TEMPLE STREET DAM REMOVAL / SOUTH RIVER RESTORATION PROJECT

BASIS OF DESIGN REPORT

Town of Duxbury, Plymouth, MA

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Prepared for

COMMONWEALTH OF MASSACHUSETTS
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EXECUTIVE SUMMARY

Background

Massachusetts Department of Fish and Game (DFG), Division of Ecological Restoration (DER) is supporting the Town of Duxbury, MA in plans for removal of the Temple Street Dam, also known as the Boys & Girls Club Dam #2, located on the South River in Plymouth County, Massachusetts.

In 2016 the DER identified the South River Restoration Project as a priority project. The project includes the potential removal of three dams. In upstream to downstream order, they include Temple Street, Chandler Pond, and Veteran's Memorial Park Dams. DER's current objective is to develop 75% design plans for the removal of the Temple Street Dam.

Project partners are pursuing a dam removal alternative that includes removing the dam outlet structure while sizing the breach and constructing two riffles to meet the following project goals.

- Remove the Temple Street Dam to restore ecological processes and natural hydrology while limiting, to the extent practical, downstream impacts to infrastructure and structures,
- Restoring fish and wildlife passage, particularly for river herring, American eel, and other anadromous fish species,
- Reduce or eliminate the need for dam maintenance,
- Mitigate the impacts of climate change (e.g., through carbon sequestration and flood attenuation).

The existing Temple Street Dam consists of an approximately 15-foot-long concrete outlet structure with earthen embankments extending to the east and west. The impoundment elevation is controlled by two 4-foot-long dagger flashboards at the outlet set at a crest elevation of approximately El. 35.2 feet, in the North American Vertical Datum of 1988 (NAVD88). The dam is currently a barrier to aquatic organism passage.

Previous Studies

This study builds upon the findings from previous investigations and conceptual feasibility assessments of the Temple Street dam removal. The findings from studies between 2016 and 2021 were reviewed and expanded upon in the development of the 75% design. Most recently between 2018 and 2021 a 2-dimensional HEC-RAS was developed to assess various dam removal concepts by Pare and Inter-Fluve. This hydraulic model was reviewed and expanded upon in this phase to assess the fish passage potential of the design.

Data Collection & Analyses

The following data collection efforts and analyses were conducted to support the design of the dam removal:

- Topographic & bathymetric survey
- Sediment probing and volume analyses
- Sediment sampling and development of a sediment management plan
- Wetland delineation
- Hydrologic & hydraulic analysis

- Infrastructure analysis

Alternatives Analysis

Various alternatives were considered for the Temple Street dam removal, including no action, dam removal with one riffle feature downstream of the dam and in-stream (passive) sediment management, and dam removal with active sediment management two riffle features and active sediment management. Major design constraints included limiting impacts to infrastructure downstream of the dam including houses, utilities, and stream crossings (roads and culverts).

Proposed Design

Dam Breach/Fish Passage

The proposed dam breach will be replaced with a stone riffle feature with a V-shaped low-flow channel with an invert approximately 2.5 feet below the current dagger board elevation. The side slopes of the breach will be 2H:1V. The width and invert elevation of the dam breach was selected based on trying to limit impacts to flood storage at the dam while keeping river depths with specific criteria for fish passage. This riffle feature will have a channel slope of approximately 0.015 ft/ft.

The riffle feature at River Street will be constructed of stone downstream of the crossing and the Town water line. Similar to the riffle feature at the dam location it was designed with a low flow channel to provide adequate depths for fish during the upstream migration season while keeping velocities within a reasonable range for higher flows. This riffle feature will have a channel slope of approximately 0.023 ft/ft.

The proposed riffles were designed to accommodate river herring's maximum burst speed of approximately 5-7 feet per second and minimum flow depth requirement of about 0.7 feet for successful passage through a range of reasonable of flows.

Floodplain Enhancements

The proposed project includes adding 10-foot-long large woody debris pieces (logs) across the floodplain to increase floodplain roughness and flood attenuation in the impoundment when the dam is breached. Approximately 40-50 logs would be installed within the impoundment area in a series of single or multiple log features (log jams). The logs would be anchored with either 2-ft diameter boulders and cable or with a pair of duckbill earth anchors.

Additionally, the project will also include the construction of beaver dam analogs in two strategic locations in the impoundment. These structures are essentially man-made beaver dams consisting of brush, sediment, large woody debris, and other materials. These structures will be constructed at approximately 3-4 feet high to elevation 38.0 and be tied into existing grade on each end to prevent flows from flanking around the sides of them. The purposes of these structures are to provide additional flood storage. Logs will be placed in strategic locations to protect them.

A low flow channel is proposed through one of the beaver dam analogs to provide a path through them for recreation users and fish to navigate their way between the dam and the upper impoundment.

Property & Infrastructure

Structures and Residences

A two-dimensional hydraulic model was used to compare the existing water surface elevations downstream of proposed water surface conditions with the dam removed. The proposed design hydraulic modeling results show a slight increase in the WSEL at three residential properties under the 5-year and 25-year storm events and essentially no change resulting from the 100-year storm event. Based on historic flow conditions (current climate conditions), the increase in the WSEL downstream with the dam removed ranges from 0.02 to 0.16 ft or 0.2 to 1.9 inches based on hydraulic modeling. This suggests that the proposed conditions result in essentially no increased flood risk at these structures

<u>Utilities</u>

Approximately 100 feet downstream of Temple Street Dam, water flows through an abandoned road crossing at River Street. The channel at this location is armored with cobble/stone. There is an active buried 12” diameter ductile iron water line which was installed at this crossing circa 2007. The approximate top of pipe elevation is approximately 2 feet below grade at the stream crossing and 5 feet below grade underneath the River Street embankments. The water line runs between the intersection of Keene Street and River Street to the intersection of River Street and Temple Street. Impacts to the water line are not anticipated as the proposed two riffle design proposes fill on the downstream side of the existing armor over the pipe. The riffle feature will provide additional stone protection for the pipe and allow fish to migrate past River Street once the project is complete.

Sediment Management

The quantity and quality of the sediment behind the Temple Street Dam was evaluated in this study. The existing dam interrupts natural sediment transport processes, causing sediment to accumulate within the reservoir. Based on sediment depth mapping performed during this study it was determined that for the recommended design up to 200 cubic yards of sediment could mobilize and settle downstream. However, due to the mild slopes of the South River downstream of Temple Street Dam there is the potential that this sediment could settle out at structures downstream. This accumulation could in turn reduce hydraulic capacities of in-stream structures and increase flooding. Therefore, a sediment management plan was developed to address this.

A passive sediment management approach of letting sediment mobilize and settle downstream naturally is not recommended as sediment buildup at downstream crossings or structures would increase water surface elevations. Therefore, it is proposed that an active approach be utilized consisting of dredging approximately 200 CY of sediment immediately upstream of the concrete outlet structure and dredging a pilot channel approximately 600 feet upstream.

After the sediment has been dredged several potential options for on-site re-use were identified including applying it as loam to the River Street and dam embankments, washing it into the voids of the riffle features, or burying the brick remains adjacent to the western dam embankment. Clean sediment could also be regraded elsewhere on the upland portion of the property (i.e., outside of the top-of-bank lines, which is known as “Upland Material Reuse”).

Project Implementation

<u>Regulatory Reviews</u>

Anticipated permitting and regulatory review requirements for the project include consultation with the Massachusetts Historical Commission; a determination of applicability from the Massachusetts Department of Environmental Protection (DEP) Chapter 91 Waterways Program; a Massachusetts

Environmental Policy Act Environmental Notification Form; a Section 401 Water Quality Certification from DEP; a Preconstruction Notice submitted to the US Army Corps of Engineers; a Wetlands Protection Act Notice of Intent for an Ecological Restoration Project submitted to the Conservation Commission.

Opinion of Probable Construction Costs

The opinion of probable construction costs (OPCC) for the project estimates approximately $510,000 for direct construction costs and about $100,000 for permitting support, engineering services during design finalization, bidding, and construction phase services.

Potential Resource Impacts and Benefits

Wetland Resources

The proposed project will involve up to 200 cubic yards (CY) of dredging of existing sediment and up to 230 CY of fill below the mean annual high-water line comprised of stone and fine material used to construct the riffle features.

Despite temporary wetland impacts due to construction and anticipated changes in wetland classifications, the dam removal will result in an overall, long-term benefit for the restoration of the South River watershed. The existing dam is a barrier to fish passage. The proposed dam removal would result in a potential reduction in bordering vegetated wetlands, but also the possible creation of new riparian habitat areas.

Other Regulated Resource Areas

The South River in this location is located within the Massachusetts Division of Fisheries and Wildlife Natural Heritage & Endangered Species Program's mapped Estimated and Priority Habitats of rare species and is classified as an Outstanding Resource Water (ORW) by the Massachusetts Department of Environmental Protection. In addition, the United States Fish and Wildlife Service's online IPaC website indicates the Threatened species, the Northern Long-eared Bat, may be a potential species of concern in the project area. The proposed project will provide an overall benefit to these regulated resource areas. More natural ecological processes will be restored and both aquatic and riparian habitat will be improved. Potential impacts to species downstream of the project will be temporary in nature and minimized with best management practices during construction and active sediment management.

Water Quality & Aquatic Habitat

The proposed project will improve the capacity of the South River to sustain its designated use of aquatic life. The existing dam concrete outlet structure will be replaced with a grade control/riffle feature and a second riffle feature will be constructed at the abandoned River Street crossing. These improvements will restore ecological processes and natural hydrology and restore fish and wildlife passage, particularly for river herring and American eel.

Summary

In summary, the proposed dam removal of the concrete outlet structure at the Temple Street dam, riffle features, and floodplain enhancements is expected to benefit wetland and other resource areas, water quality and aquatic habitat, fisheries, and climate change resilience. The design will limit downstream impacts to infrastructure while providing ecological benefits such as fish passage and flood storage.

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ACE	Annual Chance Exceedance
ACEC	Area of Critical Environmental Concern
BVW	bordering vegetated wetlands
cfs	cubic feet per second
cm	centimeters
CY	cubic yards
DCR	Massachusetts Department of Conservation and Recreation
DEP	Massachusetts Department of Environmental Protection
DER	Massachusetts Division of Ecological Restoration
DFG	Massachusetts Department of Fish and Game
DFW	Massachusetts Division of Fisheries and Wildlife
DOT	Massachusetts Department of Transportation
EL	elevation
ENF/EENF	Expanded Environmental Notification Form
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
ft	feet
ft/s	feet per second
Gomez and Sullivan	Gomez and Sullivan Engineers, DPC
GSE	Gomez and Sullivan Engineers, DPC
HEC-RAS	Hydraulic Engineering Center River Analysis System
LiDAR	Light Detection and Ranging
LOMR	Letter of Map Revision
MAHW	mean annual high water
MassGIS	Massachusetts Office of Geographic Information
MCP	Massachusetts Contingency Plan
MEPA	Massachusetts Environmental Policy Act
MGD	million gallons per day
MHC	Massachusetts Historical Commission
NAD 83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NHESP	Natural Heritage & Endangered Species Program
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NOI	Notice of Intent
ODS	Office of Dam Safety
OHW	ordinary high water
OPCC	opinion of probable construction cost
ORW	Outstanding Resource Water
PEC	Probable Effects Concentration
PNF	Project Notification Form
RTE	rare, threatened, and endangered
RTK	real-time kinematic
sf	square feet
TOY	time-of-year

USACE	United States Army Corps of Engineers
USFWS	US Fish and Wildlife Service
USGS	US Geological Survey
WQC	Water Quality Certification
WSE	water surface/water surface elevation

1 BACKGROUND

1.1 Project Purpose & Background

The Massachusetts Department of Fish and Game (DFG), Division of Ecological Restoration (DER) is supporting the Town of Duxbury, MA in plans for removal of the Temple Street Dam, also known as the Boys & Girls Club Dam #2, located on the South River in Plymouth County, Massachusetts. A location map is shown in **Figure 1.1-1** in **Appendix A**.

In 2016 the DER identified the South River Restoration Project as a priority project. The project includes the potential removal of three dams. In upstream to downstream order, they include Temple Street, Chandler Pond, and Veteran's Memorial Park Dams. DER's current objective is to develop 75% design plans for the removal of the Temple Street Dam.

Project Partners

Project partners include the dam owner (Town of Duxbury, MA), DER, and the North and South Rivers Watershed Association.

Project Goals

Project partners are pursuing dam removal that includes removing the dam outlet structure while sizing the embankment breach and constructing a two riffle features to meet the following project goals.

- Remove the Temple Street Dam in order to restore ecological processes and natural hydrology while limiting, to the extent practical, downstream impacts to infrastructure and structures,
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- Restoring fish and wildlife passage, particularly for river herring, American eel, and other anadromous fish species,
- Reduce or eliminate the need for dam maintenance,
- Mitigate the impacts of climate change (e.g., through carbon sequestration and flood attenuation).

Dam removal will provide upstream fish access to approximately 2.5 miles of additional habitat for migratory fish and restore natural river processes for downstream benefits (e.g. sediment and organic matter transport). The primary target for fish passage design improvements are the anadromous alewife (*Alosa pseudoharengus*) and blueback herring (*Alosa aestivalis*), known collectively as river herring.

Previous Studies

This study builds upon the findings from previous investigations and conceptual feasibility assessments of the dam removal. Since the South River Restoration Project was identified in 2016 several investigations for the overall project and removal of the Temple Street Dam have been completed including:

- 2015, Tighe & Bond: hydrology and hydraulics (H&H) study and 1-D HEC-RAS model of the lower South River from Main Street (Route 3A) to Chandler's Pond Dam (DER-funded).
- 2016, Pare: Temple Street Dam site reconnaissance study (contracted by DER).
- 2017, FEMA: H&H analyses for the South River and Zone A tributaries were completed for the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for Plymouth County, revised July 6, 2021 (FIS 25023CV001D).

- 2018, Pare: H&H study including a 1-D unsteady state HEC-RAS model for the upper South River from upstream of the Temple Street Dam impoundment to approximately 1,000 feet downstream of Route 3 (contracted by DER and Town of Duxbury).
- 2020, Pare: H&H study including a 2-D unsteady state HEC-RAS model from above the Temple Street Dam to the tidal portion of the river, to determine impacts of removal of the Temple Street Dam (contracted by DER).
- 2021, Inter-Fluve (with Pare as subcontractor): conceptual design and refined 2-D hydraulic modeling effort (contracted by DER).

Current Scope

In 2021, DER contracted with Gomez and Sullivan Engineers, DPC (GSE) to conduct additional investigations and develop approximately 75% complete design for the removal of the Temple Street Dam. Tasks completed during this phase included a site inspection, infrastructure evaluation, topographic and bathymetric survey, sediment probing and volume estimation, H&H analyses, sediment sampling and preparation of preliminary design plans, cost estimate, and this basis of design report.

Existing Information

The following key background information was collected and reviewed for this report:

- Technical Memorandum – Temple St. Conceptual Design Alternatives and Hydraulic Modeling Results, June 2021.
- Technical Memorandum – South River Hydrology and Hydraulic Analysis: Temple Street Dam Removal by Pare Corporation, August 2020.
- Hydraulic Study Report for South River Restoration / Temple Street Dam Removal, Hydrologic and Hydraulic Analysis of Upper South River – Duxbury, MA by Pare Corporation, June 2018
- Flood Insurance Study (FIS) report for Plymouth County, MA(Federal Emergency Management Agency (FEMA), 2021). The FEMA panels cover the South River from its tributary waters upstream of the project area down all the way through the project area to the Atlantic Ocean.
- Hydrologic and Hydraulic Evaluation of the Lower South River by Tighe & Bond, June 2015.
- StreamStats basin delineation and flow computations at the dam and other locations (US Geological Survey (USGS), 2022)
- Streamflow gage data for USGS Gage No. 01105730 (Indian Head River at Hanover, MA) (USGS, 2022)
- List of threatened and endangered species that may occur in the project area (US Fish and Wildlife Service (USFWS), IPaC 2022)
- Relevant GIS data layers (Massachusetts Office of Geographic Information (MassMapper), 2022)

1.2 Project Description

Watershed

The South River flows through Plymouth County, Massachusetts in a northeasterly direction for approximately 15 miles from its headwaters in the Town of Duxbury to its tidal estuary in the Town of Marshfield, where it joins with the North River and flows into Massachusetts Bay. This river is located within the Massachusetts Division of Fisheries and Wildlife Natural Heritage & Endangered Species Program’s mapped Estimated and Priority Habitats of rare species and is classified as an Outstanding Resource Water (ORW) by the Massachusetts Department of Environmental Protection.

The drainage area at the Temple Street Dam is approximately 5.5 square miles and is comprised of about 24 percent wetlands, which provide flood storage, and is approximately 59 percent forested. The mean basin slope is about 3.38 percent (USGS, 2022).

Major features and structures along the river, from upstream to downstream, include:

- South River Reservoir
- Temple Street Dam
- River Street Crossing (abandoned)
- Myrtle Street
- Route 3 South
- Route 3 North
- Feinberg Bog Control Structure
- South River Trail Control Structure
- Old Ocean Street
- Chandlers Pond Dam Primary/Auxiliary Spillways
- Old Ocean Street/Pudding Hill Lane
- Cross Street/Old Ocean Street
- Route 139 (Plain Street)
- Veterans Memorial Park Dam
- Main Street
- Willow Street
- Francis Keville Bridge
- South River (mouth)

Figure 1.2-1 in **Appendix A** shows these features.

Regulated Resource Areas

This river is located within the Massachusetts Division of Fisheries and Wildlife Natural Heritage & Endangered Species Program's (NHESP) mapped Estimated and Priority Habitats of rare species. Priority habitat is based on the known geographical extent of habitat for all state-listed rare species, both plants and animals and is codified under the Massachusetts Endangered Species Act (MESA). Habitat alteration within Priority Habitats may be subject to regulatory review by NHESP. Estimated habitats are a subset of Priority Habitats and are based on the geographical extent of habitat of state-listed rare wetland wildlife species codified under the Wetlands Protection Act. The project area is listed as Priority Habitat (PH 814) and Estimated Habitat (EH 642).

In addition, federally listed endangered species and critical habitat was obtained using the USFWS' online IPaC assessment. Based on the this, the Northern Long-eared Bat is listed as a Threatened species within the project area. The South River at is classified as an Outstanding Resource Water (ORW) by the Massachusetts Department of Environmental Protection.

The water quality classification of the South River through the Project area (Segment ID MA94-08) is Class B and has designated uses including habitat for fish, other aquatic life, and wildlife, including for their reproduction, migration, growth, and other critical functions, and for primary and secondary contact recreation. Class B waters shall be suitable for irrigation and other agricultural uses and for compatible industrial cooling and process uses. These waters shall also have consistently good aesthetic value.

Dam

Representative photographs of the Temple Street Dam are provided in **Appendix B**. A plan of existing conditions is provided in Drawings 3 and 4 of **Appendix C**.

The existing Temple Street Dam consists of an approximately 15-foot-long concrete outlet structure with earthen embankments extending approximately 70 feet from the outlet structure on river right[1] and approximately 130 feet from the outlet structure towards river left. The earthen embankments have side slopes of approximately 2H:1V and a pedestrian access trail runs along its centerline. The impoundment elevation is controlled by two 4-foot-long dagger flashboards at the outlet set at a crest elevation of approximately El. 35.2 feet, in the North American Vertical Datum of 1988 (NAVD88)[2]. The bottom of the outlet on the downstream side is protected by a concrete apron overlain with cobble. The height of the concrete training walls is approximately 8 feet from the top of the concrete apron. The invert elevation of the concrete apron is approximately El. 30.4 feet. The dam is a barrier to aquatic organism passage.

The Massachusetts Office of Dam Safety lists the dam as non-jurisdictional, and as such has no hazard classification.

Surrounding Infrastructure

River Street

Approximately 100 feet downstream of Temple Street Dam, water flows through an abandoned road crossing at River Street. The channel at this location is armored with cobble/stone and is trapezoidal in shape with an approximate 4 feet wide bottom and 2H:1V side slopes. There was once a culvert at this location which was washed out (Pare 2018). Based on outreach to the Duxbury Water & Sewer Department there is an active buried 12" diameter ductile iron water line which was installed at this crossing circa 2007; the approximate top of pipe elevation of the water line is 30.9 feet NAVD88 (approximately 2 feet below grade at the stream crossing and 5 feet below grade underneath the River Street embankments. The water line runs between the intersection of Keene Street and River Street to the intersection of River Street and Temple Street. Drawings showing the approximate location of the water line are provided in **Figures 1.2-2 and 1.2.-3 in Appendix A**.

Myrtle Street

Approximately 200 feet downstream of Temple Street Dam, the South River flows through a 5-foot high by 10-foot-wide concrete box culvert underneath Myrtle Street which is an active roadway.

Downstream Infrastructure

There are two dams downstream of Temple Street Dam including Chandler Pond Dam (~2.2 miles downstream) and Veteran's Memorial Park Dam (~3 miles downstream). There are also three residential structures of interest downstream along the South River at 229 Old Ocean Street (~1.5 miles downstream), 108 Cross Street (~2.3 miles downstream), and 60 Cross Street (~2.5 miles downstream).

[1] River right assumes one is looking in a downstream direction.

[2] Unless otherwise noted all elevations are based on NAVD88.

2 DATA COLLECTION & ANALYSES

The following data collection efforts and analyses were conducted to support the 75% design of the Project:

- Topographic & bathymetric survey
- Sediment probing and volume analyses
- Sediment sampling and development of a sediment management plan
- Wetland delineation
- Hydrologic & hydraulic analysis
- Infrastructure analysis

These analyses are described in more detail in the following sections.

2.1 Topographic & Bathymetric Survey

A topographic and bathymetric survey was conducted to support hydraulic model development and sediment management plan development and fill in the data gaps from previous studies. The survey work was conducted on December 15 – 17th, 2021. A total station was used in conjunction with a survey grade real-time kinematic (RTK) GPS unit. Accuracy for this system is typically within ±0.05 feet horizontally and ±0.1 feet vertically. The survey area extended from approximately 600 feet upstream of the dam to the southern end of the Lower Impoundment to just downstream of Myrtle Street. The survey was tied into the North American Datum of 1983 (NAD 83) State Plane Massachusetts Mainland (feet) and NAVD88. Surveyed features included the existing dam outlet structure, embankment profile, stream profile, stream cross-sections, utilities, wetland flags, trees, control points, and upstream and downstream structures. LiDAR elevation data (from 2011) was used to provide upland topography outside of the survey area in the previous hydraulic modeling studies.

Additional details are provided below. The existing site plan is shown in **Drawings 3 and 4** of **Appendix C**. Elevation and section views of the dam are shown in **Drawing 8** of **Appendix C**.

- Existing Structure – The locations and elevations of key features of the existing dam were surveyed, including the upstream and downstream invert elevations at the dam, as well as the concrete training walls, concrete apron, and earthen embankments.
- Stream Profile – A detailed profile of the stream thalweg was surveyed for approximately 600 feet upstream of the dam and downstream to Myrtle Street to evaluate the potential for fish passage alternatives.
- Sediment Probing Cross-Sections – Sediment probing to refusal was conducted at a total of 6 stream cross-sections. The cross sections were surveyed at approximately 600, 360, 170, 100, 55, and 25 feet upstream of the dam. Estimated volumes of potentially mobile and immobile sediment post-dam removal were based on these cross sections.
- Wetland Flags – The location of wetland delineation flags placed by LEC Environmental Consultants, Inc., were surveyed.
- Recoverable Control Points –two recoverable control points were established to allow for tie-in for future surveys and construction stakeout.

2.2 Sediment Analyses & Management Plan

Dams can interrupt the natural continuity of sediment transport, causing sediment to accumulate within a reservoir and depriving downstream reaches of sediments needed to maintain channel form and to support the riparian ecosystem. Over time a dam will retain a certain volume of sediment with occasional flushing of some surface sediment during high flow events only to be replenished again. By removing a dam, it allows nutrient filled sediments to naturally flow downstream. However, due to the mild slopes of the South River downstream of Temple Street Dam sediment accumulation can occur at structures downstream. This accumulation can in turn reduce hydraulic capacities of in-stream structures and increase flooding. Therefore, a sediment management plan was developed to address this.

A sediment management plan is developed based on the quantity and quality of sediment present in the impoundment and in upstream and downstream reaches as well as the results of a due diligence analysis to assess the potential for contaminants in the watershed upstream of the dam. Management alternatives generally fall under one of two approaches—active or passive management. Active management includes more traditional methods to remove or otherwise control the sediment, such as mechanical dredging and channel reconstruction or in-place stabilization. Conversely, passive management, also known as "in-stream management," involves the natural erosion and downstream repositioning of impounded sediments over time. The approach is based on the premise that most (if not all) of the accumulated sediments in impoundments resulted from the presence of the dam, and that the accumulated material would have been transported downstream in the absence of the barrier. In fact, substrate in reaches downstream of dams are often lacking in finer sediments and would benefit from a gradual release of sediments from behind the breached dam.

Dam removal projects in Massachusetts and elsewhere in New England have demonstrated that in-stream management of the appropriate types of sediments can be an acceptable sediment management strategy. While minor short-term impacts to downstream receiving areas may occur (e.g., deposition of sediment in pools), the potential for numerous medium- and long-term ecological benefits exists, including benthic habitat improvements and an influx of organic matter. Natural channel formation (versus a constructed channel) is also preferred as it is more likely to result in a dynamically stable stream form, involves far less cost, and avoids related impacts from the use of heavy equipment in recently dewatered soft wetland areas.

Because the proposed project will involve dredging of more than 100 cubic yards (CY) and dredging in an ORW, a Section 401 Water Quality Certification (WQC), and Chapter 91 license will be required from the Massachusetts Department of Environmental Protection (DEP) and the project will be subject to sediment quality analysis requirements in accordance with 314 CMR 9.00.

The quantity and quality of sediment impounded upstream of the Temple Street Dam were evaluated as discussed below.

2.2.1 Due Diligence Review for Potential Sediment Contamination

The potential for contaminated sediments flowing into the dam's impoundment, retained in the impoundment, or located downstream of the dam is evaluated as part of due diligence. Having information on the potential for contaminated sediment in the project area helps to further inform the development of a sediment management plan for removal of the dam. Many contaminants released into rivers and streams in the form of industrial wastes, accidental spills, or urban runoff commonly adhere to solids suspended in the water column of a stream and ultimately accumulate in slow moving environments, such as impoundments behind dams.

Although less obvious, sediment contamination should also be a consideration for restoration alternatives that leave a dam intact, including the “no action” alternative. Contaminants trapped in sediment behind dams are often considered buried, but they cannot be assumed to be immobile. Some contaminants are easily exchanged between bottom sediment and the overlying water column, allowing them to become biologically available under certain environmental conditions. Sediment-bound contaminants can also be scoured, re-suspended, transported downstream, and redeposited during storm events, potentially affecting aquatic organisms, including fish, far from the original source. Additionally, benthic organisms, which live on or within the bottom sediment, may be directly exposed to hazardous levels of these contaminants and, in turn, indirectly expose fish and other wildlife to the contaminants through food-web magnification. Humans may be exposed through ingestion of affected wildlife or by direct physical contact (Breault et al., 2013).

Additionally, sediment quantity and quality should be factored into the overall hazard classification associated with a dam along with its structural integrity and downstream risks, although dam safety inspections required by the ODS do not currently consider this information.

This section includes a due diligence review of existing information relating to the potential for sediment contamination in the Temple Street Dam impoundment.

Releases of oil and/or hazardous material to the environment are required to be reported to the MassDEP’s Bureau of Waste Site Cleanup, in accordance with M.G.L. Chapter 21E and procedures established within the Massachusetts Contingency Plan (MCP) (310 CMR 40.0000). All reported releases are given a period of one year to either be cleaned up or be classified as either Tier I (indicating groundwater contamination in a current drinking water resource area, presence of an imminent hazard or Critical Exposure Pathway, or ongoing Immediate Response Action that involves remedial action) or Tier II (all other sites) in order to undergo a comprehensive assessment and cleanup program. Failure to comply with cleanup or “tier classify” in the one-year timeframe results in the site being automatically classified as a Tier ID (Default) site. In cases where cleanup cannot be achieved to the most protective use, a Notice of Activity and Use Limitation (AUL) must be attached to the deed of the contaminated property to document the location of residual contamination and specify restricted and permitted activities and uses of the property in this location (AUL area).

MassDEP maintains a searchable online database of waste sites and reportable releases, as well as a file viewer that can be used to access electronic reports and forms for those sites (MassDEP, 2016). MassGIS periodically publishes the MassDEP waste sites database in a spatial format that can be used to identify potential sources of sediment contamination in a particular watershed. According to the most recent (2018) MassGIS publication of the MassDEP waste site data, there are currently no classified sites listed in the drainage area upstream of the Temple Street Dam. The contributing watershed down to Veterans Park Dam was assessed as well to assess the potential for any potential sediments to affect areas downstream of the site. There were no sites between Temple Street Dam and the Veterans Park Dam for which a Permanent Solution (permanent site closure) has not yet been achieved (i.e., “active sites”). Of the sites with a Permanent Solution (i.e., “closed sites”) [3] in the sites in the vicinity of the Project (at least

[3] The total number closed sites (including those with AULs and those without) is not included in the MassGIS database, but can be found in the MassDEP database, which is searchable by town/city.

2.65 miles downstream), three have AULs. **Figure 2.2.1-1** shows a map of oil and/or hazardous material sites in the watershed.

Based on a search of the data portal conducted in January 2022, there are no active release sites within the South River watershed upstream of the Temple Street Dam. There are two closed sites with AULs implemented at each located 2.65 and 3.27 miles downstream of Temple Street Dam. There are no waste or reportable sites within the boundary of the South River watershed upstream of the dam as delineated by StreamStats, shown in **Figure 2.2.1-1**. A summary of the closed sites in the vicinity of the project site is provided in **Table 2.2.1-**1. As the releases were all relatively minor in nature and located at a sufficient distance downstream from the project site, it is assumed that they would not have resulted in an increased risk for sediment contamination at the site.

Table 2.2.1-1: Summary of Reportable Releases within the Vicinity of the Project Site

RTN	Site Address	Town	Distance Downstream of Dam (mi)	Within Watershed	Release Date	Chemical(s)	Amount	Status
4-0000789	2170 Ocean Street	Marshfield	2.65	Yes	10/24/1989	Petroleum	Unknown	Closed (AUL implemented)
4-0015251 (Primary) 4-0026173 (Secondary)	1901 Ocean Street	Marshfield	3.27	Yes	01/11/2000	Ethene, 1,2-Dichloro, (z) MTBE TCE PCE TPH	390 ug/L 2000 ug/L 290 ug/L 350 ug/L 500 ug/L	Closed (AUL implemented)

Source: MassDEP, 2021. More information and documents available by searching by RTN at https://eeaonline.eea.state.ma.us/portal#!/search/wastesite

A search of the US Environmental Protection Agency (EPA) Superfund Enterprise Management System returned one result for a Superfund site in Duxbury, but it was located outside of the watershed. Two results were returned for Marshfield, but those sites were both located north of the South River and outside the watershed.

The Duxbury Conservation Commission was also consulted for any relevant information in the project area watershed, but no additional information regarding potential sediment contamination was uncovered.

In summary, the due diligence review did not reveal any potential sources of oil or hazardous material contamination for the sediment proposed to be managed.

2.2.2 Sediment Quantity Assessment

To quantify the volume of sediment impounded by the dam, sediment depth mapping was conducted at transects throughout the impoundment.

Methods

Sediment depth mapping was conducted on December 14-15, 2021. A total of six transects were collected, ranging from approximately 1 foot to 2 feet upstream of the dam face and generally varied from 3-5 feet in within the sediment probing area further away from the dam with some depths exceeding 6 feet at T-6. For each transect, a steel rod marked in 0.5-foot increments was driven with a hammer into the sediment until refusal at points spaced approximately 15 feet apart on average. Sediment composition (e.g., silt, sand, muck, gravel, etc.) was roughly characterized based on feel and any vertical changes in composition were documented. Water depths were measured relative to the water surface elevation (WSE), which was surveyed using an RTK GPS at each transect. Transect proving locations were horizontally georeferenced using RTK GPS.

The resulting data were used to compute elevations for the top and bottom of sediment at each probing station. The cross-sectional area of sediment in each transect was calculated and then interpolated between transects to estimate the total volume of impounded sediment.

Results

The location of the sediment transects are shown in **Figure 2.2.2-1**. Dots along the transects in **Figure 2.2.2-1** correspond to individual probe locations, which are color-coded according to sediment depth as detailed in the legend. Cross-sectional plots of each transect showing the water surface, sediment top, and sediment bottom elevations are presented in **Figures 2.2.2-2** through **Figure 2.2.2-7**.

Total Sediment Volume

The volume of sediment estimated to lie between the Temple Street Dam and the proposed upstream grade control at transect T-6 is approximately 14,900 cubic yards (CY). The sediment upstream of the dam consists primarily of fines with organic matter which becomes covered with aquatic vegetation during the growing season.

Mobile Sediment Volume

The volume of potentially mobile sediment in the lower impoundment was estimated by calculating the approximate channel bankfull width for the site that would be anticipated if a channel formed naturally through the lower impoundment. It also assumes all of the sediment within those bankfull limits above the potential lower limit of the vertical adjustment (headcut) line through the lower impoundment would mobilize under proposed dam removal conditions between the proposed grade control at the dam and at transect T-6. The current design concept would lower the elevation at the dam to approximately El. 33.0 ft and raise the channel at transect T-6 to approximately El. 33.00 ft. Based on these elevations it is anticipated that only sediment in the vicinity of the dam and transects T-1 and T-6 will be excavated/actively managed to develop the streambed modifications. The top of sediment in the thalweg at transects T-2 through T-5 is below the anticipated headcut line and is expected to remain in-situ. The anticipated bankfull channel dimension calculations and headcut line elevations based on the proposed grade control elevations are provided in the sediment probing data attached to the Sediment Sampling plan approved by DEP.

Regional regression equations were used to estimate the bankfull width of the river in a post-dam removal scenario. The United States Geological Survey (USGS) report titled "Equations for Estimating Bankfull Channel Geometry and Discharge for Streams in Massachusetts" provides two equations for calculating channel bankfull width: 1) a simple regression equation based on drainage area (Equation 1 below) and 2) a multiple regression equation based on drainage area and mean basin slope (Equation 2 below). These equations are presented below:

Equation 1 (Simple Regression): Bankfull Width (ft) = 15.0418 x (Drainage Area (mi^2))$^{0.4038}$

Equation 2 (Multiple Regression): Bankfull Width (ft) = [10.6640 x (Drainage Area (mi^2))$^{0.0.3935}$] x [(Mean Basin Slope (%))$^{0.1751}$]

Additionally, a 2015 article in the Journal of the American Water Resources Association titled "Development and Evaluation of Bankfull Hydraulic Geometry Relationships for the Physiographic Regions of The United States" presents three different regression equations for sites across the United States depending on the resolution/level of analysis. The most applicable regression equation for the project area is based on drainage area (presented below).

Equation 3 (New England Region): Bankfull Width (m) = 5.90 x (Drainage Area (km^2))$^{0.280}$

Table 2.2.2-1 below shows a comparison of the predicted bankfull width at the Temple Street dam removal site based on the equations presented above.

Table 2.2.2-1: Summary of Bankfull Width Estimates

Source	Description		Bankfull Width (ft)
USGS, 2013	Regional Regression Equation 1 – Simple Regression		29.85
	Regional Regression Equation 2 – Multiple Regression[4]	1:250,000 Scale DEM (low resolution, state, or regional assessments)	19.13
		10 m DEM (medium resolution, regional assessments)	25.73
		Surveyed Slope DS of Myrtle Street (site-specific assessment)	18.09
Bieger et. al, 2015	Regional Regression Equation 3 – New England		40.64
Alpha Survey Group, LLC (1/15/18)	Surveyed channel width approx. 150 feet downstream of Myrtle Street		26.72
Average of Calculated Bankfull Widths (ft)			**26.69**

The average bankfull width of the various estimates is 26.69 feet, which is similar to the on-site measurement of the bankfull width of the river measured approximately below Myrtle Street (26.72 feet). In addition, the banks at the Indian Head River Gage location as measured from Google Earth (2018 Aerial Imagery) are approximately 40 feet apart where the annual median flow is approximately 42 cfs. Based on the ratio of bankfull width to flow at the Indian Head River Gage and an annual median flow at Temple Street of 8 cfs the bankfull width would be approximately 8 feet. Therefore, we give greater deference to the actual field measurements and thus used a bankfull width of 26.7 feet as it would be more conservative for the purposes of sediment quantification. Using the predicted bankfull width of approximately 26.7 feet, the anticipated channel limits for potentially mobile sediment were delineated[5],[6]. Note that the calculated average bankfull width is similar to the channel width surveyed downstream of Myrtle Street.

Based on the data above the potentially mobile sediment volume assumes the minimum bankfull width in a dam removal scenario is a minimum of 26.7 feet and all of the sediment within that channel down to observed refusal elevations would mobilize. Based on the predicted bankfull width, sediment probing data, and proposed elevations of the grade control structures, the volume of sediment that could potentially mobilize from within the lower impoundment is approximately 200 CY and the overall sediment volume of sediment in the lower impoundment between transects T1 and T6 is approximately 15,100 CY.

It should be noted that the estimated volume of potentially mobile sediment is conservative. One of the project goals is to minimize the change in downstream water levels, so the proposed dam breach width

[4] In addition to drainage area, multiple regression equations for bankfull width depend on the slope. USGS StreamStats reports provide mean basin slopes based on digital elevation models (DEMs) at different scales. For this study, a sensitivity analysis was performed using the StreamStats slopes as well as the surveyed channel slope downstream of the Temple Street Dam.

[5] The bank stations were drawn based on field conditions observed during the sediment probing. If the observed bank width observed in the field exceeded the calculated bankfull width of 26.7 feet the observed stations were used. If the calculated bankfull width was wider than what was observed the bank stations were adjusted/widened based on the calculated bankfull width.

[6] It was assumed the anticipated stream channel would form along the thalweg points identified during the sediment depth probing.

will be limited to that necessary to decommission the structure, and a grade control feature will be installed in the breach opening to mitigate downstream impacts. Stream restoration designs will also include the use of large woody debris to stabilize sediment within the impoundment and help maintain flood storage capacity. Therefore, it is anticipated that upstream sediments will remain relatively stable and no significant headcutting will occur post-dam removal.

2.2.3 Sediment Quality Assessment

To characterize the quality of sediment impounded by a dam and inform management options, sediment samples are collected and analyzed for chemical and physical parameters in compliance with the Massachusetts 401 Water Quality Certification guidelines (314 CMR 9.00). Typically, for dam removal projects, samples are collected within the dam impoundment and at other locations in the river as follows:

- **Upstream of Dam Impoundment** – At least one sample is usually collected upstream of the area impounded by the dam to characterize sediment that is likely to mobilize during future storm events and be transported and deposited in the impoundment or transported downstream of the dam regardless of whether the dam is removed.

- **Mobile Sediment within Dam Impoundment** – Several samples are usually collected within the dam impoundment from sediment deposits that are expected to mobilize post-dam removal to characterize contaminant levels potentially present in sediment requiring either active or passive management.

- **Stable Sediment within Dam Impoundment** – At least one sample is usually collected within the dam impoundment from sediment deposits that are expected to stabilize as floodplain wetlands post-dam removal to characterize potential risks to human health from newly exposed sediment.

- **Downstream of Dam Impoundment** – At least one sample is usually collected downstream of the dam in depositional areas that would be expected to receive sediment mobilized from the impoundment post-dam removal to characterize potential ecological risks from pollutants that might be bound to or otherwise associated with the mobilized sediment. The finding of similar or higher pollutant levels downstream, for example, might lead to a conclusion of limited ecological risk from sediment with similar or lower contaminant levels moving downstream due to dam removal. It should be noted that the sediment management strategy discussed in Section 2.2.4 relies on active sediment management and removing potentially mobile sediment.

Sediment sampling for Temple Street Dam was conducted on March 17, 2022. A total of five samples were collected from the following locations:

- **Upstream of Dam Impoundment (US-1)** – One sample in the upper impoundment, approximately 1,300 feet upstream of the dam.

- **Mobile Sediment within Dam Impoundment (IMP-1 & IMP-2)** – Two samples within the lower impoundment along the potentially mobile edge of the sediment deposits, approximately 50 feet and 500 feet upstream of the dam respectively.

- **Stable Sediment within Dam Impoundment – (IMP-3)** One sample within the lower impoundment in a potentially immobile sediment deposit on river left, approximately 140 feet upstream of the dam.

- **Downstream of Dam Impoundment** (**DS-1) –** One sample downstream of the Myrtle Street crossing, approximately 270 feet downstream of the dam

The sampling locations are shown in the **Drawings** in **Appendix C**. Samples were collected with a stainless-steel hand core system outfitted with a Cellulose Acetate Butyrate (CAB) liner. The push core system was advanced up to five 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 Con-Test Analytical Laboratory, a Massachusetts-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)):

- Heavy metals (Arsenic, Cadmium, Chromium III and VI, Copper, Lead, Mercury, Nickel, and Zinc)
- PAHs
- PCBs (incl. congeners and extractable or total hydrocarbons (%))
- Organochlorine pesticides
- TOC
- Volatile Organic Compounds (VOCs)
- Percent water
- Grain size distribution (Sieve Nos. 4, 10, 40, 60, and 200)

Appendix F presents the findings of the sediment testing. Results are compared to screening criteria MacDonald's Threshold Effects Concentration (TEC) and Probable Effects Concentration (PEC) (MacDonald et al., 2000) as well as the Massachusetts Contingency Plan Method 1 Cleanup Standards for S-1 (soils) and GW-1 (groundwater).

In general, the sediment samples collected by Gomez and Sullivan show little contamination in the Lower Impoundment, upstream of the Lower Impoundment, or downstream of Myrtle Street. Concentrations of metals including arsenic, cadmium, lead, and mercury were detected in the lower impoundment exceeding TECs, but below PECs and MCP Method 1 Standards. Lead concentrations exceeding TECs but below PECs and MCP values were detected in the upstream sample. Pesticides including DDD, DDE, and DDT were detected at each sampling location exceeding PEC values but below MCP values. Only one Volatile Organic Compound (VOC), acetone, was detected. In the Lower Impoundment and downstream samples, acetone concentrations exceeded the MCP value but was below all Direct Contact (Method 2) human exposure threshold values. The impoundment samples also contained very low levels of total PCBs, below the TEC and MCP values. EPHs were also detected at each sampling location in concentrations well below MCP values.

There were several PAHs detected at each of the sediment sampling locations which exceeded PEC thresholds; however, only one of them, benzo(a)pyrene exceeded the MCP value. Benzo(a)pyrene exceeded the MCP threshold at each sampling location. The downstream sample exceeded the Direct Contact S-1 threshold for residential exposure. The maximum and mean concentrations within the impoundment exceeded the Direct Contact S-2 threshold but not the S-3 threshold. The upstream sample also exceeded the Direct Contact S-2 threshold for benzo(a)pyrene.

2.2.4 Sediment Management Approach

Sediment management options considered as part of the dam removal include:

1 A passive approach to "in-stream" management, and sediment redistribution over time;

2 An active approach utilizing mechanical dredging and channel re-construction upstream of the dam; and,
3 A combined approach where select locations are excavated to initiate a low flow channel for fish passage through high points observed in the lower impoundment. A portion of the excavated sediment would be re-used to fill in the voids of the proposed riffle features for fish passage. Other than these select few locations the rest of the approximately 30 cubic yards of potentially mobile sediment would be allowed to mobilize naturally.

For projects in clean, semi-rural locations, in-stream sediment management are generally recommended. Natural channel formation (compared to a constructed, man-made channel) is preferred and often more likely to result in a stream form which will be more dynamically stable, cost much less, involve less impacts to natural resources including wetlands. Mechanical dredging of all the sediment in the lower impoundment with heavy equipment would be costly and damage new potential floodplains and adjacent wetlands.

Based on the proposed grade control elevation at the Temple Street Dam location and sediment depth mapping conducted in December 2021 described in Section 4, the potentially mobile volume of sediment is 200 CY. Once the dam flashboards are removed and the impoundment has been lowered approximately 200 cubic yards of sediment will be removed behind the dam and at the upstream end of the lower impoundment.

Our sediment management plan is based on the following rationale:

- The dam removal will be implemented outside the most sensitive times of year for migratory fish species (4/15 – 7/15)
- Contaminant concentrations are similar upstream, within the impoundment, and downstream;
- Mechanical removal of the impoundment sediments would require unnecessary significant impacts to natural resources;
- In addition to attenuating flood flows is it anticipated that the large woody debris being installed are also expected to stabilize sediment;
- Cost effectiveness; and,
- Timing of the work and careful sequencing, along with construction oversight will ensure the proper implementation of this sediment management approach.

The sediment volumes described above, to be managed are already within the stream and do not include any upland sediment. Sedimentation from upland sources will be prevented using structural and non-structural Best Management Practices (BMPs) as shown/described in the 75% construction drawings.

This dam removal will benefit anadromous fish in terms of allowing fish to migrate past the Temple Street Dam location and opening up the habitat upstream of the dam to migratory fish and restoring natural ecological processes and sediment transport processes.

Actively removing the entire volume of sediment in the Lower Impoundment would require a much larger construction area for dewatering and managing the sediment prior to its onsite or offsite disposal. The limits of work and temporary impacts from this would greatly exceed those shown in the 75% drawings.

Due to the potential impacts of sediment aggradation on downstream flooding an active sediment approach is recommended. Of the 200 CY of potentially mobile sediment approximately 200 CY would be actively excavated and re-used on-site. Excavated sediment will be used to fill in the voids of two riffle features being proposed for fish passage and applied as loam during site restoration.

Dewatering Methods

The locations of the proposed dewatering of dredged material are shown in the 75% drawings in **Appendix C**. Most of the project impacts will be to LUW. The Lower and Upper Impoundments will be dewatered by approximately 2 feet after removing the dam and after installing the proposed riffle grade control features. After the impoundment has been lowered the large woody debris within the impoundments will be installed.

For water control a temporary supersack cofferdam system installed at the locations of the proposed riffle features is recommended. The cofferdam systems will divert flow through the dam and riffle construction areas. Only minimal maintenance pumping of runoff entering the riffle construction areas is anticipated. The discharge water is not expected to be contaminated.

Once the riffle feature at River Street downstream of the concrete dam is constructed the concrete outlet structure will be removed. Flows will be diverted through one half of the dam breach at a time as the riffle feature in the dewatered half is completed until the grade control grading is completed.

During construction, temporary erosion, sedimentation, water, and pollution controls will be utilized in accordance with BMP guidelines recommended by MassDEP. To prepare the site, vegetation will be preserved to the extent practicable. Erosion of proposed access routes (on existing paved and grassed areas) will be controlled by installing a stabilized construction entrance and gravel access roads with geotextile underlayment. For the installation of large woody debris in the impoundment the contractor shall use swamp mats as needed to access wetland areas as needed. Erosion and sedimentation due to stormwater runoff will be managed with approved measures such as silt socks or entrenched silt fences installed at the limits of all work/disturbances. Disturbed areas and stockpiles in upland areas will receive temporary seeding/ mulching/riprap as appropriate. Dust will be controlled as necessary. Any pump discharge (due to dewatering) will be directed into pumped filter bags or approved equivalent BMPs to capture fine sediments.

Dredging and dewatering activities will be timed appropriately per TOY restrictions and conducted in accordance with BMPs and applicable permit conditions to avoid and minimize adverse impacts on water quality, physical processes, marine productivity, and public health.

Dredging

<u>Dredging Quantity</u>

The proposed project will involve dredging of approximately 200 CY of sediment within a footprint of approximately 4,500 square feet.

<u>Dredging Alternatives</u>

The design process sought to avoid, minimize, and/or mitigate (in that order of preference) potential adverse impacts to land under water. Specific to dredging, no practicable alternatives were identified that could avoid related impacts while still meeting the project goals of restoring upstream fish passage and channel roughening in the impoundment. These impacts will be minimized and mitigated using BMPs as discussed previously. An alternatives analysis for the project as discussed below.

No Action

The no action alternative would involve keeping the existing dam in place. The dam requires maintenance occasionally by the Town. If the dam fails, the buried water line at River Street(which is only 2 feet below grade) could be compromised, causing significant additional costs to repair the damage. The wave of water from a dam failure could potentially present a risk to property owners downstream. The flashboards would continue to present a barrier to aquatic organism passage and create artificially impounded conditions upstream. Therefore, the no action alternative is not recommended.

Dam Removal and Passive (In-Stream) Sediment Management

The proposed dam removal would eliminate the need for maintenance by the Town and remove the dam as a fish passage barrier. The selected alternative includes a dam breach, the two riffle features and two beaver dam analogues and installing large woody debris in the floodplain. Based on the proposed elevations of 32.7 feet at the dam and 33 feet at the upstream end of the lower impoundment approximately 200 CY would mobilize downstream post-removal.

Due to the flat nature of the South River downstream of the Temple Street Dam sediment tends to build up and settle out. When sediment settles at structures it can reduce the hydraulic capacities of bridges, culverts, and storage behind dams. Based on concerns about downstream flooding allowing sediment to passively move downstream and build up in locations downstream is not recommended. Therefore, this option was not selected.

Dam Removal and Active Sediment Management

The proposed dam removal would include the same elements as those described in above in the Dam Removal and Passive Sediment Management approach except for the proposed sediment management strategy. To reduce the potential for sediment aggradation downstream of the project to impact hydraulic conditions and flooding worse active removal through excavation is the preferred alternative. This option would involve removing approximately 50 CY of sediment at the dam and approximately 120 CY of sediment from a low flow channel at the upstream end of the lower impoundment, leaving only 30 CY of sediment to mobilize downstream.

To reuse the sediment on-site approximately 70 CY of the sediment could be washed into the riffle features at the dam and downstream of River Street while the remaining 100 CY of excavated sediment could be re-used as loam prior to seeding along the tops of the dam and River Street embankments once the project is substantially complete during the restoration of the site.

This alternative is the preferred option as limits the potential for sediment buildup to contribute to flooding downstream and re-uses the sediment without having to haul it off site.

<u>Sediment Characterization</u>

The grain size analyses identified the upstream sediments as lean silt (United Soil Classification System (USCS) class “ML”) consisting of approximately 48.7% sand and approximately 51.2% fines and less than 1% gravel. The three impoundment samples were all classified as silty sand (SM) and consisted of 44.2% fines on average, 55.1% sands, and less than 0.7% gravel. The sample taken downstream of Myrtle Street was classified as well-graded sand with some silt (SW-SM) and had the smallest concentration of fines at approximately 8.7%, 86.3% sands, and approximately 5% gravel.

Disposal Site

The preferred approach for dredged sediment is to utilize approximately 50 120 CY of the low flow channel dredged sediment from the impoundment and approximately 50 CY of sediment just upstream of the dam on-site. The approach would utilize approximately 70 CY to fill in the voids in the two riffle features with fine material and 120 CY would be placed along the top of the embankments at River Street and the dam breach and at the edges of the riffle riprap above the water surface. Alternatively, there are brick remains adjacent to the western dam embankment. Sediment may be used to bury this structure. The location is provided in the **Drawings in Appendix C**.to wash into the proposed riffle features to fill in the voids in the placed cobble/gravel.

2.3 Wetland Delineation

A wetland delineation was conducted by LEC Environmental Consultants on December 16 and 17, 2021 from approximately 220 feet downstream of the dam at Myrtle Street to 680 feet upstream of Temple Street Dam. The survey limits were based on areas that could be impacted by dam removal including laydown areas, access routes, a reach below the dam and around the impoundment that could be impacted by lowered water levels. Wetland boundaries in the vicinity of the dam are shown in **Drawing 4** of **Appendix C**.

Wetland boundaries outside the survey area, as well as other regulated resource areas, were obtained from MassGIS. Regulated wetland resource areas, including Land Under Water (LUW), Bordering Vegetated Wetlands (BVW), Bordering Land Subject to Flooding (BLSF), and Riverfront Area (RFA), are depicted in **Figure 2.3-1 and Figure 2.3-2** in **Appendix A**. The 100-foot bank buffer and 100-foot bordering vegetated wetland (BVW) buffer zones are also delineated in **Drawings 3 and 4** of **Appendix C**.

2.4 Hydrologic & Hydraulic Analysis

A hydrologic analysis was conducted to develop flows for use in a hydraulic model to estimate water surface profiles, velocities, depths, and other parameters under existing and proposed conditions.

2.4.1 Hydrologic Analysis

Two different types of flows are useful for the analysis and design of a dam removal project: 1) period based (average daily) flows representative of low/normal flow periods, and 2) event-based (peak discharge) flows. Average daily flows are used to evaluate the potential impact of dam removal on water-level dependent resources; assess fish passage and inform the design for the care and diversion of water during construction. Event-based flows are used to evaluate the impact of dam removal on infrastructure and peak water surface elevations (WSEs); and determine the required extent of spillway removal.

Period Based (Average Daily) Flows

In previous studies the hydrologic analysis was limited to one non-peak flow, a "Sunny Day" or baseflow value of 2 cubic feet per second per square mile of drainage area (cfsm). This flow may have been based on documentation regarding the New England Flow Policy (USFWS, 2002)[7]. This value is not site specific; therefore, to build upon the range of period-based flows assessed in previous studies Gomez and Sullivan

[7] Although the source of the baseflow estimate is not explicitly stated in the 2018 or 2020 Pare reports, New England Flow Policy documentation was provided as a reference, and this policy documentation estimates an average annual flow of approximately 2 cfsm for the New England Region.

compared the stream flows measured at two nearby USGS gages as there are no active USGS streamflow gages on the South River.

Table 2.4.1-1 summarizes key basin parameters for each of these locations. The parameters are generally from StreamStats, except for the drainage area, which is reported on the USGS website. The drainage area, reported in square miles (mi^2), for Temple Street is from the existing HydroCAD hydrologic model.

Table 2.4.1-1: Basin Parameters

Basin Parameter	Temple Street Dam	USGS Gage 01105730 (Indian Head River at Hanover, MA)	USGS Gage 01105870 (Jones River at Kingston, MA)
Drainage Area (mi^2)	5.9	30.3	19.8
ELEV (ft)	74.5	101	80.2
STOR (%)	24.84	20.33	26.51
Basin Slope (%)[1]	3.375	2.803	4.043
Impervious (%)	5.01	17.8	5.26
Developed (%)	23.6	50.7	19.8
Forested (%)	59.24	26.72	63.54

Notes:

1. Streamstats reports the storage from two separate sources. This table displays BSLDEM10M.

Despite the similarity in results between the two gages and the similarity in basin metrics between the Jones River gage and the Temple Street Dam, the Indian Head River gage flows were used, due to uncertainties due to the impacts of regulation on flows at the Jones River gage[8], and the wider range of design flows provided by the Indian Head River gage. To determine the range of daily flows (Q) for evaluation at the Temple Street Dam site the daily flows measured at the Indian Head River gage were prorated by the ratio of drainage areas (DA) as shown below:

Equation 4 (Avg Daily Flows):

$$Q_{Temple\ St(cfs)} = Q_{Indian\ Head\ St(cfs)} x \ \frac{DA\ @\ Temple\ St\ (5.9\ mi2)}{DA\ @\ Indian\ Head\ River\ \ (30.3\ mi2)}$$

Table 2.4.1-2 shows the calculated minimum, maximum, median and mean flows calculated for the period of record at the Temple Street Dam. An annual and monthly average daily flow duration curve based on the analysis are provided in **Figure 2.4.1-1** through **Figure 2.4.1-7** in **Appendix A**.

Table 2.4.1-2: Estimated Flow Statistics for South River at Temple Street Dam

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Min	2	2	3	3	2	1	0	0	0	0	1	1	0
Max	154	158	245	128	154	184	125	107	123	230	97	185	245
Median	12	13	17	15	10	6	3	2	2	4	8	11	8
Mean	16	18	24	20	13	10	5	4	4	8	12	16	12

All units in cubic feet per second (cfs). Period of Record: July 8, 1966 to December 12, 2021

[8] The USGS notes that flow at the Jones River gage may be affected by upstream regulation, wastage from Silver Lake, ground water that enters from or moves into adjacent basins, and occasional backwater from tidal surge.

Table 2.4.1-3 provides flows anticipated during fish passage, construction, and duck hunting seasons[9].

Table 2.4.1-3: Estimated Fish Migration, Construction, Duck Hunting Season Flow Statistics for South River at Temple Street Dam

	Upstream Migration (4/15-7/15)	Downstream Migration (7/1-12/31)	Construction Period (7/15-10/31)	Duck Hunting Season (10/1-11/26)
Min	0.9	0.4	0.3	0.7
Max	184	230	230	230
Median	8	4	3	5
Mean	11	8	5	9

All units in cubic feet per second (cfs). Period of Record: July 8, 1966 to December 12, 2021

Event-Based Flows

Several sources were considered in the assessment of event-based flows for this project. Gomez and Sullivan compared three sources of data for peak discharges including the existing hydrologic model developed for the project by Pare in 2018, regional regression equations, and peak frequency analyses tools using available streamflow data. A more detailed summary of the hydrology review is provided in **Appendix D**.

Existing Hydrologic Model

The model was developed in 2018 using the HydroCAD Version 10.1-3a software, which applies rainfall depths and distribution curves over 14 drainage areas within the South River watershed. The model evaluates three annual chance exceedance (ACE) events under current and projected climate conditions. The three events evaluated were the 5-year (20% ACE), 25-year (4% ACE) and 100-year (1% ACE) events.

The National Oceanic and Atmospheric Administration (NOAA) Atlas 14 (NOAA, 2019) was used to identify the 24-hour rainfall depths for current climate conditions, while the Resilient Massachusetts Action Team (RMAT) Climate Resilience Design Standards Tool was used to develop 24-hour precipitation storm depths, considering a Teir 2 methodology and mid-century planning horizon, for projected climate conditions. **Table 2.4.1-4** provides a summary of the precipitation depths used for this study.

Table 2.4.1-4: 24-hour Total Precipitation Depths for Duxbury, MA

Climate Condition	Total Precipitation Depth (in)		
	5-Year	25-Year	100-Year
Current	4.28	6.04	7.61
Projected	4.62	6.52	8.45

Regional Regression Equations

Regional regression equations utilize different basin parameters for a given location (e.g., drainage area, elevation, surface water storage area) to estimate the anticipated peak flow for various recurrence interval events (e.g., 10% AEP). The development of these equations (e.g., basin parameters, exponents)

[9] The current impoundment/surrounding area is open to duck hunting; therefore, impacts to the impoundment water surface during duck hunting season were evaluated.

are based on statistical analyses of the magnitude and frequency of flows observed at stream gages within a given region. These equations allow for the estimation of the magnitude and frequency of flows for locations which do not have stream gages. The USGS published regional regression equations for the state of Massachusetts (Zarriello, 2017), which depend on three basin parameters to estimate flows: drainage area (DA), mean elevation of the basin (ELEV), and total storage as defined as the percent of wetlands and open water for the basin (STOR)[10].

The USGS has developed a webtool called StreamStats to implement regional regression equations for most states. This study (GSE 2022) utilized the StreamStats Version 4.6.2, to compute basin parameters and applicable flow estimates at Temple Street Dam (42.07950, -70.74543).

Peak Frequency Analysis

A peak flow frequency analysis performs a statistical analysis on a series of annual instantaneous maximum flows recorded for a given location, along with other available peak flow information, to estimate the frequency and magnitude of flows for that location. A Bulletin 17C statistical analysis was performed using the PeakFQ Version 7.3 software (USGS, 2019c).

Bulletin 17C recommends at least 10 years of annual maximum peak flow data. Since there are no active USGS streamflow gages on the South River the Indian Head River gage was used to determine period-based flows at the dam. For this study the peak frequency analysis utilized over 50 consecutive years of annual maximum peak flow data. The PeakFQ results were prorated based on a ratio of drainage area to estimate peak flows at the Temple Street Dam.

Climate Change Projections

In the future, the occurrence of heavy precipitation events is projected to increase, with a slight increase in the number of dry days (Easterling et al., 2017). In order to consider climate change in the design, precipitation multipliers were reviewed, and a separate set of model runs were performed using projected future flows for the South River. Recommendations for future flow estimation from Massachusetts and the surrounding states were considered in this study. Massachusetts Climate Change Adaptation Report (MACCC n.d.) and New Hampshire (UNH 2019) provided guidance on precipitation multipliers to be applied when considering climate change, while New York (NYSDEC 2020) provided guidance on flow multipliers to be applied when considering climate change. **Table 2.4.1-5** summarizes the proposed precipitation multipliers. Since the Massachusetts recommendations bound the range of recommendations, these multipliers were applied to the precipitations used in the existing HydroCAD model. The resulting flows are presented in **Table 2.4.1-6**, along with the flows from the existing HydroCAD model with New York's proposed flow multipliers applied. New York's recommendations are similar to Massachusetts recommendations for Mid-century 2050/7070, while Massachusetts recommendations for Late-Century are much higher.

[10] The effective Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for Plymouth County, revised July 6, 2021, utilized the Massachusetts regional regression equations, to develop peak flows for the South River. However, the FIS does not report the locations or flows developed for this effort.

Table 2.4.1-5: Summary of Precipitation Multipliers for Climate Change Consideration

State	Scenario	Precipitation Increase (%)		
		20% AEP	4% AEP	1% AEP
Massachusetts	Mid-Century (2030/2050)	8%	8%	11%
New Hampshire	High to Medium Flood Risk Tolerance	15%	15%	15%
New Hampshire	Low to Very Low Flood Risk Tolerance	>15%	>15%	>15%
Massachusetts	Late Century (2070/2090)	20%	20%	27%

Table 2.4.1-6: Summary of Peak Flows at Temple Street Dam with Climate Change Consideration

Annual Exceedance Probability	Recurrence Interval (years)	Peak Discharge (cfs)			
		Current (2020 Pare)	Massachusetts: Mid-Century 2050/2070	Massachusetts: Late Century 2070/2090	New York: 2025-2100
20%	5	160	193	238	192
4%	25	320	366	441	384
1%	100	480	584	729	576

The period-based and event-based flows used for simulation in the hydraulic model are compiled in **Table 2.4.1-7** below.

Table 2.4.1-7: Flows Utilized in in HEC-RAS Model of Temple Street Dam Removal

Type	Rationale	Name	Description	Flow (cfs)	Source
Average Daily Flows	To evaluate the impacts of dam removal under normal flows, fish passage, and compare model results to measured flows	Normal Flow	Mean annual flow	8	Flow duration analysis, based on prorating Flows by Drainage Area from the Indian Head River
		Fish Passage Flows	95% exceedance Downstream Migration flow	1	
			5% exceedance Upstream Migration flow	33	
	Evaluate average flow capacity required for water controls during construction.	Median Construction Flow	Median Flow 7/15 - 10/31	3	
	Evaluate impacts on Duck Hunting in the Temple Street Impoundment		Median Duck Hunting Season 10/1 – 11/26[11]	5	
Current and Projected Climate Change Peak Flows	Evaluate impacts on natural resources and high flows during construction.	Current 2-Year Flow	50% ACE Flow	119	HydroCAD Model
	To evaluate impacts of dam removal on infrastructure, inform the design, and determine the required extent of spillway removal to no longer constrict flood flows	Current 5-Year Flow	20% ACE flow	160	
		Current 25-Year Flow	4% ACE flow	320	
		Current 100-Year Flow	1% ACE flow	480	
		Projected 5-Year Flow	20% ACE flow	193	
		Projected 25-Year Flow	4% ACE flow	366	
		Projected100-Year Flow	1% ACE flow	584	

Note: Flows represent peak values at Temple Street Dam. The flood flows used in HEC-RAS were time varying hydrographs and included flow inputs at other locations within the South River watershed.

[11] The project is in the Central hunting zone as classified by the Massachusetts Division of Fisheries and Wildlife (https://www.mass.gov/doc/2022-2023-migratory-game-bird-regulations/download). Regular duck hunting season is listed as from October 10 – November 26 while the falconry duck hunting is October 1 – February 2. For the purposes of this study the October 1 – November 26 timeframe was used for the analysis as it is anticipated that flows during this hunting season timeframe would be the lowest and be impacted the most.

2.4.2 Hydraulic Analysis

An approximate hydraulic model of the South River was developed for the Plymouth County FIS dated July 6, 2021, using the United States Army Corps of Engineers' (USACE's) Hydraulic Engineering Center River Analysis System (HEC-RAS) program. Although useful for comparison purposes, the FIS model does not include significant hydraulic control features such as bridges. A detailed two-dimensional (2D) hydraulic model of the South River was developed for the Temple Street Dam Conceptual Design Alternatives (Inter-Fluve, 2021) using the USACE's HEC-RAS program. Although useful for comparing downstream impacts, a more refined model was necessary for the dam removal design.

Hydraulic Model Development

The existing 2D hydraulic model, herein referred to as the Large-Scale Model, extends from approximately 1.5 miles upstream of Temple Street Dam to approximately 4.5 miles downstream of Temple Street Dam. In addition to utilizing the Large-Scale Model, a new 2D hydraulic model was developed for this analysis using the USACE's HEC-RAS Version 6.1, herein referred to as the Refined Model. The Refined Model extends from approximately 1,500 feet upstream of Temple Street Dam to approximately 370 feet downstream of Temple Street Dam.

Model Inputs

Model geometry was georeferenced using HEC-RAS. The model was developed using the following input data:

- Existing survey data
- Topographic/bathymetric survey data
- Sediment probing data
- LiDAR elevation data for upland topography in the areas not covered by the above
- Bridge plans
- Field observations of channel, bank, and floodplain substrates/cover types

Roughness coefficients (Manning's 'n' values) used in the model for existing conditions ranged from 0.03 to 0.035 in the channel and 0.015 to 0.16 on the banks and floodplains.

A downstream "boundary" condition is needed for hydraulic models. A rating curve, a relationship between flow and water surface elevation, was used at the downstream end of the model for the Large-Scale Model, and the normal depth method was used at the downstream end of the model for the Refined Model.

Existing Conditions

A range of flows were run in each model to simulate water surface elevations, depths, and velocities under existing conditions.

Proposed Conditions

The existing conditions models were modified to reflect proposed dam removal conditions. To simulate the removal of the spillway, the dam structure was replaced in the model with the proposed breach section shown in **Section C** of **Drawing 8** in **Appendix C**. The width of the dam breach was selected considering resulting changes in water levels at select locations downstream of the dam under several unsteady flow conditions. Further, the invert of the dam breach was guided by water depths and velocities conducive to upstream and downstream fish passage and avoiding impacts to the existing 12” ductile iron water line approximately 2 feet below the streambed at River Street. The stream thalweg was modified within the lower impoundment to reflect post-construction conditions.

Roughness coefficients were also adjusted to reflect proposed conditions including increased roughness within areas of large woody debris and beaver dam analog installation. As part of the conceptual design, roughness coefficients were increased to 0.16 within all inundation areas upstream of Myrtle Street. The current proposed conditions adjusted roughness coefficients in a more targeted manner based on field reconnaissance during different seasons (winter/early spring/summer).

The limits of large woody debris installation and the beaver dam analogues in **Appendix C** represent the extent of changes to existing roughness coefficients, and the magnitude of the proposed roughness coefficient. This model assumes does not assume growth of any new vegetation.

Manning coefficients were adjusted based on the values presented in Tables 2 and 3 in the Guide for Manning’s Roughness Coefficients for Natural Channels and Floodplains published by the United States Geological Survey in 1989 (Arcement and Schneider 1989). The roughness coefficients are based on channel/floodplain geometry, number of obstructions, amount of vegetation and channel substrate. The guide also includes pictures correlating to specific coefficient values for reference.

Model Outputs

Model outputs were analyzed to evaluate potential impacts of dam removal on fish passage and infrastructure, as described in the following sections. Pertinent parameters considered during these analyses include WSE, depth, and velocity.

2.5 Fish Passage Analysis

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. These characteristics vary among the target species. Species being considered for this project include herring (e.g., alewife, blueback herring, American shad), American eel, and sea lamprey. Proposed channel flow conditions were evaluated for the ability to support safe, timely, and effective fish passage. Considerations for seasonal flow magnitudes were addressed in the hydrologic analysis discussed above.

2.5.1 Existing Conditions

Within the vicinity of the Temple Street Dam there are two primary obstacles in terms of migration including the Temple Street Dam itself and the River Street just downstream. The flashboards at the dam prevent fish from entering the current impoundment and there is a drop in the riprap at the River Street crossing of approximately 2 feet which can create high velocities and difficult swimming conditions for fish migrating upstream. A profile showing the drops in the water surface under existing conditions at both of these locations is provided in **Figure 2.5.1-1** in **Appendix E**.

2.5.2 Design Criteria

The following design criteria were utilized to address fish passage designs at the dam and River Street.

