling but manufactured linoleum flooring at the mill until shutting down in the 1990s (Frazier, 1985).

7.2.2 Baird & McGuire Superfund Site

Baird & McGuire is a 20-acre Superfund site less than 2 miles upstream of Hollingsworth Pond and 500 feet from the Cochato River, which feeds into the Monatiquot River, eventually leading to Hollingsworth Pond. Baird & McGuire operated as a chemical mixing and batching company from 1912 to 1983. Products they manufactured included pesticides, disinfectants, soaps, floor waxes and solvents. Hazardous wastes were historically disposed of on-site in a lagoon or cesspool, or into the soil, a nearby brook, surrounding wetlands or former gravel pit. A plume of hazardous waste affected the groundwater, rendering a nearby drinking water aquifer unusable. Hazardous substances historically disposed of on-site include heavy metals such as lead and arsenic, VOCs, PAHs, other organic compounds, pesticides such as dichloro-diphenyl-trichloroethane (DDT) and Chlordane, as well as dioxin (EPA, 2016).

7.2.3 AUL Sites

Several facilities within the drainage area are regulated via Activity and Site Use Limitations (AULs). AULs
specify the allowable and prohibited use of a property by establishing limits and conditions for the future use of contaminated property, thereby allowing cleanups to be tailored to these uses. Locations within the watershed include a light manufacturing facility, a floor laminate facility, Speedy Lube (an automotive service and maintenance facility), a Veteran of Foreign Wars hall and two residences. The majority of AULs deal with gasoline or oil contamination (MassDEP, 2015). Details of the AUL locations within the drainage area are included the Approved Sediment Sampling Plan in Appendix I.

7.2.4 21E Sites

Several 21E locations also exist within the drainage area. Contaminants include but are not limited to: aromatic hydrocarbons, petroleum products, VOCs, pesticides, lead and other metals. Petroleum products, especially #2 fuel oil, comprise the largest recorded volume of contamination within the watershed. Additional information about the 21E contamination locations within the drainage are included the Approved Sediment Sampling Plan in Appendix I.

7.2.5 TRI Sites

Massachusetts companies that use large quantities of certain toxic chemicals are required to evaluate and plan for pollution prevention, implement the plan if practical, and measure/report the results annually as required under the Toxics Use Reduction Act of 1989. The Toxic Release Inventory (TRI) keeps track of industrial use (i.e., recycling, combustion, destruction, disposal etc.) of certain chemicals that pose a threat to human health and the environment. The TRI also helps track the reduction of chemical waste generation by industry regulations. TRI locations within the Armstrong Dam drainage area are included the Approved Sediment Sampling Plan in Appendix I.

7.3 Sediment Sampling

Sediment sampling was conducted on June 13 and 14, 2016 in cooperation with *MarineFisheries* staff. A total of eleven samples were collected[24]. The sampling locations are shown in Figures 7.3-1 to 7.3-3. The samples were collected with a hand core system outfitted with a Cellulose Acetate Butyrate liner. The push core system was advanced up to two feet or until refusal. Each sediment core was composited. The samples were processed on shore, including completion of chain of custody forms, and were delivered to Alpha Analytical, Inc., a MA-certified laboratory, for testing. Laboratory analysis included the following parameters (reported within detection limits meeting or exceeding those found in 314 CMR 9.07(2)(b)(6)) using the standard methods noted:

- 6010/7471 Metals
- 8270 PAH
- 8082 NOAA PCB Congeners – Total
- MADEP Extractable Petroleum Hydrocarbons (EPH)
- 8260 Volatile Organic Compounds (VOC)
- Walkley Black TOC
- ASTM 2216 % Moisture
- ASTM D422 Grain Size –Sieve
- MCP Pesticides – EPA 8081B

The samples were collected from the following locations:

- **Upstream of the Armstrong Dam Impoundment (04 & 05)** – Two samples were collected in free-flowing sections of the Monatiquot River that were outside the reach impounded by Armstrong Dam to characterize sediment that could become mobilized and transported downstream under free-flowing conditions. These sample results would provide information on background conditions of the sediment upstream of the Project influence. Sample 04 was collected just downstream of the confluence of the Farm and Cochato Rivers, and Sample 05 was collected upstream of the Braintree Golf Course Bridge, approximately 5,000 feet and 3,700 feet upstream of the dam, respectively.

- **Mobile Sediment within Dam Impoundment (02 & 09)**– Two samples were collected within the dam impoundment from sediment deposits that would be expected to mobilize post-dam removal to characterize contaminant levels potentially present in sediment requiring either active or passive management. Sample 02 was collected from the large sediment deposit near the boat ramp, approximately 300 feet upstream of the dam. Sample 09 was taken near the upstream face of the dam and is a composite sample made up of 2 aliquots, or subsamples, that are spaced about 16 feet apart and over 20 feet away from the west abutment of the dam, to be far enough away from LEA-SS-003 as requested by MassDEP[25].

[24] At the recommendation of MassDEP two additional samples were collected within the impoundment and frozen in case further analysis were required at a later date.

[25] MassDEP requested that Sample 09 be made up of three aliquots; however, there were insufficient sediment deposits to fulfill this request.

- **Stable Sediment within Dam Impoundment (03, 06, 10 & 11)** – Four samples were collected within the dam impoundment from sediment deposits that would be expected to become exposed, but not necessarily be mobilized upon dam removal. These sediments would be exposed and thus were tested to characterize potential risks to human health. Sample 03 was collected approximately 500 feet upstream of the dam in a large sediment deposit in the middle of the Hollingsworth Pond. Sample 06 was collected between the Plain and Washington Street bridges, approximately 1,100 feet upstream of the dam. Samples 10 and 11 were collected on the west side of the impoundment. Samples 10 and 11 were about 15 feet East and 60 feet west of Sample LEA-SS-003.

- **Downstream of Dam Impoundment (01, 07 & 08)** – Three samples were collected downstream of the dam in depositional areas that would be expected to receive sediment mobilized from the impoundment post-dam removal (this is under the assumption that the sediment would be permitted to naturally transport downstream, which has not been determined). The purpose for collecting these samples is to characterize potential ecological risks and to determine background levels of potential sediment contamination below the dam. Sample 01 was collected approximately 500 feet downstream of the Armstrong Dam at a point approximately 70 feet upstream of Ames Pond Dam. Sample 07 and 08 were collected approximately 1,500 feet and 5,700 feet downstream of the Armstrong Dam, respectively.

7.4 Sediment Analysis

Appendix J contains a summary of the findings along with the lab reports provided by Alpha Analytical, Inc. Tables 7.4-1 to 7.4-2 compares the sediment results against both human risk and ecological risk. The ecological screening criteria are categorized into threshold effects concentration (TEC) and probable effects concentration (PEC). Note that TEC values are screening thresholds below which adverse effects to freshwater ecosystems are unlikely. PEC values are screening thresholds above which adverse effects to freshwater ecosystem are likely.

The chemical results of the nine sediment samples were compared against human risk (MCP S1/GW1) and ecological risk (TEC and PEC) as shown in Tables 7.4-1 and 7.4-2. If the concentration exceeds a) the TEC, the background is shaded green, b) the PEC, the background is shaded in blue and c) the MCP S1/GW1, the value is shown in red.

Metals

Upstream- Sample 05 exceeded the TEC for arsenic, otherwise Sample 04 and 05 represented low ecological risk. There were no exceedances of the PEC or human health risk.

Impoundment- Sample 02 exceeds the PEC for copper. Sample 09 exceeds the PEC for Lead and Zinc. Samples 09 and 06 exceed the human risk threshold for cadmium. There are several exceedances of the TEC at Sample 06, 02 and 09.

Downstream- The level of ecological and human risk was higher at the samples collected downstream of the dam, particularly Sample 08. Sample 08 exceeded the human health risk threshold for arsenic, cadmium, chromium, lead and zinc. In addition, Sample 08 exceeded the PEC for cadmium, chromium, lead, mercury and zinc. Sample 01 exceeded the human health risk threshold for cadmium and zinc and the PEC for zinc.

PAHs

Upstream- The upstream samples had a few exceedances of the TEC for specific PAHs, but no exceedances of the PEC and human health.

Impoundment- There are several PAHs exceeding the PEC and the human risk threshold concentrations within the impoundment. All of the samples exceed TEC, PEC or human health thresholds for foranthracene, Benzo[a]anthracene, Benzo(a)pyrene, Benzo[b]fluoranthene, Chrysene, Dibenz[a,h]anthracene, Fluoranthene, Fluorene, Phenanthrene, Pyrene, and total PAHs (calculated). Sample 02, which had the highest concentrations and located near the boat ramp also exceeds human health standards for Indeno[1,2,3-cd]pyrene.

Downstream- Human health concentration thresholds were exceeded for benzo[a]anthracene for Samples 07 and 08 here was also several exceedances of both the TEC and PEC at Samples 07 and 08.

PCBs

Upstream- Samples 04 and 05 exceeded the TEC for most individual congeners and the total PCBs for both samples were below the TEC.

Impoundment- Total PCBs for Samples 02 and 03 were above the TEC and Sample 06 was above the PEC.

Downstream- Total PCBS for Sample 07 were above the TEC and Sample 08 was above the PEC. Generally total PCBs in the impoundment and downstream samples were similar.

Pesticides

Upstream- There were no exceedances of the TEC or human health thresholds at the two upstream samples. Some pesticides exceeded the TEC at Samples 04 and 05.

Impoundment- Sample 06 exceeds the PEC for 4,4'-DDD (a DDT), but for that sample and all others in the impoundment total DDTs or other pesticides do not exceed PEC, though several exceed TEC.

Downstream- Sample 08 exceeds the PEC for 4,4'-DDD, 4,4—DDE and total DDTs (calculated) and there were several exceedances of the TEC at primarily Samples 07 and 08.

The upstream sampling results (Samples 04 and 05) generally indicate lower concentrations for metals, PAHs and PCBs. DDT pesticides exceed the TEC for total DDTs. The concentration levels downstream are generally similar to those found in the as human health thresholds and the PEC were exceeded suggesting the sediment presents both human and ecological risk.

Physical characteristics from all of the samples are also presented in Table 7.4-2, including total organic carbon, percent water and solids, and grain size distribution.

7.5 Total Sediment Volume

This section describes the methods used to quantify the total sediment volume between the Armstrong Dam and Plain Street, including the sediment depth mapping, data collection, processing methods and results.

Data Collection

Sediment depth mapping was conducted at seven transects within the pond on December 18, 2015. Two *MarineFisheries* and one Gomez and Sullivan employee utilized a small boat and a GPS displaying the boat's location in real-time to collect the data. At each probing station, a threaded stainless steel rod marked in 0.5-foot increments was lowered into the water. The depth of water was recorded, and then the rod was driven into the sediment with a hammer until refusal. The depth of sediment was recorded. An RTK-GPS system was used to determine the latitude and longitude of each probing station[26] and water and sediment depths were recorded on average, for the entire survey, every eight feet along the transects. A map of the processed sediment probing transects and their sediment depths is shown in Figure 7.5-1.

Data Processing Methods

Water and sediment depth notes were merged with the post-processed RTK-GPS data. Elevations for the top and bottom of sediment were calculated based on a constant pond WSEL of 94.1 feet. Probing locations were not directly on a transect line, but are relatively close, as shown in Figure 7.5-2. The collected points were projected onto a line using a Python script, and cross-sectional sediment profiles were developed.

The cross-sectional area of sediment at each transect was calculated. These areas are presented in Figures 7.5-3 to 7.5-9. Note that some minor adjustments were made to the data by adding a point on the channel banks about 0.1 feet from the shore and inputting a sediment depth of zero. Also sediment depth information was extrapolated to the river left side at the transect immediately upstream of the dam (T-5) as no data were collected at this station.

After the cross-sectional area of sediment at each transect was computed, it was multiplied by one half the distance to the next upstream and downstream transect to compute a sediment volume. Based on this approach, the total sediment volume within the Armstrong Dam pond is estimated to be 3,200 cubic yards (CY). Field observations and probing indicate that the sediment deposits within the pond are predominantly medium sand to fine sand. There are also large areas of densely packed gravel, cobble or bedrock; generally, these areas are closer to the dam, and sandier sediments are located at Transects T-2, T-2.5 and T-3 (Figure 7.5-2). In addition, there is generally more sediment located along the river left bank.

7.6 Mobile Sediment Volume

The volume of sediment expected to mobilize if the Armstrong Dam were removed is a portion of the total sediment volume. The mobile sediment volume was estimated by considering only the sediment

[26] The transect directly in front of the dam, T5, could not utilize GPS because of interference from the Armstrong Cork Building. To track the probing locations, measurements were taken 5 ft in front of the upstream face of the Armstrong Dam.

that is deposited within the existing channel form through the impoundment. To estimate the mobile sediment volume, the following steps were taken:

1. A representative channel cross-section was "formed" by examining the free-flowing sections of the Monatiquot River above and below the Armstrong Dam. Since conditions below the dam are generally more ravine-like below the Armstrong Dam and/or the channel morphology is affected by the Ames Pond Dam the representative transect was formed using engineering judgement to represent the expected channel morphology (depth, width, side slopes) of the newly created channel in the impoundment if the dam was removed.
2. Based on the bathymetric and sediment probing, it was estimated —in plan view— where the channel would become established through the impoundment. Based on the sediment probing, it appears the channel would become established along the right side of the impoundment looking downstream as it is deeper in this location. The approximate edges of this "main channel" are visible in Figure 7.5-1 (denoted by the dashed blue line, described as "Approx extent of mobile sediments" in the legend).
3. To determine the mobile sediment volume, the representative transect was overlain on the seven transects where sediment thickness mapping was obtained and that sediment falling within the confines of the representative transect was assumed be mobile. The width of the representative transect is shown in the seven transects in Figures 7.5-3 through 7.5-9.
4. From this, the mobile sediment volume was computed at each transect and was multiplied by one half the distance to the next upstream and downstream transect.
5. Note that some adjustment to the mobile sediment volume was made at the downstream end of the pond due to having to disturb some sediment to access the upstream side of the dam.

Based on the above, the estimated mobile sediment volume is 1,100 CY.

Figure 7-1: 2012 Sediment Samples

Table 7-1: Data Summary Table from Loureiro's 2012 Sediment Sampling (Loureiro, 2012)

TABLE 1
SEDIMENT ANALYTICAL DATA SUMMARY OF DETECTS & EXCEEDANCES (November 2, 2012)
HOLLINGSWORTH POND, BRAINTREE, MA

Parameter	Units	LEA-SS-001	LEA-SS-002	LEA-SS-003	LEA-SS-004	MCP Reportable Conc.		MCP Method 1 Standards			
		1268055	1268056	1268057	1268058						
		11/2/2012	11/2/2012	11/2/2012	11/2/2012	RCS1	RCS2	S1GW1	S1GW2	S1GW3	UCLs
Percent Moisture	%	28	36	47	43	NA	NA	NA	NA	NA	NA
VOCs (8260C)											
1,1,2,2-Tetrachloroethane	mg/Kg	< 0.0031	< 0.0043	<0.0058	NA	0.005	0.02	0.005	0.02	0.8	400
1,2,4-Trimethylbenzene	mg/Kg	< 0.0031	< 0.0043	**0.006**	NA	1,000	10,000	NA	NA	NA	NA
1,4-Dioxane	mg/Kg	< 0.310	< 0.430	< 0.5580	NA	0.2	6	0.2	6	70	5,000
2-Butanone (MEK)	mg/Kg	**0.055**	**0.150**	**0.190**	NA	4	50	4	50	400	10,000
Acetone	mg/Kg	< 0.310	**0.510**	**0.620**	NA	6	50	6	50	400	10,000
Chlorobenzene	mg/Kg	< 0.0031	< 0.0043	**0.016**	NA	1	3	1	3	100	10,000
Chlorodibromomethane	mg/Kg	< 0.0031	< 0.0043	< 0.0058	NA	0.005	0.03	0.005	0.03	20	5,000
PAHs (8270D)											
2-Methylnaphthalene	mg/Kg	< 1.3	< 1.4	< 1.8	< 1.7	0.7	80	0.7	80	300	5,000
Acenaphthene	mg/Kg	**1.6**	< 1.4	**2.7**	< 1.7	4	3000	4	1000	1000	10,000
Acenaphthylene	mg/Kg	< 1.3	< 1.4	< 1.8	< 1.7	1	10	1	600	10	10,000
Anthracene	mg/Kg	**5.6**	< 1.4	< 1.8	< 1.7	1000	3000	1000	1000	1000	10,000
Benzo[a]anthracene	mg/Kg	**16**	**1.6**	**2.4**	< 1.7	7	40	7	7	7	3,000
Benzo[a]pyrene	mg/Kg	**15**	< 2.8	< 3.6	< 3.3	2	4	2	2	2	300
Benzo[b]fluoranthene	mg/Kg	**21**	**1.8**	**3.5**	< 1.7	7	40	7	7	7	3,000
Benzo[g,h,i]perylene	mg/Kg	**5**	< 1.4	< 1.8	< 1.7	1000	3000	1000	1000	1000	10,000
Benzo[k]fluoranthene	mg/Kg	**9.7**	< 1.5	< 1.9	< 1.8	70	400	70	70	70	10,000
Chrysene	mg/Kg	**16**	**1.6**	**2.4**	< 1.7	70	400	70	70	70	10,000
Dibenz(a,h)anthracene	mg/Kg	**1.9**	< 1.4	< 1.8	< 1.7	0.7	4	0.7	0.7	0.7	300
Fluoranthene	mg/Kg	**40**	**3.5**	**6.5**	< 1.7	1000	3000	1000	1000	1000	10,000
Fluorene	mg/Kg	**3.6**	< 1.4	**2.2**	< 1.7	1000	3000	1000	1000	1000	10,000
Indeno[1,2,3-cd]pyrene	mg/Kg	**5.7**	< 2.8	< 3.6	< 3.3	7	40	7	7	7	3,000
Phenanthrene	mg/Kg	**31**	**3.2**	**8.2**	< 1.7	10	1000	10	500	500	10,000
Pyrene	mg/Kg	**32**	**2.9**	**5.7**	< 1.7	1000	3000	1000	1000	1000	10,000
EPH (MA-EPH)											
Carbon Fractions											
C11-C22 Aromatics	mg/Kg	**370**	**180**	**1200**	**310**	NA	NA	NA	NA	NA	NA
C11-C22 Aromatics (Adjusted)	mg/Kg	< 4.6	< 5.1	< 6.2	< 5.8	1,000	3,000	1,000	1,000	1,000	10,000
C19-C36 Aliphatics	mg/Kg	**22**	**23**	< 180	**100**	3,000	5,000	3,000	3,000	3,000	20,000
C9-C18 Aliphatics	mg/Kg	< 13	< 14	< 180	< 17	1,000	3,000	1,000	1,000	1,000	20,000
Target PAHs											
2-Methylnaphthalene	mg/Kg	< 1.3	< 1.4	< 18	< 1.7	0.7	80	0.7	80	300	5,000
Acenaphthene	mg/Kg	1.5	< 1.4	< 18	< 1.7	4	3,000	4	1,000	1,000	10,000
Acenaphthylene	mg/Kg	< 1.3	< 1.4	< 18	< 1.7	1	10	1	600	10	10,000
Anthracene	mg/Kg	4.7	< 1.4	< 18	< 1.7	1,000	3,000	1,000	1,000	1,000	10,000
Benzo[a]anthracene	mg/Kg	**13**	**1.9**	< 18	**1.9**	7	40	7	7	7	3,000
Benzo[a]pyrene	mg/Kg	**13**	**2.7**	< 18	**3.1**	2	4	2	2	2	300
Benzo[b]fluoranthene	mg/Kg	**7.1**	**1.9**	< 18	**2.5**	7	40	7	7	7	3,000
Benzo[g,h,i]perylene	mg/Kg	**6.9**	< 1.4	< 18	< 1.7	1,000	3,000	1,000	1,000	1,000	10,000
Benzo[k]fluoranthene	mg/Kg	**19**	**3.1**	< 18	**3.4**	70	400	70	70	70	10,000
Chrysene	mg/Kg	**18**	**3.7**	**19**	**4.4**	70	400	70	70	70	10,000
Dibenz(a,h)anthracene	mg/Kg	**2.1**	< 1.4	< 18	< 1.7	0.7	4	0.7	0.7	0.7	300
Fluoranthene	mg/Kg	**39**	**3.8**	< 18	**2.8**	1,000	3,000	1,000	1,000	1,000	10,000
Fluorene	mg/Kg	3.1	< 1.4	< 18	< 1.7	1,000	3,000	1,000	1,000	1,000	10,000
Indeno[1,2,3-cd]pyrene	mg/Kg	**8.5**	< 1.4	< 18	< 1.7	7	40	7	7	7	3,000
Naphthalene	mg/Kg	< 1.3	< 1.4	< 18	< 1.7	4	40	4	40	500	10,000
Phenanthrene	mg/Kg	**28**	**3.2**	< 18	< 1.7	10	1,000	10	500	500	10,000
Pyrene	mg/Kg	**33**	**3.9**	**18**	**4.9**	1,000	3,000	1,000	1,000	1,000	10,000
METALS(6010/7471A)											
Antimony	mg/Kg	**0.72**	< 0.78	**68**	**2.2**	20	30	20	20	20	300
Arsenic	mg/Kg	**3.7**	**5.3**	**17**	**15**	20	20	20	20	20	200
Barium	mg/Kg	**20**	**48**	**1200**	**56**	1,000	3,000	1,000	1,000	1,000	10,000
Beryllium	mg/Kg	**0.34**	**0.45**	**0.59**	**0.71**	100	200	100	100	100	2,000
Cadmium	mg/Kg	< 0.26	**0.51**	**4.3**	**1.6**	2	30	2	2	2	300
Chromium	mg/Kg	**20**	**6.9**	**35**	**150**	30	200	30	30	30	2,000
Lead	mg/Kg	**110**	**46**	**1400**	**160**	300	300	300	300	300	3,000
Mercury	mg/Kg	< 0.14	< 0.16	**5.6**	**1.4**	20	30	20	20	20	300
Nickel	mg/Kg	**5.2**	**6.5**	**38**	**11**	20	700	20	20	20	7,000
Selenium	mg/Kg	< 0.64	**0.97**	**0.99**	< 0.86	400	800	400	400	400	8,000
Silver	mg/Kg	< 0.64 ^	< 0.78 ^	< 0.89 ^	0.88 ^	100	200	100	100	100	2,000
Thallium	mg/Kg	< 1.3	< 1.6	< 1.8	< 1.7	8	60	8	8	8	800
Vanadium	mg/Kg	**14**	**15**	**24**	**38**	600	1,000	600	600	600	10,000
Zinc	mg/Kg	**79**	**100**	**13000**	**360**	2,500	3,000	2,500	2,500	2,500	10,000

Note
G:\Projects\31BF201 FX Messina Armstrong Cork Dam Removal\[2012.11.27.TBL.1.Sediment Results Data Summary.xls]Sediment Data

% - percent
mg/Kg - milligrams per kilogram
BOLD analyte detected above reporting limit
BOLD/SHADE analyte detected above reporting limit and RC and/or Method 1 cleanup standard.
* - LCS or LCSD exceeds the control limits
^ - ICV,CCV,ICB,CCB, ISA, ISB, CRI, CRA, DLCK or MRL standard: Instrument related QC exceeds the control limits.

