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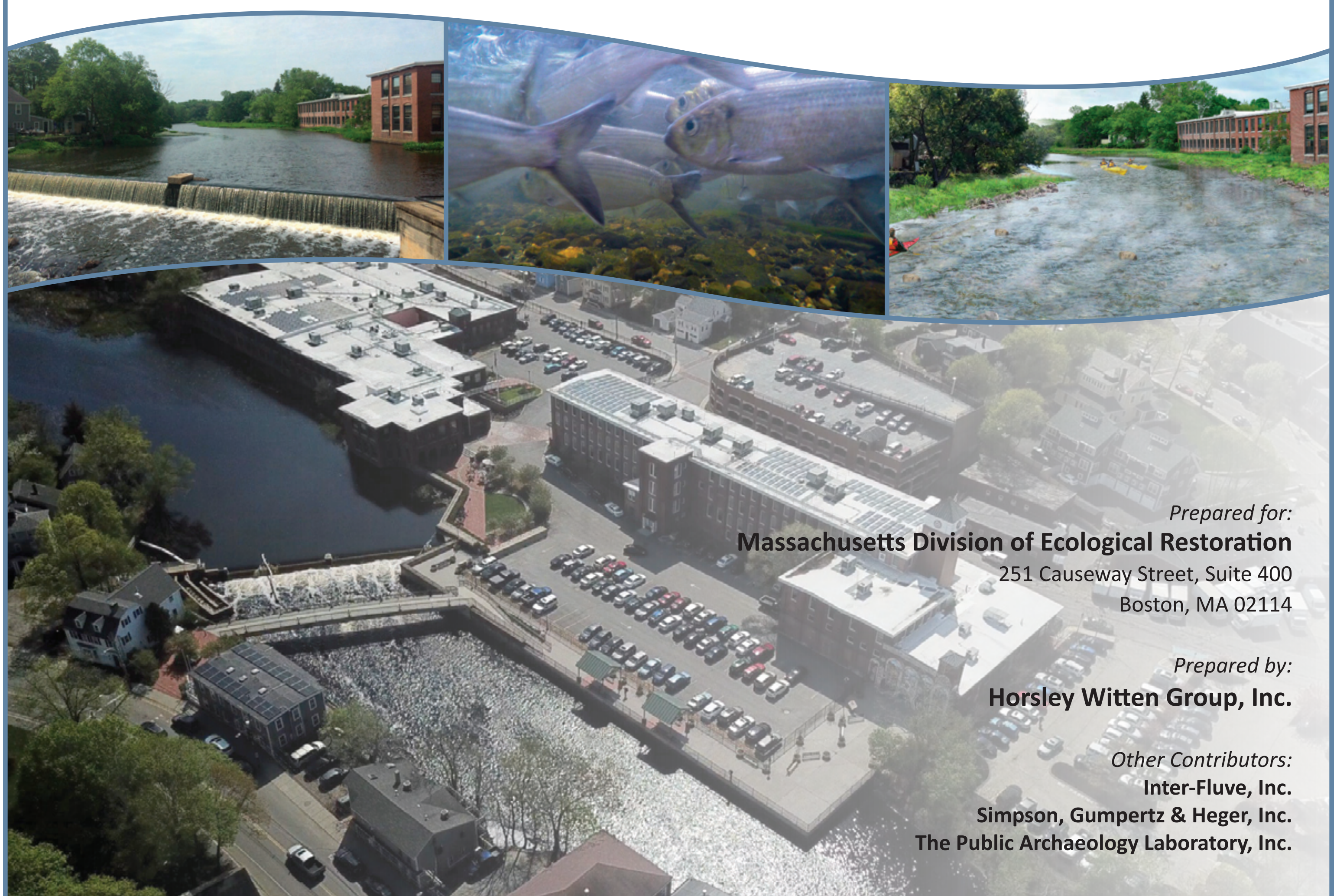
Ipswich Mills Dam Removal Feasibility Study

Ipswich, Massachusetts

March 2019



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Attachments:

1.0 INTRODUCTION

The Horsley Witten Group, Inc. (HW) is pleased to submit to the Massachusetts Division of Ecological Restoration (DER) this report documenting the Ipswich Mills Dam Removal Feasibility Study in Ipswich, Massachusetts. Other significant Project Partners include:

- The Town of Ipswich (the Town), including the Planning and Development Department, the Conservation Commission, the Department of Public Works, and the Historic Commission;
- The Ipswich River Watershed Association (IRWA);
- The United States Fish and Wildlife Service (USFWS);
- The National Oceanic and Atmospheric Administration (NOAA) Restoration Center;
- EBSCO Publishing, Inc.; and
- Trout Unlimited (TU), Northeast Chapter.

Funding was provided by:

- The National Fish and Wildlife Foundation (NFWF);
- NOAA;
- U.S. Department of the Interior;
- DER; and
- The Massachusetts Environmental Trust (MET).

In addition to HW who led and managed the feasibility effort, the HW Team who completed the project included:

- Inter-Fluve, Inc. (IF) responsible for hydrologic and hydraulic (H&H) modeling, primary conceptual design development, and other technical assistance;
- Simpson, Gumpertz, and Heger, Inc. (SGH) responsible for structural assessment of the potential impacts of dam removal on the structural stability of the EBSCO facility, located immediately upstream of the dam and adjacent to the river; and
- Public Archaeological Laboratory, Inc. (PAL) who conducted historical and archaeological research for the dam site and surrounding area.

The Ipswich Mills Dam is located at the head of tide on the Ipswich River, in downtown Ipswich, approximately 750-feet south (upstream) of the Route 133/South Main Street/Choate Bridge crossing (Figure 1-1). The dam is currently owned and operated by the Town of Ipswich Utilities Department (Haley & Aldrich, 2009). The river flows approximately south to north (left to right in Figure 1-1), ultimately discharging to the ocean waters of Plum Island Sound, shortly out of view to the right on Figure 1-1. In this report, all left and right directional references are relative to the direction of river flow looking downstream; river left refers to the river's left (generally approximately west) bank and river right refers to the river's right (generally east) bank. For the purposes of this report, we use the term "lower impoundment" to describe the channel immediately upstream of the dam and "impoundment" when referring to the entire length of channel upstream of the dam that is hydraulically affected by the dam structure. All elevation data given in this report are relative to the NAVD88 vertical datum in units of feet.

Figure 1-1 identifies key features discussed in this Feasibility Study, including:

- the EBSCO facility, located immediately upstream of the dam and adjacent to the river on its left bank;
- active and abandoned fish ladders immediately downstream of the dam on river right;
- the pedestrian bridge crossing the river shortly downstream from the dam;
- the historic Choate Bridge located approximately 750 feet downstream from the dam;
- Sally's Pond located several hundred feet upstream from the dam, up on its right bank; and
- The railroad bridge crossing of the river approximately a mile and a half upstream from the dam that defines the practical extent of the current impoundment created by the dam.

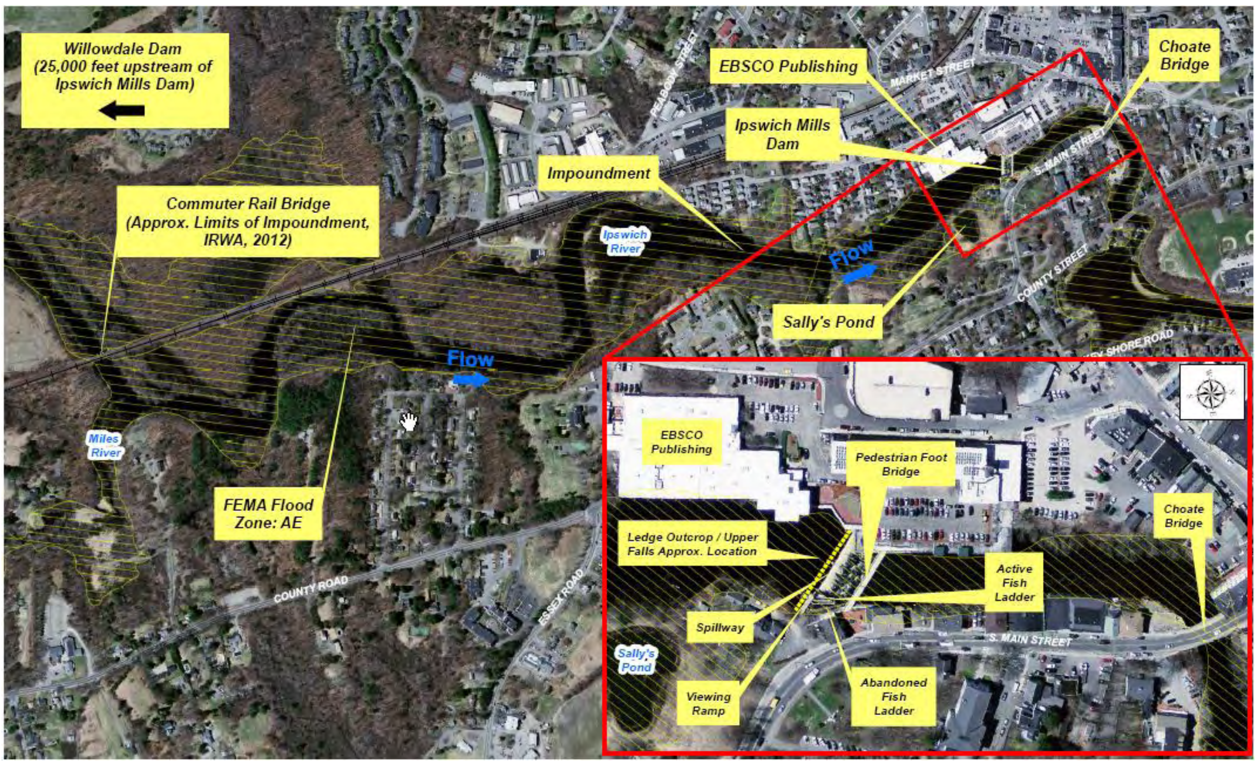

Figure 1-1. Key Project Area Features

As a head of the tide dam, the dam is the first man-made fish passage barrier encountered moving inland up the Ipswich River. Because the dam no longer serves an economic purpose, and because removal of the dam would improve fish passage to approximately 49 upstream river miles (DER Restoration Potential Tool, https://www.mass.gov/service-details/ders-restoration-potential-model-tool), removal of the dam is being evaluated as a potential option. The next dam upstream of Ipswich Mills is the Willowdale Dam where an improved fish ladder is slated to be constructed in 2019. An efficient fish ladder at Willowdale would add an additional 6 river miles of fish passage beyond the 49 associated with the Ipswich Mills dam. Further upstream, removal of the South Middleton dam is currently in the permitting stage. Removal of that dam would add an additional 57 miles of fish passage for a grand total of 112 miles. This report summarizes the methods and results of a Feasibility Study undertaken to better understand the factors favoring and disfavoring dam removal.

Prior to completing this current study, an initial Partial Feasibility Study was completed by HW in 2014 with assistance from a diverse team including Clean Soils Environmental, Ltd. (CSE), IF, Roux Associates, Alpha Analytical, IRWA, DER, NOAA, and Town departments. Funding was provided by the Conservation Law Foundation (CLF), NOAA, and DER. The 2014 Partial Feasibility Study is included herein as Attachment 1. Also in 2014, a bathmetric survey of the lower impoundment (from the dam to approximately 1,000 feet upstream) was completed by Norde-East Survey. That survey is included herein as Attachment 2.

Work on the current Project was organized into a series of primary tasks with a summary memorandum, or similar deliverable, submitted for each task. Those summary memoranda or deliverables are included herein as Attachments. The project tasks were as follows:

1. <u>Task 1 Existing Conditions Summary:</u> This task consisted of a summary of existing conditions including ecology, historical resources, and physical/ infrastructure conditions. Much of the information on existing ecological conditions was provided by the IRWA, based on its decades of in-house knowledge and experience, with input from IF and HW. Preliminary historical research and reporting were conducted by PAL. HW compiled existing data on physical conditions and conducted on-the-ground survey to create a basemap of existing conditions suitable for use in completing forthcoming design and other project goals. The Project basemap is included herein as Attachment 3 and the Task 1 Summary Memorandum is included herein as Attachment 4.

2. <u>Task 2 H&H Analysis:</u> This task included modeling of river flow under existing and dam-out conditions with a variety of different hydrologic scenarios in order to evaluate the potential hydraulic impacts of dam removal on key infrastructure, ecological functions, recreational uses, and other functions of the river. Hydrologic and hydraulic (H&H) modeling was primarily conducted by IF with input from HW. The Task 2 Summary Memorandum is included herein as Attachment 5.

3. <u>Task 3 Potential Impacts on EBSCO Facility:</u> This task included subsurface field investigations and a review of existing information to assess the potential impacts of dam removal on the structural stability of the EBSCO facility buildings. This work focused primarily on evaluations of the potential presence and, if present, significance of timber piling supports for the buildings and compressible soils beneath the buildings. This work was conducted by SGH. The Task 3 Summary Memorandum is included herein as Attachment 6.

4. Task 4 Potential impacts on Other Structures: This task assessed the potential hydraulic impacts of dam removal on other structures along the river besides the EBSCO facility. This task was not a structural assessment as in Task 3 but, instead, focused on how potential changes in river flow dynamics following dam removal might affect retaining walls and other structures from a standpoint of erosion or water level changes. An assessment of potential groundwater impacts to drinking water wells was also included. This task was led by HW with input from IF. The Task 4 Summary Memorandum is included herein as Attachment 7.
5. Task 5 Conceptual Plans and Cost Estimates: This task was effectively the culmination of the preceding tasks leading to the creation of conceptual plans for dam removal and river restoration. This task was led by IF with input from HW. The Task 5 Conceptual Plans and Cost Estimates are included herein as Attachment 8.
6. Task 6 Conceptual Renderings: This task built off of the conceptual plans to create visual representations of what the area might look like under dam-out conditions for the purpose of public information. This task was completed by HW. The Task 6 Conceptual Renderings are included herein as Attachment 9.
7. Task 7 Public Outreach: This task included public meetings to present the project to the public and solicit feedback, input, and ideas. This task was completed by HW and Project Partners with support from other members of the HW Team.
8. Task 8 Report: The Task 8 Project Report culminated in this document summarizing all the work conducted and findings resulting from the Project.

2.0 TASK 1 - EXISTING CONDITIONS SUMMARY

As more fully detailed in the Task 1 Existing Conditions summary memorandum (Attachment 4), Task 1 consisted of summarizing existing conditions including ecology, historical resources, and physical/ infrastructure conditions.

2.1 Existing Conditions Summary

Historical records show that a dam has existed in the vicinity of the Ipswich Mills Dam site since 1637 (Haley & Aldrich, 2009). Historic accounts indicate that the Ipswich Mills Dam was built upon or just downstream of a rock ledge outcrop or small rock rapids, referred to as the Upper Falls (Refer to Attachment 3 for further detail). The Lower Falls is a currently existing rock rapids section of the river (except for at high tide) located just below the County Road bridge (at the right edge of Figure 1-1). The elevation of bedrock ledge underlying surficial boulders at the dam site is currently uncertain. However, the following information suggests a potential approximate elevation of the bedrock:

- The hard surface of ledge and/or large boulders that has been observed spanning the width of the river approximately 10-20 feet upstream of the dam was observed during the field survey conducted by HW as part of the current study in August 2016 to consist, at least at the surface, of boulders, as opposed to bedrock ledge (Figure 2-1). The boulder surface is undulating but has an average elevation of approximately 6 feet (NAVD88). IRWA staff was able to use a steel pry bar to penetrate through the boulders to a consistent depth of refusal at approximately 3-5 feet; confirming the makeup of the surface of the feature as loose boulders, and also confirming that any bedrock ledge, if present, likely has a minimum elevation of roughly 1-3 feet (NAVD88).
- As part of the Task 3 structural assessment of the current project a test pit excavated in the river at the edge of the EBSCO building foundation, near the western edge of the dam, revealed bedrock at approximately elevation 3.2 feet (NAVD 88). This suggests that, at least near the western edge of the dam, bedrock ledge may be present in the general vicinity of the dam several feet below the elevation of the observed boulder surface.

Figure 2-1. Ipswich Mills Dam during August 2016 drawdown, left facing west, right facing east

In 1908, the dam was modified to its current structural design to supply nearby mill buildings (at the time) with a reliable source of power. The current dam is constructed out of cut stones with concrete at some locations, with the spillway extending across most of the width of the river. The main spillway is 132 feet wide. A three-foot-wide low level stop-log spillway is at the river-right end of the main spillway with an invert elevation approximately 0.4 feet lower than the spillway. Further to the right, the dam also has a 4.5-foot-wide by 3-foot-high low level gated outlet with an invert elevation approximately two feet lower than the main spillway. Further still to the right is a functional fish way that was installed in 1996 (IRWA). Furthest to the right is an abandoned fish ladder of older construction (Haley & Aldrich, 2009). According to a recent records review by IRWA, the dam is currently classified by the Office of Dam Safety (ODS) as a Low Hazard Dam, meaning that failure may cause minimal property damage to others and loss of life is not expected. The dam's condition was reported as "Fair" in 2009 but will be re-evaluated in 2019.

A run of the river dam provides minimal storage above it and is operated such that the volume of water released below the dam is equal to the volume of water flowing in the stream or river above the dam on a normal, continuous basis. Put another way, water is not stored in the impoundment to be released later. Rather, the dam simply increases the head in the river, providing a potential power source that can be captured. It does not serve to prevent or mitigate flooding downstream of the dam since it allows water to flow over the dam during most typical flows. The dam receives river flow contributions from a 148 square-mile watershed upstream of the dam that is made up of primarily forested land, wetland areas, residential properties, agricultural land, and some commercial/industrial zones. The soils in the watershed primarily include somewhat excessively or excessively drained, loamy and sandy soils that were formed in outwash deposits and well drained, loamy soils formed in glacial till (Fuller and Francis, 1984). The river flows nearly 40 miles from its headwaters in Wilmington and North Andover to its mouth in Plum Island Sound, dropping approximately 115 feet in elevation along its course.

The USGS maintains a flow and stage gauge on the Ipswich River located 200 feet downstream from the Willowdale Dam, or approximately 4.6 miles upstream of the Ipswich Mills Dam and has water surface elevation and discharge data from as far back as June 1930. The drainage area to the gauge is 125 square miles. Monthly mean flows at the Willowdale Dam between 1930 and 2009 range from 42.0 cubic feet per second (cfs) in August to 446 cubic feet per second in March. The highest flow on record (4,600 cfs) occurred on May 16, 2006. Two photographs of the Ipswich Mills Dam on May 16, 2006 are provided below showing that the dam is virtually drowned out by the discharge in the river (Figure 2-2). In the right-most photograph, water can be seen flowing just over the surface of the viewing platform on river right, which was surveyed during this current study to be at elevation 13.46 feet (NAVD88 datum). According to IRWA this photograph was taken at or near peak river stage at the dam.

Figure 2-2. Ipswich Mills Dam on May 16, 2006, facing southwest (left) and northwest (right) (photos by IRWA)

The second highest flow on record (3,950 cfs) at the Willowdale Gauge occurred on March 17, 2010. After that flood event the USGS visited watersheds throughout eastern Massachusetts and documented and surveyed high-water indicators (Zariello and Bent, 2011). The closest high-water indicator to the Ipswich Mills dam was at the pedestrian bridge immediately downstream and visible in the right-most photograph of Figure 2-2. The observed high-water indicator from the 2010 flood event at the pedestrian bridge was at elevation 13.52 (NAVD88), an elevation that correlates well with the Figure 2-2 photograph from 2006 and indicates that, perhaps, that photograph was taken a little before or after the peak river stage.

The potential for contamination in the sediments contained within the impoundment upstream of the dam was assessed by Clean Soils Environmental, Ltd., with assistance from IF and IRWA, in the Ipswich Mills Dam Partial Feasibility Study (HW, 2014). The preliminary sediment quality assessment opined that the sediments found behind the Ipswich Mills Dam have a very low likelihood of toxicity when viewed independently and in relation to other dams across Massachusetts. This opinion was based on the review of data from two sediment cores previously collected behind the dam by The U.S. Geological Survey (USGS) and three cores collected by IRWA and IF in 2012 as part of the Clean Soils Environmental preliminary assessment. Generally, data from both sampling events indicate that the sediment is below applicable ecological impact benchmark limits and that a condition of 'No Significant Risk' may exist from the sediment behind the dam. The concentrations of metals, SVOCs, pesticides, VOCs, and EPHs measured within the sediment appear to be consistent with surface water runoff from non-point sources (e.g., roadways and farming). More extensive sediment sampling and analyses conducted closer to the time of anticipated permitting will be required as part of the environmental permitting process if dam removal is further pursued.

2.2 Ecological Assessment

The ecology of the Ipswich River watershed is well studied. The IRWA, DER, and their municipal, state and federal partners have conducted regular monitoring of water quality (clarity, temperature, dissolved oxygen, flow and conductivity), herring passage at fish ladders, fish populations and macroinvertebrates. The overall health of the system has been summarized in various documents, including Bowling and Mackin (2003) and Armstrong et. al (1999). IRWA publishes strategic planning, best management plan (BMP) assessments, and annual reports documenting efforts in the watershed.

Historically, the Ipswich River watershed supported abundant fisheries resources including significant populations of diadromous (sea-run) fish. Diadromous fish species common in the Ipswich and its estuary included river herring (alewife and blueback herring), American shad, rainbow smelt, sea lamprey, Atlantic sturgeon and Atlantic salmon (Jerome et. al 1968). The River's first name, Agawam, a Native American term which translates to "place where fishes of passage resorted" speaks to the abundance of this former fishery (Jerome et. al 1968). Alewife spawning returns once numbered in the millions of fish, supporting a substantial commercial fishery.

Armstrong et. al (1999) showed that over 90 percent of fish in the Ipswich River are generalists tolerant of lentic or lake conditions, whereas the historic native fishery was composed of lotic or riverine species requiring flowing water to thrive. The dam maintains a consistent water level in the impoundment that extends over a mile upstream from the dam and produces unnatural lentic habitat conditions in this reach of the river, favoring habitat generalist fish species and pond-like invertebrate communities. River herring runs are monitored yearly, but totals are now typically less than 1,000 spawners per year. Purinton et. al (2003) estimated that the Ipswich River is currently supporting less than 1% of its total spawning potential. Belding & Corwin (1921) blamed alewife decline in the Ipswich on a number of factors, but primarily on the combined influence of conversion of historic spawning ponds (e.g. Wenham Lake and Suntaug Lake) to water supply use, and to obstruction of migration pathways by dams. A 2004 Massachusetts Division of Marine Fisheries (DMF) report notes that the Ipswich River likely has good restoration potential for river-spawning anadromous fish such as American shad and blueback herring (Reback et.al 2004).

The Ipswich Mills Dam is located at the head of tide (upstream limit of tidal influence) roughly 3.7 miles from the Ipswich River's mouth. In addition to limiting many migratory fish species from moving upstream into the watershed to spawn or feed, the dam also presents a problem to freshwater resident species that pass over the dam for one reason or another, including many freshwater fish, turtles and other species that cannot survive long-term below the dam. With the exception of wildlife that are strong swimmers or good climbers many of these animals are likely to be permanently trapped below the dam.

The area in and around the current impoundment supports abundant wildlife populations. Semi-aquatic animals commonly seen in the water and the riparian areas include mammals (e.g. beaver, muskrat, river otter), birds (e.g. blue heron, wood duck, mallard duck, kingfisher, Canada goose), and reptiles (e.g. painted turtle, musk turtle, snapping turtle). The impoundment also has considerable populations of unionid freshwater mussels. Rare animal species (including endangered, threatened, special concern and watch list) that have been documented in the Ipswich River Watershed include bridle shiner, piping plover, least tern, least bittern, golden-winged warbler, pied-billed grebe, Cooper's hawk, northern harrier, salamanders (spotted, blue-spotted, marbled and four-toed), eastern pond mussel, box turtles (spotted, Blandings and eastern), and a number of invertebrates.

Restoration of sizable populations of diadromous fish to the Ipswich River Watershed would have ecosystem-wide importance. Large spawning runs of anadromous species such as river herring and shad bring large influxes of marine-derived food and nutrients to the freshwater system. They are also important as a forage fish, serving as prey for numerous piscivorous predators while at sea (e.g., tuna, cod, dolphins, billfish, gannets), in estuaries (e.g., striped bass, bluefish, weakfish, harbor seals, cormorants), and in rivers (e.g., ospreys, white perch, herons, river otters). The current low populations of diadromous forage species have important implications throughout marine and freshwater food webs.

2.3 Potential Ecological Impacts from Dam Removal

One possible short-term impact from the removal of the dam is the release of mobile sediment that has accumulated behind it. Downstream sediment transport is a natural riverine process. That natural process is altered by the presence of dammed impoundments which tend to capture and accumulate sediment migrating from upstream sources while thereby depriving downstream areas of the sediment supply needed to support a vibrant riverine ecology. Following dam removal, there tends to be an accelerated process of removing sediment from the impounded area and redistributing it to downstream areas. In time, a new equilibrium is reached that reflects the river's hydraulics and sediment dynamics post dam-removal.

While sessile communities (e.g. some invertebrates) can suffer significant impacts downstream of dam removals, fish are able to move upstream or downstream of the impact zone and thus avoid many of the negative impacts. Fish species can respond quickly to the increases in turbidity, bedload and temperature following dam removals. Timing the Ipswich Mills Dam removal to begin releasing sediment well ahead of fish migration periods would also help to minimize impacts. Temporary deposition may occur above the Choate and County Road bridges before larger storm flows continue to flush that sediment downstream. More significant deposition may occur in the Great Cove immediately downstream of County Street (approximately 0.3 mile downstream of the dam) where the channel is artificially widened, and flow velocities decreased. The tide will help to move sediment through the system, eventually delivering it to the estuary, helping to build and sustain the estuary.

Wetland delineation by the Massachusetts Department of Environmental Protection (Mass DEP, 2009) shows areas of deep marsh, shallow marsh, wooded swamp, and shrub swamp bordering the main channel through the impounded reach upstream of Ipswich Mills Dam. In the longer-term following dam removal, normal water levels will fall, and it is likely that some of the shallow water wetland areas will evolve into a different type of wetland, or potentially also upland habitat at the highest elevations. Areas currently shown as deep marsh and existing backwater areas are likely to remain as shallow water wetland habitat. Given that these areas are anticipated to experience cyclical water level fluctuations as a result of downstream tidal fluctuations (though the potential extent of actual salt water encroachment remains unknown), the resulting wetlands may be characterized as tidal freshwater wetlands, one of the rarest wetlands habitats in Massachusetts. These wetlands would be capable of supporting rare freshwater plant species currently uncommon in the Ipswich River watershed, or other nearby areas.

For typical small dams, removal results in the restoration of a river's natural water temperature regime through the former impoundment area and downstream of the dam (e.g., Pawloski and Cook 1993). Removal of the dam will encourage active flow and help reduce water temperatures, making this part of the river more hospitable to flow dependent and fluvial fish species. Removal of the dam will also allow free movement of motile aquatic organisms past the dam site to take advantage of food resources and to escape periodic, unsuitable conditions in currently impounded area.

In general, following dam removal, overall lotic macroinvertebrate abundance and diversity tends to increase relative to that of impoundment communities as a new channel is formed and more heterogeneous in-channel habitat becomes available for both invertebrates and fish (Bushaw-Newton et al. 2002, Calaman and Ferreri 2002, Pollard and Reed-Anderson 2001). Such a change is anticipated following removal of the Ipswich Mills Dam. Restoration of sediment continuity through this reach would be beneficial over the long term, not only for restoring habitat locally, but also for replenishment of sediment in the estuary downstream.

2.4 Cultural Resources Summary

A Historical and Cultural Resources Summary was completed by PAL and is attached herein as Appendix A to the Attachment 3 Task 1 Summary Memorandum. The PAL report summarizes what is known about the pre-and post-settlement history of the dam site based upon information obtained from the Ipswich Historical Commission, the Massachusetts Historical Commission (MHC), and other sources. It contains the following information: identification of historic properties and previously surveyed archaeological and architectural resources within and immediately adjacent to the Project area; cultural context relating the pre-history and history of the dam site including former dams and their date(s) of construction; and recommendations concerning potential impacts to cultural resources or additional cultural resources survey efforts that may be needed if the project proceeds into design and permitting.

The PAL report identifies a rich cultural history for the downtown Ipswich area near the dam site beginning with multiple pre-contact Native American cultures and proceeding through colonial era European activity, early American history, the Industrial Revolution, and the modern period. While highly informative, the PAL report completed as part of the current project is not a formal historical review that may be required if the project moves further towards construction. It does, however, provide an initial understanding of the bigger picture cultural and historical resources likely important to the project, as well as a foundation for future coordination between the dam owner, state and federal permitting agencies, and the MHC, should the project progress into design and permitting.

2.5 Project Basemap

With assistance from IRWA, the Town, and DER, HW compiled existing information around the Project area on topography, bathymetry, wetlands, hydrography, structures, utilities, roads, and other infrastructure relevant to the assessment of potential Ipswich Mills Dam removal. HW then supplemented these existing data by field-surveying 26 transects across the river, physical attributes of the dam and four bridges, other key infrastructure, and hydrologic indicators. These GIS data, HW field survey data, and prior bathymetric survey data of the impoundment conducted by Norde-East, Inc. in 2014 were all converted to common vertical (NAVD 88, feet) and horizontal (NAD 83, feet) datums and compiled into a single project basemap. It is a four-sheet set in 24X36-inch page size format that includes a large scale view of the entire (approximately five river mile) project area from above the railroad bridge down to below the Lower Falls, an intermediate scale view of the key project area from the impoundment down to the Choate Bridge, a zoomed-in view of the immediate dam area, and a longitudinal profile along the entire project area. The basemap is included herein as Attachment 3 and reduced to a 11x17 page size.

2.6 Tidal Data

In order to document existing tidal conditions immediately above and below the dam, HW collected continuous water level data at a six-minute interval immediately upstream and downstream of the dam from September 7th to November 6th, 2016. To complement water level data at the dam site, HW also obtained continuous water level data collected by the Plum Island Ecosystems (PIE) Long Term Estuarine Research (LTER) from the Ipswich Yacht Club in Plum Island Sound near the mouth of the Ipswich River. All water level data were corrected to the NAVD88 vertical datum based on the HW survey and information from PIE. Figure 2-3, below, compiles water level data from all three locations, and also includes National Weather Service precipitation data from the Beverly Airport. Figure 2-4 depicts a closer scale view of the spring tide period between October 13th and 23^{d}, 2016. The following are some key observations regarding the water level data.

- Water levels were recorded while the river was still recovering from the 2016 drought and the August 2016 drawdown. Water levels above the dam illustrate a slow rise as the river responds to smaller precipitation events in September and quicker responses to two larger precipitation events in early to middle October. Water levels below the dam are dominated by tidal hydraulic influence and show only a moderate increase beginning with the largest precipitation event in early October.
- No tidal response can be observed above the dam as the dam crest is above maximum tidal elevation. There is no observed periodicity of rising and falling water levels above the dam in response to rising and falling tidal levels below the dam.
- When fresh water river flows are high (upstream river stage above 8 feet) and the tidal cycle is in a neap period, river flow overwhelms the tidal influence of those lowest high tides below the dam. The converse is true for periods of low river flow and spring high tides. Due to the drought conditions prevalent for most of the monitoring period, no data are available to inform the relative influence of river flow versus tides below the dam for high flows and spring tides. According to IRWA, hydraulic responses to spring high tides are observed at the toe of the dam even during these higher river flow periods, though not at flood events.
- The water level is always a little higher at the toe of the dam than it is downstream at the yacht club. This is due to outgoing river flow riding on top of the incoming tide (for

the higher high tides), or due to the outgoing river flow simply being held back by the hydraulic influence of the incoming tides (for the lower high tides).

- The rock pile below the dam retains water in a pool at the toe of the dam even during outgoing tides.
- There is about an hour time lag between the high tide at the yacht club and at the toe of the dam.
- High tides at the toe of the dam reach to between approximate elevations 4 to 7 feet. These are all above the estimated bedrock controlling elevation in a potential dam-out scenario. Therefore, in a potential dam-out scenario, it appears that high tides would exert hydraulic influence above the current dam location. The available data are not sufficient to predict whether any saline water would reach above the current dam site. According to IRWA, salt water has rarely been detected above the Lower Falls in the dozens of water samples collected by the Division of Marine Fisheries (DMF) over the years, and only reaches to the dam site for spring high tides that occur during periods of low river flow. Given that reported limited presence of saline water at the toe of the dam, and the preference for higher density salt water to remain low and fresh water to sit above it, the likelihood of significant amounts of saline water reaching above the dam appears low but may require further study if warranted.

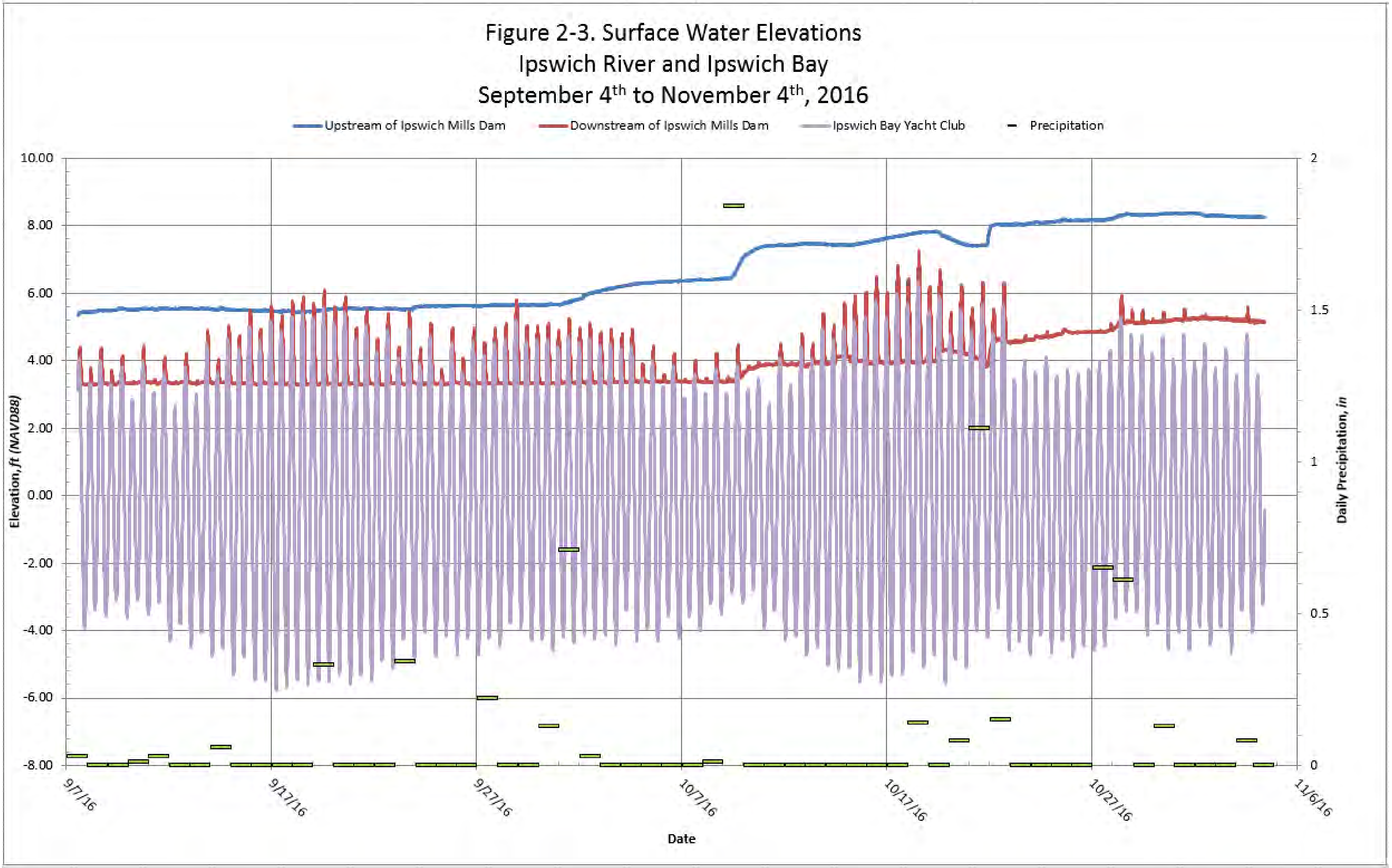
Figure 2-3. Surface Water Elevations
Ipswich River and Ipswich Bay
September 4th to November 4th, 2016
Upstream of Ipswich Mills Dam
Downstream of Ipswich Mills Dam
Ipswich Bay Yacht Club
Precipitation
Elevation, ft (NAVD88)
Daily Precipitation, in
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
-4.00
-6.00
-8.00
2
1.5
1
0.5
0
9/7/16
9/17/16
9/27/16
10/7/16
10/17/16
10/27/16
11/6/16
Date

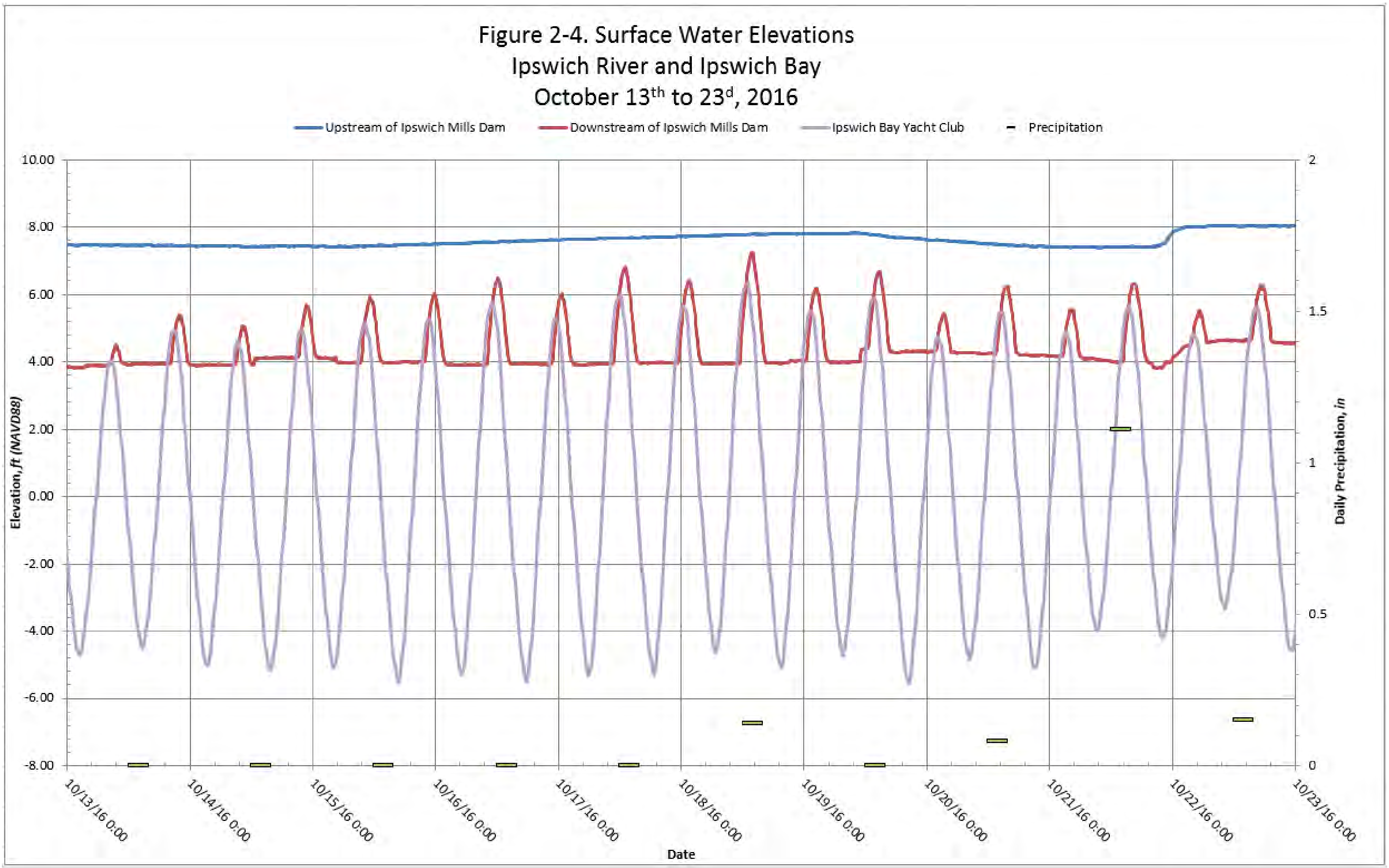
Figure 2-4. Surface Water Elevations
Ipswich River and Ipswich Bay
October 13th to 23d, 2016
Upstream of Ipswich Mills Dam
Downstream of Ipswich Mills Dam
Ipswich Bay Yacht Club
Precipitation
Elevation, ft (NAVD88)
Daily Precipitation, in
Date
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
-4.00
-6.00
-8.00
2
1.5
1
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3.0 TASK 2 – HYDROLOGIC & HYDRAULIC ANALYSIS

Task 2 consisted primarily of modeling the hydrology and hydraulics (H&H) of the subject stretch of the river under existing and potential dam-out conditions. Hydrology, in this context, refers to the conveyance of precipitation-derived water from the watershed into the river under different storm events; while hydraulics refers to the flow characteristics of the river resulting from those hydrologic inputs under the same set of various storm conditions. This task was led by IF with input from HW. The Task 2 summary memorandum is included herein as Attachment 5.

3.1 Evaluation of USGS Willowdale Gauge Data

The nearest USGS flow gage (ID 01102000, Ipswich River near Ipswich, MA) is located approximately 200 feet downstream of the Willowdale Dam and 4.6 miles upstream of the Ipswich Mills Dam (https://waterdata.usgs.gov). The drainage area to the gage is 125 square miles. Instantaneous annual peak flows and daily average flows were downloaded for the period of record (1930-present). Annual peak flow rates are plotted in Figure 3-1.

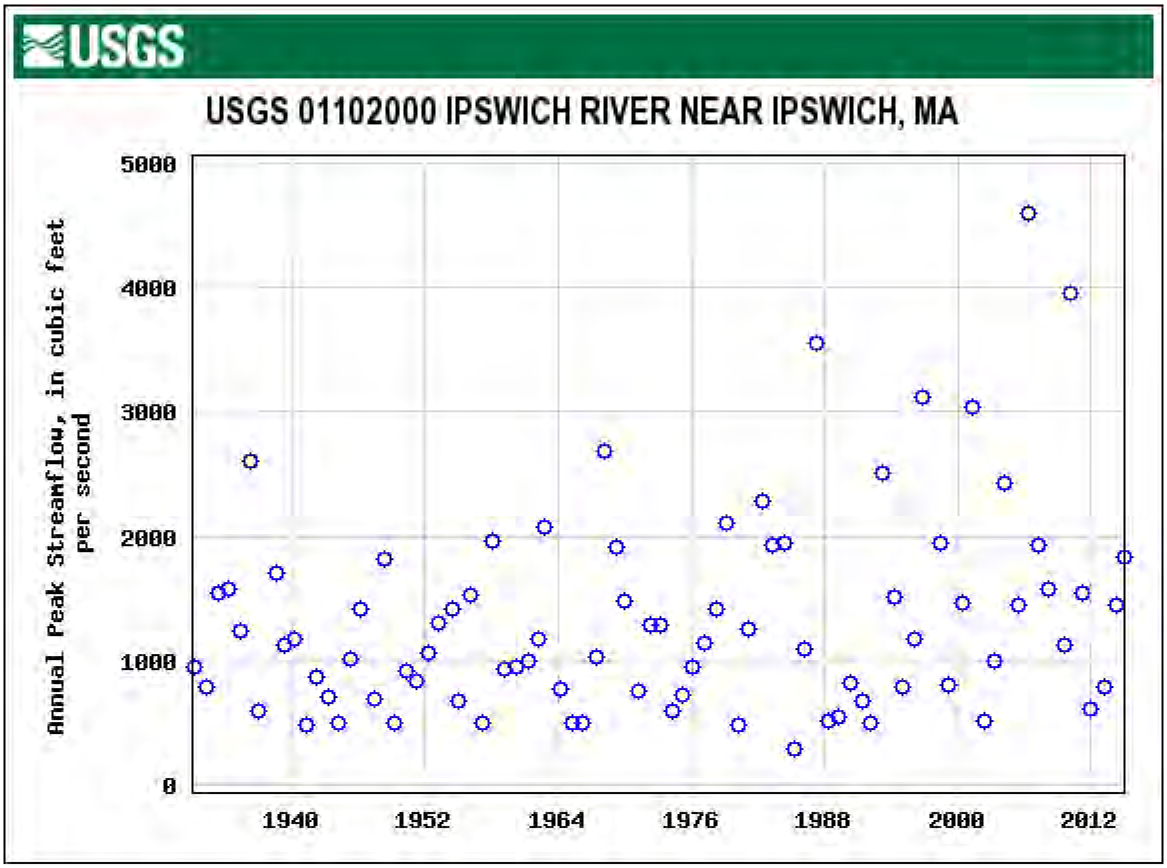


Figure 3-1. Instantaneous annual peak flows at USGS Willowdale Gauge

Analysis of the flow data from 1930 to present suggests a trend of increasing magnitude of runoff events since around 1970, with the highest flows on record occurring after 1980. The flood record at Ipswich is consistent with observations from across New England that indicate a climatic shift towards increased magnitude and frequency of floods (Collins, 2009; Armstrong et al., 2011). Changes since around 1970 may also reflect in part land cover changes and/or upstream flow management. Table 3-1 lists the estimated flood flows for the dam site based on the full flood record, the record since 1970, and a comparison with the latest flood insurance study (FIS; FEMA, 1985). The period of record for the 1985 FIS did not include the significant flood events experienced since the 1980s.

Table 3-1. Peak discharges for a range of recurrence intervals at Ipswich Mills Dam

Recurrence Interval (years)	2	10	25	50	100	200	500
1930-present (cfs)	1,324	2,824	3,791	4,609	5,514	6,514	8,003
1970-present (cfs)	1,439	3,316	4,569	5,644	6,846	8,187	10,203
FIS values (Ipswich River at Central St/ Choate Bridge)	-	2,023	-	3,016	3,251	-	4,196

In order to provide the most conservative results, the peak discharges calculated from the post-1970 dataset were used in the hydraulic model.

3.2 Model Development

A one-dimensional, mixed, steady-state flow model was developed using the U.S. Army Corps of Engineers (USACE) Hydraulic Engineering Center River Analysis System (HEC-RAS, v. 5.0.3) to investigate the potential hydraulic implications of removing the Ipswich Mills Dam. One dimensional HEC-RAS models are well-suited for situations like the current study where hydraulic changes occur predominantly in one-dimension (i.e. from upstream to downstream along the centerline of the channel. Two and three (adding vertical variance) dimensional models are more complex and, require significantly more input data, as well as more advanced modeling software.

Cross sections for input into HEC-RAS were compiled from several sources. Bathymetric data for the lower impoundment was collected by Norde-East Survey in August 2014 and five cross sections were defined through the lower impoundment based on those data. Surveyed cross-section data was collected by HW in August and September 2016, and in April 2018. A total of 26 channel and bridge cross sections were surveyed from a location approximately 1,100 feet upstream of the railroad bridge, downstream through an area called “lower falls” located just downstream of the County St. Bridge where a bedrock outcrop forms the river bed and provides downstream grade control. Additionally, a cross section surveyed by IRWA in 2013 approximately 10 feet upstream of the dam was incorporated into the model. Finally, a cross section was included immediately downstream of the dam to define the scour pool present at the foot of the dam. All cross sections were extended across the floodplain using available LiDAR data (US Geological Survey North East Project 2011 LiDAR, 1m grid resolution) (Figure 3-2).

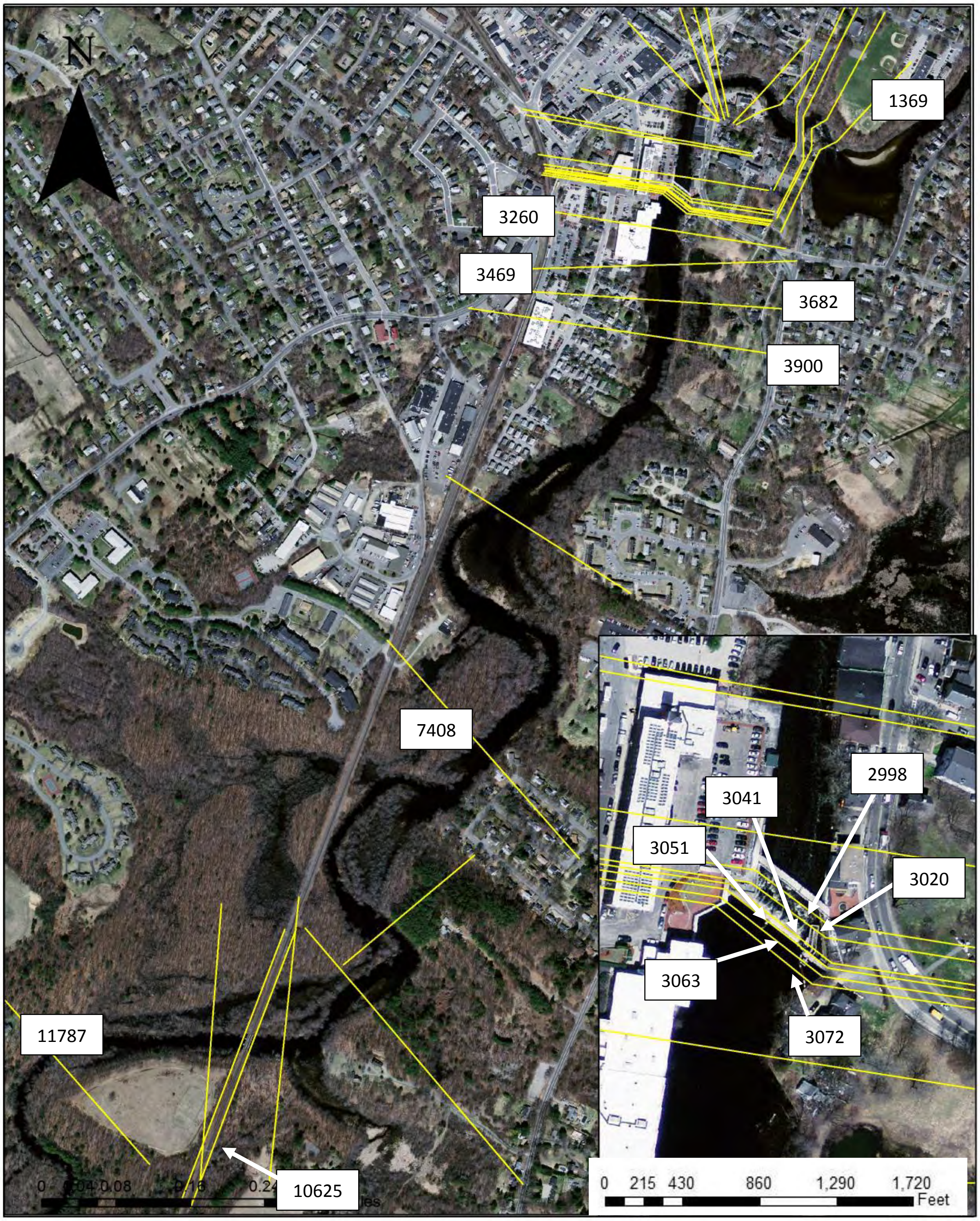


Figure 3-2. Map showing HEC-RAS cross section locations with inset of area around dam and selected cross section stations labeled for reference (Source: USGS color ortho imagery, 2009)

The downstream boundary condition of the HEC-RAS model is represented by tidal conditions estimated by scaling short-term tidal data from the Ipswich Bay Yacht Club at the mouth of the river against long-term data from the National Oceanic and Atmospheric Administration (NOAA) gauge in Boston Harbor. Scaling was accomplished by conducting a regression analysis of September 2016 tidal data (a month with tidal elevations typical to those occurring on average throughout the year) for the Yacht Club and NOAA stations. The regression relationship between the two stations was applied to the published long-term datums from the NOAA station in order to estimate the values of those same datums for the Yacht Club. The relevant tidal data used in the model is shown in Table 3-2.

Table 3-2. Tidal datums and exceedance probability tide levels in feet relative to NAVD88

	Boston, MA	Scaled to Ipswich Bay Yacht Club
1% exceedance probability stillwater tide level (FEMA, 1985)	N/A	8.7
1% exceedance probability	9.58	8.45
10% exceedance probability	8.46	7.49
50% exceedance probability	7.64	6.78
Mean Higher High Water (MHHW)	4.77	4.32
Mean High Water (MHW)	4.52	4.10
Mean Low Water (MLW)	-4.96	-4.55

The existing conditions model was compared to field-observed mean high-water markers, to the 1985 FEMA FIS, and to the high-water indicator from the March 2010 flood (Zariello and Bent, 2011) to validate its general accuracy for simulating existing conditions. The model simulated comparable water levels to all three water level comparison indicators, based on their respective flow conditions. The primary differences between the existing conditions model developed for this study and the FEMA model are: (1) updated channel geometry data based on recent survey, and (2) inclusion of the channel and bridge structures downstream of the dam in the hydraulic model. Results show that upstream of the dam, the simulated flood levels provide a reasonable match to the reported FEMA flood levels. Downstream of the dam, FEMA reported stillwater tide levels only and did not simulate flow through these reaches and bridge structures. This explains the discrepancy between the model results for this study and FEMA results. Over the course of many studies, it has been shown that differences should be expected between simulation results from coarsely resolved, older FEMA studies (in this case over 30 years), and more highly resolved, current, project-scale models. The comparison of results in this study are consistent with trends seen on many other rivers and many similar projects.

The existing conditions model was then modified to create a dam-out conditions model. The dam-out model included the following:

- Removal of the full vertical height and lateral extent of the dam and associated structures where possible;
- Limited channel restoration or modifications to other river infrastructure; and

- A conservative approach to defining the post-removal bed surface at the dam site in order to gain an understanding of worst-case risk to upstream and downstream infrastructure in terms of effects on hydraulic conditions.

This scenario involves leaving the abandoned fishway integral with the existing river in place in order to avoid destabilizing the river right wall (looking downstream) during demolition. An approximate 10-foot section of the existing viewing platform would be retained as a part of this minimum measure to protect the river right wall (Figure 3-3). All other elements of the dam would be removed to the full vertical extent.

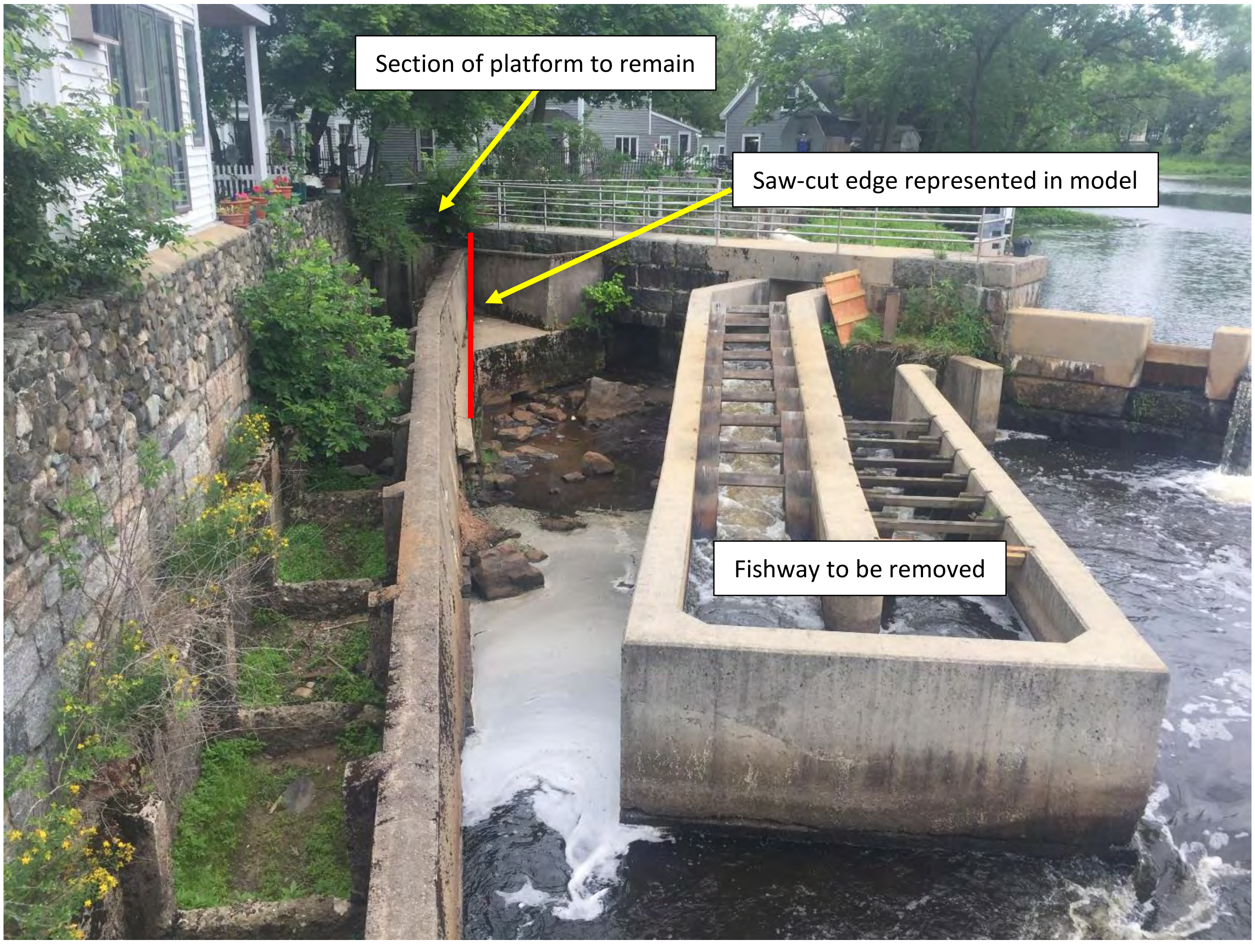


Figure 3-3. Modeled limits of dam removal on river right

Because of the uncertainty concerning the elevation of bedrock at the dam site, the dam-out model has taken a conservative (i.e., safe) approach from the perspective of infrastructure risk by assuming any bedrock is present at a low enough elevation so that the long-term bed profile will align with the average upstream and downstream bed profiles. This means that the proposed bed profiles are modeled at their likely lowest possible elevation, thus presenting the highest possible "risk" to upstream infrastructure. Further investigation will be required to clarify the presence or absence and elevation of bedrock at the site. Figure 3-4 shows the dam-out condition represented by the model as compared with existing conditions.

The dam-out conditions model geometry includes removal of the dam, filling of the scour pool below the dam, removal of the fishway from the cross sections immediately downstream of the dam, lowering the bed levels at Sta 3072 (21 feet upstream of former dam) by 3.04 feet to reflect mechanical regrading or removal of the accumulated rock immediately behind the dam, and lowering the bed levels at Sta 3020 (between dam and pedestrian bridge) by 1.39 feet to depict mechanical regrading of accumulated bed material downstream of the scour pool at the base of the dam. Bed levels elsewhere were left unchanged.

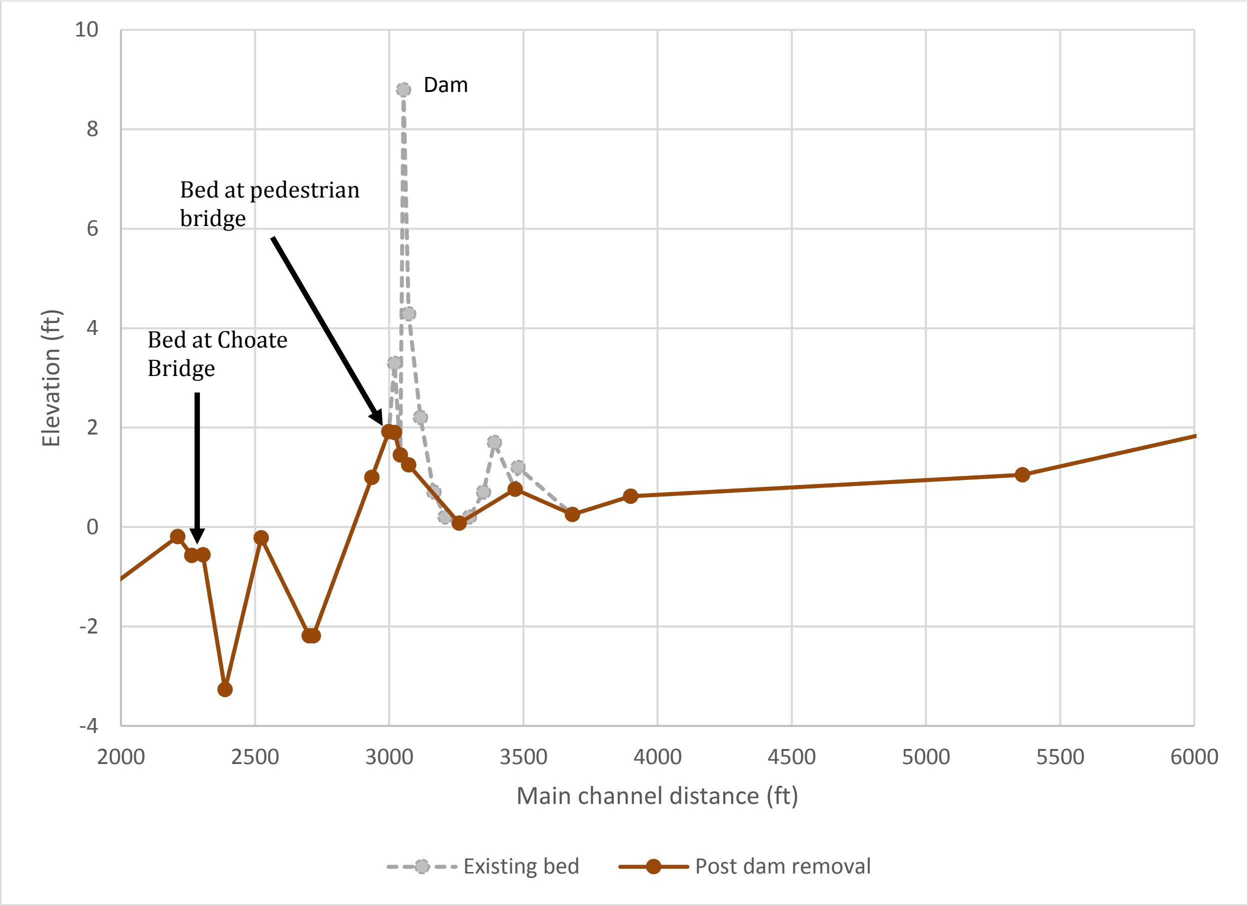


Figure 3-4. Comparison of existing and modeled long-term longitudinal channel bed profiles. Flow is from right to left.

3.3 Model Simulations

Table 3-3 summarizes the various combinations of flow and downstream boundary conditions used in the modeling to investigate flood risk impacts, channel stability, and hydraulics during high flows and fish passage conditions. The scenarios examined for impacts to flood risk combine high tides with flood flows to provide worst-case inundation extents and depths. The scenarios examined for impacts to structures consider hydraulic conditions with the greatest scour potential (i.e. highest velocities and shear stresses). These conditions are most likely to occur during high flows when the tide is out (i.e., low tide). For tide-out scenarios, normal flow depth was selected as the downstream boundary condition because MLW, as shown in Table 3-2, is below bed level at the downstream limit of the model (base of lower falls).

Table 3-3. Combinations of flow and downstream boundary condition for model runs[1]

River Flow Recurrence Interval (years)	Purpose of Run	Flow (cfs)	Downstream Boundary Condition
100	Flood risk	6,846	8.7 feet stillwater tide level
100	Flood risk	6,846	4.10 feet MHW tide
50	Flood risk	5,644	4.10 feet MHW tide
25	Flood risk	4,569	4.10 feet MHW tide
10	Flood risk	3,316	4.10 feet MHW tide
2	Flood risk	1,439	4.10 feet MHW tide
100	Channel stability/ hydraulics	6,846	Normal flow depth
25	Channel stability/ hydraulics	4,569	Normal flow depth
2	Channel stability/ hydraulics	1,439	Normal flow depth
95% exceedance (daily flow series)	Tidal influence	47	4.10 feet MHW tide
Fish passage flow – 5% exceedance (daily flow series)	Fish passage	1,142	Normal flow depth
Fish passage flow – 50% exceedance (daily flow series)	Fish passage	288	Normal flow depth
Fish passage flow – 95% exceedance (daily flow series)	Fish passage	47	Normal flow depth

3.4 Model Results

Sediment Results:

One potential short-term impact of dam removal is the release of sediment that has accumulated behind the structure. The retention of coarse sediment behind the dam has resulted in downstream areas that are sediment deprived. During this transitional period,

[1] Exceedance flows calculated over the period March through June

softer/more mobile sediments currently retained behind the dam will migrate downstream, begin to fill in voids in currently sediment deprived locations, and continue to migrate downstream until they are deposited in locations where the flow energy regime is supportive of deposition (e.g. the Great Cove downstream of County Street and approximately 0.3 mile downstream of the dam where the channel is artificially widened and flow velocities reduced). This process will continue over the transitional period until the river's sediment dynamics approach equilibrium with the post-dam flow energy dynamics.

Depth-of-refusal survey data suggests that a relatively small volume of sediment is present within the lower impoundment in the vicinity of the dam. Depths of accumulation along the channel thalweg are minimal with most of the material stored along the margins of the impoundment are partially vegetated. Further upstream, there appears to be potentially mobile sediment stored along the bed of the channel, but depths and sediment volume are unknown at this time and may require additional investigation. Construction activity and breaching of the dam will mobilize some fine organic and inorganic sediment, which will be held in suspension resulting in short term/temporary and occasional increased turbidity downstream of the dam. The existing rock scour protection at the railroad bridge represents the likely upstream limit of sediment mobilization since it will likely act as a grade control feature following dam removal. Fine sediment that is released as a result of dam removal is likely to be dispersed by fluvial flows and tidal fluctuations in the downstream channel. The spatial and temporal scale of the impacts will depend on the volume of material and how rapidly it is released.

Coarse sediment released from upstream may potentially accumulate temporarily at the Choate Bridge downstream of the dam site where modeling suggests the structure currently restricts flow during large magnitude events. Model results indicate that bed shear stress conditions at the bridge are sufficient to transport gravel up to cobbles over the range of flows investigated; therefore, the effects are likely to be temporary with material transported past the bridge during subsequent high flows.

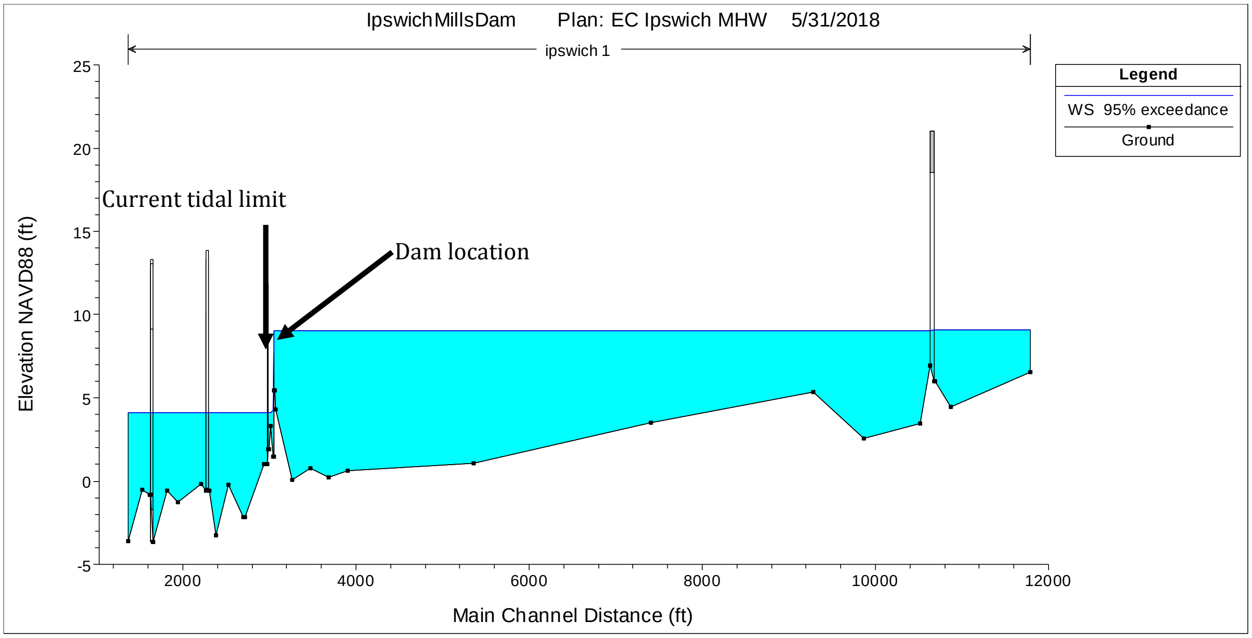
The impact on aquatic species depends on the concentration, exposure time, and time of year. Sessile communities are more susceptible to sediment impacts than fish which can adjust quickly to changes in turbidity and bedload. Further investigation into the volume of fine sediment stored over the whole length of the impoundment is necessary before short-term impacts can be fully assessed. Timing the Ipswich Mills Dam removal so that sediment is released well ahead of fish migration periods will help to minimize impacts to migratory fish. It is recommended that potential impacts associated with deposition downstream of the dam are monitored following dam removal.

Over the long term, sediment eroded from newly exposed banks, the bed, or supplied by headcutting and bank erosion along tributaries may be transported downstream. Restoration of a more consistent sediment transport regime will be beneficial over the long term not only for restoring habitat locally but also for replenishment of sediment in downstream reaches currently sediment-starved and artificially dominated by cobble-sized material.

Tidal Influence:

A combination of high tide (MHW) and low flow (95% exceedance) under existing and dam-out conditions were simulated to examine the impact of dam removal on the extent of hydraulic tidal influence on river conditions. Comparison of the water surface profiles in Figure 3-5 shows that in the absence of the dam, the hydraulic tidal influence is predicted to extend upstream to near Upper River Road, or approximately 4,350 feet upstream of the existing dam and current tidal limit.

Dam removal will also impact the range of tidal freshwater wetlands, important rare wetlands formed near the limits of the tidal range. Because fresh water is less dense than salt water, fresh water tends to flow on top of salt water as the salt water moves upstream with an incoming tide. The water surface elevation rises and falls with the tide, but the river banks and vegetation community interact primarily with the portion of the water column that is fresh water. The mixing dynamics within the tidal range of the Ipswich River are unknown; however, with the dam removed and the range of tidal influence increased, tidal freshwater wetlands may be able to expand their range within the study area.



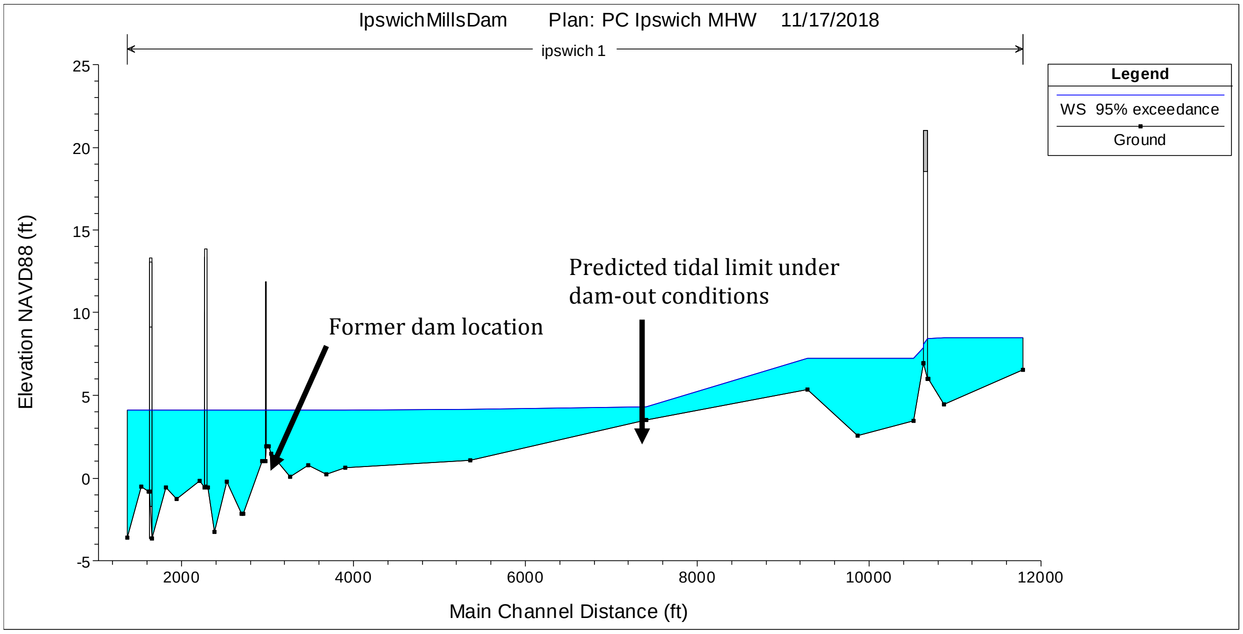


Figure 3-5. Predicted low-flow water surface profiles for existing (top) and dam-out (bottom) conditions during high tide

Flood Risk:

As shown in Table 3-4, flood levels under dam-out conditions are simulated to decline relative to existing conditions throughout the impoundment upstream to the railroad bridge. Rip rap scour protection beneath the bridge and conveyance capacity through the bridge section appear to control flood levels upstream of the railroad bridge. During the 100-year event, backwater from the Choate Bridge affects water surface profiles upstream through the dam site in both the existing and dam-out scenarios, resulting in very little predicted change in the flood profile following dam removal. The results show a slight, localized increase in water levels between the dam and pedestrian bridge for both the 2-year and 100-year events, which likely reflects a modeled change from rapidly varied, turbulent flow conditions under existing conditions to smoother, more stable flow conditions with the dam removed. In actuality, the ability of the one-dimensional model to accurately predict precise water surface elevations in areas of rapidly varied flow (existing conditions case) is limited, so this simulated change should be considered within the overall uncertainty of the modeling itself and does not necessarily mean that water levels would increase in reality. No changes in water surface elevations are predicted downstream of the pedestrian bridge for the scenarios tested. Flood profiles for existing and dam-out conditions are provided in Appendix B and inundation maps in Appendix C of Attachment 5 of this report.

Fish Passage:

Model results indicate that predicted water surface profiles and flow velocities through the former dam location during low flows will be favorable to fish passage (Table 3-5). The flows modeled were calculated by taking into account records over the entire migration period from March through June. Predicted average flow velocities are less than 4 ft/s, and maximum flow depths are greater than 0.5 feet at all of the cross sections in the immediately vicinity of the dam removal. At high tide, tide levels will extend past the former dam location, as discussed previously, and no issues with fish passage are therefore anticipated. The effects of dam removal on low flow water surface profiles are predicted to extend upstream to the railroad bridge. Compared with existing conditions, water depths are shallower and average flow velocities are predicted to increase, particularly immediately downstream of the railroad bridge as flow passes over the high spot beneath the bridge where rip rap has been placed on the bed of the channel. Modeling indicates turbulent flow at this location with a hydraulic jump forming immediately downstream as flow transitions back to smoother, more stable conditions. The results suggest that depending on the design, dam removal may make fish passage conditions more challenging at the railroad bridge than they are at present. However, irregularities in the rock bed at the bridge may provide diverse flow conditions and opportunities for passage over this relatively short distance. It is recommended that fish passage conditions continue to be evaluated and optimized at this location as the project moves into later stages of design.

Ecology:

Following dam removal, over the long term, normal water levels will fall, and it is likely that shallow water wetland areas will evolve into a different type of wetland or upland habitat. Areas that are currently deep marsh are likely to become shallow water wetland habitat. Vegetation cover and succession upstream of the dam will likely be affected by the increased tidal range upstream of the former dam location. For typical small dams, removal results in a long-term decline in water temperatures through the former impoundment area and downstream of the dam (e.g., Pawloski and Cook, 1993). The narrowed cross section and increased velocity through the former impoundment area would result in reduced residence time from the ponded condition. This combination equates to cooler temperatures capable of supporting higher dissolved oxygen concentrations resembling conditions of the stream upstream of the dam's influence (Zaidel, 2018). Decreased post-dam removal water temperatures favor those stream fishes adapted to cool or coldwater environments (Born et al., 1998). Removal of the dam will encourage active flow and help reduce water temperatures, making this part of the river more hospitable to fluvial fish species. Removal of the dam will also facilitate movement of other aquatic cool-water organisms past the dam site. Turtles, resident freshwater fish, and other aquatic organisms will have improved movement with the dam removed. As described earlier, the rare tidal freshwater wetland range may be able to be expanded following the dam removal.

Table 3-4. Predicted flood water elevations for existing and dam-out conditions (ft NAVD88)

	100-year flow and 8.7 ft stillwater tide		100-year flow and 4.10 ft MHW tide		2-year flow and 4.10 ft MHW tide	
River station	Existing (ft)	Dam Removed (ft)	Existing (ft)	Dam Removed (ft)	Existing (ft)	Dam Removed (ft)
11787	21.29	21.29	21.29	21.29	13.00	13.00
10867	21.31	21.31	21.31	21.31	12.79	12.79
10689	20.58	20.58	20.58	20.58	12.33	12.32
10657	Railroad bridge					
10625	15.79	15.69	15.71	15.59	11.85	9.75
10513	16.81	16.74	16.76	16.67	12.04	10.95
9865	16.70	16.62	16.64	16.55	11.99	10.84
9283	16.44	16.35	16.37	16.27	11.89	10.62
7408	15.79	15.67	15.69	15.56	11.59	9.44
5359	15.32	15.18	15.21	15.04	11.44	8.27
3900	14.82	14.66	14.69	14.49	11.34	7.11
3682	14.77	14.60	14.64	14.43	11.33	6.97
3469	14.73	14.56	14.59	14.38	11.31	6.68
3260 (EBSCO)	14.66	14.48	14.51	14.29	11.30	6.54
3072	14.42	14.41	14.24	14.21	11.24	6.43
3063	14.39	-	14.19	-	11.22	-
3051 (Dam)	14.39	-	14.19	-	11.22	-
3041	14.38	14.35	14.19	14.15	7.03	6.34
3020	14.29	14.33	14.07	14.13	6.05	6.28
2998	14.32	14.32	14.11	14.11	6.17	6.17
2990	Pedestrian bridge					
2934	14.30	14.30	14.08	14.08	6.14	6.14
2717	14.25	14.25	14.03	14.03	6.04	6.04
2701	14.25	14.25	14.03	14.03	6.04	6.04
2522	14.21	14.21	13.99	13.99	5.82	5.82
2387	14.06	14.06	13.84	13.84	5.79	5.79
2306	13.36	13.36	13.12	13.12	5.62	5.62
2302	Choate Bridge					
2264	10.65	10.65	10.27	10.27	5.32	5.32

Table 3-5. Predicted cross-sectionally averaged velocities during fish passage flows during low tide conditions

	5% exceedance		50% exceedance		95% exceedance	
River station	Existing (ft/s)	Dam Removed (ft/s)	Existing (ft/s)	Dam Removed (ft/s)	Existing (ft/s)	Dam Removed (ft/s)
11787	2.94	02.94	1.27	1.77	0.35	0.52
10867	1.30	1.30	0.54	0.70	0.13	0.15
10689	4.57	4.57	2.32	3.85	0.63	0.95
10657	Railroad bridge					
10625	4.34	11.35	2.41	8.17	0.64	3.35
10513	1.17	1.50	0.43	0.64	0.08	0.14
9865	0.84	1.10	0.32	0.48	0.06	0.11
9283	1.38	2.01	0.58	1.42	0.13	0.61
7408	1.14	1.99	0.40	1.77	0.08	3.35
5359	0.74	1.83	0.25	1.48	0.05	0.57
3900	1.11	2.84	0.36	1.49	0.07	0.43
3682	0.93	2.37	0.30	1.19	0.05	0.33
3469	0.99	3.30	0.32	1.75	0.06	0.52
3260	0.91	2.34	0.28	1.14	0.05	0.29
3072	1.53	2.21	0.51	1.12	0.10	0.31
3051	Former dam location					
3041	2.55	2.84	0.94	1.48	0.20	0.43
3020	7.39	3.19	4.78	1.88	3.12	0.64
2998	3.93	3.93	8.23	3.19	1.33	1.66
2990	Pedestrian bridge					
2934	3.01	3.01	2.32	2.32	3.25	3.25
2717	2.02	2.02	1.17	1.17	0.58	0.58

Recreation:

Based on the assumptions made for this study, it will be possible to paddle past the former dam site, creating a new opportunity for boats to pass directly from the existing boat launches downstream to the estuary and vice versa. Even if bedrock is found beneath the dam at a higher elevation than assumed here, modeling suggests that the increased tidal range will help facilitate upstream and downstream movement at least twice a day during high tide. With the dam removed, boating hazards associated with the dam will be eliminated, though the bedrock may be challenging to navigate depending on the water levels and tide.

At the upstream end of the impoundment, portage may be required underneath the railroad bridge during low water periods. Other high spots on the bed within the impoundment may also present challenges for paddlers and could require portage during very low flows and low tide. Overall, there is no evidence to suggest that the river through the former impoundment will not remain usable for paddlers. A primary impact of dam removal will be more variability in paddling conditions as flow levels vary with changes in discharge and tidal conditions. Impacts should be reconsidered as the design progresses with access modifications and portage provisions incorporated as necessary to allow for access over a range of flow and tidal conditions.

Infrastructure:

Modeling results were used to inform an evaluation of potential impacts from dam removal to infrastructure along the river. Potential impacts to the EBSCO facility are discussed in Section 4 below. Potential impacts to other infrastructure are discussed in Section 5.

4.0 TASK 3 – POTENTIAL IMPACTS ON EBSCO FACILITY

As more fully detailed in the Task 3 Potential Impacts on the EBSCO Facility summary memorandum (Attachment 6), the Project included a detailed assessment of the potential impacts from dam removal on the structural stability of the EBSCO facility buildings located on the river's left edge immediately upstream of the dam. This assessment was completed by the structural engineering firm SGH and focused on the two primary concerns associated with the potential influence of dam removal on the structural stability of the facility:

1. Is the facility, or significant portions of the facility supported on timber piles? If so, are those piles located at a high enough elevation that groundwater might drop below the tops of those piles following dam removal, thereby exposing the piles to the potential for accelerated fungal rot and decay leading to building settlement? This issue was raised as a potential concern due to the age of the facility (estimated construction from 1901-1918) during an era when construction with timber piles was a common practice for locations characterized by a significant thickness of soft soils that would not be

suitable for building load bearing. There is no direct evidence at this time indicating that the EBSCO building was constructed with the use of timber piles.

2. Is the facility foundation or other structural elements sitting on top of soft, compressible soils that might compress as groundwater levels drop following dam removal, thereby exposing the facility to the potential for settlement of slabs-on-grade, shallow footings, or buried utilities? This issue was raised as a potential concern due to the location of the facility along the river's edge, a place where silt, clay, organic deposits, and similar compressible soils might exist if they were not removed during building construction.

Due to constraints placed by EBSCO preventing access to the interior of the facility in order to minimize disturbance to company operations, all field work conducted as part of this study occurred outside of the facility's exterior walls. SGH's work on this task was conducted in three phases:

1. Existing Information was reviewed to investigate if historical plans or other documentation might help inform the two primary questions posed above.
2. An initial subsurface investigation was conducted in the fall of 2016 that consisted of two underwater test pits dug adjacent to the facility's riverfront foundation wall, and three borings were completed at locations on Town property adjacent to either the EBSCO property or the dam itself (Figure 4-1).
3. A second round of subsurface borings in the spring of 2018 was completed at seven locations on EBSCO property, around the building perimeter, as close as possible to the facility's exterior walls (Figure 4-1).

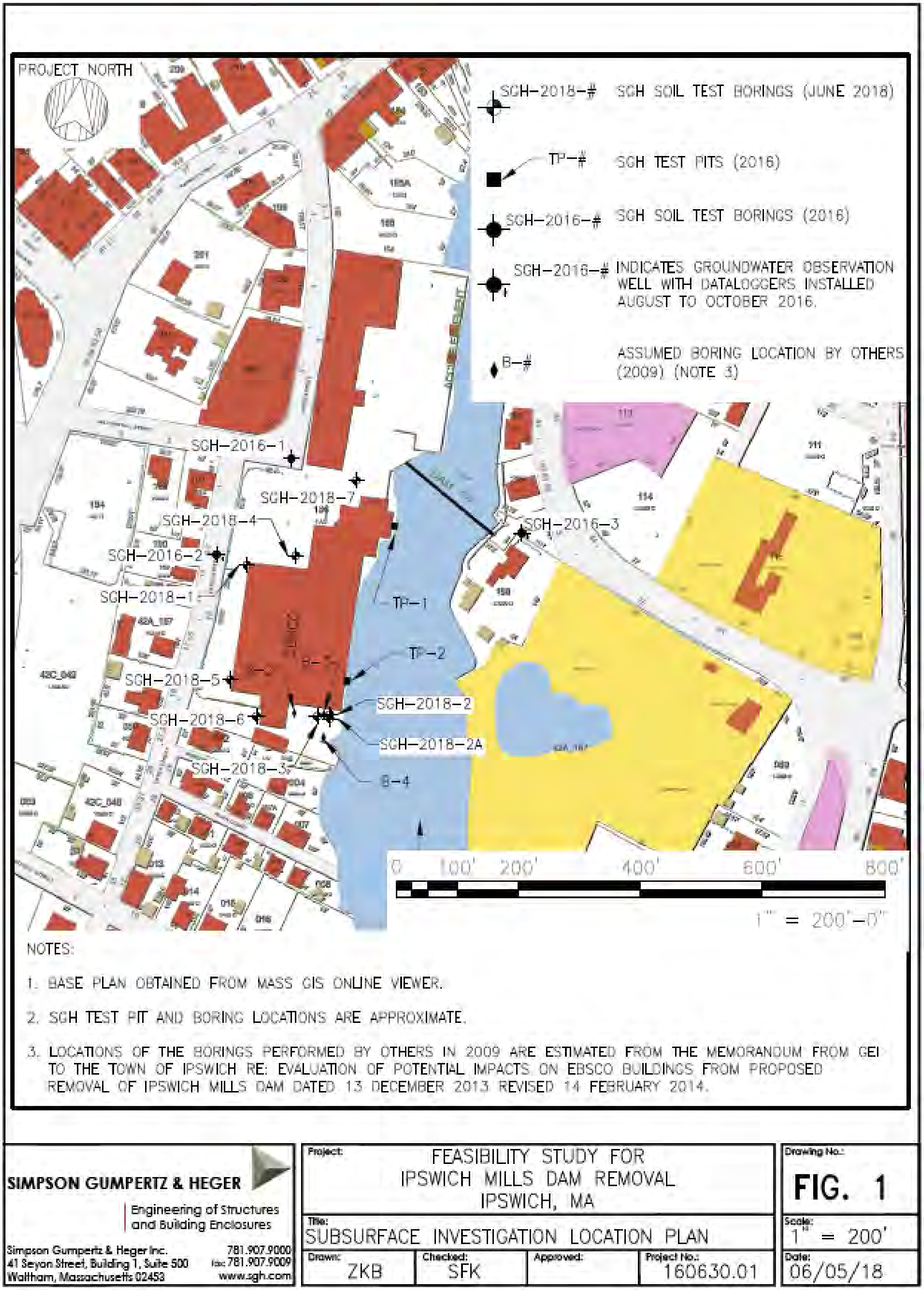


Figure 4-1. SGH Subsurface Exploration Locations

SGH concluded the following:

- Timber piles were not found in the test pits dug along the building's river edge foundation wall and the test pits were dug deep enough to determine that the foundation wall is bearing on either rock, competent soils, or piled foundations at an elevation lower than the currently estimated low water level following dam removal. Therefore, no indication was observed at the two test pit locations along the riverfront wall of the potential for fungal attack and rot of timber piles in a post-dam removal scenario.
- If timber piles exist at other locations supporting the EBSCO Facility, it is anticipated that the tops of the timber piles are at a low enough elevation to remain submerged after dam removal and, therefore, fungal deterioration of the tops of the timber piles would not occur.
- Due to the shallow depth (4.5 to 7.5 feet below grade) to dense glacial till observed along the north/northwest perimeter of the EBSCO facility it is unlikely that timber piles would have been installed in these areas as the piles would have only been between 3.5 to 6.5 feet long. It is likely that the original foundation in these areas included over excavation to place shallow footings bearing directly on the glacial till.
- Due to the shallow to moderate depth (5 to 13 feet below grade) to dense glacial till observed along the west/southwest perimeter of the EBSCO facility it is possible but unlikely that timber piles would have been installed in these areas as the piles would have only been between 4 to 12 feet long if driven all the way through to glacial till, and only 1.5 to 5 feet long if bearing on the shallower clayey silt stratum. It is highly likely that shallow soil bearing foundations bearing on the medium-stiff to hard natural fine-grained soils (Clayey Silt stratum) were used in this area. Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No.11 are founded on shallow spread footings bearing on the Clayey Silt stratum.
- The top of the glacial till stratum is generally deeper along the south/southeastern perimeter of the EBSCO facility. The depth to the top of the glacial till stratum in these borings ranged from 13 to 19 feet below grade. The glacial till stratum is overlain by 4 to 13 feet of soft to medium-stiff, fine-grained soils, including about 2.5 feet of organic silt. The depth and thickness of the observed compressible soils in this area is such that timber piles may have been driven through the soft compressible soils to bear on the glacial till stratum below to support the building structure in these areas.
- Soft, compressible soils were not identified along the northern and western margins of the EBSCO facility, away from the river, but were identified along the south/southeastern margins, near the river. SGH observations from the three 2018 borings completed in this area indicated that the compressible organic stratum may not

be as thick as had been indicated from logs of two previous borings conducted by others in 2009. Visual observation and laboratory testing of samples from the 2018 borings indicate a change of the description of the compressible organic soils in this area from peat (as indicated from the 2009 boring logs) to organic silt.

- Because no borings were completed inside the EBSCO facility it is uncertain to what extent, if any, compressible soils may underlie portions of the facility closest to where such soils were observed along the south/southeastern perimeter. If compressible soils are present beneath that portion of the facility, if groundwater levels were lowered by between one to five feet as a result of dam removal (the potential minimum to maximum range anticipated), the corresponding total potential settlement of those compressible soils after 50 years is estimated to be between approximately 0.9 to 1.5 inches, respectively.

River water levels were at approximately elevation 6 feet (NAVD88) during the 2016 drawdown period when the initial SGH field investigations were conducted. If groundwater levels cannot be maintained at approximately 6-feet elevation in a post-dam scenario, SGH recommended the following next steps to further assess the potential settlement of structures bearing on compressible soils, if present:

- Conduct a targeted subsurface investigation consisting of test pits and borings within the EBSCO facility, focused on Buildings 10A and 11A where the foundation construction is unknown and compressible soils may potentially be present. SGH estimated a planning-level cost of approximately $200,000 to conduct this work.
- Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during and after dam removal. The instrumentation should be installed prior to construction and acceptable settlement limits should be established with approval from EBSCO. Further consultation with a qualified structural engineering team may be warranted to refine the details and acceptable tolerances of this monitoring program.

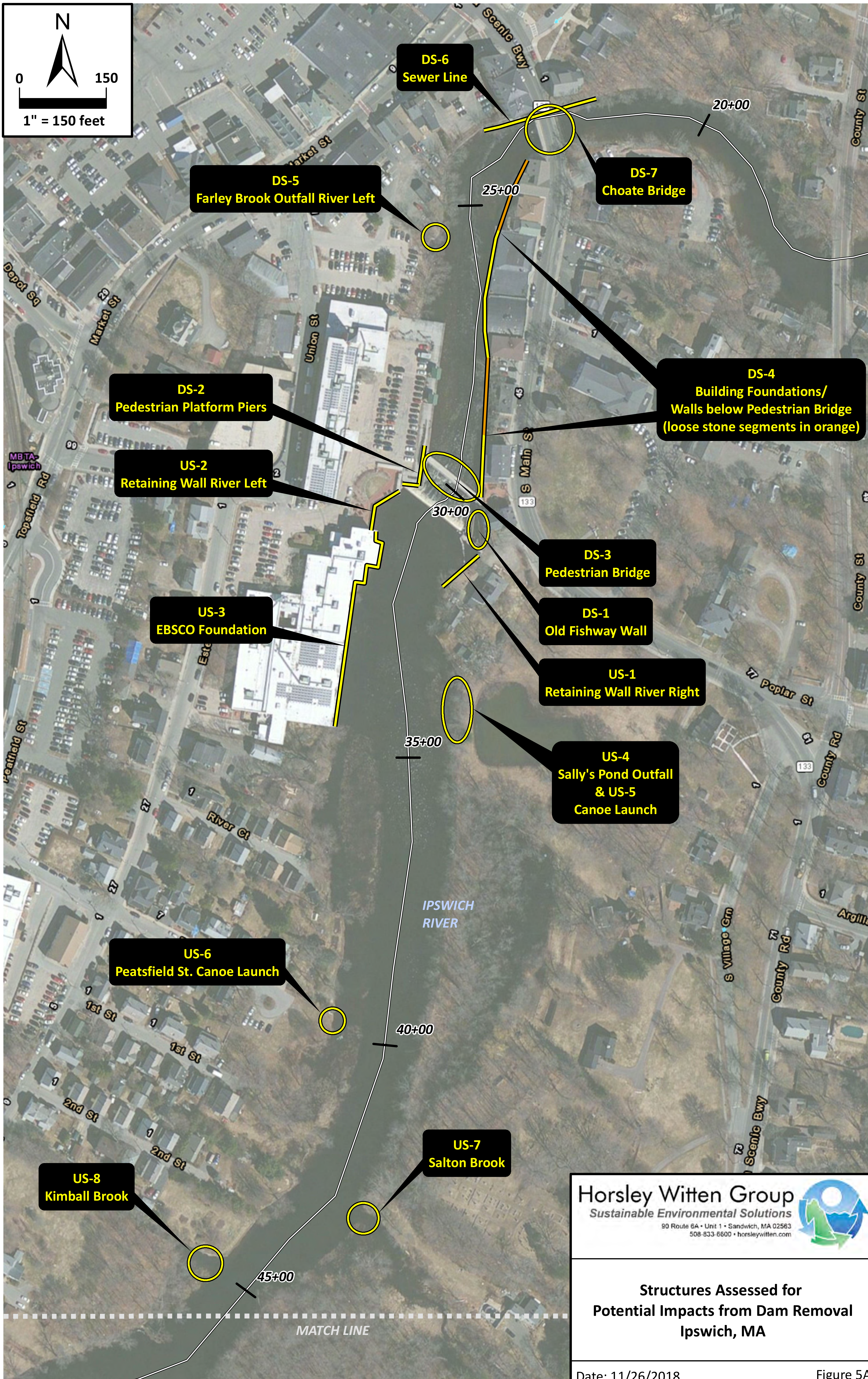
5.0 TASK 4 – POTENTIAL IMPACTS ON OTHER STRUCTURES

As more fully detailed in the Task 4 Potential Impacts to Other Structures summary memorandum (Attachment 7), the Project included an assessment of the potential impacts from dam removal on other structures (in addition to the EBSCO facility addressed in Task 3) along the river. This assessment was intended as a high-level, screening evaluation based on visual observation and a comparison to modeled dam-out river flow conditions, as presented in the Task 2 Hydrologic and Hydraulic (H&H) Analysis conducted using a HEC-RAS model built for this purpose. No subsurface investigations or structural analyses for specific infrastructure items were conducted as part of this Task 4 assessment. HEC-RAS model simulations were run for existing and dam-out conditions under high and low tide and various river flow scenarios, including 2-year storm, 10-year storm, 25-year storm, 50-year storm, 100-year storm, 500-year storm, 5% exceedance, 50% exceedance, and 95% exceedance.

The following process was followed to conduct the Task 4 assessment discussed in this memorandum:

1. Review aerial photography to identify potential structures in the project vicinity to evaluate;
2. Discuss with IRWA and other Technical Team members potential structures to evaluate to take advantage of local knowledge;
3. Field observe the length of river from the railroad bridge down to the lower falls (downstream from the County Road Bridge) from either the water side, the land side, or both to further vet those potential structures identified in steps 1 and 2, and to search for additional structures with potential to be impacted;
4. Visit structures with potential to be impacted to visually observe and photograph their conditions;
5. Compare the locations of identified structures with modeled changes in river level, velocity and erosive shear stress under dam-out conditions to evaluate if potential hydraulic changes might impact those structures; and
6. Make recommendations to protect potentially at-risk structures and/or mitigate against potential damages.

Twenty-one structures (or groups of structures along a contiguous river stretch) were identified for comparison to hydraulic modeling results for further evaluation. Seven of those structures are downstream of the dam and 14 are upstream. Photographs of all evaluated structures are included in the Task 4 Summary Memorandum (Attachment 7). Figures 5A (downstream) and 5B (upstream) depict the locations of the structures evaluated and discussed in this memorandum. River stationing for the hydraulic model and the design plans is also shown on these figures. Potential hydraulic impacts from dam removal are modeled to dissipate rapidly downstream of the dam under all modeled scenarios. The modeled extent of significant impact for either river stage or flow velocity is approximately 100 feet downstream from the dam, shortly downstream of the pedestrian bridge.

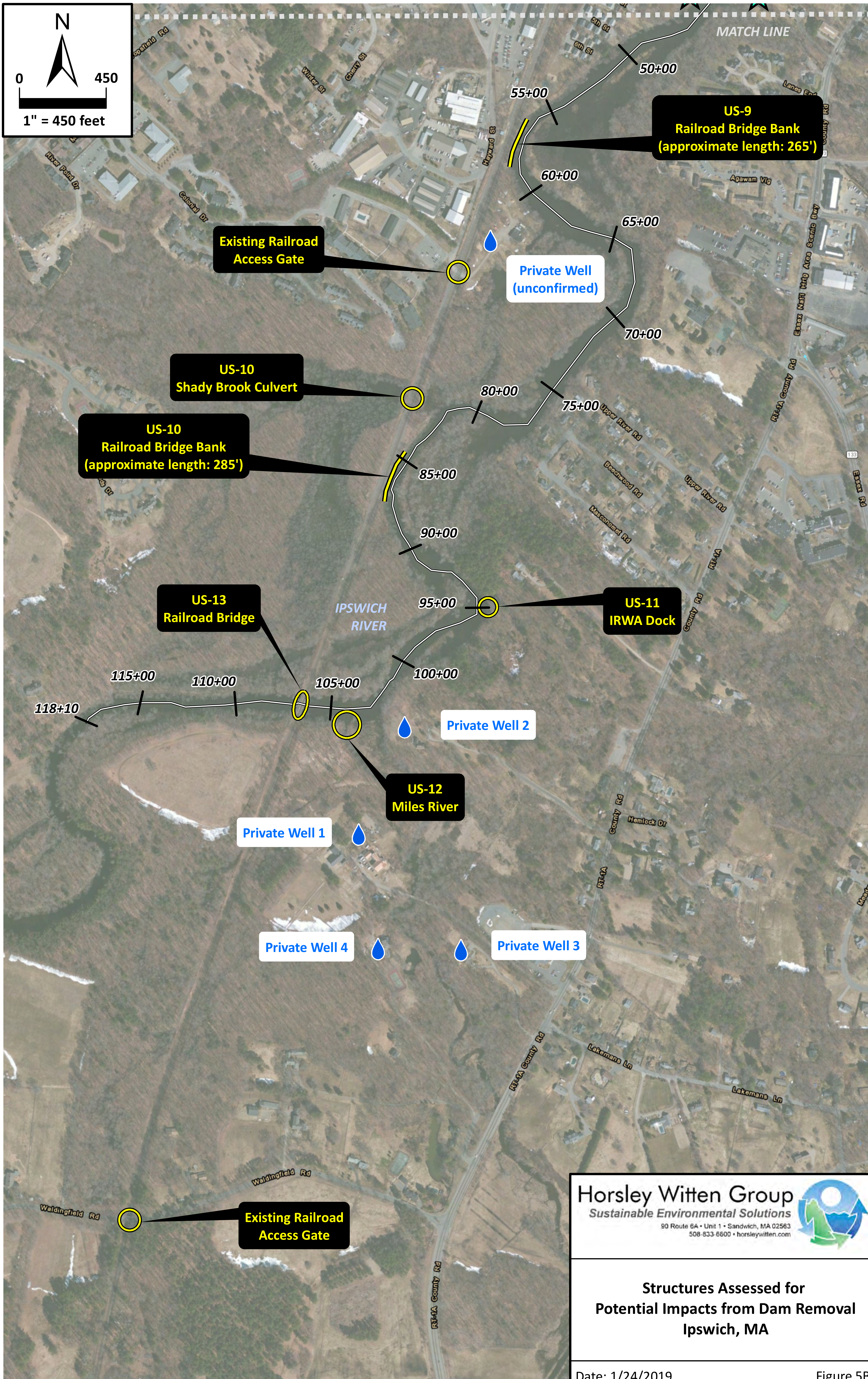


Structures Assessed for
Potential Impacts from Dam Removal
Ipswich, MA

Date: 11/26/2018

Figure 5A

Document Path: H:\Projects\2016\16041 DER_Ipswich Dam Removal Feasibility Study\GIS\Maps\181030_Structures_North.mxd



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Structures Assessed for Potential Impacts from Dam Removal Ipswich, MA

Date: 1/24/2019

Figure 5B

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In contrast, some potential hydraulic impacts under at least some flow scenarios are modeled to extend more than a mile and a half upstream, at least 1,000 feet upstream of the railroad bridge at the upper limits of currently impounded conditions. Water levels at the most upstream model transect (Station 118+10) are predicted to drop by approximately 0.9 feet under 50% exceedance flow conditions. No water level changes at this most upstream transect are modeled to occur for any other flow scenarios, and no velocity changes are modeled to occur for any scenarios. More significant water level and velocity changes are modeled to occur beginning just below the railroad bridge under all modeled flow scenarios. The river bed beneath the railroad bridge has elevated rip rap placed for scour protection of the bridge's support piers that create a small hydraulic drop that appears to prevent significant hydraulic impacts from extending much further upstream.

As discussed above, and more fully in the Task 1 Summary Memorandum (Attachment 4), the elevation of competent bedrock ledge at the dam site is not yet accurately known. Therefore, the hydraulic modeling conducted for this project under Task 2 and used to inform this Task 4 evaluation of potential impacts to structures, takes a conservative approach by assuming that bedrock is not present higher than the observed river bottom elevation upstream and downstream of the dam. This assumption leads to the conservative prediction of faster and more erosive river flows that would tend to cause more sediment migration and greater impacts to adjacent structures than would be anticipated if there were competent bedrock at a higher elevation than assumed to date for this project.

5.1 Downstream Structures

Despite the fact that hydraulic impacts from potential dam removal are modeled to dissipate within approximately 100 feet downstream of the dam, potential impacts to structures were evaluated and discussed down to below the Choate Bridge, approximately 1,000 feet downstream. Discussion was extended over this longer downstream area due to the high density of infrastructure in the area (such as the Town's main sewer interceptor and siphon) and the historic significance of the Choate Bridge and other structures. Table 5-1 lists the downstream structures evaluated for potential impacts from dam removal, their likelihood of potential impact, and whether or not mitigation is proposed. All distances are approximate river channel distances and left and right directions are relative to downstream river flow.

Table 5-1. Downstream Structures Evaluated for Potential Impacts

ID	Description	~ Feet from Dam	Nearest Model Station	Potential Impact	Further Action
DS-1	Old Fishway Wall	0-50 Right	3,020 & 3,041	Moderate	Reinforcement
DS-2	Pedestrian Platform Piers	0-300 Left	3,020 & 3,041	Moderate	Reinforcement
DS-3	Pedestrian Bridge	60	2,998	No	No
DS-4	Building Foundations/ Walls Downstream of Pedestrian Bridge	90-700 Right	2,387; 2,522; 2,701; 2,717; & 2,934	No	Monitoring
DS-5	Farley Brook Outfall River Left	440	2,522 & 2,701	No	No
DS-6	Sewer Interceptor and Siphon	450-1,000	2,387	No	No
DS-7	Choate Bridge	750	2,306	No	Monitoring

The only downstream structures identified to have risk of impact are the old fish ladder walls immediately downstream of the dam on river right, and the support piers for the pedestrian platform immediately downstream of the dam on river left. Both of those structures are proposed to be reinforced and protected as part of the conceptual design for dam removal (Task 5, included herein as Attachment 8). Some retaining walls downstream of the dam were determined to not be at significant risk from dam removal but are currently in somewhat deteriorated condition and, therefore, monitoring of those walls is recommended. Similarly, while no significant changes in river stage or velocity are modeled to occur as a result of potential dam removal, the Choate Bridge is also recommended to be monitored. This is partly due to the historic and practical significance of the bridge, and partly due to the fact that the bridge is a flow restriction during larger flow events under current conditions and will remain so under dam-out conditions. Under a potential dam-out scenario sediment currently retained behind the dam will migrate downstream. No long-term sedimentation impacts at the bridge are anticipated, however, it's possible that some sediment may be temporarily retained beneath the bridge before it can be remobilized and transported past the bridge during subsequent high flow events.

5.2 Upstream Structures

Due to the relatively long upstream extent of potential hydraulic impact from dam removal, potential impacts to structures are evaluated and discussed up to and including the railroad bridge, approximately 7,500 feet upstream. Table 5-2 lists the upstream structures evaluated for potential impacts from dam removal, their likelihood of potential impact, and if mitigation is proposed. All distances are approximate river channel distances and left and right directions are relative to downstream river flow. Under a potential dam removal scenario, the greatest changes in river hydraulics and geometry are expected at and shortly upstream of the dam site. As such, this area would also be expected to experience the greatest potential risks to infrastructure.

Table 5-2. Upstream Structures Evaluated for Potential Impacts

ID	Description	~ Feet from Dam	Nearest Model Station	Potential Impact	Further Action
US-1	Retaining Wall River Right	0-150 Right	3,072	Low	Reinforcement
US-2	Retaining Wall River Left	0-100 Left	3,072	Low	Reinforcement
US-3	EBSCO Foundation	100-440 Left	3,260	Low	Reinforcement
US-4	Sally's Pond Outfall	250-450*	3,496	Unknown	Monitoring
US-5	Sally's Pond Canoe Launch	300	3,496	Low	Not Needed Post Dam Removal
US-6	Peatfield St. Canoe Launch	920 Left	3,900	Low	Adaptive Management
US-7	Saltonstall Brook	1,200 Right	3,900 & 5,359	Low	Monitoring
US-8	Kimball Brook	1,400 Left	3,900 & 5,359	Low	Monitoring
US-9	Railroad Bridge Bank	2,500-2,800 Left	5,359	Low	Further Study
US-10	Shady Brook Culvert	5,200 left	7,408	Low	Monitoring
US-11	Railroad Bridge Bank	5,300-5,600 Left	7,408 & 9,283	Low	Further Study
US-12	IRWA Dock	6,300 Right	9,283	Low	Monitoring
US-13	Miles River	7,200 Right	10,513	Low	Monitoring
US-14	Railroad Bridge	7,500	10,625 &10,689	Low	Further Study

*Outfall not observed and not on record plans. Existence hypothetical.

The potential impact assessment for upstream structures determined the following:

- Retaining walls immediately upstream of the dam on both river right and left have the greatest risk for potential impact. Both of those structures are proposed to be reinforced and protected as part of the conceptual design for dam removal (Task 5, included herein as Attachment 8). Despite the fact that no significant risk to the EBSCO foundation wall was anticipated by the Task 3 EBSCO structural investigation, that foundation wall is also recommended for protection by the same river left reinforcement feature.