Fish Migration Flows

Hydraulic analyses of the fish passage design were based on flows between 1 cfs and 33 cfs which represent the 95% and 5% exceedance flows during the upstream fish passage migration season as described in **Section 2.4.1**.

Water 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[12]. Since American shad is the largest of the target species in terms of body thickness, its dimensions serve as a conservative surrogate for all the target species. Assuming an average body thickness to total body length ratio of 30% and an adult body length of 14 inches, body thickness would be about 4 inches, and the minimum depth required for passage would be about 6 to 8 inches, which when rounded to the nearest 0.1 feet (ft) becomes 0.5 to 0.7 ft. As such, maps showing the anticipated water depths in the proposed design (**Figure 2.5.2-1 and Figure 2.5.2-2)** in **Appendix E** use three bins [i.e., >0.7 ft (blue), 0.5 - 0.7 ft (yellow), and <0.5 ft (red)].

Water Velocities

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 long periods of time; employed for general movement and migration.
- Sustained speed: A speed that can be maintained for minutes; employed for passage through difficult areas.
- Burst speed: A single effort that is not sustainable; employed for feeding or escape purposes.

Table 2.5.2-1 below provides a summary of the various swimming speeds in feet per second (ft/s) of the target fish species for the upstream migrant life stage (i.e., adults). It should be noted that the Casto-Santos (2005) reference includes information for Alewife, Blueback herring, and American shad. The Casto-Santos (2005) reference generally reports swimming speeds in terms of body length per second (BL/s). Gomez and Sullivan has developed its own set of swimming speeds from the Casto-Santos (2005) reference, in which it took the range of swimming speeds (BL/s) provided in Figure 4 of the reference and multiplied them by the range of mean fork lengths (a surrogate for body length), converted to feet. The resulting swim speeds are generally higher than other two references.

[12] Brad Chase, Division of Marine Fisheries, personal communication, 2014. Federal fish passage design criteria recommend 2 times body thickness (USFWS, 2019).

Table 2.5.2-1: Summary of Swimming Speeds for Target Species

Family	Species	Swimming Speed (ft/s)[1]			Source
		Cruising	Sustained	Burst	
Herring	Alewife	-	-	6.0	(Turek, Haro and Towler, 2016)
		0 - 3	3 - 5	5 - 7	(Bell, 1991)
		-	-	11.5 - 12.5	(Dow, 1962)
		-	-	4 - 16	(Castro-Santos, 2005)
	Blueback herring	-	-	6.0	(Turek, Haro and Towler, 2016)
		0 - 3	3 - 5	5 - 7	(Bell, 1991)
		-	-	5 - 18	(Castro-Santos, 2005)
	American Shad	-	-	8.25	(Turek, Haro and Towler, 2016)
		0 - 3	3 - 7	8 - 13.5	(Bell, 1991)
		-	4 - 10	10 - 22	(Castro-Santos, 2005)
Eel	American Eel	-	0.25 - 0.5	-	(Bell, 1991)
Lamprey	Sea Lamprey	-	-	6.0	(Turek, Haro and Towler, 2016)
		0 - 1	1 - 3	3 - 7	(Bell, 1991)

Notes

Swimming speeds are reported for the upstream migrant life stage (i.e., adults)

The most important swimming speed for fish passage considerations is sustained speed. Eel and lamprey generally have lower sustained swimming speeds than those of the herring family target species, but they exhibit climbing and/or attachment behaviors that may help them navigate obstructions that are impassable to herring. The current dam precludes the upstream passage of all fish except American eel. Of the three herring 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 2.5.2-1**, a maximum water velocity of 5 ft/s to 7 ft/s was selected as the appropriate criteria to ensure that most target species should be able to navigate barriers using either sustained or burst speeds. As such, Gomez and Sullivan proposed to present velocity figures using three bins [i.e., <5 ft/s (blue), 5 - 7 ft/s (yellow), and >7 ft/s (red)], and depth figures (**Figure 2.5.2-3 and Figure 2.5.2-4)** in **Appendix E** using three bins [i.e., , 7 ft (red), 5 – 7 ft (yellows) and <5 ft (blue)].

2.5.3 Design Assessment

The depth and velocity resulting from proposed conditions was reviewed for scenarios representing the full range of design flows (the 95% exceedance flow and the 5% exceedance flow) considering typical upstream and downstream migration seasons within the extents of the proposed work area and the average depth was generally more than 0.7 ft. However, under low flow conditions (e.g., 1 cfs and 2 cfs), the channel width having a depth of 0.7 ft is very narrow throughout the rock riffle area. Within the extents of the proposed work area the average velocity was less than 5 ft/s. While velocities exceeded the 1 ft/s criteria for American Eel, this species exhibits climbing and/or attachment behavior helping them navigate high velocity areas. Water depth maps and water surface extents from the hydraulic model in the lower impoundment in the anticipated work area are mapped and in **(Figures 2.5.3-1 through Figure 2.5.3-9) Appendix E**.

2.6 Infrastructure Analysis

The infrastructure analysis included evaluation of existing information, evaluation of the data collected, and observations made during the site inspection, identification of any potential hydraulic impacts to infrastructure due to dam removal, and recommendations for design approaches and mitigation alternatives as needed.

2.6.1 Potential Impacts & Recommendations on Existing Infrastructure

The location of most of these features is provided in **Figure 1.2-1** of **Appendix A**. Representative photographs are provided in **Appendix B**.

River Street

River Street is an abandoned road approximately 200 feet downstream of the Temple Street Dam. Based on 2007 engineering drawings from Amory Engineers, P.C., there is a 12” ductile iron waterline crossing underneath the stream. Sheet 2 of 4 indicates there are two layers of cover over the water line including approximately 12” of riprap stone (M2.02.3) on top of approximately 12” of ¾” crushed stone on top of the pipe. Based on the potential location of this water line, an exclusion zone is proposed where no equipment shall either operate or travel over the water line as shown on the **Drawings** in **Appendix C**.

Rock fill is proposed downstream of the water line crossing to build up the proposed riffle feature just downstream of River Street. No excavation is being proposed near the location of the water line.

The proposed water surface elevations at River Street (see **Table 2.6.1-1**) increase and as a result the velocities decrease for storm events relative to existing conditions due to the additional fill being placed. **Figure 2.6.1-1 through Figure 2.6.1-6** show peak water surface elevations and velocities at this location and are provided in **Appendix E**.

Table 2.6.1-1: Hydraulics at River Street (Existing vs Proposed)

Recurrence Interval	Water Surface Elevation (ft)		Mean Channel Velocity (fps)	
	Existing	Proposed	Existing	Proposed
20%	33.75	34.65	5.4	4.2
4%	34.85	35.6	6.0	4.4
1%	36.5	36.7	6.2	4.4

The streambed designs for the riffle features at River Street and the dam location are described in **Section 4**[13].

Myrtle Street

Myrtle Street is an active roadway carried by a concrete box culvert. Peak water surface elevations and velocities were examined 10 feet upstream of the crossing to determine if there were any hydraulic impacts from the proposed riffle features and the dam removal. The water surface elevations at this location (see **Table 2.6.1-2**) were generally similar for existing and proposed conditions varying by less than a foot and velocities varying between 0 – 0.2 feet per second. **Figure 2.6.1-7 through Figure 2.6.1-12** showing peak water surface elevations and velocities at this location are provided in **Appendix E**.

[13]A scour analysis is recommended during final design at the River Street location and the findings should be compared to the findings from the streambed design in described in Section 4.

Table 2.6.1-2: Hydraulics at Myrtle Street (Existing vs Proposed)

Recurrence Interval	Water Surface Elevation (ft)		Velocity (fps)	
	Existing	Proposed	Existing	Proposed
20%	32.85	33.4	0.2	0.25
4%	34.55	34.8	1.6	1.7
1%	36.4	36.35	1.6	1.7

Downstream Residential Structures

During the development of the conceptual design three residential structures were identified as potentially being affected by changes to Temple Street Dam, as follows:

- 228 Old Ocean Street (Barn Structure)
- 108 Cross Street (Garage Structure)
- 60 Cross Street (Primary Residence Structure)

Tables 2.6.1-3 and 2.6.1-4 below summarize the estimated change in WSE at the three downstream residential structures for certain storm events considering current and projected conditions. Freeboard, as reported in these tables, is the difference between the peak WSE and finished floor elevation (FFE). The FFE for each structure was determined by a site survey performed in April 2021. A freeboard with a positive value indicates that the WSE is below the FFE, while a freeboard with a negative value indicates that the WSE is above the FFE. A positive change in water surface elevation indicates that the proposed conditions have caused an increase in the WSE relative to existing conditions, while a negative change in water surface elevation indicates that the proposed conditions have caused a decrease in the WSE relative to existing conditions.

Table 2.6.1-3: Hydraulic Model Results at Select Residential Structures under Current Climate Conditions

Model Condition	Parameter	Current 5-Year Storm Event (20% ACE)			Current 25-Year Storm Event (4% ACE)			Current 100-Year Storm Event (1% ACE)		
		229 Old Ocean Street	108 Cross Street	60 Cross Street	229 Old Ocean Street	108 Cross Street	60 Cross Street	229 Old Ocean Street	108 Cross Street	60 Cross Street
FFE		**28.63**	**17.05**	**13.06**	**28.63**	**17.05**	**13.06**	**28.63**	**17.05**	**13.06**
Existing Conditions	Peak WSE (NAVD88)	28.11	13.62	12.75	28.73	14.05	13.21	29.37	14.86	13.61
	Freeboard (ft)	0.52	3.43	0.31	-0.10	3.00	-0.15	-0.74	2.19	-0.55
Proposed Conditions	Peak WSE (NAVD88)	28.27	13.72	12.78	28.81	14.13	13.23	29.35	14.80	13.62
	Freeboard (ft)	0.36	3.33	0.28	-0.18	2.92	-0.17	-0.72	2.25	-0.56
WSEL Change due to Proposed Conditions (ft)		0.16	0.1	0.03	0.08	0.08	0.02	-0.02	-0.06	0.01

Table 2.6.1-4: Hydraulic Model Results at Select Residential Structures under Projected Climate Conditions

Model Condition	Parameter	Projected 5-Year Storm Event (20% ACE)			Projected 25-Year Storm Event (4% ACE)			Projected 100-Year Storm Event (1% ACE)		
		229 Old Ocean Street	108 Cross Street	60 Cross Street	229 Old Ocean Street	108 Cross Street	60 Cross Street	229 Old Ocean Street	108 Cross Street	60 Cross Street
FFE		**28.63**	**17.05**	**13.06**	**28.63**	**17.05**	**13.06**	**28.63**	**17.05**	**13.06**
Existing Conditions	Peak WSEL (NAVD88)	28.22	13.71	12.84	28.78	14.13	13.33	29.65	15.57	14.12
	Freeboard (ft)	0.41	3.34	0.22	-0.15	2.92	-0.27	-1.02	1.48	-1.06
Proposed Conditions	Peak WSEL (NAVD88)	28.39	13.81	12.86	28.85	14.17	13.34	29.62	15.49	14.04
	Freeboard (ft)	0.24	3.24	0.2	-0.22	2.88	-0.28	-0.99	1.56	-0.98
WSEL Change due to Proposed Conditions (ft)		0.17	0.1	0.02	0.07	0.04	0.01	-0.03	-0.08	-0.08

The last row of Tables 2.6.1-3 and 2.6.1-4 show that under proposed conditions, the hydraulic modeling results show a slight increase in the WSEL at the three properties under the 5-year and 25-year storm events and essentially no change under the 100-year flood. Based on historic flow conditions (current climate conditions), the increase in the WSEL at the properties with the dam removed ranges from 0.02 to 0.16 ft or 0.2 to 1.9 inches based on hydraulic modeling.

This suggests that the proposed conditions result in essentially no increased flood risk at these structures. Water surface extents results are provided as **Figure 2.6.1-13 through Figure 2.6.1-15** in **Appendix E**.

2.6.2 FEMA Modeling Requirements

A FEMA flood insurance study presents the flood risk data along watercourses including lakes, and coastal flood hazard areas. These studies provide water depth and water extents information for various flows with varying levels of detail. There are specific rules regulations for building and adding fill in areas within the flood mapping zones. The two primary zones are Zone A and Zone AE. In Zone AE[14] detailed hydraulic modeling is performed to calculate estimated water surface elevations along a watercourse called base flood elevations (BFEs). This method involves gathering survey data throughout the reach being studied and is typically modeled in HEC-RAS or another approved model. For Zone A, approximate zones, inundation mapping water surface elevations are based on contour maps or other local water surface elevation data.

The current effective FEMA FIS for Plymouth County provides approximate zone (i.e., Zone A) inundation mapping for the South River as shown in **Figure 2.6.2-1 in Appendix A**. This means that no BFEs were developed. Based on our preliminary inquiries with FEMA, it appears that the decision to request a LOMR for a project in an approximate zone rests solely with the affected community(s). One potential reason to request a LOMR when it is not required would be to attempt to remove properties/structures from the

[14] Detailed modeling also results in special zones called floodways located within Zone AE which are areas where development, including fill placement is prohibited as it would contribute to exacerbating flood conditions. Since the project is in Zone A there is no floodway.

SFHA if the proposed project would reduce BFEs in the area. Since the FEMA uses steady state models, water levels would not be impacted at any locations downstream of the project area. Further, based on the effective FIRM, it does not appear that any structures are currently located in the SFHA in the proposed project's upstream area of potential effect (APE). Therefore, pursuing a LOMR is not recommended as it would not make financial sense to do so.

3 ALTERNATIVES ANALYSIS

3.1 Design Constraints & Considerations

During the data collection and analysis phase, the following key design constraints were identified.

<u>Hydraulic Impacts to Infrastructure</u>

The dam breach was sized such that impacts to infrastructure along the South River (i.e. houses, road crossings, and utilities) and maximize the flood storage in the impoundment to the extent practical while restoring natural ecological processes. The model results in Section 2.6.1 above indicate that water surface increases at the homes identified in this study would be limited to 0.02 – 0.16 ft. While the riffle feature at River Street will result in lower velocities through the River Street and Myrtle Street embankments.

There is no infrastructure along the impoundment therefore no impacts upstream are anticipated.

<u>Town of Duxbury 12” Ductile Iron Waterline</u>

As discussed in Section 2.6.1 there is an active 12” diameter water line buried approximately 2 feet below the streambed. Therefore, it was assumed that any fish passage features required to facilitate fish passage by River Street would need to be provided so as to not disturb or endanger the water line. It was assumed that a work exclusion zone in the vicinity of the water line and limitation of where excavation can occur is required.

Design Considerations

The following design considerations were used to evaluate the various alternatives and select the preferred alternative, as discussed in **Section 3.2** below.

- Site/Design Constraints
- Ease of Construction
- Potential for Erosion, Head-Cutting, and Stream Channel Adjustment
- Storm Flow Conveyance / Climate Change Resilience
- Geomorphic Compatibility
- Impacts to Wetlands
- Potential to Affect Property or Infrastructure
- Regulatory Review Requirements
- Cost

3.2 Alternatives Analysis

No Action

The no action alternative would involve keeping the existing dam in place. The dam requires maintenance occasionally by the Town. If the dam fails, the buried water line at River Street(which is only 2 feet below grade) could be compromised, causing significant additional costs to repair the damage. The wave of water from a dam failure could potentially present a risk to property owners downstream. The flashboards would continue to present a barrier to aquatic organism passage and create artificially impounded conditions upstream. Therefore, the no action alternative is not recommended.

Alternative 1 –Dam Removal with one continuous riffle between River Street and the Dam, no addition of large woody debris or roughness elements or beaver dam analogues

This alternative would include the removal of the entire concrete outlet structure including the training walls and apron. The breach width and invert would be sized to avoid increasing upstream water surface elevations during peak discharges downstream. A proposed riffle feature would be constructed allowing fish passage beyond the dam from downstream of River Street to the dam location. This single riffle alternative would require excavating the fill at River Street and relocating or burying the active 12" ductile iron water line at River Street which could be costly and potentially impact nearby property owners. The length and slope of the riffle feature would also likely require a resting pool for migratory fish.

This alternative would not include roughness elements or beaver dam analogues in the impoundment. Without these roughness elements in the impoundment, the hydraulic modeling showed that infrastructure downstream could potentially be inundated during peak flow events.

Due to the concerns with having to relocate the existing water line and the potential flooding of downstream structures, this alternative was not selected.

Alternative 2 –Dam Removal with one continuous riffle between River Street and the Dam, no addition of large woody debris or roughness elements or beaver dam analogues

This alternative was the same as Alternative 1 with the exception that a step pool weir nature-like fishway constructed of stone would be installed at River Street and the dam location instead of a riffle/rock ramp. This option would involve installing a set of boulders across the stream, tightly together, with a gap in them somewhere for low flows to pass through.

While step pools generally require less earthwork/fill than riffle alternatives, sizing the breach or low flow notches in them so they perform well under various fish passage flows is difficult. Due to the low flows at the site it's not possible to match the criteria for nature like fishways (Turek, J., A. Haro, and B. Towler 2016) for the low flow weir opening notch flow depth and widths for the target species.

Additionally, gravels and sands could be used to fill in the gaps in the large stones forming the weirs but during low flows water seeping through those gaps would become problematic as flow may not flow over the low flow notch. The stones used to construct these weirs are also typically large and trying to enact any type of adaptive management for fish passage in terms of re-arranging the stones/adjusting the weirs once construction equipment leaves the site could be difficult.

Due to the concerns about lack of adaptive management potential and the incompatibility with fish passage criteria with low flows at this site this alternative was not selected.

Alternative 3 –Dam Removal with two riffle features at River Street and the Dam, constructing beaver dam analogues, and adding large woody debris throughout the entire Lower and Upper Impoundments, and passive sediment release downstream

This alternative would include the removal of the entire concrete outlet structure including the training walls and apron similar to Alternative 1. The breach would be sized to minimize hydraulic impacts to downstream infrastructure; however, to address fish passage two separate riffles would be constructed-one just downstream of River Street and one at the dam location. This fish passage strategy would eliminate the need for excavating in the vicinity of the active 12" ductile iron water line at River Street. Stone placed for the riffle at River Street would provide additional scour protection for the water line than is there now as there is a two-foot drop in grade just downstream of the crossing. In terms of fish passage, the design would allow River Street to provide a resting pool just upstream of it during higher migration flows prior to swimming through the dam breach.

In this alternative two beaver dam analogues would be included, one along the east side of the Lower Impoundment and one at the southern end of the Lower Impoundment. The beaver dam analogues would help retain flows up to the 1% recurrence interval event. This alternative would also include the addition of roughness elements throughout the extent of the impoundments (Upper and Lower) across approximately 147 acres of wetlands.

This alternative included allowing up to 200 CY of sediment mobilize downstream based on the projected headcut line through the impoundment and sediment probing transects.

Due to concerns over sediment aggradation at downstream infrastructure and reduced hydraulic capacities downstream a passive sediment management approach and the addition of large woody debris across the 147-acre floodplains/impoundment area would be costly and invasive with respect to the existing wetlands therefore this alternative was not selected.

Selected Alternative –Dam Removal with two riffle features at River Street and the Dam, constructing beaver dam analogues, and strategically adding large woody debris throughout the Lower Impoundment

The selected alternative includes dam removal, constructing two riffle features and two beaver dam analogs and addition of roughness elements to the floodplain. The primary difference in the selected alternative – is the addition of roughness elements upstream of the dam would focus on the installation of large woody debris in strategic locations within the Lower Impoundment. The selected alternative would involve adding roughness elements across an area of approximately 4 acres as opposed to 147 acres for Alternative 3. Based on the hydraulic analysis described in **Section 2** the incremental differences in water surface elevations downstream of the dam between Alternative 3 and the selected alternative were minimal (less than 0.1 feet), and the costs and environmental impacts associated with the selected alternative are anticipated to be much less than Alternative 3. This alternative would also eliminate dam maintenance and provide fish passage at the site from below River Street through the dam location. This alternative includes an active sediment management strategy.

4 PROPOSED DESIGN

Based on the findings of the engineering assessment and resource studies, a preliminary design for removal of the Temple Street Dam was developed for the selected alternative. Design drawings (approximately 75% complete) are presented in **Appendix C**.

4.1 Overview

The proposed removal of Temple Street Dam involves demolition of the full vertical extent of the approximately 11-foot-wide concrete outlet structure.

The proposed design would involve instream repositioning or dredging of approximately 140 CY of sediment immediately upstream of the dam, a proposed low flow channel 500 feet upstream of the dam, and the placement of about 230 CY of stone fill to protect the dam removal side slopes and establish riffle features at the Temple Street Dam location and River Street crossing, and installation of large woody debris and other floodplain modifications upstream of the dam.

4.2 Riffle Feature Design

The streambed design consists of two main elements, which are discussed in more detail below:

- Riffle Feature Stone
- Filter stone

Riffle Feature Stone

The stone is in the riffle is needed to maintain the riffle grades for fish passage and protect the water line at River Street from erosion. To size the riprap, flow depths and velocities at River Street were reviewed for several peak discharge events. Several methods were evaluated to determine and check the stone size, including:

- **Maynord/USACE EM 1601** – This method is recommended by the FHWA Hydraulic Engineering Circular No. 23 (HEC-23) (2009) and the National Cooperative Highway Research Program (NCHRP) Report No. 568 (2006). The equation is primarily a function of the velocity and depth of flow over the riprap, the riprap shape (angular versus round), and the bank side slopes for revetment designs. The recommended factor of safety of 1.5 was applied for the 100-year flow event. This method resulted in a D_{50} size of 6 inches for rounded riprap.

- **HEC-11** – The FHWA's *Design of Riprap Revetment* (HEC-11) (1989) was reviewed, although it is now an archived publication. Riprap is sized as a function of velocity and depth, with coefficients to account for stone shape and slope. A D_{50} size of 4" was obtained by this method.

- **Abt and Johnson** – The Abt and Johnson (1991) formula as described in *Rock Ramp Design Guidelines* by Mooney, Holmquist-Johnson, and Broderick (2007) was reviewed. Riprap is sized primarily as a function of unit discharge along with coefficients to account for shape and slope. This method utilizes a safety factor of 1.35 on the unit discharge. A D_{50} size of 15" was obtained by this method.

- **ARS Rock Chutes** – The ARS equation as described in Technical Supplement 14C of Part 654 of the NRCS National Engineering Handbook was evaluated. Stone is sized based on a relationship between discharge, channel slope, and the channel width. A D_{50} size of 6" was obtained by this method.

A summary of the riprap D_{50} sizes obtained by the various methods is provided in **Table 4.2-1** below. The conservatively highest 15-inch size obtained by the Maynord/USACE EM 1601 method was selected for the design. By comparison the gradation of the existing MassDOT M2.02.3 riprap at River Street is provided as well. Riprap design calculations are provided in **Appendix G**.

Table 4.2-1: Summary of Riprap Sizes Obtained by Various Methods

Method	Riprap D_{50} (in)
Maynord/ USACE EM 1601	**6”**
HEC-11	4”
Abt and Johnson	15”
ARS Rock Chutes	7”
MassDOT M2.02.3 - **D75**	12”

A proposed gradation for the rounded riprap is provided in **Table 4.2-2** below.

Table 4.2-2: Proposed Gradation for Buried Rounded Riprap Protection

Sieve Size	Percent Finer by Weight
2.25'	100
1.7'	85
15"	50
8"	15

Minimum layer thickness for stone at t is typically a factor of 1.5 times the D_{50} size. Assuming a D_{50} of 15 inches, a minimum layer thickness of 22.5 inches (1.9 feet) was determined. Therefore, a layer thickness of 24” was selected with a 6” layer of filter stone. If anything this stone would provide additional protection downstream of the water line.

Filter Stone

A filter is a transitional layer of gravel, small stone, or fabric placed between a riprap layer and the underlying sediment or soil. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the riprap particles to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils.

The NCHRP Report No. 568 (2006) recommends the Cistin-Ziems method for designing an appropriate granular filter for riprap. This method determines a maximum allowable D_{50} size for the filter to ensure that both the underlying native sediment won’t erode through the voids and interstitial spaces in the filter, and the filter material won’t erode through the riprap. Based on this analysis, MassDOT’s dense-graded crushed stone for subbase (Item No. M2.01.7) was selected as an appropriate material for the riprap filter.

4.3 Large Woody Debris

In order to increase the roughness within the limits of the former impoundment the proposed design includes the addition of large woody debris (10-feet long, 12” diameter) to the floodplain upstream of the dam. The following design forces/scenarios were considered based on guidance from the U.S. Department of the Interior Bureau of Reclamation Large Woody Material – Risk Based Design Guidelines (USBR 2014):

- Buoyancy forces
- Overturning
- Rotation

Typical forces used to counteract these forces include piles driven adjacent to the floodplain pieces, backfilling, and or boulders. Initial calculations were based on assessing the number, size, and embedment depth required if vertical timber piles were selected for stabilizing the logs.

4.3.1 Pile Anchors

Pile anchors rely on vertical or diagonal members or being driven into the ground and either tying them with cable or driving rebar through them into the logs/large woody debris pieces they're anchoring. This installation relies on the properties of the soils to resist buoyancy and other lateral/rotational forces.

Buoyancy

The buoyant force acting on the large woody debris is equal to the volume of water displaced by the submerged logs since logs are less dense than water. It was assumed for these calculations that the logs were placed flat across the top of the existing grade and were completely submerged based on the projected 1% recurrence interval event.

Overturning/Rotation

Calculations to review the requirements for resisting overturning were assessed assuming the log pieces lied perpendicular to the direction of flow.

The findings from the calculations provided in **Appendix H** are summarized in **Table 4.3.1-1** below. The various pile embedment depths, diameters, and number of piles and the associated factors of safety are compared to the standard factors of safety recommended by the Bureau of Reclamation 2014 guidance. Log pile anchor design calculations are provided in **Appendix H**.

Table 4.3.1-1: Pile Design Alternatives

Pile Arrangement/Log	Buoyancy	Rotation	Overturning
Bureau of Reclamation Factor of Safety (FOS)	**2.0**	**1.75**	**1.75**
1 Piles– 6" Dia., 7 feet embedment	2.43	2.68	1.32
2 Piles– 6" Dia., 7 feet embedment	2.42	2.54	1.32
1 Pile– 8" Dia., 9 feet embedment	2.30	2.58	1.29
2 Piles– 8" Dia., 5 feet embedment	3.13	3.03	1.44
3 Piles– 4" Dia., 7 feet embedment	4.96	4.28	1.76
2 Piles– 6" Dia., 7 feet embedment	4.86	4.74	1.75
1 Pile– 8" Dia., 9 feet embedment	5.35	4.86	1.83
2 Piles– 8" Dia., 7 feet embedment	6.36	5.35	2.01

4.3.2 Boulder Anchors

In order to counteract the buoyant force of a 10 foot, 12" diameter log piece a 2-foot diameter boulder with an eyehook and cable would be able to resist the buoyancy of the log pieces with an adequate FOS. Alternatively, a cable could be secured around the log in a manner which would limit the log's ability to float/rotate from its original position. The primary disadvantage with this alternative is having to import logs and the cables/chains/eyehooks etc. are all unnatural and would need to be stainless steel/galvanized to last in a semi-aquatic environment.

4.3.3 Earth Anchors

Earth anchors can require less equipment to install than other methods such as pile anchors or importing boulders. These anchors can take the form of small steel spades or duckbill anchors that are attached to cable and driven into the ground either with manual hydraulic tools or by pushed in by hand. Once the spade is driven into the ground the cable is pulled back at an angle and the spade rotates under the soil until it resists to the point it stops rotating and can't be pulled any more. Utilizing these anchors would be labor intensive but would potentially be less invasive than bringing in heavy equipment into the lower impoundment. The ends of the anchors sticking above the soil would be connected to stainless steel or galvanized cable or an approved equal wrapped around the logs. Based on the lowest capacities of available standard duckbill anchors available two anchors would be required per 10' log.

4.3.4 Impoundment Bed Scour Effects on Anchors

The D_{50} grain size observed in the impoundment was approximately 0.0029 inches (0.074 mm). According to output from the hydraulic model during the 20% and 1% recurrence interval events maximum shear stresses in the impoundment are limited to 0.05 lb/ft^2 in the floodplain with values in localized areas within the thalweg as high as 0.24 lb/ft^2. The shear stresses in the floodplain exceed the critical shear stress of 0.002 lb/ft^2 required to mobilize very fine sands found in the impoundment (Berenbrock and Tranmer 2008)[15]. Because of this the logs may shift slightly from their original positions but the logs have been placed in low shear stress locations and there is existing aquatic vegetation across a large portion of the impoundment which will also serve to stabilize sediment.

4.4 Beaver Dam Analogues

In order to provide additional flow attenuation upstream of the dam post-removal, two beaver dam analogues are proposed in the locations shown on **Drawing 5** in **Appendix C**. A 130-foot-long beaver dam analogue is proposed 160 feet upstream of the dam, connecting an island to the mainland. A second 530-foot-long beaver dam analogue is proposed 660 feet upstream of the dam crossing the impoundment from east to west connecting the same island to the mainland with a 20' wide gap in it for the fish to be able to pass upstream and allow duck hunters to pass through the area by boat so they can access conservation lands further upstream. The beaver dam analogues will be tied into elevation 38.0 at each end to prevent "flanking" of flows around the structures.

Details for construction of the beaver dam analogues are provided on **Drawing 8** in **Appendix C**. The design details were based on guidance from Chapter 4 of the Low-Tech Process-Based Restoration of Riverscapes Design Manual Version 1.0 (Wheaton et. al 2019).

The selected design utilizes native materials stacked on top of each other in 6-12" lifts and includes wood stake reinforcement every 2.5 feet on center to provide extra stability. Based on the topographic data these structures would be 3-4 feet high and have a crest of approximately 38.0 feet to hold back flows during events up to the 1% recurrence storm event. Key pieces (12" larger diameter) logs are proposed to be included to give the structures added strength.

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[15] It should be noted that the sediment sampling technique used to gather sediment samples for analytical testing can sometimes underestimate the average grain sizes in the impoundment as a 2-inch corer is used.

4.5 Construction Methods

Timing

Dam removal implementation may be feasible during a range of flows in nearly any season. Depending on input from DMF/DFW, the following seasonal restrictions should be considered when planning the construction period:

- April 15 – July 15: Upstream fish migration season
- June 1 – July 31: Potential restriction on cutting trees that may provide nesting habitat for northern long eared bats
- October 1 – November 30: Potential time-of-year (TOY) restriction for trout

Sediment Management

An active sediment management approach is recommended for the project to prevent 200 CY of sediment from mobilizing downstream. Sediment dredging is proposed just upstream of the dam and through a low flow channel upstream located 600 feet upstream of the dam. Sediment from these two locations will be re-used in one of several ways including washing it into the voids of proposed riffle features, applying it to disturbed upland slopes and embankments as loam, burying the brick remains adjacent to western dam embankment, or on other upland areas on the property (specific location to be determined).

Access & Staging

The proposed access and staging plan is shown on **Drawing 4** in **Appendix C** and in **Figure 4.5-1** in **Appendix A**. Photographs of potential access and staging area locations are shown in **Photos 3** through **5, 10, and 15-18** in **Appendix B**. At this time, it is envisioned that the parking area off of Myrtle Street/Temple Street, the dam embankments, and the abandoned River Street embankment can be used as a staging area. Temporary access ramps will be constructed out of crushed stone to facilitate access down the banks of the channel as needed. Swamp mats will be utilized for access through the wetlands as needed to establish water control measures and to facilitate other work including installation of large woody debris and beaver dam analogues.

Two additional potential access routes/staging areas identified on the Drawings in **Appendix C**. For the work at the downstream end of the project where the dam removal and riffle features are proposed access from River Street off of Keene Street may be considered. This access is currently restricted by two posts with a chain across the driveway but the remains of the asphalt road down river street extends to within approximately 250 feet from the river. This driveway may be utilized for staging/laydown of materials if there isn't enough space on the eastern River Street and dam embankments. Clearing an approximately 14 feet wide/approximately 250-foot path along the top of the former roadway would be required to get equipment to the river/near the work area.

Additionally, there is a currently unused cart path off of Keene Street which leads to a peninsula across from the Beaver Dam analogue location. This access and the Town property lines off of Keene Street are provided in **Figure 4.5-1**. The path starts out approximately 10 -12 feet wide and narrows as it gets closer to the river. Photographs of this path are provided in **Appendix B**. This access is being identified so it may be considered as an option for access by the contractor when evaluating means and methods.

Large Woody Debris/Roughness Elements Installation in Floodplain

Several options for the installation of large woody debris are possible including floating the logs into place. Instead of waiting to dewater the impoundment to install the large woody material it might be less impactful to bring the logs out to their desired locations by floating them out to their desired location and anchoring them in place a garden stake and rope temporarily. Once the logs are in place the flashboards at the dam could be pulled and the impoundment drained to install earth anchors. This combination of floating logs and using earth anchors would avoid significant disturbance to the soils in the impoundment area due to heavy equipment.[16]

An alternative method for large woody material installation would be to use specialized heavy equipment with wide treads built for working in a wetland environment. This method would cause significant disturbance within the floodplain. However, if boulders or other piles were used to anchor the logs it would be beneficial to have a piece of equipment to move and install those types of anchors.

The least preferred alternative is clearing/constructing a road along the west side of the Lower Impoundment from the dam to access the impoundment with conventional heavy equipment. Soils in the impoundment are fine and equipment becoming stuck is a concern. This option would also require the most impacts to wetland resources.

Water Control

The suggested water control plan for construction of the riffle features at the dam and downstream of River Street is shown on **Drawing 6** in **Appendix C**. During Phase I prior to removing the dam it is recommended that the logs are floated into place with the dagger boards in place and temporarily anchor the logs rather than drag or lift the logs into place with equipment.

During Phase I of construction once the logs pieces are floated into place the first phase of temporary water controls will be installed.

During Phase II it is proposed that the first riffle feature and water control installed occur at the River Street feature. The proposed water control calculations are based on the median flow of 3 cfs between 7/15 and 10/31 and the 2-year (50% recurrence interval) storm event (119 cfs) presented in Section 2.4.1. Based on the analysis of potential alternatives at the site a 36" diameter pipe would be able to divert river flows up approximately 35 cfs. During the anticipated construction period 35 cfs is exceeded less than 2% of the time and 6% of the time annually. This option will not be able to pass storm events such as bankfull (50% recurrence interval) flows.

A second alternative investigated utilizing sandbags/super sacks to divert the river without pipes was investigated. Using the Manning "n" equation and normal depth method anticipated depths of flow were determined for 119 cfs assuming half of the riffles were blocked off for construction. The analyses showed for a 5' wide bottom width the depth of flow at 119 cfs would be approximately 3.46 feet. Therefore, a 5-foot-high cofferdam is recommended to divert water around the riffles while providing over 1' of freeboard during a bankfull flow. Supporting calculations are provided in Appendix I.

[16] Floating logs into place may require either doing it in the spring either before or during the anticipated fish migration season if the dagger boards remain in their current position. Alternatively, it could be possible to temporarily raise the impoundment by 1-1.5 feet from its normal elevation in the summer (when South River flows are low) by installing additional or new dagger boards.

During Phase III, once the riffle feature at River Street is completed the water controls (i.e.sandbags/supersacks) will be relocated to the dam location for the removal and channel work at the dam in Phase III. Following removal of the dam and construction of the riffle feature at the former dam location water controls will be removed. In the event of flows higher than the capacity of the temporary diversion channel, work will be suspended until it is safe to resume work to re-establish water controls. The contractor will be required to prepare a water control plan to be approved by the owner and engineer including 1) a proposed cofferdam and temporary bypass system plan, details, and calculations, 2) a water control contingency plan, and 3) dewatering and sedimentation control measures. The water control plan will conform to applicable environmental permit requirements and conditions. The contractor will be responsible for taking all necessary precautions to prevent damage to the work or equipment by high water or storms.

Erosion, Sedimentation, & Pollution Control

Proposed erosion, sedimentation, and pollution controls are shown on **Drawing 4** in **Appendix C**. All work will be conducted in accordance with the local erosion and sedimentation control guidelines and best management practices. Erosion, sedimentation, and pollution controls will be installed prior to any major soil or stream disturbance and maintained until permanent protection is established. Recommended controls include compost filter socks around the access and staging areas, an oil boom across the channel downstream of the dam, a stabilized construction entrance, and temporary access ramps and/or swamp mats as needed to minimize soil disturbance while accessing channel areas, and erosion control blankets to be placed on any slopes greater than 3H:1V. The proposed upland disturbance area is not expected to exceed 1 acre however the area impacted below the OHWM may potentially exceed 1 acre and require a National Pollutant Discharge Elimination System (NPDES) permit.

Stormwater Management

The proposed project is a restoration project, not a development project; therefore, only stormwater standards related to temporary construction impacts (Standard 8) would apply. The project is not anticipated to have significant stormwater impacts. Recommended erosion and sedimentation control measures are described above and included on the construction drawings. The selected contractor will be responsible for developing and implementing a plan to control construction-related impacts, including erosion, sedimentation, and other pollutant sources during construction and land disturbance activities. The plan will be required to comply with all conditions contained in relevant permits and must be approved by the engineer and the owner. During construction, temporary erosion, sedimentation, water, and pollution controls will be utilized in accordance with BMP guidelines recommended by DEP.

Material Disposal

Sediment to be dredged is assumed to not be contaminated based on the due diligence review and sediment sampling results presented in **Section 2.2**. Any dredged sediment suspected to be contaminated will be stockpiled onsite, dewatered, and tested per DEP recommendations to inform disposal options. Otherwise, dredged sediment may be reused if deemed suitable for this purpose (i.e., within the specified gradation range). There are several ways to re-use dredged sediment at the site as described in Section 2.2.4 including but not limited to applying it as loam to the River Street and dam embankments, washing it into the voids of the riffle features, burying the brick remains adjacent to the western dam embankment. , Clean sediment could also be regraded elsewhere on the upland portion of the property (i.e., outside of the top-of-bank lines, which is known as “Upland Material Reuse”). Lastly, either clean or contaminated sediment could be disposed of at an in-state landfill or a hazardous waste facility per DEP regulations. Unsuitable material will be removed from the site for lawful disposal or upland reuse as appropriate. The material could be disposed of at a local sand/gravel borrow pit or other location accepting clean fill. If it is clean and relatively sandy, it could potentially be accepted as landfill cover (i.e., "Landfill Reuse" in accordance with COMM-97-001).

Air Quality Control

Construction and operation activities shall not cause or contribute to a condition of air pollution due to dust, odor, or noise in accordance with 310 CMR 7.09 and 7.10. Excessive idling during the construction period will be prohibited. The methods of reducing idling will include posting signage limiting idling to five minutes or less at the project site, driver training, and periodic inspections by site supervisors to ensure compliance with this regulation once the project is occupied. Finally, staging areas will be established in a manner that minimizes impacts to abutting properties from construction equipment emissions.

Invasive Species Prevention and Control Plan

Invasive species are not anticipated to be a significant issue for this project. Some invasive species are present in the area, but standard precautions will be implemented to prevent the spread of any potential invasive species during construction (e.g., use of a stabilized construction entrance, cleaning of equipment, etc.). Disturbed upland slopes, such as the roadway embankments, will be seeded with a native seed mixture during site restoration. The wetland area will be allowed to revegetate naturally from the seed bank in the sediment. During the revegetation period, Project Partners will monitor the site regularly and hand pull any observed invasive species as soon as possible before they can spread. Monitoring will occur for at least two years or until native vegetation has become established.

Construction Sequence

The proposed construction sequence is as follows:

<u>CONSTRUCTION SEQUENCE</u>

<u>SITE PREPARATION AND ACCESS</u>

1. Contractor shall prepare a construction sequence plan to be approved by owner and engineer. The following general sequence shall be adapted for the site-specific requirements.

2. Survey and stake the proposed limit of disturbance and limit of erosion controls. Install erosion controls and containment measures as indicated in the plans.

3. Flag limits of clearing, to be approved by owner prior to any tree removal. Clear and grub along approved access routes as needed.

4. Install staging area and temporary access ramps/routes as needed. Utilize swamp mats (or approved equal) to minimize disturbance to wetland areas.

5. Install oil boom and turbidity curtain.

<u>PHASE I - (RIVER WORK)</u>

1. Leave dagger boards in place at the dam.

2. Float log pieces into the impoundment and temporarily anchor them in place.

PHASE II - (RIVER WORK)

1. Install supersack cofferdam (or approved equal) to facilitate flow through site while constructing riffle feature at river street.

2. Construct proposed riffle feature at river street as shown. Riffle shall be constructed in 15-inch lifts above filter layer and fine material shall be used to choke each lift prior to placement of the subsequent lift. The contractor shall wash the fine material into the lift of coarse material with a sufficient quantity of water.

3. Relocate supersacks at the ends of the cofferdam as required to switch flows to the other side to construct the other half of the riffle feature.

4. Once riffle feature at river street is constructed relocate supersack cofferdams/controls to facilitate the removal of the dam in phase iii.

PHASE III - (RIVER WORK)

1. Install supersack cofferdam (or approved equal) to facilitate flow through site while removing the dam and constructing the upstream riffle feature.

2. Remove spillway dagger boards to drain the impoundment.

3. Remove the full vertical extent of the concrete training walls, pier, deck, and apron of the dam spillway outlet structure. Remove all concrete from the river.

4. Construct proposed riffle feature at the dam as indicated. Riffle shall be constructed in 15-inch lifts above filter layer and fine material shall be used to choke each lift prior to placement of the subsequent lift. The contractor shall wash the fine material into the lift of coarse material with a sufficient quantity of water.

5. Remove water controls from the dam location.

FLOODPLAIN ENHANCEMENTS

1. Install beaver dam analogs (BDA) to crest elevation 38.0 ft as shown on the plans and construct the low flow channel at the boundary of the lower and upper impoundments. BDA's shall tie into grade at each end except at low flow channel breach.

2. Remove temporary anchors and install permanent anchors for large woody debris as shown on the plans.

3. Plant live stakes in the floodplain areas with native species per the planting plan along the water's edge.

SITE RESTORATION

1. Remove any remaining water controls from the site.

2. Remove crushed stone, stone fill and geotextile fabric for temporary access paths and at the construction entrance.

3. Repair paved parking area, to the satisfaction of the owner's representative, if necessary.

4. Remove erosion control and other containment measures only after all areas are stabilized with vegetative cover to the satisfaction of owner's representative.

5. Any remaining excavated sediment shall be spread across any upland disturbed areas, used to bury the brick remains shown on the plans, or disposed of at an upland location.

6. Any re-used sediment or disturbed upland areas be seeded with an approved native seed mixture. Erosion controls shall remain in place until 85% of the disturbed upland areas have been stabilized with a stand of grass/vegetation.

4.6 Opinion of Probable Construction Cost

A preliminary opinion of probable construction cost (OPCC) for the proposed removal of Temple Street Dam is provided in **Table 4.6-1** below. The OPCC was developed using MassDOT's published weighted average bid prices[17], R. S. Means Construction Cost Data, and available final costs from comparable projects. The OPCC itemizes costs for mobilization/demobilization, controls and protections, access, site work, and site restoration. At this stage, a contingency of 25% was included.

[17] Prices for all districts from the period of June 2021 to June 2022. DOT's Standard Specifications for Highways and Bridges provide more detail about methods and included services for each item.

Table 4.6-1: Opinion of Probable Construction Cost for Temple Street Dam Removal

Category	Item	MassDOT Item No.	Description	Unit	Qty	Unit Cost	Ext. Cost
Mobilization/ Demobilization	Contractor's general requirements	748	15% of remaining costs excluding precast structures	LS	15%	$352,505	$52,876
						SUBTOTAL	$52,876
Access & Erosion Control	Sediment control barriers	761.121	Compost filter socks	LF	900	$7	$6,300
	Clearing & grubbing	101	Clear brush and small trees along proposed access routes	AC	0.50	$47,150	$23,575
	Swamp mats	-	To access areas in floodplain	LS	1	$19,000	$19,000
	Access Ramps/Roads Gravel	402	M2.01.7 dense graded crushed stone for access roads/ramps	CY	250	$74	$18,500
						SUBTOTAL	$67,375
Water Control	Oil containment boom	697.2	Across channel downstream of work area	LF	40	$49	$1,960
	Cofferdam & water diversion system	252.3	Bulk sandbags and installation	LS	1	$24,000	$24,000
	Dewatering pump system	-	For initial dewatering & seepage/runoff control	DAY	10	$1,100	$11,000
	Sediment filter bag	-	For sediment control during dewatering	EA	3	$100	$300
						SUBTOTAL	$37,260
Removals	Dam Demolition and Disposal	-	Partial removal of existing dam structure	LS	1	$11,000	$11,000
	Channel/Sediment Excavation	123	Excavation of sediment as needed to facilitate partial dam removal.	CY	54	$60	$3,240
	Embankment Excavation	120	To facilitate Dam Removal	CY	40	$37	$1,480
						SUBTOTAL	$15,720
Channel Construction	Muck excavation	123	Excavation of low flow channel	CY	110	$60	$6,600
	Channel bed material (sand borrow)	154	Sand borrow for channel construction	CY	70	$48	$3,360
	Riffle Features	983.1	Rounded riprap (D50 = 15")	TON	270	$89	$24,030
	Filter stone	402	M2.01.7 dense graded crushed stone for subbase	CY	50	$74	$3,700
	Channel construction	-	Excavator, operator, and three laborers	DAY	10	$3,100	$31,000
						SUBTOTAL	$68,690
Floodplain Construction	Install Woody Debris/log jams	-	To roughen the floodplain areas	EA	50	$ 1,500	$75,000
	Install Boulder Anchors	-	Boulders to ballast 10' long, 12" Dia. Logs	EA	50	$ 540	$27,000
	Beaver Dam Analogue Material	-	Beaver Dam Analogue Material	LS	1	$ 21,700	$21,700
	Installation of Beaver Dam Analogues	-	Foreman, and three laborers	DAY	10	$ 2,200	$22,000
						SUBTOTAL	$145,700
Restoration	Live stakes	-	Placement in between large woody debris in floodplain	EA	350	$30	$10,500
	Seeding	765	For upland disturbed areas only - wetland seeding not included	SY	2420	$3.00	$7,260
	Loam (Spread 4" of excavated material)	170	For upland disturbed areas only - wetland seeding not included	SY	2420	$10.00	$2,686
						SUBTOTAL	$17,760
						SUBTOTAL Direct Construction Cost	$405,381
						Contingency Allowance (25%)	*$101,345*
						TOTAL DIRECT CONSTRUCTION COST *(2022 $, rounded to nearest $1,000)*	$510,000
Engineering Services	Final design & permitting services (FY2023)						$40,000
	Bid Phase Services (FY2024)						$10,000
	Construction Phase Services (FY 2024)						$50,000
						TOTAL OPINION OF PROBABLE CONSTRUCTION COST (2022 $)	$610,000

** In 2022 dollars, rounded to nearest $1,000.*

4.7 Regulatory Review Process

Table 4.7-1 below provides a summary of regulatory submittals, reviews, and permits that are anticipated to be required for this project.

Table 4.7-1: List of Anticipated Required Regulatory Submittals/Reviews/Permits

Permit	Agency	Applicability
Massachusetts Endangered Species Act Review	Natural Heritage and Endangered Species Program (NHESP)	Projects proposed in estimated rare or endangered species habitat, as delineated on the NHESP database.
Section 106 Historical Review / Project Notification Form (PNF)	MA Historical Commission (MHC)	Projects that require state funding, licenses, or permitting.
Environmental Notification Form (ENF)	MA Environmental Policy Act (MEPA) Office	Alteration of 5,000+ sf of bordering vegetated wetlands, alteration of one-half acre of other wetlands, new fill in a regulatory floodway, and possibly other thresholds.
401 Water Quality Certificate (WQC)	MA Dept. of Environmental Protection (DEP)	Any activity that would result in a discharge of dredged material, dredging, or dredged material disposal greater than 100 CY that is also subject to federal regulation.
Chapter 91		
Wetlands Protection Act Notice of Intent (NOI) for Ecological Restoration Projects	DEP / Municipality	Any construction in or near a wetland resource. Qualifies for a Restoration Order of Conditions as a dam removal project.
Clean Water Act Section 404 Programmatic General Permit	US Army Corps of Engineers (USACE)	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 or any fill, and for any restoration project.
Letter of Map Revision (LOMR)	Federal Emergency Management Agency (FEMA) / Town	May be required to officially revise the current Flood Insurance Rate Map to show changes to floodplains, floodways, or flood elevations.

5 REFERENCES

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Appendix A – Figures

Appendix A – Figures

Figure 1.1-1: Location Map

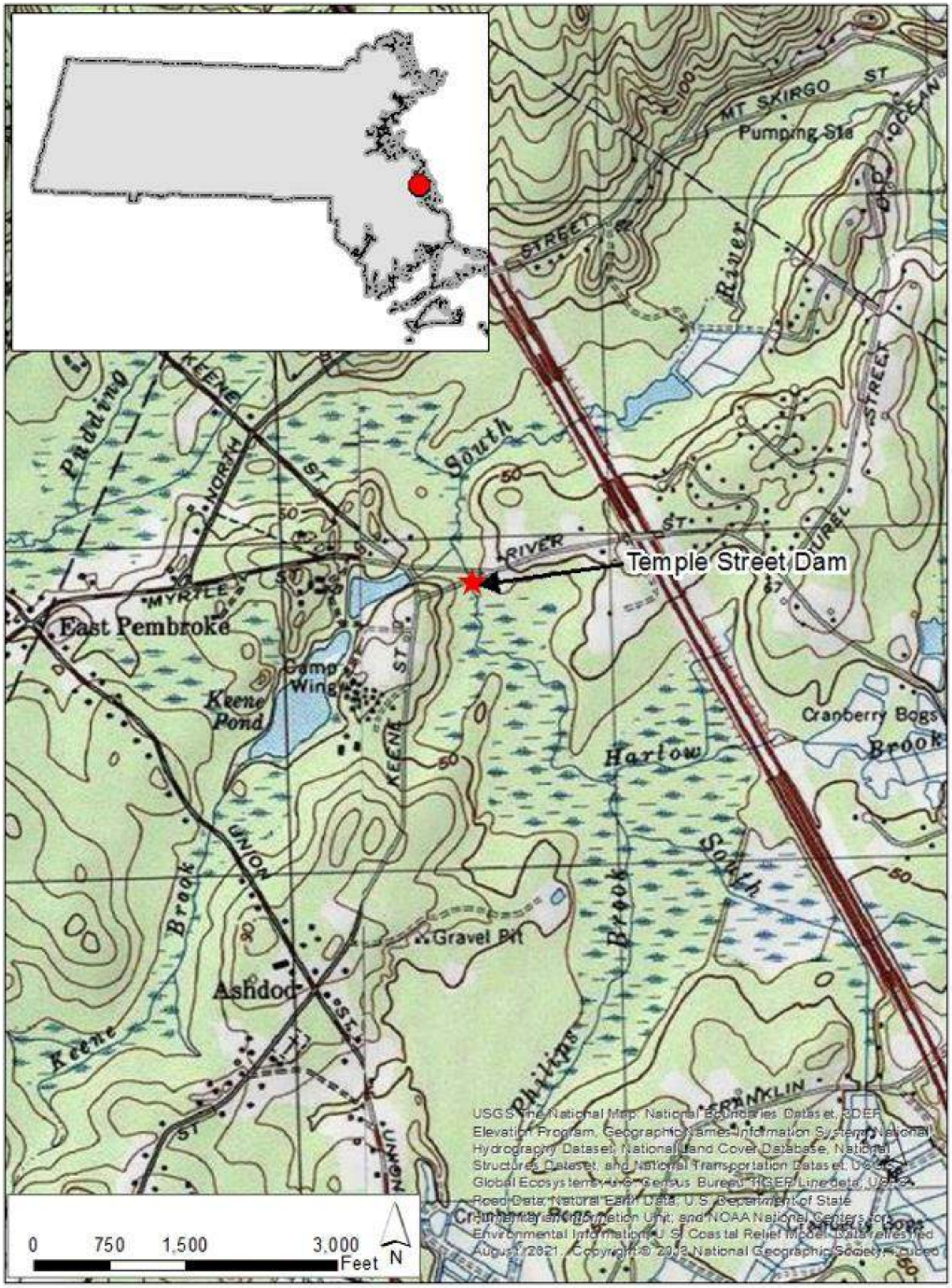


Figure 1.2-1: South River Watershed Map

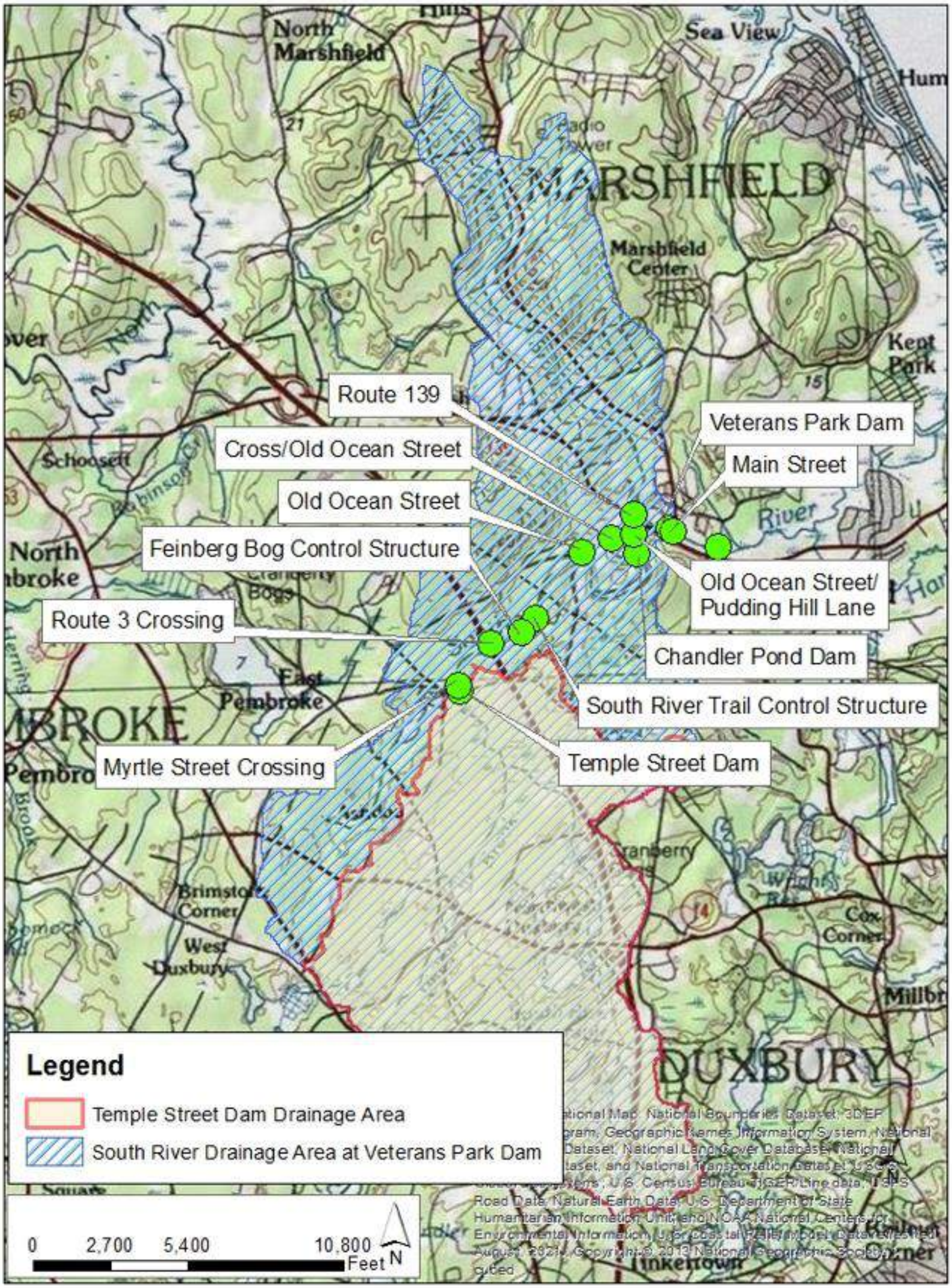


Figure 1.2-2 – Water Line Plans from Amory Engineers (2007)

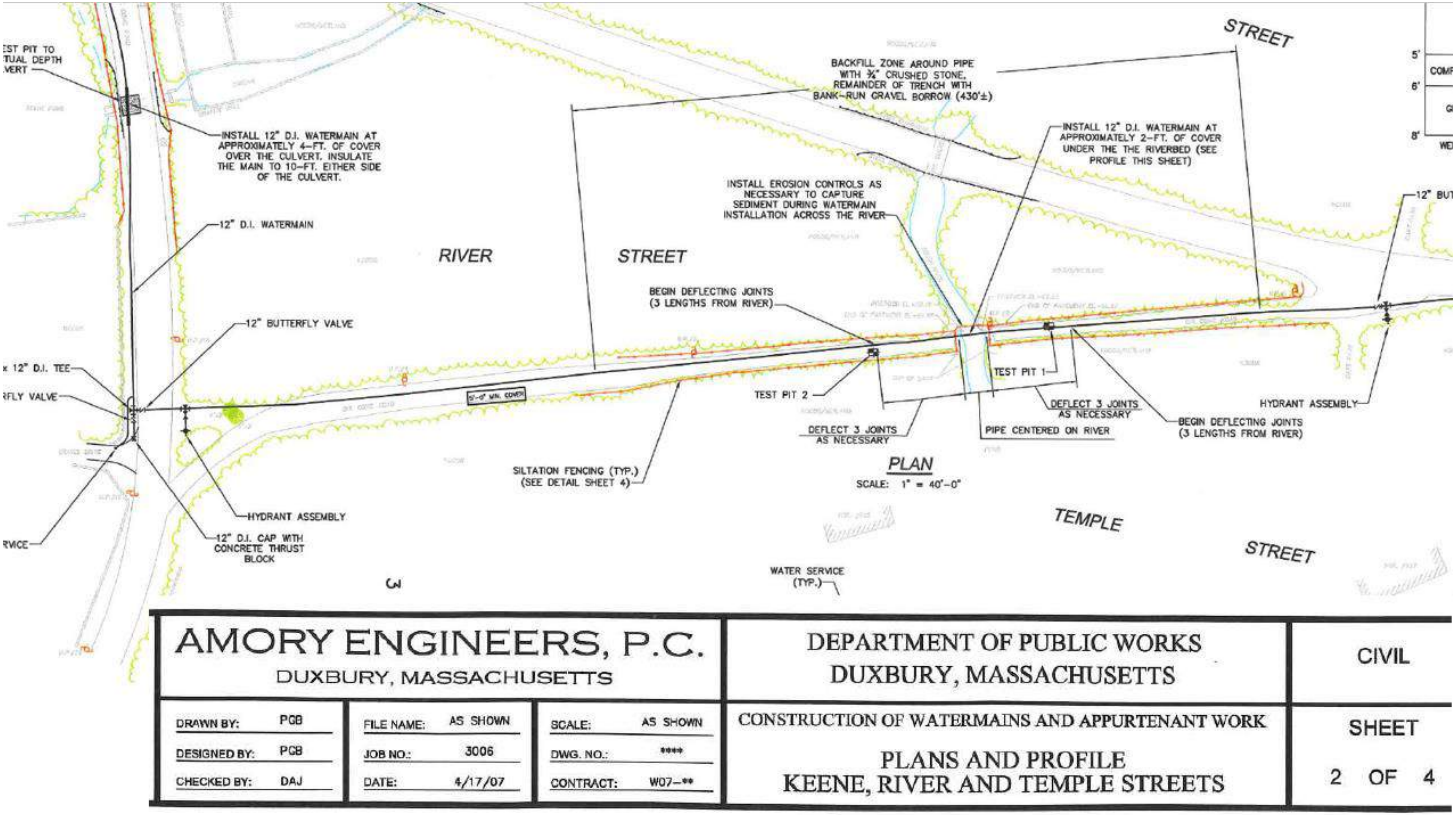


Figure 1.2-3 – Water Line Plans from Amory Engineers (2007) - Continued

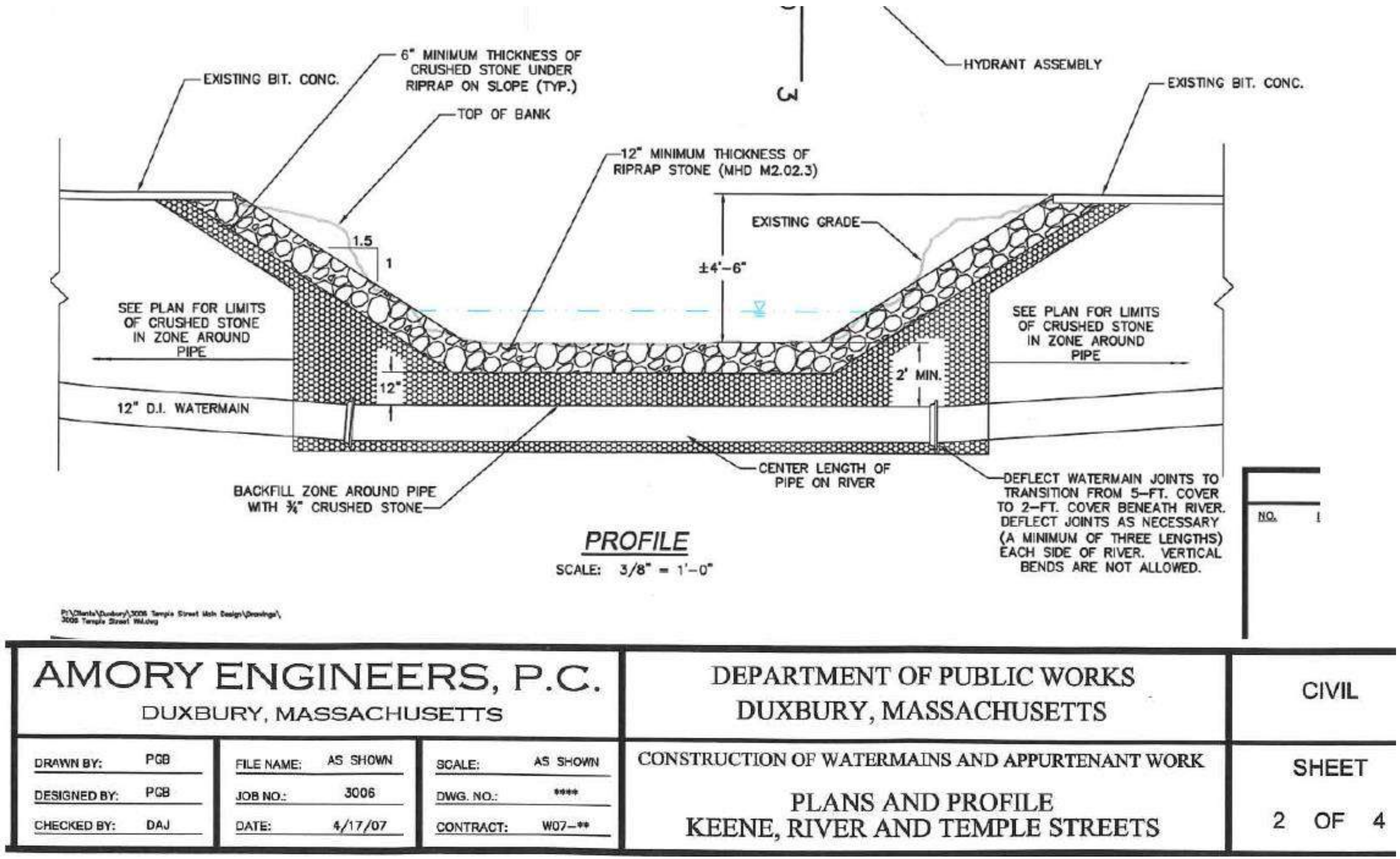


Figure 2.2.1-1: Waste Sites and Reportable Releases within the Vicinity of the Project