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Table 7-2: Sediment Analysis Requirements (MA 314 CMR9.07(2)(b)6)

Parameter[1]	Reporting Limit mg/kg (dry weight) – unless otherwise noted[2]
Arsenic	0.5
Cadmium	0.1
Chromium	1.0
Copper	1.0
Lead	1.0
Mercury	0.02
Nickel	1.0
Zinc	1.0
Polycyclic Aromatic Hydrocarbons (PAHs)	0.02
Polychlorinated Biphenyls (PCBs)-by NOAA Summation of Congeners	0.01
Extractable Petroleum Hydrocarbons[3]	25
Volatile Organic Compounds (VOC)[4]	0.1
Total Organic Carbon	0.1%
Percent Water	1.0%
Toxicity Characteristic Leaching Procedure[5]	As applicable
Grain Size Distribution – wet sieve (ASTM D422)	Sieve Nos. 4, 10, 40, 60, 200

1 *The applicant shall use the results of the due diligence review to determine whether additional parameters should also be analyzed.*

2 *If one or more of the Reporting Limits could not be met; the applicant shall include a discussion of the reason(s) for the inability to achieve the reporting limit (e.g., matrix interference).*

3 *Current method for the determination of Extractable Petroleum Hydrocarbons (EPH) MADEP January 1998*

4 *Required for sediment to be reused or disposed of in the upland environment unless the due diligence review indicates that VOC contamination is unlikely to be present.*

5 *Required to be performed when sediment is to be managed in the upland environment and if the total concentrations of metals or organic compounds are equal to or greater than the theoretical concentration at which TCLP criteria may be exceeded: As > 100 mg/kg, Cd > 20 mg/kg, Cr > 100 mg/kg, Pb > 100 mg/kg, Hg > 4 mg/kg.*

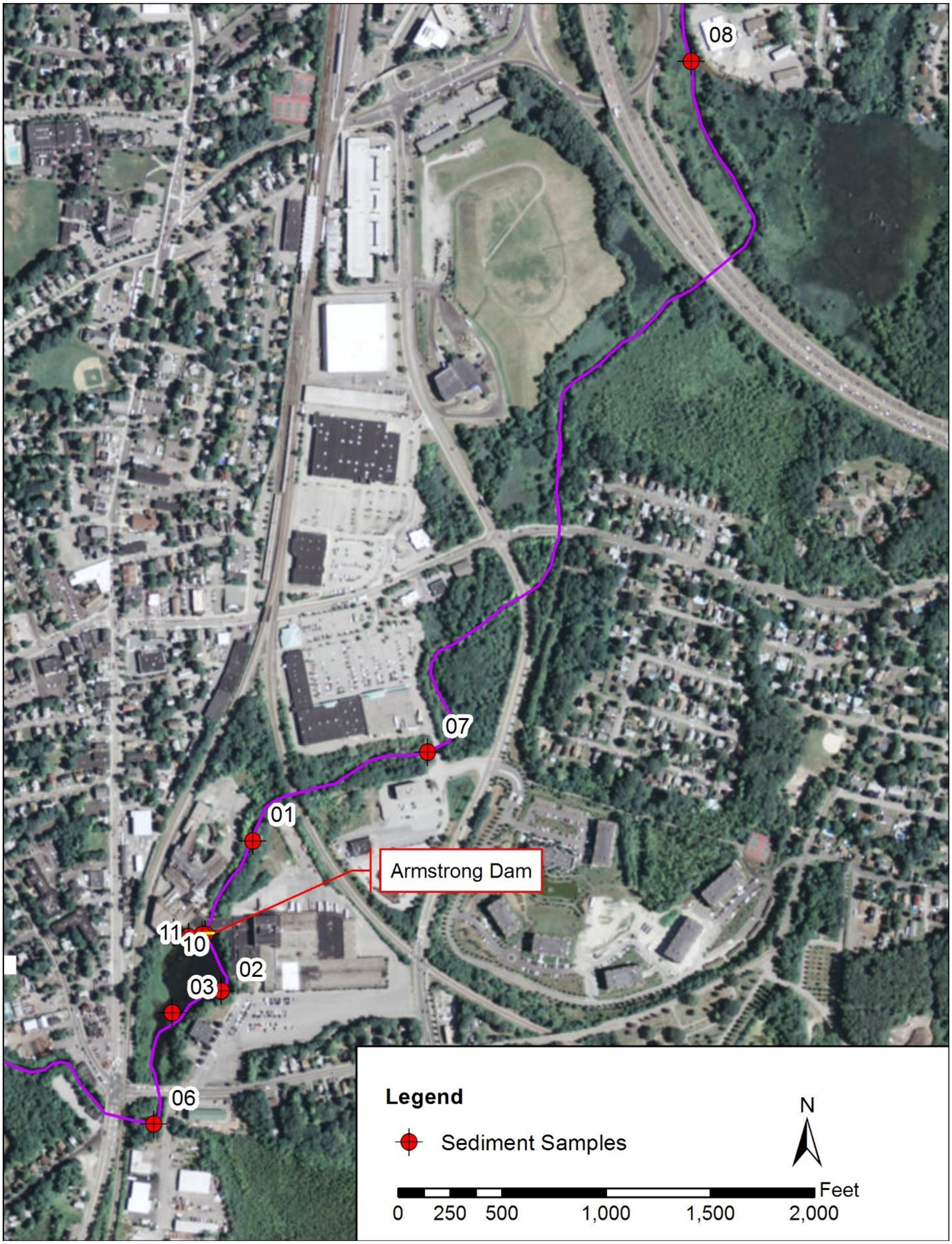

Figure 7.3-1: Downstream Sediment Sampling Location

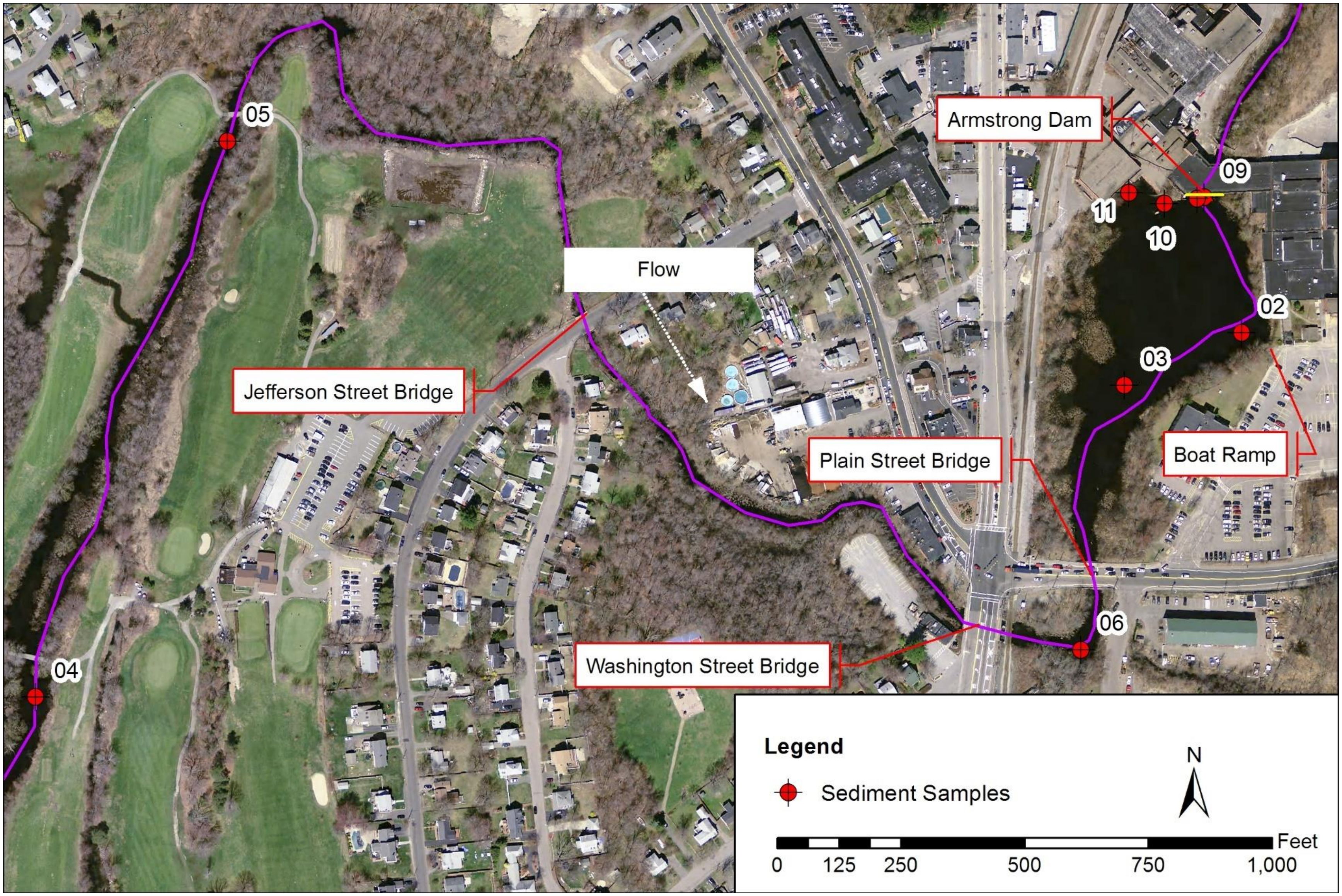

Figure 7.3-2: Upstream Sediment Sampling Locations

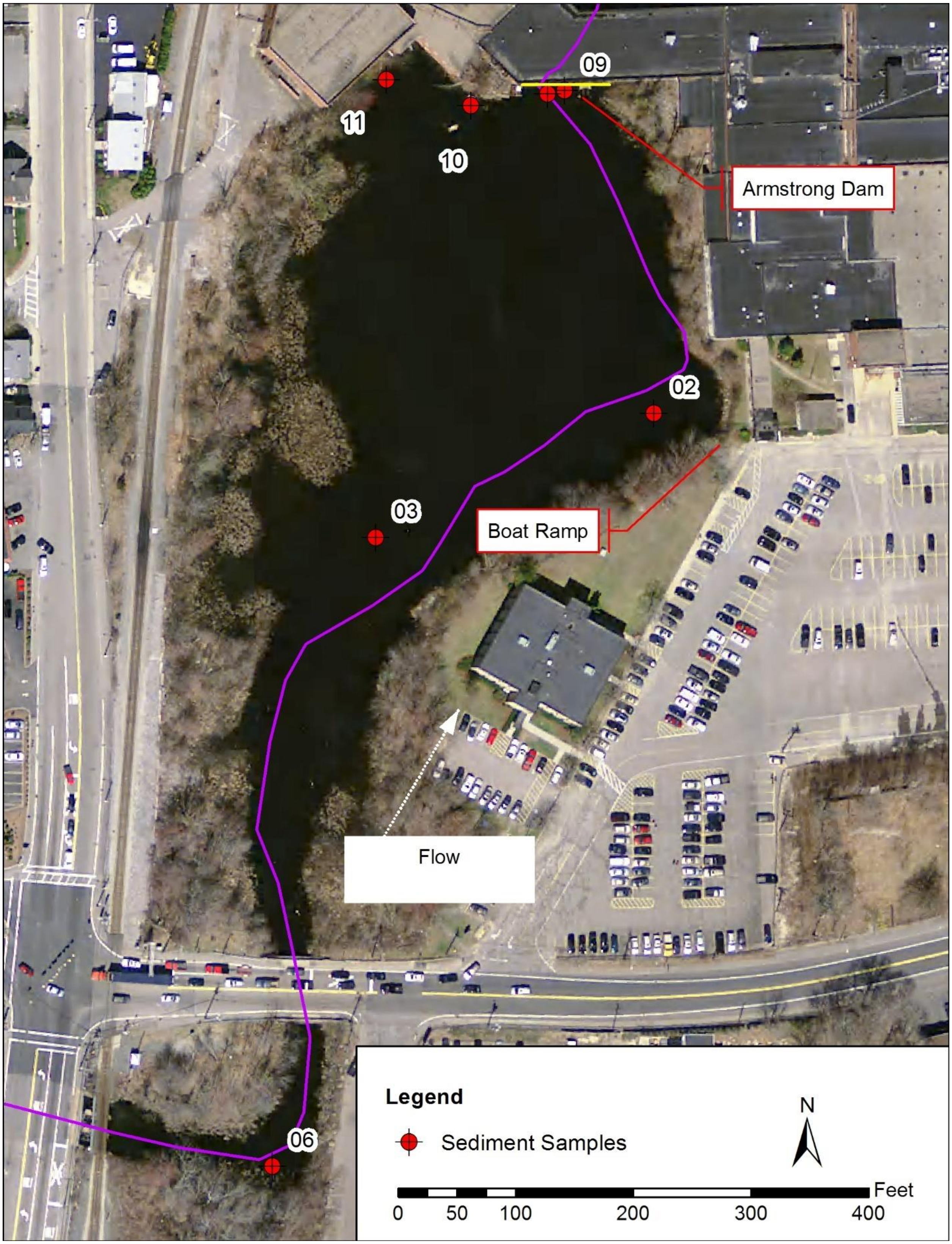

Figure 7.3-3: Impoundment Sediment Sampling Locations

Table 7.4-1: Sediment Sampling Results (Metals, PAHs, PCBs)

Parameter	Screening Benchmarks			Dam Impoundment Samples				Downstream Samples			Upstream Samples		Impoundment Statistics *(Immobile and Mobile Sediment Areas)*			Downstream Statistics *(Receiving Areas)*			Upstream Statistics *(Contributing Areas)*		
				9	2	3	6	1	7	8	4	5									
(Important: Units listed by category below)	**MCP S1/GW1** *Human Health*	**TEC** *Freshwater*	**PEC** *Freshwater*	L1618042-08	L1618042-07	L1618042-06	L1618042-05	L1618042-11	L1618042-04	L1618042-03	L1618042-01	L1618042-02	Min	Max	Mean	Min	Max	Mean	Min	Max	Mean
Metals [mg/kg]																					
Arsenic	20.0	9.79	33.0	10.0	4.4	3.2	15.0	3.5	3.3	**23.9**	3.0	12.5	3.2	15.0	8.1	3.3	**23.9**	10.2	3.0	12.5	7.8
Cadmium	0.9	0.99	4.98	**1.0**	0.6	0.3	**2.1**	**1.0**	0.6	**21.9**	0.4	0.5	0.3	**2.1**	**1.0**	0.6	**21.9**	**7.8**	0.4	0.5	0.5
Chromium (TOTAL)	100.0	43.4	111.0	21.1	35.4	6.3	17.0	5.6	9.3	**161.0**	5.2	5.5	6.3	35.4	20.0	5.6	**161.0**	58.6	5.2	5.5	5.3
Chromium (III)																					
Chromium (VI)	100.0																				
Copper		31.6	149.0	101.0	314.0	8.2	54.3	139.0	23.2	140.0	6.2	8.9	8.2	314.0	119.4	23.2	140.0	100.7	6.2	8.9	7.5
Lead	200.0	35.8	128.0	144.0	93.6	32.7	116.0	26.5	52.1	**401.0**	12.6	24.7	32.7	144.0	96.6	26.5	**401.0**	159.9	12.6	24.7	18.7
Mercury	20.0	0.18	1.06	0.6	0.1	0.0	0.2	0.1	0.2	1.1	0.0	0.0	0.0	0.6	0.2	0.1	1.1	0.4	0.0	0.0	0.0
Nickel	600.0	22.7	48.6	9.3	8.4	5.3	24.5	4.3	8.2	30.5	4.3	5.8	5.3	24.5	11.9	4.3	30.5	14.3	4.3	5.8	5.0
Zinc	1,000.0	121.0	459.0	626.0	148.0	75.0	450.0	**2,210.0**	272.0	**1,310.0**	67.2	89.0	75.0	626.0	324.8	272.0	**2,210.0**	**1,264.0**	67.2	89.0	78.1
SVOCs (PAHs) [ug/kg]																					
Acenaphthene	4,000.0			1,480.0	3,100.0	48.1	76.6	18.0	62.4	205.0	6.4	6.4	48.1	3,100.0	1,176.2	18.0	205.0	95.1	6.4	6.4	6.4
Acenaphthylene	1,000.0			168.0	189.0	84.6	160.0	6.4	28.0	133.0	6.4	6.4	84.6	189.0	150.4	6.4	133.0	55.8	6.4	6.4	6.4
Anthracene	1,000,000.0	57.2	845.0	750.0	14,400.0	172.0	224.0	6.4	138.0	269.0	6.4	6.4	172.0	14,400.0	3,886.5	6.4	269.0	137.8	6.4	6.4	6.4
Benzo[a]anthracene	700.0	108.0	1,050.0	**3,860.0**	**36,600.0**	**819.0**	**2,960.0**	6.4	**950.0**	**2,470.0**	20.9	58.2	**819.0**	**36,600.0**	**11,059.8**	6.4	**2,470.0**	**1,142.1**	20.9	58.2	39.6
Benzo(a)pyrene	2,000.0	150.0	1,450.0	**2,100.0**	**38,000.0**	746.0	**2,780.0**	6.4	980.0	**2,760.0**	25.8	65.6	746.0	**38,000.0**	**10,906.5**	6.4	**2,760.0**	1,248.8	25.8	65.6	45.7
Benzo[b]fluoranthene	7,000.0	27.3	13,400.0	2,650.0	**39,900.0**	705.0	4,820.0	6.4	1,420.0	3,870.0	36.1	93.9	705.0	**39,900.0**	**12,018.8**	6.4	3,870.0	1,765.5	36.1	93.9	65.0
Benzo[g,h,i]perylene	1,000,000.0			1,220.0	26,400.0	501.0	2,240.0	6.4	817.0	2,320.0	27.9	57.3	501.0	26,400.0	7,590.3	6.4	2,320.0	1,047.8	27.9	57.3	42.6
Benzo[k]fluoranthene	70,000.0			1,660.0	33,400.0	642.0	2,030.0	6.4	673.0	2,110.0	35.3	59.8	642.0	33,400.0	9,433.0	6.4	2,110.0	929.8	35.3	59.8	47.6
Chrysene	70,000.0	166.0	1,290.0	2,500.0	43,700.0	838.0	3,670.0	6.4	1,160.0	3,070.0	34.5	83.9	838.0	43,700.0	12,677.0	6.4	3,070.0	1,412.1	34.5	83.9	59.2
Dibenz[a,h]anthracene	700.0	33.0	260.0	364.0	**6,340.0**	124.0	521.0	6.4	172.0	529.0	6.4	12.9	124.0	**6,340.0**	**1,837.3**	6.4	529.0	235.8	6.4	12.9	9.7
Fluoranthene	1,000,000.0	423.0	2,230.0	6,190.0	102,000.0	1,660.0	6,300.0	6.4	2,280.0	4,880.0	52.3	137.0	1,660.0	102,000.0	29,037.5	6.4	4,880.0	2,388.8	52.3	137.0	94.7
Fluorene	1,000,000.0	77.4	536.0	1,860.0	5,410.0	108.0	173.0	6.4	98.3	251.0	6.4	6.4	108.0	5,410.0	1,887.8	6.4	251.0	118.6	6.4	6.4	6.4
Indeno[1,2,3-cd]pyrene	7,000.0			1,350.0	**30,200.0**	532.0	2,380.0	6.4	883.0	2,310.0	37.9	70.4	532.0	**30,200.0**	**8,615.5**	6.4	2,310.0	1,066.5	37.9	70.4	54.2
2-Methylnaphthalene	700.0			70.0	69.0	69.0	64.0	68.0	38.0	61.0	59.0	49.0	64.0	70.0	68.0	38.0	68.0	55.7	49.0	59.0	54.0
Naphthalene	4,000.0	176.0	561.0	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9
Phenanthrene	10,000.0	204.0	1,170.0	4,500.0	**59,200.0**	881.0	2,330.0	6.4	1,090.0	2,000.0	18.2	62.1	881.0	**59,200.0**	**16,727.8**	6.4	2,000.0	1,032.1	18.2	62.1	40.2
Pyrene	1,000,000.0	195.0	1,520.0	7,790.0	74,000.0	1,400.0	4,680.0	6.4	1,730.0	4,310.0	43.6	121.0	1,400.0	74,000.0	21,967.5	6.4	4,310.0	2,015.5	43.6	121.0	82.3
Total PAHs (calculated)		1,610.0	22,800.0	38,515.9	512,911.9	9,333.6	35,412.5	179.5	12,523.6	31,551.9	427.4	900.6									
PCBs (ug/kg)																					
2,4'-Dichlorobiphenyl				0.6	0.6	0.6	1.8	0.6	0.6	0.6	0.6	0.6	0.6	1.8	0.9	0.6	0.6	0.6	0.6	0.6	0.6
2,2',5-Trichlorobiphenyl				0.6	0.6	1.4	2.1	0.6	0.6	0.6	0.6	0.6	0.6	2.1	1.2	0.6	0.6	0.6	0.6	0.6	0.6
2,4,4'-Trichlorobiphenyl				0.6	2.3	3.8	6.3	0.6	1.3	0.6	0.6	0.6	0.6	6.3	3.3	0.6	1.3	0.8	0.6	0.6	0.6
2,2',3,5'-Tetrachlorobiphenyl				0.6	2.3	2.2	4.0	0.6	0.8	11.4	0.6	0.6	0.6	4.0	2.3	0.6	11.4	4.3	0.6	0.6	0.6
2,2',4,5'-Tetrachlorobiphenyl				0.6	1.2	1.9	3.5	0.6	4.9	10.9	0.6	0.6	0.6	3.5	1.8	0.6	10.9	5.5	0.6	0.6	0.6
2,2',5,5'-Tetrachlorobiphenyl				0.6	5.8	2.8	7.1	0.6	2.2	40.5	0.6	0.6	0.6	7.1	4.1	0.6	40.5	14.4	0.6	0.6	0.6
2,3',4,4'-Tetrachlorobiphenyl				0.6	2.3	1.8	4.8	0.6	2.9	9.6	0.6	0.6	0.6	4.8	2.4	0.6	9.6	4.4	0.6	0.6	0.6
2,2',3,4,5'-Pentachlorobiphenyl				0.6	2.7	1.3	5.8	0.6	1.6	14.3	0.6	0.6	0.6	5.8	2.6	0.6	14.3	5.5	0.6	0.6	0.6
2,2',4,5,5'-Pentachlorobiphenyl				0.6	7.1	2.4	21.3	0.6	5.9	35.1	0.6	0.6	0.6	21.3	7.9	0.6	35.1	13.9	0.6	0.6	0.6
2,3,3',4,4'-Pentachlorobiphenyl				0.6	0.6	0.6	5.2	0.6	0.6	0.6	0.6	0.6	0.6	5.2	1.8	0.6	0.6	0.6	0.6	0.6	0.6
2,3',4,4',5-Pentachlorobiphenyl				0.6	5.1	2.1	8.6	0.6	5.1	31.2	0.6	0.6	0.6	8.6	4.1	0.6	31.2	12.3	0.6	0.6	0.6
2,2',3,3',4,4'-Hexachlorobiphenyl				0.6	0.6	0.8	12.1	0.6	1.9	7.8	0.6	0.6	0.6	12.1	3.5	0.6	7.8	3.4	0.6	0.6	0.6
2,2',3,4,4',5'-Hexachlorobiphenyl				3.4	12.0	5.3	88.9	0.6	7.8	44.4	0.6	1.1	3.4	88.9	27.4	0.6	44.4	17.6	0.6	1.1	0.9
2,2',4,4',5,5'-Hexachlorobiphenyl				2.3	10.9	4.8	81.6	0.6	5.5	37.0	0.6	0.6	2.3	81.6	24.9	0.6	37.0	14.4	0.6	0.6	0.6
2,2',3,3',4,4',5-Heptachlorobiphenyl				0.6	5.6	2.4	52.6	0.6	1.5	17.4	0.6	0.6	0.6	52.6	15.3	0.6	17.4	6.5	0.6	0.6	0.6
2,2',3,4,4',5,5'-Heptachlorobiphenyl				2.7	11.2	4.7	84.2	0.6	2.0	38.1	0.6	0.6	2.7	84.2	25.7	0.6	38.1	13.6	0.6	0.6	0.6
2,2',3,4,4',5',6-Heptachlorobiphenyl				0.6	2.6	1.2	19.8	0.6	0.6	8.0	0.6	0.6	0.6	19.8	6.1	0.6	8.0	3.1	0.6	0.6	0.6
2,2',3,4,4',6,6'-Heptachlorobiphenyl				0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
2,2',3,4',5,5',6-Heptachlorobiphenyl				1.4	6.0	2.2	43.5	0.6	1.1	21.1	0.6	0.6	1.4	43.5	13.3	0.6	21.1	7.6	0.6	0.6	0.6
2,2',3,3',4,4',5,6-Octachlorobiphenyl				0.6	1.6	0.6	9.2	0.6	0.6	4.4	0.6	0.6	0.6	9.2	3.0	0.6	4.4	1.9	0.6	0.6	0.6
2,2',3,3',4,4',5,5',6-Nonachlorobiphenyl				0.6	1.1	0.6	4.3	0.6	0.6	3.9	0.6	0.6	0.6	4.3	1.7	0.6	3.9	1.7	0.6	0.6	0.6
Decachlorobiphenyl				2.6	12.7	0.6	1.6	0.6	2.0	62.4	0.6	1.4	0.6	12.7	4.4	0.6	62.4	21.7	0.6	1.4	1.0
Total PCBs (calculated)	1,000.0	59.8	676.0	41.6	177.1	79.8	878.8	23.0	86.2	733.7	23.0	25.5									