- Two canoe launch areas (Sally's Pond and Peatfield Street) may potentially be minimally impacted to the extent that the distance from the current path to the water's edge will increase. According to IRWA, the Sally's Pond launch is used primarily for portage around the dam and will therefore be less used under a dam-out scenario. Also, according to IRWA, the river bottom at both locations is relatively firm such that canoe access to the water should not be significantly impacted even considering a greater distance to the water's edge. Monitoring of both locations is recommended, and inexpensive mats or other soft path reinforcement can be considered if canoe access becomes an issue.
- There are four tributaries entering the Ipswich between the dam and the railroad bridge (Saltonstall Brook, Kimball Brook, Shady Brook, and the Miles River). Under dam-out conditions water levels in the main stem Ipswich will drop under most flow scenarios. Depending upon the competency of the river bed sediments at these confluences, some degree of temporary headcutting of the river bed may occur as the Ipswich River and its tributaries come to equilibrium to dam-out flow energy and sediment dynamic regimes to re-establish pre-dam conditions. No significant infrastructure was identified anywhere in the vicinity of these confluences and so no significant impact is anticipated. Monitoring is recommended.
- The embankment for the Metropolitan Boston Transit Authority (MBTA) railroad line currently touches the river at two locations. Under a dam-out scenario, the water's edge will retreat away from these embankments under most flow conditions while flow velocities in the narrower river channel will increase under low to average flow conditions. While no significant impact to these embankments is anticipated, further study and consultation with the MBTA is recommended, due to the importance of the railroad line. Such study will include an evaluation of the sediment characteristics of the river bed and embankment, and a scour analysis to determine the potential for any increased erosion at the embankments under dam-out conditions.
- The MBTA railroad line crosses the river approximately 7,500 feet upstream from the dam near the modeled upstream limit of hydraulic impact from dam removal. The bridge narrows the river channel at its location and rip rap protection of the bridge's support piers creates a shallow water zone. Therefore, river flow velocities at the bridge are relatively high under current conditions. Those velocities are modeled to increase under most low to moderate flow scenarios under a dam-out scenario. Therefore, it is recommended that further field study and a scour analysis be undertaken, as well as consultation with the MBTA, in order to ensure that the bridge is protected under potential dam-out conditions. The shallow conditions created by the rip rap beneath

the bridge may also potentially impede fish passage during low flow conditions under a dam-out scenario. Further study is recommended to evaluate design options to maintain low-flow fish passage results under the bridge under dam-out conditions.

5.3 Drinking Water Wells

The potential concerns regarding dam removal on drinking water wells are that groundwater levels might drop sufficiently to reduce well yields, and that, if saline water were to migrate further upstream than currently occurs, such saline water might potentially impact water quality in the aquifer surrounding the river. Under potential dam-out conditions, tidal hydraulic influence (though not necessarily actual saline water) is anticipated to extend approximately 4,350 feet upstream of the dam to the vicinity of Upper River Road. Therefore, actual water level conditions within that river reach will vary two times per day with the rising and falling tides, as well as with the seasonal fluctuation of river flows. The magnitude of the difference between modeled high tide versus low tides is greatest furthest downstream and declines steadily upstream. Tidal influence is also more evident for low flow events than for high ones. For larger storm events in particular, the tidal influence is overwhelmed by the downstream river flow, even as far downstream as the dam location.

Regarding potential declines in groundwater level from dam removal the following should be considered:

- Because hydraulic changes in groundwater occur much slower than in surface water due to the restrictive nature of the solid aquifer matrix through which groundwater must move, groundwater levels tend to respond more to longer-term, average, surface water boundary condition levels than to shorter term fluctuations. Therefore, the average tidal condition in the river influences neighboring groundwater levels more significantly than does either low or high tide conditions. Similarly, the average, climatically-influenced river level over periods of weeks or months is more significant than hourly or daily fluctuations.
- The restrictive nature of the aquifer also dampens the influence of boundary condition elevation changes as you move landward away from the river boundary. Estimated groundwater declines from changes in river level dissipate rapidly on the order of hundreds of feet away from the river, even for areas of the river proximal to the dam where river level declines would be greatest. As one moves further upstream from the dam, the estimated declines in river level decrease so that the corresponding distance laterally away from the river in which significant declines in groundwater might occur also decreases.

Regarding potential salinity impacts the following should be considered:

- While we know that tidal influence currently extends up to the dam, and would extend approximately 4,350 feet upstream from the dam under dam-out conditions, we do not know how saline the actual water chemistry is at the dam site (or the vertical distribution of salinity within the water column), and we therefore do not how far upstream of the dam saline water might reach under dam-out conditions. According to IRWA, salt water is rarely detected above the lower falls in the dozens of water samples collected by the Division of Marine Fisheries (DMF) over the years, and only reaches to the dam site for spring high tides that occur during periods of low river flow.

- Since the prevalent groundwater flow gradient is from the aquifer into the river (the river is a discharge boundary for the aquifer), any salt water intrusion from the river into the surrounding aquifer is limited to what may infiltrate during periods when the pressure head in the river is greater than the underlying aquifer (e.g. high tides). At a position as upstream along the river as the dam, where salt water content reportedly is currently relatively low, the significance of salt water influence from the river on the aquifer is likely minimal in terms of both the actual salinity and the horizontal extent of any such influence away from the river. Further upstream, the likely significance diminishes still further.

IRWA researched wells located from the dam site upstream to the identified limit of potential water level impact from dam removal shortly upstream of the railroad bridge (as identified by the Task 2 H&H analysis) and extending out 1,000 feet to either side of the river (Figure 5B). IRWA research included Board of Health (BOH) and water department records for direct evidence of private wells and public drinking water connection records for indirect evidence. Any developed property not recorded as receiving public water supply was assumed to have a private well. This research did not include the possibility of irrigation wells on properties connected to the public water system. IRWA research revealed the following:

- All public water supply sources are located far outside of the zone of potential dam-removal impact area.
- According to IRWA there is no potential for increased public water withdrawals since the regulatory safe yield for the Ipswich River basin has been exceeded, thus prohibiting the permitting of additional withdrawals within the basin over what is allowed currently. In addition, any new surface water withdrawals would not be practical due to the marginal amount of storage provided by the current dam and the need to provide advanced treatment for a river water source.
- Three known private wells were identified, all located along the Miles River near its confluence with the Ipswich over a mile upstream from the dam and therefore unlikely to experience potential significant impact from dam removal.
- IRWA is also aware of another potential, but unconfirmed, well at a landscape company located approximately one mile upstream of the dam which is not on town water. Further research is required to determine, first, if a private well even exists at this property. If there is a well, it should be determined where it is located relative to the

river, how deep it is, and what its withdrawal rate is. Unless the potential well is very shallow and very close to the river it is unlikely that it would be impacted by dam removal.

- Any potential impacts felt by private wells (known or unknown) as a result of dam removal could be readily mitigated by connecting to town water.

While the potential for significant impact from dam removal to any private wells along the river is low based on available information, it is recommended that additional efforts be made to identify any additional private wells (e.g., irrigation wells) beyond those discussed herein. Any wells identified within the zone of potential influence from dam removal should have their baseline depths to water and salinity documented. That would allow for a comparison of future well conditions to baseline in the event that those well owners believe that their wells have been impacted following potential dam removal.

6.0 TASK 5 – CONCEPTUAL PLANS AND COST ESTIMATES

Task 5 consisted of the creation of conceptual design plans for dam removal and an accompanying estimate of planning level construction costs based on that design. Task 5 was led by Inter-Fluve, Inc. (IF) with input from HW. Both the conceptual design plans and cost estimate are included herein as Attachment 8. The conceptual design is presented as a five-sheet planset that includes existing conditions, construction access, materials staging locations, demolition plan, longitudinal profile along the river orientation, four cross sections across the river, a restoration and planting plan, and typical details.

The conceptual design was based on information learned during the previous tasks that informed the conditions that the design would need to accommodate - Task 1 for existing conditions, Task 2 for the river hydraulic changes anticipated from dam removal, Task 3 for the potential impact on the EBSCO facility, and Task 4 for the potential impacts on other structures. Because, as stated above, the controlling bedrock elevation at the dam site has not yet been determined, the hydraulic modeling, potential impacts assessment, and conceptual design were all undertaken based on the conservative assumption that there is no bedrock outcrop present at the dam site at elevations above the prevailing upstream to downstream river bottom profile. This assumption led to the conservative prediction in Task 2 of relatively lower river water levels and higher flow velocities. Those predictions, in turn led to Task 3 and 4, respectively, considering relatively greater potential impacts to structures. Finally, those relatively greater potential impacts led to the conceptual design including relatively more robust protective measures for the riverbanks in the area surrounding the dam location.

The following are key components of the conceptual design:

- Construction equipment and materials staging would occur in the municipal parking lot across South Main Street from the dam site on river right.

- Construction access to the dam site will occur on river right from South Main Street through the Town's existing easement to the fish passage and viewing platform.
- The dam will be completely removed except for a 10-foot length on river right necessary to ensure against damage to connected retaining walls.
- The newer fish ladder will be completely removed.
- The older fish ladder will be partially removed, and the remainder filled with salvaged rock and/or concrete to protect the connected retaining wall.
- Piles of accumulated boulders both upstream and downstream of the dam will be regraded to fill the existing plunge pool beneath the dam and used to protect river edge features including the support piers for the pedestrian platform downstream of the dam on river left.
- The retaining walls and properties on river right immediately upstream and downstream of the dam will be protected with a hard "toe" of boulders and fiber-encapsulated soil lifts (FES) uphill of the stone toe. FES lifts are soft, bio-engineered, bank stabilization measures that include lifts at successfully higher elevation of soil wrapped in bio-degradable fabric. The FES lifts are planted with appropriate native vegetation and, over time, will evolve into natural, stabile, and vegetated river banks.
- The entire length of the EBSCO retaining wall and foundation on river left will be protected with the same FES lifts.
- An additional area on river right, to approximately the same upstream distance as the EBSCO facility on the opposite bank, will be seeded and planted to accelerate the grow-in period following dam removal. The need for seeding and planting of this area should be further evaluated in future design phases.

7.0 TASK 6 – CONCEPTUAL RENDERINGS

For Task 6, HW made a series of artistic renditions of the potential dam removal to aid the public outreach process. These artistic renditions included:

- A colorized version of the proposed conceptual design restoration plan view from Task 5.
- A colorized version of the conceptual design proposed conditions cross section through the current dam site from Task 5.
- Renderings of the anticipated view from the pedestrian bridge looking upstream under dam-out conditions for three water level conditions. The three water level conditions were a typical spring conditions, high water level flow (exceeded approximately 5% of the time) at high tide; an overall average water level flow (exceeded approximately 50% of the time) at low tide; and a typical late summer conditions, low water flow (exceeded approximately 95% of the time) at low tide.

- A compiled poster board showing all three water level condition renderings on one board.

The colored plan, cross section, and renderings are included herein as Attachment 9.

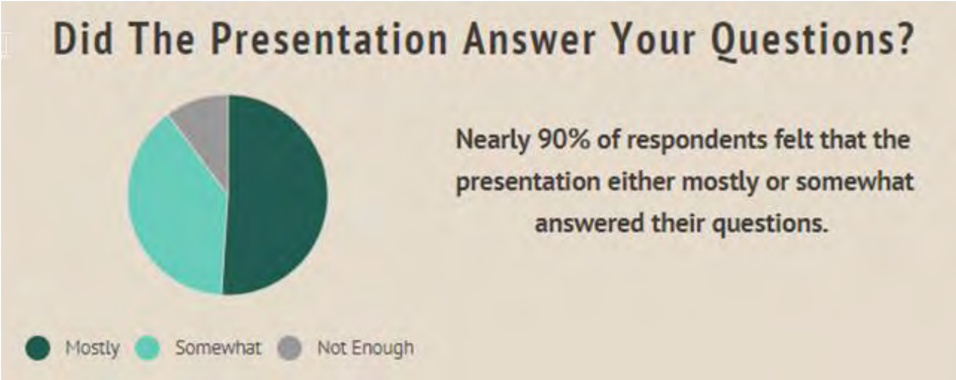
8.0 TASK 7 – PUBLIC OUTREACH

Two public outreach meetings were held as part of this study. Both meetings were held in the Selectmen's Room at Town Hall and were filmed and broadcast for community access television. The first meeting was held on June 8, 2016 for the purpose of introducing the study to interested stakeholders. That meeting outlined the intent of the feasibility study, the organizations and people responsible for different components of the study, the work that was to be conducted as part of the study, and the opportunities for community comment and engagement.

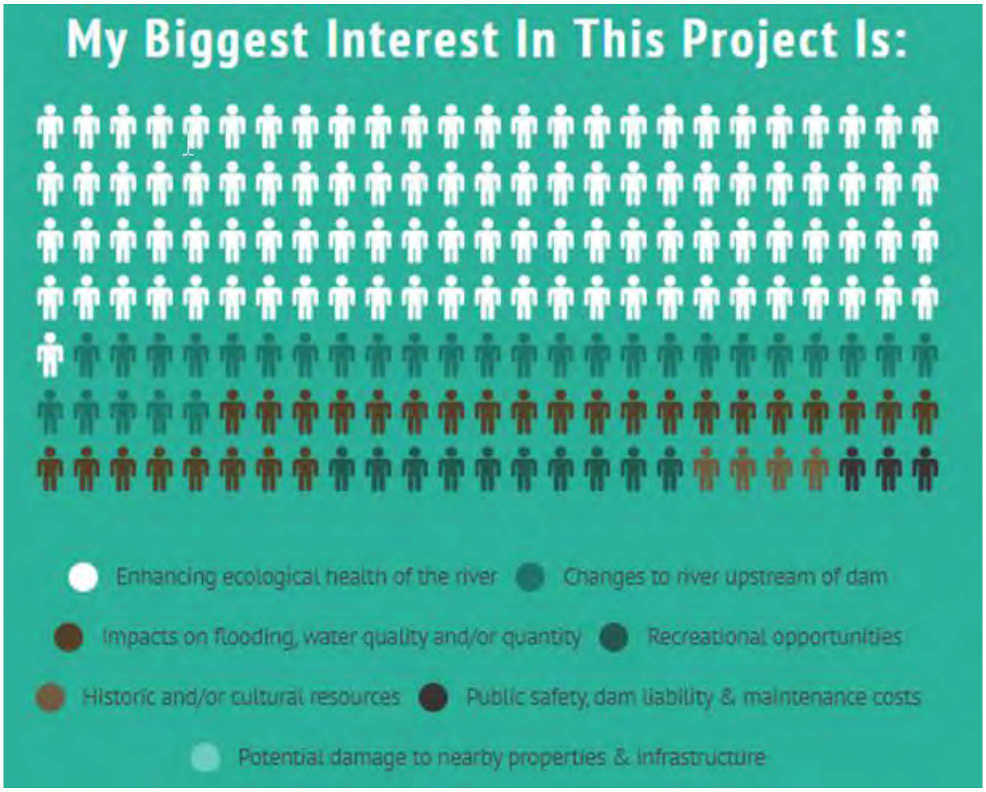
The second meeting was held on December 12, 2018 for the purposes of presenting the results of the study, soliciting public comment on the work completed, and asking questions of the public regarding the study. Following the presentation attendees participated in a public input exercise where each attendee voted on different questions concerning the potential for dam removal by placing a sticky dot next to the item or answer that they most supported. Each attendee could vote for each question. Staff responsible for the Feasibility Study and/or arranging the meeting did not participate in the voting. Please note that while 55 people signed up as attending the meeting, more or less than 55 total votes were placed for individual questions. This suggests that either more people attended and voted than had signed in, or that some people did not vote for all questions and, instead, used the stickers they saved from some questions to place more than one sticker on other questions. If this occurred, it was contrary to the instructions given. The results given below simply present the numbers based on the votes that were cast. The results of the voting exercise are as follows:

- Of 58 total votes cast concerning whether or not they supported dam removal, 62% of respondents were generally in support of dam removal, 19% were generally concerned, and 19% did not feel fully informed.

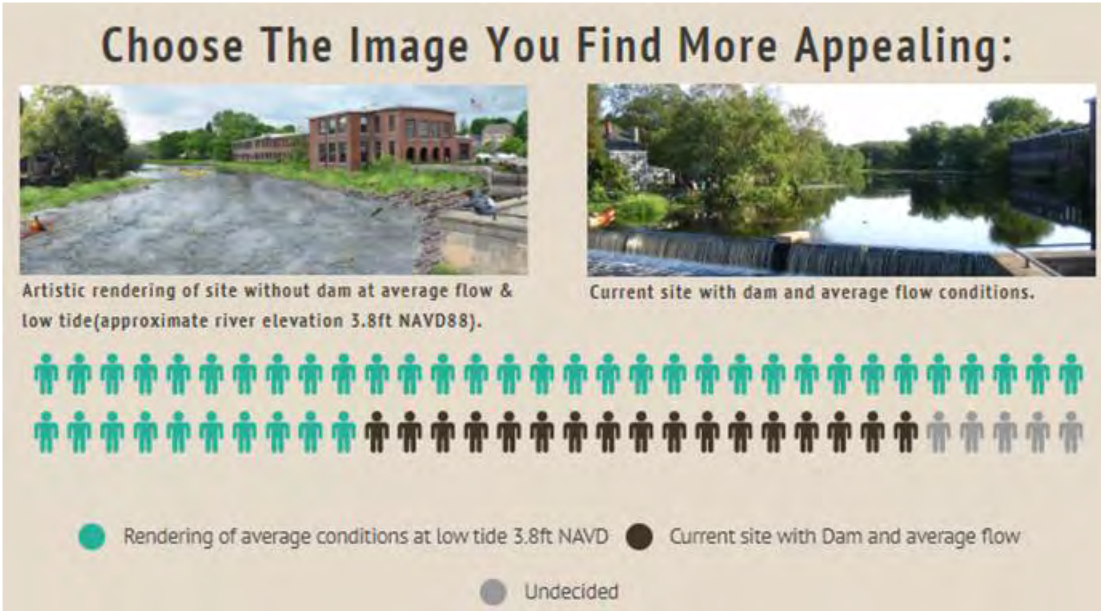
- Of 49 total votes cast concerning whether or not the presentation answered the attendees' questions, 51% of respondents felt that the presentation mostly answered their questions, 39% felt that the presentation somewhat answered their questions, and 10% felt that the presentation did not address their questions enough.



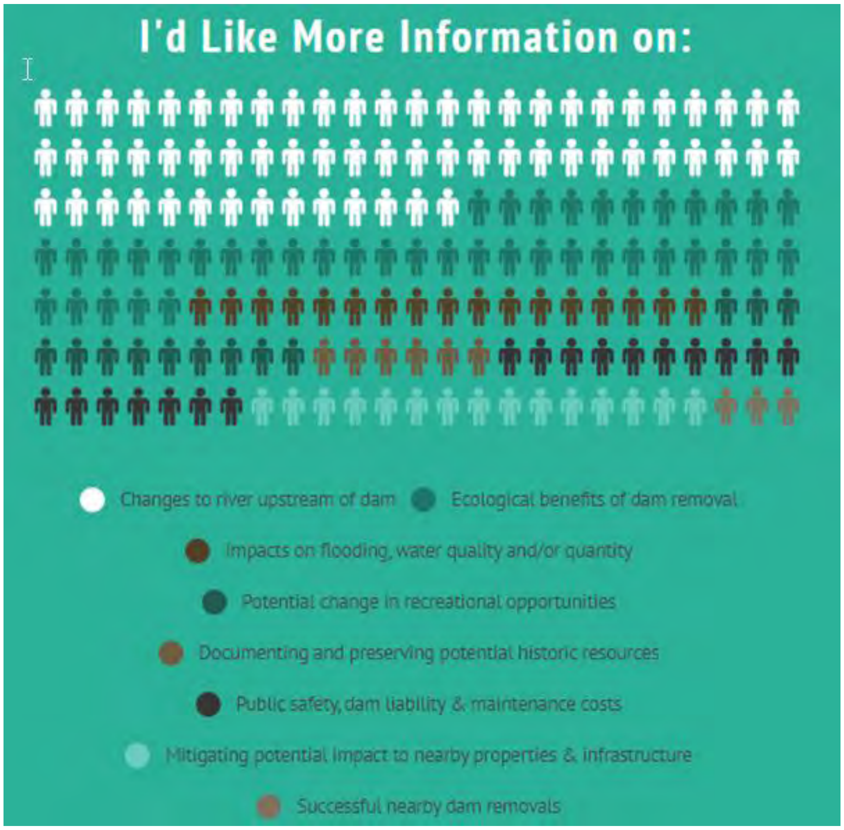
- Of 66 total votes cast indicating what participants' biggest interest was in the project, 58% of respondents chose that their biggest interest in the project was enhancing the ecological health of the river; 17% chose changes to the river upstream of the dam; 16% chose impacts on flooding, water quality, and/or quantity; 6 % chose recreational opportunities; 2% chose historic and/or cultural resources; 2 % chose public safety, dam liability, and maintenance costs; and none chose potential damage to nearby properties and infrastructure.



- Of 53 total votes cast concerning whether or not participants preferred an artistic rendering image of the river in a dam-out condition as opposed to a current picture of the dam site, 66% of respondents chose the dam-out rendering as most appealing; 26% chose the opposite; and 8% were undecided.



- Of 60 total votes cast indicating preferences for desired additional information, 37% of respondents chose changes to the river upstream of the dam; 23% chose ecological benefits of dam removal, 10% chose impacts on flooding, water quality, and/or quantity; 10% chose public safety, dam liability, and maintenance costs ; 8% chose mitigating potential impact to nearby properties and infrastructure; 7% chose potential changes in recreational opportunities; 3% chose documenting and protecting historic resources; and 2% chose successful nearby dam removals.



9.0 SUMMARY & CONCLUSIONS

This Feasibility Study conducted from 2016 through 2018 to evaluate the potential removal of the Ipswich Mills Dam was able to consolidate valuable existing information about the dam site area, help answer some long-standing questions, and better refine potential next steps to resolve significant outstanding questions. The study answered many questions sufficiently to inform Town consideration of dam removal, identified questions requiring more study, and suggested appropriate methods to better answer remaining questions. While there are still some aspects of this project which require further study, at this stage and based on the existing information, no issues were identified during this study that would categorically eliminate dam removal as a feasible option.

Highlights of the existing conditions summary effort include:

- The dam is currently owned by the Town of Ipswich, is in fair condition and ranked as a low-hazard dam (per the Office of Dam Safety as of the last inspection in 2009), and no longer serves an industrial/economic purpose. As the owner, the Town bears responsibility for damages that might occur as a result of dam failure.
- A run of the river dam provides minimal storage above it and is operated such that the volume of water released below the dam is equal to the volume of water flowing in the stream or river above the dam on a normal, continuous basis. Put another way, water is

not stored in the impoundment to be released later. Rather, the dam simply increases the head in the river, providing a potential power source that can be captured. It does not serve to prevent or mitigate flooding downstream of the dam since it allows water to flow over the dam during most typical flows.

- As a head of the tide dam, the dam is the first man-made fish passage barrier encountered moving inland up the Ipswich River. Removal of the dam would improve fish passage to approximately 49 upstream river miles, with connection to an additional 63 upstream river miles following the completion of a dam removal and fish ladder installation at the two next upstream dams on the river.

- Historical records show that a dam has existed in the vicinity of the Ipswich Mills Dam site since 1637, and that the dam was built upon or just downstream of a rock ledge outcrop or small rock rapids referred to as the Upper Falls. In 1908, the dam was modified to its current structural design to supply nearby mill buildings (at the time) with a reliable source of power.

- As documented in the Historical and Cultural Resources Summary created for this study, the downtown Ipswich area near the dam site has a rich cultural history beginning with multiple pre-contact Native American cultures and proceeding through colonial era European activity, early American history, the Industrial Revolution, and the modern period. The summary document identified historic properties and previously surveyed archaeological and architectural resources within and immediately adjacent to the Project area; detailed the pre-history and history of the dam site including former dams and their date(s) of construction; and made recommendations concerning potential impacts to cultural resources or additional cultural resources survey efforts that may be needed if the Project proceeds into design and permitting.

  The summary document is not a formal historical review as will likely be required if the Project moves further towards construction. It does, however, provide an initial understanding of the bigger picture cultural and historical resources likely to be important to the Project, as well as a foundation for future coordination between the dam owner, state and federal permitting agencies, and the MHC, should the Project progress into design and permitting.

- Historically, the Ipswich River watershed supported abundant fisheries resources including significant populations of diadromous (sea-run) fish. Diadromous fishes common in the Ipswich and its estuary included river herring (alewife and blueback herring), American shad, rainbow smelt, sea lamprey, Atlantic sturgeon and Atlantic salmon. River herring runs are monitored yearly, but the Ipswich River is currently supporting less than 1% of its total spawning potential. Alewife decline in the Ipswich has no doubt been driven by a number of factors, but primarily by the combined influence of conversion of historic spawning ponds (e.g. Wenham Lake and Suntaug Lake) to water supply use and to obstruction of migration pathways by dams.

- Restoration of sizable populations of diadromous fish to the Ipswich River Watershed would have ecosystem-wide importance. Large spawning runs of anadromous species such as river herring and shad bring large influxes of marine-derived food and nutrients to the freshwater system. They are also important as a forage fish, serving as prey for numerous piscivorous predators while at sea (e.g., tuna, cod, dolphins, billfish, gannets), in estuaries (e.g., striped bass, bluefish, weakfish, harbor seals, cormorants), and in rivers (e.g., ospreys, white perch, herons, river otters). The current low populations of diadromous forage species have important implications throughout marine and freshwater food webs.

- Dam removal typically results in the restoration of a river's natural water temperature regime through the former impoundment area and downstream of the dam. Removal of the dam will encourage active flow and help reduce water temperatures, making this part of the river more hospitable to flow dependent and fluvial fish species such as brook trout and fallfish. Removal of the dam will also allow free movement of motile aquatic organisms past the dam site to take advantage of food resources and to escape periodic, unsuitable conditions in currently impounded area.

- A Project basemap was created as part of this study that combines new survey data collected during this study with all available existing geographic data regarding topography, infrastructure, utilities, and wetlands resources within the project area. New topographic survey was conducted of 25 transects across the river as well as details of bridges and other significant infrastructure to inform the basemap and subsequent modeling evaluations.

Highlights of the modeling and other evaluations conducted during this study include:

- Tidal monitoring and modeling were conducted that showed that tidal influence currently does not reach above the dam, but that the hydraulic influence of tides would be expected to extend approximately 4,000 feet upstream of the dam site if the dam were removed. Although modeling predicts that flow conditions during low tide will be favorable for fish passage, the extended tidal range will further facilitate fish passage as well as boat passage past the former dam site.

  Limited existing salinity data shortly below the dam suggests that the influence of actual saline water above the dam site under dam-out conditions would be relatively limited and largely restricted to astronomically higher tides occurring during low-flow conditions. Because fresh water floats on top of denser salt water, the upper part of the water column, at least, is expected to be primarily fresh water at and above the dam site. Tidally-created, diurnal, water level changes raising and lowering predominantly fresh water levels above the dam site may foster the creation and expansion of tidal freshwater ecosystems, one of the rarest plant communities in Massachusetts.

- In the short term, removal of the dam would result in the release of potentially mobile sediment that has accumulated behind the dam. Following dam removal, sediment from the impounded area will be redistributed to downstream areas currently deprived of the sediment supply needed to support a vibrant riverine ecology. In time, a new equilibrium is reached that reflects the river's hydraulics and sediment dynamics post dam-removal. Fine sediment that is released as a result of dam removal is likely to be dispersed by fluvial flows and tidal fluctuations in the downstream channel. Mobilization of coarse gravel along the channel bed or banks following removal of the dam could be a potential issue for flow conveyance at the Choate Bridge, but impacts are likely to be temporary with material transported past the bridge during subsequent high flows. It is recommended that deposition in the downstream channel is monitored following dam removal.

  Available depth-of-refusal survey in the lower impoundment shows relatively little sediment accumulated along the thalweg of the channel with greater depths of accumulation at the margins of the impoundment. The risk of substantial headcutting along the main river channel in this area is therefore low, but some material may be mobilized from the margins. Vegetation growth following a drop in normal water levels should help to stabilize marginal deposits in some places.

- Flood levels downstream of the dam would be unchanged by dam removal. The Choate Bridge is currently a restriction during flood flows and would remain as such if the dam were removed. Flood levels through the impoundment upstream to the railroad bridge are predicted to decrease as a result of dam removal. Upstream of the railroad bridge, flood levels are controlled by conveyance and bed levels through the bridge section. The bridge also represents the likely upstream extent of bed incision following dam removal. Survey documents fine sediment accumulation on the bed of the channel in the upper impoundment, and depth-of-refusal survey extending the full length of the impoundment is required to better assess the risk of incision and fine sediment mobilization and to estimate impounded sediment volume.

- Model results indicate that predicted water surface profiles and flow velocities through the former dam location during low flows would be favorable to fish passage. The effects of dam removal on low flow water surface profiles are predicted to extend upstream to the railroad bridge. Compared with existing conditions, water depths are shallower and average flow velocities are predicted to increase, particularly immediately downstream of the railroad bridge as flow passes over the high spot beneath the bridge where rip rap has been placed on the bed of the channel. Modeling results suggest that depending on the design, dam removal may make fish passage conditions more challenging at the railroad bridge than they are at present. However, irregularities in the rock bed at the bridge may provide diverse flow conditions and opportunities for passage over this relatively short distance. It is recommended that fish passage conditions continue to be evaluated and optimized at this location as the project moves into later stages of design.

- Following dam removal, normal water levels would fall, and it is likely that shallow water wetland areas will evolve into a different type of wetland or upland habitat. Areas that are currently deep marsh are likely to become shallow water wetland habitat. Vegetation cover and succession upstream of the dam will likely be affected by the increased tidal range upstream of the former dam location.

- Modeling results indicate that it will be possible to paddle past the former dam site under dam-out conditions, creating a new opportunity for boats to pass directly from the existing boat launches downstream to the estuary. Even if bedrock is found beneath the dam at a higher elevation than assumed here, modeling suggests that the increased tidal range will help facilitate upstream and downstream movement at least twice a day during high tide. With the dam removed, boating hazards associated with the dam will be eliminated, though the bedrock may be challenging to navigate depending on the water levels and tide.

  At the upstream end of the impoundment, portage may be required underneath the railroad bridge. Other high spots on the bed within the impoundment that could be exposed upon dam removal may also present challenges for paddlers and could require portage during very low flows. Overall, there is no evidence to suggest that the river through the former impoundment will not remain usable for paddlers. A primary impact of dam removal will be more variability in paddling conditions as flow levels vary with changes in discharge and tidal conditions.

- The structural investigation of the exterior of the EBSCO facility indicated that the riverfront foundation wall is bearing on either rock, competent soils, or piled foundations at an elevation lower than the currently estimated low water level following dam removal. Observations made at the two test pit locations along the riverfront wall did not indicate the potential for fungal attack and rot of timber piles in a post-dam removal scenario. If timber piles exist at other locations supporting the EBSCO Facility, it is anticipated that the tops of the timber piles are at a low enough elevation to remain submerged after dam removal and, therefore, fungal deterioration of the tops of the timber piles would not occur.

- Due to the shallow depths to competent, load-bearing soils around the majority of the perimeter of the EBSCO facility it is unlikely that timber piles would have been installed with the exception of the south/southeastern perimeter of the EBSCO facility where soft, compressible soils were identified and the depth to competent soils is greater. Because no borings were completed inside the EBSCO facility it is uncertain to what extent, if any, compressible soils may underlie portions of the facility closest to where such soils were observed along the south/southeastern perimeter. Subsurface exploration from the interior of the EBSCO building would be necessary to conclusively determine if compressible soils, and potentially timber piles exist under that southeastern corner of the building, or if such soils were excavated out during construction. If compressible soils are present beneath that portion of the facility, lowering of groundwater levels by 1-5 feet as a result of dam removal is estimated to create a total potential settlement of those compressible soils after 50 years of between approximately 0.9 to 1.5 inches, respectively.

- River water levels were at approximately elevation 6 feet (NAVD88) during the 2016 drawdown period when the initial SGH field investigations were conducted. SGH opined that maintaining groundwater levels at about that elevation following dam removal would likely not result in adverse impacts to the EBSCO facility.
- Structures immediately upstream and downstream of the dam along both river banks were identified as having the potential to be impacted by dam removal (retaining walls along both banks and the pedestrian platform on river left). The conceptual design for dam removal completed as part of this study included protection for all of those structures in the form of hard stone reinforcement and soft bio-engineered protection.
- Potential impacts to the MBTA railroad bridge are identified due to the modeled increase of flow velocities under low to moderate flow scenarios under a dam-out scenario. Therefore, it is recommended that further field study and a scour analysis be undertaken, as well as consultation with the MBTA, to determine the potential for any increased erosion and to ensure that the bridge is protected under potential dam-out conditions. While no significant impacts are anticipated to the two embankments for the railroad line further downstream from the bridge at this time, further study is also recommended for these locations if dam removal design were to advance.
- No public supply wells are located such that they could be potentially impacted by dam removal due to either changing water levels or salinity. No known private wells were identified that could potentially be impacted. The one parcel identified that potentially may have a private well should be further evaluated to determine if a well exists and, if so, how deep it is and how far from the river it is located.
- A summary of anticipated hydraulic risks at key infrastructure and recommended measures to mitigate these risks is provided in Table 9-1. Additional risks may be identified, or certain risks resolved as the project progresses through design and more information becomes available.

Table 9-1. Summary of risks and recommendations at key infrastructure

Location	Potential Risks	Recommendations
Channel downstream of former dam, particularly Choate Bridge	• Temporary deposition of sediment eroded from the impoundment & restricted conveyance at bridges	• Further investigation of impounded sediment volume • Post-dam removal monitoring and contingency planning
River retaining walls at dam location and abutments supporting pedestrian and parking area on river left	• Exposure of walls, foundations, and abutments to hydraulic forces resulting in scour and structural instability	• Scour protection such as bank construction or placement of rock in front of walls and abutments • Structural investigation including foundation depths and stability; and/or • Incorporation of structural and/or additional scour mitigation into design and/or construction methods if necessary
EBSCO building foundation wall	• Exposure of foundation to hydraulic forces resulting in scour and structural instability	• Proactive management of impoundment margin through bioengineering bank stabilization
EBSCO building southeast corner	• Potential presence of compressible soils. If supported on compressible soils, building settlement up to a maximum of 1.5 inches under worst case scenario of lowered groundwater levels.	• Subsurface investigation of interior of this section of building to evaluate if supported on compressible soils; and/or • Maintenance of groundwaer at or above approximate elevation of 6 feet; and/or • Perform precision movement monitoring to detect and mitigate any potential settlement.
Channel headcutting at tributaries	• Unknown potential for bed incision along main Ipswich River channel and upstream along tributaries • More limited access to boat launches	• Further investigation of impounded sediment depth • Collection of additional data, including thalweg elevations, along tributaries • Post-dam removal monitoring and contingency planning
Railroad bridge	• Exposure to hydraulic forces resulting in scour and structural instability should incision occur immediately downstream of the bridge • More challenging fish passage conditions	• Further investigation of impounded sediment depth, particularly in upper impoundment • Improved characterization of scour risk and fish passage conditions • Incorporation of mitigation into design if necessary

Location	Potential Risks	Recommendations
Railroad embankments	• Exposure to hydraulic forces resulting in scour and structural instability	• Further investigation of impounded sediment depth; • Improved characterization of scour risk; • Incorporation of mitigation into design if necessary

- The public input component of the public meeting held at the conclusion of the study indicated that the majority felt that the presentation mostly or somewhat answered their questions, that there was general support amongst the attendees for dam removal, and that the ecological health of the river was their issue of greatest interest.

The following items are recommended for further study:

- Because the controlling bedrock elevation at the dam site has not yet been determined, the hydraulic modeling, potential impacts assessment, and conceptual design were all undertaken based on the conservative assumption that there is no bedrock outcrop present at the dam site at elevation above the prevailing upstream to downstream river bottom profile. This assumption led to conservative modeled estimates of relatively lower river water levels and higher flow velocities under dam-out conditions. That conservative river modeling led, in turn, to estimating relatively greater potential impacts to the EBSCO facility and to other structures under dam-out conditions. The actual depth to competent bedrock should be identified so that all of the assumptions, modeling, and impact assessments can be revisited during subsequent project design phases under the light of improved data.
- Additional sediment probing should be conducted throughout the entire impoundment up to the railroad bridge to better identify the thickness and characteristics of potentially mobile sediment in the impoundment. This information will help better inform revised evaluations of the potential for channel incision, headcutting, downstream sediment migration, and impacts to adjacent structures.
- Scour analyses should be conducted for the railroad bridge and the railroad embankments to better inform an evaluation of the potential risks to those structures. In additional coordination with the MBTA should occur to solicit their feedback on the potential risks.
- Based upon the above-recommended investigation of the elevation of controlling bedrock at the dam site, the average estimated river level at the EBSCO building should be re-evaluated, along with the resulting changes in groundwater level beneath the southeastern corner of the EBSCO building. If it is determined that average groundwater levels cannot be maintained at approximately 6-feet elevation in a post-

dam scenario, SGH recommended the following next steps to further assess the potential settlement of structures bearing on compressible soils, if present:

- Conduct a targeted subsurface investigation consisting of test pits and borings within the EBSCO facility, focused on Buildings 10A and 11A where the foundation construction is unknown and compressible soils may potentially be present.
- Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during and after dam removal. The instrumentation should be installed prior to construction and acceptable settlement limits should be established with approval from EBSCO.

Table 9-2, below, lists the permits that are anticipated to be required prior to dam removal potentially going to construction. Actual permitting requirements may vary depending upon funding sources and other factors to be determined during subsequent project phases.

Table 9-2. Anticipated Permitting Requirements

Permits, Reviews, or Authorizations Required
Local
Ipswich Conservation Commission Restoration Order of Conditions
State
Massachusetts Environmental Policy Act (MEPA) Review / Secretary's Certificate
Massachusetts Office of Dam Safety (ODS) Chapter 253 Permit
Massachusetts Dept. of Environmental Protection (DEP) 401 Water Quality Certification
Massachusetts Dept. of Environmental Protection (DEP) Chapter 91 Dredge Permit
Massachusetts Div. of Marine Fisheries (DMF) Fishway Construction Permit
Massachusetts Historical Commission (MHC) Memorandum of Agreement
Federal
United States Army Corps of Engineers (USACE) Section 404 / Section 10 Permit
National Historic Preservation Act (NHPA) Section 106 Consultation
Federal Emergency Management Agency (FEMA) Letter of Map Revision
National Environmental Policy Act (NEPA) Review
National Pollutant Discharge Elimination System (NPDES) Construction General Permit

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ATTACHMENT 1

Ipswich Mills Dam Partial Feasibility Study

Preliminary analysis of three primary factors that may influence the cost and feasibility of the removal of the Ipswich Mills Dam, Ipswich, MA

April 23, 2014

conservation law foundation

Division of Ecological Restoration

Horsley Witten Group
Sustainable Environmental Solutions

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The Town of Ipswich thanks the following companies and organizations for their contribution to the Study:

- Clean Soils Environmental, Ltd. and Horsley Witten Group, Inc., provided pro-bono services to prepare sections of this report.
- Inter-Fluve, Inc. assisted with sediment sampling and field processing.
- Roux Associates, Inc. and Alpha Analytical, Inc. partnered to provide sediment analysis at reduced rates.
- The Ipswich River Watershed Association provided project management support and community outreach to this study in addition to coordinating ongoing citizen science monitoring efforts that complement the project.

A technical team planned and guided the Study. The Technical Team was comprised of the NOAA Restoration Center, MA DER, Ipswich River Watershed Association, Horsley Witten Group, Inc., and various Town of Ipswich departments and committees (e.g. Planning, Conservation, Public Works, Town Manager, Fire and Rescue Services). Technical Team members were integral to developing detailed scopes of work for the project components, reviewing draft documents and engaging other stakeholders.

EXECUTIVE SUMMARY

This document is a compilation of three preliminary assessment studies that comprise a partial feasibility study to evaluate the removal of the Ipswich Mills Dam. This work was partially funded by a grant jointly awarded from the Conservation Law Foundation and the National Oceanic and Atmospheric Administration (NOAA) Restoration Center, and managed by a Steering Committee representing the Town of Ipswich, the Ipswich River Watershed Association (IRWA), the Massachusetts Division of Ecological Restoration (MA DER), and the NOAA Restoration Center. The three studies presented herein are as follows:

1. Preliminary Hydraulic/Hydrologic Assessment of the Potential Removal of the Ipswich Mills Dam (Horsley Witten Group, Inc.)
2. Evaluation of Potential Impacts on EBSCO Buildings from the Proposed Removal of Ipswich Mills Dam (GEI Consultants)
3. Sediment Management Preliminary Review (Clean Soils Environmental, Ltd.)

Together these preliminary assessments provide a basis for future investigation, analysis and decision-making with regard to the potential removal of the Ipswich Mills Dam. In short, the reports provide competent professional assessments that conclude the following:

- The removal of the dam would lower the level of the water upstream of the dam such that the water elevation likely would be governed by the rock ledge identified by IRWA in a preliminary site survey extending approximately 10 feet upstream from the dam structure;
- The preliminary assessment of the dam environment suggests that sediment trapped by the dam may have little contamination and may not pose a threat to human or aquatic health; and
- The lowering of the water elevation upstream of the site as a result of dam removal could pose a biodeterioration threat to the foundation of the EBSCO building on the river bank just upstream of the Ipswich Mills Dam. Methods exist to mitigate these potential impacts.. More information is required to understand better the existing foundation structure and elevation.

Background

The Ipswich Mills Dam is a run of the river dam that was built for the purpose of generating power for nearby buildings and manufacturing processes. It no longer serves that purpose and now stands as a relic structure in the river. A run of the river dam is operated such that the volume of water released below the dam is equal to the volume of water flowing in the stream or river above the dam on a continuous, real-time basis. Put another way, water is not stored in the impoundment to be released at a later time. Rather, the dam simply increases the head in the river, providing a power source that can be captured. This is typical of many small New England dams.

The current dam is constructed out of cut stones with concrete at some locations and is a run of the river dam with the spillway extending across most of the width of the river. The main spillway is 132 feet wide. A 3-foot-wide low level stop-log spillway is at the right end of the main spillway. The spillway crest is at El. 9.71 and the low level stop-log spillway invert is at El. 8.7. The dam also has a

4.5-foot-wide by 3-foot-high low level gated outlet with an invert at El. 7.5 on the right side of the dam. The right side of the dam also includes a fish ladder and a non-overflow granite block wall or pier that extends approximately 45 feet into the river and abuts the right end of the spillway. It has five low level gates that, when originally installed, could be removed manually to adjust the water level in the River. However, as described in the 2009 dam safety inspection report (Haley and Aldrich, 2009), three of those gates have since been plugged, one has been fitted with a stainless steel slide gate operated by a handwheel and one controls flow to the fish ladder.

A number of buildings have been built over the years adjacent to the Ipswich River and the Ipswich Mills Dam. Most notable is the EBSCO complex, which includes one particular building that sits directly on the edge of the river upstream of the dam, such that the foundation appears to be submerged. This suggests that lowering the elevation of the river water along the building foundation could potentially expose the foundation to air, which could cause biodeterioration of the foundation.

In addition, the long history of development upstream and surrounding the historic dam suggests that there is a potential for contaminated sediments to build up behind the dam over time. Therefore, evaluating these sediments and managing them appropriately during any dam removal process is essential to protect the health of humans and the environment.

Preliminary Hydraulic/Hydrologic Assessment

The Ipswich Mills Dam itself was not constructed to provide flood control for the area downstream of the dam, and does not serve that purpose by default. The dam provides relatively little storage (small head pond) by detaining flow behind the dam, and what is detained is actually occupying or using up a small portion of the flood storage capacity that would naturally be available in the flood plain in the absence of the dam. Because of its minimal storage capacity, this dam does not provide flood mitigation for areas downstream of the dam. Flows downstream of the river are essentially equal to what they would be in the absence of the dam because the river has created an equilibrium in which water flowing to the dam equals water flowing over and downstream from the dam. It is presumed that a ledge outcrop and falls extend from the dam toe to approximately 10 feet upstream of the dam.

According to Haley and Aldrich (2009), the impoundment from the dam extends upstream approximately 12,500 feet at an average width of 70 feet, and has a total surface area less than 1% of its contributing watershed area. Conversely, preliminary field reconnaissance by IRWA suggests that the impoundment extends only 7,500 feet upstream to the commuter rail bridge (MacDougall, email correspondence, November 6, 2010).

A preliminary site survey by IRWA staff identified a rock ledge extending from the dam toe to approximately 10 feet upstream of the Ipswich Mills Dam. In this case, it is reasonable to expect that once the dam is removed, the falls will become the new defining element in the river and will establish the new upstream water surface elevation during normal or low flow conditions. However, during flood flows, the existing dam and the rock ledge outcrop (or Upper Falls as it is commonly referenced) appear to have little impact on the water surface elevation or the river discharge due to the presence of numerous other impediments to flow, including the Choate Bridge, the pedestrian foot bridge, downstream tidal influence, and the sharp bend in the river downstream of the Choate

Bridge. The amount of influence each of these impediments has on the current system is unknown at this time but can be estimated with future evaluation.
Many factors must be considered when deciding whether to remove a dam, including the hydrologic and hydraulic factors presented in this preliminary assessment. Based upon the information compiled and reviewed for this assessment, it seems relatively clear that the dam no longer serves its initial intended purpose of providing a small-scale energy source for the surrounding mill activities. Because of the dam's basic design and relatively small size, it does not provide active (regulated) flood mitigation services for areas downstream of the dam. While the head pond created behind the dam is relatively small in comparison to the average annual and average monthly discharge passing over the dam, the dam does raise the surface elevation of the water upstream of the dam above what would exist in the absence of the dam.

Based on historical records and anecdotal observations reported during low flow conditions, it is generally believed that the dam was constructed on top of or at the toe of a rock ledge outcrop that created the Upper Falls. The extent of that ledge is yet to be determined, but it is expected that, in the absence of the dam, the height of the rock ledge will be a primary factor in determining the normal or low water surface elevations.

The next steps for this feasibility assessment are to develop a more detailed understanding of the flows (discharge, surface elevation, velocities) in the river under existing conditions from the area upstream of the Ipswich Mills Dam to downstream of the Choate Bridge, and to use that information to predict the conditions in the river under the potential dam removal scenario. It is important for the town to understand what impact the dam is having on the flow regime in the river (both high flows and low flows) and to develop an understanding of the potential risks and benefits from dam removal. This includes estimating the future river water surface elevations and the flow velocities in the area of the dam if it were to be removed. This would need to be evaluated under all flow conditions (i.e., low, normal, and flood flows) to gain an informed understanding of the impact of dam removal. HW recommends using the HEC-RAS model, which is publically available from the Army Corps of Engineers and is the industry standard in modeling river and stream hydraulics, together with current detailed cross-sectional data and flow data, using current data from the USGS gauge at the Willowdale Dam located just upstream of the Ipswich Mills Dam.

Evaluation of Potential Impacts on EBSCO Buildings from Proposed Removal of Ipswich Mills Dam

The portions of the EBSCO buildings along the Ipswich River are supported most likely on timber piles given the soil conditions along the river and the age of the buildings. However, the preliminary assessment was not able to identify any information regarding the elevation of the tops of the suspected timber piles. GEI reviewed logs from three borings performed in 2009 immediately south of the southeast corner of EBSCO's Building No 10-A, and concluded that at least some portion of the EBSCO buildings along the river are likely supported on deep foundations that consist of driven timber piles.

GEI also observed that some of the existing and former buildings pre-date the construction or reconstruction of the existing dam. It is possible that the tops of the foundations supporting the buildings that pre-date the current dam were constructed when the impoundment behind the dam was maintained at a lower elevation. Consequently, the tops of the timber piles supporting these older buildings could have been established based on a lower impoundment elevation and may not be at risk of biodeterioration from removal of the dam.

Assuming that portions of the EBSCO buildings are supported on timber piles, the tops of the timber piles need to be below water to protect them from rapid deterioration (biodeterioration). Methods that have been implemented on other projects to protect timber piles have included artificially raising groundwater levels to keep the piles submerged, lowering the tops of the piles below the expected future groundwater level, or a combination of raising groundwater levels and cutting off the tops of the piles.

Consequently, it is still uncertain if the removal of the dam would likely expose the tops of the piles causing them to deteriorate resulting in damage to the building. The assessment concludes with recommendations to perform the following additional work: literature search for historic records of the former dams; lowering of the impoundment for maximum exposure of the EBSCO foundation wall along the river; soil probing along the EBSCO foundation wall in the river; a more extensive river sounding program; and possibly coring through the EBSCO foundations or excavating test pits inside the EBSCO building to expose the foundations.

Sediment Management Preliminary Review

At this time, the impounded sediment within the future channel is (conceptually) proposed to be discharged downstream within the tidal waters of the Ipswich River. This study evaluates the quality of the impounded sediment in relation to human and ecological health thresholds, an important factor in evaluating dam removal options.

The sediments found behind Ipswich Mills Dam have a very low likelihood of toxicity when viewed independently and in relation to other dams across Massachusetts. The U.S. Geological Survey (USGS) and the Massachusetts Department of Fish and Game, Division of Ecological Restoration (MassDFG) collaborated to collect baseline information on the quantity and quality of sediment impounded behind 32 selected dams in Massachusetts, which can be used as a point of reference for other dams. As part of this study, USGS collected two sediment cores in the vicinity of the Ipswich Mills Dam impoundment. That study concluded that the Ipswich Mills impoundment had a 13% likelihood of toxicity of bottom sediments. In addition, the IRWA and staff from Interfluve, Inc. collected three (3) sediment cores from the impoundment area on May 31, 2012 and had them analyzed in the laboratory for Total Heavy Metals, SVOCs, PAHs, Volatile Organic Compounds (VOCs), Extractable Petroleum Hydrocarbons (EPH), and Physical Characteristics such as Percent of Total Organic Carbon (TOC), Percent of Water, and Percent of Grain Size Distribution. Generally, both sampling events indicate that the sediment is below applicable ecological impact benchmark limits. The assessment concludes that laboratory data to date indicate a condition of 'No Significant Risk' may exist within the sediment from the impoundment of the Dam. This is logical based on the past development and uses in the vicinity of the dam, which include mainly residential uses with little industrial use. The concentrations of metals, SVOCs, pesticides, VOCs, and EPHs measured within the sediment appear to be consistent with surface water runoff non-point sources (e.g., roadways and farming).

Recommendations from the assessment include estimating the volume of sediment that is contained within the impoundment and the volume of sediment that would be dredged or mobilized as part of a dam removal project so that the required number of samples can be estimated and collected, as well as conducting further testing of sediments above and below the impoundment with an emphasis on the area downstream of the impoundment. In particular, testing is suggested immediately upstream of the Ipswich Mills Dam, as well as further upstream in depositional areas subject to potential mobilization during storm events, which will help evaluate material that is 'moving through the system' regardless of actions taken at the dam.

SECTION 1:

Preliminary Hydraulic/Hydrologic Assessment of the Potential Removal of the Ipswich Mills Dam

Horsley Witten Group, Inc.

18 Pages

Preliminary Hydraulic/Hydrologic Assessment
of the Potential Removal of the Ipswich Mills Dam
Ipswich, MA

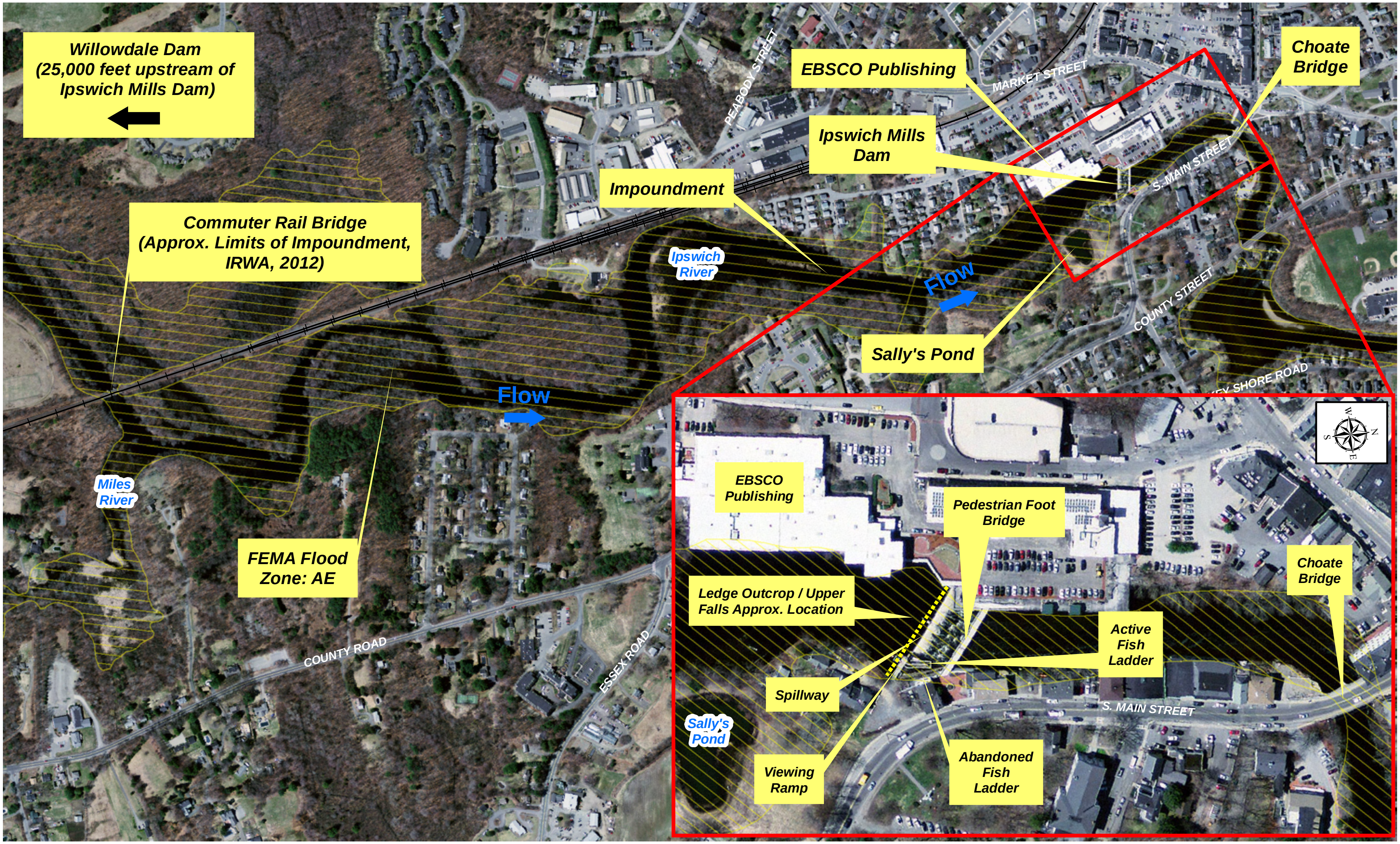
1. Introduction

The Horsley Witten Group (HW) has been retained by the Town of Ipswich to compile and present a basic assessment of the hydrologic and hydraulic implications associated with the removal of the Ipswich Mills Dam on the Ipswich River, using existing information. This work is a portion of a partial feasibility study to evaluate the removal of the Ipswich Mills Dam, funded by a grant jointly awarded from the Conservation Law Foundation and the National Oceanic and Atmospheric Administration (NOAA) Restoration Center, and managed by a Steering Committee representing the Town of Ipswich, the Ipswich River Watershed Association (IRWA), the Massachusetts Division of Ecological Restoration (MA DER), and the NOAA Restoration Center. To perform this task, HW has reviewed existing data and reports on the subject and performed a site visit to aid in the investigation. This report describes the results of the assessment and presents recommendations for a future, more detailed dam removal feasibility study.

2. History and Background

The Ipswich Mills Dam is located on the Ipswich River in the Town of Ipswich, Massachusetts, approximately 700-feet south (upstream) of the Route 133/South Main Street/Choate Bridge crossing. The spillway spans 136-feet from the western riverbank, near the EBSCO Publishing Company, to the eastern riverbank, near a private residence on Route 133. A locus map is presented in Figure 1 depicting the dam location and other significant surrounding features. The dam is currently owned and operated by the Town of Ipswich Utilities Department (Haley & Aldrich, 2009).

Historical records show that a dam has existed in the vicinity of the Ipswich Mills Dam site since 1637 (Haley & Aldrich, 2009). In 1908, the structure was modified to its current structural design to supply nearby mill buildings with a reliable source of power; however, the dam no longer serves its industrial purpose. The design of the Ipswich Mills Dam is referred to as a “run-of-the-river” dam or “channel dam.” For the purposes of this study, the term “run-of-the-river” dam can be defined as a dam that creates an impoundment that is completely contained within the banks of a river and provides only limited, short-term, storage capacity (ICF Consulting, 2005). Typically, these types of dams are no more than fifteen feet tall. They are designed to allow all flowing water to pass over the crest of the dam (ICF Consulting, 2005). A run of the river dam serves a different purpose than a flood control dam; It is used to create head and therefore generate small scale power from the change in elevation between the top of the dam and the downstream water surface. It does not serve to prevent or mitigate flooding downstream of the dam since it is generally sized to allow water to flow over the dam during all typical flows.



Path: H:\Projects\2011\11101 Ipswich Mills Dam Feas.Study\GIS\Maps\Figure_1.mxd

Legend

Commuter Rail Service

FEMA AE Zone: The base floodplain (100-yr flood zone), or areas with a 1% annual chance of flooding and a 26% chance of flooding over the life of 30-year mortgage. Base flood elevations have been provided.

Feet
750

Horsley Witten Group
Sustainable Environmental Solutions

Ipswich Mills Dam and
Surrounding Area
Ipswich, MA

Date: 3/15/2012

Figure 1

During recent years, a number of modifications to the dam have occurred to allow and improve fish passage. There is a three-foot wide low level spillway at the eastern end of the dam that can be controlled with stop-logs (Haley & Aldrich, 2009). There are two additional outlets in the spillway, one that regulates flow to an active denil style fish ladder and one that is controlled by a manually operated slide-gate. The slide-gate was closed, and the outlet non-operational, when HW observed the dam on November 3, 2011.

The Ipswich Mills Dam receives river flows contributed from a 148 square-mile watershed of the Ipswich River upstream of the dam. The watershed is made up of primarily forested land, wetland areas, residential properties, agricultural land, and some commercial/industrial zones. About 160,000 people, in parts of 21 towns, live throughout the watershed (IRWA, 2012). The Ipswich River flows nearly 40 sinuous miles from its headwaters in Burlington, Wilmington, and Andover, MA, to its mouth in Plum Island Sound, and loses approximately 115 feet in elevation along its course. The soils in the watershed are comprised primarily of Merrimac-Hinckley-Urban land and Paxton-Montauk-Urban land. The former is an excessively drained, loamy, sandy soil that was formed in outwash deposits. The latter is a well drained, loamy soil formed in glacial till. Canton-Woodbridge-Freetown soil also exists in the upper parts of the watershed but to a lesser extent than the other soil types. This soil type is a well to moderately-well drained, loamy soil formed in glacial till (USDA SCS, 1981).

3. Summary of Existing Data

A wealth of data and reports exists for the Ipswich Mills Dam and the surrounding area of the Ipswich River, as well as rich photographic and historic documentation of the dam during various periods in history and during recent severe flood events. Flood studies, dam inspections, and bathymetric and sediment surveys have been completed in recent decades. The particularly pertinent items received and reviewed by HW for this assessment include the following:

- Flood Insurance Study, Town of Ipswich, Massachusetts, Essex County. Federal Emergency Management Agency. February 5, 1985.

- Ipswich Mills Dam Phase I Inspection / Evaluation. Haley & Aldrich. October 20, 2009.

- Ipswich River Longitudinal Profile and Cross-Sectional Data (upstream of Mills Dam). Ipswich River Watershed Association (IRWA). November 3, 2011.

- Feasibility Study for Willowdale Dam Fish Passage Project. Alden Research Laboratory, Inc., for MA Division of Marine Fisheries. August 2006.

In addition, the USGS maintains a gage located 200 feet downstream from the Willowdale Dam, or 25,000 feet upstream of the Ipswich Mills Dam, and has continuously recorded water surface elevation and discharge data as far back as June 1930. Monthly mean flows at the Willowdale Dam between 1930 and 2009 range from 42.0 cubic feet per second in August to 446 cubic feet per second in March. Appendix A presents the monthly gage data according to the USGS analysis of data between June 1930 and 2009. The highest flow on record of 4,600 cfs occurred on May 16, 2006. Two photos of the

Ipswich Mills Dam on May 16, 2006 are provided below (Figures 2a and 2b), showing that the dam is virtually drowned out by the discharge in the river.

Figure 2a. Ipswich Mills Dam on May 16, 2006, facing southwest (photo provided by IRWA)

Figure 2b. Ipswich Mills Dam on May 16, 2006, facing northwest (photo provided by IRWA)

Because of the severity of this flood event, as well as other lesser but significant flood events in recent years, the area of the dam during flood flows has been very well photographed. Some of these photographs have been provided by IRWA for this assessment and are useful in describing the functionality of the dam with respect to flood flows.

The Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) describes the existence and severity of flood hazards in the Town of Ipswich, MA (1985). Peak discharges and peak elevations along the Ipswich River are calculated and presented for the 10-, 50-, 100-, and 500-year floods, and summarized in Table 1. Since no gauging station was present at the Ipswich Mills Dam, peak discharges just below the dam (i.e., Central Street as described in the FIS) were obtained by scaling measured upstream flows at the Willowdale Dam. Peak flood elevations were computed through the use of a U.S. Army Corp of Engineers (ACOE) HEC-2 (Hydrologic Engineering Center) step-backwater computer analysis. A longitudinal profile with expected flood elevations is presented in the report for the nearly 30,000-foot length of the Ipswich River, beginning at the river mouth near Plum Island Sound and ending at the headwaters.

Table 1: Published FEMA flood results at Ipswich Mills Dam, Ipswich, MA (1985)

Flood Recurrence Interval	Water Surface Elevation* (feet)	Peak Discharge (cubic feet / second)
10-year	12.8	2,023
50-year	13.8	3,016
100-year	14.1	3,251
500-year	14.7	4,196

*Elevations referenced are based on the National Geodetic Vertical Datum of 1929 (NGVD)

There are several reasons to consider the current applicability of the FIS-predicted flood elevations for the area of the Ipswich Mills Dam:

- As is common throughout eastern Massachusetts, increasing development has altered the natural hydrologic response of the watershed to precipitation. As impervious coverage in a watershed increases, runoff conveyed quickly and directly to the river increases while groundwater recharge decreases. As a result streamflow tends to become “flashier” with higher peak storm flows and lower summer baseflows. According to Ipswich River Watershed Association (www.ipswichriver.org//issues/land-use/), the population of the watershed increased by 9 percent between 1980 and 2000, yet residential land use increased by 35 percent. In addition, between 1971 and 1999, the area of forested land in the Ipswich River watershed is estimated to have decreased by more than 15 percent. In addition, increasing groundwater withdrawals for irrigation and drinking water, combined with wastewater transfers out of the watershed have decreased the quantity of groundwater available to support summer baseflows (www.ipswichriver.org/issues/low-flows-floods/).

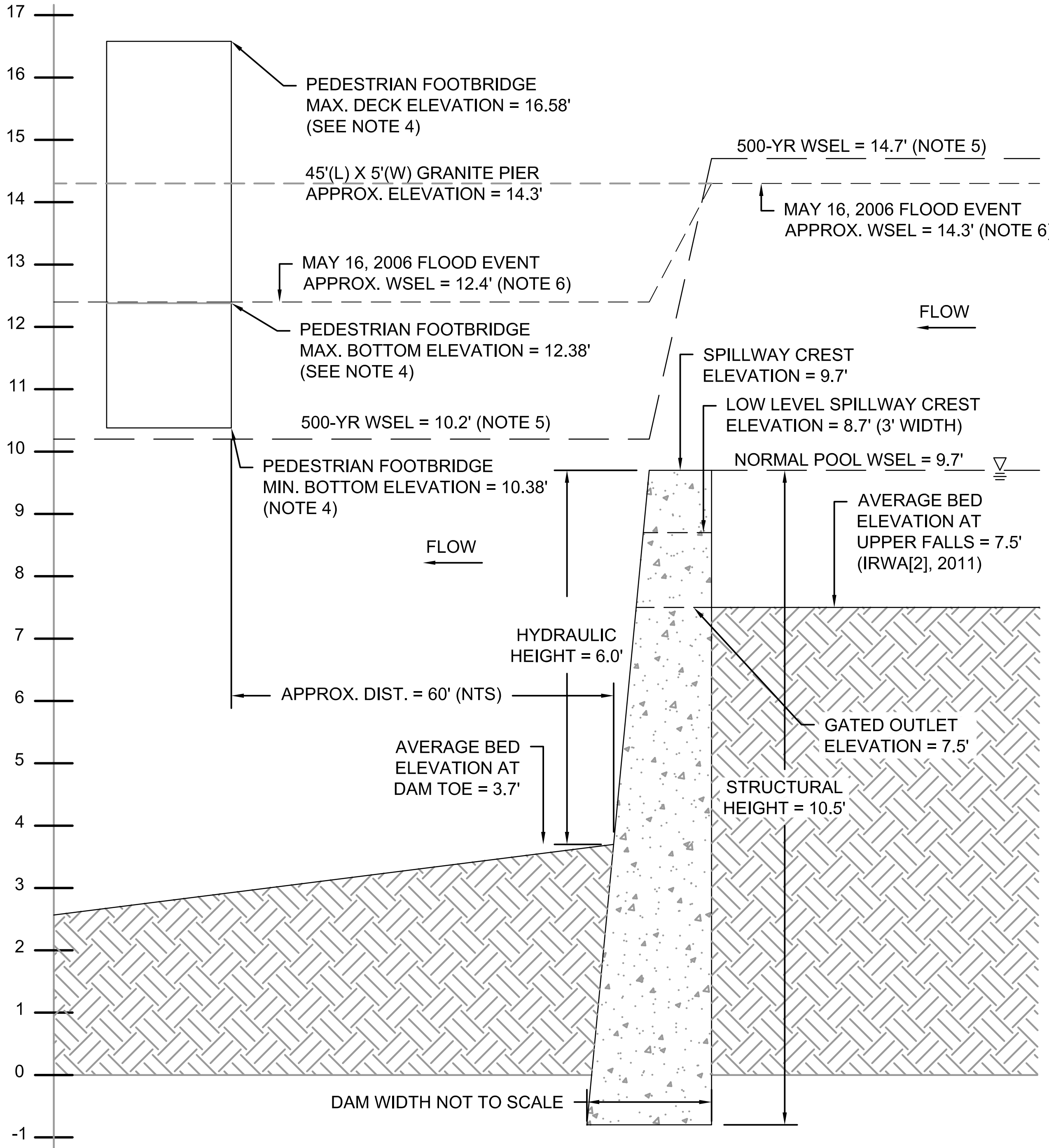
- There is also documented evidence showing that average annual precipitation in New England has increased, particularly among higher frequency storm events, and that flood frequency and intensity have also increased, particularly since 1970. This upward trend in flood series has been observed in 25 out of 28 New England watersheds with dominantly natural streamflow (meaning that climatically-induced impacts on hydrology have occurred independently of the effects from human development) (Collins, 2009). Collins analysis suggests that FEMA flood prediction methodologies should be more flexible and should be looking at precipitation after the 1970 climatologic shift to more conservatively estimate the more frequent (1-10 year) flood events, since rainfall patterns appear to have shifted in the early 1970s. As concluded in a subsequent research paper by Armstrong, Collins and Snyder (2011), "New England rivers appear to be shifting toward flow regimes that flood more frequently and with greater magnitude. Statistical flood frequency estimates calculated using pre-1970 data, or even the entire record, may underestimate discharges calculated for post-1970 data alone -particularly for high-frequency, low-magnitude events." In short, rainfall patterns and resulting flood frequency in New England are changing and the 1985 FIS does not capture that change in its assessment.

- Limited cross-sectional data in the area of the Ipswich Mills Dam for the 1985 FIS. The cross-sections for the HEC-2 model in the vicinity of the dam stop at the toe of the dam, upstream of the Choate Bridge. Therefore, the impacts of the Choate Bridge are not accurately reflected in the model. A new model would need to be developed with additional cross-sectional data to present more accurate predictions of water surface elevations.

- FIS-mapped flood plains do not appear to reflect real world experience. According to comments from the Dam Removal Feasibility Technical Committee, the water surface elevations observed in the river during specific flood events appear to be noticeably higher than those predicted by the FEMA FIS.

The Ipswich Mills Dam Phase I Inspection / Evaluation was completed by Haley & Aldrich, Inc. in October, 2009. The report highlights the significant features of the Ipswich Mills Dam, including dimensions, elevations, hydraulic capacity, structure design, and condition of the dam. The significant design elevations and dimensions for the dam and its appurtenances are summarized and illustrated in Figure 3.

As stated by Haley & Aldrich (2009), this dam is classified by the MA Office of Dam Safety as an Intermediate dam with Significant Hazard Potential, and failure of the dam would cause property damage and may result in loss of life if the failure occurred without warning and people were within the initial flood-wave. The report also documents the structural condition of the dam at the time of inspection and recommends minor repairs to prevent failure. The safety status of the Ipswich Mills Dam with respect to the need for repair was judged as “satisfactory” by Haley & Aldrich.



NOTES:

1. ALL ELEVATIONS REPORTED ARE IN FEET AND ARE BASED ON THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD).
2. WSEL = WATER SURFACE ELEVATION
3. UNLESS OTHERWISE NOTED, REPORTED ELEVATIONS AND DIMENSIONS IN THIS FIGURE WERE OBTAINED FROM THE IPSWICH MILLS DAM PHASE INSPECTION / EVALUATION REPORT (HALEY & ALDRICH, 2009).
4. ELEVATIONS WERE OBTAINED FROM THE *PEDESTRIAN BRIDGE OVER THE IPSWICH RIVER CONSTRUCTION PLANS* (BETA GROUP, INC. & MASSHIGHWAY, 2001).
5. ELEVATIONS WERE OBTAINED FROM THE IPSWICH, MA FLOOD INSURANCE STUDY (FEMA, 1985).
6. ELEVATIONS WERE APPROXIMATED BY IRWA FROM PHOTO DOCUMENTATION ON MAY 16, 2006 (SEE FIGURES 2A & 2B) IN CONJUNCTION WITH THE REFERENCES LISTED IN NOTES 3 AND 4.

Project Number: *11101*	Registration:	Prepared For: Town of Ipswich 25 Green Street Ipswich, MA 01938	Plan Set: *IPSWICH MILLS DAM CROSS-SECTION* *IPSWICH, MASSACHUSETTS*	**Horsley Witten Group, Inc.** **Sustainable Environmental Solutions** **90 Route 6A** **Sandwich, MA 02563** **508-833-6600 voice** **508-833-3150 fax**
Sheet Number: *1 of 1*			Plan Title: *FIGURE 3*	Date: 06/04/2012 · Designed By: -- · Drawn By: KH · Checked By: EB

Historic accounts indicate that the Ipswich Mills Dam was built upon or just downstream of a rock ledge outcrop or small rock rapids, referred to as the 'upper falls.' According to Dam Removal Feasibility Technical Committee, the 'upper falls' likely extends from the Ipswich Mills Dam upstream approximately 10 feet. Reference to the upper falls is made in the historic accounts the Proceedings at the Annual meeting of the Ipswich Historical Society XIII, December 7, 1903, Page 24, and is supported by a diagram of downtown Ipswich in the late 17th century indicating a fording location on the river, which would naturally be a shallow firm surface, at approximately the location of the Ipswich Mills Dam before it was built (Ipswich in the Massachusetts Bay Colony 1633-1700; Thomas Franklin Waters, 1905). IRWA performed a preliminary field survey of the stream cross section approximately 10 feet upstream of the dam and observed refusal at the stream bottom, strongly suggesting the presence of ledge rather than sediment on the river bottom. This ledge outcrop is not identified in the FEMA FIS analysis but is visible during low flows.

4. Results and Discussion

The Ipswich Mills Dam is a run of the river dam that was built for the purpose of generating power for nearby buildings and manufacturing processes. It no longer serves that purpose and now stands as a relic structure in the river. A run of the river dam is operated such that the volume of water released below the dam is equal to the volume of water flowing in the stream or river above the dam on a continuous, real-time basis. Put another way, water is not stored in the impoundment to be released at a later time. Rather, the dam simply increases the head in the river, providing a power source that can be captured. This is typical of many small New England dams. It has five low level gates that, when originally installed, could be removed manually to adjust the water level in the River. However, as described in the 2009 dam safety inspection report (Haley and Aldrich, 2009), three of those gates have since been plugged, one has been fitted with a stainless steel slide gate operated by a handwheel and one controls flow to the fish ladder.

The dam itself was not constructed to provide flood control for the area downstream of the dam, and does not serve that purpose by default (Figure 4a). The dam provides relatively little storage (small head pond) by detaining flow behind the dam, and what is detained is actually occupying or using up a small portion of the flood storage capacity that would naturally be available in the flood plain in the absence of the dam. Because of its minimal storage capacity, this dam does not provide flood mitigation for areas downstream of the dam. Flows downstream of the river are essentially equal to what they would be in the absence of the dam because the river has created an equilibrium in which water flowing to the dam equals water flowing over and downstream from the dam.

As presented in Figure 3, there is only one foot of elevation difference between the low flow spillway crest and the overflow spillway crest. The storage volume between the two spillway crests is therefore minimal. The FEMA FIS plainly states in Section 2.4 Flood Protection Measures under the description of the Area Studied that 'These dams [including the Ipswich Mills Dam] are used for water power, and none affects flood flows." The exact extent of the impoundment is not clear from previous reports and estimates. According to Haley and Aldrich (2009), the impoundment from the dam extends upstream approximately 12,500 feet at an average width of 70 feet, and has a total surface area less than 1% of its contributing watershed area. The total volume of the impoundment at these measurements would be about 100 acre-feet, or about half the volume between the Mills and Willowdale dams. Conversely, preliminary field reconnaissance by IRWA suggests that the impoundment extends only 7,500 feet upstream to the commuter rail bridge (MacDougall, email correspondence, November 6, 2010).

In contrast, flood control dams and large power generation dams generally do not allow significant flows over the dam because the flows are regulated through a designated discharge near the base of the dam (Figure 4b). The significant storage volume behind the dam allows the dam to detain water so that flows downstream can be regulated, thus mitigating floods.

Typically, the removal of a run of the river dam would result in a reduction of the water surface elevation at and upstream of the dam location such that a new equilibrium is established (or rather, restored), while downstream water surface elevations typically do not change. Essentially, the relatively small volume of water stored behind the dam is 'released', but the flow toward the dam still equals the flow out below the dam and the natural water surface elevation is re-established.

In the case of the Ipswich Mills Dam, where it is presumed that a ledge outcrop and falls extends from the dam toe to approximately 10 feet upstream of the dam, it is reasonable to expect that once the dam is removed, the falls will become the new defining element in the river and will establish the new upstream water surface elevation during normal or low flow conditions. However, during flood flows, the existing dam and the rock ledge outcrop (or Upper Falls as it is commonly referenced) appear to have little impact on the water surface elevation or the river discharge due to the presence of numerous other impediments to flow, including the Choate Bridge, the pedestrian foot bridge, downstream tidal influence, and the sharp bend in the river downstream of the Choate Bridge. The amount of influence each of these impediments has on the current system is unknown at this time but can be estimated with future evaluation.

One of the most accurate and widely used methods for predicting and quantifying flood flows and water surface elevations is to create a hydraulic model using U.S. ACOE HEC software. A good example of a relevant HEC model was performed as part of the 2006 Feasibility Study for the Willowdale Dam Fish Passage Project (Alden Research Laboratory, 2006) upstream of the Ipswich Mills Dam. HEC analyses are also commonly used by FEMA for estimating flood zones and flood elevations. Models such as these can provide insight into how a dam or other flow impediments may contribute and affect flood elevations. In order to create such a model, it is necessary to obtain detailed cross-sectional geometry for the stream reach of interest. In the case of the Ipswich Mills Dam, cross-sections would be needed above and below the following locations: the Choate Bridge crossing at Route 133/South Main Street, the Ipswich Mills Dam, the Boston/Maine railroad crossing near the confluence with the Miles River, the Willowdale Dam, and any intermediate road crossings. Some of these data were collected by FEMA in 1985 and modeled for the 1985 Flood Insurance Study but how well they reflect current conditions is uncertain.

Figure 4a. The Ipswich Mills Dam is a run-of-the-river mill dam formerly used to power the nearby mills. (Horsley Witten Group, November 2011)

Figure 4b. The Westville Lake Flood Control Dam on the Quinnebaug River in Southbridge, MA serves to control flooding to downstream areas. This dam serves an entirely different purpose than the Ipswich mills Dam. (US Army Corps of Engineers)

A HEC analysis is not part of this report's scope of work, but would be a key task in the next phase of a feasibility study, should such a study be undertaken. Instead, two other dam removal feasibility studies that rely on HEC analyses for flood predictions are presented and discussed as examples here in order to correlate results and make educated assumptions for the Ipswich Mills Dam project area.

In 2008, a dam removal feasibility study was completed for the Mill River in Taunton, MA which utilized HEC-RAS to estimate flood elevation changes associated with the removal of three existing "run-of-the-river" dams. The modeling results show that the depths at and upstream of the dam locations were expected to decrease under all flow conditions up to and including the 100-year frequency storm event (Woodlot Associates, Inc. & Inter-fluve, Inc., 2008). Typically, the largest decreases in water levels were shown just upstream of the existing dam locations, or at the deepest part of the reservoirs. The expected reduction in water depth declined upstream, moving away from the influence of the impoundment. Little to no change in water surface elevation was shown at the most downstream location following dam removal.

These same hydraulic changes were predicted by a similar dam removal feasibility study completed in 2010 for the Curtis Pond Dam in Danvers, MA. The Curtis Pond Dam removal feasibility study evaluated the expected water level change upstream and downstream of the dam for the 2-, 10-, 25-, 50-, and 100-year frequency events. Water levels were anticipated to drop approximately five-feet in Curtis Pond following removal and no change in water level was predicted downstream of the dam (Pare Corporation, Kleinfelder/SEA Consultants, IRWA; 2010).

When considering the effects on the Ipswich Mills Dam system, a predicted decrease of upstream water depth and water surface elevation will likely result in proportionally decreased local groundwater levels (greatest groundwater declines closest to the dam removal site). Loss of nearby wetland resource areas upstream of the dam is also a possibility, but further investigation would be required to identify the wetland areas of concern and determine if they are primarily groundwater or surface water dependant. It is not expected that a groundwater level decrease would impact drinking water availability since there are no documented pumping sources in the vicinity of the impoundment area (Ipswich Utilities, 2012). Long-term downstream water elevations are not anticipated to change because the amount of water impounded by the dam is insignificant in relation to the normal flow of the river.

One factor at the project area that may play an important role in governing the downstream flow conditions, perhaps more so than the Ipswich Mills Dam itself, is the Choate Bridge river crossing. The Choate Bridge currently acts as a flow restriction due to its limited open cross-sectional area. The extent of this flow restriction and the impacts of the bridge span on the flow of the river during various high flow scenarios are not known at this time. In addition, given that the Ipswich Mills Dam represents the approximate head of tide in the river, the tide itself creates an additional influence on the downstream flow in the river and the resulting flood elevations. In order to more fully understand the likely impact of the Choate Bridge and head of tide on the river flow and elevation under both current and potential dam removal conditions, a detailed hydraulic and hydrologic analysis, including a HEC model, must be performed. This involves measuring the cross-sections of the river and evaluating the surficial characteristics (roughness) at various key locations extending from upstream of the dam to well downstream of the Choate Bridge, and then creating a three-dimensional model of the river through which various flows can be input to estimate the water surface elevations throughout the study area.

5. Conclusions

Many factors must be considered when deciding whether or not to remove a dam, including the hydrologic and hydraulic factors presented in this preliminary assessment. Based upon the information compiled and reviewed for this assessment, it seems relatively clear that the dam no longer serves its initial intended purpose of providing a small scale energy source for the surrounding mill activities. Because of the dam's basic design and relatively small size, it does not provide significant active (regulated) flood mitigation services for areas downstream of the dam. While the head pond created behind the dam is relatively small in comparison to the average annual and average monthly discharge passing over the dam, the dam does raise the surface elevation of the water upstream of the dam above what would exist in the absence of the dam. Based on historical records and anecdotal observations reported during low flow conditions, it is generally believed that the dam was constructed on top of or at the toe of a rock ledge outcrop that created the Upper Falls. The extent of that ledge is yet to be determined, but it would be expected that in the absence of the dam, the height of the rock ledge rock ledge will be a primary factor in determining the normal or low water surface elevations. The Mill River and Curtis Pond dam removal feasibility studies serve as good examples of how to evaluate the expected conditions associated with removing a run of the river mill dam.

6. Scope of Work for Full Hydrologic and Hydraulic Analysis of Ipswich Mills Dam Removal

The next steps for this feasibility assessment are to develop a more detailed understanding of the flows (discharge, surface elevation, velocities) in the river under existing conditions from the area upstream of the Ipswich Mills Dam to downstream of the Choate Bridge, and to use that information to predict the conditions in the river under the potential dam removal scenario. It is important for the town to understand what impact the dam is having on the flow regime in the river (both high flows and low flows) and to develop an understanding of the potential risks and benefits from dam removal. This includes estimating the future river water surface elevations and the flow velocities in the area of the dam if it were to be removed. This would need to be evaluated under all flow conditions (i.e., low, normal, and flood flows) to gain an informed understanding of the impact of dam removal.

A basic tool in developing this understanding is the HEC-RAS model, which is publically available from the Army Corps of Engineers and is the industry standard in modeling river and stream hydraulics. Data requirements for this type of analysis include obtaining detailed cross-sectional data and Manning's roughness coefficient inputs that are representative of current conditions in the river. As previously described, data originally used by FEMA in the 1985 Flood Insurance Study are unlikely to represent current conditions due to continued development in the watershed since the time the data were collected and subsequent changes to flow rates, flow patterns, rainfall, erosion and sedimentation. It is highly probable that the current hydrology for the Ipswich River Watershed varies greatly from the conditions observed at the time the Flood Insurance Study was prepared. Development and land use changes can have significant impacts on a river's flow regime. Therefore, the flow rates at the Ipswich Mills Dam should be reassessed, which can accurately be performed since flow measurements have been recorded for the last 80 years at the Willowdale Dam. The USGS has published and made available sufficient resources and documentation to develop a new discharge-frequency relationship for the project area. Any necessary revisions can be applied to the HEC analysis and used to develop more representative flood predictions for current hydrologic conditions, as well as possible future scenarios.

7. References

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APPENDIX A. USGS Water Data Report 2009, Ipswich River Gage near Willowdale Dam

01102000 IPSWICH RIVER NEAR IPSWICH, MA

MASSACHUSETTS-RHODE ISLAND COASTAL BASIN
IPSWICH RIVER SUBBASIN

LOCATION.--Lat 42°39′35″, long 70°53′39″ referenced to North American Datum of 1927, Essex County, MA, Hydrologic Unit 01090001, on left bank 200 ft downstream from Willowdale Dam, 1.5 mi downstream from Howlett Brook, and 4 mi upstream from Ipswich.

DRAINAGE AREA.--125 mi².

SURFACE-WATER RECORDS

PERIOD OF RECORD.--Discharge: June 1930 to current year. Water-quality records: water years 1954, 1976-79.

REVISED RECORDS.--WSP 1621: 1930-58 (monthly runoff). WDR MA-RI-84-1: Drainage area.

GAGE.--Water-stage recorder with satellite telemeter. Datum of gage is 20.63 ft above National Geodetic Vertical Datum of 1929.

COOPERATION.--Massachusetts Department of Conservation and Recreation, Water Resources Commission; Massachusetts Department of Environmental Protection, Office of Watershed Management; and Massachusetts Executive Office of Energy and Environmental Affairs.

REMARKS.--Records good except those for estimated daily discharge, which are poor. Diversions upstream for municipal supply of Reading, Lynn, Peabody, Danvers, Salem, and Beverly. Some regulation by reservoirs upstream.

01102000 IPSWICH RIVER NEAR IPSWICH, MA—Continued

DISCHARGE, CUBIC FEET PER SECOND
WATER YEAR OCTOBER 2008 TO SEPTEMBER 2009
DAILY MEAN VALUES
[e, estimated]

Day	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1	442	135	380	e569	192	506	320	227	89	253	977	211
2	431	131	410	e530	199	416	328	206	87	295	590	203
3	391	130	440	e481	204	411	327	189	82	352	441	179
4	351	125	443	e436	198	429	349	175	78	410	389	152
5	314	123	413	380	192	417	375	168	64	451	348	130
6	274	122	358	e359	186	391	412	176	68	456	311	114
7	241	137	313	326	177	397	485	200	66	431	273	103
8	211	150	e264	330	175	443	541	230	58	431	239	95
9	189	161	203	e344	178	474	558	254	54	423	213	89
10	166	167	187	e335	180	530	521	261	51	411	193	82
11	148	164	198	299	183	589	479	249	47	387	170	78
12	134	150	416	285	207	654	451	227	67	368	150	91
13	122	135	767	e275	251	646	423	203	77	342	135	96
14	112	129	1,020	e250	287	603	397	182	103	314	122	94
15	104	126	1,050	e236	313	547	365	167	138	292	110	93
16	95	136	925	e214	323	498	334	152	154	262	103	96
17	88	135	801	193	313	459	303	145	157	230	97	95
18	83	134	697	179	297	424	279	139	157	211	91	91
19	78	135	595	169	298	393	260	134	175	190	85	84
20	74	126	441	165	319	367	241	120	185	170	80	76
21	72	116	398	162	341	343	260	108	194	154	73	71
22	71	108	376	158	361	319	312	96	219	148	69	68
23	67	100	e390	157	420	297	368	87	244	145	68	65
24	67	95	e379	156	462	278	403	80	261	267	74	60
25	67	123	e385	159	471	262	400	74	268	477	86	56
26	79	177	e442	159	449	243	372	68	269	583	93	50
27	85	242	460	157	437	235	341	63	256	588	99	48
28	99	316	597	154	498	233	312	67	233	527	100	50
29	115	358	897	161	---	246	287	71	218	444	133	61
30	129	359	740	172	---	272	252	81	231	429	172	61
31	134	---	e636	181	---	297	---	87	---	542	204	---
Total	5,033	4,745	16,021	8,131	8,111	12,619	11,055	4,686	4,350	10,983	6,288	2,842
Mean	162	158	517	262	290	407	368	151	145	354	203	94.7
Max	442	359	1,050	569	498	654	558	261	269	588	977	211
Min	67	95	187	154	175	233	241	63	47	145	68	48
Cfsm	1.30	1.27	4.13	2.10	2.32	3.26	2.95	1.21	1.16	2.83	1.62	0.76
In.	1.50	1.41	4.77	2.42	2.41	3.76	3.29	1.39	1.29	3.27	1.87	0.85

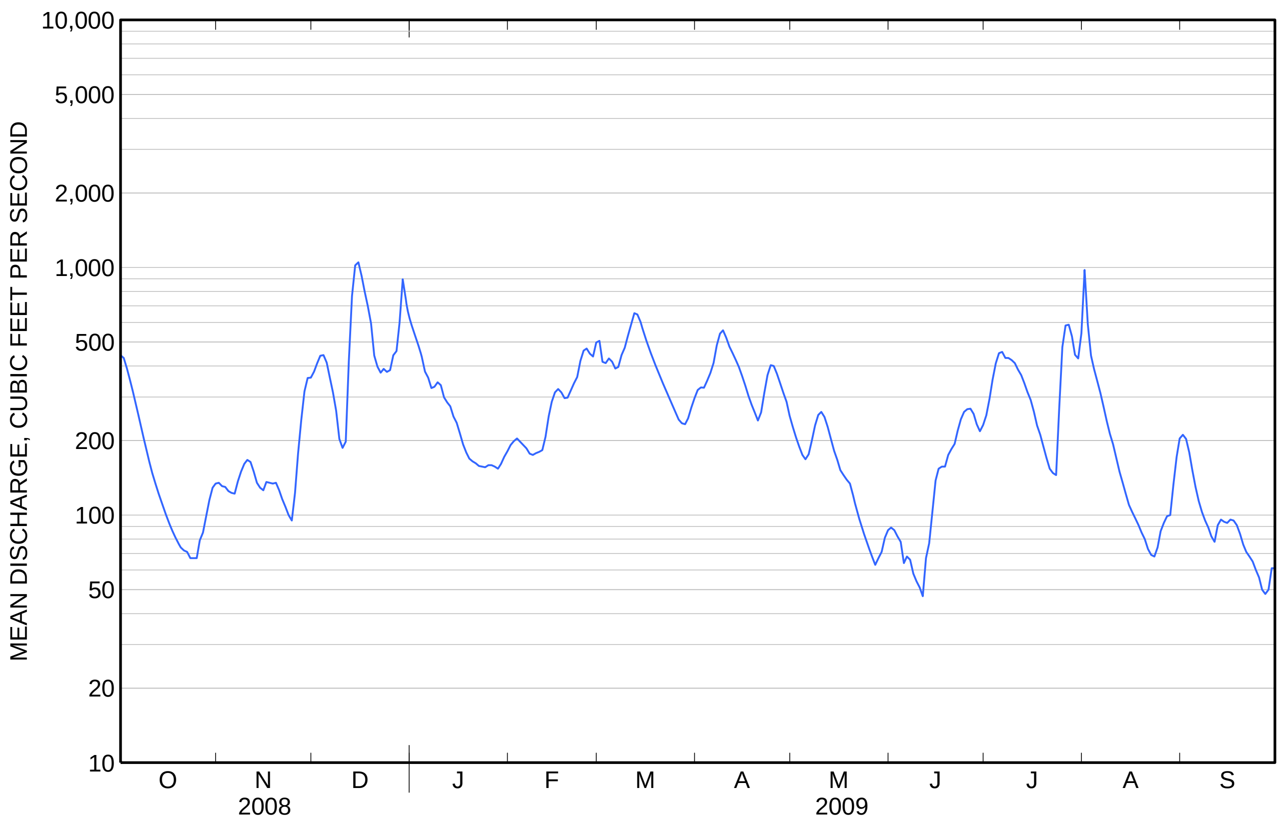
STATISTICS OF MONTHLY MEAN DATA FOR WATER YEARS 1930 - 2009, BY WATER YEAR (WY)

	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Mean	81.5	140	198	212	247	446	443	257	160	64.7	42.0	43.7
Max	749	525	621	566	675	1,158	1,233	1,309	821	518	356	390
(WY)	(1997)	(1933)	(1997)	(1958)	(2008)	(1983)	(1987)	(2006)	(1982)	(1938)	(1938)	(1954)
Min	4.75	6.87	11.5	14.4	16.4	75.0	97.1	83.5	25.6	5.75	2.13	1.49
(WY)	(1998)	(1966)	(1966)	(1966)	(1980)	(1989)	(1985)	(1999)	(1976)	(1957)	(1965)	(2005)

01102000 IPSWICH RIVER NEAR IPSWICH, MA—Continued

SUMMARY STATISTICS

	Calendar Year 2008		Water Year 2009		Water Years 1930 - 2009	
Annual total	105,459		94,864			
Annual mean	288		260		194	
Highest annual mean					374	2006
Lowest annual mean					57.7	1966
Highest daily mean	1,560	Mar 10	1,050	Dec 15	4,550	May 16, 2006
Lowest daily mean	24	Sep 5	47	Jun 11	0.59	Sep 21, 1978
Annual seven-day minimum	33	Aug 31	55	Sep 24	0.90	Oct 1, 2005
Maximum peak flow			1,120	Aug 1	4,600	May 16, 2006
Maximum peak stage			5.54	Aug 1	10.53	May 16, 2006
Instantaneous low flow			20	Jun 5	0.34	Sep 20, 1978
Annual runoff (cfsm)	2.31		2.08		1.55	
Annual runoff (inches)	31.38		28.23		21.11	
10 percent exceeds	668		472		454	
50 percent exceeds	178		211		118	
90 percent exceeds	55		78		12	



SECTION 2:

Evaluation of Potential Impacts on EBSCO Buildings from the Proposed Removal of Ipswich Mills Dam

GEI Consultants

16 Pages

Memo

To:	Mr. Glenn Gibbs, Town of Ipswich
From:	Michael A. Yako, P.E.
c:	Brian Kelder, Ipswich River Watershed Association Giuliana Zelada-Tumialan, P.E., Simpson Gumpertz & Heger, Inc.
Date:	Revised February 14, 2014,December 13, 2013
Re:	Evaluation of Potential Impacts on EBSCO Buildings from Proposed Removal of Ipswich Mills Dam Ipswich, Massachusetts GEI Project No. 1325760

The purpose of this memorandum is to present the results of our evaluation of the potential impacts of the removal of the Ipswich Mills Dam on the EBSCO buildings. We prepared this interim memorandum based on: our review of documents provided by the Town of Ipswich, Ipswich River Watershed Association, and EBSCO; discussions with you and Mr. Brian Kelder; and our August 27, 2013 site visit.

Our work for this project was authorized by a signed agreement between the Town of Ipswich and GEI dated May 14, 2013. GEI was assisted on this project by Ms. Giuliana Zelada-Tumialan of Simpson Gumpertz & Heger, Inc. (SGH).

Summary

The portion of the EBSCO buildings along the Ipswich River are likely supported on timber piles given the soil conditions along the river and the age of the buildings. We were unable to identify any information regarding the elevation of the tops of the suspected timber piles; consequently, we don't know if the removal of the dam would likely expose the tops of the piles causing them to deteriorate resulting in damage to the building.

As discussed in more detail below, we recommend performing the following additional work: literature search for historic records of the former dams; lowering of the impoundment for maximum exposure of the EBSCO foundation wall along the river; soil probing along the EBSCO foundation wall in the river; a more extensive river sounding program; and possibly coring through the EBSCO foundations or excavating test pits inside the EBSCO building to expose the foundations.

Background Information

We understand that the Town would like to remove the Town-owned Ipswich Mills Dam on the Ipswich River. The location of the dam is shown in Figs. 1 and 2. It is expected that removal of the dam would enhance fish passage and ecological connections between the river, estuary, and ocean, and reduce the Town's liability and maintenance costs associated with the dam. However, removal of the dam could affect the foundations supporting the EBSCO buildings. In particular if the buildings are supported on timber piles and if the river level drops below the tops of the piles, the piles could deteriorate leading to settlement and damage to the buildings.

400 Unicorn Park Drive
Woburn, MA 01801
781.721.4000 fax 781.721.4073

Our work is part of a larger study that also included an evaluation of the influence of the dam on upstream and downstream flooding and an evaluation of the quality of the sediment behind the dam.

Ipswich Mills Dam and River Bed Topography

The first dam at this location was constructed in 1637. A number of larger dams were constructed over the years with the current dam being constructed or reconstructed in about 1908. The dams were constructed to provide power to the industries along the river. The dam is currently owned by the Town of Ipswich and no longer serves its original purpose.

The current dam is located about 4 miles from the mouth of the Ipswich River and sits on a bedrock outcrop referred to as the Upper Falls. The river is tidal below the dam.

The current dam is constructed out of cut stones with concrete at some locations and is a run of the river dam with the spillway extending across most of the width of the river. The main spillway is 132 feet wide. A 3-foot-wide low level stop-log spillway is at the right end of the main spillway. The spillway crest is at El. 9.7[1] and the low level stop-log spillway invert is at El. 8.7. The dam also has a 4.5-foot-wide by 3-foot-high low level gated outlet with an invert at El. 7.5 on the right side of the dam. The right side of the dam also includes a fish ladder and a non-overflow granite block wall or pier that extends approximately 45 feet into the river and abuts the right end of the spillway.

We were provided the results of soundings performed across the width of the river 10 feet upstream of the dam. The information from the soundings is provided in Appendix A. We were told that the soundings represent the top of bedrock upstream of the dam. Consequently, if the dam is removed, the river should not drop below the lowest elevation of the bedrock upstream of the dam or El. 6.3. This would represent a drop of 3 feet below the dam spillway elevation and a drop of 1.2 feet below the invert of the low level outlet.

EBSCO Buildings

The EBSCO Information Services' buildings that are the subject of this evaluation are the buildings located on the west side of the river immediately upstream of the dam as shown in Figs. 1 and 2. These buildings, which are identified as Nos. 9, 10, and 10-A in the plan in Appendix B, were constructed between approximately 1901 and 1912. Building Nos. 6, 7, 8, and a portion of No. 9 no longer exist. No other plans are reportedly available for the buildings.

As shown in the plan in Appendix B and in Figs. 1 and 2, the east side of Building Nos. 9, 10, and 10-A are located immediately on the property line and buildings directly abut the river for most of their length. As shown in Figs. 1 and 2 and based on our observations and review of historic aerial photographs, the ground surface is above the river level along a portion of the buildings.

Mr. Thomas Wheeler of EBSCO provided GEI with the logs of three soil borings that were performed in 2009 immediately south of the southeast corner of Building No. 10-A. The general location of the borings is shown in Fig. 1 and a more detailed plan showing the boring locations is provided in Appendix C. Logs of the borings are also contained in Appendix C. The two borings closest to the river (B-3 and B-4) both encountered about 16 feet of loose fill, soft to medium stiff

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[1] Elevations in this report are in feet and are based on the National Geodetic Vertical Datum (NGVD).

clay, and organic soils (peat) overlying very dense glacial till. Boring B-2, which is located further from the river encountered 10 feet of fill overlying very dense glacial till. B-2 did not encounter clay or peat.

The fill, clay, and peat are not a suitable bearing layer for support of the buildings. Consequently, it is our opinion that at least some portion of the EBSCO buildings along the river are likely supported on deep foundations. Given the age of the buildings, we anticipate that the deep foundations likely consist of driven timber piles. Portions of the buildings further from the river are likely supported on spread footings where the glacial till is much shallower and where the foundation excavations could be dewatered and not impacted by large groundwater inflows from the river.

Some of the existing and former buildings pre-date the construction or reconstruction of the existing dam. The crest elevation of the former dams and the elevation of the river at the time of construction of the older buildings are not known. It is possible that the tops of the foundations supporting the buildings that pre-date the current dam may have been constructed when the impoundment behind the dam was maintained at a lower elevation. Consequently, the tops of the timber piles supporting these older buildings could have been established based on a lower impoundment elevation and may not be at risk of biodeterioration from removal of the dam.

We were able to view a very limited portion of the exterior brick façade at the southeast corner of Building 10-A. The façade appeared to be in good condition with no readily visible signs of distress.

Mr. Wheeler provided a tour of the interior of their buildings to Ms. Zelada-Tumialan of SGH and Mr. Michael Yako of GEI. We did not observe any building features that provided any indication as to the type of foundations supporting the buildings along the river. We observed some dishing of the first floor slab indicating that the slab is a slab-on-grade and that the soils underlying the slab are compressible and had settled. This is consistent with the soils encountered in the borings discussed above.

Following our August 27, 2013 site visit, we visited the Building Inspector’s office and reviewed their files for the EBSCO buildings; however, we were not successful in finding any information in the Town’s files about the EBSCO building foundations.

We requested permission to probe along the foundation wall along the river to try and identify the bottom of the foundation wall. This would provide some information about the tops of the piles for the exterior foundation wall along the river. However, EBSCO required that the Town and GEI assume all liability in the event contaminated materials were encountered while probing or performing any other intrusive investigations. Since the Town and GEI couldn’t assume the liability, no intrusive investigations were performed.

Methods of Protecting EBSCO’s Timber Piles

Assuming that portions of the EBSCO buildings are supported on timber piles, the tops of the timber piles need to be below water to protect them from rapid deterioration (biodeterioration).

Methods that have been implemented on other projects to protect timber piles have included artificially raising groundwater levels to keep the piles submerged, lowering the tops of the piles below the expected future groundwater level, or a combination of raising groundwater levels and cutting off the tops of the piles.

Artificially raising the groundwater level involves installing a series of discrete points or pipes below the building floor slab through which water is recharged into the ground. Instrumentation to monitor groundwater levels is needed to confirm that the groundwater level is maintained above the tops of the piles and below the floor slab. A significant amount of work would need to be performed from inside the building. To improve the effectiveness of the recharge system, a barrier wall may be needed in the river immediately alongside the EBSCO building. The purpose of the barrier wall is to retain the recharged water below the building to reduce the volume of water required to be recharged and to help maintain relatively uniform water levels below the building. The 550- to 600-foot long barrier wall could be constructed using steel or vinyl sheet piles. A barrier wall may not be needed if the existing foundation walls extend into the clay layer, which would serve to retain the water below the building because of its relatively low permeability.

Other significantly more costly and intrusive methods include cutting off the tops of the timber piles that extend above the future water level and replacing the cutoff section of pile with a steel post encased in concrete, or providing entirely new foundations such as drilled mini-piles or helical piers to replace the existing timber piles. To perform this work, groundwater levels below the building would need to be lowered at least 5 to 6 feet below the bottom of the existing floor slab. To lower the groundwater below the building, a steel sheet pile cofferdam would need to be installed along the river side of the building and the south side of the building. Sheet piles cannot be driven on the north side of the building because of the existing granite block retaining wall. Some other means such as jet grouting would need to be used to create a cofferdam.

As discussed below, additional information would be required to evaluate the most appropriate method(s) to protect the timber piles. Based on our experience on other projects, we would expect the cost of installation of a recharge system plus a barrier wall to be in the hundreds of thousands of dollars, and we would expect the cost to cut and post the timber piles would be in excess of a million dollars. These estimates are very preliminary and are intended to only provide an indication of the relative magnitude of the possible cost for the methods discussed. Additional information would be required to refine the estimates.

Recommendations for Additional Work

We recommend that the town consider performing the following additional work:

1. Literature Search – Review the Town's historic records for information on the former dams. In particular information on the date of construction and spillway elevations of the dams may provide information on the top of pile elevations for the various buildings. In addition, newspaper articles from the early 1900s may include information on the construction of the EBSCO buildings.

2. Lower the Impoundment – During a period of relatively low flow, fully open the low level gated outlet to lower the impoundment as far as possible. This will provide maximum exposure of the EBSCO foundation wall along the river and may provide some useful information. However, we expect that the EBSCO foundation walls extend deeper than El. 7.5, the invert of the low level gated outlet. It would be helpful to confirm this.

3. Probe Along the EBSCO Foundation Wall – Based on the plan in Appendix B, the EBSCO buildings are located immediately on the property line. The Town could probe the river bottom along the building off of EBSCO property. This information will supplement the information from the borings, and provide some indication of the extent of the EBSCO

buildings that may be supported on timber piles. No soil or sediment samples would be collected during the probing. If the probe contacts the foundation wall, it may be possible to estimate the elevation of the bottom of the foundation wall/pile cap, which would provide information on the elevation of the tops of the piles along the river side of the EBSCO buildings. The piles supporting interior columns and other portions of the building could be cutoff at different elevations. This probing should be performed in conjunction with lowering the impoundment.

4. Perform River Soundings – Perform a more extensive sounding program to better define the top of bedrock elevations upstream of the dam to evaluate the expected elevation of the impoundment adjacent to the EBSCO building should the dam be removed.

5. Coring Through EBSCO Foundations – If EBSCO provides access to the interior of their buildings, it may be possible to core vertically down through the foundation walls of the three buildings along the river and through the foundations supporting the interior columns of the buildings. This would provide information on the bottom of the foundations/pile caps and the tops of the piles. The coring could be performed on weekends to limit disturbance to EBSCO's operations. A site visit would be required to evaluate whether coring is feasible based on ceiling heights, access to the work areas, and existing use of the space.

6. Test Pits in EBSCO Building – The most definitive way to determine whether the EBSCO buildings are supported on timber piles and the elevations of the timber piles is to perform a series of test pit excavations from inside the building. We would expect that a minimum of 6 test pits would be required. Dewatering of the test pits could be very difficult and costly given the close proximity of the river.

We appreciate the opportunity to be of service to the Town and would be pleased to meet with you to discuss the results of our evaluation.

Please call (781-721-4043) or e-mail (myako@geiconsultants.com) if you have any questions.