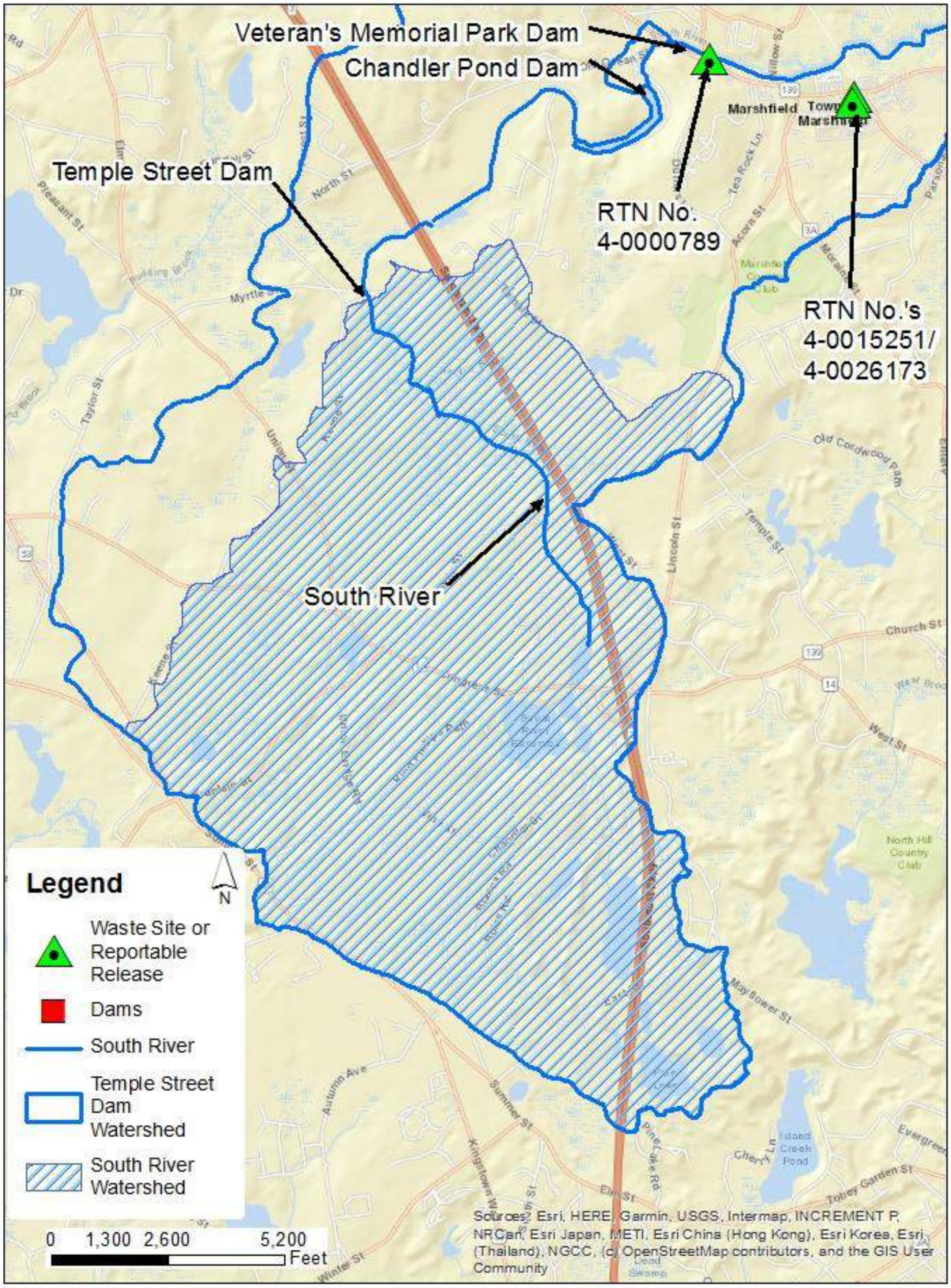


Figure 2.2.2-1: Sediment Probing Transects

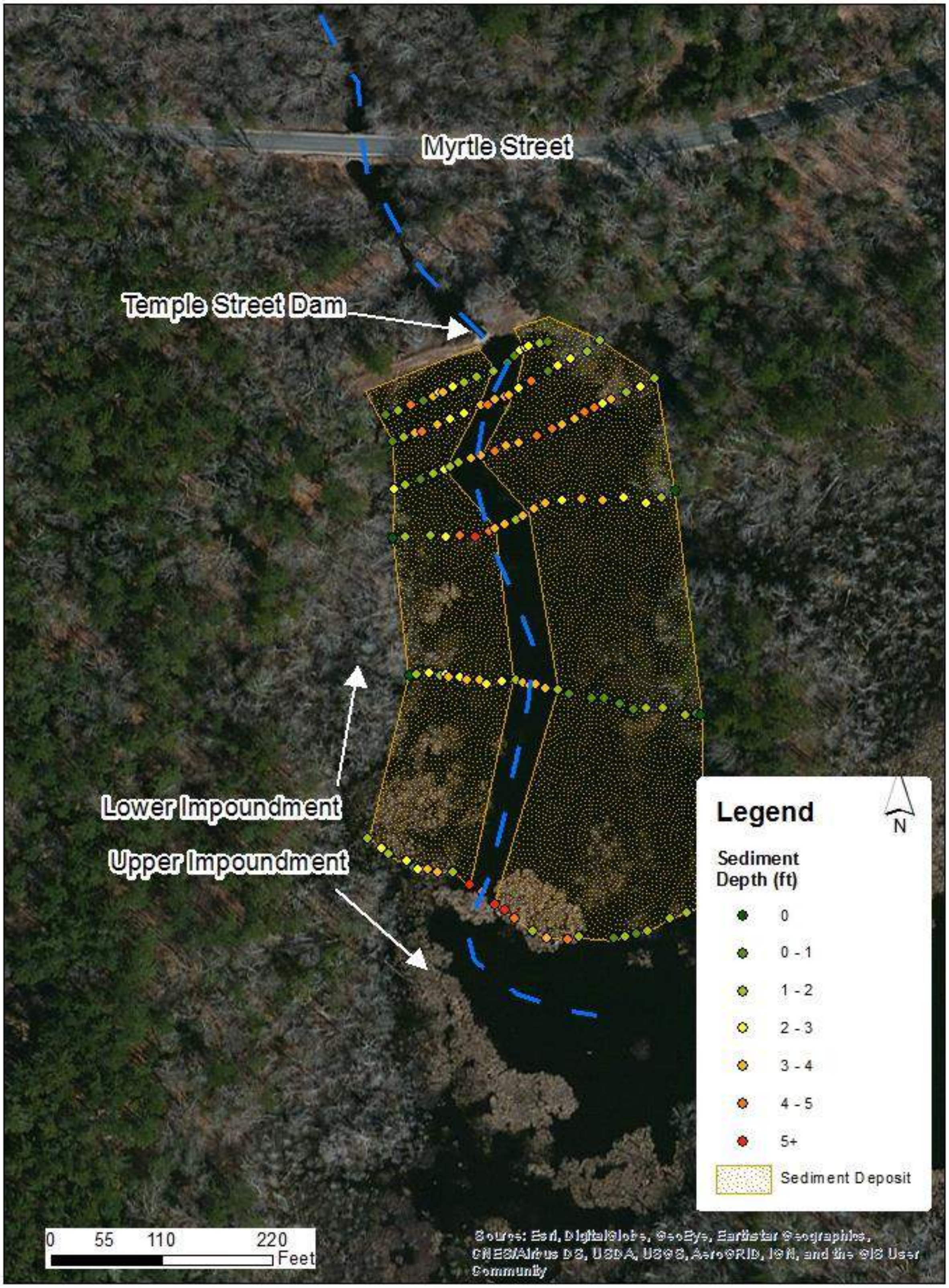


Figure 2.2.2-2: Sediment Probing Transect T-1 Cross-Sectional Plot

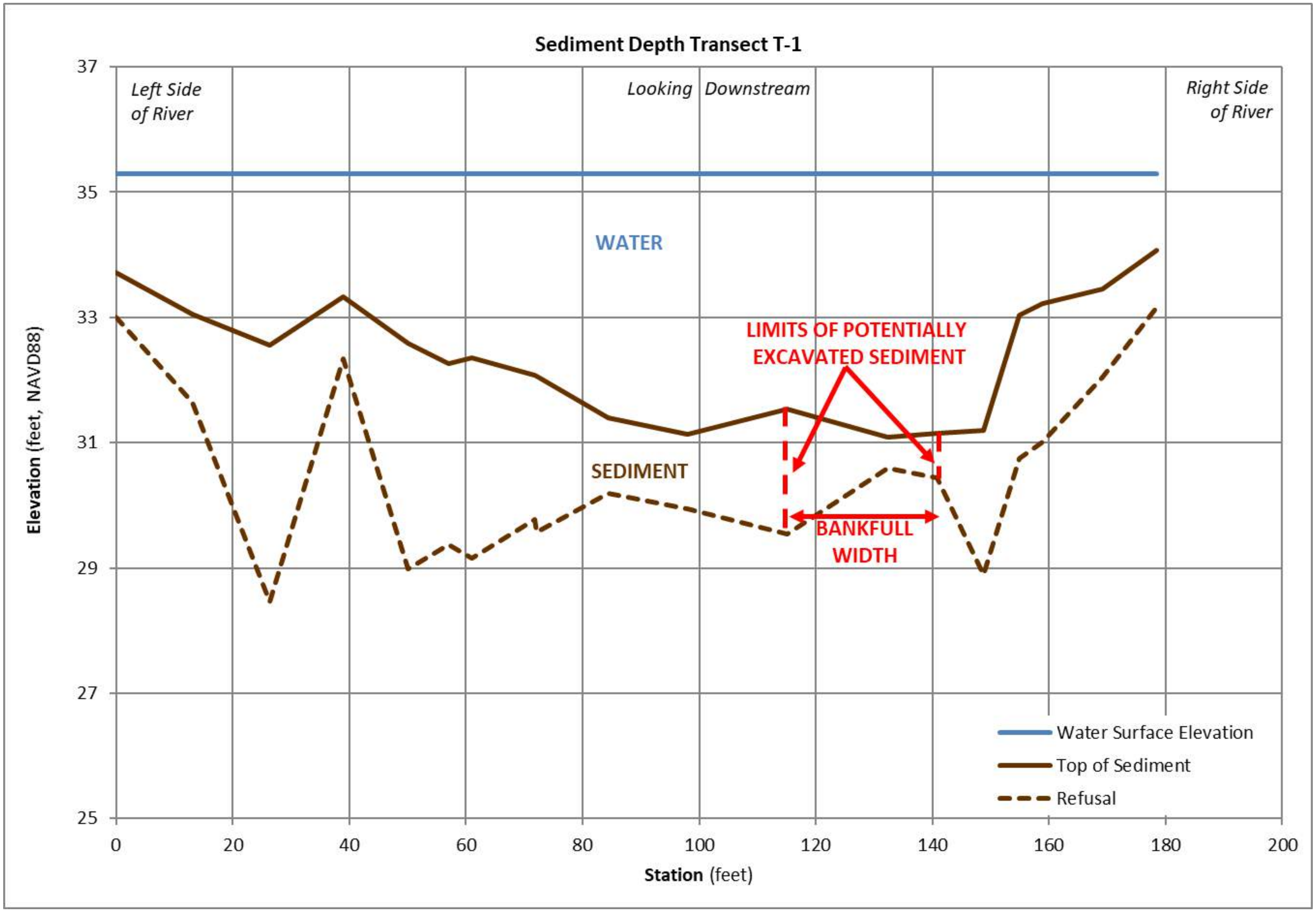


Figure 2.2.2-3: Sediment Probing Transect T-2 Cross-Sectional Plot

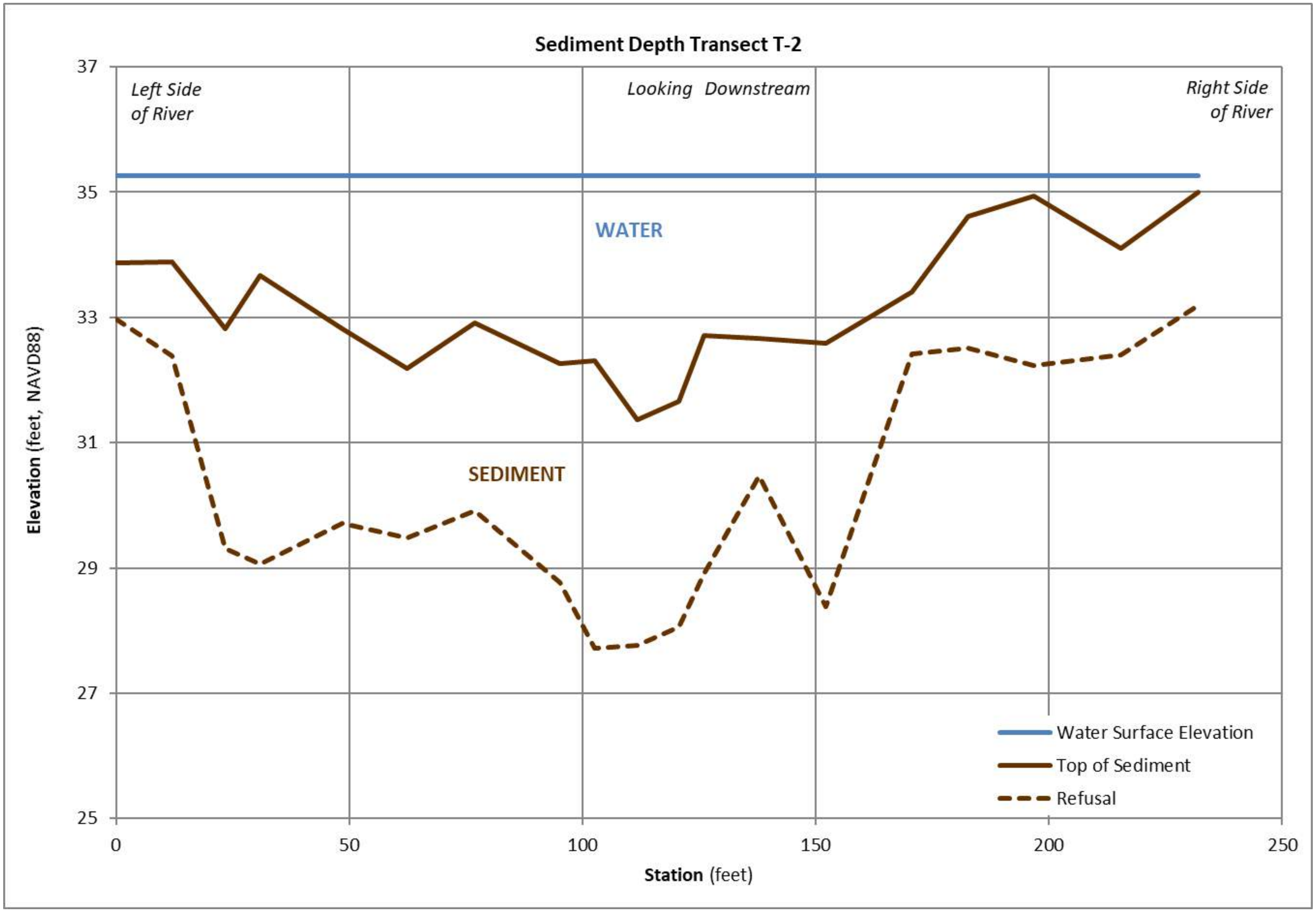


Figure 2.2.2-4: Sediment Probing Transect T-3 Cross-Sectional Plot

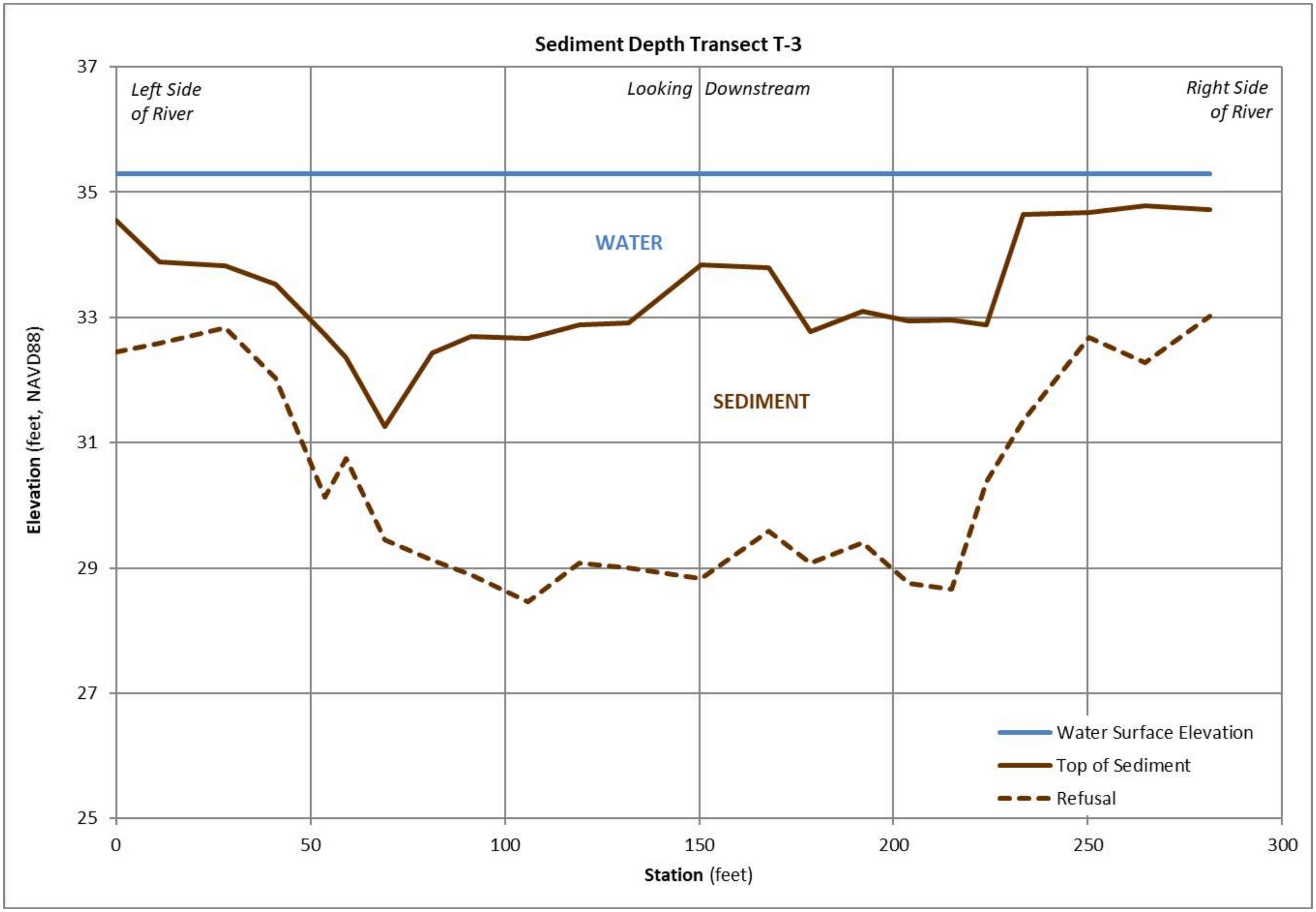


Figure 2.2.2-5: Sediment Probing Transect T-4 Cross-Sectional Plot

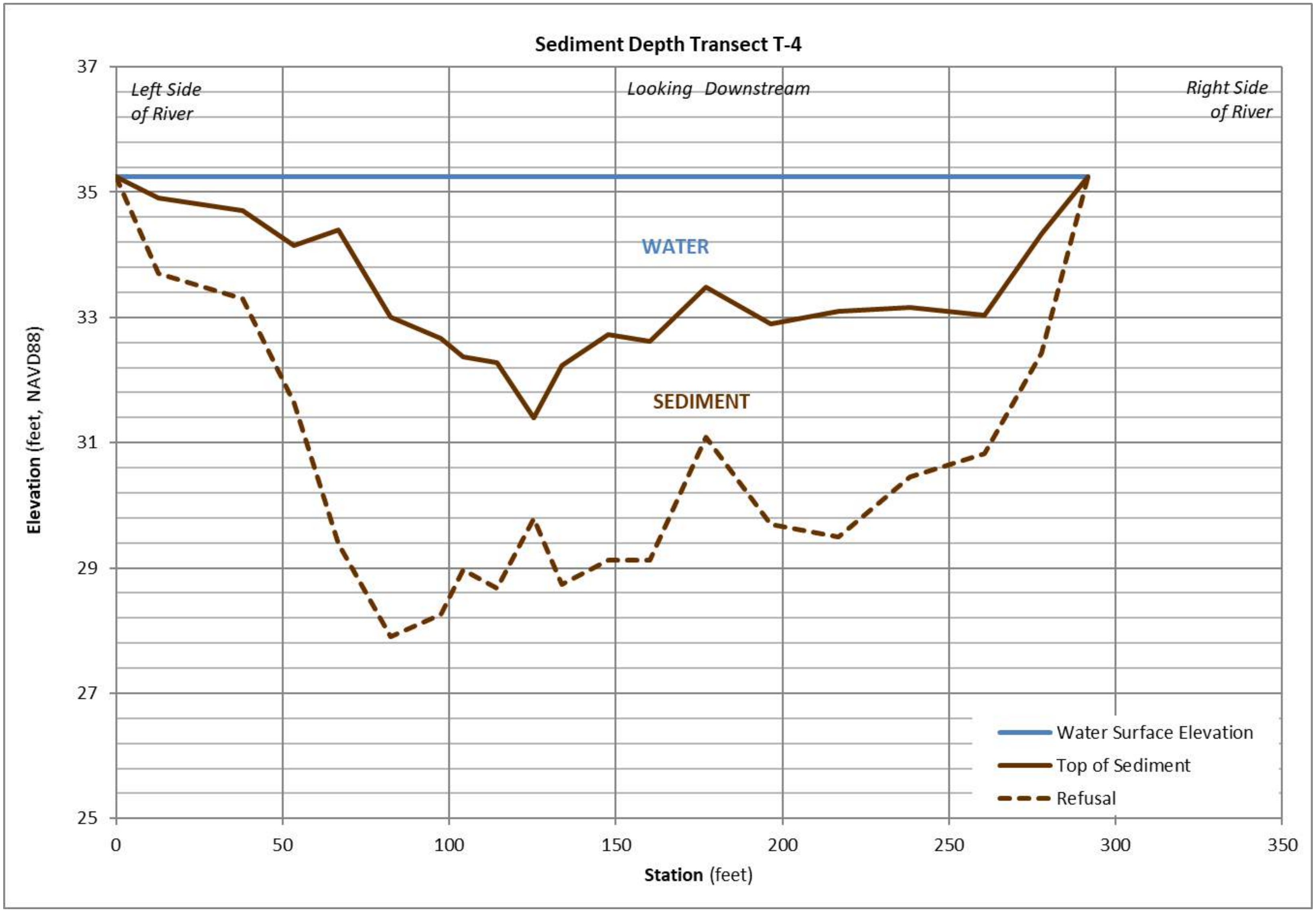


Figure 2.2.2-6: Sediment Probing Transect T-5 Cross-Sectional Plot

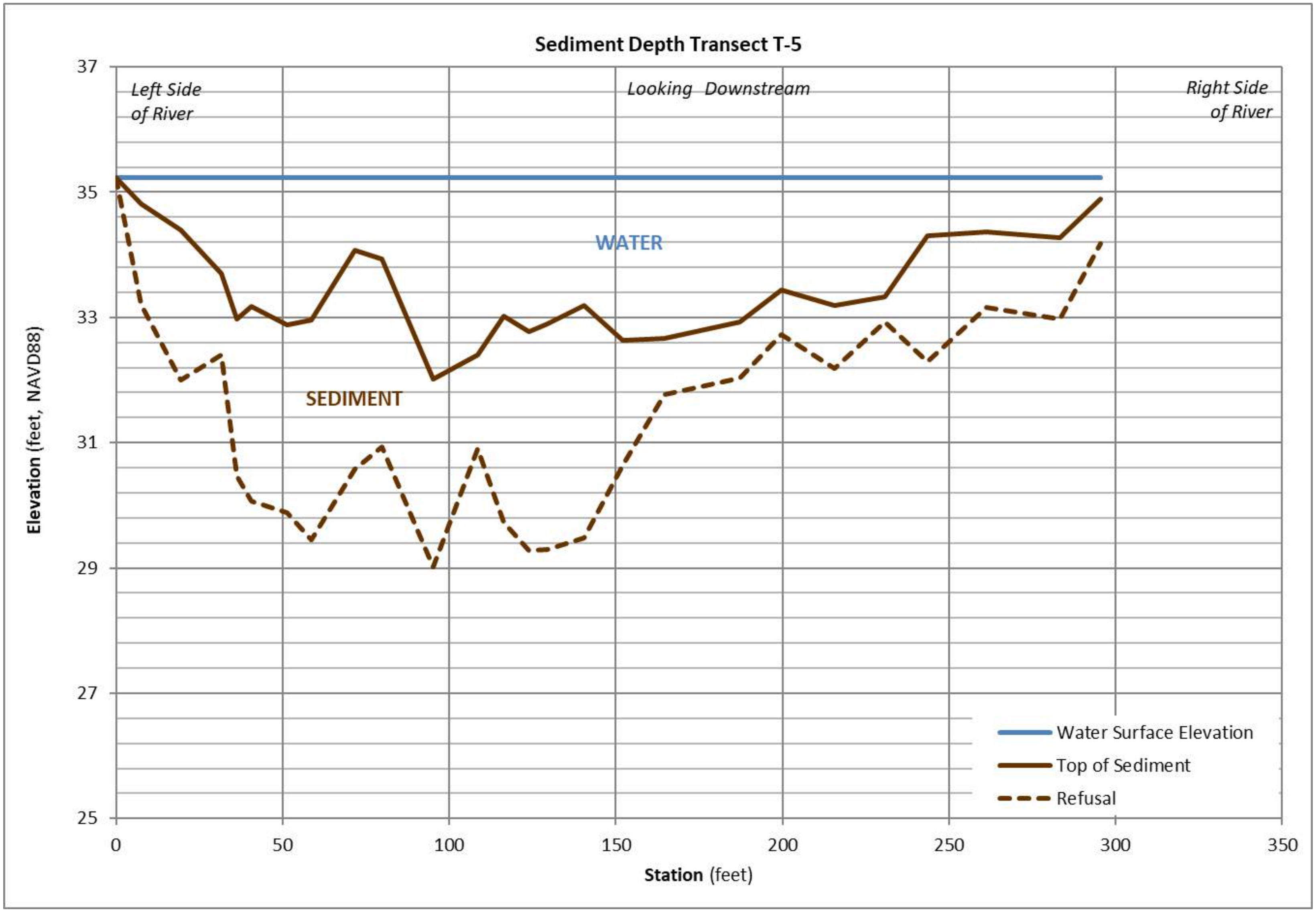


Figure 2.2.2-7: Sediment Probing Transect T-6 Cross-Sectional Plot

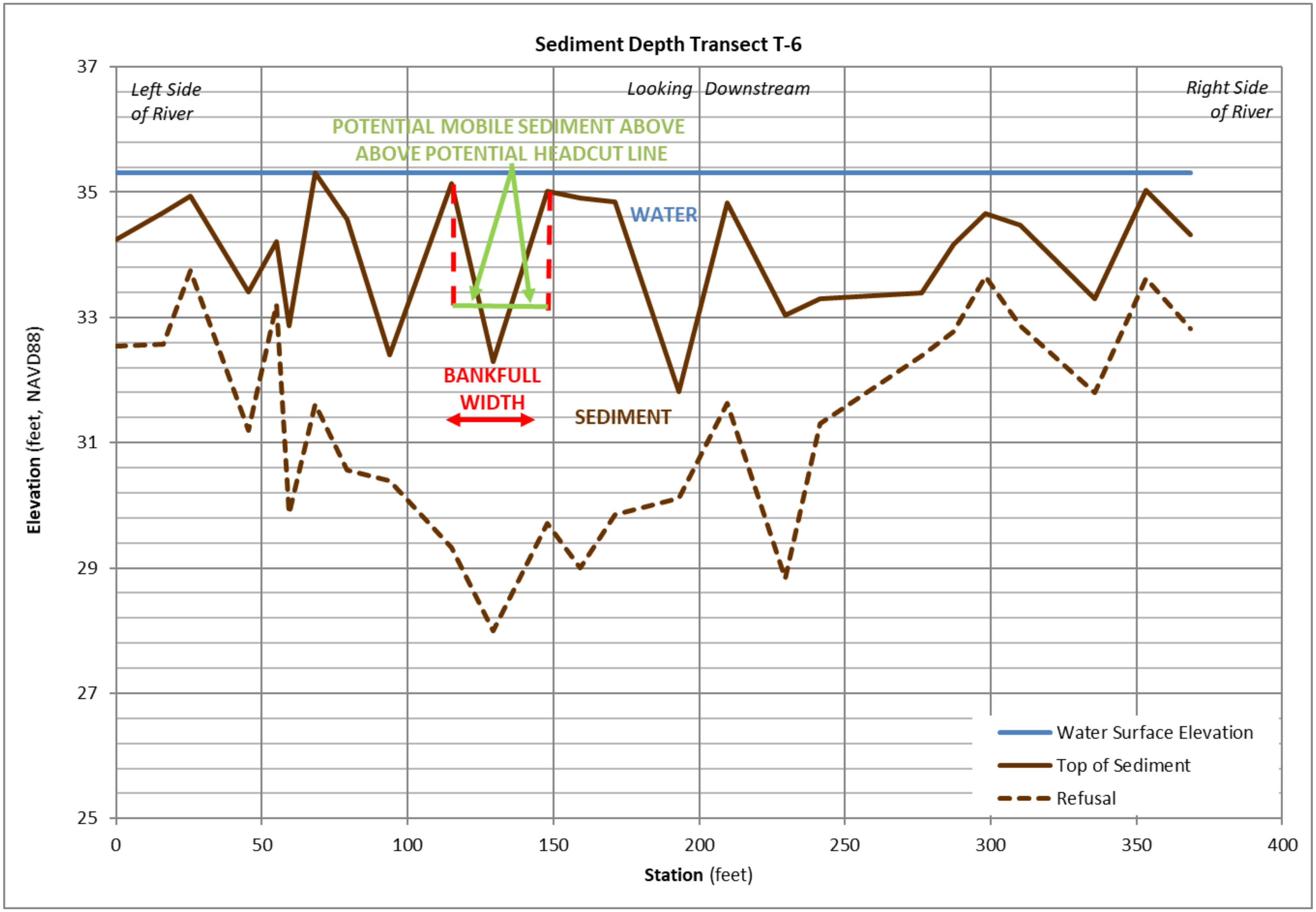


Figure 2.3-1: Regulated Wetland Resource Areas – Existing Conditions

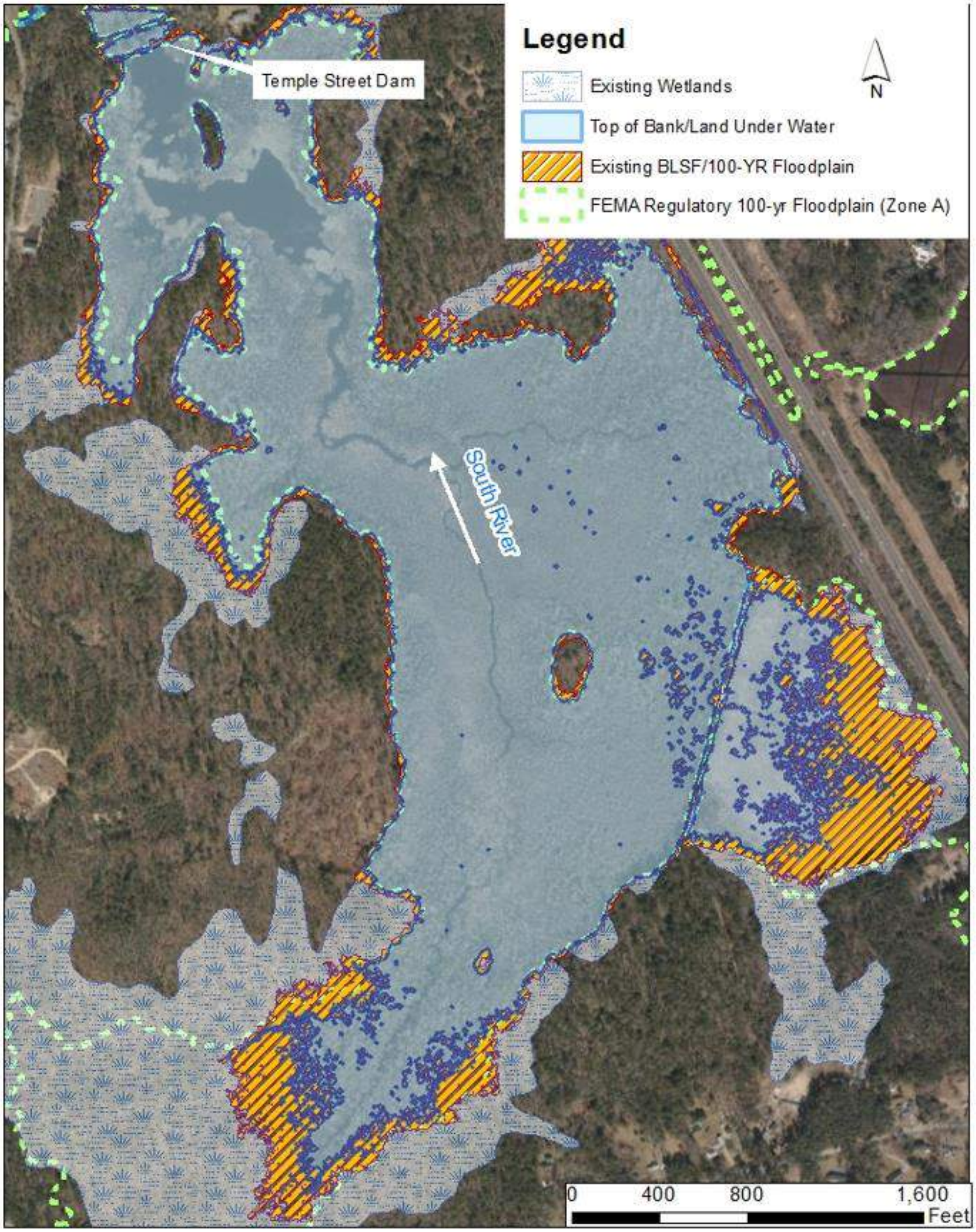


Figure 2.3-2: Regulated Wetland Resource Areas – Proposed Conditions

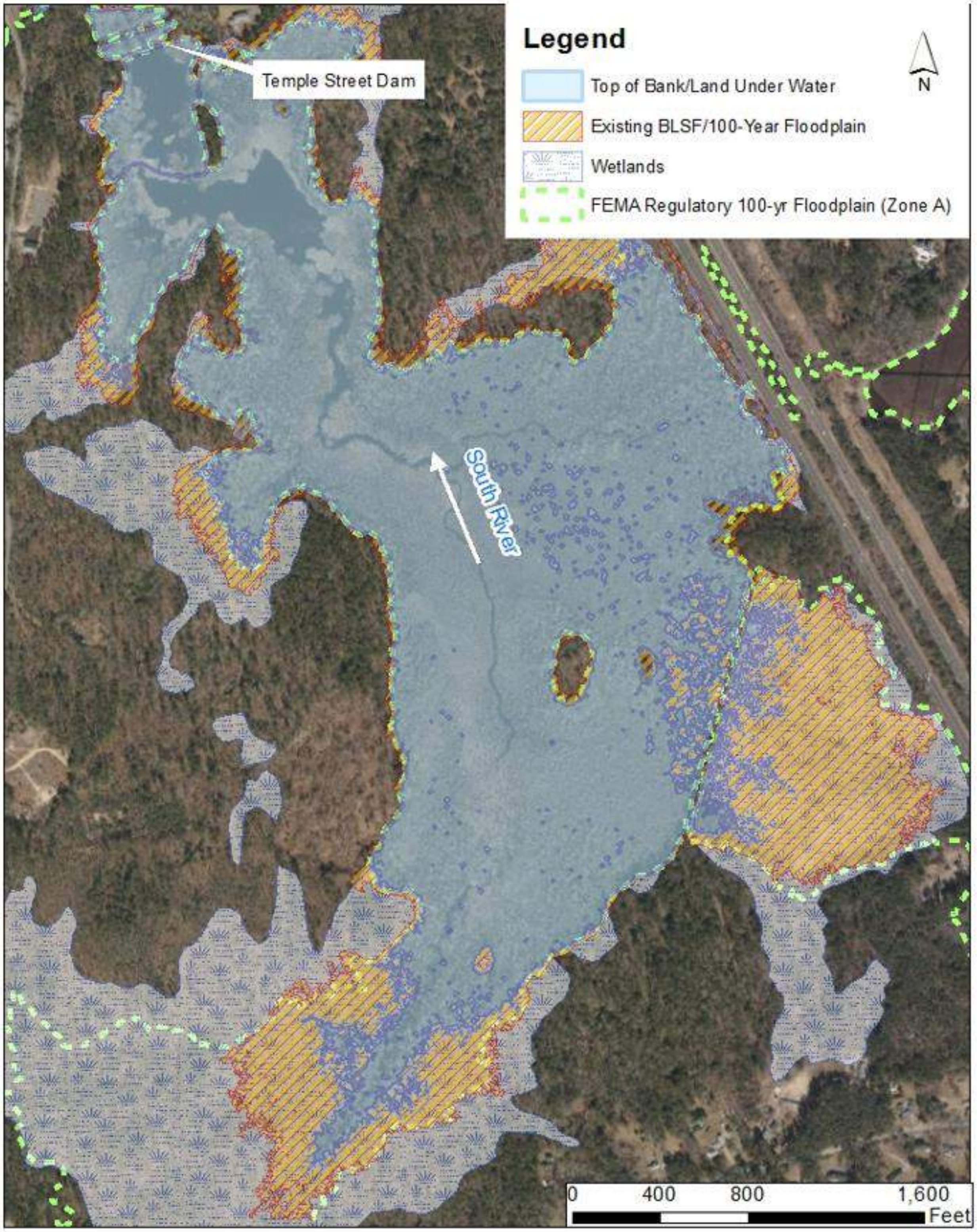


Figure 2.4.1-1: Annual Flow Duration Curve

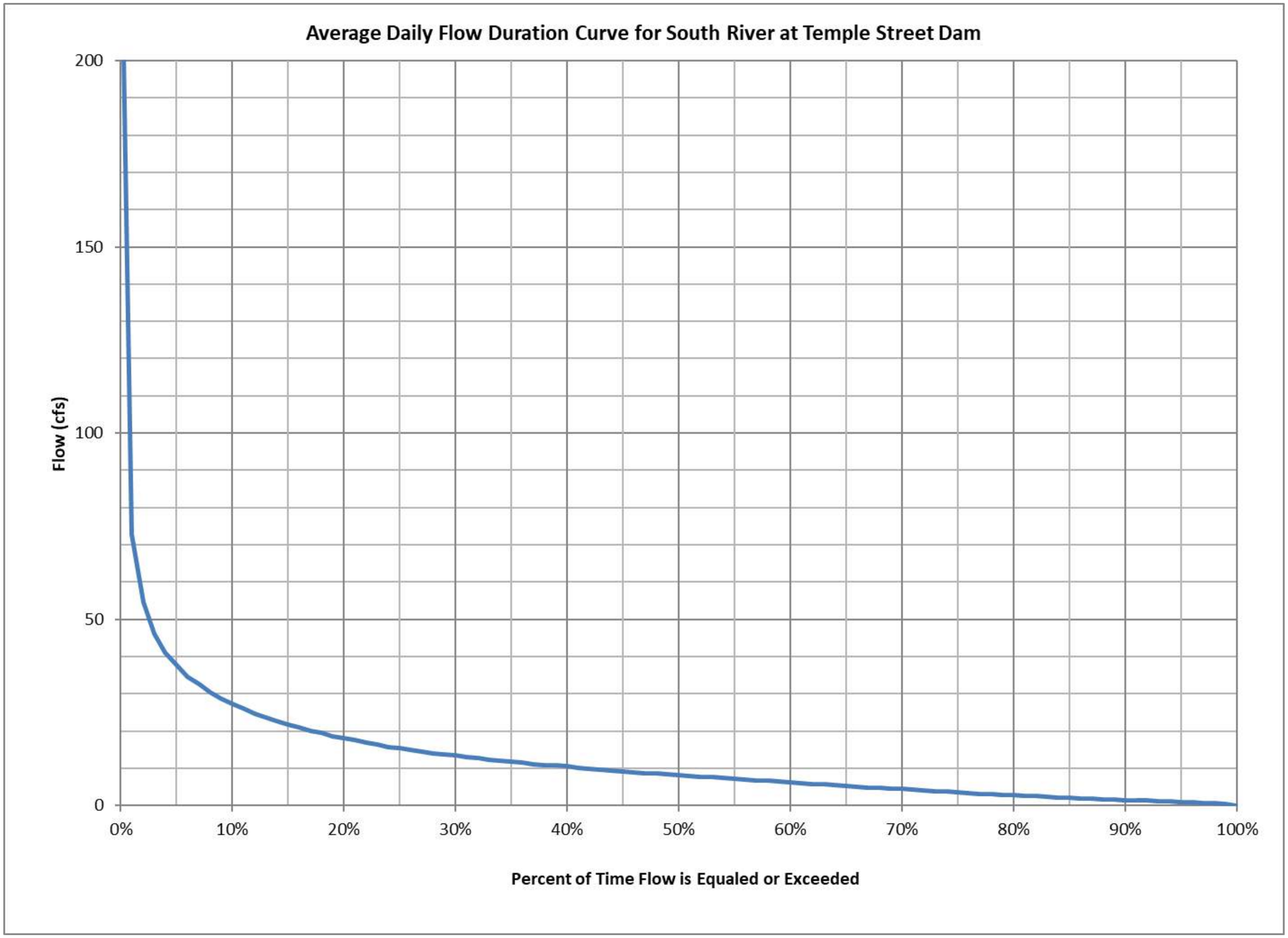


Figure 2.4.1-2: Flow Duration Curves – January, February, and March

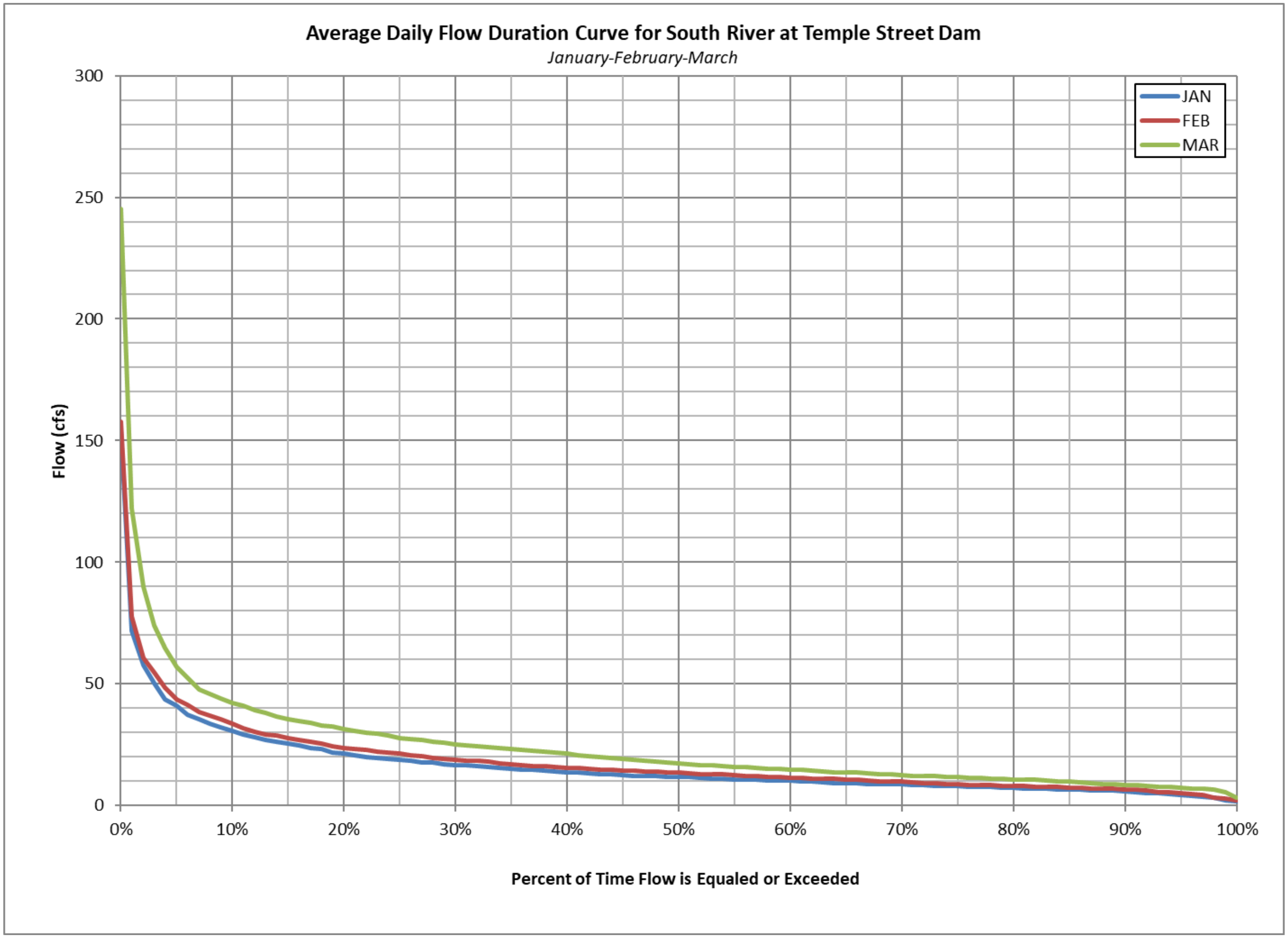


Figure 2.4.1-3: Flow Duration Curves – April, May, and June

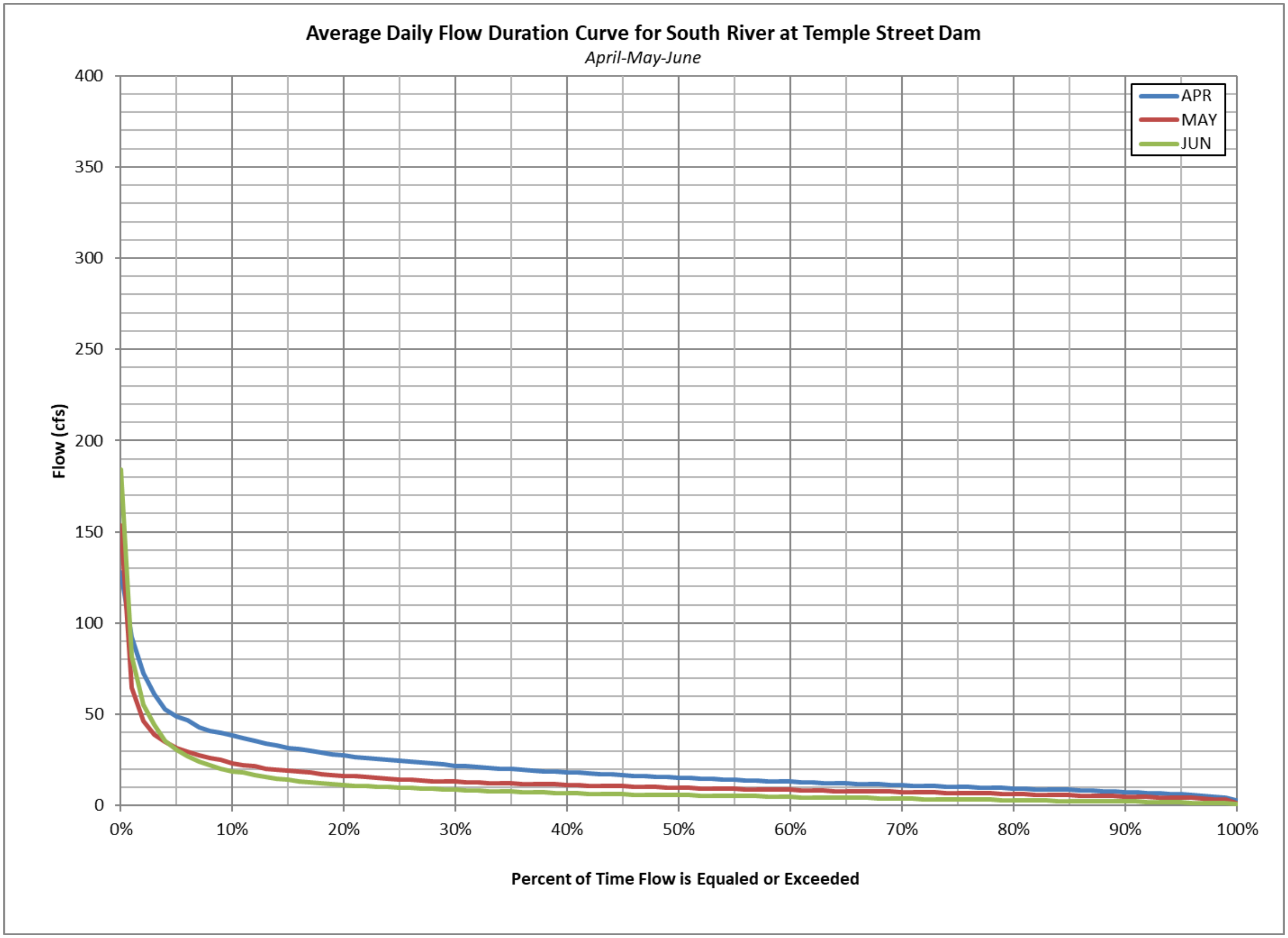


Figure 2.4.1-4: Flow Duration Curves – July, August, and September

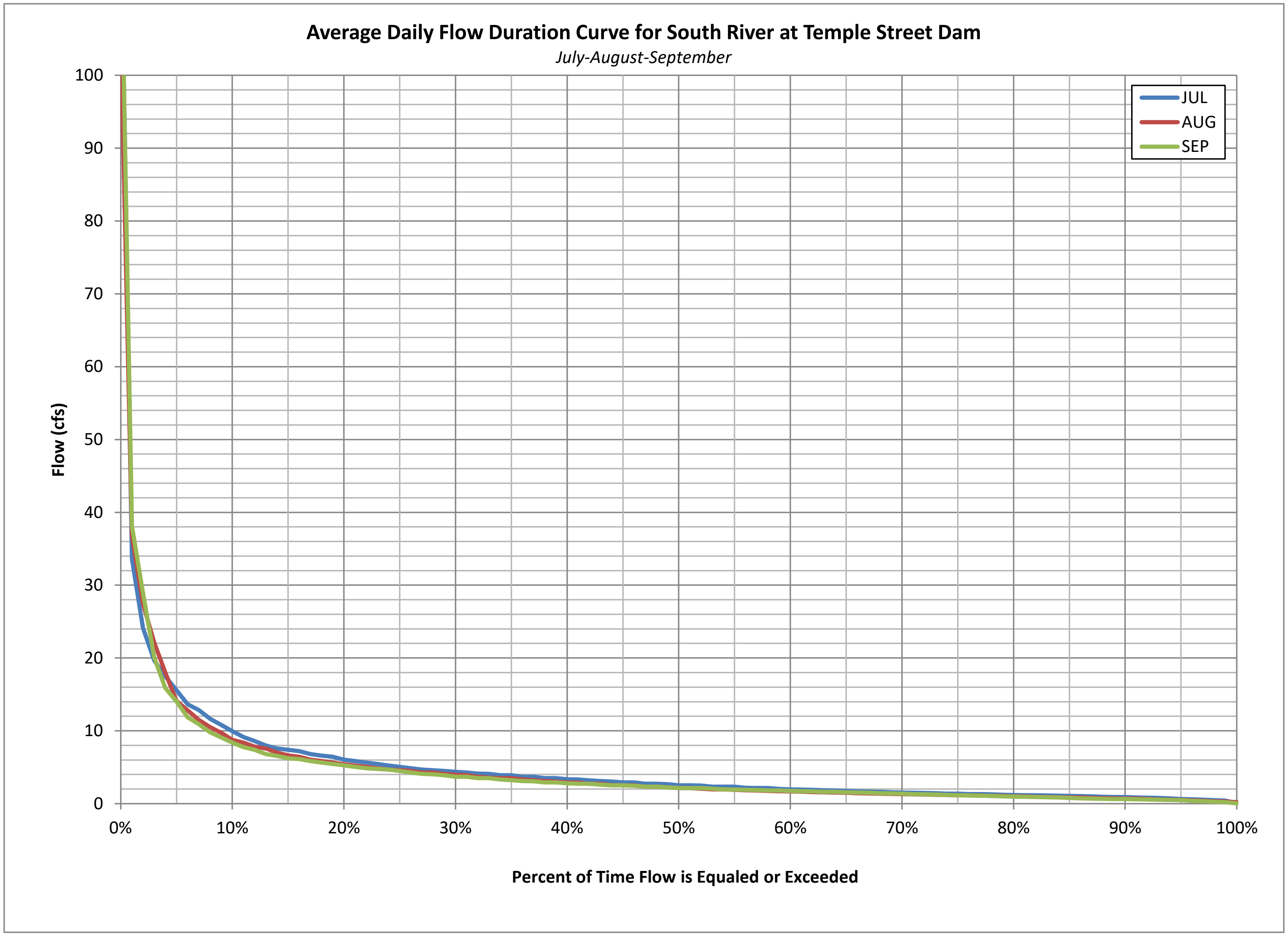


Figure 2.4.1-5: Flow Duration Curves – October, November, December

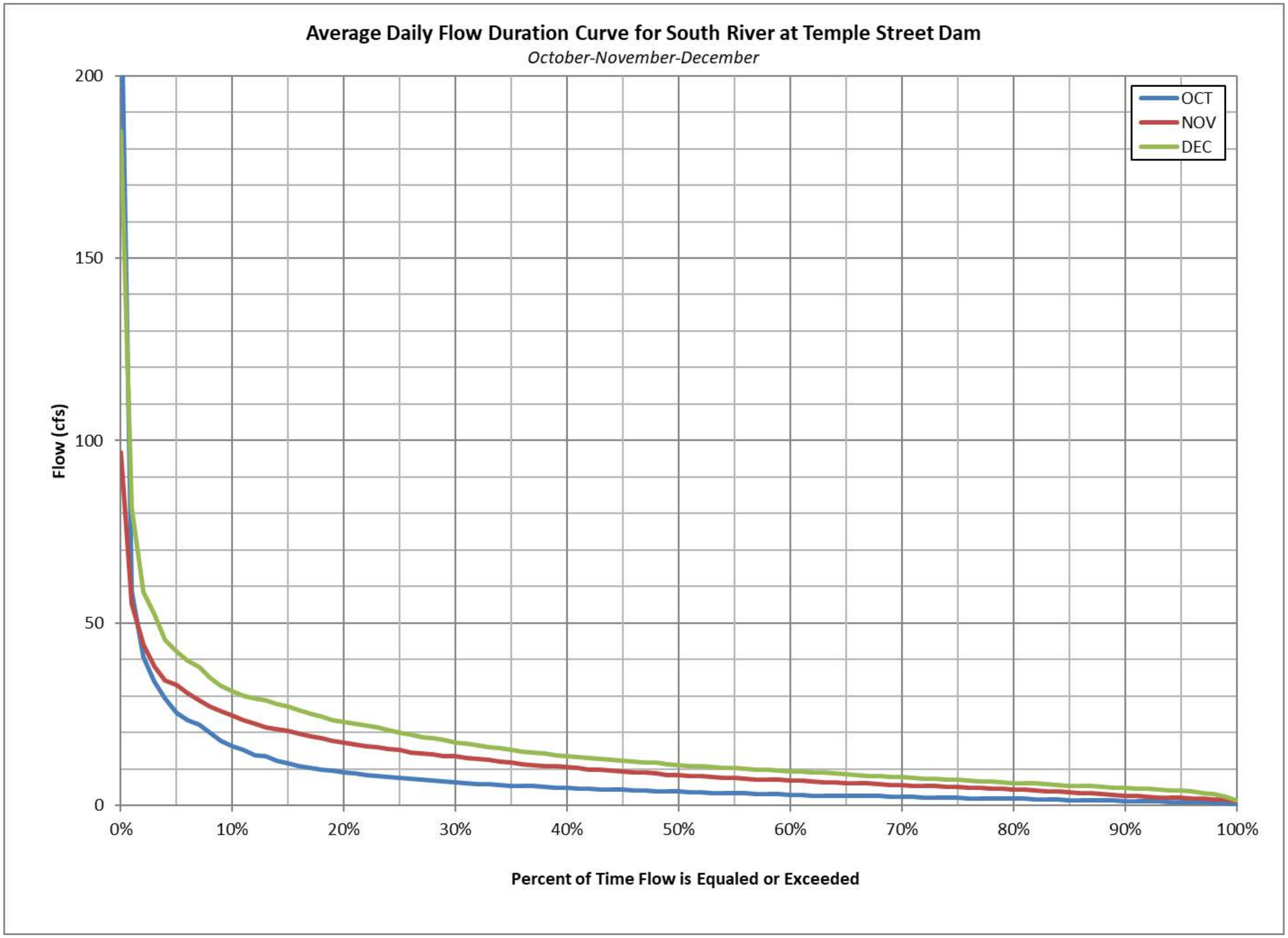


Figure 2.4.1-6: Flow Duration Curves – Upstream Fish Passage Migration Season

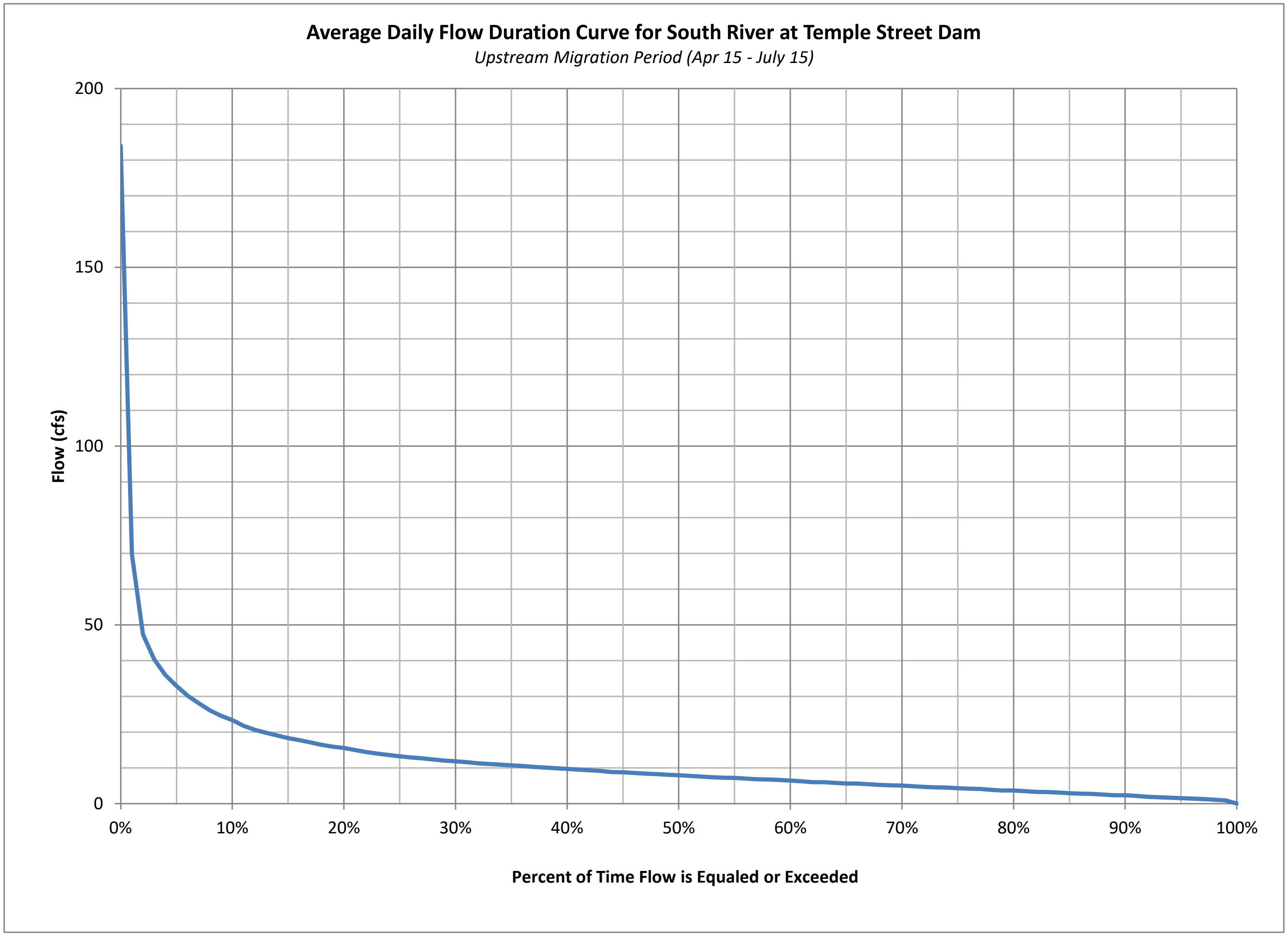


Figure 2.4.1-7: Flow Duration Curves - Downstream Fish Passage Migration Season

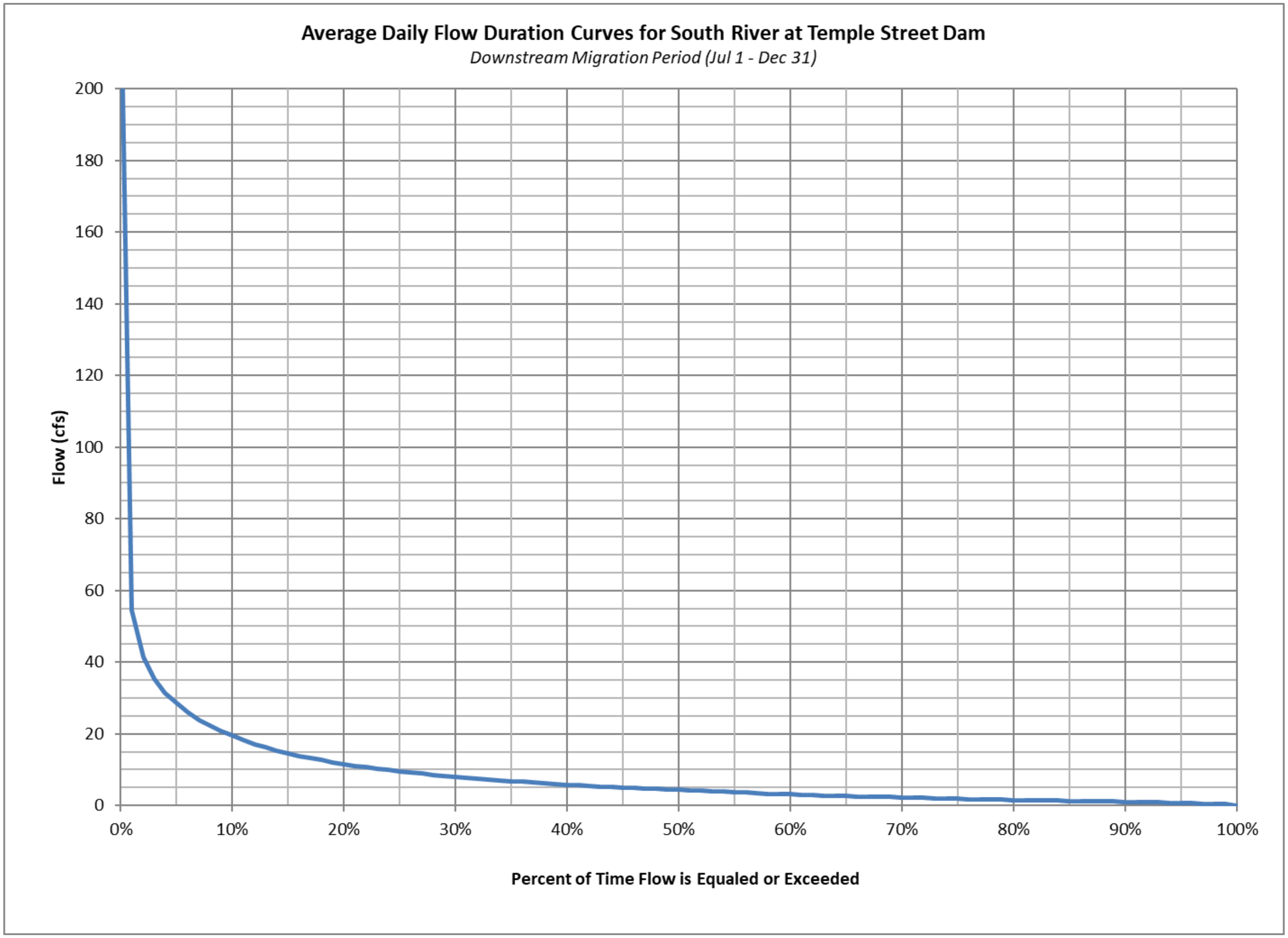


Figure 2.6.2-1 – FEMA FIRMETTE of Project Area – July 6, 2021 Flood Insurance Study Report – Plymouth County, MA

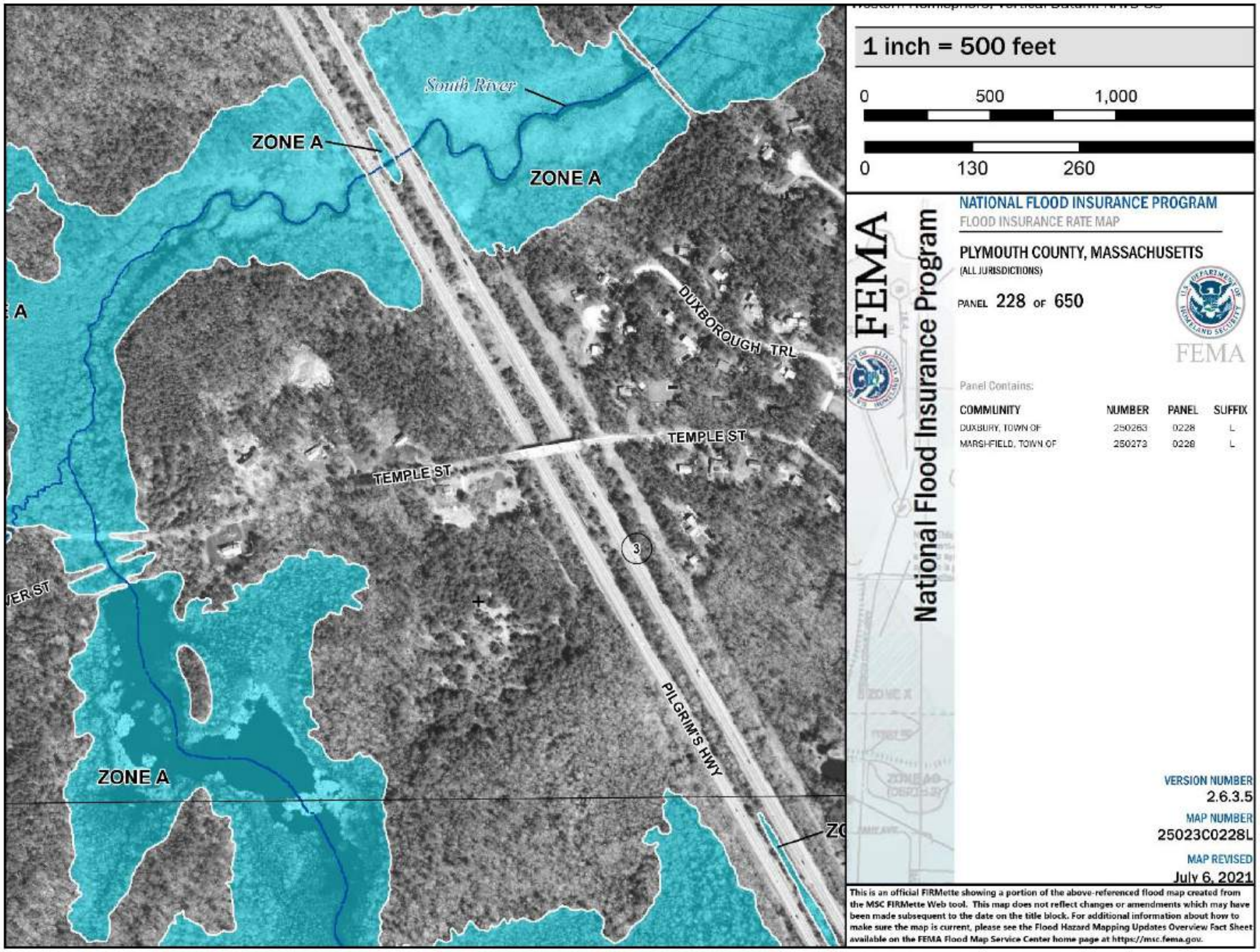


Figure 4.5-1: Parcel and Access Map

Appendix B – Photographs

Appendix B – Photographs

Photo 1: Temple Street Dam Outlet Control Structure (Looking East)

GSE | 06/25/2022

Photo 2: Temple Street Dam Outlet Control Structure (Looking West)

GSE | 06/25/2022

Photo 3: Pull-off along Myrtle/Temple Street

GSE | 06/25/2022

Photo 4: Cart Path from Myrtle/Temple Street towards Temple Street Dam

GSE | 06/25/2022

Photo 5: Cart Path between Pull-off and Dam (Looking East)

GSE | 06/25/2022

Photo 6: Temple Street Dam Embankment and Cart Path (Looking West)

GSE | 06/25/2022

Photo 7: Temple Street Dam Outlet Structure Dagger Boards

GSE | 06/25/2022

Photo 8: Temple Street Concrete Training Walls (note pier in the middle)

GSE | 06/25/2022

Photo 9: Utility Pole at Myrtle/Temple Street Pull-off (looking west)

GSE | 06/25/2022

Photo 10: River Street (Abandoned Road) (looking east)

GSE | 06/25/2022

Photo 11: Riprap Armor at River Street Crossing

GSE | 06/25/2022

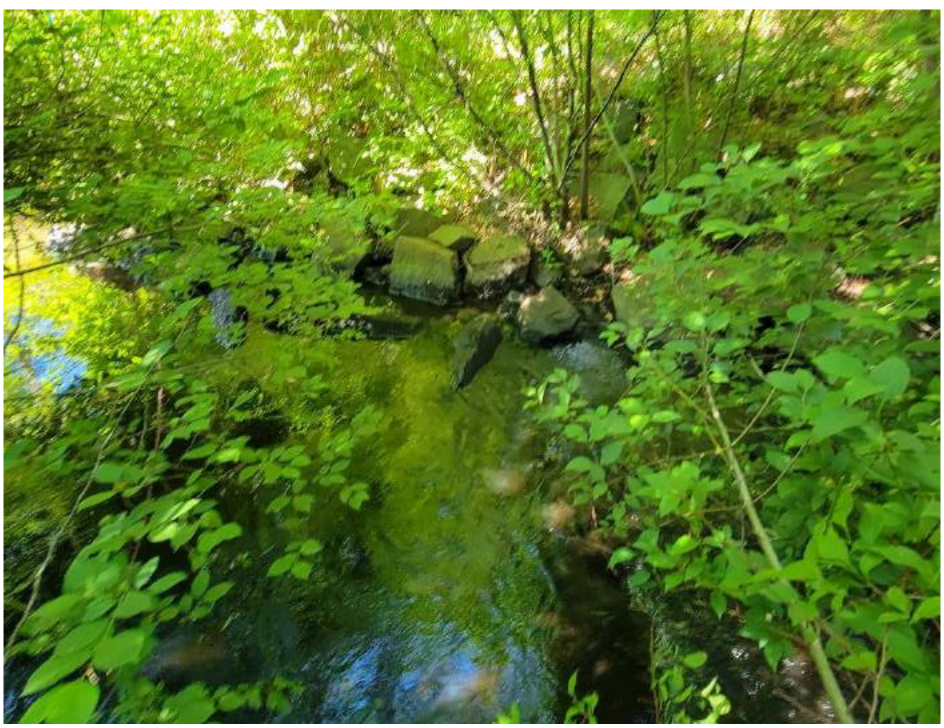


Photo 12: River Street Crossing (looking west)

GSE | 06/25/2022

Photo 13: Myrtle Street Bridge (looking upstream)

GSE | 06/25/2022

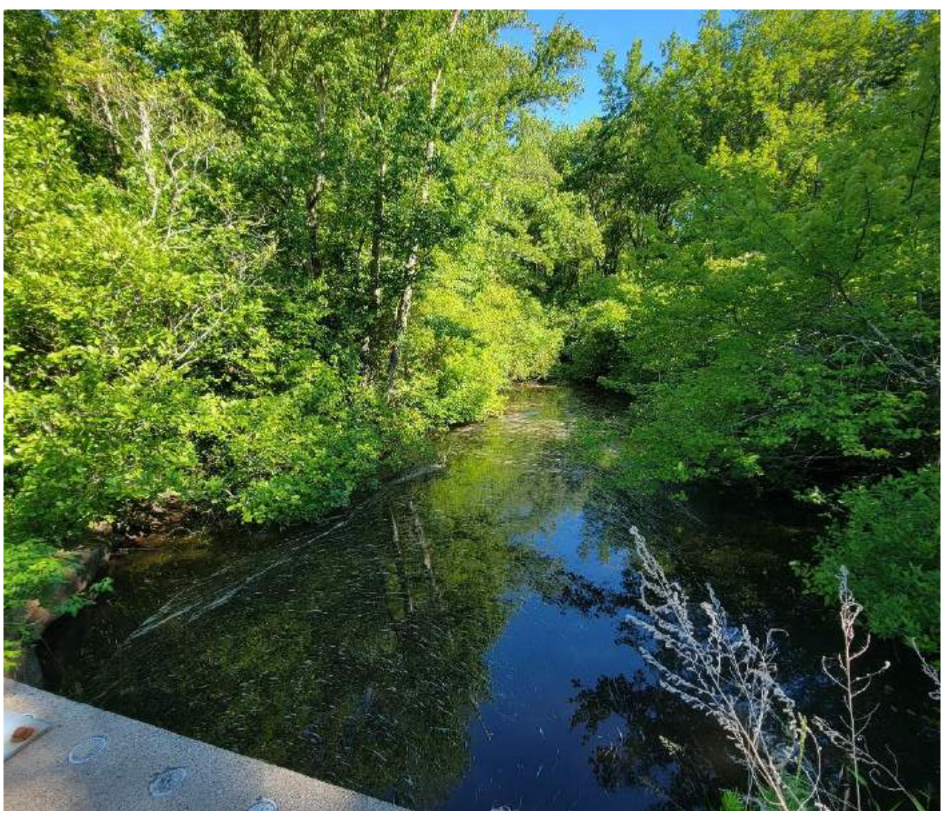


Photo 14: Myrtle Street Bridge (looking downstream)

GSE | 06/25/2022

Photo 15: River Street Access from Keene Street (Looking East)
GSE | 06/25/2022

Photo 16: River Street Access from Keene Street (Looking West)
GSE |06/25/2022

Photo 17: Cart Path Entrance from Keene Street (Looking West)

GSE |06/25/2022

Photo 18: Cart Path from Keene Street (Looking East)

GSE |06/25/2022

Photo 19: View of Impoundment from Cart Path Peninsula (Looking North)

GSE |06/25/2022

Photo 20: Wetlands at Eastern Beaver Dam Analogue Location (Looking South towards Island)

GSE |06/25/2022

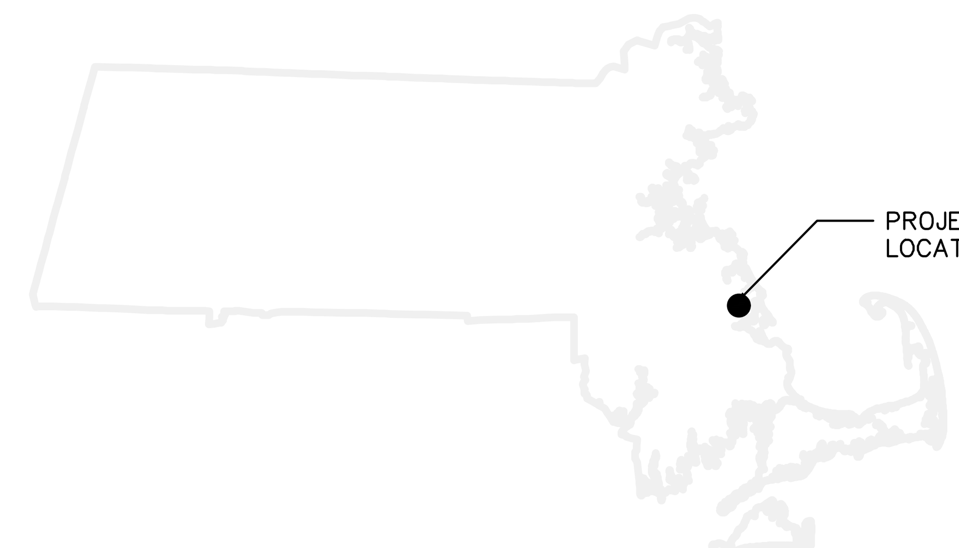
Appendix C – Design Plans

TEMPLE STREET DAM REMOVAL
DUXBURY, MA

MASSACHUSETTS DIVISION OF ECOLOGICAL RESTORATION

251 CAUSEWAY STREET, SUITE 400
BOSTON, MA 02114

SUPPORTED BY PROJECT PARTNERS:
NORTH AND SOUTH RIVERS WATERSHED ASSOCIATION
TOWN OF DUXBURY

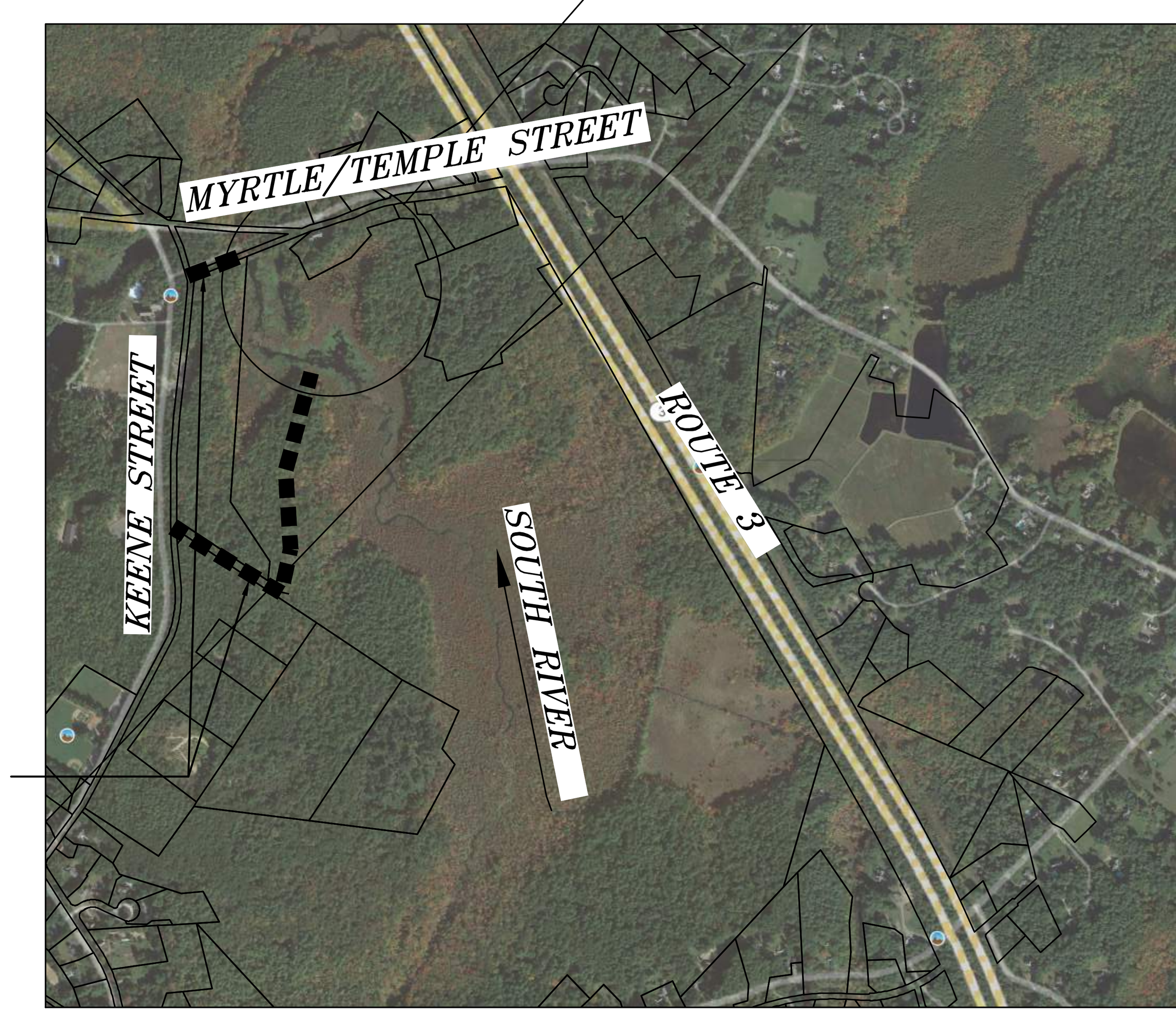


N

SOURCE: USGS SCALE: 1" = 2000'

PROJECT SITE

N



EXISTING CART PATH/POTENTIAL SECONDARY ACCESS

SOURCE: GOOGLE SCALE: 1" = 1000'

75% DESIGN DRAWINGS

DRAWING NO.	TITLE
1	COVER SHEET
2	GENERAL NOTES
3	EXISTING OVERALL SITE PLAN
4	EXISTING TEMPORARY ACCESS, EROSION CONTROL AND WATER CONTROL PLAN
5	PROPOSED OVERALL SITE PLAN
6	PROPOSED PLAN AND PROFILE AND WATER CONTROL PLAN
7	PROPOSED BEAVER DAM ANALOGUE AND LOW FLOW CHANNEL DETAILS
8	DAM REMOVAL ELEVATION AND TYPICAL SECTIONS
9	STREAM RESTORATION DETAILS
10	EROSION AND SEDIMENT CONTROL DETAILS
11	SITE RESTORATION/PLANTING PLAN

COMMONWEALTH OF MASSACHUSETTS
Division of Ecological Restoration

DUXBURY MASSACHUSETTS INCORPORATED JUNE 17 1637

NSRWA

DIG-SAFE
CONTRACTOR SHALL CALL DIG-SAFE CALL CENTER AT 811 OR 1-888-344-7233 AT LEAST 72 HOURS PRIOR TO STARTING ANY EXCAVATION. SATURDAYS, SUNDAYS, AND LEGAL HOLIDAYS ARE NOT TO BE INCLUDED IN THE REQUIRED 72 HOUR NOTICE.