Table 7.4-2: Sediment Sampling Results (Pesticides and Physical Characteristics)

Parameter	Screening Benchmarks			Dam Impoundment Samples				Downstream Samples			Upstream Samples		Impoundment Statistics *(Immobile and Mobile Sediment Areas)*			Downstream Statistics *(Receiving Areas)*			Upstream Statistics *(Contributing Areas)*		
	Sample Number --->			9	2	3	6	1	7	8	4	5									
(Important: Units listed by category below)	**MCP S1/GW1** *Human Health*	**TEC** *Freshwater*	**PEC** *Freshwater*	L1620719-08	L1620719-07	L1620719-06	L1620719-05	L1620719-09	L1620719-04	L1620719-03	L1620719-01	L1620719-02	Min	Max	Mean	Min	Max	Mean	Min	Max	Mean
Pesticides (ug/kg)																					
Alachlor				N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Aldrin	80.0			0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
alpha-BHC				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
beta-BHC				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
delta-BHC				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
gamma-BHC		2.4	5.0	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Chlordane	5,000.0	3.2	17.6	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7	15.7
4,4'-DDD	4,000.0	4.88	28.0	7.2	23.2	11.8	33.9	0.3	12.0	551.0	2.8	11.7	7.2	33.9	19.0	0.3	551.0	187.8	2.8	11.7	7.3
4,4'-DDE	3,000.0	3.16	31.3	10.8	11.7	5.7	27.3	0.3	3.6	126.0	2.7	9.2	5.7	27.3	13.9	0.3	126.0	43.3	2.7	9.2	5.9
4,4'-DDT	3,000.0	4.16	62.9	0.3	20.7	0.3	48.9	0.3	5.5	53.0	0.3	0.6	0.3	48.9	17.6	0.3	53.0	19.6	0.3	0.6	0.5
Total DDTs (calculated)		5.28	572.0	18.3	55.6	17.8	110.1	0.9	21.0	730.0	5.8	21.5	17.8	110.1	50.4	0.9	730.0	250.6	5.8	21.5	13.7
Dieldrin	80.0	1.9	61.8	0.3	16.6	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	16.6	4.4	0.3	0.3	0.3	0.3	0.3	0.3
Endosulfan I				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Endosulfan II				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Endosulfan Sulfate				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Endrin	10,000.0	2.2	207.0	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Endrin Aldehyde				N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Endrin Ketone				0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Heptachlor	300			0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Heptachlor Epoxide	100.0	2.5	16.0	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Hexachlorobenzene	700.0			1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Methoxychlor				3.1	237.0	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	237.0	61.6	3.1	3.1	3.1	3.1	3.1	3.1
	MCP S1/GW1 *Human Health*	**TEC** *Freshwater*	**PEC** *Freshwater*	L1618042-08	L1618042-07	L1618042-06	L1618042-05	L1618042-11	L1618042-04	L1618042-03	L1618042-01	L1618042-02									
Physical Characteristics																					
Total Organic Carbon (mg/kg)				5.18	3.66	2.47	16.3	11.7	11.9	17.3	5.97	4.71									
Percent Solids (%)				49.7	44	62.4	24.6	36.7	56.4	21.3	30.9	32.2									
Percent Water (%)				50.3	56	37.6	75.4	63.3	43.6	78.7	69.1	67.8									
Grain Size Distribution (%)																					
Sieve No. 4				19.6	4	1.3	1	0.05	8.9	0.2	0.6	3.7									
Sieve No. 10				11.4	4.4	3.8	6.9	1.1	16.5	1.7	2.3	4									
Sieve No. 40				18.4	27	63.5	20.2	3.7	43.8	7.5	9.8	15.7									
Sieve No. 60				32.2	47	25.5	41.2	22.2	24.4	14.4	65.7	58.8									
Sieve No. 200				18.4	17.6	5.9	30.7	73	6.4	76.2	21.6	17.8									

NOTES: Values in green are below the laboratory detection limit (BDL); a value of 1/2 the detection limit is provided. No TEC or PEC values exist for 4'4 DDD, DDE, or DDT. This sheet used the TEC and PEC values for the SUM of DDE, DDD, and DDT, respectively, to provide a conservative value for comparison. Total PCBs are calculated as the sum of aroclars; total PAHs are similarly calculated by summing values. Percent water is inferred from percent solids.