Attachments:

Fig. 1 - Aerial View of Project Area
Fig. 2 – EBSCO Building Nos. 9, 10, and 10-A
Appendix A – Riverbed Soundings Data and Plot
Appendix B – Historic Plan Showing EBSCO Building Locations
Appendix C – Boring Location Plan and Boring Logs

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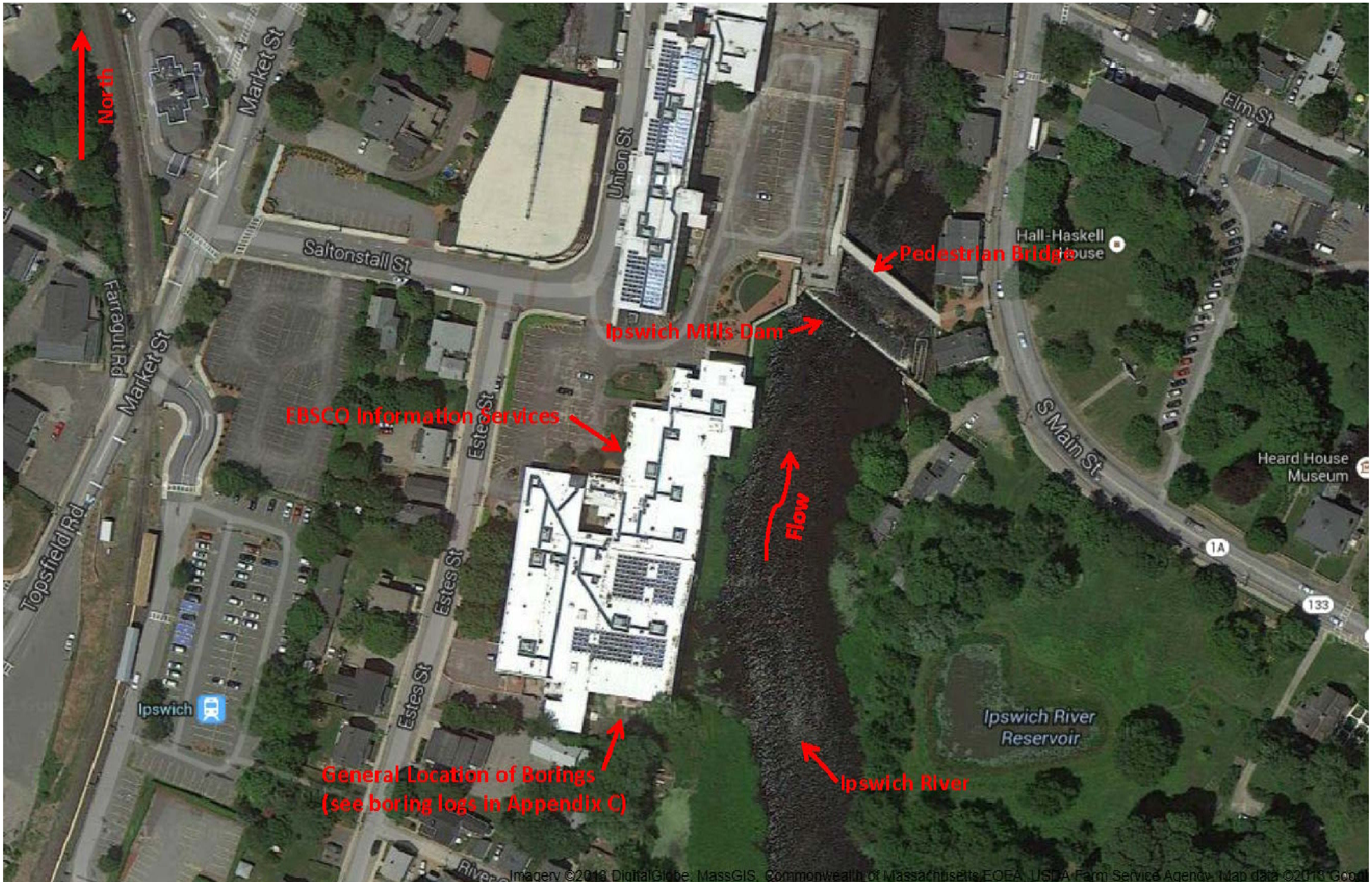


Source: **Google Maps**

0 150 300

APPROXIMATE SCALE, FEET

Evaluation of Ipswich Mills Dam Removal on EBSCO Buildings Ipswich, Massachusetts	GEI Consultants	AERIAL VIEW OF PROJECT AREA
Town of Ipswich Ipswich, Massachusetts	Project 1325760-0	December 2013 Fig. 1

M:\DRAFTING\2013\1325760 Ipswich Mills Dam\1325760-01

Source: **Bing Maps**

Evaluation of Ipswich Mills Dam Removal on EBSCO Buildings Ipswich, Massachusetts	GEI Consultants	EBSCO BUILDING Nos. 9, 10, AND 10-A
Town of Ipswich Ipswich, Massachusetts	Project 1325760-0	December 2013 Fig. 2

M:\DRAFTING\2013\1325760 Ipswich Mills Dam\1325760-01

Appendix A

Riverbed Soundings Data and Plot

Pt		Depth	Station	Elevation
48	0	0.0	0.0	9.3
47	0	1.6	3.4	7.7
45	0	2.6	10.9	6.7
46	0	2.1	13.4	7.2
44	0	1.6	17.4	7.7
43	0	0.6	25.3	8.7
42	0	2.1	35.5	7.2
41	0	2.5	42.3	6.8
40	0	2.4	57.1	6.9
39	0	1.4	65.7	7.9
37	0	0.6	66.6	8.7
38	0	1.9	70.3	7.4
36	0	0.7	73.4	8.6
23	2	2.0	77.6	7.3
32	0	1.1	80.5	8.2
33	0	1.1	82.1	8.2
24	3	3.0	88.7	6.3
34	0	1.1	90.3	8.2
31	0	1.3	97.7	8.0
30	0	2.2	100.0	7.1
26	3	3.0	108.5	6.3
29	0	0.2	110.1	9.1
28	0	2.2	115.9	7.1
51	0	2.9	130.0	6.4
49	0	1.6	140.4	7.7
50	0	0.0	143.1	9.3

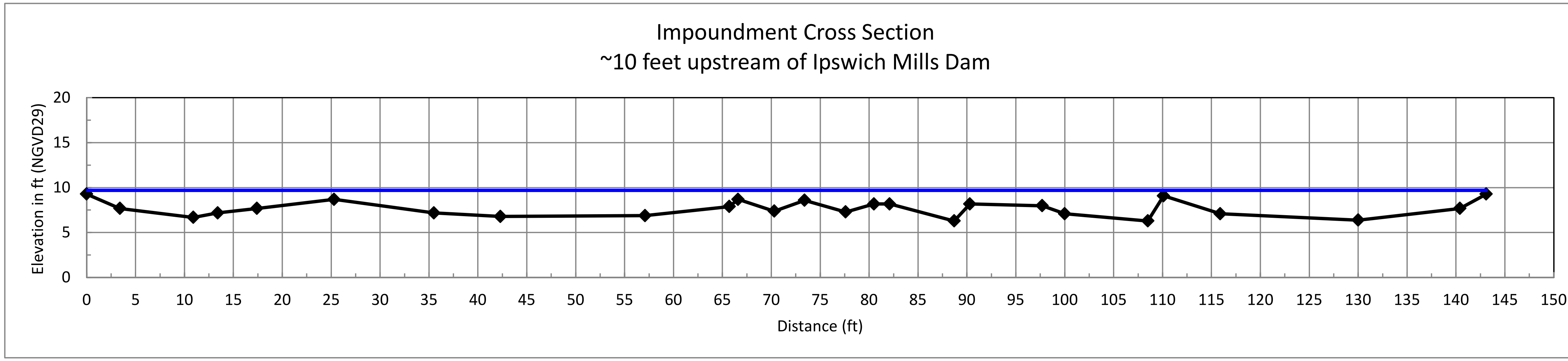
Data not used in the cross section

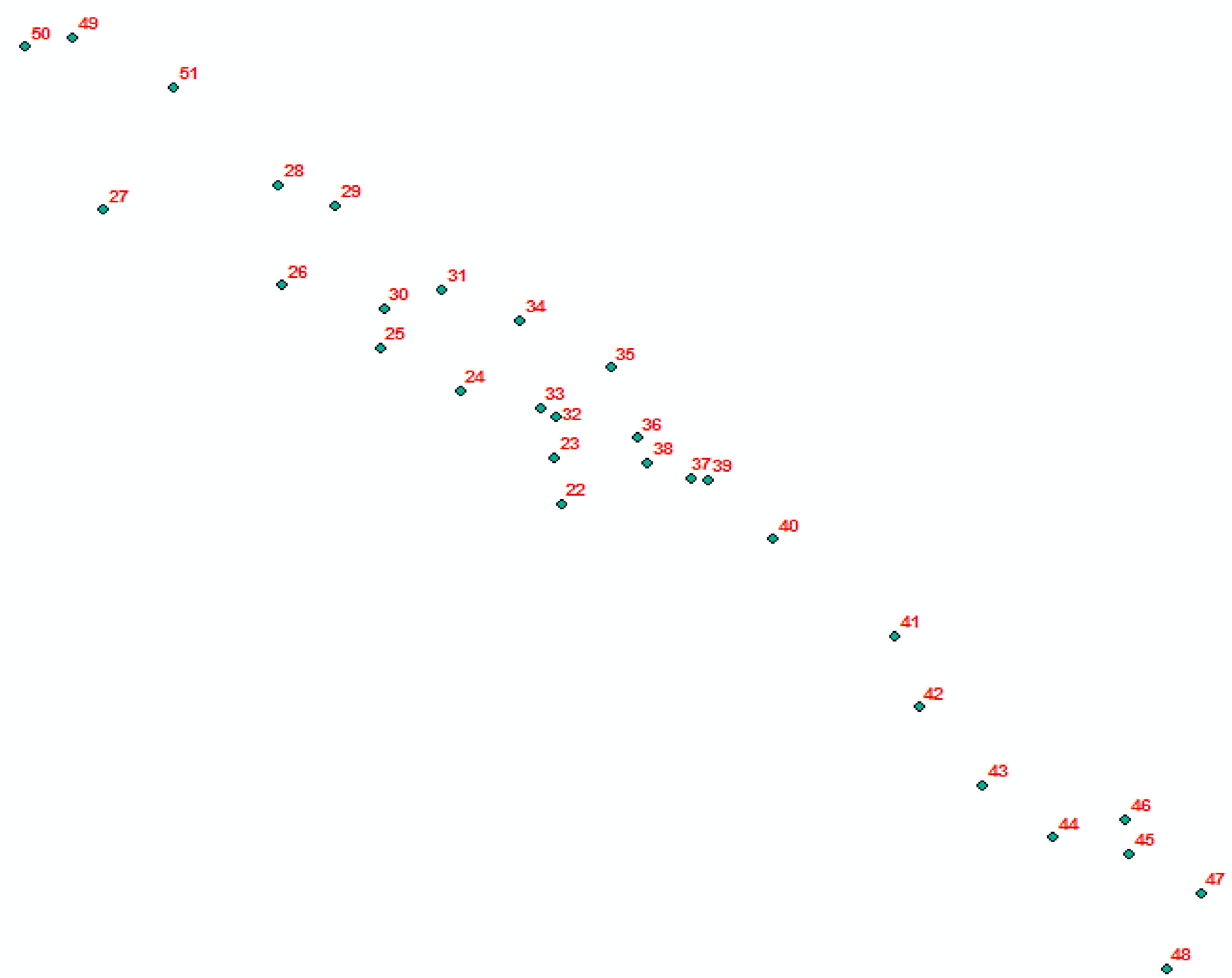
35	0	0.6		8.7
22	3	3.0		6.3
25	2	2.0		7.3
27	4	4.0		5.3

5 inches below spillway

9.3

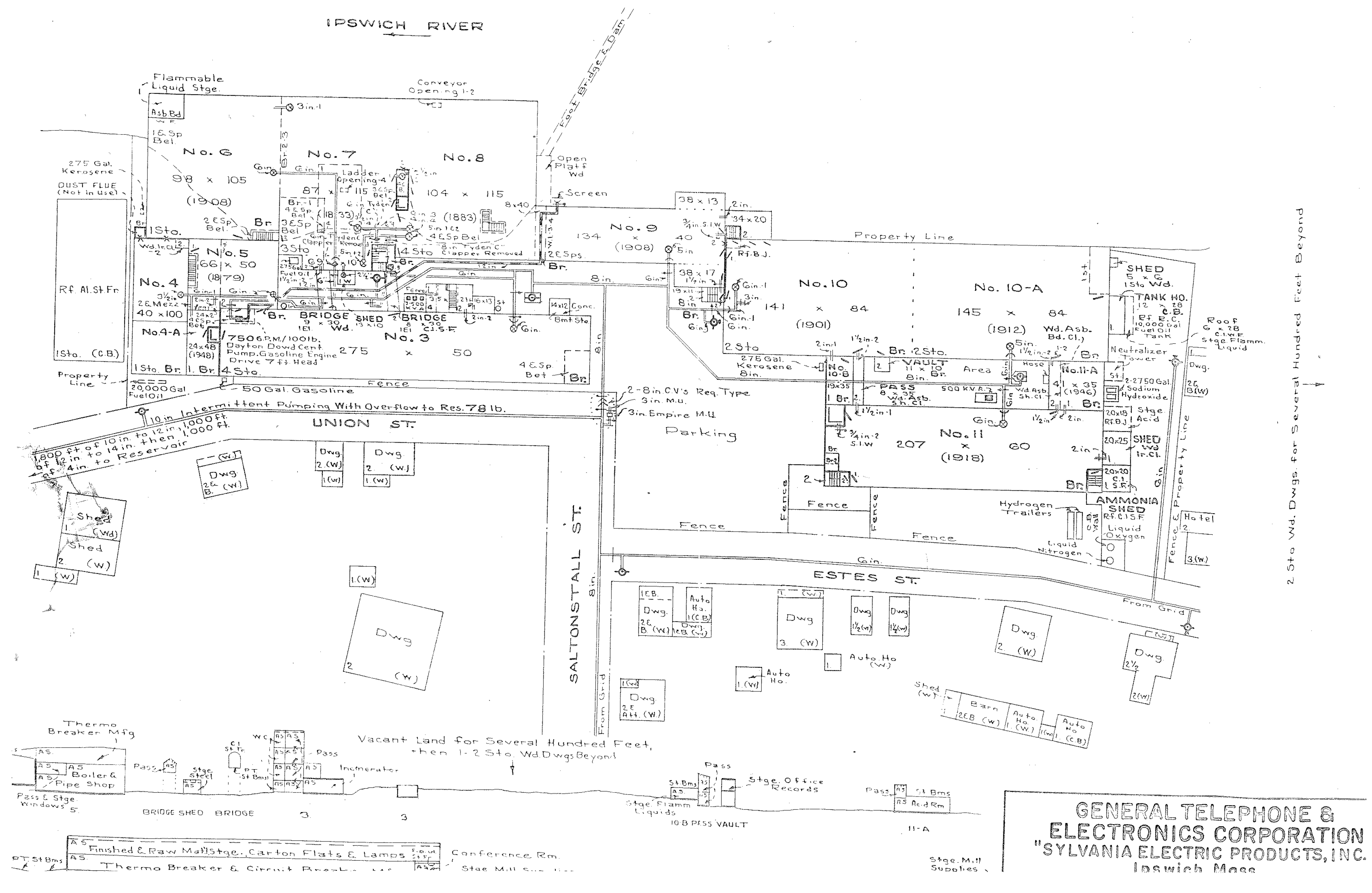
Spillway crest	
0	9.7
143.1	9.7





Appendix B

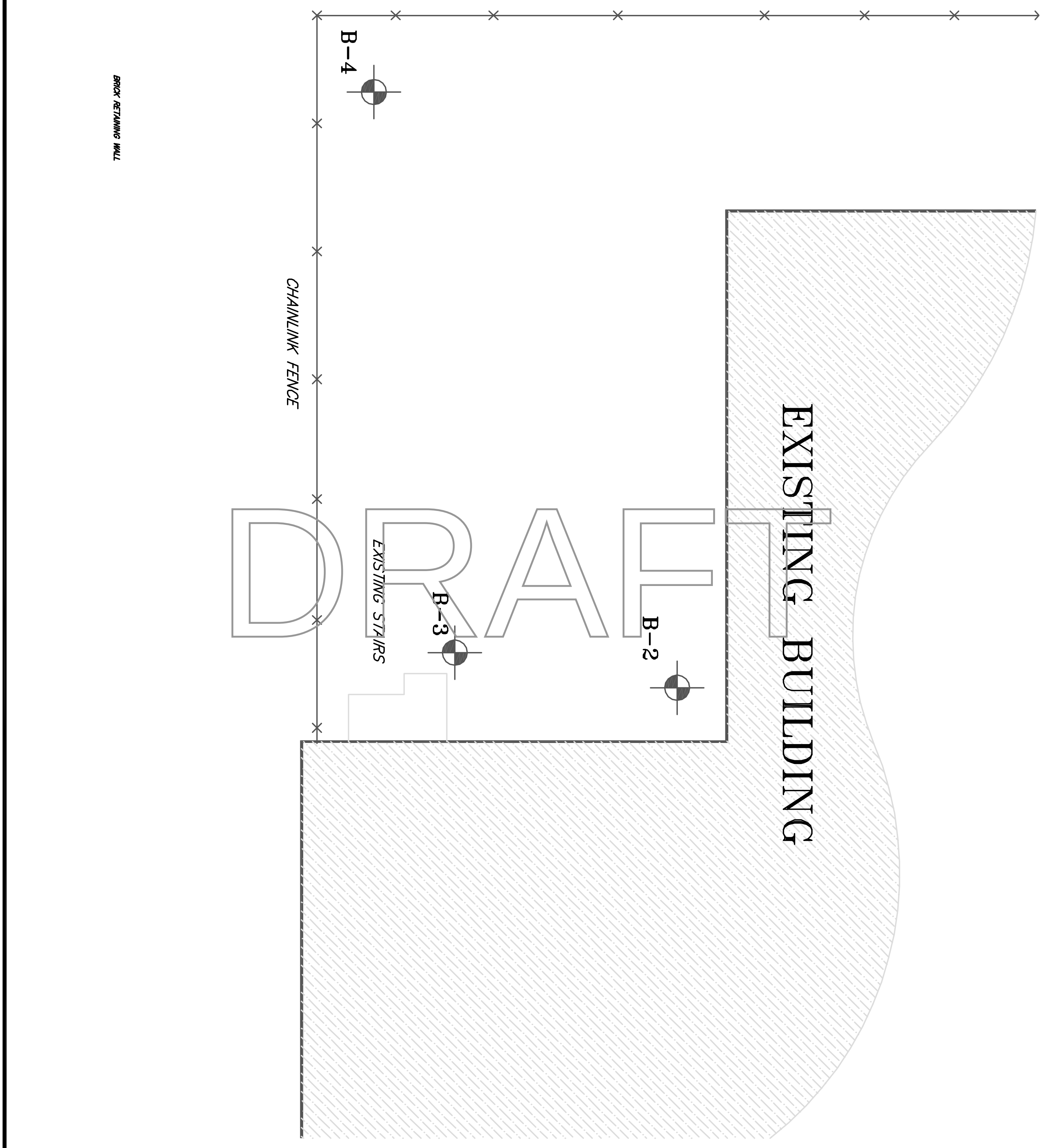
Historic Plan Showing EBSCO Building Locations



IPSWICH RIVER
Flammable Liquid Stge.
Conveyor Opening 1-2
Foot Bridge & Dam
No. 6
98 x 105
(1908)
No. 7
87 x 115
No. 8
104 x 115
(1883)
Open Platf Wd
Screen
275 Gal. Kerosene
DUST FLUE (Not in use)
Rf. Al. St. Fr.
1Sto. (C.B.)
No. 5
66 x 50
(1879)
No. 4
2 & Mezz
40 x 100
No. 4-A
24x48
(1948)
BRIDGE
SHED
BRIDGE
No. 3
275 x 50
750 G.P.M./100 lb. Dayton Dowd Cent. Pump. Gasoline Engine Drive 7 ft. Head
Property Line
20,000 Gal Fuel Oil
50 Gal. Gasoline
Fence
38 x 13
34 x 20
No. 9
134 x 40
(1908)
38 x 17
Property Line
No. 10
141 x 84
(1901)
No. 10-A
145 x 84
(1912)
Wd. Asb. Bd. Cl.
SHED
5 x 6
1 Sto. Wd.
TANK HO.
12 x 28
C.B.
Rf. B.C.
10,000 Gal Fuel Oil Tank
Roof 6 x 28 C.I.W.F. Stge. Flamm. Liquid
275 Gal. Kerosene
No. 10-B
VAULT 11 x 10
Area
No. 11-A
41 x 35
(1946)
Neutralizer Tower
2-2750 Gal. Sodium Hydroxide
PASS 8 x 35 Wd. Asb. Sh. Cl.
500 KVA
Acid
SHED Wd Ir. Cl.
No. 11
207 x 60
(1918)
Hydrogen Trailers
AMMONIA SHED
Liquid Oxygen
Liquid Nitrogen
2-8 in. C.V.'s Req. Type
3 in. M.U.
3 in. Empire M.U.
Parking
10 in. Intermittent Pumping With Overflow to Res. 78 lb.
UNION ST.
1,800 ft. of 10 in. to 12 in., 1,000 ft. of 12 in. to 14 in. then 1,000 ft. of 14 in. to Reservoir
Dwg.
Shed
SALTONSTALL ST.
From Grid
Fence
Fence & Property Line
Hotel
ESTES ST.
From Grid
Auto Ho.
Barn
Vacant Land for Several Hundred Feet, then 1-2 Sto. Wd. Dwgs Beyond
2 Sto Wd. Dwgs for Several Hundred Feet Beyond
Thermo Breaker Mfg
Boiler & Pipe Shop
Pass & Stge. Windows
Stge. Steel
Incinerator
Pass
BRIDGE SHED BRIDGE
Stge. Office Records
Stge. Flamm. Liquids
10-B PASS VAULT
St. Bms
Acid Rm
11-A
Finished & Raw Mat'l Stge., Carton Flats & Lamps
Thermo Breaker & Circuit Breaker
Conference Rm.
Stge. Mill Supplies
GENERAL TELEPHONE & ELECTRONICS CORPORATION
"SYLVANIA ELECTRIC PRODUCTS, INC.
Ipswich Mass

Appendix C

Boring Location Plan and Boring Logs



TITLE:
BORING LOCATION PLAN

PROJECT: **EBSCO PUBLISHING WAREHOUSE EXPANSION**
IPSWICH, MASSACHUSETTS

CLIENT: **PARK CONSTRUCTION CORPORATION**
FITZWILLIAM, New Hampshire

GEOTECHNICAL SERVICES INC.
18 COTE AVENUE, UNIT #11, GOFFSTOWN, NH 03045
TEL. (603) 624-2722 FAX. (603) 624-3733

DATE: JUNE, 22 2009

DESIGN BY:	H. WETHERBEE, P.E.	PROJECT No.	???
DRAWN BY:	D. HAYNER	SHEET No.	1 of 1
CHECKED BY:	H. WETHERBEE, P.E.	SCALE:	NONE

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GSI

TEST BORING LOG

Boring No. B - 2

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel	Truck		Skid		Hammer Type:
Type	HS		SS		Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8		Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140		Tripod		Other		Automatic
Hammer Fall (in.)			30		Winch		Cat Head	X	Roller Bit / Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	5	5-8			Top 3" loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					7-5			
2		S2	2-4	7	1-4			Very Loose to loose wet Brown/Black fine Sand and Silt, trace to little Organics (FILL)
3					6-6			
4								
5								
6		S3	5-7	12	2-1			Very Loose, Moist, Brown/Black fine Sand and Silt, trace organics
7					1-1			
8								
9								
10		S4	10-12	16	10-17			Light Brown, medium dense, wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
11					19-15			
12								
13								
14								
15								
16		S5	15-17	9	15-15			Light Brown, medium dense, wet fine to coarse Sand and Gravel, trace to little Silt
17					21-16			
18								
19								
20								
21		S-6	20-22	18	44-45			Light Brown, dense, wet fine to coarse Sand and Gravel, trace to little Silt
22					38-45			
23								Boring terminated at 22.0 feet without refusal
24								
25								
26								
27								
28								
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	11:30	n/a	n/a	1.0 ft

Sample Identification
O = Open Ended
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30: Very Stiff
Over 30: Hard

Granular Soils N-Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

Notes:

B-2

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TEST BORING LOG

Boring No. B - 3

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit / Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data: No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	5	2-5			Top 3" Very loose to loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					4-3			
2								
3		S2	2-4	18	7-5			Top 6" Loose Dark Brown fine to medium Sand and gravel, little Silt, trace to little Organics (FILL)
4					5-5			Loose, Moist, Light Brown fine Sand and Silt (FILL)
5								
6		S3	5-7	18	1-1			Grey, wet, very soft Clay ,trace black fine sand (in seams) (CLAY)
7					1-1			--- q_u = 1.0 tsf using a pocket penetrometer
8								
9								
10		S4	10-12	18	1-2			Grey, wet, very soft Clay ,trace black fine sand (in seams) --- q_u = 1.0 tsf using a pocket penetrometer
11					2-2			Bottom 4" Black, Wet, fine Sand and Silt with some organics (PEAT)
12								
13								
14								
15								
16		S5	15-17	16	2-7			Light Brown, wet, loose to medium dense fine to medium Sand and Silt with some Clay (TILL)
17					14-17			Bottom 4", Light Brown, medium dense, Wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
18								
19								
20								
21		S-6	20-22	14	33-44			Light Brown, dense, wet, fine to coarse Sand and Gravel, trace to little Silt
22					48-35			
23								
24								
25		S-7	25-27	14	5-19			Light Brown, medium dense, wet, fine to coarse Sand and Gravel, trace to little Silt
26					25-28			
27								
28								Boring terminated at 27 feet without refusal
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	9:30	n/a	n/a	3.5 ft

Sample Identification
- O = Open Ended
- U = Undisturbed
- S = Split Spoon
- C = Rock Core
- G = Geoprobe

Cohesive Soils N-Value
- 0 to 2: Very Soft
- 2 to 4: Soft
- 4 to 8: Medium Stiff
- 8 to 15: Stiff
- 15 to 30: Very Stiff
- Over 30: Hard

Granular Soils N-Value
- 0 to 4: Very Loose
- 4 to 10: Loose
- 11 to 30: Medium Dense
- 31 to 50: Dense
- Over 50: Very Dense

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

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TEST BORING LOG

Boring No. B - 4

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel						Hammer Type:
Type	HS		SS		Truck		Skid			
Type	HS		SS		Track	X	ATV			Safety Hammer
Inside Diameter (in.)	2.25		1-3/8		Bomb		Geoprobe	X		Doughnut
Hammer Weight (lb)			140		Tripod		Other			Automatic
Hammer Fall (in.)			30		Winch		Cat Head	X	Roller Bit	Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data: No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	4	4-4			Top 3" Very loose to loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					4-3			
2		S2	2-4	18	2-1			Very Loose Dark Brown fine to medium Sand and gravel, little Silt, trace to little Organics (FILL)
3					2-3			
4								
5								
6		S3	5-7	18	3-3			Very Loose, Moist, Brown/Black fine Sand and Silt, trace organics
7					2-3			
8								
9								
10		S4	10-12	18	1-5			Grey, Wet, soft Clay ,trace black fine sand (in seams) --- q_u = 1.0 tsf using a pocket penetrometer
11					3-3			Bottom 5" Black, Wet, fine Sand and Silt with some organics (PEAT)
12								
13								
14								
15								
16		S5	15-17	9	16-21			Light Brown, medium dense to dense, wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
17					33-22			
18								
19		S-6	18-20	5	67-38			Light Brown, dense to very dense, wet fine to coarse Sand and Gravel, trace to little Silt
20					60-5"			
21								Refusal at 19.5 feet
22								Boring terminated at 19.5 feet
23								
24								
25								
26								
27								
28								
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	1:30	n/a	n/a	3.5 ft

Sample Identification	Cohesive Soils N-Value	Granular Soils N-Value
O = Open Ended	0 to 2: Very Soft	0 to 4: Very Loose
U = Undisturbed	2 to 4: Soft	4 to 10: Loose
S = Split Spoon	4 to 8: Medium Stiff	11 to 30: Medium Dense
C = Rock Core	8 to 15: Stiff	31 to 50: Dense
G = Geoprobe	15 to 30: Very Stiff	Over 50: Very Dense
	Over 30: Hard	

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

Notes:

B-4

SECTION 3:

Sediment Management Preliminary Review

Clean Soils Environmental, Ltd.

10 Pages

November 22, 2013 Via e-Mail Only: bkelder@ipswichriver.org

Mr. Brian Kelder
Restoration Program Manager
Ipswich River Watershed Association
143 County Road
Ipswich, MA 01938

Re: **Sediment Management Preliminary Review**
Ipswich River Watershed Association
Ipswich Mills Dam Removal Project
Ipswich, MA

Dear Mr. Kelder:

Clean Soils Environmental, Ltd. (CSE) is pleased to provide the Ipswich River Watershed Association (IRWA) with a preliminary review of sediment testing results collected from the impoundment of the Ipswich Mills Dam on the Ipswich River, hereinafter referred to as the "Dam". CSE understands the Town of Ipswich is investigating the option to remove the Dam. The Dam is owned by the Town of Ipswich and located behind EBSCO Publishing in the vicinity of the Historic Ipswich Riverwalk. See the attached Figure for the location of the Dam, Historic Ipswich Riverwalk, and EBSCO's facility locations in this vicinity.

INTRODUCTION

The preliminary study being conducted by the IRWA is focused, at this time, on determining contaminant levels upstream of the Dam, the Dam's effect on upstream and downstream flooding, and potential effects of removing the Dam (i.e., lowering the water table) on the foundations of the historic mill buildings in the vicinity of the Dam. The results of the preliminary study will help the Town of Ipswich decide whether to undertake a more detailed study to further investigate the possibility of removing the Dam or continue to maintain the Dam in place.

CSE'S SCOPE OF WORK

The IRWA has requested Pro bono services from CSE to assist with a small portion of the preliminary study concerning the existing data from sediment testing to date and prepare a short letter report with CSE's opinion and recommendations. This preliminary review of sediment testing by CSE is meant to help the IRWA evaluate any potential contaminants in relation to ecological and human health thresholds. This preliminary assessment is the first step towards determining an appropriate sediment management strategy, should the Town decide to remove the dam.

Typical sediment management strategies associated with dam removal include 1) downstream release; 2) excavation and on-site reuse for grading; and 3) excavation and off-site reuse or disposal. Each dam removal project is unique, and the appropriate sediment management strategy for each project is developed based on chemical analysis of the sediment; evaluation of downstream infrastructure and ecosystems; and thorough discussions and coordination with agencies such as the MA Department of Environmental Protection, the MA Division of Marine Fisheries, and the MA Division of Ecological Restoration.

The most cost-effective way to manage sediment trapped by a dam is to allow the sediment to migrate slowly downstream over time. This approach has been used successfully in several dam removal projects in Massachusetts and many other projects across the country. Many factors go into making the decision to release sediment downstream. One of those factors is the chemical analysis of the sediment and comparison of the results with ecological and human health thresholds.

CSE's preliminary sediment assessment may help determine whether the sediment behind the impoundment of the Dam can be discharged naturally downstream according to 314 CMR 9.00 without significant affects to human health and the environment. This is a preliminary assessment only with the goal of comparing the sediment quality with human and ecological health thresholds. Future work will include 1) additional sediment testing, 2) quantification of the volume of sediment expected to be dredged or released downstream; and 3) evaluation of upstream and downstream infrastructure that could be affected under various sediment management strategies.

IRWA'S JUSTIFICATION FOR THE PROPOSED DAM REMOVAL

The primary goal of the Dam removal is to restore the habitat and passage for species throughout the Dam impoundment, restore ecological conditions and processes such as the movement of sediment and organic matter via cooler, free flowing water and tidal fluctuations, and eliminate further maintenance costs and liability to the Town of Ipswich associated with the Dam. Therefore, the primary goal of this project is to restore the natural ecological system that exists within the vicinity of Dam before its construction many years ago.

HUMAN HEALTH AND THE ENVIRONMENT

At this time, the impounded sediment within the future channel is conceptually proposed to be discharged downstream within the tidal waters of the Ipswich River. See the attached Figure for this location of the Ipswich River including the tidal areas. Please note it is likely some sediment will remain in place or on the banks of the redeveloped channel above the Dam, though restoration will occur in these areas over time that will likely develop a new vegetative bank and/or meadow. The sediment released from the impoundment will be discharged and/or reused downstream of the Dam. Therefore, this sediment management plan has the potential of affecting human health and the environment. Many factors must go into a decision to release sediment downstream. This study evaluates one such factor, the quality of the sediment in relation to human and ecological health thresholds.

SEDIMENT SAMPLES COLLECTED FROM THE IMPOUNDMENT BY USGS

The U.S. Geological Survey (USGS) and the Massachusetts Department of Fish and Game, Division of Ecological Restoration (MassDFG) collaborated to collect baseline information on the quantity and quality of sediment impounded behind selected dams in Massachusetts, including sediment thickness and the occurrence of contaminants potentially toxic to benthic organisms. The thicknesses of impounded sediments were measured, and cores of sediment were collected from 32 impoundments in 2004 and 2005. Cores were chemically analyzed, and concentrations of 32 inorganic elements and 108 organic compounds were quantified. As described below, the sediments found behind Ipswich Mills Dam have a very low likelihood of toxicity when viewed independently and in relation to other dams across Massachusetts.

On September 8, 2005, the United States Geological Survey (USGS) collected two (2) cores in the vicinity of the Ipswich Mills impoundment (Site 7, located at Lat: 42.677648, Long: -70.837756) shown on the attached Figure. CSE understands the USGS collected sediment samples and laboratory analyzed the samples for Total Heavy Metals, Semi Volatile Organic Compounds (SVOC), Polycyclic Aromatic Hydrocarbons (PAHs), and Total Polychlorinated Biphenyls (PCBs).

According to Table 6 in the report, “Estimated Sediment Thickness, Quality, and Toxicity to Benthic Organisms in Selected Impoundments in Massachusetts”;

> The Ipswich Mills Impoundment has a mean probable effects concentration quotient (PECQ) of 0.132, in other words, the estimated likelihood of toxicity of bottom-sediment cores is 13%.
>
> The average probable effects concentration quotient for a site (PECQx) is the average of the PECQs for arsenic, cadmium, chromium, copper, lead, nickel, and zinc, total polycyclic aromatic hydrocarbons (PAHs), total polychlorinated biphenyls (PCBs), and total dichlorodiphenyl-trichloroethylene (DDTs). For the purposes of this report, total DDTs comprise the sum of dichlorodiphenyl-trichloroethane and dichlorodiphenyl-dichloroethylene compounds. Total PAHs comprise the sum of the concentrations of anthracene, 9H-fluorene, naphthalene, phenanthrene, benzo(a)anthracene, benzo(a)pyrene, chrysene, fluoranthene, and pyrene.

According to Table 7 in the report, “Estimated Sediment Thickness, Quality, and Toxicity to Benthic Organisms in Selected Impoundments in Massachusetts”;

> The total drainage area is 150 mi^2, with 53 dams within the drainage area, with 11.2% being impervious. There are 45 21E sites within the drainage area and 22 factories in the 1830s.

Based upon the results of all 32 impoundments, the estimated probability of toxicity of bottom sediment ranged from about 8 to 70 percent among the sampling locations and averaged slightly under 30 percent. This put the Ipswich Mills impoundment at the low end of toxic bottom sediment range with a 13% likelihood.

SEDIMENT SAMPLES COLLECTED FROM THE IMPOUNDMENT BY IRWA

On May 31, 2012, the IRWA and staff from Interfluve, Inc. collected three (3) sediment cores from the impoundment area (Lat: 42.6825, Long: -70.8236). The sediment samples were identified as IM-1, IM2, and IM-3. See the attached Figure for the approximate sample locations. Sediment samples were laboratory analyzed for Total Heavy Metals, SVOCs, PAHs, Volatile Organic Compounds (VOCs), Extractable Petroleum Hydrocarbons (EPH), and Physical Characteristics such as Percent of Total Organic Carbon (TOC), Percent of Water, and Percent of Grain Size Distribution.

IRWA SAMPLE LABORATORY ANALYSES

The laboratory results are tabulated on the attached table that was developed by the MassDFG. This table and/or spreadsheet were used to compare the initial sediment testing results to a screening benchmarks or criteria. The sample results from the USGS and IRWA were tabulated within this MassDFG table and/or spreadsheet for this preliminary review.

The table and/or spreadsheet compare the sediment sampling results to the conservative Massachusetts Contingency Plan (MCP) cleanup standards for soil only within a residential scenario including groundwater suitable for human consumption (i.e., drinking water). These conservative cleanup standards are being used since sediment will likely be left in place below the existing water levels and/or above the newly developed channel after the Dam is removed. Thus, it is possible that the impounded sediment may come in contact with humans in the future, and therefore it’s important to know how its quality compares with human health thresholds.

The table and/or spreadsheet also compare the sediment sampling results to Threshold Effects Concentrations (TECs), Probable Effects Concentrations (PECs), Threshold Effect Levels (TELs), and Probable Effect Levels (PELs). The TECs and TECs are considered background concentrations and typically are interpretive as ‘No Significant Risk’ to the ecological environment. The PECs and PELs are considered potential actions levels and a significant exceedance might indicate that negative ecological affects are possible, such as impairments to benthic dwelling organisms.

INTERPRETATION OF THE LABORATORY DATA COLLECTED BY THE IRWA

Generally, both sampling events indicate that the sediment is below applicable ecological benchmark limits in regard to the freshwater PEC, marine PEL, and human health MCP Method 1 Cleanup Standard S1/GW1 screening criteria measured at this time. Therefore, it appears, the laboratory data to date indicates that a condition of ‘No Significant Risk’ may exist within the sediment from the impoundment of the Dam.

These results do make some sense, at this time, since the upstream past history in the vicinity of the area has mainly been residential with little industrial effects. The concentrations of metals, SVOCs, pesticides, VOCs, and EPHs measured within the sediment appear to be mainly from surface water run off non-point sources (e.g., roadways and farming).

RECOMMENDATIONS

CSE recommends the continuation of the feasibility study to remove the Dam. However, a significant volume of work is still required to permit the removal of the Dam and manage the sediment in place and downstream of the Dam.

At this time, CSE recommends estimating the volume of sediment that is contained within the impoundment and the volume of sediment that would be dredged or mobilized as part of a dam removal project. CSE understands the regulators typically request one (1) sediment sample per 1,000 cubic yards of dredged or mobilized material. Therefore, the next step should focus on estimating the sediment volume to help determine how many more samples should be collected in order to complete the sediment contaminate level study. It appears to CSE that this is likely a very important component to the entire study to help permit the removal of the Dam. CSE also believes a focus on characterizing the sediment immediately upstream of the Dam is also important since these are likely to be the quickest sediments to mobilize and discharge to the environment or tidal waters of the Ipswich River following removal of the dam. This is also the location of the former Ipswich Mills and may exhibit different contamination levels than the sites sampled upstream of the former mill.

CSE also believes further sediment testing should be conducted above and below the impoundment with an emphasis on downstream of the impoundment. CSE suggests at least three to four (3 – 4) sample locations downstream of the impoundment, one recommendation being the meander or cove between Country Street and Turkey Shore Road as shown on the Map. A significant volume of sediment from street sanding has accumulated within this vicinity for years including fines from organic matter and possibly discharges from the former mills. This is also likely the location where sediment will accumulate within the tidal waters of the Ipswich River (see Figure for this location).

One or more upstream samples (from depositional areas subject to potential mobilization during storm events) will help evaluation material that is ‘moving through the system’ regardless of actions at the dam. If upstream source areas are contaminated, then actions such as dredging with the dam impoundment may not affect sediment quality in the longer-term.

Please note the intention of the above interpretation and recommendation are preliminary and this project will be complicated by the entire regulatory process required for this project in the Commonwealth of Massachusetts. Please do not hesitate to call if you have any questions.

Respectfully submitted,
CLEAN SOILS ENVIRONMENTAL, LTD.

William H. Mitchell, Jr., LSP
President/Geologist

Kevin L. McAndrews
Environmental Geologist

Enclosures: Figure
Table Instructions
Table
Cross Sections

REFERENCES

Breault, R.F., Sorenson, J.R., and Weiskel, P.K., 2013, Estimated sediment thickness, quality, and toxicity to benthic organisms in selected impoundments in Massachusetts: U.S. Geological Survey Scientific Investigations Report 2012–5191, 42 p., at http://pubs.usgs.gov/sir/2012/5191/.

Ipswich Mills Dam Removal Feasibility Study

Ipswich River Watershed Association Sediment Core Location:

Sample #	Location	Agency	Date	Latitude	Longitude	Determined by	Description
IM-1	Impoundment	IRWA	5/31/2012	42.661640	-70.844764	GPS (Android-EpiCollect)	0.1 ft fines & organics over 0.8 ft sand to refusal
IM-2	Impoundment	IRWA	5/31/2012	42.670725	-70.841567	GPS (Android-EpiCollect)	fine silt above fine sand core depth ~2.5 ft to refusa
IM-3	Impoundment	IRWA	5/31/2012	42.675602	-70.838249	GPS (Android-EpiCollect)	~3ft fines over 3 inches fine sand to refusal

Figure: Sediment core locations and sediment depth cross sections from 2012 surveys of Ipswich Mills impoundme

Prepared by the Ipswich River Watershed Association (IRWA including notes from Clean Soils Environmental, Ltd. (CSE.

Sediment Quality Spreadsheet Updated December 2010

Please send comments, questions, or suggested corrections to:
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The purpose of this spreadsheet is to organize sediment quality data and provide comparison to relevant ecological and human health screening values.
The format was originally developed by ERM during donated services to the Ox Pasture Project (Rowley) through CWRP (2008).
It has been modified extensively by the Mass Division of Ecological Restoration to include additional parameters, notes, imbedded calculations, and thresholds.
The spreadsheet is structured to provide comparisons useful during **401 Water Quality Certification** via Mass DEP.

Disclaimer: The Department of Fish and Game and the Division of Ecological Restoration (DER) takes no responsibility for the accuracy of the screening threshold values presented in this workbook.
Staff has made every effort to ensure accuracy, but standards change and errors are possible.
Users are encouraged to double check the accuracy of values based upon the most recently available screening thresholds.
All threshold values are rounded to one decimal place (place cursor in cell to see true value)
Evaluating sediment quality findings can be complex, and users are encouraged to consider concentrations upstream, downstream, and in the impoundment for context.

Instructions

1 Enter all data for samples taken from the dam impoundment, upstream, and downstream.
2 Create additional columns if necessary to house data from your sampling locations.
3 If additional columns have been added, check the equations under "Impoundment Sample Statistics" to ensure that all values are being utilized in the automated calculations.
4 For results that are below the laboratory detection limit...
Enter a value **1/2** of the value of the laboratory detection limit in the appropriate space and color code it green.
For example, the following cell value indicates a lab result of "below detection limit" and a detection limit of 0.5.
0.25 The use of 1/2 the detection limit is for developing mean values.
5 Check laboratory methods and units, and update specific parameters if necessary.
6 The table uses conditional formatting to evaluate to following:
Maximum impoundment values above MCP S1/GW1 standards are show as: **35.34** These values assist in evaluating potential human health risks from the area of highest concentration
Mean impoundment values above freshwater PECs are show as: **124.5** These values assist in evaluating potential ecological risks from the average concentration of impounded sediment
7 Include information about sample locations and characteristics on the sheet entitled "Map and Sample Info". It is critical to understand how samples were collected (i.e. cores via surficial grab samples) to interpret your results.
8 Note that this spreadsheet uses the MCP Method 1 Cleanup Standards for S-1 (soils) and GW-1 (groundwater). Depending on your project location, a different soil and/or groundwater category may be appropriate.
S-1/GW-1 is the most conservative. Please refer to 310 CMR 40.0930 (Identification of Site Groundwater and Soil Categories)

Guidance of interpreting values

1 Ecological screening values are important for evaluating downstream release of sediment, including (1) as a sediment management option and (2) for precautions needed during dam removal.
In general, our experience in MA has been that PECs are the important value. Given our long history of human impact in MA, the TEC values often are considered to be background levels. This may not be the case for more pristine rivers.
In evaluating downstream release of sediment, it is also important to compare concentrations in the impoundment to those found in downstream depositional areas.

2 Human health screening values (MCP) are important for evaluating shoreline placement and upland re-use options.
In many cases, adequate regulation under 401 Water Quality Certification may prevent entry into the MCP system, even when values exceed the MCP cleanup standards {see 314 CMR 9.07 (9)]

3 To evaluate off-site disposal options, it may also be necessary to compare sediment quality data to MA DEP screening values for landfill reuse.

References

401 Water Quality Certification Regulation (314 CMR 9.00)
Massachusetts Contingency Plan (MCP; 310 CMR 40.0000)
MCP Method 1 Cleanup Standards for S-1 soils
MA DEP Interim Policy Comm 97-004 (for reuse and disposal at permitted landfills)

COMMONWEALTH OF MASSACHUSETTS
Division of Ecological Restoration

Questions, comments, or concerns?
Please contact Alex Hackman, Restoration Specialist, Mass Division of Ecological Restoration
alex.hackman@state.ma.us / (617) 626-1548

Parameter	CAS No.	Method	Screening Criteria					Dam Impoundment Samples			
(Important: Units listed by category below)			MCP S1 / GW1	TEC	PEC	TEL	PEL	IM-1	IM-2	IM-3	USGS
			Human Health	Freshwater		Marine					
Metals [mg/kg]											
Antimony	7440-36-0	6020A	20.0								0.32
Arsenic	7440-38-2	6020A	20.0	9.8	33.0	7.2	41.6	4.76	13.8	9.68	3.6
Barium	7440-39-3	6020A	1,000.0								
Beryllium	7440-41-7	6020A	100.0								
Cadmium	7440-43-9	6020A	2.0	1.0	5.0	0.7	4.2	0.139	0.503	0.515	0.53
Chromium (TOTAL)	7440-47-3	6020A	30.0	43.4	111.0	52.3	160.4	7.48	15.7	16.4	36
Chromium (III)	7440-47-3		1,000.0								
Chromium (VI)	7440-47-3		30.0								
Copper	7440-50-8	6020A	NC	31.6	149.0	18.7	108.2	3.45	10.7	15.1	13.7
Lead	7439-92-1	6020A	300.0	35.8	128.0	30.2	112.2	9.37	32.7	43.5	33.8
Mercury	7439-97-6	7471A	20.0	0.2	1.1	0.1	0.7	0.047	0.216	0.185	0.066
Nickel	7440-02-0	6020A	20.0	22.7	48.6	15.9	42.8	5.25	9.79	10.9	9.2
Selenium	7782-49-2	6020A	400.0								0
Silver	7440-22-4	6020A	100.0			0.7	1.8				0.22
Thallium	7440-28-0	6020A	8.0								
Vanadium	7440-62-2	6020A	600.0								
Zinc	7440-66-6	6020A	2,500.0	121.0	459.0	124.0	271.0	32.9	80	102	41.4
SVOCs (**PAHs**)[ug/kg]											
Acenaphthene	83-32-9	8270/8100	4,000.0			6.7	88.9	5.7	5.7	33.5	76
Acenaphthylene	208-96-8	8270/8100	1,000.0			5.9	127.9	27.7	19.5	124	84
Anthracene	120-12-7	8270/8100	1,000,000.0	57.2	845.0			20.3	21.2	145	330
Benz[a]anthracene	56-55-3	8270/8100	700.0	108.0	1,050.0			129	110	673	730
Benzo[a]pyrene	50-32-8	8270/8100	2,000.0	150.0	1,450.0			140	97.9	610	670
Benzo[b]fluoranthene	205-99-2	8270/8100	7,000.0	27.3	13,400.0			135	129	718	550
Benzo[g,h,i]perylene	191-24-2	8270/8100	1,000,000.0					84.3	70	412	360
Benzo[k]fluoranthene	207-08-9	8270/8100	70,000.0					121	104	571	550
Chrysene	218-01-9	8270/8100	70,000.0	166.0	1,290.0	107.8	846.0	153	132	702	740
Dibenz[a,h]anthracene	53-70-3	8270/8100	700.0	33.0	260.0	6.2	134.6	19.2	16.1	108	200
Fluoranthene	206-44-0	8270/8100	1,000,000.0	423.0	2,230.0	112.8	1,493.5	356	279	1410	1500
Fluorene	86-73-7	8270/8100	1,000,000.0	77.4	536.0	21.2	144.4	14.5	10.2	62.8	150
Indeno[1,2,3-cd]pyrene	193-39-5	8270/8100	7,000.0					96.6	77.2	491	380
Phenanthrene	85-01-8	8270/8100	10,000.0	204.0	1,170.0	86.7	543.5	181	93	590	
Pyrene	129-00-0	8270/8100	1,000,000.0	195.0	1,520.0	152.7	1,397.6	296	239	1240	1200
2-Methylnaphthalene	91-57-6	8270/8100	700.0			20.2	201.3				39
Naphthalene	91-20-3	8270/8100	4,000.0	176.0	561.0	34.6	390.6	25.4	15.6	55.3	78
Total PAHs				1,610.0	22,800.0	1,684.1	16,770.4				
Pesticides (ug/kg)											
2,4'-DDD	-	8151a									
4,4'-DDD	72-54-8	8151a	4,000.0			1.2	7.8				
Sum DDD				4.9	28.0						
2,4'-DDE	-	8151a									
4,4'-DDE	72-55-9	8151a	3,000.0			2.1	374.2				
Sum DDE				3.2	31.3						
2,4'-DDT	-	8151a									
4,4'-DDT	50-29-3	8151a	3,000.0			1.2	4.8				
Sum DDT				4.2	62.9						
Total DDTs				5.3	572.0	3.9	51.7				
alpha-Chlordane	57-74-97	8081a		0.5	6.0						
Aldrin	30-90-02		40.0			NC	NC				
Chlordane	57-74-9			3.2	17.6	2.3	4.8				
Dieldrin	60-57-1	8081a	50.0	1.9	61.8	0.7	4.3				
Endrin	72-20-8	8081a	8,000.0	2.2	207.0						
gamma-BHC/Lindane	-	8081a		2.4	5.0	0.3	1.0				
gamma-Chlordane	-	8081a									
Heptachlor epoxide	1024-57-3	8081a	90.0	2.5	16.0						
Hexachlorobenzene	118-74-1	8081a	700.0								
PCBs (ug/kg)											
Aroclor 1016	12674-11-2	8082									
Aroclor 1221	11104-28-2	8082									
Aroclor 1232	11141-16-5	8082									
Aroclor 1242	53469-21-9	8082									
Aroclor 1248	12672-29-6	8082									
Aroclor 1254	11097-69-1	8082									
Aroclor 1260	11096-82-5	8082									
Total PCBs	1336-36-3	8082	2,000.0	59.8	676.0	21.6	188.8				11
VPH (mg/kg)											
C5-C8 Aliphatic Hydrocarbons		MADEP	100.0								
C9-C12 Aliphatic Hydrocarbons		MADEP	1,000.0								
C9-C10 Aromatic Hydrocarbons		MADEP	1,000.0								
Unadjusted C5-C8 Aliphatic Hydrocarbons		MADEP	NC								
Unadjusted C9-C12 Aliphatic Hydrocarbon		MADEP	NC								
VOCs (mg/kg)											
Methyl tert-butyl ether (MTBE)	1634-04-4	MADEP	0.1					3.25	3.25	3.25	
Benzene	71-43-2	MADEP	2.0					1.65	1.65	1.65	
Toluene	108-88-3	MADEP	40.0	4.5	4.5			2.45	2.45	2.45	
Ethylbenzene	100-41-4	MADEP	30.0					1.65	1.65	1.65	
m&p-Xylenes	1330-20-7	MADEP	400.0					3.25	3.25	3.25	
o-Xylene	95-47-6	MADEP	400.0					3.25	3.25	3.25	
EPH (mg/kg)											
C9-C18 Aliphatics		MADEP	1,000.0					5.05	5.05	5.05	
C19-C36 Aliphatics		MADEP	3,000.0					5.05	5.05	16.8	
C11-C22 Aromatics		MADEP	1,000.0					10.3	5.05	22.4	
Unadjusted C11-C22 Aromatics		MADEP						10.3	5.05	22.4	
Physical Characterisitcs											
Total Organic Carbon (%)								0.9835	3.84	2.985	
Percent Water (%)								36.2	62.2	52.1	
Grain Size Distribution (%)		ASTM D422									
Sieve No. 4								0.1	0.1	0	
Sieve No. 10								2.2	1	0.4	
Sieve No. 40								50	17.4	17.4	
Sieve No. 60								40.7	73.2	63.2	
Sieve No. 200								7	8.3	19	

Prepared by:

Massachusetts Department of Fish and Game

Division of Ecological Restoration (DER)

Updated December 2010

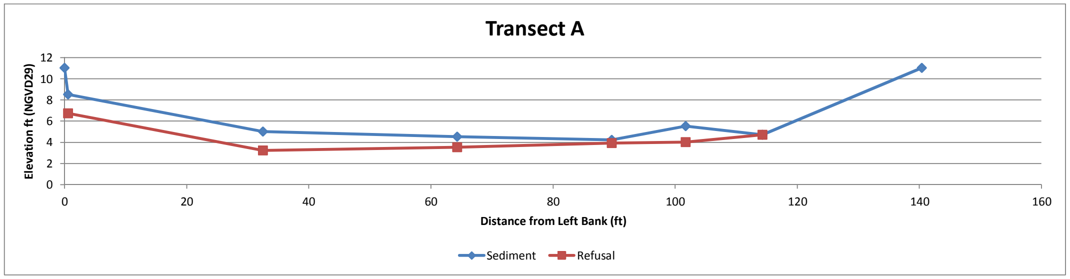
This table is prepared and offered by DER for the benefit of all parties pursuing river restoration projects within Massachusetts.

mg/kg = milligrams per killogram

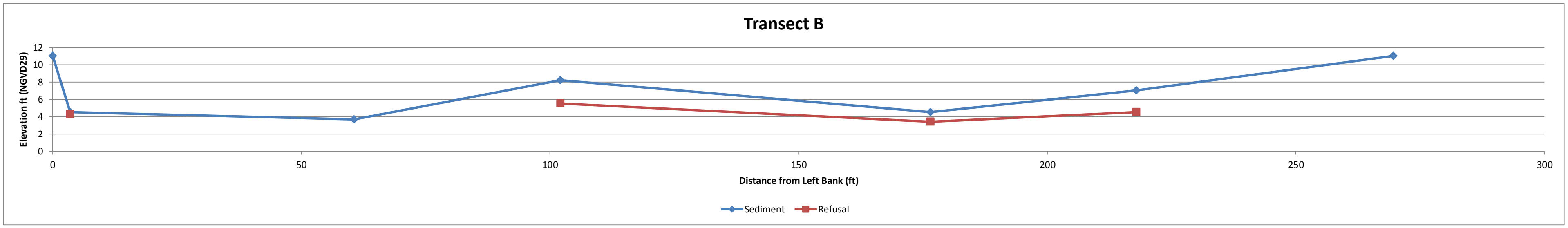
ug/kg = micrograms per killogram

TEC - Threshold Effect Concentration

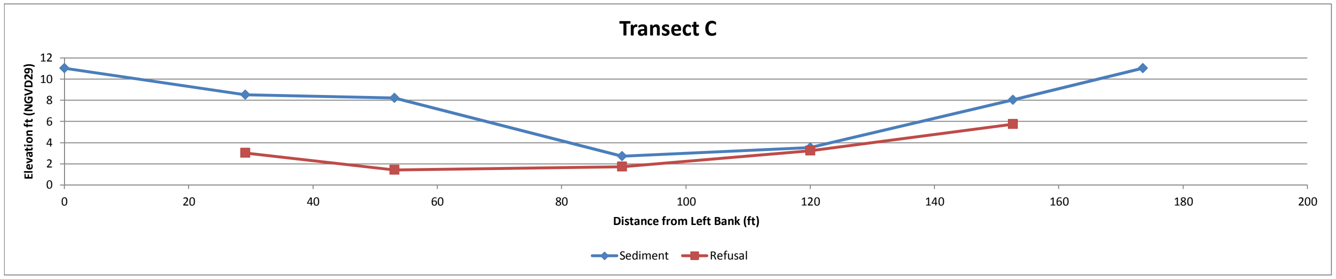
PEC - Probable Effect Concentration



Transect A
Elevation ft (NGVD29)
12
10
8
6
4
2
0
0
20
40
60
80
100
120
140
160
Distance from Left Bank (ft)
Sediment
Refusal



Transect B
Elevation ft (NGVD29)
12
10
8
6
4
2
0
0
50
100
150
200
250
300
Distance from Left Bank (ft)
Sediment
Refusal



Transect C
Elevation ft (NGVD29)
12
10
8
6
4
2
0
0
20
40
60
80
100
120
140
160
180
200
Distance from Left Bank (ft)
Sediment
Refusal

ATTACHMENT 2

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- ***Land Survey***
- ***GPS - Mapping***
- ***Hydrographic Survey***

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August 26, 2014

Project: Ipswich River Mills Dam survey

Surveyor's Report

Project goals:

1. Obtain bathymetry data for the upstream section and to the face of the Ipswich River Mills Dam.
2. Measure sediment depths at select points in the area closer to the dam.
3. Prepare a 'Riverbed Survey' plan to present the field measured data.

Equipment list:

1 ODEC Bathy MF500 echo sounder coupled with a 200-kHz transducer – a survey grade accuracy shallow water measuring system, variable power settings for very shallow depths to reduce reverberation (multi-echo) effects. Digital depth output/input to the navigation software.

1 LEICA 1200 system dual-frequency (L1/L2) GPS receiver – a precision real-time-kinematic (RTK) differential navigation signal output/input to the navigation software.

1 LEICA AT502 L1/L2 GPS antenna coupled with a cell modem providing GPS correction data from the Maine Technical Source Inc. (MTS) virtual reference network (VRN). Corrections applied in the 1200 GPS receiver for output/input of X,Y,Z position data to the navigation software.

1 HYPACK INC. licensed copy software for boat track guidance and real-time data recording of the digital depth and positional data. Coordinate collection in the State Plane Coordinate System NAD83 (SPCS83) MA Zone 2001 M in US survey feet.
Depth to riverbed from echo sounder corrected to elevations on the National Geodetic Vertical Datum of 1929 (NGVD29).

1 shallow draft aluminum craft 12 ½ ft. in length powered by electric motor and oars, 2 persons onboard.

1 single-section of rebar marked in decimal feet for sediment probe depth measure.

1 SOKKIA SR530 reflectorless digital total station for shore-based collection of dam structure and riverbank limits.
Coupled with a **Juniper Technology Inc. Allegro** field computer for digital data collection.

Shore (land) survey:

Survey field crew located dam structure detail, fish ladder, river bank limits and river way walls, including the EBSCO Inc. building location. Instrument set-up points on river walk and pedestrian bridge tied to the horizontal and vertical reference data by static GPS observations.

Proposed navigation run lines determined from this shore survey, drafted in Carlson / AutoCAD software input/output to Hypack Inc. navigation software.

Bathymetric survey:

Starting in the area of Peatfield Street the survey crew observed a series of sounding lines by the upstream river section for 900 linear feet to the face of the dam. The sounding lines were spaced at approximately 15 feet (average) intervals and perpendicular to the line of flow or centerline of the river channel.
The boat was brought as close to the dam face as possible for riverbed depth measure whilst considering the accelerating spill flow rate and concern for security of the precision survey instrumentation. Riverbed and sediment depths by the upstream face of the dam would later be measured directly by hand from the dam crest.
The top of water surface elevation was measured from fixed reference points on the shore both at the start and end of the workday. The gradient of the river surface from Peatfield Street to the area of the dam was measured but found to be negligible in relation to the overall precision of the survey method. The assigned water surface elevation was used to correct all measured depths streaming to the laptop based navigation software.
Considering the very shallow depths observed each sounding line can be considered a continuous line across the riverbed at the echo sounder ping rate of 10 per second.

Depth sediment survey:

Sediment depth samples were observed starting 400 feet upstream of the dam face and numbered 57 in total. Fourteen of the depths were obtained from riverbank and dam crest direct measure, the remainder from the boat.
Once the boat was secure at anchor the GPS would be recorded at that location, the rebar section then hand-pushed to a firm point of refusal. Type of material was approximated by resistance and feel; in some instances the sound of sand and gravel disturbance could audibly be discerned.
Depth of sediment and any apparent gradation levels were recorded in the field notebook.

Riverbed Survey plan generation:

1. Each sounding run line was edited for false spikes or lows in depth data, for example; debris, grass or riverbed tree limbs causing multi-echo or false bottom readings. A filter and smoothing process is applied to generate a dataset of soundings that represent the most likely riverbed.
2. Output of 12,200 discrete position and depth points to the Carlson / AutoCAD software.
3. Addition of observed riverbed depths and bank elevations measured from shore and the dam crest.
4. Creation of one continuous terrain model from all observed data points by the triangular irregular network method (TIN).
5. Contour and elevation label generation from the terrain model.
6. Selection of transect locations for profile views along the length of the 900 ft. river section.
7. Draft the transect profiles with sediment depth locations next to the plan view of the river section.
8. Plan copies in color and black and white sent to Client along with the complete digital CAD drawing file, .pdf view files of drawing included in transmittal.
9. Contour and transect polyline data output to shape files for GIS Client database use.

Statement of accuracy:

The ground control survey (shore-based instrument measure points) are *accurate* to plus or minus 0.04' for north and east coordinates on the State Plane Coordinate System, at a 1-sigma estimate
The vertical control determination on the NGVD29 site datum elevation is estimated at an *accuracy* of plus or minus 0.07', at a 1-sigma estimate.
The *relative precision* for the transfer of elevation to the surface of water is estimated at plus or minus 0.02'
The precision of the echo sounder at shallow depth is plus or minus 0.10', considering the point at which a reliable ping returns and is properly identified from a silted riverbed can add to the uncertainty of real precision at any one sounding point. The return echo from a solid or hard bottom is always more precise for obvious reasons.
Considering the density of sounding lines and the process of automatic gate and smoothing filters at both the field measure and office edit stages it can be assumed that the accumulated system error behaves in a random manner. The estimate for the ***vertical precision*** of any single sounding point, as a cumulative system error, is plus or minus 0.13' at 95% confidence.
The estimate for the ***horizontal precision*** of any single sounding point, as a cumulative system error, is plus or minus 0.20' at 95% confidence, being a function of boat motion and applied real-time filters by the navigation software, GPS signal corrections and process latencies.
The measure of sediment depth to firm point of resistance is estimated at 0.05' precision relative to the river water surface.

Report written by:

8/26/14

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Norde-East Survey

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- ***GPS - Mapping***
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November 6, 2014

Project: Ipswich River Dam survey

Surveyor's Report – additional depth & sediment samples.

Project goals:

1. Obtain additional bathymetry data for the upstream section closer to the Ipswich River Dam.
2. Measure sediment depths at select points in the area adjacent to the dam structure.
3. Prepare a 'Riverbed Survey' exhibit plan to represent the field-measured data.

Equipment list:

1 LEICA 1200 system dual-frequency (L1/L2) GPS receiver – a precision real-time-kinematic (RTK) differential navigation system.

1 shallow draft aluminum craft 12 ½ ft. in length, crewed by Patrick McCormack & Brian Kelder.

1 section of rebar marked in decimal feet for sediment probe depth measure.

Riverbed survey:

9/14/14
Survey crew measured 45 location points for depth of water and thickness of riverbed sediment.
Sample locations were tied to the horizontal datum and recorded by real-time kinematic GPS observation.
Depth to riverbed measured by probe and related to the known elevation of the water surface over the period of work.
The type of riverbed and thickness of the sediment were noted.

A metal 6-inch diameter pipe was found and is shown on the survey plan. The pipe is buried for a portion of its length.
The utility type and present day use of this service pipe was not determined.

A series of large boulders, some cut in a regular shape, were located and are shown on the survey plan.

Riverbed sample exhibit plan:

The depth and sediment sample locations are shown by an exhibit plan at 1" = 30' scale, and presented with this report. The exhibit plan is dated 11/06/14, and noted as an attachment to the 'Riverbed Survey Plan' with a revision date of 11/06/14.
A table showing the sample identifying number with thickness of sediment and nature of riverbed is shown on the exhibit plan.

Riverbed survey plan revisions:

The additional riverbed depths were used to re-compute the entire riverbed terrain model.
Contoured elevations on the revised survey plan dated 11/06/14 reflect the re-computed triangular irregular network (TIN).

An additional transect line (A1) is presented on the Riverbed Survey Plan. Location of this profile by request of Brian Kelder.

Please refer to the Surveyor's Report dated August 26, 2014 for a complete and detailed project summary.

This report outlines only the efforts in obtaining additional sediment depth and thickness observed when the river was in a low flow state. It was thus easier to maneuver by boat and measure in the immediate area of the dam crest.

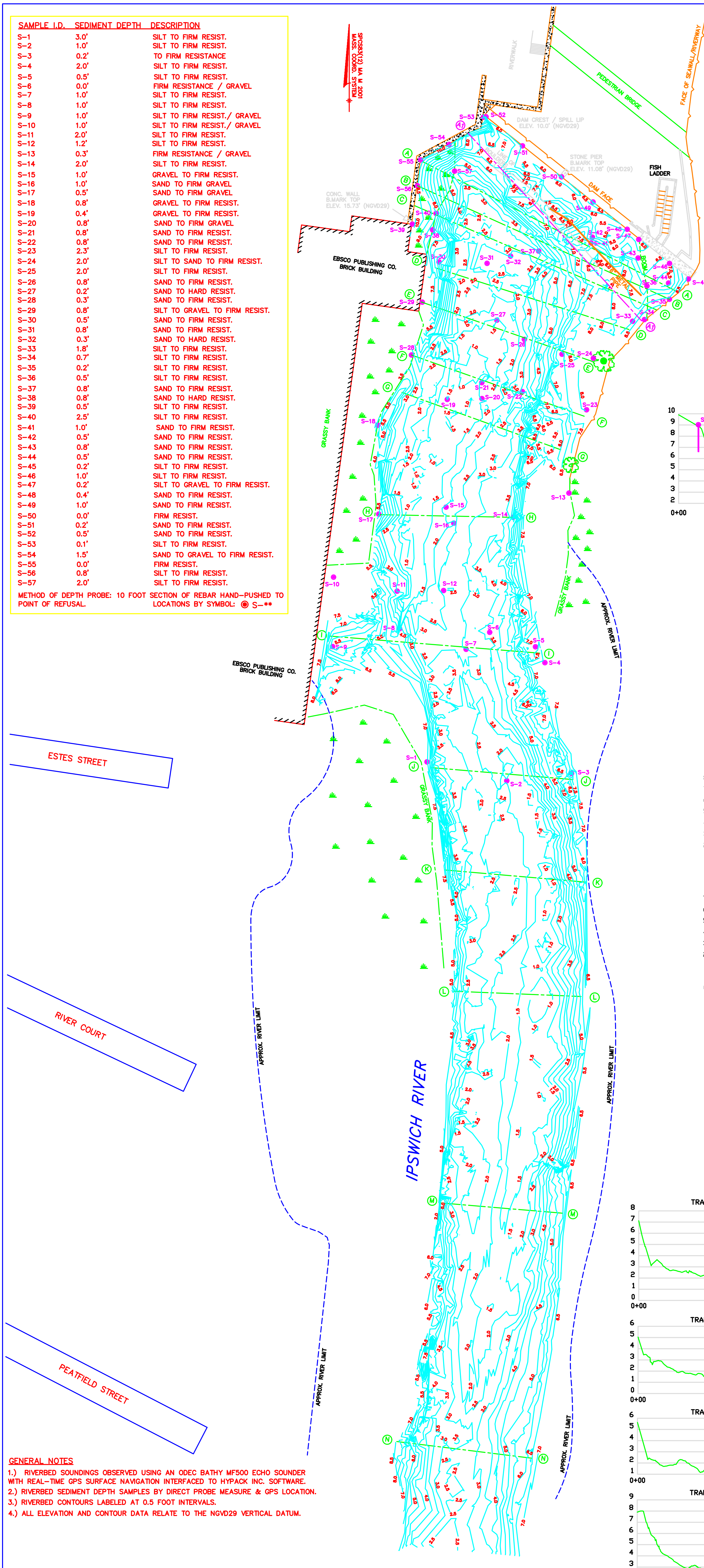
Report written by:

Patrick J. McCormack, PLS
Norde-East Survey

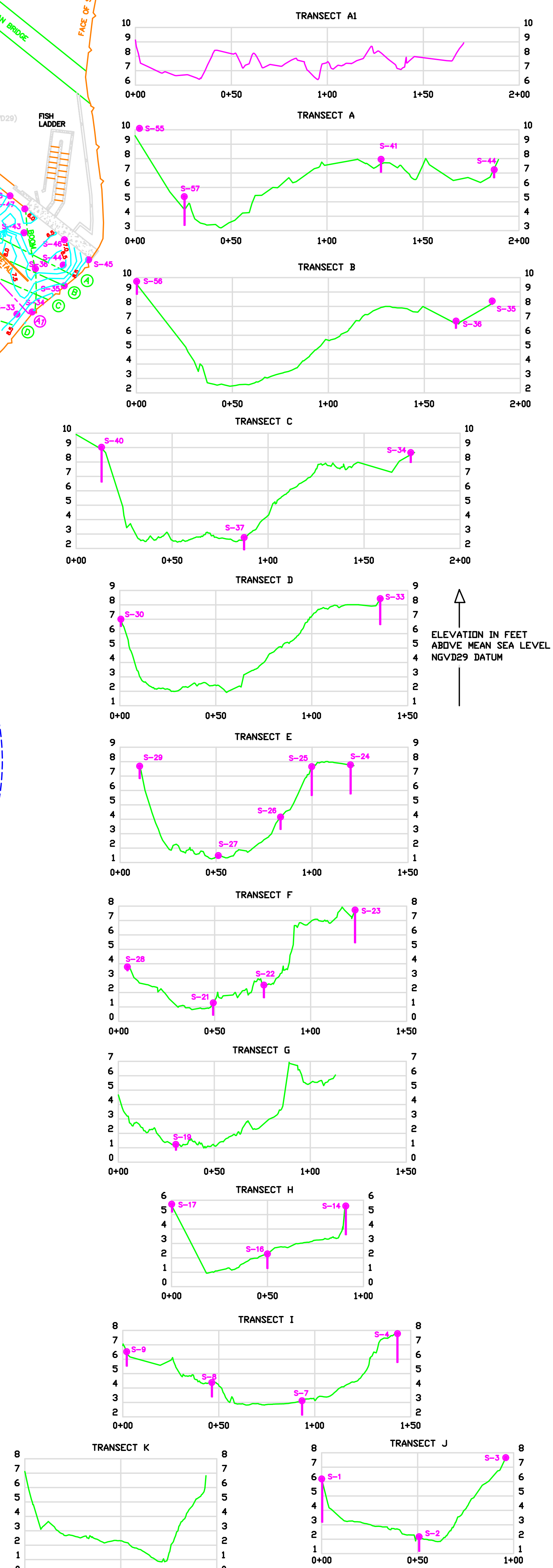
11/6/14
Date

SAMPLE I.D.	SEDIMENT DEPTH	DESCRIPTION
S-1	3.0'	SILT TO FIRM RESIST.
S-2	1.0'	SILT TO FIRM RESIST.
S-3	0.2'	TO FIRM RESISTANCE
S-4	2.0'	SILT TO FIRM RESIST.
S-5	0.5'	SILT TO FIRM RESIST.
S-6	0.0'	FIRM RESISTANCE / GRAVEL
S-7	1.0'	SILT TO FIRM RESIST.
S-8	1.0'	SILT TO FIRM RESIST.
S-9	1.0'	SILT TO FIRM RESIST./ GRAVEL
S-10	1.0'	SILT TO FIRM RESIST./ GRAVEL
S-11	2.0'	SILT TO FIRM RESIST.
S-12	1.2'	SILT TO FIRM RESIST.
S-13	0.3'	FIRM RESISTANCE / GRAVEL
S-14	2.0'	SILT TO FIRM RESIST.
S-15	1.0'	GRAVEL TO FIRM RESIST.
S-16	1.0'	SAND TO FIRM GRAVEL
S-17	0.5'	SAND TO FIRM GRAVEL
S-18	0.8'	GRAVEL TO FIRM RESIST.
S-19	0.4'	GRAVEL TO FIRM RESIST.
S-20	0.8'	SAND TO FIRM GRAVEL
S-21	0.8'	SAND TO FIRM RESIST.
S-22	0.8'	SAND TO FIRM RESIST.
S-23	2.3'	SILT TO FIRM RESIST.
S-24	2.0'	SILT TO SAND TO FIRM RESIST.
S-25	2.0'	SILT TO FIRM RESIST.
S-26	0.8'	SAND TO FIRM RESIST.
S-27	0.2'	SAND TO HARD RESIST.
S-28	0.3'	SAND TO FIRM RESIST.
S-29	0.8'	SILT TO GRAVEL TO FIRM RESIST.
S-30	0.5'	SAND TO FIRM RESIST.
S-31	0.8'	SAND TO FIRM RESIST.
S-32	0.3'	SAND TO HARD RESIST.
S-33	1.8'	SILT TO FIRM RESIST.
S-34	0.7'	SILT TO FIRM RESIST.
S-35	0.2'	SILT TO FIRM RESIST.
S-36	0.5'	SILT TO FIRM RESIST.
S-37	0.8'	SAND TO FIRM RESIST.
S-38	0.8'	SAND TO HARD RESIST.
S-39	0.5'	SILT TO FIRM RESIST.
S-40	2.5'	SILT TO FIRM RESIST.
S-41	1.0'	SAND TO FIRM RESIST.
S-42	0.5'	SAND TO FIRM RESIST.
S-43	0.8'	SAND TO FIRM RESIST.
S-44	0.5'	SAND TO FIRM RESIST.
S-45	0.2'	SILT TO FIRM RESIST.
S-46	1.0'	SILT TO FIRM RESIST.
S-47	0.2'	SILT TO GRAVEL TO FIRM RESIST.
S-48	0.4'	SAND TO FIRM RESIST.
S-49	1.0'	SAND TO FIRM RESIST.
S-50	0.0'	FIRM RESIST.
S-51	0.2'	SAND TO FIRM RESIST.
S-52	0.5'	SAND TO FIRM RESIST.
S-53	0.1'	SILT TO FIRM RESIST.
S-54	1.5'	SAND TO GRAVEL TO FIRM RESIST.
S-55	0.0'	FIRM RESIST.
S-56	0.8'	SILT TO FIRM RESIST.
S-57	2.0'	SILT TO FIRM RESIST.

METHOD OF DEPTH PROBE: 10 FOOT SECTION OF REBAR HAND-PUSHED TO POINT OF REFUSAL. LOCATIONS BY SYMBOL: ◉ S-**



RIVERBED TRANSECTS: WEST TO EAST
HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 4'
SEDIMENT DEPTH LOCATIONS BY SYMBOL: ◉ S-**
WESTERLY BANK — EASTERLY BANK



GENERAL NOTES

1.) RIVERBED SOUNDINGS OBSERVED USING AN ODEC BATHY MF500 ECHO SOUNDER WITH REAL-TIME GPS SURFACE NAVIGATION INTERFACED TO HYPACK INC. SOFTWARE.
2.) RIVERBED SEDIMENT DEPTH SAMPLES BY DIRECT PROBE MEASURE & GPS LOCATION.
3.) RIVERBED CONTOURS LABELED AT 0.5 FOOT INTERVALS.
4.) ALL ELEVATION AND CONTOUR DATA RELATE TO THE NGVD29 VERTICAL DATUM.

PATRICK J. McCORMACK – PROFESSIONAL LAND SURVEYOR

DATE

RIVERBED SURVEY IPSWICH RIVER MILLS DAM TOWN OF IPSWICH, MA

SCALE 1" = 30' AUGUST 9, 2014

SURVEYED BY
NORDE-EAST SURVEY
27 CONGRESS STREET, Suite 205-8, SALEM, MA 01970.
Tel.(978) 5421920 E-mail: norde-east@verizon.net

NO.	DATE	REVISIONS / DESCRIPTION	BY
1.	10/3/14	REVISE CONTOURS BY DAM	P.McC
2.	11/6/14	ADD TRANSECT A1	P.McC

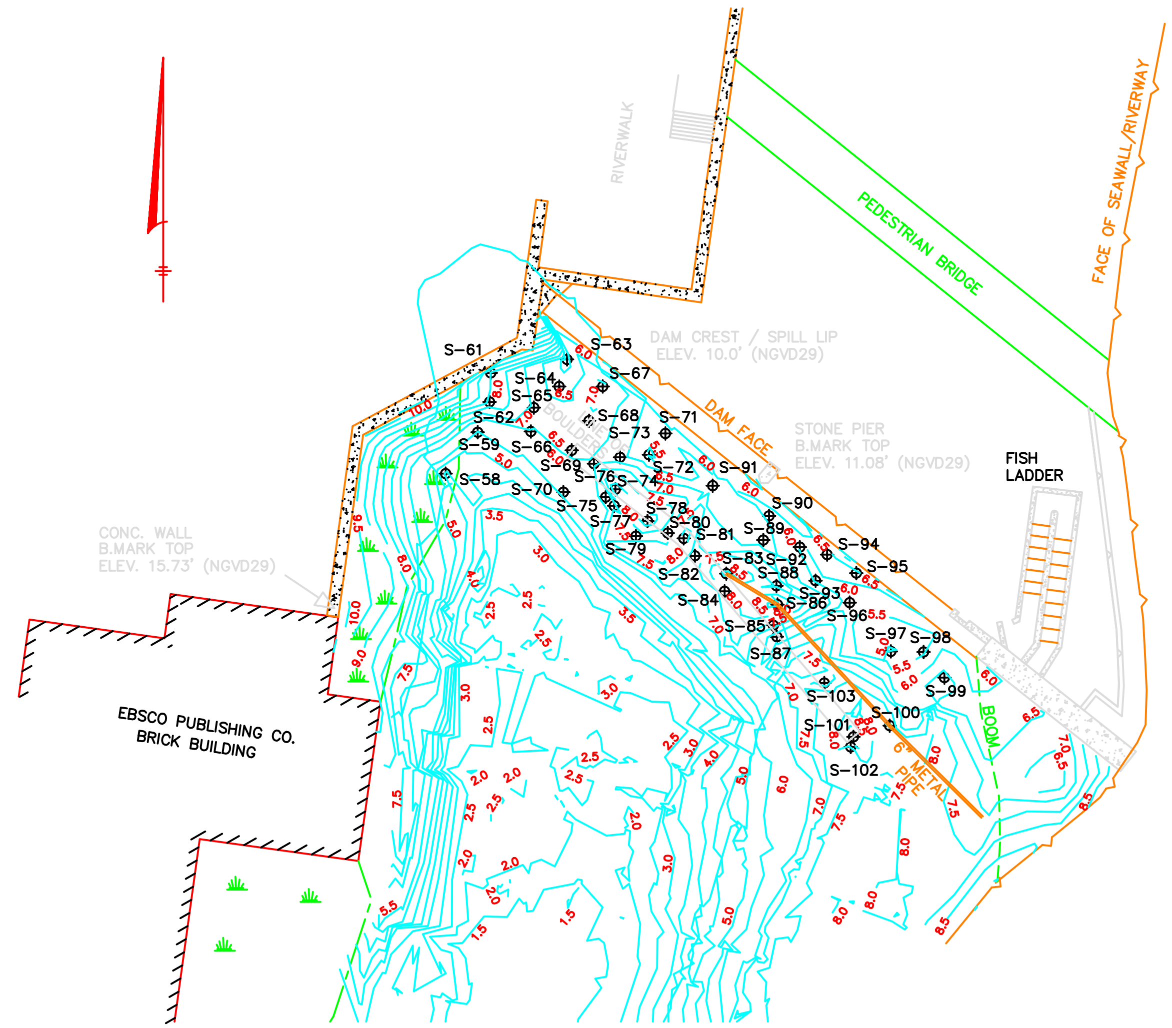
DRAWN BY: P.McCORMACK
DATE: 08/09/14
FIELD CREW: P.McCORMACK, D.JONES, R.GUILIZIA
DRAWING NO.: 777HYDRO.dwg
JOB NO.: 777
SHEET 1 OF 1

ADDITIONAL SAMPLES MEASURED 9/7/14

SAMPLE I.D.	SEDIMENT DEPTH	DESCRIPTION
S-58	1.2'	COARSE/SAND TO FIRM RESIST.
S-59	1.5'	SAND/COARSE TO FIRM RESIST.
S-61	1.7'	SILT TO FIRM RESIST.
S-62	0.3'	SAND TO FIRM RESIST.
S-63	0.1'	HARD PACK STONE.
S-64	0.8'	HARD PACK STONE.
S-65	0.8'	SILT/GRAVEL TO FIRM RESIST.
S-66	0.0'	HARD PACK GRAVEL.
S-67	0.0'	SOLID ROCK.
S-68	2.5'	LOOSE GRAVEL TO FIRM RESIST.
S-69	1.0'	SAND/GRAVEL TO FIRM RESIST.
S-70	1.5'	SILT/COARSE SAND TO FIRM RESIST.
S-71	0.5'	SILT/SAND TO FIRM RESIST.
S-72	0.3'	COBBLE TYPE SCOUR AREA.
S-73	1.3'	COBBLE TYPE SCOUR AREA.
S-74	0.0'	TOP OF BOULDER LINE.
S-75	1.2'	SILT/SAND TO FIRM RESIST.
S-76	0.5'	SILT/SAND TO FIRM RESIST.
S-77	0.1'	SILT TO FIRM RESIST.
S-78	0.0'	TOP OF BOULDER LINE.
S-79	0.2'	SILT/SAND TO FIRM RESIST.
S-80	0.8'	COARSE SAND TO FIRM RESIST.
S-81	0.0'	TOP OF BOULDER LINE.
S-82	0.3'	SAND TO FIRM RESIST.
S-83	___	TOP OF METAL PIPE.
S-84	0.5'	SILT/SAND TO FIRM RESIST.
S-85	0.0'	HARD PACK STONE.
S-86	___	TOP OF METAL PIPE.
S-87	0.0'	HARD PACK GRAVEL.
S-88	2.5'	SILT/SAND TO FIRM RESIST.
S-89	0.5'	SILYT/SAND TO FIRM RESIST.
S-90	0.0'	LARGE ROCK.
S-91	0.8'	COARSE SAND TO FRIM RESIST.
S-93	0.8'	SILT/SAND TO FIRM RESIST.
S-94	0.3'	SILT/SAND TO FIRM RESIST.
S-95	0.3'	SILT/SAND TO FIRM RESIST.
S-96	0.3'	SILT TO HARD RESIST.
S-97	0.8'	SILT/SAND TO FIRM RESIST.
S-98	0.3'	SILT TO HARD RESIST.
S-99	0.8'	SILT/GRAVEL TO FIRM RESIST.
S-100	___	TOP OF METAL PIPE.
S-101	0.0'	TOP OF BOULDER LINE.
S-102	0.5'	SILT/SAND TO FIRM RESIST.
S-103	0.3'	SILT/SAND TO FIRM RESIST.

S-60 SAMPLE SERIES NUMBER NOT ASSIGNED, FIELD NOTEBOOK OMISSION.

METHOD OF DEPTH PROBE: 10 FOOT SECTION OF REBAR HAND-PUSHED TO POINT OF REFUSAL.
LOCATIONS BY SYMBOL: ⊕ S-**



GENERAL NOTES

1.) RIVERBED SOUNDINGS OBSERVED USING AN ODEC BATHY MF500 ECHO SOUNDER WITH REAL-TIME GPS SURFACE NAVIGATION INTERFACED TO HYPACK INC. SOFTWARE.
2.) RIVERBED SEDIMENT DEPTH SAMPLES BY DIRECT PROBE MEASURE & GPS LOCATION.
3.) RIVERBED CONTOURS LABELED AT 0.5 FOOT INTERVALS.
4.) ALL ELEVATION AND CONTOUR DATA RELATE TO THE NGVD29 VERTICAL DATUM.

PATRICK J. McCORMACK – PROFESSIONAL LAND SURVEYOR   DATE

0' 30' 60' 90'

RIVERBED SURVEY
IPSWICH RIVER MILLS DAM
TOWN OF IPSWICH, MA

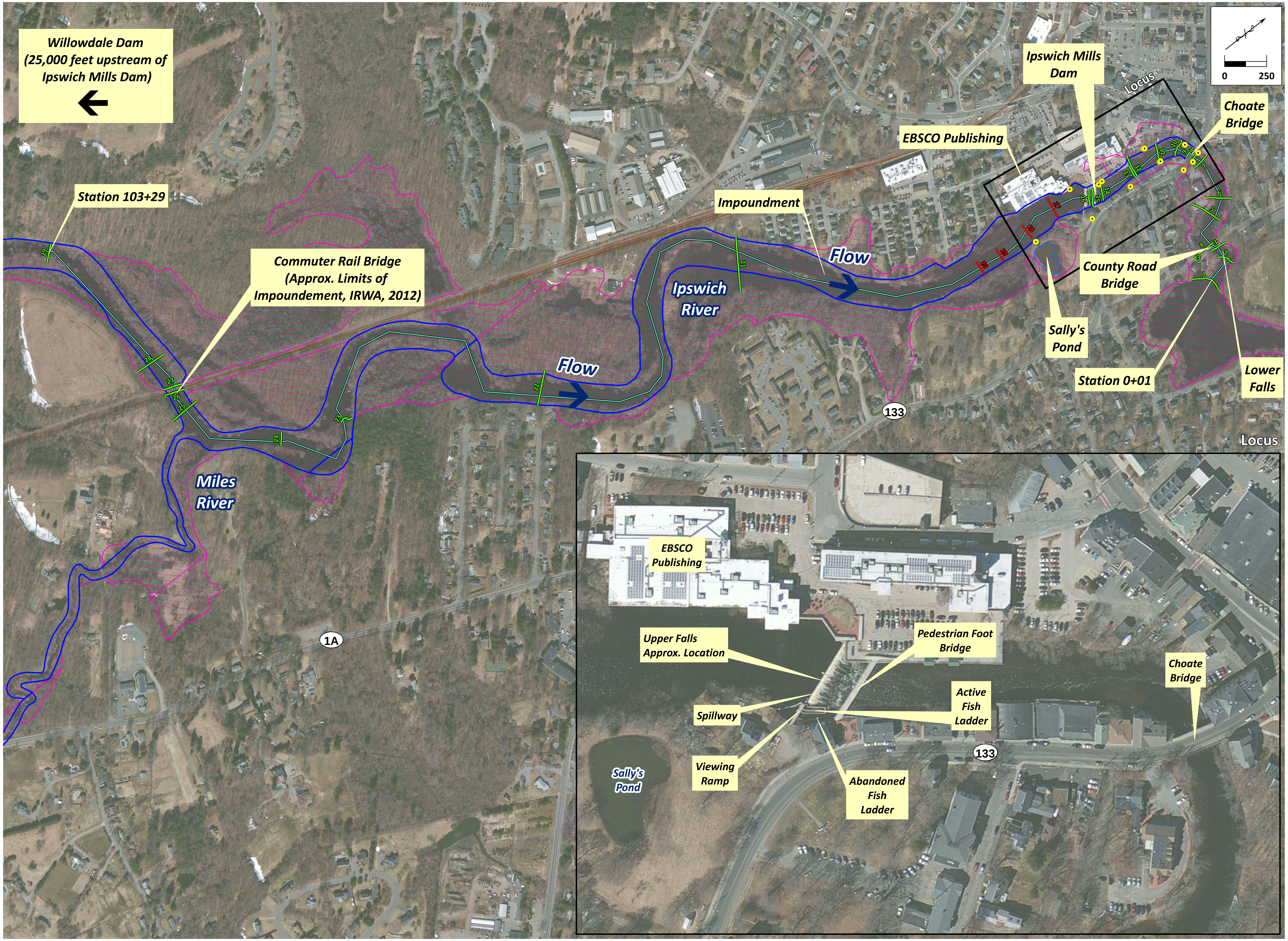
SCALE 1" = 30'   AUGUST 9, 2014

SURVEYED BY
NORDE-EAST SURVEY
27 CONGRESS STREET, Suite 205-8, SALEM, MA 01970.
Tel.(978) 5421920 E-mail: norde-east@verizon.net

NO.	DATE	DESCRIPTION	BY
		REVISIONS	
1.	10/3/14	REVISE CONTOURS BY DAM	P.McC
2.	11/6/14	ADDITIONAL TRANSECT A1	P.McC
3.	11/6/14	ADDITIONAL SEDIMENT DEPTHS	P.McC

DRAWN BY: P.McCORMACK	DATE: 08/09/14
FIELD CREW: P.McCORMACK, D.JONES, R.GUILIZIA	DRAWING NO.: 777HYDRO.dwg
JOB NO.: 777	EXHIBIT ATTACHMENT TO SHEET 1

ATTACHMENT 3



Prepared For:
Department of Fish and Game, Division of Ecological Restoration
Riverway Program
251 Causeway Street, Suite 400
Boston, MA 02114

Prepared By:
Horsley Witten Group, Inc
90 Route 6A
Sandwich, MA 02563
Phone: (508) 833-660
Fax: (508) 833-3150
Dated: March 23, 2017

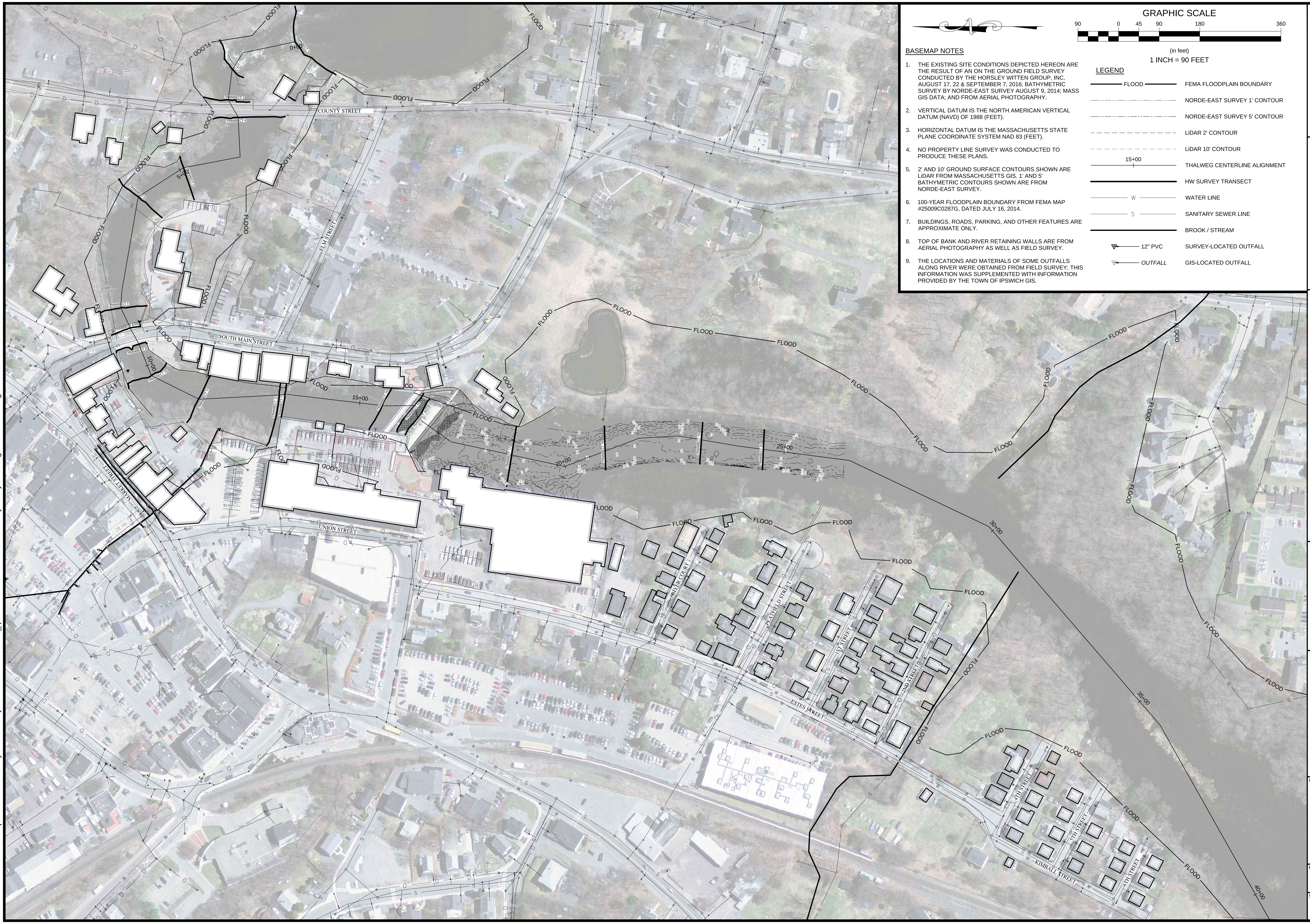
Plan Set:
Ipswich Mills Dam
Removal Feasibility Study
Ipswich, Massachusetts

Plan Title:
Basemap Plan (1)

Legend
- Regulatory Floodway
- Outfall
- Commuter Rail Service
- HW Survey Transects
- Norde-East Survey Transects
- Longitudinal Profile
- FEMA Zone AE - 1% Annual Chance Flood Hazard

Project Number:	Sheet:
16041	1 of 3

Sheet Number:
C-1



GRAPHIC SCALE

90 0 45 90 180 360

(in feet)

1 INCH = 90 FEET

BASEMAP NOTES

1. THE EXISTING SITE CONDITIONS DEPICTED HEREON ARE THE RESULT OF AN ON THE GROUND FIELD SURVEY CONDUCTED BY THE HORSLEY WITTEN GROUP, INC. AUGUST 17, 22 & SEPTEMBER 7, 2016; BATHYMETRIC SURVEY BY NORDE-EAST SURVEY AUGUST 9, 2014; MASS GIS DATA; AND FROM AERIAL PHOTOGRAPHY.
2. VERTICAL DATUM IS THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988 (FEET).
3. HORIZONTAL DATUM IS THE MASSACHUSETTS STATE PLANE COORDINATE SYSTEM NAD 83 (FEET).
4. NO PROPERTY LINE SURVEY WAS CONDUCTED TO PRODUCE THESE PLANS.
5. 2' AND 10' GROUND SURFACE CONTOURS SHOWN ARE LiDAR FROM MASSACHUSETTS GIS. 1' AND 5' BATHYMETRIC CONTOURS SHOWN ARE FROM NORDE-EAST SURVEY.
6. 100-YEAR FLOODPLAIN BOUNDARY FROM FEMA MAP #25009C0287G, DATED JULY 16, 2014.
7. BUILDINGS, ROADS, PARKING, AND OTHER FEATURES ARE APPROXIMATE ONLY.
8. TOP OF BANK AND RIVER RETAINING WALLS ARE FROM AERIAL PHOTOGRAPHY AS WELL AS FIELD SURVEY.
9. THE LOCATIONS AND MATERIALS OF SOME OUTFALLS ALONG RIVER WERE OBTAINED FROM FIELD SURVEY; THIS INFORMATION WAS SUPPLEMENTED WITH INFORMATION PROVIDED BY THE TOWN OF IPSWICH GIS.

LEGEND

FLOOD	FEMA FLOODPLAIN BOUNDARY
	NORDE-EAST SURVEY 1' CONTOUR
	NORDE-EAST SURVEY 5' CONTOUR
	LiDAR 2' CONTOUR
	LiDAR 10' CONTOUR
15+00	THALWEG CENTERLINE ALIGNMENT
	HW SURVEY TRANSECT
W	WATER LINE
S	SANITARY SEWER LINE
	BROOK / STREAM
12" PVC	SURVEY-LOCATED OUTFALL
OUTFALL	GIS-LOCATED OUTFALL

Revisions

Rev.	Date	By	Appr.	Description
1	---	---	---	---
2	---	---	---	---
3	---	---	---	---
4	---	---	---	---
5	---	---	---	---
6	---	---	---	---
7	---	---	---	---

Horsley Witten Group, Inc.
Sustainable Environmental Solutions
www.horsleywitten.com
90 Route 6A
Sandwich, MA 02563
508-833-6600 voice
508-833-3150 fax

Date: MAY, 2017
Designed By:
Drawn By: MCL
Checked By: NP

Plan Set:

IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title:

BASE MAP PLAN (2)

Prepared For:

Mass. Dept. of Fish and Game
Div. of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Survey Provided By:

Horsley Witten Group, Inc.
90 Route 6A
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Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

DRAFT
NOT FOR CONSTRUCTION

Project Number: 16041

Sheet : 2 of 4

Sheet Number: C - 2

last modified: 07/11/18 printed: 07/11/18 by ml H:\Projects\2016\16041 DER_Ipswich Dam Removal Feasibility Study\Drawings\16041 EX.dwg

Revisions

Rev.	Date	By	Appr.	Description

Horsley Witten Group, Inc.
Sustainable Environmental Solutions
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Sandwich, MA 02563
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Date: MAY, 2017 | Designed By: | Drawn By: MCL | Checked By: NP

Plan Set: IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title: BASE MAP PLAN (3)

Prepared For:
Mass. Dept. of Fish and Game
Div. of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Survey Provided By:
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Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

DRAFT NOT FOR CONSTRUCTION

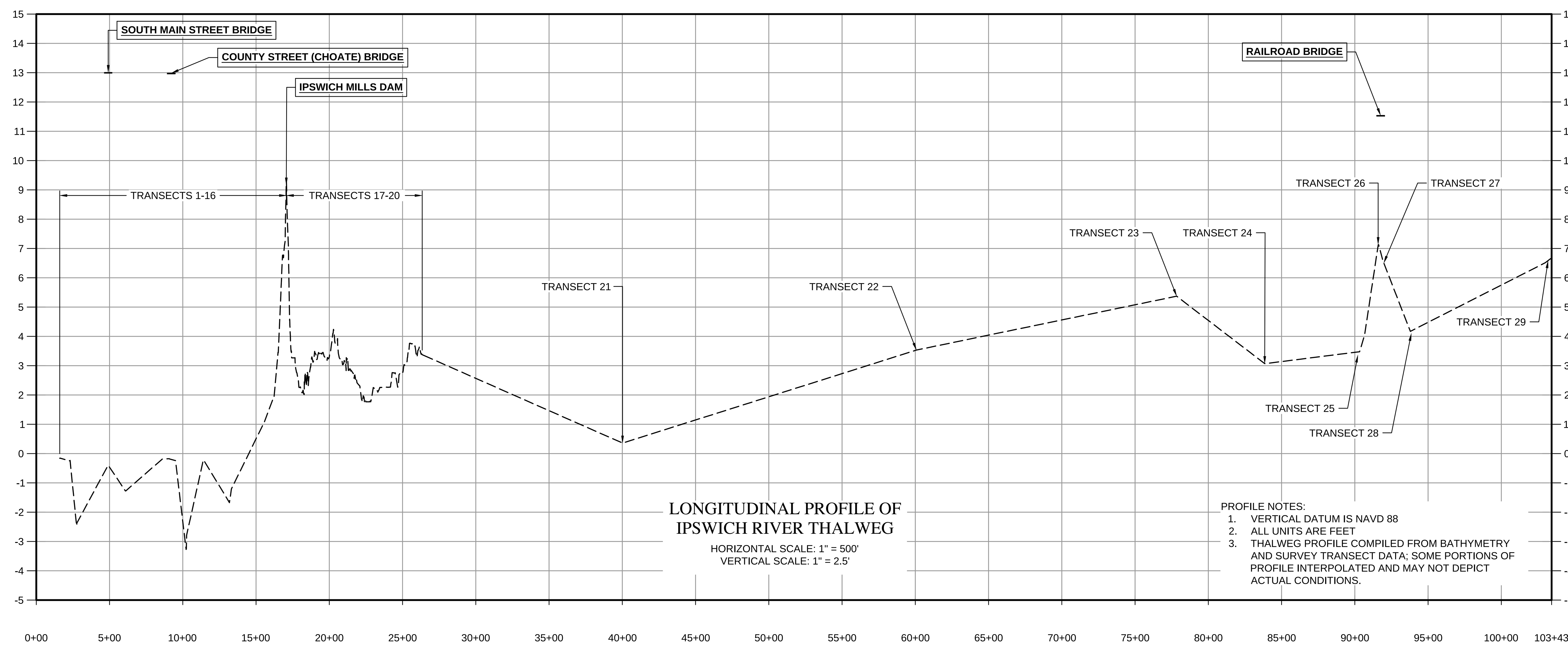
Project Number: 16041
Sheet: 3 of 4
Sheet Number: C - 3

last modified: 07/11/18 printed: 07/11/18 by ml H:\Projects\2016\16041 DER_Ipswich Dam Removal Feasibility Study\Drawings\16041 EX.dwg

last modified: 07/11/18 printed: 07/11/18 by ml H:\Projects\2016\16041 DER_Ipswich Dam Removal Feasibility Study\Drawings\180626 16041 TRANSECTS mcl.dwg

TRANSECT INFORMATION TABLE

OLD NAME	NEW NAME	STATION
TRANSECT 16	TRANSECT 1	0+01
TRANSECT 15	TRANSECT 2	1+62
TRANSECT 14	TRANSECT 3	2+30
TRANSECT 13	TRANSECT 4	2+75
TRANSECT 12	TRANSECT 5	4+91
TRANSECT 11	TRANSECT 6	6+10
TRANSECT 10	TRANSECT 7	8+59
TRANSECT 9	TRANSECT 8	9+09
TRANSECT 8	TRANSECT 9	9+49
TRANSECT 7	TRANSECT 10	10+23
TRANSECT 6	TRANSECT 11	11+41
TRANSECT 5	TRANSECT 12	13+20
TRANSECT 4	TRANSECT 13	13+33
TRANSECT 3	TRANSECT 14	15+50
TRANSECT 2	TRANSECT 15	16+22
TRANSECT 1	TRANSECT 16	16+51
NORDE-EAST	TRANSECT 17	18+79
NORDE-EAST	TRANSECT 18	20+95
NORDE-EAST	TRANSECT 19	23+15
NORDE-EAST	TRANSECT 20	24+54
TRANSECT 25	TRANSECT 21	39+76
TRANSECT 24	TRANSECT 22	60+18
IRWA DOCK	TRANSECT 23	77+83
IRWA DOCK	TRANSECT 24	83+86
TRANSECT 23	TRANSECT 25	90+32
RR BRIDGE D/S	TRANSECT 26	91+60
RR BRIDGE U/S	TRANSECT 27	91+97
TRANSECT 22	TRANSECT 28	93+97
TRANSECT 21	TRANSECT 29	103+29



LONGITUDINAL PROFILE OF IPSWICH RIVER THALWEG

HORIZONTAL SCALE: 1" = 500'
VERTICAL SCALE: 1" = 2.5'

PROFILE NOTES:
1. VERTICAL DATUM IS NAVD 88
2. ALL UNITS ARE FEET
3. THALWEG PROFILE COMPILED FROM BATHYMETRY AND SURVEY TRANSECT DATA; SOME PORTIONS OF PROFILE INTERPOLATED AND MAY NOT DEPICT ACTUAL CONDITIONS.

Revisions

Rev.	Date	By	Appr.	Description

Horsley Witten Group, Inc.
Sustainable Environmental Solutions
www.horsleywitten.com
90 Route 6A
Sandwich, MA 02563
508-833-6600 voice
508-833-3150 fax

Date: July 11, 2018
Designed By:
Drawn By: MCL
Checked By: NP

Plan Set:
IPSWICH DAM REMOVAL FEASIBILITY STUDY
IPSWICH, MASSACHUSETTS

Plan Title:
RIVER PROFILE

Prepared For:
Mass. Dept. of Fish and Game
Div. of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Survey Provided By:
Horsley Witten Group, Inc.
90 Route 6A
Sandwich, MA 02563
Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:
DRAFT
NOT FOR CONSTRUCTION

Project Number: 16041
Sheet: 4 of 4
Sheet Number: C - 4

ATTACHMENT 4

MEMORANDUM

To: Kris Houle, DER; Wayne Castonguay, IRWA, Ethan Parsons, Town of Ipswich

From: Neal Price

Date: June 30, 2017; Revision 1 June 2018; Revision 2 November 2018

Re: Ipswich Mills Dam Removal Feasibility Study – Task 1 Summary

---

The Horsley Witten Group, Inc. (HW) is pleased to submit to the Massachusetts Division of Ecological Restoration (DER) and the Ipswich River Watershed Association (IRWA) the following memorandum summarizing Task 1 work completed as part of the Ipswich Mills Dam Removal Feasibility Project (the Project), located in Ipswich, Massachusetts approximately 700-feet south (upstream) of the Route 133/South Main Street/Choate Bridge crossing (Figure 1). The dam is currently owned and operated by the Town of Ipswich Utilities Department (Haley & Aldrich, 2009). Task 1 consisted of a summary of existing conditions including ecology, historical resources, and physical/ infrastructure conditions. Much of the information on existing ecological conditions was provided by the IRWA, based on its decades of in-house knowledge and experience, with input from Inter-Fluve, Inc. (IF) and HW. Preliminary historical research and reporting were conducted by the Public Archaeological Laboratory,Inc. (PAL). HW compiled existing data on physical conditions and conducted additional on-the-ground survey to create a basemap of existing conditions suitable for use in completing forthcoming design and other project goals.

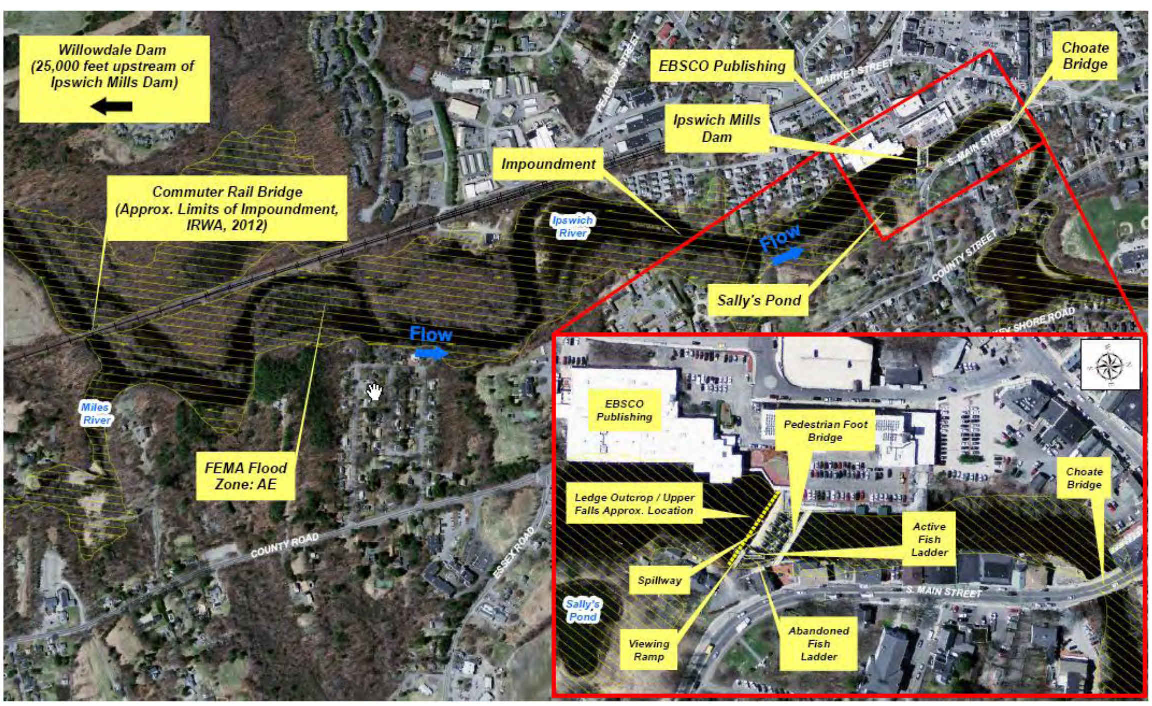


Figure 1. Key Project Area Features

Overview

Existing conditions at the Ipswich Mills Dam Project site were previously summarized by HW in the Ipswich Mills Dam Partial Feasibility Study (HW, 2014). The following is an abbreviated summary of the existing conditions overview from that report, supplemented with additional information learned during the current study. Historical conditions mentioned here are more fully detailed in the Cultural Resources Summary completed by PAL as part of this current Project (Appendix A).

Historical records show that a dam has existed in the vicinity of the Ipswich Mills Dam site since 1637 (Haley & Aldrich, 2009). Historic accounts indicate that the Ipswich Mills Dam was built upon or just downstream of a rock ledge outcrop or small rock rapids, referred to as the 'Upper Falls.' Reference to the Upper Falls is made in historic accounts of the proceedings of the annual meeting of the Ipswich Historical Society XIII, December 7, 1903, Page 24, and is supported by a diagram of downtown Ipswich in the late 17th century indicating a fording location on the river, which would naturally be a shallow firm surface, at approximately the location of the Ipswich Mills Dam before it was built (Ipswich in the Massachusetts Bay Colony 1633-1700; Thomas Franklin Waters, 1905).

A hard surface that may have been ledge and/or large boulders was observed spanning the width of the river approximately 10-20 feet upstream of the dam during an IRWA preliminary field survey in 2010, and during a bathymetric survey conducted by Norde-East, Inc. in 2014. During the field survey conducted by HW during a drawdown of the impoundment as part of this current study in August, 2016, at least the hard surface layer of this feature was observed to consist of boulders, as opposed to bedrock ledge (Figure 2). Therefore it is uncertain at what elevation bedrock ledge may underlie the surficial boulders at the dam site. There is, however, some information that suggests a potential approximate elevation of the bedrock, even if it cannot be accurately identified at this time:

- As part of the Task 3 structural assessment of this current project conducted by Simpson, Gumpertz, and Heger, Inc. (SGH), a test pit excavated in the river at the edge of the EBSCO building foundation, near the western edge of the dam, revealed bedrock at approximately elevation 3.2 feet (NAVD 88). This suggests that, at least near the western edge of the dam, bedrock ledge may be present in the general vicinity of the dam several feet below the elevation of the observed boulder surface.
- During the drawdown, IRWA staff was able to jostle the surface boulders with a steel pry bar confirming the makeup of the surface of the feature as loose boulders. The boulder surface is undulating but has an average elevation of approximately 6 feet (NAVD88). IRWA probed approximately 150 locations across the boulder feature. Of those, 20 went down to a maximum penetration of depth of approximately 5 feet and the remainder penetrated to between 1 and 4 feet (all depths relative to the high point of the boulder feature). In the opinion of the IRWA staff who conducted the probing, the refusal depths are indicative of bedrock ledge. SGH staff, who was onsite at the time of the IRWA probing conducting the test pits mentioned in the above bullet, conducted a level survey of several of the probing locations to relate elevations at probing locations to elevations on top of the dam previously surveyed by HW. The surveyed elevation of the high point along the top of the boulder ridge that the IRWA probing depths were reported relative to is 6.81 feet. Therefore, the lowest elevation of bedrock beneath the boulder ridge

estimated by IRWA staff for 20 probing locations is approximately 1.8 feet. The elevation of bedrock beneath the boulder ridge estimated by IRWA staff for the other 130 probing locations is higher, with variable elevations between approximately 2 and 6 feet.

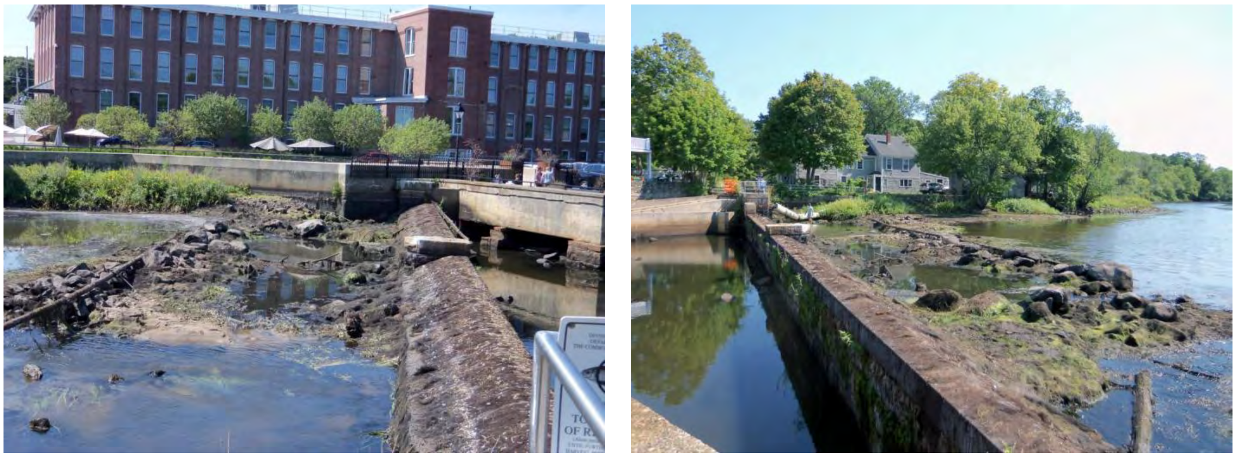


Figure 2. Ipswich Mills Dam during drawdown in August, 2016, left facing west, right facing east

In 1908, the structure was modified to its current structural design to supply nearby mill buildings (at the time) with a reliable source of power. The Ipswich Mills Dam is a run of the river dam that was built for the purpose of generating power for nearby buildings and manufacturing processes. It no longer serves that purpose and now stands as a relic structure in the river. A run of the river dam is operated such that water is not stored in the impoundment to be released at a later time. Rather, the dam simply increases the head in the river, providing a power source that can be captured. It does not serve to prevent or mitigate flooding downstream of the dam since it is generally sized to allow water to flow over the dam during all typical flows. This is typical of many small New England dams.