ANY ERRORS OR OMISSIONS SHALL BE REPORTED TO THE ENGINEER WITHOUT DELAY. ALL DESIGNS AND DRAWINGS ARE INSTRUMENTS OF SERVICE OF GOMEZ AND SULLIVAN ENGINEERS, D.P.C. REPRODUCTION OR USE FOR ANY PURPOSE OTHER THAN THAT AUTHORIZED BY GOMEZ AND SULLIVAN, D.P.C. IS DONE AT THE LIABILITY OF THOSE RESPONSIBLE FOR SUCH REPRODUCTION OF USE.

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
6/30/22	1	75% DRAWINGS	BAS	KJC

DRAWN BY: BAS
CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

COVER SHEET

Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
41 Liberty Hill Road
PO Box 2179
Henniker, NH 03242

SCALE: NONE DRAWING: 1

DATA SOURCES

1. HORIZONTAL DATUM IS NORTH AMERICAN DATUM (NAD) 1983, MASSACHUSETTS STATE PLANE COORDINATE SYSTEM, MAINLAND ZONE (FT). VERTICAL DATUM IS NORTH AMERICAN VERTICAL DATUM (NAVD) 1988 FEET.
2. TAX PARCELS ARE BASED ON TAX PARCEL DATA FROM MASSGIS AND THE TOWN OF DUXBURY'S ASSESSOR'S GIS PARCEL MAPS & PROPERTY DATA ONLINE. ACCESSED DECEMBER 2021.
3. TOPOGRAPHIC SURVEYS WERE PERFORMED BY ALPHA SURVEY GROUP, LLC ON JANUARY 9&10, 2018, BY INTER-FLUVE ON APRIL 14-19, 2021 AND JUNE 7, 2021, AND GOMEZ AND SULLIVAN ENGINEERS ON DECEMBER 14-15, 2021.
4. ALL OTHER TOPOGRAPHY OUTSIDE THE SURVEY AREA WAS DERIVED FROM THE "2011 LIDAR FOR THE NORTHEAST" DATASET. OBTAINED FROM THE MASSMAPPER (FORMERLY MASSGIS) CLEARINGHOUSE.
5. WETLAND BOUNDARIES WERE DELINEATED BY LEC ENVIRONMENTAL CONSULTANTS, INC, ON DECEMBER 15, 2021 AND SURVEYED BY GOMEZ AND SULLIVAN ENGINEERS ON DECEMBER 15, 2021. WETLAND BOUNDARIES BEYOND THE LIMITS OF DISTURBANCE ARE FROM THE MASSGIS WETLAND LAYER DEVELOPED BY THE MASSACHUSETTS DEPARTMENT OF ENVIRONMENTAL PROTECTION (MASSDEP) IN 2005.
6. HYDRAULIC ANALYSIS WAS CONDUCTED BY GOMEZ AND SULLIVAN ENGINEERS, DPC AS SUMMARIZED IN A REPORT DATED JUNE 30, 2022.
7. BORDERING LAND SUBJECT TO FLOODING (BLSF) DEPICTED ON THE PLANS IS BASED ON THE 100-YEAR FLOOD INUNDATION MAPPING DEVELOPED BY GOMEZ AND SULLIVAN ENGINEERS FOR EXISTING AND PROPOSED CONDITIONS. THE FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA) 100-YEAR REGULATORY FLOODPLAIN (ZONE A) IS NOT SHOWN FOR REFERENCE SINCE IT IS ASSUMED TO BE LESS ACCURATE THAN THE MODELED BLSF. ZONE A IS SHOWN IN A REPORT DATED JUNE 30, 2022
8. UTILITY LOCATIONS AND ELEVATIONS SHOWN ON THE PLANS ARE BASED SURVEYS AND PLANS OBTAINED FROM THE DUXBURY CONSERVATION COMMISSION BY AMORY ENGINEERS, P.C. DATED OCTOBER 8, 2007. THIS INFORMATION IS NOT TO BE RELIED UPON AS BEING EXACT OR COMPLETE.

GENERAL NOTES

1. CONTRACTOR SHALL CONFIRM THE LOCATION OF ALL UTILITIES PRIOR TO THE COMMENCEMENT OF EXCAVATION. CONTRACTOR SHALL NOTIFY DIG SAFE MASSACHUSETTS AT 811 OR 1-888-344-7233 AT LEAST 72 HOURS PRIOR TO COMMENCING ANY EXCAVATION. SATURDAYS, SUNDAYS, AND LEGAL HOLIDAYS ARE NOT TO BE INCLUDED IN THE REQUIRED 72 HOUR NOTICE.
2. CONTRACTOR SHALL MAINTAIN ALL UTILITIES IN WORKING ORDER AND FREE FROM DAMAGE DURING THE DURATION OF THE PROJECT. NO EQUIPMENT SHALL ENTER OR WORK IN THE EXCLUSION ZONE SHOWN ON THE DRAWINGS OVER THE WATER LINE CROSSING AT RIVER STREET. ANY DAMAGE TO EXISTING UTILITY LINES OR STRUCTURES INCURRED DURING CONSTRUCTION OPERATIONS SHALL BE REPAIRED BY CONTRACTOR AT NO COST TO THE TOWN OR UTILITY COMPANIES. EXCAVATION REQUIRED WITHIN THE PROXIMITY OF EXISTING UTILITY LINES SHALL BE COMPLETED BY HAND.
3. CONTRACTOR SHALL MAINTAIN ALL CONTROL POINTS DURING CONSTRUCTION, INCLUDING BENCHMARKS AND ELEVATIONS AT CRITICAL AREAS. SITE LAYOUT SURVEY REQUIRED FOR CONSTRUCTION SHALL BE PROVIDED BY THE CONTRACTOR AND PERFORMED BY A MASSACHUSETTS' REGISTERED PROFESSIONAL LAND SURVEYOR. ALL GRADE STAKES SET BY SURVEYOR SHALL BE MAINTAINED BY CONTRACTOR UNTIL FINAL INSPECTION OF THE ITEM HAS BEEN COMPLETED BY ENGINEER.
4. EXCESSIVE IDLING DURING THE CONSTRUCTION PERIOD IS PROHIBITED. SIGNS SHALL BE POSTED AT THE SITE LIMITING IDLING TO 5 MINUTES OR LESS. PERIODIC INSPECTIONS SHALL BE CONDUCTED BY SITE SUPERVISORS TO ENSURE COMPLIANCE. STAGING AREAS SHALL BE LOCATED TO MINIMIZE EMISSION IMPACTS TO ABUTTING PROPERTIES.

CONSTRUCTION WASTE MANAGEMENT

1. SITE SHALL BE KEPT WELL ORGANIZED, SIGNED, AND FREE OF WASTE MATERIALS, DEBRIS, AND RUBBISH AT ALL TIMES. GOOD HOUSEKEEPING PRACTICES SHALL BE MAINTAINED ON A CONTINUOUS BASIS FROM WORK SITE TO WORK SITE. DISPOSAL OF ANY WASTE MATERIALS ON THE CONSTRUCTION SITE IS PROHIBITED.
2. SANITARY, WASTE DISPOSAL, AND EMPLOYEE FACILITIES SHALL BE PROVIDED BY CONTRACTOR.
3. ALL WATER RESOURCES (E.G., GROUND AND SURFACE WATERS), INCLUDING ALL DRAINS AND CATCH BASINS, SHALL BE PROTECTED FROM LEACHING AND/OR RUN-OFF OF CHEMICAL POLLUTANTS, SOLID WASTES, AND CONSTRUCTION SITE DEBRIS. ALL CATCH BASINS SHALL BE MAINTAINED FREE FLOWING.
4. ALL COMBUSTIBLE WASTE MATERIALS SHALL BE PLACED IN COVERED METAL CONTAINERS AND PROMPTLY DISPOSED OF IN AN APPROVED MANNER AT AN APPROVED WASTE DISPOSAL FACILITY.
5. STORAGE AND/OR USE OF CHEMICALS, FUELS, OILS, GREASES, BITUMINOUS MATERIALS, SOLIDS, WASTE WASHINGS, AND CEMENT SHALL BE HANDLED APPROPRIATELY AS TO PREVENT LEACHING OR SURFACE RUNOFF INTO PUBLIC WATERS OR DRAINS. ALL APPROVED STORAGE AREAS FOR THESE MATERIALS MUST BE DIKED.
6. ALL ROADWAYS SHALL BE MAINTAINED FREE OF DEBRIS. STABILIZED CONSTRUCTION ENTRANCES SHALL BE CONSTRUCTED TO CAPTURE DEBRIS FROM WHEELS OF CONSTRUCTION VEHICLES. VEHICLES SHALL BE INSPECTED AT ENTRANCES BEFORE TURNING ONTO THE ROADWAY AND EXCESS DEBRIS SHALL BE REMOVED.
7. ALL EXCESS MATERIALS SHALL BE REMOVED FROM THE SITE AS SOON AS POSSIBLE AND IN ACCORDANCE WITH FEDERAL, STATE, AND LOCAL REGULATIONS FOR REUSE AND DISPOSAL.

TRAFFIC CONTROL

1. CONTRACTOR SHALL IMPLEMENT TRAFFIC CONTROL IN ACCORDANCE WITH MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (MUTCD) AND MASSACHUSETTS AMMENDMENTS TO THE MUTCD OR AS DIRECTED OR ORDERED BY OWNER, ENGINEER, OR MUNICIPAL POLICE DEPARTMENT.

TEMPORARY ACCESS ROUTE STABILIZATION

1. DEFINITION: THE STABILIZATION OF TEMPORARY CONSTRUCTION ACCESS ROUTES, ON-SITE VEHICLE TRANSPORTATION ROUTES, AND CONSTRUCTION PARKING AREAS.
2. PURPOSE: TO CONTROL EROSION ON TEMPORARY CONSTRUCTION ROUTES AND PARKING AREAS.
3. CONDITION WHERE PRACTICE APPLIES: ALL TRAFFIC ROUTES AND PARKING AREAS FOR TEMPORARY USE BY CONSTRUCTION TRAFFIC.
4. DESIGN CRITERIA: CONSTRUCTION ROADS SHOULD BE LOCATED TO REDUCE EROSION POTENTIAL, MINIMIZE IMPACT ON EXISTING SITE RESOURCES, AND MAINTAIN OPERATIONS IN A SAFE MANNER. HIGHLY EROSIVE SOILS, WET OR ROCKY AREAS, AND STEEP SLOPES SHOULD BE AVOIDED. ROADS SHOULD BE ROUTED WHERE SEASONAL WATER TABLES ARE DEEPER THAN 18 INCHES. SURFACE RUNOFF AND CONTROL SHOULD BE IN ACCORDANCE WITH OTHER STANDARDS.
5. ROAD GRADE: A MAXIMUM GRADE OF 12% IS RECOMMENDED, ALTHOUGH GRADES UP TO 20% ARE ACCEPTABLE FOR SHORT DISTANCES.
6. ROAD WIDTH: 14 FT (9 FT MINIMUM) FOR ONE-WAY TRAFFIC, OR 24 FT MINIMUM FOR TWO-WAY TRAFFIC.
7. SIDE SLOPE OF ROAD EMBANKMENT: 2:1 OR FLATTER.
8. COMPOSITION: USE AN 8-INCH LAYER OF STATE DOT APPROVED GRAVEL SUB-BASE OR EQUIVALENT, PLACED ON A GEOTEXTILE FABRIC.
9. MAINTENANCE: ACCESS ROUTES AND PARKING AREAS SHALL BE INSPECTED PERIODICALLY FOR CONDITION OF SURFACE AND TOPDRESSED WITH NEW GRAVEL AS NEEDED.
10. RESTORATION: UPON COMPLETION OF THE WORK, ALL TEMPORARY MATERIALS SHALL BE REMOVED AND THE SITE SHALL BE RESTORED TO PRE-PROJECT CONDITIONS.

SOIL EROSION AND SEDIMENTATION CONTROL

1. ALL WORK SHALL BE CONDUCTED IN ACCORDANCE WITH THE MASSACHUSETTS DEPARTMENT OF ENVIRONMENTAL PROTECTION EROSION AND SEDIMENTATION CONTROL GUIDELINES AND APPLICABLE NPDES STANDARDS.
2. ALL APPLICABLE SOIL EROSION AND SEDIMENT CONTROL PRACTICES ARE TO BE INSTALLED PRIOR TO ANY SOIL OR STREAM DISTURBANCE, OR IN THEIR PROPER SEQUENCE, AND MAINTAINED UNTIL PERMANENT PROTECTION IS ESTABLISHED.
3. ALL DISTURBED AREAS THAT WILL BE LEFT EXPOSED MORE THAN FOURTEEN (14) DAYS, AND NOT SUBJECT TO CONSTRUCTION TRAFFIC, SHALL IMMEDIATELY RECEIVE A TEMPORARY SEEDING WITH A NATIVE SEED MIXTURE. MULCH, WATER AND ANCHOR AS NECESSARY TO ESTABLISH GRASS AND PREVENT LOSS TO WIND OR EROSION. IF THE SEASON PREVENTS THE ESTABLISHMENT OF A TEMPORARY COVER, THE DISTURBED AREAS SHALL BE MULCHED WITH SMALL GRAIN STRAW AT A RATE OF TWO (2) TONS PER ACRE IN ACCORDANCE WITH STATE STANDARDS.
4. PERMANENT VEGETATION SHALL BE SEEDED WITH A NATIVE SEED MIXTURE ON ALL EXPOSED AREAS IMMEDIATELY AFTER FINAL GRADING. MULCH SHALL BE USED AS NECESSARY FOR PROTECTION UNTIL SEEDING IS ESTABLISHED.
5. ALL CRITICAL AREAS SUBJECT TO EROSION SHALL RECEIVE A TEMPORARY SEEDING WITH AN APPROVED NATIVE SEED MIXTURE IN COMBINATION WITH STRAW MULCH, AT A RATE OF TWO (2) TONS PER ACRE IN ACCORDANCE WITH STATE STANDARDS.
6. SHOULD THE CONTROL OF DUST AT THE SITE BE NECESSARY, THE SITE SHALL BE SPRINKLED WITH WATER UNTIL THE SURFACE IS WET, TEMPORARY VEGETATIVE COVER SHALL BE ESTABLISHED, OR MULCH SHALL BE APPLIED IN ACCORDANCE WITH STATE STANDARDS FOR EROSION CONTROL.
7. ALL SOIL WASHED, DROPPED, SPILLED, OR TRACKED OUTSIDE THE LIMIT OF DISTURBANCE OR ONTO PUBLIC RIGHTS-OF-WAY SHALL BE REMOVED IMMEDIATELY.
8. STOCKPILE AND STAGING LOCATIONS DETERMINED IN THE FIELD SHALL BE PLACED WITHIN THE LIMIT OF DISTURBANCE. ALL SOIL STOCKPILES SHALL BE TEMPORARILY STABILIZED IN ACCORDANCE WITH NOTE #3 AND PROTECTED BY COMPOST FILTER SOCKS ON THE DOWNHILL SIDES.
9. THE CONTRACTOR SHALL INSPECT DISTURBED AREAS OF THE CONSTRUCTION SITE, AREAS USED FOR STORAGE OF MATERIALS THAT ARE EXPOSED TO PRECIPITATION AND THAT HAVE NOT BEEN FINALLY STABILIZED, STABILIZATION PRACTICES, STRUCTURAL PRACTICES, AND OTHER CONTROLS AT LEAST ONCE EVERY SEVEN (7) CALENDAR DAYS AND WITHIN 24 HOURS AFTER THE END OF ANY STORM THAT PRODUCES AT LEAST 0.5 INCHES OF RAINFALL AT THE SITE. WHERE SITES HAVE BEEN FINALLY STABILIZED, SUCH INSPECTION SHALL BE CONDUCTED AT LEAST ONCE EVERY MONTH UNTIL FINAL COMPLETION. CRITICAL AREAS AND AREAS WHERE VEHICLES EXIT THE SITE SHALL BE INSPECTED DAILY.

CONSTRUCTION SEQUENCE

SITE PREPARATION AND ACCESS

1. CONTRACTOR SHALL PREPARE A CONSTRUCTION SEQUENCE PLAN TO BE APPROVED BY OWNER AND ENGINEER. THE FOLLOWING GENERAL SEQUENCE SHALL BE ADAPTED FOR THE SITE-SPECIFIC REQUIREMENTS.
2. SURVEY AND STAKE THE PROPOSED LIMIT OF DISTURBANCE AND LIMIT OF EROSION CONTROLS. INSTALL EROSION CONTROLS AND CONTAINMENT MEASURES AS INDICATED IN THE PLANS.
3. FLAG LIMITS OF CLEARING, TO BE APPROVED BY OWNER PRIOR TO ANY TREE REMOVAL. CLEAR AND GRUB ALONG APPROVED ACCESS ROUTES AS NEEDED.
4. INSTALL STAGING AREA AND TEMPORARY ACCESS RAMPS/ROUTES AS NEEDED. UTILIZE SWAMP MATS (OR APPROVED EQUAL) TO MINIMIZE DISTURBANCE TO WETLAND AREAS.
5. INSTALL OIL BOOM AND TURBIDITY CURTAINS.

PHASE I - (RIVER WORK)

1. LEAVE DAGGER BOARDS IN PLACE AT THE DAM.
2. FLOAT LOG PIECES INTO THE IMPOUNDMENT AND TEMPORARILY ANCHOR THEM IN PLACE.

PHASE II - (RIVER WORK)

1. INSTALL SUPERSACK COFFERDAM (OR APPROVED EQUAL) TO FACILITATE FLOW THROUGH SITE WHILE CONSTRUCTING RIFFLE FEATURE AT RIVER STREET.
2. CONSTRUCT PROPOSED RIFFLE FEATURE AT RIVER STREET AS SHOWN. RIFFLE SHALL BE CONSTRUCTED IN 15-INCH LIFTS ABOVE FILTER LAYER AND FINE MATERIAL SHALL BE USED TO CHOKE EACH LIFT PRIOR TO PLACEMENT OF THE SUBSEQUENT LIFT. THE CONTRACTOR SHALL WASH THE FINE MATERIAL INTO THE LIFT OF COARSE MATERIAL WITH A SUFFICIENT QUANTITY OF WATER.
3. RELOCATE SUPERSACKS AT THE ENDS OF THE COFFERDAM AS REQUIRED TO SWITCH FLOWS TO THE OTHER SIDE TO CONSTRUCT THE OTHER HALF OF THE RIFFLE FEATURE.
4. ONCE RIFFLE FEATURE AT RIVER STREET IS CONSTRUCTED RELOCATE SUPERSACK COFFERDAMS/CONTROLS TO FACILITATE THE REMOVAL OF THE DAM IN PHASE III.

PHASE III - (RIVER WORK)

1. INSTALL SUPERSACK COFFERDAM (OR APPROVED EQUAL) TO FACILITATE FLOW THROUGH SITE WHILE REMOVING THE DAM AND CONSTRUCTING THE UPSTREAM RIFFLE FEATURE.
2. REMOVE SPILLWAY DAGGER BOARDS TO DRAIN THE IMPOUNDMENT.
3. REMOVE THE FULL VERTICAL EXTENT OF THE CONCRETE TRAINING WALLS, PIER, DECK, AND APRON OF THE DAM SPILLWAY OUTLET STRUCTURE. REMOVE ALL CONCRETE FROM THE RIVER.
4. CONSTRUCT PROPOSED RIFFLE FEATURE AT THE DAM AS INDICATED. RIFFLE SHALL BE CONSTRUCTED IN 15-INCH LIFTS ABOVE FILTER LAYER AND FINE MATERIAL SHALL BE USED TO CHOKE EACH LIFT PRIOR TO PLACEMENT OF THE SUBSEQUENT LIFT. THE CONTRACTOR SHALL WASH THE FINE MATERIAL INTO THE LIFT OF COARSE MATERIAL WITH A SUFFICIENT QUANTITY OF WATER.
5. REMOVE WATER CONTROLS FROM THE DAM LOCATION.

STREAM RESTORATION

1. INSTALL BEAVER DAM ANALOGS (BDA) TO CREST ELEVATION 38.0 FT AS SHOWN ON THE PLANS AND CONSTRUCT THE LOW FLOW CHANNEL AT THE BOUNDARY OF THE LOWER AND UPPER IMPOUNDMENTS. BDA'S SHALL TIE INTO GRADE AT EACH END EXCEPT AT LOW FLOW CHANNEL BREACH.
2. REMOVE TEMPORARY ANCHORS AND INSTALL PERMANENT ANCHORS FOR LARGE WOODY DEBRIS AS SHOWN ON THE PLANS.
3. PLANT LIVE STAKES IN THE FLOODPLAIN AREAS WITH NATIVE SPECIES PER THE PLANTING PLAN ALONG THE WATER'S EDGE.

SITE RESTORATION

1. REMOVE ANY REMAINING WATER CONTROLS FROM THE SITE.
2. REMOVE CRUSHED STONE, STONE FILL AND GEOTEXTILE FABRIC FOR TEMPORARY ACCESS PATHS AND AT THE CONSTRUCTION ENTRANCE.
3. REPAIR PAVED PARKING AREA, TO THE SATISFACTION OF THE OWNER'S REPRESENTATIVE, IF NECESSARY.
4. REMOVE EROSION CONTROL AND OTHER CONTAINMENT MEASURES ONLY AFTER ALL AREAS ARE STABILIZED WITH VEGETATIVE COVER TO THE SATISFACTION OF OWNER'S REPRESENTATIVE.
5. EXCAVATED SEDIMENT SHALL BE SPREAD ACROSS ANY DISTURBED AREAS IN A 4" LAYER AND WILL BE SEEDED WITH AN APPROVED NATIVE SEED MIXTURE.

ROAD
TAX PARCEL BOUNDARIES
TREE LINE
TOP OF BANK (TOB)
MEAN ANNUAL HIGH WATER (MAHW)
ORDINARY HIGH WATER (OHW)
BORDERING VEGETATED WETLAND (BVW)
100' BANK BUFFER
200' RIVERFRONT AREA LIMIT
100-YR FLOODPLAIN
100-YR FLOODPLAIN WITH DAM REMOVED
OVERHEAD ELECTRICAL LINES — OHE — OHE —
UNDERGROUND WATER PIPE — W — W — W —
COMPOST FILTER SOCK — X — X — X — X —
OIL BOOM
EXISTING CONTOUR
PROPOSED CONTOUR
CONCRETE
RIPRAP (D_{50} = 12")
RIPRAP (D_{50} = 9")
GRAVEL ACCESS SURFACE
CLEARING EXTENTS
REMOVAL EXTENTS
SEDIMENT AND SEED
SEDIMENT DISPOSAL AREA
DISTURBED AREA
ACCESS ROUTE

RESOURCE AREA IMPACTS

RESOURCE	AREA (SQUARE FEET, SF)			TEMPORARY (SF)	PERMANENT (SF)
	EXISTING	CHANGE	TOTAL		
BANK (FT)	40,404	3,516	43,920	3,435	3,435
BORDERING VEGETATED WETLANDS	5,776,234	1,020,141	6,796,375	4,197	110
ISOLATED VEGETATED WETLANDS	0	0	0	0	0
LAND UNDER WATER/OHW/MAHW	7,432,053	-1,183,394	6,248,659	246,923	10,026
BORDERING LAND SUBJECT TO FLOODING	1,602,029	1,140,571	2,742,600	1,769	686
ISOLATED LAND SUBJECT TO FLOODING	0	0	0	0	0
RIVERFRONT AREA	110,491	0	110,491	10,202	8,050
FISH RUNS (LF)	216	4,977	5,193	879	380

NOTE:

1. RESOURCE AREAS BASED ON THE DOWNSTREAM LIMIT OF WORK JUST UPSTREAM OF THE SOUTH RIVER MYRTLE STREET CROSSING UP TO THE UPSTREAM END OF THE IMPOUNDMENT.

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
6/30/22	1	75% DRAWINGS	BAS	KJC

DRAWN BY: BAS
CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

GENERAL NOTES

Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
41 Liberty Hill Road
PO Box 2179
Henniker, NH 03242

SCALE: NONE
DRAWING: 2

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

EXISTING OVERALL SITE PLAN

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
6/30/22	1	75% DRAWINGS	BAS	KJC

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CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

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Gomez and Sullivan Engineers, D.P.C.
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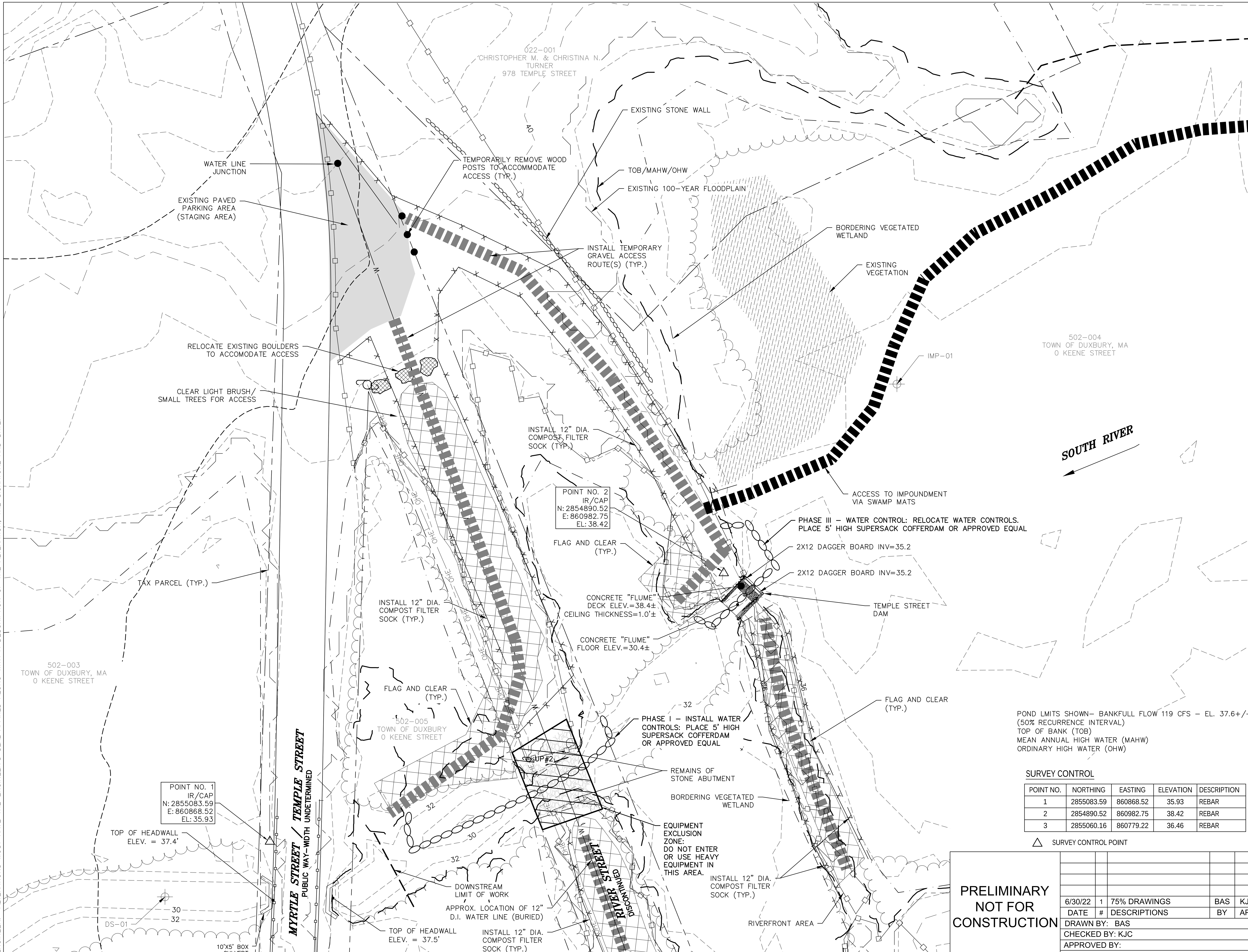
Henniker, NH 03242

SCALE: 1" □ 50□

DRAWING: 3



CARE AND DIVERSION OF WATER

1. CONTRACTOR SHALL PROVIDE, MAINTAIN, AND REMOVE ALL DIVERSIONS, COFFERDAMS, PIPING, AND/OR PUMPS AS REQUIRED TO PERFORM THE WORK AND BYPASS SOUTH RIVER FLOWS AROUND THE WORK SITE DURING CONSTRUCTION.
2. ALL DIVERSIONS, COFFERDAMS, PIPING AND PUMPS SHALL BE DESIGNED BY A LICENSED PROFESSIONAL ENGINEER REGISTERED TO PRACTICE IN MASSACHUSETTS HIRED BY THE CONTRACTOR AND SUBMITTED TO OWNER AND THE ENGINEER FOR REVIEW AND APPROVAL PRIOR TO USE.
3. CONTRACTOR SHALL COORDINATE INSTALLATION OF ALL DIVERSIONS AND COFFERDAMS WITH OWNER AND IN ACCORDANCE WITH ALL PERMIT REQUIREMENTS.
4. CONTRACTOR SHALL DEVELOP A PLAN TO SECURE THE WORK SITE AND BREACH THE DIVERSION AND/OR COFFERDAM WITH MINIMAL RELEASE OF SEDIMENT IN THE EVENT OF A STORM EVENT GREATER THAN THE BYPASS SYSTEM CAPACITY. THE PLAN SHALL ADDRESS MEASURES REQUIRED TO PASS THE STORM FLOW AND SHALL BE SUBMITTED TO OWNER FOR APPROVAL.
5. MEAN FLOW IN THE SOUTH RIVER IS APPROXIMATELY 5 CFS FOR THE MONTHS OF JULY THROUGH DECEMBER WITH A 10% CHANCE OF EXCEEDANCE OF 11 CFS BASED ON DATA INTERPOLATED FROM THE INDIAN HEAD RIVER AT HANOVER, MA USGS GAGE NO. 01105730 FOR THE PERIOD OF RECORD ADJUSTED TO PROJECT SITE BASED ON DRAINAGE AREA RATIO.
6. WORK SHALL COMPLY WITH TIME-OF-YEAR RESTRICTIONS RELATIVE TO WORK IN THE RIVER. NO IN-WATER CONSTRUCTION OR ACTIVITIES CONTRIBUTING SILT OR SEDIMENT TO THE SOUTH RIVER SHALL BE CONDUCTED FROM APRIL 15 THROUGH JULY 15.

LEGEND

30 MAJOR CONTOUR
28 MINOR CONTOUR
TOP OF BANK (TOB)/MAHW/OHW
100-YEAR FLOODPLAIN
WETLAND BOUNDARY
TAX PARCEL
APPROXIMATE THALWEG
100' WETLAND BUFFER
TREELINE
SEDIMENT SAMPLING LOCATION

SURVEY CONTROL

POINT NO.	NORTHING	EASTING	ELEVATION	DESCRIPTION
1	2855083.59	860868.52	35.93	REBAR
2	2854890.52	860982.75	38.42	REBAR
3	2855060.16	860779.22	36.46	REBAR

△ SURVEY CONTROL POINT

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TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

EXISTING CONDITIONS, ACCESS, EROSION CONTROL, AND WATER CONTROL PLAN

Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
41 Liberty Hill Road
PO Box 2179
Henniker, NH 03242

SCALE: 1" □ 20□
DRAWING: 4

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

PROPOSED OVERALL SITE PLAN

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
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APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

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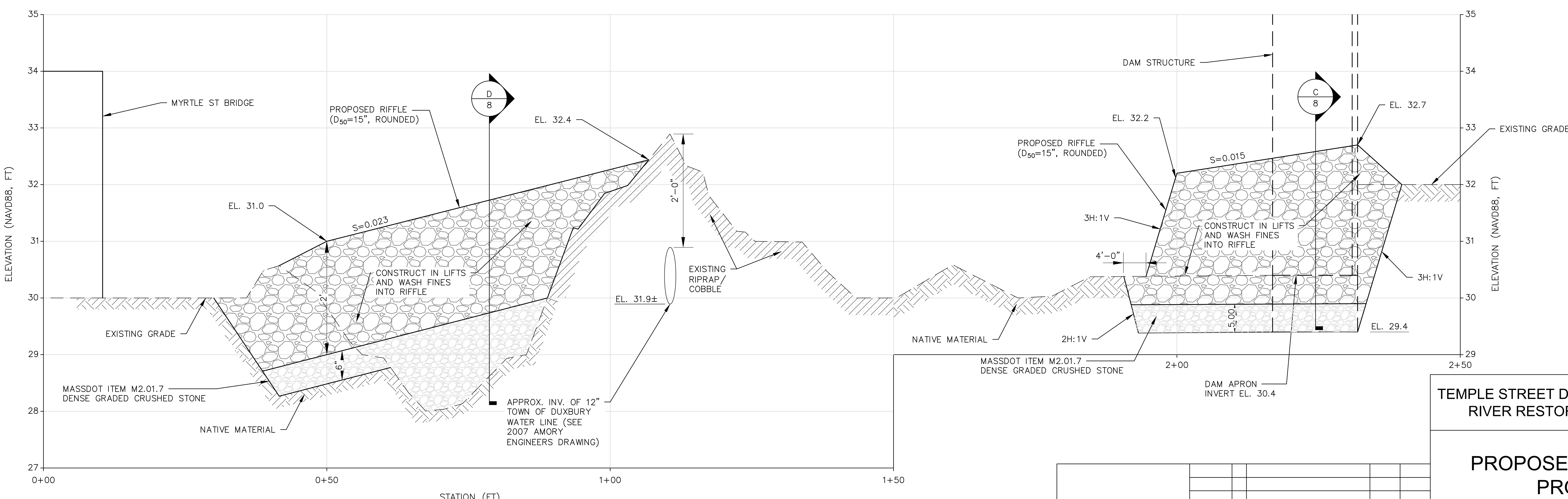
SCALE: 1" □40□ DRAWING: 5

1 / 6 PLAN VIEW
SCALE: 1" = 10'

10' 0 10' 20'
SCALE: 1" = 10'



1' 0 1' 2'
SCALE: 1" = 1'

10' 0 10' 20'
SCALE: 1" = 10'

2 / 6 PROPOSED THALWEG PROFILE
HORIZONTAL SCALE: 1" = 10' VERTICAL SCALE: 1"=1'

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CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

PROPOSED PLAN AND PROFILE

Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
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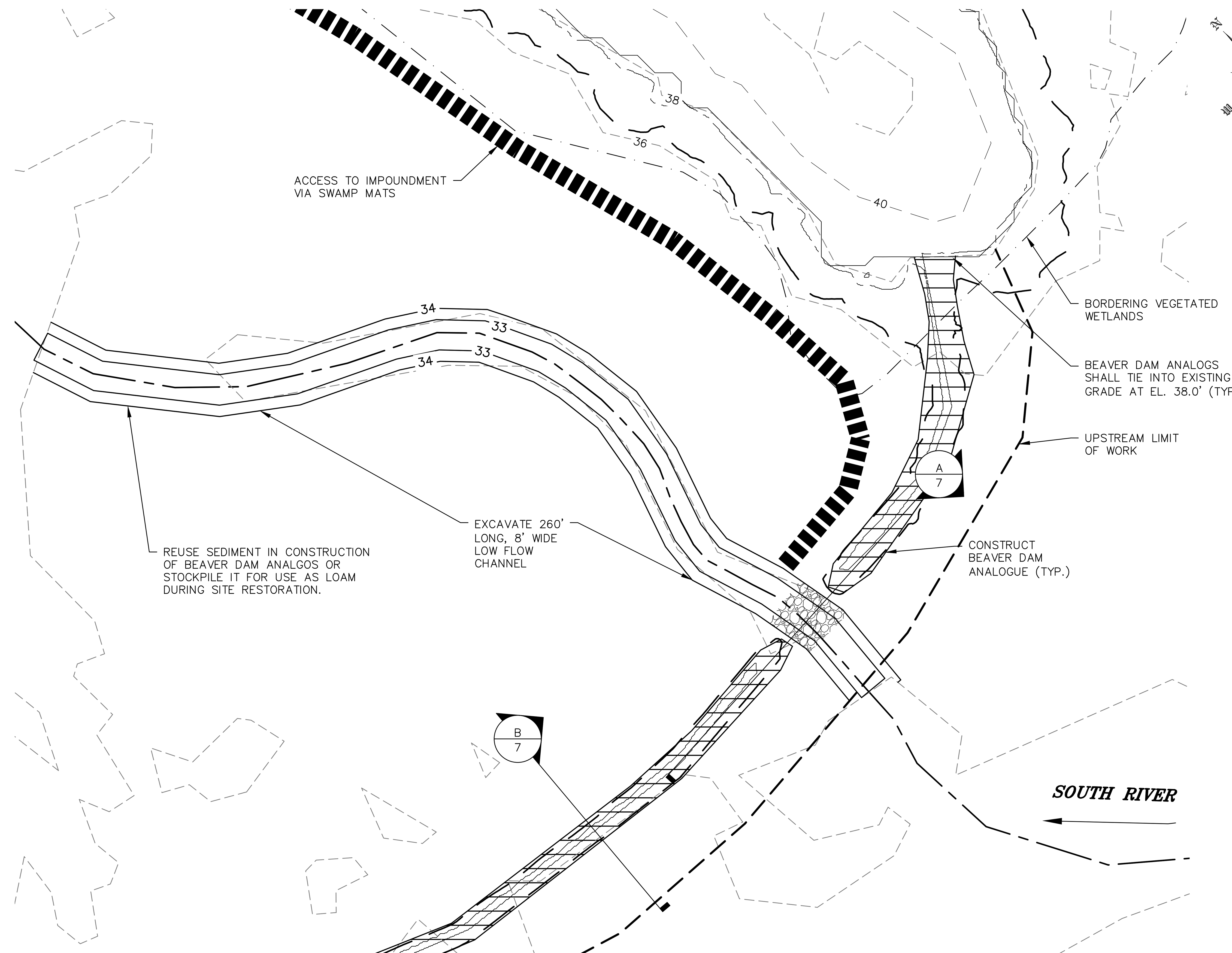
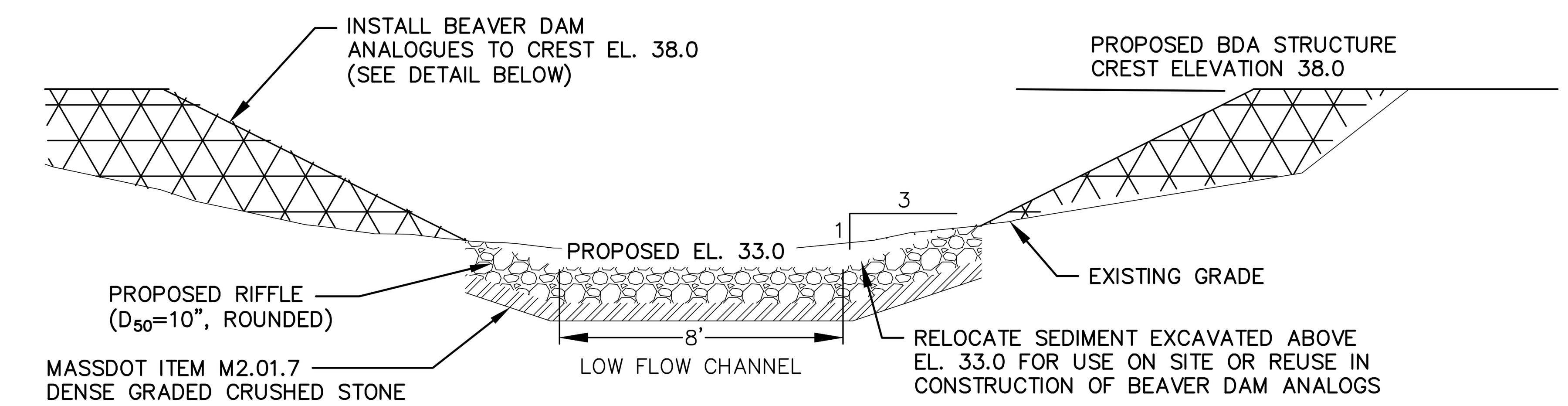
PO Box 2179
Henniker, NH 03242

SCALE: 1" □ 10□
DRAWING: 6

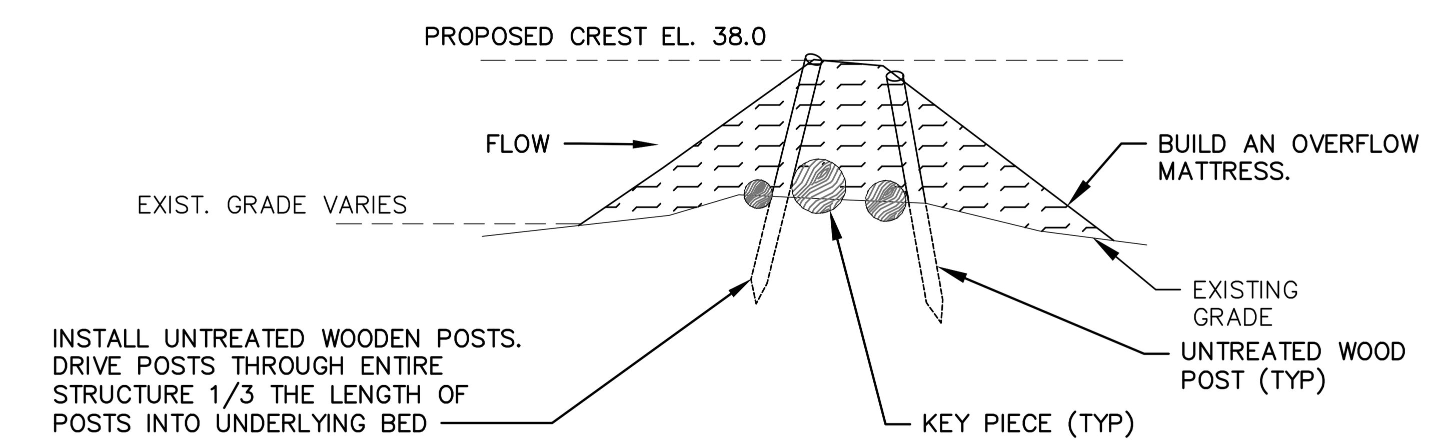


POND LMITS SHOWN – MEDIAN FLOW (3 CFS) EL. 33.2+/– DURING CONSTRUCTION PERIOD (JULY 1 – OCT. 31)
TOP OF BANK (TOB)
MEAN ANNUAL HIGH WATER (MAHW)
ORDINARY HIGH WATER (OHW)

1/7 BEAVER DAM ANALOGUE AND LOW FLOW CHANNEL
SCALE: 1" = 20'



A/7 BEAVER DAM ANALOGUE– LOW FLOW CHANNEL DETAIL
SCALE: 1" = 4'



B/7 BEAVER DAM ANALOGUES
SCALE: 1" = 4'

BDA MATERIALS

1. WOODY FILL MATERIALS: BRANCHES, LIMBS, SMALL LOGS, BRUSHY FILL.
2. FINER FILL MATERIAL: ORGANIC (E.G., TURF MATS, ROOTS, LEAVES, CONIFER NEEDLES, GRASS, ETC.) AND INORGANIC (E.G., FINE SEDIMENT, SILT, CLAY, SOIL, AND GRAVEL).

BDA CONSTRUCTION SEQUENCE

1. FOLLOW THE SEQUENCE BELOW FOR BDA INSTALLATION.
   1.1. BUILD UP FIRST LAYER OR COURSE BY WIDENING BASE UPSTREAM AND DOWNSTREAM OF CREST TO FLAT HEIGHT OF 6 TO 12" ABOVE EXISTING WATER SURFACE, AND MAKE SURE IT HOLDS BACK WATER.
   1.2. LAY OUT FIRST LAYER OF LARGER FILL MATERIAL, BEING CAREFUL NOT TO GO TO HIGHER THAN 6" TO 12" ABOVE EXISTING WATER SURFACE. THE FIRST LAYER SHOULD BE JUST HIGH ENOUGH TO BACKWATER A FLAT WATER SURFACE BEHIND IT.
   1.3. USING MUD, BED MATERIAL & TURF (TYPICALLY SOURCED FROM BACKWATER AREA OF POND) AS FINE FILL MATERIAL TO PLUG UP LEAKS, COMBINE WITH STICKS AND BRANCHES OF VARIOUS SIZES TO BUILD A WIDE BASE. MAKE SURE BASE IS WIDE ENOUGH TO ACCOMMODATE ANTICIPATED DAM HEIGHT WITH A 1.5H:1V SLOPE.
2. BUILD UP SUBSEQUENT LAYER(S) IN 6" TO 12" LIFTS, PACKING WELL WITH FINE FILL MATERIAL UNTIL PONDING WATER TO ITS NEXT TEMPORARY CREST ELEVATION.
3. REPEAT STEP 2 AS MANY TIMES AS NECESSARY TO BUILD UP TO DESIGN CREST ELEVATION.
4. WORK A WILLOW MATTRESS (LAYING BRANCHES PARALLEL TO FLOW) INTO DAM ON DOWNSTREAM SIDE AND BUILD TO PROVIDE ENERGY DISSIPATION TO OVERTOPPING FLOWS.
5. PLUG UP BDA WITH MUD AND ORGANIC MATERIAL (SMALL STICKS AND TURF) TO CREST ELEVATION.

1' 0 1' 2'
SCALE: 1" = 1'

10' 0 10' 20'
SCALE: 1" = 10'

PRELIMINARY NOT FOR CONSTRUCTION

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APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

PROPOSED BEAVER DAM ANALOGUE AND LOW FLOW CHANNEL DETAILS

Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
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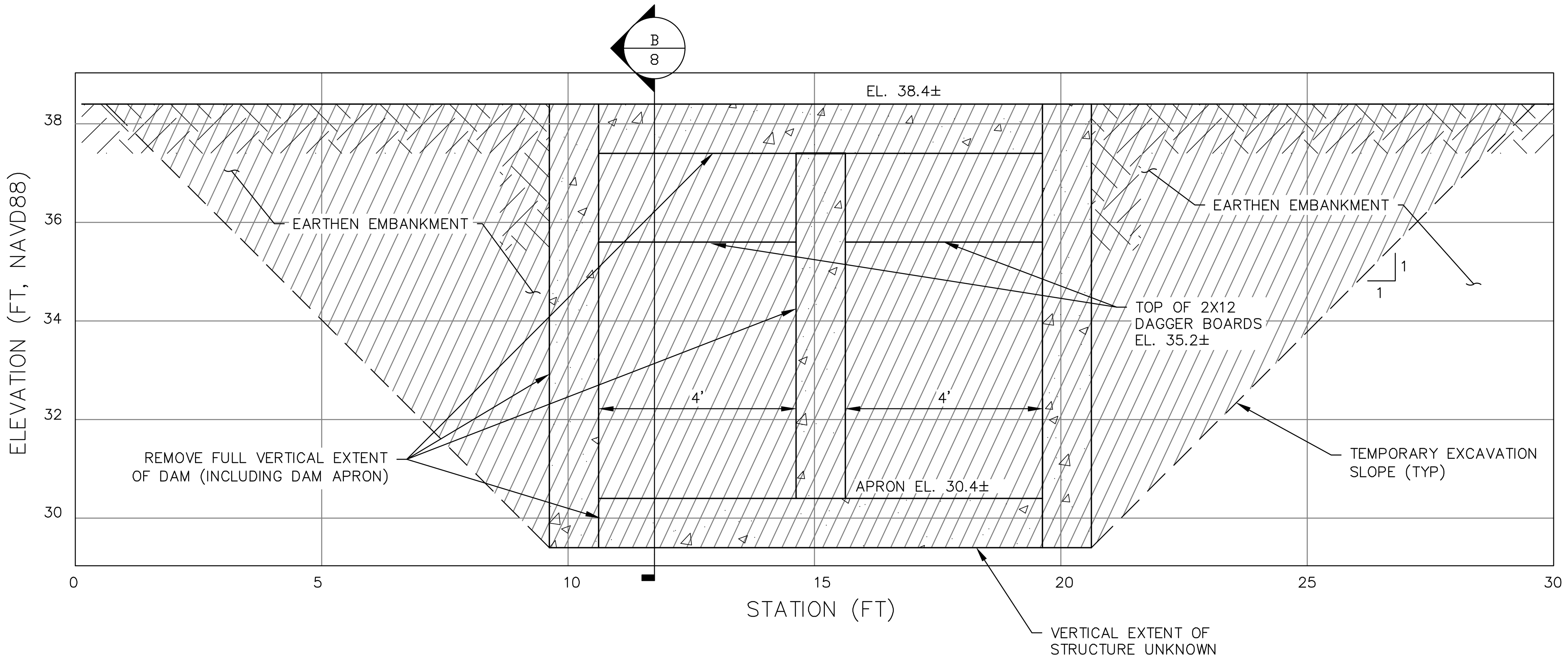
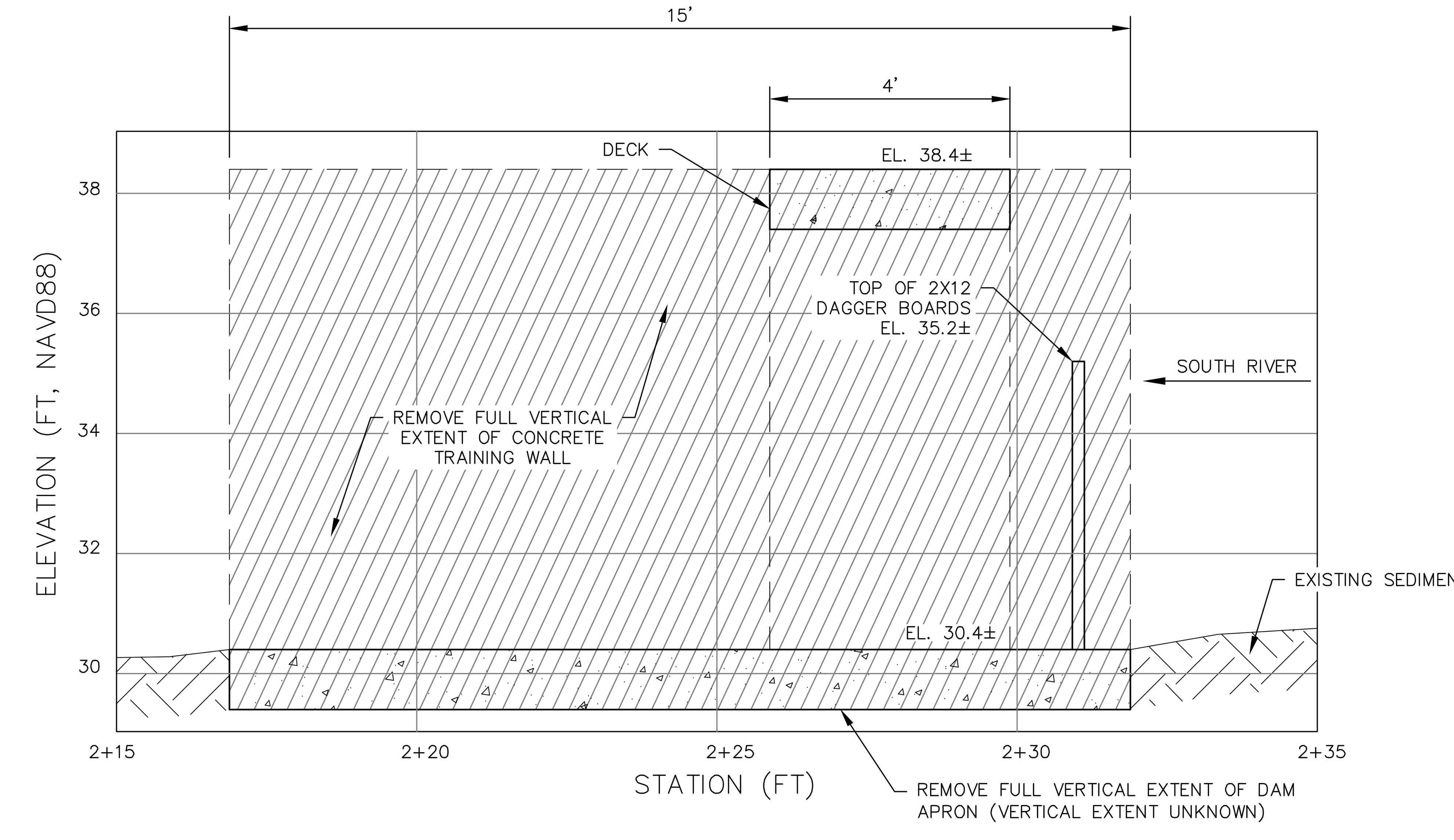
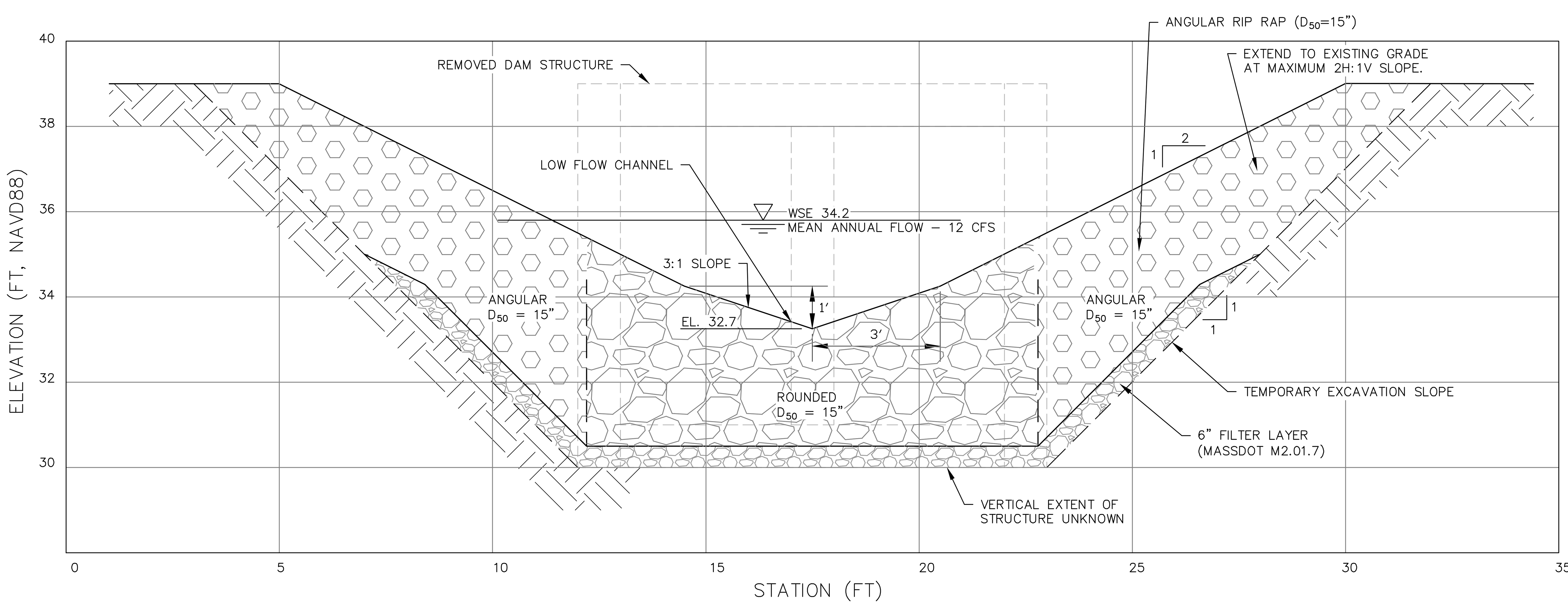
SCALE: AS SHOWN
DRAWING: 7



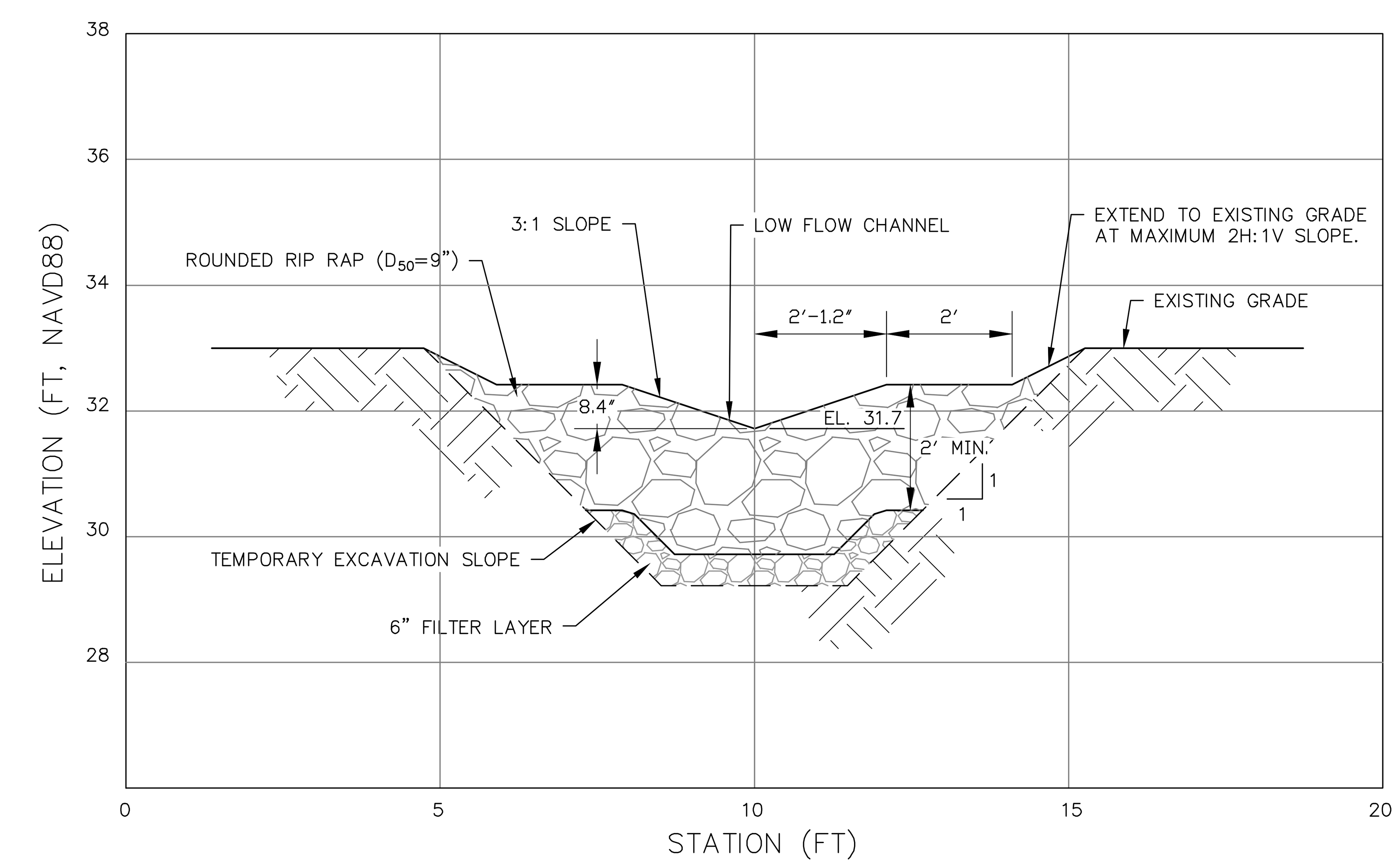
A TEMPLE ST DAM ELEVATION – REMOVALS
8 SCALE: 1" = 2'



B TEMPLE ST DAM PROFILE – REMOVALS
8 SCALE: 1" = 2'



C TEMPLE ST DAM ELEVATION – LOW FLOW CHANNEL
8 SCALE: 1" = 2'



D RIFFLE CROSS-SECTION – LOW FLOW CHANNEL
8 SCALE: 1" = 2'

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

DAM REMOVAL ELEVATION AND TYPICAL SECTIONS

PRELIMINARY NOT FOR CONSTRUCTION

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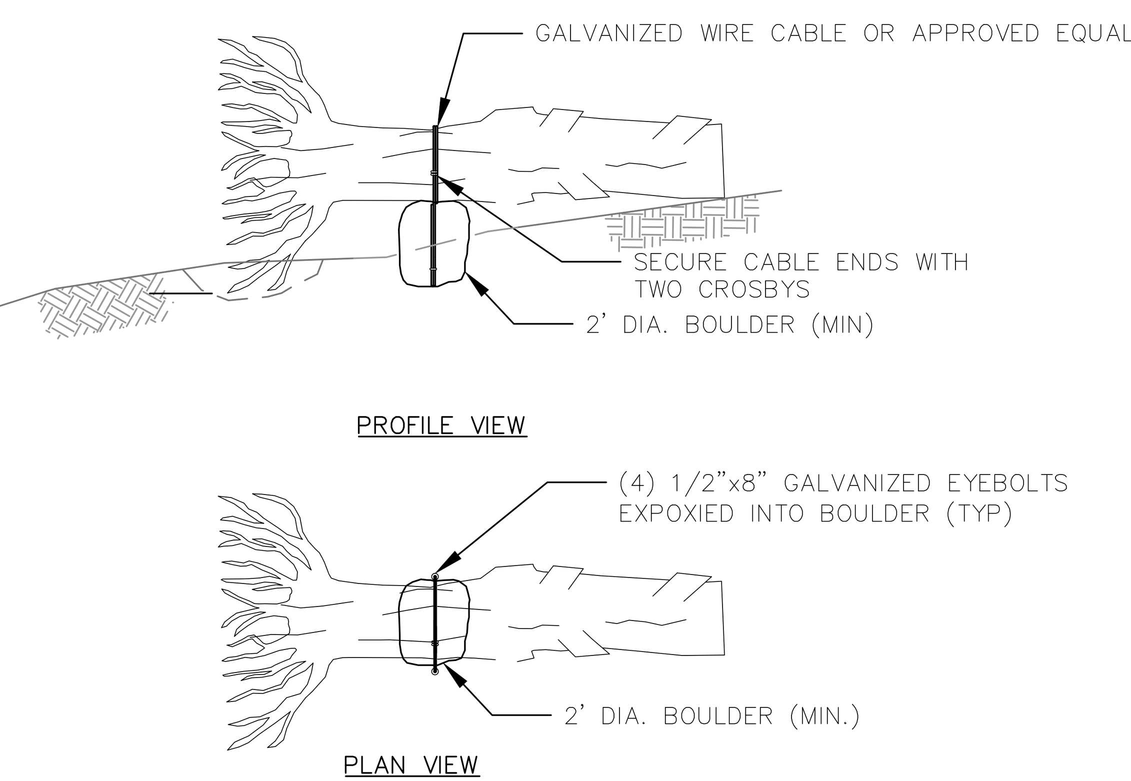
PO Box 2179
Henniker, NH 03242

SCALE: NOT TO SCALE
DRAWING: 8

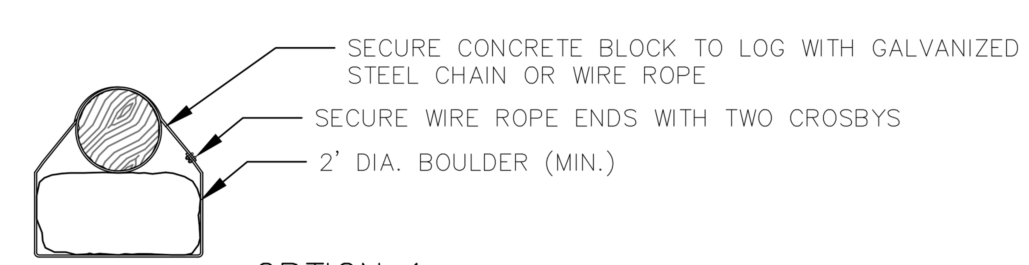
IT IS A VIOLATION OF THE LAW FOR ANY PERSON TO ALTER THIS DRAWING IN ANYWAY UNLESS HE IS ACTING UNDER THE DIRECTION OF A LICENSED PROFESSIONAL ENGINEER. ALTERATIONS MUST HAVE THE ENGINEER'S SEAL AFFIXED ALONG WITH A DESCRIPTION OF THE ALTERATION, THE SIGNATURE AND DATE.