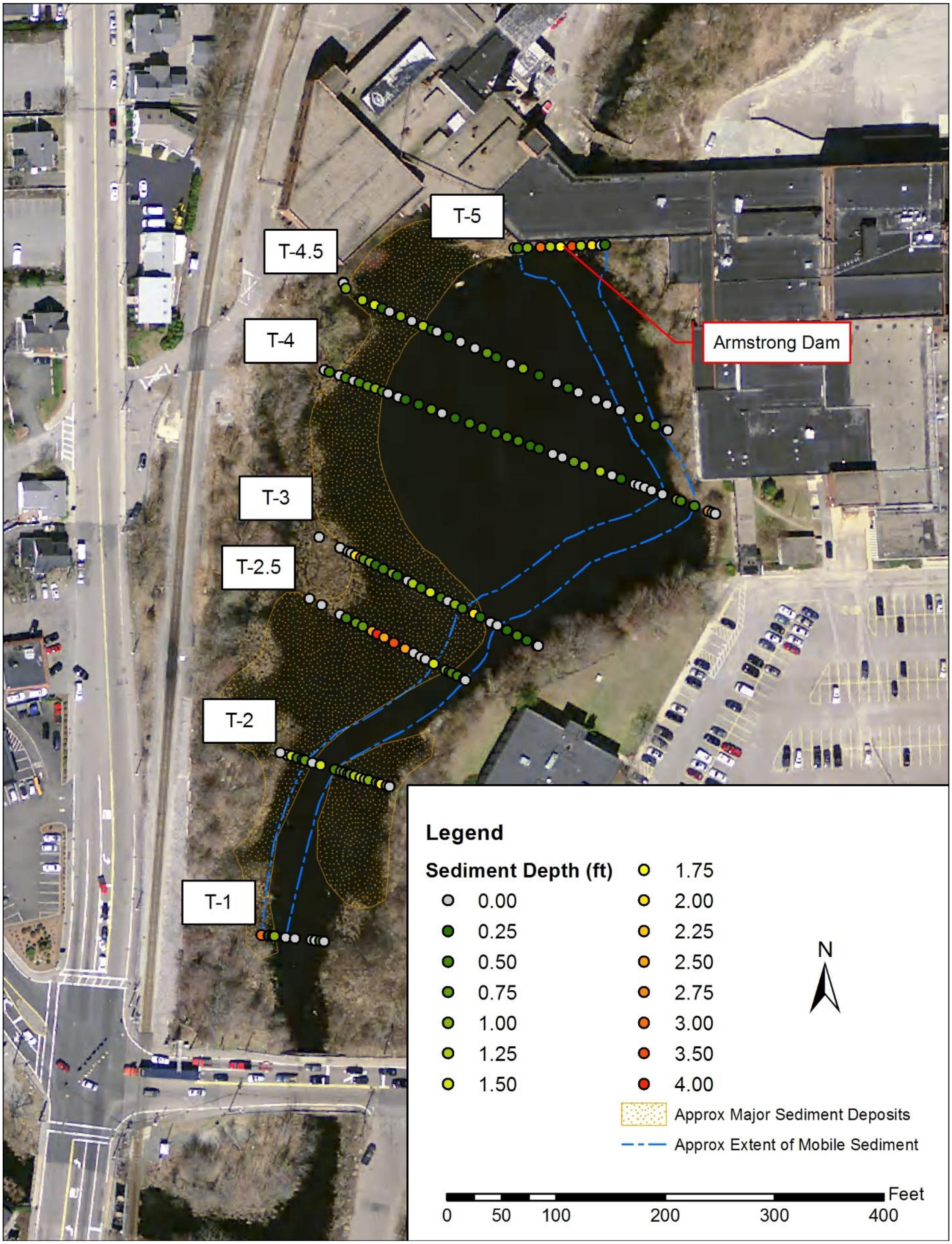

Figure 7.5-1: Sediment Depth

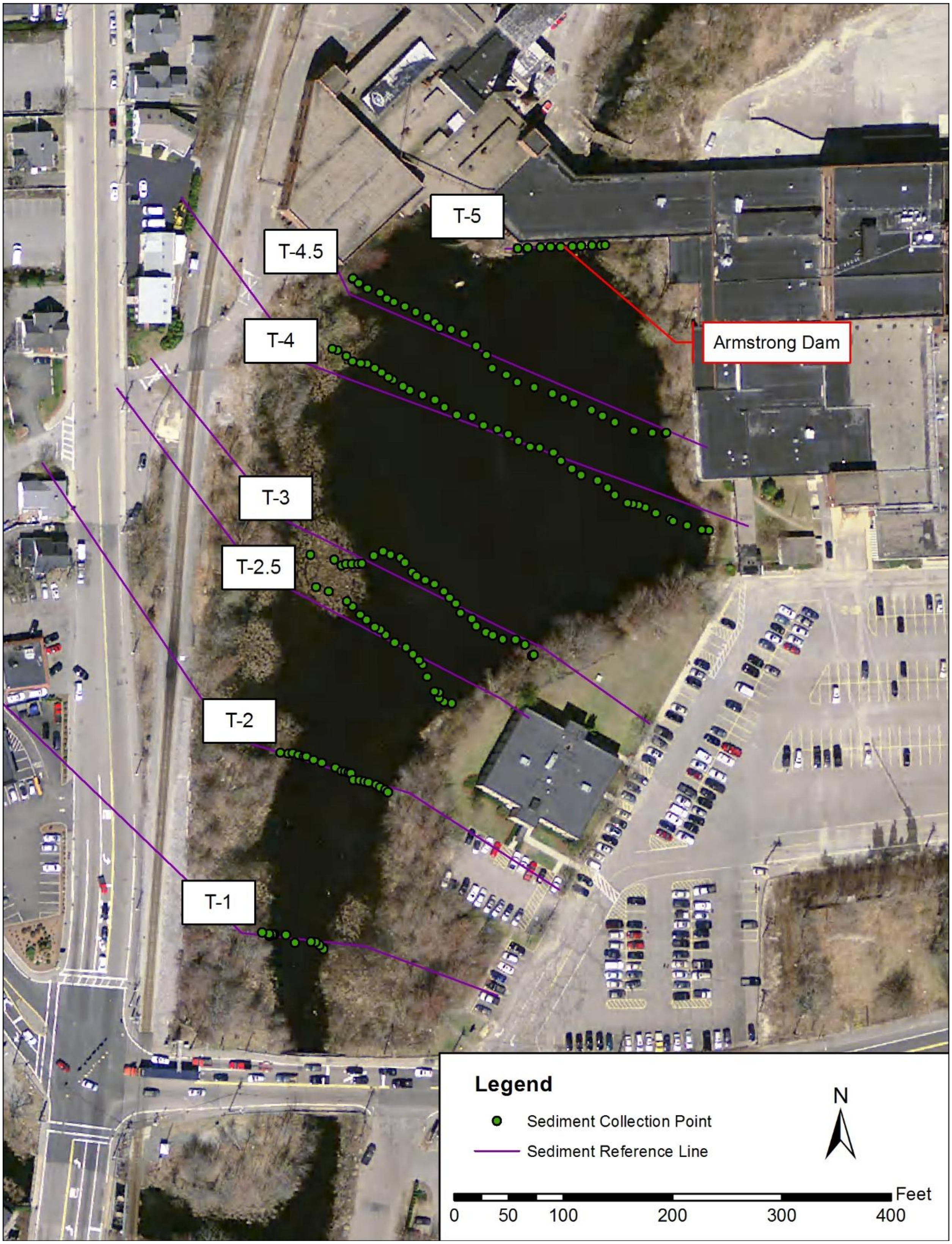

Figure 7.5-2: Sediment Probing Collection Points and Reference Lines

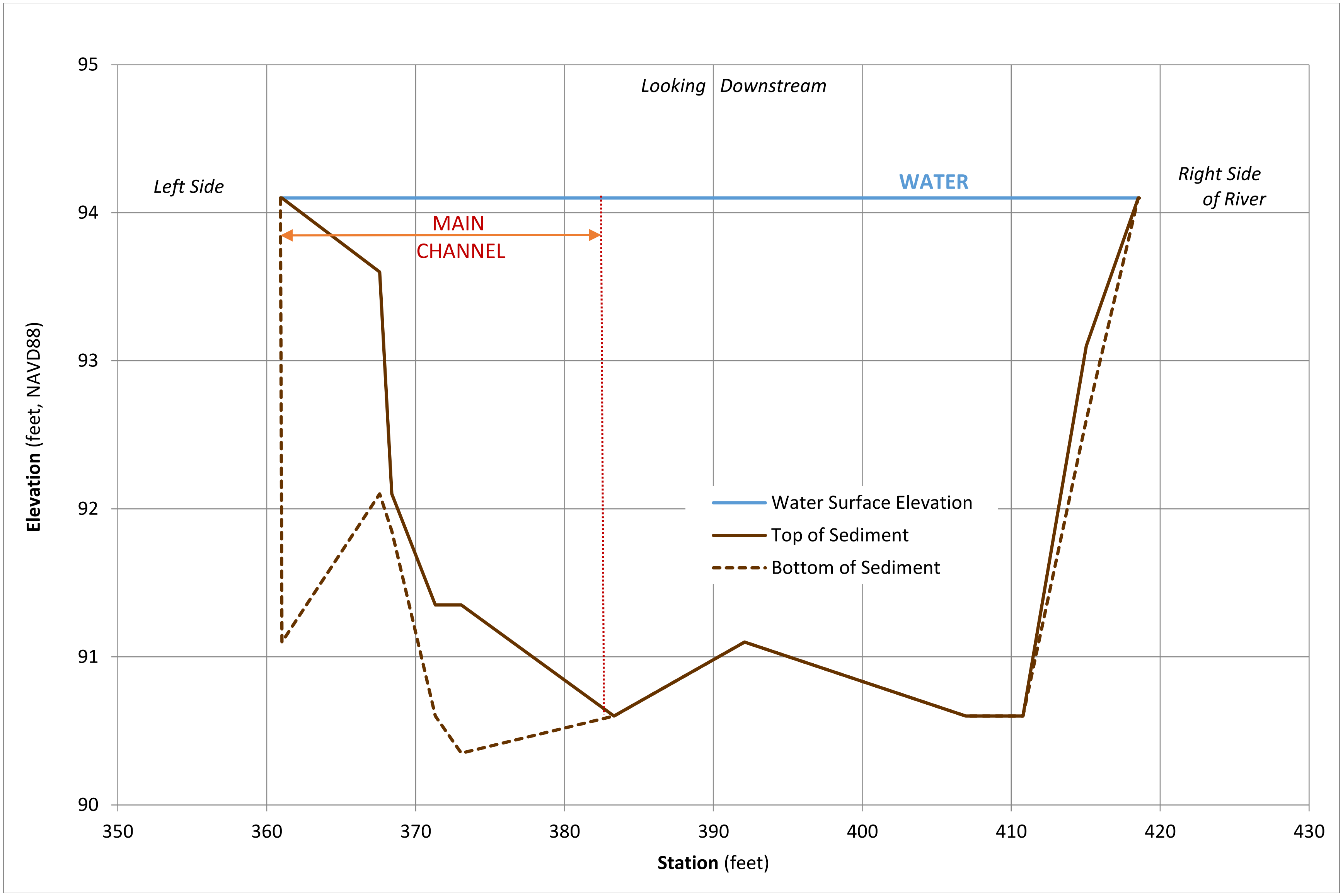

Figure 7.5-3: Sediment Depth Transect T-1

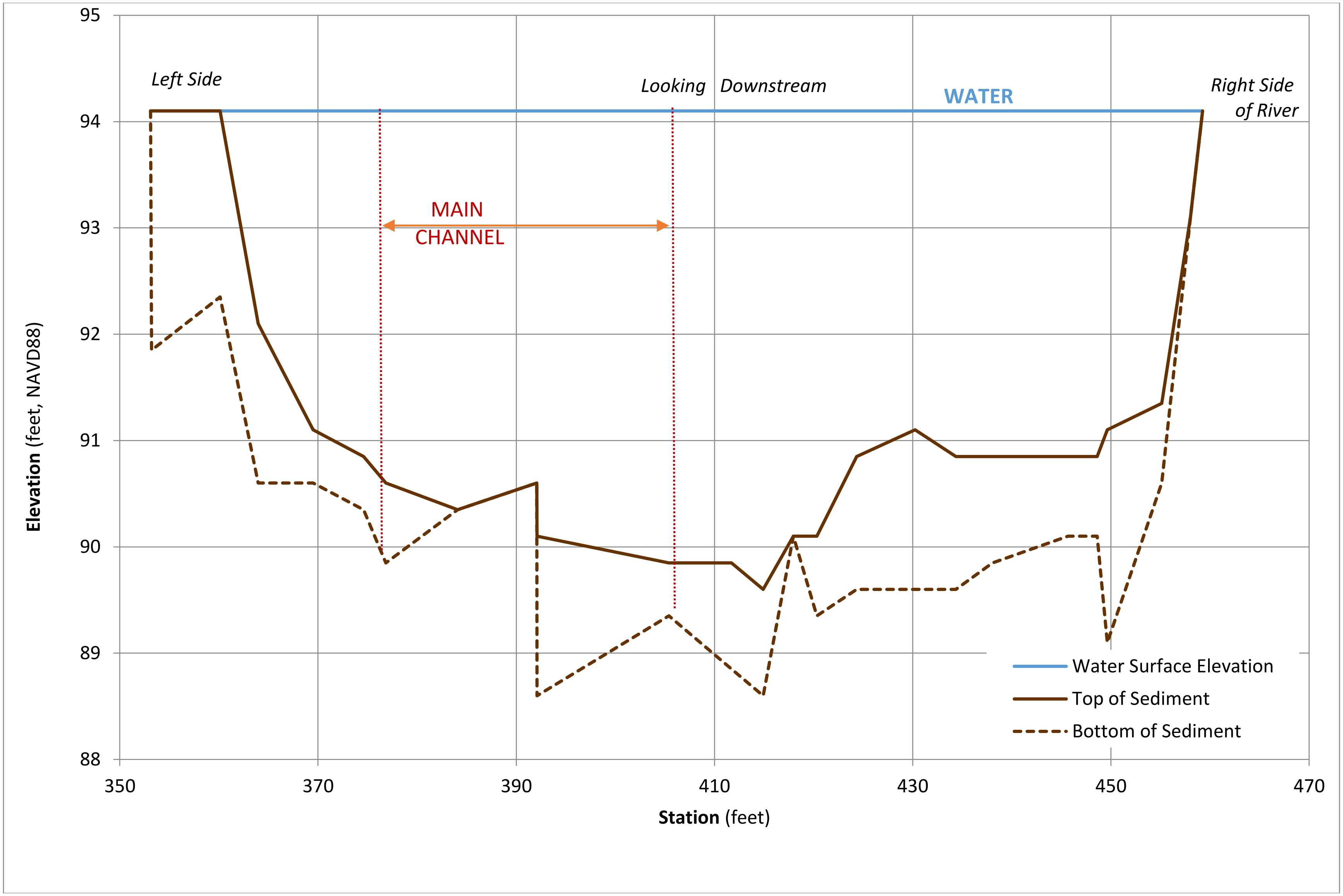

Figure 7.5-4: Sediment Depth Transect T-2

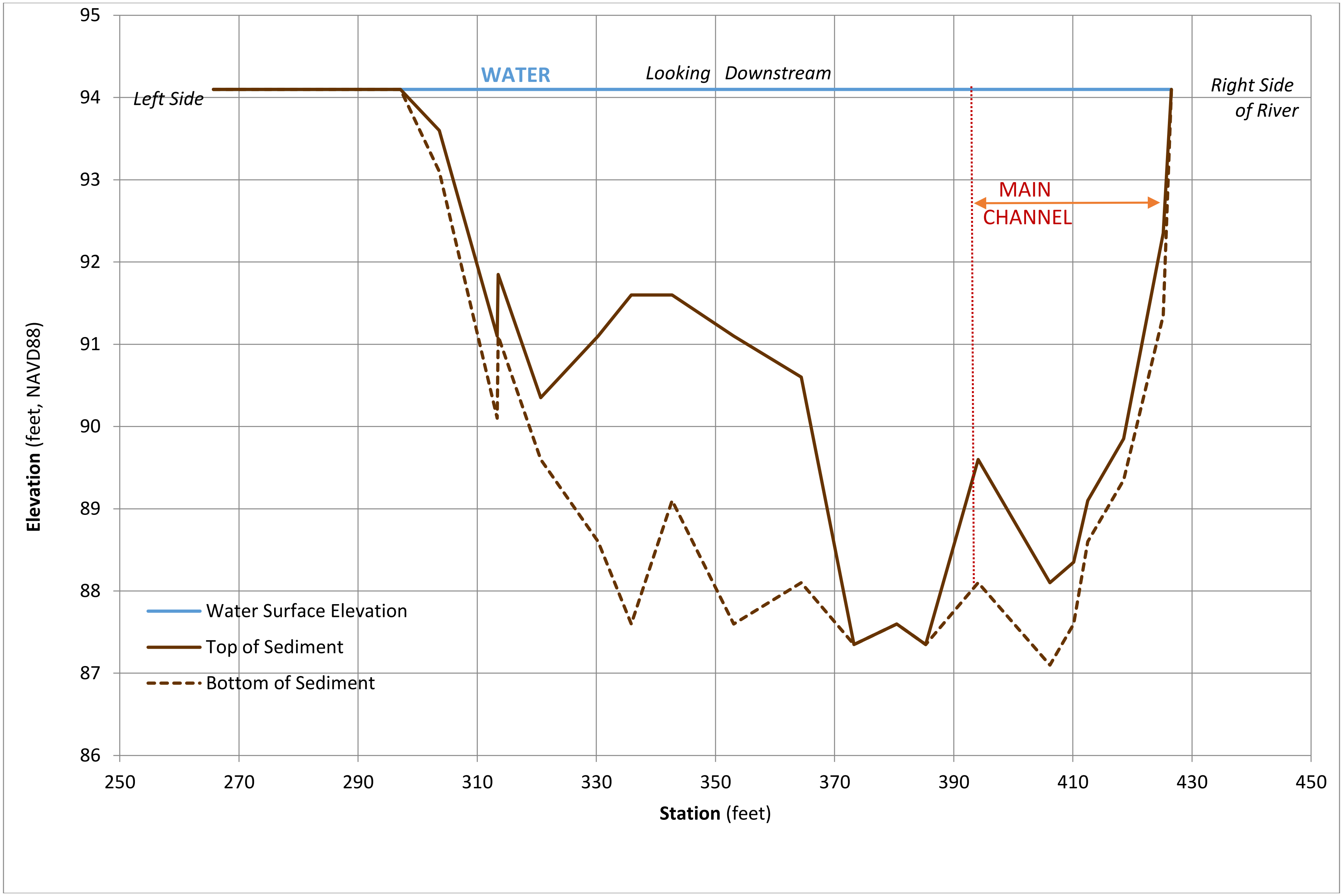

Figure 7.5-5: Sediment Depth Transect T-2.5

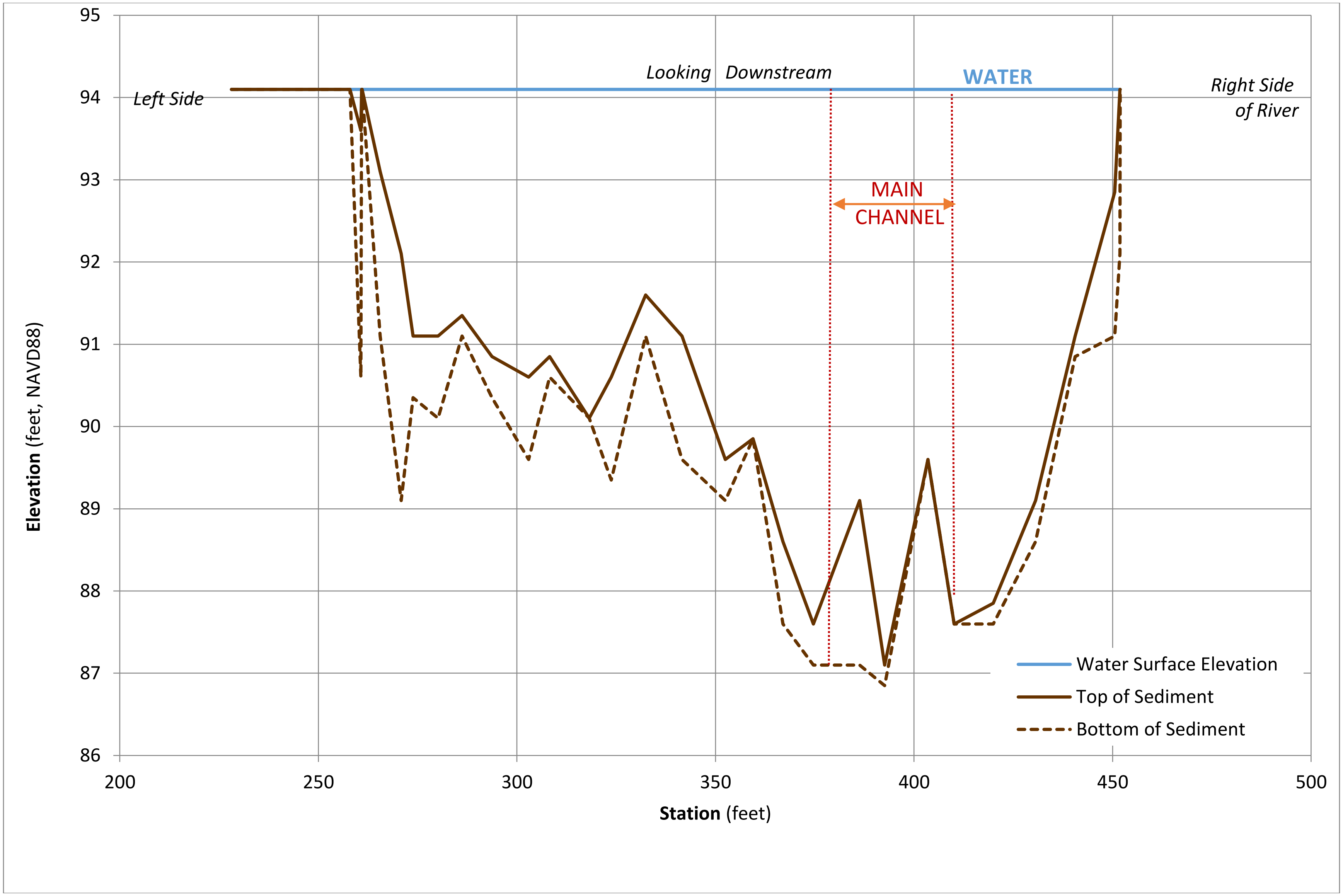

Figure 7.5-6: Sediment Depth Transect T-3

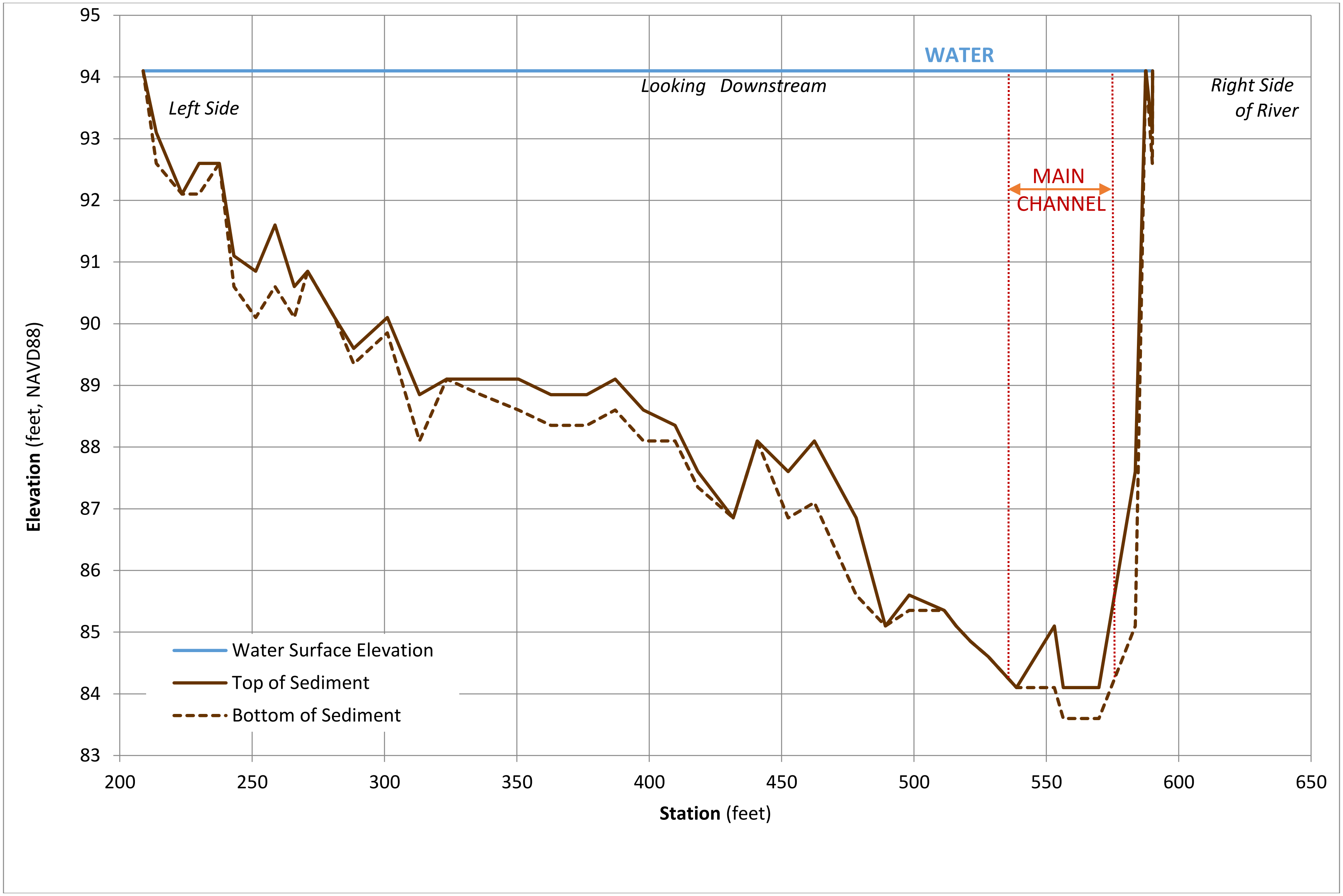

Figure 7.5-7: Sediment Depth Transect T-4

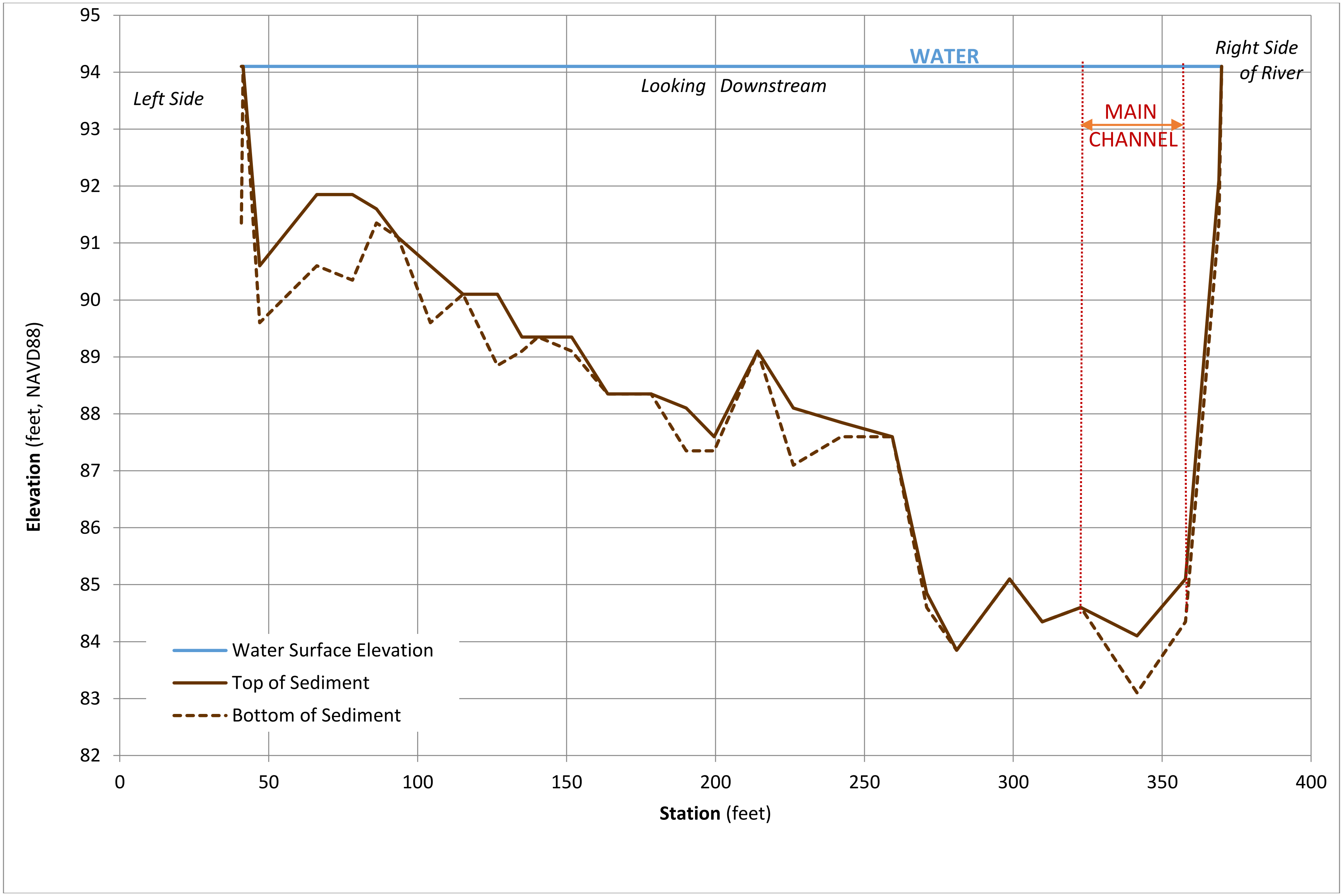

Figure 7.5-8: Sediment Depth Transect T-4.5

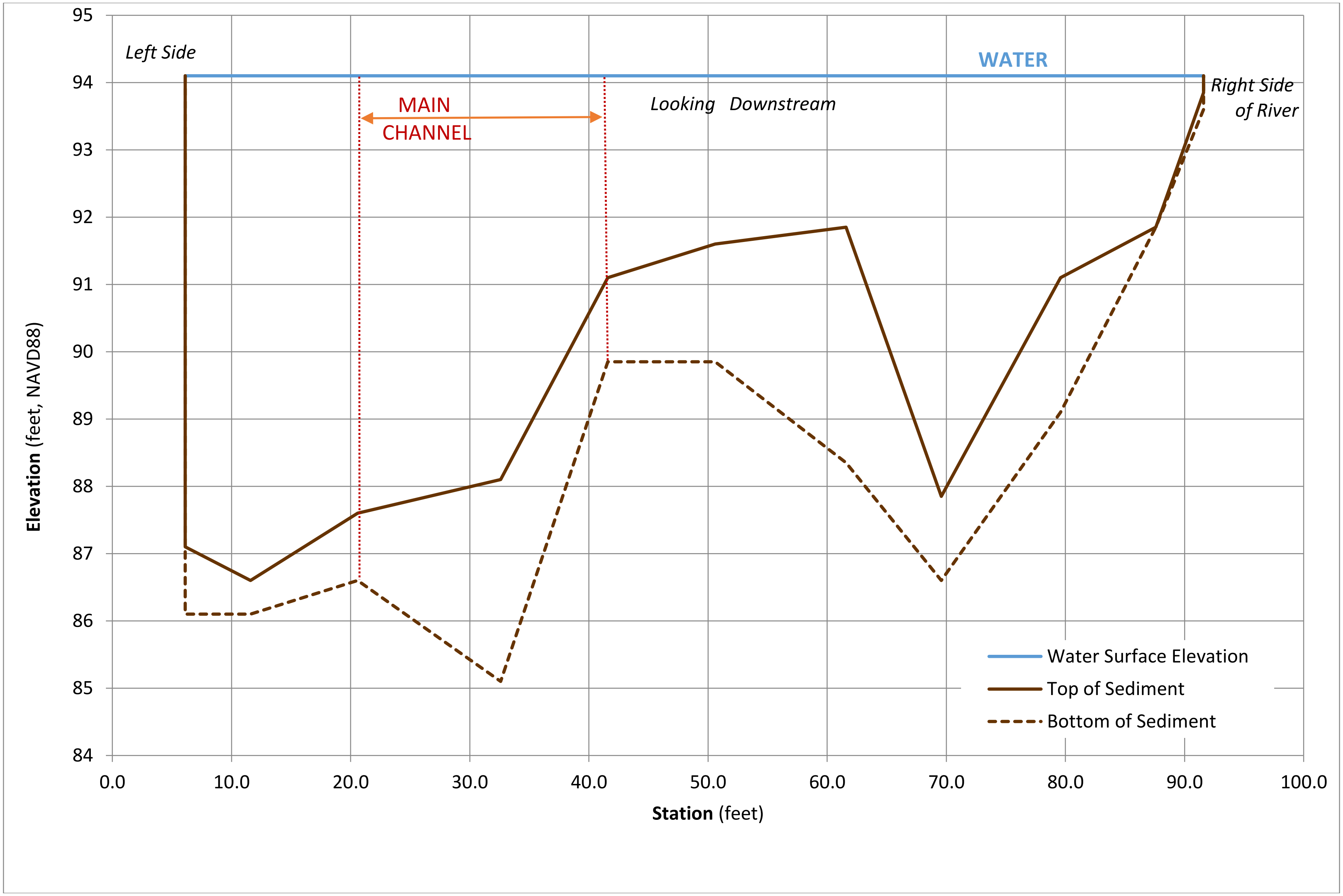

Figure 7.5-9: Sediment Depth Transect T-5 (Dam)

8. Hydrologic Analysis

A hydrological assessment of the Monatiquot River watershed was conducted for the 2009 FS (GSE, 2009) to determine the magnitude and seasonal availability of flow during the upstream and downstream river herring migration seasons. The critical factors investigated included: a) is there sufficient streamflow under current conditions to permit river herring migration to Great Pond assuming barriers are mitigated and b) if streamflow is a limiting factor, are there options relative to dam releases, reduced water supply withdrawals or other measures to facilitate river herring migration. For the current FS, hydrological data used in the hydraulic model were extracted from the 2009 FS, i.e. the analysis was not updated. This section summarizes how the 2009 hydrological data were calculated.

8.1 Hydrological Assessment

A United States Geological Survey (USGS) flow monitoring gage (Gage No. 01005583) was installed on the Monatiquot River on March 31, 2006. Because the period of record includes 10 years of record (still considered a limited period from a hydrologic perspective, it was placed into context with another USGS gage having a longer period of record, a similar size drainage area and similar basin characteristics. As described in the 2009 FS report, the East Branch Neponset River USGS gage (Gage No. 01105500) was selected as it has a long period of record (installed in 1952), is in relatively close proximity to the Monatiquot River, and a regression analysis showed a relatively close relationship[27] between flows on each river for the common period of record[28]. The drainage areas of the Monatiquot and East Branch Neponset River gages are 28.7 and 27.2 square miles, respectively. The flows on the East Branch Neponset River were adjusted by a ratio of drainage areas to estimate the flow at the USGS gage on the Monatiquot River.

Flows were subsequently estimated at key locations in the basin using a) the adjusted East Branch Neponset River gage flows (57 years of data) and b) the observed Monatiquot River gage flows (10 years of data). Flows at locations other than at the USGS gage were estimated by a ratio of drainage areas. Shown in Table 8.1-1, below, is the estimated average annual flow at key locations on the Monatiquot River.

Table 8.1-1: Drainage Area and Average Annual Flow Data

Location	Drainage Area (mi^2)	Monatiquot River 03/31/2006-05/06/2008	Adjusted East Branch Neponset River flow 10/01/1952-05/06/08
Farm River at confluence with Monatiquot River	12.9	23.7 cfs	24.8 cfs
Cochato River at confluence with Monatiquot River	11.1	20.4 cfs	21.4 cfs
Monatiquot River at Armstrong Dam	25.9	47.6 cfs	49.9 cfs
Monatiquot River at USGS Gage	28.7	52.7 cfs	55.2 cfs

[27] The coefficient of determination, or R^2, provides a measure of how closely related the flows correlate. The annual coefficient of determination was $R^2 = 0.85$, although monthly regression analyses were not as high for March, September and October which were 0.65, 0.25 and 0.62 respectively.
[28] The common period of record at the time of the 2009 Report was March 31, 2006 – May 06, 2008.

8.2 Flow Used in the Hydraulic Assessment

As described in Section 9, a hydraulic model was developed for this FS. The purpose of the model is to evaluate water depth and velocities under dam-in and dam-out conditions and determine the effects on infrastructure and fish passage. For hydraulic modeling purposes flood flows, low flows, and flows during river herring migration season were simulated.

Flood Flows

The FEMA conducted a flood insurance study (FIS) of the Monatiquot River and predicted the 10-, 50-, 100- and 500-year flood flows at various locations. The FIS estimated flood flows on the Monatiquot River at virtually the same location at the Armstrong Dam. Shown in Table 8.2-1 are the flood flows reported in the Braintree FIS, which were simulated in the hydraulic model.

Table 8.2-1: Flood Flows at the Armstrong Dam used in Hydraulic Model

Condition	Flow	Source
50-year flood	1,700 cfs	Braintree Flood Insurance Study
100-year flood	2,100 cfs	Braintree Flood Insurance Study

Fish Passage Flows

Flows during river herring migration months were also examined for this FS. The peak upstream migration month is May, while the downstream migration may last from September through November. As noted above, because the period of record for the Monatiquot River gage is relatively short, the mean monthly flows based on the East Branch Neponset River flows were prorated by the ratio of drainage areas for hydraulic modeling purposes. Shown in Table 8.2-2, below, are the May, September, October and November average monthly flows based on adjusting the East Branch Neponset River flows to represent flow at Armstrong Dam. The period of record from the 2009 Feasibility Study is shown; these data were not updated. Shown in the far right-hand column of Table 8.2-2 are the flows used in the hydraulic model; the October flow was dropped since it is already bracketed by flows in September[29] and November.

Table 8.2-2: Estimated Average May, September and October Flows at Armstrong Dam

Peak Migration Months	East Branch 10/01/1952-05/06/08	Flows Used in Hydraulic Model
May- upstream migration	53.2 cfs	53 cfs
Sep- downstream migration	20.6 cfs	21 cfs
Oct- downstream migration	30.3 cfs	----
Nov- downstream migration	45.1 cfs	45 cfs

[29] From the 2009 FS report, the correlation coefficient for the common period of record for September was 0.25. It is suspected that 21 cfs is a high estimate due to water supply diversions and hydrological conditions.

8.3 Hydrology Check

Although the hydrological analysis from the 2009 Report was not updated for the purposes of this study the assumption of prorating the flow from the East Branch of the Neponset River by the ratio of drainage areas to the Monatiquot River was confirmed. Table 8.2-3, below, provides a comparison of the average annual flow data for the period of hydrological record at each USGS gage.

Table 8.3-1: Drainage Area and Average Annual Flow Data

Location	Drainage Area (mi^2)	Monatiquot River 03/31/2006 - 06/30/2016 (cfs)	Adjusted East Branch Neponset River Flow 03/31/2006-06/30/2016 (cfs)	Adjusted East Branch Neponset River flow 10/01/1952-06/30/2016 (cfs)
Farm River at confluence with Monatiquot River	12.9	21.8	26.0	24.9
Cochato River at confluence with Monatiquot River	11.1	18.8	22.4	21.4
Monatiquot River at Armstrong Dam	25.9	43.8	52.2	50.0
Monatiquot River at USGS Gage	28.7	48.5	57.8	55.4

Since the hydrological analysis from the 2009 Report (Table 8.1-1) average annual flows have increased slightly on the East Branch of the Neponset River for the period of record. As Table 8.3-1 shows, there is a slight increase in the flow over an approximately ten-year period. Reanalyzing the period of record for average annual flows on the Monatiquot River a slight decrease in flow has occurred at the gage, from 52.7 cfs to 48.5 cfs. The ratio of flows at the two rivers has deviated slightly from the 2008 Report; however, the period of record from the Monatiquot River is still too short to justify doing another hydrological analysis.

9. Hydraulic Analysis

9.1 HEC-RAS Model Background

A hydraulic model was developed using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center River Analysis System (HEC-RAS) program. The modeling software is publically available free-of-charge on the USACE's website. The HEC-RAS hydraulic model predicts water surface elevations (WSELs), river depths, and mean channel velocities at various transects under a range of flows. This section provides brief technical background on how HEC-RAS predicts water depths, WSELs, velocities, and water surface profiles (WSP) along the studied reach of the Monatiquot River. This section contains technical terms relating to hydraulics and hydrology. Whenever possible, effort has been made to simplify hydraulic concepts presented; however, if further clarification or explanation is desired, the reader is referred to the HEC-RAS Hydraulic Reference Manual (Brunner, 2010) http://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS_4.1_Reference_Manual.pdf or any standard open channel flow text.

HEC-RAS is designed to perform one-dimensional, steady (flow does not change over time), gradually varied flow calculations in natural and manmade channels, as well as to perform unsteady (flow changes over a time) flow routing. The model can simulate depths, WSELs and velocities for a single reach, a branched system, or a full network of channels. HEC-RAS can simulate subcritical, supercritical, and mixed flow regimes.

Hydraulic analyses performed by HEC-RAS are based upon a step-wise solution of the one-dimensional energy equation. In instances of rapid change in the WSEL causing turbulence and energy loss, HEC-RAS uses the momentum equation. In HEC-RAS, rapid changes in the WSEL may occur under the following conditions: bridge constrictions, inline structures (dams and weirs), confluences of two or more flows, rapid changes in the channel bed elevation, and hydraulic jumps. Energy losses in the channel are associated with friction (solved with Manning's equation) or with contraction and expansion (solved by multiplying a loss coefficient by the change in velocity head between transects).

Manning's equation states:

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}},$$

Where:
V = Velocity (ft/s)
R = Hydraulic radius (ft)
S = Bed slope (ft/ft)
n = Manning's coefficient of roughness (accounts for friction)

Head losses due to contraction and expansion are evaluated in HEC-RAS by the following equation:

$$h_{ce} = C \left| \frac{\alpha_1 V_1^2}{2g} - \frac{\alpha_2 V_2^2}{2g} \right|$$

Where:
h_{ce} = Head loss due to contraction and expansion (ft)
C = Contraction or expansion coefficient
α_n = Velocity weighting coefficient at transect n
V_n = Velocity at transect n (ft/s)
g = Gravitational constant (ft/s^2)

HEC-RAS also permits the modeler to include gate structures that accompany inline structures such as dams. Flows over weirs and other inline structures (dams) are determined with the standard weir equation. The standard weir equation states:

$$Q = CLH^{\frac{3}{2}},$$

Where:
Q = Volumetric flow rate (cfs)
C = Discharge coefficient for the weir structure
L = Width of weir (ft)
H = Height of the water over the weir (ft)

The discharge coefficient in the equation above is a constant that represents the efficiency of a particular weir at passing water (Brater and King, 1996).