The current dam is constructed out of cut stones with concrete at some locations with the spillway extending across most of the width of the river. The main spillway is 132 feet wide. A three-foot-wide low level stop-log spillway is at the river-right end of the main spillway with an invert elevation approximately 0.4 feet lower than the spillway. Further to the right, the dam also has a 4.5-foot-wide by 3-foot-high low level gated outlet with an invert elevation approximately two feet lower than the main spillway. Further still to the right is a functional fish way that was installed in 1996 (IRWA). Furthest to the right is an abandoned fish ladder of older construction (Haley & Aldrich, 2009).

The dam had been classified by the Massachusetts Office of Dam Safety (ODS) as an Intermediate Dam with Significant Hazard Potential at the time of the 2009 Haley and Aldrich inspection. Failure of the dam would cause property damage and may result in loss of life if the failure occurred without warning and people were within the initial flood-wave (Haley & Aldrich, 2009). The safety status of the Ipswich Mills Dam with respect to the need for repair was judged as “satisfactory” by Haley & Aldrich. However, the dam is currently classified by the ODS as a Low Hazard Dam (IRWA). Dam located in an area where failure may cause minimal property damage to others and loss of life is not expected.

The Ipswich Mills Dam receives river flows contributed from a 148 square-mile watershed of the Ipswich River upstream of the dam. The watershed is made up of primarily forested land, wetland areas, residential properties, agricultural land, and some commercial/industrial zones. About 160,000 people, in parts of 21 towns, live throughout the watershed (IRWA, 2012). The Ipswich River flows nearly 40 sinuous miles from its headwaters in Burlington, Wilmington, and Andover, MA, to its mouth in Plum Island Sound. It loses approximately 115 feet in elevation along its course. The soils in the watershed are comprised primarily of the Merrimac-Hinckley-Urban land and Paxton-Montauk-Urban land associations. The former includes areas of urban land and deep, nearly level to steep, somewhat excessively or excessively drained, loamy and sandy soils that were formed in outwash deposits. The latter incorporates urban areas and deep, nearly level to steep, well drained, loamy soils formed in glacial till. The Canton-Woodbridge-Freetown soil association also exists in the upper parts of the watershed but to a lesser extent. This grouping includes deep, nearly level to steep, well drained or moderately well drained, loamy soils formed in glacial till and deep, nearly level, very poorly drained, mucky soils formed in organic deposits (USDA SCS, 1981; Fuller and Francis, 1984).

The USGS maintains a gage located 200 feet downstream from the Willowdale Dam, or approximately 4.6 miles upstream of the Ipswich Mills Dam, and has continuously recorded water surface elevation and discharge data as far back as June 1930. Monthly mean flows at the Willowdale Dam between 1930 and 2009 range from 42.0 cubic feet per second in August to 446 cubic feet per second in March. The highest flow on record of 4,600 cfs occurred on May 16, 2006. Two photos of the Ipswich Mills Dam on May 16, 2006 are provided below showing that the dam is virtually drowned out by the discharge in the river (Figure 3).


Figure 3. Ipswich Mills Dam on May 16, 2006, facing southwest (left) and northwest (right) (photos by IRWA)

The Ipswich Mills Dam Partial Feasibility Study (HW, 2014) also addressed the potential for contamination in the sediments contained within the impoundment, and the potential for structural impacts to the EBSCO building adjacent to the impoundment on the river-left side. The preliminary sediment quality assessment was led by Clean Soils Environmental, Ltd., with assistance from IF and IRWA. The preliminary structural assessment of the EBSCO building was conducted by GEI, Inc.

The preliminary sediment quality assessment opined that the sediments found behind the Ipswich Mills Dam have a very low likelihood of toxicity when viewed independently and in relation to other dams across Massachusetts. This opinion was based on the review of data from two sediment cores previously collected behind the dam by The U.S. Geological Survey (USGS) and three cores collected by IRWA and IF in 2012 as part of the Clean Soils Environmental preliminary assessment. Generally, data from both sampling events indicate that the sediment is below applicable ecological impact benchmark limits and that a condition of 'No Significant Risk' may exist from the sediment behind the dam. The concentrations of metals, SVOCs, pesticides, VOCs, and EPHs measured within the sediment appear to be consistent with surface water runoff from non-point sources (e.g., roadways and farming). More extensive sediment sampling and analyses conducted closer to the time of anticipated permitting will be required as part of the environmental permitting process if dam removal is further pursued at Ipswich Mills.

The preliminary structural assessment (GEI, 2014) of the EBSCO building was limited to a review of existing information, including logs from three borings performed in 2009 immediately south of the southeast corner of EBSCO's Building No 10-A. The assessment opined that the portions of the EBSCO buildings along the Ipswich River could be supported on timber piles given the soil conditions along the river and the age of the buildings. However, the preliminary assessment was not able to confirm the presence of timber piles nor identify any information regarding the elevation of the tops of the suspected timber piles, if they were present. GEI also observed that some of the existing and former buildings pre-date the construction or reconstruction of the existing dam. It is possible that the tops of the foundations supporting the buildings that pre-date the current dam were constructed when the impoundment behind the dam was maintained at a lower elevation.

If portions of the EBSCO buildings are supported on timber piles, the tops of the timber piles need to remain below water to protect them from rapid deterioration (biodeterioration). Methods that have been implemented on other projects to protect timber piles have included maintaining groundwater levels high enough to keep the piles submerged, lowering the tops of the piles below the expected future groundwater level, or a combination of raising groundwater levels and cutting off the tops of the piles.

As part of this current study, a more detailed structural investigation of the EBSCO buildings and their potential to be supported by timber piles at risk from exposure has been conducted by Simpson, Gumpertz, and Heger, Inc. (SGH). The results of that investigation will be separately submitted as a Task 3 Summary Report.

Ecological Summary

The ecology of the Ipswich River watershed is well studied. The IRWA, DER, and their municipal, state and federal partners have conducted regular monitoring of water quality (clarity, temperature, dissolved oxygen, flow and conductivity), herring passage at fish ladders, fish populations and macroinvertebrates. The overall health of the system has been summarized in various documents, including Bowling and Mackin (2003) and Armstrong et. al (1999). IRWA publishes strategic planning, best management plan (BMP) assessments, and annual reports documenting efforts in the watershed.

Historically, the Ipswich River watershed supported abundant fisheries resources including significant populations of diadromous (sea-run) fish. Diadromous fishes common in the Ipswich and its estuary included river herring (alewife and blueback herring), American shad, rainbow smelt, sea lamprey, Atlantic sturgeon and Atlantic salmon (Jerome et. al 1968). The River's first name, Agawam, a Native American term which translates to "place where fishes of passage resorted" speaks to the abundance of this former fishery (Jerome et. al 1968). Alewife spawning returns once numbered in the millions of fish, supporting a substantial commercial fishery. At its peak, the Ipswich alewife fishery harvested thousands of barrels of herring from the stretch of river just downstream of the dam, near Choate Bridge (Belding & Corwin 1921). The Town of Topsfield established a public alewife fishery near Hood Pond (then called Prichard's Pond) in 1803 (Belding & Corwin 1921).

Reduced base flow caused by development of the watershed and groundwater withdrawals has been shown to be a major driver in fish and wildlife assemblages in the Ipswich River. Dams also cause serious impacts to fish and wildlife populations. Armstrong et. al (1999) showed that over 90 percent of fish in the Ipswich River are generalists tolerant of lentic or lake conditions, whereas the historic native fishery was composed of lotic or riverine species requiring flowing water to thrive. According to IRWA, the frequency of low water and reduced water quality episodes in portions of the watershed may be preventing some species from reaching reproductive age. Species now common in the Ipswich River include American eel, golden shiner, yellow perch, largemouth bass, pumpkinseed, brown bullhead and white sucker. Species that are extremely rare or have been locally extirpated in the river upstream of the Ipswich Mills Dam include Johnny darter, white perch, rainbow smelt, and Atlantic salmon. River herring runs are monitored yearly, but totals are typically less than 1,000 spawners per year. Purinton et. al (2003) estimated that the Ipswich River is currently supporting less than 1% of its total spawning potential. Belding & Corwin (1921) blamed alewife decline in the Ipswich on a number of factors, but primarily on the combined influence of conversion of historic spawning ponds (e.g. Wenham Lake and Suntaug Lake) to water supply use and to obstruction of migration pathways by dams. The fact that the Ipswich Mill dam did not have any functional fish passage between 1906 and 1996 probably eliminated the bulk of the anadromous fish gene pool.

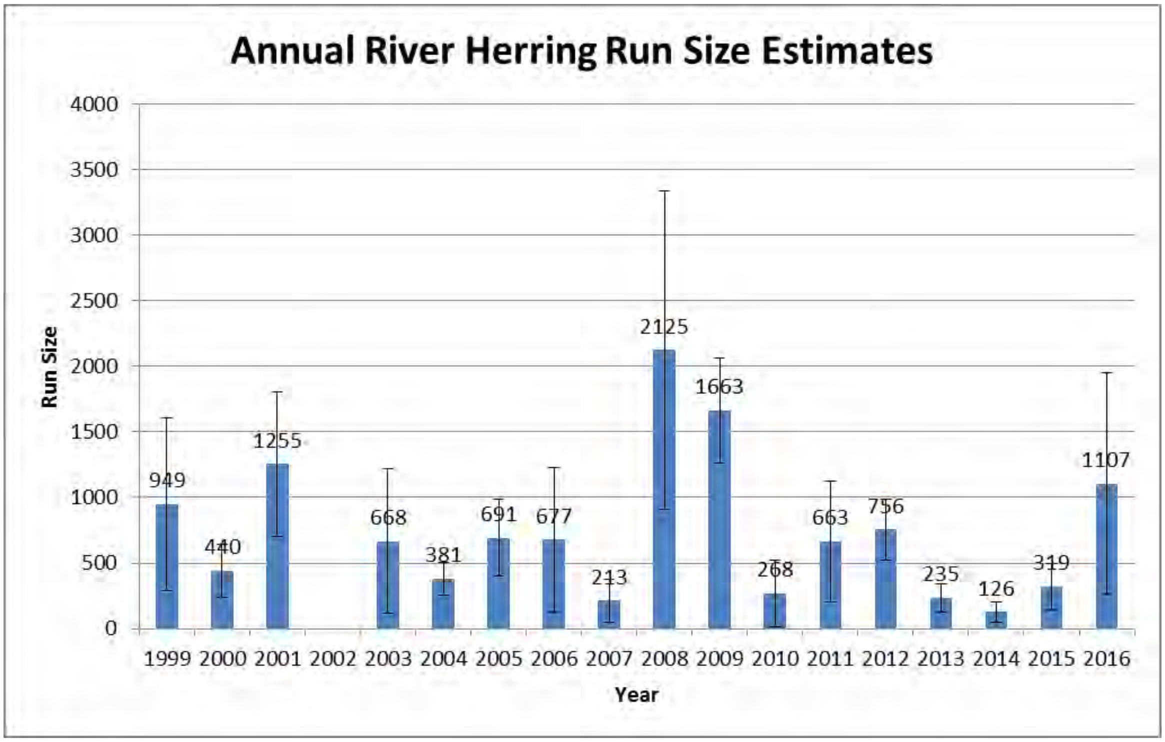


Figure 4. Annual run-size estimates for Ipswich River herring counts, 1999-2016. (from O'Donnell 2016)

The Ipswich Mills Dam is located at the head of tide (upstream limit of tidal influence) roughly 3.7 miles from the Ipswich River's mouth. The dam's location at the interface between fresh and brackish water makes its ecological impact particularly profound as these estuarine transition zones are very rare and productive habitats. Rainbow smelt are known to spawn in the tidal waters immediately downstream of the dam which limits the upstream extent of available spawning habitat for this species (Chase 2006). In addition to limiting many migratory fish species from moving upstream into the watershed to spawn or feed, the dam also presents a problem to freshwater resident species that pass over the dam for one reason or another, including many freshwater fish, turtles and other species that cannot survive long-term below the dam. With the exception of wildlife that are strong swimmers or good climbers many of these animals are likely to be permanently trapped below the dam.

The natural flow regime of a river organizes and defines its ecosystem through physical processes that create and continually adjust habitat characteristics (Poff et al., 1997). Dams disrupt the natural flow regime of river systems, short circuiting many of these habitat-forming processes, altering water chemistry and temperature profiles, and slowing or stopping natural downstream transport of sediment and large organic material.

The Ipswich Mills Dam is a run of river structure that maintains a very consistent water level in the impoundment that extends over a mile upstream from the dam. This consistently elevated water level produces unnatural lentic habitat conditions in this reach of the river, favoring habitat generalist fish species and pond-like invertebrate communities as noted above. At Ipswich Mills, the water level still rises and falls with storms, but many natural stream processes are impaired since water levels no longer drop below the crest of the dam, within the banks of the natural river channel. This persistent inundation of the historic floodplain has essentially halted channel forming processes in the impoundment and has likely contributed to a general lack of habitat complexity in the river channel and benthic portions of this stretch of river.

The area in and around the current impoundment supports abundant wildlife populations. Semi-aquatic animals commonly seen in the water and the riparian areas include mammals (e.g. beaver, muskrat, river otter), birds (e.g. blue heron, wood duck, mallard duck, kingfisher, Canada goose), and reptiles (e.g. painted turtle, musk turtle, snapping turtle). The impoundment also has considerable populations of unionid freshwater mussels. During a temporary drawdown of the impoundment in August 2016, more than 5,000 mussels were relocated from shallow areas to deeper portions of the impoundment (IRWA unpublished data).

Rare animal species (including endangered, threatened, special concern and watch list) that have been documented in the Ipswich River Watershed include bridle shiner, piping plover, least tern, least bittern, golden-winged warbler, pied-billed grebe, Cooper's hawk, northern harrier, salamanders (spotted, blue-spotted, marbled and four-toed), eastern pond mussel, box turtles (spotted, Blandings and eastern), and a number of invertebrates.

A 2004 Massachusetts Division of Marine Fisheries (DMF) report notes that the Ipswich River likely has good restoration potential for river-spawning anadromous fish such as American shad and blueback herring (Reback et.al 2004). Results from field work conducted by DMF from 2006-2009 identified five or fewer of each of these species passing above the Ipswich Mills fish ladder on any given year (O'Donnell, 2014). It is not known whether the low numbers of shad and blueback are associated with passage conditions at the fish ladder or lack of a remnant Ipswich River population, but this fish ladder design is known to be inefficient at passing shad upstream.

As noted above, there are a number of factors that likely contributed to the decline and continued low populations of diadromous fishes in the Ipswich River. While fish passage at dams and other structures is seen as a major factor, it is not the only one. Similarly, restoring passage at one barrier cannot in itself restore healthy fish runs if additional barriers continue to block access to important spawning and rearing habitat. For instance, in the Ipswich River, the best spawning habitat for alewife is in lakes and ponds located above two or more dams further upstream. We can only expect alewife populations to rebound significantly when we restore efficient access to some of those historically productive upstream water bodies.

While much of the historic spawning habitat for alewife is impacted by use for public water supply reservoirs, some of the historically productive ponds remain potential restoration opportunities including Martins Pond (North Reading) and Hood Pond (Topsfield/Ipswich), which were two of the major spawning areas for alewife in the system. Alewife have not had access to either of these ponds for over a century, but studies suggest the ponds remain potentially viable spawning and nursery habitat (IRWA unpublished report, June 2018). The Ipswich River Watershed Association is currently working with the Division of Marine Fisheries to assess spawning habitat suitability at both ponds with the hope of restoring a sustaining alewife population when migratory access can be restored. A project underway to remove the South Middleton Dam, upstream of the Ipswich Mills Dam, will restore access to Martins Pond.

Restoration of sizable populations of diadromous fish to the Ipswich River Watershed would have ecosystem-wide importance. Large spawning runs of anadromous species such as river herring and shad bring large influxes of marine-derived food and nutrients to the freshwater system. They are also important as a forage fish, serving as prey for numerous piscivorous predators while at sea (e.g., tuna, cod, dolphins, billfish, gannets), in estuaries (e.g., striped bass, bluefish, weakfish, harbor seals, cormorants), and in rivers (e.g., ospreys, white perch, herons, river otters). The current low populations of diadromous forage species have important implications throughout marine and freshwater food webs.

Potential Ecological Impacts from Dam Removal

Potential ecological impacts that might occur from removal of the Ipswich Mills Dam will be more fully addressed in the Task 2 Hydrology & Hydraulics Assessment completed as part of this current study, and to be submitted separately. A generalized discussion is presented here. In the short-term, perhaps the most significant impact from the removal of a dam is the release of potentially mobile sediment that has accumulated behind the dam. Down stream sediment transport is a natural riverine process. That natural process is altered by the presence of dammed impoundments which tend to capture and accumulate sediment migrating from upstream sources while thereby depriving downstream areas of the sediment supply needed to support a vibrant riverine ecology. Following dam removal, there tends to be an accelerated process of removing sediment from the impounded area and redistributing it to downstream areas. In time, a new equilibrium is reached that reflects the river's hydraulics and sediment dynamics post dam-removal.

The short-term impact from sediment migration on aquatic species depends on the concentration and exposure time, both of which can vary dramatically in a dam removal. Suspended sediment occurring after every rainfall event in natural, stable streams does not produce mortality in fish, and laboratory experiments exposing fish to suspended sediment showed mortality only at extremely high concentrations (e.g., Bisson and Bilby 1982, Berg and Northcote 1985, Cordone and Kelley 1961, Gradall and Swenson 1982). While sessile communities like invertebrates can suffer significant impacts downstream of dam removals, fish are able to move upstream or downstream of the impact zone and thus avoid many of the negative impacts. Fish species can respond quickly to the increases in turbidity, bedload and temperature following dam removals.

A bathymetric and depth-to-refusal survey was carried out within the impoundment in August 2014 (Norde-East Survey 2014) showing 0.2 to 3 ft of sand and silt accumulation above firm subgrade, or what may have been the historical stream bed. However, the boulders, and potentially underlying ledge, located upstream of the Ipswich Mills Dam may continue to create some semi-impounded conditions reducing the mobility of sediment out of the impoundment area if the dam were to be removed. Timing the Ipswich Mills Dam removal to begin releasing sediment well ahead of fish migration periods would also help to minimize impacts. The tide will also help to move sediment through the system, eventually delivering it to the estuary, helping to build and sustain the estuary. Some deposition may occur in the Great Cove immediately downstream of County Street (approximately 0.3 mile downstream of the dam) where the channel is artificially widened and flow velocities decreased.

Wetland delineation by the Massachusetts Department of Environmental Protection (Mass DEP, 2009) shows areas of deep marsh, shallow marsh, wooded swamp, and shrub swamp bordering the main channel through the impounded reach upstream of Ipswich Mills Dam. In the longer term following dam removal, normal water levels will fall, and it is likely that some of the shallow water wetland areas will evolve into a different type of wetland or upland habitat. Areas currently shown as deep marsh and existing backwater areas are likely to remain as shallow water wetland habitat. Given that these areas are anticipated to experience cyclical water level fluctuations as a result of downstream tidal fluctuations (though the potential extent of actual salt water encroachment remains unknown), the resulting wetlands may be characterized as tidal freshwater wetlands, one of the rarest wetlands habitats in Massachusetts. These wetlands would be capable of supporting rare freshwater plant species currently uncommon in the Ipswich River watershed, or other nearby areas.

For typical small dams, removal results in the restoration of a river's natural water temperature regime through the former impoundment area and downstream of the dam (e.g., Pawloski and Cook 1993). The narrowed cross-section and increased velocity through the former impoundment area equates to cooler temperatures resembling those of the stream upstream of the dam's influence. Decreased post-dam removal water temperatures favor those stream fishes adapted to cool or coldwater environments (Born et al. 1998). In the Ipswich River, within the impounded area, stagnating flows result in rising water temperatures. Removal of the dam will encourage active flow and help reduce water temperatures, making this part of the river more hospitable to flow dependent and coldwater fish species such as brook trout and fallfish. Removal of the dam will also allow free movement of motile aquatic organisms past the dam site to take advantage of food resources and to escape periodic, unsuitable conditions in currently impounded area.

In general, following dam removal, overall lotic macroinvertebrate abundance and diversity tends to increase relative to that of impoundment communities as a new channel is formed and more heterogeneous in-channel habitat becomes available for both invertebrates and fish (Bushaw-Newton et al. 2002, Calaman and Ferreri 2002, Pollard and Reed-Anderson 2001). Such a change is anticipated following removal of the Ipswich Mills Dam. Restoration of sediment continuity through this reach would be beneficial over the long term, not only for restoring habitat locally, but also for replenishment of sediment in the estuary downstream. Water quality and habitat improvements coupled with restoration of aquatic organism passage will have long-term ecosystem benefits that outweigh the anticipated short-term impacts.

Historical/ Cultural Resources Summary

As part of this current Project, PAL completed an initial historical and cultural resources summary. That PAL report is attached to this memorandum as Appendix A. It provides a cultural resources narrative that includes a summary of what is known about the pre-and post-settlement history of the dam site based upon information obtained from the Ipswich Historical Commission, the Massachusetts Historical Commission (MHC), and other sources. The report contains the following information: identification of historic properties and previously surveyed archaeological and architectural resources within and immediately adjacent to the Project area; cultural context relating the pre-history and history of the dam site including former dams and their date(s) of construction; and recommendations concerning potential impacts to cultural resources or additional cultural resources survey efforts that may be needed if the Project proceeds into design and permitting.

The initial historic and cultural summary completed as part of the current project is not a formal "Section 106" historical review as will likely be required if the Project moves further into construction. It does, however, provide an initial understanding of the bigger picture cultural and historical resources likely important to the Project, as well as a foundation for future coordination between the dam owner, state and federal permitting agencies, and the MHC, should the Project progress into design and permitting.

Physical Conditions Survey and Basemap

With assistance from IRWA, the Town, and DER, HW compiled existing information around the Project area on topography, bathymetry, wetlands, hydrography, structures, utilities, roads, and other infrastructure relevant to the assessment of potential Ipswich Mills Dam removal. These data included the following:

- MassGIS LiDAR topography, aerial photography, wetlands, and hydrography;
- Impoundment bathymetry from Norde-East, Inc. (2014);
- Town GIS drainage, wastewater, drinking water, and other utilities;
- FEMA flood zone mapping; and
- USGS National Hydrography Dataset.

HW then supplemented these existing data with three days of additional field survey on August 17th and 22d, and September 7th, 2016, followed by an additional day on April 5th, 2018. The survey was conducted in the NAD83 horizontal datum, feet, and the NAVD88 vertical datum, feet. The Norde-East bathymetric survey was converted from the NGVD29 vertical datum, feet to the NAVD88 datum in order to allow all Project data to be presented consistently in NAVD88, feet. The HW survey collected the following data:

- 26 river transects;
- Dam details and related infrastructure;
- Details of the Pedestrian Bridge, Choate Bridge, County Road Bridge, and Railroad Bridge;
- Details of the pedestrian riverwalk area on river-left immediately downstream of the dam;
- Details of key building and wall locations immediately adjacent to the river in the dam area;
- Stormwater pipes and outfalls entering the river in the dam area;
- Sewer main downstream of the dam in the river channel; and
- Mean high water indicators along the river channel.

In addition, bathymetric data from the impoundment (Norde-East, 2014) were converted to the NAVD88 vertical datum and processed to create six transects through the impoundment. These six impoundment transects, along with the 26 river transects were all merged with upland LiDAR topography data from MassGIS to create extended transects across the entire flood plain for hydraulic modeling. These 30 extended transects were supplied to IF to inform the Task 2 HEC-RAS hydraulic modeling for the Project (to be separately submitted as a Task 2 Summary Memorandum).

All existing and surveyed data were combined together to create a project basemap that includes all available information likely to be relevant to forthcoming dam removal conceptual design activities. The basemap was submitted separately to DER and IRWA. It is a four-sheet set in 24X36-inch page size format that includes a large scale view of the entire (approximately five river mile) Project area from above the Railroad Bridge down to below the Lower Falls, an intermediate scale view of the key Project area from the impoundment down to the Choate Bridge, a fine scale view of the immediate dam area, and a longitudinal profile along the entire Project area.

HW also collected continuous water level data at a six minute interval immediately upstream and downstream of the dam from September 7th to November 6th, 2016 in order to assess the extent of tidal influence of the Project area. To complement water level data at the dam site, HW also obtained continuous water level data collected by the Plum Island Ecosystems (PIE) Long Term Estuarine Research (LTER) from the Ipswich Yacht Club in Plum Island Sound near the mouth of the Ipswich River. All water level data were corrected to the NAVD88 vertical datum based on the HW survey and information from PIE. Figure 4, below, compiles water level data from all three locations and also includes National Weather Service precipitation data from the Beverly Airport. Figure 5 depicts a closer scale view of the spring tide period between October 13th and 23d, 2016. The following are some key observations regarding the water level data.

- Water levels were recorded while the river was still recovering from the 2016 drought and the August 2016 drawdown. Water levels above the dam illustrate a slow rise as the river responds to smaller precipitation events in September and quicker responses to two larger precipitation events in early to middle October. Water levels below the dam are dominated by tidal hydraulic influence and show only a moderate increase beginning with the largest precipitation event in early October.

- No tidal response can be observed above the dam in Figure 4. In the closer scale Figure 5 there is still no observed periodicity of rising and falling water levels above the dam in response to rising and falling tidal levels below the dam. However, there is a gradual and subtle rise in water level above the dam (on the order of a tenth of a foot or so) over approximately the highest four days of the peak tidal cycle. The bulk of that observed upstream water level response corresponds to a small precipitation event on October 18th, however water levels began subtly rising above the dam at least a day before that rain event. The reason for this is uncertain.

- Even the lowest yacht club tide over the period (2.7 feet) created at least a minimal hydraulic response at the toe of the dam during the early, drier part of the record (upstream river stage below around 7 feet). Later on, when the river flow has increased (upstream river stage around 8 feet), even yacht club tides of around 3.5 feet don’t create much hydraulic response at the dam. This indicates that when fresh water flows are high, that influence overwhelms the tidal influence of the lowest high tides. Note that the later period of this data record when upstream river stages are above 8 feet corresponds to a neap period of the tidal cycle. It is likely that hydraulic responses to spring high tides would be observed at the toe of the dam even during these higher river flow periods, though perhaps not flood events.

- The water is always a little higher at the toe of the dam than it is down at the yacht club. This is due to outgoing river flow riding on top of the incoming tide (for the higher high tides), or for the outgoing river flow simply being held up a bit by the hydraulic influence of the incoming tides (for the lower high tides). This is borne out by the survey data. The rock pile below the dam is mostly above elevation 4 feet with only couple of spots dipping just below 3 feet. Therefore, lower yacht club high tides of around 2.7 feet would not be likely to put actual tidal water up to the toe of the dam; just slow down the outflow of river water. In addition, there is some frictional tidal resistance as you move upriver from the yacht club, so the height of actual tidal salt water will be progressively less as you move up towards the dam.
- The effectiveness of the rock pile below the dam at retaining water in a pool at the toe of the dam even during outgoing tides is illustrated by the fact that water levels at the toe of the dam do not drop below approximate elevation 3.5 feet through early October, and approximate elevation 5 feet through later October and November.
- The closer scale Figure 5 indicates that there is about an hour time lag between the high tide at the yacht club and the toe of the dam.
- Peak high tides at the toe of the dam reach to approximate elevation 7 feet for the highest spring tide. This is above the water level observed above the dam during the drawdown and also well above the estimated bedrock controlling elevation in a potential dam-out scenario. Therefore, in a potential dam-out scenario, it appears that high tides would exert hydraulic influence above the current dam location. Whether any actual saline water would reach above the current dam site cannot be predicted with the available data, though the likelihood of significant amounts of saline water reaching above the dam appears low due the preference for higher density salt water to remain low and fresh water to sit above it.

Attachments: Appendix A - Historical and Cultural Resources Summary by PAL, Inc.

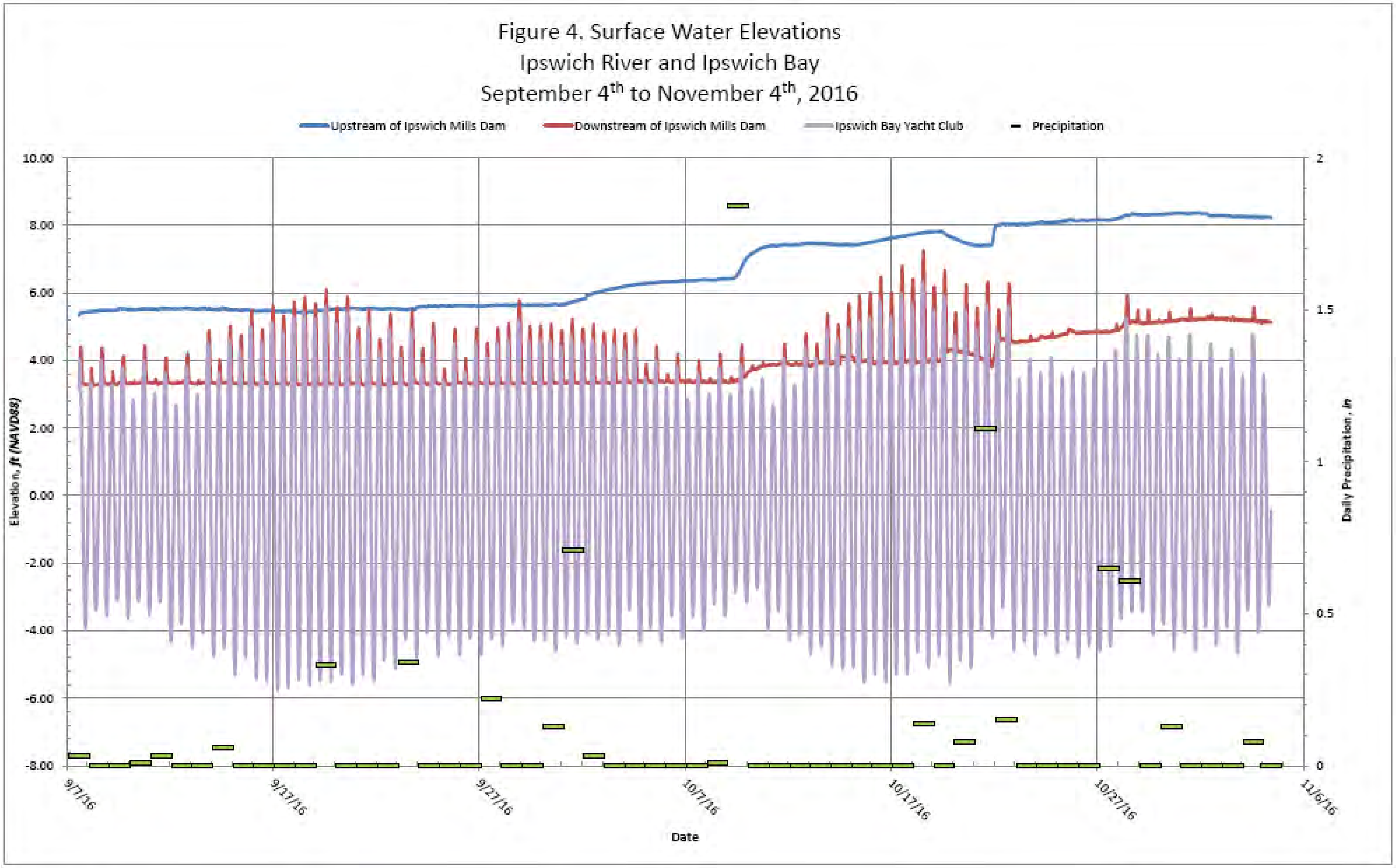


Figure 4. Surface Water Elevations
Ipswich River and Ipswich Bay
September 4th to November 4th, 2016
Upstream of Ipswich Mills Dam
Downstream of Ipswich Mills Dam
Ipswich Bay Yacht Club
Precipitation
Elevation, ft (NAVD88)
Daily Precipitation, in
Date
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
-4.00
-6.00
-8.00
2
1.5
1
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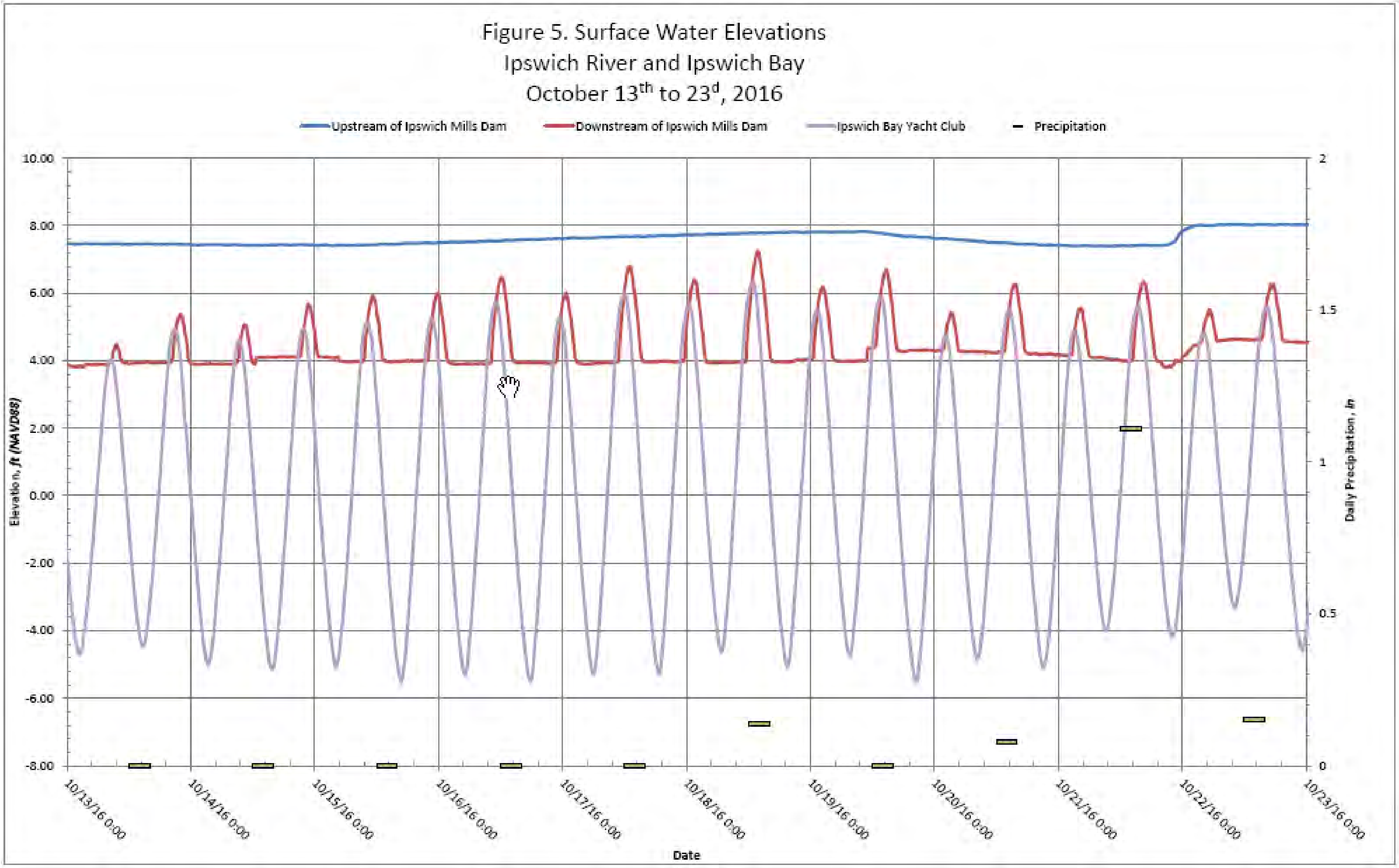
Figure 5. Surface Water Elevations
Ipswich River and Ipswich Bay
October 13th to 23d, 2016
Upstream of Ipswich Mills Dam
Downstream of Ipswich Mills Dam
Ipswich Bay Yacht Club
Precipitation
Elevation, ft (NAVD88)
Daily Precipitation, in
10.00
8.00
6.00
4.00
2.00
0.00
-2.00
-4.00
-6.00
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2
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1
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10/13/16 0:00
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Date

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APPENDIX A – Cultural Resources Summary

Report
Ipswich Mills Dam Removal-Feasibility Study
Ipswich, Massachusetts

Cultural Resources Summary

February 21, 2017

Submitted to:

Horsley Witten Group
90 Route 6A
Sandwich, Massachusetts 02563

This report provides a Cultural Resources Summary for the Ipswich Mills Dam Removal Feasibility Study (the Project). As part of the Project's Feasibility Study, the Massachusetts Division of Ecological Restoration (MA DER) requested a cultural resources narrative that includes a summary of what is known about the pre-and post-settlement history of the dam site using information from the Ipswich Historical Commission, the Massachusetts Historical Commission (MHC), and other sources. The report contains the following information: identification of historic properties and previously surveyed archaeological and architectural resources within and immediately adjacent to the feasibility study area; cultural context relating the pre-history and history of the dam site including former dams and their date(s) of construction; and recommendations concerning potential impacts to cultural resources or additional cultural resources survey efforts that may be needed if the Project proceeds into design and permitting.

The narrative serves two purposes: 1) provide information for the public; and 2) provide a foundation for future coordination between the dam owner, agencies, and the MHC, should the Project progress into design and permitting. The Public Archaeology Laboratory, Inc. (PAL) completed the cultural resources summary under contract with the Horsley Witten Group on behalf of MA DER.

Information Sources

MHC and PAL Repositories

PAL conducted a review of the Massachusetts Historical Commission's (MHC) *Inventory of the Historic and Archaeological Assets of the Commonwealth* (MHC Inventory) files to identify previously recorded cultural resources within and immediately adjacent to the feasibility study area. The online file review included historic properties (those that are listed or evaluated as eligible for listing in the National Register), resources that are included in the State Register, and surveyed properties that have not been evaluated for registration. Readily available cultural resource management (CRM) reports, town histories, and historic maps salient to the study area were also consulted. Pre-contact Native American settlement in the dam section of the Ipswich River drainage was researched using information contained in previous CRM reports and scholarly studies on file at PAL and the MHC.

Local Historical Research and Contacts

PAL reviewed historical documents and town maps available online through the Ipswich Historical Commission and/or Historical Society websites, to supplement existing dam histories and assist in preparing the cultural resource summary for the Feasibility Study. PAL conducted research at the Ipswich Public Library Archives, which contains a wide selection of materials on local history including several hundred documents relating to the history of Ipswich from 1634 to 1985, postcards, and photographs. PAL also reviewed all available dam inspection reports and supporting documentation provided by the Project team.

PAL also reviewed the Ipswich Museum Collection of the Ipswich Museum (formerly the Ipswich Historical Society). The collection is housed in the Ipswich Room at the Public Library, and includes correspondence, deeds, genealogical notes, notebooks, diaries, account books, and other materials. The *Inventory of Archives Committee Materials*, a collection finding aid, was used to facilitate the site-specific research at the Ipswich Public Library Archives. PAL limited the local research to documents in these various collections that relate specifically to the dam and associated mill businesses.

PAL interviewed multiple local informants to identify research repositories and pertinent documents and to collect additional information concerning the history of the Ipswich Mills Dam. Gordon Harris, Town of Ipswich Historian, shared his research and writing concerning the history of the dam. Stephen Stickney, current owner of the masonry company Stephen A. Stickney Co. of Boxford, Massachusetts, provided information concerning work at the dam completed in the 1970s. Katherine Chaison, Curator of the Ipswich Museum, and John Stump, an Ipswich Museum volunteer and local historian, provided valuable insights into museum collections, and John Stump also shared his history of the dam site. Finally, PAL consulted with John Fiske of the Ipswich Historical Commission regarding local preservation restrictions and bylaws pertaining to historic properties including the Town's Demolition Delay Bylaw (Chapter XVI of Town of Ipswich Bylaws) and their potential application to the proposed removal of the Ipswich Mills Dam.

Cultural Context

Pre-Contact Period (12,500–300 B.P.)

Essex County including the Ipswich River drainage in northeastern Massachusetts has long been recognized as a core area of pre-contact land use and settlement. The record of occupation has been well documented, first by avocational archaeologists and more recently by professional archaeologists undertaking cultural resource management (CRM) surveys that have filled in some of the gaps in the archaeological record over the past twenty years. The large number of recorded sites and broad range of represented temporal periods in Essex County reflects the favorable environmental conditions that existed throughout the pre-contact period. Essex County has one of the highest densities of known **PaleoIndian Period** (12,500–10,000 years before present [B.P.]) sites in southern New England. Extensive depositions have been recovered from the Bull Brook Site and Bull Brook II Site in Ipswich, the largest PaleoIndian deposition known in the region (Byers 1954, 1959; Grimes et al. 1984). Three hundred meters separate the two depositions that are located on a knoll, which separates the estuarine lower portions of the Egypt River and the Muddy Run. A saltwater marsh is located on three sides of this peninsula. Analysis of the artifact assemblages from both of these sites has revealed that the diverse tool classes are indistinguishable and that both are dominated by chert (Grimes et al. 1984). A fluted point was recovered on a farm east of Bull Brook on the opposite bank of Muddy Run (19-ES-103) and another was discovered on top of North Ridge, a drumlin that overlooks Plum Island Sound (19-ES-294), both located in Ipswich. The Ipswich Cove Site located on the Heard House property off South Main Street may also contain evidence of a PaleoIndian component in the form of two possible channel flakes (Mailhot 2013).

Unlike the PaleoIndian Period, the **Early Archaic Period (10,000–8000 B.P.)** is not well represented in Essex County either by specific sites or find spots. At Bull Brook only two diagnostic bifurcate-base projectile points from this period were recovered. A single bifurcate-base projectile point was discovered at the Pine Swamp Site (19-ES-306) in Ipswich and at Eastern Point in Gloucester (MHC site files). The distribution and somewhat higher density of **Middle Archaic Period (8000–5000 B.P.)** sites indicates that a multisite seasonal settlement system was firmly established by this time. More than 35 sites are known from this time period in Essex County including the Bull Brook Site, which has yielded diagnostic Middle Archaic Neville-like projectile points. Site 19-ES-103, located along the banks of Muddy Run Brook in Ipswich, contained diagnostic Middle Archaic Stark-like projectile points. Johnson and Mahlstedt (1982) assigned a collection of 15 stemmed points, described as a cross between Neville-like and Neville-variant, to the Middle Archaic Period on the North Shore. At many sites, these hybrid points are the sole evidence of Middle Archaic activity in the Ipswich River drainage.

Land use patterns in the Ipswich River drainage during the **Late Archaic Period (5000–3000 B.P.)** appear to reflect population increases and environmental changes similar to those observed across New England during this period. Sites are present in almost all environmental niches, and the utilization of a wide variety of plant and animal resources is suggested by small, special-purpose sites found along the edges of streams, bogs, and kettle hole swamps. Evidence of fishing and shellfish collecting is visible in the archaeological record for this period. Artifacts dating to this period have been recovered from the Bull Brook Site as well as in the area located between it and Bull Brook II in Ipswich (Grimes et al. 1984). Bullen excavated the shell midden on Treadwell's Island (Site 19-ES-98) off the coast of Ipswich where the earliest deposition appeared to date to the Late Archaic Period (Bullen 1949; MHC site files). The assemblage from this site includes shell (oyster, clam, quahog and mussel) as well as Brewerton and Small Stem Tradition projectile points. The Ipswich Cove Site (19-ES-853) on the Heard House property yielded a single Neville Variant projectile point suggestive of a Late Archaic component (Mailhot 2013).

The period of transition between the Archaic and Woodland periods is not well defined. Sites that contain artifacts diagnostic of the **Transitional Archaic Period (3600–2500 B.P.)** and particularly the **Early Woodland Period (3000–2000 B.P.)** are few. Only two sites in Essex County have yielded more than a single diagnostic point from the Early Woodland Period and these are found in Salem and Danvers (Johnson and Mahlstedt 1982). A Meadowood projectile point, an Early Woodland Period diagnostic type, was recovered from 19-ES-103 near Muddy Run Brook in Ipswich. A lack of information for the **Middle Woodland Period (2000–1000 B.P.)** continues from earlier periods in terms of both known sites and artifacts contained in local collections. Site 19-ES-318 contained a possible Middle Woodland Period occupation located on the Egypt River in Ipswich based on the recovery of aboriginal ceramics and flakes. Sites that have yielded diagnostic **Late Woodland Period (1000–450 B.P.)** Levanna projectile points include Treadwell's Island and Eagle Hill both in Ipswich. Thin-bodied, shell-tempered ceramics found at the Sewer Site (19-ES-475) are also indicative of the Late Woodland Period and are found on many multicomponent sites and collections, but not in high numbers. The Ipswich Cove Site (19-ES-853) yielded stone tool and pottery artifacts dating from the Transitional through Early Woodland periods (Mailhot 2013).

During the **ProtoHistoric and Contact Period (450–300 B.P.)** Ipswich was part of the native territory called Agawam, meaning "resort for fish of passage." Native settlements in this area may have shifted seasonally exploiting anadromous fish resources. Permanent settlements may have existed at or near the mouths of several coastal rivers. Ipswich was inhabited by members of the native Pawtucket group, which extended from the Saugus/Salem area north to the York, Maine area. This group is commonly referred to as the Agawams, most likely a sub-tribe of the Massachusetts under the leadership of the Penacooks (MHC 1985). Known sites from this time period are in coastal areas of Ipswich particularly near the mouths of the Ipswich and Castle Neck rivers and on Treadwell's Island (Site 19-ES-98). In the past, this area may also have been the mouth of the Merrimack River, which would further increase ProtoHistoric and Contact Period settlement possibilities (MHC 1985).

Post-Contact Period (1620–Present)

The first Europeans to obtain land rights in the Ipswich area were the owners of the Plymouth Company who established trading posts and fishing stations between the Charles and Merrimack rivers as early as 1620 during the **Plantation Period (1620–1675)**. In 1621 John Mason obtained land rights to the territory between the Namkeag and Merrimack rivers from the Plymouth Company. The first permanent colonial settlement was in 1633 when John Winthrop Jr. and 12 other men settled on the north side of the Ipswich River west of Jeffrey's Neck. In 1634 a second group of about 100 settlers arrived and the General Court incorporated the Agawam area as Ipswich. Ipswich was settled as a centralized village around a meetinghouse, burial lot and green (MHC 1985).

The combined use of agriculture and husbandry were important aspects in the economic development of the early settlement at Ipswich. Fishing also became an important economic enterprise during the early settlement of Ipswich. Ipswich was in a prime position for the exploitation of anadromous fish runs within larger rivers. Along with the North, Muddy, and Ipswich rivers, the town contained several principal streams including Winthrop's, Norton's, Howlet's, Mile and Bull Brooks, which were all used for fishing being well-stocked with pickerel and trout (Perley 1888). Good wharf areas were provided in the Castle Neck area and Plum Island provided a good breakwater for harborage in the Ipswich and Eagle Hill rivers. The components of the economic base of Ipswich settlers that began during the seventeenth century continued to grow and flourish during the **Colonial Period (1675–1775).** By 1700 most of the Ipswich workforce was engaged in various fishing activities. Numerous mills were also constructed on river drainages during this period.

Geography was perhaps the primary obstacle preventing Ipswich from developing into an important port town during the eighteenth century. The extensive coastal marsh necessitated building the town center far inland and the winding Ipswich River made it difficult for ships to reach the main settlement areas. Ipswich merchants did however own a small fleet of fishing and coasting trade vessels. Many of these vessels were built in Chebacco Parish, a part of Ipswich until Essex incorporated as a separate town in 1819. While the fishing and coastal trade continued to grow throughout the later Colonial Period and during the **Federal Period (1775-1830)**, the Embargo of 1807-1808 diminished foreign trade. By 1830 the Ipswich involvement in the West Indies trade had practically ceased and the distillery, lacking molasses, was forced to close (MHC 1985).

In addition to the coastal fisheries, Ipswich also had a valuable stream fisheries business that originated in the seventeenth century and continued throughout the nineteenth century. The stream fisheries focused on catches of sturgeon, bluefish, shad, and alewives (or herring), which were a source of revenue locally to the town and of some commercial importance in trade with the West Indies (Perley 1888). In 1674 in the river near the Ipswich Mills Dam, the town permitted Nathaniel Rust and Samuel Hunt to construct a stone fish weir at "the Falls" if it did not "hinder the mill nor passage thereto" (Felt 1834:108). The Upper Falls was a natural location in the river where "millions of herring, shad, salmon, and alewife swam upstream each year to their spawning grounds (Stump 2011). Town records indicate the weir had stone walls built down the stream that connected at a forty-five-degree angle, where a cage built of hoops with twigs fastened to them was placed. The walls directed the fish down to the cage where they were reportedly collected in great numbers (Felt 1834:108).

In 1803 a public fishery for alewives was established by the neighboring town of Topsfield on Pritchard's Pond, connected with the Ipswich River through Howlett's and Mile brooks. Thousands of barrels of alewives were collected on the Ipswich River above Choate Bridge and salted, and shipped to the West Indies (Belding 1920). The importance of the stream fisheries caused numerous petitions by residents to protect fish passage, including one petition dated May 1768 that stated "the Ipswich River has been reported from age to age one of the best fish streams, particularly for shad, bass, and alewives, in the county if not in the country" (Perley 1888:636). In 1788 the first law protecting the local alewife fishery was passed, followed by voluminous state legislation for the towns of Ipswich, Hamilton, Topsfield, Reading, Danvers, and Middleton, including laws passed in 1821, 1825, and 1829 that required fishways with definite construction and size specifications at factory dams (Belding 1920).

Agricultural production increased dramatically during the **Early Industrial Period (1830–1870)** despite the loss of almost 10,000 acres of farmland following the incorporation of Hamilton in 1792 and Essex in 1819. Large potato, vegetable, fruit, and grain crops as well as an increased number of people employed on farms contributed to the growing agricultural economy. This expansion helped the economy overcome the virtual collapse of the fishing and coastal trade. Tonnage of ships registered in Ipswich fell from 2331 in 1830 to 428 in 1855 (MHC 1985). The manufacturing sector also developed despite the collapse of the lace industry in 1833 as former lace manufacturers turned to the manufacture of hosiery. Large stone or brick hosiery mills, including the Ipswich Woolen Mills (1863), were established during this period. Other manufacturing operations included a shoe factory (ca. 1836), 14 small shoe shops, two tanneries, three coopers and eight cabinetmakers shops (MHC 1985).

The number of farms and acres under tillage declined after 1875 during the **Late Industrial Period (1870–1915)** as farmers turned increasingly to dairying. Dairy farms required increased pasturage and agricultural production focused on providing increased amounts of livestock fodder (primarily corn) rather than grain for human consumption. The enlargement of the Ipswich Hosiery Mills and the introduction of modern weaving machinery resulted in dramatic growth in manufacturing during the period. Other important industrial activity in the last quarter of the nineteenth century was the factory production of shoes. By 1885 five shoe factories were in operation; auxiliary industries included nine blacksmith and machine shops and two box-making factories. During this period, Plum Island beaches on Ipswich's coastline became an important attraction to summer tourists. A hotel and numerous summer cottages were built (MHC 1985).

Despite the courts' efforts to maintain fish passage at industrial dams including those on the Ipswich River, the importance of stream fisheries in the town steadily declined in the second half of the nineteenth century, and was basically defunct by the late 1880s. In 1875, for example, the capital employed in stream fisheries was $9,000 and the value of fish caught was $20,948, while in 1885 the capital had dropped to $2,200 and the value of fish caught was $2,784 (Perley 1888). By the turn-of-the-century, little to no alewives were observed in the Ipswich River, and by 1920 there was no alewife fishery reported in the town. In Ipswich, the demise of the stream fisheries has been attributed to the utilization of the spawning grounds for municipal water supplies, the obstruction of streams and rivers by dams without functioning fishways, trade-waste pollution, and the diminution of the quantity of water in the river and its tributaries (Belding 1920).

With the closing of the Ipswich Mill Company in 1929 Ipswich became primarily a residential community, which has continued throughout the **Modern Period (1915–present).** Single-family homes were built in large numbers within the central village and along the town's rural highways. Seasonal cottages were constructed on Jeffrey's Neck and Little Neck during this period. Agriculture continued to decline in the twentieth century with farming limited to southern Ipswich. A small industrial fringe emerged immediately north of the central village near Town Hill.

History of the Ipswich Mills and Dam Site

A dam has existed at the Ipswich Mills site (the site) since at least 1637, and possibly as early as 1635, making it the earliest water power privilege to be developed on the Ipswich River by English settlers. The first dam at the site was built by Richard Saltonstall, who obtained exclusive rights to the privilege, to power a grist mill. The dam was likely constructed of logs and stones and was located at a series of natural waterfalls on the Ipswich River known as Upper Falls that which were roughly 30 feet upstream (south) of the present dam (Haley & Aldrich 2009; Harris 2015; Stump 2011; Waters 1905:77).

Richard Saltonstall (1610–1694) was the son of Sir Richard Saltonstall (1586-1661), First Assistant to Governor Winthrop of the Massachusetts Bay Colony and a Patentee of Connecticut. Saltonstall graduated from Emmanuel College in Cambridge, England, in 1627 and accompanied his father to New England in 1630. He settled permanently in Ipswich in 1635 and was involved as a deputy at the court in the town. Over the course of the remainder of his life, he regularly travelled between Ipswich and England, where he died in 1694 (Saltonstall 1897: 86-87).

Saltonstall's grist mill remained the only one on the Ipswich River until 1687. The vicinity of the dam had become a nexus of early industrial activity by the end of the seventeenth century and was known as "Mill Garden" (Waters 1905:329) due to the presence of fulling mills, sawmills, woolen mills, bark mills, dye houses, tanneries, and other establishments at this and nearby mill privileges, including the Lower Falls. In 1729, the Saltonstall family divested themselves of their financial interest in the site, selling their interest in the mills and dam to John Waite, a clothier, and Samuel Dutch, a bricklayer. At this point, the dam powered a grist mill and fulling mill on the west bank of the Ipswich River, and a sawmill established by unknown persons on the east bank of the river. Waite and Dutch sold their interests in the property a few years later and it changed hands several times before being acquired by Michael and Nathaniel Farley by 1755. The fulling mill likely "went out of use as the hand weaving in the weavers' shops all about the Town gave place to factories" (Waters 1905: 329-330). By 1792, Asa Andrews was operating the sawmill as well as a scythe mill on the east side of the river (Harris 2015; Stone 1930:414; Stump 2011; Waters 1905:329). During Farley's involvement at the privilege, it was sometimes referred to as "Farley's Falls" (MGC 1832).

The industrial revolution ushered in the next significant phase of development at the Ipswich Mills Dam, shifting milling at the site away from small-scale production for the local market towards manufacturing. In the early nineteenth century, George Washington Heard (1793–1863)—a wealthy merchant in the China trade, his brother Augustine Heard (1785-1868), and their brother-in-law Joseph Farley became interested in establishing a lace industry in Ipswich. In 1822, the Heards and Farley convinced Benjamin Fewkes and George Warner to smuggle a lace machine (the first in the country) into Ipswich from England. Fewkes (1788–1869) and Warner were from Loughborough, England, and were skilled textile workers from mechanized hosiery knitting and lace weaving trades. In 1824, Joseph Farley and the Heard brothers opened the Boston & Ipswich Lace Company. The company produced lace until 1828, when it ceased operation, likely due to English trade interference and competition making the industry less profitable. English interference in the nascent American lace industry culminated in heavy English tariffs on thread exported to the United States and put most lace-manufacturing operations out of business by 1834. The three partners began a new venture by chartering the Ipswich Manufacturing Company to produce cotton cloth (Fewkes 1938:43-53; Hartmann 1996:13-15; Hurd 1888:638; Stone 1930:414).

The Ipswich Manufacturing Company was chartered with $200,000 in capital in 1828 and was the first sizeable manufacturing corporation in Ipswich. The company expanded its cotton cloth production for several years, reaching 450,000 yards of cloth annually. Preparations for this new enterprise had begun in 1827, when Joseph Farley replaced the dam with a higher, more substantial stone dam. During the construction of the new dam, Farley was given permission to "fill up the town [ford]way, as a watering-place" (Felt 1834:101), which was located just below the dam (Harris 2015; Waters 1917:636). The new mill for the Ipswich Manufacturing Company was constructed of stone between 1828 and 1829. The mill, identified as a 'cotton factory,' and also known as the "stone mill," is indicated at the west bank of the river on the 1832 (Anderson) map of Ipswich, with its dam adjacent thereto (Figure 1) (Anderson 1832; Felt 1834:101; Harris 2015; Hartmann 1996:15; Stone 1930:414).

In 1830, 12-inch flashboards were added to the dam to increase the size of the impoundment. From the 1830s throughout 1850s, a regular series of compensations were made by Augustine Heard for flood damage due to the dam. By 1836, the Ipswich Manufacturing Company was encountering financial difficulties (Hartmann 1996:15; Stump 2011). In 1845, the Massachusetts General Court (MGC) passed legislation mandating that the Ipswich Manufacturing Company "construct…a good and sufficient passage-way for the fish to pass over said dam up Ipswich River" (MGC 1845).

Along with the industrial development of the west bank of the Ipswich River, the earlier sawmill on the east bank of the river remained in operation, utilizing the waterpower of the dam. By the 1830s, the sawmill was under the operation of Benjamin Hoyt. In 1843, Hoyt signed a 10-year lease with the Ipswich Manufacturing Company that granted him the rights to build a new sawmill at the site of the old sawmill and to utilize waterpower at the site. Circa (ca) 1858, Hoyt's sawmill building was purchased and moved several blocks away to 17 County Street, where it still stands today (Waters 1905:637: Harris 2015; Stump 2011).

Due to the financial difficulties faced by the Ipswich Manufacturing Company, the stone mill was sold to the Dane Manufacturing Company in 1846, which continued to produce coarse cotton cloth known as ‘drilling’ at the factory. The Heard family remained involved, with George W. Heard, serving as president of the new company. The Ipswich Mills Dam was not included in this purchase, remaining under the ownership of Augustine Heard. The Dane Manufacturing Company manufactured cotton cloth at the stone mill until about 1868 (Figure 2) (Waters 1917). In 1858, Heard had raised the flashboards on the dam from 12 to 18 inches, resulting in additional property damage up river from the dam and commensurate compensation from Heard (Adams 1856:69; Harris 2015; Hartmann 1996:15; Harvard Business School 2011; Stump 2011; Walling 1856).

The Ipswich Mills site entered its next significant phase in 1868 when the property, presumably including the dam, was purchased by Amos A. Lawrence (1814–1886) for $70,000 and renamed The Ipswich Mills Company, which produced hosiery and at one point became the “largest stocking mill in the country” (Stone 1930:414). Hosiery became an important industry in Ipswich, with three companies employing 451 workers in the town by 1880. Lawrence was heavily involved in the textile industry and was from a prominent family for which Lawrence, Massachusetts, was named. The Ipswich Mills complex expanded in size after ownership changed to the well-capitalized Lawrence family. By 1872, several new structures were present on the site, including the hosiery mill. In approximately 1880, the Ipswich Mills Dam may have been reconstructed in place based on technique used to cut the stone, size of the stones, and two maps showing different shaped dams (Stump 2011). A footbridge was established atop the dam structure. By 1884, the Ipswich Mills property consisted of 9 buildings on the west bank of the Ipswich River, adjacent to the dam, known as the Ipswich Hosiery Mills. Between South Main Street and the east bank of the river below (north of) the dam, wood-frame buildings (unrelated to the Ipswich Mills) set on stone retaining walls and wood pilings lined the river. The area was known locally as “Little Venice” for its working-class shops, residences, and mill tenements (Beers 1872; Hartmann 1996:16; Sanborn 1887, 1892, 1897, and 1902; Stone 1930:414; Stump 2011; Walker 1884; Walling 1856).

Late-nineteenth century photographs and insurance plans show that the Ipswich Mills Dam was a stone masonry structure with a distinctive “dog-leg” footprint. The western two-thirds or three-quarters of the structure ran on a northwest–southeast footprint. The remaining eastern end of the structure ran almost due east–west (Associated Mutual Insurance Companies 1904; Sanborn 1887, 1892, and 1897). Several piers of indeterminate construction[1] extended laterally and above the spillway and supported a wood footbridge (Figures 3 and 4). A fishway to allow for passage of alewives had been installed at the east side of the dam in the 1880s (Belding 1920).

The year 1908 was a time of great expansion at Ipswich Mills, with considerable construction at the mill site including the demolition of the 1829 stone mill to make way for a new knitting mill. The complex continued to grow as the company entered a period of peak prosperity. The knitting mill (no longer extant), was located immediately adjacent to the dam on the west bank of the Ipswich River. Ipswich Mills also owned two small wood buildings, possibly worker housing, adjacent to and upstream of the dam on the east bank of the river. Ipswich Mills reached its peak in prosperity and productivity during WWI, with strong demand from European armies and rising domestic demand, and was the reportedly the largest hosiery manufacturer in the world from 1916 until 1919 (Harris 2015; Hartmann 1996:13-17, 21; Sanborn 1907, 1916; Stump 2011; Walker 1910).

According to John Stump’s (2011) article on the history of the Ipswich Mills Dam, the dam appears to have been rebuilt in 1908, presumably to increase or improve the reliability of the available

[1] The eastern-most piers are stone, the others may be wood or concrete (see Figures 3 and 4).

waterpower. A comparison of insurance plans predating (Sanborn 1892, 1897, 1902, and 1907) and postdating 1908 (Sanborn 1916, 1929, and 1944) supports the dam rebuild around that date. The plans indicate that the dam footprint was realigned at its southeast end, eliminating the "dog leg" that had previously existed (Figures 5 and 6). The dam's location also appears to have been shifted slightly to the north (downstream), which would also explain the elimination of the "dog leg" on the east side. To further support a 1908 rebuild of the dam, an 1896 photograph shows masonry buttresses on the downstream side that are not present in later photographs and today (see Figure 3).

A description of the dam included in the May 21, 1912 inspection report indicates the structure was "rebuilt in 1908." The report further notes that the dam was in good condition and constructed of "coarsed [sic] stone masonry with timber flash-boards", and that recent repairs consisted of putting in "one new gate and pointing a few joints." The 1925 dam inspection report indicates that the dam was of "cut granite on the face" and the owner had a "plan of it on file in their office drawn by Charles T. Hain when the structure was rebuilt about 1908"—the plan shows rock foundation beneath the dam (ACOE 1980). Low water conditions during August 2016 exposed what appears to be the stone remains of an earlier dam at the location depicted on the insurance maps pre-dating 1908. The 1916 insurance map confirms that the dam at that time had a foot bridge over it (see Figure 6), likely the steel truss structure that appears in a mid-twentieth-century photograph.

The 1880s fishway at the dam was reportedly entirely destroyed in 1916, and a new fishway was installed at the same location (east side of the dam) in 1919 in an effort by the state's Division of Fisheries and Game to re-establish the fishery (Belding 1920). The 1912 dam inspection report notes that the dam included a "fish run on the east side" and the 1925 inspection report mentions that the "fish-way has recently been built", presumably the replacement one installed in 1919.

After WWI, Ipswich Mills experienced a rapid decline, with a 50 percent slowdown in production in late 1920 due to consumer demand for higher grade hosiery than the dated circular cotton stockings that were being produced. Ipswich Mills also faced increased competition, evidenced by the organization of the Hayward Hosiery Company in Ipswich in 1922. The Ipswich Mills Company ceased operation in 1928 and the machinery was sold to mills in Moscow, Russia (Hartmann 1996:21; Sanborn 1929; Stone 1930:414; Stump 2011). The 1930 dam inspection report indicates that the mills were closed and the machinery had been sold, and "from all appearances there have been no changes since the last inspection [in 1928], and the dam seems to be in good condition" (ACOE 1980).

The Ipswich Mills complex sat empty until Ernest Currier purchased it for unknown uses in 1932 for $13,000. The 1932 dam inspection report indicates that the "dam is in good condition and there is no change", while the 1934 inspection report notes that the dam "belongs to E.B. Currier, Real Estate Agent" and "is in good condition and there have been no changes. The water power is used occasionally. There is a watchman on duty all the time. Water is flowing over the dam today." The 1936 dam inspection report indicates that the dam was now owned by the Tanning Process Company, a subsidiary of United Shoe Machinery Company (USMC). The tanning company used the dam for power when there was sufficient water, and there was "some leakage around the old gates." The 1936 report also notes that the owner intended to repair the "old gates and stop all leaks" within the following year, and that he would "probably build a dike up river to hold out the water while making repairs." It is not clear if these repairs were made, since the 1938 dam inspection report still notes that "some of the timber gates need repairing." The 1938 report also makes note of a "concrete wall at the westerly end of the dam." In October 1940, the dam inspection report indicates that "new gates and timber work have just been built at the westerly end of the dam, where the wheels are. Last year new draw-off gates were built at the easterly end of the dam…and the dam is in good condition" (ACOE 1980).

According to dam inspection reports, from 1940 to 1948 Hygrade Sylvania Corporation (Sylvania) leased part of the "old mills" from the USMC, to produce products including "proximity fuses, military and commercial transformers as well as tungsten coils" (ACOE 1980; Stump 2011). During WWII, Sylvania participated in then-secret war work for the Navy, employing 1,200 workers to build proximity fuses that aided in the Allied victory. The dam was maintained by the USMC and used by Sylvania to supply water power for its machinery. The 1942 dam inspection report notes that there was a "slight seepage at the easterly end of the dam at the fishway" but that it was otherwise in good condition with "water flowing over the flashboards which have been renewed since the last [October 1940] inspection." Two years later, the 1944 dam inspection report indicates that the only repair made during that period was the replanking of the floor of the footbridge over the spillway. No leaks were observed, but the "hoisting machinery of one of the gates needs to be repaired" (ACOE 1980).

In 1946 the dam inspection report indicates that "new timbers have been placed under east gates. Center pier of bridge has been braced with timbers" and the condition of the dam was the same as 1944 report. By 1948 Sylvania Electric Products, Inc. had taken over ownership of the mill property including the dam, and the inspection conducted in October noted "leaks under spillway at westerly end" and disintegration in the concrete in the fishway. In 1950 the dam inspection report notes that the leaks at the west end of the spillway were still "very bad" although some work had been done on the gates. The concrete sidewall of the fishway was also still "badly disintegrated." The 1952 dam inspection report notes that the owner attempted to stop the leakage at the west end of the dam by backfilling the back of the dam with gravel, but the leakage continued. In 1954 no repairs had been made to the dam, although the inspection report notes that the owner was cooperating to keep it a "safe structure." The dam inspection reports from 1956, 1958, and 1959 indicate the same dam conditions, and that the owner continued to cooperate by keeping the dam "under constant observation to keep the structure safe." In 1961 all gates were reported to be closed except for the fishway, which was kept open, and in 1962, the fishway was "kept about half way open."

The 1962 dam inspection report indicates continued leakage, and that the owners had arranged for a contractor to "gunite the face of the dam and the fishway, and backfill with impervious material the back of the dam near the mill" during the summer months at low water (ACOE 1980). There are no other details about these repairs in the available dam inspection reports, although there are recollections that the 1960s work may have involved an unconfirmed rebuild of the entire dam (Dick Dunn quoted in Stump 2011). A vertical slab of concrete lining the upstream face of the dam and exposed during drawdown in the summer of 2016 may date to this period of dam repairs and/or rebuild. Dam inspection reports dated 1964, 1966, and 1968 indicate the dam conditions were good, with no mention of repairs or rebuilding (ACOE 1980).

In 1971 the dam inspection report does not mention any particular issues with the dam. The next inspection, conducted on September 18, 1973, notes that the owner, GTE Sylvania Inc., wanted to close all spillway and sluiceway openings. Also at that time, two of the gates on the east end of the dam appeared operable, but all others were inoperable. The 1973 dam inspection report also indicates that the owner was in the process of removing the steel truss foot bridge supported by granite piers over the dam spillway (ACOE 1980). Later that month, on September 28, 1973, GTE Sylvania Inc. filed construction drawings and specifications with the Commonwealth of Massachusetts, Department of Public Works, Division of Waters to undertake a series of alternations to the dam structure, consisting of removing the slide gates on the east and west sides of the dam and closing the openings; and closing the opening in the building foundation walls, where water flows from the west slide gates. By filling in the openings of the inoperable gates, the owners hoped to eliminate leakages and create a continuous flow of water over the spillway crest, which would enhance the appearance of the dam. The state application to alter the dam also reported that the dam was constructed entirely of granite blocks with a 185-ft long spillway that extended the full width of the river channel. There is no mention of concrete as a material used in the dam and spillway structures. No changes were proposed for the fish ladder at the east side of the dam. The dam alterations were approved by the Commonwealth of Massachusetts Department of Public Works, Division of Waterways on October 17, 1973. GTE Sylvania Inc. commenced the alterations at Ipswich Mills Dam on/about September 23, 1974 and the work was completed by December 27, 1974, except for an alternate concrete wall planned at the east slide gates that was not implemented due to the high quality of foundation material found at this location (ACOE 1980). Stephen Stickney, current owner of the Stephen A. Stickney Co. of Boxford, Massachusetts, confirmed that his company did the work in the 1970s, and that they would have used concrete masonry blocks to fill in the gate opening, since they have never done concrete formwork (Stephen Stickney, personal communication 2016). In addition to the dam alternations in 1973-1974, GTE Sylvania Inc. demolished approximately 50 percent of the Ipswich Mills complex, including the machine shop and the knitting mill adjacent to the dam.

The Town of Ipswich purchased the dam from GTE Sylvania in 1982. The February 4, 1993 dam inspection report indicates a 1900 date of original construction with modifications in 1908, and the approximately 180 ft-long, and 9 ft-high structure being made of granite blocks. The report notes that there were four low level outlets and one mid-level outlet at the dam's right abutment, but no gates, the openings having been "blocked off with masonry products", which corresponds to the personal account of the work undertaken by Stephen A. Stickney Co. in the 1970s. The 1993 report also mentions the presence of a 70–80-ft long concrete fish ladder (constructed over a cut stone foundation) at the right abutment of the dam. Inflow to the ladder was via an opening through the granite outlet pier where the ladder joins the dam, but no gate to control flows through the fish ladder existed. The report also notes that the fish ladder was likely the one noted in dam safety files as having been built in 1925, and that the history of the structure's functionality was unknown, but that it "had not functioned properly for many years." No original construction drawings for the dam are on file at the Office of Dam Safety (DEM Office of Dam Safety 1993). Osram Sylvania sold the remaining Ipswich Mills buildings to EBSCO Publishing in 1995, who rehabilitated the mills in 1996 and continue to utilize them currently (Harris 2015; Hartmann 1996:13; Newton et al. 2001:22; Sanborn 1944; Stump 2011; Varrell 2006:76-77).

<u>Timeline of Ipswich Mill Dam History</u>

1635 or 1637	Richard Saltonstall builds first dam on Upper Falls, roughly 30 feet upstream of the current dam, for a gristmill. The dam is probably log and stone.
1729	Saltonstall family sells dam. The structure passes through multiple owners before acquisition by Michael and Nathaniel Farley by 1755.
Ca. 1824	George Washington Heard (1793–1863), Augustine Heard (1785-1868), and Joseph Farley establish Boston & Ipswich Lace Company factory at the site, possibly purchasing the dam structure.

1827–1828	Heards and Farley replace the dam with a new stone dam and organize the Ipswich Manufacturing Company for cotton textile manufacturing.
1830	12-inch flashboards added to the dam to increase the size of the impoundment.
By 1858	Heard raised the flashboards on the dam from 12 to 18 inches. The property is leased to Dane Manufacturing Company.
1868	Amos A. Lawrence (1814–1886) and the Ipswich Mills Company purchase dam.
Ca. 1880	Ipswich Mills Company reconstructs the dam (at the same location), and adds a fishway on or around this date.
1908	Ipswich Mills Company appears to rebuild the dam as part of plant expansion. The new dam is slightly downstream of the older dam, and the jog at the east end is eliminated.
1912	Dam repairs include installing one new gate and pointing a few joints.
1916	Flood destroys the ca. 1880 fishway.
1919	New concrete fishway installed at the east end of the dam (same location as before).
1928-30	Ipswich Mills closed, machinery sold, and dam in good condition.
1936	Dam owned by the Tanning Process Company, a subsidiary of United Shoe Machinery Company, which proposes repairs to “old gates” to stop leaks.
1939–1940	United Shoe Machinery Company installs new draw-off gates at east end of dam, and new gates and timber work at west end of dam.
1940-1948	Hygrade Sylvania Corporation (Sylvania) leases the mills from United Shoe Machinery Company.
1952	Sylvania attempts to stop leakage at the west end of the dam by backfilling the back of the dam with gravel, but the leakage continued.
1961	Sylvania has closed all the gates in the dam except for the fishway gate.
1962	Sylvania tells Dam Inspector that a contract has been let to “gunite the face of the dam and the fishway, and backfill with impervious material the back of the dam near the mill” during the summer months at low water.
Mid-1960s	Unconfirmed recollection from former Sylvania employee Dick Dunn that the entire dam was rebuilt. A vertical slab of concrete lining the upstream face of the dam was noted during drawdown in the summer of 2016 and may date to this period of dam repairs and/or rebuild.

1973 GTE Sylvania Inc. removes the steel truss foot bridge supported by granite piers over the dam spillway.

1974 GTE Sylvania Inc. undertakes alterations at the dam to close the inoperable gates at the east and west sides of the spillway. Concrete masonry blocks were used to fill in the gate openings. Work completed by Stephen A. Stickney Co. of Boxford, Massachusetts.

1982 The Town of Ipswich purchases the dam from GTE Sylvania Inc.

Known and Expected Cultural Resources

Archaeological Resources

There are over 60 recorded pre-contact sites along the Ipswich River and at its outlet in Ipswich Harbor. While most of these sites have been recorded on the basis of avocational collections many of which are housed at the Peabody Essex Museum in Salem, their presence indicates a long history of human occupation focused on the river's estuarine resources including herring and other species by both Native American and early English settlers. The fish were used for consumption and commercial sale by the English settlers. A review of the state's inventory of archaeological records indicates two pre-contact Native American sites on the east side of the river within approximately 600 feet of the Ipswich Mills Dam between the river and County Street. No information other than location is recorded for the unnamed site MHC #19-ES-101, although artifacts recovered from this area are reportedly on file at the Peabody Essex Museum in Salem. The other site, MHC #19-ES-853, is known as the Ipswich Cove Archaeological Site, located near the Heard House on South Main Street. Both avocational and professional archaeologists have investigated the site, resulting in the recovery of over 300 pre-contact artifacts consisting of chipped and groundstone tools, pottery, and a possible lithic workshop. The site is multi-component, featuring artifacts from the Late Archaic, Transitional Archaic, Early Woodland, and possible PaleoIndian Period (Mailhot 2013; Mailhot and Donohue 2013).

In addition, there are six recorded post-contact archaeological sites in the same geographic area between the river and County Road within 600 feet of the Ipswich Mills Dam. All the sites are related to residential (homestead) occupations dating from the seventeenth through twentieth centuries and associated with the earliest town settlement in the South Green Historic District (MHC #IPS.J) described in detail below for the historic resources. They were identified through archival research as part of a town-wide historical survey conducted by Boston University in the late 1970s (Starbuck et al. 1979). One of these sites, the Rachel Haffield Homestead Site (MHC #IPS-HA-52), is situated on the Samuel Dutch Homestead Property (MHC #IPS.26) that borders the east side of the dam between the river and South Main Street. In 1655 the town gave Widow Rachel Haffield a small lot near the mill dam on the Ipswich River on which she erected a dwelling. She later was one of only a few individuals in Ipswich to be brought to trial for witchcraft in 1692 and was acquitted the following year. The extant house on the lot was built ca. 1723 by Samuel Dutch, a mariner, who purchased the Haffield house lot and 2/3 interest in Nathaniel Saltonstall's sawmill standing on the south (east) side of the river and 2/3 interest in the dam. He sold his homestead along with mill and dam interests to John Treadwell, an innkeeper, in 1742, and it continued to change ownership through the nineteenth century. The remains of the seventeenth century dwelling may have been destroyed or incorporated into the Dutch House when it was built in 1723. The property is considered to have the potential to contain material cultural and structure remains dating to the seventeenth and eighteenth centuries (Savulis 1979 in MHC site files).

A review of town histories and historical maps indicates that a stone fish weir was established near the dam in 1674 by English settlers, and a sawmill was located on the east side of the dam between the extant houses at 41 and 45 South Main Street (MHC #IPS.26 and #IPS.31). The sawmill appears on both the 1795 and 1832 maps of Ipswich. As noted above in the dam site's history, the sawmill was in existence by 1729 when Nathaniel Saltonstall sold his 2/3 interest in the mill and the dam to John Waite and Samuel Dutch. The mill was near a ford way or footbridge for crossing the river in the early 1600s, but the ford way fell into disuse after the County Street bridge was built in 1647 when South Main Street from the dam north to the junction with Market Street was likely opened (Waters 1905). The sawmill remained in operation for over 100 years when it was sold in 1836 along with the adjoining water way land to the Ipswich Manufacturing Co. that operated on the west side of the dam. A scythe mill may have operated in conjunction with the sawmill on the east side of the dam for a short period of time when it was owned by Asa Andrews from 1794 to 1813 (Stump 2011). In 1846 the water way from the bend in South Main Street west to the dam was closed by permission of the Town and County with provision for a public right-of-way and access by the neighboring landowners. The original sawmill was reportedly taken down and a new mill for veneer sawing was erected ca. 1843 by Benjamin Hoyt on the same site. The new mill at the dam site operated under the name "Hoyt's Veneer Mill" until it was moved by James Wellington ca. 1858 to 17 County Street where it operated as "Perkins & Daniels Stocking Factory" in the upper story and by Wellington as a dwelling in the lower story (Waters 1905; Harris 2016).

On the west side of the dam the documented seventeenth and early eighteenth century mills including a gristmill, fulling mill, and hemp mill opposite the river from the sawmill were all supplanted by the lace factory (Ipswich Manufacturing Co., later Ipswich Hosiery Mills) in the 1820s including the construction of a new stone dam (discussed above in the dam site's history). The National Register nomination form for the Ipswich Mills Historic District ((MHC #IPS.I) indicates a high potential for pre-contact Native American and post-contact mill-related resources including legacy dams that may be deeply buried in fill deposits and river sediments on both sides of the current dam and north/west river shoreline (Hartman and Friedberg 1996).

Historic Resources

The Ipswich Mills Dam is not currently an historic property—it has not been listed in nor determined eligible for listing in the National Register of Historic Places (the National Register), and it is not included in the MHC's Inventory. The Ipswich Mills Dam is immediately adjacent to, but not included within the bounds of two historic properties listed in the National Register and the State Register: the Ipswich Mills Historic District (MHC #IPS.I), listed July 9, 1996; and the South Green District (MHC #IPS.J), listed September 17, 1980 (Hartman and Friedberg 1996; Welden 1978).

The Ipswich Mills Historic District is a well-preserved example of a hosiery manufacturing complex and related worker housing set within approximately 20 acres of land bounded by Union and Saltonstall streets on the north, following Estes and Kimball streets on the west, and bounded by the Ipswich River on the south and east. The historic district is significant for its associations with the Ipswich Mills and the collection of mill and residential buildings that make up the district provide an important visual narrative of the key industry in Ipswich's manufacturing economy, as well as illustrating the broad patterns of New England industrial development and the role of immigrant groups therein. The buildings of the Ipswich Mills Hosiery Manufacturing Company (MHC #IPS.356), which contribute to the significance of the historic district, are located at the west end of the Ipswich Mills Dam. The dam, although it is historically associated with the operations of the Ipswich Mills, is excluded from the historic district boundaries. The period of significance for the property extends from 1850 until 1946 (Hartman and Friedberg 1996).

The South Green Historic District is a collection of seventeenth, eighteenth, and nineteenth century homes, a Unitarian Church, and the South Green—a public common. The district encompasses approximately 17 acres bounded by Elm Street to the north, following Elm Street on the east, bounded by Saltonstall Creek on the south, and bounded by the Ipswich River on the west. South Green was established as common land in 1636 developed as a residential, religious, and educational center for the Ipswich community from that date until ca. 1900. The property is significant because of its associations with the social development of Ipswich and for its collection of distinguished residential architecture. Ipswich Mills Dam abuts the northwest corner of the historic district, and does not appear to have any substantial associations with the development of that property. The period of significance for the South Green Historic District is not defined in the National Register documentation, but likely extends from 1636 until ca. 1900 (Welden 1978). Two contributing resources to the South Green District—the Samuel Dutch House (MHC #IPS.26) and the Dr. Philomen Dean House (MHC #IPS.31)—are located immediately adjacent to the dam.

Management Recommendations

If the Project progresses into design and permitting, it may be subject to review under federal, state, and local legislation that provide protections for significant historical and archaeological properties.

The National Historic Preservation Act

Federal funding and/or permitting of the Project will trigger review under Section 106 of the National Historic Preservation Act (NHPA), as amended, and its implementing regulations (36 CFR 800). Section 106 requires that Federal agencies having direct or indirect jurisdiction over a proposed Federal or federally assisted undertaking shall "take into account the effect of the undertaking on any district, site, building, structure, or object that is included in or eligible for inclusion in the National Register [collectively termed historic properties]." As outlined in 36 CFR 800—Protection of Historic Properties, the process to meet this requirement (collectively termed the Section 106 Process) is consultative; involving the federal agency official, the Advisory Council for Historic Preservation (ACHP), the State Historic Preservation Officer (SHPO—in Massachusetts, the MHC), Tribal Historic Preservation Officers, local governments (typically in the form of a local Historical Commission such as the Ipswich Historical Commission), and other interested organizations and individuals (collectively, the consulting parties). There are four steps in the Section 106 Process: 1) initiate the process; 2) identify historic properties; 3) assess adverse effects; and 4) resolve adverse effects.

As detailed above, the Ipswich Mills Dam is not currently an historic property, nor does it contribute to the significance of any historic properties. If the Project proceeds within the Section 106 Process, then the historic property identification (step 2) as it relates to historic resources would likely consist of a survey of the Ipswich Mills Dam to determine if the structure is an historic property. The scope of such a survey effort would be subject to the recommendations of the SHPO/MHC. If the Ipswich

Mills Dam is determined to be an historic property, then any substantial alteration or wholesale removal of the structure that is unavoidable would likely result in a finding of adverse effect (step 3) that would need to be resolved.

Also, as described above, the dam is not recorded as an archaeological site, but there are several pre-contact and post-contact archaeological sites inventoried on the east side of the river within 600 feet of the dam and river shoreline. The river channel and the west and east shorelines and adjacent areas have been previously identified as possessing generally high sensitivity for known and previously undocumented archaeological resources including pre-contact Native American habitation sites and post-contact seventeenth through early twentieth-century residential and mill-related sites including legacy dams in the river channel sediments. If the Project proceeds within the Section 106 Process, similar to historic properties, there would most likely need to be an archaeological survey identification and evaluation effort (step 2), subject to review and recommendations by the SHPO/MHC. Any archaeological survey would need to be conducted under a State Archaeologist's permit issued by the MHC to ensure that the technical team has the appropriate qualifications and expertise, and that the scope of work (proposed archaeological survey research design, field methodology, and reporting standards) meets the regulatory and legislative needs of the Project. If any significant archaeological sites are identified, and avoidance of Project impacts is not deemed feasible, then similar to historic resources, there would be a finding of adverse effect (step 3) that would need to be resolved.

The resolution of adverse effects (step 4) for dam removals including both above- and belowground cultural resources is typically accomplished by minimizing or mitigating the adverse effects. The resolution measures are typically formalized through a Memorandum of Agreement (MOA) developed by and executed among the consulting parties. Minimization of the effects is accomplished through a modification of the Project design (where technically feasible) in accordance with the Secretary of the Interior's *Standards for the Treatment of Historic Properties* (36 CFR 68). Mitigation on dam removal projects in Massachusetts is usually accomplished by producing a permanent record of the historic property through archival photographic and written documentation for aboveground resources, and through data recovery and/or construction monitoring for archaeological resources. Elements of a historic property may also be left in place, salvaged and reused in a sensitive fashion, or donated to a museum. Finally, a public interpretive component such as a wayside panel, internet site, or brochure is often included in dam removal mitigation work.

Massachusetts General Laws

If state funding and/or permitting is utilized and/or permitting through the Massachusetts Environmental Policy Act (MEPA) is needed, the Project will be subject to review under Massachusetts General Laws (MGL), Chapter 9, sections 26-27C (950 CMR 70/71) and MEPA (301 CMR.11). Both 301 CMR.11 and 950 CMR 70/71 provide for the protection of historic properties listed in the National Register or State Register of Historic Places, accomplished through a consultative review process similar to the Section 106 Process. Where both Section 106 and 950 CMR 70/71/301 CMR.11 are applicable, the two review processes are typically coordinated at the same time to facilitate the agency consultations.

General By-laws of the Town of Ipswich

General By-laws of the Town of Ipswich include two provisions protecting historic properties (Town of Ipswich 2016). At present, neither provision would appear to apply to the removal of the Ipswich Mills Dam, although consultation with the Ipswich Historical Commission will be required to determine this with certainty (John Fiske, Ipswich Historical Commission; personal communication with John Daly, PAL; April 28, 2016).

Chapter XVI—Procedure for Delaying the Demolition of Historically or Architecturally Significant Buildings allows the Ipswich Historical Commission to prohibit demolition of significant buildings over 75 years of age for a 1-year period. For purposes of this Chapter, buildings are defined as "any combination of materials, whether portable or fixed, having a roof, the purpose of which is the shelter of persons, animals, property, or processes." (Town of Ipswich 2015:9). The Ipswich Mills Dam would not appear to meet the definition of a "building" (Town of Ipswich 2016:110).

Chapter XXII—Architectural Preservation District establishes an Architectural Preservation District (APD) within which an Architectural Preservation District Commission (APDC) exercises review of proposed alterations to buildings and their settings. The Ipswich Mills Dam (as well as the Ipswich Mills Historic District) is not within the boundaries of the APD and the Project is therefore not subject to APDC review (Town of Ipswich 2014, 2016:143).

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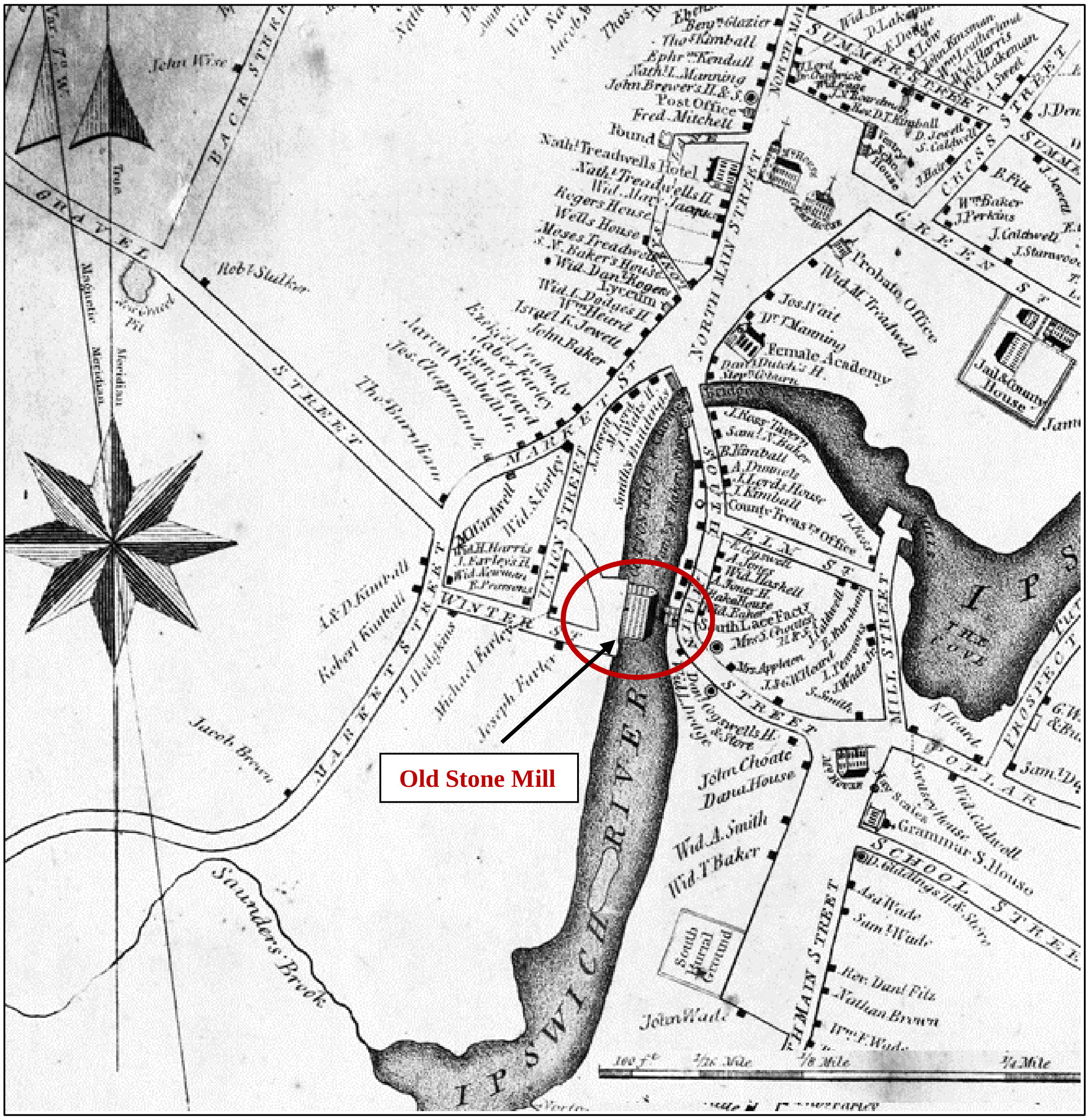

Figure 1. 1832 *Plan of Ipswich Village*, showing location of the Old Stone Mill (source: Anderson 1832).

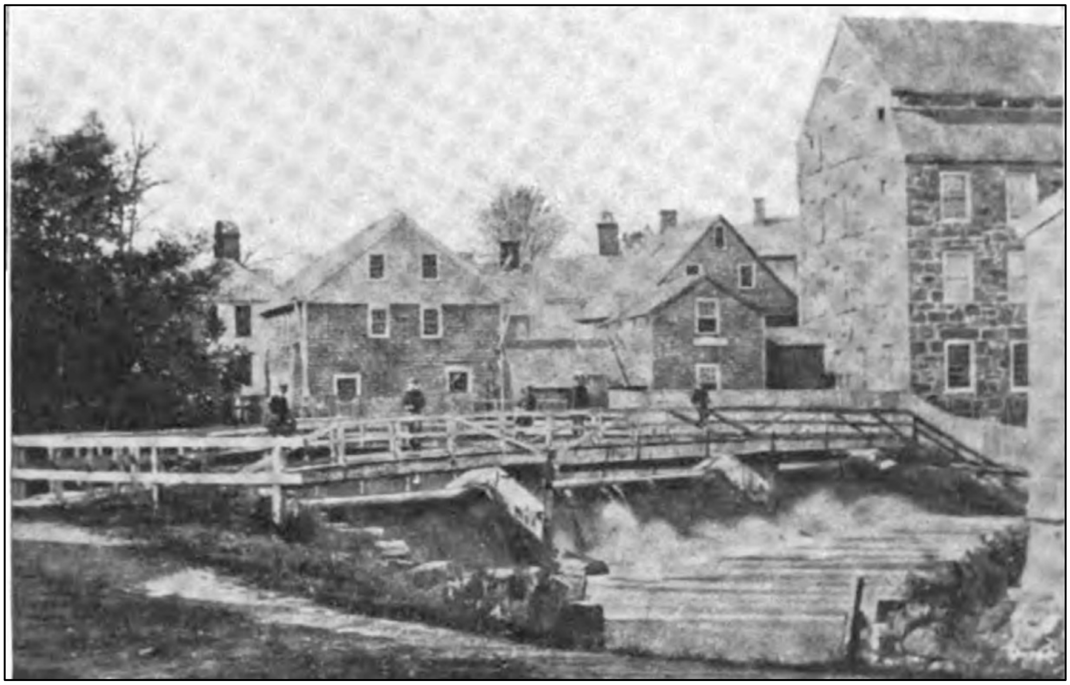

Figure 2. Ca. 1867 photograph of Ipswich Mills Dam, showing "foot-bridge" and "water-way", view looking southwest (stone mill at right) (source: Waters 1917:678).

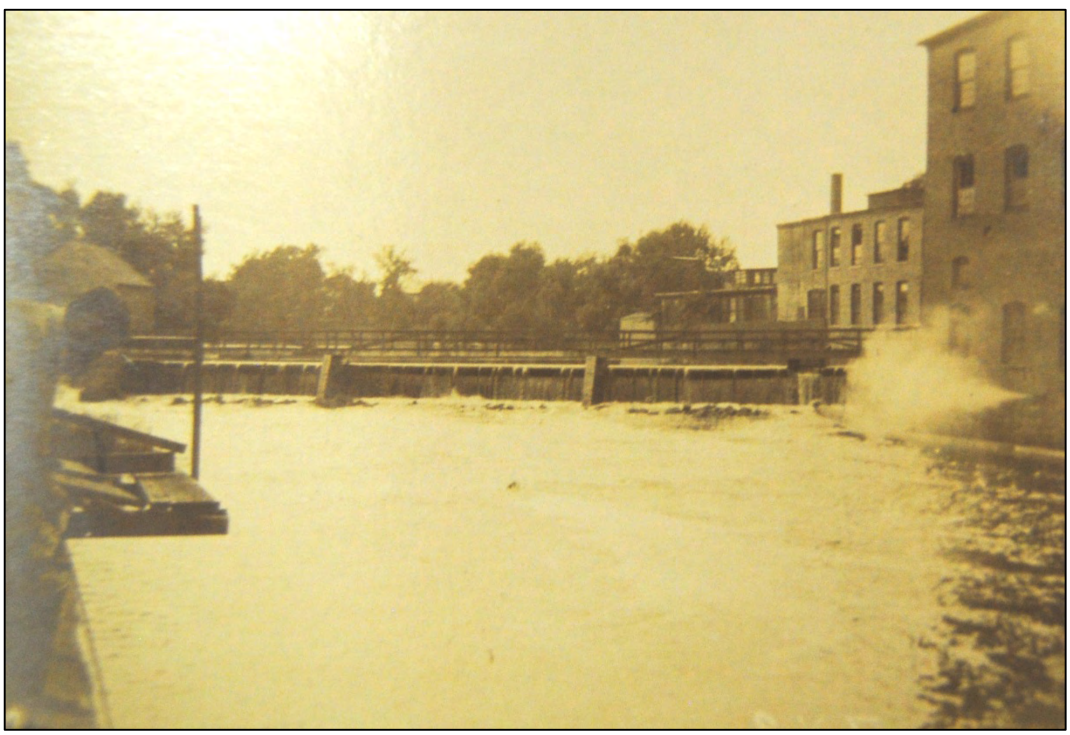

Figure 3. Ca. 1896 photograph of the Ipswich Mills Dam, looking south (upstream) towards dam (source: Courtesy Ipswich Public Library, Ipswich, MA).

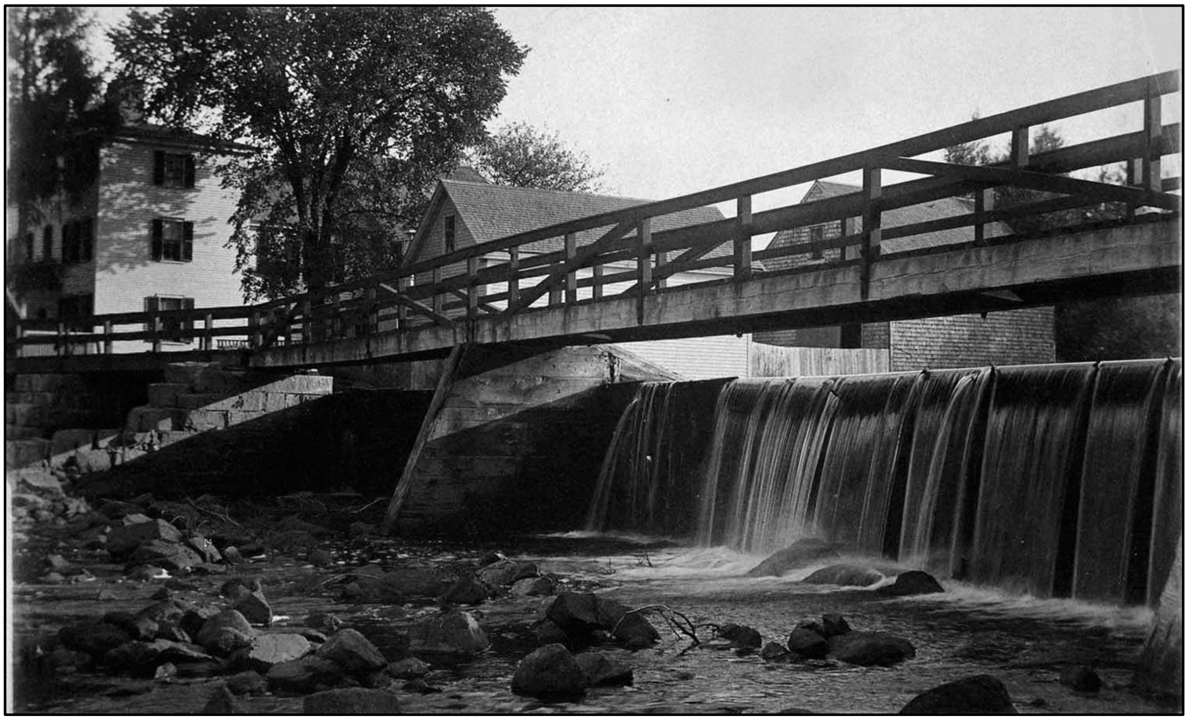

Figure 4. Ca. 1900 photograph by Arthur Wesley Dow showing the northeast end of the Ipswich Mills Dam (source: electronic document: StoriesFromIpswich.org).

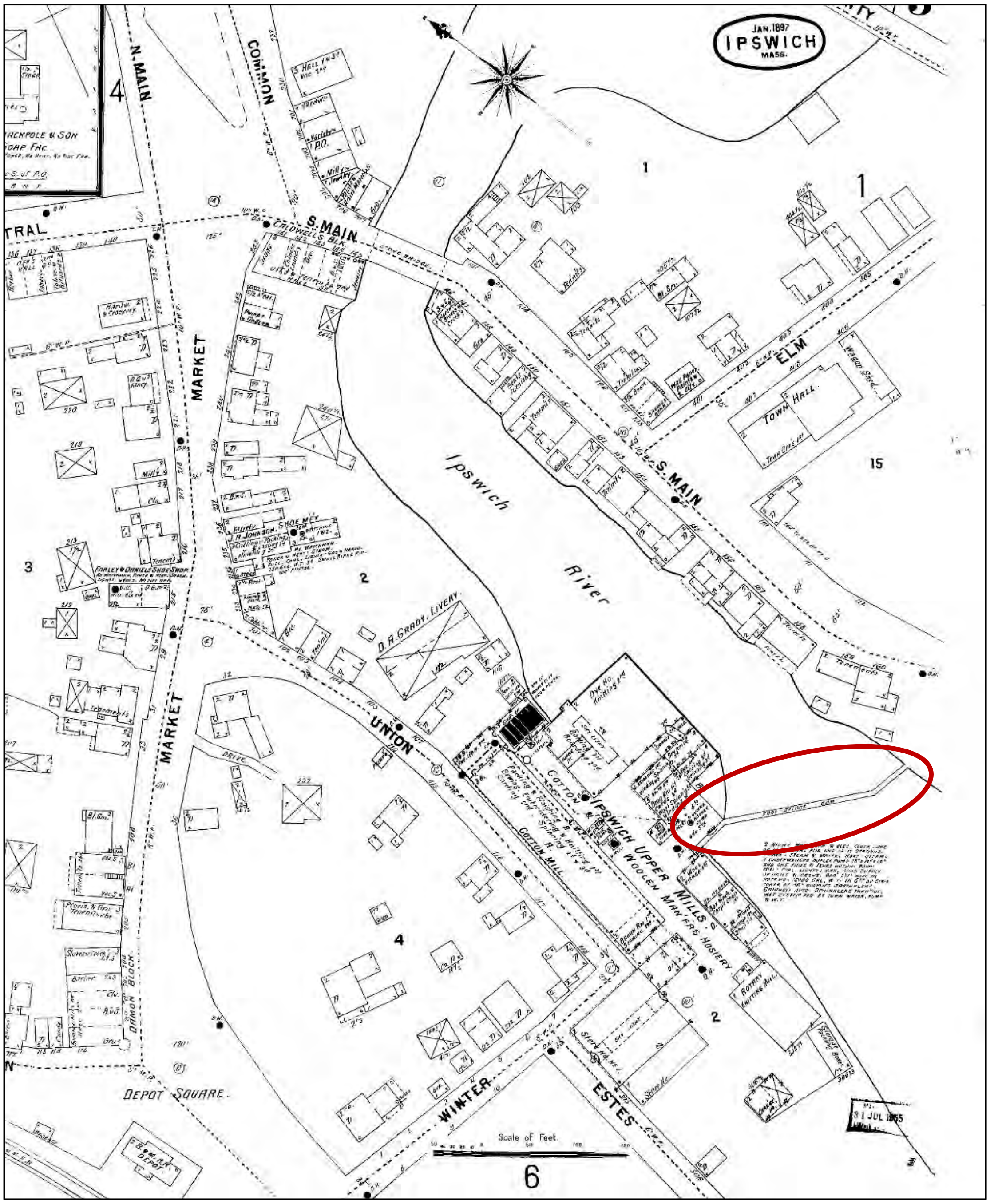

Figure 5. 1897 insurance map showing the Ipswich Mills and Dam at lower right (source: Sanborn 1897).

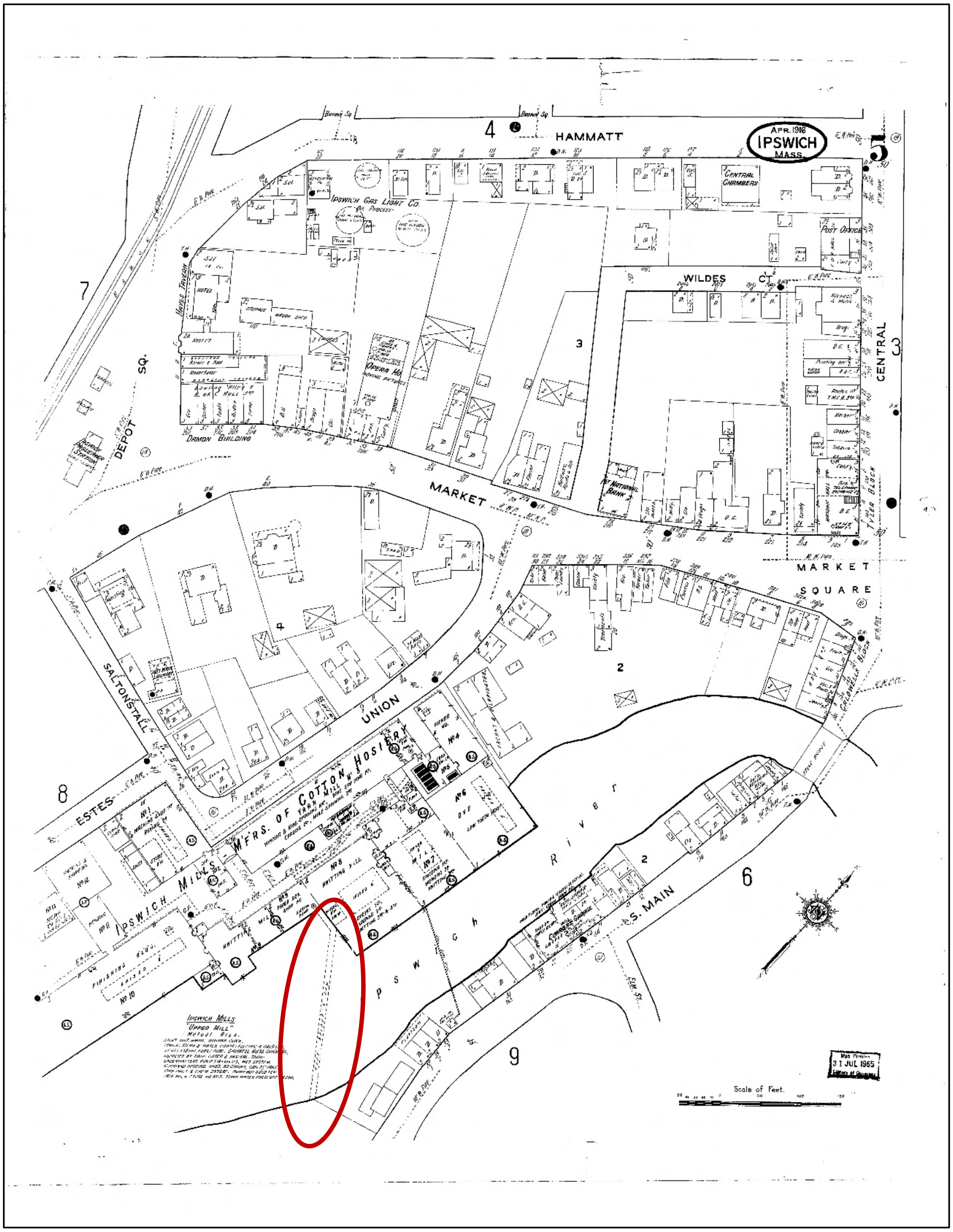

Figure 6. 1916 insurance map showing the Ipswich Mills and Dam at lower left (source: Sanborn 1916).

ATTACHMENT 5

TECHNICAL MEMORANDUM

To: Neal Price, Horsley Witten Group

From: Candice Constantine and Kristen Coveleski, Inter-Fluve

Date: November 13, 2018

Re: Technical Memorandum on Task 2: Hydrology and Hydraulics Summary

Introduction

The Ipswich Mills Dam is a "run-of-the-river"[1] dam located on the Ipswich River in the Town of Ipswich, Massachusetts (Figure 1). It is currently owned and operated by the Town of Ipswich Utilities Department and is directly adjacent to the EBSCO Publishing Company. The dam once provided surrounding mills with a reliable source of power but today no longer serves its industrial purpose and is being considered for removal.

The dam receives flow from a contributing watershed with an area of 148 square miles. The Merrimac-Hinckley-Urban land and Paxton-Montauk-Urban land associations are the primary soil associations in the watershed. The former includes areas of urban land and deep, nearly level to steep, somewhat excessively or excessively drained, loamy and sandy soils that were formed in outwash deposits. The latter incorporates urban areas and deep, nearly level to steep, well drained, loamy soils formed in glacial till. The Canton-Woodbridge-Freetown soil association also exists in the upper parts of the watershed but to a lesser extent. This grouping includes deep, nearly level to steep, well drained or moderately well drained, loamy soils formed in glacial till and deep, nearly level, very poorly drained, mucky soils formed in organic deposits (Fuller and Francis, 1984). Land cover within the watershed is largely forest, wetland and agriculture with significant developed areas comprising residential, commercial and industrial uses.

The river flows nearly 40 miles from its spring-fed headwaters in Wilmington and North Andover to its mouth in Plum Island Sound, dropping approximately 115 feet in elevation along its course. The river's estuary is part of an extensive salt marsh ecosystem known for its outstanding ecological, economic and recreational value and is one of the most important shellfish areas in the state. The river supports a diverse ecosystem, including valuable aquatic habitat. In addition to physical barriers, groundwater extraction for drinking water presents a significant threat to the health of the river[2].