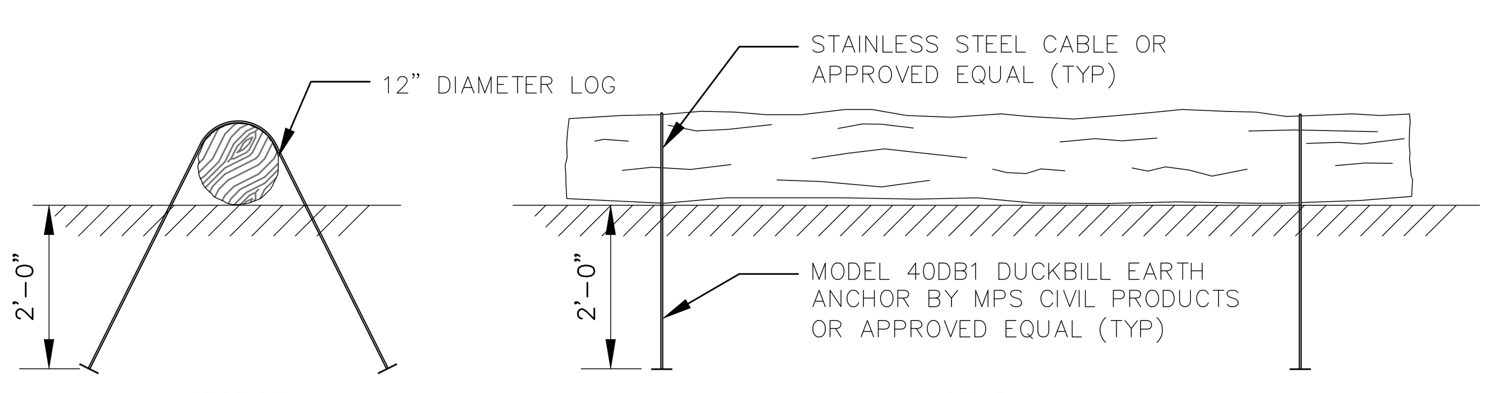
A / 9 WOODY DEBRIS STRUCTURE DETAIL
SCALE: NTS

WOODY DEBRIS INSTALLATION NOTES:

1. LARGE WOODY DEBRIS SHALL BE DECAY RESISTANT.
2. LOGS FOR LOG JAMS SHALL BE CEDAR, SPRUCE, PINE, OR FIR. LIMBS AND BRANCHES SHALL BE IN TACT TO THE FULLEST EXTENT POSSIBLE. OTHER TYPES OF LOGS MAY BE USED IF APPROVED PRIOR TO CONSTRUCTION WITH THE OWNER'S REPRESENTATIVE. STRUCTURE MEMBERS ARE TO BE A MINIMUM OF 1-FOOT IN DIAMETER.
3. BALLAST EACH 10' LONG WITH AT LEAST ONE BOULDER 2-FEET IN DIAMETER PER 10' LONG, 12" DIAMETER LOG.
4. ANCHOR ALL MEMBERS TOGETHER AS SHOWN IN ANCHOR DETAIL. PRE-DRILL 1 INCH HOLE AND USE AN A307 HOT-DIPPED GALVANIZED, 1-INCH DIAMETER, THREADED STAINLESS STEEL ROD WITH HEAVY PLATE WASHERS AND NUTS.
5. EXPOSED LOG ENDS SHALL HAVE BROKEN ENDS RATHER THAN SAW CUT ENDS.
6. EXISTING LOG AND ROOT MATERIALS EMBEDDED WITHIN THE FLOODPLAIN SHALL BE SUPPLEMENTED, NOT COLLECTED FOR REDISTRIBUTION ACROSS THE FLOODPLAIN.
7. ORIENTATION AND LOCATION SHALL BE DETERMINED BY THE OWNER'S REPRESENTATIVE IN THE FIELD.



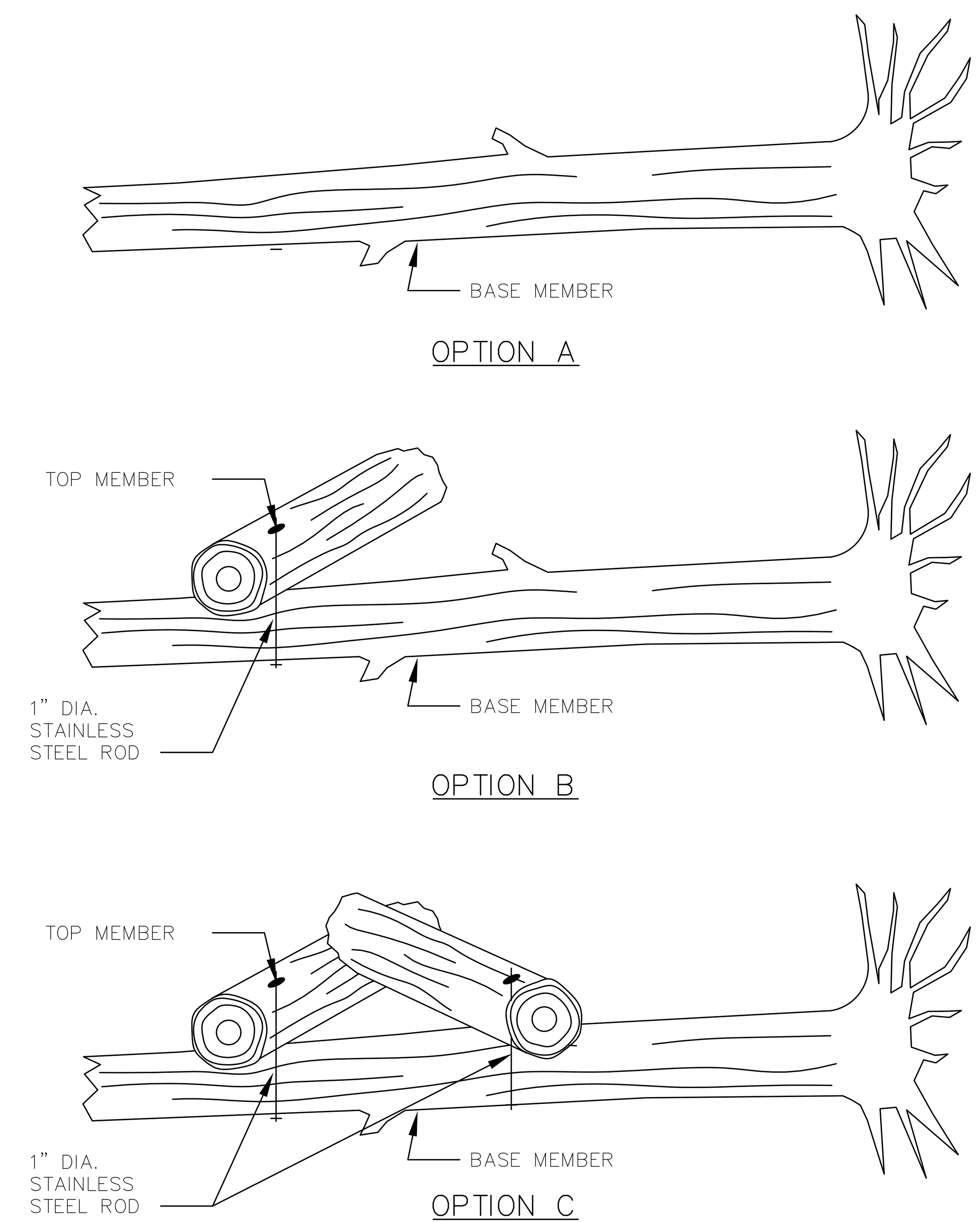
OPTION 1



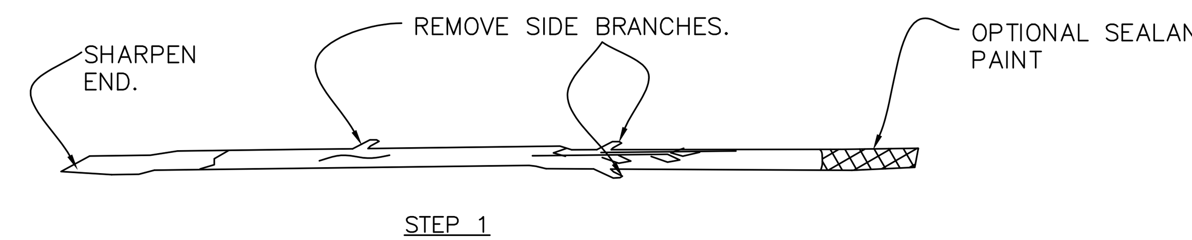
NOTE: INSTALL (2) EARTH ANCHORS PER 10' LOG

OPTION 2

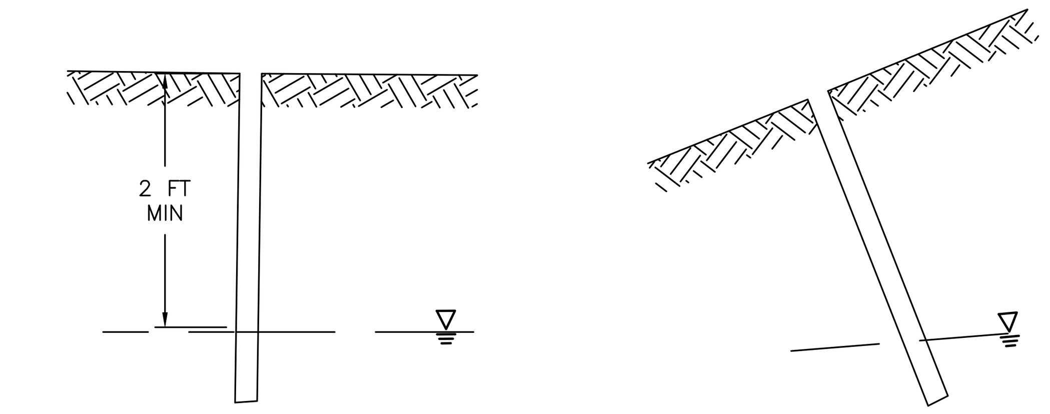
B / 9 LOG ANCHOR DETAIL
SCALE: NTS



C / 9 LOG PLACEMENT OPTIONS
SCALE: NTS



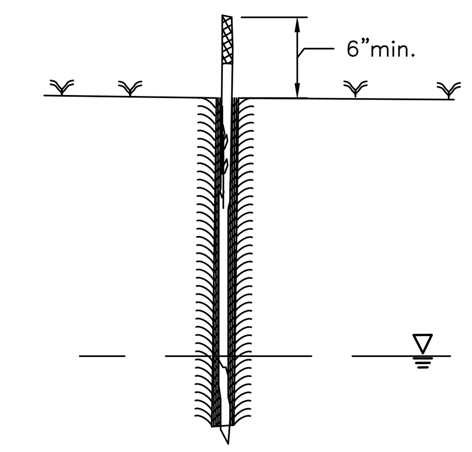
OBTAIN WILLOW OR WILLOW TYPE ADVENTITIOUSLY ROOTABLE STOCK. MATERIAL SHOULD BE FROM AN AREA WITH SIMILAR SOIL, CLIMATE, AND LOCATION RELATIVE TO THE STREAM. THE MATERIAL SHALL BE AT LEAST TWO YEARS OLD AND FREE OF DISEASE, ROT, OR INSECT INFESTATION. MATERIAL SHALL BE HARVESTED WHILE DORMANT AND SOAKED (1 TO 14 DAYS) BEFORE INSTALLATION.



NOTE: A WATERJET (HYDRODRILL) MAY BE USED TO CREATE THE PILOT HOLE IN SILT, LOAM, AND SOME CLAY SOILS. IT DOES NOT WORK WELL IN LARGE GRAVELS AND COBBLES.

STEP 2

CREATE A PILOT HOLE THAT IS PERPENDICULAR TO THE GROUND SURFACE AND DEEP ENOUGH TO REACH THE LOWEST WATER TABLE OF THE YEAR. THE HOLE SHALL BE ⅔ TO ¾ THE LENGTH OF THE LIVE POLE.



NOTE: IF PILOT HOLE IS MADE WITH A WATER JET (HYDRO DRILL), BACKFILLING WITH WATER SOIL SLURRY MAY NOT BE NECESSARY.

STEP 3

TAMP LIVE POLES INTO HOLE. TOP OF CUTTING SHALL BE ABOVE COMPETING VEGETATION. BACK FILL HOLE WITH WATER AND SOIL MIX TO ACHIEVE GOOD SOIL TO STEM CONTACT. SPACE STAKES IN A 1 TO 3 FOOT RANDOM PATTERN.

D / 9 LIVE STAKE DETAIL
SCALE: NTS

IT IS A VIOLATION OF THE LAW FOR ANY PERSON TO ALTER THIS DRAWING IN ANYWAY UNLESS HE IS ACTING UNDER THE DIRECTION OF A LICENSED PROFESSIONAL ENGINEER. ALTERATIONS MUST HAVE THE ENGINEER'S SEAL AFFIXED ALONG WITH A DESCRIPTION OF THE ALTERATION, THE SIGNATURE AND DATE.

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
6/30/22	1	75% DRAWINGS	BAS	KJC

DRAWN BY: BAS
CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

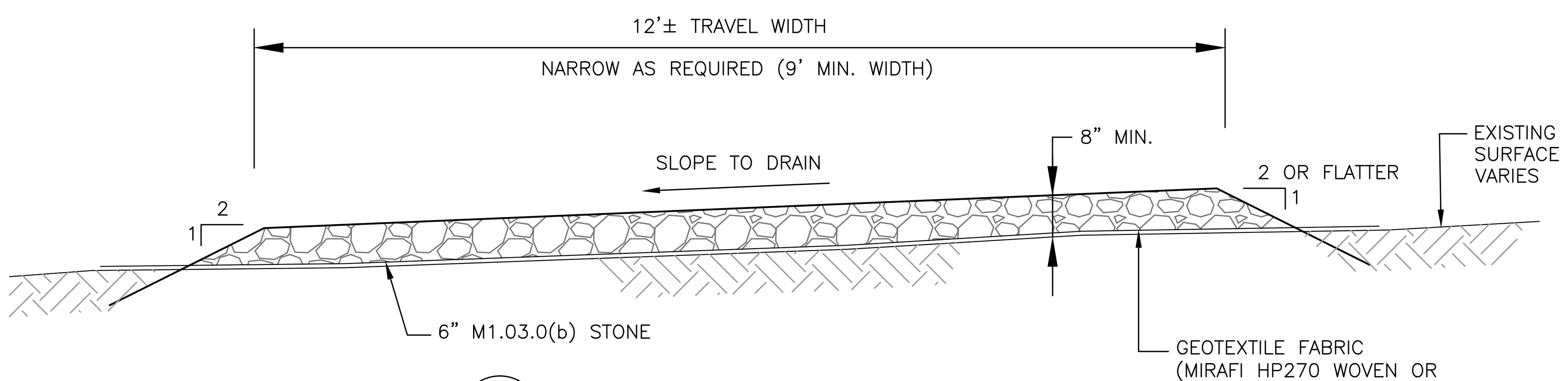
TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

STREAM RESTORATION DETAILS

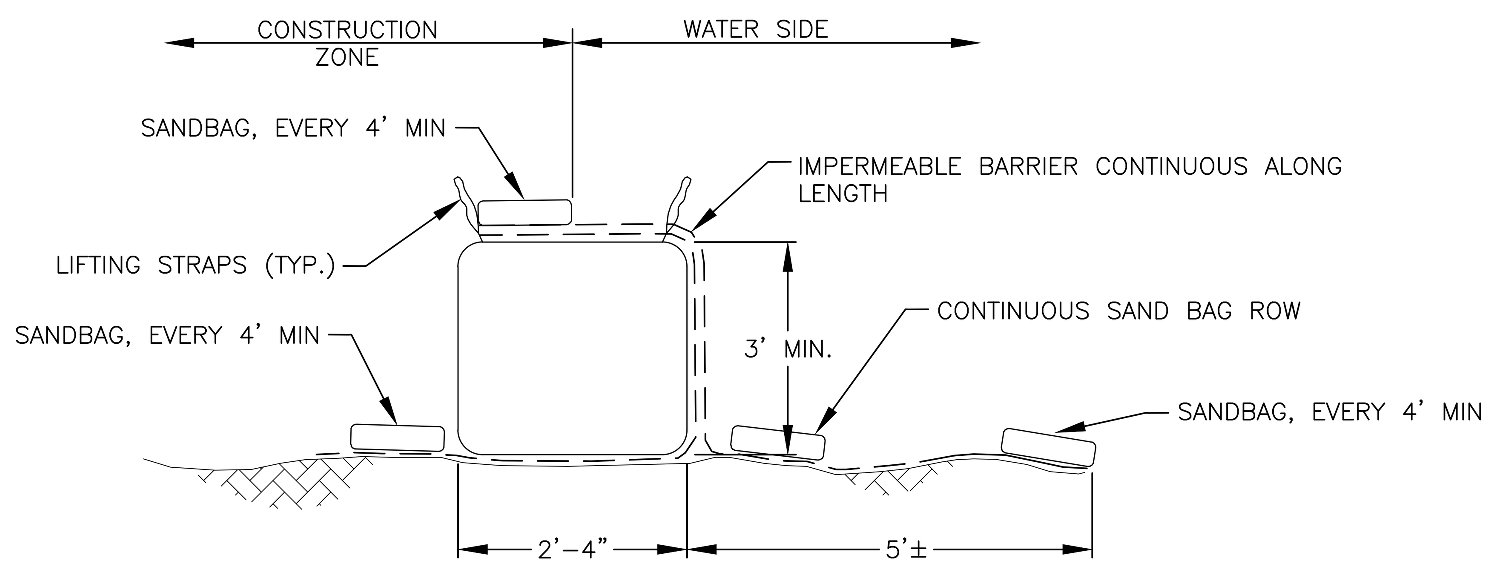
Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
41 Liberty Hill Road
PO Box 2179
Henniker, NH 03242

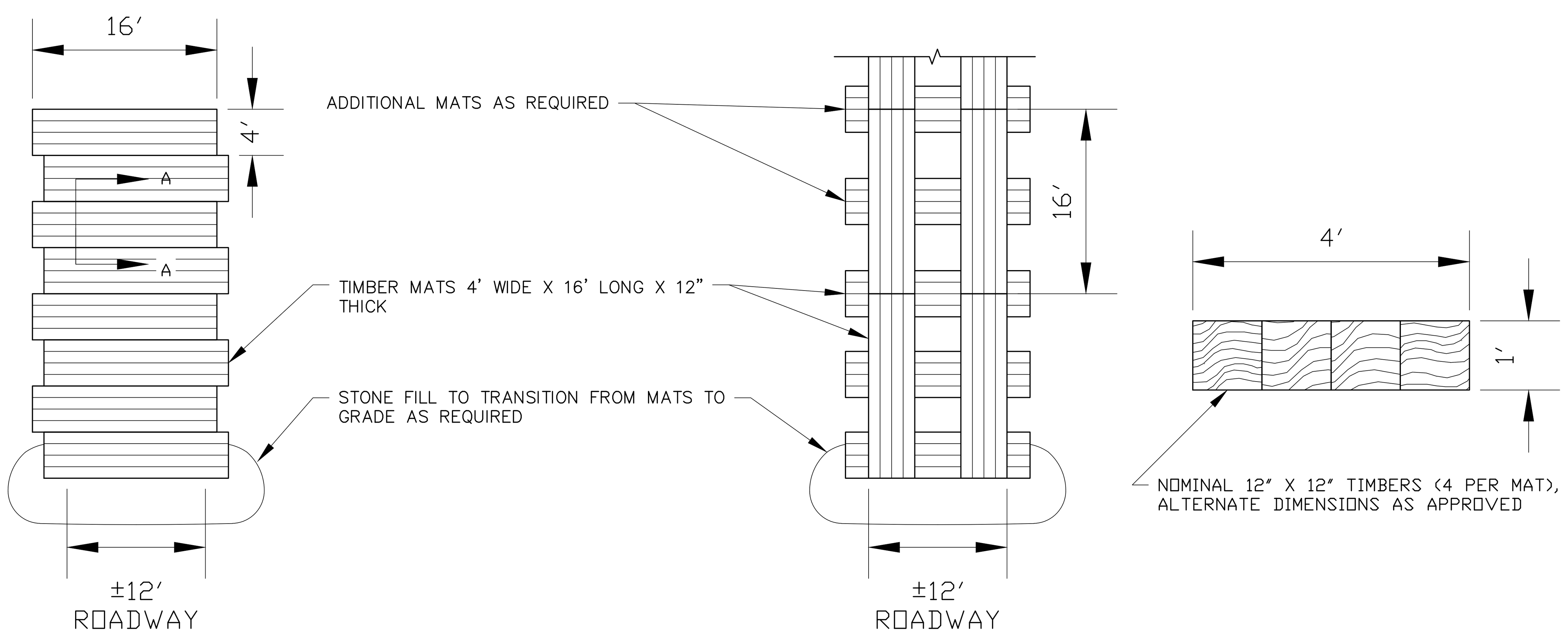
SCALE: AS NOTED
DRAWING: 9



A / 10 TEMPORARY CONSTRUCTION ROAD
NTS



B / 10 SUPER SACK DETAIL
NTS



SINGLE LAYER PLAN

DOUBLE LAYER PLAN

SECTION A–A

INSTALLATION

1. MATS SHALL BE IN GOOD CONDITION TO ENSURE PROPER INSTALLATION, USE AND REMOVAL.
2. OPERATING HEAVY EQUIPMENT IN WETLANDS SHALL BE MINIMIZED, AND SUCH EQUIPMENT OTHER THAN FIXED EQUIPMENT (DRILL RIGS, FIXED CRANES, ETC.) SHALL NOT BE STORED, MAINTAINED, FUELED OR REPAIRED IN WETLANDS UNLESS THE EQUIPMENT IS BROKEN DOWN AND CANNOT BE EASILY REMOVED.
3. AN ADEQUATE SUPPLY OF SPILL CONTAINMENT EQUIPMENT SHALL BE MAINTAINED ON SITE.
4. MINIMIZE IMPACTS TO WETLAND AREAS DURING INSTALLATION, USE, AND REMOVAL.
5. INSTALL ADEQUATE EROSION AND SEDIMENT CONTROLS AT APPROACHES TO MATS TO PROMOTE A SMOOTH TRANSITION TO, AND MINIMIZE SEDIMENT TRACKING ONTO, SWAMP MATS.
6. IN MOST CASES, CONSTRUCTION MATS SHOULD BE PLACED ALONG THE TRAVEL AREA SO THAT THE INDIVIDUAL BOARDS ARE RESTING PERPENDICULAR TO THE DIRECTION OF TRAFFIC. NO GAPS SHOULD EXIST BETWEEN MATS. PLACE MATS FAR ENOUGH ON EITHER SIDE OF THE RESOURCE AREA TO REST ON FIRM GROUND.

MAINTENANCE

1. MAT INSTALLATIONS SHOULD BE MONITORED TO ASSURE CORRECT FUNCTIONING OF THE MATS. INSPECT MATS AFTER USE. LOOK FOR ANY DEFECTS OR STRUCTURAL PROBLEMS. MATS WHICH BECOME COVERED WITH SOILS OR CONSTRUCTION DEBRIS SHOULD BE CLEANED AND THE MATERIALS REMOVED AND DISPOSED OF IN AN UPLAND LOCATION. THE MATERIAL SHOULD NOT BE SCRAPED AND SHOVELED INTO THE RESOURCE AREA. MATS WHICH BECOME IMBEDDED MUST BE RESET OR LAYERED TO PREVENT MUD FROM COVERING THEM OR WATER PASSING OVER THEM.

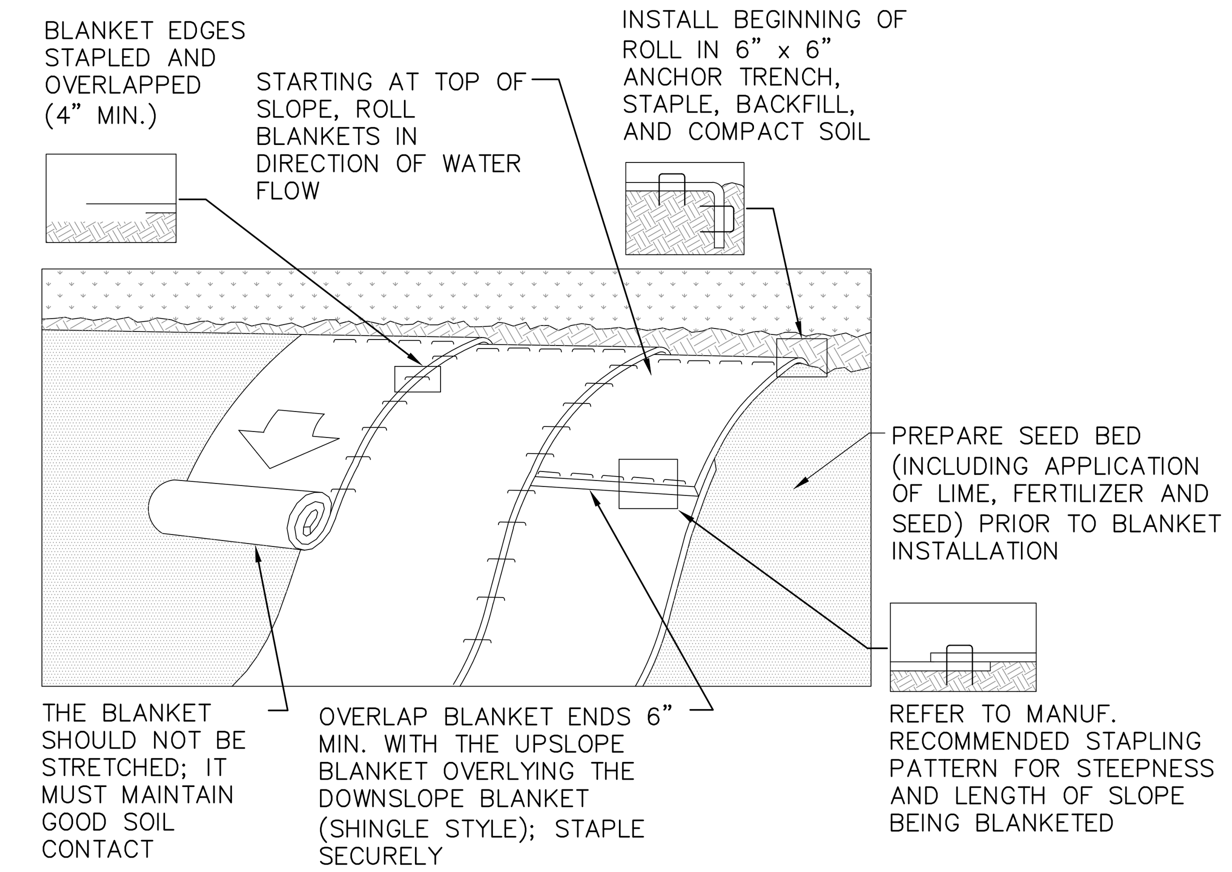
REMOVAL

1. MATTING SHOULD BE REMOVED BY "BACKING" OUT OF THE SITE, REMOVING MATS ONE AT A TIME. ANY RUTTING OR SIGNIFICANT INDENTATIONS IDENTIFIED DURING MAT REMOVAL SHOULD BE REGRADED IMMEDIATELY, TAKING CARE NOT TO COMPACT SOILS.
2. MATS SHOULD BE CLEANED BEFORE TRANSPORT TO ANOTHER WETLAND/STREAM LOCATION TO REMOVE SOIL AND ANY INVASIVE PLANT SPECIES SEED STOCK OR PLANT MATERIAL.
3. MATS SHALL BE CLEANED OF SOIL AND ANY INVASIVE PLANT SPECIES SEED STOCK OR PLANT MATERIAL FROM BEFORE INSTALLATION.
4. CLEANING METHODS MAY INCLUDE BUT ARE NOT LIMITED TO SHAKING OR DROPPING MATS IN A CONTROLLED MANNER WITH A PIECE OF MACHINERY TO KNOCK OFF ATTACHED SOIL AND DEBRIS, SPRAYING WITH WATER OR AIR, AND SWEEPING.

RESTORATION

1. SPECIAL PRECAUTIONS SHOULD BE TAKEN TO PROMPTLY STABILIZE AREAS OF DISTURBED SOIL LOCATED NEAR WETLANDS AND STREAMS. MATTED AREAS WITHIN WETLANDS SHALL BE RESTORED TO THEIR ORIGINAL CONDITION AND ELEVATION. THIS MAY INVOLVE NATURAL REVEGETATION FROM EXISTING ROOT AND SEED STOCK OF NATIVE PLANT SPECIES. CONDITIONS MAY WARRANT PLANTING AND THE BROADCAST OF A WETLAND SEED MIX OVER THE MATTED AREA TO SUPPLEMENT THE EXISTING SEED AND ROOTSTOCK. SEED MIXES AND VEGETATION SHALL CONTAIN ONLY PLANT SPECIES NATIVE TO NEW ENGLAND. THE USE OF MULCH IN WETLANDS SHALL CONSIST OF WEED- FREE MULCH TO MITIGATE THE RISK OF THE SPREAD OF INVASIVE PLANT SPECIES.

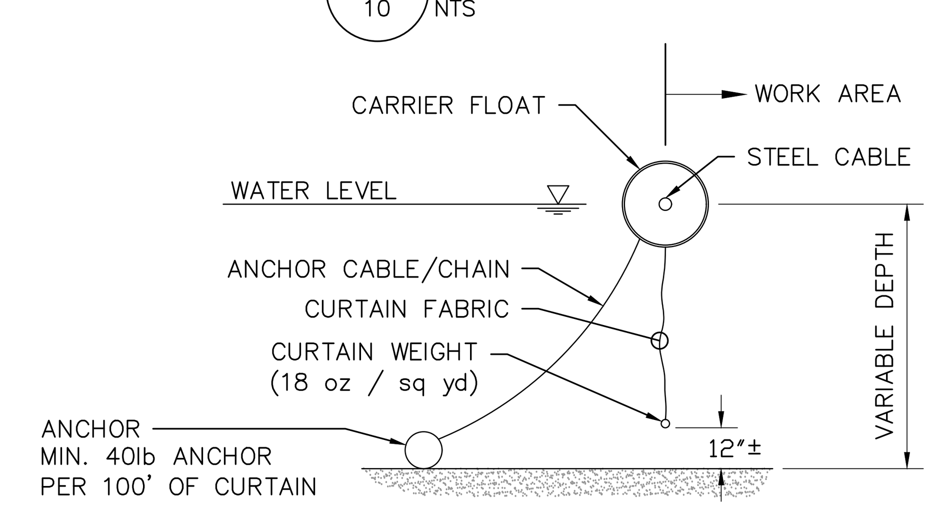
C / 10 SWAMP MAT DETAIL
NTS



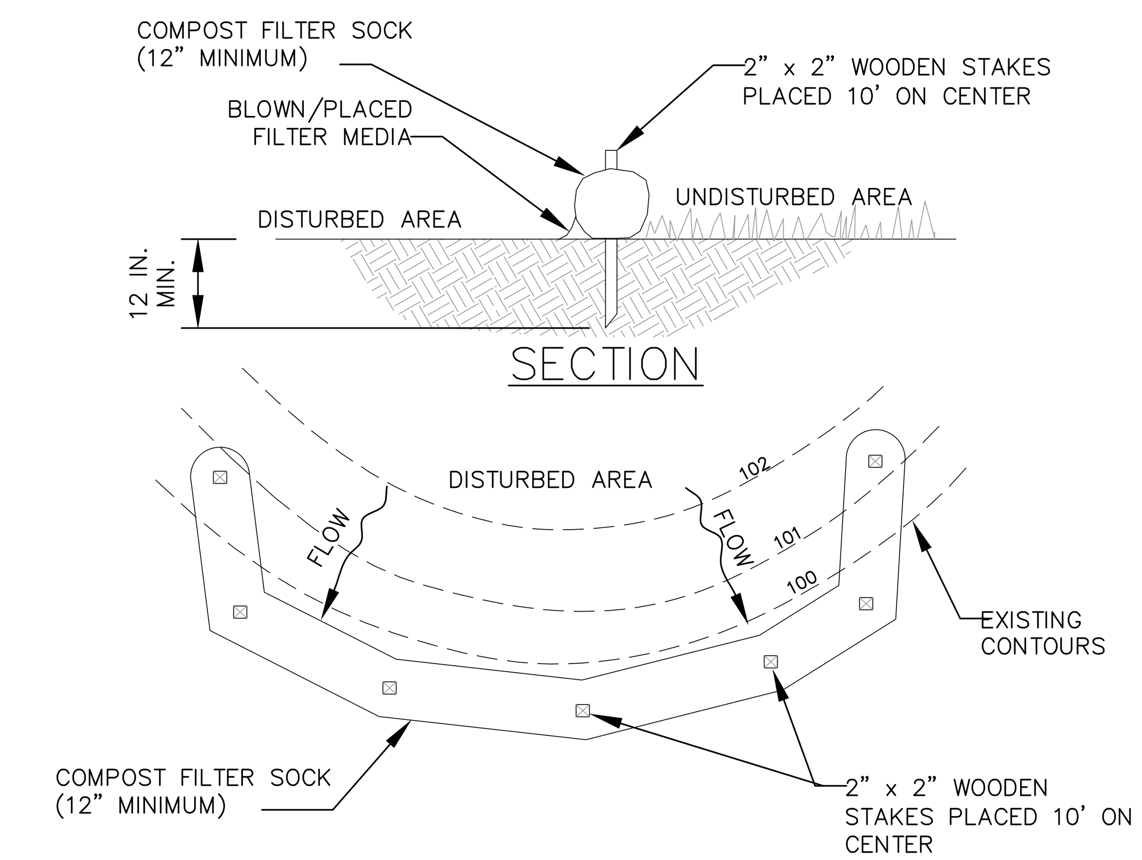
INSTALLATION NOTES
1. PREPARE SOIL BEFORE INSTALLING BLANKETS, INCLUDING ANY NECESSARY APPLICATION OF LIME, FERTILIZER, AND SEED.
2. BEGIN AT THE TOP OF THE SLOPE BY ANCHORING THE BLANKET IN A 6" (15CM) DEEP X 6" (15CM) WIDE TRENCH WITH APPROXIMATELY 12" (30CM) OF BLANKET EXTENDED BEYOND THE UP-SLOPE PORTION OF THE TRENCH. ANCHOR THE BLANKET WITH A ROW OF STAPLES/STAKES APPROXIMATELY 12" (30CM) APART IN THE BOTTOM OF THE TRENCH. BACKFILL AND COMPACT THE TRENCH AFTER STAPLING. APPLY SEED TO COMPACTED SOIL AND FOLD REMAINING 12" (30CM) PORTION OF BLANKET BACK OVER SEED AND COMPACTED SOIL. SECURE BLANKET OVER COMPACTED SOIL WITH A ROW OF STAPLES/STAKES SPACED APPROXIMATELY 12" (30CM) APART ACROSS THE WIDTH OF THE BLANKET.
3. ROLL THE BLANKETS DOWN THE SLOPE. BLANKETS WILL UNROLL WITH APPROPRIATE SIDE AGAINST THE SOIL SURFACE. ALL BLANKETS MUST BE SECURELY FASTENED TO SOIL SURFACE BY PLACING STAPLES/ STAKES IN APPROPRIATE LOCATIONS AS SHOWN IN THE STAPLE PATTERN GUIDE. WHEN USING OPTIONAL DOT SYSTEM, STAPLES/STAKES SHOULD BE PLACED THROUGH EACH OF THE COLORED DOTS CORRESPONDING TO THE APPROPRIATE STAPLE PATTERN.
4. THE EDGES OF PARALLEL BLANKETS MUST BE STAPLED WITH APPROXIMATELY 2"–5" (5CM–12.5CM) OVERLAP DEPENDING ON BLANKET TYPE. TO ENSURE PROPER SEAM ALIGNMENT, PLACE THE EDGE OF THE OVERLAPPING BLANKET (BLANKET BEING INSTALLED ON TOP) EVEN WITH THE COLORED SEAM STITCH ON THE PREVIOUSLY INSTALLED BLANKET.
5. CONSECUTIVE BLANKETS SPLICED DOWN THE SLOPE MUST BE PLACED END OVER END (SHINGLE STYLE) WITH AN APPROXIMATE 3" (7.5CM) OVERLAP. STAPLE THROUGH OVERLAPPED AREA, APPROXIMATELY 12" (30CM) APART ACROSS ENTIRE BLANKET WIDTH.

NOTES:
1. INSTALLATION TO BE COMPLETED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.
2. DO NOT SCALE DRAWINGS.
3. IN LOOSE SOIL CONDITIONS THE USE OF STAPLE OR STAKE LENGTHS GREATER THAN 6" (15CM) MAY BE NECESSARY TO PROPERLY SECURE THE BLANKETS.
4. SLOPE SURFACE SHALL BE FREE OF ROCKS, CLODS, STICKS, AND GRASS.
5. SEED AND SOIL AMENDMENTS SHALL BE APPLIED ACCORDING TO THE RATES IN THE PLAN DRAWINGS PRIOR TO INSTALLING THE BLANKET.
6. BLANKETED AREAS SHALL BE INSPECTED WEEKLY AND AFTER EACH RUNOFF EVENT UNTIL PERENNIAL VEGETATION IS ESTABLISHED TO A MINIMUM UNIFORM 70% COVERAGE THROUGHOUT THE BLANKETED AREA. DAMAGED OR DISPLACED BLANKETS SHALL BE RESTORED OR REPLACED WITHIN 4 CALENDAR DAYS.
7. EROSION CONTROL BLANKET SHALL BE PLACED ON ANY VEGETATED SLOPES GREATER THAN 3H:1V.

D / 10 EROSION CONTROL BLANKET
NTS



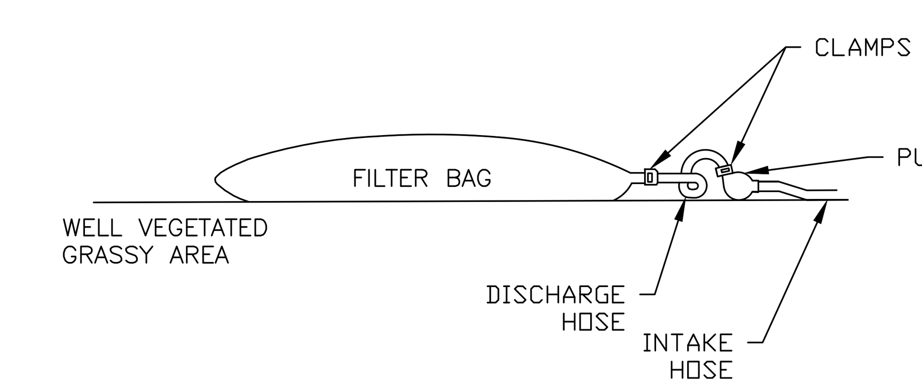
E / 10 OIL BOOM
NTS



NOTES:

1. SOCK FABRIC SHALL MEET MADEP STANDARDS.
2. COMPOST FILTER SOCK SHALL BE PLACED AT EXISTING LEVEL GRADE. BOTH ENDS OF THE BARRIER SHALL BE EXTENDED AT LEAST 8 FEET UP SLOPE AT 45 DEGREES TO THE MAIN BARRIER ALIGNMENT. MAXIMUM SLOPE LENGTH ABOVE ANY BARRIER SHALL NOT EXCEED THAT SPECIFIED FOR THE SIZE OF THE SOCK AND THE SLOPE OF ITS TRIBUTARY AREA.
3. TRAFFIC SHALL NOT BE PERMITTED TO CROSS COMPOST FILTER SOCKS.
4. ACCUMULATED SEDIMENT SHALL BE REMOVED WHEN IT REACHES 1/2 THE ABOVE GROUND HEIGHT OF THE BARRIER AND DISPOSED IN THE MANNER DESCRIBED ELSEWHERE IN THE PLAN.
5. COMPOST FILTER SOCKS SHALL BE INSPECTED WEEKLY AND AFTER EACH RUNOFF EVENT. DAMAGED SOCKS SHALL BE REPAIRED ACCORDING TO MANUFACTURER'S SPECIFICATIONS OR REPLACED WITHIN 24 HOURS OF INSPECTION.
6. UPON STABILIZATION OF THE AREA TRIBUTARY TO THE SOCK, STAKES SHALL BE REMOVED. THE SOCK MAY BE LEFT IN PLACE AND VEGETATED OR REMOVED. IN THE LATTER CASE, THE MESH SHALL BE CUT OPEN AND THE MULCH SPREAD AS A SOIL SUPPLEMENT.
7. IF THERE ARE SEAMS WHERE ONE SOCK ABUTS ANOTHER, THERE NEEDS TO BE OVERLAP AND STAKING PER MANUFACTURERS SPECIFICATIONS.

F / 10 COMPOST FILTER SOCK
NTS



NOTE:
1. BAGS SHALL BE LOCATED IN WELL-VEGETATED (GRASSY) AREA, AND DISCHARGE ONTO STABLE, EROSION RESISTANT AREAS. WHERE THIS IS NOT POSSIBLE, A GEOTEXTILE UNDERLAYMENT AND FLOW PATH SHALL BE PROVIDED. BAGS MAY BE PLACED ON FILTER STONE TO INCREASE DISCHARGE CAPACITY. BAGS SHALL NOT BE PLACED ON SLOPES GREATER THAN 5%. FOR SLOPES EXCEEDING 5%, CLEAN ROCK OR OTHER NON-ERODIBLE AND NON-POLLUTING MATERIAL MAY BE PLACED UNDER THE BAG TO REDUCE SLOPE STEEPNESS.

G / 10 SEDIMENT DEWATERING BAG DETAIL
SCALE: NONE

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
6/30/22	1	75% DRAWINGS	BAS	KJC

DRAWN BY: BAS
CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

EROSION AND SEDIMENT CONTROL DETAILS

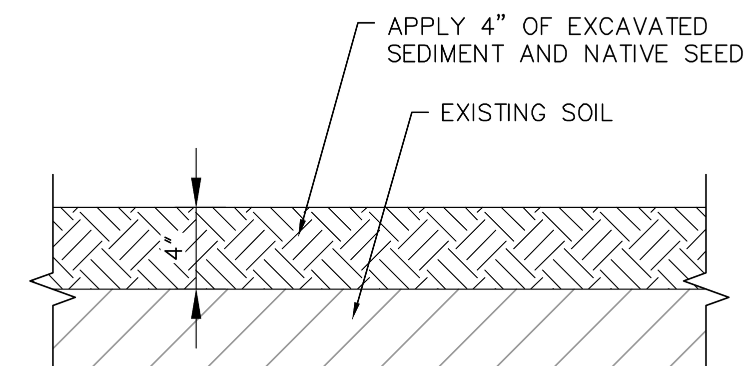
Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
41 Liberty Hill Road
PO Box 2179
Henniker, NH 03242

SCALE: NOT TO SCALE
DRAWING: 10

Zone	Wetland Restoration Habitat Type (Elevation Range)	Size	Strategy	Plant Form	Location and (SF)
UPL	Upland (>36.5)	0.46 (20,000 SF)	Seed distributed upland areas with native upland mix as needed.	Seed	Former Embankment Area, other disturbed upland areas (20,000)
Zone 1	Wet Meadow/Shrub-Scrub (33-36.5)	0.4 Acres (18,500 SF)	1-3" diameter stakes 18-36" long at 2' on center	Stake	Former Impoundment Area, adjacent to the edge of water (18,500)

Woody Cuttings (Stakes) (Zone 1) - Wetland Restoration Area		
Common Name	*Scientific Name*	*Specification*
Red-Osier Dogwood	Cornus stolonifera	1-3" diameter/18-36" long/2' o.c.
Pussy Willow	Salix discolor	1-3" diameter/18-36" long/2' o.c.
Silky Willow	Salix sericea	1-3" diameter/18-36" long/2' o.c.



A
11
UPLAND RESTORATION DETAIL
SCALE: NTS

PLANTING NOTES:

1. CONTRACTOR MUST BE FAMILIAR WITH SPECIFICATIONS AND HAVE THEM AVAILABLE AT ALL TIMES.
2. PLANTS SHALL BE PROVIDED AS PER SPECIES NAMED. NO SUBSTITUTIONS WITHOUT PRIOR WRITTEN APPROVAL.
3. PLANTS MUST BE NATIVE TO SOUTHEASTERN MASSACHUSETTS AND SHALL BE FROM A NURSERY SPECIALIZING IN NATIVE WETLAND PLANTS.
4. PROVIDE RECORD OF CONFIRMATION FOR PLANTS-ON-ORDER WITHIN 30 DAYS OF CONTRACT DATE.
5. ALL PLANTS MUST BE HEALTHY AND VIGOROUS.
6. EACH TRAY OF STAKES SHALL BE LEGIBLY LABELED WITH THE SPECIES SCIENTIFIC NAME. PROVIDE BILL OF LADING LISTING ALL PLANTS AND QUANTITIES FOR ALL MATERIAL ON SITE.
7. PLANTS SHALL BE INSTALLED IN THE CORRECT ZONES, AS INDICATED ON THE PLAN. MARK ALL BED AREAS WITH (STAKES, ETC.). BED AREA LOCATION AND SIZE TO BE ACCEPTED BY THE OWNER'S REPRESENTATIVE PRIOR TO INSTALLATION.
8. ALL PLANTS SHALL BE KEPT WATERED AS NECESSARY UNTIL THE END OF THE ESTABLISHMENT/GUARANTEE PERIOD (THE PERIOD ENDS WHEN ALL PLANTS HAVE BEEN INSTALLED TO THE SATISFACTION OF THE OWNER'S REPRESENTATIVE).
9. DEAD OR REJECTED PLANTS SHALL BE REMOVED FROM THE PROJECT AND REPLACED DURING THE ESTABLISHMENT/GUARANTEE PERIOD (NO MORE THAN TWICE IN THE SAME AREA).
10. PLANTS MUST BE INSTALLED IN SPECIFIC ZONES AS SPECIFIED. THE OWNER'S REPRESENTATIVE WILL ASSIST IN THE LAYOUT OF PLANT MATERIAL

PRELIMINARY NOT FOR CONSTRUCTION

DATE	#	DESCRIPTIONS	BY	APP
6/30/22	1	75% DRAWINGS	BAS	KJC

DRAWN BY: BAS
CHECKED BY: KJC
APPROVED BY:
PROJECT NO. 2336 DATE: 6/30/2022

TEMPLE STREET DAM REMOVAL/ SOUTH RIVER RESTORATION PROJECT

RESTORATION/PLANTING PLAN

Massachusetts Division Of Ecological Restoration
Massachusetts Dept. of Fish & Game
251 Causeway Street, Suite 400
Boston, MA 02114

Gomez and Sullivan Engineers, D.P.C.
41 Liberty Hill Road
PO Box 2179
Henniker, NH 03242

SCALE: 1" 40
DRAWING: 11

Appendix D – Hydrology Review Memo

TEMPLE STREET DAM REMOVAL HYDROLOGIC ANALYSIS


Final Report

June 2022

Prepared for

Massachusetts Department of Fish and Game
Division of Ecological Restoration
Invested in Nature and Community

251 Causeway Street, Suite 400
Boston, MA 02114

Prepared by

Gomez and Sullivan Engineers

41 Liberty Hill Road
Henniker, NH 03242

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LIST OF ABBREVIATIONS

AEP	Annual Exceedance Probability
cfs	Cubic feet per second
cfsm	Cubic feet per second per square mile of drainage area
CN	Curve number
CRRA	New York State Community Risk and Resiliency Act
DER	Massachusetts Department of Fish and Game Division of Ecological Restoration
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
LiDAR	Light Detection and Ranging
MACCC	Massachusetts Climate Change Clearinghouse
MassGIS	Massachusetts Bureau of Geographic Information
NOAA	National Oceanic and Atmospheric Administration
NRCC	Cornell's Northeast Regional Climate Center
NYS	New York State
NYSDEC	New York State Department of Environmental Conservation
NYSDOT	New York State Department of Transportation
Pare	Pare Corporation
RCP	Representative concentration pathways
RMAT	Resilient Massachusetts Action Team
UNH	University of New Hampshire
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey

1 Introduction

The South River flows through Plymouth County, Massachusetts in a northeasterly direction for approximately 15 miles from its headwaters in Town of Duxbury to its tidal estuary in the Town of Marshfield, where it joins with the North River and flows into Massachusetts Bay. This river is located within the Massachusetts Division of Fisheries and Wildlife Natural Heritage & Endangered Species Program's mapped Estimated and Priority Habitats of rare species and is classified as an Outstanding Resource Water (ORW) by the Massachusetts Department of Environmental Protection. The South River Restoration Project was identified in 2016 by the Massachusetts Department of Fish and Game's Division of Ecological Restoration (DER) as a priority project. The project includes the potential removal of three dams: Temple Street, Chandler Pond, and Veteran's Memorial Park Dams. DER's current objective is to develop 75% design plans for the removal of the Temple Street Dam. The Temple Street Dam, also known as the Boys & Girls Club Dam #2, is located in the Town of Duxbury just upstream from Temple/Myrtle Street in the Camp Wing Conservation Area. Previous studies regarding the removal of the Temple Street Dam have been performed by Tighe & Bond (2015), Pare (2016, 2018, and 2020), and Inter-Fluve (2021). The goals of the current hydrologic analysis are to:

- verify the current hydrologic approach for storm events,
- develop fish passage flow estimates during migration periods, and
- consider the potential effects of climate change.

The results of this hydrologic analysis will be used as hydraulic model inputs during the development of the 75% design plans for the removal of Temple Street Dam.

2 Methodology

Considerations during the design of the Temple Street Dam removal, include flooding and fish migration. The hydrology for flood control design is generally based on isolated storm events (i.e., event based flows), while the hydrology for fish passage design is generally based on the range of flows anticipated during a particular period (i.e., period based flows). The analysis approach differs for the event based and period based flows, and as such, this analysis presents them separately.

2.1 Event Based Flows

The most recent hydrologic analysis performed by Pare Corporation (Pare) in 2020, included updating an event based hydrologic model of the South River watershed. This analysis includes a review of the existing model developed by Pare, as well as other readily available event based hydrologic information.

2.1.1 Existing Model Review

The existing hydrologic model was developed using the HydroCAD Version 10.1-3a software. This model applies rainfall depths and distribution curves over 14 drainage areas within the South River watershed. Ten different rainfall events were evaluated, whose 24-hour depths had an Annual Exceedance Probability (AEP) ranging from 100% (1-year recurrence interval) to 0.1% (1000-year recurrence interval). Runoff from each drainage area is generally computed using the standard Soil Conservation Service unit hydrograph (i.e., Gamma = 484). However, two of drainage areas upstream of Temple Street Dam utilized unit hydrographs with a Gamma value of 350 to account for flow attenuation through cranberry bogs which were not modeled as separate hydraulic structures. This unit hydrograph methodology requires a time of concentration and a curve number to be defined for each drainage area. The time of concentration was developed for each drainage area within the existing model using the segmental method (e.g., sheet flow, shallow concentrated flow, channel flow), while a composite curve number was developed for each drainage area within the model based on the area weighted land cover/soil type combinations. In addition to the 14 drainage areas, the existing model includes 23 hydraulic structures. These hydraulic structures represent dams, control structures, and roadway crossings throughout the watershed, and were modeled using level pool routing techniques. Hydrologic reach routing was not included between the drainage area and hydraulic structure features.

The model uses National Oceanic and Atmospheric Administration (NOAA) Atlas 14 (NOAA, 2019) for rainfall inputs. Soil type information from the Natural Resource Conservation Center (NRCS) soil survey dataset (NRCS, 2015) was combined with land cover information from the Massachusetts Bureau of Geographic Information (MassGIS) land cover dataset (MassGIS, 2019) to develop the curve numbers. The model utilized an initial abstraction to retention ratio of 0.2. Stage-storage-discharge information for each hydraulic structure was developed using field collected data, previous reports, LiDAR terrain data, and aerial imagery. A three minute computational time step was used within the existing hydrologic model.

The methods, assumptions, and parameters utilized in the existing hydrologic model were generally considered to be reasonable. It should be noted that Cornell's Northeast Regional Climate Center (NRCC) webtool for extreme precipitation in New England is an alternative source for rainfall inputs. This tool provides precipitation depths for Plymouth County which are slight lower than used in the existing model for the 5-year recurrence interval and slightly higher precipitation depths than used in the existing model for the 25-year and 100-year recurrence intervals.

2.1.2 Regional Regression Equations

Regional regression equations utilize different basin parameters for a given location (e.g., drainage area, elevation, surface water storage area) to estimate the anticipated peak flow for various recurrence interval events (e.g., 10% AEP). The development of these equations (e.g., basin parameters, exponents) are based on statistical analyses of the magnitude and frequency of flows observed at stream gages within a given region. These equations allow for the estimation of the magnitude and frequency of flows for locations which do not have stream gages. The USGS published regional regression equations for the state of Massachusetts (Zarriello, 2017), which depend on three basin parameters to estimate flows: drainage area (DA), mean elevation of the basin (ELEV), and total storage as defined as the percent of wetlands and open water for the basin (STOR). The USGS has developed a webtool called StreamStats to implement regional regression equations for most states.

The effective Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for Plymouth County, revised July 6, 2021, utilized the Massachusetts regional regression equations, to develop peak flows for the South River. However, the FIS does not report the locations or flows developed for this effort. This study (GSE 2022) utilized the StreamStats Version 4.6.2, to compute basin parameters and applicable flow estimates at Temple Street Dam (42.07950, -70.74543).

2.1.3 Peak Frequency Analysis

A peak frequency analysis performs a statistical analysis on a series of annual maximum flows recorded for a given location, along with other available peak flow information, to estimate the frequency and magnitude of flows for that location. Bulletin 17C outlines the latest accepted methods for peak frequency analysis and utilizes an expected moments algorithm to accommodate interval data and a multiple Grubbs-Beck test for identifying low outliers (USGS, 2019a). The methods require that the regional skew and mean-square error estimates be obtained using the Bayesian Weighted Least

Squares/Bayesian Generalized Least Squares regression procedure. Regional skew and mean-square error estimates appropriate for Bulletin 17C methodology have been developed for Massachusetts (USGS, 2019b). The Bulletin 17C methodology can be implemented using the PeakFQ software (USGS, 2019c), provided that a sufficient period of observed maximum annual flow data is available. Bulletin 17C recommends at least 10 years of annual maximum peak flow data.

There are no active USGS streamflow gages on the South River. However, nearby gages are often utilized for peak frequency analyses with adjustments made to the results. These adjustments are typically based on drainage area. Ideally, the nearby gages would have similar basin characteristics as the location of interest. This study utilized PeakFQ Version 7.3 to perform a peak frequency analysis on two nearby gages with similar drainage areas at Temple Street Dam and over 50 consecutive years of annual maximum peak flow data[1]. A regional skew of 0.37 and standard error of 0.374166 was utilized for these analyses, consistent with the recommendations for New England.

The PeakFQ results were prorated using a drainage area ratio in order to estimate flows at Temple Street Dam. A regional exponent is sometimes applied to the drainage area ratio. These exponents can vary depending on the storm magnitude, and are often obtained from the exponent applied to the drainage area in the regional regression equations. The drainage areas used to develop the regional regression equations ranged from 0.16 square miles (mi^2) to 512 mi^2, while the drainage areas considered in this study ranged from 5.9 mi^2 (Temple Street Dam) to 30.3 mi^2 (Indian Head River gage). Similarly, other basin characteristics that impact basin runoff characteristics (e.g., storage, impervious area) for sites used to develop the regional regression equations exhibited a much wider range than the sites considered in this study. Due to the close proximity of the gages to the Temple Street Dam and their relatively similar basin characteristics, this study used a simple drainage area ratio (i.e., exponent of 1) when prorating flows.

2.2 Period Based Flows

Previous studies have evaluated a “Sunny Day” or baseflow value of 2 cubic feet per square mile (cfsm), which may be based on documentation regarding the New England Flow Policy (USFWS, 2002)[2]. This hydrologic information is not specific to the project site. Therefore, the same two USGS gages identified for the peak flow frequency analysis were evaluated to identify the 50% exceedance flows throughout the year.

For the design of fish passage facilities, recent guidance defines the high and low design flows as the daily average river flow that is exceeded 5% and 95% of the time during the migration/emigration period (USFWS, 2019). The migration period refers to when diadromous fish are typically migrating to and from their spawning habitat. The timing varies by species and region of the country. **Table 2.2-1** below summarizes key timeframes for the various life stages and events of the target species based on general guidelines provided by the Massachusetts Department of Fish and Game’s Division of Marine Fisheries and United States Fish and Wildlife Service (USFWS), as well as input from project partners regarding upstream herring migration. Since American Eel passage is not a significant issue at the project site, this study utilized April 15 through July 15 to define the upstream migration period and July 1 through December 31 to define the downstream migration period. The same two USGS gages identified for the

[1] The peak flows for one of the gages was noted to be affected by regulation or diversion for all of the available peaks. However, the qualifications codes for 12 years of data allow them to be used in a peak frequency analysis without activating the option to include regulated peaks. Exclusion of the other 40 years of regulated discharges at this site results in higher peak flow estimates.

[2] Although the source of the baseflow estimate is not explicitly stated in the 2018 or 2020 Pare reports, New England Flow Policy documentation was provided as a reference, and this policy documentation estimates an average annual flow of approximately 2 cfsm for the New England Region.

peak flow frequency analysis were evaluated to identify the 5% and 95% exceedance flows during the identified upstream and downstream migration periods.

Table 2.2-1: Timing of Important Life Cycle Events for Target Species

Species	Life Stage	Event	Month									
			MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
River herring	adults	*upstream migration*		4/15		6/15						
	juveniles	*downstream emigration*					7/1				11/30	
American shad	adults	*upstream migration*			5/1		7/15					
	juveniles	*downstream emigration*					7/1				11/30	
American eel	glass eels & elvers	*upstream migration*		4/1						10/31		
	silver eels	*downstream emigration*							9/1			12/31
Sea lamprey	adults	*upstream migration*			5/1	6/30						
	trans-formers	*downstream emigration*							9/1			12/31

2.3 Climate Change Projections

The occurrence of heavy precipitation events are projected to increase, with a slight increase in the number of dry days (Easterling et al., 2017). Generally, this may lead to longer periods of drought, punctuated by intense downpours. As such, many agencies have begun evaluating how to address infrastructure resilience in the face of a changing climate, including Massachusetts and surrounding states. Recommendations for climate change consideration with regards to flow estimation were identified within Massachusetts, New Hampshire, and New York. Rhode Island has not developed their own standards, but references Massachusetts and New York in their stream crossing design manual. Finally, no recommendations were found in Connecticut or Maine. The following section outlines the guidance found for Massachusetts, New Hampshire, and New York. It should be noted that most infrastructure resilience recommendations focus on flood events. While the Massachusetts Climate Change Clearinghouse (MACCC) indicates that low flow projections are "coming soon" (MACCC, n.d.), no good guidelines were found to currently exist regarding potential impacts to low flows for fish passage design.

2.3.1 Massachusetts

The Resilient MA Action Team (RMAT) is an inter-agency team tasked with implementation of the State Hazard Mitigation and Climate Adaptation Plan, which was adopted by the Commonwealth of Massachusetts in 2018. The RMAT Climate Resilience Design Standards Tool is a webtool which provides a preliminary assessment of climate risk and recommended climate resilience design standards for state-funded projects. This study used the beta version of the webtool to determine the appropriate approach for the hydrologic analyses of potential future climate change scenarios. After entering basic project data, the webtool recommended using the Tier 2 Methodology for determining projected total precipitation depths for 24-hour design storms. This methodology consists of applying precipitation multipliers to the 24-hour precipitation depths for each design storm provided in NOAA Atlas 14 as shown in **Table 2.3.1-1**.

Table 2.3.1-1: Total Precipitation Depth Design Criteria for Tier 2 Methodology from RMAT Climate Resilience Design Standards

Location	Design Storm	Mid-Century 2030/2050	Late Century 2070/2090
Massachusetts (all counties except Hampden)	More Frequent Design Storm*	8%	20%
	100-yr Design Storm	11%	27%
	Extreme Design Storm**	15%	36%

* More Frequent includes 2-yr, 5-yr, 10-yr, 25-yr, 50-yr Design Storms

** Extreme includes 200-yr, 500-yr Design Storms

Table 2.3-2 provides projected 24-hour precipitation depths for the AEP Events utilized in this study. These depths were used as input to the existing hydrologic model[3], in order to estimate the projected flows due to climate change.

Table 2.3-2: Projected 24-hour Total Precipitation Depths for Duxbury, MA using RMAT Tier 2 Methodology

	Precipitation (inches)		
Frequency (years):	**5**	**25**	**100**
NOAA Atlas 14, 24-hr:	**4.28**	**6.04**	**7.61**
with Mid-Century multiplier	4.62	6.52	8.45
with Late-Century multiplier	5.14	7.25	9.66

2.3.2 New Hampshire

The University of New Hampshire (UNH) recently published two reports – New Hampshire Coastal Flood Risk Summary Part I: Science (2019) and Part II: Guidance for Using Scientific Projections (2020) – which were prepared in partnership with the New Hampshire Coastal Flood Risk Science and Advisory Panel and the New Hampshire Department of Environmental Services. The Part I report describes the projected increases to extreme precipitation events under the representative concentration pathways (RCP) 4.5 and 8.5 scenarios by end of century. It is projected that the precipitation falling on the wettest day of the year will increase in magnitude by 8-18% under RCP 4.5 and 13-24% under RCP 8.5 by the end of the century (UNH, 2019).

The Part II: Guidance for Using Scientific Projections report recommends applying a 15% increase to extreme precipitation estimates for projects with high to medium tolerance for flood risk, and a greater than 15% increase for projects with low to very low tolerance for flood risk (UNH, 2020).

2.3.3 New York

In response to projected changes in climate, New York State (NYS) passed the Community Risk and Resiliency Act (CRRA) in 2014. In accordance with the guidelines of the CRRA, in 2020 the NYS Department of Environmental Conservation (NYSDEC) released the New York State Flood Risk Management Guidance for Implementation of the Community Risk and Resiliency Act. In the report, two methods for estimating

[3] The analysis also utilized NOAA Atlas 14 temporal distribution for 24-hour duration events for each design storm, as recommended by the RMAT webtool for Tier 2 projects.

projected future discharges were discussed: an end of design life multiplier and the USGS FutureFlow Explorer map-based web application (NYSDEC, 2020).

FutureFlow Explorer[4] was developed by the USGS in partnership with the NYS Department of Transportation (NYSDOT). This application is an extension for the USGS *StreamStats* map-based web application and projects future stream flows in New York State. However, this tool is still being tested and the USGS recommends using this tool only as qualitative guidance to inform selection of appropriate design flows.

While quantitative methods are under development, the NYSDEC recommends that future peak flow conditions should be adjusted by multiplying relevant peak flow parameters by a factor specific to the expected service life of the structure and geographic location of the project. For Eastern New York, the recommended design-flow multiplier is 20% increased flow for an end of design life of 2025-2100 (NYSDEC, 2020).

3 Results

Some of the hydrologic information developed for this analysis was based on locations other than Temple Street Dam. **Figure 3.0-1** provides an overview of the location and drainage area associated with the dam, and the two USGS gage locations utilized, while **Table 3.0-1** summarizes key basin parameters for each of these locations. The parameters in this table are generally from StreamStats, except for the drainage area. The drainage area for Temple Street is from the existing hydrologic model, while the drainage area for each gage is from its respective USGS National Water Information System webpage.

Table 3.0-1: Basin Parameters

Basin Parameter	Temple Street Dam	USGS Gage 01105730 (Indian Head River at Hanover, MA)	USGS Gage 01105870 (Jones River at Kingston, MA)
Drainage Area (mi^2)	5.9	30.3	19.8
ELEV (ft)	74.5	101	80.2
STOR (%)	24.84	20.33	26.51
Basin Slope (%)[1]	3.375	2.803	4.043
Impervious (%)	5.01	17.8	5.26
Developed (%)	23.6	50.7	19.8
Forested (%)	59.24	26.72	63.54

Notes:

1. Streamstats reports the storage from two separate sources. This table displays BSLDEM10M.

3.1 Event Based Flows

Table 3.1-1 provides a summary of the event based flows developed for this analysis. The existing HydroCAD model generally provided the highest estimated flows. The 20% AEP was only 6 cfs (<5%) lower than the estimate flow based on the prorated Indian Head River frequency analysis.