Steady and Unsteady-State Conditions
As noted above, the HEC-RAS model can operate in either steady-state or unsteady-state conditions. Steady state means flows remain constant; unsteady state means flow can vary over time. The hydraulic model for this study is steady state where a constant flow was used to determine the WSEL at various locations.

9.2 Data

Model transects were obtained from several sources including survey data collected exclusively for this hydraulic model, survey data collected for the sediment depth transects, surveyed transects from the 2009 FS and the FIS. Table 9.2-1 summarizes the sources of bathymetric and topographic data. Figure 9.2-1 shows the transect locations and model extents.

Survey Data

Upstream of the Armstrong Dam, the Monatiquot River was surveyed at three transects on November 3, 2015. A line, survey rod and level were used to collect station and elevation data. At each transect, benchmarks, headpins and tailpins were surveyed in with RTK-GPS to determine transect locations and tie elevation data into the Project datum. The surveyed sediment depth transects located in the Armstrong Dam pond (described earlier) were also included in the hydraulic model.

Downstream of Armstrong Dam, and underneath the Armstrong Cork Building, two additional transects were surveyed on December 17, 2015. A total station was utilized for this survey and temporary benchmarks set outside the building served to triangulate the data and place it into the Project datum.

The accuracy of the survey data for the upstream and downstream transects is about 0.1 ft as described in Section 6.2.

Water Level Logger Data

The 2009 hydraulic model terminated at the confluence of Farm and Cochato Rivers; however, it was unclear if the Armstrong Dam caused the Monatiquot River to backwater further upstream—particularly under high flows-- as it could impact the hydraulics (depths and velocities) if the dam were removed. To help tease out the influence of the dam's backwater and to help calibrate the hydraulic model, six WLLs were installed on September 8, 2015 to help establish the upstream boundary of the Armstrong Dam Pond under a range of flows. Shown in Figure 9.2-2 and summarized below are the WLL locations:

- Just upstream of Armstrong Dam
- Just upstream of the Jefferson Street Bridge
- Just downstream of the confluence of the Farm and Cochato Rivers
- On the Farm River
- On the Cochato River

Water Level Logger Accuracy

The WLLs used to measure the WSELs were HOBO U20-001-01. These loggers measure absolute pressure and have an operation range of approximately 0 to 30 feet. They have a maximum water level accuracy of ± 0.03 ft, assuming accurate barometric compensation data, with an approximately 0.007 ft resolution. The logger used to record the barometric pressure was the HOBO U20-001-04[30]. The maximum raw pressure accuracy of this logger is 0.063 pounds per square inch (psi), with a resolution of approximately 0.002 psi. Converting the barometric pressure to head, this could contribute to an additional 0.15 ft of inaccuracy. A composite accuracy of ± 0.18 ft is found by summing up the level of accuracy due to water level and the level of accuracy due to barometric pressure.

Water Level Logger Analysis

A plot of the WSELs and the flow at the Monatiquot River USGS gage over the duration of the study is displayed in Figure 9.2-3. For the flow conditions during which the WLL were operating, the WLL data shows that there is a hydraulic gradient of at least one foot from the Armstrong Dam to the WLL placed right upstream of the Jefferson Street Bridge. This indicates that the upstream extent of the impoundment ends before the Jefferson Street Bridge under the flow conditions observed; this is further supported by the hydraulic modeling results as described later.

Figure 9.2-3 shows a decline in the WSEL at the Farm River from about September 13, 2015 to September 28, 2015, but nothing similar at the other WLLs during this period. It appears that water is being pulled from Farm River during this period (and again from October 10, 2015 to October 29, 2015). *MarineFisheries* confirmed with the Town that they were diverting water from the Farm River to Richardi Reservoir during this period. One possible explanation is that water may also be pumped from an underground well or the Farm and Cochato Rivers, and under dry conditions, the pumping and diversion activity are potentially causing a flow reversal. However, the evidence of this flow reversal may

[30] A HOBO U20-001-04 was also used in the Farm River.

be due to WLL or survey instrumentation error. More information is needed on downstream migration flows.

MassGIS Datalayers

To supplement upland topography for the hydraulic model development, Light Detection and Ranging (LiDAR) elevation data available from MassGIS was used. Upland topography is needed to simulate high flow events, such as the 100-year flood, as the transects in the model need to contain all of the flow. LiDAR data for this region were collected between November 2013 and April 2014 and have an average point spacing of about 2.3 feet and a fundamental vertical accuracy of about 0.33 feet.

The distance between transects, a hydraulic model input, was determined using the Measure Tool in ArcGIS and MassGIS Orthoimagery. Orthoimages for this region were collected in April, 2013 and have an accuracy of about 5 feet.

2009 Feasibility Study and FEMA Model Data

The transects downstream of Armstrong Dam developed for the 2009 FS (GSE, 2009) hydraulic model were utilized to supplement the newly acquired transect data. The model was in NGVD29[31] and a datum shift of -0.79 ft was applied to all of the model geometry, using the built-in Vertical Datum Adjustment Tool in HEC-RAS, to convert the data to NAVD88. The hydraulic model for the 2009 FS used some transect data from the FEMA FIS (2000). The transect data for that model is from low-resolution 1976 survey data (e.g. one of the transects is comprised of only ten survey points over the approximately one-thousand-foot-wide transect with an assumed (non-surveyed) trapezoidal channel). The stationing at some of the 2009 model transects were updated based on measurements obtained using ArcGIS.

Bridge Plans

The geometry of the bridges upstream of the Armstrong Dam were updated for this FS based on plans obtained from MassDOT, MBTA, field survey and measurement. It was noted in the 2009 FS that more up-to-date bridge geometry data were needed as the FEMA hydraulic model did not reflect the current bridge geometry. All bridge plans are available in Appendix D.

9.3 Hydraulic Modeling

The hydraulic model extended from the stone arch MBTA Railroad Bridge to just downstream of the confluence of the Farm and Cochato Rivers. It was calibrated to dam-in conditions and the known geometry of the dam and bridges. The operating conditions at the Armstrong Dam, specifically, the number of low-level outlet stoplogs installed, were used to represent existing dam-in conditions. Note that between the 2009 FS and this FS, it appears that some of the low level outlet stoplogs may have been removed as the overall WSEL at the low-level outlet crest currently is lower than it was in 2009. The conditions simulated in the current hydraulic model reflect dam-in conditions, no stop logs installed in the bays, and the auxiliary low-level outlet stoplogs at elevation (El.) 93.1 feet. Again, this differs from the conditions during the 2009 FS, for which stop logs were installed.

Ames Pond Dam was simulated as being in place for both the dam-in and dam-out models. A cursory hydraulic analysis was conducted on the impacts of removing the Ames Pond Dam on the hydraulics in

[31] The 2009 Feasibility Study Report incorrectly labeled the datum NAVD88.

the Armstrong Dam impoundment if the Armstrong Dam were removed. It showed that removing the Ames Pond Dam does not impact the hydraulics in the Armstrong Dam impoundment. However, should removal of the Ames Pond Dam proceed, a more detailed analysis is recommended.

Calibration Data

The mean flow from the Monatiquot River USGS gage from 9 pm to midnight on November 24, 2015 was used as calibration data. This period was selected since flows and water levels were relatively stable (averaging the data over a three-hour period further smooths out minor fluctuations) and flows were relatively high for the period of time during which the water level loggers were installed. The flow at the Monatiquot USGS gage was prorated by the ratio of drainage areas to represent inflow the Armstrong Dam of 21.2 cfs. To put this flow into perspective, it is equaled or exceeded approximately 68.9 % of the time, when compared to the prorated flows on the Neponset River.

Boundary Conditions

For the downstream boundary condition, critical depth was chosen since the hydraulic depth switches from subcritical to supercritical at Rock Falls. For an upstream boundary condition, the normal depth method was chosen, which assumes that flow is steady and uniform and the slope of the water surface equals that of the channel.

Model Calibration

Once the model was set up, an initial analysis was conducted to calibrate the model by comparing model results to the calibration (or observed) data set. The observed data included the flow as measured at the Monatiquot USGS Gage and the WSEL as measured with the WLLs. Manning's roughness coefficients (n), contraction/expansion coefficients, and ineffective flow areas were adjusted appropriately to model the observed conditions.

Roughness coefficients, which account for friction in the channel, were adjusted to match the model predicted and observed WSEL at the WLLs. Increasing roughness coefficients slows water and makes it deeper, decreasing roughness coefficients speeds up the water and makes it more shallow. Roughness coefficients at transects below the Armstrong Dam were set to 0.07 in the channel and 0.08 overbank, based on the steep gradient and large boulders in this more ravine-like area. The roughness coefficients above the dam were 0.045 in the main channel and 0.06 in the overbank area to represent a more riverine reach (Chow, 1959).

The weir coefficient, which can range from 2.6 to 3.2 depending on the configuration of the dam, was back calculated for the Armstrong Dam's low level outlet. It was back calculated by finding the time at which the WSEL at the dam was at its crest elevation, 94.1 ft. The flow at the Monatiquot USGS gage was adjusted to the Armstrong Dam by prorating the flow by the ratio of drainage areas. The head atop the low level outlet was 1 foot. Based on the known head and flow, the weir coefficient was calculated to be 2.93. The weir coefficients at the other Armstrong Dam gates were set to 3.2 as 2.93 seems low for these gates due to their increased ability to pass flow due to a longer width and a rounded weir crest shape on both the upstream and downstream sides. At Ames Pond Dam the weir coefficient was set to 3.0.

Calibration Results

The hydraulic model simulated dam in conditions under a flow steady flow of 21.2 cfs. The model produced WSELs were compared to the observed WSELs measured at the four WLLs above Armstrong Dam. While HEC-RAS transects were not located at these exact locations of the WLLs, the WSELs could be interpolated between transects. Table 9.3-1 summarizes the comparison of modeled and observed WSELs. The results show a difference of less than 0.3 ft, which are within the calibration tolerances.

Table 9.3-1: Comparison of Modeled vs. Observed Water Surface Elevations

Water Level Logger	Nearby Transect	Observed WSEL (ft, NAVD 88)	Modeled WSEL (ft, NAVD 88)	Difference (ft)
Upstream of Armstrong Dam	19819	94.22	94.00	-0.22
Upstream of Jefferson Street Bridge	22405	95.43	95.16	-0.26
Downstream of Confluence	24540	96.33	96.17	-0.16

Dam-Out Conditions

Dam-out conditions were simulated in the HEC-RAS model to allow for comparison between depth and velocities in the Project area. The dam-out alternative included the following:

- removal of the spillway and low level outlet,
- Removal of the existing vertical column extending from the spillway crest elevation to the low chord of the building;
- The building footprint and abutments would remain. It was assumed that vertical structural support columns would extend from the low chord of the building to the underlying bedrock based on the existing pattern and spacing of columns supporting the remainder of the building. Note that no structural evaluation was conducted as part of this study—the vertical supports were added only to simulate hydraulics.
- An existing access route across the river[32], would remain over the dam in the same place of the existing building.

To simulate the dam-out conditions and preserve an access route, the primary spillway was replaced in the model with a transect representing estimated natural conditions directly beneath the dam. As noted above, the existing piers between the spillway bays were replaced by projecting the structural columns supporting the building about 27 feet back from crest of the dam forward to the crest. To maintain the 24-foot-on-center spacing between columns, a column was added on river left.

The historic drawings of the Armstrong Cork Building did not depict a detailed cross-section of the dam and underlying bedrock[33]. Thus, the transect surveyed immediately upstream of Armstrong Dam, representing the bottom of sediment, was used to replace the primary spillway in the model. The transect used to replace the dam is shown in Figure 9.3-1.

[32] Currently Messina Enterprises relies on an easement from the MBTA to access the west side of the Armstrong Cork Building.

[33] It is assumed that the Armstrong Dam is founded on bedrock as downstream of the dam bedrock is visible.

To simulate dam-out conditions, the results of the sediment depth mapping were used to remove potentially mobile sediment within the expected future channel throughout the length of the impoundment. As described earlier a transect representing free-flowing conditions was "formed" to "represent" the channel morphology through the pond. The thalweg of the representative transect was "projected" onto the thalweg elevation estimated by the sediment probing at each transect in the impoundment to form a proposed channel.

9.4 Comparison of Upstream Hydraulics with and without the Armstrong Dam

Once the model was calibrated, the flows in Table 8.2-1 and Table 8.2-2 were simulated under dam-in and dam-out conditions.[34] The following section discusses the results of the modeling runs to simulate WSELs and velocities under dam-in and dam-out conditions. WSP plots under dam-in and dam-out conditions under flows of 100-year (2,100 cfs), 50-year (1,700 cfs), May (53 cfs), November (45 cfs), September (21 cfs), and Maximum Impoundment Flow (586 cfs, see below) are shown in Figures 9.4-1 through 9.4-6, respectively. Table 9.4-1 shows the average channel velocity and maximum channel water depth at all locations for dam-in and dam-out conditions for fish flow.

Flood Flows (100-yr, 50-yr, Extent of Armstrong Dam Impoundment)

To check the upstream extent of the pond's influence, the dam was removed from the model and the other transects were modified such that the mobile sediment was removed. The 100-year flood flow was simulated to depict the extent of the dam's influence. The extent of the impoundment is typically defined as the transect at which the WSEL is the same under dam-in and dam-out conditions for the 100-year flood flow. The point at which this occurs is at the upstream face of the Jefferson Street Bridge. A WSP comparing WSELs under dam-in and dam-out conditions for the 100-year flood flow, 50-year flood flow and the maximum impoundment flow are shown in Figures 9.4-1 to 9.4-3.

The 100-year profile shows that for at least the Plain Street Bridge it is undersized to pass the 100-yr and 50-yr flood flows (under both dam-in and dam-out conditions) at it becomes overtopped. Because of undersized hydraulic capacity at the Plain Street Bridge it creates a backwater that also overtops the Washington and Jefferson Street Bridges. If the Plain Street Bridge was not hydraulically undersized it is unclear if the Washington and Jefferson Street Bridges would be overtopped. All three bridges are overtopped under the 50—year and 100-year flood.

Fish Passage Flows (May-53 cfs, September- 21 cfs and November- 45 cfs)

In order for resident and migratory fish to readily pass to and from their spawning habitat, certain physiological and behavioral needs and physical river conditions must be met, including seasonal flow magnitudes, depths, and velocities. These characteristics vary among the target species. Water depth in the river channel and through obstacles such as road crossings must be sufficient to accommodate the physical dimensions of fish navigating upstream and downstream. Additionally, migratory riverine species often encounter zones of high velocity flow that may impede their migrations, such as where flow is restricted through a road crossing or narrow channel section or a natural falls occurs.

Shown in Figures 9.4-4, 9.4-5 and 9.4-6 are the WSP plots under dam-in and dam-out conditions for May (53 cfs), November (45 cfs) and September (21 cfs), respectively. Based on the WSP plot for all three

[34] As part of this FS, Ames Pond Dam was not resurveyed or its removal evaluated (i.e. Ames Pond Dam was included in the Dam-in and Dam-out model).

flows, under dam-out conditions, two known vertical barriers to fish passage are present. As noted earlier, downstream of the Armstrong Dam there is currently one vertical barriers to fish passage—the Rock Falls. Measures would be needed to permit river herring to move beyond these barriers. In addition, a potential vertical barrier exists directly beneath the Armstrong Dam. Typically, dams are constructed atop falls and bedrock is visually present immediately below the dam. Sediment probing was also conducted to refusal immediately upstream of the dam. It is possible that a falls does exist at the site, with perhaps an even greater drop pending the profile immediately beneath the dam, which could preclude upstream passage of river herring unless measures were taken during a potential removal to create a zone passage route. Those measures could include removing the bedrock or determining if a passage route around the bedrock was possible.

Fish Passage Thresholds Check

The model was used to evaluate whether stream hydraulics (depth/velocity) would prevent fish passage. Specifically, the model was evaluated to determine if velocities were too high resulting in a velocity barrier to fish passage. In addition, river herring cannot leap or jump thus, the model was also evaluated to determine if there were any vertical barriers to fish passage.

Relative to velocity barriers, Table 4.2-1 listed the cruising, sustained and burst speed for river herring. The sustained and burst speeds were listed as 3-5 ft/sec and 5-7 ft/sec, respectively. In addition, the estimated minimum depth needed for river herring passage ranged from 6-8 inches (0.5-0.67 ft). With this information as background, Table 9.4-1 provides the average channel velocity and maximum channel depth[35] at each transect under dam-out conditions based on a low flow (mean September flow of 21 cfs) and high spring migratory fish passage flow (mean May flow of 53 cfs).

In comparing the velocity findings under the 53 cfs flow against the burst speed (5-7 ft/sec) there was only one transect (HEC-RAS station 19200, at Rock Falls) that was in this range (5.2 ft/sec). A high velocity at this location is expected as the water velocity increases as it moves over the falls. However, this transect was located below the Ames Pond Dam and outside the influence of the potential dam removal Project. Note that the same high velocity (5.2 ft/sec) occurs under dam-in conditions as expected (see Table 9.4-1). The velocities through the "new" channel created through the impoundment, were all very low. As a reminder the velocities presented herein are based on the average transect velocity. There are locations along the transect where velocities will be higher or lower than the mean transect velocity, meaning that upstream migrating river herring can likely navigate the channel.

In comparing the depth findings under the 21 cfs flow against the minimum depth requirements (6-8 inches or 0.5-0.67 ft) there is one transect (HEC-RAS station 20975) located between the Plain Street and upstream railroad bridge where the maximum channel depth is less than 0.5 feet (0.36 feet).

Rock Falls presents a depth, velocity and vertical barrier downstream of Armstrong Dam. Ames Pond Dam provides a velocity barrier under high flows. It is the intent of the Project Partners to remove this dam as well; however, the Ames Pond Dam is likely founded on bedrock and the profile of the top of the bedrock is unknown.

[35] The maximum depth is determined from the WSEL and the thalweg (or lowest surveyed point along the transect).

9.5 Bridge Scour Analysis

A potential concern with removing the Armstrong Dam is the resulting increase in velocities around the four bridge piers (if present) and abutments at the Plain, Railroad, Washington and Jefferson Street Bridges (under flows where the Plain Street Bridge is not overtopped). Increased velocities could result in scour along the pier or abutment, which could potentially jeopardize the structural integrity of the bridge. A HEC-18 scour analysis was not conducted as part of this FS. Instead, visual observations and probing of sediment near the bridge abutments and/or piers was conducted. In addition, MassDOT and others were contacted for any bridge reports and specifically any previously conducted bridge scour analysis. Table 9.5-1 shows the average channel velocity at all locations for dam-in and dam-out conditions for the maximum flow that does not overtop the bridges (586 cfs).

Downstream Railroad Bridge

The WSELs and velocities downstream of the Armstrong Dam will not be affected by its removal. This includes Ames Pond Dam and the downstream railroad bridge.

Plain Street Bridge

If the Armstrong Dam were removed, water velocities through the bridges will increase and WSEL will decrease under the same flow. According to an underwater inspection report (MassDOT, Appendix D), and corroborated in the field for this FS, no footings are currently exposed. Sediment probing revealed that the sediment around the Plain Street Bridge is a fairly deep and well-compacted sand, making it difficult to get a sense of the sediment closer to the foundation. The sand is built up approximately two to three feet. The bridge plans (MassDOT, Appendix D) show no pile supports, so a scour analysis is recommended. Boring logs from the Plain Street Bridge plans are available on Sheet 2 of 2 for Bridge No. B-21-14 in Appendix D. These indicate the bridge is founded upon mostly firm fine blue sand and gravel and some clay. Water velocities will increase by approximately 2.1 ft/s at the Plain Street Bridge under the maximum impoundment flow (586 cfs).

According to Massachusetts Department of Transportation (MassDOT) inspection and underwater inspection reports, there is some undermining at the left abutment, but no scour is evident. Photos of the Plain Street Bridge area available in Appendix A (Photos 14 –15).

Upstream Railroad Bridge

Scour may be of concern based on visual inspection at the upstream railroad bridge. The piers of the upstream railroad bridge currently have minor scour holes along their faces. There is another area in the channel where finer sediments have been washed out leaving a larger scour hole. Sediment probing revealed that the substrate around the bridge is mainly small boulder and cobble with the river left side having a buildup of sand.

Inspection and underwater inspection reports prepared by Green International Affiliates, Inc. and Child Engineering Corporation are available in Appendix D. Photos of the upstream railroad bridge area available in Appendix A (Photos 17 –18). Water velocities will increase by approximately 0.2 ft/s at the Upstream Railroad Bridge under the maximum impoundment flow (586 cfs).

Washington/Hancock Street Bridge

The Washington Street Bridge did not show any signs of scour during field investigation. The bridge was rebuilt in 1979 and the waterway appears to be paved with cobble overlain with sand. According to plans (MassDOT, Appendix D), it is not pile supported. Boring logs for the Washington Street Bridge are available on Sheet 2 of 8 for Bridge No. B-12-16 in Appendix D. These indicate that the Washington Street bridge is founded upon a moist cemented gray inorganic silt with some fine sand and gravel. During sediment probing there was no penetration of the underlying substrate with the steel rod. Bridge drawings, inspection and underwater inspection reports of the Washington Street bridge are located in Appendix D. Also, see Photo 18 in Appendix A). Water velocities will increase by approximately 0.3 ft/s at the Washington Street Bridge under the maximum impoundment flow (586 cfs).

Jefferson Street Bridge

The Jefferson Street Bridge is not pile supported. The banks have been heavily armored with boulder sized riprap, and the sediment is well compacted sand and cobble. There was little to no penetration during sediment probing. Bridge drawings, inspection and underwater inspection reports of the Jefferson Street Bridge are located in Appendix D. A photo of the Jefferson Street Bridge is available in Appendix A (Photo 20). Water velocities at the Jefferson Street Bridge should have little to no increase at the Jefferson Street Bridge.

9.6 Utility Scour Analysis

Another concern with the Armstrong Dam removal is the potential for headcuts (unraveling and subsequent lowering of the channel bed from the dam progressing upstream) or scour that could potentially expose utility lines crossing beneath the channel bed. There are both water main and sewer mains crossing the Monatiquot River upstream of Armstrong Dam. As noted earlier, the town does not have any profile drawings of the water lines thus it is unclear how deep they are relative to the channel bed. In addition, the town has some profile drawings of the sewer lines, but not all that cross the Monatiquot in the potentially impacted reach.

Water Main Crossing

A 12-inch-diameter cast iron water main crosses the Monatiquot River near the downstream side of the Plain Street Bridge. No profile drawing of this water line is available; however, water velocities will increase by approximately 2.06 ft/s at this location under the maximum impoundment flow (586 cfs), thus it is recommended that the line be located to determine if it could be impacted if the dam is removed.

A 12-inch-diameter cast iron water main crosses the Monatiquot River immediately upstream of the Washington Street Bridge; however, it is affixed to the upstream face of the Washington Street Bridge, thus it would not be impacted by dam removal.

A 10-inch-diameter ductile iron water main crosses the Monatiquot River at the Jefferson Street Bridge. Again, no profile drawing of the water line is available. It appears that this water line would not be impacted by removal of the Armstrong Dam as velocities will only increase by 0.01 ft/s this far upstream.

Sewer Main Crossing

There are three sewer mains crossing the Monatiquot River upstream of the Armstrong Dam. A 24-inch-by-38-inch reinforced concrete elliptical sewer main crosses the Monatiquot River at a point approximately 25 feet upstream of the Plain Street Bridge. According to Sewer Assessment Plan 6638 (Appendix E), the sewer main is encased in concrete 6 inches all around, and the top of the concrete is buried about 6 inches below the channel bed. Water velocities will increase by approximately 0.25 ft/s under the maximum impoundment flow at this location. It would be unlikely that this line would become uncovered due to such a small increase in velocity.