[1] A run-of-the-river dam is operated such that water is not stored in the impoundment to be released at a later time. Rather, the dam simply increases the head in the river, providing a potential power source that can be captured. It does not serve to prevent or mitigate flooding downstream of the dam since it is generally sized to allow water to flow over the dam during all typical flows.
[2] www.ipswichriver.org/low-flows-floods. Accessed 1/4/17

Figure 1. Ipswich Mills Dam

The Ipswich Mills Dam consists of a cut stone spillway that extends across much of the width of the river. On the right side of the dam is a granite pier incorporating low level gates used to control water levels in the river. Most of the outlets have been plugged, although one still controls flow into an active fish ladder. The dam is thought to have been built on or just downstream of a bedrock outcrop forming the river bed (known as the "upper falls"). It is the downstream-most barrier to fish passage on the river and is located at the head of tide, approximately 3.7 miles from the river mouth. Tidal influence and numerous bridges downstream of the dam have an effect on flood flows at the dam location.

This report builds on feasibility work completed in an earlier phase (Town of Ipswich, 2014) and supports an ongoing wider feasibility assessment led by Horsley Witten Group of the potential impacts of dam removal on the lower Ipswich River. The objectives of this report are to:

- Provide updated hydrologic information;
- Assess current and post-removal hydraulic conditions during both low and flood flows;
- Assess potential hydraulic impacts, including impacts to infrastructure within the study extents, flooding, fish passage, ecology, and recreation; and
- Inform the selection of a preferred approach for the dam removal project.

Hydrology

FLOOD FLOWS

Flows at the dam site are affected to an unknown degree by a number of upstream dams on the mainstem and tributaries. The nearest USGS flow gage (ID 01102000, Ipswich River near Ipswich, MA) is located approximately 200 feet downstream of the Willowdale Dam and 4.6 miles upstream of the Ipswich Mills Dam. The drainage area to the gage is 125 square miles. Instantaneous annual peak flows and daily average flows were downloaded for the period of record (1930-present). Annual peak flow rates are plotted in Figure 2.

Figure 2 suggests a trend of increasing magnitude of runoff events since around 1970 with the highest flows on record occurring after 1980. The highest peaks on record are consistent with known flood events and so were taken as part of the systematic record (i.e. not outliers). The flood record at Ipswich is consistent with observations from across New England that indicate a climatic shift towards increased magnitude and frequency of floods (Collins, 2009; Armstrong et al., 2011). Changes since around 1970 may also reflect in part land cover changes and/or upstream flow management. For this study, peak flows were analyzed for the whole period of record and for the period of record since 1970 to provide a comparison.

Statistical analysis of the peak flow series was carried out using the USGS program PeakFQ which uses the methodology outlined in USGS Bulletin 17B (U.S. Interagency Advisory Committee on Water Data, 1982). The magnitudes of the annual events are assumed to follow a log-Pearson Type III probability distribution, the parameters of which are used to calculate flows for selected exceedance probabilities. The resulting flood frequency information was transferred from the gage site to the Ipswich Mills Dam site using the Drainage Area Ratio Method and a drainage area ratio of 1.18 (148/125 square miles).

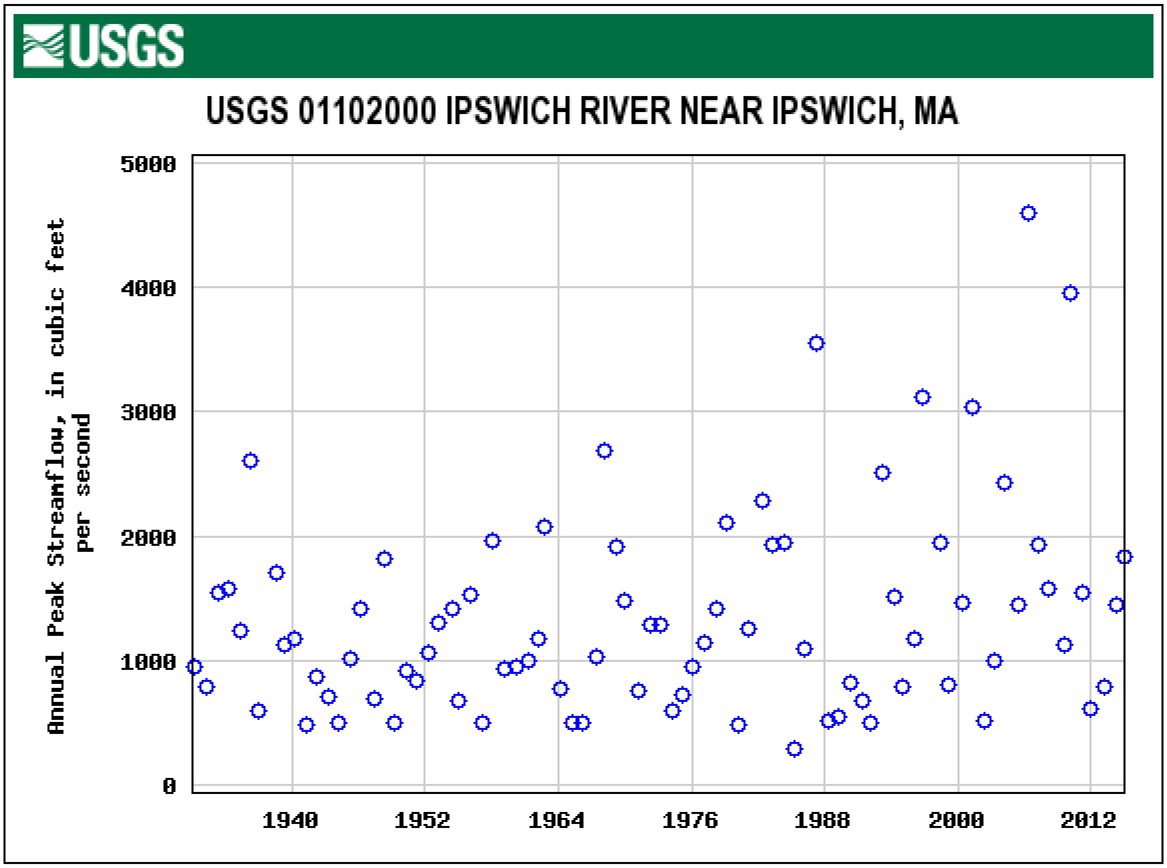

Figure 2. Instantaneous annual peak flows at USGS gage 01102000 located 4.6 miles upstream of the dam (Source: https://waterdata.usgs.gov)

The results of the analysis are given in Table 1 and Appendix A for both the full flood record and the record since 1970, and a comparison with the latest flood insurance study (FIS; FEMA, 1985) results is provided. Flood discharges used in the FIS had also been computed via statistical analysis of peak flow data recorded at USGS gage 01102000; however, the period of record was much shorter at the time and did not include the significant flood events experienced since the 1980s. In order to provide the most conservative results, the peak discharges calculated from the post-1970 dataset were used in the hydraulic model.

Table 1. Peak discharges for a range of recurrence intervals at Ipswich Mills Dam

Recurrence Interval (years)	2	10	25	50	100	200	500
1930-present (cfs)	1,324	2,824	3,791	4,609	5,514	6,514	8,003
1970-present (cfs)	1,439	3,316	4,569	5,644	6,846	8,187	10,203
FIS values (Ipswich River at Central St/ Choate Bridge)	-	2,023	-	3,016	3,251	-	4,196

FISH PASSAGE FLOWS

Mean daily flow data from 1970 to present was analyzed by month using the Duration Analysis function in HEC-DSSVue v.2.0. A sample duration curve for the month of April is shown in Figure 3.

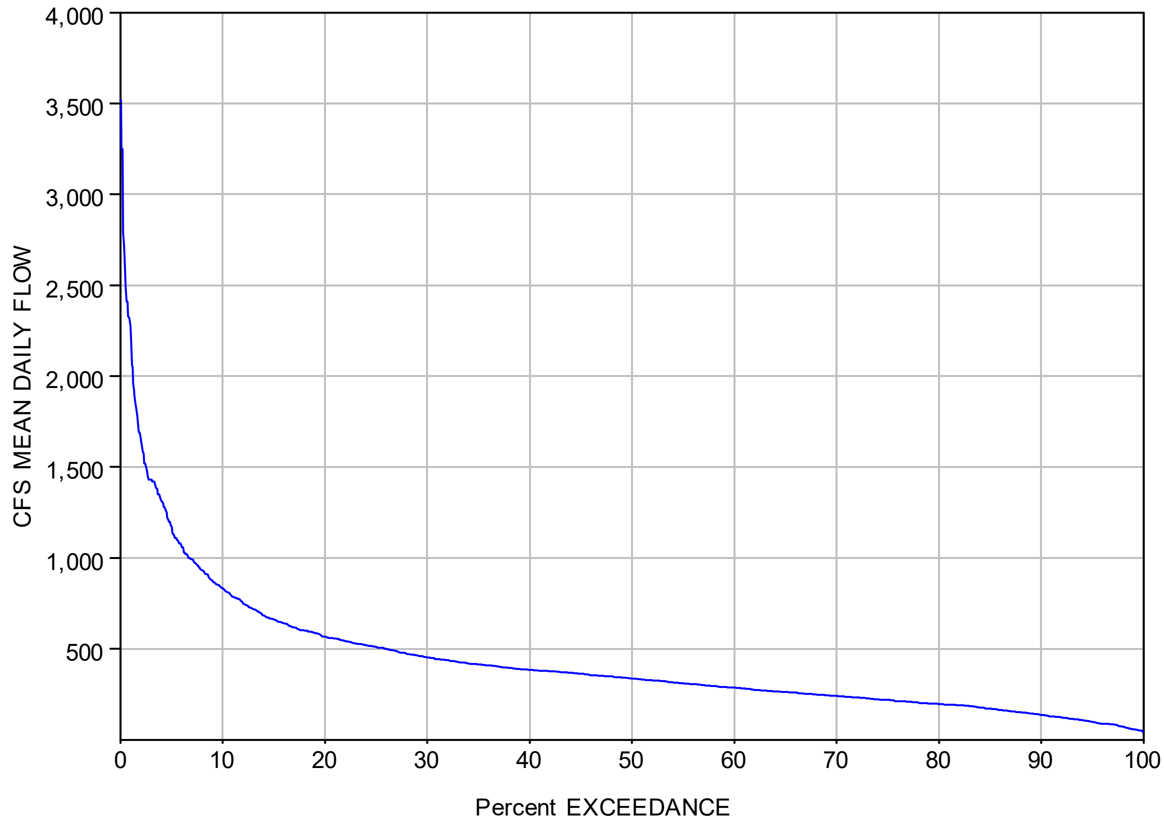

Figure 3. Duration curve for April mean daily flows at Ipswich Gage (USGS 01102000), 1970 to present

The results were exported in tabular form for common exceedance probabilities and scaled by drainage area. Table 2 provides a summary of the results for March through June, spanning the typical upstream migration periods for river herring and shad.

Table 2. Flow rates in cfs corresponding to 5, 50, and 95% exceedance probabilities at Ipswich Mills Dam

	March	**April**	**May**	**June**	**Entire period**
5% exceedance	1,445	1,391	571	721	1,142
50% exceedance	390	398	249	118	288
95% exceedance	113	117	72	26	47

Model Development

DATA SOURCES

We have assembled existing data sources and developed a one-dimensional, mixed, steady-state HEC-RAS model (v. 5.0.3) to investigate the potential hydraulic implications of removing the Ipswich Mills Dam to the fullest vertical and lateral extent practical. Features included in the HEC-RAS model and/or considered in this study are shown in Figure 4.

Cross sections for input into HEC-RAS were compiled from a number of sources; all surveys completed for the project were tied to known datums using survey grade differential global positioning technology (RTK-DGPS). Elevation data given in this report are relative to the NAVD88 vertical datum in units of feet. Bathymetric data was collected by Norde-East Survey in August 2014 and surveyed cross-section data was collected by Horsley Witten Group in August and September 2016, and in April 2018. A total of 25 channel and bridge cross sections were surveyed from a location approximately 1,100 feet upstream of the railroad bridge, downstream through an area called "lower falls" located just downstream of the County St. Bridge where a bedrock outcrop forms the river bed and provides downstream grade control (Horsley Witten Group, 2016). Two cross sections were surveyed at the lower falls: one at the top and one at the base of the exposed bedrock. The cross section across the dam structure itself was input into the model as an inline structure (Sta 3051) and defined using point data from the same Horsley Witten survey, with a minimum crest elevation of 8.79 feet.

For the purposes of this report, we use the term "lower impoundment" to describe the channel immediately upstream of the dam and "impoundment" when referring to the entire length of channel upstream of the dam that is hydraulically affected by the dam structure. Five cross sections were defined through the lower impoundment based on bathymetric data collected by Norde-East Survey in August 2014. Additionally, a cross section surveyed by IRWA in 2013 approximately 10 feet upstream of the dam was incorporated into the model (Sta 3063; reported in Town of Ipswich, 2014). Finally, a cross section was included immediately downstream of the dam (Sta 3041) to define the scour pool present at the foot of the dam. All cross sections were extended across the floodplain using available LiDAR[3] data (Figure 5). At the upstream extent of the study area near the railroad bridge, the floodplain includes secondary channels outside the limits of the surveyed cross sections that focused on the main channel. There is no known opening through the railroad embankment other than the one surveyed for this study (see Figure 6).

[3] US Geological Survey North East Project 2011 LiDAR, 1m grid resolution.

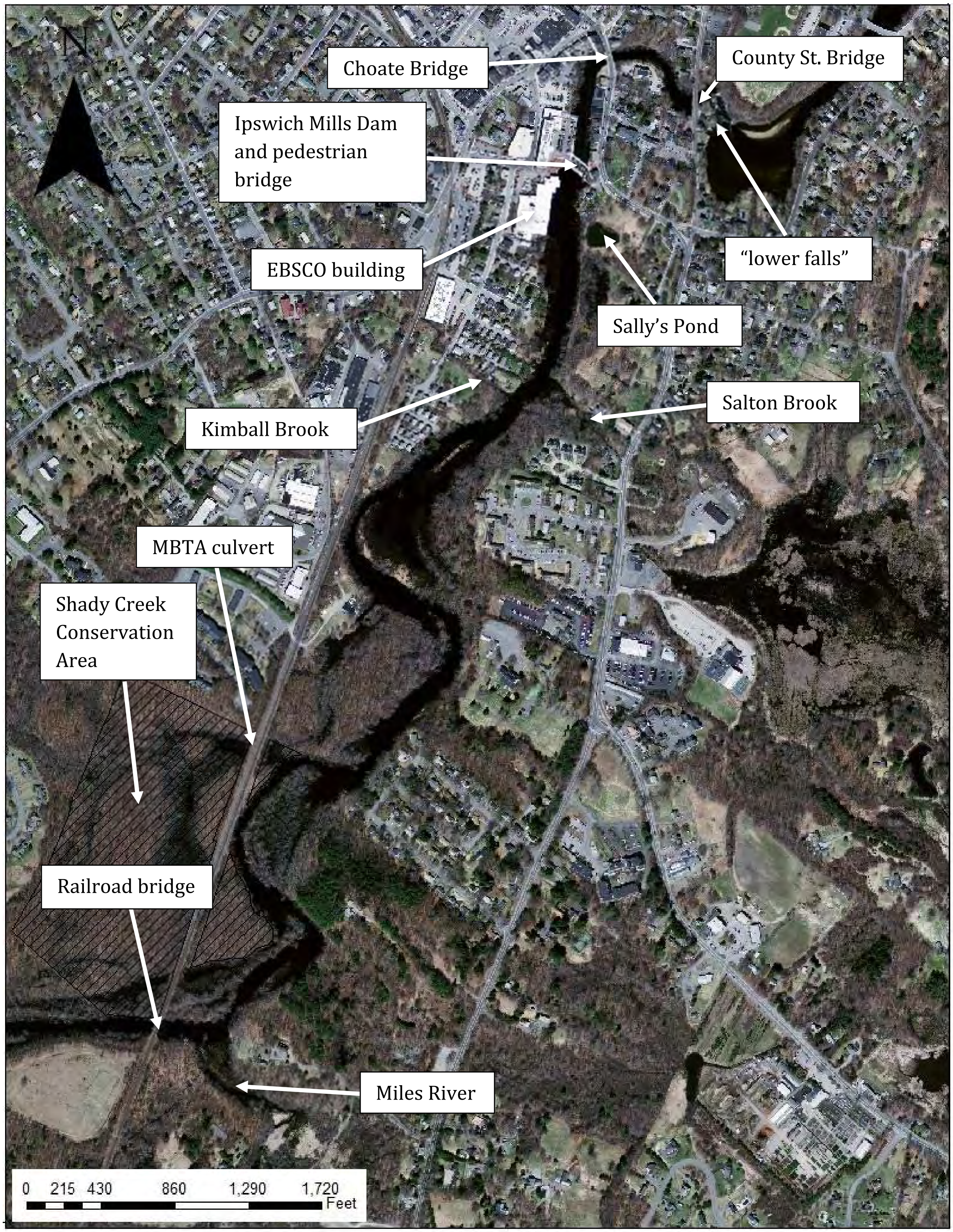

Figure 4. Aerial photograph of study area (Source: USGS color ortho imagery, 2009)

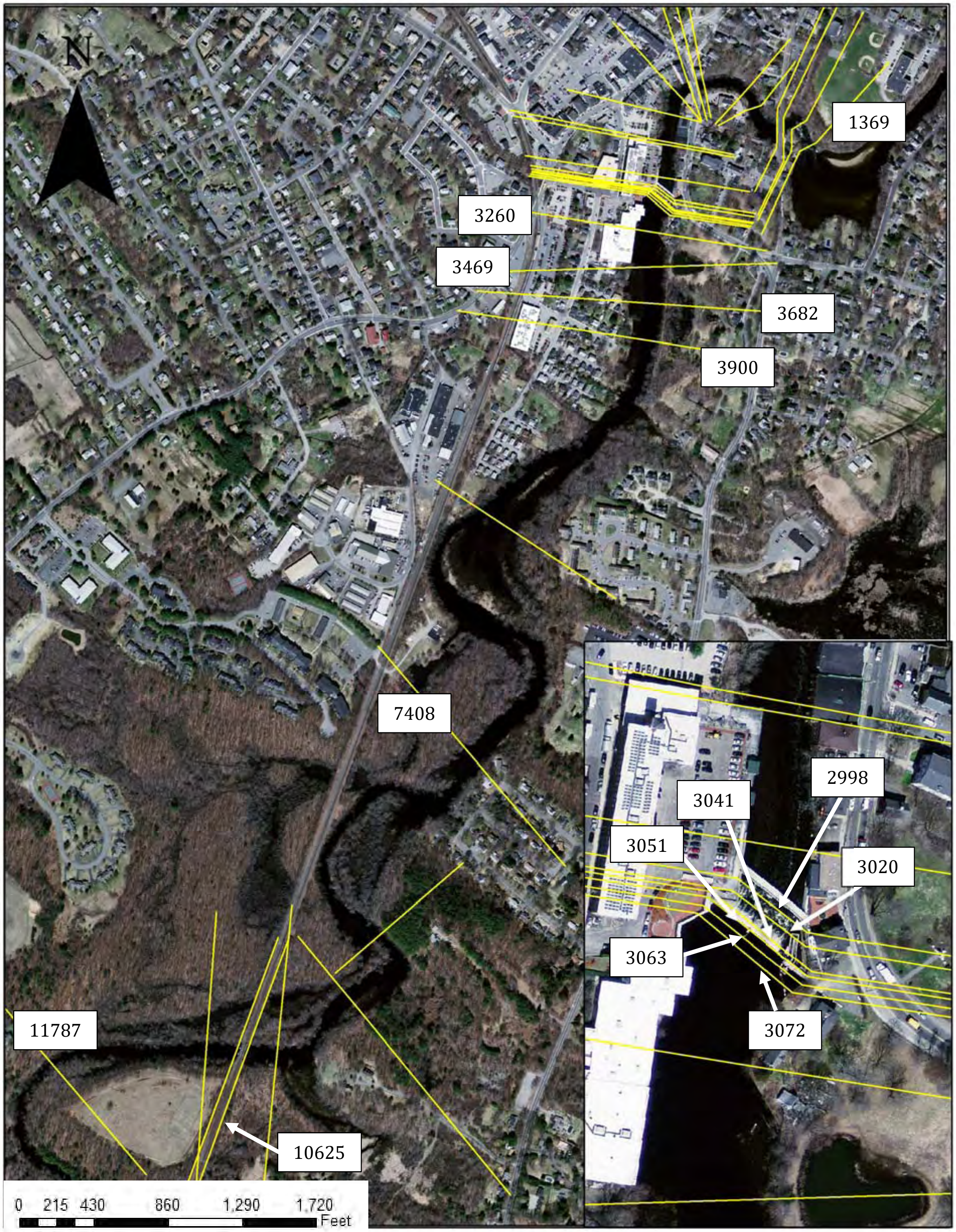

Figure 5. Map showing HEC-RAS cross section locations with inset of area around dam and selected cross section stations labeled for reference (Source: USGS color ortho imagery, 2009)

Manning's n values used in the FEMA model (1985) ranged from 0.02 to 0.04 in the Ipswich River channel, and 0.03 to 0.1 in overbank areas. For this study, a Manning's n value of 0.04 was used for the channel and impoundment, corresponding with an average value for clean, winding channels with some pools and shoals (Chow, 1959). On the floodplains, a Manning's n of 0.1 was used in heavily vegetated areas with lower values of 0.016 in paved areas lacking vegetation and 0.04 in developed areas with curbs, vegetation, or other obstructions. The selected values of Manning's n are consistent with observed field conditions, typical for values used in many hydraulic studies for similar roughness conditions, and similar to those used in a previous project on the Ipswich River (Inter-Fluve, Inc., 2015).

BRIDGES

Railroad Bridge

The railroad bridge located near the upstream limit of the study area spans approximately 90 feet and has two primary central piers and six rows of piles in the main channel (Figure 6). The surveyed low chord of the bridge is 12 to 13 feet above the channel thalweg through the bridge section. Bed levels are higher along the downstream side of the bridge where rock has likely been placed to provide scour protection. The high chord of the bridge was estimated from LiDAR data to be 21 feet. The energy (standard step) computation method was used for the low flow bridge modeling approach, and pressure and/or weir was used for the high flow modeling. The upstream water surface elevation was used to check for pressure flow.

Figure 6. Looking upstream at the railroad bridge taken when the impoundment was drawn down showing accumulated rock on the bed (photo credit: Horsley Witten Group)

Choate Bridge

The Choate Bridge is a historic stone bridge built in 1764. It has a total length of approximately 72 feet with two stone arches, each spanning approximately 30 feet and 13 feet in height. Two arch culverts placed in parallel were used to model the Choate Bridge (Figure 7). A Manning's value of 0.015 was used for the top of the structure (brickwork lined with cement mortar) and 0.04 was used for the bottom to replicate the in-channel conditions. At the bridge, a roadway elevation of 16 feet was estimated from the surrounding LiDAR as ground survey of the bridge deck was unavailable at the time of the study. The stone railing was estimated to be three feet tall from photographs; therefore, the upstream high chord was assumed to be 19 feet. The energy (standard step) computation method was used for the low flow bridge modeling approach, and pressure and/or weir was used for the high flow modeling. The upstream water surface elevation was used to check for pressure flow.

Figure 7. Looking upstream at Choate Bridge during low flow and low tide conditions

County St. Bridge

County St. Bridge is composed of three arches, two tandem arches (spanning approximately 30 feet and 13 feet high) along the mainstem of the channel and a third, smaller arch (spanning approximately 15 feet and 9 feet high) on river right that transports water into an abandoned mill raceway (Figure 8). A Manning's value of 0.015 was used for the top of the structure (brickwork lined with cement mortar) and 0.04 was used for the bottom to replicate the in-channel conditions. Cross sections immediately upstream and downstream were manually updated to reflect the in-channel survey data, including sediment deposition that was observed. A high chord of 16 feet was estimated from the surrounding LiDAR as ground survey of the bridge deck was unavailable at the time of the study. The energy (standard step) computation method was used for the low flow bridge modeling approach, and pressure and/or weir was used for the high flow modeling. The upstream water surface elevation was used to check for pressure flow.

Figure 8. Looking downstream at County Street Bridge at low flow and low tide conditions

DOWNSTREAM BOUNDARY CONDITIONS

Currently, the Ipswich Mills Dam forms the upstream boundary of tidal influence on the river. Numerous sources of data were investigated in the selection of appropriate tide levels to define the downstream boundary conditions for the hydraulic analysis. These sources included: (1) Results from data loggers deployed immediately upstream and downstream of the dam from September 7, 2016 through November 7, 2016[4]; (2) Plum Island Ecosystems LTER water level data collected at the Ipswich Bay Yacht Club, Plum Island Sound from 2011-2016[5]; and (3) NOAA tidal gage records for Boston, MA (ID 8443970) and Fort Point, NH (ID 8423898).

The downstream extent of the model is the base of the "lower falls" located immediately downstream of the County St. Bridge. Downstream of the falls, the river has been artificially widened to form a cove (the Great Cove). Based on the nature of the channel from this location downstream to the coast, it is likely that tide levels are very similar at the Great Cove and the Ipswich Bay Yacht Club.

Although high-quality tidal data exist for the yacht club site for the past six years, statistical analyses of long-term trends and predictions of extreme levels are unavailable. For this reason, it was decided that long-term data from a NOAA gage would be scaled for use in the model based on a relationship between water levels at the yacht club and water levels at a specific NOAA gage. We compared high and low tide levels between the Ipswich Bay Yacht Club and both the Boston and Fort Point gages for every tidal cycle in September 2016, a month during which tidal elevations represent typical tide levels throughout the year. We found a slightly stronger linear correlation with recorded levels at the Boston gage ($r^2 = 0.991$ for high tides and 0.968 for low tides; see Figure 9 and Figure 10) than with recorded levels at the Fort Point gage ($r^2 = 0.987$ for high tides and 0.964 for low tides). It was therefore decided to scale tidal data from the Boston site.

The site-specific high-tide levels captured by loggers deployed by Horsley Witten Group also appear to correlate better with data from the Boston gage than from the Fort Point gage (Figure 11). Note that water levels at the site are influenced by bed levels and river flow as well as tidal fluctuations.

[4] Data collected by Horsley Witten Group

[5] http://pie-lter.ecosystems.mbl.edu/content/data-catalog-research-area. High-quality water level data relative to NAVD88 are available for years 2011-2016.

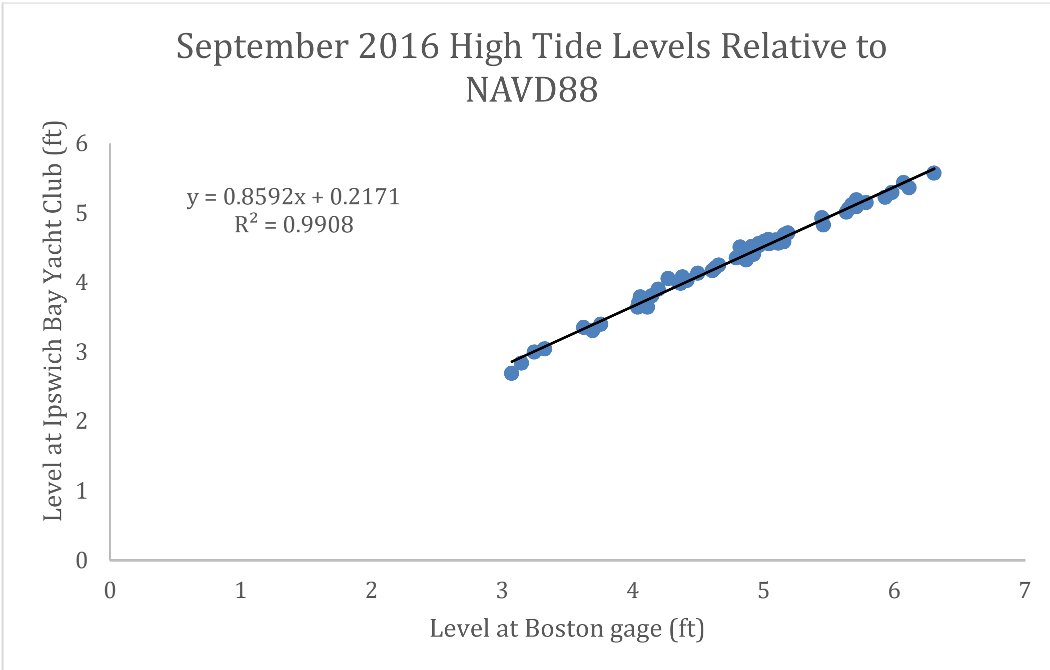

Figure 9. September 2016 high tide levels at Ipswich Bay Yacht Club plotted against high tide levels at the Boston gage (NOAA ID 8443970)

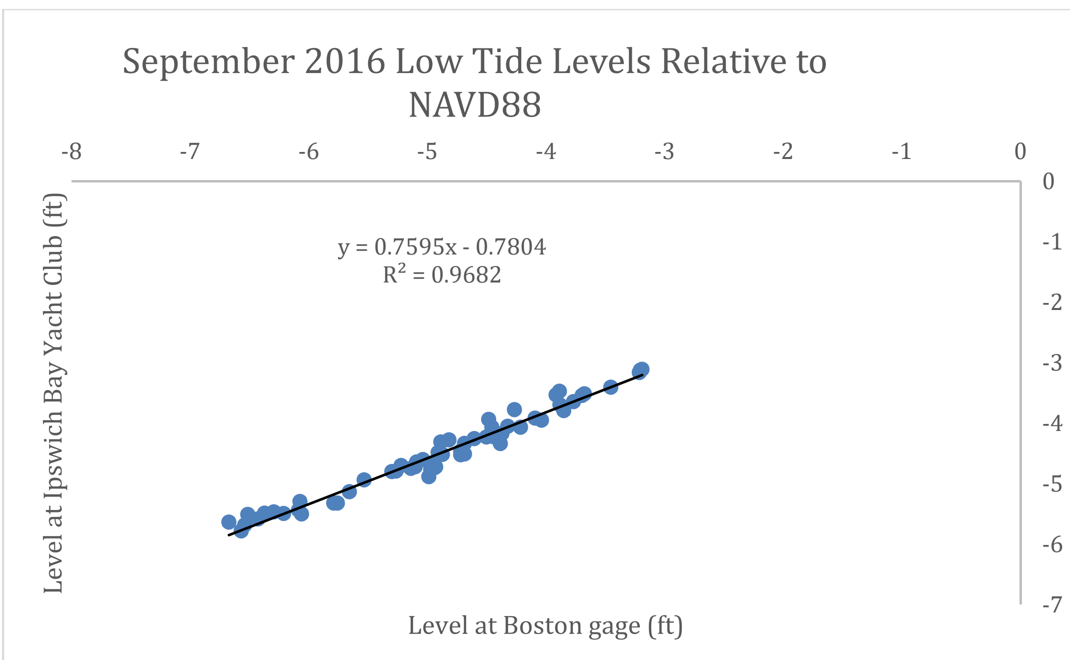

Figure 10. September 2016 low tide levels at Ipswich Bay Yacht Club plotted against high tide levels at the Boston gage (NOAA ID 8443970)

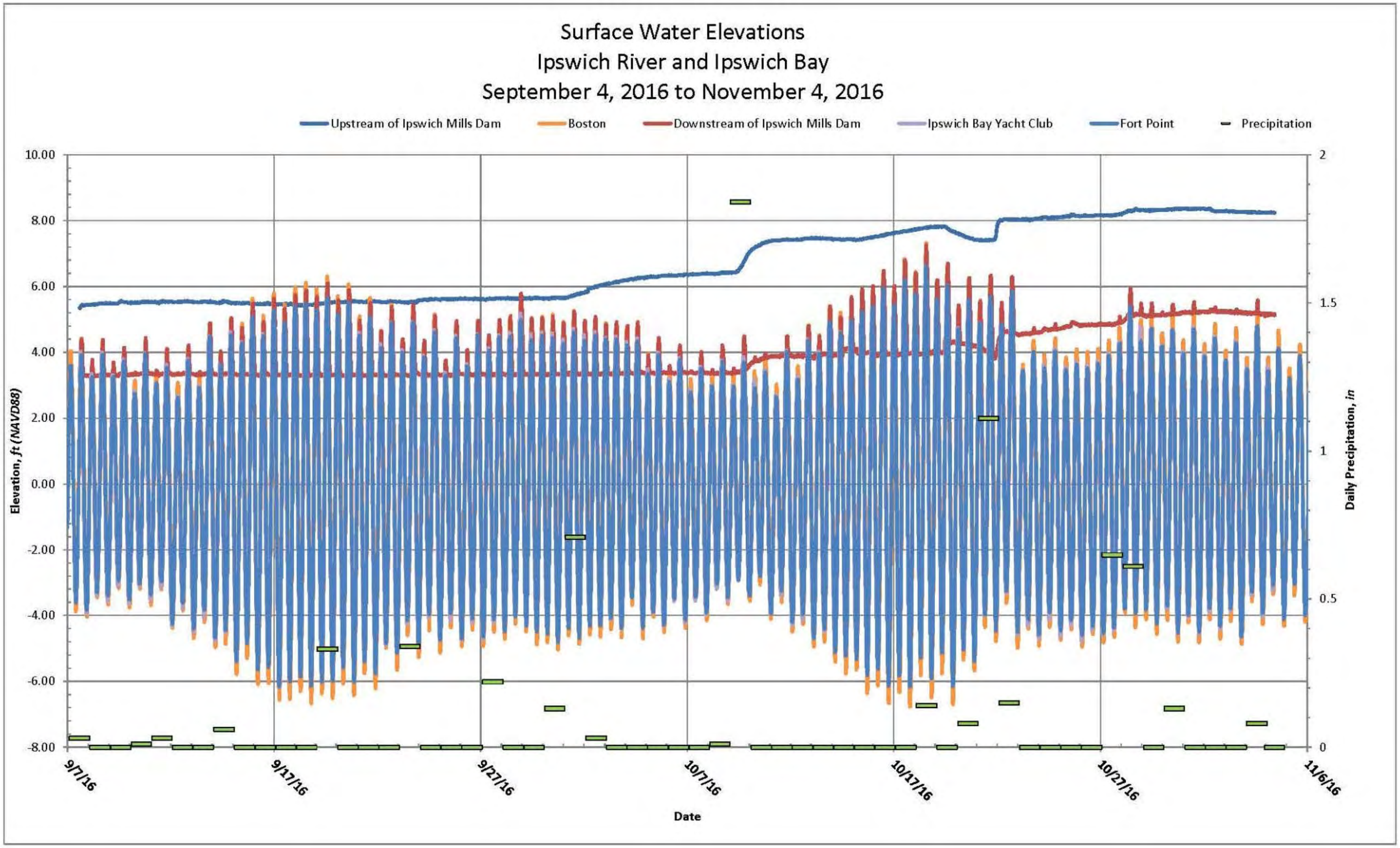

Figure 11.Comparison of water levels monitored by Horsley Witten (Upstream of Ipswich Mills Dam and Downstream of Ipswich Mills Dam) with records at tidal gages. Figure by Horsley Witten.

We tested our high and low tide linear correlations derived from the September 2016 data by applying them to the ten highest and ten lowest tide levels recorded during the period of record at the yacht club (i.e. from 2011 through 2016). This includes high tide of 8.33 feet on January 3, 2014, which is the eighth highest tide level on record at the Boston site (period of record 1921 – 2016; for context, the highest on record is 9.59 feet on February 7, 1978). The linear relationships predicted tide levels at the yacht club well with an average difference between predicted and actual levels of 6%, or 0.4 feet. The relationships were therefore considered suitable for use in converting Boston tidal datums to datums for the yacht club site. More recent tidal data, including high tides experienced in early 2018, will be incorporated in updated hydrology and hydraulic analyses in the next design phase.

NOAA estimates tidal datums and extreme tide levels for the current year based on linear historic trends over the most recent tidal epoch. For current (2016) predictions, that epoch is 1983-2001. Figure 12 gives 2016 predictions for various datums and exceedance probability levels in meters relative to the Mean Sea Level datum at the Boston gage (-0.3 feet NAVD88). These predictions are likely underestimates given that the rate of sea level rise has been increasing in recent decades (Church and White, 2011).

Mean High Water (MHW) and Mean Low Water (MLW) were selected to represent long-term average tide levels for this study and the datums calculated in feet relative to NAVD88. The 1, 10 and 50% exceedance probability levels were also calculated in feet relative to NAVD88. These values for the Boston gage were then scaled to the Ipswich Bay Yacht Club location using the linear relationships in Figure 9 and Figure 10. The results are given in Table 3.

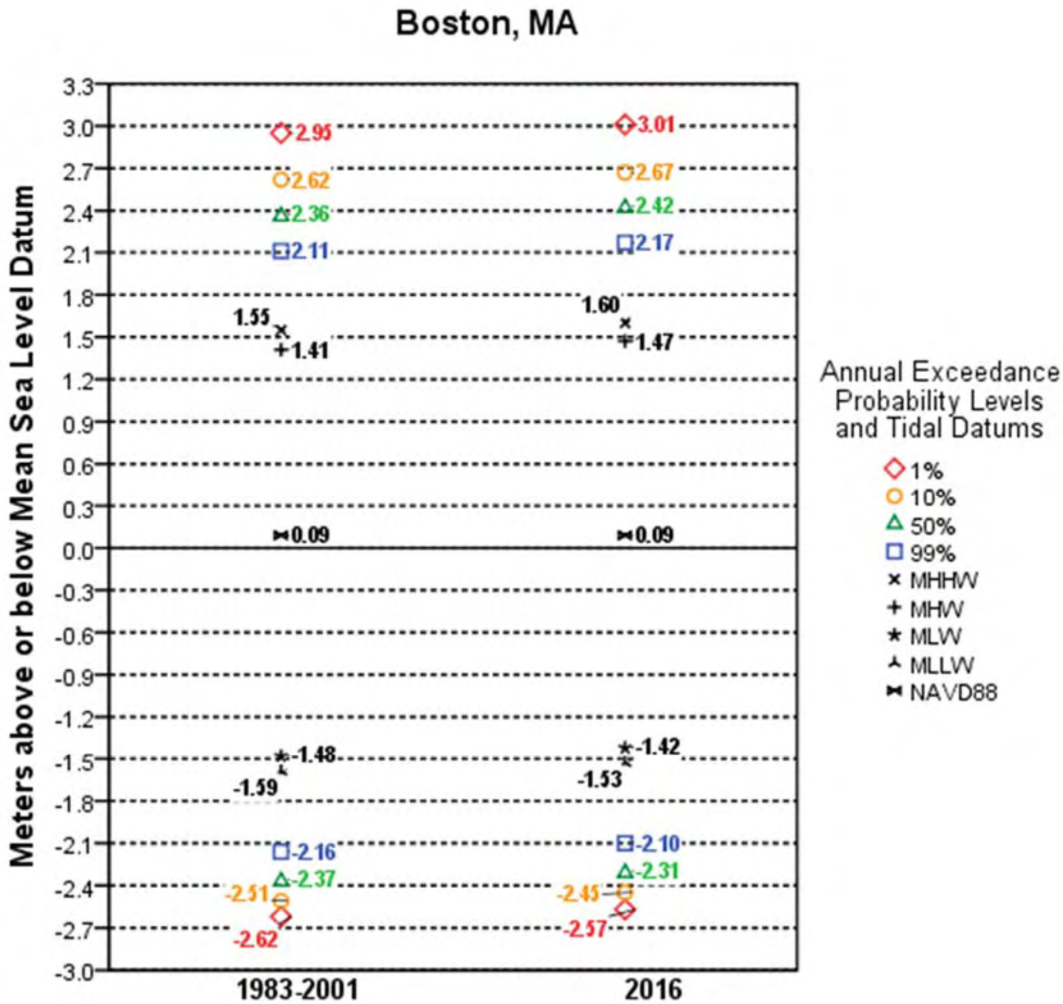

Figure 12. Tidal datums and exceedance probability levels relative to Mean Sea Level. On the left are the exceedance probability levels for the mid-year of the tidal epoch currently in effect for the station. On the right are projected exceedance probability levels and tidal datums assuming continuation of the linear historic trend. (Source: https://tidesandcurrents.noaa.gov/est/stickdiagram.shtml?Staid=8443970)

Table 3. Tidal datums and exceedance probability tide levels in feet relative to NAVD88 as compared with FEMA stillwater tide level

	Boston, MA	**Scaled to Ipswich Bay Yacht Club**
1% exceedance probability stillwater tide level (FEMA, 1985)	N/A	8.7
1% exceedance probability	9.58	8.45
10% exceedance probability	8.46	7.49
50% exceedance probability	7.64	6.78
Mean Higher High Water (MHHW)	4.77	4.32
Mean High Water (MHW)	4.52	4.10
Mean Low Water (MLW)	-4.96	-4.55

The 1% exceedance probability projection for 2016 likely corresponds to the storm surge and limited wave setup caused by breaking waves but not for wave runup, similar to FEMA's stillwater elevation as referenced in the 1985 FIS (FEMA, 1985). The difference between the FEMA level of 8.7 feet and the value computed for this study is likely a function of the uncertainty in the scaling applied to the Boston 2016 prediction. For consistency with the FEMA study and for the purpose of providing conservative results, the FEMA stillwater tide level of 8.7 feet was used to represent extreme high tide levels for this study.

ACCURACY TESTING OF EXISTING CONDITIONS MODEL

The survey conducted by Horsley Witten included the elevations of eight distinctive stain lines on structures (i.e. walls or bridges) downstream of the dam that were thought to correlate with mean annual high water. These data points were compared to the existing conditions model results for the 2-year flow event with a normal depth downstream boundary condition (Figure 13). A normal depth downstream boundary condition represents a tide out scenario where the water depth at the downstream boundary of the model is a function of discharge and channel geometry, slope, and roughness. The model results support the general trend of the collected mean annual high-water survey shots, with some variability likely due to tidal influences not replicated in the model and the inherent variability and lack of precision of this type of data.

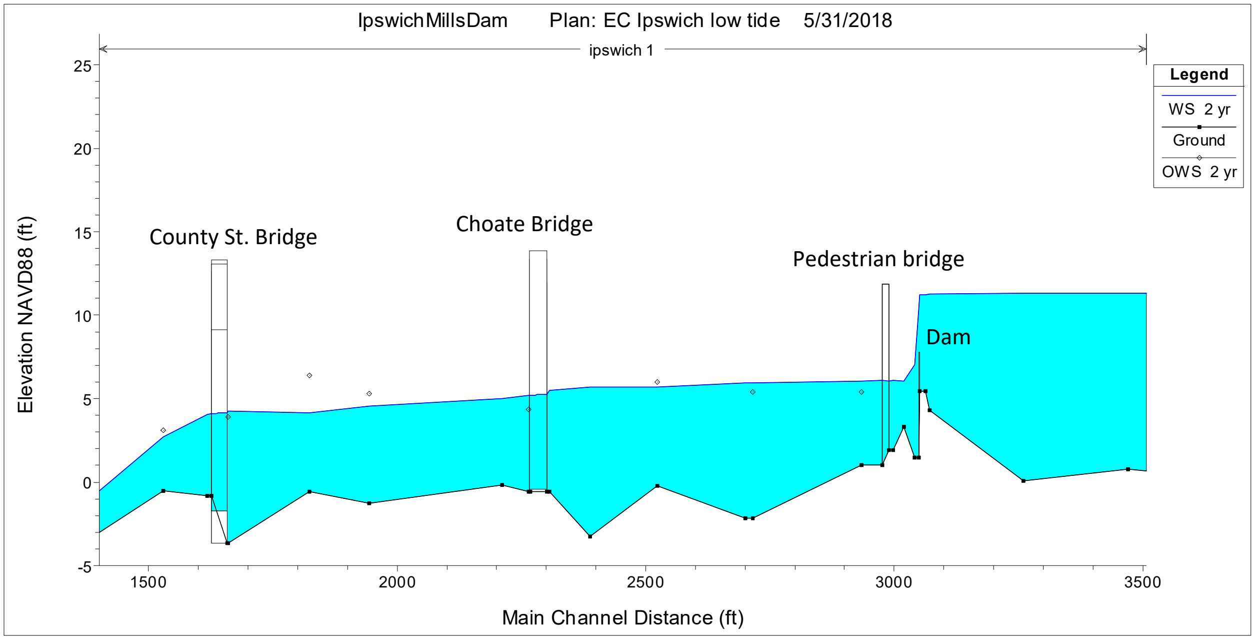

Figure 13. Existing conditions run showing the predicted water surface (labeled as WS 2yr) relative to field observations of Mean Annual High Water collected during the survey (labeled as OWS 2yr)

Results for larger flood flows were examined against previous results from the 1985 FIS. Discharges from the FEMA study (see Table 1) were run through the current existing conditions model using the stillwater tide downstream boundary condition of 8.7 feet, and the resulting water surface elevations were plotted against the FEMA results (Figure 14). The primary differences between the existing conditions model developed for this study and the FEMA model are: (1) updated channel geometry data based on recent survey, and (2) inclusion of the channel and bridge structures downstream of the dam in the hydraulic model. The model developed by FEMA used the stillwater tide elevation to depict backwater from the ocean all the way upstream to the dam; downstream structures were not included in the model.

Results show that upstream of the dam, the simulated flood levels provide a reasonable match to the reported FEMA flood levels. Downstream of the dam, FEMA reported stillwater tide levels only and did not simulate flow through these reaches and bridge structures, explaining the discrepancy between the model results for this study and FEMA results. Over the course of many studies, it has been shown that differences should be expected between simulation results from coarsely resolved, older FEMA studies (in this case over 30 years), and more highly resolved, current, project-scale models. The comparison of results in this study are very consistent with trends seen on many other rivers.

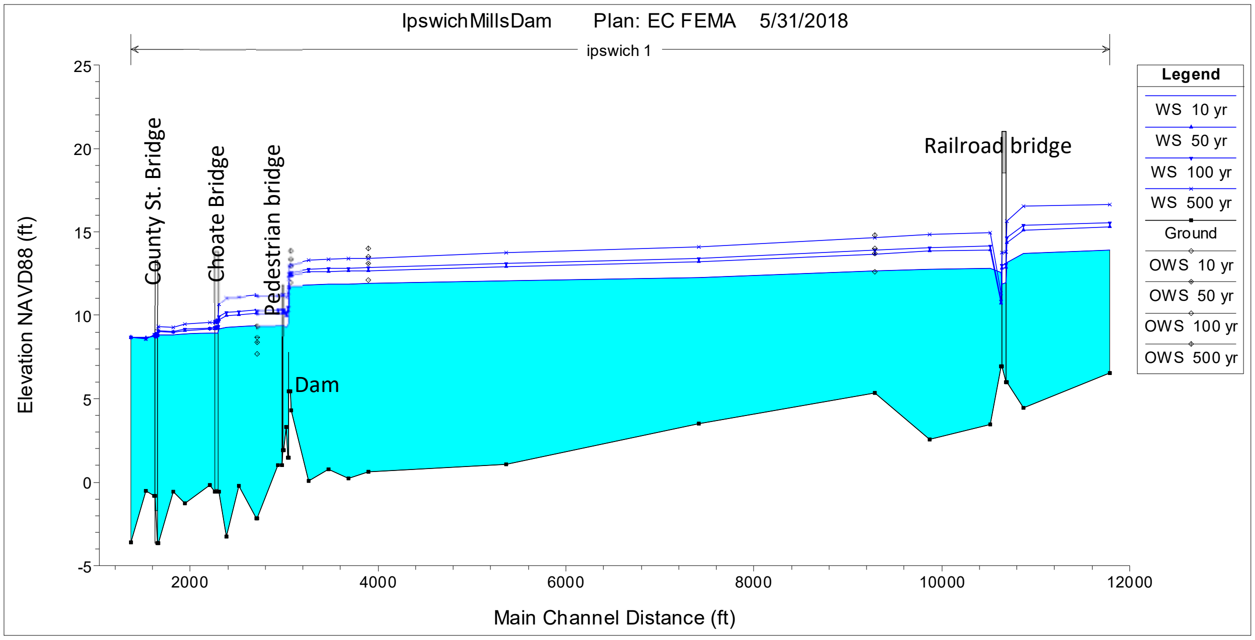

Figure 14. Existing conditions run using FEMA flows. Predicted water surfaces (labeled as WS 10yr, for example) plotted with water surface elevations from the 1985 flood insurance study (data points labeled as OWS 10yr, for example)

Finally, we simulated a known high-flow event for which peak flood stage had been recorded. Peak flood stage was recorded at various points along the Ipswich River and other rivers during the March-April 2010 flood event and reported in a U.S. Geological Survey Open-File Report (Zariello and Bent, 2011). The peak flow recorded at the Ipswich River gage used for this study (ID 01102000, Ipswich River near Ipswich, MA) was 3,950 cfs. Stage was reported at one site within the study area: 13.52 feet elevation at Footbridge behind cemetery US1, which is near Sta 3900 in the existing conditions model. Applying the drainage area ratio of 1.18 to the recorded peak flow, the corresponding peak at the dam site would have been approximately 4,660 cfs. We ran this flow through the existing conditions model and predictions indicate a peak flood stage of 13.58 feet at Sta 3900. The close agreement of these two sources suggests that the model provides a reasonable representation of flood levels for existing condition.

DAM-OUT CONDITIONS MODEL GEOMETRY

As discussed with the project partners, the goal of the dam removal project at this site is to eliminate the barrier to fish passage while minimizing risk to infrastructure and adverse impacts to ecology and existing river uses. We developed a basic dam-out scenario for the purposes of testing the hydraulic impacts of full dam removal, including:

- Removal of the full vertical height and lateral extent of the dam and associated structures where possible;
- Limited channel restoration or modifications to other river infrastructure; and
- A conservative approach to defining the post-removal bed surface at the dam site in order to gain an understanding of worst-case risk to upstream and downstream infrastructure in terms of effects on hydraulic conditions.

This scenario involves leaving the disused fishway integral with the existing river in place in order to avoid destabilizing the river right (looking downstream) wall during demolition. An approximate 10-foot section of the existing viewing platform would be retained as a part of this minimum measure to protect the wall (Figure 15). All other elements of the dam would be removed down the full vertical extent.

We examined existing survey data to estimate the long-term bed profile through the project site following dam removal. Figure 16 shows the longitudinal channel bed thalweg profile compiled from the Horsley Witten surveys along with additional points and depth-of-refusal data extracted from Norde-East (2014) survey data. The depth-of-refusal surface was determined by probing and represents the elevation of competent material beneath impounded fine sediment. The extent of the depth-of-refusal data is limited, and available data along the thalweg are shown in Figure 16.

The bed within the impoundment immediately upstream of the dam is composed of cobbles and boulders at levels just below the dam crest. The origin of this material is unknown and it may have been placed as a part of a previous project to support or protect a disused pipe running across the channel. Upstream of this area, bed levels drop off away from the dam. The depth-to-refusal survey shows from 0 to up to 3 feet of sand and silt accumulation above firm subgrade, or what may have been the historical stream bed, through the lower impoundment. Accumulation along the channel thalweg is minimal with greater depths of sediment detected at the channel margins.

Figure 15. Modeled limits of dam removal on river right

Historical, anecdotal, and recent information suggest that bedrock is present in the bed of the channel at the dam site. The elevation of the bedrock surface is unknown, however. The following is an excerpt from the Task 1 summary for this project (Horsley Witten Group, 2018):

A hard surface that may have been ledge and/or large boulders was observed spanning the width of the river approximately 10-20 feet upstream of the dam during an IRWA preliminary field survey in 2010, and during a bathymetric survey conducted by Norde-East, Inc. in 2014. During the field survey conducted by HW during a drawdown of the impoundment as part of this current study in August, 2016, at least the hard surface layer of this feature was observed to consist of boulders, as opposed to bedrock ledge. Therefore it is uncertain at what elevation bedrock ledge may underlie the surficial boulders at the dam site. There is, however, some information that suggests a potential approximate elevation of the bedrock, even if it cannot be accurately identified at this time:

- *As part of the Task 3 structural assessment of this current project conducted by Simpson, Gumpertz, and Heger, Inc. (SGH), a test pit excavated in the river at the edge of the EBSCO building foundation, near the western edge of the dam, revealed bedrock at approximately elevation 3.2 feet (NAVD 88).*

This suggests that, at least near the western edge of the dam, bedrock ledge may be present in the general vicinity of the dam several feet below the elevation of the observed boulder surface.

- *During the drawdown, IRWA staff was able to jostle the surface boulders with a steel pry bar confirming the makeup of the surface of the feature as loose boulders. The boulder surface is undulating but has an average elevation of approximately 6 feet (NAVD88). IRWA probed approximately 150 locations across the boulder feature. Of those, 20 went down to a maximum penetration of depth of approximately 5 feet and the remainder penetrated to between 1 and 4 feet (all depths relative to the high point of the boulder feature). In the opinion of the IRWA staff who conducted the probing, the refusal depths are indicative of bedrock ledge. SGH staff, who was onsite at the time of the IRWA probing conducting the test pits mentioned in the above bullet, conducted a level survey of several of the probing locations to relate elevations at probing locations to elevations on top of the dam previously surveyed by HW. The surveyed elevation of the high point along the top of the boulder ridge that the IRWA probing depths were reported relative to is 6.81 feet. Therefore, the lowest elevation of bedrock beneath the boulder ridge estimated by IRWA staff for 20 probing locations is approximately 1.8 feet. The elevation of bedrock beneath the boulder ridge estimated by IRWA staff for the other 130 probing locations is higher, with variable elevations between approximately 2 and 6 feet.*

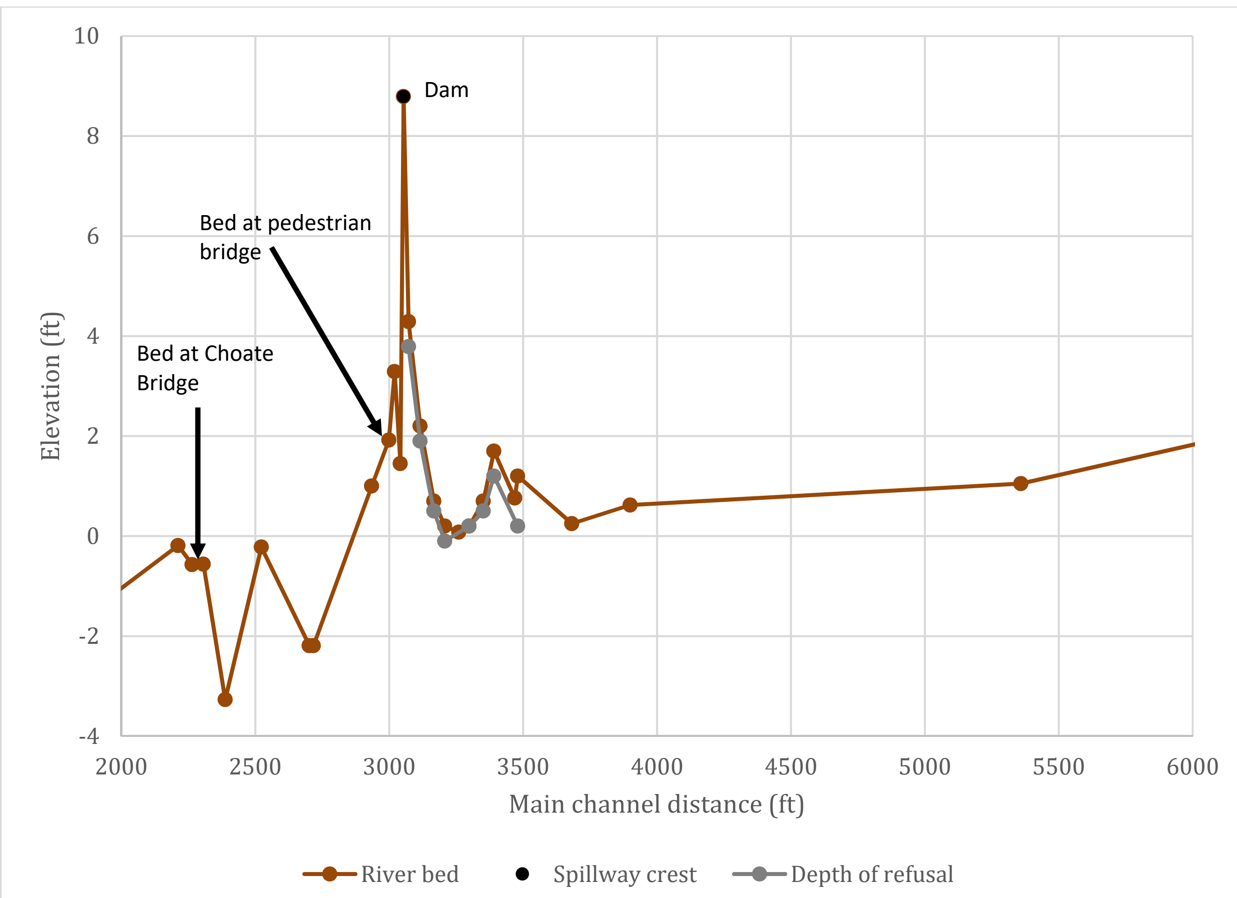

Figure 16. Longitudinal thalweg bed and depth-of-refusal profiles through the dam site. Flow is from right to left.

Because of the uncertainty in the level of bedrock at the dam site, we have taken a conservative approach from the perspective of infrastructure risk and have approximated the long-term bed profile through the former dam site by assuming bedrock is present at a low level and that the long-term bed profile will align with the average upstream and downstream bed profiles. Further investigation will be required to clarify the presence or absence and elevation of bedrock at the site.

It is likely that the fine sediment present on the bed of the impoundment will be mobilized following dam removal; however, based on the limited available data, substantial headcutting and incision of the channel bed in the lower impoundment is not anticipated. The smaller fractions of the coarse material immediately upstream of the dam will also likely be mobilized, and the larger rock will likely be regraded or possibly removed as a part of dam removal. To anticipate potential bed level changes further upstream, additional depth-of-refusal survey extending through the entire impoundment will be required.

Within the model geometry, we removed the dam, the IRWA cross section at Sta 3063, and the fishway from the cross sections immediately downstream of the dam. We also lowered bed levels at Sta 3072 (21 feet upstream of former dam) by 3.04 feet to reflect mechanical regrading or removal of the accumulated rock immediately behind the dam and at Sta 3020 (between dam and pedestrian bridge) by 1.39 feet to depict mechanical regrading of accumulated bed material downstream of the scour pool at the base of the dam. Bed levels elsewhere were left unchanged.

Figure 17 shows the dam-out condition represented by the model as compared with existing conditions. The assumed dam-out profile still shows a high spot on the bed at the pedestrian bridge at Sta 2998 which may be capturing the downstream limit of the existing bed material accumulation associated with the scour at the base of the dam. Over time, this material may be redistributed or could be mechanically regraded as part of the dam removal project, resulting in bed levels approximately 1 foot lower than those shown at this location. To maintain simplicity in our assumptions, we did not alter the bed levels at Sta 2998 for this study, although we did examine the model results at this location to check for localized effects.

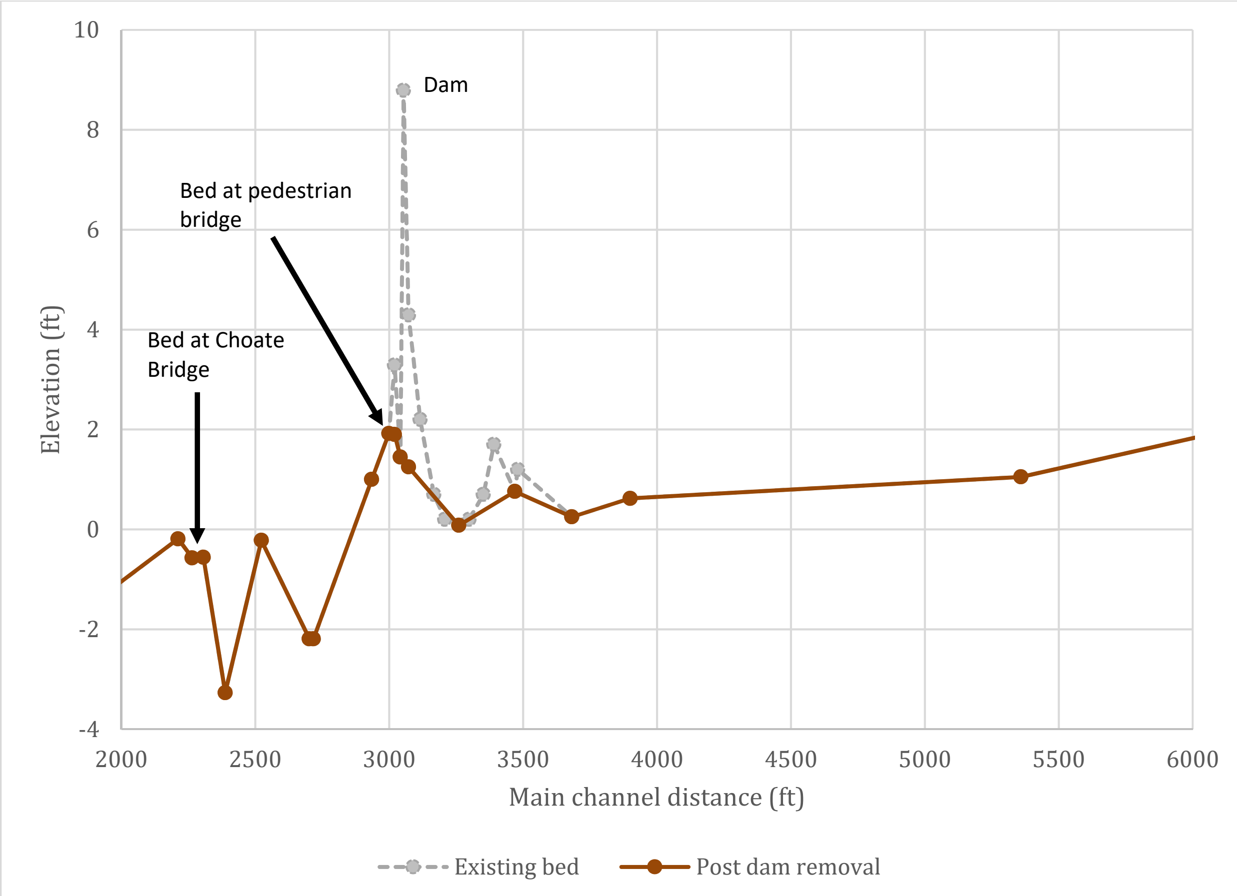

Figure 17. Comparison of existing and modeled long-term longitudinal channel bed profiles. Flow is from right to left.

MODEL SIMULATIONS

Table 4 summarizes the various combinations of flow and downstream boundary conditions used in the modeling to investigate flood risk impacts, channel stability and hydraulics during high flows, and fish passage conditions. The scenarios examined for impacts to flood risk combine high tides with flood flows to provide worst-case inundation extents and depths. The hydraulic conditions with the greatest scour potential (i.e. highest velocities and shear stresses), on the other hand, are likely to occur during high flows when the tide is out. For tide-out scenarios, normal flow depth was selected as the downstream boundary condition because MLW as given in Table 3 is below bed level at the downstream limit of the model (base of lower falls).

Table 4. Combinations of flow and downstream boundary condition for model runs[6]

River Flow Recurrence Interval (years)	Purpose of Run	Flow (cfs)	Downstream Boundary Condition
100	Flood risk	6,846	8.7 feet stillwater tide level
100	Flood risk	6,846	4.10 feet MHW tide
50	Flood risk	5,644	4.10 feet MHW tide
25	Flood risk	4,569	4.10 feet MHW tide
10	Flood risk	3,316	4.10 feet MHW tide
2	Flood risk	1,439	4.10 feet MHW tide
100	Channel stability/ hydraulics	6,846	Normal flow depth
25	Channel stability/ hydraulics	4,569	Normal flow depth
2	Channel stability/ hydraulics	1,439	Normal flow depth
95% exceedance (daily flow series)	Tidal influence	47	4.10 feet MHW tide
Fish passage flow – 5% exceedance (daily flow series)	Fish passage	1,142	Normal flow depth
Fish passage flow – 50% exceedance (daily flow series)	Fish passage	288	Normal flow depth
Fish passage flow – 95% exceedance (daily flow series)	Fish passage	47	Normal flow depth

[6] Exceedance flows calculated over the period March through June

Impact Assessment Results

SEDIMENT

Model results for low flow conditions (95% exceedance; 47 cfs) show that the impoundment effects extend upstream through the railroad bridge (Figure 18). Accumulated sediment within these limits could be mobilized when the dam is removed.

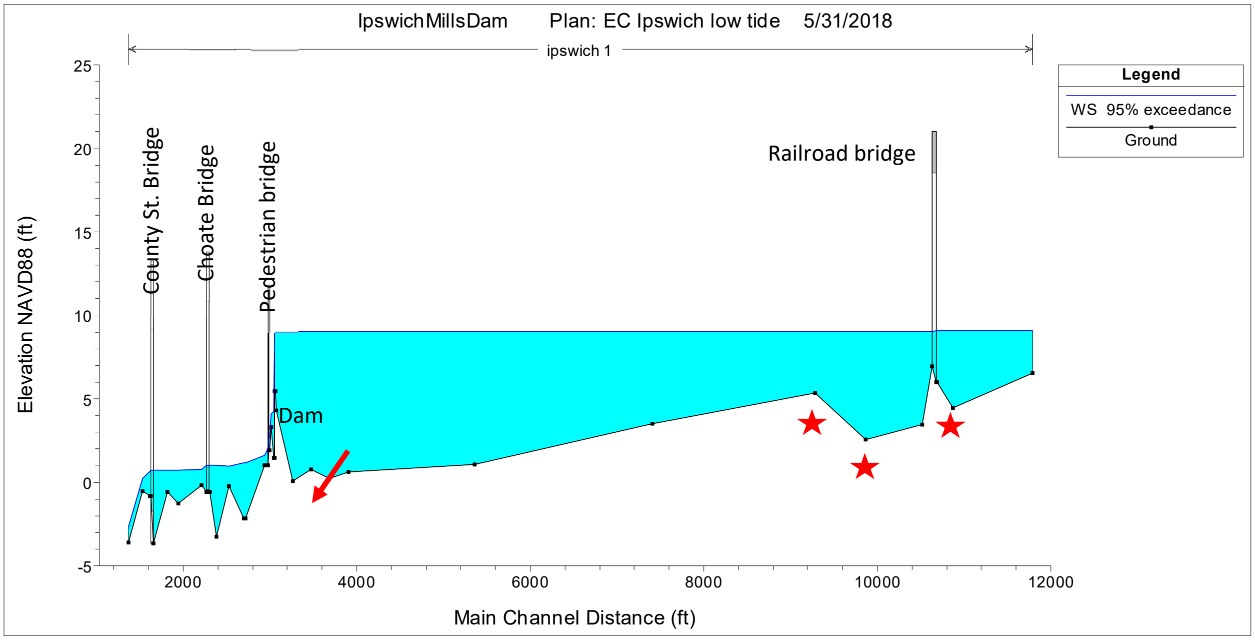

Figure 18. Predicted low-flow water surface profile for existing conditions during low tide. Extent of Norde-East (2014) depth-of-refusal survey indicated by red arrow. Red stars indicate where survey notes are available near the upstream limit of the impoundment and indicate a soft channel bottom.

As described above, the Norde-East bathymetric and depth-to-refusal survey (2014) shows from 0 to up to 3 feet of sand and silt accumulation above firm subgrade through the lower impoundment from the dam to approximately 430 feet upstream (the upstream extent of the data). Measured sediment depths are greater along the margins of the impoundment than along the thalweg. Because of the relatively shallow depths of sediment accumulation, the risk of substantial headcutting along the main river channel in this area is low. The limited fine sediment present on the bed of the lower impoundment is likely to be mobilized following dam removal with some additional mobilization of sediment stored along the margins. In some areas, existing or new growth of vegetation is expected to help stabilize marginal deposits when normal water levels drop and these areas become more regularly exposed. In addition to fine sediment, the smaller fractions of the coarse material currently stored immediately upstream of the dam would likely be mobilized.

Further upstream, detailed survey notes are available for cross sections near the upstream end of the impoundment and indicate that the channel bed is soft (mud or sand) (Figure 18). Recently collected (April 2018) topographical data in this area has revealed a possible wedge of fine sediment that suggests accumulated sediment volume in the upper impoundment may be greater than previously thought. It is recommended that additional depth of refusal survey extending through the entire impoundment be carried out to reduce the unknowns (sediment volume and caliber) and uncertainty related to sediment impacts.

The rock scour protection at the bridge represents the likely upstream limit of sediment mobilization following dam removal. Limited fine sediment accumulation immediately upstream of the railroad bridge may be partly associated with the impoundment but is also likely precipitated by backwater effects from the bridge and rock scour protection.

Fine sediment that is released as a result of dam removal is likely to be dispersed by fluvial flows and tidal fluctuations in the downstream channel. The spatial and temporal scale of the impacts will depend on the volume of material and how rapidly it is released. Release of a substantial volume of coarse sediment in particular could be a potential issue at the Choate Bridge downstream of the dam site where modeling suggests the structure restricts flow during large magnitude events. Model results indicate that bed shear stress conditions at the bridge are sufficient to transport gravel up to cobbles over the range of flows investigated; therefore, the effects are likely to be temporary with material transported past the bridge during subsequent high flows. It is recommended that potential impacts associated with deposition downstream of the dam are monitored following dam removal.

FLOOD RISK

Predicted water surface elevations are given in Table 5 and show a reduction in flood levels through the impoundment upstream to the railroad bridge. High bed levels along the downstream face of the bridge (see rock exposed on bed in Figure 6 and model profiles in Appendix B) and conveyance capacity through the bridge section appear to control flood levels upstream of the railroad bridge.

During the 100-year event, backwater from the Choate Bridge affects water surface profiles upstream through the dam site in both the existing and dam-out scenarios, resulting in very little predicted change in the profiles following dam removal. During lower return period events, predicted reductions in water surface elevation through the impoundment resulting from dam removal are more pronounced, although backwater from the Choate Bridge appears to affect predicted water surface elevations through the former dam site during all flood flows simulated with the exception of the 2-year event.

The results show a slight, localized increase in flood level at Sta 3020 (between the dam and pedestrian bridge) during the 2-year and 100-year events, which likely reflects a change from rapidly varied flow conditions in the existing scenario to subcritical flow conditions with the dam removed. In actuality, the ability of the one-dimensional model to accurately predict precise water surface elevations in areas of rapidly varied flow (existing conditions case) is limited and this difference should be considered within the overall uncertainty of the modeling software itself. No changes in water surface elevations are predicted downstream of the pedestrian bridge for the scenarios tested. Flood profiles for existing and dam-out conditions are provided in Appendix B and inundation maps in Appendix C.

Table 5. Predicted flood water surface elevations for existing and dam-out conditions (ft NAVD88)

	100-year flow and 8.7 ft stillwater tide		100-year flow and 4.10 ft MHW tide		2-year flow and 4.10 ft MHW tide	
River station	**Existing (ft)**	**Dam Removed (ft)**	**Existing (ft)**	**Dam Removed (ft)**	**Existing (ft)**	**Dam Removed (ft)**
11787	21.29	21.29	21.29	21.29	13.00	13.00
10867	21.31	21.31	21.31	21.31	12.79	12.79
10689	20.58	20.58	20.58	20.58	12.33	12.32
10657	Railroad bridge					
10625	15.79	15.69	15.71	15.59	11.85	9.75
10513	16.81	16.74	16.76	16.67	12.04	10.95
9865	16.70	16.62	16.64	16.55	11.99	10.84
9283	16.44	16.35	16.37	16.27	11.89	10.62
7408	15.79	15.67	15.69	15.56	11.59	9.44
5359	15.32	15.18	15.21	15.04	11.44	8.27
3900	14.82	14.66	14.69	14.49	11.34	7.11
3682	14.77	14.60	14.64	14.43	11.33	6.97
3469	14.73	14.56	14.59	14.38	11.31	6.68
3260 (EBSCO building)	14.66	14.48	14.51	14.29	11.30	6.54
3072	14.42	14.41	14.24	14.21	11.24	6.43
3063	14.39	-	14.19	-	11.22	-
3051 (Dam)	14.39	-	14.19	-	11.22	-
3041	14.38	14.35	14.19	14.15	7.03	6.34
3020	14.29	14.33	14.07	14.13	6.05	6.28
2998	14.32	14.32	14.11	14.11	6.17	6.17
2990	Pedestrian bridge					
2934	14.30	14.30	14.08	14.08	6.14	6.14
2717	14.25	14.25	14.03	14.03	6.04	6.04
2701	14.25	14.25	14.03	14.03	6.04	6.04
2522	14.21	14.21	13.99	13.99	5.82	5.82
2387	14.06	14.06	13.84	13.84	5.79	5.79
2306	13.36	13.36	13.12	13.12	5.62	5.62
2302	Choate Bridge					
2264	10.65	10.65	10.27	10.27	5.32	5.32

TIDAL INFLUENCE

We simulated a combination of high tide (MHW) and low flow (95% exceedance) under existing and dam-out conditions to examine the impact of dam removal on the extent of hydraulic tidal influence. Comparison of the water surface profiles in Figure 19 shows that in the absence of the dam, the hydraulic tidal influence is predicted to extend upstream to Sta 7408 near Upper River Road, or approximately 4,350 feet or 0.8 mile upstream of the existing dam and current tidal limit (Figure 20). Potential implications of the greater reach of tidal influence are changes to sediment dynamics and hydrologic conditions as a result of tidal fluctuations.

Dam removal will also impact the range of tidal freshwater wetlands, important rare wetlands formed near the limits of the tidal range. Because fresh water is less dense than salt water, fresh water tends to flow on top of salt water as the salt water moves upstream with an incoming tide. The water surface elevation rises and falls with the tide, but the river banks and vegetation community interact primarily with the portion of the water column that is fresh water. The mixing dynamics within the tidal range of the Ipswich River are unknown; however, with the dam removed and the range of tidal influence increased, tidal freshwater wetlands may be able to expand their range within the study area.

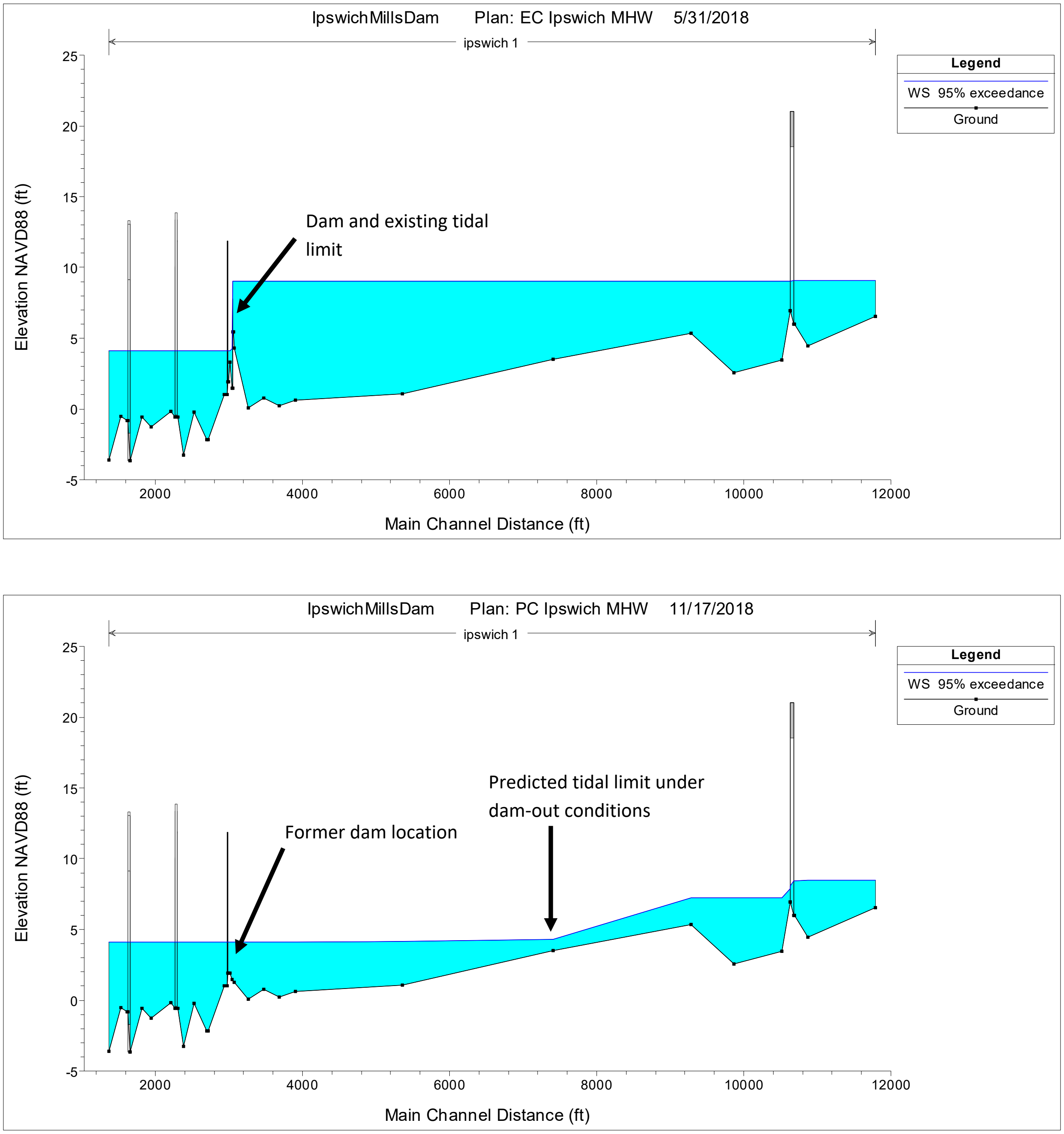

Figure 19. Predicted low-flow water surface profiles for existing (top) and dam-out (bottom) conditions during high tide

Figure 20. Comparison of the existing tidal limit at the dam and the predicted tidal limit under modeled dam-out conditions

INFRASTRUCTURE

Our assessment of hydraulic impacts at key infrastructure utilizes simulations of low tide when hydraulic conditions (i.e. flow velocities and shear stresses) are anticipated to have the highest scour potential. The modeled dam-out geometry assumes that bed levels immediately upstream of the dam would evolve or be regraded such that the gradient of the channel through the former dam location would approximate that of upstream and downstream reaches. Under this conservative assumption, bed levels at and immediately upstream of the current spillway could be reduced by as much as 7 feet in elevation (from approximately 8 feet to approximately 1 foot) with a similar magnitude drop in water surface elevation during low flows (Table 6 and Figure 21). It is likely that the greatest change would occur at and around the thalweg. At the former spillway location, it is unclear whether such a reduction in bed levels alone (i.e., without taking into account changes to local hydraulics) would threaten the stability of adjacent river walls. This would need to be considered, foundation depths investigated, and measures put in place to prevent undermining as necessary.

Average channel velocities at cross sections bounding the former dam site (Sta 3041 at the toe of the existing dam and Sta 3072 located 21 feet upstream of the existing spillway) are predicted to decrease during the 100-year flood event and increase during the 2-year flood event (Table 7). We examined detailed cross section model output for the 2-year event and found that bed shear stresses are predicted to increase along the channel sides at both cross sections as a result of dam removal. At Sta 3041, the increase is approximately 25% to 0.3 lb/ft^2 and at Sta 3072, the increase is approximately 100% to 0.2 lb/ft^2. Values predicted for pre- and post-removal scenarios remain within the range of shear stresses for mobilizing gravel smaller than 0.8 inch in diameter. It should be noted that for the existing conditions, the one-dimensional model does not adequately capture turbulence at the base of the dam so actual shear stress conditions may surpass model predictions. Similarly, model predictions may underestimate hydraulic forces associated with impinging flow against the river retaining wall and abutments supporting the pedestrian area and parking lot on river left and the retaining wall on river right in the absence of the dam.

Upstream along the margins of the lower impoundment, sediment has accumulated adjacent to existing river retaining walls and the EBSCO building (Sta 3260) and has been colonized by wetland vegetation. Predicted changes in water surface elevations (Table 6 and Figure 21) suggest that these areas where ground levels are 7 and 8 feet in elevation will be above the 2-year flood water surface after the dam is removed. Provided this material remains in place and continues to support vegetation growth following dam removal, it will define a new bankfull cross section width approximately 40 feet narrower than the former impoundment. It may also help to buffer adjacent infrastructure, potentially including some areas of the EBSCO building foundations, from direct hydraulic forces and undermining. However, with the change in base level adjacent to the accumulated sediment, some sloughing of the material could occur with evacuation of impounded sediment elsewhere. The change in hydraulic and hydrological conditions may also affect rates of erosion through fluvial shear and the establishment and continued growth of vegetation.

Table 6. Predicted water surface elevations at low tide for existing and dam-out conditions (ft NAVD88)

	100-year flow and normal depth		25-year flow and normal depth		2-year flow and normal depth	
River station	Existing (ft)	Dam Removed (ft)	Existing (ft)	Dam Removed (ft)	Existing (ft)	Dam Removed (ft)
11787	21.29	21.29	17.07	17.07	13.00	13.00
10867	21.31	21.31	16.96	16.96	12.79	12.79
10689	20.58	20.58	16.04	16.04	12.33	12.32
10657	Railroad bridge					
10625	15.71	15.59	11.46	11.46	11.85	9.75
10513	16.76	16.67	15.20	14.62	12.04	10.94
9865	16.64	16.55	15.10	14.49	11.99	10.84
9283	16.37	16.27	14.88	14.20	11.89	10.62
7408	15.69	15.56	14.28	13.20	11.59	10.62
5359	15.21	15.04	13.90	12.48	11.44	8.26
3900	14.69	14.49	13.54	11.74	11.34	7.07
3682	14.64	14.43	13.50	11.66	11.33	6.93
3469	14.59	14.38	13.46	11.51	11.31	6.63
3260 (EBSCO building)	14.51	14.29	13.40	11.38	11.30	6.48
3072	14.24	14.21	13.15	11.30	11.24	6.36
3063	14.19	-	13.06	-	11.22	-
3051 (Dam)	14.19	-	13.06	-	11.22	-
3041	14.19	14.15	11.22	11.13	7.03	6.28
3020	14.07	14.13	10.74	11.08	6.05	6.22
2998	14.11	14.11	10.98	10.98	6.09	6.09
2990	Pedestrian bridge					
2934	14.08	14.08	10.93	10.93	6.06	6.06

Average velocities at cross sections through the impoundment are predicted to increase following dam removal (Table 7) with the greatest changes predicted to occur during lower magnitude events when the channel is not experiencing backwater effects from the Choate Bridge. However, average bed shear stresses through the impoundment remain low, below 0.36 lb/ft^2 up through the 100-year event (Figure 22), corresponding with a shear stress sufficient to mobilize coarse gravel up to 0.9 inch in diameter.

Average bed shear stresses likely underestimate actual local shear stresses at the toe of the bank on the outside of meander bends. This may be a consideration at the EBSCO building (Sta 3260) where the river bends slightly to the right. Detailed model output at Sta 3260 predicts that during the 2-

year flood event, bed shear stress at the toe of the left bank will increase from 0.04 lb/ft^2 to 0.28 lb/ft^2 (i.e., conditions likely to mobilize very fine to fine gravel). Maximum predicted post-removal bed shear stresses for all events modeled are between 0.3 and 0.4 lb/ft^2 at Sta 3260. However, the model does not represent all the forces acting on flow around meander bends. Empirical data from other lowland rivers suggests that local bed shear stress at the outside of a meander bend may be up to twice the magnitude of the cross-sectionally averaged value as a result of centrifugal forces that push the core of high-velocity flow towards the outer bank creating a steep velocity gradient near the boundary. Thus, maximum local shear stresses around the bend may reach up to 0.8 lb/ft^2 which is within the range to mobilize very coarse gravel up to 2 inches in diameter.

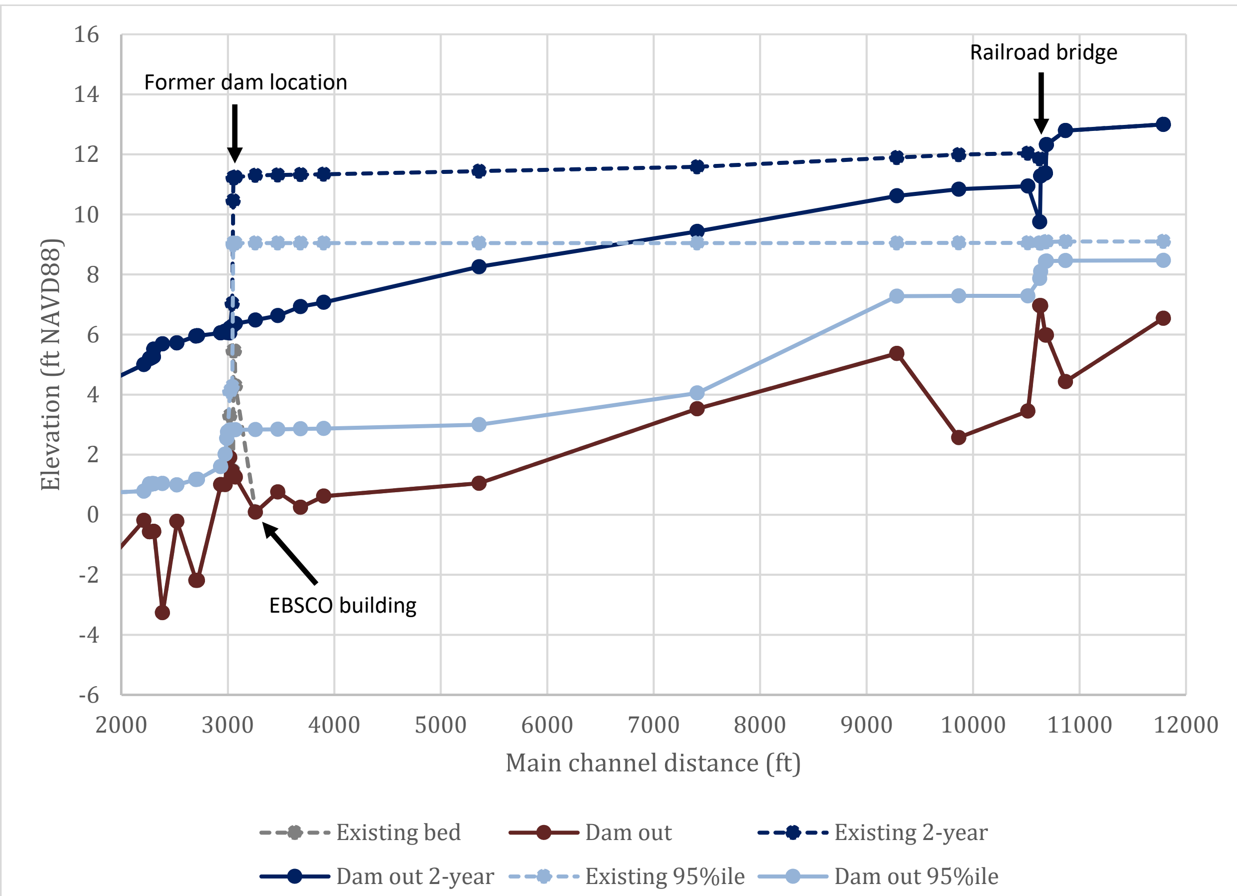

Figure 21. Existing and predicted dam-out water surface profiles for the 2-year flood event (1,439 cfs) and 95% exceedance probability flow (period March through June; 47 cfs) during low tide conditions

Table 7. Predicted average channel velocities at low tide for existing and dam-out conditions

	100-year flow and normal depth		25-year flow and normal depth		2-year flow and normal depth	
River station	**Existing (ft/s)**	**Dam Removed (ft/s)**	**Existing (ft/s)**	**Dam Removed (ft/s)**	**Existing (ft/s)**	**Dam Removed (ft/s)**
11787	4.48	4.48	4.92	4.92	3.31	3.31
10867	1.68	1.68	2.06	2.06	1.46	1.46
10689	6.51	6.51	7.22	7.22	4.93	4.93
10657 (Railroad bridge)	7.85	7.85	12.76	12.76	8.70	8.70
10625	10.48	10.66	17.06	17.06	4.75	11.99
10513	2.88	2.91	2.45	2.71	1.34	1.68
9865	2.28	2.30	1.86	2.03	2.28	1.22
9283	3.52	3.57	2.90	3.24	1.57	2.16
7408	3.01	3.07	2.54	3.13	1.33	2.16
5359	2.40	2.44	1.88	2.32	0.87	1.85
3900	3.88	3.98	3.00	3.91	1.32	3.10
3682	3.38	3.46	2.58	3.34	1.11	2.60
3469	3.13	3.24	2.53	3.66	1.19	3.54
3260 (EBSCO building)	3.08	3.21	2.51	3.46	1.10	2.52
3072	4.57	3.23	4.04	3.20	1.81	2.38
3063	4.89	-	4.60	-	2.14	-
3051 (Dam)	-	-	-	-	-	-
3041	3.99	3.90	5.04	4.34	2.96	3.06
3020	4.94	3.99	7.13	4.53	7.97	3.37
2998	4.09	4.09	5.01	5.01	4.02	4.02
2990 (Pedestrian bridge)	3.32	3.32	5.08	5.08	4.06	4.06
2934	3.58	3.58	4.61	4.61	3.23	3.32

It is recommended that the potential risks to infrastructure in the lower impoundment continue to be evaluated and managed proactively in subsequent project phases, either through monitoring and contingency planning, or through focused stabilization activities. At the EBSCO building in particular, bioengineering bank stabilization is recommended to stabilize impounded sediment currently deposited along the margins of the channel between the thalweg and the building foundations.

Immediately downstream of the railroad bridge, flow velocities during existing and dam-out conditions are high along the steep face of the existing rock on the bed of the channel where the model predicts supercritical flow conditions. Predicted shear stress results suggest that this location is susceptible to scour in both scenarios. Refinement of the modeling in this area with higher resolution data and additional cross sections may help to improve the accuracy of modeling results in this area of rapidly varied flow; however, the result that this area is susceptible to scour both currently and in the event of dam removal is unlikely to change. Further consideration of the risk of scour and potential impacts to the bridge should be incorporated into future design phases to determine whether remedial measures should form part of the dam removal scope.

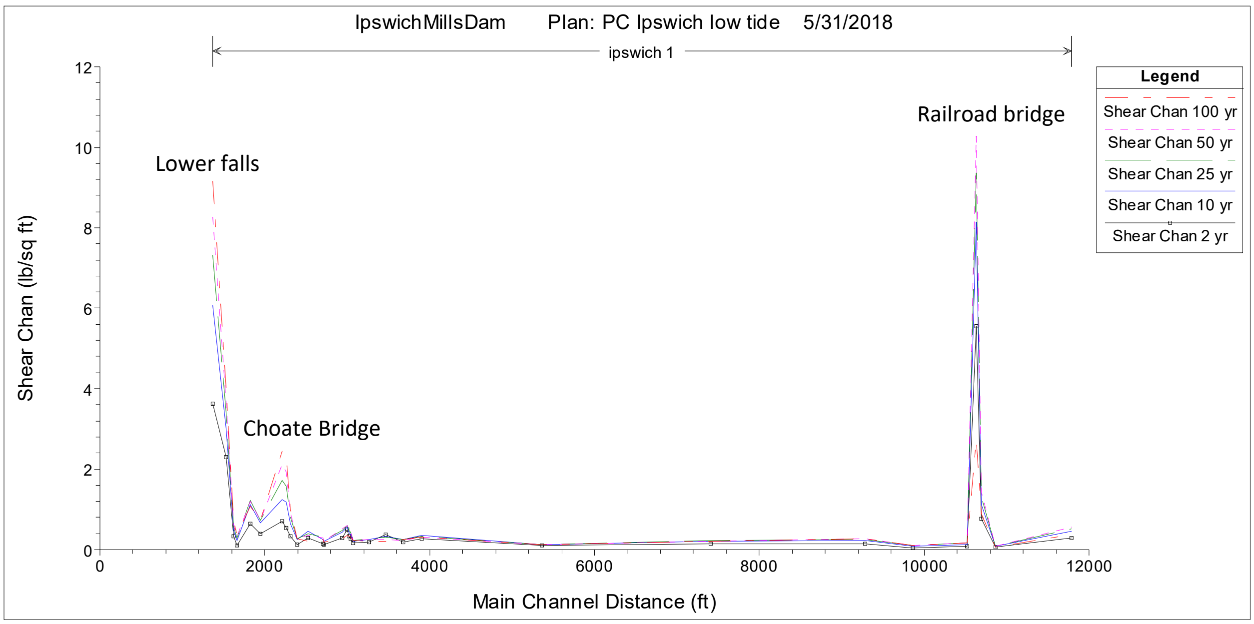

Figure 22. Predicted average bed shear stress in the channel at low tide for the dam-out condition

Other local peaks in averaged bed shear stress are predicted to occur elsewhere where the bed surface is steep such as at the Choate Bridge and across lower falls. However, predicted post dam removal hydraulic conditions at these locations are the same as for existing conditions as explained below, so no additional infrastructure risk as a result of dam removal has been identified using the methods employed in this study.

Downstream of the dam, predicted water levels, flow velocities, and shear stresses with the dam removed remain the same as for the existing condition except where bed levels were modified (Sta 3020).

FISH PASSAGE

Model results indicate that predicted water surface profiles and flow velocities through the former dam location during low flows will be favorable to fish passage (Figure 23 and

; Table 5 in Turek et al., 2016 for comparison with fish passage requirements by species). The flows modeled were those calculated by taking into account records over the entire migration period from March through June. Predicted average flow velocities are less than 4 ft/s, and maximum flow depths are greater than 0.5 feet at all of the cross sections in the immediately vicinity of the dam removal. At high tide, tide levels will extend past the former dam location as discussed previously, and no issues with fish passage are therefore anticipated.

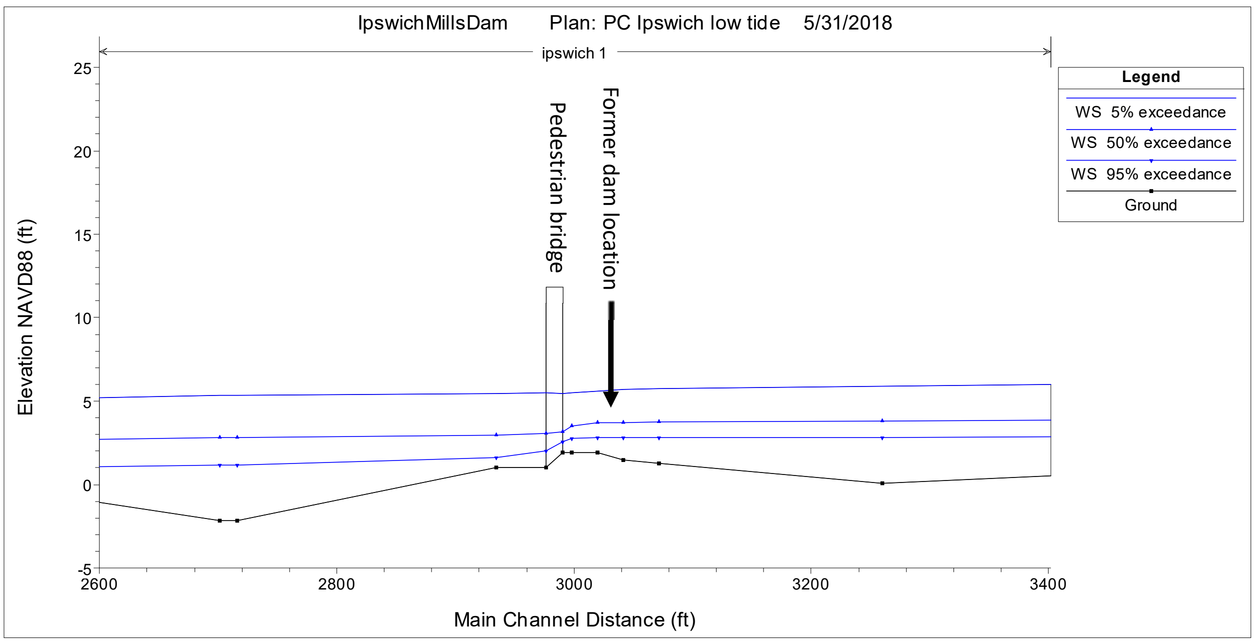

Figure 23. Predicted water surface profiles through the former dam site during fish passage flows and under low tide conditions

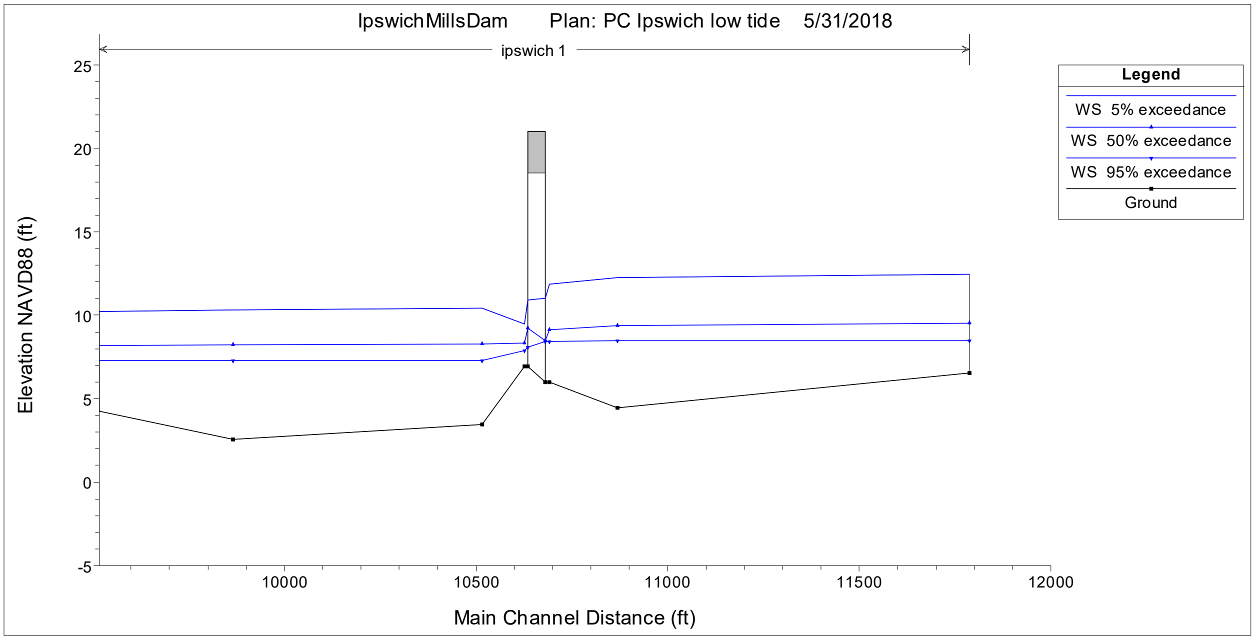

Figure 24. Predicted water surface profiles at the railroad bridge during fish passage flow and under low tide conditions

The effects of dam removal on low flow water surface profiles are predicted to extend upstream to the railroad bridge (Figure 24). Compared with existing conditions, water depths are shallower and average flow velocities are predicted to increase, particularly immediately downstream of the bridge as flow passes over the high spot where rock has been placed on the bed of the channel (Sta 10625) (). Modeling indicates supercritical flow at this location with a hydraulic jump forming immediately downstream as flow transitions back to subcritical. The results suggest that depending on the design, dam removal may make fish passage conditions more challenging at the railroad bridge than they are at present. Irregularities in the rock bed at the bridge may provide diverse flow conditions and opportunities for passage over this relatively short distance; however, it is recommended that fish passage conditions continue to be evaluated and optimized at this location as the project moves into later stages of design.

Table 8. Predicted cross-sectionally averaged velocities during fish passage flows during low tide conditions

	5% exceedance		50% exceedance		95% exceedance	
River station	Existing (ft/s)	Dam Removed (ft/s)	Existing (ft/s)	Dam Removed (ft/s)	Existing (ft/s)	Dam Removed (ft/s)
11787	2.94	02.94	1.27	1.77	0.35	0.52
10867	1.30	1.30	0.54	0.70	0.13	0.15
10689	4.57	4.57	2.32	3.85	0.63	0.95
10657	Railroad bridge					
10625	4.34	11.35	2.41	8.17	0.64	3.35
10513	1.17	1.50	0.43	0.64	0.08	0.14
9865	0.84	1.10	0.32	0.48	0.06	0.11
9283	1.38	2.01	0.58	1.42	0.13	0.61
7408	1.14	1.99	0.40	1.77	0.08	3.35
5359	0.74	1.83	0.25	1.48	0.05	0.57
3900	1.11	2.84	0.36	1.49	0.07	0.43
3682	0.93	2.37	0.30	1.19	0.05	0.33
3469	0.99	3.30	0.32	1.75	0.06	0.52
3260	0.91	2.34	0.28	1.14	0.05	0.29
3072	1.53	2.21	0.51	1.12	0.10	0.31
3051	Former dam location					
3041	2.55	2.84	0.94	1.48	0.20	0.43
3020	7.39	3.19	4.78	1.88	3.12	0.64
2998	3.93	3.93	8.23	3.19	1.33	1.66
2990	Pedestrian bridge					
2934	3.01	3.01	2.32	2.32	3.25	3.25
2717	2.02	2.02	1.17	1.17	0.58	0.58

TRIBUTARIES

Three tributaries enter the impoundment: Kimball Brook and Salton Brook about 1,300 feet upstream of the dam, an unnamed intermittent stream via a culvert under the railroad embankment and an outlet upstream of the impoundment, and the Miles River at the upstream limit of the impoundment. Low-flow water surface elevations near the Kimball Brook and Salton Brook confluences are predicted to decrease by up to 3 to 6 feet upon removal of the dam based on the assumptions made for this study (Figure 25). The confluences are close to the lower impoundment where depth-of-refusal data suggests that thalweg bed levels in the main Ipswich River channel may not change substantially following dam removal. However, headcutting through the newly formed banks along the margins of the impoundment and up the brooks is a risk. Sediment contribution from these smaller tributaries is likely to be small; however, potential impacts of incision should be assessed further in future project phases.

Farther upstream, an MBTA culvert is present through the railroad embankment at the Shady Creek Conservation area (Figure 4 and Figure 25). The culvert is a 3-foot by 3-foot granite masonry box at its downstream end that has been extended using twin 12-inch-diameter vitrified clay pipes at its upstream end. The downstream invert of the culvert is 7.63 feet according to 2015 Peer Consultants, PC design plans for the upstream extension, which were obtained by IRWA for this study. The MBTA culvert is one of two outlets for a small (0.23-square-mile drainage area) intermittent stream. The other is a direct connection to the Ipswich River upstream of the railroad bridge. According to StreamStats, 35% of the stream's drainage area is wetlands, some of which form a buffer between the MBTA culvert outlet and the Ipswich River. The small drainage area and dense vegetative land cover surrounding the stream suggest that the area is inundated by Ipswich River flows during high magnitude events and likely drains slowly as floodwaters recede. No observations from aerial photographs or the field (as observed by IRWA or Horsley Witten) indicate that the culvert regularly conveys large volumes of water. As such, flow velocities along the stream and through the culvert are likely low. Hydraulic modeling results suggest that the culvert invert is currently at or just below water surface elevation in the Ipswich River during low flow. With the dam removed, modeling predicts that river levels will drop to approximately 5 feet, or approximately 2.5 feet below the invert of the culvert outlet. However, we consider the risk of headcutting affecting the culvert to be low as a result of low flow rates and the presence of a densely vegetated buffer between the river and culvert. Post-project monitoring is recommended to capture future changes that may require management.

The risk of substantial headcutting and bank erosion along the Miles River is also low given its location at the upstream limit of the impoundment; however, there is still some uncertainty around the scale of bed level adjustment along the Ipswich River near the confluence. Impacts along all tributaries, including fish passage, should be given further consideration as the dam removal design is progressed.

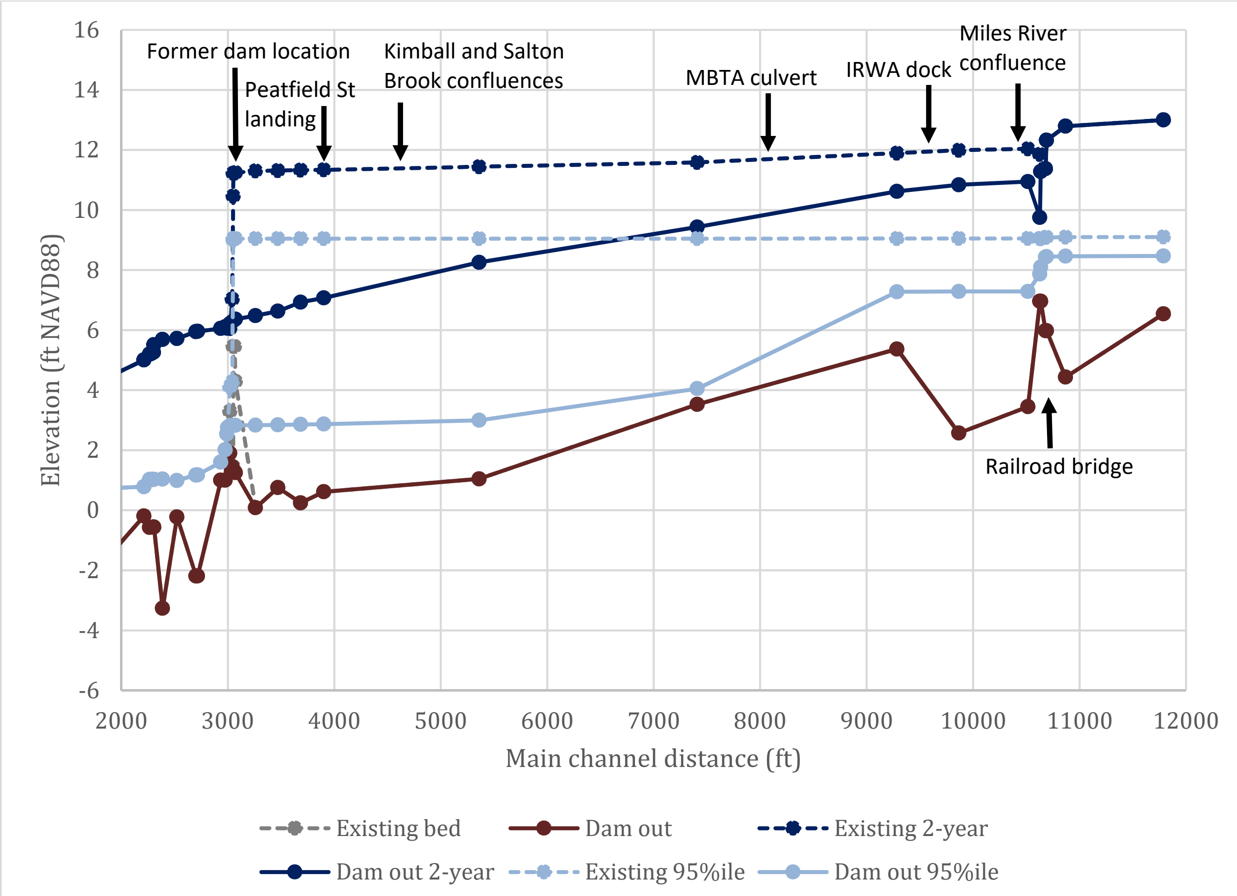

Figure 25. Existing and predicted dam-out water surface profiles for the 2-year flood event (1,439 cfs) and 95% exceedance probability flow (period March through June; 47 cfs) during low tide conditions showing locations of confluences and recreational assets

SALLY'S POND

Lowering of impoundment levels will affect local groundwater level which may result in a lowering of water levels within Sally's Pond (refer to Figure 4 for pond location). The pond was dug by the Town of Ipswich in 1974 and is currently owned by the Ipswich Museum. It is unclear whether the pond and the river share a direct hydrologic connection. Drawings submitted with the original Notice of Intent show a pipe draining the pond towards South Main Street; however, utility records from the Town show a drain pipe leading directly from the pond into the river. We recommend additional field investigation of these pipe locations to better understand how the pond and river are hydrologically connected. Regardless of the influence of pipes, the two bodies of water are connected via groundwater flow paths.

ECOLOGY

Short-term impacts

Perhaps the most notable potential short-term impact of rapid removal of a dam is the release of sediment that has accumulated behind the structure. At the Ipswich Mills Dam site, depth-of-refusal survey suggests that a relatively small volume of sediment is present within the lower impoundment in the vicinity of the dam. Depths of accumulation along the channel thalweg are minimal with most of the material stored along the margins of the impoundment and partially vegetated. Further upstream, there appears to be fine sediment stored along the bed of the channel, but depths and sediment volume are unknown at this time.