[4] https://ny.water.usgs.gov/maps/floodfreq-climate/

Table 3.1-1: Comparison of Peak Discharge Estimates at Temple Street Dam

Annual Exceedance Probability (%)	Recurrence Interval (yrs)	Peak Discharge (cfs)			
		Prorated Jones River[1]	Prorated Indian Head River[2]	HydroCAD (2020 Pare)[3]	StreamStats
20%	5	110	167	160	147
10%	10	142	201	230	191
4%	25	188	249	320	254
2%	50	226	287	400	306
1%	100	267	328	480	361

Notes:

1. The period of annual maximum flow records available for the Jones River gage at the time of this analysis was 1967 to 2018. However, only 1967 through 1978 were utilized, due to the qualification codes used regarding regulation and diversion. The Jones River PeakFQ results were adjusted by a drainage area ratio of approximately 0.30 (5.9 mi^2/19.8 mi^2 = 0.30).
2. The period of annual maximum flow records available for the Indian Head River gage at the time of this analysis was 1967 to 2020. The Indian Head River PeakFQ results were adjusted by a drainage area ratio of approximately 0.19 (5.9 mi^2/30.3 mi^2 = 0.19).
3. Flows from HydroCAD at Temple Street Dam represent discharge from four drainage areas routed through three hydraulic structures.

3.2 Period Based Flows

Table 3.2-1 provides a summary of the period based flows developed for this analysis. The baseflow at Temple Street Dam, used in previous analyses was approximately 12 cfs. The prorated flows between the two gages are fairly similar for all periods and exceedances analyzed. The Jones River drainage area is more similar to the Temple Street Dam drainage area in terms of impervious area, developed, and forested areas. These metrics typically have a large influence on the low flow regime within a watershed. However, the USGS notes that flow at the Jones River gage may be affected by upstream regulation, wastage from Silver Lake, ground water that enters from or moves into adjacent basins, and occasional backwater from tidal surge.

Table 3.2-1: Summary of Period Based Flows for the Hydraulic Model

Period	Exceedance (%)	Flow (cfs)	
		Prorated Jones River[1]	Prorated Indian Head River[2]
Annual (1/1-12/31)	50%	10	8
Upstream Passage (4/15-7/15)	5%	33	33
	95%	4	2
Downstream Passage (7/1-12/31)	5%	26	30
	95%	2	1

Notes:

1. The period of daily average flow records available for the Jones River gage at the time of this analysis was 8/1/1966 – 12/12/2021. The Jones River PeakFQ results were adjusted by a drainage area ratio of approximately 0.30 (5.9 mi^2/19.8 mi^2 = 0.30).
2. The period of daily average flow record available for the Indian Head River gage at the time of this analysis was 7/8/1966 – 12/12/2021. The Indian Head River PeakFQ results were adjusted by a drainage area ratio of approximately 0.19 (5.9 mi^2/30.3 mi^2 = 0.19).

3.3 Climate Change Projections

Massachusetts and New Hampshire provided guidance on precipitation multipliers to be applied when considering climate change, while New York provided guidance on flow multipliers to be applied when considering climate change. **Table 3.3-1** summarizes the proposed precipitation multipliers. Since the Massachusetts recommendations bound the range of recommendations, these multipliers were applied to the precipitations used in the existing HydroCAD model. The resulting flows are presented in **Table 3.3-2**, along with the flows from the existing HydroCAD model with New York's proposed flow multipliers applied. New York's recommendations are similar to Massachusetts recommendations for Mid-century 2050/7070, while Massachusetts recommendations for Late-Century are much higher.

Table 3.3-1: Summary of Precipitation Multipliers for Climate Change Consideration

State	Scenario	Precipitation Increase (%)		
		20% AEP	4% AEP	1% AEP
Massachusetts	Mid-Century (2030/2050)	8%	8%	11%
New Hampshire	High to Medium Flood Risk Tolerance	15%	15%	15%
New Hampshire	Low to Very Low Flood Risk Tolerance	>15%	>15%	>15%
Massachusetts	Late Century (2070/2090)	20%	20%	27%

Table 3.3-2: Summary of Peak Flows at Temple Street Dam with Climate Change Consideration

Annual Exceedance Probability	Recurrence Interval (yrs)	Peak Discharge (cfs)			
		Current (2020 Pare)	Massachusetts: Mid-Century 2050/2070	Massachusetts: Late Century 2070/2090	New York: 2025-2100
20%	5	160	193	238	192
4%	25	320	366	441	384
1%	100	480	584	729	576

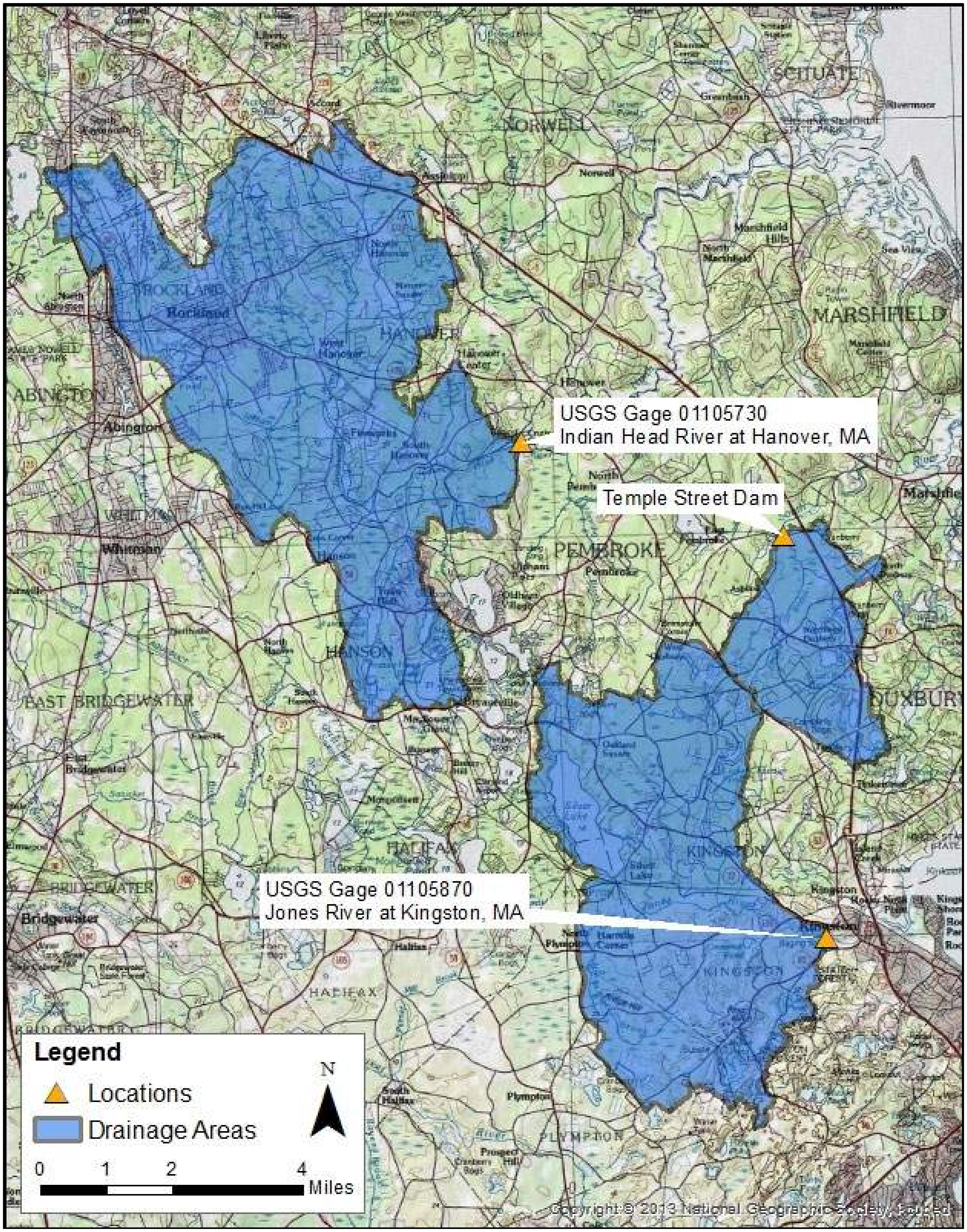
4 Conclusions

Table 4.0-1 presents the flows recommended for analysis during the 75% design, based on the analysis performed for this study and consultation with the project partners. For event-based flows, it is proposed that the existing hydrologic model results be used for current hydrologic conditions, and that the Massachusetts Mid-Century 2050/2070 precipitation multipliers be applied to the existing model for projected hydrologic conditions. For period based flows, despite the similarity in results between the two gages and the similarity in basin metrics between the Jones River gage and the Temple Street Dam, it is proposed that the Indian Head River gage flows be used, due to uncertainties with the accuracy of flows at the Jones River gage, and the wider range of design flows provided by the Indian Head River gage. The time-varying inflow hydrograph from the HydroCAD model will be used for the event based flows, while a steady inflow hydrograph will be used for the period based flows.

Table 4.0-1: Summary of 75% Design Basis Flows

Flow Type	Description	Peak Flow at Temple Street Dam (cfs)
Event Based	20% AEP (Current)	160
	4% AEP (Current)	320
	1% AEP (Current)	480
	20% AEP (Projected)	193
	4% AEP (Projected)	366
	1% AEP (Projected)	584
Period Based	Sunny Day	8
	Upstream Migration (4/15-7/15) High Flow (5%)	33
	Upstream Migration (4/15-7/15) Low Flow (95%)	2
	Downstream Migration (7/1-12/31) High Flow (5%)	30
	Downstream Migration (7/1-12/31) Low Flow (95%)	1

Figure 3.0-1: Overview of Analysis Locations



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Appendix E – Hydraulic Model Output

Appendix E – Hydraulic Model Output

Figure 2.5.1-1 – Water Surface Profile through Riffle Features – 95% Upstream Fish Passage Season Exceedance Flow (33cfs) Water Surface Profile

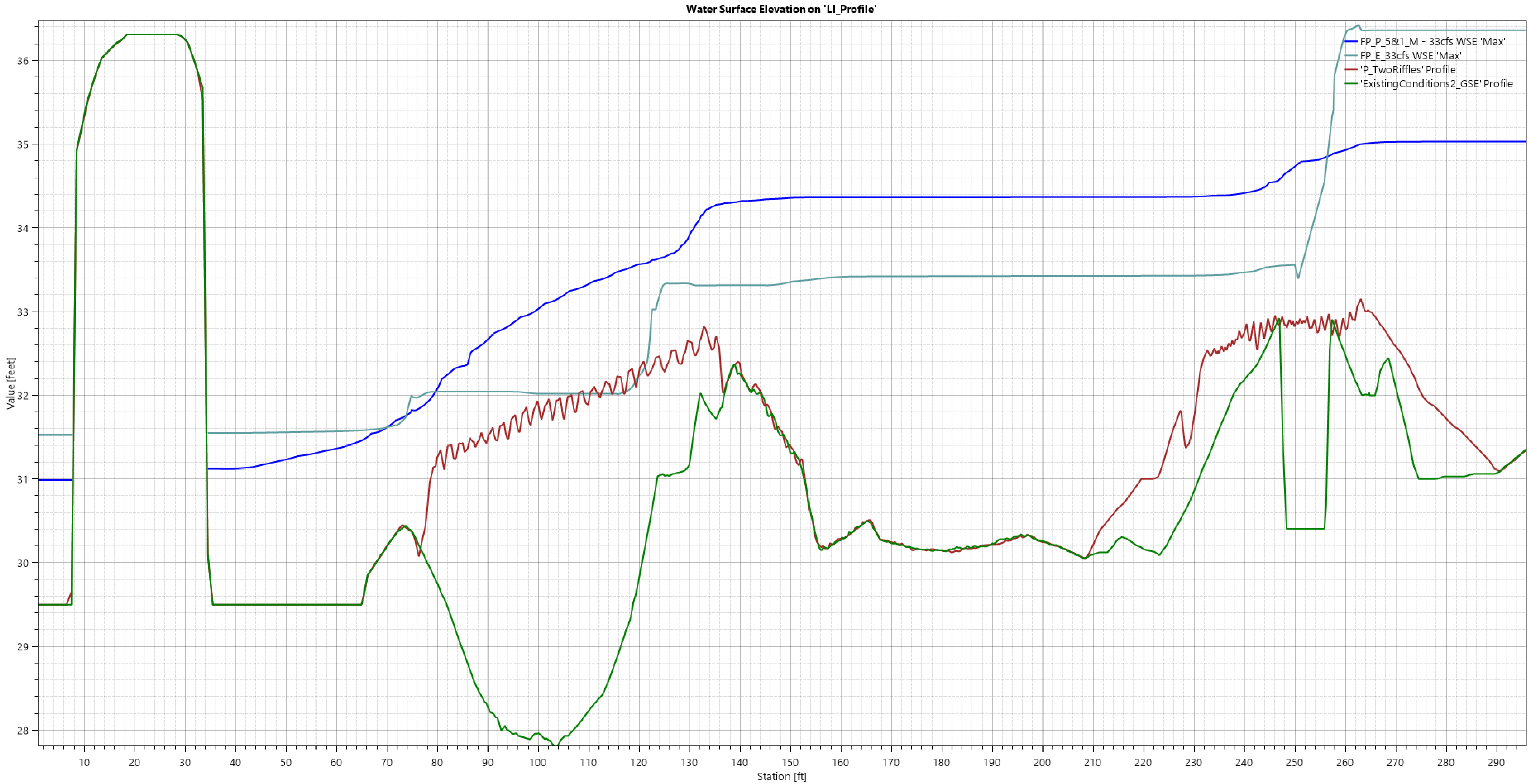


Figure 2.5.2-1 – Riffle Hydraulics – 1 cfs (95% Upstream Fish Passage Season Exceedance Flow) – Depth Map

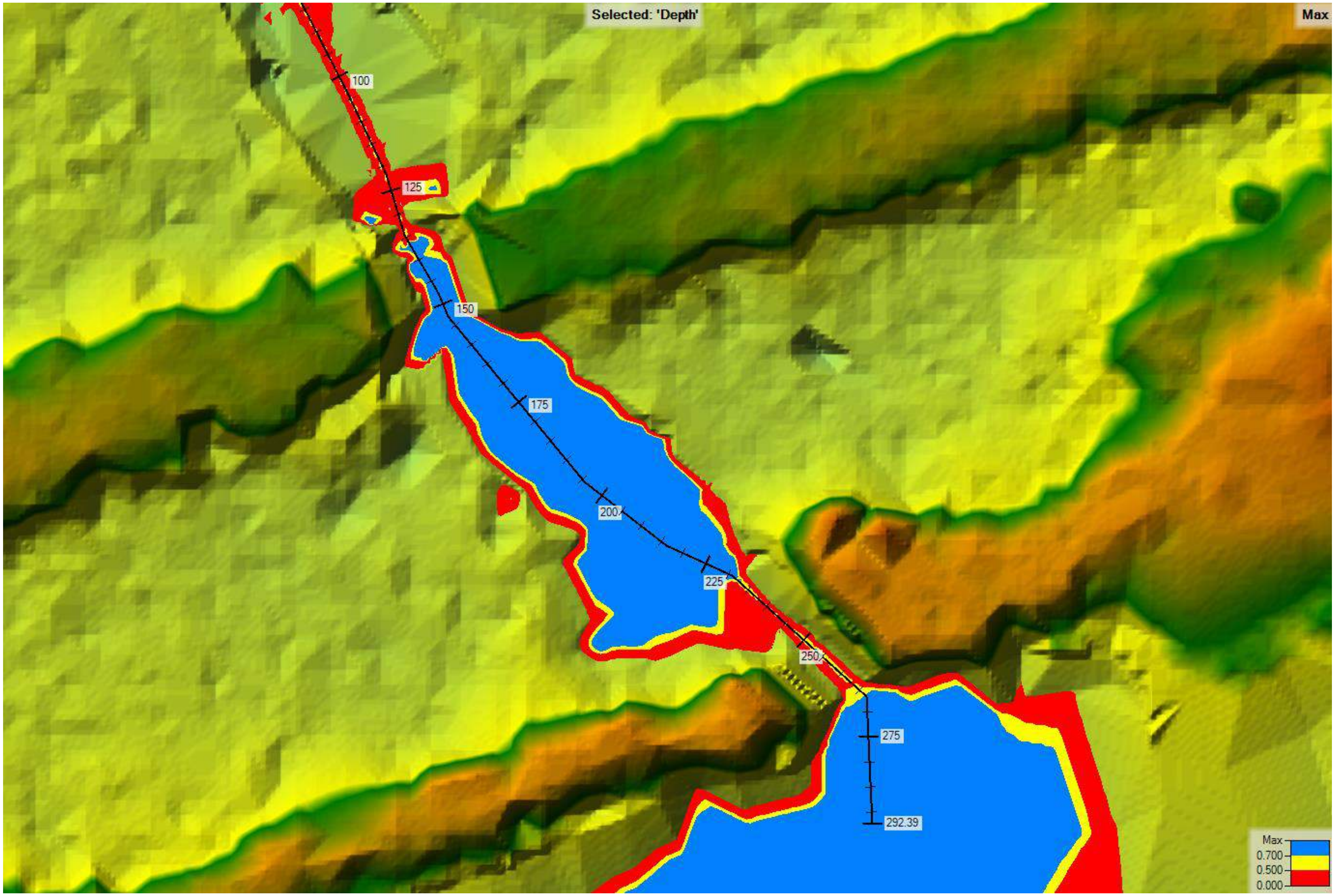


Figure 2.5.2-2 – Riffle Hydraulics – 33 cfs (95% Upstream Fish Passage Season Exceedance) – Depth Map

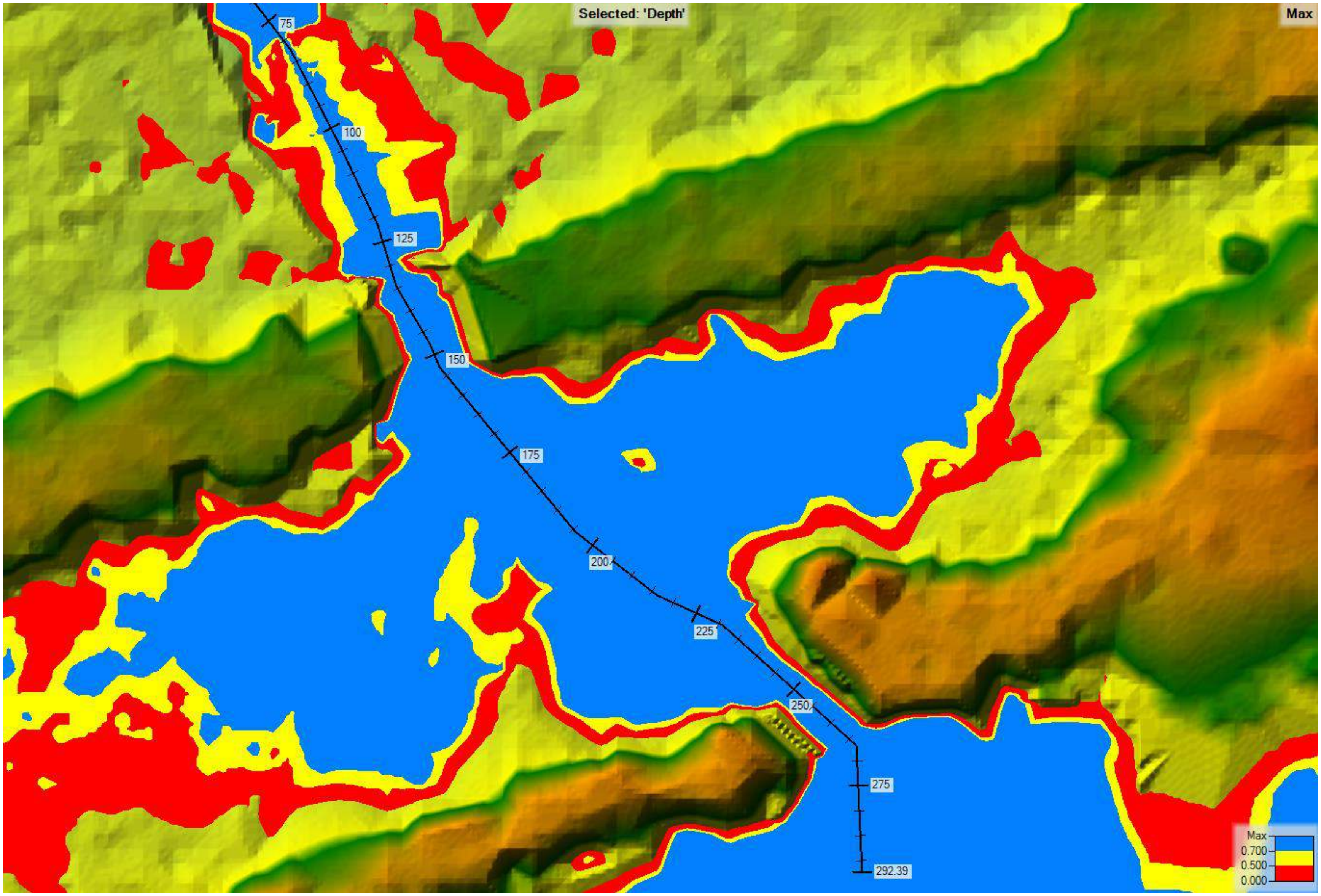


Figure 2.5.2-3 – Riffle Hydraulics – 1 cfs (95% Upstream Fish Passage Season Exceedance) – Velocity Map

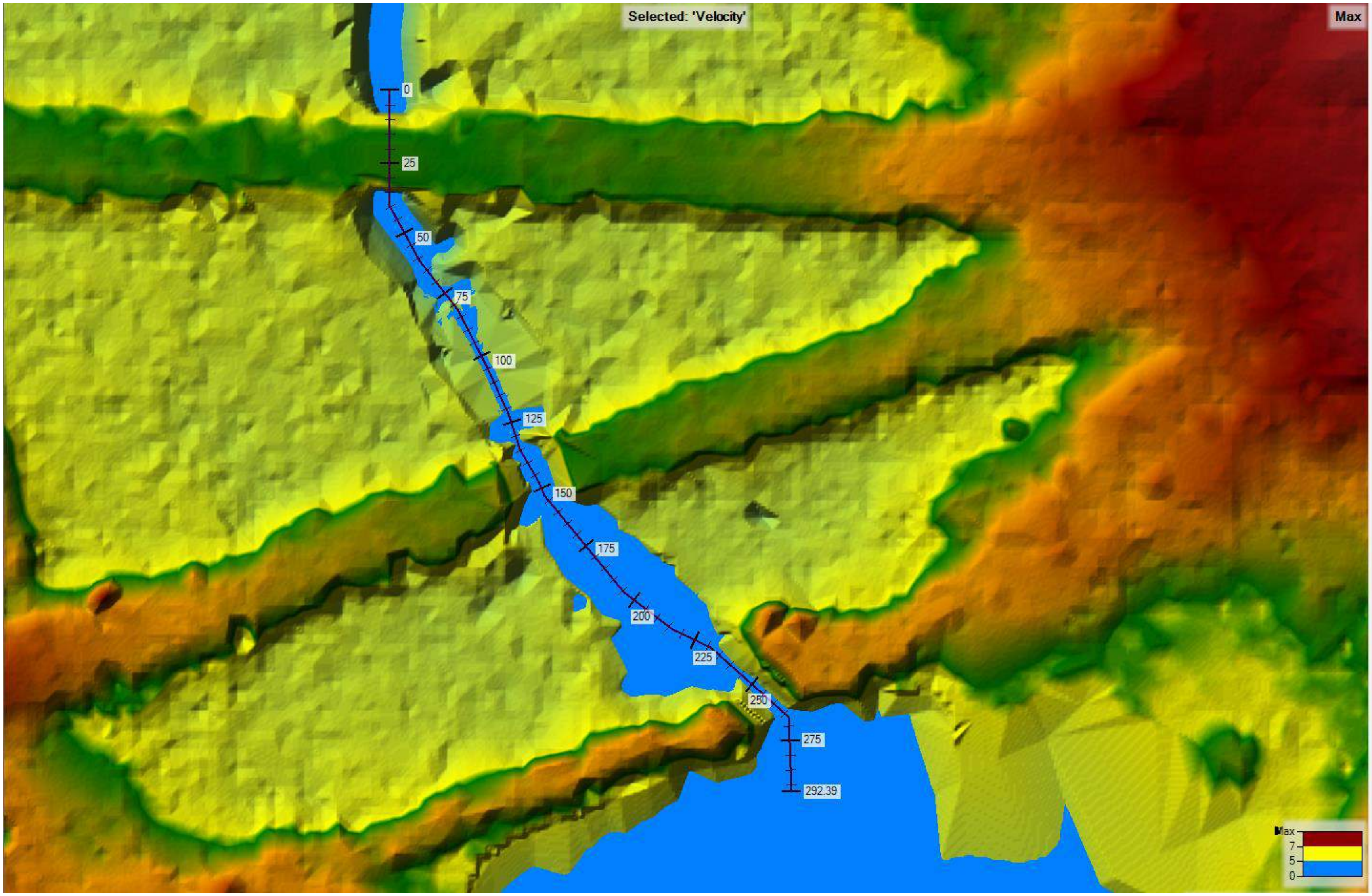


Figure 2.5.2-4 – Riffle Hydraulics – 33 cfs (95% Upstream Fish Passage Season Exceedance) – Velocity Map

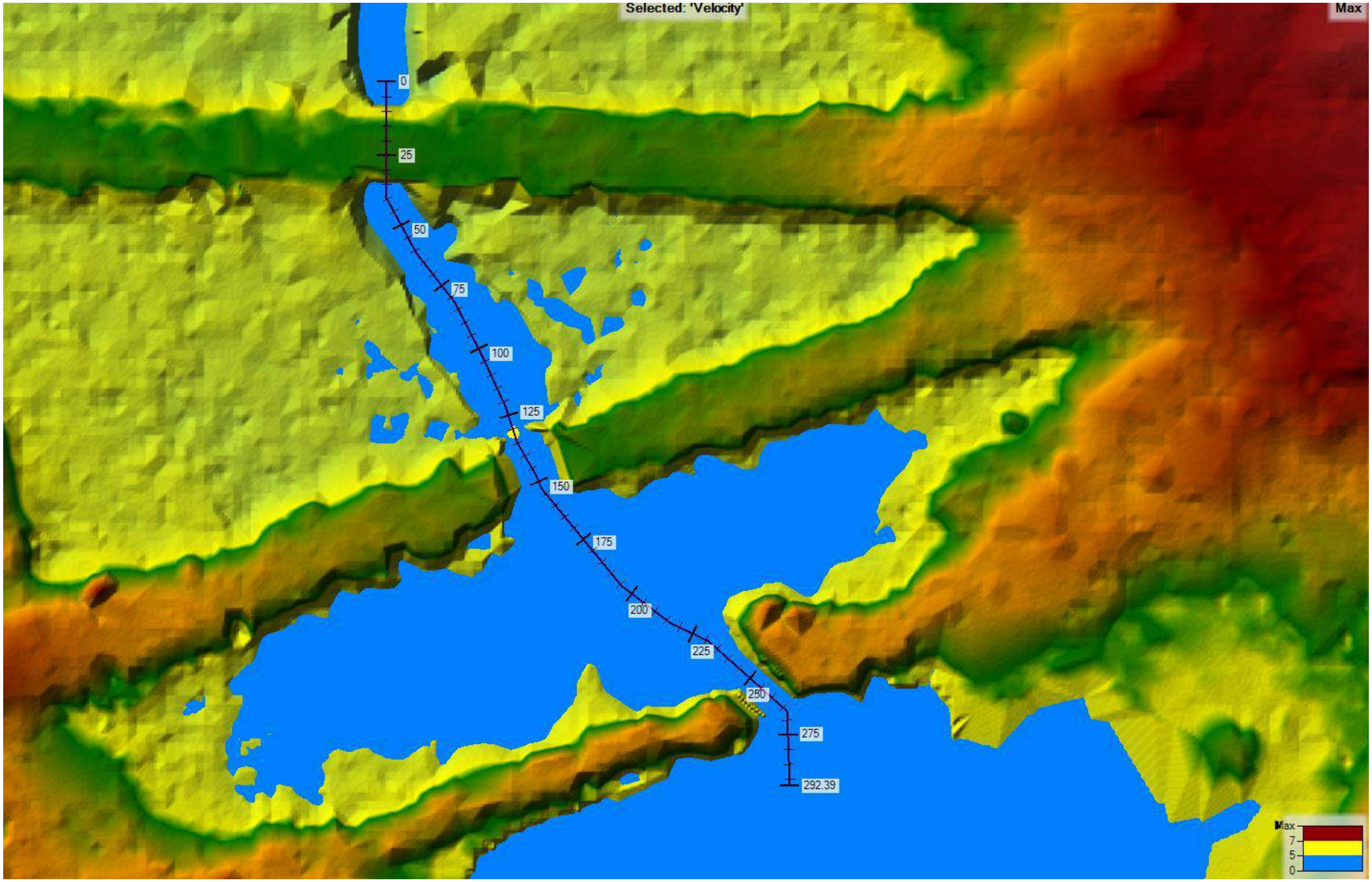


Figure 2.5.3-1 – Lower Impoundment - 1 cfs (95% Upstream Fish Passage Season Exceedance) –Depth Map

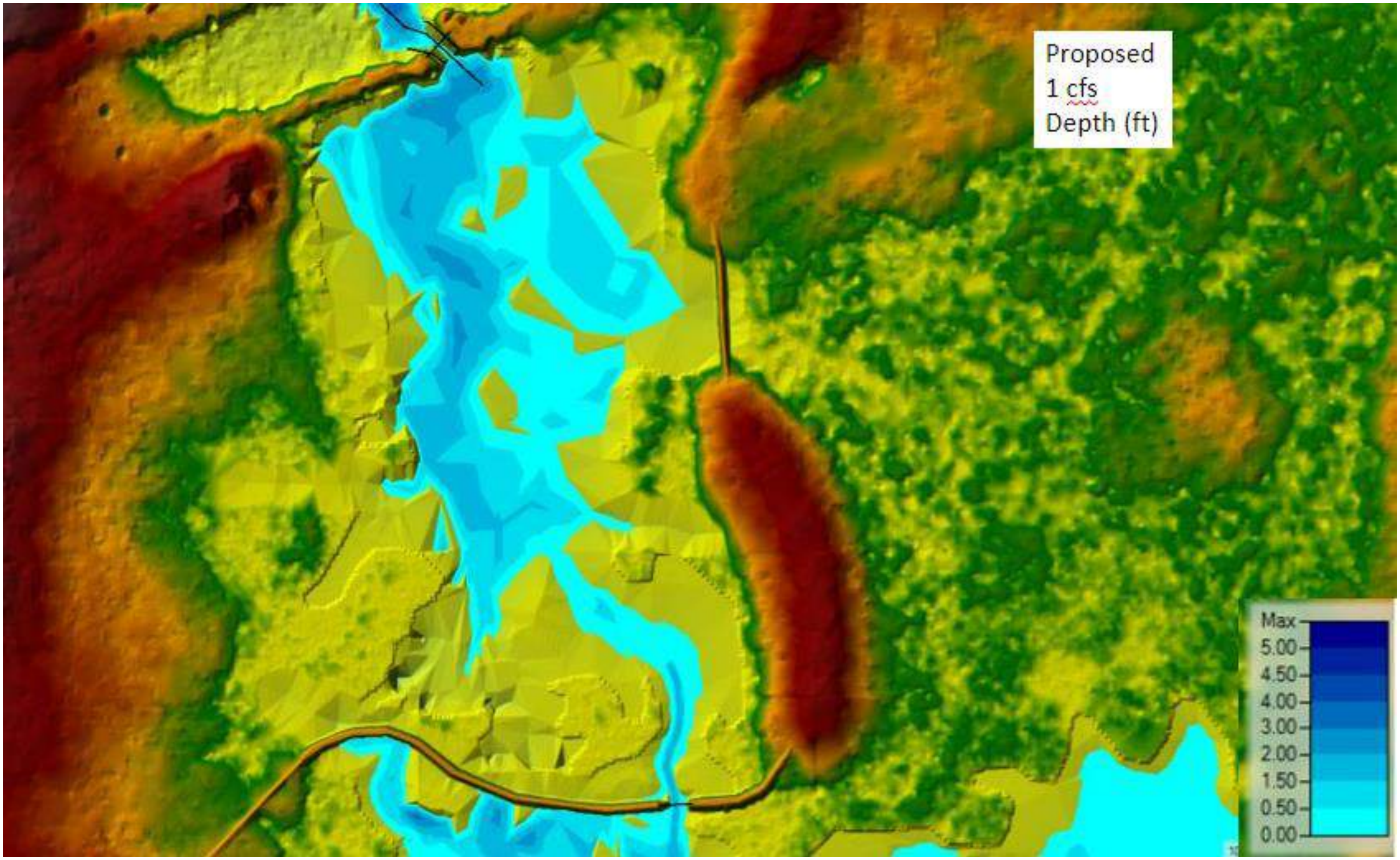


Figure 2.5.3-2 – Lower Impoundment - 33 cfs (95% Upstream Fish Passage Season Exceedance) – Depth Map

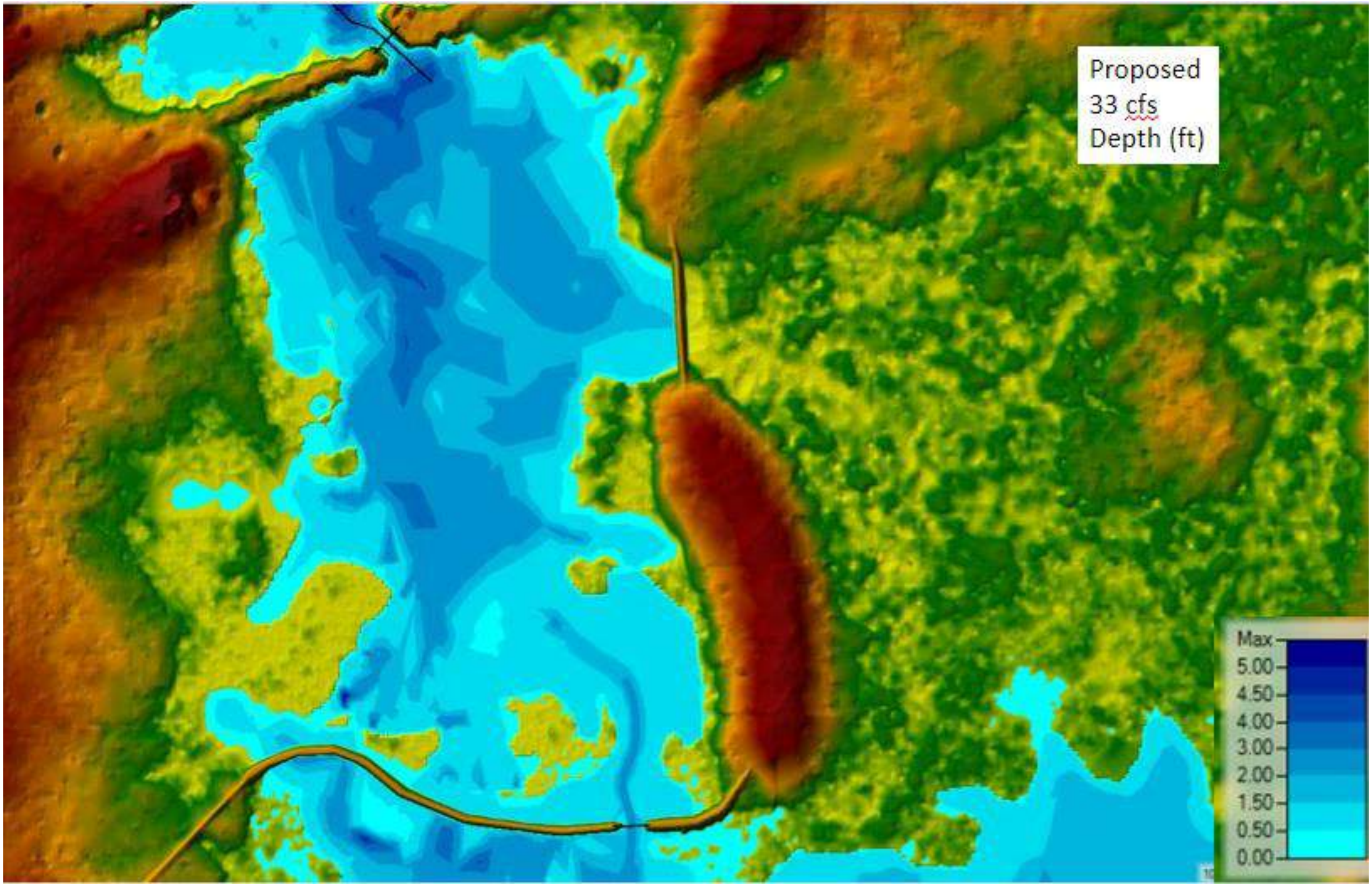


Figure 2.5.3-3 – Water Surface Extents – 95% Upstream Fish Passage Season Flow (1 cfs) (April 15 – July 15)– Existing (Blue) VS. Proposed (Green) Conditions

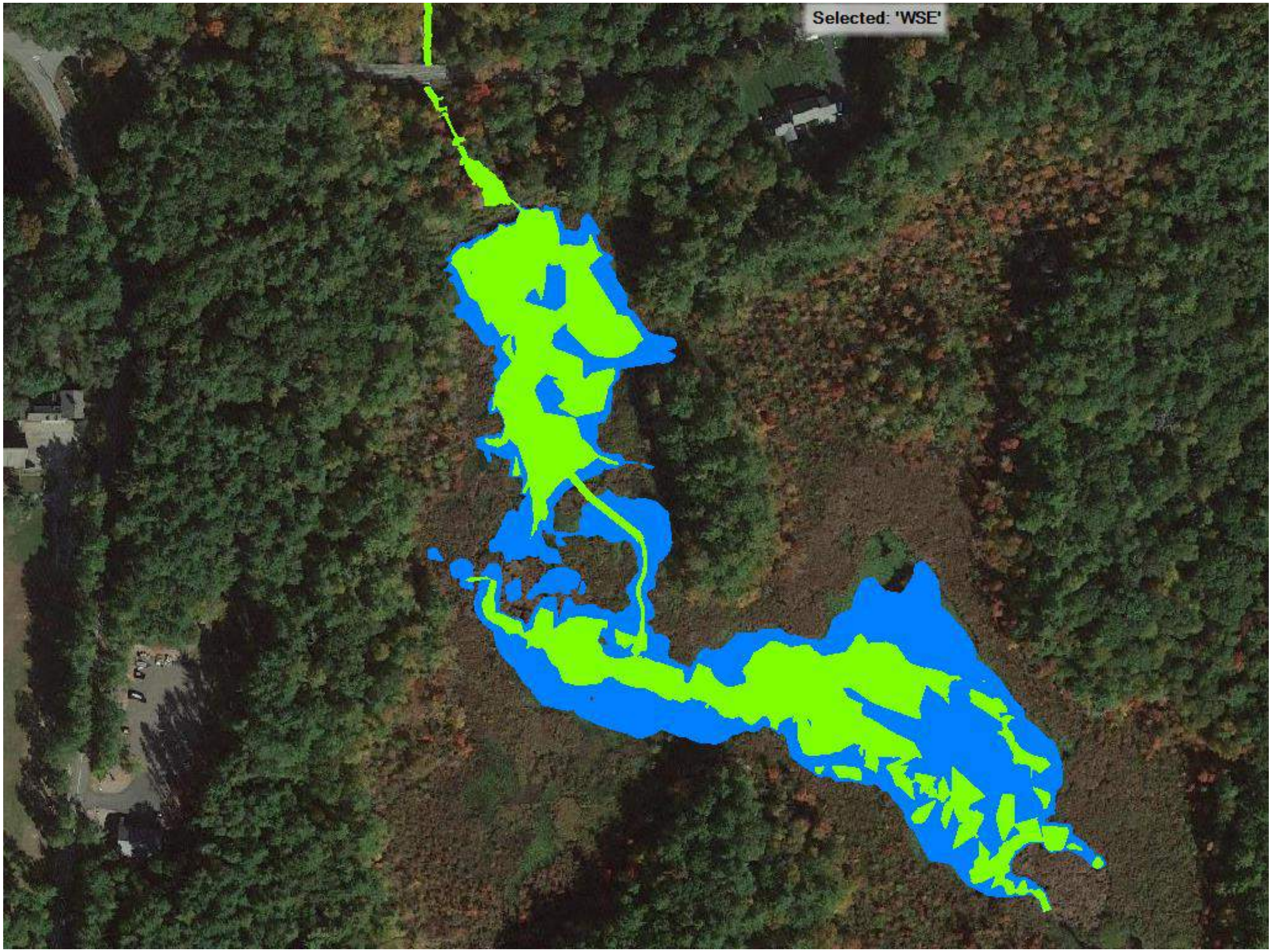


Figure 2.5.3-4 – Water Surface Extents - 5% Upstream Fish Passage Season Flow (33 cfs) (April 15 – July 15)– Existing (Blue) VS. Proposed (Green) Conditions

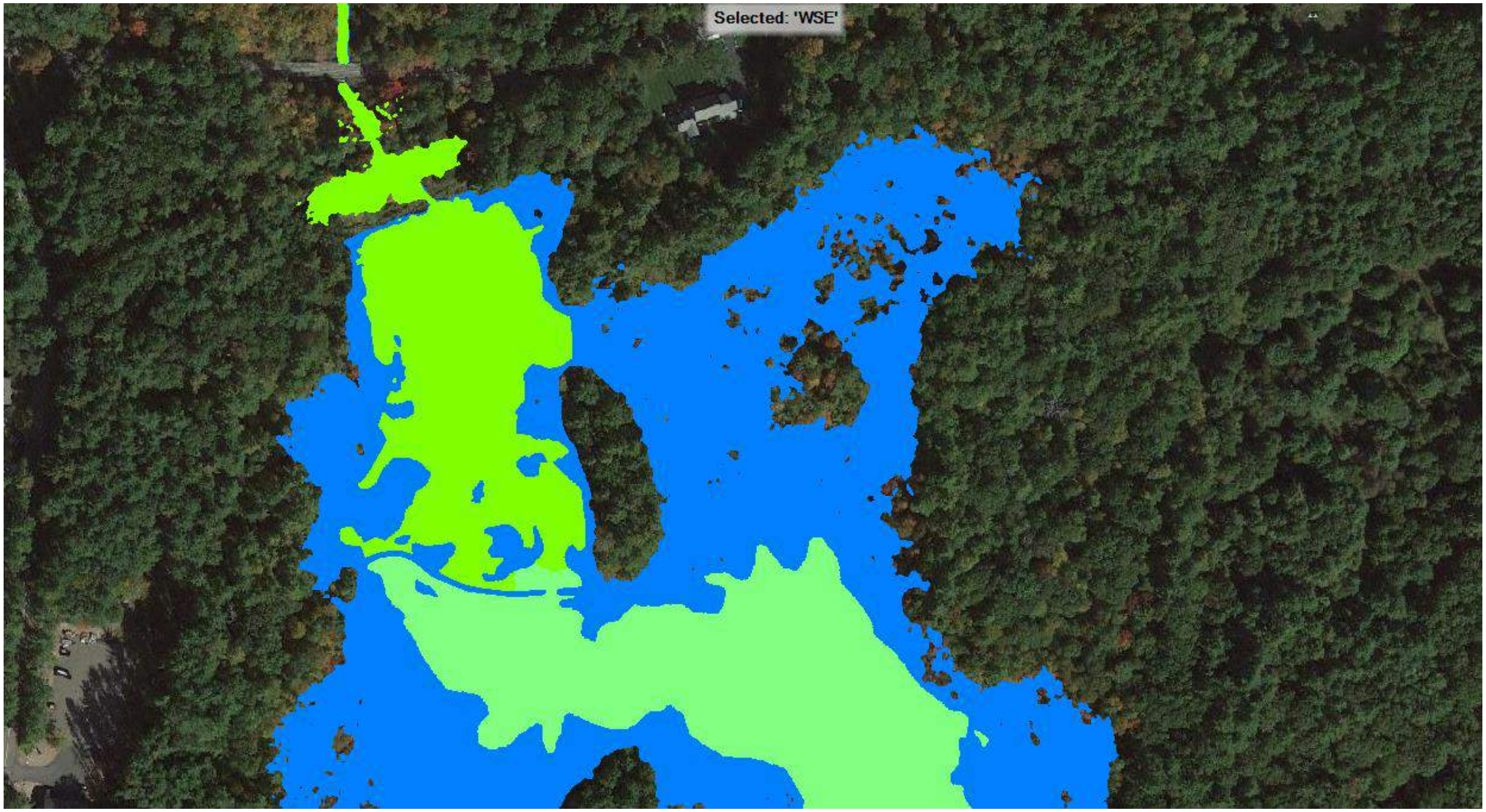


Figure 2.5.3-5 – Water Surface Extents – Bankfull Flow (119 cfs)– Existing (Blue) VS. Proposed (Green) Conditions

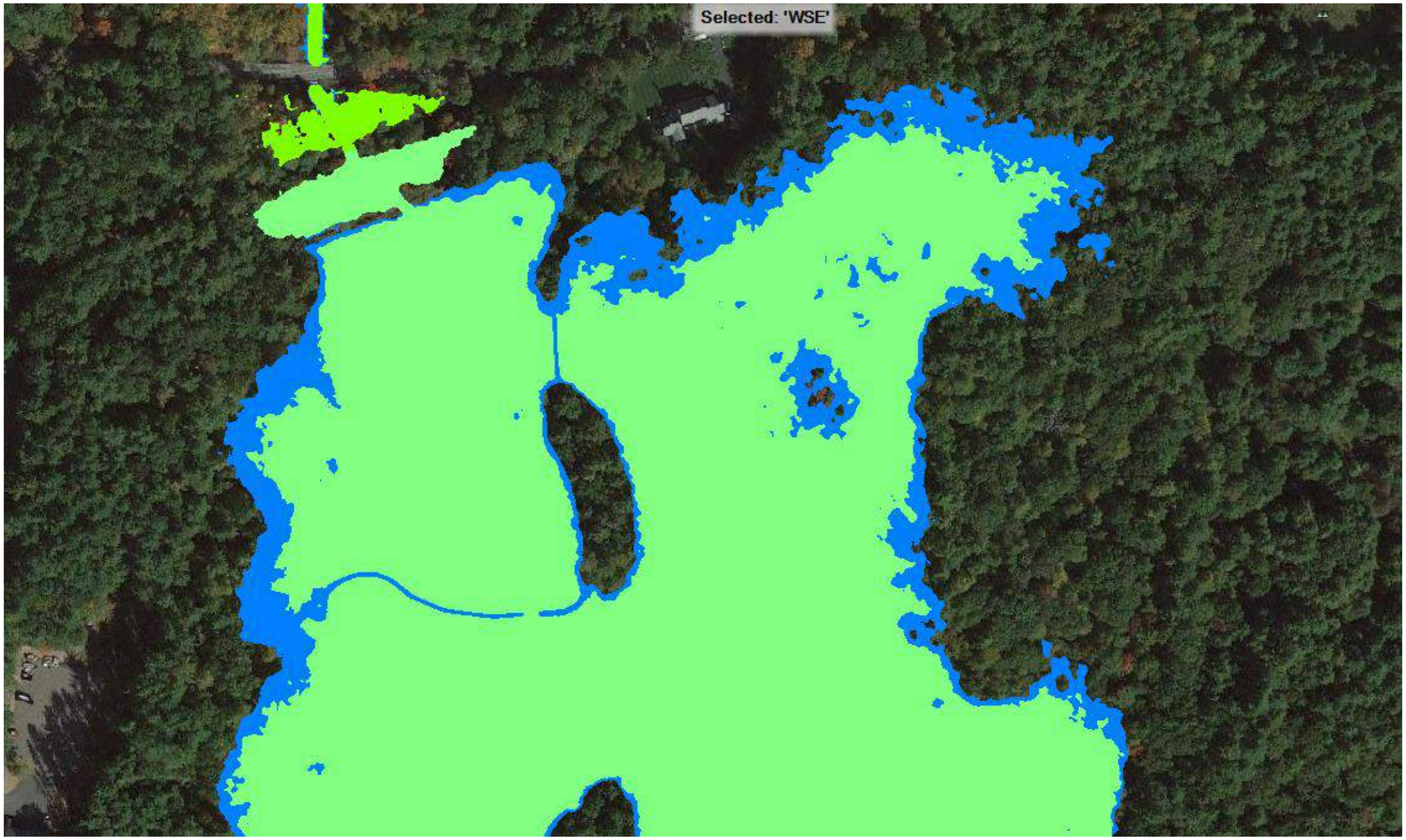


Figure 2.5.3-6 – Water Surface Extents – Median Duck Hunting Season Flow (5 cfs) (Oct. 10 – Nov. 26)– WSE Map – Existing (Blue) VS. Proposed (Green) Conditions

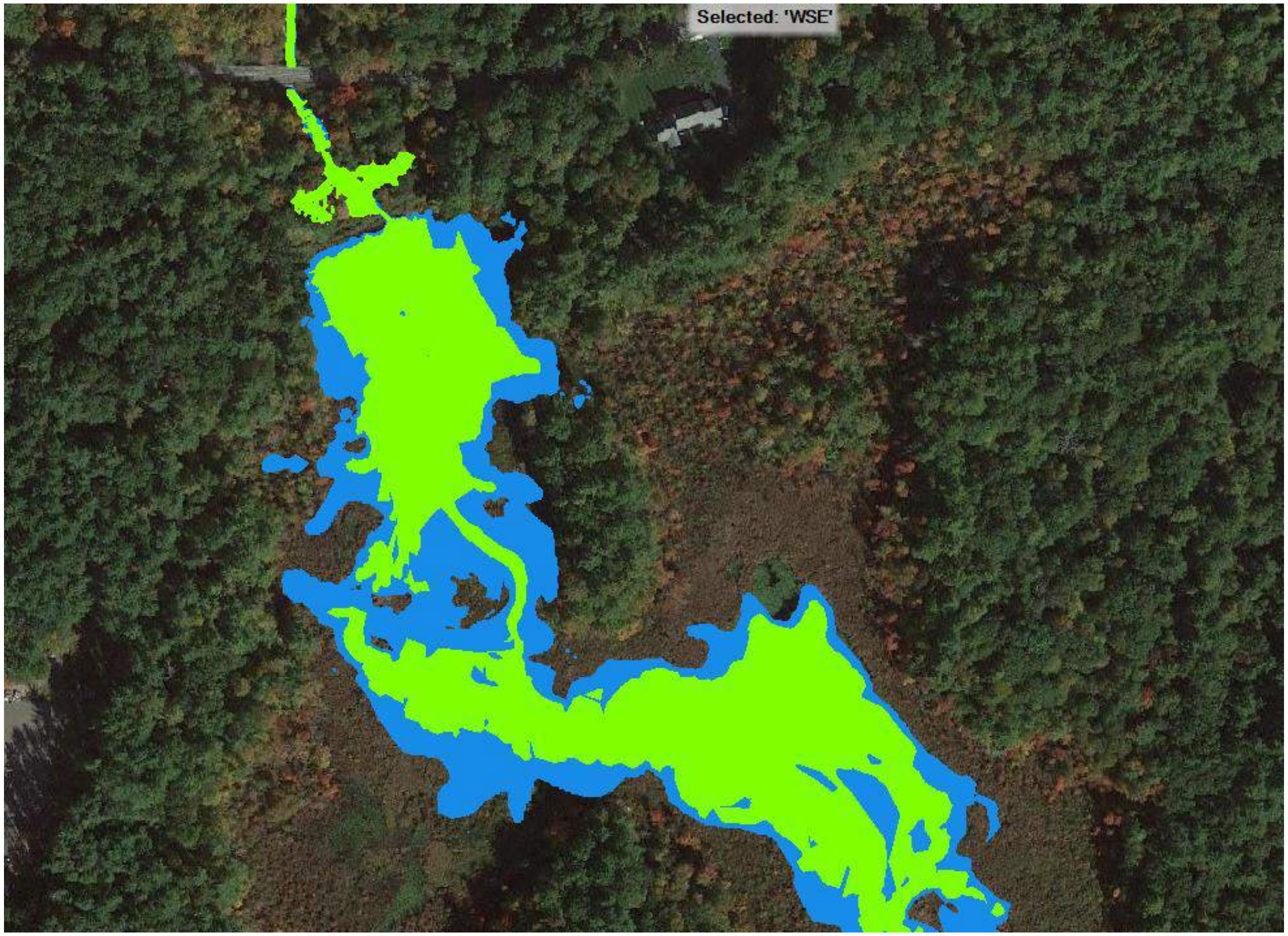


Figure 2.6.1-1: River Street Water Surface Elevations – (Existing vs Proposed Conditions) – 1% Recurrence Interval

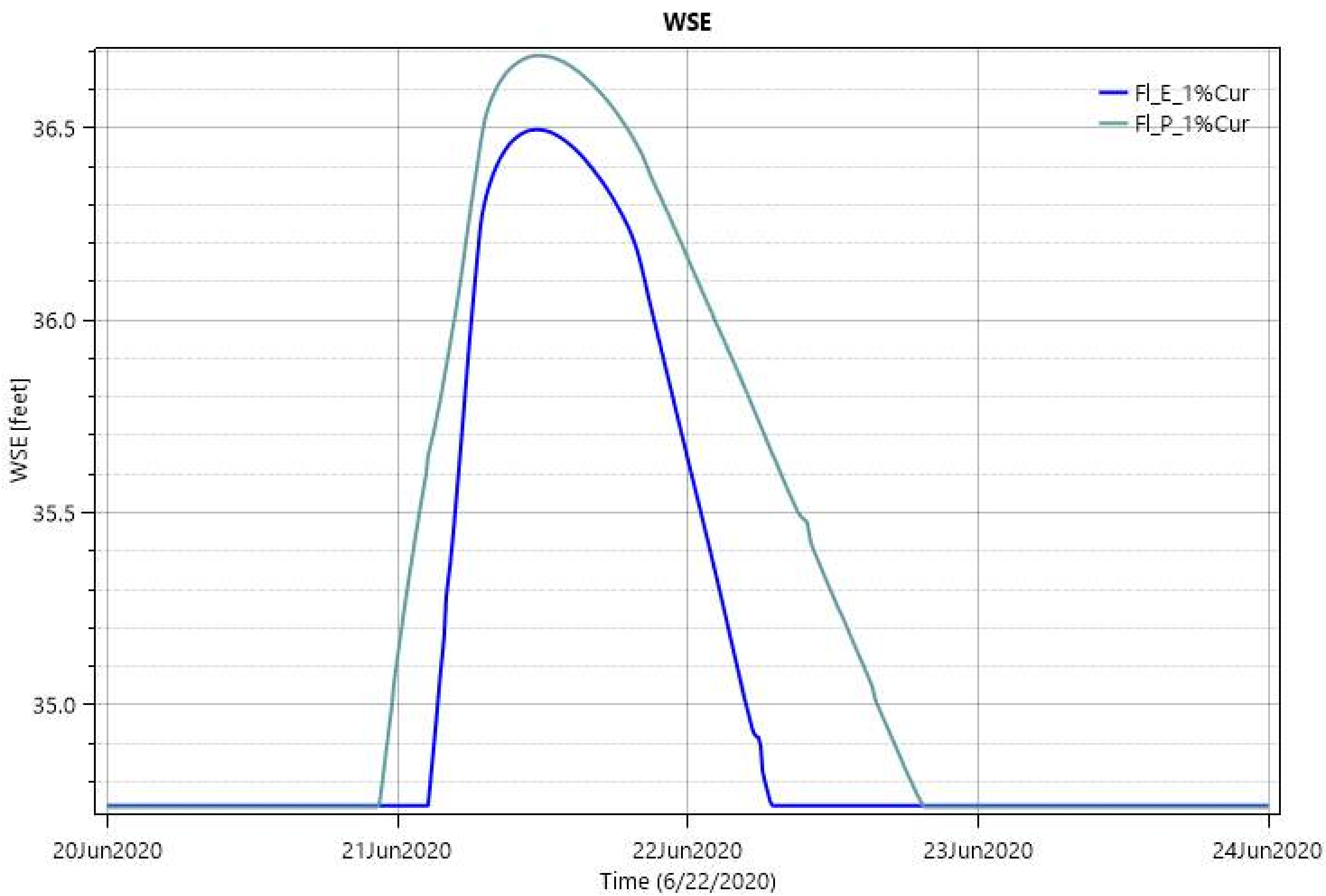


Figure 2.6.1-2: River Street Water Velocities – (Existing vs Proposed Conditions) – 1% Recurrence Interval

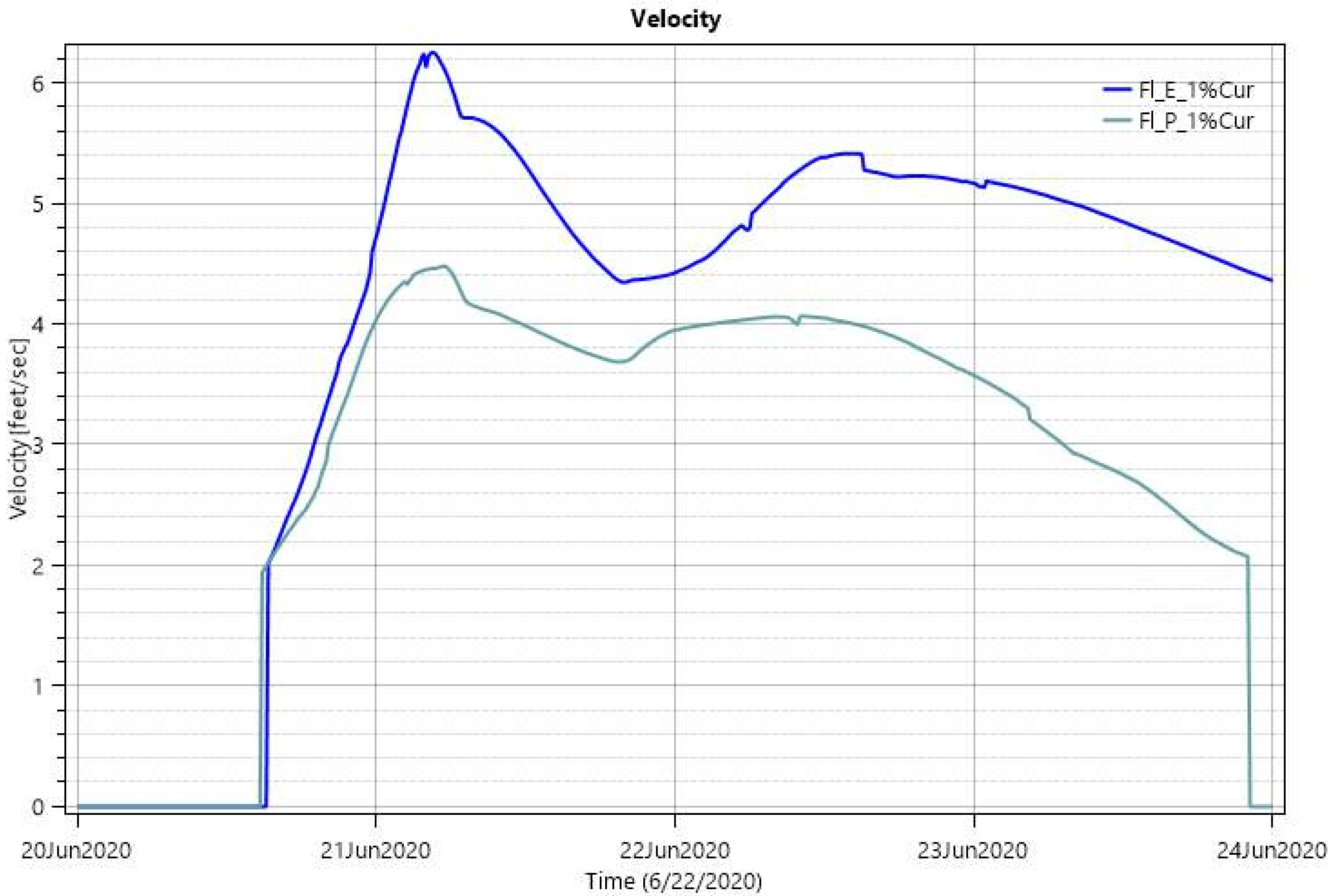


Figure 2.6.1-3: River Street Water Surface Elevations – (Existing vs Proposed Conditions) – 4% Recurrence Interval

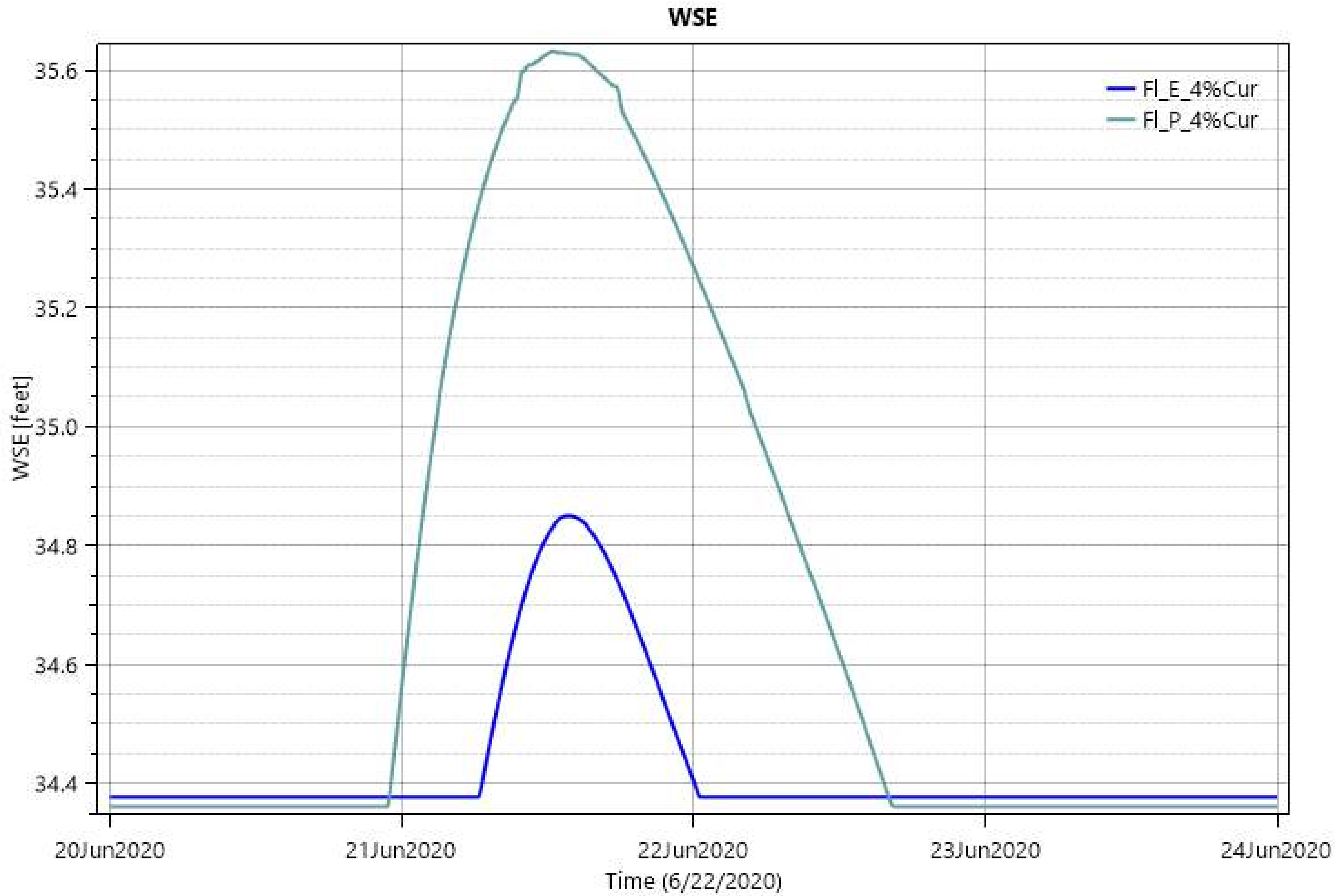


Figure 2.6.1-4: River Street Water Velocities – (Existing vs Proposed Conditions) – 4% Recurrence Interval

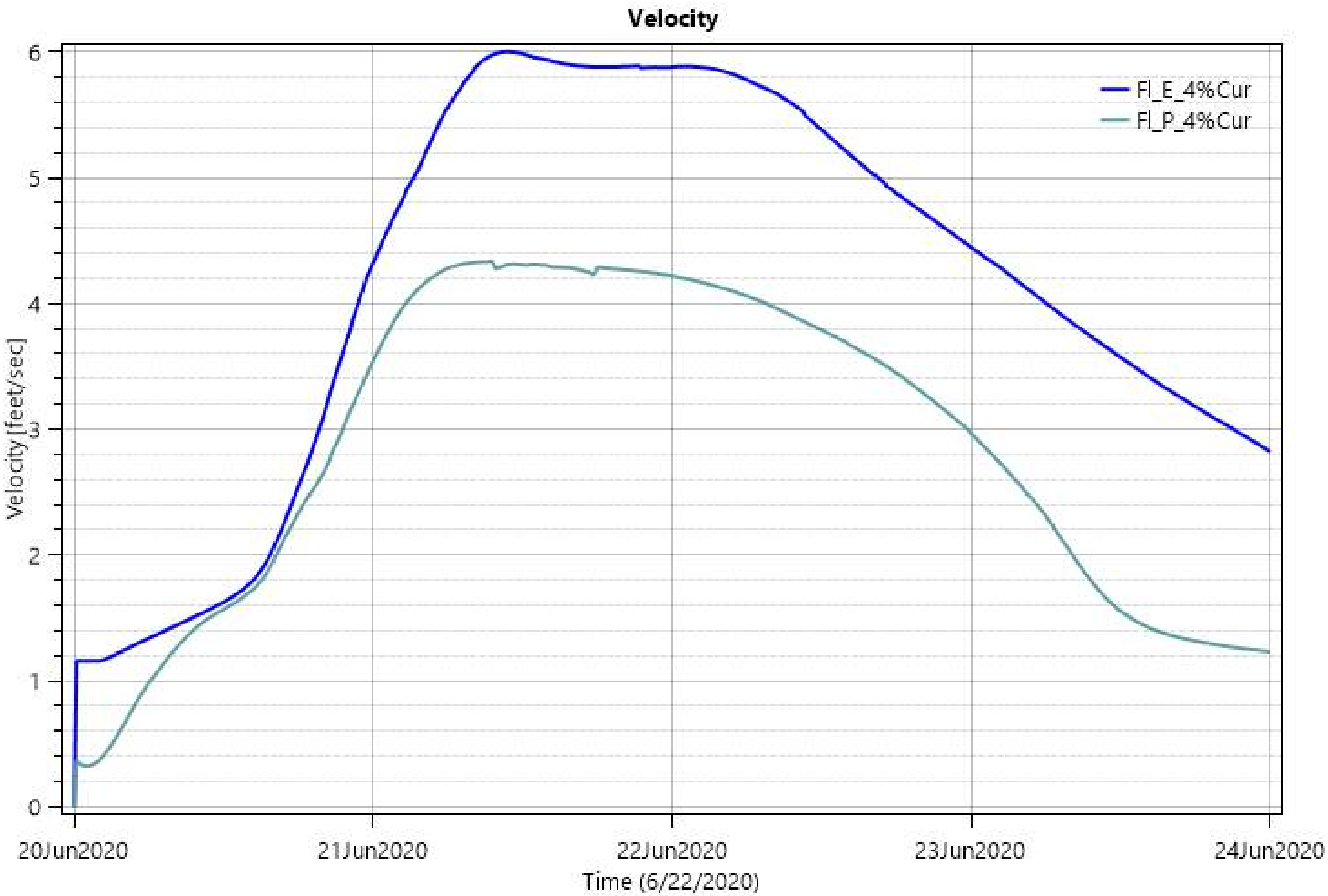


Figure 2.6.1-5: River Street Water Surface Elevations – (Existing vs Proposed Conditions) – 20% Recurrence Interval