A 12-inch-diameter cast iron sewer main pipe crosses the Monatiquot River immediately upstream of the Washington/Hancock Street Bridge. The sewer main is encased in concrete with 2.5 in. of concrete above the sewer main, 12 inches on the sides and 12 inches of 1-inch crushed stone below. According to the plan, the sewer main was buried about one foot below the channel bottom. The Town (Personal Communication, 2/25/16) believes the sewer line is no longer active, but is not entirely sure. Water velocities will increase by approximately 0.22 ft/s under the maximum impoundment flow at this location. It would be unlikely that this line would become uncovered due to such a small increase in velocity.

An 8-inch-diameter cast iron sewer pipe crosses the Monatiquot River less than 100 ft downstream of the Jefferson Street Bridge. The 1956 plan (Appendix E) shows the pipe is encased in concrete with the top elevation of the concrete just below the channel bottom. Water velocities are virtually unaffected (~0.02 ft/s) this far upstream, so this pipe would not become uncovered due to the potential removal of the dam.

Stormwater Outfall

If the Armstrong Dam were removed, the water levels downstream of Jefferson Street Bridge would be lowered under non-flood flow events. The stormwater discharges that currently empty directly into the water may not travel further along the streambank before discharging into the river if the Armstrong Dam were removed. The stormwater discharge could erode the newly exposed bank if not protected.

There are two culverts on the east side of the impoundment (Figure 3.3-1), which if the dam were removed, would become perched. The upstream culvert, by the entrance causeway to the gym, would most likely need to be extended and riprap armoring would be needed from the discharge location to the new river channel to prevent erosion. With the dam removed, the WSEL would drop approximately 5.5 feet. Scour at the more downstream culvert may be prevented by armoring alone, since the main channel will be relatively close to the current outfall location; again, the WSEL would be reduced on the order of 5.5 feet.

The two culverts on river right and upstream of the Plain Street bridge (Figure 3.3-1) may need to be armored with more riprap as the WSEL will decrease on the magnitude of 0.4 feet at these locations. The remaining culverts upstream of the Washington Street Bridge are built into the abutments and either inactive or sufficiently armored. There are two culverts in close proximity to the Jefferson Street Bridge. WSELs are virtually unaffected by the proposed removal of the Armstrong Dam at this location.

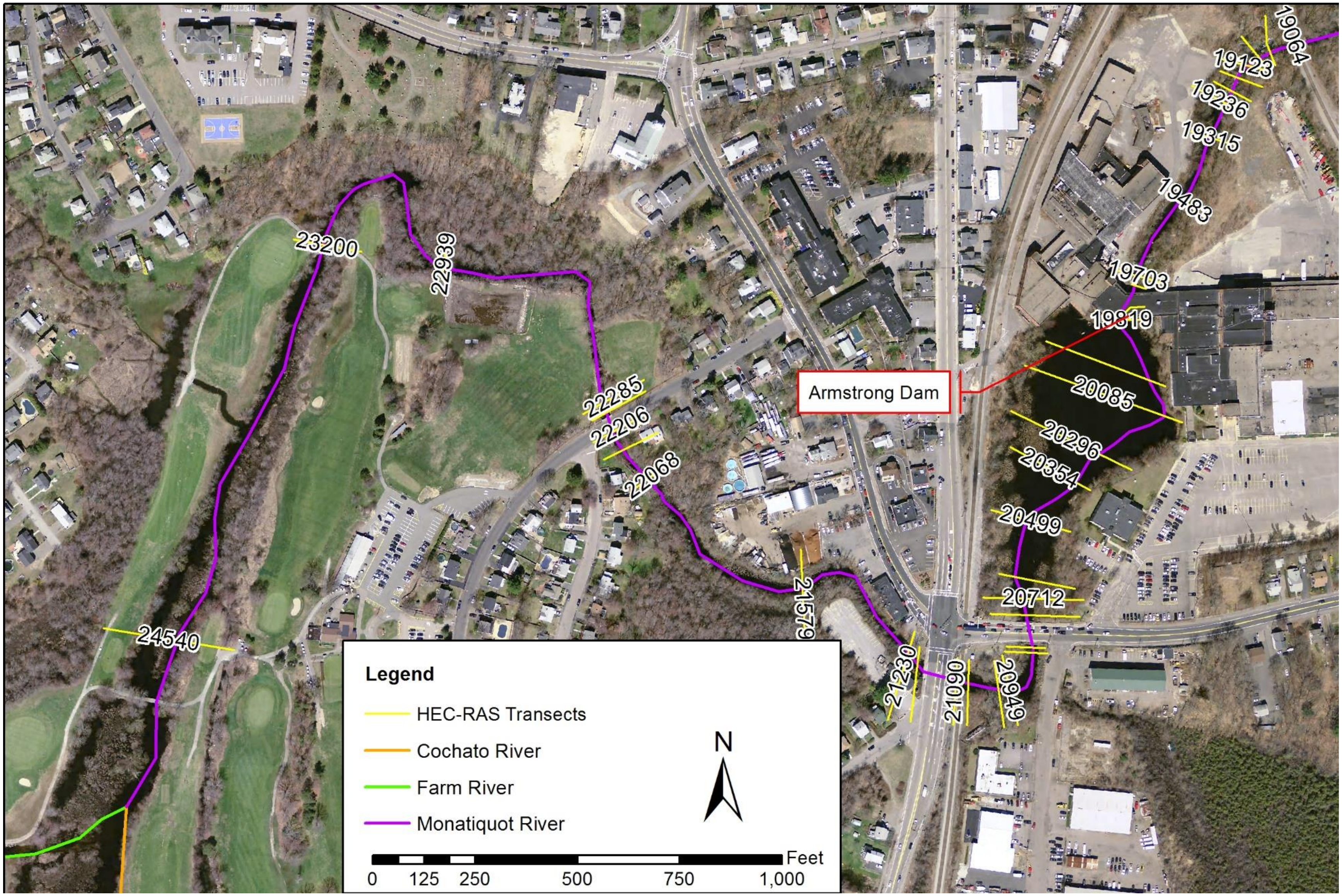

Figure 9.2-1: Hydraulic Model Transects

Table 9.2-1: Summary of Bathymetry and Topography Sources Used in the Hydraulic Model

Type	Source	Date	Coverage	Utilized for
Topo Survey	Gomez and Sullivan Engineers	November 2015 – December 2015	Underneath Armstrong Cork Building – Braintree Golf Course	Channel cross-sections below dam; Channel cross-sections and upland topography upstream of dam; structures
Sediment Depth Mapping	Gomez and Sullivan Engineers	December 2015	7 transects from dam to approximately 850 ft upstream	Impoundment channel cross-sections; estimation of proposed dam removal channel geometry
HEC-RAS Model Input Data	Gomez and Sullivan Engineers	2008	Downstream of Armstrong Dam	Channel geometry and upland topography.
LiDAR	MassGIS	2014	Upstream of Armstrong Dam	Upland topography for sediment and topo survey cross-sections[36]
FIS HEC-2[37] Model Input Data	FEMA	1989	Entire model extent	Channel geometry and upland topography.

[36] Upland topography for existing model cross-sections were not updated with the new LiDAR data
[37] HEC-2 is a previous version of HEC-RAS.

Figure 9.2-2: Water Level Logger Locations

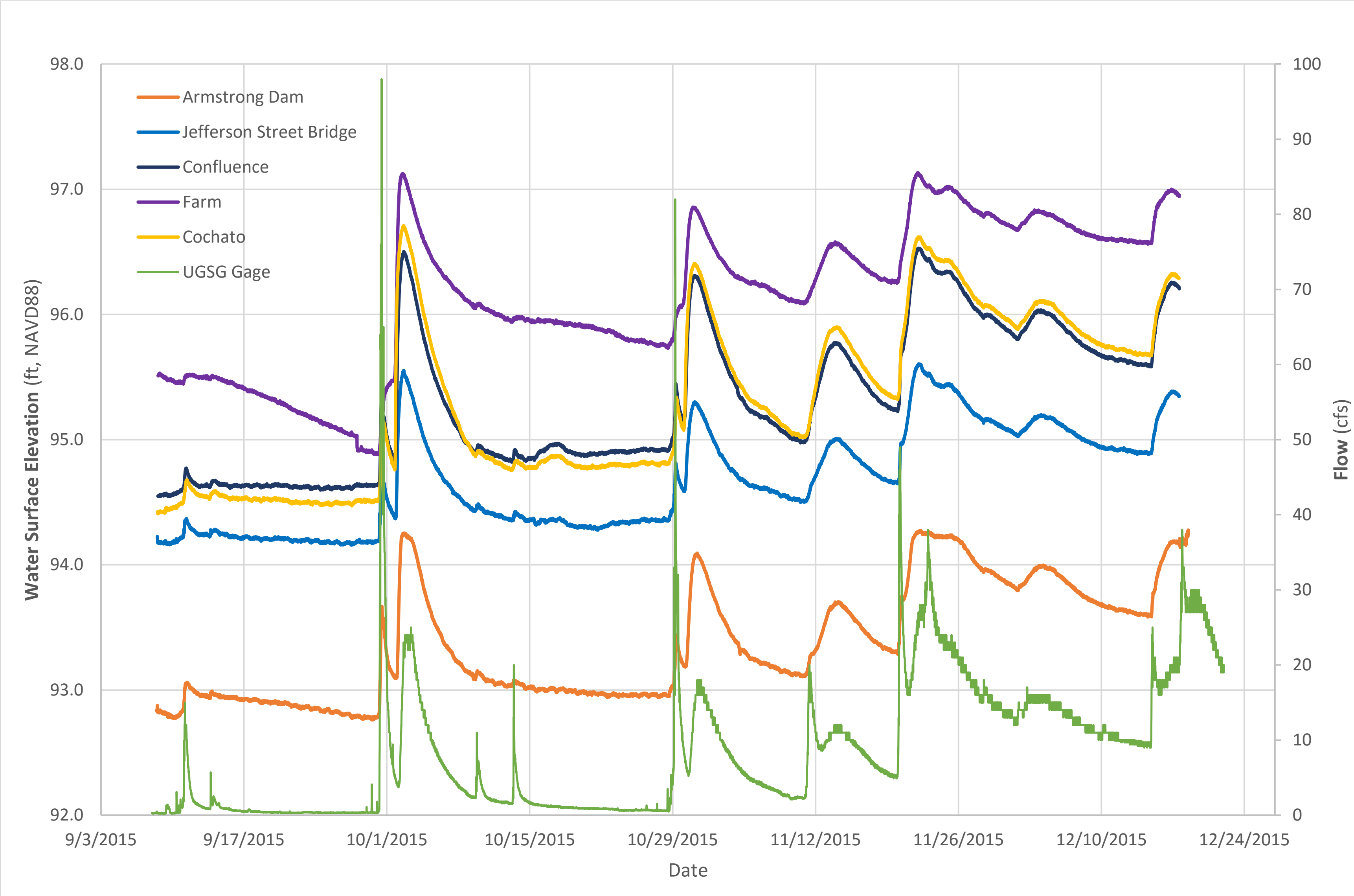

Figure 9.2-3: Water Level Logger Data

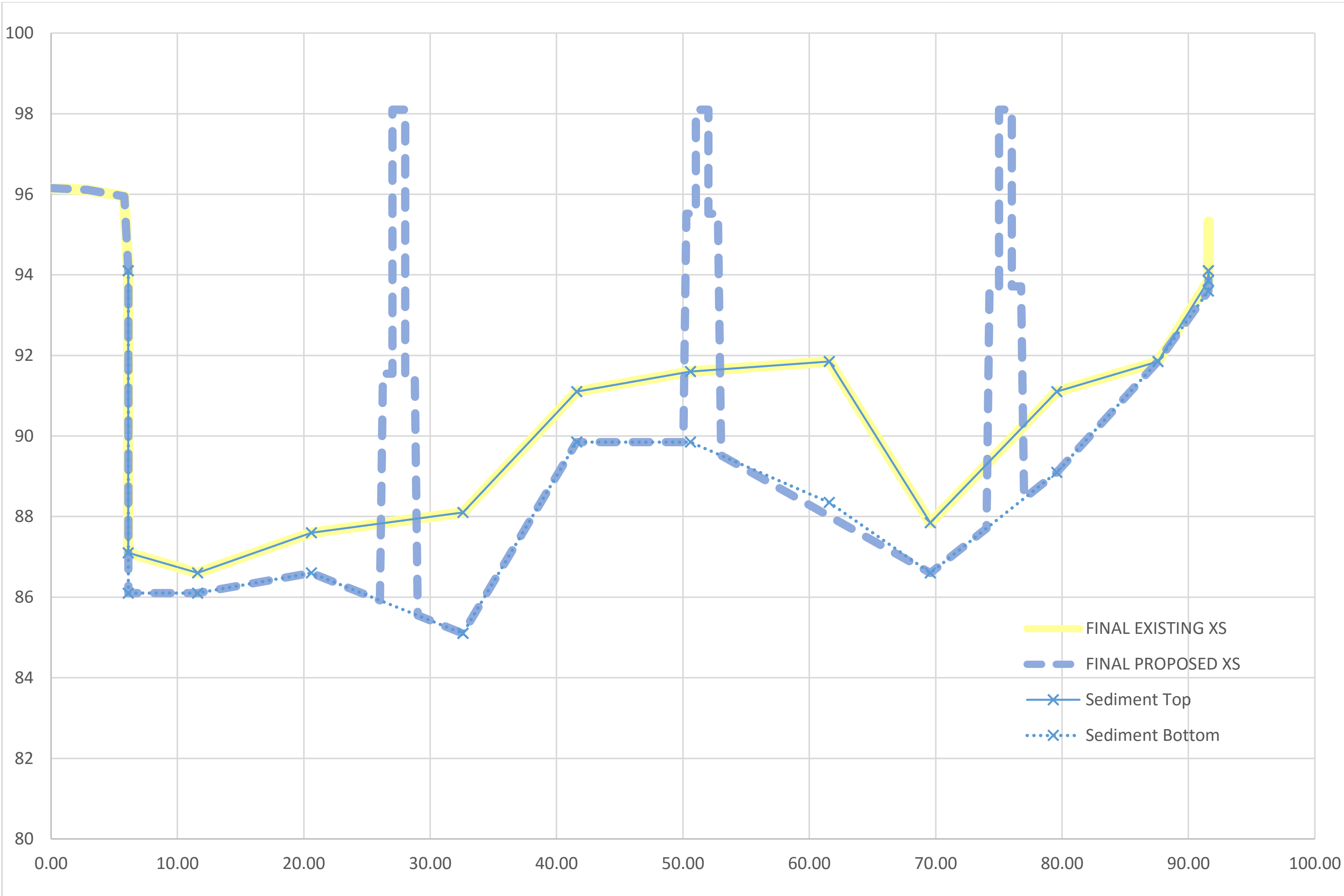

Figure 9.3-1: Proposed Cross Section Used Instead of Armstrong Dam (Looking Downstream)

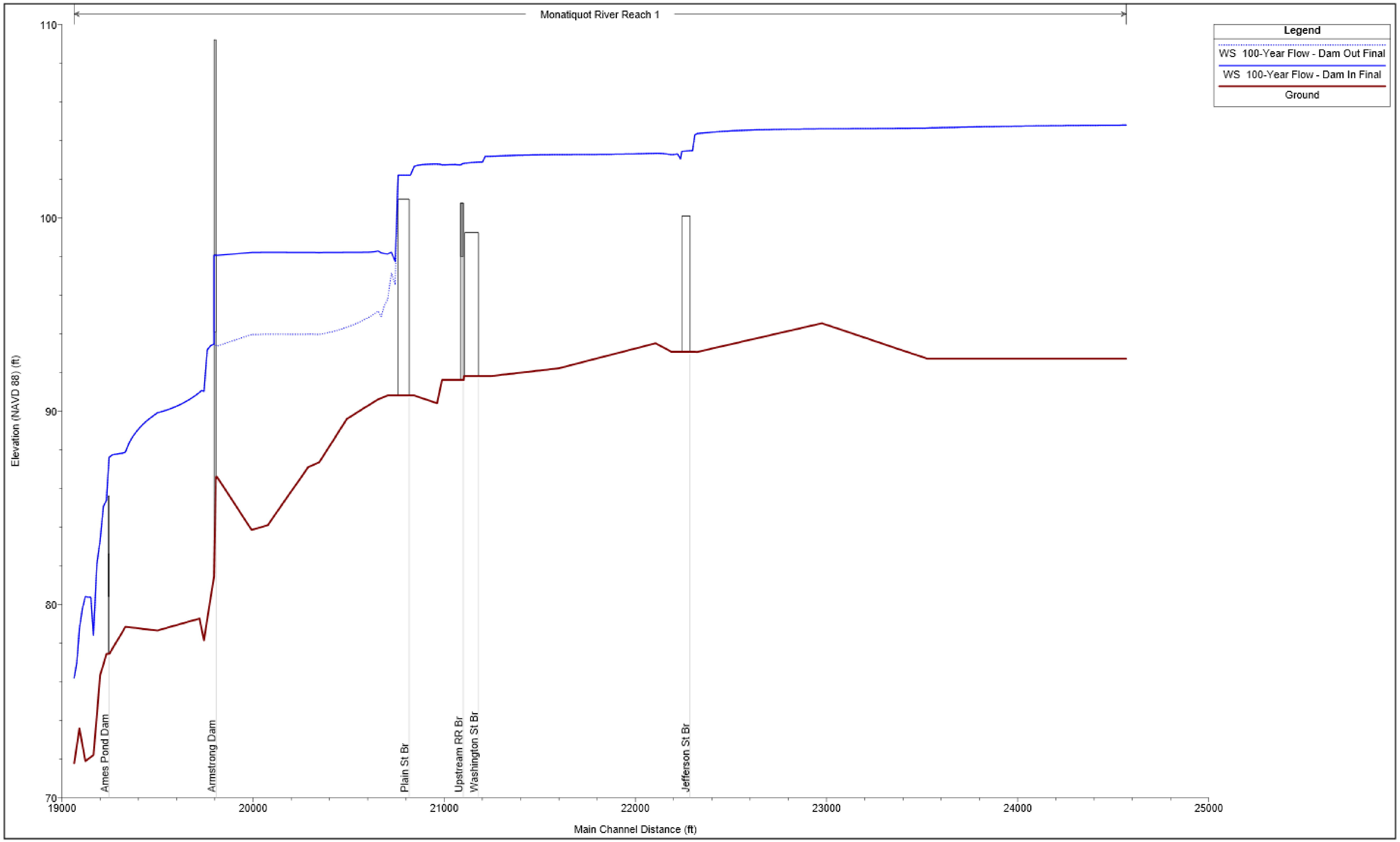

Figure 9.4-1: 100-Year Flood Flow Water Surface Profiles for Existing and Proposed Conditions

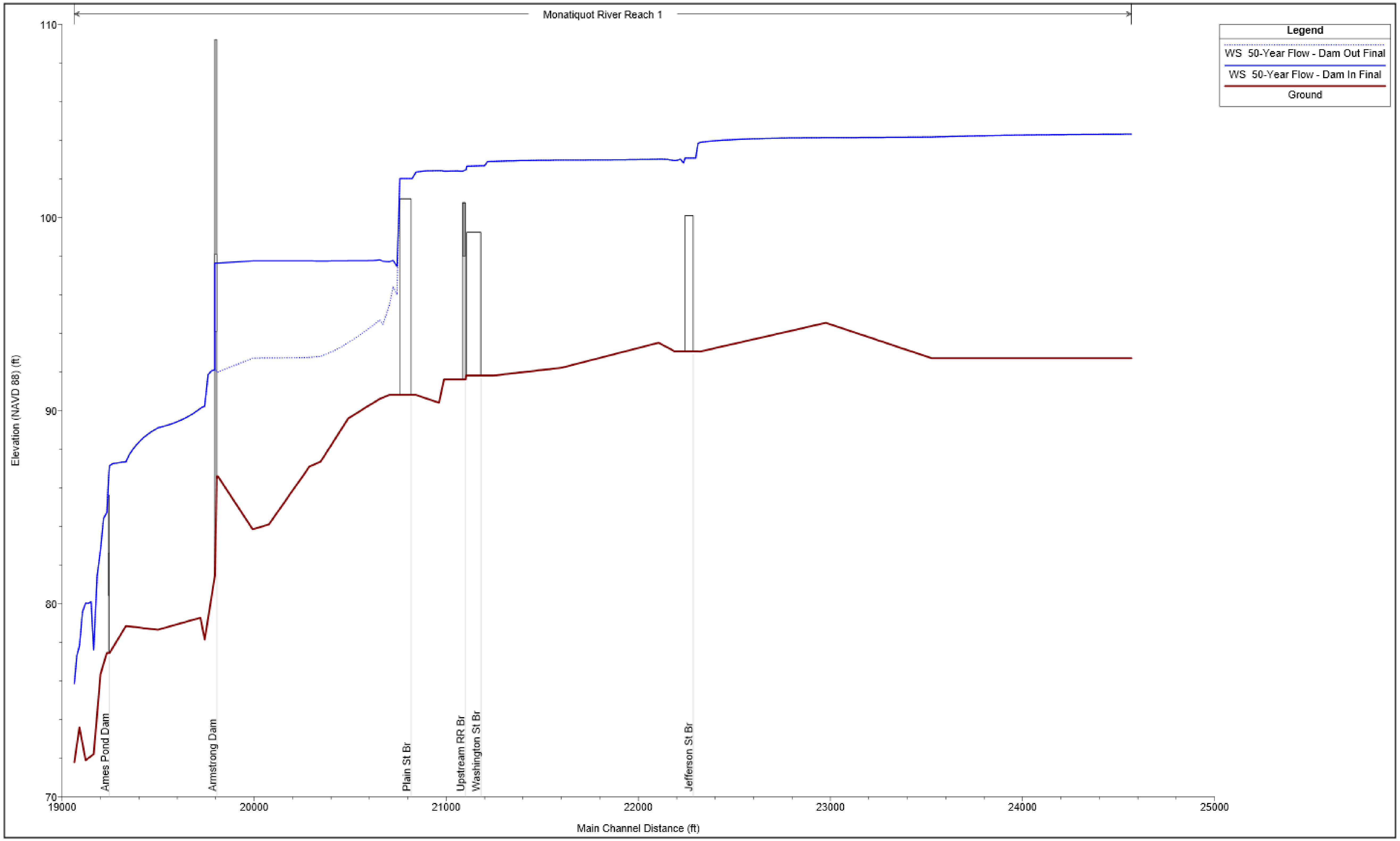

Figure 9.4-2: 50-Year Flood Flow Water Surface Profiles for Existing and Proposed Conditions