Construction activity and breaching of the dam will mobilize some fine organic and inorganic sediment, which will be held in suspension resulting in short term/temporary increased turbidity downstream of the dam. The impact on aquatic species depends on the concentration, exposure time, and time of year. Suspended sediment occurring after every rainfall event in natural, stable streams does not produce mortality in fish, and laboratory experiments exposing fish to suspended sediment showed mortality only at extremely high concentrations (e.g., Bisson and Bilby, 1982; Berg and Northcote, 1985; Cordone and Kelley, 1961; Gradall and Swenson, 1982). Sessile communities like invertebrates are more susceptible to sediment impacts than fish which can adjust quickly to changes in turbidity and bedload. Further investigation into the volume of fine sediment stored over the whole length of the impoundment is necessary before short-term impacts can be fully assessed.

Timing the Ipswich Mills Dam removal to begin releasing sediment well ahead of fish migration periods will help to minimize impacts to migratory fish. Tidal exchange will also help to move sediment through the system, with some limited deposition likely in the Great Cove immediately downstream of County Street (approximately 0.3 mile downstream of the dam) where the channel is artificially widened and flow velocities reduced.

Long-term impacts

Wetland delineation by the Massachusetts Department of Environmental Protection (Mass DEP, 2009) shows areas of deep marsh, shallow marsh, wooded swamp, and shrub swamp bordering the main channel through the impounded reach upstream of Ipswich Mills Dam (Figure 26). Following dam removal, normal water levels will fall, and it is likely that shallow water wetland areas will evolve into a different type of wetland or upland habitat. Areas currently shown as deep marsh and existing backwater areas are likely to remain as shallow water wetland habitat. Vegetation cover and succession upstream of the dam will likely be affected by the increased tidal range upstream of the former dam location.

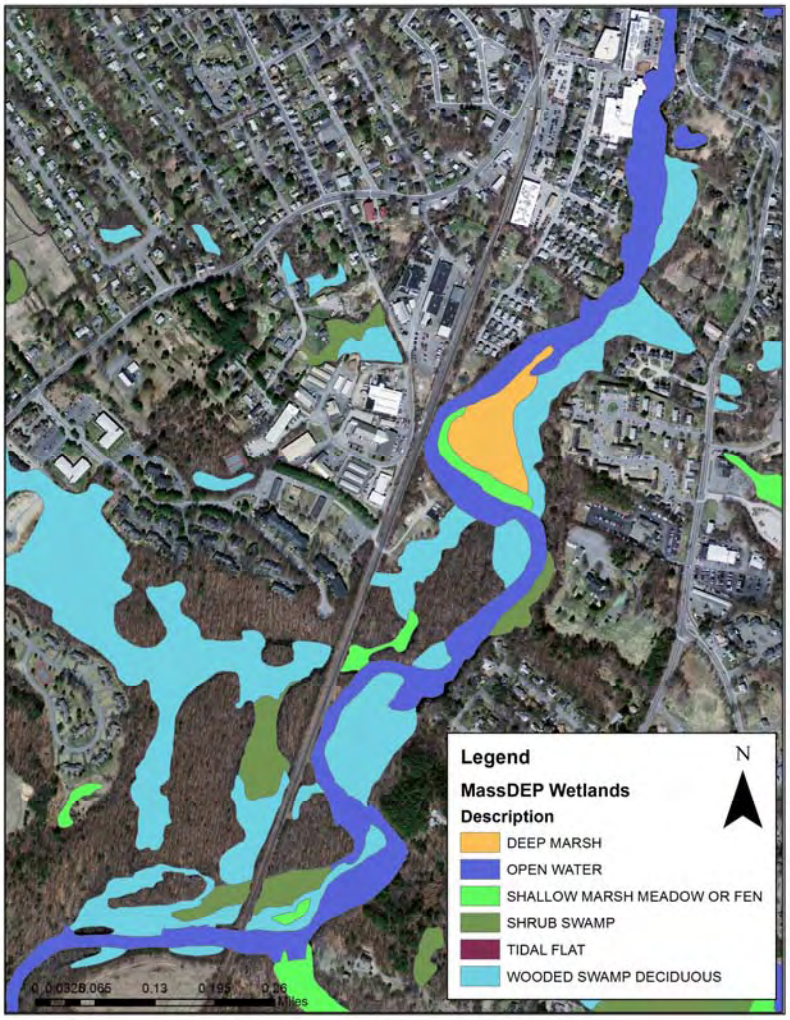

Figure 26. Mass DEP wetlands (Sources: USGS color ortho imagery, 2009; Mass DEP, 2009)

For typical small dams, removal results in a long-term decline in water temperatures through the former impoundment area and downstream of the dam (e.g., Pawloski and Cook, 1993). The narrowed cross section and increased velocity through the former impoundment area result in reduced residence time from the ponded condition. This combination equates to cooler temperatures and frequently higher dissolved oxygen concentrations resembling conditions of the stream upstream of the dam's influence (Zaidel, 2018). Decreased post-dam removal water temperatures favor those stream fishes adapted to cool or coldwater environments (Born et al., 1998). Removal of the dam will encourage active flow and help reduce water temperatures, making this part of the river more hospitable to coldwater fish species. Removal of the dam will also facilitate movement of other aquatic cool-water organisms past the dam site. Turtles, resident freshwater fish, and other aquatic organisms will have improved movement with the dam removed. As described earlier, the rare tidal freshwater wetland range may be able to be expanded following the dam removal.

Over the long term, sediment eroded from newly exposed banks, the bed, or supplied by headcutting and bank erosion along tributaries may be transported downstream. Restoration of more consistent sediment continuity will be beneficial over the long term not only for restoring habitat locally but also for replenishment of sediment in downstream reaches currently dominated by cobble-sized material. Water quality and habitat improvements coupled with restoration of aquatic organism passage at the dam site will have long-term ecosystem benefits that are expected to outweigh anticipated short-term impacts.

RECREATION

There are two major public boat access points through the impoundment: Peatfield St. landing (left bank) and, farther upstream, the IRWA dock (right bank) (Figure 25). The predicted change in the low-flow (95% exceedance probability flow) water surface elevation is approximately 6.2 feet at the Peatfield St. landing and 1.8 feet at the IRWA dock under the dam-out conditions assumed for this study. It is likely that modifications will be required at Peatfield St. to maintain access during periods of low flow following dam removal. The IRWA dock is location nearer to the upstream limit of the impoundment and predicted changes are less substantial although some uncertainty remains around potential changes in bed levels. It is anticipated, however, that lesser modifications will be necessary to maintain access at this location.

Based on the assumptions made for this study, it will be possible to paddle past the former dam site, creating a new opportunity for boats to pass directly from the existing boat launches downstream to the estuary. Even if bedrock is found beneath the dam at a higher elevation than assumed here, modeling suggests that the increased tidal range will help facilitate upstream and downstream movement at least twice a day during high tide. With the dam removed, boating hazards associated with the dam will be eliminated, though the bedrock may be challenging to navigate depending on the water levels and tide.

At the upstream end of the impoundment, portage may be required underneath the railroad bridge. Other high spots on the bed within the impoundment that could be exposed upon dam removal may also present challenges for paddlers and could require portage during very low flows. Overall, there is no evidence to suggest that the river through the former impoundment will not remain usable for paddlers. A primary impact of dam removal will be more variability in paddling conditions as flow levels vary with changes in discharge and tidal conditions. Impacts should be reconsidered as the design progresses with access modifications and portage provisions incorporated as necessary to allow for access over a range of flow and tidal conditions.

Conclusions

We have assembled existing data sources and developed a one-dimensional, steady-state HEC-RAS model to investigate the potential hydraulic implications of removing the Ipswich Mills Dam to the fullest vertical and lateral extent practical. For simulating post-dam removal conditions, we approximated the long-term bed profile through the former dam site by assuming it will align with the average upstream and downstream bed profiles.

Based on our assumed post-removal bed profile, the results of the model predict that the hydraulic tidal influence will extend an additional 0.8 mile upstream of its current limit at the toe of the dam. This change in sediment and flow dynamics may have an impact on vegetation type and growth along the channel margins in the lower impoundment. Although modeling predicts that flow conditions during low tide will be favorable for fish passage, the extended tidal range will further facilitate fish passage as well as boat passage past the former dam site.

Flood levels through the impoundment upstream to the railroad bridge are predicted to decrease as a result of dam removal. Upstream of the railroad bridge, flood levels are controlled by conveyance and bed levels through the bridge section. The bridge also represents the likely upstream extent of bed incision following dam removal. Survey documents fine sediment accumulation on the bed of the channel in the upper impoundment, and depth-of-refusal survey extending the full length of the impoundment is required to better assess the risk of incision and fine sediment mobilization and to estimate impounded sediment volume.

In the lower impoundment (to 430 feet upstream of the dam), depth-of-refusal survey is available and shows little sediment accumulated along the thalweg of the channel with greater depths of accumulation at the margins of the impoundment. The risk of substantial headcutting along the main river channel in this area is therefore low, but some material may be mobilized from the margins. Vegetation growth following a drop in normal water levels should help to stabilize marginal deposits in some places, although this effect may be tempered by local sloughing of material. At the Kimball Brook and Salton Brook confluences, there is a risk of headcutting beginning through these deposits and progressing up the brooks. The risk of incision and bank erosion along the Miles River should be lower given that the confluence is located near the upstream limit of the impoundment; however, this needs to be examined when more is known about the depth of impounded sediment stored on the bed of the main Ipswich River channel near the confluence.

Fine sediment that is released as a result of dam removal is likely to be dispersed by fluvial flows and tidal fluctuations in the downstream channel. The spatial and temporal scale of the impacts will depend on the volume of material and how rapidly it is released. Mobilization of very coarse and coarse gravel along the channel bed or banks following removal of the dam could be a potential issue for flow conveyance at the Choate Bridge, but impacts are likely to be temporary with material transported past the bridge during subsequent high flows. It is recommended that deposition in the downstream channel is monitored following dam removal.

A summary of anticipated geomorphic and predicted hydraulic risks at key infrastructure is provided in Table 9. Risks were primarily identified at and upstream of the former dam location. Downstream of the dam, modeling predicted no change from existing conditions except at Sta 3020 where the post-removal channel geometry had been modified. Additional risks may be identified or certain risks resolved as the project progresses through design and more information becomes available. Incorporation of measures to mitigate these risks should be developed in future design phases.

Table 9. Summary of risks and recommendations at key infrastructure

Location	Risks	Potential Implications	Recommendations
Channel downstream of former dam, particularly Choate Bridge	• Temporary deposition of sediment eroded from the impoundment, particularly coarse material	• Restricted conveyance at bridges	• Further investigation of impounded sediment volume • Post-dam removal monitoring and contingency planning
River retaining walls at former dam location and abutments supporting pedestrian and parking area on river left	• Exposure of walls, foundations, and abutments to hydraulic forces and impinging flow as a result of dam removal and lowering of bed levels	• Structural instability • Scour and undermining of walls and abutments	• Scour protection such as bank construction or placement of rock in front of walls and abutments • Structural investigation including foundation depths and stability; and/or • Incorporation of structural and/or additional scour mitigation into design and/or construction methods if necessary

Location	Risks	Potential Implications	Recommendations
River retaining walls through former impoundment	• Increased flow velocities and shear stresses through former impoundment • Erosion of impounded sediment along channel margins between thalweg and wall foundations and subsequent exposure of walls to hydraulic forces	• Scour or undermining	• Monitoring and contingency planning or focused stabilization
EBSCO building	• Increased flow velocities and shear stresses through former impoundment • Erosion of impounded sediment along channel margin between thalweg and EBSCO foundations and subsequent exposure of foundations to hydraulic forces	• Scour or undermining	• Proactive management of impoundment margin through bioengineering bank stabilization
Infrastructure along tributaries	• Reduction in water surface elevations • Unknown potential for bed incision along main Ipswich River channel • Headcutting through channel margin deposits and upstream along tributaries	• Scour or undermining • More limited access to boat launches	• Scour analysis • Further investigation of impounded sediment depth • Collection of additional data, including thalweg elevations, along tributaries • Incorporation of mitigation into design if necessary • Post-dam removal monitoring and contingency planning where risk is considered too low to warrant immediate mitigation

Location	Risks	Potential Implications	Recommendations
Railroad bridge	• Reduction in downstream water surface elevations and thus a steeper water surface profile along downstream face of bridge	• Scour and undermining of existing scour protection should incision occur immediately downstream of the bridge • More challenging fish passage conditions following dam removal	• Further investigation of impounded sediment depth, particularly in upper impoundment • Improved characterization of scour risk and fish passage conditions • Incorporation of mitigation into design if necessary

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Appendix A

PeakFQ Output

```
1
  Program PeakFq           U. S. GEOLOGICAL SURVEY             Seq.002.000
  Version 7.1        Annual peak flow frequency analysis      Run Date / Time
  3/14/2014                                                   01/04/2017 06:31

                         --- PROCESSING OPTIONS ---

                      Plot option         = None
                      Basin char output   = TAB-SEPARATED
                      Print option        = Yes
                      Debug print         = No
                      Input peaks listing = Long
                      Input peaks format  = WATSTORE peak file

                      Input files used:
                         peaks (ascii)  -
C:\Users\cconstantine\Desktop\Jobs\Ipswich\Streamflow data\PEAKS.INP
                         specifications -
C:\Users\cconstantine\Desktop\Jobs\Ipswich\Streamflow data\PKFQWPSF.TMP
                      Output file(s):
                         main -
C:\Users\cconstantine\Desktop\Jobs\Ipswich\Streamflow data\PEAKS.PRT
                         bcd  -
C:\Users\cconstantine\Desktop\Jobs\Ipswich\Streamflow data\PEAKS.BCD

1


  Program PeakFq           U. S. GEOLOGICAL SURVEY             Seq.001.001
  Version 7.1        Annual peak flow frequency analysis      Run Date / Time
  3/14/2014                                                   01/04/2017 06:31

               Station - 01102000  IPSWICH RIVER NEAR IPSWICH, MA


                    I N P U T   D A T A   S U M M A R Y

                 Number of peaks in record           =       45
                 Peaks not used in analysis          =        0
                 Systematic peaks in analysis        =       45
                 Historic peaks in analysis          =        0
                 Beginning Year                      =     1971
                 Ending Year                         =     2015
                 Historical Period Length            =        0
                 Generalized skew                    =    0.616
                      Standard error                 =    0.550
                      Mean Square error              =    0.303
                 Skew option                         =   WEIGHTED
```

PEAKS - Post-1970 PeakFQ results at gage.PRT

```
Gage base discharge                  =      0.0
User supplied high outlier threshold =   --
User supplied PILF (LO) criterion    =   --
Plotting position parameter          =     0.00
Type of analysis                       BULL.17B
PILF (LO) Test Method                      GBT
Perception Thresholds           =   Not Applicable
Interval Data                   =   Not Applicable
```

```
*********  NOTICE  --  Preliminary machine computations.        *********
*********  User responsible for assessment and interpretation.  *********

  WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                  0.0
  WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.           222.2
  WCF163I-NO HIGH OUTLIERS OR HISTORIC PEAKS EXCEEDED HHBASE.     6885.6
```

Kendall's Tau Parameters

	TAU	P-VALUE	MEDIAN SLOPE	No. of PEAKS
SYSTEMATIC RECORD	0.169	0.104	13.728	45

1

```
Program PeakFq           U. S. GEOLOGICAL SURVEY          Seq.001.002
Version 7.1        Annual peak flow frequency analysis    Run Date / Time
3/14/2014                                                 01/04/2017 06:31
```

Station - 01102000 IPSWICH RIVER NEAR IPSWICH, MA

ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

	FLOOD BASE		LOGARITHMIC		
	DISCHARGE	EXCEEDANCE PROBABILITY	MEAN	STANDARD DEVIATION	SKEW
SYSTEMATIC RECORD	0.0	1.0000	3.0923	0.2734	0.000
BULL.17B ESTIMATE	0.0	1.0000	3.0923	0.2734	0.168

BULL.17B ESTIMATE OF MSE OF AT-SITE SKEW   0.1138

ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	<-- FOR BULLETIN 17B ESTIMATES --> VARIANCE OF EST.	95% CONFIDENCE INTERVALS LOWER	UPPER
0.9950	270.0	244.4	----	187.4	353.2
0.9900	309.2	285.9	----	220.1	397.8
0.9500	453.0	439.1	----	345.1	557.8
0.9000	558.8	552.0	----	440.4	674.0
0.8000	724.9	728.1	----	592.8	856.9
0.6667	929.9	943.0	----	781.3	1087.0
0.5000	1215.	1237.	----	1038.0	1421.0
0.4292	1360.	1384.	----	1165.0	1598.0
0.2000	2089.	2101.	----	1768.0	2551.0
0.1000	2801.	2772.	----	2317.0	3566.0
0.0400	3859.	3724.	----	3089.0	5183.0
0.0200	4767.	4507.	----	3722.0	6650.0
0.0100	5782.	5351.	----	4408.0	8362.0
0.0050	6915.	6261.	----	5153.0	10350.0
0.0020	8617.	7573.	----	6238.0	13460.0

1

Program PeakFq U. S. GEOLOGICAL SURVEY Seq.001.003
Version 7.1 Annual peak flow frequency analysis Run Date / Time
3/14/2014 01/04/2017 06:31

Station - 01102000 IPSWICH RIVER NEAR IPSWICH, MA

I N P U T D A T A L I S T I N G

WATER YEAR	PEAK VALUE	PEAKFQ CODES	REMARKS
1971	762.0		
1972	1290.0		
1973	1290.0		
1974	589.0		
1975	727.0		
1976	943.0		
1977	1150.0		
1978	1420.0		
1979	2110.0		

1980	480.0
1981	1250.0
1982	2280.0
1983	1930.0
1984	1940.0
1985	289.0
1986	1100.0
1987	3550.0
1988	524.0
1989	550.0
1990	821.0
1991	682.0
1992	505.0
1993	2510.0
1994	1520.0
1995	791.0
1996	1170.0
1997	3120.0
1998	1950.0
1999	798.0
2000	1460.0
2001	3040.0
2002	522.0
2003	1000.0
2004	2420.0
2005	1450.0
2006	4600.0
2007	1930.0
2008	1570.0
2009	1120.0
2010	3950.0
2011	1550.0
2012	619.0
2013	785.0
2014	1450.0
2015	1830.0

Explanation of peak discharge qualification codes

PeakFQ CODE	NWIS CODE	DEFINITION
D	3	Dam failure, non-recurrent flow anomaly
G	8	Discharge greater than stated value
X	3+8	Both of the above
L	4	Discharge less than stated value
K	6 OR C	Known effect of regulation or urbanization

PEAKS - Post-1970 PeakFQ results at gage.PRT

H 7 Historic peak

- Minus-flagged discharge -- Not used in computation
  -8888.0 -- No discharge value given
- Minus-flagged water year -- Historic peak used in computation

1

Program PeakFq U. S. GEOLOGICAL SURVEY Seq.001.004
Version 7.1 Annual peak flow frequency analysis Run Date / Time
3/14/2014 01/04/2017 06:31

Station - 01102000 IPSWICH RIVER NEAR IPSWICH, MA

EMPIRICAL FREQUENCY CURVES -- WEIBULL PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	SYSTEMATIC RECORD	B17B ESTIMATE
2006	4600.0	0.0217	0.0217
2010	3950.0	0.0435	0.0435
1987	3550.0	0.0652	0.0652
1997	3120.0	0.0870	0.0870
2001	3040.0	0.1087	0.1087
1993	2510.0	0.1304	0.1304
2004	2420.0	0.1522	0.1522
1982	2280.0	0.1739	0.1739
1979	2110.0	0.1957	0.1957
1998	1950.0	0.2174	0.2174
1984	1940.0	0.2391	0.2391
1983	1930.0	0.2609	0.2609
2007	1930.0	0.2826	0.2826
2015	1830.0	0.3043	0.3043
2008	1570.0	0.3261	0.3261
2011	1550.0	0.3478	0.3478
1994	1520.0	0.3696	0.3696
2000	1460.0	0.3913	0.3913
2005	1450.0	0.4130	0.4130
2014	1450.0	0.4348	0.4348
1978	1420.0	0.4565	0.4565
1972	1290.0	0.4783	0.4783
1973	1290.0	0.5000	0.5000
1981	1250.0	0.5217	0.5217
1996	1170.0	0.5435	0.5435
1977	1150.0	0.5652	0.5652

2009	1120.0	0.5870	0.5870
1986	1100.0	0.6087	0.6087
2003	1000.0	0.6304	0.6304
1976	943.0	0.6522	0.6522
1990	821.0	0.6739	0.6739
1999	798.0	0.6957	0.6957
1995	791.0	0.7174	0.7174
2013	785.0	0.7391	0.7391
1971	762.0	0.7609	0.7609
1975	727.0	0.7826	0.7826
1991	682.0	0.8043	0.8043
2012	619.0	0.8261	0.8261
1974	589.0	0.8478	0.8478
1989	550.0	0.8696	0.8696
1988	524.0	0.8913	0.8913
2002	522.0	0.9130	0.9130
1992	505.0	0.9348	0.9348
1980	480.0	0.9565	0.9565
1985	289.0	0.9783	0.9783

1

End PeakFQ analysis.
Stations processed : 1
Number of errors : 0
Stations skipped : 0
Station years : 45

Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4, or *.)
(2, 4, and * records are ignored.)

For the station below, the following records were ignored:

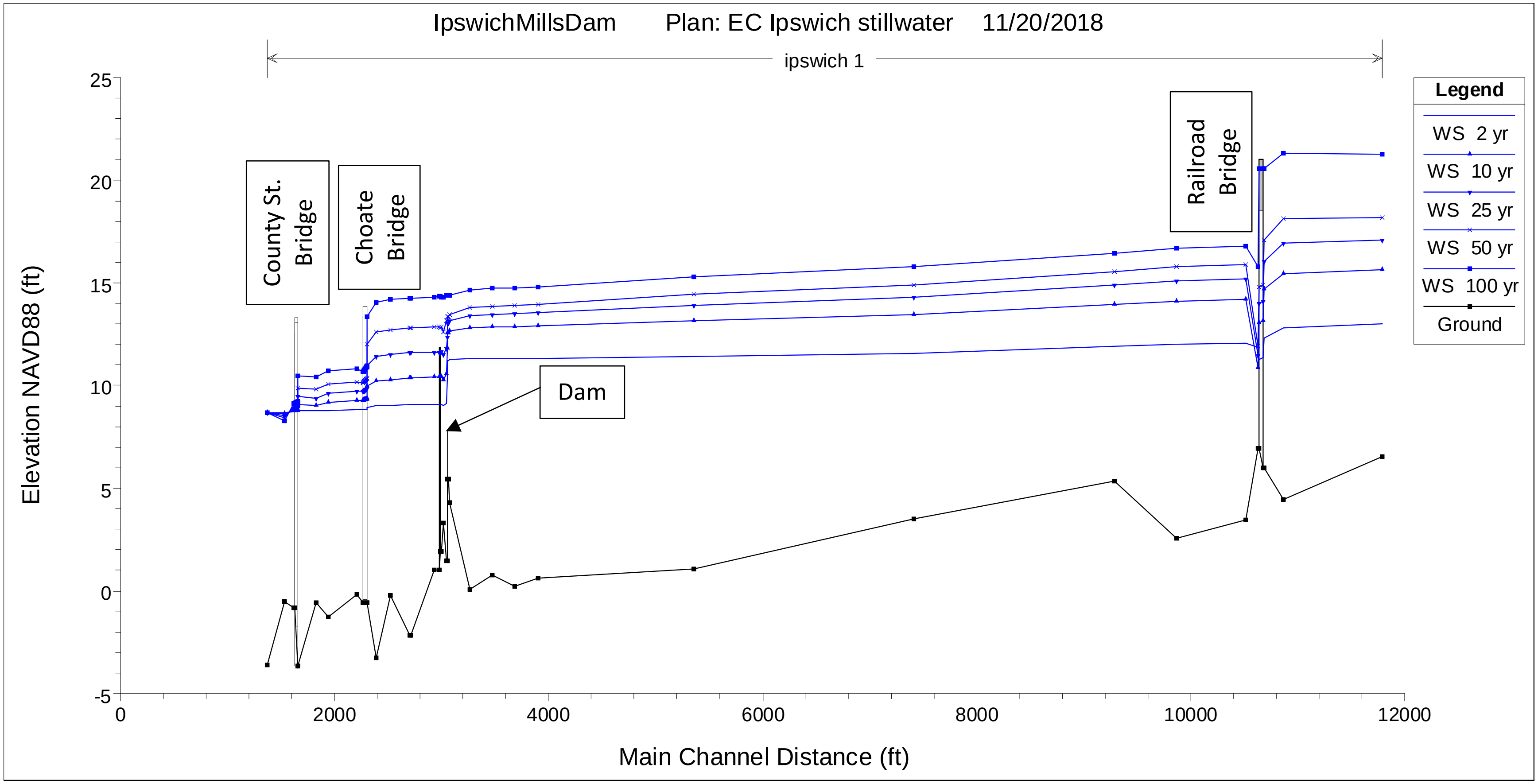
FINISHED PROCESSING STATION: 01102000 USGS IPSWICH RIVER NEAR IPSWICH, M

For the station below, the following records were ignored:

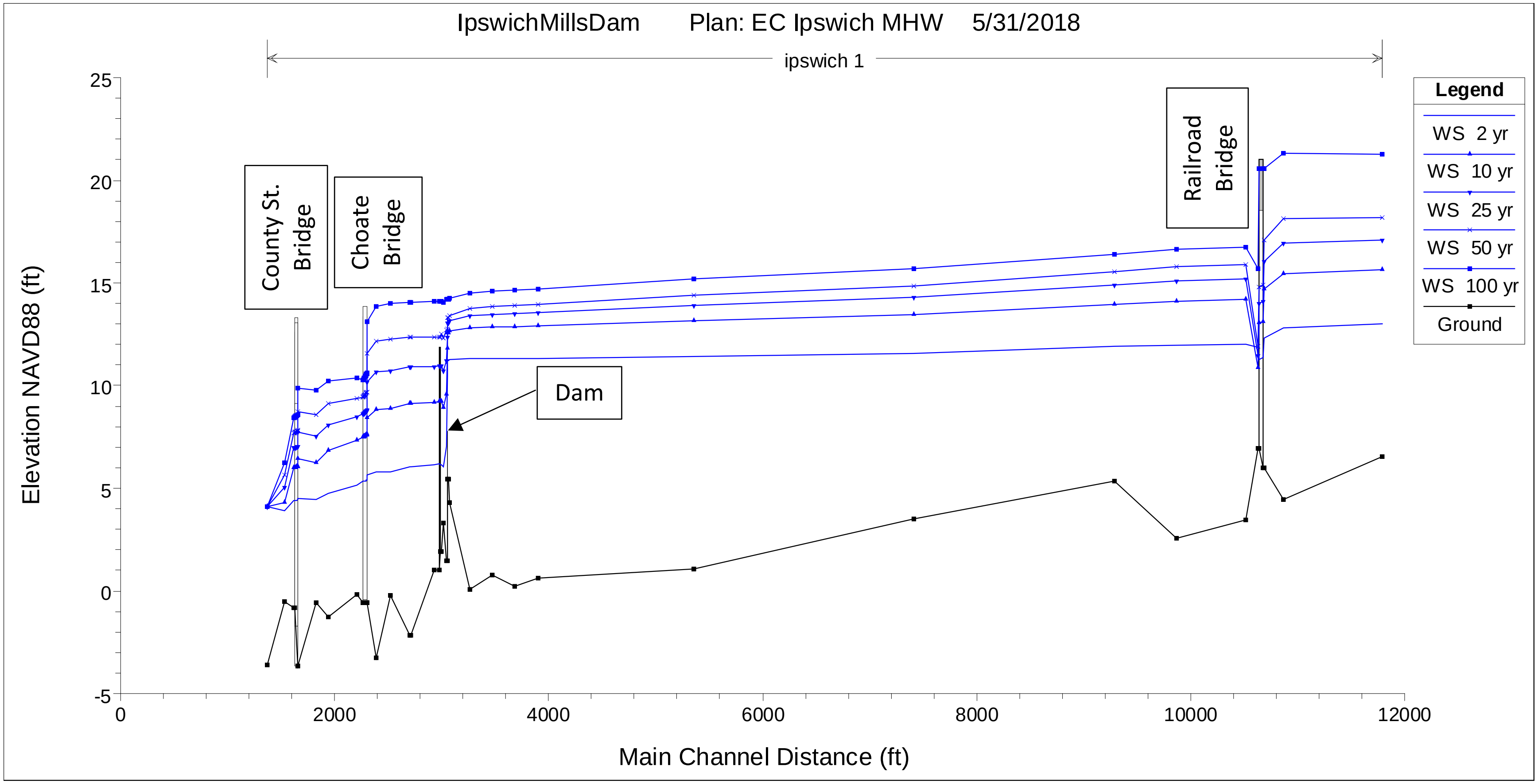
FINISHED PROCESSING STATION:

Appendix B

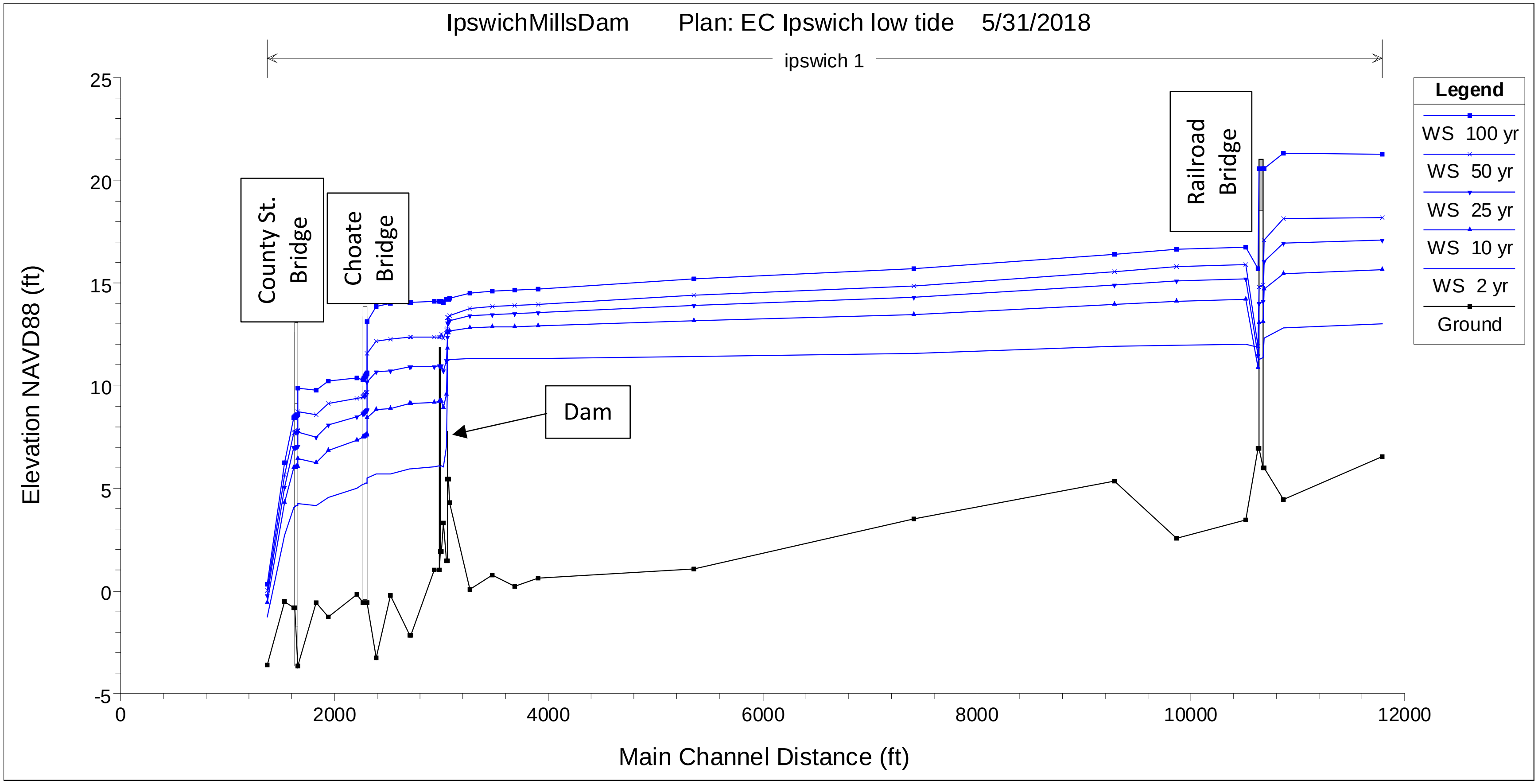
Flood Profiles



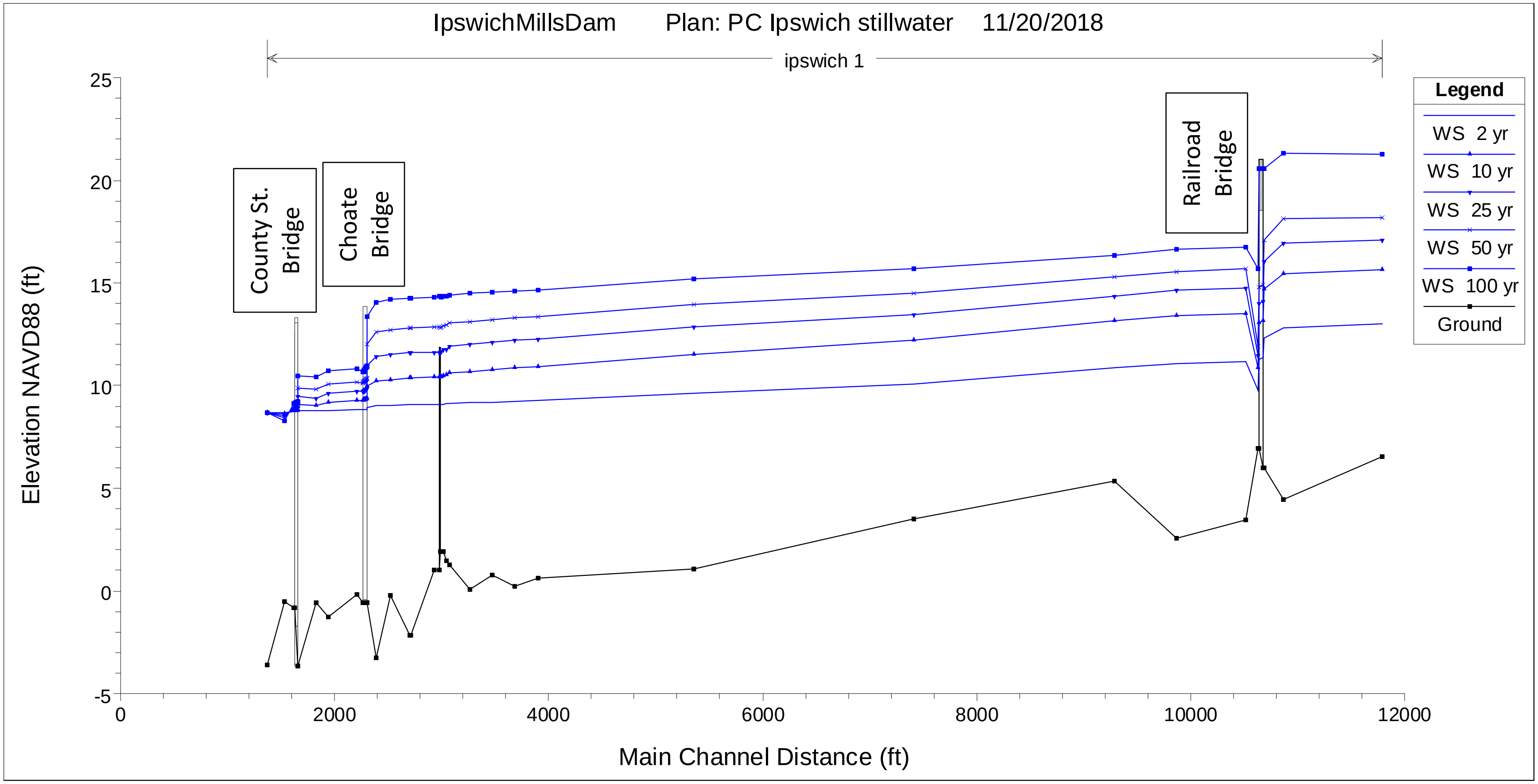
B.1. Existing conditions – Stillwater tide downstream boundary condition (8.7 feet NAVD88)



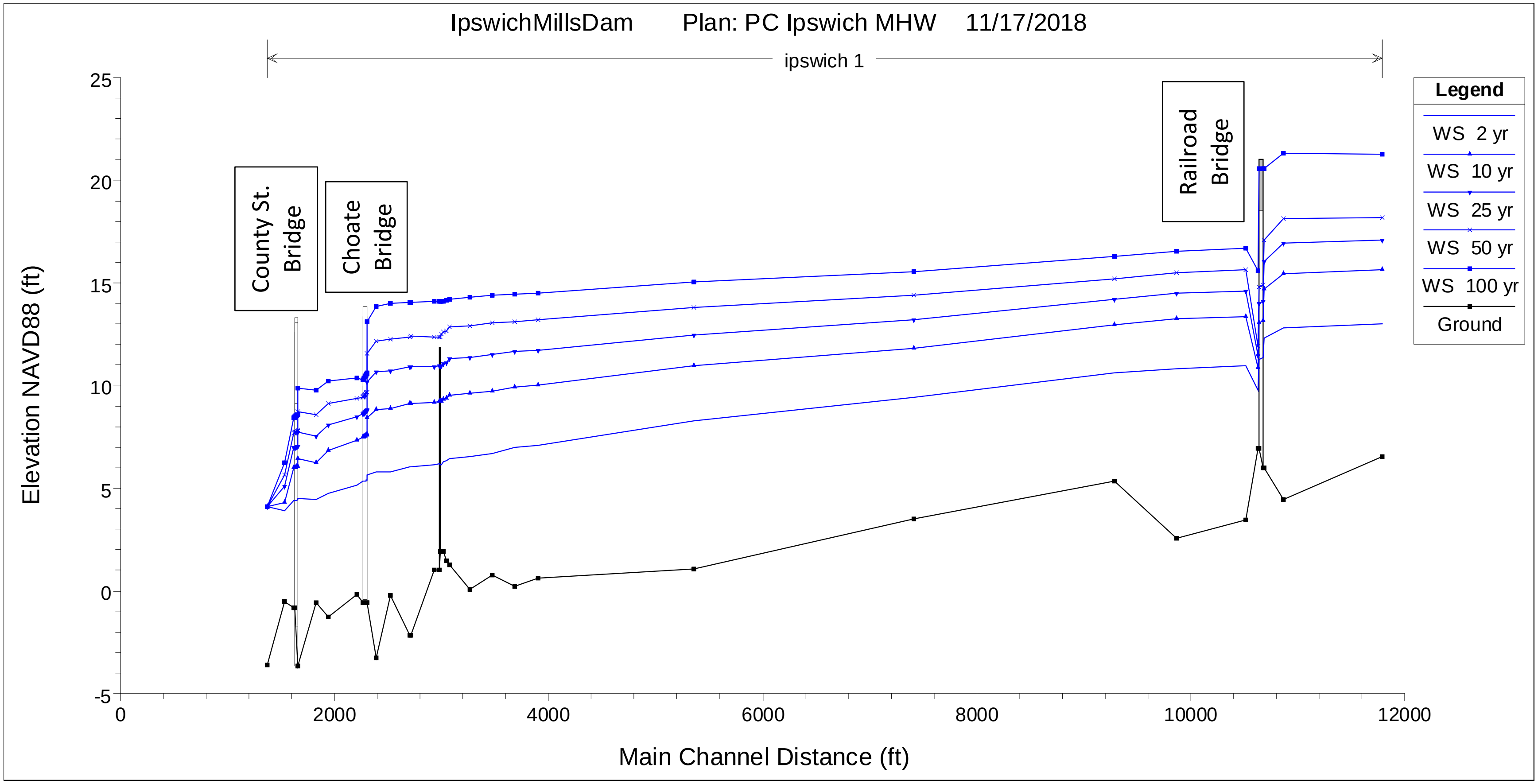
B.2. Existing conditions – Mean High Water (MHW) tide downstream boundary condition (4.1 feet NAVD88)



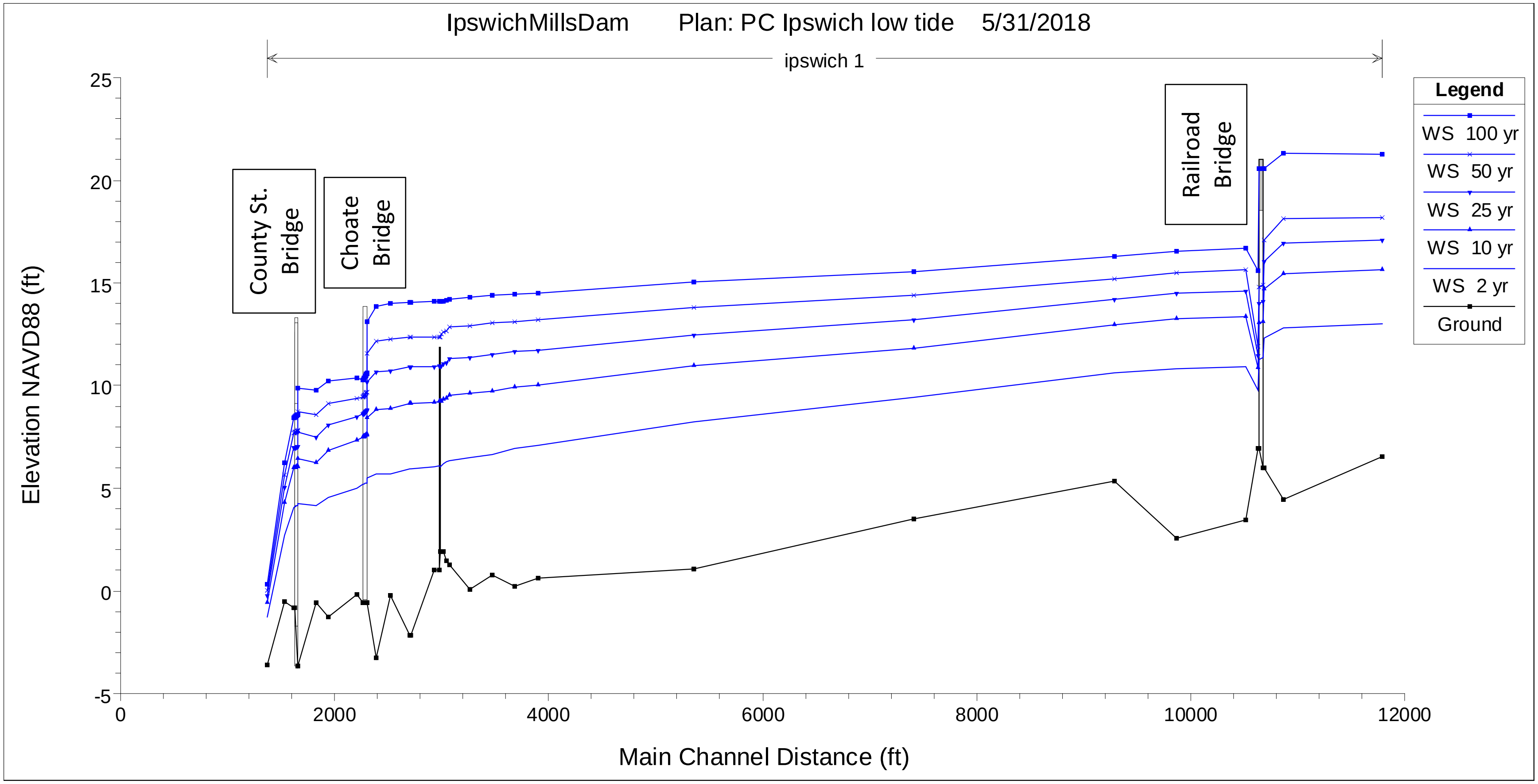
B.3. Existing conditions – Normal depth downstream boundary condition



B.4. Dam-out conditions – Stillwater tide downstream boundary condition (8.7 feet NAVD88)



B.5. Dam-out conditions – Mean High Water (MHW) tide downstream boundary condition (4.1 feet NAVD88)

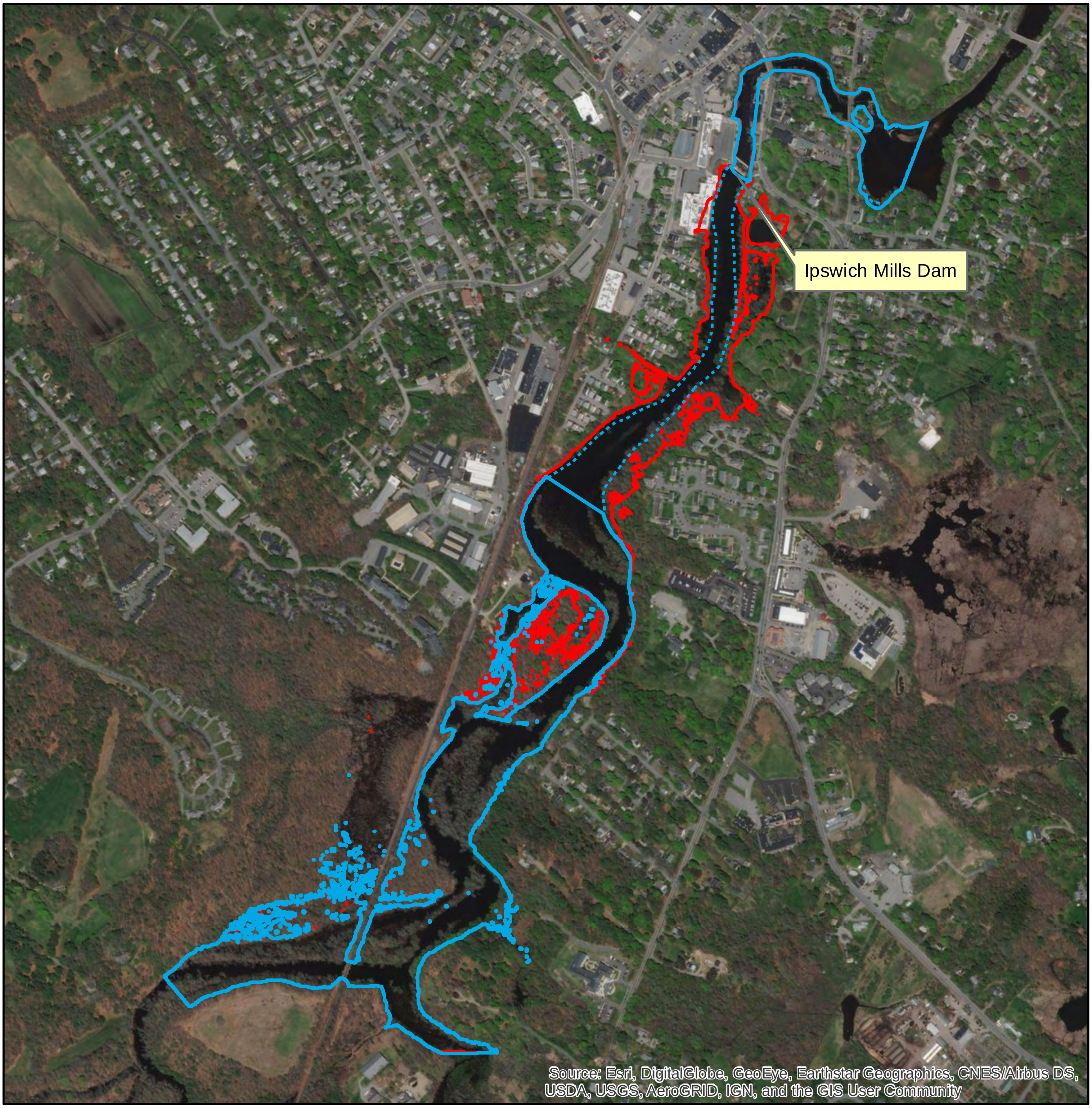


B.6. Dam-out conditions – Normal depth downstream boundary condition

Appendix C

Flood Inundation Maps for

Existing Conditions and Dam-out Conditions



Note: Dashed line depicts approximate channel boundary where predicted water surface levels in dam-out conditions are below existing elevation as provided by LiDAR data.

Ipswich Mills, Ipswich, MA

2-Year Flood Inundation Map

Normal Depth (low tide) Downstream Boundary Condition

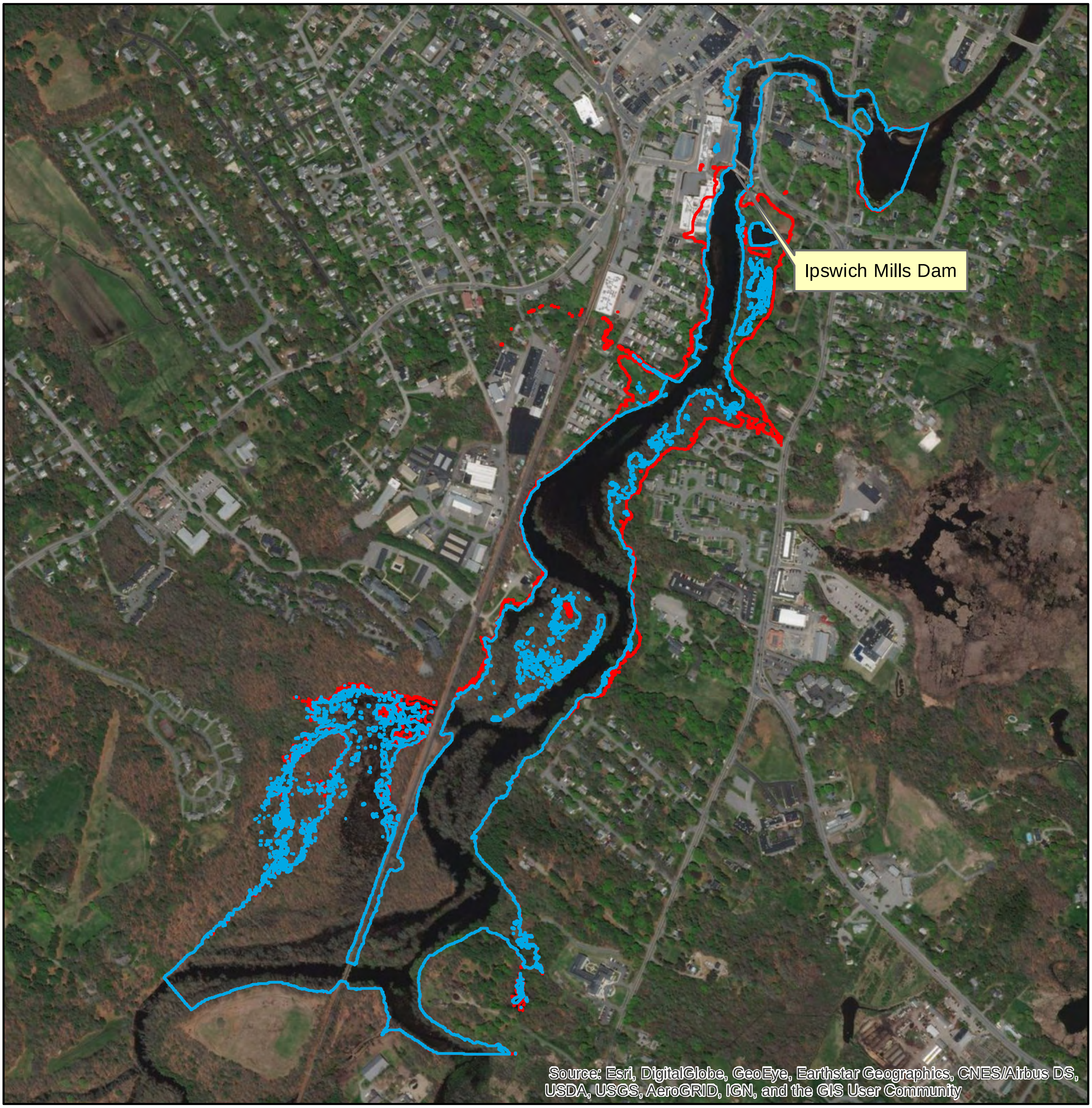
N

0 2,500 5,000 Feet

Legend

Dam Out 2 yr

Existing Conditions 2 yr



Ipswich Mills, Ipswich, MA

10-Year Flood Inundation Map

Normal Depth (low tide) Downstream Boundary Condition

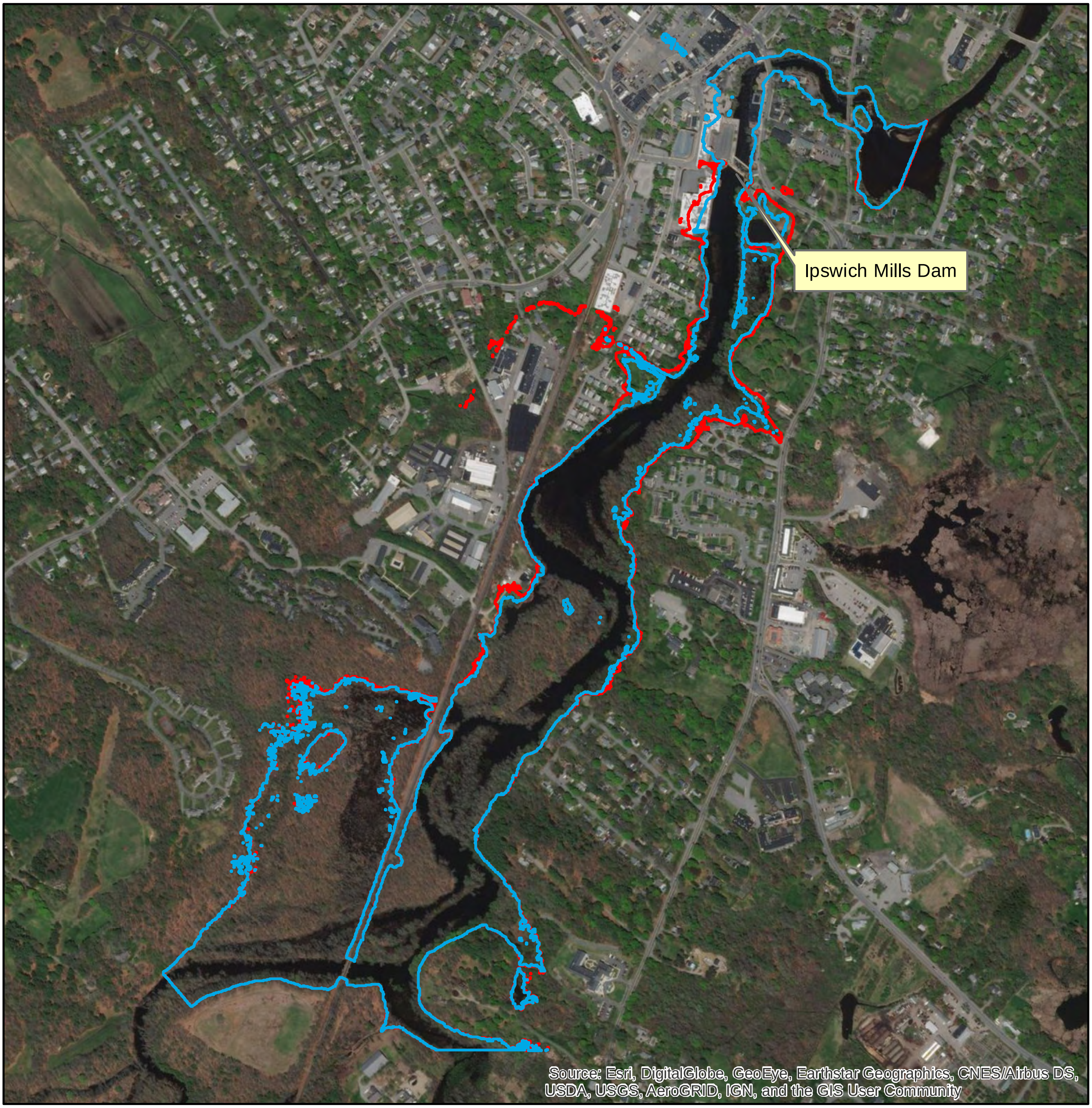
N

0 2,500 5,000 Feet

Legend

Dam Out 10 yr

Existing Conditions 10 yr



Ipswich Mills, Ipswich, MA

25-Year Flood Inundation Map

Normal Depth (low tide) Downstream Boundary Condition

N

0 2,500 5,000 Feet

Legend

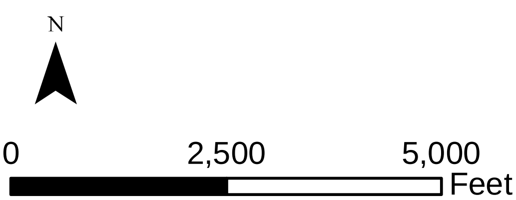
Dam Out 25 yr

Existing Conditions 25 yr

Ipswich Mills, Ipswich, MA

50-Year Flood Inundation Map

Normal Depth (low tide) Downstream Boundary Condition



Legend

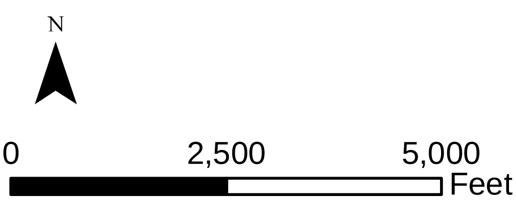
Dam Out 50 yr

Existing Conditions 50 yr

Ipswich Mills, Ipswich, MA

100-Year Flood Inundation Map

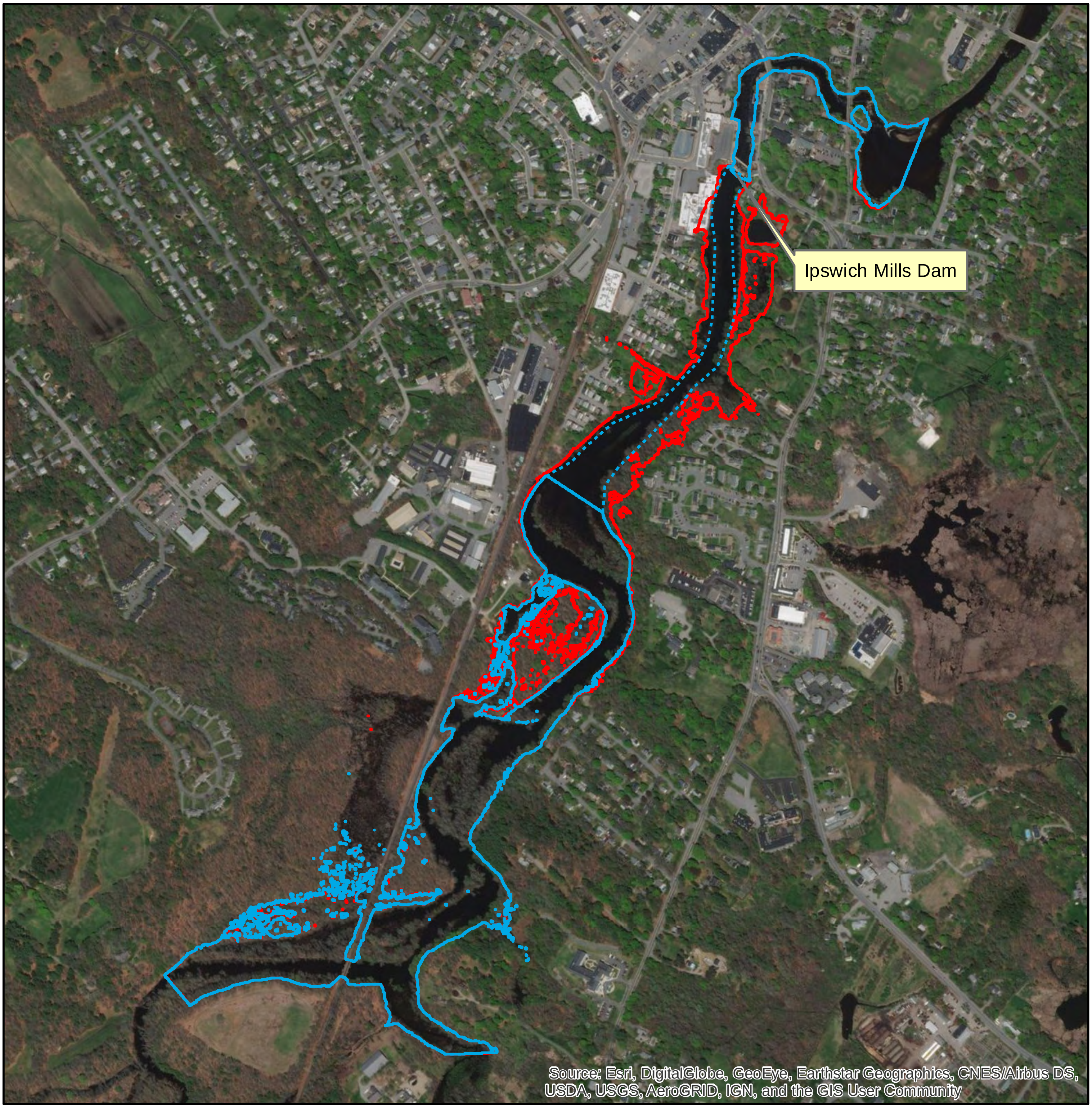
Normal Depth (low tide) Downstream Boundary Condition



Legend

Dam Out 100 yr

Existing Conditions 100 yr



Note: Dashed line depicts approximate channel boundary where predicted water surface levels in dam-out conditions are below existing elevation as provided by LiDAR data.

Ipswich Mills, Ipswich, MA

2-Year Flood Inundation Map

MHW Tide Downstream Boundary Condition (4.1 feet NAVD88)

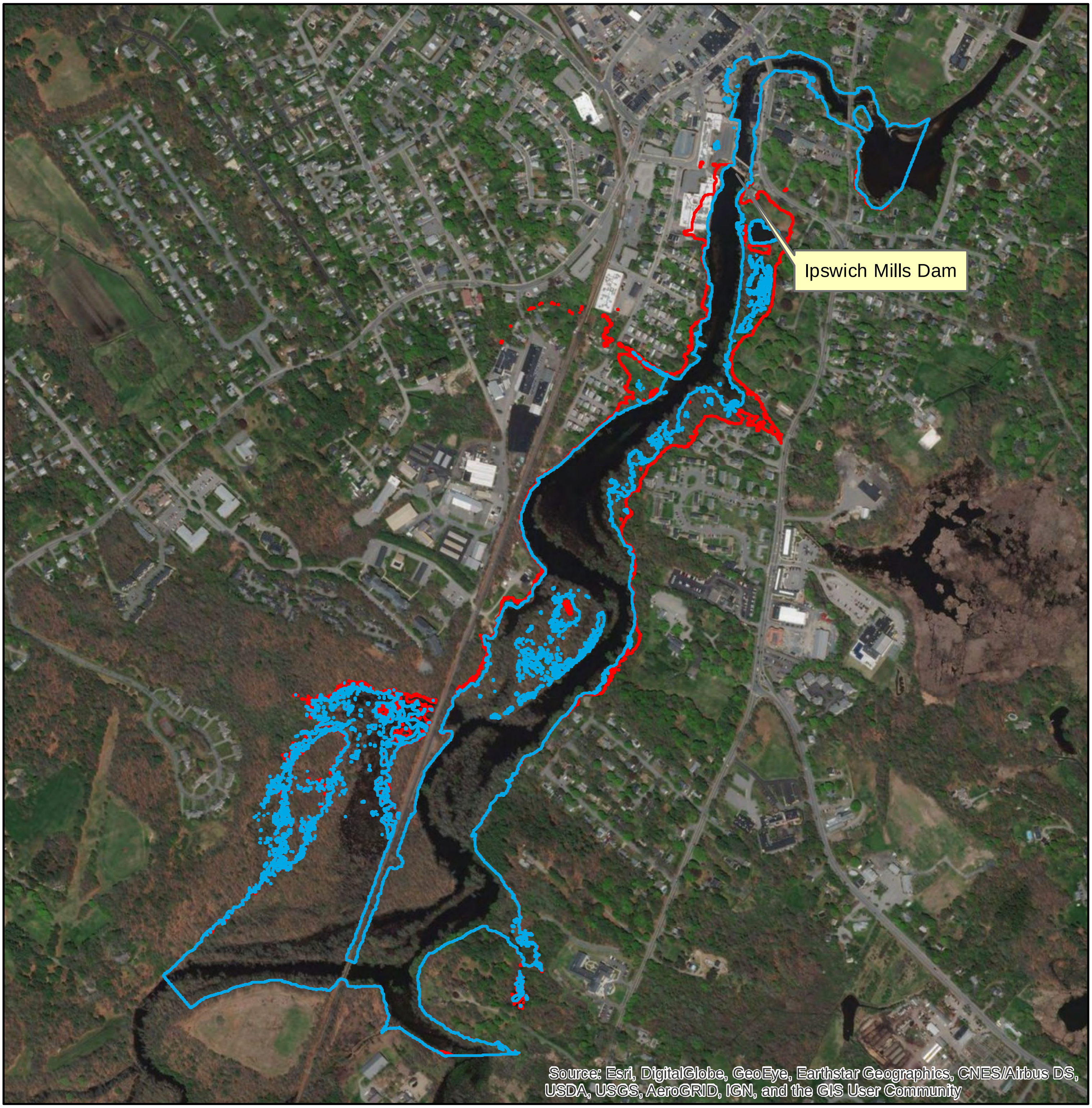
N

0 2,500 5,000 Feet

Legend

Dam Out 2 yr

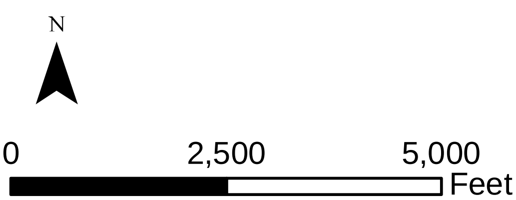
Existing Condition 2 yr



Ipswich Mills, Ipswich, MA

10-Year Flood Inundation Map

MHW Tide Downstream Boundary Condition (4.1 feet NAVD88)



Legend

Dam Out 10 yr

Existing Condition 10 yr

Ipswich Mills, Ipswich, MA

25-Year Flood Inundation Map

MHW Tide Downstream Boundary Condition (4.1 feet NAVD88)

N

0 2,500 5,000 Feet

Legend

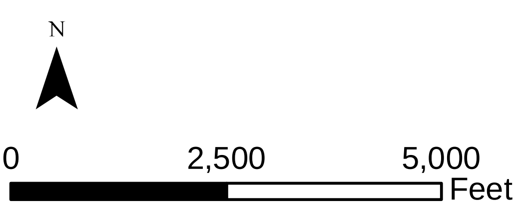
Dam Out 25 yr

Existing Condition 25 yr

Ipswich Mills, Ipswich, MA

50-Year Flood Inundation Map

MHW Tide Downstream Boundary Condition (4.1 feet NAVD88)



Legend

Dam Out 50 yr

Existing Condition 50 yr

Ipswich Mills, Ipswich, MA

100-Year Flood Inundation Map

MHW Tide Downstream Boundary Condition (4.1 feet NAVD88)

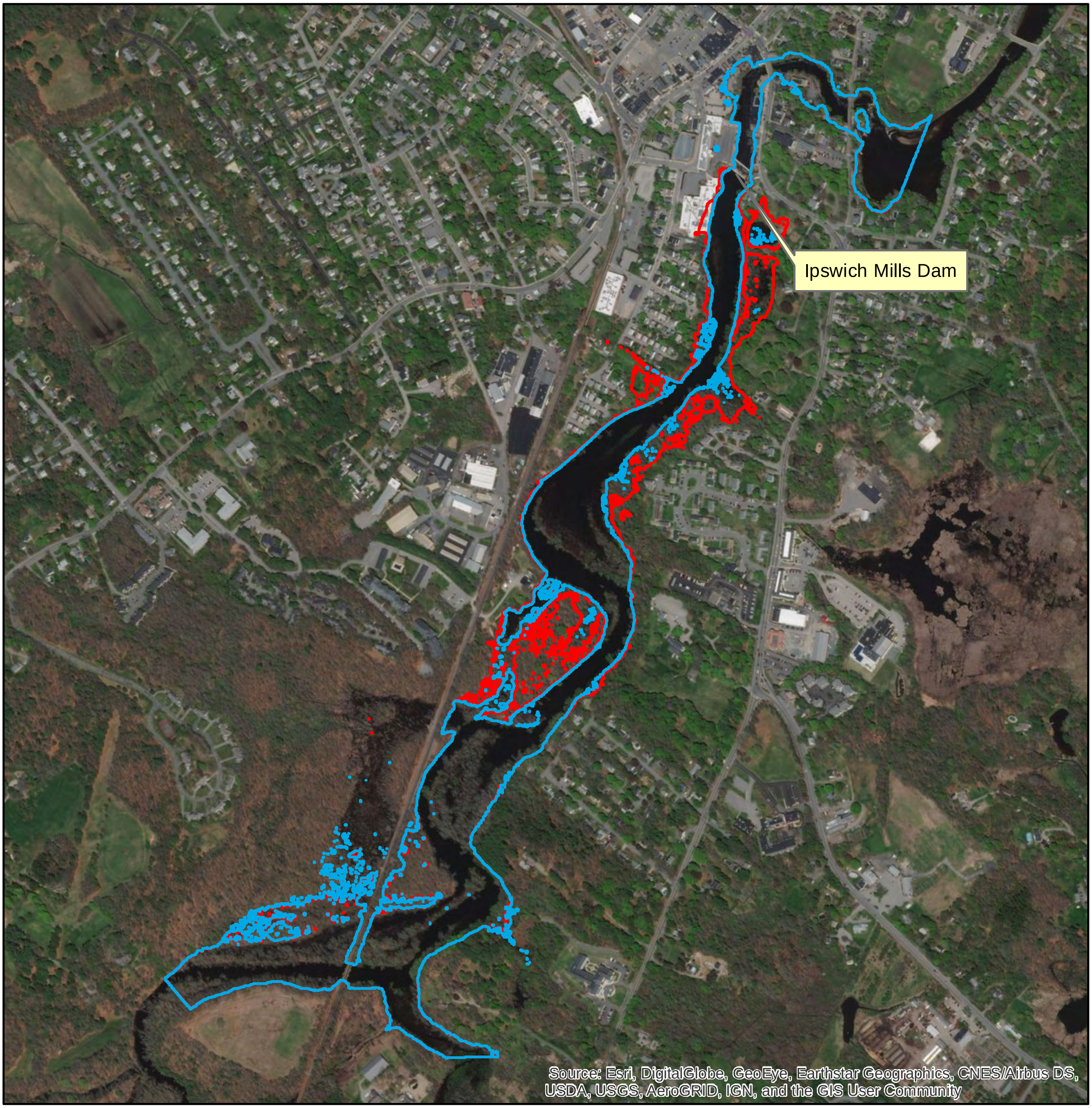
N

0 2,500 5,000 Feet

Legend

Dam Out 100 yr

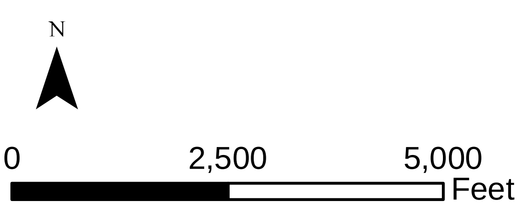
Existing Condition 100 yr



Ipswich Mills, Ipswich, MA

2-Year Flood Inundation Map

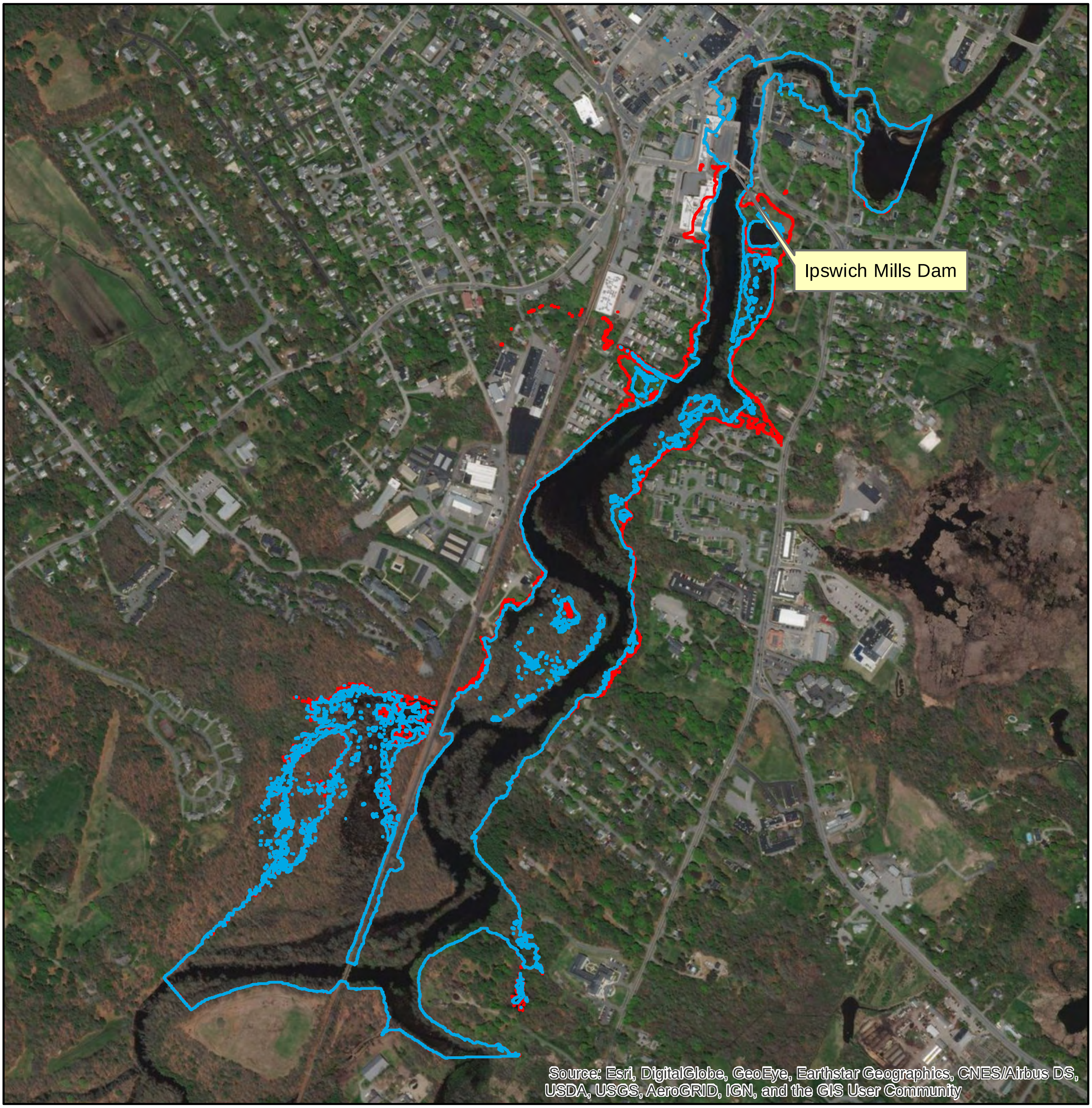
Stillwater Tide Downstream Boundary Condition (8.7 feet NAVD88)



Legend

Dam Out 2 yr

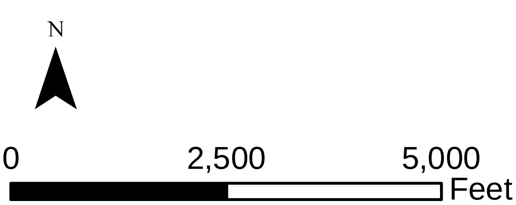
Existing Conditions 2 yr



Ipswich Mills, Ipswich, MA

10-Year Flood Inundation Map

Stillwater Tide Downstream Boundary Condition (8.7 feet NAVD88)



Legend

- Dam Out 10 yr
- Existing Conditions 10 yr

Ipswich Mills, Ipswich, MA

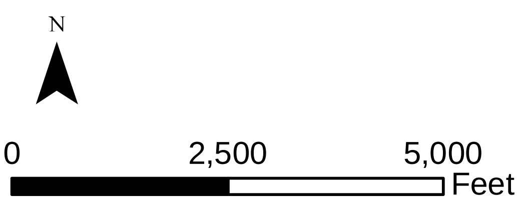
25-Year Flood Inundation Map

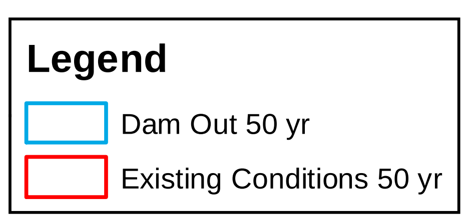
Stillwater Tide Downstream Boundary Condition (8.7 feet NAVD88)

Ipswich Mills, Ipswich, MA

50-Year Flood Inundation Map

Stillwater Tide Downstream Boundary Condition (8.7 feet NAVD88)





Ipswich Mills, Ipswich, MA

100-Year Flood Inundation Map

Stillwater Tide Downstream Boundary Condition (8.7 feet NAVD88)

N

0 2,500 5,000 Feet

Legend

Dam Out 100 yr

Existing Conditions 100 yr

ATTACHMENT 6

29 June 2018
(Revised 6 July 2018)

Mr. Neal Price
Senior Hydrogeologist / Senior Project Manager
Horsley Witten Group
90 Route 6A
Sandwich, MA 02563

Project 160630.01 – Ipswich Mills Dam Removal Feasibility Study: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Supplemental Limited Subsurface Investigation, Ipswich, MA

Dear Mr. Price:

This letter report summarizes our observations, findings, and conclusions regarding the potential impact(s) of the proposed removal of the Ipswich Mills Dam and the subsequent lowering of the water table on the EBSCO Facility Building. The current study supplements the findings from our initial investigation as documented in our report to you dated 17 February 2017 and revised on 20 February 2018.

If additional information becomes available, we reserve the right to supplement or modify the material presented herein.

1. INTRODUCTION

All elevations in this report are in feet referenced to the North American Vertical Datum of 1988 (ft NAVD 88) unless otherwise noted.

1.1 Background

Simpson Gumpertz & Heger Inc. (SGH) completed an investigation to evaluate the potential impacts of the proposed dam removal on the EBSCO Facility Building located adjacent to the Ipswich Mills Dam; refer to our investigation report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, dated 17 February 2018 and revised 20 February 2018, referred to herein as the February 2018 SGH Report (Appendix A). SGH's scope of work was part of a larger feasibility study and concept plan for the dam removal, led by Horsley Witten Group (HWG) and prepared for the Ipswich Mills Dam Removal Feasibility Study Project Team (Project Team). The Project Team includes the Town of Ipswich, the Ipswich River Watershed Association, EBSCO, the NOAA Restoration Center, US Fish and Wildlife Service, the Massachusetts Division of Marine Fisheries, Trout Unlimited, the Massachusetts Division of Ecological Restoration, and others.

The February 2018 SGH Report was limited to two test pit investigations adjacent to the EBSCO Facility Riverfront Foundation Wall at the north end (Building No. 9, constructed in 1908) and the south end (Building No. 10-A, constructed in 1912). SGH concluded that the riverfront wall foundations of Buildings No. 9 and 10-A are bearing on rock and/or are bearing on soils or piled foundations at an elevation lower than the currently estimated low-water level of the Ipswich River at the site after dam removal (El. 3 ft to El. 6 ft). SGH did not observe the foundations supporting interior walls or columns of Buildings No. 9 and 10A or the other buildings on the EBSCO campus (Buildings No. 10, 11, and 11A, constructed in 1901, 1918, and 1946, respectively).

The three borings directed by SGH in August 2016 were located outside of the EBSCO site and did not encounter compressible soils. The 2009 borings performed by others at the south end of Building No. 10A indicate the presence of localized soft compressible soils, including organics, along the riverfront. Where organics are present, which is likely near the river, lowered groundwater levels could result in settlement of pavement, slabs-on-grade, structures on spread footings, or buried utilities supported above the soft compressible soils.

SGH recommended that additional test pits be excavated in the interior and exterior of the EBSCO Facility to obtain more definitive information regarding the presence of timber piles and soft compressible soils within the footprint of the EBSCO Facility. Alternatively, if EBSCO did not provide access to the inside of its facility or access for test pit investigations on the exterior of the facility, SGH recommended that a limited soil test boring investigation be performed around the building exterior to provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and timber piles, if any were deemed to be present.

The Massachusetts Division of Ecological Restoration (MA DER) issued a Request for Proposal (RFP) on 12 April 2018 to perform a limited subsurface investigation. MA DER authorized HWG to retain SGH to perform the work.

1.2 Objective

The objective of the supplemental limited subsurface investigation is to provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and assess the likelihood of the presence of timber piles based on the depth to an adequate soil bearing stratum. The current limited subsurface investigation on the EBSCO Facility site will supplement the existing February 2018 SGH Report.

1.3 Scope of Work

Our Scope of Work included the following:

- Perform eight soil test borings around the perimeter of the EBSCO Facility.
- Retain a third-party soil testing laboratory to perform testing on relatively undisturbed soft compressible soil samples.
- Prepare this letter report summarizing our findings, conclusions, and recommendations.

2. FIELD INVESTIGATION – SOIL TEST BORINGS

SGH performed the supplemental limited subsurface investigation at the site on 1 and 2 June 2018. The investigation consisted of eight soil test borings (SGH-2018-1, SGH-2018-2, SGH-2018-2A, SGH-2018-3, SGH-2018-4, SGH-2018-5, SGH-2018-6, and SGH-2018-7) located around the perimeter of the EBSCO Facility. Soil test boring locations are shown in Figs. 1 and 2. Soil test borehole drilling was performed by Carr-Dee Corporation (Carr-Dee), of Medford, Massachusetts under SGH supervision. Site access was provided by EBSCO, and SGH coordinated boring locations with facilities personnel from the EBSCO Facility. EBSCO requested that SGH not perform soil test borings located adjacent to the north elevation of the EBSCO Facility due to existing buried utilities (including a buried fiber optic cable), the specific locations of which are currently unknown. EBSCO also requested that we not disturb paver site finishes for the patio area. There was also limited access due to steep-sloped site finishes at portions of the facility on the west elevation.

Steven Keppel and Zachary Boswell from SGH were present during the field work to observe drilling, assist in obtaining samples, and prepare a descriptive log of each test boring. The sampling intervals, soil descriptions, Standard Penetration Test (SPT) blowcounts, and other pertinent field data are summarized in the individual soil boring logs included in Appendix B. The soil test borings were performed on the north, south, and west elevations of the EBSCO Facility (Photos 1, 2, and 3). The east elevation of the EBSCO Facility (referred to as the Riverfront Foundation wall in our previous report) borders the Ipswich River.

Carr-Dee drilled soil test borings with a Mobile soil scout track drill rig or truck-mounted drill rig using the case and wash method with a 4.5 in O.D. casing. One borehole (SGH-2018-3) was drilled using a 2-1/4 in. I.D. hollow stem auger. Soil samples were obtained using 2 in. O.D. split spoon samplers driven using a 140 lb donut hammer falling 30 in. with a rope cathead. Relatively undisturbed soil samples of soft compressible soils were obtained using a thin-walled Shelby tube.

Soil test borings extended into dense Glacial Till or to split spoon refusal, which ranged between El. -11 ft and El. 8.6 ft. (i.e., 7.5 to 24 ft below ground surface, bgs). Split spoon samples were obtained at 5 ft intervals, except in soft compressible soils where continuous SPT sampling and Shelby tubes were obtained. We encountered wood debris in the wash water while drilling through the soft compressible soils at Boring SGH-2018-2. We had poor sample recovery rates at the same depths where we observed wood in the wash. We drilled Boring SGH-2018-2A, located approximately 4 ft from SGH-2018-2, in order to collect Shelby tube samples in the soft compressible soils. Therefore, we terminated this boring prior to reaching the Glacial Till stratum or refusal. All samples were secured, sealed, and transported to the SGH office at the end of the soil test boring program.

We attempted to drill a soil test boring through an existing concrete pad adjacent to Building No. 10-A. After two attempts we abandoned this location after reaching refusal at a depth of approximately 11 in. on steel reinforcement placed both ways within the slab. We moved the drill rig just outside the slab and completed Soil Test Boring SGH-2018-3.

We estimated elevations based on our measurements for the top of the Riverfront Foundation Wall in the February 2018 SGH Report.

3. ANALYSIS

3.1 Subsurface Conditions

Ground surface conditions consist of asphalt pavement or topsoil. The asphalt pavement is 4 in. to 9 in. thick and was encountered at Soil Test Borings SGH-2018-1, SGH-2018-5, and SGH-2018-6. The topsoil consists of 3 to 6 in. of a brown, dry, sandy silty loam. Topsoil was encountered in Soil Test Borings SGH-2018-2, SGH-2018-2A, SGH-2018-3, SGH-2018-4, and SGH-2018-7.

We prepared a subsurface profile, transverse to the Ipswich River, along the south elevation of the EBSCO Facility based on the results of the soil test borings performed by SGH (SGH-2018-1 to SGH-2018-7) and others (B-2 to B-4) (Fig. 3). We summarize the subsurface strata encountered in the following sections.

3.1.1 Subsurface Conditions - Southeast of EBSCO Facility

We summarize the soil strata encountered southeast of the EBSCO Facility (Soil Test Borings B2, B-3, B-4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3) as follows:

- **Stratum 1 – Fill:** This stratum consists of 3 to 10 ft of a loose to medium dense, brown, dry to wet, silty sand to sandy gravel, fine to coarse grained, poorly graded, subangular, with trace wood and trace brick. SPT blow counts ranged from 2 to 15 blows per foot (bpf).

- **Stratum 2a – Sand and Silt:** This stratum consists of 3 ft of very loose brown, sand and silt, fine grained. SPT blow counts were 2 bpf. This stratum was encountered in Soil Test Boring SGH-2018-2 underlying the Fill stratum.

- **Stratum 3 – Upper Silty Clay:** This stratum consists of 1 to 5.5 ft of very soft to stiff, gray to olive gray, moist to wet silty clay. SPT blow counts ranged from 2 to 10 bpf. This stratum was encountered in Soil Test Borings SGH-2018-3, B-3, and B-4 underlying the Fill stratum and in Soil Test Boring SGH-2018-2 and SGH-2018-2A underlying the Sand and Silt stratum. Fine- to coarse-grained silty sand seams were observed within this stratum at Soil Test Borings SGH-2018-2A, B-3, and B-4. We encountered wood debris in the wash water while drilling at Soil Test Boring SGH-2018-2.

- **Stratum 4 – Organic Silt:** This stratum consists of 1.5 to 2.5 ft of very soft to firm, grey to black, wet, organic silt, with trace to some fine sand. SPT blow counts range from 2 to 5 bpf. This stratum was encountered at Soil Test Borings SGH-2018-2, SGH-2018-2A, SGH-2018-3, B-3, and B-4, underlying the Fill or Upper Silty Clay strata. The measured organic content ranges from 10.3% to 11%.

  A 5 ft thick organic silt stratum was identified as a Peat stratum by others in Soil Test Borings B-3 and B-4. However, the soil description in the logs indicates “fine Sand and Silt with some organics (PEAT).” Furthermore, only 12 in. of this organic soil material was sampled; no other sampling was performed within this stratum. In our soil profile (Fig. 3) we classify this stratum as Organic Silt with a thickness of 2.5 ft (instead of 5 ft shown on logs for B-3 and B-4) based on the description of the soils in these boring logs, the lack of continuous sampling by others through this stratum, and our visual observations and laboratory test results at the soil test borings performed in 2018 that are located within close proximity to Soil Test Borings B-3 and B-4.

- **Stratum 5 – Lower Silty Clay:** This stratum consists of 2 to 6 ft of firm to very stiff, gray to olive gray, moist to wet silty clay. SPT blow counts ranged from 5 to 20 bpf. This stratum was encountered in Soil Test Borings SGH-2018-2 (Photo 4), SGH-2018-2A, and SGH-2018-3 underlying the Organic Silt stratum. Fine- to coarse-grained silty sand seams were observed within this stratum at Soil Test Boring SGH-2018-2A.[1] We encountered wood debris in the wash water while drilling at Soil Test Boring SGH-2018-2.

- **Stratum 6 – Glacial Till**: This stratum consists of medium dense to very dense, light reddish brown to olive grey, dry to wet, sand to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 21 bpf to refusal. This stratum was encountered at Soil Test Boring B-2, underlying the Fill stratum; at Soil Test Boring SGH-2018-2, underlying the Lower Silty Clay stratum; and at Soil Test Borings B-3 and B-4, underlying the Organic Silt stratum. Soil test borings were terminated in the glacial till layer and SGH or others did not determine the stratum thickness at these locations. See Table 1 below for the approximate elevation of the top of the Glacial Till stratum.

- **Stratum 7 – Rock Ledge:** The elevation of the top of the rock ledge varies at the site. Prior test pit investigations and bathymetric survey results indicate that the rock ledge varies between approximately El. 3.0 ft and El. 7.5 ft near the Ipswich Mills Dam.

3.1.2 Subsurface Conditions North, West, and Southwest of EBSCO Facility

We summarize the strata encountered on the north, west, and southwest elevations of the EBSCO Facility (Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-5, SGH-2018-6, SGH-2018-7, SGH-2016-1, and SGH-2016-2) as follows:

- **Stratum 1 – Fill:** This stratum consists of 2.5 to 8 ft of a loose to very dense, brown, dry to wet, silty sand to sandy gravel, fine to coarse grained, poorly graded, subangular, with trace wood and trace brick. SPT blow counts ranged from 4 to 73 bpf.

- **Stratum 2b – Silty Sand:** This stratum consists of 2 to 4.5 ft of medium dense to very dense, light orange brown, dry, silty sand to gravelly sand, fine to coarse grained, uniform to well-graded, subangular. SPT blow counts ranged from 16 to 99 bpf. This stratum was encountered at Soil Test Borings SGH-2016-2 and SGH-2016-3 underlying the Fill stratum.

- **Stratum 2c – Clayey Silt:** This stratum consists of 2.5 to 5 ft of medium stiff to hard, brown, gray, or olive, clayey silt with trace fine sand. SPT blow counts ranged from 6 to 39 bpf. This stratum was encountered in Soil Test Borings SGH-2018-5 and SGH-2018-6, located at the southwest corner of the EBSCO Facility, underlying the Fill stratum.

[1] The laboratory reports Sample SGH-2018-3 US-3 (depth 13.5 to 15.5) as Grey Varved Soil. We did not observe varved soil during drilling.

- **Stratum 7 – Glacial Till**: This stratum consists of medium dense to very dense, light reddish brown to olive grey, dry to wet, sand to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 28 bpf to refusal.

  This stratum was encountered at Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-7, and SGH-2016-1 underlying the Fill stratum; at Soil Test Borings SGH-2018-5 and SGH-2018-6 underlying the Clayey Silt stratum, and at Soil Test Borings SGH-2016-2 and SGH-2016-3 underlying the Silty Sand Stratum. See Table 1 below for approximate elevations of the top of the Glacial Till stratum.

3.1.3 Summary of Top of Glacial Till Stratum Elevations

Table 1 summarizes the depth and elevation of the top of the Glacial Till stratum:

Table 1: Top of Glacial Till Stratum

	Soil Test Borings											
	SGH-2018-1	**SGH-2018-2**	**SGH-2018-4**	**SGH-2018-5**	**SGH-2018-6**	**SGH-2018-7**	**SGH-2016-1**	**SGH-2016-2**	**SGH-2016-3**	**B-2**	**B-3**	**B-4**
Depth (ft)	6.0	19.0	6.0	13.0	5.0	5.0	4.5	7.5	8.0	10.0	16.0	16.0
Elevation (ft)(1)	11.0	-6.0	10.0	3.0	8.0	7.0	12.5	10.5	6.0	3.0	-3.0	-3.0

(1) Elevations are estimated and referenced to the NAVD88 datum.

3.2 Groundwater Conditions

SGH did not measure groundwater levels during drilling at the soil test borings due to the cased-and-washed drilling method artificially raising the water levels within the borehole. After drilling was completed, SGH observed the groundwater level at approximately El. 8 ft at Soil Test Borings SGH-2018-2, SGH-2018-2A, and SGH-2018-3 located on the south elevation of the EBSCO Facility, which was generally consistent with the level of the Ipswich River during drilling.

We did not observe groundwater in the soil test borings performed in 2016 in Estes Street and Saltonstall Street (SGH-2016-1 and SGH-2016-2), which were terminated at approximately El. 6 ft and El. 7.5 ft respectively.

3.3 Settlement of Compressible Soils

Soft compressible soils are present at the southeast elevation of the EBSCO Facility (Soil Test Borings B3, B4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3). For our settlement analysis, we assumed a soil profile similar to the conditions encountered at Soil Test Borings SGH-2018-2/2A and B-3. We assumed that the soil profile consists of a 3 ft thick Fill stratum overlying a 1 ft thick Sand stratum overlying a 5 ft thick Upper Silty Clay stratum overlying a 2.5 ft thick Organic Silt stratum overlying a 6 ft thick Lower Silty Clay stratum. We estimated soil properties for the Upper Silty Clay, Organic Silt, and Lower Silty Clay strata based on laboratory consolidation tests performed on relatively undisturbed soil samples obtained from Soil Test Boring SGH-2018-2A.

We estimated the potential settlement of compressible soils due to primary consolidation imposed by an increase in effective stress due to lowered groundwater levels, and secondary compression of the organic soils after primary consolidation is complete[2]. For this analysis we assumed that the organic soils are normally consolidated and the additional stress applied to the organic soils due to groundwater drawdown will reinitiate secondary compression.

The laboratory test results show that the Lower Silty Clay stratum is over-consolidated; we estimate an over-consolidation ratio of approximately 4 for this stratum[3]. We assumed that the Upper Silty Clay stratum has similar consolidation parameters as the test sample from the Lower Silty Clay stratum, except that we assumed the Upper Silty Clay is normally consolidated to match the underlying Organic Silt stratum conditions determined from the laboratory test results.

In the February 2018 SGH Report, we estimated an initial groundwater level at approximately El. 6 ft based on our observations of the river staff gauge during our investigation in August 2016 and on groundwater data collected at one observation well. We assumed that the overburden soils under the EBSCO Facility have experienced groundwater levels as low as El. 6 ft; we calculated the range of potential settlement of the clay and organic soils resulting from a 1, 2, and 3 ft drop in groundwater levels. We understand that HWG has not yet completed the hydraulic study of post-dam-removal river levels; however, the revised preliminary estimated lower-bound water level after the proposed dam removal is at approximately El. 1 ft (i.e. a 5 ft drop in groundwater level), and will likely be higher (between El. 3 ft and El. 6 ft). The water level is subject to change pending results from the hydraulic analysis performed by HWG. For our analysis of the potential settlement of compressible soils, we considered a range of potential low-water river elevations between El. 1 ft and El. 5 ft.

Table 2 summarizes the soil consolidation soil parameters used in our analysis:

Table 2 – Clay and Organic Silt Strata Consolidation Parameters(1)

Soil Stratum	Depth to Mid-Layer [ft, bgs]	σ'_{vo}(2) [psf]	σ'_{vf}(2) [psf] Case 1	Case 2	Case 3	Case 4	Case 5	σ'_p(2) [psf]	Initial Void Ratio e_0	C_c(3)	C_r(3)	C_α(3)
Upper Silty Clay	7.75	872	919	919	919	919	919	872	0.897	0.36	0.05	--
Organic Silt	10.25	991	1,054	1,116	1,178	1194	1194	1,000	2.87	1.18	0.17	0.021
Lower Silty Clay	14.5	1,206	1,268	1,331	1,393	1455	1518	5,000	0.897	0.36	0.05	--

(1) Case Nos. 1 through 5, correspond with a groundwater level drawdown of 1 ft through 5 ft respectively (i.e. groundwater level at El. 5 ft, El. 4 ft, El. 3 ft, El. 2 ft, and El. 1 ft respectively).
(2) σ'vo is the estimated existing overburden or in situ vertical effective stress at midlayer (prior to dam removal). σ'vf is the estimated vertical effective stress after dam removal (lowered groundwater level). σ'p is the maximum past pressure experienced by the soil estimated from laboratory test results for SGH-2018-2A US-1 and US-3. The soil profile is estimated from Soil Test Borings SGH-2018-2 and SGH-2018-2A.
(3) Cc is the primary consolidation index, Cr is the recompression index, Cα is the secondary compression index. We did not estimate secondary compression for the Silty Clay.
(3) We assumed the lower half (depths ranging between 6.5 ft and 9 ft) of the Upper Silty Clay stratum contributes to settlement and is normally consolidated. We assumed consolidation parameters for the Upper Silty Clay are similar to the test results for the Lower Silty Clay stratum.

[2] Primary consolidation settlement is load-dependent and occurs when load is transferred to the soil structure and pore water is squeezed out of the soil mass. Secondary compression settlement is time-dependent and occurs after primary consolidation is complete. Secondary compression occurs under constant load and can be significant for organic soils due to creep, and compression and degradation of the organic material.
[3] Normally consolidated and over-consolidated are terms that refer to the current vertical overburden pressure on the soil relative to the maximum vertical overburden pressure the soil has ever experienced. A normally consolidated soil has a current pressure equal or nearly equal to the maximum experienced pressure. An over-consolidated soil has previously experienced a higher pressure, which can be due to natural or man-made causes, compared to the current pressure. The over-consolidation ratio is the ratio of the maximum past pressure relative to the current pressure.

Based on the assumptions listed above, we estimate that the average primary settlement due to lowering groundwater levels by between 1 ft and 5 ft in the post-dam-removal conditions is in the order of between 0.4 in. and 1.0 in., in those areas where compressible soils are present.

We estimate that secondary compression, which is time-dependent strain, of the organic silt stratum will be about 0.5 in. For the purpose of this calculation we assumed a remaining service life of 50 yrs for the EBSCO Facility (we have not performed a service life evaluation of the structure). Including primary consolidation for the 1 ft to 5 ft drawdown scenarios, the total estimated settlement of the soft compressible soils is approximately between 0.9 in. and 1.5 in. after 50 yrs from drawdown.

Table 3 summarizes our estimated settlement of compressible soils resulting from drawdown of groundwater levels:

Table 3 - Estimated Settlement due to Drawdown

Case No. / Drawdown [ft]	Primary Consolidation Settlement(1) [in.]	Secondary Compression(2) [in.]	Total Settlement [in.]
1	0.4	0.5	0.9
2	0.7	0.5	1.2
3	0.9	0.5	1.4
4	1.0	0.5	1.5
5	1.0	0.5	1.5

Notes
(1) See Table 2 for soil properties.
(2) Estimated secondary compression 50 yrs after end of primary consolidation, assuming normally consolidated soils.

4. DISCUSSION

4.1 Supplemented EBSCO Facility Elevations

We estimated ground surface and soil strata elevations based on our reported elevations for the Riverfront Foundation Wall in the February 2018 SGH Report. Table 4 below summarizes elevations pertinent to the EBSCO Facility updated to include additional elevations for the top of Glacial Till based on the recent field investigation. We understand from HWG that the preliminary estimate for the low river level is likely in the range of El. 1 ft to El. 6 ft after dam removal (elevation is subject to change pending the results of the hydrologic and hydraulic analysis).

Table 4 – Water Level, Glacial Till, and EBSCO Foundation Wall Elevations

Description	Elevation [ft, NAVD 88]
Water Levels	
Estimated Low River Level Elevation After Dam Removal (Preliminary Estimate from HWG)	1 to 6
South End of the EBSCO Facility	
Top of Foundation Wall at Building No. 10-A.	12.5
Maximum Elevation of Bottom of Foundation Wall at Building No. 10-A / Bottom of Test Pit No. 2 (TP-2)	–0.5
Approximate Range of top of Organic Silt Stratum at South End of Building No. 10A	1.5 to 4
Approximate Range of top of Glacial Till Stratum at South End of Building No. 10A	-6 to 3
North End of the EBSCO Facility (Closest to Dam)	
Top of Foundation Wall at Building No. 9.	11.4
Apparent Bottom of Foundation Wall at Building No. 9 / Bottom of Test Pit No. 1 (TP-1)	3.2
Dam Crest	8.9
Approximate Top of Glacial Till Stratum at North End of Building No. 9	7
Average Elevation of Rock Ledge at Toe of Dam	2.9
Top of Abandoned Timber Formwork and Abandoned Timber Wall	5.7

4.2 Likelihood of Presence of Timber Pile Foundations at EBSCO Facility

We encountered a shallow depth to the top of the Glacial Till stratum in the soil test borings located on the northern end of Buildings No. 9, No. 10, No. 10B and No. 11 (Soil Test Borings SGH-2018-1, SGH-2018-4, SGH-2018-7, SGH-2016-1, and SGH-2016-2). The depth to the top of the Glacial Till stratum in this area ranged from 4.5 ft to 7.5 ft below ground surface (bgs) (i.e., El. 7 to 12.5). Considering a minimum depth to bottom of footing of approximately 3 ft bgs for exterior foundations and approximately 1 ft below top of slab-on-grade for interior foundations, it is unlikely that timber piles were installed in these areas, as the timber piles would be in the order of 3.5 to 6.5 ft long at most. It is likely that the original foundation construction in this area included over-excavation to place shallow footings bearing directly on the Glacial Till stratum. Based on the limited subsurface information gathered to date, it is very likely that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.

We encountered the top of the Glacial Till stratum on the south end of Building No. 11 (Soil Test Borings SGH-2018-5 and SGH-2018-6) at depths ranging between 5 to 13 ft bgs (El. 3 to 8 ft). At Soil Test Boring SGH-2018-5, where the depth to the top of Glacial Till was 13 ft bgs, the Glacial Till stratum was overlain by 2.5 to 5 ft of medium stiff to hard fine-grained soils (Clayey Silt stratum); that is, the top of the Clayey Silt stratum is at 2.5 to 8 ft bgs in this area. Considering a minimum depth to bottom of footing of approximately 3 ft bgs for exterior foundations and approximately 1 ft below top of slab-on-grade for interior foundations, it is possible but unlikely that timber piles were installed in this area, as the timber piles would be in the order of 4 to 12 ft long at most if bearing on the Glacial Till stratum, and 1.5 to 5 ft long at most if bearing on the Clayey Silt stratum. It is highly likely that shallow soil bearing foundations bearing on the medium-stiff to hard natural fine-grained soils (Clayey Silt statum) were used in this area. Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No. 11 are founded on shallow spread footings bearing on the Clayey Silt stratum.

The top of the Glacial Till stratum is generally deeper on the south end of Building No. 10A and Building No. 11A (Soil Test Borings B-2, B-3, B-4, SGH-2018-2, SGH-2018-2A, and SGH-2018-3). The depth to the top of the Glacial Till stratum in these borings ranged from 13 to 19 ft bgs (El. -6 ft to 3 ft). The Glacial Till stratum is overlain by 4 to 13 ft of soft to medium-stiff fine-grained soils, including about 2.5 ft of Organic Silt. In our February 2018 SGH Report we noted that it was unlikely that the Riverfront Foundation Wall at Building No. 10A was founded on timber piles given the depth to which excavation was performed to install the wall (greater than 10 ft) and the maximum 5.5 ft depth to top of Glacial Till stratum from the estimated maximum elevation of bottom of Riverfront Foundation Wall (El. -0.5 ft). The construction of the land-side exterior foundation walls and interior column foundations at Buildings No. 10A and No. 11A is not known. The depth and thickness of the observed compressible soils in this area is such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas.

4.3 Settlement of Compressible Soils

Lowered groundwater levels due to potential drawdown after dam removal would increase effective stresses in soft compressible soils and result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported above these soft compressible soils. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river; however, we observed soft compressible soils at all soil test borings on the southeast elevation, near the river. Our observations at Soil Test Borings SGH-2018-2, SGH-2018-2A, and SGH-2018-3 indicate that the compressible organic stratum may not be as thick as indicated on prior soil test borings performed by others. Prior soil test borings also indicated the presence of peat. Our visual observations and laboratory results show soils consistent with organic silt. The laboratory test results for organic content measured between 10.3% and 11% percent organic content of the two samples tested, supporting this change in soil description.

In the February 2018 SGH Report we estimated settlement of the soft compressible soils observed by others using assumed soil properties based on ranges of published values for organic soils and local clays. We updated our calculations using revised strata thicknesses, depths, and consolidation parameters determined from our laboratory testing. We tested relatively undisturbed (Shelby tube) samples for one Lower Silty Clay sample and one Organic Silt sample, and we estimated consolidation parameters from the test results. The tested samples are representative of the soil strata, but the in-situ soil properties may vary compared to our limited sample testing results. The laboratory test results show that the Organic Silt is nearly normally consolidated; therefore, a relatively small increase in stress due to drawdown of groundwater levels will result in potentially significant consolidation settlement. Our estimated consolidation coefficients based on laboratory testing of the Lower Silty Clay are consistent with the average values used in our prior analysis, and the estimated consolidation coefficients from laboratory testing of the Organic Silt are consistent with lower-bound published values. Our refined total settlement estimate of the localized soft compressible soils based on laboratory data of the sampled in-situ soils shows a somewhat smaller magnitude of settlement compared to our previous estimate based on only published data. We estimate that the average primary settlement due to lowering groundwater levels by between 1 ft and 5 ft in the post-dam removal conditions is in the order of between 0.4 in. and 1.0 in., in those areas where compressible soils are present. At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. Based on results of the soil test boring located on the building exterior, it is unlikely that organics underlie the northern and western portion of the EBSCO Facility.

We also updated our settlement estimate to include secondary compression, which is long-term time-dependent compressive strain that will occur for many years after primary consolidation is complete. We estimated secondary compression 50 yrs after the end of primary consolidation, assuming normally consolidated organic soils. We estimate secondary compression of the organic silt stratum will be nearly 0.5 in. after 50 yrs. Including primary consolidation for scenarios ranging between 1 ft and 5 ft of drawdown, the total settlement of the soils would range between 0.9 in. and 1.5 in. respectively after 50 yrs.

5. CONCLUSIONS

Based on the recent June 2018 investigation, we have the following conclusions to supplement our February 2018 SGH Report regarding the potential impacts of the dam removal on the adjacent EBSCO Facility:

- Existing Foundations:
  - Based on the limited subsurface information gathered to date, it is very likely that the exterior walls and interior columns of Building No. 9, Building No. 10, Building No. 10B, and the northern portion of Building No. 11 are founded on shallow spread footings bearing on the Glacial Till stratum or rock.
  - Based on the limited subsurface information gathered to date, it is likely that the exterior walls and interior columns of the southern portion of Building No. 11 are founded on shallow spread footings bearing on the Clayey Silt stratum.
  - It is unlikely that the Riverfront Foundation Wall at Building No. 10A was founded on timber piles given the depth to which excavation was performed to install the wall (greater than 10 ft) and the maximum 5.5 ft depth to the top of the Glacial Till stratum from the estimated maximum elevation of the bottom of Riverfront Foundation Wall (El. -0.5 ft). The construction of the land-side exterior foundation walls and interior column foundations at Buildings No. 10A and No. 11A is not known. The depth and thickness of the observed compressible soils in this area are such that timber piles may have been driven through the soft compressible soils to bear on the Glacial Till stratum below to support the building structure in these areas.
  - At this time it is uncertain to what extent, if any, compressible soils underlie the EBSCO Facility. We did not encounter soft compressible soils in soil test borings located on the west and north elevations of the building away from the river.

- Effects of Lowering Groundwater: Lowered groundwater levels could result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported by soft compressible soils. We estimate a potential total settlement of the soft compressible soils of approximately between 0.9 in. and 1.5 in. respectively due to a water level drawdown of between 1 ft and 5 ft, assuming a remaining service life of 50 years for the EBSCO facility, in those areas where compressible soils are present.

- Subsequent Steps: If the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above El. 6 ft, the following approach could be implemented to assess potential settlement of structures bearing on compressible soils:

  - Conduct a subsurface investigation consisting of test pits and soil test borings performed within the EBSCO Facility, focused on Buildings 10A and 11A where the foundation construction is unknown and compressible soils are potentially present. This portion of the EBSCO Facility presents the highest risk of settlement due to drawdown of groundwater levels.
  - Develop and implement a precision movement monitoring program to monitor for the potential movement of structures during dam removal construction. Install the instrumentation prior to the start of construction, and also establish acceptable settlement limits with approval from EBSCO.

Limitations of Current Investigation

The information presented herein is based on the geotechnical information collected to date. The boring logs and geotechnical investigation records depict subsurface conditions only at the specific soil sampling locations. Subsurface conditions at other locations may differ from conditions observed at specific sample depths and exploration locations. There is no warranty or guarantee, either expressed or implied, that the conditions indicated by such investigations or records thereof are representative of those existing throughout such areas, or any part thereof, or that unexpected developments may not occur, or that materials other than, or in proportions different from, those indicated may not be encountered.