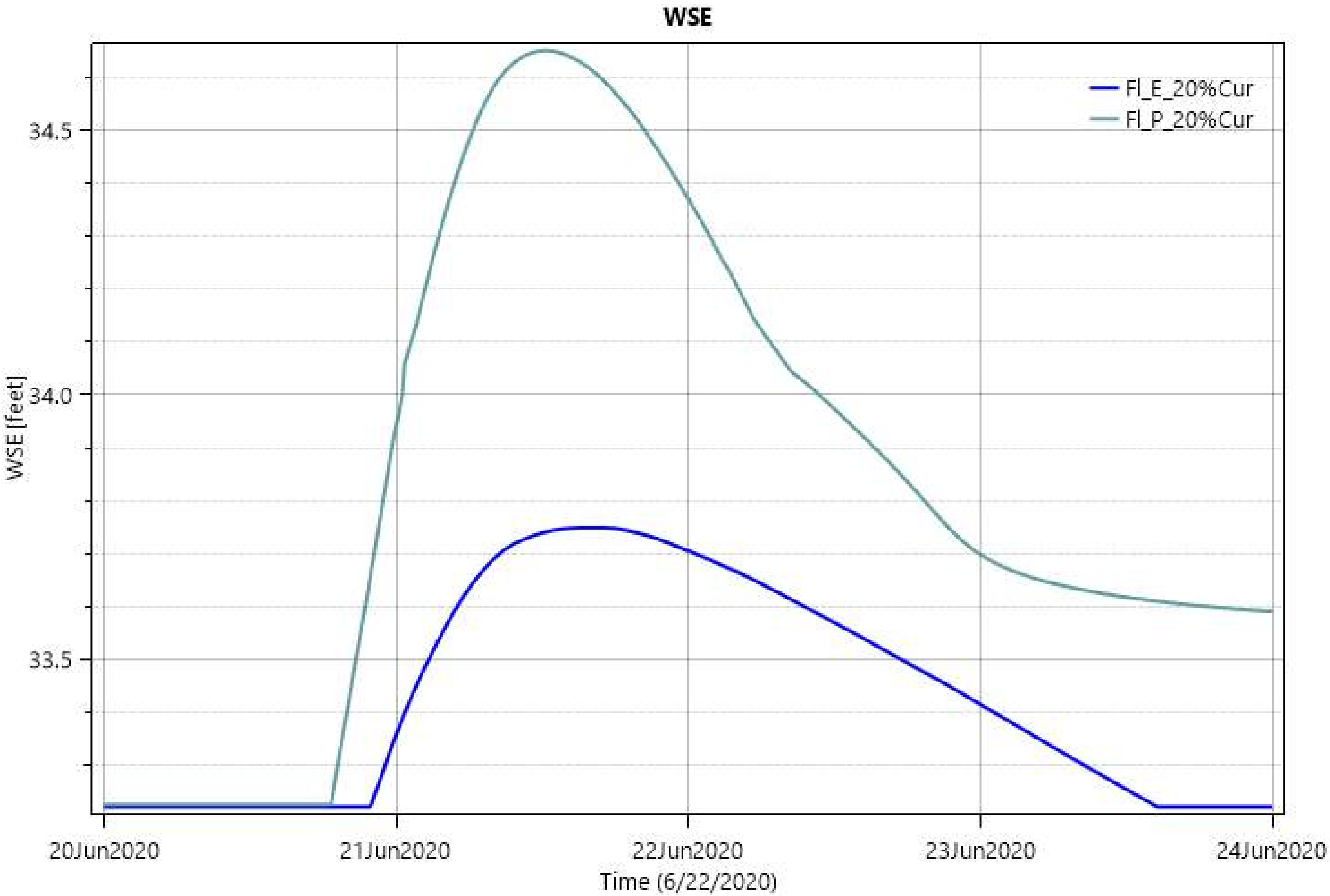


Figure 2.6.1-6: River Street Water Velocities – (Existing vs Proposed Conditions) – 20% Recurrence Interval

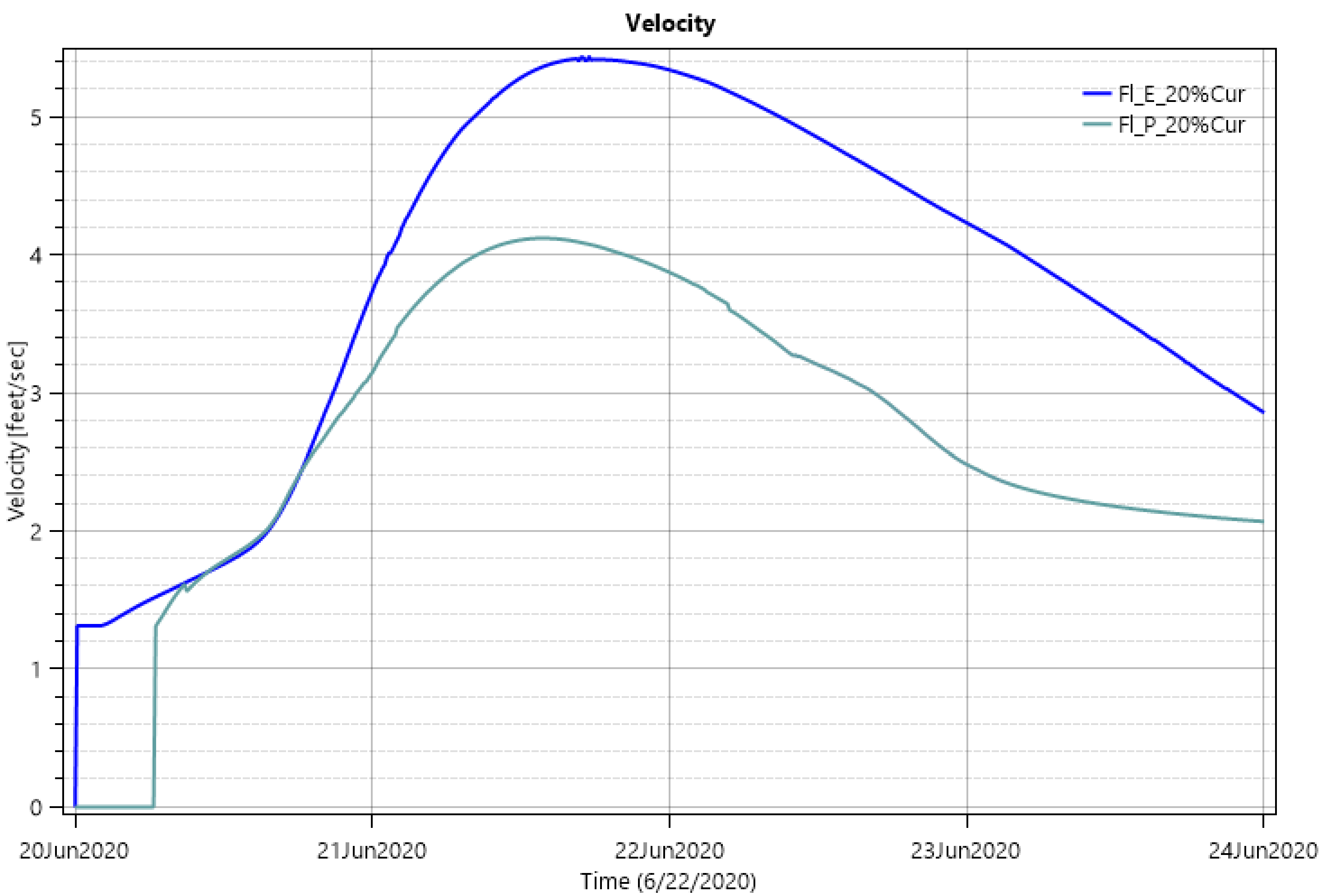


Figure 2.6.1-7: Upstream of Myrtle Street Water Surface Elevations – (Existing vs Proposed Conditions) – 1% Recurrence Interval

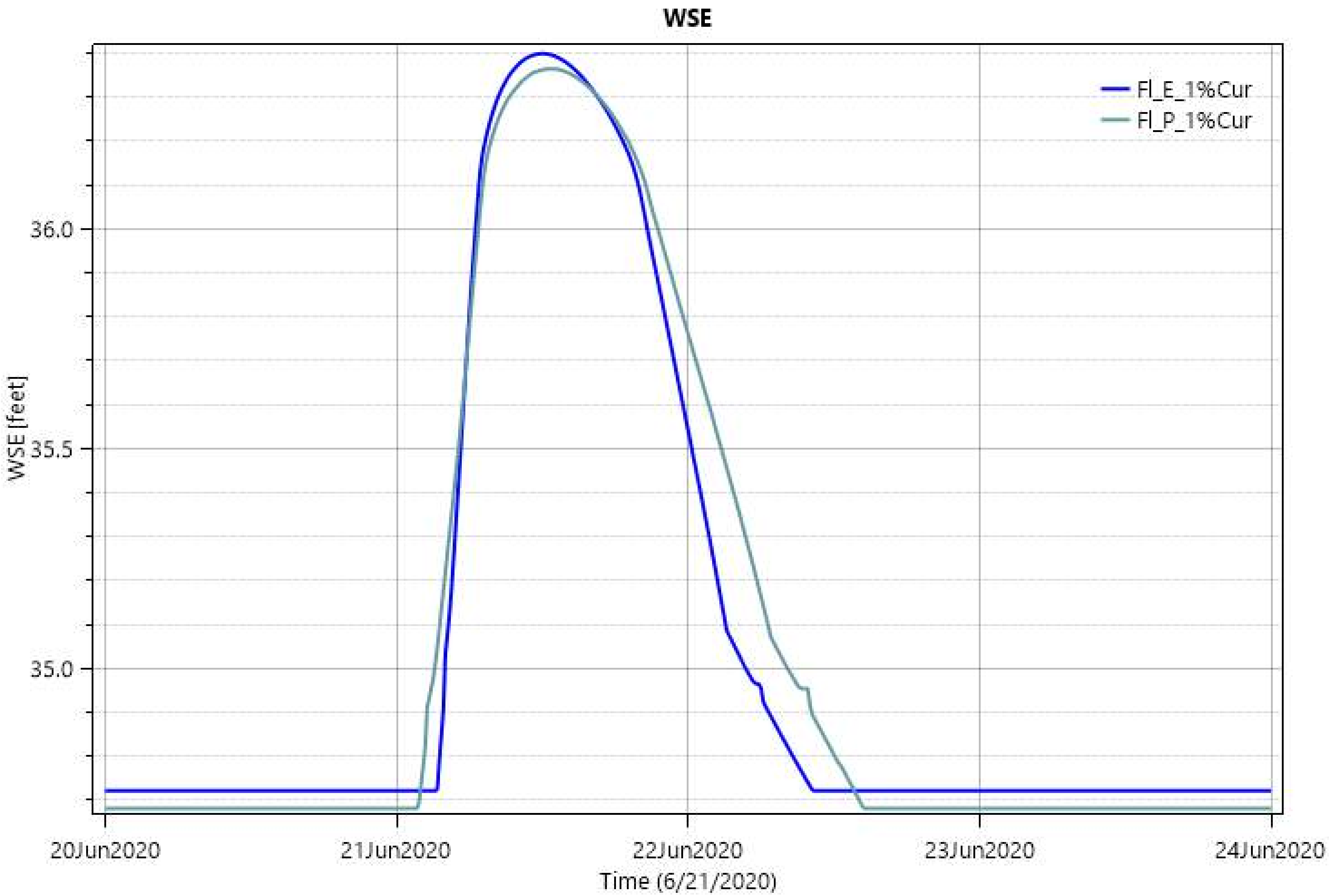


Figure 2.6.1-8: Upstream of Myrtle Street Water Velocities – (Existing vs Proposed Conditions) – 1% Recurrence Interval

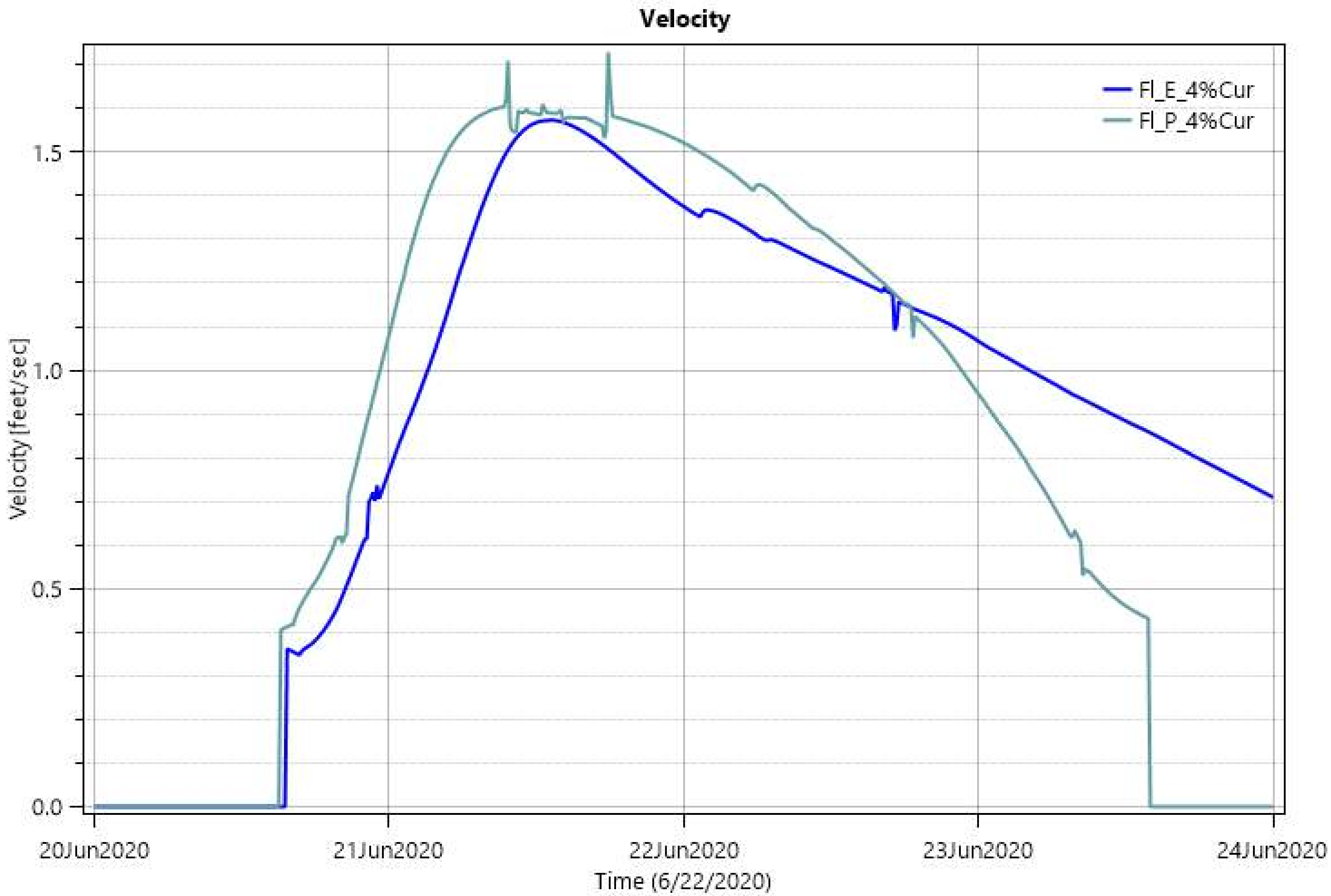


Figure 2.6.1-9: 10 Feet Upstream of Myrtle Street Water Surface Elevations – (Existing vs Proposed Conditions) – 4% Recurrence Interval

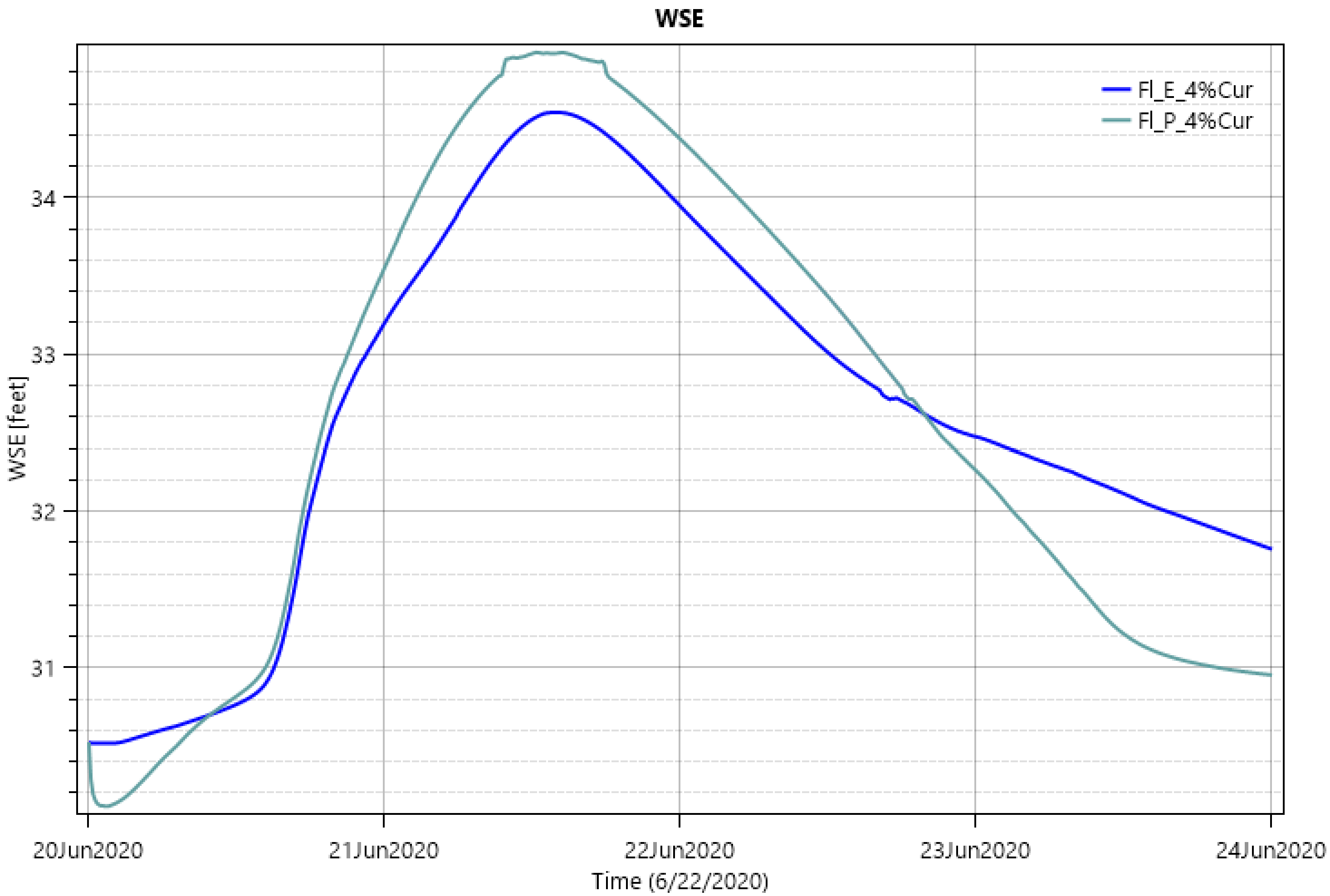


Figure 2.6.1-10: 10 Feet Upstream of Myrtle Street Water Velocities – (Existing vs Proposed Conditions) – 4% Recurrence Interval

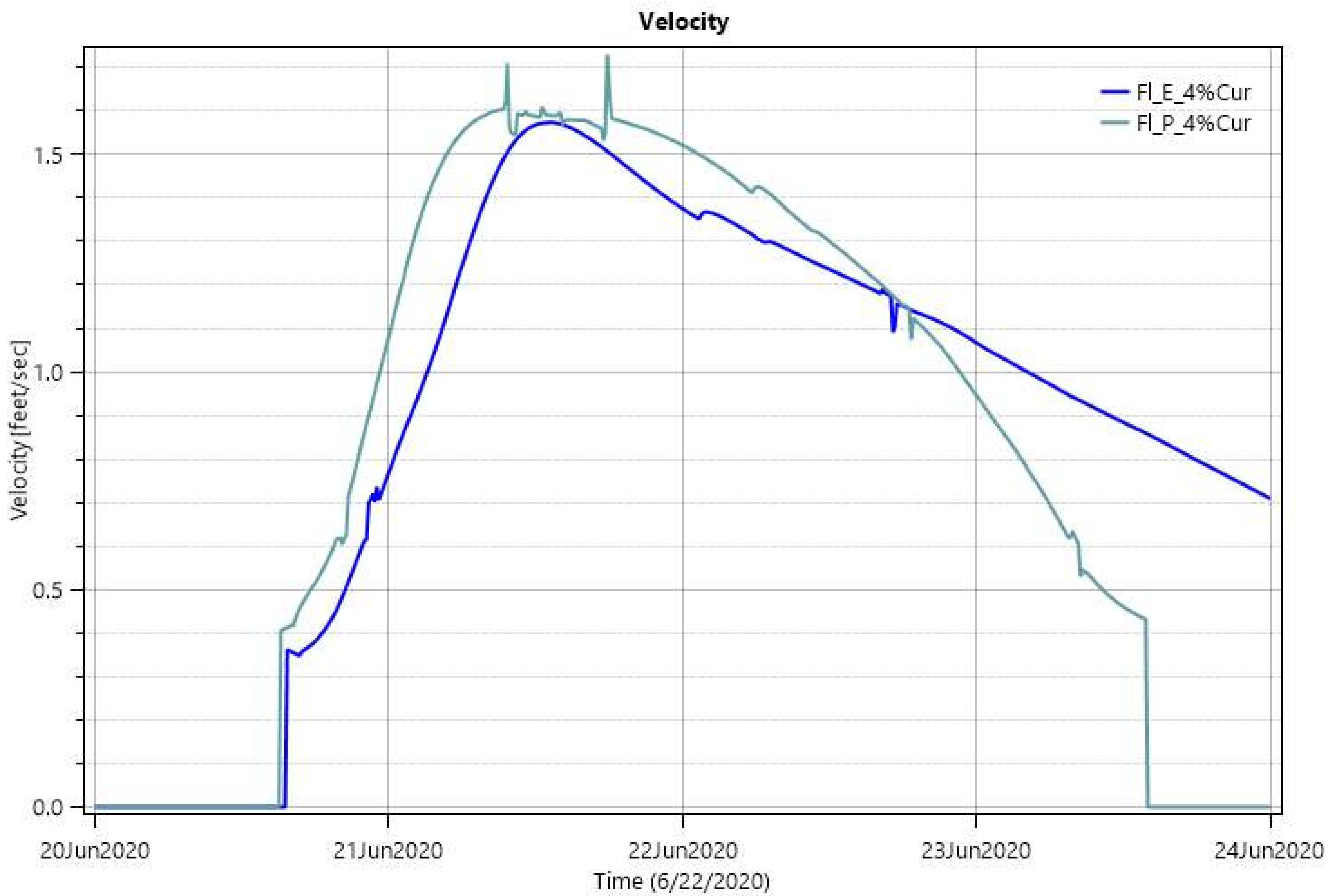


Figure 2.6.1-11: 10 Feet Upstream of Myrtle Street Water Surface Elevations – (Existing vs Proposed Conditions) – 20% Recurrence Interval

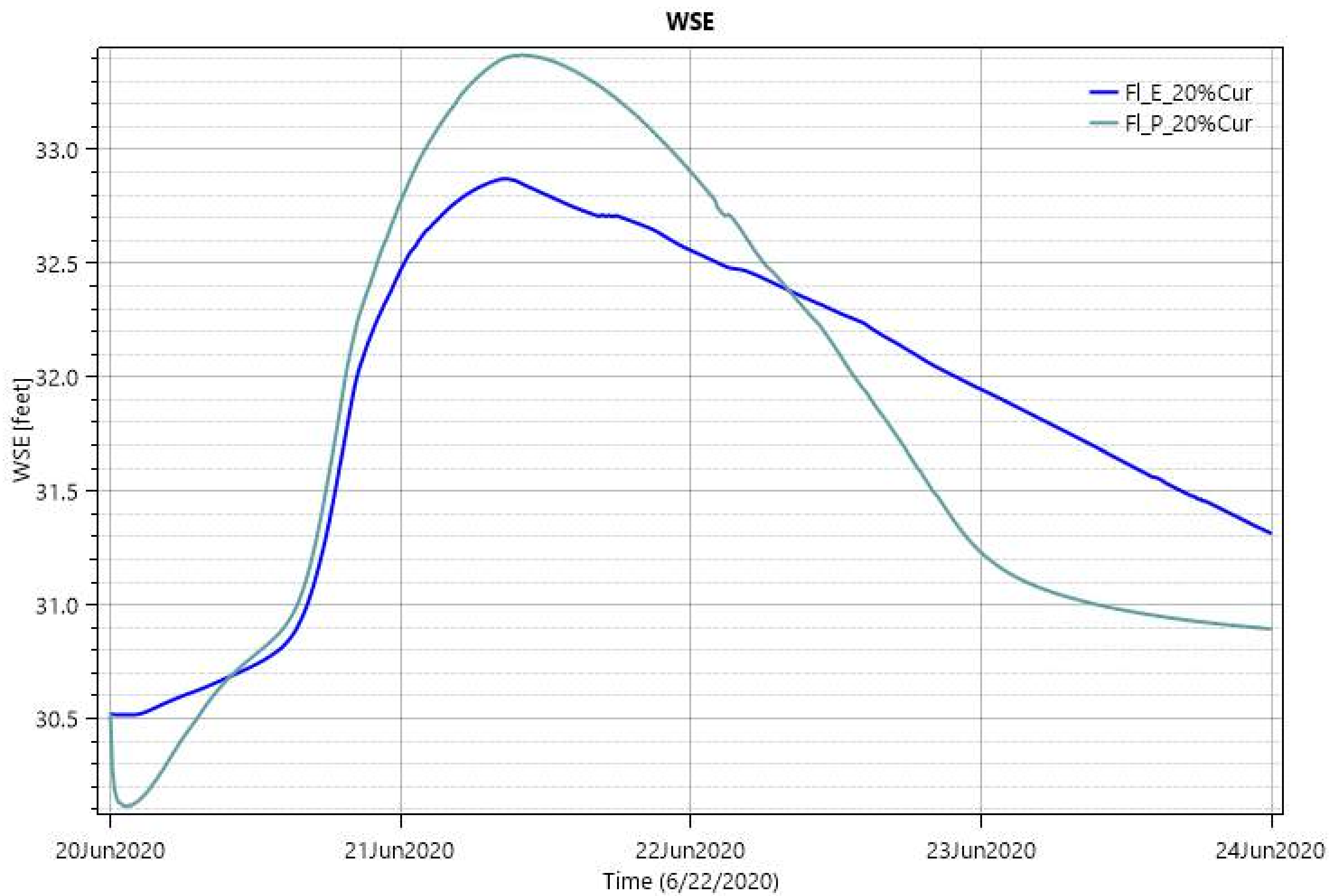


Figure 2.6.1-12: 10 Feet Upstream of Myrtle Street Water Velocities – (Existing vs Proposed Conditions) – 20% Recurrence Interval

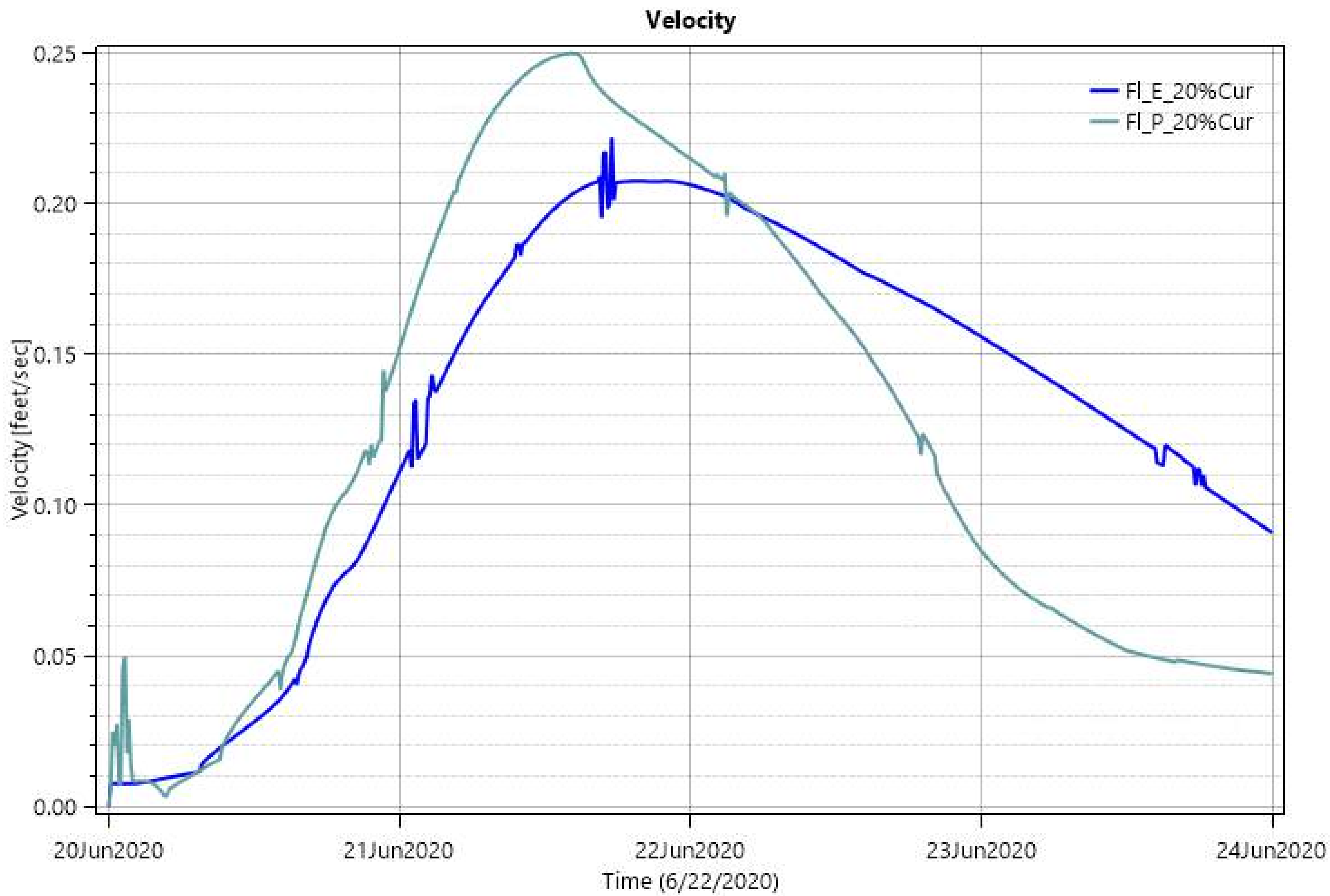


Figure 2.6.1-13– Water Surface Extents – 20% Recurrence Interval - Existing (Blue) VS. Proposed (Green) Conditions

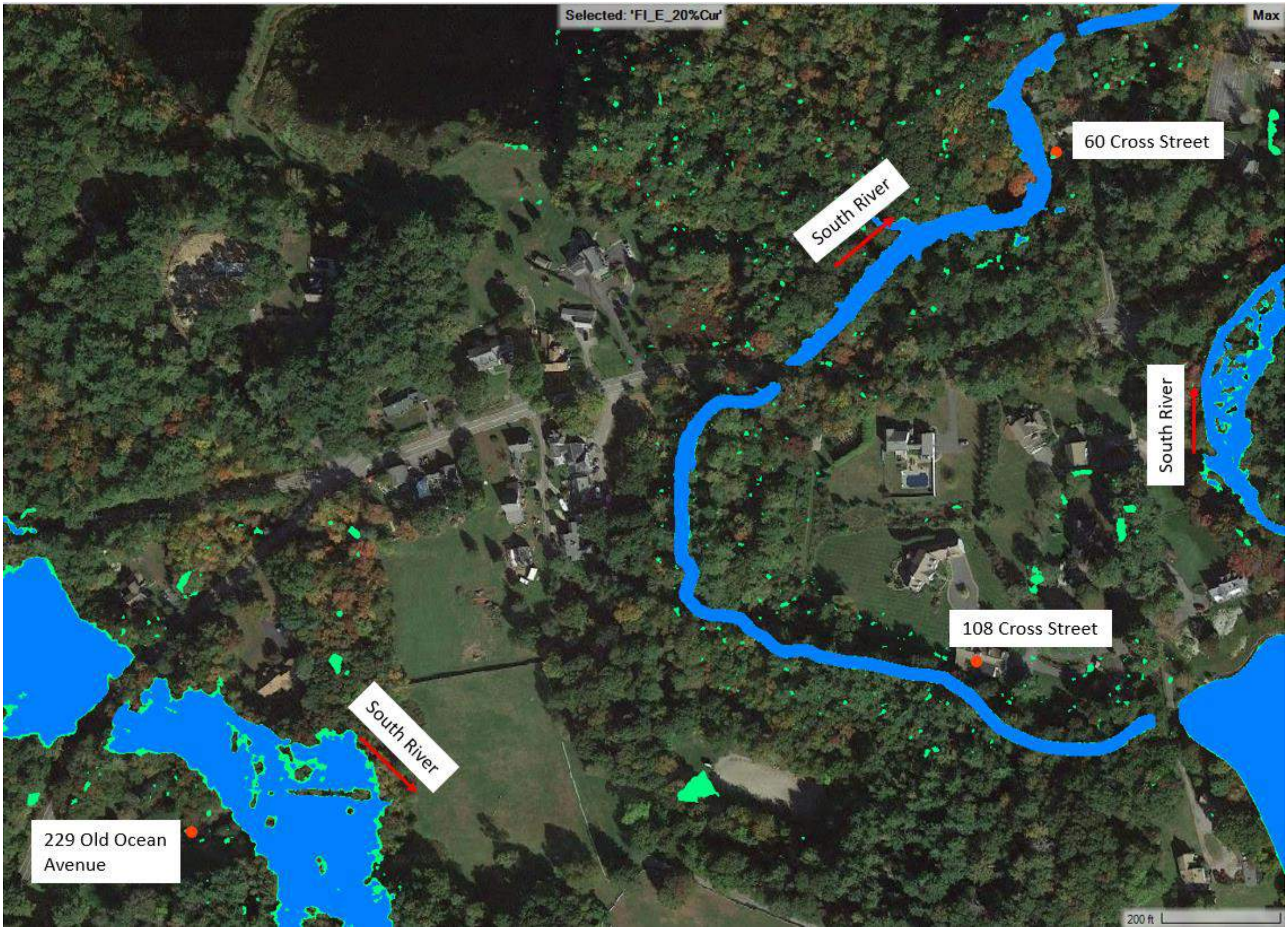


Figure 2.6.1-14 – Water Surface Extents – 4% Recurrence Interval - Existing (Blue) VS. Proposed (Green) Conditions

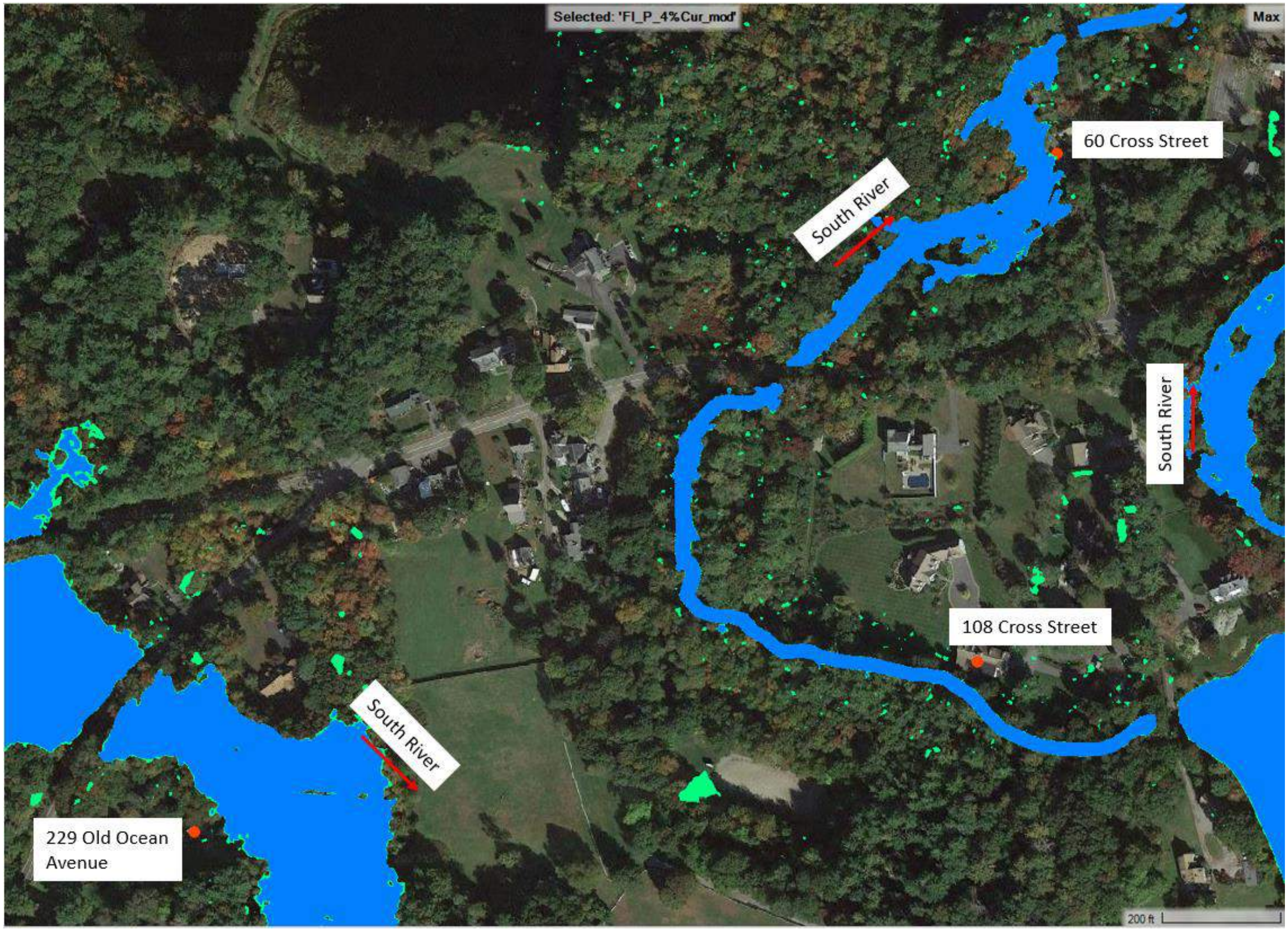
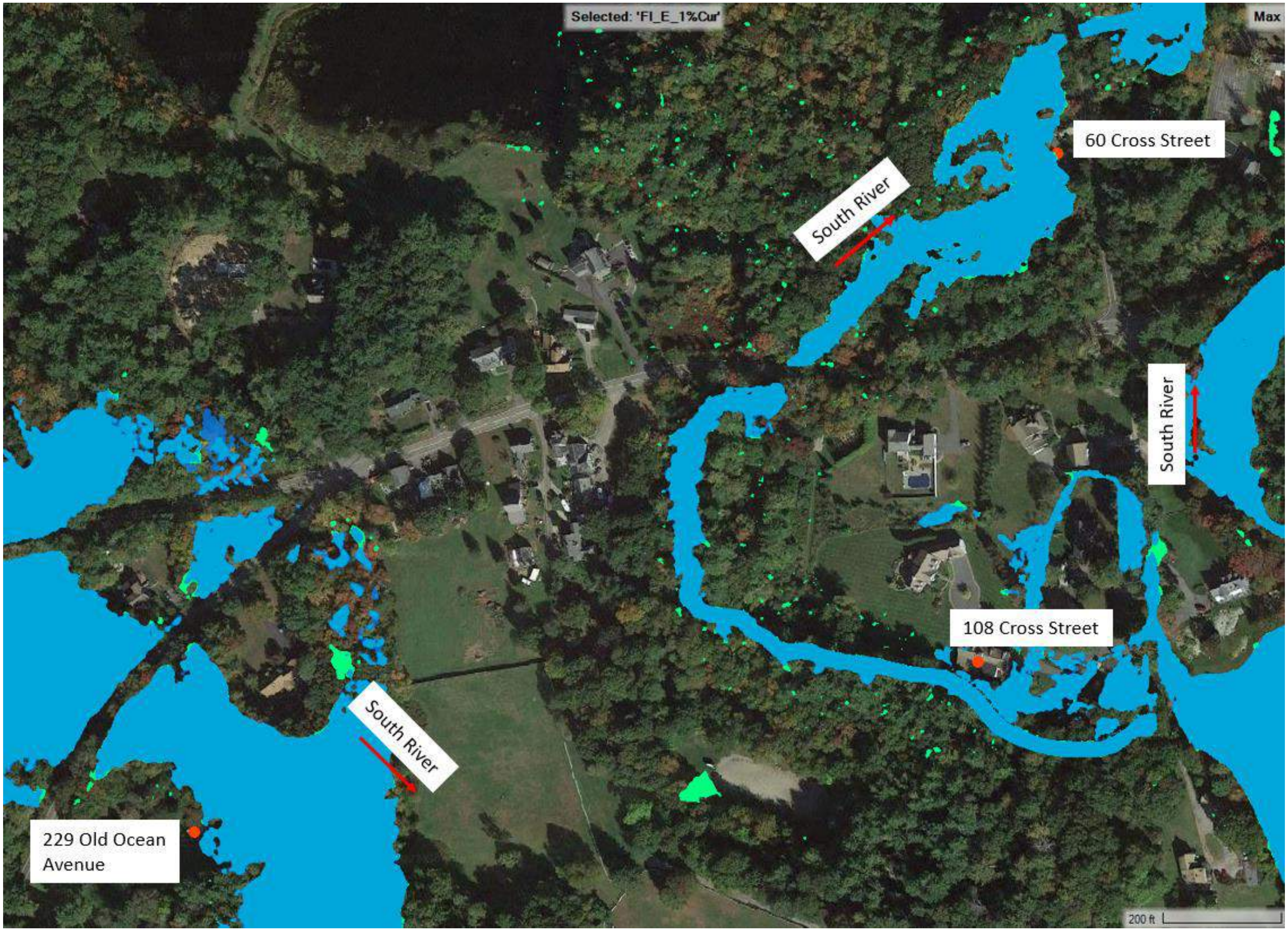


Figure 2.6.1-15 – Water Surface Extents – 1% Recurrence Interval - Existing (Blue) VS. Proposed (Green) Conditions



Appendix F – Sediment Sampling Results Summary

Standard Analyses for Dam Removal Projects (adjust if due dilligence suggests additional pollutant risks)		Ecological Thresholds (aquatic)				Human Exposure Thresholds (upland/floodplain)			Method 1 Soil Standards (MCP)	Dam Impoundment Samples			Downstream Samples Results	Upstream Samples	Summary Calculations				
Parameters	**Units**	Freshwater		Marine		Direct Contact Method 2 (S-1)	Direct Contact Method 2 (S-2)	Direct Contact Method 2 (S-3)	*(for comparison)* S-1 / GW-1	IMP-1	IMP-2	IMP-3	DS-1	US-1	Impoundment			Downstream	Upstream
		TEC/TEL	PEC/PEL	TEL	PEL										Min	Max	Mean	Mean	Mean
Metals, Total [mg/kg or ppm]																			
Arsenic (ppm)	mg/kg (ppm)	9.79	33.00	7.24	41.60	20	20	50	20	11.0	8.5	14.0	2.3	7.3	8.5	14	11.17	2.3	7.3
Cadmium (ppm)	mg/kg (ppm)	0.99	4.98	0.68	4.20	70	100	100	70	2.0	1.5	1.3	0.3	0.9	1.3	1.95	1.57	0.32	0.87
Chromium (TOTAL) (ppm)	mg/kg (ppm)	43.40	111.00	52.30	160.00	100	200	200	100	43.3	44.5	44.3	12.5	41.6	43.3	44.5	44.03	12.5	41.6
Chromium III (ppm)	mg/kg (ppm)					1,000	3,000	5,000	1,000	9.8	10.0	9.3	3.0	8.1	9.3	10	9.70	3	8.1
Chromium VI (Hexavalent) (ppm)	mg/kg (ppm)					100	200	200	100	33.5	34.5	35.0	9.5	33.5	33.5	35.0	34.3	9.5	33.5
Copper (ppm)	mg/kg (ppm)	31.60	149.00	18.70	108.00					21.0	17.0	28.0	6.3	17.0	17	28.0	22.0	6.3	17.0
Lead (ppm)	mg/kg (ppm)	35.60	128.00	30.24	112.00	200	600	600	200	43.0	40.0	60.0	11.0	37.0	40	60	47.67	11	37
Mercury (ppm)	mg/kg (ppm)	0.18	1.06	0.13	0.70	20	30	30	20	0.2	0.1	0.3	0.0	0.2	0.11	0.27	0.18	0.025	0.16
Nickel (ppm)	mg/kg (ppm)	22.70	48.60	15.90	42.80	600	1,000	1,000	600	10.0	7.8	13.0	15.0	8.0	7.8	13	10.27	15	8
Zinc (ppm)	mg/kg (ppm)	121	459	124.00	271.00	1,000	3,000	5,000	1,000	91.0	60.0	130.0	39.0	65.0	60	130	93.67	39	65
PAHs (ug/kg or ppb)																			
Acenaphthene	ug/kg (ppb)	7	89	7	89	1,000,000	3,000,000	5,000,000	4,000	430.0	445.0	450.0	125.0	430.0	430	450	441.67	125	430
Acenaphthylene	ug/kg (ppb)	6	128	6	128	1,000,000	3,000,000	5,000,000	1,000	430.0	445.0	450.0	125.0	430.0	430	450	441.67	125	430
Anthracene	ug/kg (ppb)	57	845	47	245	1,000,000	3,000,000	5,000,000	1,000,000	430.0	445.0	450.0	72.0	430.0	430	450	441.67	72	430
Benz[a]anthracene	ug/kg (ppb)	108	1,050	75	693	7,000	40,000	300,000	7,000	1300.0	940.0	1100.0	1500.0	1500.0	940	1300	1113.33	1500	1500
Benzo[a]pyrene	ug/kg (ppb)	150	1,450	89	763	2,000	7,000	30,000	2,000	17000.0	10000.0	8800.0	2900.0	12000.0	8800	17000	11933.33	2900	12000
Benzo[b]fluoranthene	ug/kg (ppb)					70,000	400,000	3,000,000	7,000	390.0	445.0	450.0	340.0	430.0	390	450	428.33	340	430
Benzo[g,h,i]perylene	ug/kg (ppb)					1,000,000	3,000,000	5,000,000	1,000,000	1800.0	1900.0	1500.0	410.0	2000.0	1500	1900	1733.33	410	2000
Benzo[k]fluoranthene	ug/kg (ppb)					70,000	400,000	3,000,000	70,000	600.0	380.0	420.0	200.0	730.0	380	600	466.67	200	730
Chrysene	ug/kg (ppb)	166	1,290	108	846	70,000	400,000	3,000,000	70,000	810.0	680.0	670.0	790.0	280.0	670	810	720.00	790	280
Dibenz[a,h]anthracene	ug/kg (ppb)	33	135	6	135	700	4,000	30,000	700	600.0	550.0	450.0	180.0	660.0	450	600	533.33	180	660
Fluoranthene	ug/kg (ppb)	423	2,230	113	1,494	1,000,000	3,000,000	5,000,000	1,000,000	600.0	445.0	450.0	740.0	430.0	445	600	498.33	740	430
Fluorene	ug/kg (ppb)	77	536	21	144	1,000,000	3,000,000	5,000,000	1,000,000	600.0	445.0	450.0	125.0	430.0	445	600	498.33	125	430
Indeno[1,2,3-cd]pyrene	ug/kg (ppb)					7,000	40,000	300,000	7,000	3400.0	2600.0	2300.0	240.0	2000.0	2300	3400	2766.67	240	2000
Naphthalene	ug/kg (ppb)	176	561	35	391	500,000	1,000,000	3,000,000	4,000	600.0	445.0	450.0	125.0	430.0	445	600	498.33	125	430
Phenanthrene	ug/kg (ppb)	204	1,170	87	544	500,000	1,000,000	3,000,000	10,000	600.0	445.0	450.0	260.0	430.0	445	600	498.33	260.0	430
Pyrene	ug/kg (ppb)	195	1,520	153	1,398	500,000	1,000,000	3,000,000	1,000,000	600.0	445.0	450.0	680.0	430.0	445	600	498.33	680.0	430
Total PAHs (ppb)	ug/kg (ppb)	1,610	22,800	1,684	16,770					30190.0	21055.0	19290.0	8812.0	23040.0	19290	30190	23511.67	8812	23040
PCBs (mg/kg or ppm)																			
Total PCBs (ppm)	mg/kg (ppm)	0.06	0.68	0.02	0.18	1	4	4	1	0.005	0.014	0.009	0.002	0.009	0.0054	0.014	0.0094	0.0016	0.009
Pesticides (ug/kg)																			
Sum DDD (ppb)	ug/kg (ppb)	4.88	28.00	1.22	7.81	8,000	40,000	60,000	8,000	100.0	110.0	480.0	38.0	93.0	100	480	230	38	93
Sum DDE (ppb)	ug/kg (ppb)	3.16	31.30	2.07	374.00	6,000	30,000	60,000	6,000	245.0	67.0	220.0	19.0	45.0	67	245	177	19	45
Sum DDT (ppb)	ug/kg (ppb)	4.16	62.90	1.19	4.77	6,000	30,000	60,000	6,000	290.0	180.0	180.0	15.0	110.0	180	290	217	15	110
Total DDTs (ppb)	ug/kg (ppb)	5.28	572.00	3.89	51.70				20,000	635.0	357.0	880.0	72.0	248.0	357	880	624	72	248
Chlordane (ppb)	ug/kg (ppb)	3.24	17.60	2.26	4.79	5,000	30,000	60,000	5,000	1250.0	900.0	90.0	245.0	340.0	90	1250	747	245	340
Dieldrin (ppb)	ug/kg (ppb)	1.90	61.80	0.72	4.30	80	500	3,000	80	245.0	180.0	180.0	49.0	70.0	180	245	202	49	70
Endrin (ppb)	ug/kg (ppb)	2.22	207.00			10,000	20,000	20,000	10,000	490.0	355.0	360.0	100.0	135.0	355	490	402	100	135
gamma-BHC (Lindane) (ppb)	ug/kg (ppb)	2.37	4.99	0.32	0.99					125.0	90.0	90.0	24.5	34.0	90	125	102	24.5	34
Heptachlor epoxide (ppb)	ug/kg (ppb)	2.47	16.00		2.74	100	900	1,000	100	305.0	225.0	225.0	60.0	85.0	225	305	252	60	85
EPH (mg/kg or ppm)																			
C9-C18 Aliphatic Hydrocarbons (ppm)	mg/kg (ppm)					1,000	3,000	5,000	1,000	60	44.5	45	12.5	43	44.5	60	50	12.5	43
C19-C36 Aliphatic Hydrocarbons (ppm)	mg/kg (ppm)					3,000	5,000	5,000	3,000	220	160	150	44	130	150	220	176.67	44	130
C11-C22 Aromatic Hydrocarbons (ppm)	mg/kg (ppm)					1,000	3,000	5,000	1,000	270	170	140	54	160	140	270	193.33	54	160
Physical Characteristics																			
Total Organic Carbon (%)	%									13.0	13.0	12.0	2.3	9.8	12.0	13.0	12.7	2.3	9.8
Percent Water (%)	%																		
Sieve No. 4 (% passing)	% passing									99.6	98.4	100.0	95.0	99.9	98.4	100.0	99.3	95.0	99.9
Sieve No. 10 (% passing)	% passing									99.3	97.7	99.8	84.4	99.9	97.7	99.8	98.9	84.4	99.9
Sieve No. 40 (% passing)	% passing									66.9	70.1	82.8	51.1	80.0	66.9	82.8	73.3	51.1	80.0
Sieve No. 60 (% passing)	% passing									58.6	62.9	75.4	33.6	72.9	58.6	75.4	65.6	33.6	72.9
Sieve No. 200 (% passing)	% passing									40.0	44.4	48.3	8.7	51.2	40.0	48.3	44.2	8.7	51.2

Standard Analyses for Dam Removal Projects (adjust if due dilligence suggests additional pollutant risks)		Ecological Thresholds (aquatic)				Human Exposure Thresholds (upland/floodplain)			Method 1 Soil Standards (MCP)	Dam Impoundment Samples			Downstream Samples Results	Upstream Samples	Summary Calculations				
Parameters	Units	Freshwater		Marine		Direct Contact	Direct Contact	Direct Contact	*(for comparison)*	IMP-1	IMP-2	IMP-3	DS-1	US-1	Impoundment			Downstream	Upstream
		TEC/TEL	PEC/PEL	TEL	PEL	Method 2 (S-1)	Method 2 (S-2)	Method 2 (S-3)	S-1 / GW-1						Min	Max	Mean	Mean	Mean
Volatile Organic Compounds (VOCs)																			
Acetone (ppm)	ug/kg (ppb)					500	1,000	3,000	6.0	380.0	420.0	190.0	32.0	415.0	190	420	330	32.0	415.0
Amyl Methyl Ether (TAME)	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Benzene	ug/kg (ppb)					40	200	1,000	2.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Bromobenzene	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Bromochloromethane	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Bromodichloromethane	ug/kg (ppb)					30	100	500	0.1	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Bromoform	ug/kg (ppb)					300	1,000	3,000	0.1	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Bromomethane	ug/kg (ppb)					90	600	600	0.5	60.0	44.5	40.5	9.0	41.5	40.5	60.0	48.3	9.0	41.5
Butylbenzene, sec-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Butylbenzene, n-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Butylbenzene, tert-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Carbon Disulfide	ug/kg (ppb)									60.0	44.5	40.5	9.0	41.5	40.5	60.0	48.3	9.0	41.5
Carbon Tetrachloride	ug/kg (ppb)					30	100	1000	10.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Chlorobenzene	ug/kg (ppb)					500	1000	3000	1.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Chlorodibromomethane	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Chloroethane	ug/kg (ppb)									115.0	90.0	80.0	18.5	85.0	80.0	115.0	95.0	18.5	85.0
Chloroform	ug/kg (ppb)					500	1000	1000	0.4	23.0	18.0	16.0	3.7	16.5	16.0	23.0	19.0	3.7	16.5
Chloromethane	ug/kg (ppb)									60.0	44.5	40.5	9.0	41.5	40.5	60.0	48.3	9.0	41.5
Chlorotoluene, 2-	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Chlorotoluene, 4-	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
1,2-Dibromo-3-chloropropane PP	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dibromoethane, 1,2- (EDB)	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Dibromomethane	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichlorobenzene, 1,3- (m-DCB)	ug/kg (ppb)					100	500	500	3.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichlorobenzene, 1,2- (o-DCB)	ug/kg (ppb)					1000	3000	5000	9.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichlorobenzene, 1,4- (p-DCB)	ug/kg (ppb)					80	400	3000	0.7	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichlorodifluoromethane (Freon 12)	ug/kg (ppb)									115.0	90.0	80.0	18.5	85.0	80.0	115.0	95.0	18.5	85.0
Dichloroethane, 1,1-	ug/kg (ppb)					500	1000	3000	0.4	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloroethane, 1,2-	ug/kg (ppb)					20	100	900	0.1	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloroethylene, 1,1-	ug/kg (ppb)					500	1000	3000	3.0	23.0	18.0	16.0	3.7	16.5	16.0	23.0	19.0	3.7	16.5
Dichloroethylene, cis-1,2	ug/kg (ppb)					100	500	500	0.3	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloroethylene, trans-1,2	ug/kg (ppb)					500	1000	3000	1.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloropropane, 1,2-	ug/kg (ppb)					30	100	1000	0.1	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloropropane, 1,3-	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Dichloropropane, 2,2-	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloropropene, 1,1-	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Dichloropropene, cis-1,3-3	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Dichloropropene, trans-1,3- 3	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Diethyl Ether OXY	ug/kg (ppb)									115.0	90.0	80.0	18.5	85.0	80.0	115.0	95.0	18.5	85.0
Diisopropyl Ether (DIPE) OXY	ug/kg (ppb)									6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Dioxane, 1,4- PP, 1	ug/kg (ppb)					20	90	500	0.2	600.0	445.0	405.0	90.0	415.0	405.0	600.0	483.3	90.0	415.0

Standard Analyses for Dam Removal Projects (adjust if due dilligence suggests additional pollutant risks)		Ecological Thresholds (aquatic)				Human Exposure Thresholds (upland/floodplain)			Method 1 Soil Standards (MCP)	Dam Impoundment Samples			Downstream Samples Results	Upstream Samples	Summary Calculations				
Parameters	Units	Freshwater		Marine		Direct Contact	Direct Contact	Direct Contact	*(for comparison)*	IMP-1	IMP-2	IMP-3	DS-1	US-1	Impoundment			Downstream	Upstream
		TEC/TEL	PEC/PEL	TEL	PEL	Method 2 (S-1)	Method 2 (S-2)	Method 2 (S-3)	S-1 / GW-1						Min	Max	Mean	Mean	Mean
Metals, Total [mg/kg or ppm]																			
Ethyl Tertiary Butyl Ether (ETBE) OXY	ug/kg (ppb)									6.0	0.0	4.1	0.9	4.2	0.0	6.0	3.4	0.9	4.2
Ethylbenzene	ug/kg (ppb)					500	1000	3000	40.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Hexachlorobutadiene	ug/kg (ppb)					30	100	100	30.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Hexanone (MNBK), 2- PP	ug/kg (ppb)									115.0	90.0	80.0	18.5	85.0	80.0	115.0	95.0	18.5	85.0
Isopropylbenzene (Cumene)2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Isopropyltoluene, p-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Methyl Ethyl Ketone (MEK) PP	ug/kg (ppb)					500	1000	3000	4.0	230.0	180.0	160.0	1.9	165.0	160.0	230.0	190.0	1.9	165.0
Methyl Isobutyl Ketone (MIBK) PP	ug/kg (ppb)					500	1000	3000	0.4	115.0	90.0	8.0	18.5	85.0	8.0	115.0	71.0	18.5	85.0
Methyl Tertiary Butyl Ether (MTBE) OXY	ug/kg (ppb)					100	500	500	0.1	23.0	18.0	16.0	3.7	16.5	16.0	23.0	19.0	3.7	16.5
Methylene Chloride	ug/kg (ppb)									115.0	90.0	80.0	18.5	85.0	80.0	115.0	95.0	18.5	85.0
Naphthalene	ug/kg (ppb)					500	1000	3000	4.0	23.0	18.0	16.0	3.7	16.5	16.0	23.0	19.0	3.7	16.5
Propylbenzene, n-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Styrene	ug/kg (ppb)					70	300	3000	3.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Tetrachloroethane, 1,1,1,2-	ug/kg (ppb)					80	400	500	0.1	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Tetrachloroethane, 1,1,2,2-	ug/kg (ppb)					10	50	400	0.005	6.0	4.5	4.1	0.9	4.2	4.1	6.0	4.8	0.9	4.2
Tetrachloroethylene	ug/kg (ppb)					30	200	1000	1.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Tetrahydrofuran (THF)	ug/kg (ppb)									60.0	44.5	40.5	9.0	41.5	40.5	60.0	48.3	9.0	41.5
Toluene	ug/kg (ppb)					500	1000	3000	30.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trichlorobenzene, 1,2,4-	ug/kg (ppb)								2.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trichlorobenzene, 1,2,3-	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trichloroethane, 1,1,1-	ug/kg (ppb)					500	1000	3000	30.0	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trichloroethane, 1,1,2-	ug/kg (ppb)					40	200	500	0.1	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trichloroethylene (TCE)	ug/kg (ppb)					30	60	60	0.3	11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trichlorofluoromethane (Freon 11)	ug/kg (ppb)									60.0	44.5	40.5	9.0	41.5	40.5	60.0	48.3	9.0	41.5
Trichloropropane, 1,2,3-	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trimethylbenzene, 1,2,4-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Trimethylbenzene, 1,3,5-2	ug/kg (ppb)									11.5	9.0	8.0	1.9	8.5	8.0	11.5	9.5	1.9	8.5
Vinyl Chloride	ug/kg (ppb)					1	7	60	0.9	60.0	44.5	40.5	9.0	41.5	40.5	60.0	48.3	9.0	41.5
Xylenes	ug/kg (ppb)					500	1000	3000	400.0	23.0	18.0	16.0	3.7	16.5	16.0	23.0	19.0	3.7	16.5

Appendix G – Riffle Feature Bed Design Calcs

Abt and Johnson Riprap Sizing

Formulas for Riprap Sizing (Mooney 2007) - Bankfull

	Bankfull, Round, River St		
	Qdesign	119	cfs
	Channel Width	12	ft*
	qdesign	9.9	cfs/ft
	qsizing	13.4	cfs/ft
iiven as 1.2	ϕ_e	1.2	
Ratio of un	r_q	1	
	ϕ_c	1.000	
Fractional I	r_s	0.45	
	a	1.40	
Slope	S_0	0.02300	
	D_{50}	**7.41**	**inches** say
	D_{50}	**0.62**	**ft**

Bankfull, Round, Dam		
Qdesign	119	cfs
Channel W	8	ft*
qdesign	14.9	cfs/ft
qsizing	20.1	cfs/ft
ϕ_e	1.2	
r_q	1	
ϕ_c	1.000	
r_s	0.45	
a	1.40	
S_0	0.01500	
D_{50}	**7.73**	**inches**
D_{50}	**0.64**	**ft**

Abt. Johnson Formulas for Riprap Sizing (Mooney 2007) - 100 Year Projected

100yr (Projected), Round, River Street		
Qdesign	338	cfs
Channel Width	12	ft*
qdesign	28.2	cfs/ft
qsizing	38.0	cfs/ft
ϕ_e	1.2	
r_q	1	
ϕ_c	1.000	
r_s	0.45	
a	1.40	
S_0	0.02300	
D_{50}	**13.29**	**inches** say
D_{50}	**1.11**	**ft**

100yr (Projected), Round, Dam		
Qdesign	346	cfs
Channel Width	8	ft*
qdesign	43.3	cfs/ft
qsizing	58.4	cfs/ft
ϕ_e	1.2	
r_q	1	
ϕ_c	1.000	
r_s	0.45	
a	1.40	
S_0	0.01500	
D_{50}	**14.06**	**inches**
D_{50}	**1.17**	**ft**

Abt. Johnson Formulas for Riprap Sizing (Mooney 2007) - 100 Year

100yr (Projected), Round, River St		
Qdesign	316	cfs
Channel Width	12	ft*
qdesign	26.3	cfs/ft
qsizing	35.6	cfs/ft
ϕ_e	1.2	
r_q	1	
ϕ_c	1.000	
r_s	0.45	
a	1.40	
S_0	0.02300	
D_{50}	**12.80**	**inches** say
D_{50}	**1.07**	**ft**

100yr (Projected), Round, Dam		
Qdesign	312	cfs
Channel W	8	ft*
qdesign	39.0	cfs/ft
qsizing	52.7	cfs/ft
ϕ_e	1.2	
r_q	1	
ϕ_c	1.000	
r_s	0.45	
a	1.40	
S_0	0.01500	
D_{50}	**13.27**	**inches**
D_{50}	**1.11**	**ft**

Qdesign	312	cfs	
Channel W	8	ft*	
qdesign	77.5	cfs/ft	
qsizing	104.6	cfs/ft	
ϕ_e	1.2		
r_q	1		
ϕ_c	1.200		
r_s	0		
a	1.00		
S_0	0.04000		
D_{50}	**25.51**	**inches**	**CHECK GUI OK**
D_{50}	**2.13**	**ft**	

Summary

	Bankfull, River St		Bankfull, Dam		100yr, River Street		100yr, Dam		100yr, River St		100yr, Dam			
Cross Sectional Average														
Q	119	cfs	119	cfs	338	cfs	346	cfs	316	cfs	312	cfs	312	cfs
w	6	ft	6	ft	12	ft	8	ft	12	ft	8	ft	8	ft
q	19.83333	cfs/ft	19.83333	cfs/ft	28.16667	cfs/ft	43.25	cfs/ft	26.33333	cfs/ft	39	cfs/ft	39	cfs/ft
Low Flow Notch														
Q	119	cfs	119	cfs	338	cfs	346	cfs	316	cfs	312	cfs	312	cfs
w	6	ft	6	ft	12	ft	8	ft	12	ft	8	ft	8	ft
q	19.83333	cfs/ft	19.83333	cfs/ft	28.16667	cfs/ft	43.25	cfs/ft	26.33333	cfs/ft	39	cfs/ft	39	cfs/ft
Ratio	1	= rq in forn	1	= rq in formula above	1	= rq in formula above	1	= rq in formula above	1	= rq in formula above	1	= rq in formula above	1	= rq in formula above

Equation E.2 $q = Q/w$

where:

q is the unit discharge (cfs/ft or ft2/s; cms or m2/s)

Q is discharge (cfs or cms)

w is the active channel width for bedload transport (ft or m) at a given cross section.

E—3

Stream Simulation

Although there are no existing guidelines for defining the active-channel width for bedload transport, we suggest using the bed width between the lower banks to represent active-channel width because it is typically the zone of active bedload transport. The unit discharge should be determined for the portion of the total flow that occurs over the active channel bed (figure E.2). Calculating unit discharge using the total discharge instead of the portion of discharge occurring over the active-channel width would overestimate the flow actually exerting force on the active-channel bed. This overestimation of unit discharge would be magnified when floodwaters inundate a wide flood plain.

ARS Rock Chutes Riprap Sizing

Q100 Projected

D50	**6.235539**	**in**
q	43.25	cfs/ft
S	0.023	ft/ft

$$\left(q_{design}\right) = \frac{\left(q_{failure}\right)}{0.74} = 1.35q_{failure} \qquad \text{(eq. TS14C–15)}$$

Stone movement occurred at approximately 74 percent of the flow, causing layer failure. It was determined from testing that rounded stone should be oversized by approximately 40 percent to provide the same protection as angular stone.

ARS rock chutes

This design technique (Robinson, Rice, and Kadavy 1998) is primarily targeted at high-energy applications. Loose riprap with a 2 D_{50} blanket thickness composed of relatively uniform, angular riprap was tested to overtopping failure in models and field scale structures. This method applies to bed slopes of 40 percent and less. This technique can be used for low slope, and thus, low-energy applications, but it is particularly useful for slopes greater than 2 percent. A factor of safety appropriate for the project should be applied to the predicted rock size. The equations are:

for $S < 0.1$

$$D_{50} = 12\left(1.923qS^{1.5}\right)^{0.529} \qquad \text{(eq. TS14C–16)}$$

$0.10 < S < 0.40$

$$D_{50} = 12\left(0.233qS^{0.58}\right)^{0.529} \qquad \text{(eq. TS14C–17)}$$

where:
D_{50} = median stone size (in)
q = highest stable unit discharge (ft^3/s/ft)
S = channel slope (ft/ft)

A spreadsheet program (Lorenz, Lobrecht, and Robinson 2000) is available to assist in sizing riprap on steep slopes. A screen capture of this spreadsheet program is shown in figure TS14C–7.

banks. Unlike most of the other available techniques, it results in a recommended minimum weight of the stone. The equation is:

$$W = \frac{0.00002}{\left(G_s - 1\right)^3} \times \frac{VM \times V^6 \times G_S}{\sin^3\left(r - a\right)} \qquad \text{(eq. TS14C–18)}$$

where:
W = minimum rock weight (lb)
V = velocity (ft/s)
VM = 0.67 if parallel flow
VM = 1.33 if impinging flow
G_S = specific gravity of rock (typically 2.65)
r = angle of repose (70° for randomly placed rock)
a = outside slope face angle to the horizontal (typically a maximum of 33°)

The weight indicated by this method should be used in conjunction with standard CALTRANS specifications and gradations.