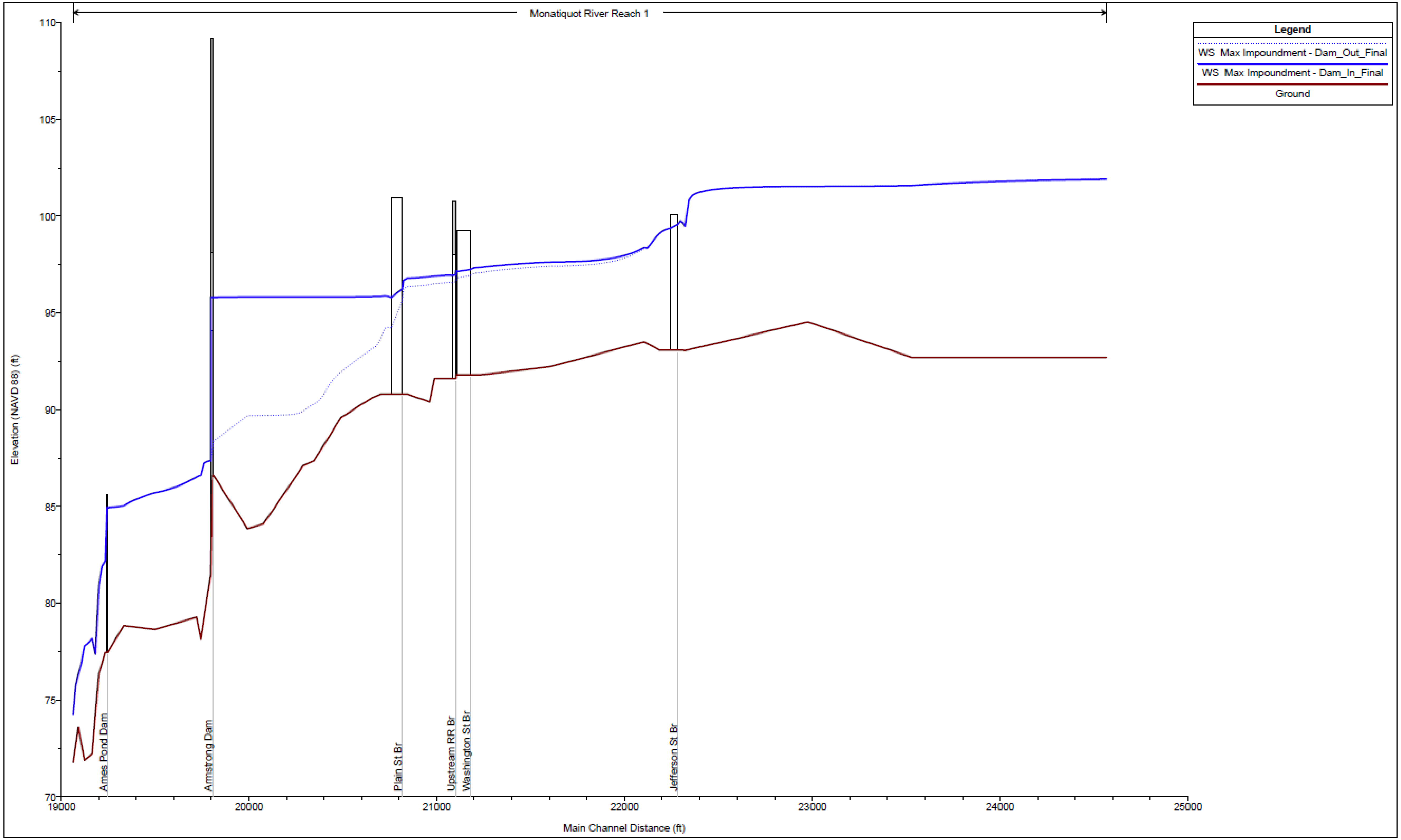

Figure 9.4-3: Maximum Impoundment Flow Water Surface Profiles for Existing and Proposed Conditions

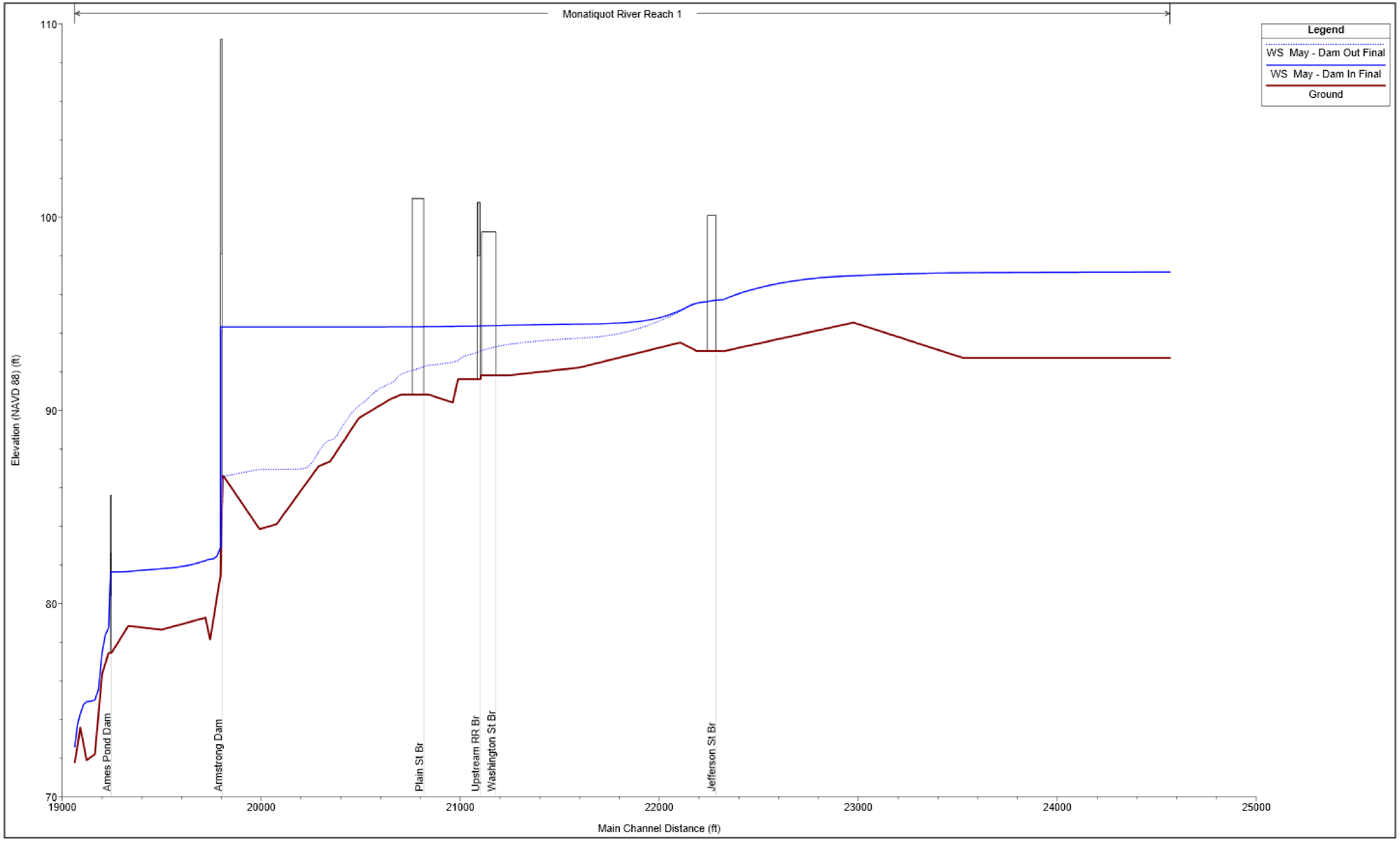

Figure 9.4-4: May Flow Water Surface Profiles for Existing and Proposed Conditions

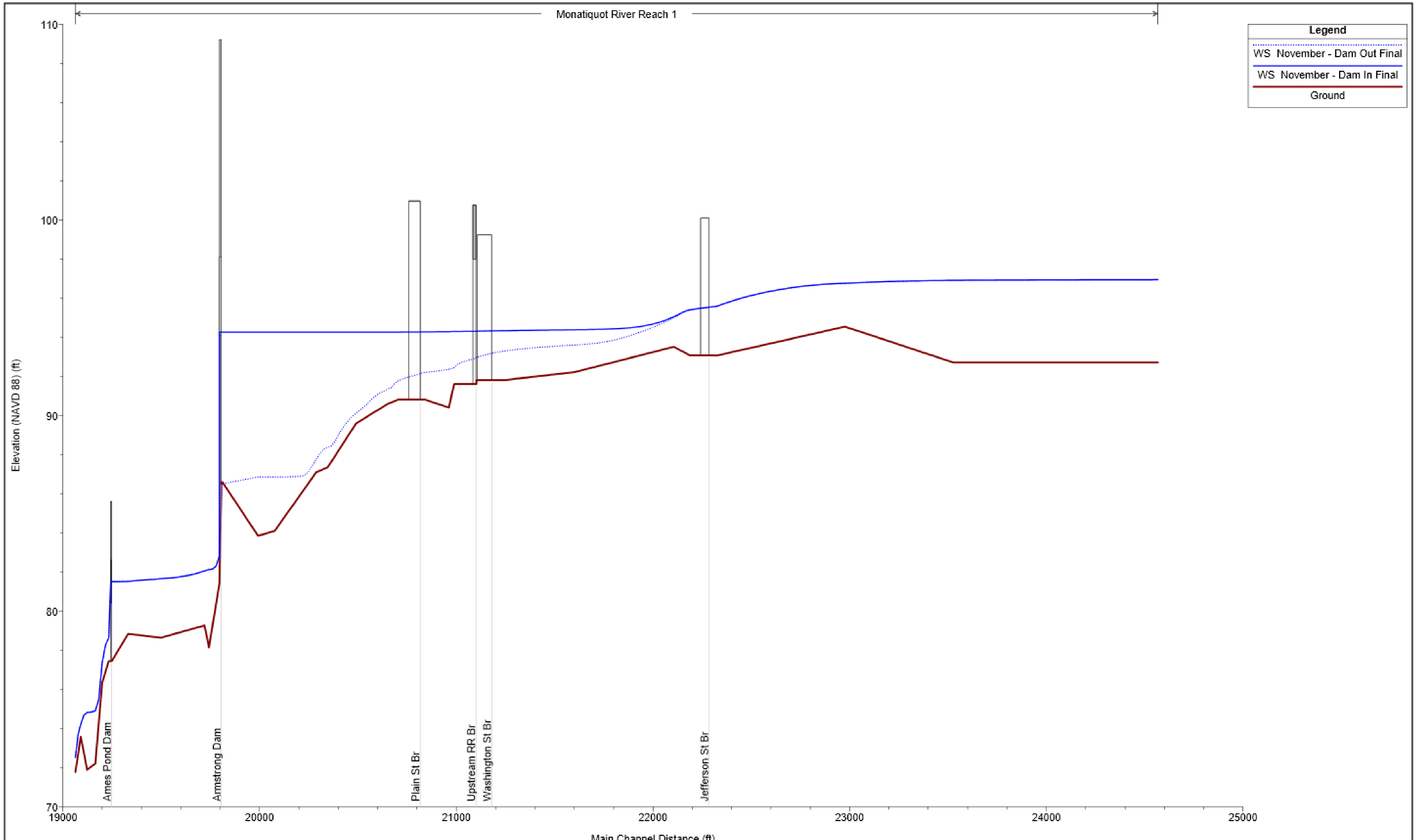

Figure 9.4-5: November Flow Water Surface Profiles for Existing and Proposed Conditions

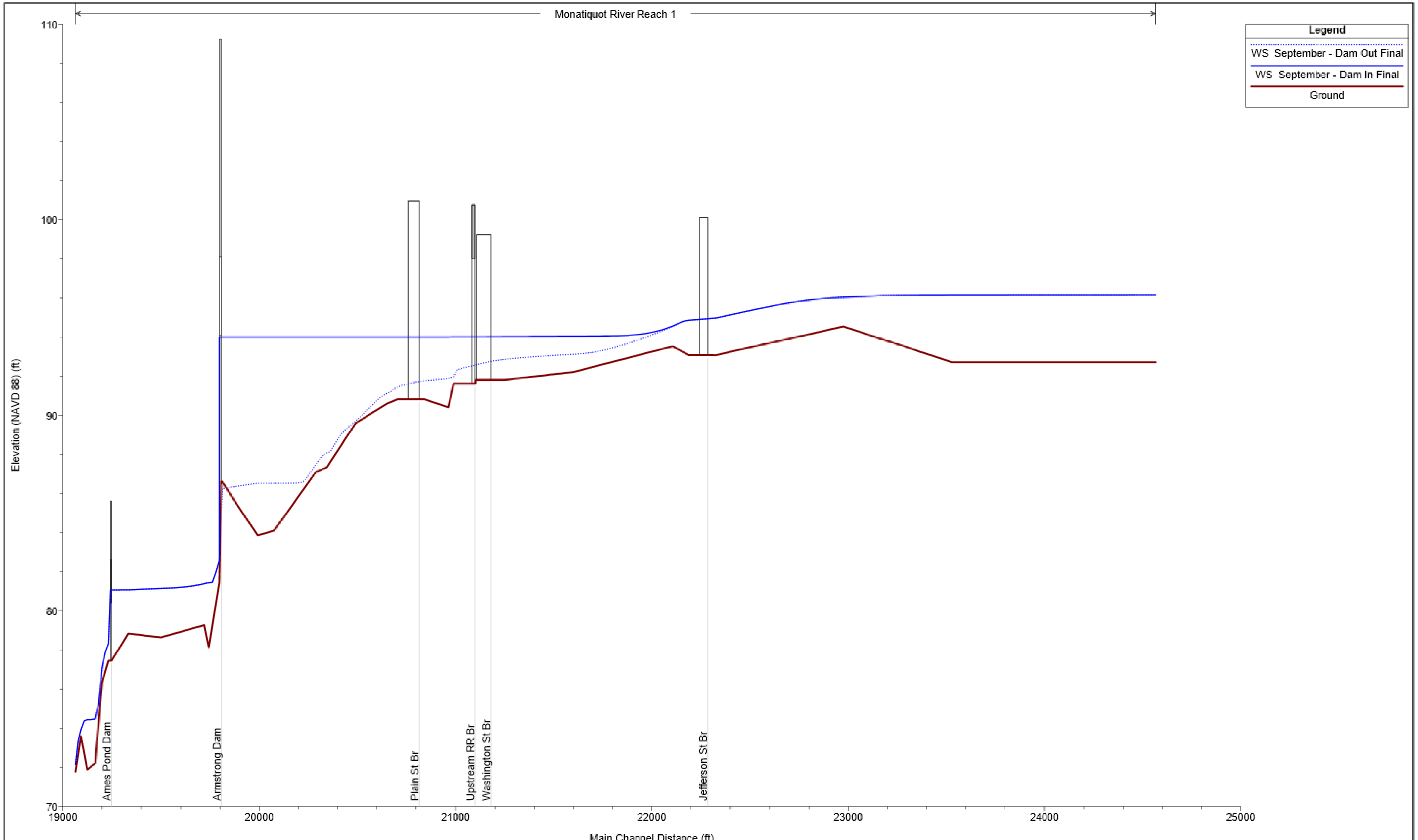

Figure 9.4-6: September Flow Water Surface Profiles for Existing and Proposed Conditions

Table 9.4-1: Channel Velocity and Depth at Low and High Fish Migration Flows – Dam-In and Dam-Out

Location	River Station (ft)	Average Channel Velocity (ft/s)				Max Channel Water Depth (ft)			
		Existing Low Flow (21 cfs)	Proposed Low Flow (21 cfs)	Existing High Flow (53 cfs)	Proposed High Flow (53 cfs)	Existing Low Flow (21 cfs)	Proposed Low Flow (21 cfs)	Existing High Flow (53 cfs)	Proposed High Flow (53 cfs)
	19064	2.12	2.12	3.57	3.57	0.81	0.81	1.26	1.26
	19091	1.16	1.16	2.27	2.27	0.94	0.94	1.29	1.29
	19123	0.60	0.60	1.18	1.18	2.55	2.55	3.03	3.03
	19165	0.96	0.96	1.59	1.59	2.26	2.26	2.80	2.80
	19200	3.85	3.85	5.21	5.21	0.72	0.72	1.11	1.11
	19236	2.59	2.59	3.28	3.28	0.88	0.88	1.28	1.28
Ames Pond Dam	19244								
	19251	0.29	0.29	0.57	0.57	3.62	3.62	4.18	4.18
	19315	0.66	0.66	1.23	1.23	2.24	2.24	2.82	2.82
	19483	0.57	0.57	1.02	1.02	2.50	2.50	3.16	3.16
	19703	1.24	1.24	1.82	1.82	2.12	2.12	2.95	2.95
	19726	0.83	0.83	1.46	1.46	3.29	3.29	4.15	4.15
	19804	0.70	0.70	1.15	1.15	1.12	1.12	1.46	1.46
Armstrong Dam	19814								
	19819	0.06	3.52	0.13	3.98	7.39	1.15	7.71	1.49
	20000	0.01	0.10	0.03	0.21	10.14	3.42	10.46	3.84
	20085	0.01	0.12	0.02	0.24	9.89	2.92	10.21	3.34
	20296	0.03	2.26	0.06	2.80	6.89	0.65	7.21	1.00
	20354	0.04	1.05	0.10	1.48	6.64	0.98	6.96	1.36
	20499	0.07	1.98	0.15	2.13	4.39	1.11	4.71	1.62
	20662	0.15	1.26	0.33	1.77	3.39	0.77	3.71	1.08
	20712	0.19	1.46	0.43	2.04	3.18	0.63	3.50	1.05
	20752	0.25	1.17	0.56	1.75	3.18	0.77	3.50	1.21
Plain St Bridge	20765								
	20831	0.21	0.93	0.47	1.35	3.19	0.93	3.52	1.48
	20851	0.19	0.91	0.42	1.32	3.19	0.95	3.52	1.51
	20949	0.34	1.33	0.73	1.99	3.60	1.50	3.94	2.09
	20975	0.26	3.10	0.54	2.39	2.39	0.36	2.74	0.96
	21059	0.26	1.06	0.54	1.53	2.39	0.88	2.75	1.35
Upstream RR Bridge	21069								
	21088	0.26	0.94	0.54	1.34	2.40	0.97	2.76	1.48
	21090	0.37	1.40	0.75	1.91	2.20	0.77	2.56	1.27
Washington St Bridge	21132								
	21186	0.37	1.05	0.74	1.51	2.20	0.98	2.58	1.53
	21230	0.36	0.98	0.74	1.42	2.21	1.03	2.59	1.60
	21579	0.29	0.68	0.57	0.93	1.83	0.90	2.25	1.52
	22068	1.88	1.94	2.69	2.79	1.06	1.03	1.66	1.61
	22150	1.15	1.16	1.86	1.88	1.78	1.77	2.47	2.45
	22198	1.11	1.12	1.78	1.80	1.82	1.82	2.55	2.54
Jefferson Street Bridge	22206								
	22261	1.07	1.08	1.69	1.71	1.87	1.87	2.64	2.63
	22285	1.74	1.75	2.88	2.90	1.90	1.90	2.67	2.66
	22939	0.71	0.71	0.90	0.90	1.49	1.49	2.43	2.42
	23200	0.32	0.32	0.60	0.60	3.43	3.43	4.40	4.40
	24540	0.18	0.18	0.34	0.34	3.45	3.45	4.44	4.44
	MIN	0.01	0.10	0.02	0.21	0.72	0.36	1.11	0.96
	MAX	3.85	3.85	5.21	5.21	10.14	3.62	10.46	4.44
	AVG	0.70	1.26	1.15	1.78	3.09	1.58	3.59	2.14

Key

Exceeds preferred fish passage thresholds (5 ft/s velocity or 0.67 ft depth)

Exceeds maximum fish passage thresholds (7 ft/s velocity or 0.5 ft depth)

Table 9.4-2: Channel Velocity and Depth at Maximum Impoundment Flows – Dam-In and Dam-Out

Location	River Station (ft)	Average Channel Velocity (ft/s)		Max Channel Water Depth (ft)	
		Existing Conditions	Proposed Conditions	Existing Conditions	Proposed Conditions
	19064	7.66	7.66	2.9	2.9
	19091	6.64	6.64	3.32	3.32
	19123	4.96	4.96	5.9	5.9
	19165	5.44	5.44	5.96	5.96
	19200	8.71	8.71	4.53	4.53
	19236	4.85	4.85	4.69	4.69
Ames Pond Dam	19244				
	19251	2.44	2.44	7.48	7.48
	19315	4.82	4.82	6.19	6.19
	19483	3.54	3.54	7.07	7.07
	19703	5.47	5.47	7.25	7.25
	19726	6.09	6.09	8.47	8.47
	19804	1.72	1.72	5.91	5.91
Armstrong Dam	19814				
	19819	1.12	7.52	9.2	3.27
	20000	0.23	0.84	11.98	6.59
	20085	0.2	0.8	11.73	6.1
	20296	0.51	4.21	8.73	3.06
	20354	0.77	4.61	8.47	3.19
	20499	1.09	4.05	6.23	3.35
	20662	2.2	4.95	5.25	2.79
	20712	2.85	6.05	5.05	2.94
	20752	4.26	6.32	5.04	3.44
Plain St Bridge	20765				
	20831	2.97	3.24	5.88	5.43
	20851	2.24	2.49	5.98	5.55
	20949	2.42	2.9	6.48	6.07
	20975	2.31	2.58	5.29	4.91
	21059	2.58	2.81	5.34	4.98
Upstream RR Bridge	21069				
	21088	2.15	2.34	5.56	5.24
	21090	2.87	3.16	5.34	5.02
Washington St Bridge	21132				
	21186	2.72	2.94	5.52	5.23
	21230	2.69	2.92	5.56	5.28
	21579	1.4	1.52	5.43	5.21
	22068	5.68	5.85	4.88	4.82
	22150	5.22	5.24	6.03	6.02
	22198	5.08	5.09	6.29	6.28
Jefferson Street Bridge	22206				
	22261	4.54	4.55	6.69	6.68
	22285	9.61	9.63	6.42	6.42
	22939	0.6	0.6	7.01	7.01
	23200	2.38	2.39	8.88	8.88
	24540	1.2	1.2	9.2	9.2

10. Alternatives Analysis and Next Steps

The following section provides a summary of this FS, including a cursory analysis of the no action and dam removal alternative. If the dam removal alternative were pursued, recommendations for additional study are included at the end of this section.

10.1 No Action Alternative

The no action alternative assumes that the Armstrong Dam would remain in place.

Fisheries

The dam serves as a physical barrier to the free movement of fish and other aquatic resources, and specifically the movement of migratory fish such as American eel and river herring. The dam prevents migratory fish from accessing historic spawning, foraging, and nursery areas within the Monatiquot River and its tributaries. Resident freshwater fish that move up and down a river to find suitable spawning, rearing, and foraging habitat are also affected. In addition to serving as physical barriers to fish passage, the dam creates an impoundment that alters natural riverine fish habitat and sediment transport.

Water Quality

A detailed water quality analysis was not conducted as part of this feasibility; however, there is ample scientific literature demonstrating the impact of dams on water quality. Three of the more common water quality issues associated with dams includes artificially increasing water temperature, lowering dissolved oxygen (DO) concentrations and causing DO supersaturation (when saturation exceeds 100%). Note that per the Massachusetts Year 2014 Integrated List of Waters, from the confluence of the Cochato and Farm Rivers forming the Monatiquot River downstream 4.4 miles, the Monatiquot River is listed as impaired due to DO (also impaired for physical substrate habitat alterations macroinvertebrates, and fecal coliform).

The Armstrong Dam impoundment is approximately 2,400 feet long. The shallow impoundment is subject to thermal loading as sunlight penetrates the majority of the water column. The impoundment water temperature warms as it takes longer for a cubic foot of water entering the impoundment to leave it compared to a natural free-flowing river.

Relative to DO, fish, mussels, macroinvertebrates and other aquatic biota require DO respiration for survival. DO is a relative measure of the amount of oxygen dissolved in water. DO in rivers is affected by three primary factors: water temperature, atmospheric pressure and dissolved solids. Also important is the amount of decaying matter in the river, turbulence at the air-water interface and the amount of photosynthesis occurring from aquatic plants within the river. Converting from an impoundment to a free-flowing river will result in increased aeration (and hence increased DO concentrations) as water tumbles over rocks. In addition, warm water holds less oxygen than cold water. As noted above, the impoundment is subject to thermal loading and thus will result in lower DO concentrations than a cooler natural river. In some instances, there can be a large diurnal swing in DO concentration when plants emit oxygen during the day potentially causing supersaturation, followed by plants consuming oxygen in the night resulting in lower DO concentrations.

Wetlands & Riparian Habitat

There are minimal wetlands located in the impounded reach. Based on the wetland delineation, vegetation throughout the forested upland portions of the Field Survey Area consists of a canopy layer of northern catalpa (*Catalpa speciosa*), Norway maple (*Acer platanoides*), northern red oak (*Quercus rubra*), white oak (*Quercus alba*), and honey locust (*Gleditsia triacanthos*). The understory includes saplings from the canopy layer and a shrub layer of staghorn sumac (*Rhus typhina*), smooth sumac (*Rhus glabra*), multiflora rose (*Rosa multiflora*) and common buckthorn (*Rhamnus cathartica*). The groundcover layer contains expansive patches of poison ivy (*Toxicodendron radicans*) (LEC, 2016).

The Project Area, including Armstrong Dam, is not located within a *Priority Habitat of Rare Species* or *Estimated Habitat of Rare Wildlife*, according to the 13th edition (October 1, 2008) of the *Massachusetts Natural Heritage Atlas* published by the Natural Heritage & Endangered Species Program (NHESP). No Certified Vernal Pools (CVP) or Potential Vernal Pools (PVP) are mapped on or within the immediate vicinity of the site either. Therefore, the Project is not subject to review under the *Massachusetts Endangered Species Act* (MESA, M.G.L. c. 131A) and its implementing *Regulations* (321 CMR 10.00).