Sincerely yours,

COMMONWEALTH OF MASSACHUSETTS
WILLIAM P. KONICKI
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REGISTERED PROFESSIONAL ENGINEER
7/6/2018

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Encls.

List of Attachments

Photos 1 through 4

Figures
Figure 1 – Subsurface Investigation Plan
Figure 2 – EBSCO Facility and Investigation Location Plan
Figure 3 – Soil Profile - EBSCO Facility

Appendices
Appendix A - Report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts, EBSCO Facility Building, Ipswich, MA" prepared by SGH dated 17 February 2018 and revised 20 February 2018
Appendix B - 2018 Soil Test Boring Logs
Appendix C - 2018 Laboratory Test Results

PHOTOS

Photo 1

Location of Soil Test Boring SGH-2018-7 on the north elevation of the site adjacent to the patio area at the EBSCO Facility

Photo 2

West elevation of Building No. 10 and north elevation of Building Nos. 10-B and 11. The construction cone indicates the location of Soil Test Boring SGH-2018-4.

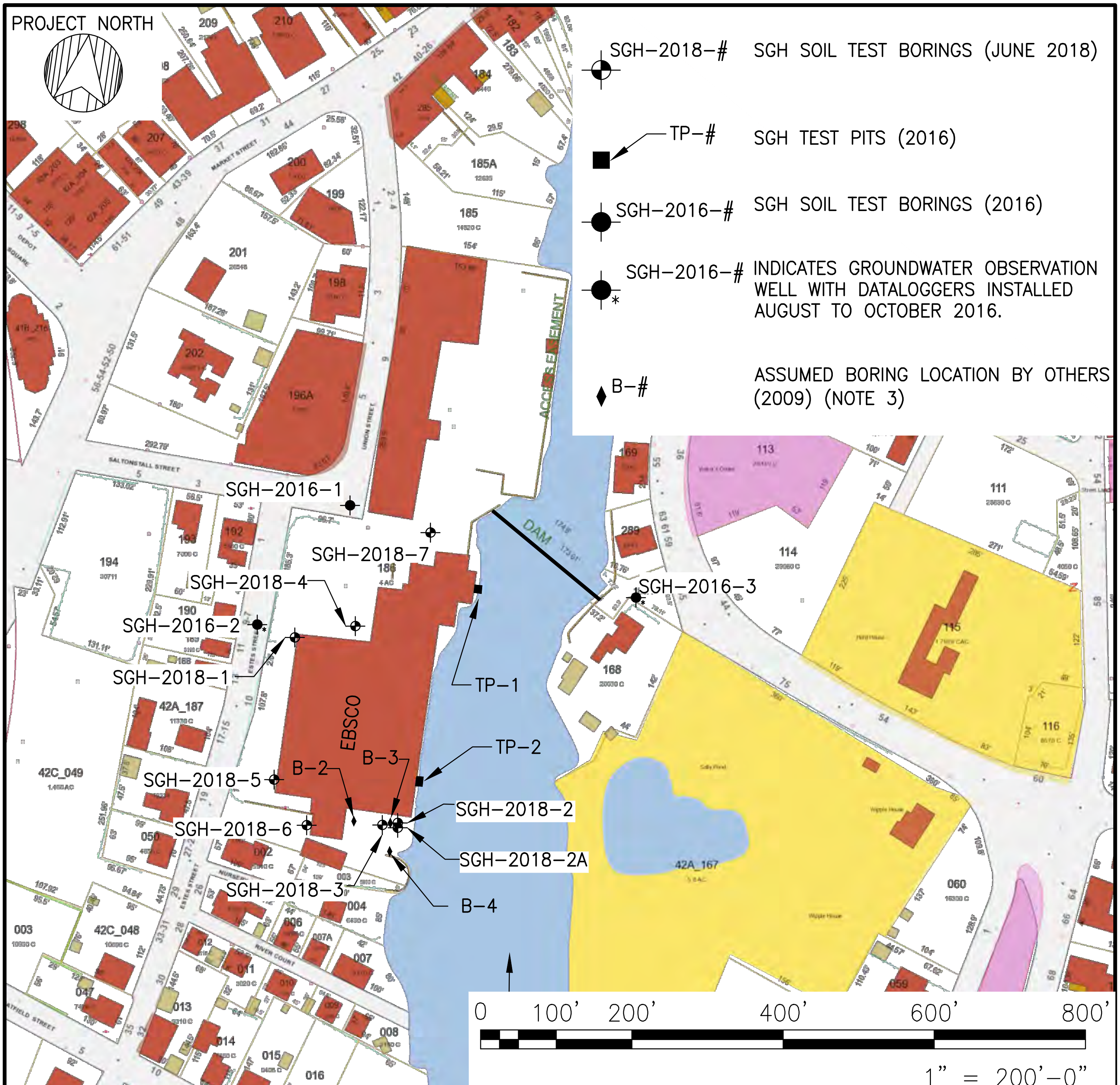
Photo 3

Drill rig set up at SGH-2018-3 on the south elevation of Building No. 10-A, adjacent to the Ipswich River. Arrow indicates the general location of Soil Test Borings SGH-2018-2 and SGH-2018-2A.

Photo 4

Split spoon sample collected in the Lower Silty Clay stratum at Soil Test Boring SGH-2018-2.

FIGURES



NOTES:

1. BASE PLAN OBTAINED FROM MASS GIS ONLINE VIEWER.

2. SGH TEST PIT AND BORING LOCATIONS ARE APPROXIMATE.

3. LOCATIONS OF THE BORINGS PERFORMED BY OTHERS IN 2009 ARE ESTIMATED FROM THE MEMORANDUM FROM GEI TO THE TOWN OF IPSWICH RE: EVALUATION OF POTENTIAL IMPACTS ON EBSCO BUILDINGS FROM PROPOSED REMOVAL OF IPSWICH MILLS DAM DATED 13 DECEMBER 2013 REVISED 14 FEBRUARY 2014.

SIMPSON GUMPERTZ & HEGER
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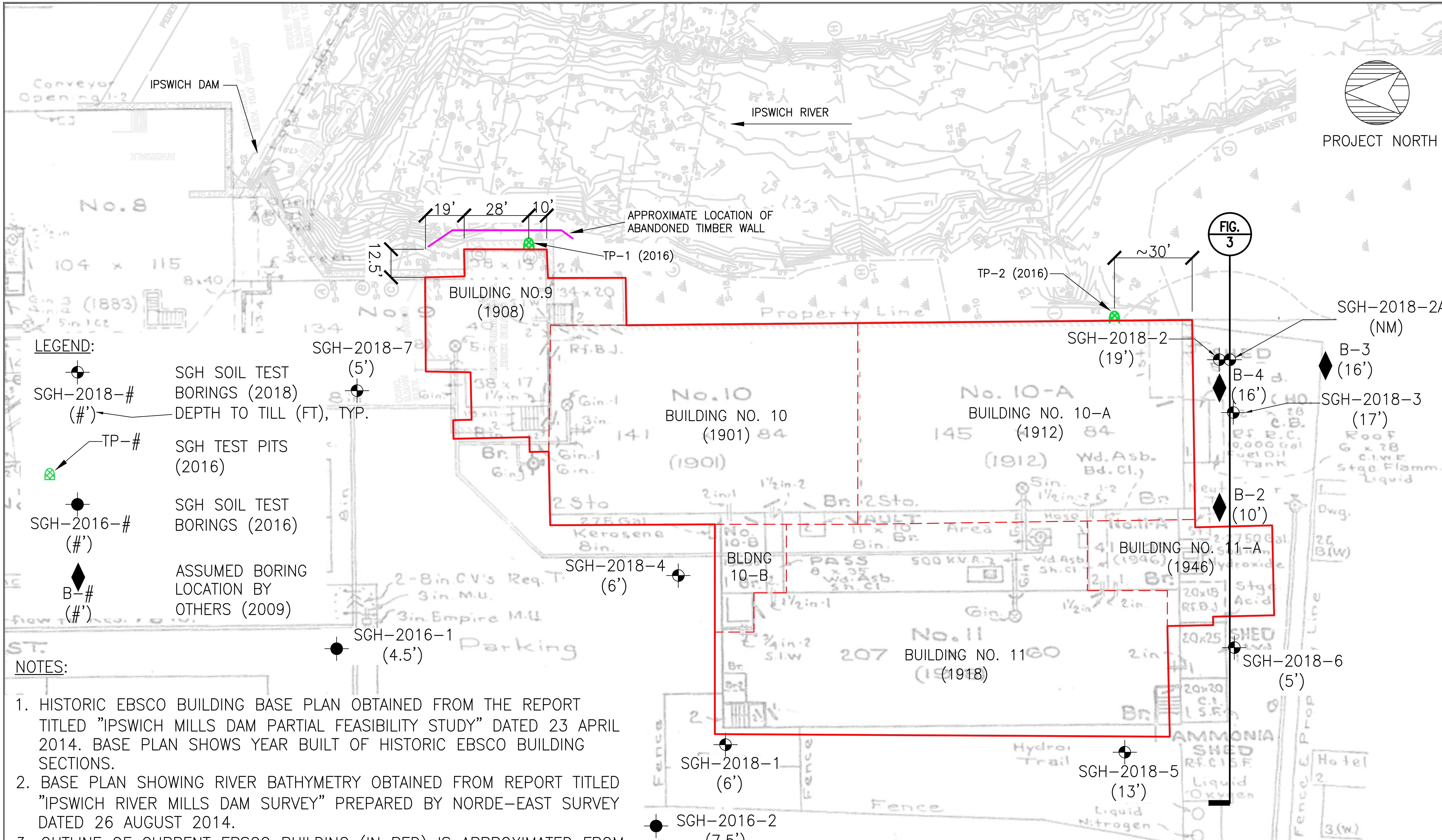
Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	SUBSURFACE INVESTIGATION LOCATION PLAN
Drawn:	ZKB
Checked:	SFK
Approved:	
Project No.:	160630.01
Drawing No.:	FIG. 1
Scale:	1" = 200'
Date:	06/05/18



NOTES:

1. HISTORIC EBSCO BUILDING BASE PLAN OBTAINED FROM THE REPORT TITLED "IPSWICH MILLS DAM PARTIAL FEASIBILITY STUDY" DATED 23 APRIL 2014. BASE PLAN SHOWS YEAR BUILT OF HISTORIC EBSCO BUILDING SECTIONS.
2. BASE PLAN SHOWING RIVER BATHYMETRY OBTAINED FROM REPORT TITLED "IPSWICH RIVER MILLS DAM SURVEY" PREPARED BY NORDE-EAST SURVEY DATED 26 AUGUST 2014.
3. OUTLINE OF CURRENT EBSCO BUILDING (IN RED) IS APPROXIMATED FROM MASS GIS ONLINE VIEWER AND LIMITED FIELD MEASUREMENTS NEAR TEST PITS.
4. DASHED LINES INDICATING EXTENTS OF HISTORIC EBSCO BUILDING SECTIONS ARE APPROXIMATE.
5. NM = NOT MEASURED

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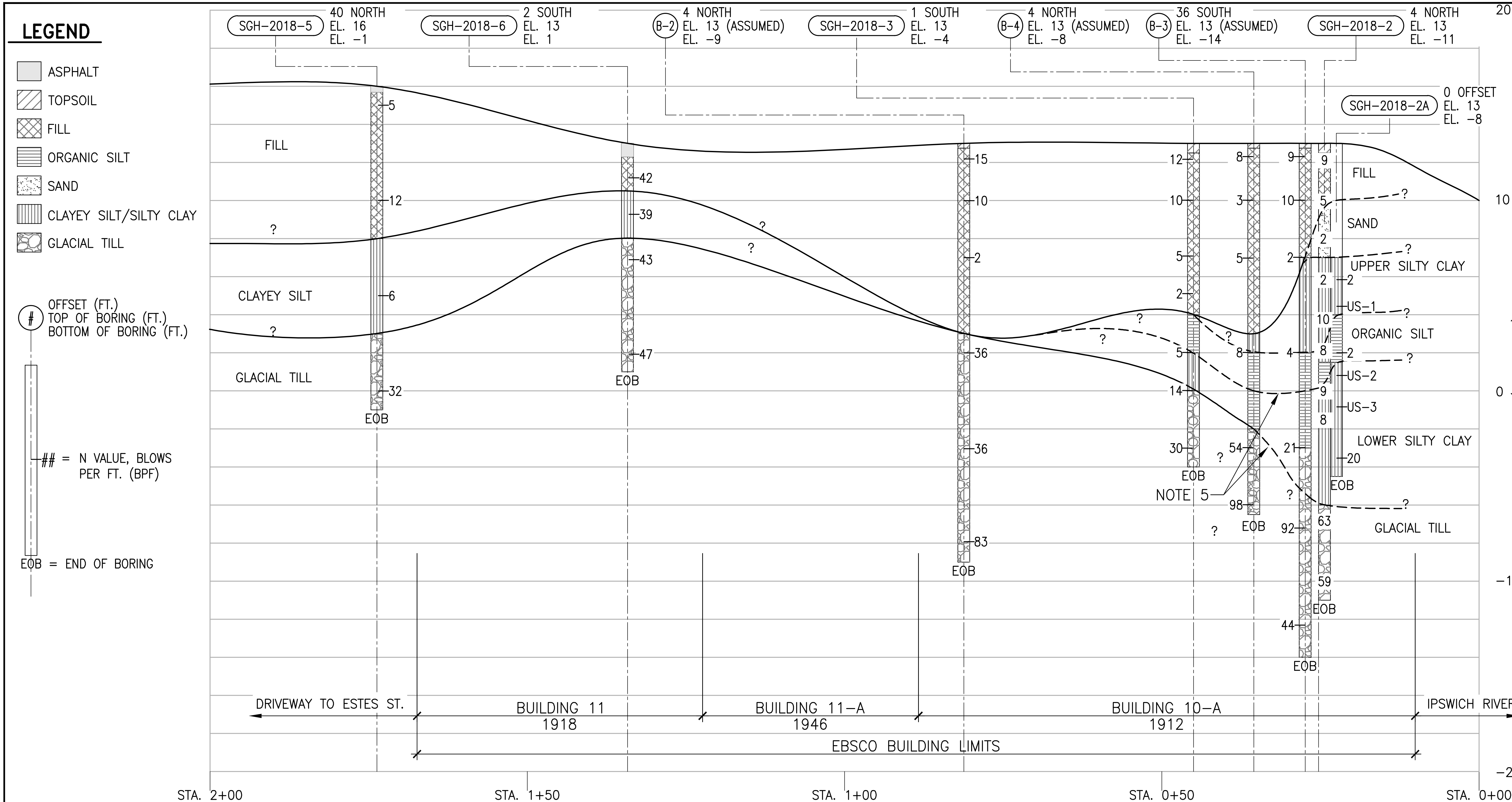
Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	EBSCO FACILITY AND INVESTIGATION LOCATION PLAN
Drawn:	ZKB
Checked:	SFK
Approved:	GAZ
Project No.:	160630.01
Drawing No.:	FIG. 2
Scale:	1"=40'-0"
Date:	06/20/18



NOTES:

1. ALL ELEVATIONS ARE ESTIMATED AND REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
2. SOIL STRATA ARE GENERALIZED PROFILES INTERPRETED FROM BORING LOGS PREPARED BY SGH AND OTHERS. REFER TO SOURCE BORING LOGS FOR MORE DETAILED SOIL DESCRIPTIONS.
3. STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES; TRANSITIONS MAY BE GRADUAL.
4. STATION NUMBERS FOR SOIL PROFILE ASSUME STATION 0+00 IS LOCATED AT THE EAST ELEVATION OF THE EBSCO FACILITY ADJACENT TO THE RIVER (I.E. THE RIVERFRONT FOUNDATION WALL). SEE FIGURE 2 FOR SECTION ALIGNMENT.
5. ORGANIC SILT AND CLAY STRATUM THICKNESS HAVE BEEN MODIFIED DUE TO THE LACK OF CONTINUOUS SAMPLING THROUGH THE ORGANIC SILT STRATUM PERFORMED BY OTHERS IN 2009, AT B-3 AND B-4.

SCALES
HORIZONTAL 1" = 15'
VERTICAL 1" = 5'

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Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA				Drawing No.: FIG. 3
Title: SOIL PROFILE – EBSCO FACILITY				Scale: AS NOTED
Drawn: CRB	Checked: ZKB/SFK	Approved: GAZ	Project No.: 160630.01	Date: 06/20/2018

APPENDICES

Appendix A

Report titled Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts, EBSCO Facility Building, Ipswich, MA” prepared by SGH dated 17 February 2018 and revised 20 February 2018

Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts

EBSCO Facility Building
Ipswich, Massachusetts

17 February 2017
(Revised 20 February 2018)

SGH Project 160630

COMMONWEALTH OF MASSACHUSETTS
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PREPARED FOR:

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PREPARED BY:

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SIMPSON GUMPERTZ & HEGER
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17 February 2017
(Revised 20 February 2018)

Mr. Neal Price
Senior Hydrogeologist / Senior Project Manager
Horsley Witten Group
90 Route 6A
Sandwich, MA 02563

Project 160630 – Feasibility Study and Conceptual Plan for Ipswich Mills Dam Removal: Evaluation of Potential Impacts on the EBSCO Facility Building Foundations, Ipswich, MA

Dear Mr. Price:

Enclosed please find our report regarding the evaluation of the potential impact(s) of the proposed removal of Ipswich Mills Dam, and the subsequent lowering of water levels upstream of the dam, on the presumed timber pile foundations of the EBSCO Information Services facilities immediately upstream of the dam. This report summarizes our observations, findings, and conclusions.

If additional information becomes available, we reserve the right to supplement or modify the material presented herein.

Sincerely yours,

COMMONWEALTH OF MASSACHUSETTS
WILLIAM P. KONICKI 32170
REGISTERED PROFESSIONAL ENGINEER

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Senior Staff II – Structures
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EXECUTIVE SUMMARY

The Ipswich Mills Dam is a run-of-the-river dam on the Ipswich River located in downtown Ipswich, Massachusetts. The dam is being considered for removal, and the purpose of the current study is to evaluate the impact the proposed dam removal could have on adjacent structures, specifically the EBSCO Information Services (EBSCO) facilities located on the west bank of the river immediately upstream of the dam.

A partial feasibility study for the Ipswich Mills Dam removal performed in 2014 opined that at least a portion of the EBSCO Facility may be supported by timber pile foundations. This opinion was based on borings performed in 2009, which encountered soft compressible soils, including peat, at the south end of the building, and the era of building construction, which was completed between 1901 and 1918. A foundation supported by untreated timber piles, if present, could be impacted by lowered water levels resulting from the removal of the Ipswich Mills Dam because the exposure of currently submerged piles could instigate fungal decay of the pile tops, resulting in settlement of the building. Additionally, lowered water levels could result in increased vertical stresses on the soil, leading to settlement of slabs-on-grade, shallow footings, buried utilities, or other buried structures, due to consolidation of soft compressible soils below the buildings, similar to the soils observed at the south end of the site.

The objective of SGH's scope of work for this project is to investigate the hypotheticals mentioned in the prior paragraph and thereby evaluate the potential impacts of the proposed dam removal on the EBSCO Facility. Our study includes investigation of the exterior foundation wall at the property line between the EBSCO Facility and the Ipswich River (referred to as the riverfront foundation wall in this report). SGH's scope of work is part of a larger feasibility study and concept plan for the dam removal, led by the Horsley Witten Group, Inc. (HW). The HW Team is performing a hydraulic analysis of the river and has provided an initial, planning-level estimate for the lowest water level likely to occur under a post–dam-removal scenario.

In August 2016, SGH performed a field investigation that consisted of two test pits located adjacent to the EBSCO Facility and excavated from the river side, and three soil test borings (two borings drilled uphill from the EBSCO Facility on the public street adjacent to the EBSCO property line, about 50 to 100 ft from the EBSCO Facility, and one boring drilled on the opposite side of the river on town property). A more comprehensive subsurface investigation was initially considered to better observe the different portions of the EBSCO Facility constructed in 1901, 1908, 1912, and 1918, as well as note any differences between interior and exterior foundations. However, due to funding and access constraints to the EBSCO facility, the current subsurface program was designed to avoid EBSCO Facility property areas in active use.

The following findings were developed from our subsurface investigation:

- Timber piles were not observed in our two test pits excavated in August 2016. The riverfront wall foundations of Building Nos. 9 and 10-A of the EBSCO Facility are bearing on rock and/or are bearing on competent soils or piled foundations at an elevation lower than the currently estimated low water level of the Ipswich River at the site after dam removal. Therefore, no indication was observed at our two test pit locations of the potential for fungal attack of timber piles in a post–dam-removal scenario.

- If timber piles exist at other locations supporting the EBSCO Facility, it is anticipated that the tops of the timber piles are at a low enough elevation to remain submerged after dam removal and, therefore, fungal deterioration of the tops of the timber piles would not occur.

- The three borings directed by SGH in August 2016 were located outside of the EBSCO site and did not encounter soft compressible soils. The 2009 borings performed by others at the southeast corner of the EBSCO Facility indicate the presence of localized soft compressible soils, including organics and clay, along the riverfront, which is common in riverfront settings. Where soft compressible soils are present, lowered groundwater levels could potentially result in settlement of pavement, slabs-on-grade, structures on shallow soil-bearing spread footings, or buried utilities supported above the soft compressible soils. We estimate a potential settlement of the soft compressible soils of approximately 1 in., 2.5 in., and 3.5 in. due to a water level drawdown of 1 ft, 2 ft, and 3 ft respectively (i.e. groundwater level at El. 5 ft, El. 4 ft, and El. 3 ft respectively). At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. We estimated the settlement assuming average soil properties from a range of published values for organic silt and clay.

Based on the results of the current investigation, we identify the following three options for the project team to determine next steps in the feasibility study for the Ipswich Dam removal:

- Option 1 – Maintain Current Groundwater Level During Post–Dam Removal. This option presents the least amount of risk for settlement due to timber pile deterioration or consolidation of compressible soils, if present, at the EBSCO Facility. Groundwater levels measured during our investigation were approximately El. 6 ft, therefore maintaining this groundwater elevation would likely not result in adverse impacts to the EBSCO Facility. Maintenance of current groundwater levels at approximately El. 6 ft would require evaluating appropriate approaches to dam removal or other engineered solutions such as groundwater recharge. Additional subsurface investigation would be required to evaluate the feasibility of applicable engineered solutions. This option also requires continuous monitoring of groundwater levels and structure movement to verify performance after the dam is removed, for the life of the structure.

If the project team anticipates that the post–dam-removal groundwater levels cannot be maintained at or above El. 6 ft, then one of the following two options may be implemented to determine risks to the EBSCO Facility and develop mitigation options if needed:

- Option 2 – Pre–Dam-Removal Supplemental Subsurface Investigation. This involves completing a supplemental foundation investigation in the building areas that were not accessible during the current investigation. Performing this investigation prior to completing the feasibility study for the dam removal would provide actionable information to perform a better assessment of the likelihood of the need for mitigation options, as it would allow the project team to identify whether timber piles are present in the remaining areas of the EBSCO Facility where test pits have not been performed, and would also allow to determine if soft compressible soils are present within the footprint of the EBSCO Facility. We consider that this option lowers the risk of adverse impacts from dam removal as it allows for timely planning and budgeting for mitigation, if needed, during the initial design phases of the project. The extent of post–dam-removal movement monitoring required to confirm adequate performance of the building would be determined based on the results of the supplemental subsurface investigation.

  An outline of the recommended supplemental investigation is included in Appendix A. We estimate that the order-of-magnitude cost for the supplemental investigation as outlined would be approximately $200,000, assuming adequate access for the investigation, minor dewatering required for test pits, and replacement of the concrete slab and asphalt pavement cut penetrations.

  If EBSCO does not provide access to the inside of its facility and access for test pit investigations on the exterior of the facility, then a limited soil test boring investigation could be performed on the building exterior. The limited investigation would include five to ten soil test borings drilled in the EBSCO Facility parking lot and other exterior areas near the building, such as the grassed area at the south end of the building. The soil test borings would provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to the potential presence of compressible soils and timber piles, if any are deemed to be present. We estimate that the order-of-magnitude cost for the limited supplemental investigation would be approximately $50,000.

- Option 3 – Perform Pre– and Post–Dam-Removal Precision Movement Monitoring, No Supplemental Subsurface Investigation. We understand a staged drawdown test in combination with precision movement monitoring could be performed for an extended period of time prior to dam removal. The pre–dam-removal precision movement monitoring would help establish a baseline against which to compare post–dam-removal performance.

  Under this option, planning for the dam removal project would proceed without further information about the foundations in building areas outside the current study, and also without further information regarding the presence of soft compressible soils within the EBSCO Facility. The project team would rely solely on pre– and post–dam-removal precision movement monitoring to assess the building performance and determine if mitigation measures are required. Precision movement monitoring helps identify problem areas; however, limits to accuracy, access, and duration of monitoring make this a more reactive approach compared to the other options. We consider that this option results in a higher risk of potential unmitigated settlement of the building because some distress to the building utilities, adjacent structure and/or slab-on-grade may occur before the post–dam-removal precision monitoring program detects measurable movement. In addition, there is a higher risk of significantly underestimating or overestimating the costs of mitigation. We note that if post–dam-removal mitigation measures are required, the costs are more likely to be higher than had mitigation been performed pre–dam removal, as the costs of repairs of any building distress (cracks, unlevelness, etc.) would need to be included. This also requires access to the interior of the EBSO Buildings to install monitoring points and at each round of survey of the monitoring points over an extended period of time.

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APPENDICES

1. INTRODUCTION

All elevations in this report are in feet referenced to the North American Vertical Datum of 1988 (ft NAVD 88) unless otherwise noted. Elevations in reports by others are reported in feet referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29). We converted elevations referenced to NGVD 29 to elevations referenced to NAVD 88 using an offset of +0.8 ft.

1.1 Background

The Ipswich Mills Dam is a run-of-the-river dam on the Ipswich River located in downtown Ipswich, Massachusetts, approximately 4 mi from the river terminus at the Atlantic Ocean. The dam was originally constructed to provide power for local industry, but no longer serves this purpose. The dam is being considered for removal, and the purpose of the current study is to evaluate the impact the proposed dam removal could have on adjacent structures, specifically the EBSCO Information Services (EBSCO) facilities located on the west bank of the river immediately upstream of the dam.

On the EBSCO campus, the brick masonry building located at 10 Estes Street (referred to as the EBSCO Facility in this report) is located along the property line abutting the Ipswich River. The EBSCO Facility is a two-story structure with brick masonry exterior bearing walls and timber floor framing supported on interior timber columns. The first-floor level (i.e., top of slab) is located approximately 3 ft above the current impounded river levels (El. 12 ft +/-). Asphalt-paved parking and employee access areas are located to the west and north of the EBSCO Facility, and a brick pavement patio area is located adjacent to the north entrance.

A partial feasibility study for the Ipswich Mills Dam removal performed in 2014 concluded that at least a portion of the EBSCO Facility may be supported by timber pile foundations. This conclusion was based on borings performed in 2009, which encountered soft compressible soils, including peat, at the south end of the building, and the era of building construction, which was completed between 1901 and 1918.

A foundation supported by untreated timber piles could be impacted by lowered water levels resulting from the removal of the Ipswich Mills Dam – as the currently submerged piles became exposed, rapid fungal decay of the pile tops could ensue, resulting in settlement of the building. Additionally, lowered water levels could result in increased vertical stresses on the soil, leading to settlement of slabs-on-grade, shallow footings, buried utilities, or other buried structures and buried utilities due to consolidation of soft compressible soils below the buildings, similar to the soils observed at the south end of the site.

1.2 Objective

The objective of our work is to evaluate the potential impacts of the proposed dam removal on the EBSCO Facility located adjacent to the Ipswich Mills Dam. Our study includes investigation of the exterior foundation wall at the property line between the EBSCO Facility and the Ipswich River (referred to as the riverfront foundation wall in this report).

Simpson Gumpertz & Heger Inc. (SGH's) scope of work is part of a larger feasibility study and concept plan for the dam removal, led by Horsley Witten Group (HW).

1.3 Scope of Work

Our scope of work included the following:

- Review prior reports prepared by others related to the EBSCO Facility; see Appendix B and Appendix C.
- Perform a field investigation, which included the following:
    - Excavate two test pits in the Ipswich River immediately adjacent to the EBSCO Facility riverfront foundation wall.
    - Perform three soil test borings, located near the EBSCO campus on the west bank of the river and near the Ipswich Mills Dam on the east bank of the river.
    - Install two groundwater observation wells and temporarily monitor groundwater levels for a period of nearly two months.
- Prepare this report.

2. DOCUMENT REVIEW

2.1 Ipswich Mills Dam Partial Feasibility Study (2014)

We reviewed the report titled "Ipswich Mills Dam Partial Feasibility Study" prepared by Horsley Witten Group, GEI Consultants, and Clean Soils Environmental Ltd, dated 23 April 2014 (Appendix B). SGH assisted GEI in reviewing existing building foundations for the partial feasibility study. We note the following pertinent information related to the foundations of the EBSCO Facility:

- The dam is likely constructed on top of or at the toe of a rock ledge. At the time of the study the extent of the rock ledge was not well understood, but the study concludes that the height of the rock ledge will likely be a primary factor in determining the normal or low water surface elevation if the dam is removed.

- The elevation of the dam spillway crest and normal pool water surface is El. 8.9 ft.

- The elevation of the dam gated outlet and the average upstream river bed elevation at the upper falls is El. 6.7 ft.

- The average bed elevation at the dam toe is El. 2.9 ft. The Ipswich river is tidal downstream of the dam.

- The Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) dated 1985 reports that the 500 yr recurrence interval flood results in a water surface at El. 13.9 ft near the Ipswich Mills Dam.

- The EBSCO Facility is composed of four buildings constructed at different time periods. Building Nos. 9, 10, 10-A, and 11 were constructed in 1908, 1901, 1912, and 1918 respectively. Building No. 9 was partially demolished. Building Nos. 9, 10, and 10-A abut the Ipswich River upstream of the dam.

- EBSCO commissioned a study to determine the feasibility of constructing a building addition in 2009. The study included three soil test borings reportedly performed at the south end of the building near the river. The 2009 soil test borings show the following:

  - At the two borings near the river (B-3 and B-4), there is 5 to 10 ft of very loose to loose fill, over 5 to 10 ft of soft to firm (medium stiff) clay and peat, over very dense glacial till. The top of the till is located approximately at 15 ft below ground surface (bgs) at both borings.
  - At the boring located further from the river (B-2), there is 10 ft of very loose to medium dense fill over very dense glacial till.

- GEI concluded that the soft compressible soils are inadequate to support the EBSCO Facility addition and that, taking into consideration the age of the buildings, the EBSCO Facility is likely supported by timber piles.

- GEI did not observe signs of distress in the EBSCO Facility facade based on limited visual observations.

- GEI noted the following: "We observed some dishing of the first floor slab indicating that the slab is a slab-on-grade and that the soils underlying the slab are compressible and had settled."

- GEI recommended further investigation of the EBSCO Facility foundations, including but not limited to test pit investigation, probing along the foundation wall, lowering the impoundment to provide access for visual observation of the foundation wall, and performing core sampling of the foundation elements.

2.2 Bathymetric Survey (2014)

We reviewed the report titled "Surveyor's Report for Ipswich Mills Dam Survey" prepared by Norde-East Survey, dated 26 August 2014 (Appendix C), which shows several transects that extend the full width of the river upstream of the dam. We note the following pertinent to the foundations of the EBSCO Facility:

- The results of the bathymetric survey performed approximately 15 ft upstream of the dam, apparently located on the rock ledge, show the following:

  - Soundings immediately adjacent to the dam show sediment thicknesses ranging between 0 and 1 ft thick.
  - Transect AI (approximately 170 ft long in a northwest to southeast direction) shows the river bed surface ranging between approximately El. 5.5 ft and El. 7.5 ft.

- The results of the bathymetric survey performed near the north (downstream) end of the EBSCO Facility show the following:

  - Soundings S-29 and S-30 show sediment ranging between 0.5 ft and 0.8 ft thick and consisting of silt to gravel and sand, respectively.
  - Transect D (approximately 150 ft long in a west to east direction) shows the river bed surface at approximately El. 6 ft near the EBSCO Facility and a low point at approximately El. 1 ft near the middle of the river.
  - Transect E (approximately 135 ft long in a west to east direction) shows the river bed surface at approximately El. 7 ft near the EBSCO Facility and a low point at approximately El. 0.5 ft near the middle of the river.

- The results of the bathymetric survey performed near the south (upstream) end of the EBSCO Facility show the following:

  - Sounding S-9, located near the EBSCO Facility shows sediment 1.0 ft thick, and consisting of silt.
  - Transect I (approximately 150 ft long in a west to east direction) shows the river bed surface at approximately El. 6 ft near the EBSCO Facility and a low point at approximately El. 2 ft near the middle of the river.

3. FIELD INVESTIGATION

SGH performed a field investigation between 22 and 24 August 2016. The field investigation consisted of two test pits adjacent to the EBSCO Facility and excavated from the river side, and three soil test borings (two borings drilled uphill from the EBSCO Facility and one boring drilled on the opposite side of the river on town property). Site access was coordinated with various personnel from EBSCO, HW, the Ipswich River Watershed Association, and the Ipswich Department of Public Works.

The EBSCO Facility incorporates four building structures (Buildings 9, 10, 10-A, and 11), each constructed at a different time, reportedly 1901, 1908, 1912 and 1918. As such, foundation conditions may vary from building to building. Due to access and budget constraints, we performed a limited investigation that included two test pit locations, both located at the riverfront foundation wall (Buildings 9 and 10-A), and three soil test borings drilled on public property. Two of the three soil test borings were located on Estes Street and Union Street outside the EBSCO property line, about 50 to 100 ft from the EBSCO Facility. In addition, access constraints also prevented SGH from obtaining sample cores of the EBSCO Facility foundation walls.

3.1 Test Pit Investigation

On 22 through 24 August 2016, Mr. Steven Keppel from SGH visited the site to observe and document the conditions at exterior test pits excavated by Pepperell Cove Marine of Portsmouth, New Hampshire (PCM). Ms. Mary Donlon from SGH also assisted with the field investigation on 24 August 2016.

The project team coordinated a drawdown of the dam impoundment during the test pit investigation. Drawdown started on the afternoon of 22 August. Prior to drawdown, the impoundment level was at approximately El. 8 ft (Photo 1). On 24 August around 3:00 p.m., the impoundment level was at approximately El. 6.2 ft (Photo 2). During the investigation, SGH measured impoundment level elevations with assistance from the IRWA using the on-site staff gauge located on the east bank upstream of the dam. We estimated test pit elevations based on the impoundment level measurements.

Both test pits were excavated in the Ipswich River adjacent to the EBSCO Facility. The test pit investigations were limited to observation of the riverfront foundation wall of the EBSCO Facility.

PCM performed underwater excavation, removing sediment with a handheld airlift or pressurized water jet and removing larger debris and small rip-rap by hand (Photos 3 and 4). Test pit locations are shown in Figs. 1 and 2. Test Pit No. 1 (TP-1) was located upstream of the dam at the north end of the EBSCO Facility (i.e., Building No. 9 constructed in 1908). Test Pit No. 2 (TP-2) was located upstream of the dam at the south end of the EBSCO Facility (i.e., Building No. 10-A constructed in 1912). Cross section sketches of each test pit are included in Figs. 3 and 4. Visibility underwater was limited and elevations of submerged soils and structures were estimated by PCM at the direction of SGH. We verified elevations with probe measurements where possible.

We summarize our field observations during test pit excavation as follows:

- Subsurface conditions in the river adjacent to the EBSCO Facility generally consist of very soft sediment overlying rip-rap. We observed the following at each test pit:

    - At TP-1, the sediment above the rip-rap is approximately 1.5 ft thick and consists of dark brown, very soft silt with trace debris (glass and brick). The rip-rap consists of subangular stones ranging between 3 in. and 12 in. diameter. PCM removed rip-rap between El. 5.2 ft and El. 3.2 ft and was unable to advance TP-1 beyond El. 3.2 ft (i.e., test pit refusal). PCM was unable to widen the test pit further by hand; however, we probed and determined that test pit refusal was likely due to a rock ledge at El. 3.2 ft.
    - At TP-2, the sediment above the rip-rap is approximately 4.5 ft thick and consists of dark brown, very soft silt with some organics, some clay, and trace debris (glass and brick). The rip-rap encountered at the bottom of the excavation consists of subangular stones generally larger than 12 in. diameter. PCM was unable to remove the rip-rap by hand at TP-2 and reached refusal at El. –0.5 ft.
    - We confirmed that the EBSCO Facility riverfront foundation wall extended to the bottom of each test pit (El. +3.2 ft and El. –0.5 ft at TP-1 and TP-2, respectively).

- Drawdown of the impoundment exposed a portion of the EBSCO Facility riverfront foundation wall that is typically submerged (Photos 5 and 6). We did not observe cracking or other indications of settlement of the concrete foundation wall. At approximately El. 10 ft and below, we observed staining and slight erosion of the concrete surface. At TP-1, the erosion of the concrete is most severe, up to 2 in. deep, at about El. 9.5 ft. The erosion of the concrete appears less severe below approximately El. 7 ft. We scanned the concrete foundation wall near TP-1 with a magnetic detection device (model Structure Scan Mini manufactured by GSSI). We were unable to detect steel reinforcement within 12 in. of the surface; however, our readings may have been impacted by latent moisture in the concrete.

- At both test pits, we observed a buried timber structure that appears to be the remnants of abandoned concrete formwork (Photo 7). The timber formwork consists of stacked horizontal planks supported by 3 in. by 3 in. vertical posts spaced at 2 ft o.c. At TP-1, the top of the remnant formwork was located at approximately El. 5.7 ft and it extended down to the bottom of the test pit (El. 3.2 ft). At Test Pit No. 2, the top of the remnant formwork was located at approximately El. 0.8 ft. The bottom of TP-2 was located at El. –0.5 ft, and we did not uncover the bottom of the remnant formwork.

- At TP-1, we observed a second timber structure, located parallel to and similar in construction to the abandoned formwork. The function of this abandoned timber wall structure is unknown. We understand from HW that the IRWA and EBSCO believe that it may be part of an abandoned dock. The abandoned timber wall is located in the river approximately 10 +/- ft from the EBSCO Facility and is shown on Fig. 2. We observed timber struts approximately 7 in. by 7 in. by 6 ft long and spaced approximately 6 ft on center connected to the abandoned wall with lapped joints. Based on probing, we estimate that the rip-rap was placed the full width between the formwork and the abandoned timber wall.

3.2 Exploratory Soil Test Borings

SGH performed a subsurface investigation at the project site on 24 August 2016. The subsurface investigation consisted of three soil test borings (Soil Test Borings SGH-2016-1 to SGH-2016-3) and installation of two groundwater observation wells (at Soil Test Borings SGH-2016-2 and SGH-2016-3). Drilling of the soil test borings and installation of observation wells were performed by Carr-Dee Corporation (Carr-Dee), of Medford, Massachusetts, under SGH supervision.

3.2.1 Soil Test Borings

Soil test boring locations are shown in Fig. 1. Two soil test borings were drilled on Saltonstall Street (Photo 8) and Estes Street, adjacent to the EBSCO campus; and one soil test boring was drilled in the gravel driveway located on the town right of way access to the dam adjacent to the private residence located at 69 South Main Street. A representative from SGH was present throughout the field work to observe drilling, assist in obtaining samples, and prepare a descriptive log of each test boring. The sampling intervals, soil descriptions, Standard Penetration Test (SPT) blowcounts, and other pertinent field data are summarized in the individual soil boring logs included in Appendix D.

All soil test borings were drilled using a truck-mounted drill rig. A 4 in. inside diameter hollow stem auger was used for drilling. Soil samples were obtained using 2 in. O.D. split spoon samplers driven using a 140 lb hammer falling 30 in. with a rope cathead.

Soil test borings were extended to refusal of the hollow stem auger to depths ranging between 10.5 ft bgs and 16.5 ft bgs. Typically, continuous SPT samples were obtained from ground surface to the end of the boring. We did not encounter groundwater in Soil Test Boring

SGH-2016-1 (Saltonstall Street) and SGH-2016-2 (Estes Street). All samples were secured, sealed, and transported to the SGH office.

3.2.2 Groundwater Observation Wells

Carr-Dee installed two groundwater observation wells at Soil Test Borings SGH-2016-2 (Estes Street) and SGH-2016-3 (S. Main Street). The observation wells extended approximately to the bottom of the borings, to a depth of 10.5 ft bgs and 16.5 ft bgs respectively. Observation wells consist of 2 in. nominal diameter PVC pipe with 10 ft of slotted screen. The slotted screen is surrounded by filter sand that extends to 4 ft above the top of the slotted screen, with the exception of SGH-2016-2, which was backfilled with cuttings at 4 ft bgs and above. The remainder of the annular space around the PVC pipe was backfilled with soil cuttings and a well cover was set in grout around the top of the PVC pipe, flush with the surrounding grade. Individual observation well installation logs are included in the soil test boring logs in Appendix D.

On 24 August 2016, SGH installed data loggers in Observation Wells SGH-2016-2 (Estes Street) and SGH-2016-3 (S. Main Street) to obtain a continuous record of groundwater levels over time. We installed a Levelogger in each observation well, SGH-2016-2 (Estes Street) and SGH-2016-3 (S. Main Street), to record groundwater levels. We installed a Barologger Edge in Observation Well SGH-2016-2 to record local atmospheric pressure. All devices are manufactured by Solinst Canada Ltd. We made an additional site visit on 11 October 2016 to collect groundwater data and remove the dataloggers.

During installation of the data loggers, SGH obtained initial groundwater depth measurements with a manual Solinst water level meter. The manual measurement at Observation Well SGH-2016-3 (S. Main Street), obtained a few hours after well installation, shows the groundwater level at about 11.75 ft bgs, i.e., at about El. 2.25 ft. We did not observe groundwater within SGH-2016-2 (Estes Street) during site visits on 24 August and 11 October 2016, nor did we detect groundwater with the Levelogger during the monitoring period at this well.

4. ANALYSIS

4.1 Subsurface Conditions

Site subsurface conditions vary depending on proximity to the Ipswich River. We prepared a subsurface profile along the EBSCO Facility riverfront foundation wall based on the results of the soil test borings performed by SGH and others, our test pit investigation, and the bathymetric survey performed by others (Fig. 5). We summarize the strata at the site as follows:

- **Stratum 1 – Fill:** This stratum consists of 3 to 10 ft of fill consisting of very loose to very dense, light to dark brown, dry to wet, sand and silt to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts range from 2 to +50 blows per foot (bpf). This stratum was encountered at Soil Test Borings SGH-2016-1 through SGH-2016-3 and Soil Test Borings B-2, B-3, and B-4, which were performed by others in 2009 at the south end of the EBSCO Facility. Some organics within the fill were encountered at Soil Test Borings B-3 and B-4 located near the river.

- **Stratum 2 – Sand:** This stratum consists of 2 to 4.5 ft of medium dense to very dense, light orange brown, dry, silty sand, fine to coarse grained, uniform to well-graded, subangular. SPT blow counts ranged from 42 to 53 bpf. This stratum was encountered at Soil Test Borings SGH-2016-1 and SGH-2016-2, underlying the Fill stratum.

- **Stratum 3 – Clay and Organic Silt:** This stratum consists of 5 to 10 ft of very soft to firm, grey to black, wet, clay with trace sand to sand and silt with organics. SPT blow counts ranged from 2 to 8 bpf. This stratum was encountered at Soil Test Borings B-3 and B-4, underlying the Fill stratum.

- **Stratum 4 – Glacial Till:** This stratum consists of 3 to more than 11 ft of medium dense to very dense, light reddish brown to olive grey, dry to wet, silty clay to sandy gravel, fine to coarse, well to poorly graded, subangular, with trace silt. SPT blow counts ranged from 23 bpf to refusal. This stratum was encountered at Soil Test Borings SGH-2016-2 and SGH-2016-3, underlying the Sand stratum, at Soil Test Borings SGH-2016-1, underlying the Granular Fill stratum, at Soil Test Boring B-2, underlying the Fill stratum, and at Soil Test Borings B-3 and B-4, underlying the Clay and Organic Silt stratum.

- **Stratum 5 – Rock Ledge:** The elevation of the top of the rock ledge varies at the site and Fig. 5 is an initial estimate of rock elevations based on limited information. The test pit investigation and the bathymetric survey results indicate that rock ledge varies between approximately El. 3.0 ft and El. 7.5 ft near the dam. We did not perform rock cores for the current study.

4.2 Groundwater Conditions

We plotted the groundwater levels measured at Observation Well SGH-2016-3 and daily local precipitation data for the monitoring period between 24 August and 11 October 2016 (Fig. 6). We obtained the daily local precipitation data from the Plum Island Ecosystems Long Term Ecological Research Network (PIE LTER) for its field station in Rowley, Massachusetts.

At the time of drilling, we encountered groundwater only at Soil Test Boring SGH-2016-3, at a depth of 11.75 ft bgs. During the monitoring period, groundwater levels at Observation Well SGH-2016-3 varied between El. 5.9 ft and El. 7.0 ft (i.e., between 7 ft and 8.1 ft bgs). We did not observe groundwater within Observation Well SGH-2016-2 during site visits on 24 August and 11 October 2016, and we did not detect groundwater with the logger installed at this observation well (the logger was located near the bottom of the observation well at El. 8.5 ft).

4.3 Stability of Riverfront Foundation Wall

We performed calculations to evaluate the stability of the EBSCO Facility riverfront foundation wall against sliding and overturning under future conditions. We used the following parameters for the calculations:

- **Load Conditions**: We considered two load cases.
    - Load Case 1 – The first case considers the lowest estimated impoundment level post-dam removal (water level at El. 3 ft) and groundwater elevation behind the foundation wall also at El. 3 ft.
    - Load Case 2 – The second case considers a flood condition (water level on the building side at El. 11.4 ft and impoundment level at El. 3.0 ft). This represents an extreme condition that may occur immediately post-flood and assumes that the floodwater on the first floor drains out the doors, leaving the groundwater level close to the slab elevation on the building side of the foundation wall (which may occur short term with limited subsurface drainage), while the river level in front of the wall has receded to its lowest estimated normal water level post-dam removal.
    - For both load cases, we assumed a 100 psf uniform surcharge representing the combined load from the weight of the ground floor slab and the surface live load inside the building.

    We did not include live load from the first floor, second floor, and the roof. These live loads do not induce any lateral pressure on the foundation wall; they will only increase the vertical load acting on the foundation wall, thereby increasing the factor of safety for sliding and overturning.

    Wind loading in the east-to-west direction is assumed to be carried by the roof and floor diaphragms to the end walls. The main wind forces on the building do not load the riverfront foundation wall. Only the local wind suction will load the exposed face with wind in the west-to-east direction; this has not been included in our evaluation.

Given the age of the buildings, we anticipate that they were not originally designed for seismic loading; therefore, we did not evaluate this condition.

- **Soil Backfill:** We used a soil friction angle of 30° and a wet unit weight of 120 pcf. These parameters are consistent with loose to medium dense mixed-fill conditions. We evaluated both at-rest and active soil pressure coefficient based on Rankine theory for a conservative estimate of soil loading on the foundation wall.

- **Base of Foundation Wall to Rock Interface Friction:** We used an interface friction angle of 35° between the concrete foundation wall and rock and we assumed the interface to be horizontal.

- **Foundation Wall:** The wall section used in our analysis is shown in Figure 3, consistent with our field observations. We used a unit weight of 150 pcf for concrete material. We assumed that the wall is 1.5 ft thick at the top and about 3 ft wide at the bottom. We assumed that the retained-soil side of the wall is vertical. We estimated dead loads from the brick masonry bearing wall and floor framing above assuming that the interior columns are located approximately 28 ft from the wall. We do not have floor plans or interior measurements, and we estimated the building dead loads using building material densities from ASCE 7-10. Based on our calculations, we estimated a dead load of 3,115 lbs/ft for the building and 2,835 lbs/ft for the foundation wall.

- We assumed that the EBSCO facility floor slab provides no restraint at the top of the riverfront foundation wall (this is a conservative estimate that assumes that the slab does not contribute to the stability of the wall).

The following table summarizes the results of our stability analysis of the retaining wall:

Table 1 – Riverfront Foundation Wall – Stability Analysis Results

Condition	Minimum Factor of Safety(1)(2) against Overturning	Minimum Factor of Safety(1)(3) against Sliding
Lowest Estimated Impoundment Level Post-Dam Removal (WL = 3.0 ft)	2.5	2.5
Flood Condition (WL = El. 11.4 ft)	1.2	1.2

Notes:
(1) The factor of safety is a measure of how much capacity a system has beyond that needed to resist an applied load. The factor of safety is calculated by dividing the estimated system capacity (ability to resist loads) by the estimated applied load (demand).
(2) Typical design factors of safety for overturning of retaining walls are 2.0 for static normal operational loading conditions and 1.1 for extreme loading conditions.
(3) Typical design factors of safety for sliding of retaining walls are 1.5 for static normal operational loading conditions and 1.1 for extreme loading conditions.

4.4 Settlement of Compressible Soils

We did not encounter compressible soils within our borings performed in 2016. For our settlement analysis, we assumed a soil profile similar to the conditions encountered at Test Boring B-3 performed by others in 2009 and located on the south side of the EBSCO Facility. We assumed that the soil profile consists of a 5 ft fill stratum overlying a 6 ft thick clay stratum overlying a 4 ft thick organic silt stratum. The boring log prepared by others identifies the stratum underlying the clay as peat; however, we assumed the stratum is organic silt based on the reported soil description.

Since the boring log does not indicate that laboratory testing was performed (i.e., no consolidation testing, moisture content, or Atterberg limits), we estimated average soil properties for the clay and organic silt strata based on general published values.[1]

We estimated the potential settlement of compressible soils due to primary consolidation imposed by an increase in effective stress due to lowered groundwater levels. We did not include secondary compression of the organic soils in our calculations; if organic soils are indeed present under the EBSCO Facility, secondary compression is occurring and will continue to occur regardless of whether groundwater levels are lowered or not.

On 24 August 2016, we observed the impoundment level at El. 6.2 ft. The results of our groundwater monitoring in observation well SGH-2016-3 show that the groundwater level was approximately at El. 6 ft for the period of record (24 August 2016 to 11 October 2016). Based on our observations of the river staff gauge during our investigation in August 2016, and groundwater data collected at one observation well, we assumed for our settlement analysis that the groundwater levels near the impoundment are similar to the impoundment level. During our investigation, the impoundment was lowered to El. 6.5 +/- ft for a period of approximately one month. Therefore, assuming that the overburden soils under the EBSCO Facility have experienced groundwater levels as low as El. 6 ft, we calculated the range of potential settlement of the clay and organic soils resulting from a 1, 2, and 3 ft drop in groundwater levels. We understand that H&W has not yet completed the hydraulic study of post-dam-removal river levels; however, the preliminary estimated lower-bound water level after the proposed dam removal is at approximately El. 3 ft, and will likely be higher (between El. 3 ft and El. 6 ft). The water level is subject to change pending results from the hydraulic analysis performed by HW.

Table 2 summarizes the soil properties used in our analysis:

[1] We estimated soil properties from published values in *An Introduction to Geotechnical Engineering* by Holtz and Kovacs (1981) and *Geological Background and Engineering Parameters of Boston Blue Clay* by Connors of the University of Massachusetts at Lowell (1993).

Table 2 – Assumed Clay and Organic Silt Strata Consolidation Parameters(1)

Soil Stratum	Boring ID	Depth to Mid-Layer [ft, bgs]	σ'_{vo}(1) [psf]	σ'_{vf}(1) [psf]	Over Consolidation Ratio OCR	Initial Void Ratio e_0	Average C_c(2)
Clay	B-3	8	529	898	1.0	1	0.350
Organic Silt	B-3	13	797	1,166	1.0	4	2.750

1. σ'_{vo} is the estimated existing overburden or in situ vertical effective stress at midlayer (prior to dam removal). σ'_{vf} is the estimated vertical effective stress after dam removal (lowered groundwater level).
2. C_c is the primary consolidation index.

Based on the assumptions listed above, we estimate that the average settlement due to lowering groundwater levels by 1 ft, 2 ft, and 3 ft in the post-dam removal conditions is in the order of 1, 2.5, and 3.5 in., respectively, in those areas where compressible soils are present.

5. DISCUSSION

5.1 EBSCO Facility Riverfront Wall Foundation

As part of the feasibility study, HW will be performing a hydrologic and hydraulic (H&H) analysis of the river and at present has not yet determined anticipated water levels near the EBSCO Facility after the proposed dam removal. We understand that the post-dam-removal river level will vary substantially depending on the river bed elevations and the local climate. HW provided a preliminary estimate that the low river level may be in the range of El. 3 ft to El. 6 ft after dam removal (elevation is subject to change pending the results of the H&H analysis).

Table 3 below summarizes elevations pertinent to the EBSCO Facility:

Table 3 – Water Level and EBSCO Foundation Wall Elevations Along the Riverfront Foundation Wall

Description	Elevation [ft, NAVD 88]
Top of Foundation Wall at Building No. 10-A.	12.5
Maximum Elevation of Bottom of Foundation Wall at Building No. 10-A / Bottom of Test Pit No. 2 (TP-2)	–0.5
Approximate Top of Glacial Till Stratum at South End of Building No. 10A	-5.0
Top of Foundation Wall at Building No. 9.	11.4
Apparent Bottom of Foundation Wall at Building No. 9 / Bottom of Test Pit No. 1 (TP-1)	3.2
Estimated Low River Level Elevation After Dam Removal (Preliminary Estimate from HW)	3 to 6
Dam Crest	8.9
Average Elevation of Rock Ledge at Toe of Dam	2.9
Top of Abandoned Timber Formwork and Abandoned Timber Wall	5.7

Based on the information collected to date, we expect that the exterior concrete foundation wall of the EBSCO Facility, which abuts the Ipswich River (referred to as the riverfront foundation wall in this report), will not be significantly impacted by the expected water-level drawdown due to removal of the dam.

The riverfront foundation wall of the EBSCO Facility appears to be bearing on rock at the north end (Building No. 9). The elevation of rock ledge detected at TP-1 (El. 3.2 ft) is consistent with the average elevation reported at the toe of the dam (El. 2.9 ft), and the low points of the river transects near TP-1 (El. 0.5 ft to El. 1 ft).

At the south end of the EBSCO Facility (Building No. 10-A), the riverfront foundation concrete wall extends below El. –0.5 ft, which is about 8.4 ft below the current normal river pool elevation. We note that the top of the glacial till stratum was encountered at about El. -5 ft in the borings performed by others at the south end of the EBSCO Facility. That is, the bottom of the riverfront foundation wall may be as much as 4.5 ft above the top of the glacial till stratum. Since it is not possible to drive timber piles any significant distance into very dense glacial till, and given the depth to which excavation was performed to construct the wall (deeper than 10 ft), it is unlikely that the riverfront foundation wall is supported on timber piles. However, even if the riverfront foundation wall at Building No. 10-A was supported on timber piles, with pile tops below El. –0.5 ft it is unlikely that the tops of the timber piles will be adversely impacted by lowered water levels after dam removal as the anticipated minimum water level due to dam removal is currently estimated to be at El. 3 ft to El. 6 ft, which is at least 3.5 ft above where the tops of the timber piles could be, if present.

The current study was limited to test pit investigations adjacent to the EBSCO Facility riverfront foundation wall at the north end (Building No. 9 constructed in 1908) and the south end (Building No. 10-A constructed in 1912). Based on visual observations of the riverfront wall and assuming the rock ledge elevations are similar along the length of the riverfront wall, the foundation construction is likely to be similar at other locations along this wall of the EBSCO Facility. However, we did not observe the foundations supporting interior walls or columns of Buildings No. 9 and 10A or the other buildings on the EBSCO campus (Building Nos. 10, 11, and 11A constructed in 1901, 1918, and 1946 respectively). Any portion of the EBSCO Facility supported on timber piles with pile top cutoffs located above the currently anticipated lowered river level (El. 3 ft to El. 6 ft), should they exist, would be subject to timber pile deterioration.

5.1.1 Mitigation of Potential Deteriorating Timber Piles

Although we did not observe timber piles supporting the EBSCO Facility riverfront foundation during our investigation, if timber piles with high cutoff elevations (i.e., top of piles above estimated lowest groundwater level) were to be present supporting interior walls, interior columns, or foundation walls for the buildings not investigated during this study, mitigation options include:

- Replace existing timber pile foundations with new micropile foundations. Drilled-in micropile foundations would be installed around the bearing walls and/or columns, extending down into the glacial till/rock strata underlying the site. Special structural connection brackets or a system of needle beams would be required to transfer the load from the bearing walls and/or columns to the micropiles. Replacing existing timber piles with new deep foundation elements represents a significant disruption to the building facility's operations, has a long construction schedule, and is very expensive compared to the more typically used cut-and-post underpinning approach described below.

- Perform cut-and-post underpinning repairs of existing timber pile foundations. Cut-and-post underpinning involves removing the top portions of the timber piles that are exposed above groundwater and replacing them with new concrete-encased steel posts. The cost of this repair is typically driven by the labor for excavating pits to access the existing foundation elements. The labor required for excavation is in turn impacted by site access and dewatering operations, among other factors. The repair would involve removing and replacing finishes around the foundation elements, excavating soils and temporarily storing/stockpiling excavated material, dewatering the access pit, mining underneath the pile cap foundation to expose the timber piles, providing temporary shoring for the existing foundation, removing the tops of all timber piles, installing concrete-filled steel posts in place of the removed timber pile tops, placing the concrete encasement around the steel posts, and backfilling the excavation. Typically a portion of the soil spoils need to be hauled and disposed of off site. Cut-and-post underpinning is typically the most effective solution to repair deteriorating timber pile tops due to lowered groundwater levels in an existing occupied structure.

- Install a groundwater recharge system to artificially raise groundwater levels to preserve the timber piles. Groundwater recharge involves installing wells and trenches to inject water into the ground to artificially raise water levels locally, thus submerging the tops of the existing timber piles. A water treatment system is required to remove impurities and biological agents that may be present in the water that could clog the system filters and screens. At this site, installing a groundwater recharge system would also involve installing a below-grade cutoff wall (e.g., jet grout wall) along the riverfront property line, and potentially on each side of the EBSCO Facility perpendicular to the river, to prevent loss of injection water toward the river. Additional engineering studies (e.g., permeability tests, injection well tests) would be required to determine if the groundwater recharge option is feasible for the EBSCO Facility. Given the need for a cutoff wall and groundwater treatment system, this option is bound to be more expensive than cut-and-post underpinning.

If mitigation of deteriorating timber piles is required at the EBSCO Facility, we consider cut-and-post underpinning to be the least costly of the options discussed above. Based on our experience on prior projects involving deteriorating timber piles, we estimate that the direct cost for cut-and-post underpinning repairs, if needed, would be on the order of $700/sq ft of foundation repaired (or $2,900/lf of foundation wall). This order-of-magnitude direct cost may vary greatly depending upon project specifics, including, but not limited to, the existing structure and subsurface conditions, the extent of the area to be repaired, access to repair areas, finishes, and any staging required to maintain building occupancy during the repair work. Also, the order-of-magnitude costs above consider the subcontracted cost, not the burdened cost to the project owner. Additional costs for general conditions, general contractor markup, owner project management, design fees, and contingencies are not included. The total burdened cost to the owner could be on the order of $1,300/sq ft (or $5,200/lf of foundation wall) assuming the following: 10% general conditions, 10% general contractor markup, 10% design fees, 50% contingency.

It is possible to perform movement monitoring of the EBSCO Facility to check the performance of the structure upon dam removal to try to detect the onset of settlement, and thus use the movement monitoring data as an indicator of the presence of timber piles. However, the rate of timber pile deterioration can be highly variable. In our experience, due to the accuracy of conventional survey methods, by the time that movement is detected, building distress, such as cracks, has already developed.

5.2 EBSCO Facility Riverfront Foundation Wall Stability

We analyzed the stability of the EBSCO Facility riverfront foundation wall and determined that the wall has enough reserve capacity to resist unbalanced loading under the proposed dam removal conditions (Table 1) and assumed water levels; our analysis indicates that the calculated factor of safety for both post-dam removal static normal operating conditions and extreme loading conditions is larger than the minimum factor of safety required for design.

Based on the preliminary lower-bound estimate for the low river level (El. 3 to 6 ft post–dam removal), erosion of the river bed could result in undermining of the rip-rap scour protection for any foundations bearing on soil (if any exist) along the EBSCO riverfront foundation wall. Pending further hydraulic analysis, it may be prudent to evaluate the need for replacement of the existing stone rip-rap after removal of the dam to prevent scour and subsequent foundation undermining. We understand that erosion of the rip-rap will be analyzed in future hydraulic studies.

5.3 EBSCO Facility Riverfront Foundation Wall Aesthetic Considerations

The estimated lowered water levels will not affect the structural stability of the foundation wall, but they could impact the visual appearance of the concrete surface.

During the investigation we observed erosion of the concrete foundation wall, likely due to freezing and thawing near the normal water levels (e.g., El. 8 ft to El. 10 ft). We observed less surface erosion and staining below the apparent current normal water levels, indicating that the concrete below the water line (and subjected to fewer freeze-thaw cycles) was somewhat better preserved than the exposed concrete subjected to more freeze-thaw cycles.

An investigation into the condition of the riverfront foundation wall concrete was not included in the scope of work for the current study. Based on the age of the concrete, we anticipate that it is not purposely air entrained, and due to the likely low air void ratio, we expect that previously submerged concrete that becomes exposed to weather due to lowered water levels under post-dam-removal conditions will be subject to some freeze-thaw deterioration, and will undergo erosion similar to the concrete that has already been exposed.

5.4 Settlement of Compressible Organic Soils

Available logs of borings performed by others at the south end of the building, within 50 ft from the Ipswich River, indicate the presence of a 5 ft to 10 ft thick clay and peat stratum. We did not encounter soft compressible soils in the three soil test borings we performed, located 50 ft or more from the river. The thickness of the clay and peat stratum appears to be largest near the river and very likely decreases with distance away from the river. Therefore, we anticipate that these problem soils are present in localized areas primarily adjacent to the river. At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility itself. Slabs supported on grade, shallow spread footings bearing above the soft compressible soils, and other shallow structures such as buried utilities overlying soft compressible soils could be at risk of some settlement due to consolidation of the soft soils initiated by lower groundwater levels resulting from the potential lower river level. We expect that where soft compressible soils are present along the river, average settlements in the order of 1 in. to 3.5 in. can be anticipated due to lowering of groundwater levels by 1 ft to 3 ft, respectively.

We observed no signs of settlement of the EBSCO Facility exterior wall along the river; therefore, we anticipate that this foundation wall is bearing on competent soils (likely glacial till) or rock. We also have not observed signs of significant settlement of the slab inside the EBSCO Facility, although some dishing of the slab has been noted, which suggests that some settlement of the slab has taken place in the past. To our knowledge, EBSCO has not reported any issues with settlement of the slab.

It is possible that settlement of soft compressible soils underlying the slab-on-grade, if they exist, could have occurred in the past and been repaired before EBSCO moved into the Facility. Based on the depth of the foundation walls near the south end of the building (TP-2), extending below El. -0.5 ft, it is also possible that a portion or all of the compressible soils that may have been present were removed as part of the excavation to construct the foundation walls. To the extent that soft compressible soils are present within the footprint of the EBSCO Facility, some settlement of the slab-on-grade may occur.

Mitigation measures for settlement of existing slabs-on-grade supported on soft compressible soils fall under two categories:

- Structural Remediation Approach – the goal is to replace the existing slab-on-grade either by 1) removing the soft compressible soils, replacing them with engineered compacted fill, and installing a new slab-on-grade, or 2) removing the existing slab-on-grade and replacing it with a structural slab supported on deep foundations (micropiles) bearing on suitable soils. Both options represent a significant disruption to a building facility's operations, have long construction schedules, and are expensive.

- Ground Improvement Approach – the goal is to reduce the compressibility of the underlying soft soils; usually maintaining the use of the existing slab-on-grade. Options for ground improvement of soft compressible soils for in-service facilities include compaction grouting or jet grouting. Both methods would involve drilling holes through the existing slab at regular intervals, for example on a 5 ft by 5 ft grid, to provide access for a drill rig to install grout columns below grade to provide supplemental support to the slab.

    - Compaction Grouting involves installing low-slump grout in lifts below grade to displace the soil and create columns of grout. More specifically, compaction grouting is performed by inserting 2 in. diameter grout injection pipes through the target weak soil stratum, then pumping low-slump grout under pressure, which forms a bulb of grout and pushes the surrounding soil, thus densifying the soil. After achieving a target pressure or volume of grout, the grout injection pipe is raised to a higher elevation, and another bulb of grout is injected. The process is repeated, extending the grout vertically through the entire weak soil stratum. The spacing of the compaction grout locations is designed by an engineer and typically depends on the subsurface conditions, the use of the building space (loads), and the capacity of the existing or new slab.

    - Jet Grouting involves installing grout (with water and/or air) at a high velocity to erode and mix with the soil to create columns of soil-crete. More specifically, jet grouting is performed by drilling a 6 in. diameter hole into the soil through the target weak soil stratum, then a specialty drill rod with a nozzle is lowered into the hole. The contractor pumps fluid (a mixture of grout, air and/or water) through the nozzle and spins the drill rod to erode a circular area. The drill rod is retracted up through the soil to create a column of soil-crete. Test borings are drilled after jet grouting to confirm strength of as-built soil-crete columns. The spacing of jet grout locations typically depends on the subsurface conditions and the ability for the soil to be eroded by the jet grout process.

Typically the soil-grout or grout columns created during compaction or jet grouting are spaced at about 5 ft on center for the typical 4 to 5 in. thick slab-on-grade, as this is typically the limit of the capacity of the slab-on-grade to resist concentrated loads acting on it. Grouting operations inside in-service facilities require significant coordination to control soil/grout spoils and maintain areas clean for use. Although grouting involves shorter construction schedules and is typically more cost-effective than the structural remediation approach, construction costs can still be high depending on the extent of the area to be treated.

All mitigation measures need to consider the presence of buried utilities under the existing slab-on-grade. Under the structural remediation approach, the buried utilities can be supported from the new structural slab. If soft compressible soils extend outside the building area to be remediated, flexible utility connections would need to be considered at the interface where existing utilities extend beyond the new structural slab or ground improvement area.

If mitigation of settling organic soils is required at the EBSCO Facility, we estimate the direct cost for repairs could be on the order of $750/sq ft, $350/sq ft, and $450/sq ft for the three repair options discussed above (structural slab with micropiles, compaction grouting, and jet grouting respectively). These order-of-magnitude costs may vary greatly depending upon project specifics, including, but not limited to, the existing structure and subsurface conditions, the depth of the micropiles or grout columns, the extent of the area to be repaired, access to repair areas, finishes, and any staging required to maintain the building occupancy during the repair work. We did not consider costs from other trades, such as plumbing for buried subsurface utilities or replacement of interior finishes. Also, the order-of-magnitude costs above consider the subcontracted cost, not the burdened cost to the project owner. Additional costs for general conditions, general contractor markup, owner project management, design fees, and contingencies are not included. The total burdened cost to the project owner could be on the order of $1,500/sq ft, $700/sq ft, and $900/sq ft for the three options above, respectively, assuming the following: 10% general conditions, 10% general contractor markup, 10% design fees, and 50% contingency.

Movement monitoring of the EBSCO Facility slab-on-grade can be performed to detect settlements and thus help identify if there is ongoing settlement of the slab-on-grade due to the presence of soft compressible soils. However, it is possible that some distress to the slab (e.g., cracks) may occur during the movement monitoring program.

6. CONCLUSIONS

We conclude the following regarding the potential impacts of the dam removal on the adjacent EBSCO Facility:

- The riverfront wall foundations of Building Nos. 9 and 10-A of the EBSCO Facility are bearing on rock and/or are bearing on soils or piled foundations at an elevation lower than the currently estimated low water level of the Ipswich River at the site after dam removal (El. 3 ft to El. 6 ft). We did not observe timber piles supporting the EBSCO Facility at these locations and, even if timber piles are present, it is anticipated that the tops of the timber piles are low enough to remain submerged in a post-dam removal scenario, and therefore, fungal deterioration of the tops of the timber piles would not occur.

- Soil test borings performed by others in 2009 indicate the presence of localized soft compressible soils, including organics, along the riverfront. Where organics are present, lowered groundwater levels could result in settlement of pavement, slabs-on-grade, and structures on spread footings or buried utilities supported above the soft compressible soils. We estimate a potential settlement of the soft compressible soils of approximately 1 in., 2.5 in., and 3.5 in. due to a water level drawdown of 1 ft, 2 ft, and 3 ft, respectively (i.e., groundwater level at El. 5 ft, El. 4 ft, and El. 3 ft, respectively). At this time it is uncertain to what extent, if any, compressible soils may or may not underlie the EBSCO Facility. We estimated the settlement assuming average soil properties from a range of published values for organic silt and clay.

- Based on the results of the current investigation, we identify the following three options for the project team to determine next steps in the feasibility study for the Ipswich Dam removal:

    - Option 1 – Maintain Current Groundwater Level During Post-Dam Removal. This option presents the least amount of risk for settlement due to timber pile deterioration or consolidation of compressible soils, if present, at the EBSCO Facility. Groundwater levels measured during our investigation were approximately El. 6 ft, therefore maintaining this groundwater elevation would likely not result in adverse impacts to the EBSCO Facility. Maintenance of current groundwater levels at approximately El. 6 ft would require evaluating appropriate approaches to dam removal or other engineered solutions such as groundwater recharge. Additional subsurface investigation would be required to evaluate the feasibility of applicable engineered solutions. This option also requires continuous monitoring of groundwater levels and structure movement to verify performance after the dam is removed, for the life of the structure.

      If the project team anticipates that the post-dam removal groundwater levels cannot be maintained at or above El. 6 ft, then one of the following two options may be implemented to determine risks to the EBSCO Facility and develop mitigation options if needed.

    - Option 2 – Pre–Dam-Removal Supplemental Subsurface Investigation. This involves completing a supplemental foundation investigation in the building areas that were not accessible during the current investigation. Performing this investigation prior to completing the feasibility study for the dam removal would provide actionable information to perform a better assessment of the likelihood of the need for mitigation options, as it would allow the project team to identify whether timber piles are present in the remaining areas of the EBSCO Facility where test pits have not been performed, and would also allow us to determine if soft compressible soils are present within the footprint of the EBSCO Facility. We consider that this option lowers the risk of adverse impacts from dam removal as it allows for timely planning and budgeting for mitigation, if needed, during the initial design phases of the project. The extent of post–dam-removal movement monitoring required to confirm adequate performance of the building would be determined based on the results of the supplemental subsurface investigation.

An outline of the recommended supplemental investigation is included in Appendix A. We estimate that the order-of-magnitude cost for the supplemental investigation as outlined would be approximately $200,000, assuming adequate access for the investigation, minor dewatering required for test pits, and replacement of the concrete slab and asphalt pavement cut penetrations.

- If EBSCO does not provide access to the inside of its facility and access for test pit investigations on the exterior of the facility, then a limited soil test boring investigation could be performed on the building exterior. The limited investigation would include five to ten soil test borings drilled in the EBSCO Facility parking lot and other exterior areas near the building, such as the grassed area at the south end of the building. The soil test borings would provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to the potential presence of compressible soils and timber piles, if any are deemed to be present. We estimate that the order-of-magnitude cost for the limited supplemental investigation would be approximately $50,000.

- Option 3 – Perform Pre– and Post–Dam-Removal Precision Movement Monitoring, No Supplemental Subsurface Investigation. We understand a staged drawdown test in combination with precision movement monitoring could be performed for an extended period of time prior to dam removal. The pre–dam-removal precision movement monitoring would help establish a baseline against which to compare post–dam-removal performance.

  Under this option, planning for the dam removal project would proceed without further information about the foundations in building areas outside the current study, and also without further information regarding the presence of soft compressible soils within the EBSCO Facility. The project team would rely solely on pre– and post–dam-removal precision movement monitoring to assess the building performance and determine if mitigation measures are required. Precision movement monitoring helps identify problem areas; however, limits to accuracy, access, and duration of monitoring make this a more reactive approach compared to the other options. We consider that this option results in a higher risk of potential unmitigated settlement of the building because some distress to the building utilities, adjacent structure, and/or slab-on-grade may occur before the post–dam-removal precision monitoring program detects measurable movement. In addition, there is a higher risk of significantly underestimating or overestimating the costs of mitigation. We note that if post–dam-removal mitigation measures are required, the costs are more likely to be higher than had mitigation been performed pre–dam-removal, as the costs of repairs of any building distress (cracks, unlevelness, etc.) would need to be included. This also requires access to the interior of the EBSCO Facility buildings to install monitoring points and during each round of survey of the monitoring points over an extended period of time.

Limitations of Current Investigation

The information presented herein is based on the geotechnical information collected to date. The boring logs and geotechnical investigation records depict subsurface conditions only at the specific soil sampling locations. Subsurface conditions at other locations may differ from conditions observed at specific sample depths and exploration locations. There is no warranty or guarantee, either expressed or implied, that the conditions indicated by such investigations or records thereof are representative of those existing throughout such areas, or any part thereof, or that unexpected developments may not occur, or that materials other than, or in proportions different from, those indicated may not be encountered.

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Photo 1

View of EBSCO building on 22 August prior to the temporary impoundment drawdown.

Photo 2

View of EBSCO building on 24 August after the temporary impoundment drawdown.

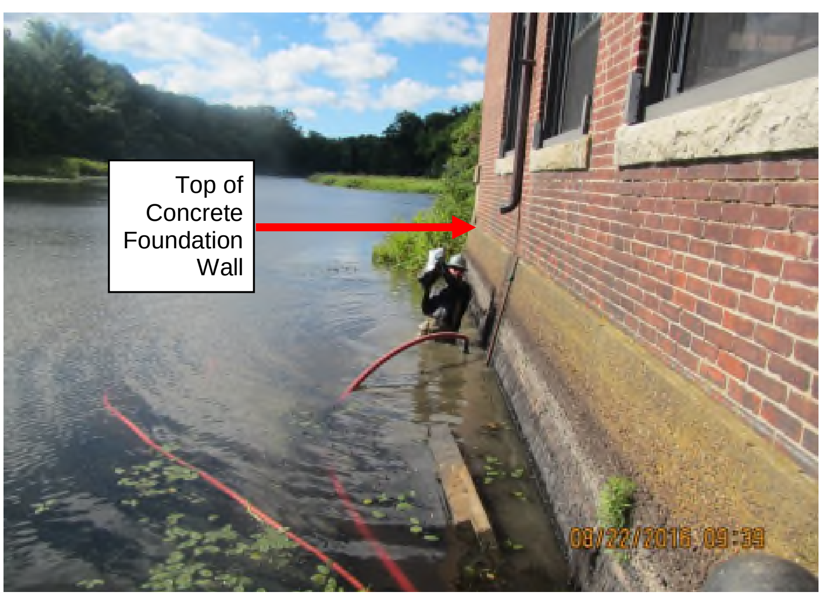


Photo 3

Diving contractor removing rip-rap by hand at TP-1, adjacent to Building No. 9.

Photo 4

Typical rip rap and debris removed from TP-1.

Photo 5

View of riverfront foundation wall at TP-1 (Building No. 9) on 24 August, looking south. Staining and concrete erosion is apparent below-normal water level.

Photo 6

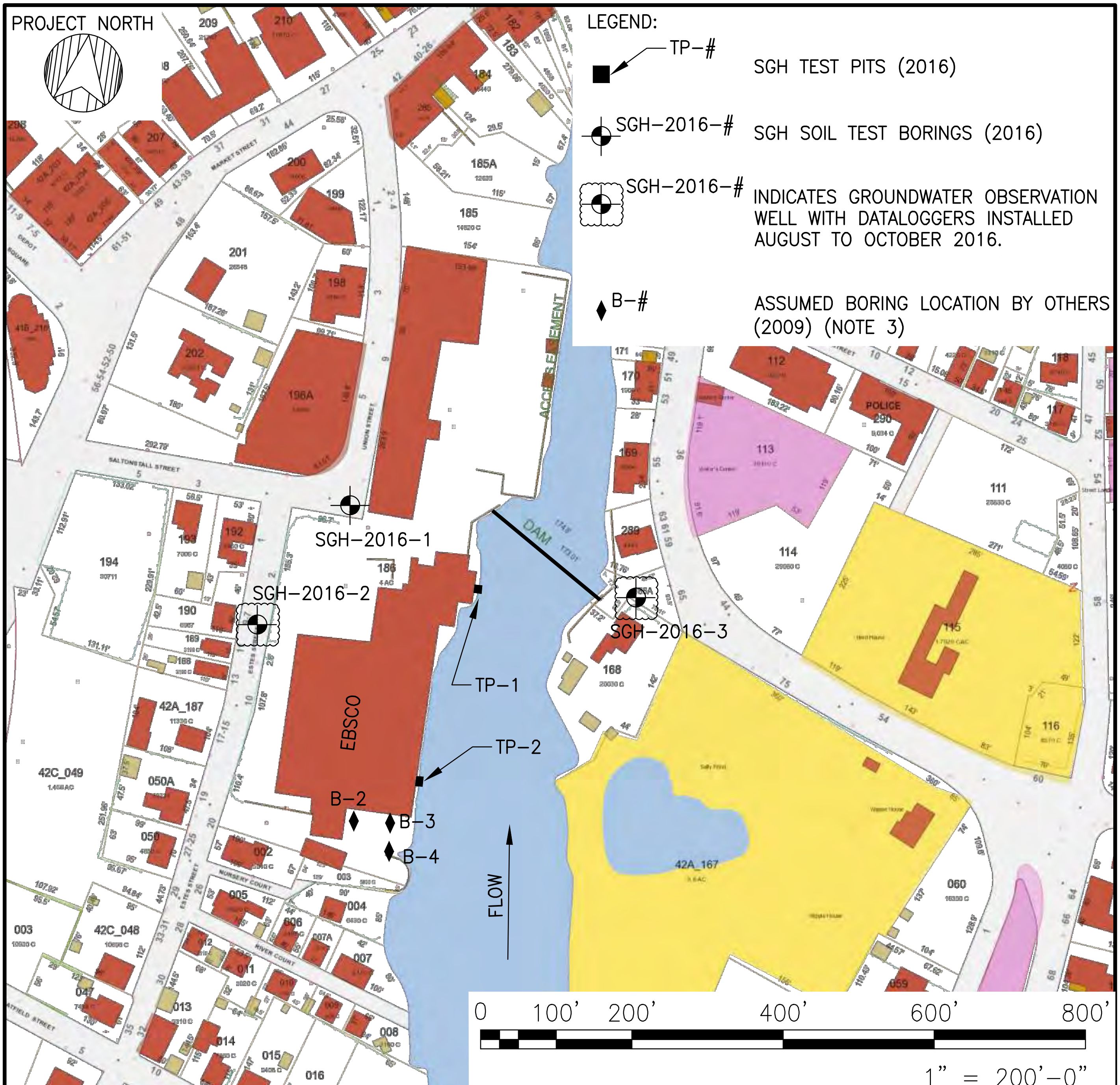
View of riverfront foundation wall at TP-2 (Building No. 10-A) on 23 August, looking north. Staining and concrete erosion is apparent below-normal water level.

Photo 7

Abandoned timber formwork removed from TP-1.

Photo 8

Drill rig set up at Soil Test Boring SGH-2016-1, located approximately 100 ft from EBSCO Building No. 9. EBSCO Building No. 3 on the right hand side.



NOTES:

1. BASE PLAN OBTAINED FROM MASS GIS ONLINE VIEWER.
2. SGH TEST PIT AND BORING LOCATIONS ARE APPROXIMATE.
3. LOCATIONS OF THE BORINGS PERFORMED BY OTHERS IN 2009 ARE ESTIMATED FROM THE MEMORANDUM FROM GEI TO THE TOWN OF IPSWICH RE: EVALUATION OF POTENTIAL IMPACTS ON EBSCO BUILDINGS FROM PROPOSED REMOVAL OF IPSWICH MILLS DAM DATED 13 DECEMBER 2013 REVISED 14 FEBRUARY 2014.

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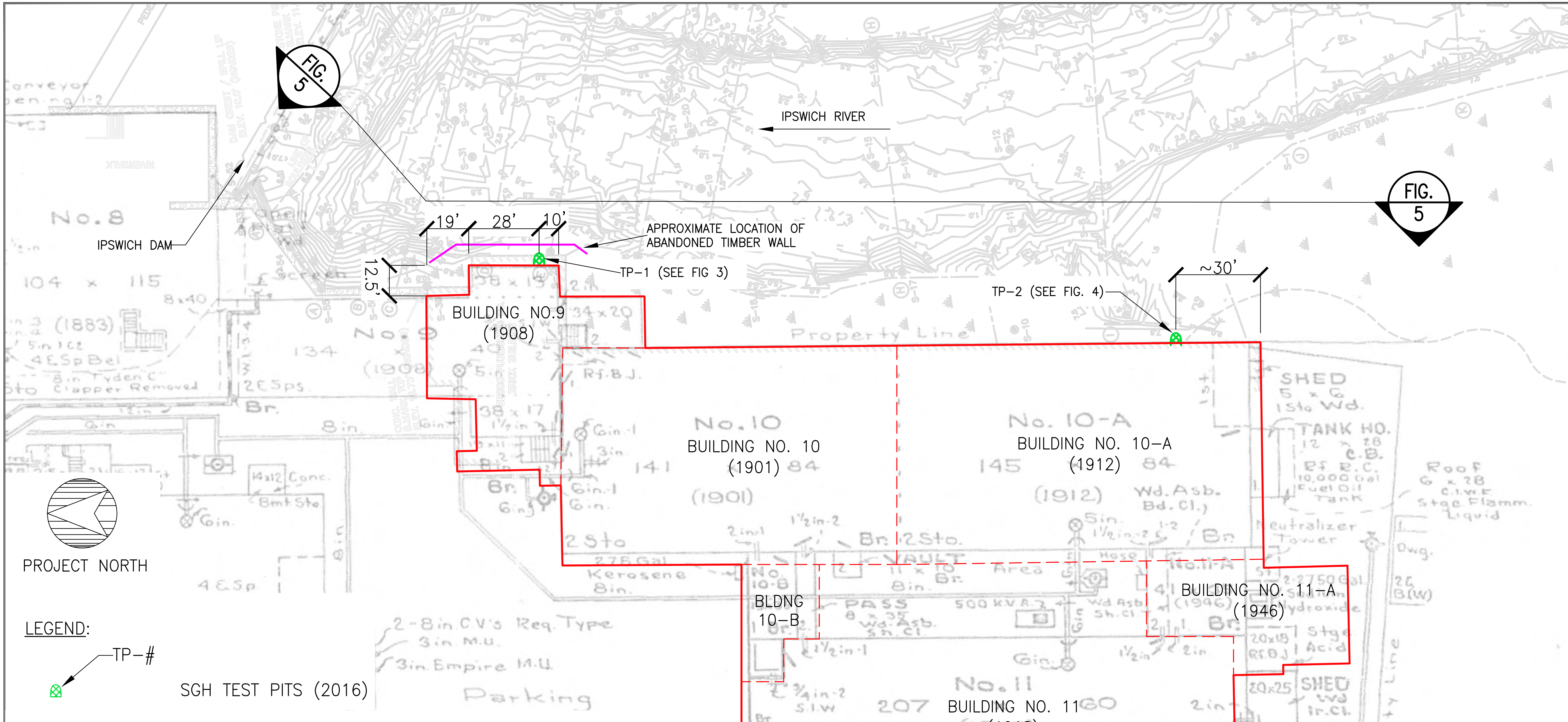
Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA				Drawing No.: FIG. 1
Title: SUBSURFACE INVESTIGATION LOCATION PLAN				Scale: 1" = 200'
Drawn: NDL	Checked: SFK	Approved:	Project No.: 160630.00	Date: 10/14/16



LEGEND:

TP-# SGH TEST PITS (2016)

NOTES:

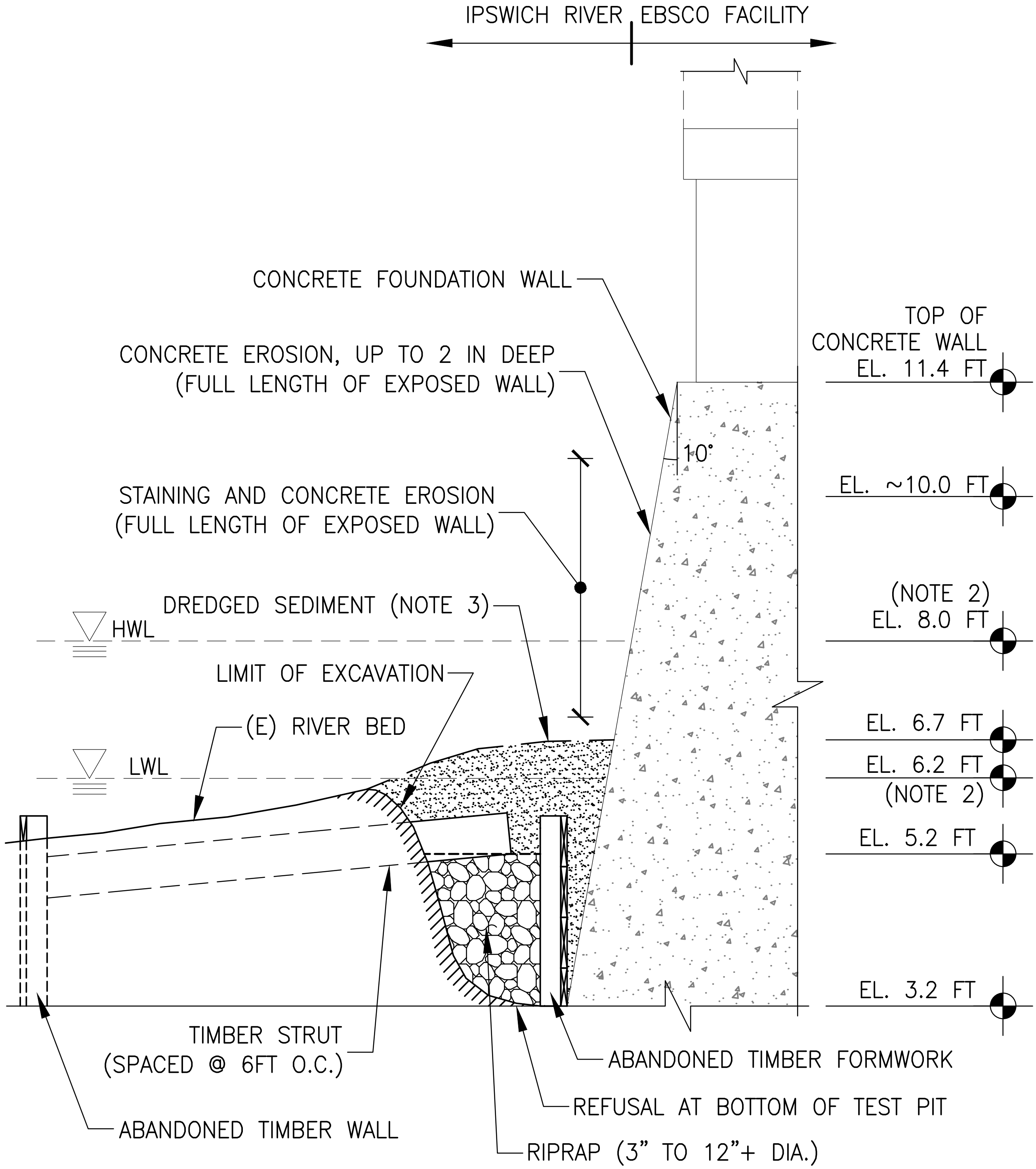
1. HISTORIC EBSCO BUILDING BASE PLAN OBTAINED FROM THE REPORT TITLED "IPSWICH MILLS DAM PARTIAL FEASIBILITY STUDY" DATED 23 APRIL 2014. BASE PLAN SHOWS YEAR BUILT OF HISTORIC EBSCO BUILDING SECTIONS.
2. BASE PLAN SHOWING RIVER BATHYMETRY OBTAINED FROM REPORT TITLED "IPSWICH RIVER MILLS DAM SURVEY" PREPARED BY NORDE-EAST SURVEY DATED 26 AUGUST 2014.
3. OUTLINE OF CURRENT EBSCO BUILDING (IN RED) IS APPROXIMATED FROM MASS GIS ONLINE VIEWER AND LIMITED FIELD MEASUREMENTS NEAR TEST PITS.
4. DASHED LINES INDICATING EXTENTS OF HISTORIC EBSCO BUILDING SECTIONS ARE APPROXIMATE.

SIMPSON GUMPERTZ & HEGER
Engineering of Structures and Building Enclosures
Simpson Gumpertz & Heger Inc.
41 Seyon Street, Building 1, Suite 500
Waltham, Massachusetts 02453
781.907.9000
fax: 781.907.9009
www.sgh.com

Project:	FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA
Title:	TEST PIT LOCATION PLAN
Drawn:	NDL
Checked:	SFK
Approved:	
Project No.:	160630.00
Drawing No.:	FIG. 2
Scale:	1"=40'-0"
Date:	02/10/17

NOTES:

1. ALL ELEVATIONS ARE APPROXIMATE AND REPORTED IN FEET REFERENCED TO NAVD88.
2. IMPOUNDMENT ELEVATIONS ESTIMATED FROM STAFF GAUGE NEAR DAM. HIGH WATER LEVEL (HWL) MEASURED ON 22 AUGUST 2016 PRIOR TO BEGINNING TEST DRAWDOWN. LOW WATER LEVEL (LWL) MEASURED ON 24 AUGUST 2016 AT 3PM.
3. SEDIMENT CONSISTS OF SILT; DARK BROWN; VERY SOFT; TRACE DEBRIS.



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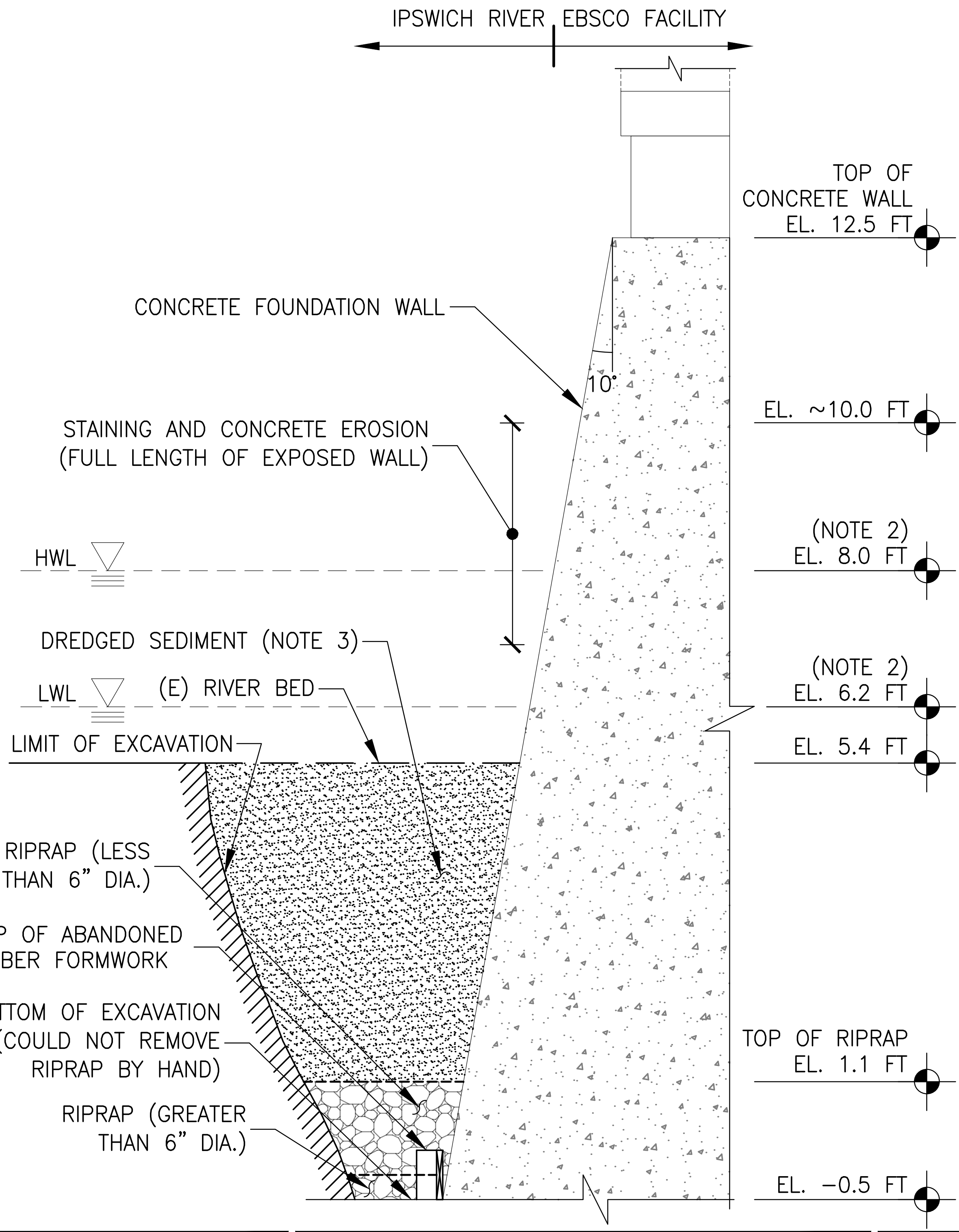
fax: 781.907.9009
www.sgh.com

Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA				Drawing No.: FIG. 3
Title: SECTION VIEW, TP-1				Scale: 1/2"=1'-0"
Drawn: NDL	Checked: SFK	Approved:	Project No.: 160630.00	Date: 02/10/17

NOTES:

1. ALL ELEVATIONS ARE APPROXIMATE AND REPORTED IN FEET REFERENCED TO NAVD88.
2. IMPOUNDMENT ELEVATIONS ESTIMATED FROM STAFF GAUGE NEAR DAM. HIGH WATER LEVEL (HWL) MEASURED ON 22 AUGUST 2016 PRIOR TO BEGINNING TEST DRAWDOWN. LOW WATER LEVEL (LWL) MEASURED ON 24 AUGUST 2016 AT 3PM.
3. SEDIMENT CONSISTS OF SILT; DARK BROWN; VERY SOFT; SOME ORGANICS; SOME CLAY; TRACE DEBRIS.



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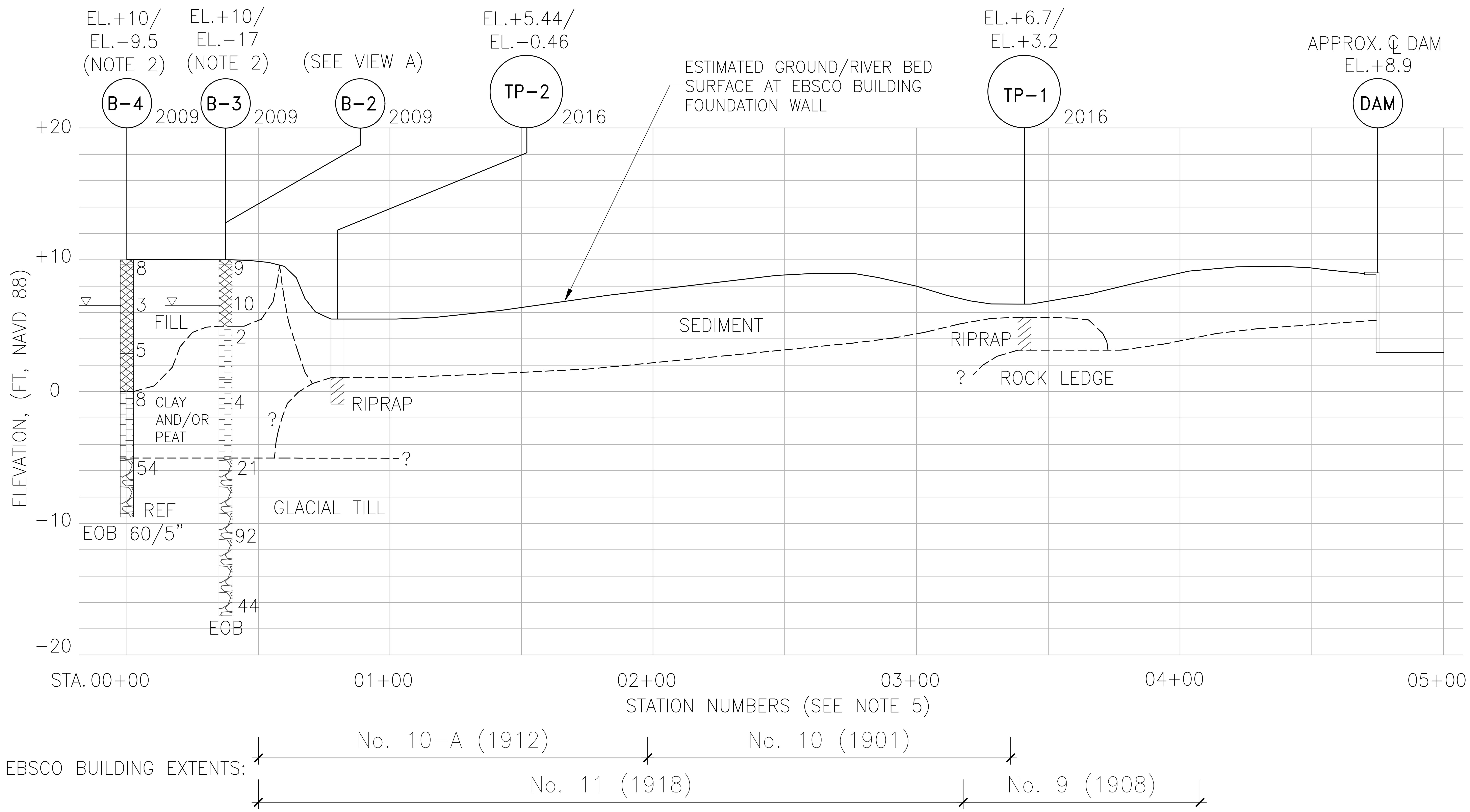
Project: FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL IPSWICH, MA

Title: SECTION VIEW, TP-2

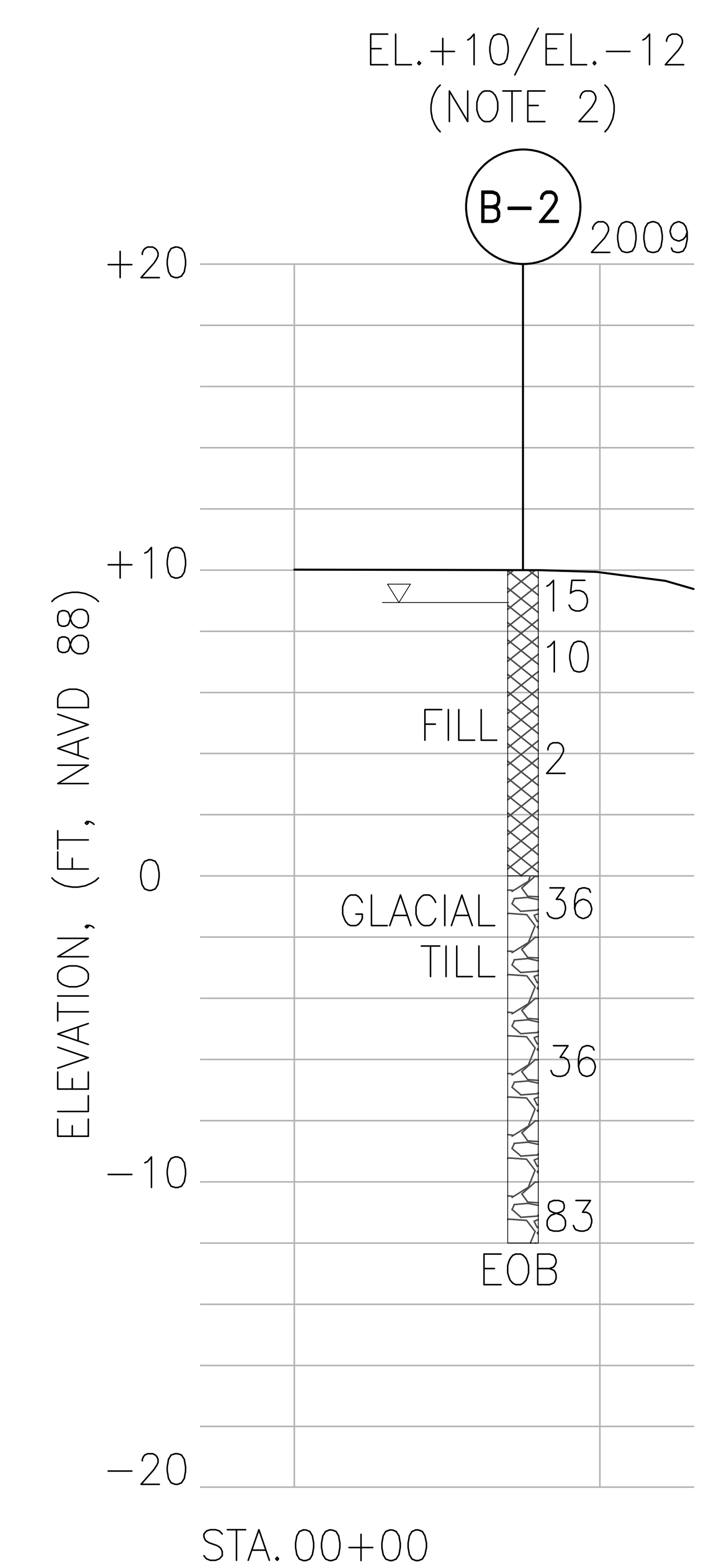
Drawing No.: FIG. 4

Scale: 1/2"=1'-0"

Drawn:	Checked:	Approved:	Project No.:	Date:
NDL	SFK		160630.00	11/22/16



SECTION AT EBSCO BUILDING RIVERFRONT FOUNDATION WALL



VIEW A

NOTES:

1. ALL ELEVATIONS ARE REFERENCED TO THE NAVD88 DATUM U.O.N.

2. THE GROUND SURFACE ELEVATIONS FOR 2009 BORINGS (B2, B3, B4) ARE NOT REPORTED. WE ASSUMED THE GROUND SURFACE IS EL.+10 FT NAVD88, WHICH WE ESTIMATED BASED ON THE BATHYMETRIC SURVEY PERFORMED BY NORDE-EAST ON 26 AUGUST 2014 AND VISUAL OBSERVATIONS NEAR TP-2 IN AUGUST 2016.

3. SOIL STRATA SHOWN ARE GENERALIZED PROFILES INTERPRETED FROM BORING LOGS PREPARED BY SGH AND OTHERS. REFER TO SOURCE BORING LOGS FOR MORE DETAILED SOIL DESCRIPTIONS.

4. STRATIFICATION LINES REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES; TRANSITIONS MAY BE GRADUAL.

5. STATION NUMBERS ALONG THE RIVERFRONT WALL OF THE EBSCO BUILDING ASSUME STATION 00+00 IS LOCATED AT BORING B-4. SEE FIGURE 2 FOR SECTION ALIGNMENT.

LEGEND

#
-OFFSET (FT)
-TOP OF BORING OR TEST PIT ELEVATION (FT)/
-BOTTOM OF BORING OR TEST PIT ELEVATION (FT)

WATER LEVEL DURING BORING ▽

#'=ELEVATION OF STRATUM CHANGE
##=N VALUE, BLOWS PER FT (BPF)
REF 100/2" = REFUSAL, BLOWS/PEN.
EOB=END OF BORING

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www.sgh.com
Boston
Chicago
Los Angeles
New York
San Francisco
Washington, DC

Consultant

No.	Date	Description	By
1	6/26/17	REPORT REVISION	SFK

FEASIBILITY STUDY FOR IPSWICH MILLS DAM REMOVAL
IPSWICH, MA

Project

SUBSURFACE PROFILE

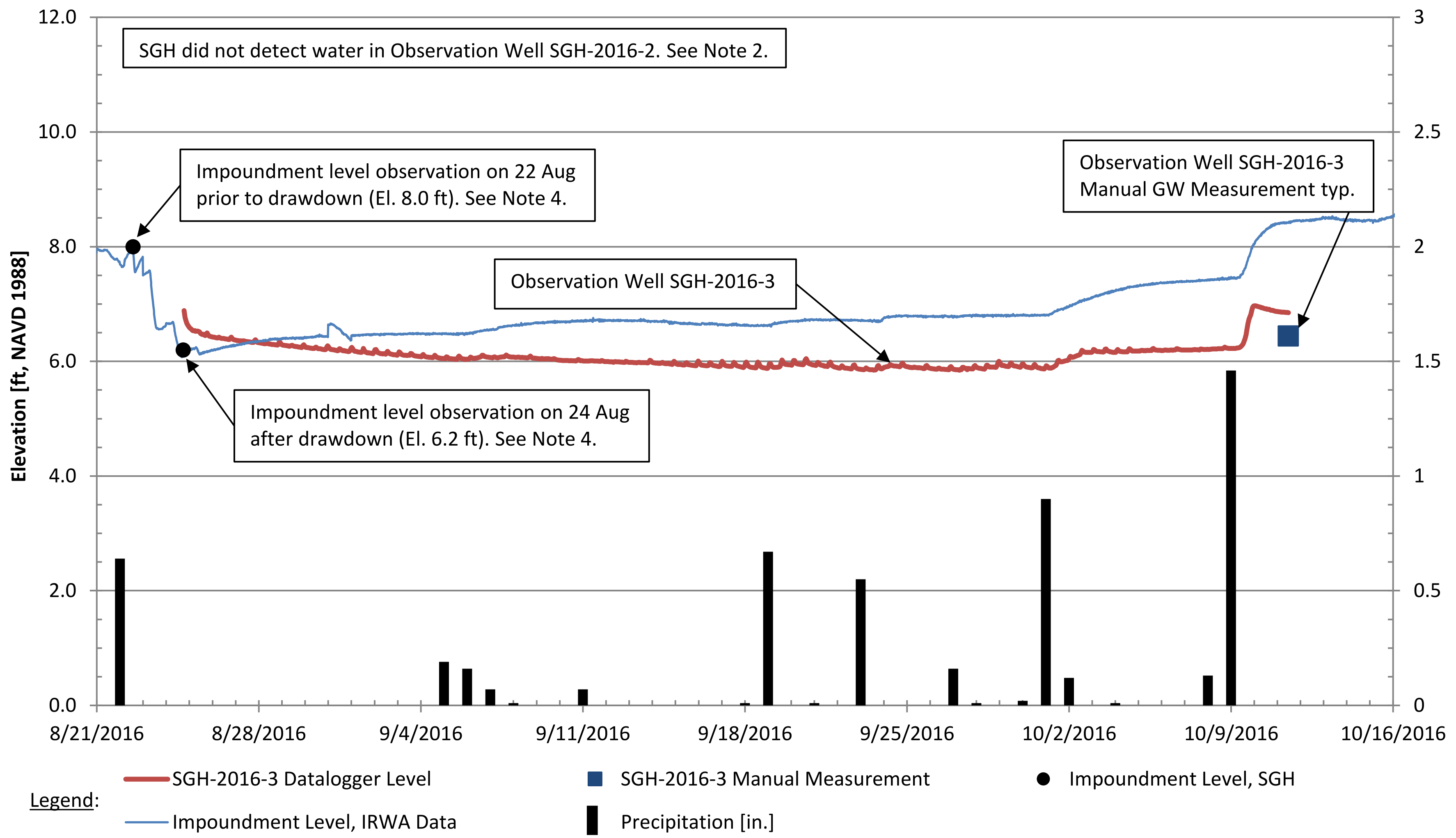
Drawing Title

Project No.	Checked	Date
160630.00	SFK	10/28/2016
Drawn	**Approved**	**Scale**
NDL	–	1V:5H

Drawing No.

FIG. 5

Seal



Notes:
1. Groundwater levels are data recorded every 1 hr at the site with data loggers (model Levelogger Gold by Solinst) installed in groundwater observation wells.
2. We observed that Observation Well SGH-2016-2 was dry on 24 August 2016 during installation and on 11 October 2016 when we removed the data logger. The logger did not detect water during the period of record. We installed the datalogger in SGH-2016-2 at El. 8.5 ft, near the bottom of the observation well (located at El. 7.5 ft).
3. Daily precipitation values are from 15 minute unverified data obtained from the Plum Island Ecosystems LTER Field Station in Rowley, MA (http://www.pielter.org/)
4. Impoundment Level, SGH data are based on SGH field observations of the IRWA staff gauge located on the upstream side of the Ipswich Mills Dam. Continuous impoundment level data provided by the IRWA and HW.
5. Groundwater observation well locations are shown on Fig. 1.