Far West states (FWS)—Lane's Method

Vito A. Vanoni worked with the Northwest E&WP Unit to develop the procedure from the ASCE paper entitled "Design of Stable Alluvial Channels" (Lane 1955a). The equation is:

$$D_{75} = \frac{3.5}{C \times K} \times \gamma_w \times D \times S_f \qquad \text{(eq. TS14C–19)}$$

where:
D_{75} = stone size, (in)
C = correction for channel curvature
K = correction for side slope
S_f = channel friction slope (ft/ft)
d = depth of flow (ft)
γ_w = density of water

This is generally considered to be a conservative technique. It assumed that the stress on the sides of the channel were 1.4 times that of the bottom. This

Maynord Riprap Sizing

Q100 (Projected)

D30	0.43054	ft
D30	5.166476	in
FS	1.2	
Cs	0.375	
Cv	1	
Ct	1	
d	7.08	ft
GammaW	62.4	lb/ft3
GammaS	165	lb/ft3
Vavg	6.05	fps
V	7.26	fps
g	32.2	ft/s2
K1	0.69728	
Angle of Rock from Horizonta	26.5	degrees
Angle of repose	40	degrees

Gradation

D15	4.13318	in
D50	**0.495121**	**ft**
D50	**5.941447**	**in**
D85	8.266361	in
D100	11.88289	in

National Cooperative Highway Research Program Report 108

This method (Anderson, Paintal, and Davenport 1970) is suggested for design of roadside drainage channels handling less than 1,000 cubic foot per second and a maximum slope of 0.10 foot per foot. Therefore, this application can be used for high- or low-energy applications. Photo documentation shows that most of the research was done on rounded stones. This method will give more conservative results if angular rock is used.

$$\tau_o = \gamma R S_e \qquad \text{(eq. TS14C–2)}$$

$$\tau_c = 4D_{50} \qquad \text{(eq. TS14C–3)}$$

therefore,

$$D_{50} = \frac{\gamma R S_e}{4} \qquad \text{(eq. TS14C–4)}$$

τ_c = critical tractive stress
γ = 62.4 lb/ft^3
R = hydraulic radius (ft)
S_e = energy slope (ft/ft)
D_{50} = median stone diameter (ft)

A similar approach has been proposed by Newbury and Gaboury (1993) for sizing stones in grade control structures. This relationship is:

tractive force (kg/m^2) = incipient diameter (cm)

USACE—Maynord method

This low-energy technique for the design of riprap is used for channel bank protection (revetments). This method is outlined in USACE guidance as provided in EM 1110–2–1601, and is based on a modification to the Maynord equation:

$$D_{30} = FS \times C_s \times C_v \times C_T \times d \times \left[\left(\frac{\gamma_W}{\gamma_S - \gamma_w} \right)^{0.5} \times \frac{V}{\sqrt{K_1 \times g \times d}} \right]^{2.5} \qquad \text{(eq. TS14C–5)}$$

where:
D_m = stone size in ft; m percent finer by weight
d = water depth (ft)
FS = factor of safety (usually 1.1 to 1.5), suggest 1.2
C_s = stability coefficient Z=2 or flatter C=0.30, (0.3 for angular rock, 0.375 for rounded rock)
C_v = velocity distribution coefficient (1.0 for straight channels or inside of bends, calculate for outside of bends)
C_T = thickness coefficient (use 1.0 for 1 D_{100} or 1.5 D_{50}, whichever is greater))
γ_w = specific weight of water (lb/ft^3)
γ_s = specific weight of stone (lb/ft^3)
V = local velocity; if unknown use 1.5 $V_{average}$
g = 32.2 ft/s^2
K_1 = side slope correction as computed below

$$K_1 = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \qquad \text{(eq. TS14C–6)}$$

where:
θ = angle of rock from the horizontal
ϕ = angle of repose (typically 40°)

Note that the local velocity can be 120 to 150 percent of the average channel velocity or higher. The outside bend velocity coefficient and the side slope correction can be calculated:

$$C_v = 1.283 - 0.2\log\left(\frac{R}{W}\right) \qquad \text{(eq. TS14C–7)}$$

where:
R = centerline bend radius
W = water surface width

In the analysis used to develop this formula, failure was assumed to occur when the underlying material became exposed. It should be noted that while many of the other techniques specify a D_{50}, Maynord (1992) specifies a D_{30} which will typically be 15 percent smaller than the D_{50}. This assumes a specific gradation of:

$$1.8D_{15} < D_{85} < 4.6D_{15} \qquad \text{(eq. TS14C–8)}$$

The USACE developed this method for the design of riprap used in either constructed or natural channels which have a slope of 2 percent or less and Froude numbers less than 1.2. As a result, this technique is not appropriate for high-turbulence areas.

Maynord's side-slope and invert equation is for cases where the protective blanket is constructed with a relatively smooth surface and has no significant projections. It is appropriate for use to size stone-toe protection. However, it has been suggested that with some adjustment to the coefficients (typically using a velocity coefficient of 1.25 and a local velocity equal to 160% of the channel velocity), Maynord's method can

USACE Riprap Sizing

Q100 (Projected)

D30	1.080038	ft
S	0.023	ft/ft
q	54.0625	cfs/ft
FS	1.25	
g	32.2	ft/s2
D85/D15	4	
D30	1.080038	ft
D50	1.714453	ft

4.3.5 USACE (1991) Bed

For a riprap on beds with slopes ranging between 2 and 20%, the U.S. Army Corps of Engineers (EM1601, 1994) presents the following dimensionless relationship, Equation 4-8.

$$D_{30} = \frac{1.95 \cdot S^{0.555} \cdot q^{2/3}}{g^{1/3}} \qquad \textbf{Equation 4-8}$$

Where,

D_{30} = rock diameter for which 30% is smaller by mass;

S = slope of the rock ramp;

q = design unit discharge, USACE (EM1601, 1994) recommends increasing the input q by a 1.25 flow concentration factor; and

g = acceleration due to gravity.

The range of applicability requires a thickness equal to 1.5 *D_{100}, angular rock with a unit weight of 167 lbs/ft^3, D_{85}/D_{15} from 1.7 to 2.7, and side slopes flatter than 2.5:1 (H:V). The D_{50} is related to the D_{30} according to Equation 4-9.

$$D_{50} = D_{30} \cdot \left(\frac{D_{85}}{D_{15}}\right)^{1/3} \qquad \textbf{Equation 4-9}$$

The Corps recommends using a filter fabric below the structure and suggests considering grouted rock instead of loose stone.

Hydraulic Model Output - Proposed Projected 100-YR Event

Qbankfull	Dam	River Street	Units		Q100 (projected)	Dam	River Street	Units
Depth	3.21	3.48	ft		Depth	7.08	5.13	ft
Velocity	5.47	3.84	fps		Velocity	6.05	6.01	fps

HEC-11 Riprap Sizing

Parameter	Scenario		Source/ Justification
	K1 as scaled frin Chart 3	*Conservative K1 Selection*	
Initial riprap size estimate, D_{50} (ft)	1	0.33	Mass Highway available sizes
Manning's n for model	0.09	0.06	Chow (1959); US DOT (1984)
Average velocity, V_a (ft/s)	6.05	6.05	HEC-RAS model (100-Year)
Average depth, d_a (ft)	7.08	7.08	HEC-RAS model (100-Year)
Bank angle, θ	1:2	1:2	Design parameter
Materia; (Angular/Rounded)	Rounded	Rounded	Design parameter
Riprap angle of repose, ϕ	39	39	HEC-11 Chart 4
Bank angle correction, K_1	0.71	0.55	HEC-11 Chart 3 (Uses ϕ)
Riprap size, D_{50} (ft)	0.14	0.20	HEC-11 Chart 1
Stability factor, SF	1.6	1.6	HEC-11, pg. 31
Riprap specific gravity, S_s	2.65	2.65	HEC-11, standard
Riprap size correction factor, C	1.54	1.54	HEC-11 Chart 2
Corrected riprap size, D_{50} (ft)	0.21	0.31	HEC-11, pg. 31
say	**3"**	**4"**	Rounded up to standard size

Material Gradation

D_{100}	1.65'	8 - 9"	Mass Highway Standard Specifications
D_{85}	1.55'	5.5 - 6.5"	
D_{50}	1.3'	4.5 - 5"	
D_{15}	0.83'	3 1/4 - 4 3/4"	
D_{10}	0.67'	3 - 4.5"	

Layer Thickness

1.5 x D_{50} (ft)	1.95	0.63	HEC-11, pg. 38
2 x D_{50} (ft)	2.6	0.83	
D_{100}	1.65	0.75	
use	2.5	1	

Chart 1

Chart 2

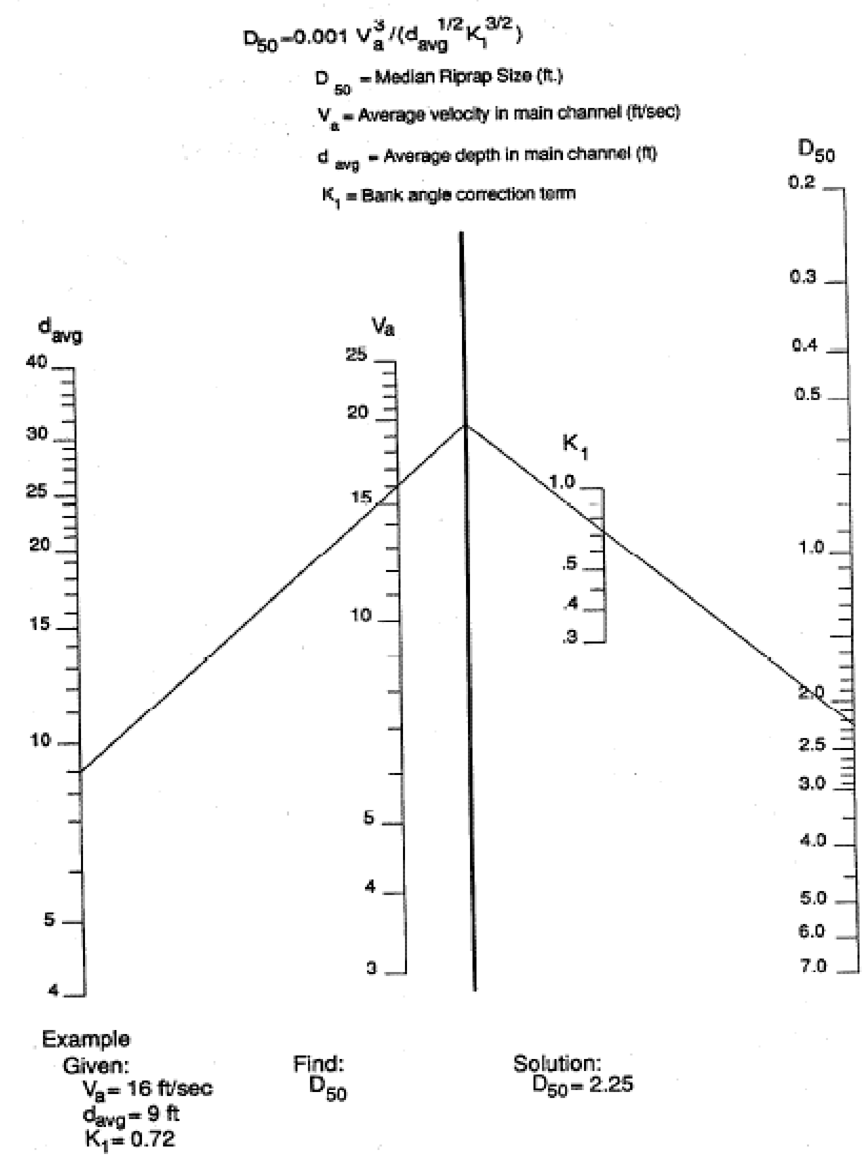


Chart 1. Riprap size relationship

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4.1.1.1 Design Relationship

A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented in equation 6. The design relationship in equation 6 is based on the assumption of uniform, gradually varying flow. The derivation of equation 6 along with a comparison with other methods is presented in appendix D. Chart 1 in appendix C presents a graphical solution to equation 6. Equation 7 can be solved using charts 3 and 4 of appendix C.

$$D_{50} = 0.001\ V_a^{3} / (d_{avg}^{0.5}\ K_1^{1.5}) \tag{6}$$

where

D_{50} = the median riprap particle size;
C = correction factor (described below);
V_a = the average velocity in the main channel (ft/s (m/s));
d_{avg} = the average flow depth in the main flow channel (ft (m)); and
K_1 is defined as:

$$K_1 = [1-(\sin^2\theta/\sin^2\phi)]^{0.5} \tag{7}$$

where

θ = the bank angle with the horizontal; and
ϕ = the riprap material's angle of repose.

The average flow depth and velocity used in equation 6 are main channel values. The main channel is defined as the area between the channel banks (see Figure 17).

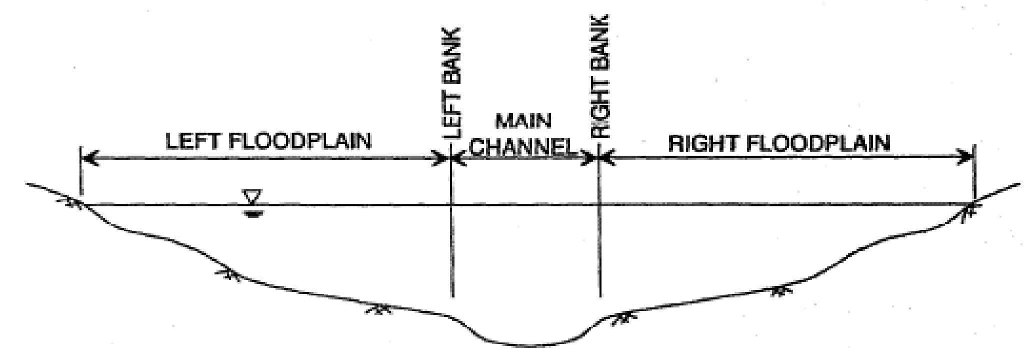


Figure 17 Definition sketch; channel flow distribution

Chart 3

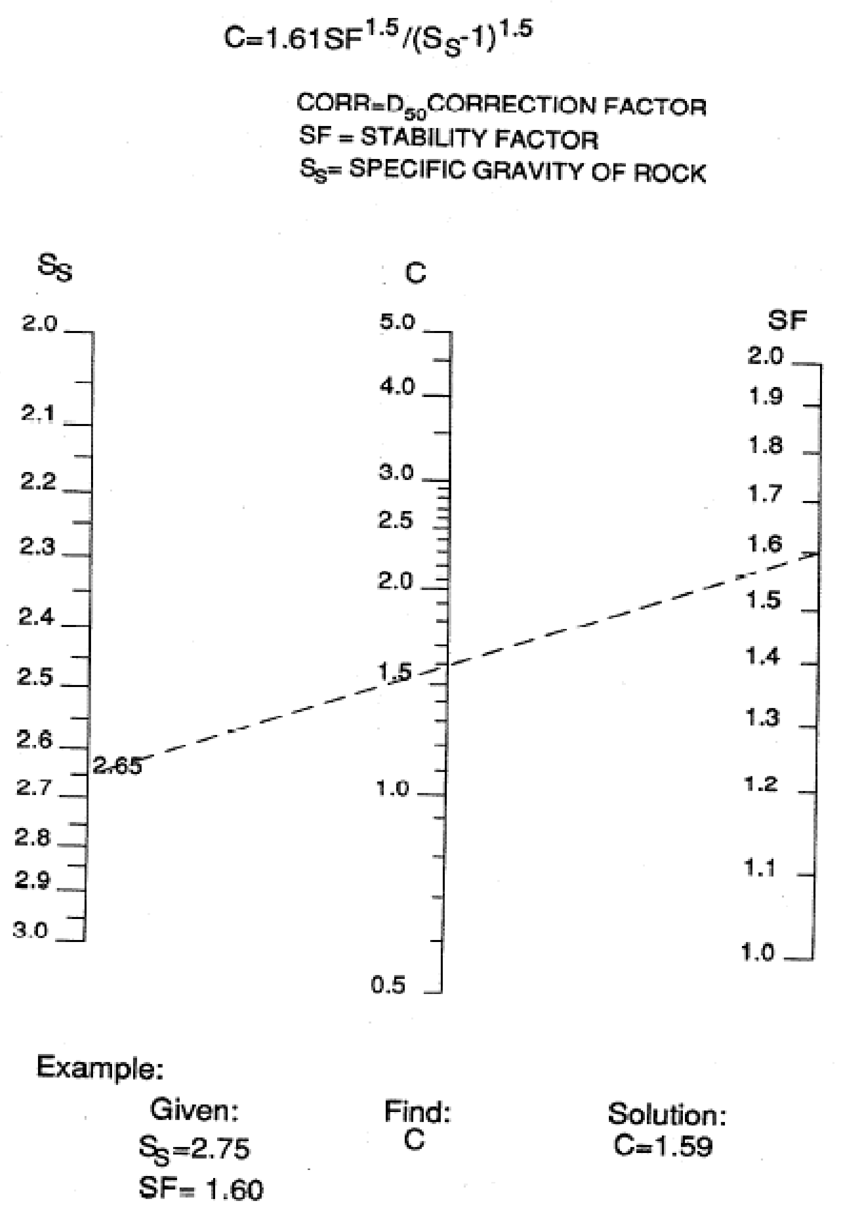


Chart 2. Correction factor for riprap size

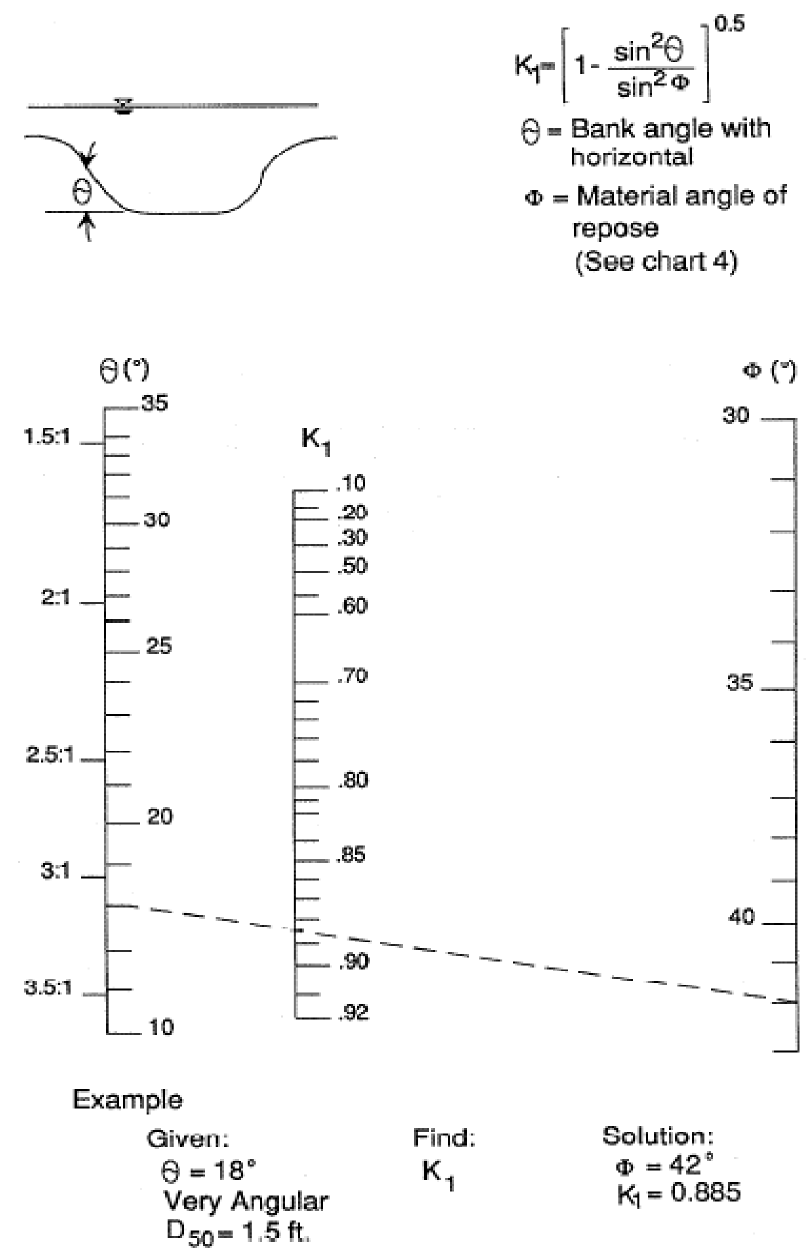


Chart 3. Bank angle correction factor (K_1) nomograph

Table 1. Guidelines for the selection of stability factors

Condition	Stability Factor Range
Uniform flow; Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; Moderate bend curvature (30 > curve radius/channel width > 10); Impact from waves or floating debris moderate.	1.3 - 1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (1 - 2 ft (.30 - .61 m)); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 - 2.0

Chart 4 Page 73 PDF Page 91

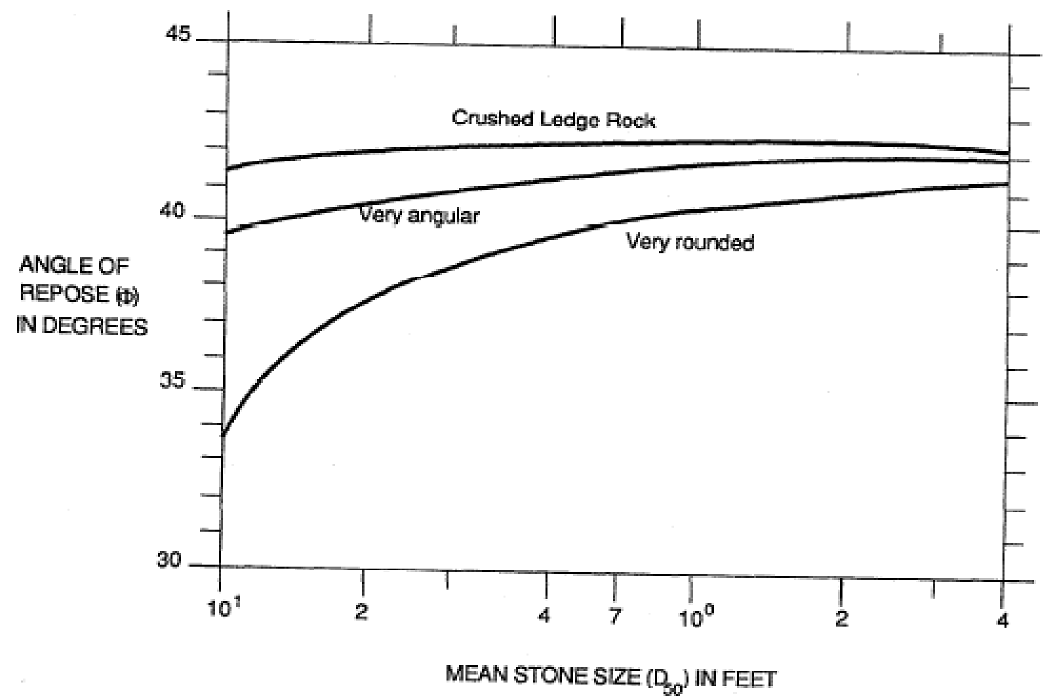


Chart 4. Angle of repose of riprap in terms of mean size and shape of stone.

Appendix H – Large Woody Debris Anchor Calculations

FOS - Buoyancy	
Fpiles	1650.63
FLWMS	-294.053
FL	-56.5088
FOS_b	**4.708527**

FOS - Sliding/Resistance
380.8033543

FOS - Overturning
7.906238542

Table 4. Minimum recommended factors of safety.

Public Safety Risk	Property Damage Risk	Stability Design Flow Criteria	$FOS_{sliding}$	$FOS_{bouyancy}$	$FOS_{rotation}$ $FOS_{overturning}$
High	**High**	100-year	1.75	2.0	1.75
High	**Moderate**	50-year	1.5	1.75	1.5
High	**Low**	25-year	1.5	1.75	1.5
Low	**High**	100-year	1.75	2.0	1.75
Low	**Moderate**	25-year	1.5	1.75	1.5
Low	**Low**	10-year	1.25	1.5	1.25

$$FOS_b = \frac{F_{LWMd}+F_{boulders}+F_{soil}+F_{piles-v}}{|F_{LWMs}+F_L|}$$ Equation 18

FOS_b = buoyancy factor of safety

$$FOS_{sliding} = \frac{|F_{hd}+F_f+F_{piles-h}+F_{passive}|}{F_d+F_{hu}+F_l}$$

BUOYANT FORCE CALCULATIONS

Buoyant force—The buoyant force is equal to the weight of the displaced water volume. The net buoyant force, $\vec{F}_b$, is equal to the difference between the weight of the structure and the weight of displaced water:

$$\vec{F}_b = \left[\rho_{wood} V_{wood} - \rho_{water} V_{water}\right]\vec{g} \qquad \text{(eq. TS14J–1)}$$

where:
ρ = density
V = volume
$\vec{g}$ = the gravitational acceleration vector in the vertical direction

For a fully submerged structure,

$$V_{wood} = V_{water} = V \text{ and } \vec{F}_b = (\rho_{wood} - \rho_{water})V\vec{g} \qquad \text{(eq. TS14J–2)}$$

Impoundment 100yr Elevation (Projected)	38.25	ft
ZBedMin	31.6	ft

Log Dimensions

L_w	Length	10	ft
D_w	Diameter	12	in
V_w	Wood Volume	7.853982	ft3
ρ_w	Wood Density	0.4	g/cm3
		24.96	lb/ft3
ρ_{h20}	Water Density	62.4	lb/ft3
V_{h20}	Water Volume	7.853982	ft3
g	Gravity	32.2	ft/s2
Fb	Buoyant Force	-294.053	lb
	Total Buoyant Force	-350.562	lb

Equivalent Boulder Size Required

FOS	2				
Stone Density	165	lb/ft3	Fl	-56.5088	lb
Req'd Stone Volume	3.564279665	ft3	Fpiles-v	0	lb
Req'd Stone Diameter	1.895211648	ft			

Table TS14J–2 Published values for design variables for LWM structures

Quantity	Used for	Typical values	Source
Density of wood in g/cm^3 (lowest, or worst-case condition[1/])	Buoyant force computation	0.4 to 0.5 0.5 0.4 to 0.5	Shields, Morin, and Cooper (2004) D'Aoust and Millar (2000) D'Aoust and Millar (1999)
Drag coefficient	Drag force computation	0.7 to 0.9 Up to 1.5 0.4 to 1.2 1.0 1.2 to 0.3 (tree) 1.2 (rootwad)	Shields and Gippel (1995) Alonso (2004) Gippel et al. (1996) Fischenich and Morrow (2000) D'Aoust and Millar (2000) D'Aoust and Millar (1999) D'Aoust and Millar (1999)
Design life for wood, yr	Planning	5 to 15	Fischenich and Morrow (2000)
Soil strength	Analysis of loads/ anchoring provided by buried members	Soil forces on buried members neglected in order to be conservative. Range of values based on soil types	Shields, Morin, and Cooper (2004)

1/ Worst case conditions presume well-dried wood. Dry wood rapidly absorbs water and may increase its density by 100% after only 24-hr submergence (Thevenet, Citterio, and Piegay 1998). However, critical conditions, especially along smaller streams, are likely to occur before wood has had time to fully absorb water.

Total Buoyant Force

The buoyant force is the sum of all vertical forces associated with a LWM structure (Equation 17). If the buoyant force is negative, the structure is anticipated to float and be unstable. If the force is positive, the structure is anticipated to remain in place.

$$F_b = F_{LWMs} + F_{LWMd} + F_L + F_{boulder} + F_{soil} + F_{piles-v} \qquad \textit{Equation 17}$$

PILE SKIN FRICTION CALCULATIONS

Impoundment 100yr Elevation (Projected)		38.25	ft
ZBedMin		31.6	ft
Log Dimensions			
N	Number of Piles	3	
D_w	Diameter	8	in
Lpiles	Embedded Pile Length	5	ft
ks	coefficient of lateral earth pressure (0.5 - 1.5)	0.5	Conservative Assumption
	Internal angle of friction of soils	30	
	Saturated Density of Soils	127	lb/ft3
r'		323	psf
p_w	Wood Density	0.4	g/cm3
		24.96	lb/ft3
p_{h20}	Water Density	62.4	lb/ft3
g	Gravity	32.2	ft/s2
Fp	Skin Friction	1650.63	lb

$$F_{piles-v} = N_{piles} * \pi * d_{piles} * L_{piles}(k_s * \tan\frac{2}{3}\emptyset * \sigma' + \frac{d_{piles}}{4} * (\gamma_{wood} - \gamma_w))$$

Equation 15

N_{piles} = number of piles

d_{piles} = diameter of piles

L_{piles} = embedded length of piles

k_s = coefficient of lateral earth pressure (0.5 to 1.5 depending on soil and density)

ϕ = internal angle of friction of soils

$$\sigma' = L_{piles} * (\gamma_{sat} - \gamma_w)$$

Equation 16

LIFT FORCE CALCULATIONS

Impoundment 100yr Elevation (Projected)		38.25	ft
ZBedMin		31.6	ft

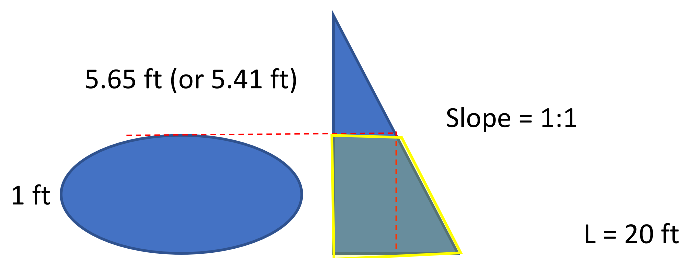
D_w	Diameter	12	in
L	Length of Logs	10	ft
	Saturated Density of Soils	127	lb/ft3
C_L	Lift coefficient	0.45	
A_{LWM}		10	sf
V_w	Wood Volume	0.785398	ft3
p_w	Wood Density	0.4	g/cm3
		24.96	lb/ft3
p_{h20}	Water Density	62.4	lb/ft3
V_{h20}	Water Volume	0.785398	ft3
g	Gravity	32.2	ft/s2
U0	Approach Velocity at Design Event	3.6	fps
FL	Lift Force	-56.5088	lb/ft3

HYDROSTATIC FORCE CALCULATIONS

Impoundment 100yr Elevation (Projected)		38.25 ft	38.25
ZBedMin		31.6 ft	
D_w	Diameter	12 in	
L	Length of Logs	10 ft	
Yu		6.65 ft	6.65
Au		10 sf	
p_w	Wood Density	0.4 g/cm3 24.96 lb/ft3	
p_{h20}	Water Density	62.4 lb/ft3	
g	Gravity	32.2 ft/s2	
Fhu	**Hydrostatic Force**	**3837.6 lb**	

Fhu=Fhd

Fhd	Hydrostatic Force	3837.6 lb



		3525.6 lb
Force =	(1)(5.65)(20)(62.4) + (1)(1)/2*(20)*(62.4)	312 lb
	(Highlighted area shown above)	**3837.6** lb

DRAG FORCE CALCULATIONS

D_w	Diameter	12	in
L	Length of Logs	10	ft
	Saturated Density of Soils	127	lb/ft3
Cd	Drag Coefficient	1.8	
A_{LWM}		10	sf
V_w	Wood Volume	0.785398	ft3
p_w	Wood Density	0.4	g/cm3
		24.96	lb/ft3
p_{h20}	Water Density	62.4	lb/ft3
V_{h20}	Water Volume	0.785398	ft3
g	Gravity	32.2	ft/s2
U0	Approach Velocity at Design Event	0.5	fps
Fd	**Drag Force**	**4.360248**	**lb**

B	0.23400936	
Ab	450	Based on section cut from island over to west bank
Ac	1473	
Vc	3.6	
Yc	5.25	
g	32.2	
Frc	0.276882121	

IMPACT FORCE CALCULATIONS

Impoundment 100yr Elevation (Projected)		38.25	ft	
ZBedMin		31.6	ft	
Wdebris	Debris Weight	196.0354	lb	
D_w	Diameter	12	in	
L	Length of Logs	10	ft	
	Saturated Density of Soils	127	lb/ft3	
Cd	Drag Coefficient	1.8		
A_{LWM}		10	sf	
V_w	Wood Volume	7.853982	ft3	
p_w	Wood Density	0.4	g/cm3	
		24.96	lb/ft3	
p_{h20}	Water Density	62.4	lb/ft3	
V_{h20}	Water Volume	7.853982	ft3	
g	Gravity	32.2	ft/s2	
U0	Approach Velocity at Design Event	0.5	fps	
Fimpact	**Impact Force**	**3.06019**	**lb**	
	weight debris	196.0354		
	delta time	1		
	Ci	1		
	Co	0.8		
	Cd	1		6.65
	Cb	1		
	Rmax	0.8		

$$F_i = \frac{\pi w_{debris} * V_{channel} * C_i * C_o * C_d * C_b * R_{max}}{2 * g * \Delta t}$$

$$F_i = impact\ force$$

$$w_{debris} = weight\ of\ debris$$

$$g = acceleration\ constant\ due\ to\ gravity$$

$$V_{channel} = water\ velocity\ in\ channel$$

$$\Delta t = time\ from\ initial\ velocity\ to\ zero\ velocity$$

Table 6. Values of coefficient of importance based on risk.

Public Safety Risk Rating	Property Damage Risk Rating	Coefficient of Importance
High	High	1.0
High	Medium	0.9
High	Low	0.8
Low	High	0.7
Low	Medium	0.6
Low	Low	0.5

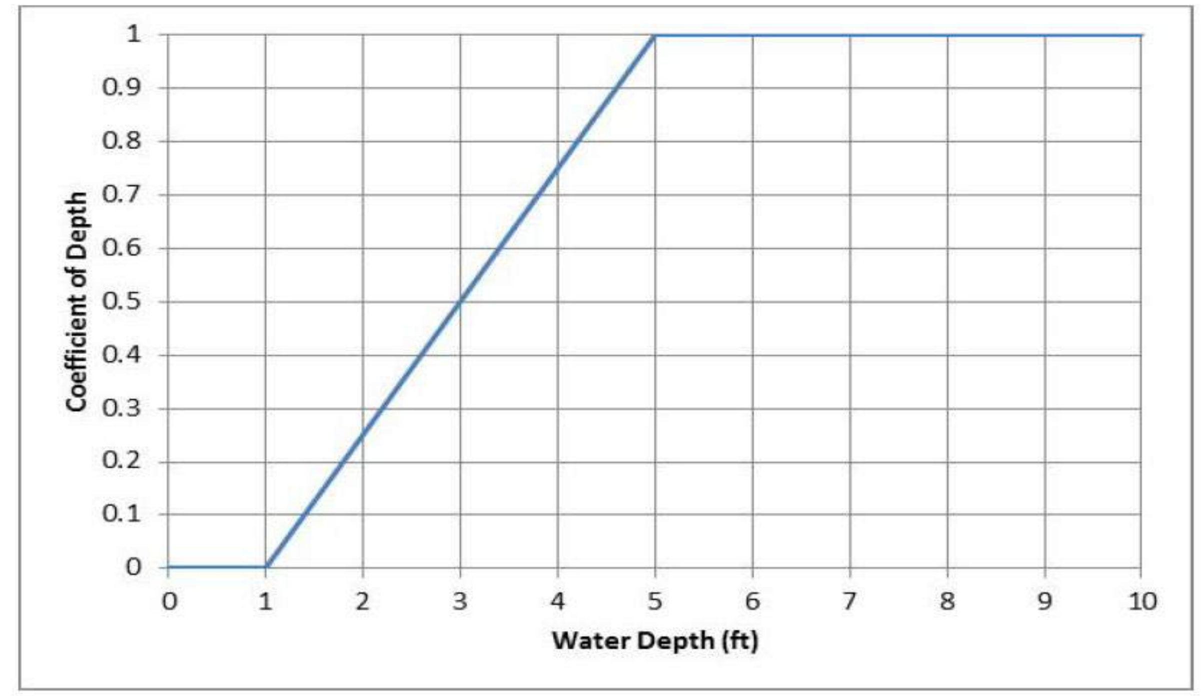


Figure 11. Coefficient of depth.

4 ft deep

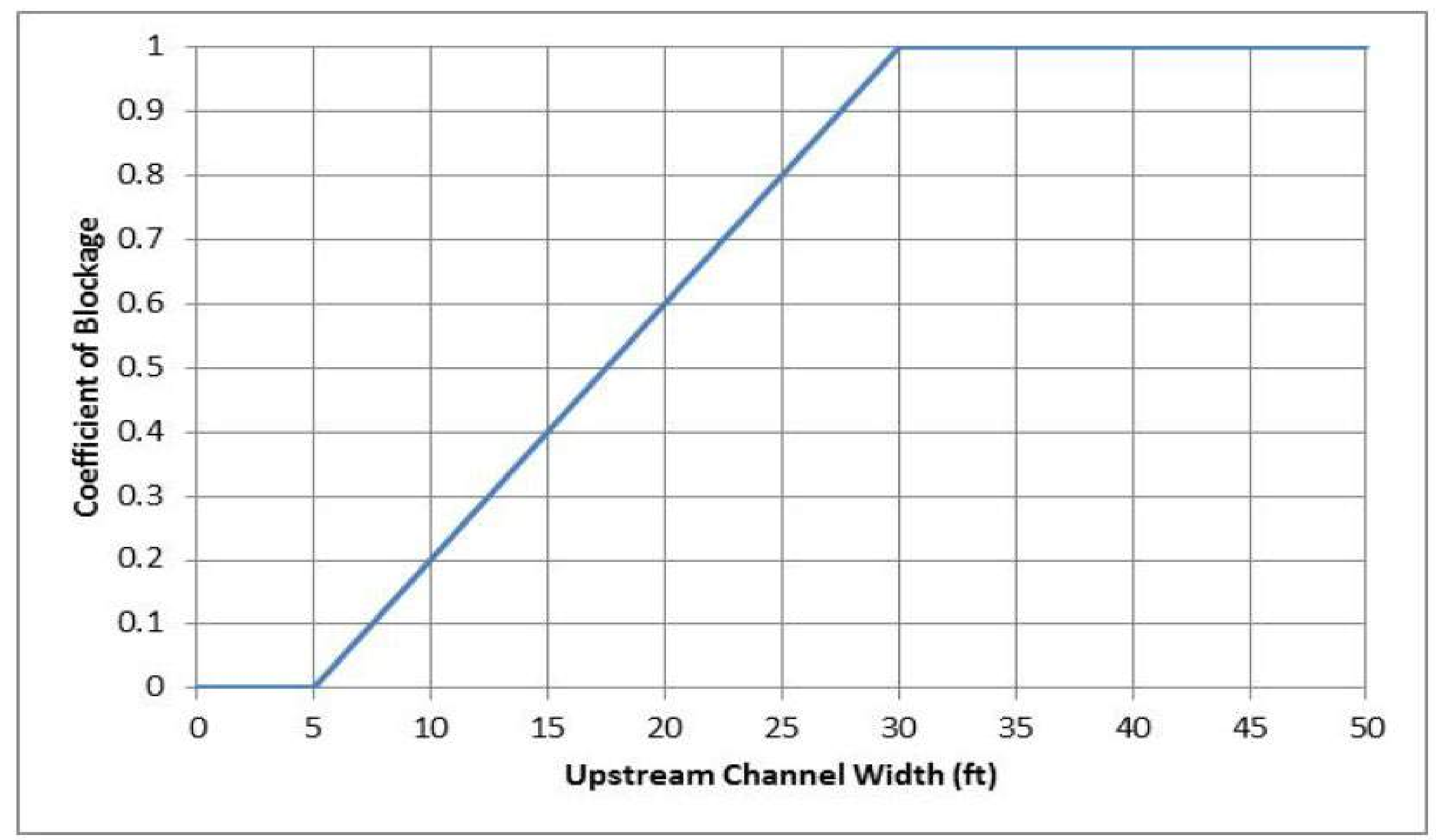


Figure 12. Coefficient of blockage.

FRICTION BED RESISTANCE CALCULATIONS

Impoundment 100yr Elevation (Projected)		38.25	ft
ZBedMin		31.6	ft
Wdebris	Debris Weight	196.0354	lb
D_w	Diameter	12	in
L	Length of Logs	10	ft
	Saturated Density of Soils	127	lb/ft3
Cd	Drag Coefficient	1.8	
A_{LWM}		10	sf
V_w	Wood Volume	7.853982	ft3
p_w	Wood Density	0.4	g/cm3
		24.96	lb/ft3
p_{h20}	Water Density	62.4	lb/ft3
V_{h20}	Water Volume	7.853982	ft3
g	Gravity	32.2	ft/s2
	Friction Angle	30	
Mbed			
Ffriction	**Friction Bed Resistance**	**202.397**	**lb**

Table 5. Substrate and soil properties, reproduced from Rafferty (2013).

Grain size (mm)	Sediment Class	Average Dry Unit Weight (lb/ft^3)	Internal Friction Angle (degrees)
Bedrock	Bedrock	165	-
256-2048	Boulder	146	42
128-256	Large Cobble	142	42
64-128	Small Cobble	137	41
32-64	Very coarse gravel	131	40
16-32	Coarse gravel	126	38
8-16	Medium gravel	120	36
4-8	Fine gravel	115	35
2-4	Very fine gravel	109	33
1-2	Very coarse sand	103	32
0.5-1	Coarse sand	98	31
0.25-0.5	Medium sand	94	30
0.125-0.25	Fine sand	93	30
0.063-0.125	Very fine sand	92	30
0.004-0.063	Silt	82	30
<0.004	Clay	78	25

$$F_f = -\mu_{bed} * (F_b - F_{piles-v})$$

F_f = *force due to frictional resistance*

$F_b - F_{piles-v} > 0$

$$\mu_{bed} = \tan \emptyset$$

PILE LATERAL RESISTANCE CALCULATIONS

Impoundment 100yr Elevation (Projected)		38.25 ft
ZBedMin		31.6 ft
Wdebris	Debris Weight	196.0354 lb
D_w	Diameter	12 in
L	Length of Logs	10 ft
	Saturated Density of Soils	127 lb/ft3
Cd	Drag Coefficient	1.8
A_{LWM}		10 sf
V_w	Wood Volume	7.853982 ft3
ρ_w	Wood Density	0.4 g/cm3 24.96 lb/ft3
ρ_{h20}	Water Density	62.4 lb/ft3
V_{h20}	Water Volume	7.853982 ft3
g	Gravity	32.2 ft/s2
	Friction Angle	30 degrees
	Lpile	5 ft
	Dpile	0.666667 ft
	hLoad	3 ft
Kp		3
N	number of Piles	3
F(Pile_horizontal)	**Pile Lateral Resistance**	**-3028.13 lb**

$$F_{piles-h} = -N_{piles} * \frac{L_{pile}^3 * \frac{1}{2} * \gamma_e * d_{pile} * K_p}{h_{load} + L_{pile}}$$

N_{piles} = *number of piles*

L_{pile} = *length of pile embedded below potential scour depth*

$$\gamma_e = \gamma_s - \gamma_w \qquad \textit{effective unit weight of soil}$$

γ_s = *dry unit weight of the soil*

γ_w = *unit weight of the soil*

d_{pile} = *diameter of the pile*

h_{load} = *height above the potential scour depth the load is applied*

$$K_p = \frac{1+\sin\emptyset}{1-\sin\emptyset}$$

Table 5. Substrate and soil properties, reproduced from Rafferty (2013).

Grain size (mm)	Sediment Class	Average Dry Unit Weight (lb/ft³)	Internal Friction Angle (degrees)
Bedrock	Bedrock	165	-
256-2048	Boulder	146	42
128-256	Large Cobble	142	42
64-128	Small Cobble	137	41
32-64	Very coarse gravel	131	40
16-32	Coarse gravel	126	38
8-16	Medium gravel	120	36
4-8	Fine gravel	115	35
2-4	Very fine gravel	109	33
1-2	Very coarse sand	103	32
0.5-1	Coarse sand	98	31
0.25-0.5	Medium sand	94	30
0.125-0.25	Fine sand	93	30
0.063-0.125	Very fine sand	92	30
0.004-0.063	Silt	82	30
<0.004	Clay	78	25

$$|F_{piles-h}| = FOS_{sliding-min} * (F_d + F_{hu} + F_i) + |F_{hd} + F_f + F_{passive}|$$

Equation 39

$FOS_{sliding-min}$ = *minimum allowed factor of safety for sliding*

$$L_{pile}^3 = \frac{|F_{piles-h}| * (L_{pile} + h_{load})}{N_{piles} * \gamma_e * d_{pile} * {}^{K_p}/_4} \qquad \textbf{Equation 40}$$

It is important to look at the ultimate pile strength (shear and moment) versus the applied loads to ensure that the pile material is structurally sound and will not snap or shear off during the design event. This is often the limiting factor on piles and can result in more piles than originally estimated from the equations above.

Sliding Factor of Safety

The sliding factor of safety is computed by dividing the resistance forces of downstream hydrostatic pressure, friction, lateral resistance from piles, and passive forces by the sum of drag forces and upstream hydrostatic forces as seen in Equation 41.

$$FOS_{sliding} = \frac{|F_{hd} + F_f + F_{piles-h} + F_{passive}|}{F_d + F_{hu} + F_i} \qquad \textbf{Equation 41}$$

If $FOS_{sliding}$ is less than the recommended minimum FOS, additional resisting forces need to be created. Additional horizontal resistance can come in the form of additional piles, anchors, or ballast to increase the frictional resistance. Passive horizontal forces could include sediment and soil located on the reactive side of a structure as is typical on apex style LWD

structure where the structure is located at the head (upstream) end of an island. The island would provide additional passive horizontal resistance to the drag force. The most common types of additional anchoring methods to resist drag forces include the addition of piles, extra ballast or backfill, and last is the addition of mechanical anchors if absolutely necessary.

ROTATION RESISTANCE CALCULATIONS

Impoundment 100yr Elevation (Projected)			38.25	ft
ZBedMin			31.6	ft
Wdebris		Debris Weight	196.0354	lb
	D_w	Diameter	12	in
	L	Length of Logs	10	ft
	Fi		3.06019	
	Fd		4.360248	
	Fhu		3837.6	
	Fhd		3837.6	
Ff			202.397	
Fpile-h			3028.125	
	Lsp		15	
	Lebp		0	
	ρ_{h20}	Water Density	62.4	lb/ft3
	V_{h20}	Water Volume	3837.6	ft3
	g	Gravity	32.2	ft/s2
	Mdrotation		28837.65329	
	Mrrotation		75721.85252	
			2.625798006	

$$MD_{rotation} = (F_i + F_d + F_{hu}) * \left(\frac{L_{sp}+L_{ebp}}{2}\right)$$ **Equation 42**

L_{sp} = *length of wood structure from tip to point of rotation measured perpendicular to flow*

L_{ebp} = *embedded length of wood structure measured perpendicular to flow*

$$MR_{rotation} = \left| F_{hd} * \left(\frac{L_{sp} + L_{ebp}}{2}\right) + F_{passive} * \frac{L_{ebp}}{2} + F_f * \frac{L_{sp}}{2} + \sum_i^n F_{pile-h_i} * L_{ph_i} \right|$$

Equation 43

$$F_{pile-h_i} = \frac{F_{piles-h}}{N_{piles}}$$ **Equation 44**

L_{phi} = *distance from pile 'i' to the point of rotation measured perpendicular to flow*

$$FOS_{rotation} = \frac{MR_{rotation}}{MD_{rotation}}$$ **Equation 45**

OVERTURNING RESISTANCE CALCULATIONS

Impoundment 100yr Elevation (Projected)			38.25	ft
ZBedMin			31.6	ft
Wdebris		Debris Weight	196.0354	lb
	D_w	Diameter	12	in
	L	Length of Logs	10	ft
	Fi		3.06019	
	Fd		4.360248	
	Fhu		3837.6	
	Fhd		3837.6	
Ff			202.397	
Fpile-h			3028.125	
dbury			0	
Ls			1	
	Lpvi		1	
	Fpile vi		550.2101	
	p_{h20}	Water Density	62.4	lb/ft3
	V_{h20}	Water Volume	3837.6	ft3
	g	Gravity	32.2	ft/s2
	Mdoverturn		208.7756707	lb-ft
	Mroverturn		1650.630254	lb-ft

Timber design strengths (NDS):

Red Pine

fb	1350	psi
fv	125	psi
fc	270	psi
Cd	0.9	
Ct	1	
Cct	1	
Cf	1	
Cb	1	
Cis	1.05	
fb'	1275.75	psi
fv'	112.5	psi
fc'	270	psi

Forces	F(lb)	Force per pile (lb)	Shear (lb)	Log Length (ft)	Pile Length (ft)	Moment Log (ftlb)	Moment Pile (ftlb)
Hydrostatic (U/S)	3837.6	1918.8	1918.8	20		9594.0	
Hyroostatic (D/S)	-3837.6	-1918.8	-1918.8	20		-9594.0	
Drag	4.36	2.2	2.2	20		10.9	
Impact	3.06019	1.5	1.5	20		7.7	
			3.7		5	18.6	18.6

Forces of Log:

Vertical (Fb)	-350.56	lb	For bearing of piles. Neglect skin friction since reactive force
Lateral (Fh)	3.71	lb	Neglect Friction and laterlaa pile capacity (i.e., soil) since these are reactive forces
Bending Moment	18.55	ftlb	Same for Pile and Log (Moment for log = FL/4; Moment for Pile = FL)

Section Properties:

Dpile	8	in
Dlog	12	in
Spile	50.3	in3
Slog	169.6	in3
bpile	8.0	in
blog	12.0	in
Qpile	42.7	in3
Qlog	144.0	in3
Ipile	201.1	in4
Ilog	1017.9	in4
Apile	50.3	in2
Alog	113.1	in2

Stresses:

Log:				
	Fb	1.31	psi	**OK**
	Fv	0.04	psi	**OK**
Pile:	Fb	4.43	psi	**OK**
	Fv	0.10	psi	**OK**
	Fc	-6.97	psi	**OK**

Appendix I – Water Control Calculations

HY-8 Culvert Analysis Report

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow
Minimum Flow: 1 cfs
Design Flow: 12 cfs
Maximum Flow: 50 cfs

Table 1 - Summary of Culvert Flows at Crossing: Crossing 1

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 1 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
31.96	1.00	1.00	0.00	1
32.65	5.90	5.90	0.00	1
33.18	12.00	12.00	0.00	1
33.45	15.70	15.70	0.00	1
33.77	20.60	20.60	0.00	1
34.07	25.50	25.50	0.00	1
34.37	30.40	30.40	0.00	1
34.66	35.30	35.30	0.00	1
34.98	40.20	40.20	0.00	1
35.34	45.10	45.10	0.00	1
35.90	50.00	50.00	0.00	1
37.00	58.29	58.29	0.00	Overtopping

Rating Curve Plot for Crossing: Crossing 1

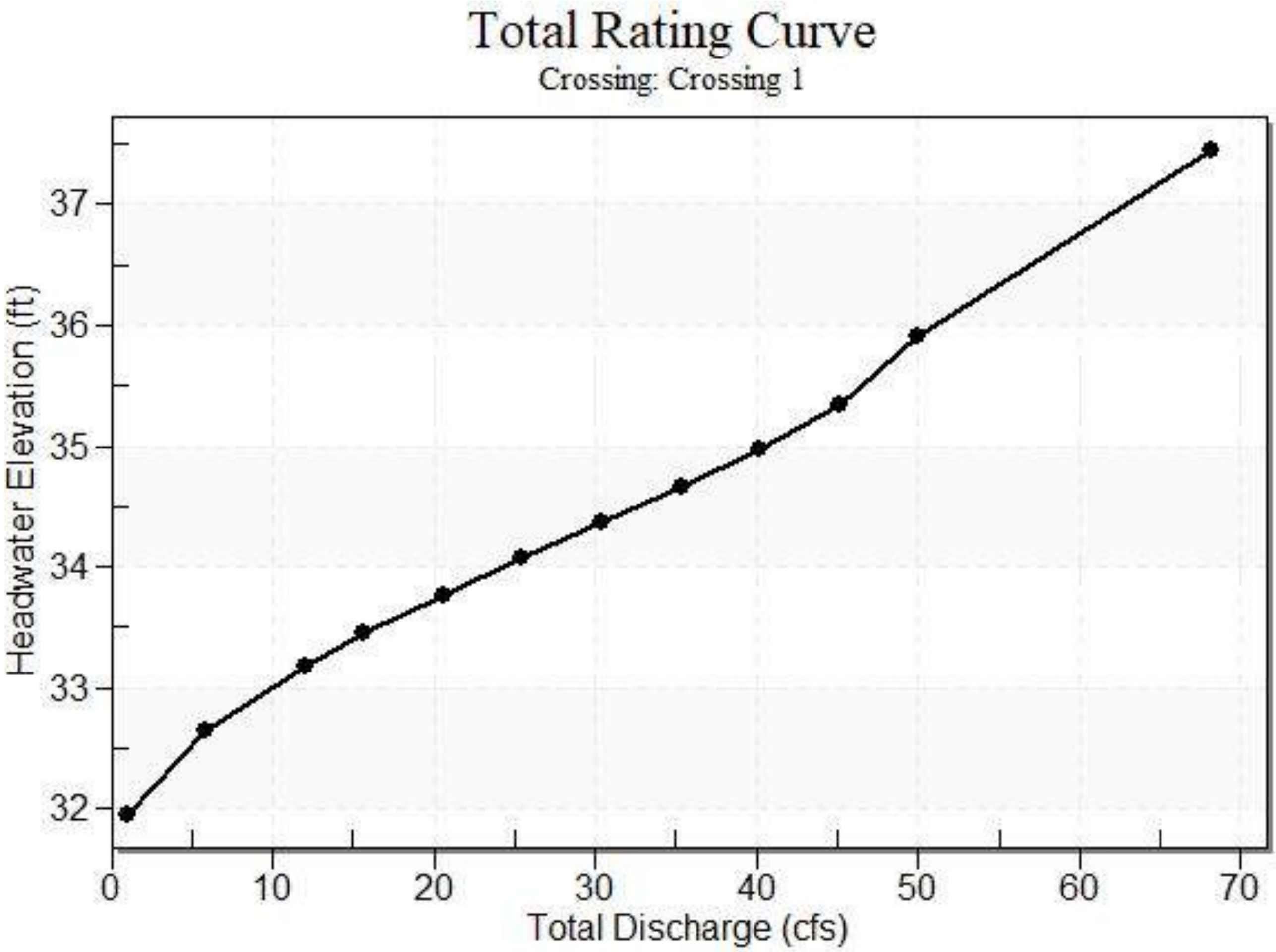


Table 2 - Culvert Summary Table: Culvert 1

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Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
1.00	1.00	31.96	0.416	0.459	2-M2c	0.341	0.309	0.309	0.203	2.604	0.985
5.90	5.90	32.65	1.039	1.151	2-M2c	0.818	0.762	0.762	0.624	4.175	1.890
12.00	12.00	33.18	1.516	1.683	2-M2c	1.189	1.099	1.099	1.000	5.113	2.399
15.70	15.70	33.45	1.770	1.950	2-M2c	1.381	1.264	1.264	1.202	5.547	2.612
20.60	20.60	33.77	2.081	2.272	2-M2c	1.621	1.457	1.457	1.452	6.048	2.837
25.50	25.50	34.07	2.376	2.574	3-M2t	1.857	1.630	1.690	1.690	6.218	3.019
30.40	30.40	34.37	2.666	2.867	3-M2t	2.105	1.786	1.918	1.918	6.372	3.170
35.30	35.30	34.66	2.962	3.163	3-M2t	2.395	1.931	2.139	2.139	6.547	3.301
40.20	40.20	34.98	3.272	3.477	3-M2t	3.000	2.064	2.355	2.355	6.753	3.414
45.10	45.10	35.34	3.603	3.842	3-M2t	3.000	2.188	2.567	2.567	7.004	3.514
50.00	50.00	35.90	3.962	4.404	7-M2t	3.000	2.301	2.775	2.775	7.323	3.604

Straight Culvert

Inlet Elevation (invert): 31.50 ft, Outlet Elevation (invert): 31.00 ft

Culvert Length: 50.00 ft, Culvert Slope: 0.0100

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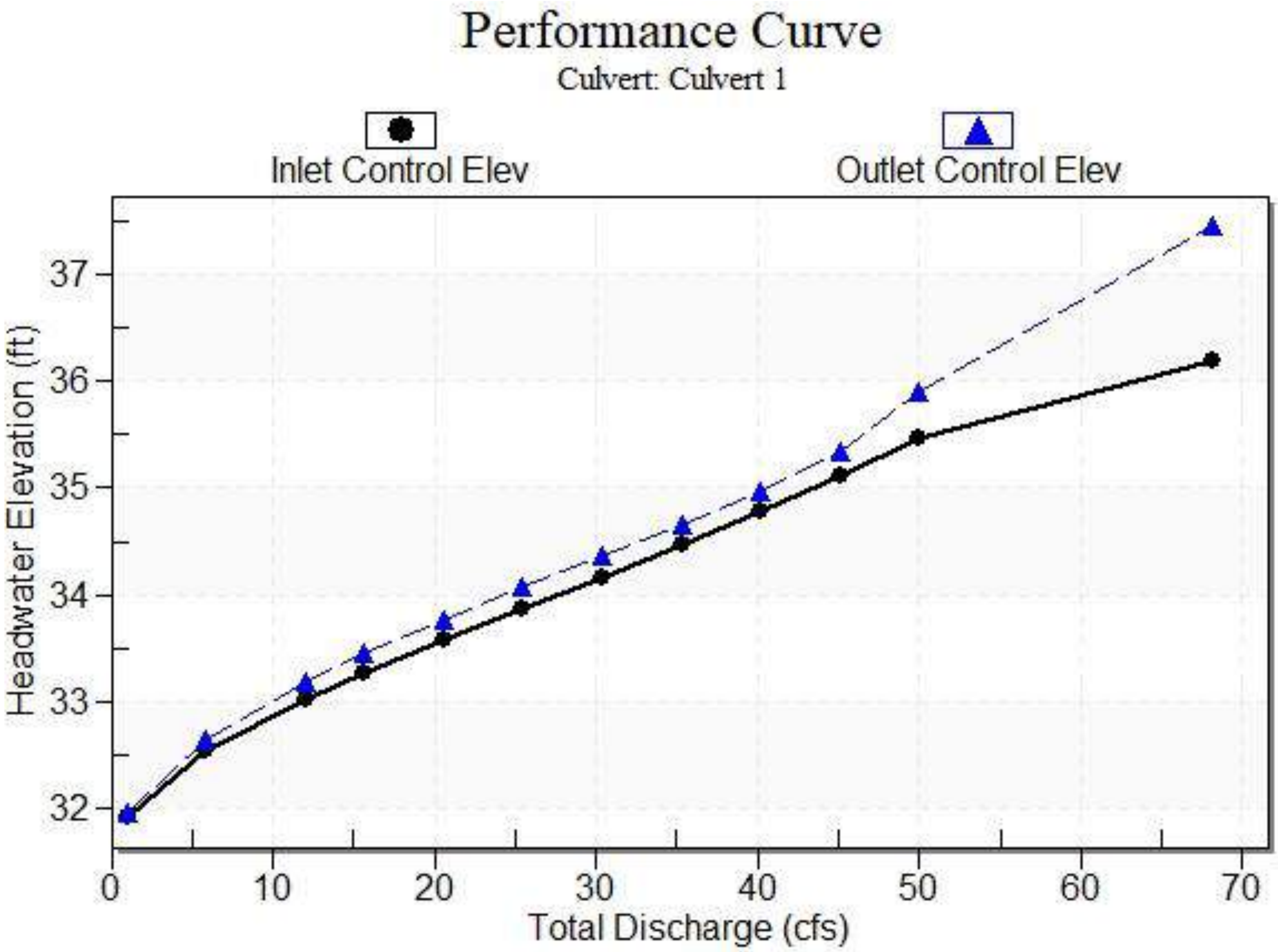
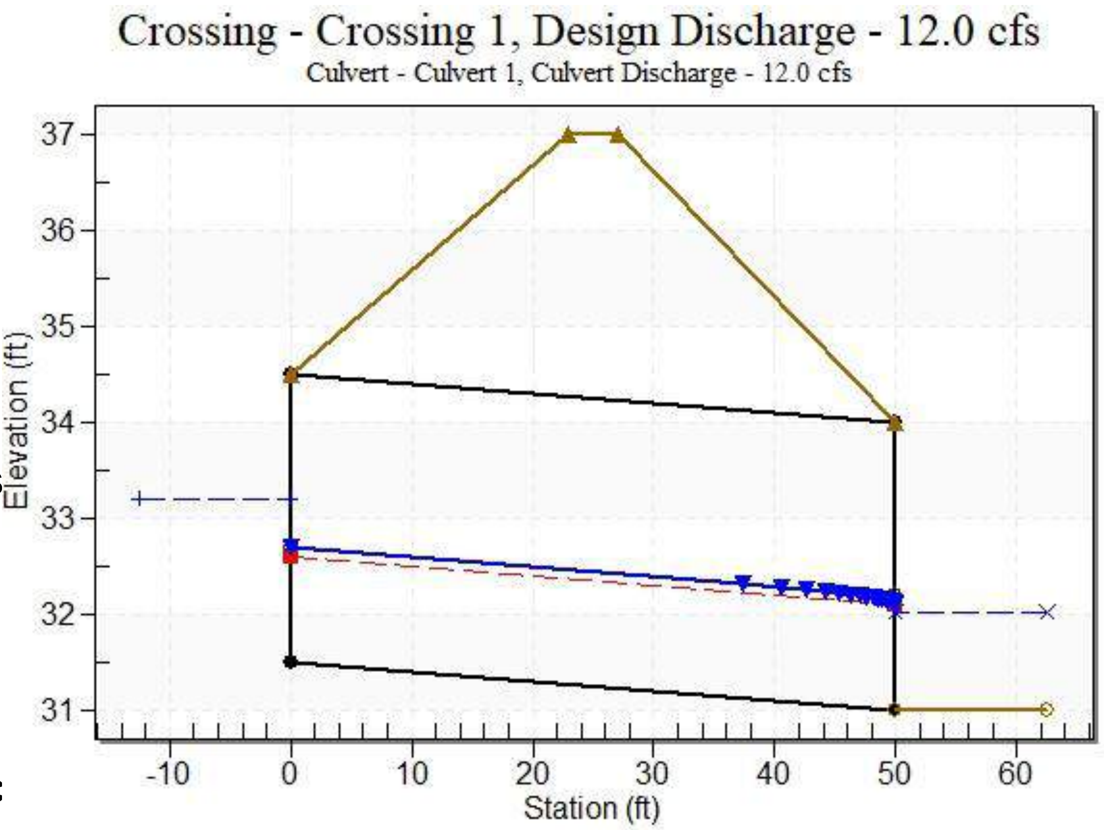
Culvert Performance Curve Plot: Culvert 1



Water Surface Profile Plot for Culvert: Culvert 1



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Barrel Diameter: 3.00 ft

Barrel Material: Corrugated PE

Embedment: 0.00 in

Barrel Manning's n: 0.0240

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Table 3 - Downstream Channel Rating Curve (Crossing: Crossing 1)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
1.00	31.20	0.20	0.98	0.06	0.39
5.90	31.62	0.62	1.89	0.19	0.42
12.00	32.00	1.00	2.40	0.31	0.42
15.70	32.20	1.20	2.61	0.38	0.42
20.60	32.45	1.45	2.84	0.45	0.41
25.50	32.69	1.69	3.02	0.53	0.41
30.40	32.92	1.92	3.17	0.60	0.40
35.30	33.14	2.14	3.30	0.67	0.40
40.20	33.36	2.36	3.41	0.73	0.39
45.10	33.57	2.57	3.51	0.80	0.39
50.00	33.77	2.77	3.60	0.87	0.38

Tailwater Channel Data - Crossing 1

Tailwater Channel Option: Rectangular Channel

Bottom Width: 5.00 ft

Channel Slope: 0.0050

Channel Manning's n: 0.0350

Channel Invert Elevation: 31.00 ft

Roadway Data for Crossing: Crossing 1

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 10.00 ft

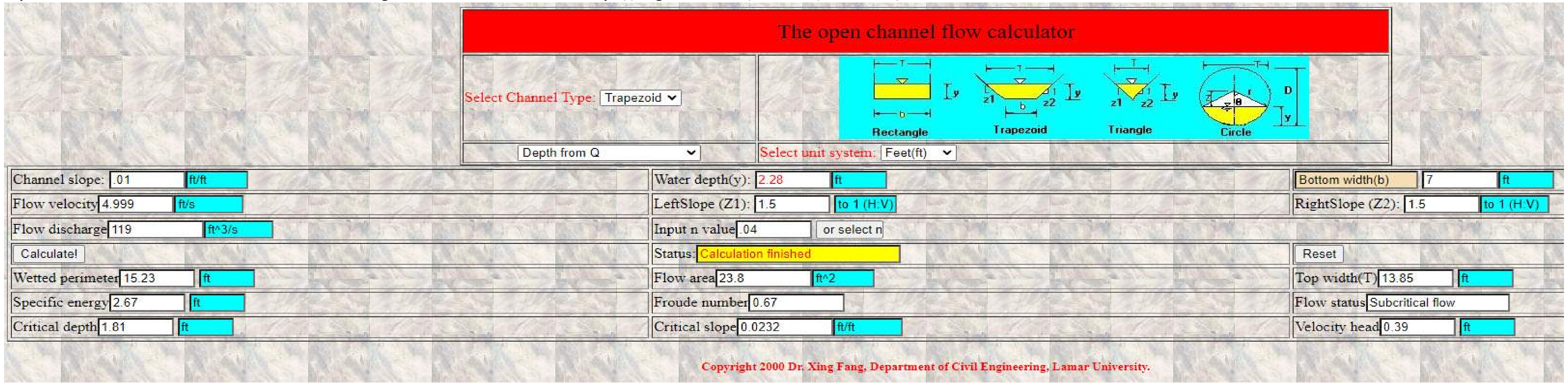
Crest Elevation: 37.00 ft

Roadway Surface: Paved

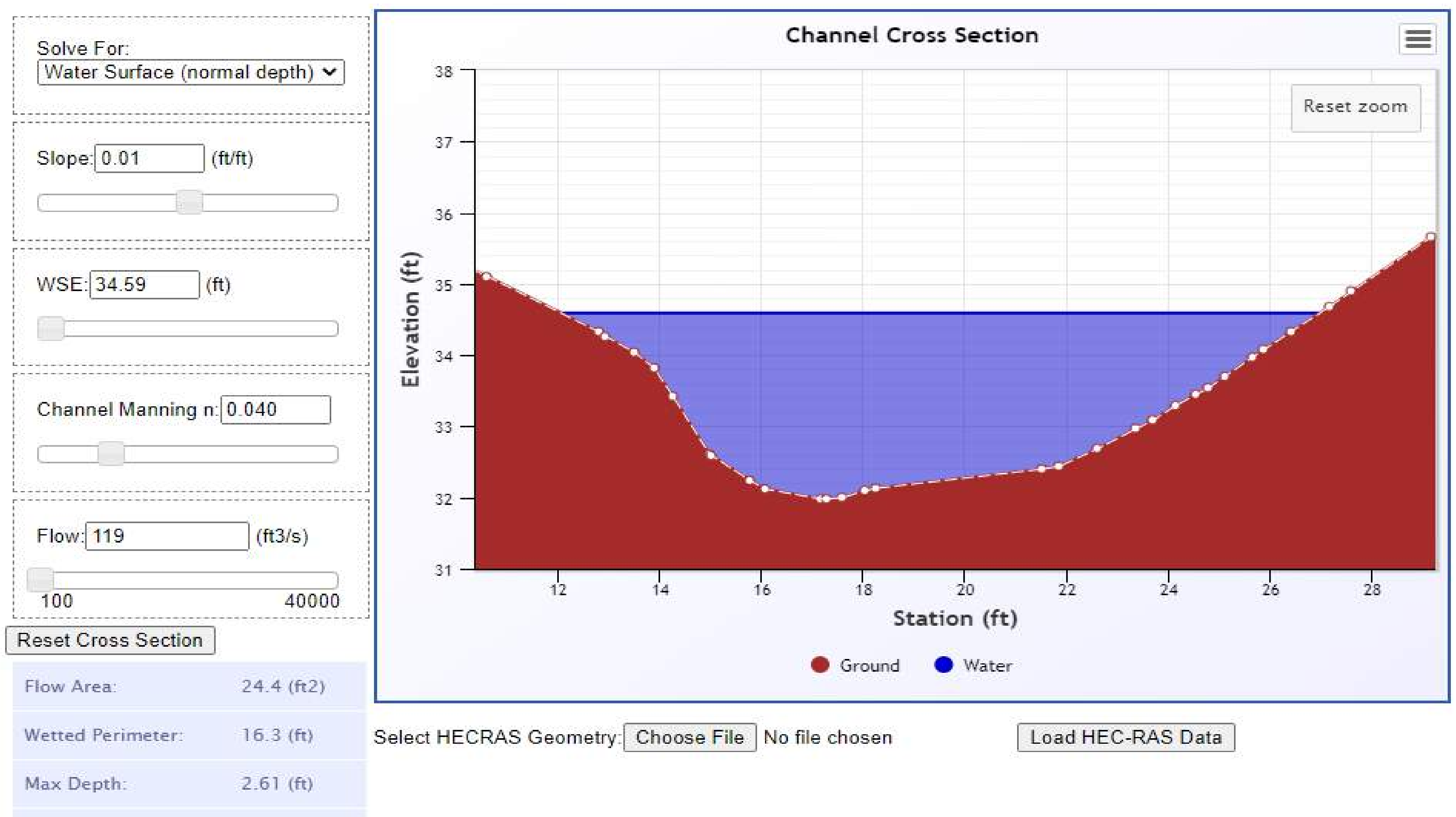
Roadway Top Width: 4.00 ft

Water Control Calculations - Open Channel Alternatives

Open Channel Flow Calculator - Bankfull Flow - Existing Conditions at River Street - https://eng.auburn.edu/~xzf0001/Handbook/Channels.html



Bankfull Width Check Using NOAA Normal Depth Calculator - https://www.bing.com/search?q=normal+depth+calculator+noaa&cvid=604388e955e948cfb01a635c1e1bcc5e&aqs=edge.0.69i59j69i57j0l7.1623j0j1&pglt=41&FORM=ANNAB1&PC=W069



Average Velocity:	4.88 (ft/s)
Top Width:	15 (ft)
Iterations:	521
Froude Number:	0.68

Open Channel Flow Calculator - Bankfull Flow - 5' Wide Sandbag Cofferdam Channel at River Street - https://eng.auburn.edu/~xzf0001/Handbook/Channels.html