Sediment Transport

Sediment that would normally stay suspended in faster moving waters and be transported downstream will settle in the slow moving water in the Armstrong Dam impoundment. This changes the availability of nutrients and the composition of plant and microbial communities downstream. Sediment impounded by dams will also accumulate and could store toxic materials that are adsorbed physically on sediment particles or absorbed actively by the biota attached to the sediments. As the sediment testing results in the impoundment showed, concentrations of several chemicals not only exceeded the PEC posing a risk to ecological biota, but some chemical concentrations were high enough to pose a risk to human health. In addition, the chemical concentrations in the impoundment were generally higher than the sediment samples obtained upstream and downstream of the impoundment.

Additionally, gravels and cobbles are retained behind dams limiting their recruitment downstream and leads to habitat changes in streams and estuaries. If the dam were to unexpectedly fail, the unplanned downstream release of sediments and potential contaminants could have significant water quality impacts.

Flooding

The Armstrong Dam is operated as a "run-of-river" dam where inflow equals outflow on a nearly continuous basis and therefore does not provide flood control as discussed in Section 2.3. this actually increases upstream water surface elevations—by at least 4.7 feet upstream of the dam for the 100-yr flood. Removal of the dam will help to reduce the area of inundation; however, as the hydraulic modeling showed, even with the dam removed, the Plain Street Bridge is under pressure flow under the 50- and 100-year floods causing the river to backwater upstream of the bridge.

Infrastructure

No impacts to public infrastructure are anticipated with this alternative, unless the dam were to unexpectedly fail. However, as the bridge inspection reports (Appendix D) and sediment probing verified there are already some scour holes in front of the piers and in the channel at the upstream railroad bridge. While structural assessments of the bridges upstream of the dam were not conducted as part of this study, the piers and abutments from the upstream railroad bridge are missing grout and have loose stones.

Scour does not appear to be a concern at the other bridges upstream of the dam. There is at least one water main and three sewer mains upstream of the Armstrong Dam. Profile drawings of the water mains were not available at the time of this report, so it is unclear if the 12-inch main near the Plain Street Bridge is currently affected by scour. Based on the currently available information and the results of the hydraulic analysis, under the no action alternative utility scour does not appear to be a concern.

Recreation and Aesthetics

A recreation assessment was not conducted as part of this Project. There is a boat launch on Armstrong Dam impoundment, but based on the overgrown vegetation on the ramp it is suspected that it is rarely used. There is no formal portage around the dam as public access is limited. Freshwater fishing on the pond is suspected to be limited given the small size of the pond and the lack of evidence of freshwater fishing observed during project site work. Based on the physical setting of the Project area and the small drainage area, little, if any, on-the-water recreation is expected. The Project is located in an urban setting and the impoundment is only 2,400 feet long. We have received reports from the FRWA of whitewater kayaking that occurs over the Rock Falls below the Armstrong Dam. Relative to aesthetics, a partial view of the impoundment can be observed when proceeding across the Plain Street Bridge. It is also visible from the parking lot located on river right of the impoundment. In the summer the impoundment become partially filled with emergent aquatic vegetation. In the winter it ices over.

Operation and Maintenance

If the Armstrong Dam remains in place Messina Enterprises will continue to be responsible for ongoing operation and maintenance costs as well as administrative support relative to communications with the Massachusetts Office of Dam Safety. Maintenance entails clearing debris and log jams at the low-level outlet and spillway gates. The Armstrong Dam currently has a high-hazard classification[38] and will require capital investment to maintain a safe and properly working structure. Although the dam was reported to be in “Fair condition in the most recent dam safety inspection report, several deficiencies were noted, and the structure will only continue to degrade over time unless appropriate maintenance measures are implement. There would still be an obligation to bring the dam into compliance with dam safety regulations as well as the continued responsibility for ongoing operation, maintenance, and liability associated with the dam. Because of the dam’s height and storage volume it is required to pass what is termed the ½ Probable Maximum Flood (PMF[39])- this is a flow higher than the 100-year flood.

10.2 Dam Removal Alternative

The dam removal alternative includes removal of the Armstrong Dam.

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[38] High Hazard Potential dam refers to dams located where failure will likely cause loss of life and serious damage to home(s), industrial or commercial facilities, important public utilities, main highway(s) or railroad(s).

[39] Probable Maximum Flood (PMF) means the most severe flood that is considered reasonably possible at a site as a result of the most severe combination of critical meteorological and hydrologic conditions possible in the region.

Fisheries

Before discussing removal of the Armstrong Dam and what it means to restoring river herring to the basin, other issues relative to fish passage, and flow management are described first. Starting downstream of Armstrong Dam, Rock Falls and Ames Pond Dam, represent vertical barriers to river herring restoration. The barriers would need to be resolved such that river herring can ascend to the Armstrong Dam area.

Above the Armstrong Dam impoundment, are two water supply reservoirs. Richardi Reservoir pumps water that is diverted into it from a diversion dam on the Farm River to Great Pond. Great Pond was found to have high suitability as river herring spawning and nursery habitat (Chase et al. 2015). Upstream and downstream fish passage structures are needed to permit river herring passage into Great Pond. In addition, any upstream and downstream fish passage structure at the Great Pond Dam must also account for the fluctuation water level. The Town of Braintree manages the Great Pond Dam and is presently in the final stages of designing and permitting the installation of a fish ladder scheduled for 2017.

Another issue to consider is flow management. Based on the water level loggers placed in the Farm and Cochato Rivers, there may be times when water is unnaturally flowing upstream in the Farm or Cochato Rivers. The logger data indicates that under really low-flow conditions[40] there is a possibility that Farm River diversions, in combination with suspected surface water or groundwater pumping, could be causing a flow reversal: meaning gravity flow is pulling water from downstream of the diversion location. It is important to note that the extent of the measured flow reversal is relatively small, happens only during really low-flow conditions and could be within the range of survey instrumentation error. We recommend further investigation into these potential flow reversals, including the potential source, magnitude and timing. It is important to understand when these conditions occur as it could impede upstream and downstream passage of river herring since they instinctively swim against the current.

Removal of the Armstrong Dam will presumably eliminate a barrier to upstream and downstream fish passage and would open up approximately 5,200 feet of free-flowing habitat on the Monatiquot River. As noted earlier, what remains unclear at this juncture is whether the bedrock profile located directly beneath the dam results in a “natural” vertical barrier to fish passage. Many dams are purposely positioned on natural falls, and until further assessment is conducted it is unclear if a natural barrier exists. If the dam removal alternative progresses to the next level of feasibility and design study, ground-penetrating radar (GPR) is recommended for the dam. GPR could be conducted above the primary spillway to attempt to map the upper surface of bedrock beneath the dam. This information could then be added to the hydraulic model to more accurately predict whether the falls will impede fish passage under a range of flows. Note that even if a natural barrier exists, it is possible to modify it during the removal process to permit upstream and downstream passage of resident and migratory fish.

Under average to high flows during the migratory fish passage season and with the dam removed, there appears to be sufficient depths and velocities to permit upstream fish passage through the river reach that would convert from impoundment to free-flowing with the dam removed.

[40] The seasonal timing of upstream and downstream river herring migration periods generally will not coincide with the driest times of the year. It is suspected that if flow reversals are occurring they only take place under very low-flow conditions.

During low flow conditions in September there are a few locations where the water depth (per the hydraulic model) were less than 0.5 foot. As noted above the ultimate goal is to move adult herring into the Great Pond water supply reservoir to spawn and provide safe emigration for juvenile herring in the fall. This may require timed-releases from the Great Pond Dam to move juvenile river herring in October/November as there is currently no minimum flow requirement below the dam. Thus, a continuous flow release would be required to move juvenile river herring out of Great Pond. It is recommended that if the full restoration effort occurs including upstream passage into Great Pond, operational releases to move juvenile river herring downstream would be coordinated with high flow precipitation events. Thus, the concern of having adequate river depths would be reduced if releases are coordinated with targeted river flow conditions.

Water Quality

Removal of the Armstrong Dam will improve water quality and aquatic habitat in the Monatiquot River by restoring natural river processes such as flow and sediment regimes. The likely establishment of new bordering vegetated wetlands along the riparian corridor (described below) will also help to filter runoff and improve water quality. Temperature and DO concentrations would be expected to improve throughout the former impoundment with the transition to a more riverine reach and the associated decreased water depths and increased velocities.

Wetlands & Riparian Habitat

The Armstrong Dam impoundment is classified as an open water wetland. Dam removal would restore free-flowing riverine conditions and continuity in the former impoundment, replacing the unnatural lacustrine conditions caused by the dam. Impacts to upstream wetland resources are anticipated to be only short-term in nature, as similar conditions are likely to re-establish at lower elevations along the restored river channel and new bordering vegetated wetlands are created in formerly impounded areas. This transition would be an overall gain for the native plant and animal community. Short-term impacts to wetlands during construction—including turbidity, altered flows, and disturbances from heavy equipment—should be minimized and timed appropriately to lessen impacts.

The new riparian area created within the current impoundment should be monitored for erosion and for the establishment of invasive species. Native shrubs and trees could be planted along the banks of the new channel in the lower impoundment to provide additional bank stabilization and reduce the potential for the establishment of invasive species. A more passive approach could allow for natural revegetation from the existing seed bank.

Sediment Transport

Under the dam removal alternative, a major issue requiring further evaluation is sediment management as the sediment testing in the impoundment showed ecological and human health risk for several chemicals. Dam removal will mobilize some of these sediments unless sediment management measures are implemented. We recommend consulting with the permitting agencies to discuss the sediment findings and to evaluate sediment management alternatives. Alternatives could include additional sediment sampling in the impoundment to isolate contaminants of concern, dredging of contaminated sediment, stabilization of some sediment in place, allowing sediment presenting low ecological risk to naturally transport downstream upon removal and potentially other options. Because of the ecological and human risk associated with the sediment, it is critical to coordinate closely with the permitting agencies on sediment management.

Even if the entire reservoir was dredged, it will not be possible to contain all of the sediment as there will be a short-term increase in turbidity levels and water quality impacts during, and immediately after, a potential dam removal.

Flooding

The dam removal alternative would lower water levels above the dam. Based on the hydraulic modeling, removal of the Armstrong Dam would reduce the floodplain between the dam and Plain Street Bridge. Removal of the dam could potentially reduce some upstream flooding; however, under flow events greater than 586 cfs Plain Street Bridge acts as a hydraulic control and would diminish the reduction in flooding farther upstream.

Infrastructure

Infrastructure that could potentially be impacted by dam removal was discussed in Sections 9.5 through 9.7 and includes bridges, utility lines and stormwater outfalls.

If the dam were removed, water levels would drop in the Armstrong Dam Impoundment, which could potentially have a scouring effect on some of the bridges upstream, including the Plain Street Bridge, located approximately 960 feet upstream of the dam and the upstream railroad bridge, located approximately 1,260 feet upstream of the dam. A buildup of deep sand at the Plain Street Bridge made visual inspection and probing an ineffective way to get a sense of the sediment at the foundations. According to the hydraulic model, water velocities will increase by approximately 2.06 ft/s under the maximum impoundment flow (586 cfs) at the Plain Street Bridge. Collection of GPR would help in refining the model velocities farther upstream which would inform the scour analysis at the Plain Street and other upstream bridges and utilities.

At the upstream railroad bridge, velocities will increase approximately 0.23 ft/s under the maximum impoundment flow (586 cfs). The substrate around the bridge is mainly comprised of small boulder and cobble with the river left side having a deep buildup of sand. However, the piers of the upstream railroad bridge currently have minor scour holes along their faces. There is another area in the channel where finer sediments have been washed out leaving a larger scour hole.

At the Washington Street Bridge velocities will increase approximately 0.29 ft/s under the maximum impoundment flow (586 cfs). The substrate around the bridge is mainly paved cobble overlain with sand. Scour from removal of the dam is unlikely.

Changes to the hydraulics at the Jefferson Street Bridges are small and the substrate around this bridges is larger so scour from the removal of the dam is unlikely. If dam removal progresses to the next level of feasibility analysis, a sediment transport study should be conducted in which the grain size of the sediment under the Plain Street and upstream railroad bridges would be characterized and the potential for predicted water velocities to transport or “scour out” the sediment in the area of the bridge piers/abutments would be evaluated.

Water mains and sewer mains cross Monatiquot River upstream of the dam. There is potential for scour at two of these mains. Near the downstream side of the Plain Street Bridge, a 12-inch-diameter cast iron water main crosses the river. Water velocities will increase by 2.06 ft/s under the maximum impoundment flow (586 cfs) at this location. There is also a 24-inch-by-38-inch reinforced concrete elliptical sewer main crossing the Monatiquot River at a point approximately 25 ft upstream of the Plain Street Bridge. According to Sewer Assessment Plan 6638 (Appendix E), the sewer main is encased in concrete 6 inches all around, and the top of the concrete is buried about 6 inches below the channel bed, so scour is less of an issue. Water velocities will increase by approximately 0.25 ft/s under the maximum impoundment flow (586 cfs) at this location.

Stormwater discharges that currently empty directly into the water may not if the Armstrong Dam were removed, which could lead to scour along the newly exposed upland area. The two culverts on the east side of the impoundment would become perched if the dam were removed. The upstream culvert, by the entrance causeway to the gym, would most likely need to be extended and armored with riprap. The change in WSEL with and without the dam is approximately 6.5 feet. Scour at the more downstream culvert may be prevented by armoring alone, since the main channel will be relatively close to the current outfall location. The two culverts on river right and upstream of the Plain Street bridge may also need to be armored with more riprap as WSEL will decrease approximately 2 feet at these locations.

Recreation and Aesthetics

A primary interest of the FRWA is to improve river access and recreation along this river corridor near the Armstrong Dam. We have received reports from the FRWA of three canoe/kayaking groups that paddle the Monatiquot River in the spring. The range of the Monatiquot River available for kayaking could be increased and improved with the dam removal. An impounded water body could be restored to a free-flowing river connecting the upstream tributaries to the lower reaches of the Monatiquot and Fore Rivers. Also, the chance to view returning diadromous fishes migrating upstream would be valued for aesthetic, recreational, and cultural purposes and as a sign of a healthy river. With the reduction in water levels, additional upland area will be exposed. It is possible that this area could be transformed into a park used for river access, or other recreational improvements. Improvements of public access along this river corridor are an interest of FWRA and the Town of Braintree and FRWA, although plans have not yet been developed on how this might occur and relate to existing property uses.

In terms of aesthetics, there would be a temporary impact of having unsightly exposed mud/sediment along the newly established river channel following dam removal. However, the “newly” created upland areas could be vegetated with plantings or permitted to naturally revegetate. In addition, as described above there is the potential for a park with river access. A pre-Feasibility Study rendering, done by Messina, of how the proposed removal could impact land use is located in Appendix K.

Operation and Maintenance

If the Armstrong Dam were removed Messina would have no further obligations to its operation and maintenance. As importantly, Messina would have no further liability of dam ownership.

10.3 Recommendations for Additional Studies and Next Steps

- **Ground-penetrating radar** – GPR could be conducted above the spillway to better map the bedrock topography beneath the dam. Having this information would help determine if there is a natural vertical barrier to fish passage. In addition, the additional information could be used to further refine the hydraulic model to evaluate depths and velocities upstream for fish passage purposes as well as scour (infrastructure) purposes.

- **Hydraulic modeling** – Additional hydraulic modeling could be conducted to incorporate the results of GPR, Messina's design and the overbank data from the FEMA FIS should be updated with LiDAR. The HEC-RAS model should be converted to a HEC-GeoRAS model to help with inundation mapping. A more formal investigation into the effects of removing Ames Pond Dam is needed. For the hydraulic model conducted for this study, it was assumed the Ames Pond Dam was in place. The hydraulics between the Armstrong Dam and Ames Pond Dam are influenced by the Ames Pond Dam.

- **Sediment sampling** – Additional samples may be needed to inform the sediment management plan. Further consultation with MassDEP is advised based on the sediment sampling findings to determine if additional sediment sampling is necessary and to discuss potential sediment management alternatives. Note that If more sediment sampling is required another sediment sampling plan will need to be developed and approved by MassDEP.

- **Sediment transport analysis** – If sediment management includes allowing some low risk sediment to naturally transport downstream, agencies may require a sediment transport evaluation to determine the locations where mobilized sediment re likely to settle downstream following dam removal. It is also recommended that a sediment transport analysis be performed at the upstream railroad and Plain Street bridges to rule out scour issues from the removal of the dam. Further sediment sampling for grain size analysis around piers and abutment would be required to inform the scour analysis.

- **Sediment Management Plan** – Due to the sediment sampling findings it is recommended that an ecological risk assessor analyze the sediment findings and assist in developing a sediment management plan.

- **Cultural resources mitigation planning** – If any federal money is used to evaluate the feasibility of dam removal, or to physically remove the dam, it will require Section 106 consultation. In short, a qualified historian would be required to evaluate if the dam is eligible for the National Register of Historic Places. In addition, a qualified archeologist would complete a Phase IA study to determine the likelihood of Native Americans and/or Euro American settling near this portion of the river. If the Phase IA study indicates the likelihood of Native American or Euro Americans utilizing the area, then a Phase IB study may be required. A Phase IB study can be more intensive and requires digging test pits to log what is found. Typically, at the end of the cultural resources study, if the dam is found to be eligible and if its removal could impact artifacts, then a Memorandum of Agreement is usually developed among consulting parties, including the State Historic Preservation Officer.

- **Public outreach** – Based on our experience with dam removal projects, public outreach and education is probably the most critical component to a successful project. The extent of public outreach and education is a function of the pond's visibility to the public as well as shoreline development. In this case, the Hollingsworth Pond is readily visible to the public, although there is no residential development around the pond. At this juncture, it is unclear if there are individuals or groups that would oppose dam removal. If Project Partners move forward with this alternative, we highly recommend holding more public meeting to notify individuals and abutters, and most importantly, to identify opponents to a potential removal. A clear plan should be developed for public outreach and education.

- **Engineering Design, Preparation of Drawings and Technical Specifications** – This requires developing plans to address: care and diversion of water during removal, demolition/removal limits and parameters, disposition of materials, channel restoration features, sediment management, and sediment and erosion control requirements during construction. Construction drawings are prepared showing plan view, elevation and sections of the dam and any work required within the impoundment. In addition to the drawings, technical specifications are needed.

- **Preparation and submittal of permit packages** – Permits would require both engineering and environmental input to complete. In Massachusetts there are federal, state and local permits required for dam removal. A summary of potentially required regulatory submittals, reviews, and permits associated with the Armstrong Dam removal is presented in Table 10.3-1

- **Meetings during engineering/permitting phase** – There will be a several meetings both with individual agencies as well as with the public during the period of engineering design and preparation of the dam removal drawings.

- **Preparation of project manual, bid documents and support during bidding** – A Project Manual includes drawings, technical specifications, general conditions, performance bond requirements, and a bid form among other items. The Project Manual is needed to bring the project to competitive bid as it is provided to prospective contractors. Bid Documents would be submitted to common contractor websites for advertising. Typically, once bids are received they would be reviewed relative to meeting the bid requirements, experience and costs.

- **Construction management during removal** – Some permits require that periodic supervision of the dam removal work be conducted.

- **Post dam removal monitoring** –Some permits require post dam removal monitoring requirements.

- **Preparation of grants and management** – If dam removal is pursued further, federal and state monies are available to help defray the cost for feasibility related-work as well as dam removal. Depending on the estimated cost of the project, there could be numerous grant submittals. These submittals require completing a grant application, and soliciting and documenting support for the dam removal. If fortunate to obtain grants, there is also a management responsibility including progress report, budget submittals, and finding “match” money. Time should be allocated for grants.

Table 10.3-1: Potential Permitting Requirements for Armstrong Dam Restoration Alternatives

Permit	Agency	Applicable Regulations	Categories	Applicability	Potential Requirements
					Dam Removal
Wetlands Protection Act Notice of Intent (NOI) & Order of Conditions	MA Dept. of Environmental Protection (DEP) / Conservation Commission	310 CMR 10.00; MGL. c.131 s.40	Order of Conditions Restoration Order of Conditions (general permit or limited project)	Any construction in or near a wetland resource. Ecological restoration projects may qualify for a Restoration Order of Conditions (either general permit or as a limited project). If the project is located within Estimated Habitat of Rare Wildlife, the NOI must also be submitted to the NHESP and DFW where it is subject to the Massachusetts Endangered Species Act (MESA) review.	X *Restoration General Permit (dam removal)*
Environmental Notification Form (ENF)	MA Environmental Policy Act (MEPA) Office	301 CMR 11.00	ENF Expanded ENF (EENF) Environmental Impact Report (EIR)	Thresholds include alteration of 5,000+ SF of bordering or isolated vegetated wetlands, alteration of one-half acre of other wetlands, alteration of 1000+ SF of outstanding resource waters, new/expanded fill or structure in a regulatory floodway, or structural alteration of a dam that causes an expansion of 20% or any decrease in impoundment capacity (triggers EIR). Restoration projects that require an EIR may request a waiver by filing an EENF.	X
Project Notification Form (PNF)	MA Historical Commission (MHC)	950 CMR 70-71; MGL c.9 s.26-27C	N/A	Projects that require state funding, licenses, or permitting.	X
Section 106 Historical Review		36 CFR 800	N/A	Projects that require federal funding, licenses, or permitting.	X
Rare Species Information Request Form	Natural Heritage and Endangered Species Program (NHESP)	321 CMR 10:00; M.G.L. c.131A	N/A	Projects proposed in estimated rare or endangered species habitat, as delineated on the NHESP database.	X
401 Water Quality Certificate (WQC)	DEP	314 CMR 9.00	Minor Project Cert. for Dredging & Disposal (> 100 CY; < 5,000 CY) Major Project Cert. for Dredging & Disposal (> 5,000 CY) Minor Project Cert. for Fill & Excavation (< 5,000 SF) Major Project Cert. for Fill & Excavation (> 5,000 SF or any ORW or special case)	Any activity that would result in a discharge of dredged material (e.g., sediment release) greater than 100 CY that is also subject to federal regulation (e.g., USACE Section 404 General Permit). Application can be combined with Ch. 91.	X
Chapter 91 Waterways License	DEP	310 CMR 9.00	Water Dependent - General	Removal of a licensed structure or dredging of a navigable waterway (most rivers & streams in MA). Application can be combined with 401 WQC.	X
Chapter 253 Dam Permit	DCR Office of Dam Safety	302 CMR 10.09-10 M.G.L c.253;	N/A	Any project to construct, repair, materially alter, breach, or remove a dam.	X
Clean Water Act Section 404 Programmatic General Permit	US Army Corps of Engineers (USACE)	40 CFR 230-232 33 CFR 320-332	Category I GP Category II GP Individual Permit	Discharge of dredged or fill material in a water of the United States, or instream construction activities. Requires Category II review for greater than 25,000 CY dredging, any fill, or other special circumstances.	X
National Pollutant Discharge Elimination System (NPDES) Permit	Environmental Protection Agency (EPA)	40 CFR 122-125	Dewatering General Permit (DGP) Construction General Permit (CGP) Remediation General Permit (RGP)	Discharges from certain construction sites, including clearing, grading, and excavation activities. If disturbance is < 1 acre and discharge is not contaminated, a DGP may be required, or the project may potentially be covered as allowable non-stormwater discharge under the host community's Small MS4 Permit. If > 1 acre, a CGP would be required. If discharge is contaminated, an RGP or Individual Permit would be required. See flowchart for details.	*X*
Conditional Letter of Map Revision (CLOMR)	Federal Emergency Management Agency (FEMA)	44 CFR 60, 65, 72	MT-2 Application: Based on Bridge, Culvert, Channel or Combination Based on Levee, Berm or Other Structural Measures Based Solely on Submission of More Detailed Data	Required to officially revise the current Flood Insurance Rate Map (FIRM) to show changes to floodplains, floodways, or flood elevations.	X

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