APPENDIX A
Recommended Supplemental Foundation Investigation

Based on the results of the current investigation, we identified options for the project team to determine next steps in the feasibility study for the Ipswich Dam removal. Option 2 is to perform a supplemental foundation investigation in the building areas that were not accessible during the current investigation. Performing this investigation prior to completing the feasibility study for the dam removal would provide actionable information to perform a better assessment of the likelihood of the need for mitigation options, as it would allow the project team to identify whether timber piles are present in the remaining areas of the EBSCO Facility where test pits have not been performed and would also allow identification of the presence of soft compressible soils within the footprint of the EBSCO Facility.

An outline of the recommended supplemental investigation is included below. We estimate that the order-of-magnitude cost for the supplemental investigation as outlined would be approximately $200,000, assuming adequate access for the investigation, minor dewatering required for test pits, and replacement of basic finishes only (concrete slab and asphalt pavement). The recommended supplemental foundation investigation includes the following tasks:

- Perform additional subsurface investigation to aid in determining the presence and extent of soft compressible soils within and around the EBSCO Facility. We recommend that at least five soil test borings be located around the exterior of the EBSCO Facility and that at least two soil test borings be located inside the EBSCO Facility. We recommend performing the soil test borings prior to the test pit investigation, as the results of the soil test borings can assist in selecting test pit investigation locations.
    - If soft compressible soils are present, obtain undisturbed soil samples for consolidation tests in a soil testing laboratory to determine soil compressibility properties.
- Perform additional test pit investigations at portions of the buildings not included in the current study, (e.g., Building Nos. 10, 11, and 11-A, which were constructed in 1901, 1918, and 1946 respectively). Additional test pits should include the following:
    - At least one test pit located inside the EBSCO Facility near a column and/or interior bearing wall. Three test pits located outside the EBSCO Facility at each of Building Nos. 10, 11, and 11-A.
    - The test pits could also be used to observe subsurface soil conditions below the first-floor slab. Dewatering is likely to be needed to confirm the depth to the bottom of the concrete foundations and determine if timber piles are present.
- Obtain three concrete core samples of the riverfront foundation wall to confirm the thickness and to obtain samples for laboratory analysis. Perform a petrographic analysis and testing to estimate long-term durability impacts due to a lowered impoundment level.

Alternatively, if EBSCO does not provide access to the inside of its facility or access for test pit investigations on the exterior of the facility, then a limited soil test boring investigation could be performed on the building exterior. The limited investigation would include five to ten soil test borings drilled in the EBSCO Facility parking lot and other exterior areas near the building, such as the grassed area at the south end of the building. The soil test borings would provide some subsurface information for the EBSCO Facility site and allow the project team to further evaluate the potential risks due to compressible soils and timber piles, if any are deemed to present. We estimate that the order-of-magnitude cost for the limited supplemental investigation would be approximately $50,000.

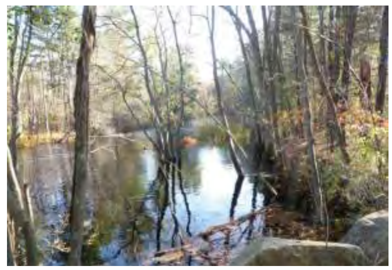
APPENDIX B
Excerpts from Report titled "Ipswich Mills Dam Partial Feasibility Study" prepared by Horsley Witten Group dated 23 April 2014

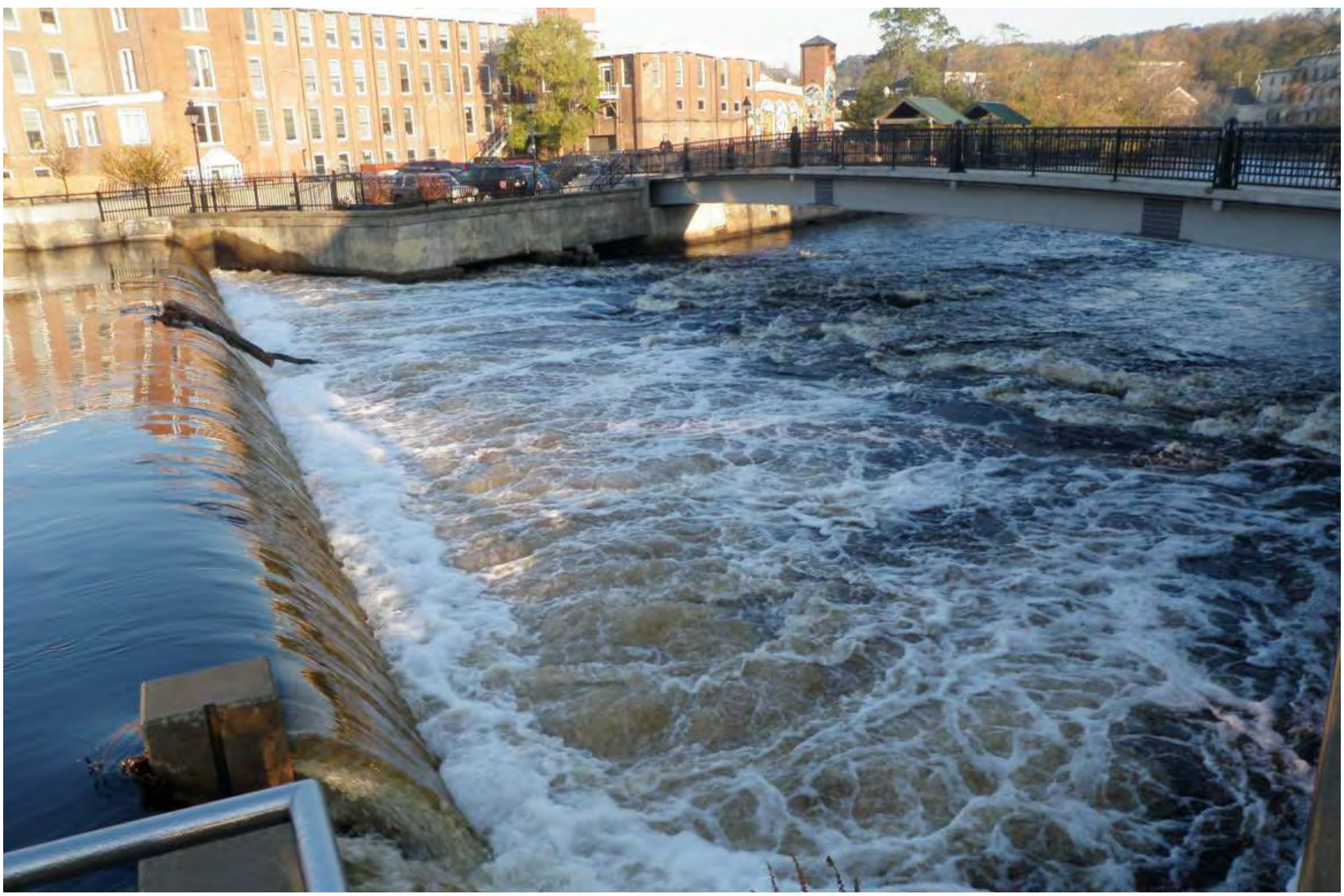
Ipswich Mills Dam Partial Feasibility Study

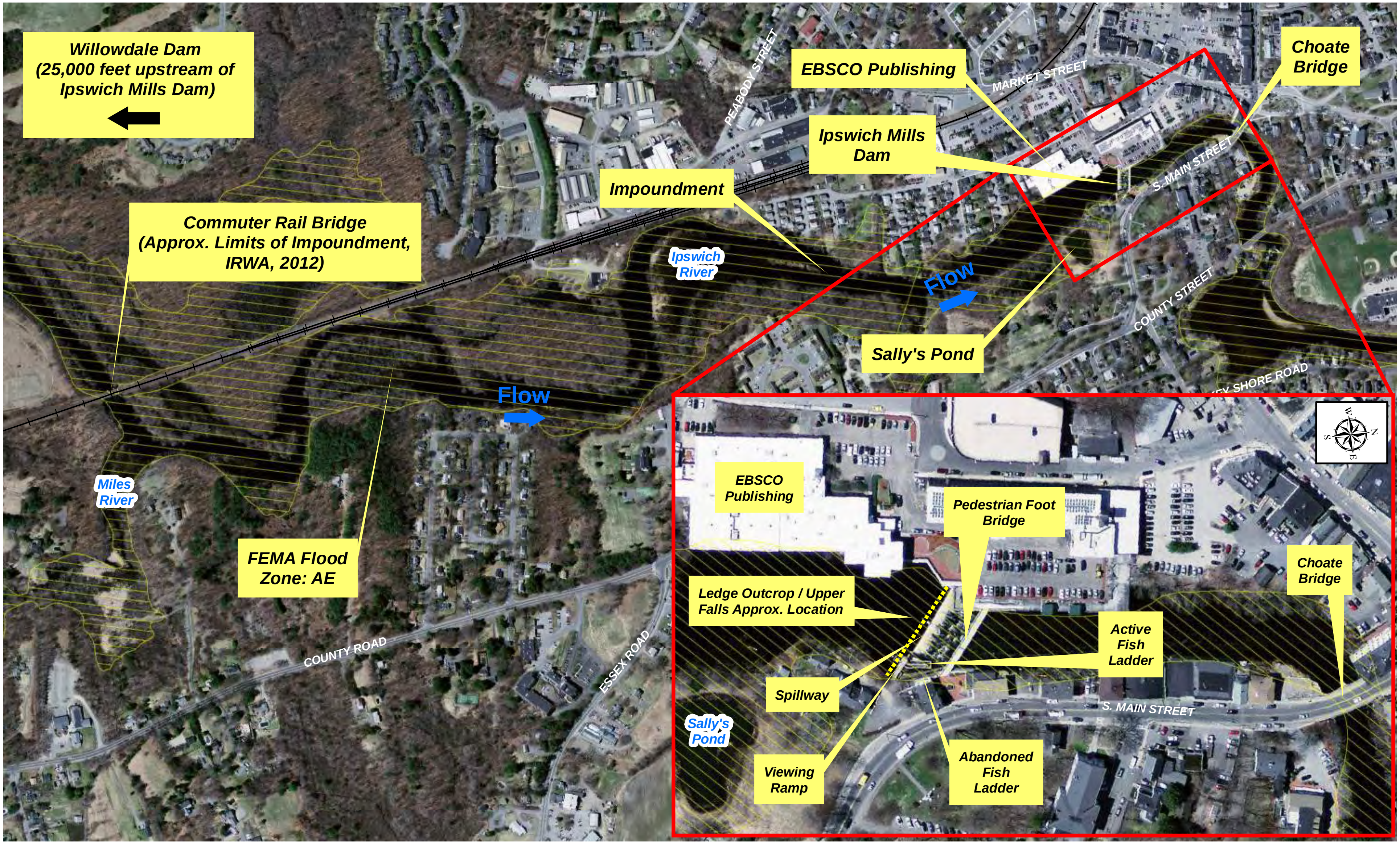
Preliminary analysis of three primary factors that may influence the cost and feasibility of the removal of the Ipswich Mills Dam, Ipswich, MA

April 23, 2014









Path: H:\Projects\2011\11101 Ipswich Mills Dam Feas.Study\GIS\Maps\Figure_1.mxd

Legend

Commuter Rail Service

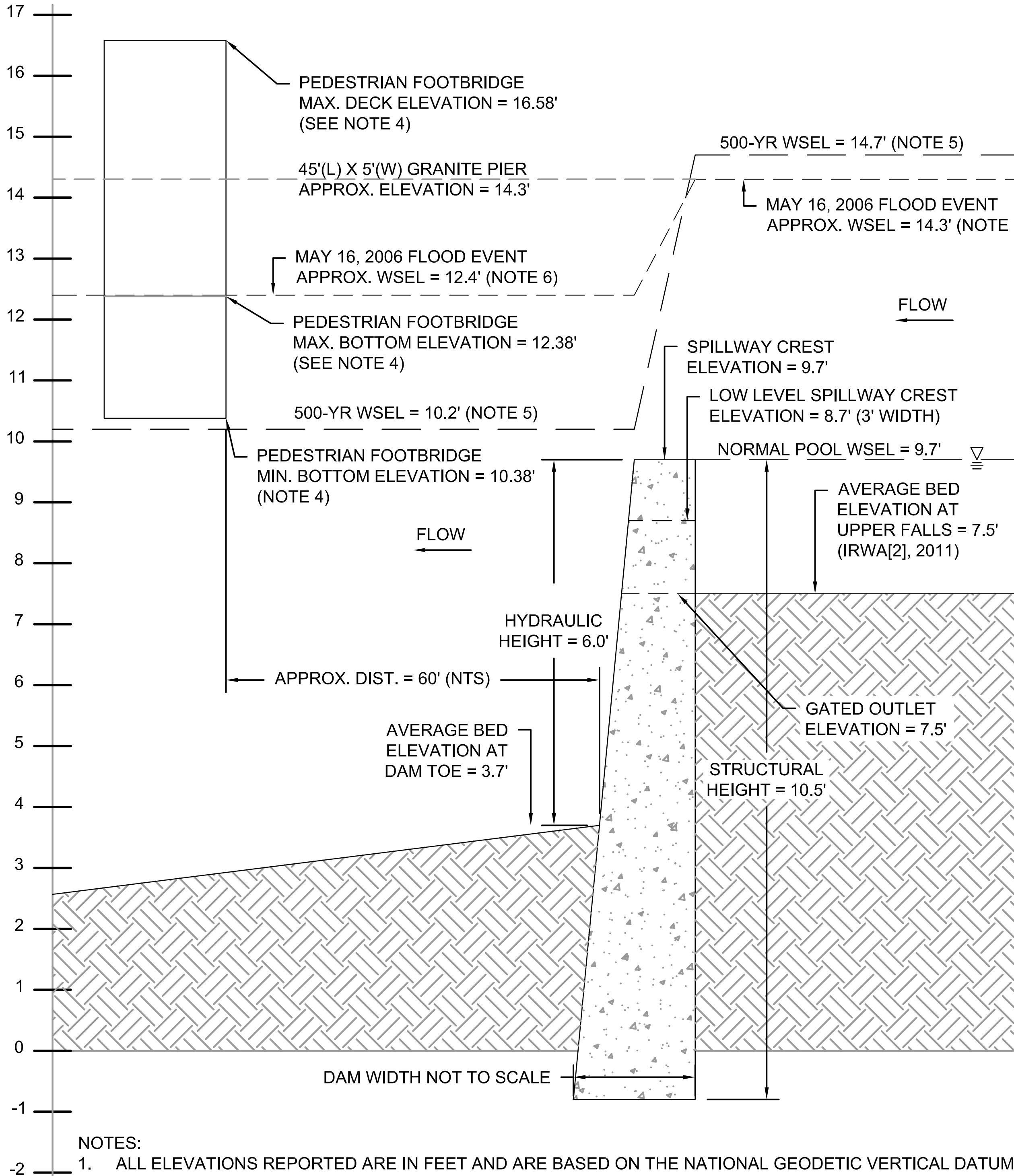
FEMA AE Zone: The base floodplain (100-yr flood zone), or areas with a 1% annual chance of flooding and a 26% chance of flooding over the life of 30-year mortgage. Base flood elevations have been provided.

Feet
750

Horsley Witten Group
Sustainable Environmental Solutions

Ipswich Mills Dam and
Surrounding Area
Ipswich, MA

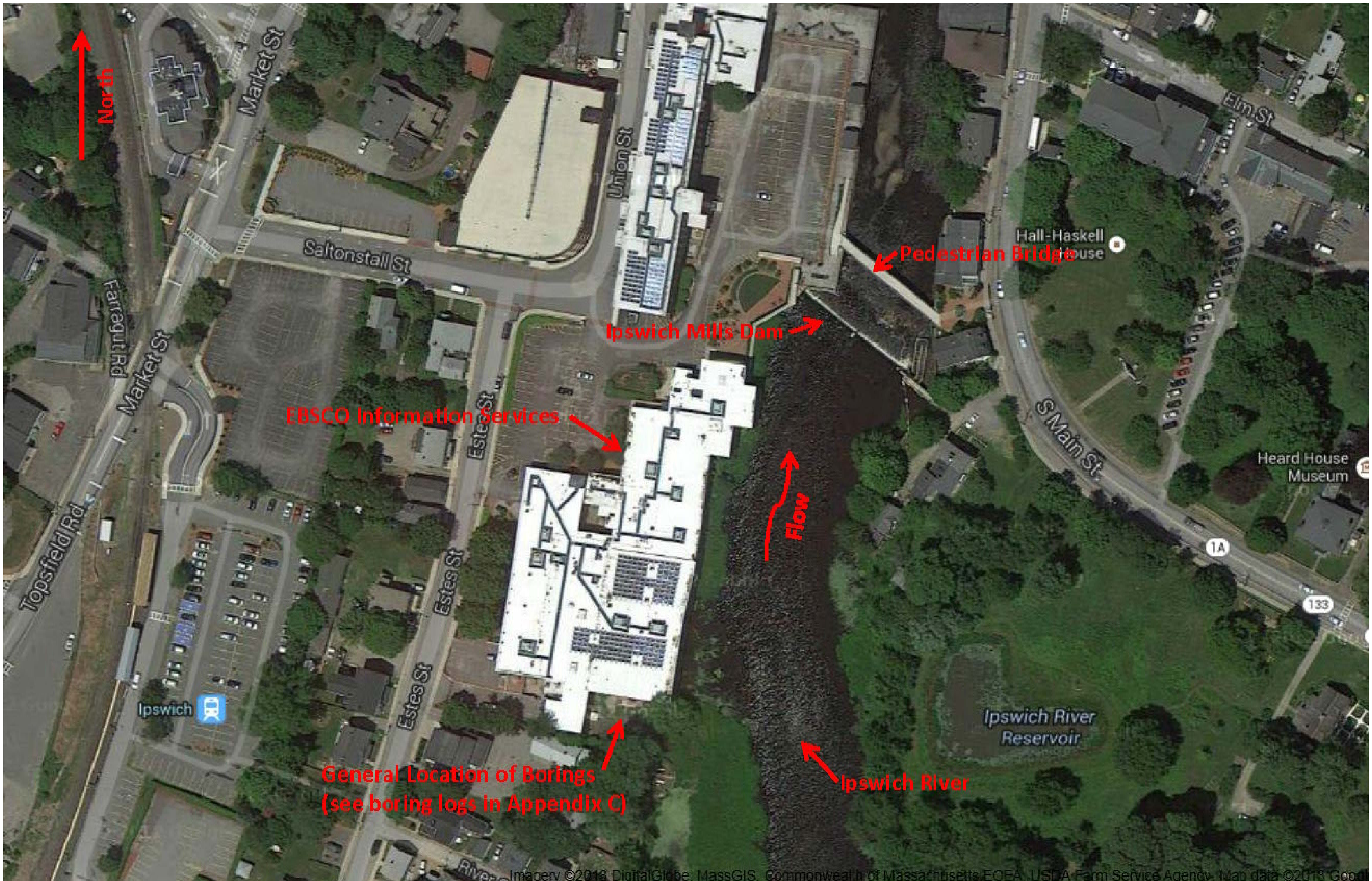
Date: 3/15/2012 Figure 1



NOTES:

1. ALL ELEVATIONS REPORTED ARE IN FEET AND ARE BASED ON THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD).
2. WSEL = WATER SURFACE ELEVATION
3. UNLESS OTHERWISE NOTED, REPORTED ELEVATIONS AND DIMENSIONS IN THIS FIGURE WERE OBTAINED FROM THE IPSWICH MILLS DAM PHASE INSPECTION / EVALUATION REPORT (HALEY & ALDRICH, 2009).
4. ELEVATIONS WERE OBTAINED FROM THE *PEDESTRIAN BRIDGE OVER THE IPSWICH RIVER CONSTRUCTION PLANS* (BETA GROUP, INC. & MASSHIGHWAY, 2001).
5. ELEVATIONS WERE OBTAINED FROM THE IPSWICH, MA FLOOD INSURANCE STUDY (FEMA, 1985).
6. ELEVATIONS WERE APPROXIMATED BY IRWA FROM PHOTO DOCUMENTATION ON MAY 16, 2006 (SEE FIGURES 2A & 2B) IN CONJUNCTION WITH THE REFERENCES LISTED IN NOTES 3 AND 4.

Project Number:	Registration:	Prepared For:	Plan Set:	
11101		*Town of Ipswich* *25 Green Street* *Ipswich, MA 01938*	*IPSWICH MILLS DAM* *CROSS-SECTION* *IPSWICH, MASSACHUSETTS*	**Horsley Witten Group, Inc.** **Sustainable Environmental Solutions** **90 Route 6A** **Sandwich, MA 02563** **508-833-6600 voice** **508-833-3150 fax**
Sheet Number: *1 of 1*			Plan Title: *FIGURE 3*	Date: 06/04/2012; Designed By: --; Drawn By: KH; Checked By: EB



Source: **Google Maps**

0 150 300

APPROXIMATE SCALE, FEET

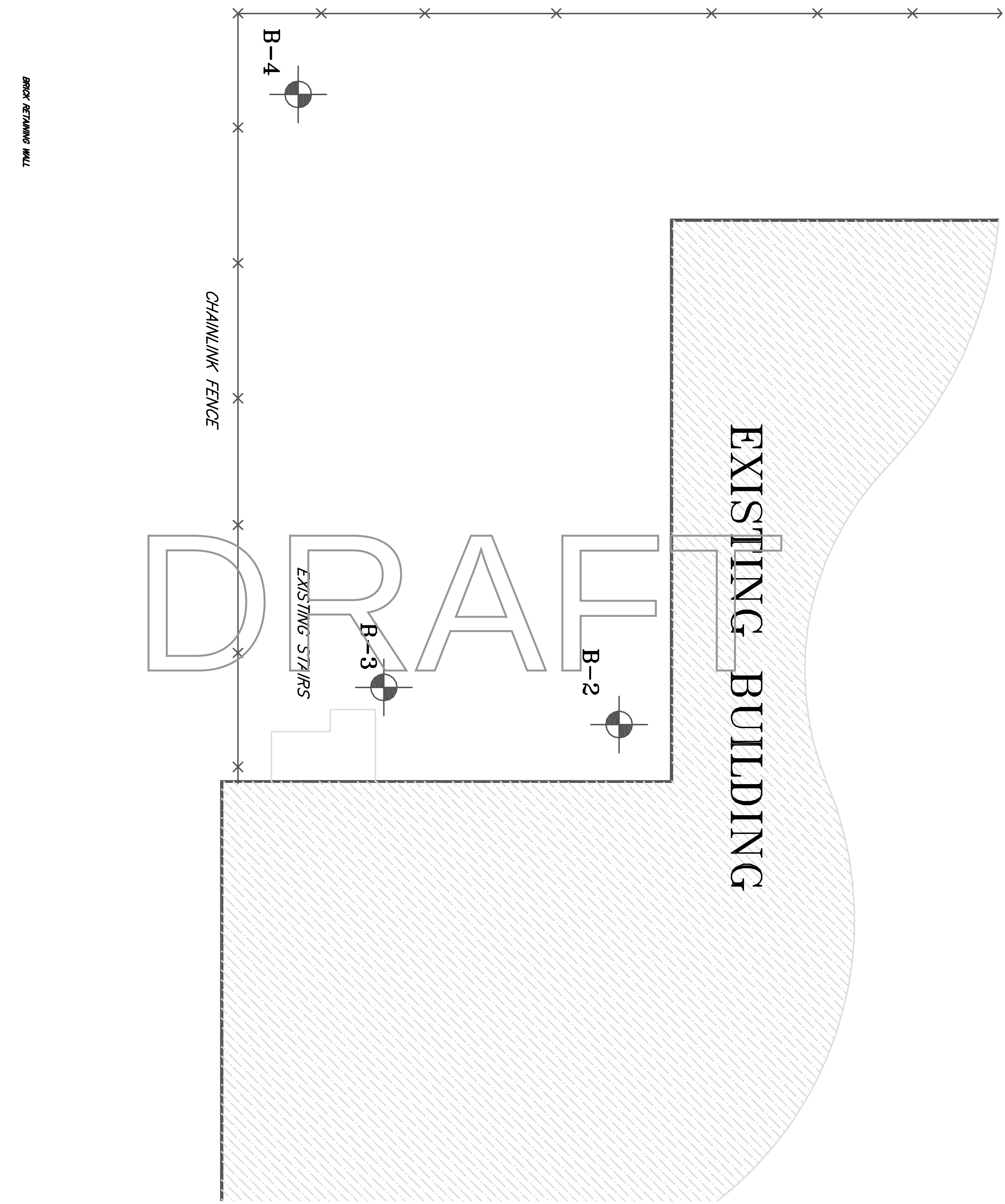
Evaluation of Ipswich Mills Dam Removal on EBSCO Buildings Ipswich, Massachusetts	GEI Consultants	AERIAL VIEW OF PROJECT AREA
Town of Ipswich Ipswich, Massachusetts	Project 1325760-0	December 2013 Fig. 1

M:\DRAFTING\2013\1325760 Ipswich Mills Dam\1325760-01

Source: **Bing Maps**

Evaluation of Ipswich Mills Dam Removal on EBSCO Buildings Ipswich, Massachusetts	GEI Consultants	EBSCO BUILDING Nos. 9, 10, AND 10-A
Town of Ipswich Ipswich, Massachusetts	Project 1325760-0	December 2013 Fig. 2

M:\DRAFTING\2013\1325760 Ipswich Mills Dam\1325760-01



TITLE: BORING LOCATION PLAN	GEOTECHNICAL SERVICES INC. 18 COTE AVENUE, UNIT #11, GOFFSTOWN, NH 03045 TEL. (603) 624-2722 FAX. (603) 624-3733
PROJECT: EBSCO PUBLISHING WAREHOUSE EXPANSION IPSWICH, MASSACHUSETTS	DATE: JUNE, 22 2009
CLIENT: PARK CONSTRUCTION CORPORATION FITZWILLIAM, New Hampshire	DESIGN BY: H. WETHERBEE, P.E. DRAWN BY: D. HAYNER CHECKED BY: H. WETHERBEE, P.E.
	PROJECT No. ??? SHEET No. 1 of 1 SCALE: NONE

Geotechnical Services, Inc. ◆ 18 Cote Avenue, Goffstown, NH 03045 Tel. 603.624.2722 Fax. 603.624.3733 ◆ 12 Rogers Road, Haverhill, MA 01835 Tel. 978.374.7744 Fax. 978.374.7799

TEST BORING LOG

Boring No. B - 2

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit / Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	5	5-8			Top 3" loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					7-5			
2		S2	2-4	7	1-4			Very Loose to loose wet Brown/Black fine Sand and Silt, trace to little Organics (FILL)
3					6-6			
4								
5								
6		S3	5-7	12	2-1			Very Loose, Moist, Brown/Black fine Sand and Silt, trace organics
7					1-1			
8								
9								
10		S4	10-12	16	10-17			Light Brown, medium dense, wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
11					19-15			
12								
13								
14								
15								
16		S5	15-17	9	15-15			Light Brown, medium dense, wet fine to coarse Sand and Gravel, trace to little Silt
17					21-16			
18								
19								
20								
21		S-6	20-22	18	44-45			Light Brown, dense, wet fine to coarse Sand and Gravel, trace to little Silt
22					38-45			
23								Boring terminated at 22.0 feet without refusal
24								
25								
26								
27								
28								
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	11:30	n/a	n/a	1.0 ft

Sample Identification
O = Open Ended
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30: Very Stiff
Over 30: Hard

Granular Soils N-Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

Notes:

B-2

Geotechnical Services, Inc. ◆ 18 Cote Avenue, Goffstown, NH 03045 Tel. 603.624.2722 Fax. 603.624.3733 ◆ 12 Rogers Road, Haverhill, MA 01835 Tel. 978.374.7744 Fax. 978.374.7799

GSI

TEST BORING LOG

Boring No. B - 3

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit	Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	5	2-5			Top 3” Very loose to loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					4-3			
2								
3		S2	2-4	18	7-5			Top 6” Loose Dark Brown fine to medium Sand and gravel, little Silt, trace to little Organics (FILL)
4					5-5			Loose, Moist, Light Brown fine Sand and Silt (FILL)
5								
6		S3	5-7	18	1-1			Grey, wet, very soft Clay ,trace black fine sand (in seams) (CLAY)
7					1-1			--- q_u = 1.0 tsf using a pocket penetrometer
8								
9								
10		S4	10-12	18	1-2			Grey, wet, very soft Clay ,trace black fine sand (in seams) --- q_u = 1.0 tsf using a pocket penetrometer
11					2-2			Bottom 4” Black, Wet, fine Sand and Silt with some organics (PEAT)
12								
13								
14								
15								
16		S5	15-17	16	2-7			Light Brown, wet, loose to medium dense fine to medium Sand and Silt with some Clay (TILL)
17					14-17			Bottom 4”, Light Brown, medium dense, Wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
18								
19								
20								
21		S-6	20-22	14	33-44			Light Brown, dense, wet, fine to coarse Sand and Gravel, trace to little Silt
22					48-35			
23								
24								
25		S-7	25-27	14	5-19			Light Brown, medium dense, wet, fine to coarse Sand and Gravel, trace to little Silt
26					25-28			
27								
28								Boring terminated at 27 feet without refusal
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	9:30	n/a	n/a	3.5 ft

Sample Identification	Cohesive Soils N-Value	Granular Soils N-Value
O = Open Ended	0 to 2: Very Soft	0 to 4: Very Loose
U = Undisturbed	2 to 4: Soft	4 to 10: Loose
S = Split Spoon	4 to 8: Medium Stiff	11 to 30: Medium Dense
C = Rock Core	8 to 15: Stiff	31 to 50: Dense
G = Geoprobe	15 to 30: Very Stiff	Over 50: Very Dense
	Over 30: Hard	

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30”, Blows are per 6” taken with an 18” long x 1.5” I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

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TEST BORING LOG

Boring No. B - 4

Project	Ebsco Publishing Warehouse Add	GSI Project No.		Elevation	n/a
Location	Ipswich, MA	Project Mgr.	Glenn Zoladz	Datum	
Client	Ebsco Publishing	Inspector	Denis Hayner	Date Started	6/19/2009
Contractor	New Hampshire Boring	Checked By		Date Finished	6/19/2009
Driller	Gregg-Mike	Rig Make & Model	Scout Rig		

Item:	Auger	Casing	Sampler	Core Barrel		Truck		Skid		Hammer Type:
Type	HS		SS			Track	X	ATV		Safety Hammer
Inside Diameter (in).	2.25		1-3/8			Bomb		Geoprobe	X	Doughnut
Hammer Weight (lb)			140			Tripod		Other		Automatic
Hammer Fall (in.)			30			Winch		Cat Head	X	Roller Bit / Cutting Head

Depth (ft)	Casing (Blows/ft)	Sample Data: No.	Depth (ft)	Rec (in.)	SPT (Blows/ 6-in.)	Rock RQD (%)	PID Rdg. (ppm)	SOIL AND ROCK CLASSIFICATION-DESCRIPTION BURMISTER SYSTEM (SOIL) U.S. CORPS OF ENGINEERS SYSTEM (ROCK)
0		S1	0-2	4	4-4			Top 3" Very loose to loose Dark Brown fine to medium Sand, little Silt, trace to little Organics (TOPSOIL)
1					4-3			
2		S2	2-4	18	2-1			Very Loose Dark Brown fine to medium Sand and gravel, little Silt, trace to little Organics (FILL)
3					2-3			
4								
5								
6		S3	5-7	18	3-3			Very Loose, Moist, Brown/Black fine Sand and Silt, trace organics
7					2-3			
8								
9								
10		S4	10-12	18	1-5			Grey, Wet, soft Clay ,trace black fine sand (in seams) --- q_u = 1.0 tsf using a pocket penetrometer
11					3-3			Bottom 5" Black, Wet, fine Sand and Silt with some organics (PEAT)
12								
13								
14								
15								
16		S5	15-17	9	16-21			Light Brown, medium dense to dense, wet fine to coarse Sand and Gravel, trace to little Silt (TILL)
17					33-22			
18								
19		S-6	18-20	5	67-38			Light Brown, dense to very dense, wet fine to coarse Sand and Gravel, trace to little Silt
20					60-5"			
21								Refusal at 19.5 feet Boring terminated at 19.5 feet
22								
23								
24								
25								
26								
27								
28								
29								
30								

Water Level Data

Date	Time	Depth (ft) to: Bott. of Casing	Bott. of Hole	Water
6/19	1:30	n/a	n/a	3.5 ft

Sample Identification
O = Open Ended
U = Undisturbed
S = Split Spoon
C = Rock Core
G = Geoprobe

Cohesive Soils N-Value
0 to 2: Very Soft
2 to 4: Soft
4 to 8: Medium Stiff
8 to 15: Stiff
15 to 30: Very Stiff
Over 30: Hard

Granular Soils N-Value
0 to 4: Very Loose
4 to 10: Loose
11 to 30: Medium Dense
31 to 50: Dense
Over 50: Very Dense

Trace (0 to 5%) Little (10 to 20%) Some (20 to 35%) And (35 to 50%)

Standard Penetration Test (SPT) = 140# hammer falling 30", Blows are per 6" taken with an 18" long x 1.5" I.D. split spoon sampler in accordance with ASTM D 1586, unless otherwise noted.

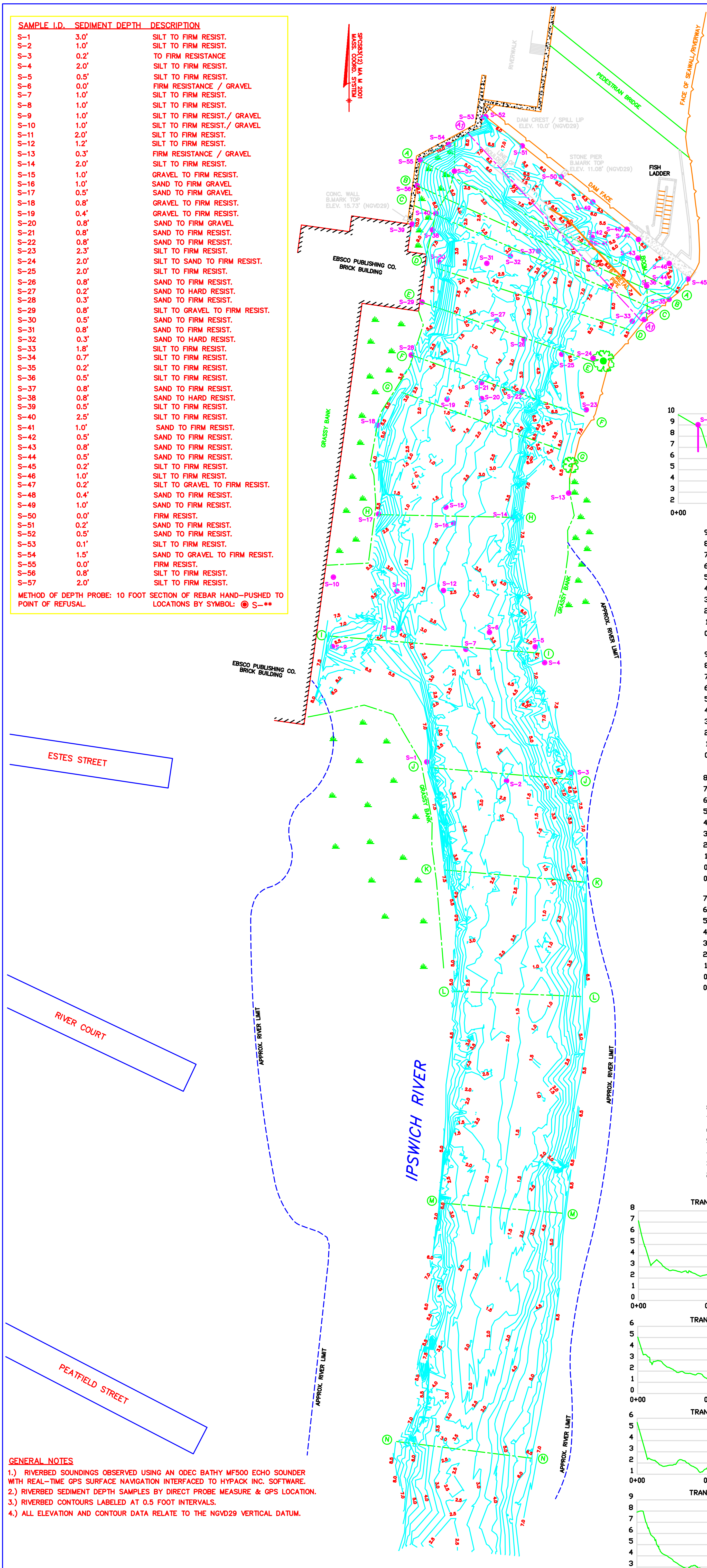
REMARKS: The stratification lines represent the approximate boundary between soil types and the transition may be gradual. Water level readings have been made in the test borings at times and under conditions stated on the test boring logs. Fluctuations in the level of the groundwater may occur due to other factors than those present at the time measurements were made.

Notes:

B-4

APPENDIX C

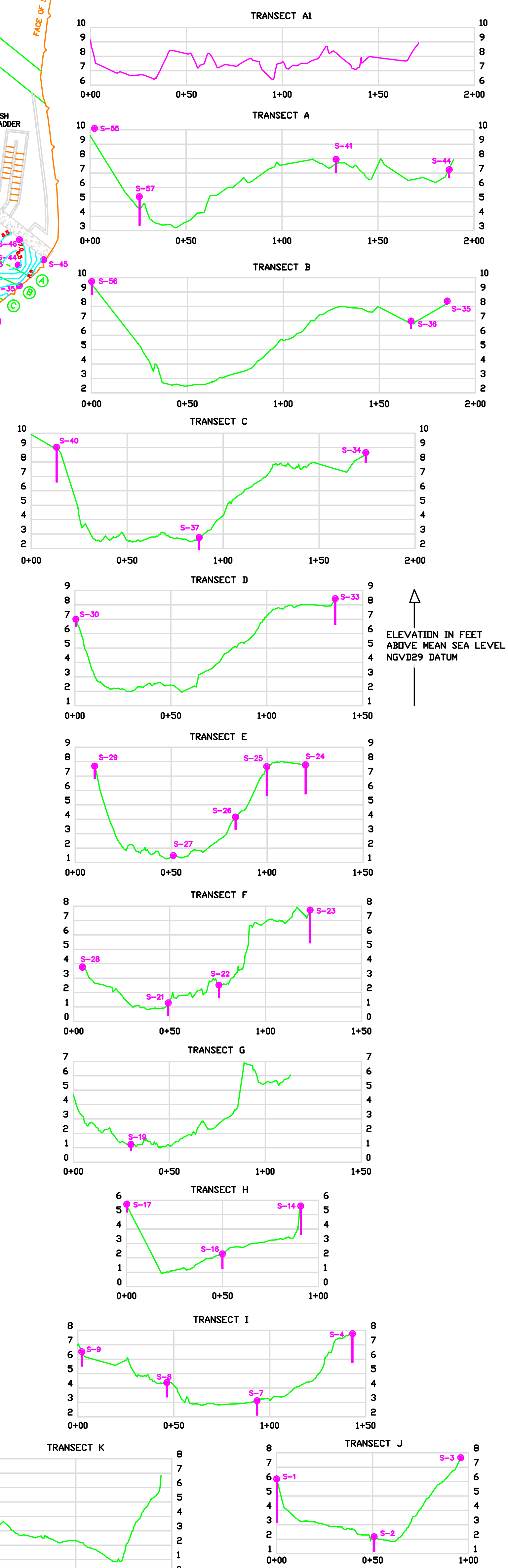
Excerpts from the Report titled “Ipswich River Mills Dam survey” prepared by Norde-East Survey dated 26 August 2014

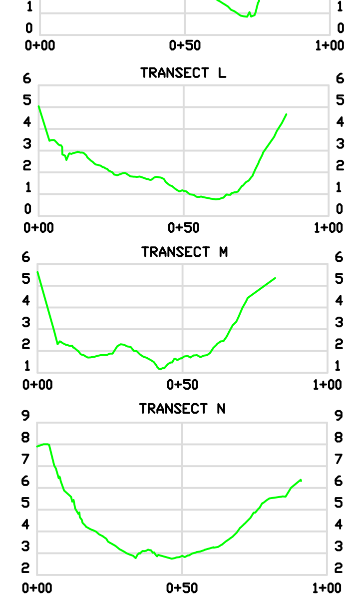


SAMPLE I.D.	SEDIMENT DEPTH	DESCRIPTION
S-1	3.0'	SILT TO FIRM RESIST.
S-2	1.0'	SILT TO FIRM RESIST.
S-3	0.2'	TO FIRM RESISTANCE
S-4	2.0'	SILT TO FIRM RESIST.
S-5	0.5'	SILT TO FIRM RESIST.
S-6	0.0'	FIRM RESISTANCE / GRAVEL
S-7	1.0'	SILT TO FIRM RESIST.
S-8	1.0'	SILT TO FIRM RESIST.
S-9	1.0'	SILT TO FIRM RESIST./ GRAVEL
S-10	1.0'	SILT TO FIRM RESIST./ GRAVEL
S-11	2.0'	SILT TO FIRM RESIST.
S-12	1.2'	SILT TO FIRM RESIST.
S-13	0.3'	FIRM RESISTANCE / GRAVEL
S-14	2.0'	SILT TO FIRM RESIST.
S-15	1.0'	GRAVEL TO FIRM RESIST.
S-16	1.0'	SAND TO FIRM GRAVEL
S-17	0.5'	SAND TO FIRM GRAVEL
S-18	0.8'	GRAVEL TO FIRM RESIST.
S-19	0.4'	GRAVEL TO FIRM RESIST.
S-20	0.8'	SAND TO FIRM GRAVEL
S-21	0.8'	SAND TO FIRM RESIST.
S-22	0.8'	SAND TO FIRM RESIST.
S-23	2.3'	SILT TO FIRM RESIST.
S-24	2.0'	SILT TO SAND TO FIRM RESIST.
S-25	2.0'	SILT TO FIRM RESIST.
S-26	0.8'	SAND TO FIRM RESIST.
S-27	0.2'	SAND TO HARD RESIST.
S-28	0.3'	SAND TO FIRM RESIST.
S-29	0.8'	SILT TO GRAVEL TO FIRM RESIST.
S-30	0.5'	SAND TO FIRM RESIST.
S-31	0.8'	SAND TO FIRM RESIST.
S-32	0.3'	SAND TO HARD RESIST.
S-33	1.8'	SILT TO FIRM RESIST.
S-34	0.7'	SILT TO FIRM RESIST.
S-35	0.2'	SILT TO FIRM RESIST.
S-36	0.5'	SILT TO FIRM RESIST.
S-37	0.8'	SAND TO FIRM RESIST.
S-38	0.8'	SAND TO HARD RESIST.
S-39	0.5'	SILT TO FIRM RESIST.
S-40	2.5'	SILT TO FIRM RESIST.
S-41	1.0'	SAND TO FIRM RESIST.
S-42	0.5'	SAND TO FIRM RESIST.
S-43	0.8'	SAND TO FIRM RESIST.
S-44	0.5'	SAND TO FIRM RESIST.
S-45	0.2'	SILT TO FIRM RESIST.
S-46	1.0'	SILT TO FIRM RESIST.
S-47	0.2'	SILT TO GRAVEL TO FIRM RESIST.
S-48	0.4'	SAND TO FIRM RESIST.
S-49	1.0'	SAND TO FIRM RESIST.
S-50	0.0'	FIRM RESIST.
S-51	0.2'	SAND TO FIRM RESIST.
S-52	0.5'	SAND TO FIRM RESIST.
S-53	0.1'	SILT TO FIRM RESIST.
S-54	1.5'	SAND TO GRAVEL TO FIRM RESIST.
S-55	0.0'	FIRM RESIST.
S-56	0.8'	SILT TO FIRM RESIST.
S-57	2.0'	SILT TO FIRM RESIST.

METHOD OF DEPTH PROBE: 10 FOOT SECTION OF REBAR HAND-PUSHED TO POINT OF REFUSAL. LOCATIONS BY SYMBOL: ◉ S-**

RIVERBED TRANSECTS: WEST TO EAST
HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 4'
SEDIMENT DEPTH LOCATIONS BY SYMBOL: ◉ S-**
WESTERLY BANK —— EASTERLY BANK





GENERAL NOTES
1.) RIVERBED SOUNDINGS OBSERVED USING AN ODEC BATHY MF500 ECHO SOUNDER WITH REAL-TIME GPS SURFACE NAVIGATION INTERFACED TO HYPACK INC. SOFTWARE.
2.) RIVERBED SEDIMENT DEPTH SAMPLES BY DIRECT PROBE MEASURE & GPS LOCATION.
3.) RIVERBED CONTOURS LABELED AT 0.5 FOOT INTERVALS.
4.) ALL ELEVATION AND CONTOUR DATA RELATE TO THE NGVD29 VERTICAL DATUM.

PATRICK J. McCORMACK – PROFESSIONAL LAND SURVEYOR — DATE

RIVERBED SURVEY
IPSWICH RIVER MILLS DAM
TOWN OF IPSWICH, MA
SCALE 1" = 30' AUGUST 9, 2014

SURVEYED BY
NORDE-EAST SURVEY
27 CONGRESS STREET, Suite 205-8, SALEM, MA 01970.
Tel.(978) 5421920 E-mail: norde-east@verizon.net

REVISIONS

NO.	DATE	DESCRIPTION	BY
1.	10/3/14	REVISE CONTOURS BY DAM	P.McC
2.	11/6/14	ADD TRANSECT A1	P.McC

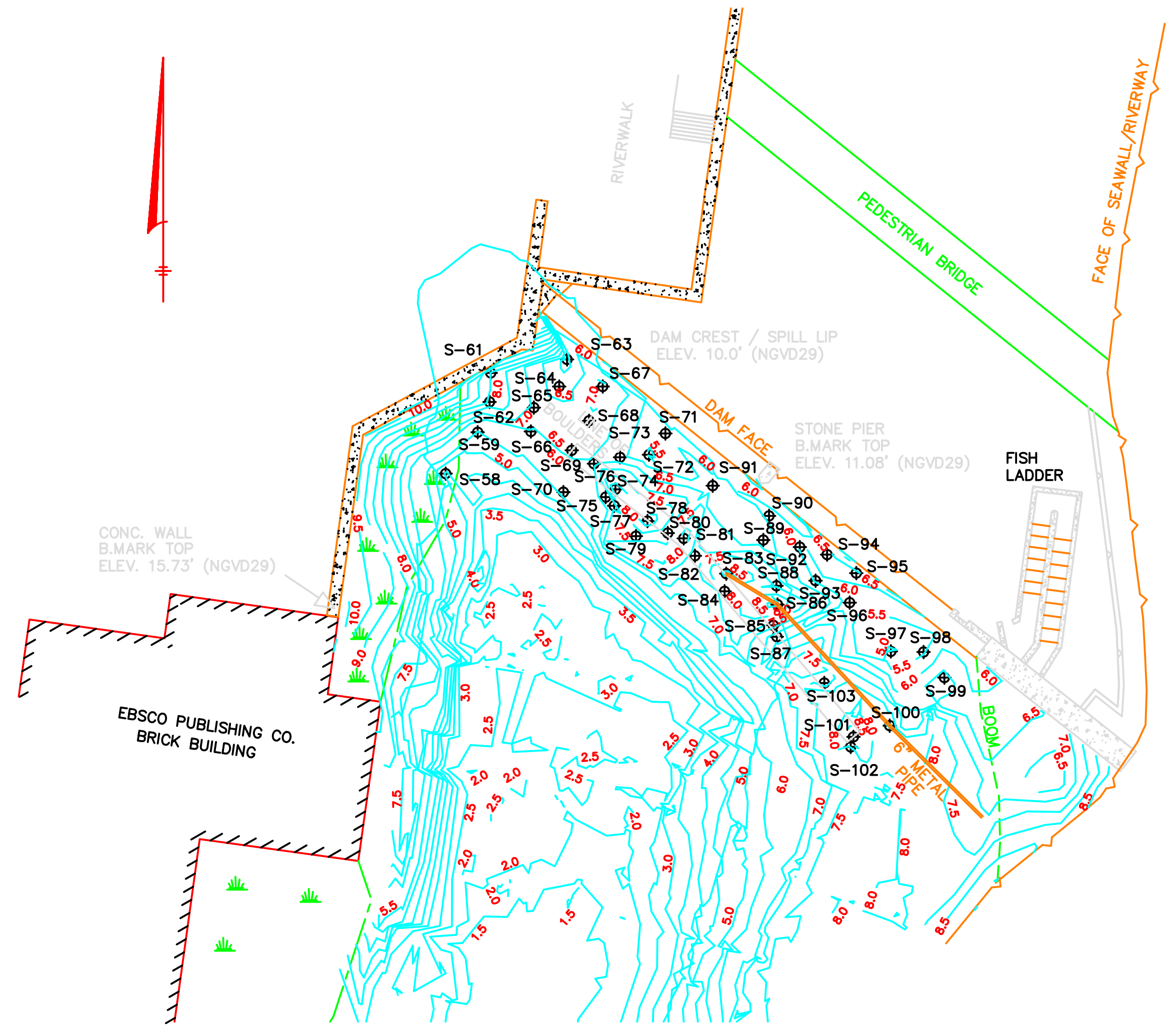
DRAWN BY: P.McCORMACK DATE: 08/09/14
FIELD CREW: P.McCORMACK, D.JONES, R.GUILIZIA DRAWING NO.: 777HYDRO.dwg
JOB NO.: 777 SHEET 1 OF 1

ADDITIONAL SAMPLES MEASURED 9/7/14

SAMPLE I.D.	SEDIMENT DEPTH	DESCRIPTION
S-58	1.2'	COARSE/SAND TO FIRM RESIST.
S-59	1.5'	SAND/COARSE TO FIRM RESIST.
S-61	1.7'	SILT TO FIRM RESIST.
S-62	0.3'	SAND TO FIRM RESIST.
S-63	0.1'	HARD PACK STONE.
S-64	0.8'	HARD PACK STONE.
S-65	0.8'	SILT/GRAVEL TO FIRM RESIST.
S-66	0.0'	HARD PACK GRAVEL.
S-67	0.0'	SOLID ROCK.
S-68	2.5'	LOOSE GRAVEL TO FIRM RESIST.
S-69	1.0'	SAND/GRAVEL TO FIRM RESIST.
S-70	1.5'	SILT/COARSE SAND TO FIRM RESIST.
S-71	0.5'	SILT/SAND TO FIRM RESIST.
S-72	0.3'	COBBLE TYPE SCOUR AREA.
S-73	1.3'	COBBLE TYPE SCOUR AREA.
S-74	0.0'	TOP OF BOULDER LINE.
S-75	1.2'	SILT/SAND TO FIRM RESIST.
S-76	0.5'	SILT/SAND TO FIRM RESIST.
S-77	0.1'	SILT TO FIRM RESIST.
S-78	0.0'	TOP OF BOULDER LINE.
S-79	0.2'	SILT/SAND TO FIRM RESIST.
S-80	0.8'	COARSE SAND TO FIRM RESIST.
S-81	0.0'	TOP OF BOULDER LINE.
S-82	0.3'	SAND TO FIRM RESIST.
S-83	___	TOP OF METAL PIPE.
S-84	0.5'	SILT/SAND TO FIRM RESIST.
S-85	0.0'	HARD PACK STONE.
S-86	___	TOP OF METAL PIPE.
S-87	0.0'	HARD PACK GRAVEL.
S-88	2.5'	SILT/SAND TO FIRM RESIST.
S-89	0.5'	SILYT/SAND TO FIRM RESIST.
S-90	0.0'	LARGE ROCK.
S-91	0.8'	COARSE SAND TO FRIM RESIST.
S-93	0.8'	SILT/SAND TO FIRM RESIST.
S-94	0.3'	SILT/SAND TO FIRM RESIST.
S-95	0.3'	SILT/SAND TO FIRM RESIST.
S-96	0.3'	SILT TO HARD RESIST.
S-97	0.8'	SILT/SAND TO FIRM RESIST.
S-98	0.3'	SILT TO HARD RESIST.
S-99	0.8'	SILT/GRAVEL TO FIRM RESIST.
S-100	___	TOP OF METAL PIPE.
S-101	0.0'	TOP OF BOULDER LINE.
S-102	0.5'	SILT/SAND TO FIRM RESIST.
S-103	0.3'	SILT/SAND TO FIRM RESIST.

S-60 SAMPLE SERIES NUMBER NOT ASSIGNED, FIELD NOTEBOOK OMISSION.

METHOD OF DEPTH PROBE: 10 FOOT SECTION OF REBAR HAND-PUSHED TO POINT OF REFUSAL. LOCATIONS BY SYMBOL: ⊕ S-**



GENERAL NOTES

1.) RIVERBED SOUNDINGS OBSERVED USING AN ODEC BATHY MF500 ECHO SOUNDER WITH REAL-TIME GPS SURFACE NAVIGATION INTERFACED TO HYPACK INC. SOFTWARE.
2.) RIVERBED SEDIMENT DEPTH SAMPLES BY DIRECT PROBE MEASURE & GPS LOCATION.
3.) RIVERBED CONTOURS LABELED AT 0.5 FOOT INTERVALS.
4.) ALL ELEVATION AND CONTOUR DATA RELATE TO THE NGVD29 VERTICAL DATUM.

PATRICK J. McCORMACK – PROFESSIONAL LAND SURVEYOR

DATE

0' 30' 60' 90'

RIVERBED SURVEY
IPSWICH RIVER MILLS DAM
TOWN OF IPSWICH, MA

SCALE 1" = 30' AUGUST 9, 2014

SURVEYED BY
NORDE-EAST SURVEY
27 CONGRESS STREET, Suite 205-8, SALEM, MA 01970.
Tel.(978) 5421920 E-mail: norde-east@verizon.net

REVISIONS			
1.	10/3/14	REVISE CONTOURS BY DAM	P.McC
2.	11/6/14	ADDITIONAL TRANSECT A1	P.McC
3.	11/6/14	ADDITIONAL SEDIMENT DEPTHS	P.McC
NO.	DATE	DESCRIPTION	BY

DRAWN BY: P.McCORMACK	DATE: 08/09/14
FIELD CREW: P.McCORMACK D.JONES R.GUILIZIA	DRAWING NO.: 777HYDRO.dwg
JOB NO.: 777	EXHIBIT ATTACHMENT TO SHEET 1

APPENDIX D
SGH Soil Test Borehole Logs and Observation Well Installation Details

BORING NUMBER SGH-2016-1

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

CLIENT Horsley & Witten Group

PROJECT NAME Ipswich Mills Dam Removal Feasibility Study

PROJECT NUMBER 160630.00

PROJECT LOCATION Ipswich MA

DATE STARTED 8/24/16 **COMPLETED** 8/24/16

GROUND ELEVATION 17 ft **HOLE SIZE** 4 inches

DRILLING CONTRACTOR Carr-Dee Corp. of MA

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger (HSA)

AT TIME OF DRILLING --- Not Encountered

LOGGED BY SFKeppel **CHECKED BY**

AT END OF DRILLING ---

NOTES Saltonsall St. and Union St. in front of EBSCO parking entrance

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	
0					0.3 4 in asphalt paving top coarse	16.7
	SS S-1	89	13-12-7 (19)		FILL: sand and gravel; light brown; dense; dry; fine to coarse grained; poorly graded; sub-angular; some silt; some asphalt fragments; trace brick Note: Borehole uncased from 0 ft to 4 ft. Borehole collapsed at an approximate depth of 2 ft.	
	SS S-2	38	9-11-21-25 (32)			
5	SS S-3	75	9-18-20-35 (38)		4.5 GLACIAL TILL: silty sand and gravel; light reddish brown; very dense; dry; fine to coarse grained; poorly graded; angular	12.5
	SS S-4	74	19-22-29-60/2"		Note: difficult to advance HSA at 7 ft to 7.5 ft.	
	SS S-5	28	82-100/1"		Note: recovered granite fragments at 8 ft.	
10	SS S-6	98	51-99-50/1"		Grading: gravel and silt; wet; some sand Note: HSA refusal at 11.25 ft (EOB). 11.3	5.8

Bottom of borehole at 11.3 feet.

Note: no indication of seasonal high water levels were observed within the borehole.

Appendix C

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

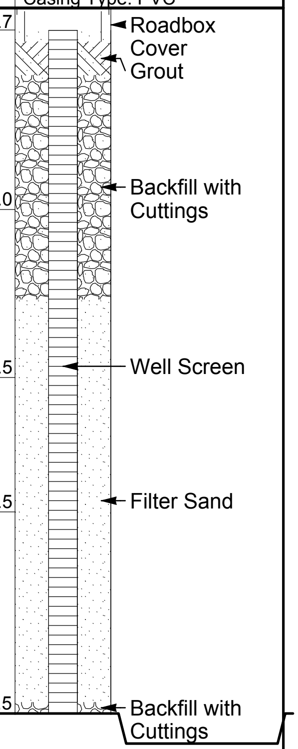
WELL NUMBER SGH-2016-2

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal Feasibility Study
PROJECT NUMBER 160630.00
PROJECT LOCATION Ipswich MA
DATE STARTED 8/24/16 **COMPLETED** 8/24/16
GROUND ELEVATION 18 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (HSA)
AT TIME OF DRILLING --- Not Encountered
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Estes St. 15' from Sidewalk (between No. 7 and No. 10)
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	Depth	MATERIAL DESCRIPTION	Elev.
0					0.3	4" of Asphalt	17.7
	SS S-1	17	20-54-19 (73)			FILL: sand; dark brown; very dense; dry; fine to medium grained; poorly graded; sub-angular	
	SS S-2	63	5-4-6-10 (10)		3.0	SILTY SAND TO SANDY SILT: light orange brown; medium dense; dry; fine grained; uniform; sub-rounded	15.0
5	SS S-3	75	3-4-12-9 (16)		5.5	SILT: light orange brown; very stiff; moist; low plasticity; some sand; trace clay.	12.5
	SS S-4	92	10-10-17-36 (27)		7.5	Note: observed gravel at tip of sampler	10.5
	SS S-5	29	19-23-47-41 (70)			GLACIAL TILL: sandy gravel; light reddish brown; very dense; moist; fine to coarse grained; poorly graded sub-angular; some silt	
10	SS S-6		50/0"		10.5	Note: HSA refusal at 10.5 ft (EOB).	7.5

WELL DIAGRAM

Casing Top Elev: 18.2 (ft)
Casing Type: PVC



Bottom of borehole at 10.5 feet.

Note: no indication of seasonal high water levels were observed within the borehole.

Appendix C

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

WELL NUMBER SGH-2016-3

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal Feasibility Study
PROJECT NUMBER 160630.00
PROJECT LOCATION Ipswich MA
DATE STARTED 8/24/16 **COMPLETED** 8/24/16
GROUND ELEVATION 14 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger (HSA)
∇ **AT TIME OF DRILLING** 11.75 ft / Elev 2.25 ft
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Driveway between No. 63 and No. 69 S. Main St.
AFTER DRILLING ---

Casing Top Elev: 14.2 (ft)
Casing Type: PVC

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elev.	WELL DIAGRAM
0					Gravel Base Pavement, fine to coarse, dense		Roadbox Cover; Grout
	SS S-1	67	32-38-33-30 (71)	1.0	FILL: sandy gravel; light brown; very dense; dry; fine to coarse grained; well graded; sub-angular; trace silt	13.0	
	SS S-2	50	9-22-19-16 (41)				Backfill with Cuttings; Riser
5	SS S-3	29	9-13-17-24 (30)				
	SS S-4	38	70-57-42-25 (99)	6.0	SAND: gravely sand; light orange brown; dense; dry; medium grained; well graded; sub-angular (outwash)	8.0	
	SS S-5	83	9-10-13-16 (23)	8.0	Grading: moist GLACIAL TILL: silty clay; olive; stiff; moist; low plasticity Qp=4.25 tsf, Sv=0.9 tsf	6.0	
10	SS S-6	25	21-31-28-20 (59)	10.0	Grading: sandy gravel; olive grey; very dense; moist; medium to coarse grained; poorly graded; sub-angular; some silty clay ∇ Grading wet at 12 ft.	4.0	Filter Sand; Well Screen
	SS S-7	50	8-13-15-30 (28)		Grading: fine to medium grained at 14 ft.		
15	SS S-8	38	36-37-100/4"	16.5	Note: HSA refusal at 16.5 ft (EOB).	-2.5	Backfill with Cuttings

Bottom of borehole at 16.5 feet.

Note: no indication of seasonal high water levels were observed within the borehole.

Appendix C

Appendix B

2018 Soil Test Boring Logs

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2018-1

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/1/18
GROUND ELEVATION 17 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	
0				0.3	ASPHALT	16.8
	SS S-1	29	7-3-3-5 (6)		FILL: sand; brown; loose; dry; fine to coarse grained; poorly graded; sub-angular; trace to some gravel; trace to little brick; trace silt	
5	SS S-2	75	8-11-30-25 (41)		grading dense at 4 ft bgs Observed casing blows increase at approximately 6 feet bgs. Possible stratum change.	
				6.0	GLACIAL TILL: gravel; gray and brown; very dense; poorly graded; rounded; little fine to coarse sand; trace clayey silt	11.0
	SS S-3	100	100/4"		Observed very hard drilling from approximately 7.5 to 8 ft bgs.	
	SS S-4	0	100/4"	8.4		8.6
					Bottom of borehole at 8.4 feet.	

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 7.5 ft bgs.

Note 3: Borehole backfilled with drill cuttings and an asphalt cold patch was placed at the ground surface.

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2018-2

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/1/18
GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Qp=Pocket Penetrometer (tsf), Sv=Pocket Torvane (tsf)
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation
0	SS S-1	50	3-3-6-7 (9)	0.5	TOPSOIL: silty loam; brown; dry	12.5
				3.0	FILL: silty sand; brown; loose; moist; fine to coarse grained; poorly graded; sub-angular; trace gravel; trace brick	10.0
	SS S-2	67	4-4-1-2 (5)			
5	SS S-3	0	1-1-1-1 (2)	6.0	SAND and SILT: olive to brown; very loose; fine grained;	7.0
	SS S-4	33	27-1-1-1 (2)		Silty CLAY: olive gray; very soft; low plasticity Solid wood observed in the wash water from a depth of 6 ft to 16 ft bgs. grading medium stiff to stiff at 8.5 ft bgs	
10	SS S-5	0	20-7-3 (10)			
	SS S-6	8	7-4-4-6 (8)	11.5		1.5
				13.0	Organic SILT: dark brown; medium stiff; low plasticity; some fine sand	0.0
	SS S-7	33	6-4-5-4 (9)		Silty CLAY: olive gray; medium stiff; low plasticity Qp = 0.75 tsf at 15 ft bgs Sv = 0.6 tsf at 15 ft bgs	
15	SS S-8	83	5-4-4-7 (8)			
				19.0		-6.0
20					Glacial TILL: gravelly sand; brown; very dense; fine to coarse grained; poorly graded; sub-angular; trace silty clay	
	SS S-9	67	77-26-37-27 (63)			
	SS S-10	100	26-39-20-12 (59)	24.0		-11.0

Bottom of borehole at 24.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings.

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2018-2A

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/2/18
GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY SFKeppel **CHECKED BY**
AT END OF DRILLING ---
NOTES Qp=Pocket Penetrometer (tsf), Sv=Pocket Torvane (tsf)
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	
0					Began sampling at 6 feet below ground surface (bgs).	
6.0	SS S-1	8	1-1-1-1 (2)		SILTY CLAY: olive gray; very soft; low plasticity; trace coarse sand	7.0
8.7	ST US-1	92			ORGANIC SILT: dark brown; trace sand; trace gravel; grading soft at 10 ft bgs	4.3
	SS S-2	0	1-1-1-2 (2)			
11.5	ST US-2	100			Silty CLAY: olive gray; stiff; low plasticity; fine sand seams 1/8 to 2 inches thick throughout sample. Observed a nail embedded in the bottom of shelby tube sample "US-2".	1.5
	ST US-3	100				
	SS S-3	100	7-9-11-13 (20)		Qp (tsf) = 0.75, 1.5, 2, 2.25 from 15.5 to 17.5 ft bgs Sv (tsf) = 0.3, 0.5, 0.55 from 15.5 to 17.5 ft bgs	
17.5					Bottom of borehole at 17.5 feet.	-4.5

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings.

Simpson Gumpertz & Heger, Inc.
41 Seyon St, Building 1 Suite 500
Waltham, MA 02453
Telephone: 781-907-9000
Fax: 781-907-9009

BORING NUMBER SGH-2018-3

CLIENT Horsley & Witten Group

PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings

PROJECT NUMBER 160630.01

PROJECT LOCATION Ipswich MA

DATE STARTED 6/2/18 **COMPLETED** 6/2/18

GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 2.5 ID/4.5 OD inches

DRILLING CONTRACTOR Carr-Dee Corp. of MA

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger

AT TIME OF DRILLING ---

LOGGED BY SFKeppel **CHECKED BY**

AT END OF DRILLING ---

NOTES Qp=Pocket Penetrometer (tsf), Sv=Pocket Torvane (tsf)

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	
0				0.5	TOPSOIL: silty loam; brown; dry	12.5
	SS S-1	42	8-4-8-6 (12)		FILL: silty sand; brown; medium dense; moist; fine to coarse grained; poorly graded; sub-angular; some gravel; trace brick; trace organics; slight chemical odor	
	SS S-2	33	5-6-4-2 (10)			
5	SS S-3	33	3-3-2-2 (5)		grading very loose to loose at 5 ft below ground surface (bgs)	
	SS S-4	75	2-1-1-1 (2)	9.0		4.0
10					Organic SILT: brown; soft; moist; trace wood fibers	
	SS S-5	58	2-2-3-4 (5)	11.0		2.0
					Silty CLAY: olive gray; wet; low plasticity; trace mottled fine sand Qp (tsf) = <0.25, 0.75, 1.5 from 11 to 12 ft bgs Sv (tsf) = 0.25, 0.35, 0.75 from 11 to 12 ft bgs	
	SS S-6	75	5-7-7-8 (14)	13.0		0.0
15					GLACIAL TILL: sand; brown; dense; fine to medium grained; poorly graded; sub-angular; some gravel; trace silty clay	
	SS S-7	88	8-13-17-18 (30)	17.0		-4.0

Bottom of borehole at 17.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings.

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BORING NUMBER SGH-2018-4

CLIENT Horsley & Witten Group

PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings

PROJECT NUMBER 160630.01

PROJECT LOCATION Ipswich MA

DATE STARTED 6/1/18 **COMPLETED** 6/1/18

GROUND ELEVATION 16 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches

DRILLING CONTRACTOR Carr-Dee Corp. of MA

GROUND WATER LEVELS:

DRILLING METHOD Case and Wash Boring

AT TIME OF DRILLING ---

LOGGED BY ZKBoswell **CHECKED BY**

AT END OF DRILLING ---

NOTES

AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION
0	SS S-1	63	1-2-2-4 (4)		FILL: silty sand and gravel; brown; loose; fine to coarse grained Hard drilling observed from approximately 4 to 7 feet bgs. Observed a change of color in the wash water from brown to gray at approximately 6 feet bgs.
5	SS S-2	75	15-17-28-28 (45)		grading medium dense at 5 ft bgs
6.0					GLACIAL TILL: fine to coarse sand and gravel; brown/gray; dense; poorly graded; sub-angular; trace to little silt Drill chatter observed at approximately 7 ft bgs. Observed drilling fluid loss at approximately 7.5 ft bgs. Observed hard drilling from approximately 7.5 to 8 ft bgs. Cuttings indiciate possible boulder. (10.0)
10	SS S-3	46	15-18-20-29 (38)		
12.0					(4.0)

Bottom of borehole at 12.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 7.5 ft bgs and open hole to termination.

Note 3: Borehole backfilled with drill cuttings.

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BORING NUMBER SGH-2018-5

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/1/18 **COMPLETED** 6/1/18
GROUND ELEVATION 16 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	Elevation
0				0.3	ASPHALT	15.7
	SS S-1	21	3-3-2-2 (5)		FILL: silty sand; brown; loose; fine to coarse grained; little silt; trace gravel	
5	SS S-2	17	2-6-6-6 (12)		grading medium dense at 5 ft bgs	
				8.0	Clayey SILT: brownish gray; medium stiff; low plasticity; trace fine sand	8.0
10	SS S-3	50	3-3-3-2 (6)			
				13.0	GLACIAL TILL: silty sand; grayish brown; dense; fine to coarse grained; sub-angular; some clayey silt; trace gravel	3.0
15	SS S-4	67	15-16-16-13 (32)			
				17.0		-1.0

Bottom of borehole at 17.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Borehole backfilled with drill cuttings and an asphalt cold patch was placed at the ground surface.

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BORING NUMBER SGH-2018-6

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/2/18 **COMPLETED** 6/2/18
GROUND ELEVATION 13 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	
0				0.8	ASPHALT	12.3
	SS S-1	67	70-32-10-16 (42)	2.5	FILL:dessicated concrete; gray; dense; angular; trace gravel; trace silt	10.5
	SS S-2	67	16-20-19-17 (39)	5.0	Clayey SILT: olive; hard; dry; low plasticity	8.0
5	SS S-3	67	20-22-21-21 (43)		GLACIAL TILL: sandy gravel; brown; dense; fine to coarse grained sand; poorly graded; rounded; trace to little clayey silt	
10	SS S-4	58	15-19-18-16 (37)	12.0		1.0

Bottom of borehole at 12.0 feet.

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 10 ft bgs.

Note 3: Borehole backfilled with drill cuttings and an asphalt cold patch was placed at the ground surface.

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BORING NUMBER SGH-2018-7

CLIENT Horsley & Witten Group
PROJECT NAME Ipswich Mills Dam Removal, Supplemental Borings
PROJECT NUMBER 160630.01
PROJECT LOCATION Ipswich MA
DATE STARTED 6/2/18 **COMPLETED** 6/2/18
GROUND ELEVATION 12 ft NAVD88 **HOLE SIZE** 4 ID/4.5 OD inches
DRILLING CONTRACTOR Carr-Dee Corp. of MA
GROUND WATER LEVELS:
DRILLING METHOD Case and Wash Boring
AT TIME OF DRILLING ---
LOGGED BY ZKBoswell **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	GRAPHIC LOG	MATERIAL DESCRIPTION	
0	SS S-1	25	1-3-4-5 (7)		FILL: silty sand; brown; loose; dry; fine to coarse grained; poorly graded; sub-angular; some silt; trace gravel; trace brick; trace cinders; trace ash	
	SS S-2	33	7-5-6-6 (11)			
5				5.0		7.0
	SS S-3	46	9-20-30-100 (50)		GLACIAL TILL: clay silt; brown; hard; some gravel; little fine to coarse sand; Observed very hard drilling at approximately 7.5 ft bgs. Rig was lifting off the ground. Split spoon refusal at approximately 7.5 ft bgs.	
				7.5		4.5
	SS S-4		100/0"		Bottom of borehole at 7.5 feet.	

Note 1: Ground surface elevation estimated from a draft plan entitled "Transects-Plan View" prepared by Horsley Witten Group, Inc., and dated September 23, 2016 and field measurements taken in August 2016 and June 2018. The elevation datum is North American Vertical Datum 1988 (NAVD 88).

Note 2: Drilled borehole with casing to approximately 7.5 ft bgs.

Note 3: Borehole backfilled with drill cuttings.

Appendix C
2018 Laboratory Test Results

THIELSCH ENGINEERING

195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
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Let's Build a Solid Foundation

Client Information:
Simpson Gumpertz & Heger, Inc
Waltham, MA
PM: Steve Keppel
Assigned By: Steve Keppel
Collected By: SK and ZB

Project Information:
Ipswitch- Supplemental Limited Subsurface Investigation
Ipswitch, MA
SGH Project Number: 160630.01

Report Date: 06.13.18

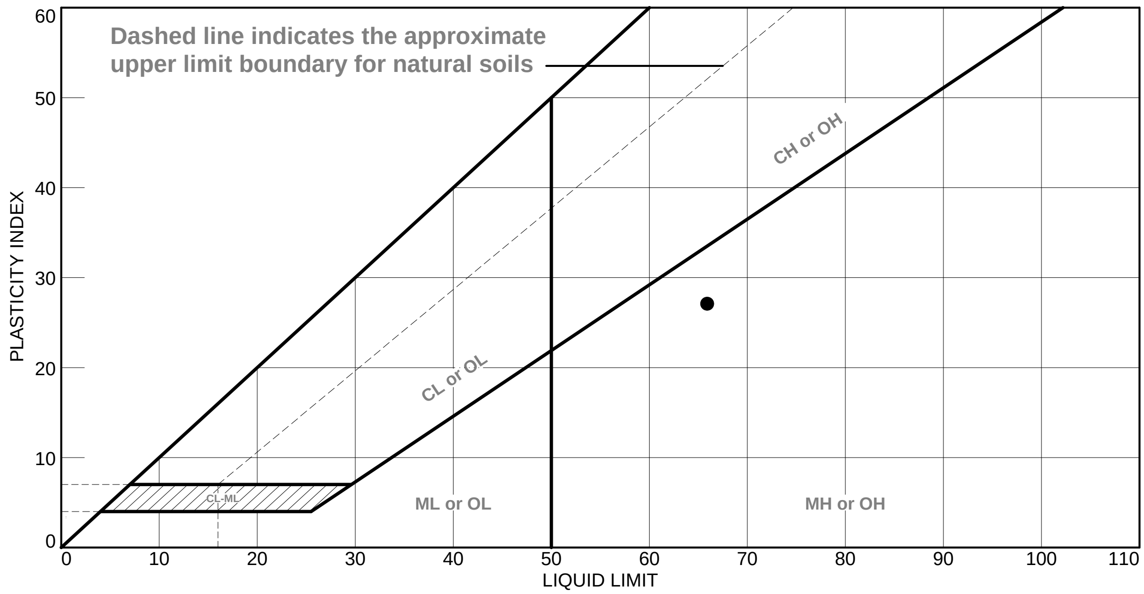
LABORATORY TESTING DATA SHEET

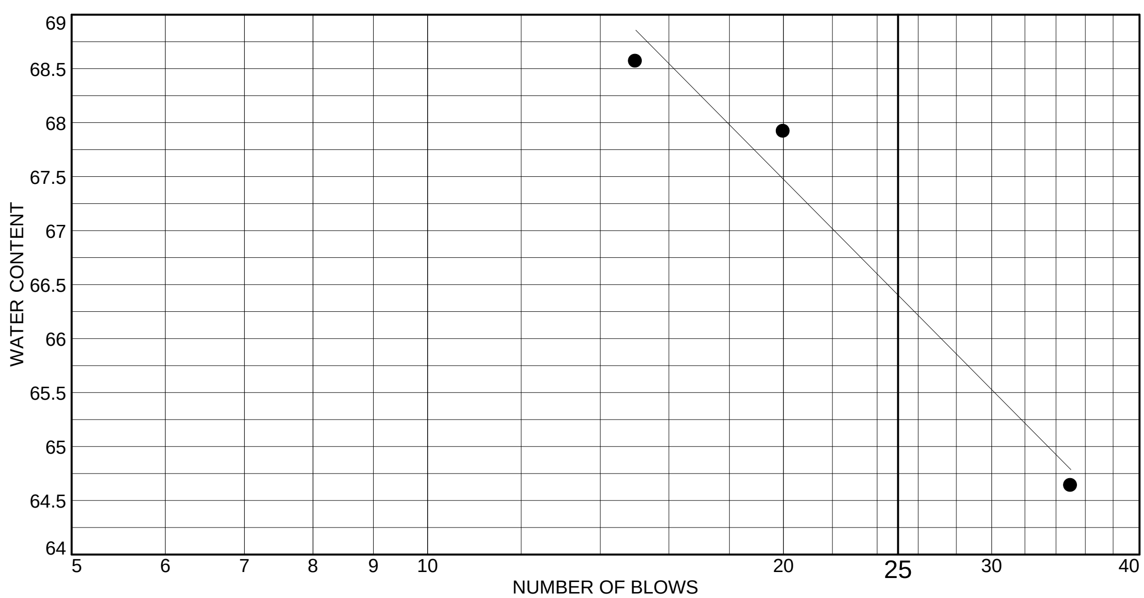
				Identification Tests								Proctor / CBR / Permeability Tests								
Boring ID	Sample No.	Depth (ft)	Laboratory No.	Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Perme-ability cm/sec	Laboratory Log and Soil Description
SGH-2018-3	S-5	10-11	18-S-737	68.3	66	39				10.3										Dark Brown Organic Silt (OH)
SGH-2018-3	S-6	12-13	18-S-738	33.1	42	21														Light Brown Lean Clay (CL)

Reviewed By ____________________

Date Reviewed *06.13.2018*

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark Brown Organic Silt (OH)	66	39	27			

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation

10 Estes Street

Source of Sample: SGH-2018-3 **Depth:** 10-11'

Sample Number: S-5

Thielsch Engineering Inc.

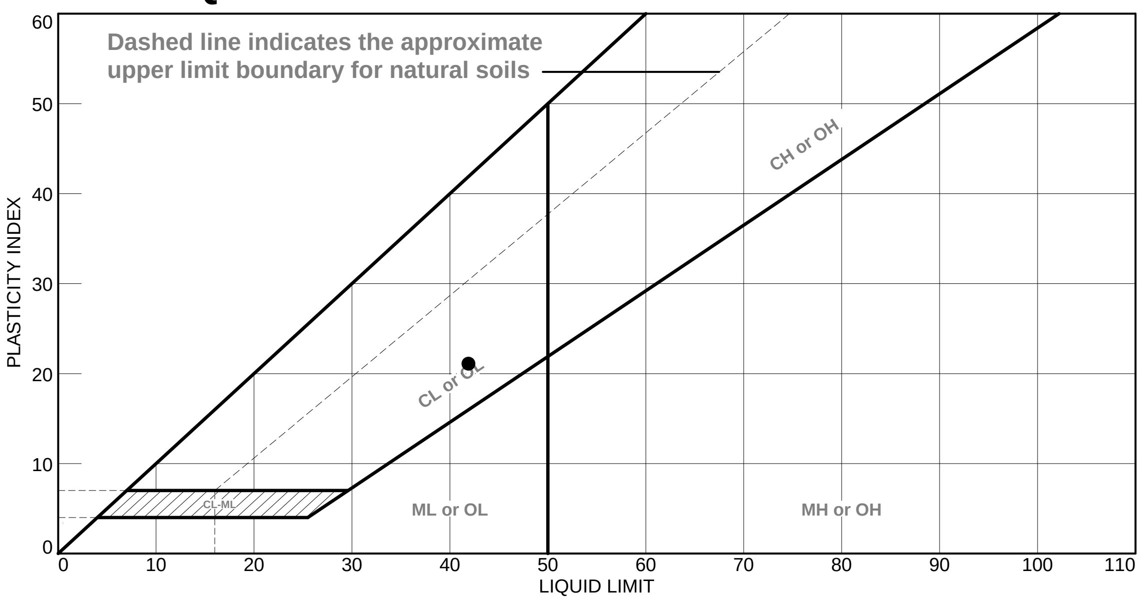
Cranston, RI

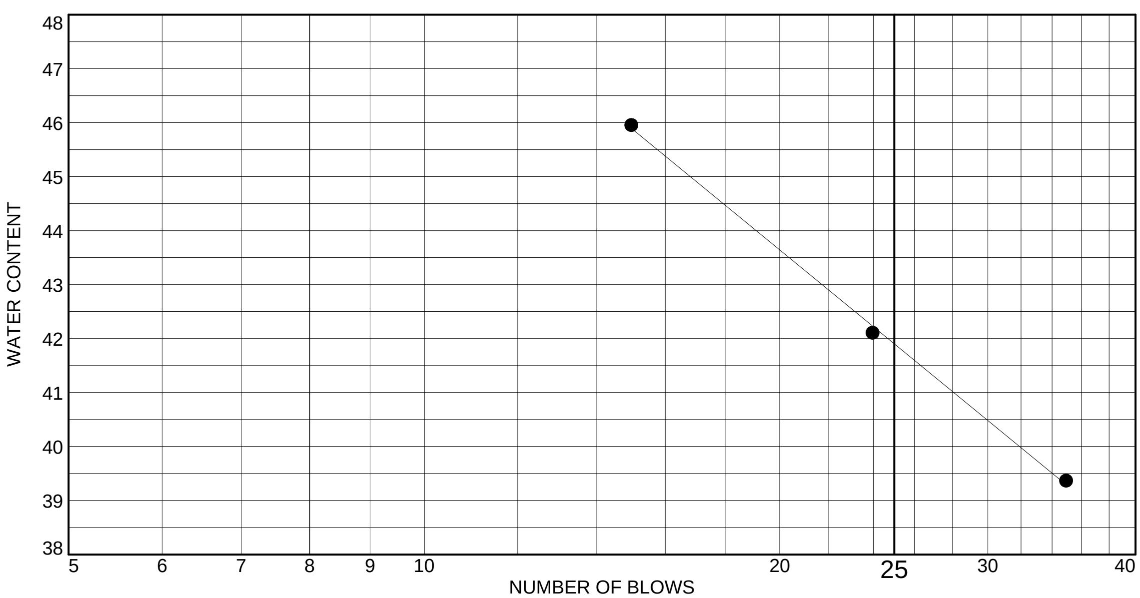
Remarks:

Figure 18-L-737

Tested By: MN **Checked By:** RR

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light Brown Lean Clay (ML)	42	21	21			

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation

10 Estes Street

Source of Sample: SGH-2018-3 **Depth:** 12-13'

Sample Number: S-6

Thielsch Engineering Inc.

Cranston, RI

Remarks:

Figure 18-L-738

Tested By: MN **Checked By:** RR

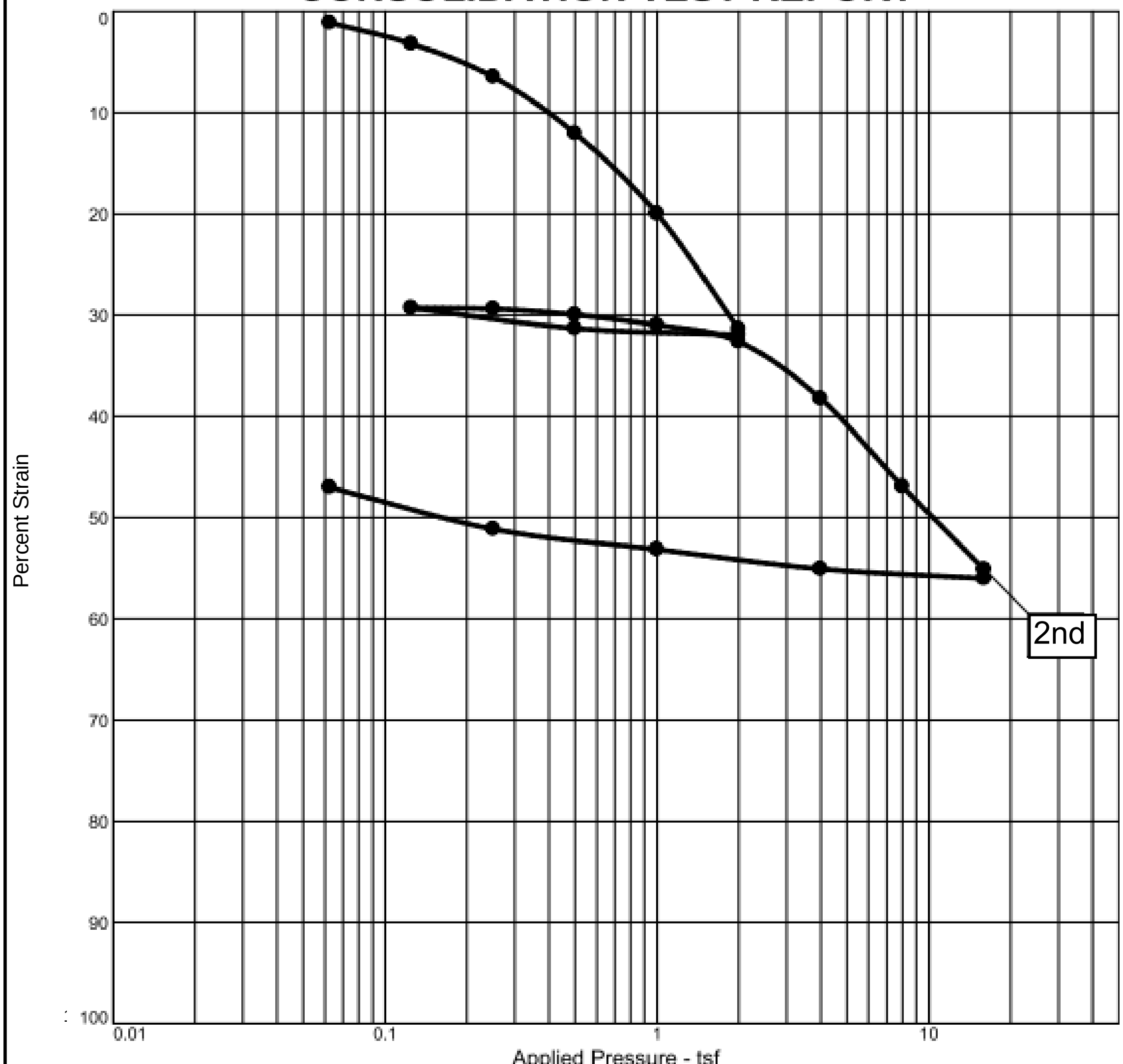
THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com *Let's Build a Solid Foundation*	Client Information: Simpson Gumpertz & Heger, Inc Waltham, MA PM: Steve Keppel Assigned By: Steve Keppel Collected By: SK and ZB	Project Information: Ipswitch- Supplemental Limited Subsurface Investigation Ipswitch, MA SGH Project Number: 160630.01 Summary Page: 2 of 3 Report Date: 06.14.18

LABORATORY TESTING DATA SHEET

				Identification Tests								Shear / Consolidation Tests								
Boring ID	Sample No.	Depth (ft)	Laboratory No.	Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Torvane or Type Test	$\bar{\sigma}_c$ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	EST. Internal Friction Angle	CR / RR	Laboratory Log and Soil Description
SGH-2018-2A	US-1	8-10'	18-T-735		Average Total Unit Weight (8-10') = 102.8 pcf															
		8.0-8.25																		Loose #4 gravel
		8.25-8.7																		Grey lean clay with gravel
		8.7-9.2		23.0																Dark brown organic silt with sand and gravel
		9.2-9.65																		Dark brown organic silt with sand and gravel * Photographed
		9.3-9.4		84.8						11.0										Dark brown organic silt and sand
		9.4-9.6		142.8								32.3	Cons						0.29 / 0.044	Dark brown organic silt and sand

Reviewed By ______________________ Date Reviewed *06.15.2018*

CONSOLIDATION TEST REPORT



MATERIAL DESCRIPTION	USCS	AASHTO
Dark Brown organic silt		

LL	PI	Sp. Gr.	Overburden (tsf)	Dry Dens. (pcf)		Moisture		Saturation		Void Ratio		P_c (tsf)	C_c
				Init.	Final	Init.	Final	Init.	Final	Init.	Final		
NV	NP	2.0		32.3	60.9	142.8	69.8	99.6 %	100.0	2.870	1.050	0.6	1.18

Preparation Process: Trimmed using cutting ring

Condition of Test: Saturated at 2 tsf

D2435 Method	C_r	Swell Press. (tsf)	%
B	0.17		

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street

Source: SGH-2018-2A **Depth:** 8-10' **Sample No.:** US-1

Thielsch Engineering Inc.

Cranston, RI

Remarks:

End of Primary Test at 9.45-9.55'.
Assumed specific gravity to be 2.0.

Checked By: sa

Title: Laboratory Manager

Figure C-735-1

Tested By: RR

CONSOLIDATION TEST DATA

6/14/2018

Client: Simpson Gumpertz & Heger
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Ipswich, MA
Project Number: 160630.01
Location: SGH-2018-2A
Depth: 8-10' **Sample Number:** US-1
Material Description: Dark Brown organic silt
Liquid Limit: NV **Plasticity Index:** NP
Preparation Process: Trimmed using cutting ring
Condition of Test: Saturated at 2 tsf
Test Method: B **Final Density:** 60.9 **Figure No.:** C-735-1
Testing Remarks: End of Primary Test at 9.45-9.55'. Assumed specific gravity to be 2.0.
Tested By: RR **Checked by:** sa **Title:** Laboratory Manager

Test Specimen Data

NATURAL MOISTURE		VOID RATIO		AFTER TEST	
Wet w+t =	205.48 g.	Spec. Gr.	= 2.0	Wet w+t =	117.77 g.
Dry w+t =	114.26 g.	Est. Ht. Solids	= 0.258 in.	Dry w+t =	90.24 g.
Tare Wt. =	50.40 g.	Init. V.R.	= 2.870	Tare Wt. =	50.80 g.
Moisture =	142.8 %	Init. Sat.	= 99.6 %	Moisture =	69.8 %
UNIT WEIGHT		**TEST START**		Dry Wt. =	39.44 g.
Height =	1.000 in.	Height	= 1.000 in.		
Diameter =	2.500 in.	Diameter	= 2.500 in.		
Weight =	100.96 g.				
Dry Dens. =	32.3 pcf				

End-Of-Load Summary

Pressure (tsf)	Final Dial (in.)	Deformation (in.)	C_V ($cm.^2$/sec.)	C_α	Void Ratio	% Strain
start	0.00394	0.00000			2.870	
0.06	0.01516	0.01122	0.0110		2.826	1.1 Comprs.
0.13	0.03592	0.03198	0.0124		2.746	3.2 Comprs.
0.25	0.06866	0.06472	0.0037		2.619	6.5 Comprs.
0.50	0.12440	0.12046	0.0031		2.404	12.0 Comprs.
1.00	0.20370	0.19976	0.0024		2.097	20.0 Comprs.
2.00	0.32350	0.31956	0.0016		1.633	32.0 Comprs.
0.50	0.31710	0.31316			1.658	31.3 Comprs.
0.13	0.29690	0.29296			1.736	29.3 Comprs.
0.25	0.29780	0.29386	0.0044		1.733	29.4 Comprs.
0.50	0.30360	0.29966	0.0068		1.710	30.0 Comprs.
1.00	0.31380	0.30986	0.0063		1.671	31.0 Comprs.
2.00	0.33020	0.32626	0.0030		1.607	32.6 Comprs.
4.00	0.38630	0.38236	0.0018		1.390	38.2 Comprs.
8.00	0.47310	0.46916	0.0014		1.054	46.9 Comprs.
16.00	0.56420	0.56026	0.0009		0.702	56.0 Comprs.
4.00	0.55470	0.55076			0.738	55.1 Comprs.
1.00	0.53550	0.53156			0.813	53.2 Comprs.
0.25	0.51540	0.51146			0.891	51.1 Comprs.
0.06	0.47410	0.47016			1.050	47.0 Comprs.

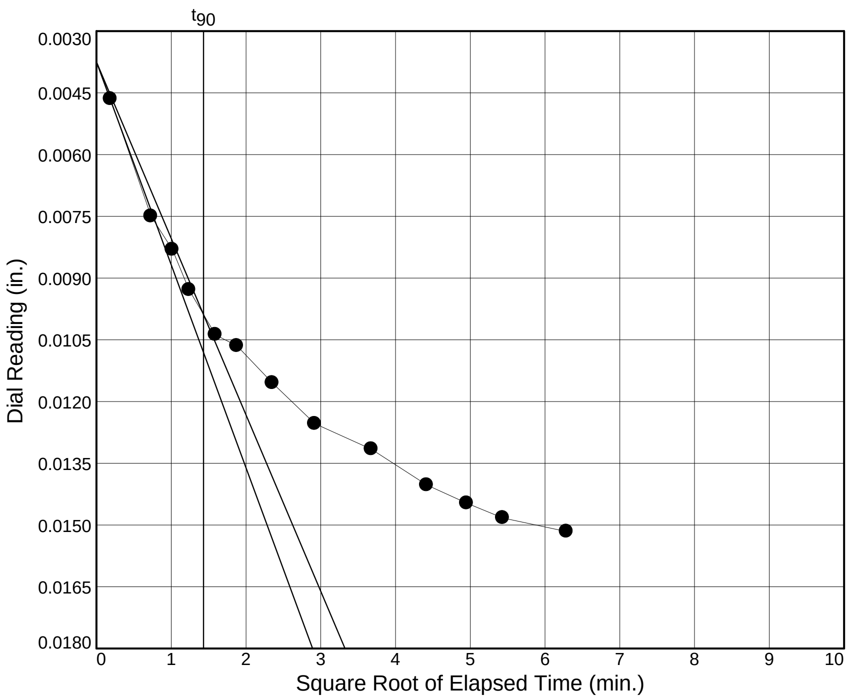
TEST RESULTS SUMMARY

Compression index (C_c), tsf = 1.18 **Preconsolidation pressure (P_p), tsf =** 0.6 **Void ratio at P_p (e_m) =** 2.327

Recompression index (C_r) = 0.17

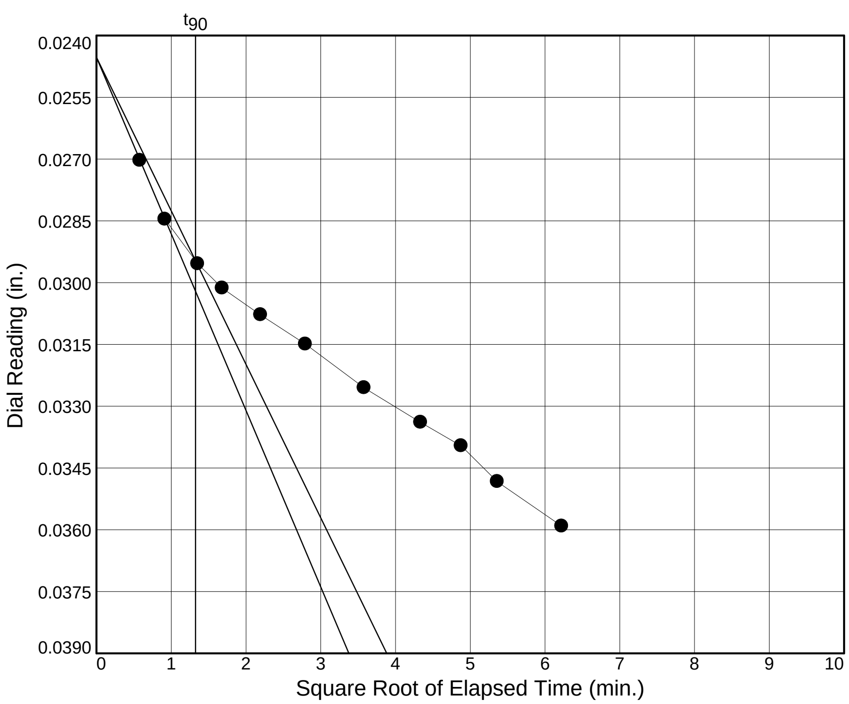
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 1
Load= 0.06 tsf
D_0 = 0.0037
D_{90} = 0.0099
D_{100} = 0.0106
T_{90} = 2.05 min.

C_v @ T_{90}
0.0110 cm.^{2}/sec.



Load No.= 2
Load= 0.13 tsf
D_0 = 0.0245
D_{90} = 0.0295
D_{100} = 0.0300
T_{90} = 1.75 min.

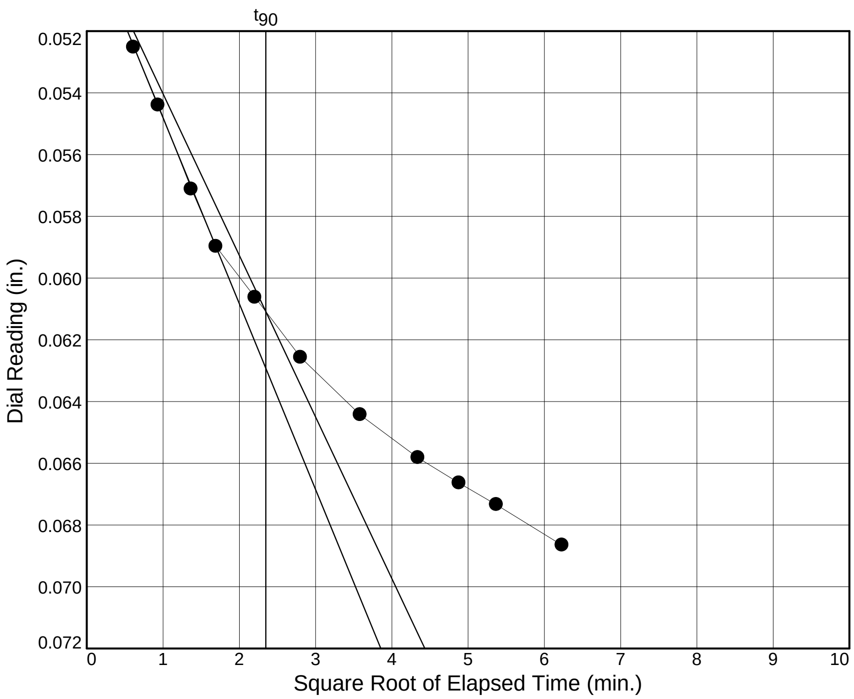
C_v @ T_{90}
0.0124 cm.^{2}/sec.

Figure C-735-2

Thielsch Engineering Inc.

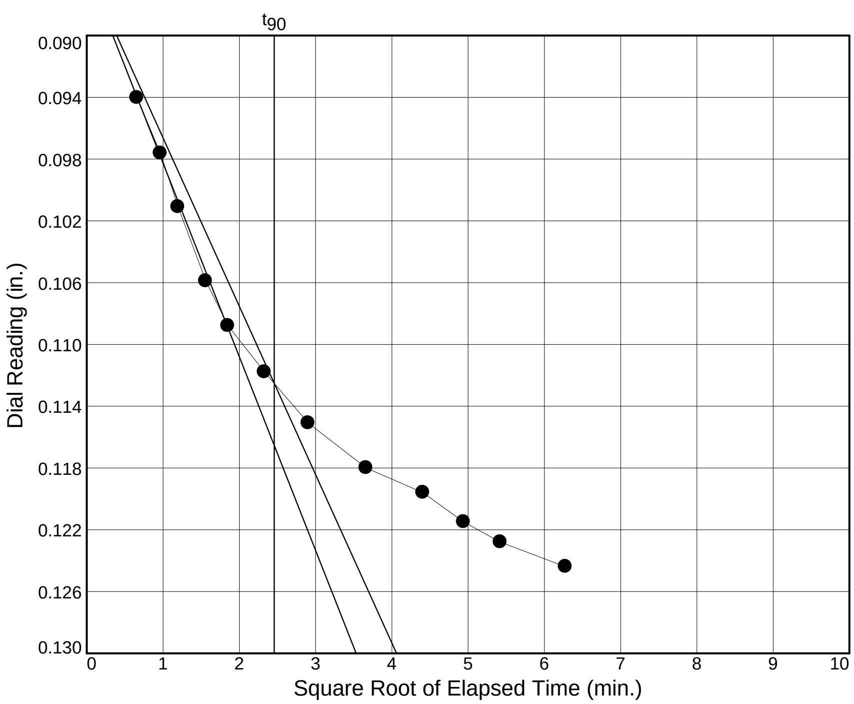
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 3
Load= 0.25 tsf
D_0 = 0.0488
D_{90} = 0.0611
D_{100} = 0.0625
T_{90} = 5.51 min.

C_v @ T_{90}
0.0037 cm.^{2}/sec.



Load No.= 4
Load= 0.50 tsf
D_0 = 0.0857
D_{90} = 0.1125
D_{100} = 0.1155
T_{90} = 6.04 min.

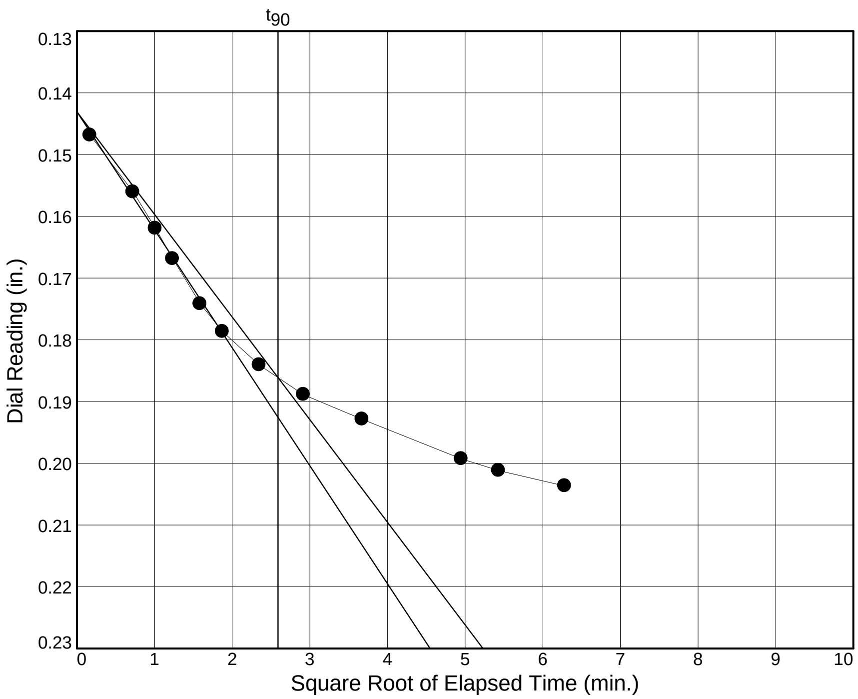
C_v @ T_{90}
0.0031 cm.^{2}/sec.

Figure C-735-3

Thielsch Engineering Inc.

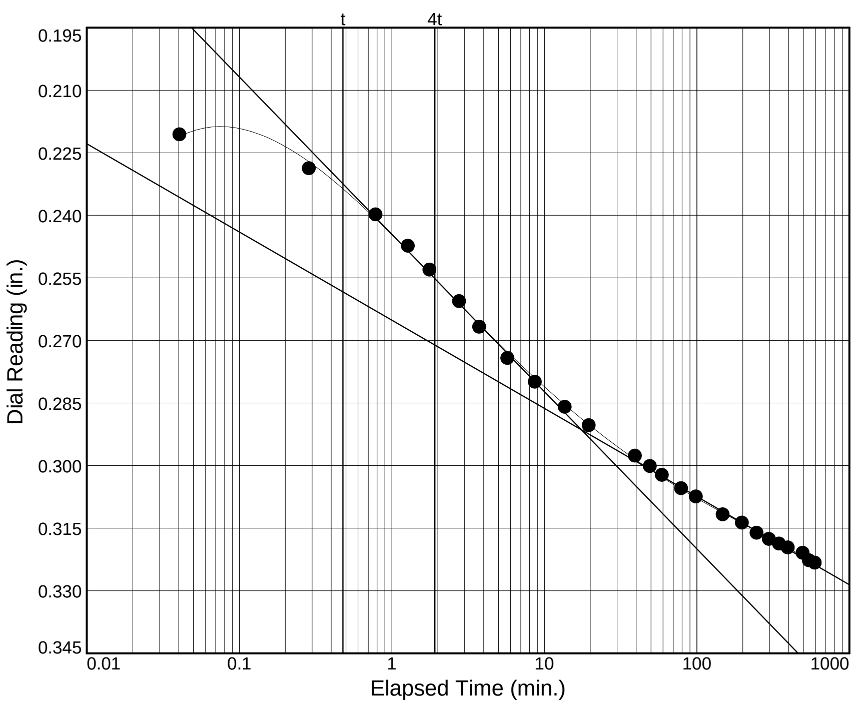
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 5
Load= 1.00 tsf
D_0 = 0.1431
D_{90} = 0.1861
D_{100} = 0.1909
T_{90} = 6.71 min.

C_v @ T_{90}
0.0024 cm.^{2}/sec.



Load No.= 6
Load= 2.00 tsf
D_0 = 0.2121
D_{50} = 0.2518
D_{100} = 0.2914
T_{50} = 1.56 min.

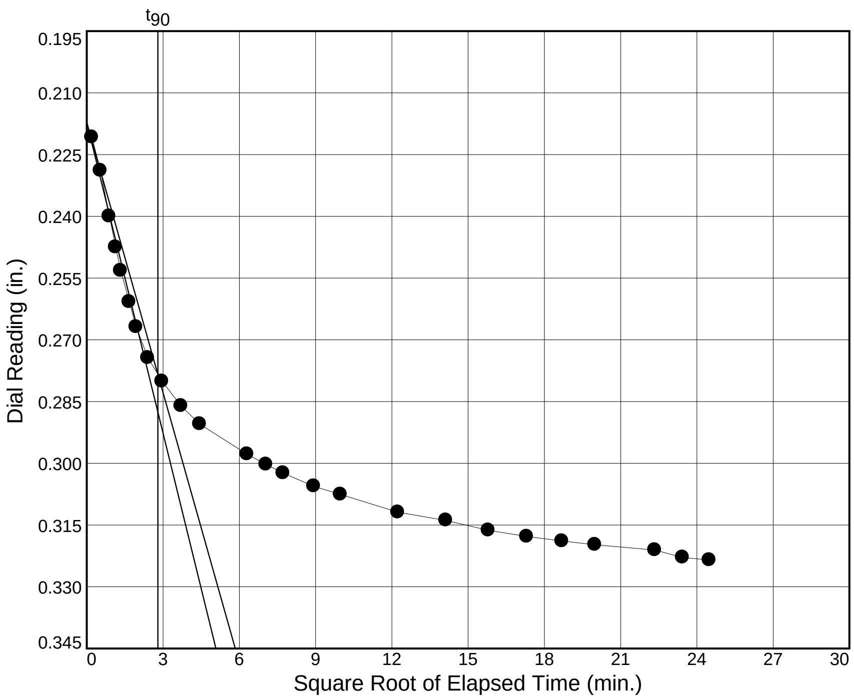
C_v @ T_{50}
0.0019 cm.^{2}/sec.

C_α = 0.021

Figure C-735-4

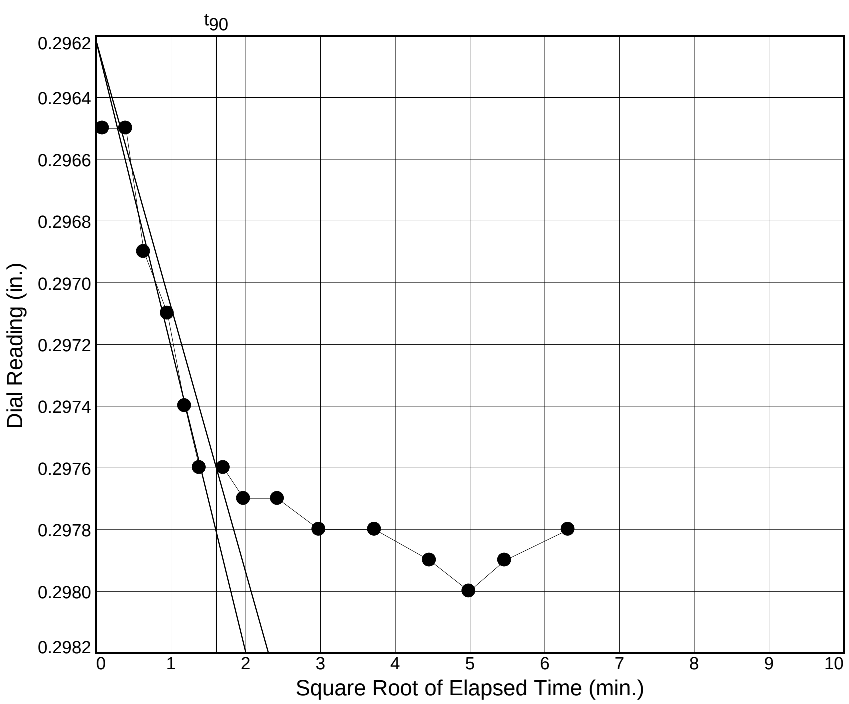
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 6
Load= 2.00 tsf
D_0 = 0.2172
D_{90} = 0.2784
D_{100} = 0.2852
T_{90} = 7.81 min.

C_V @ T_{90}
0.0016 cm.^{2}/sec.



Load No.= 9
Load= 0.25 tsf
D_0 = 0.2962
D_{90} = 0.2976
D_{100} = 0.2978
T_{90} = 2.58 min.

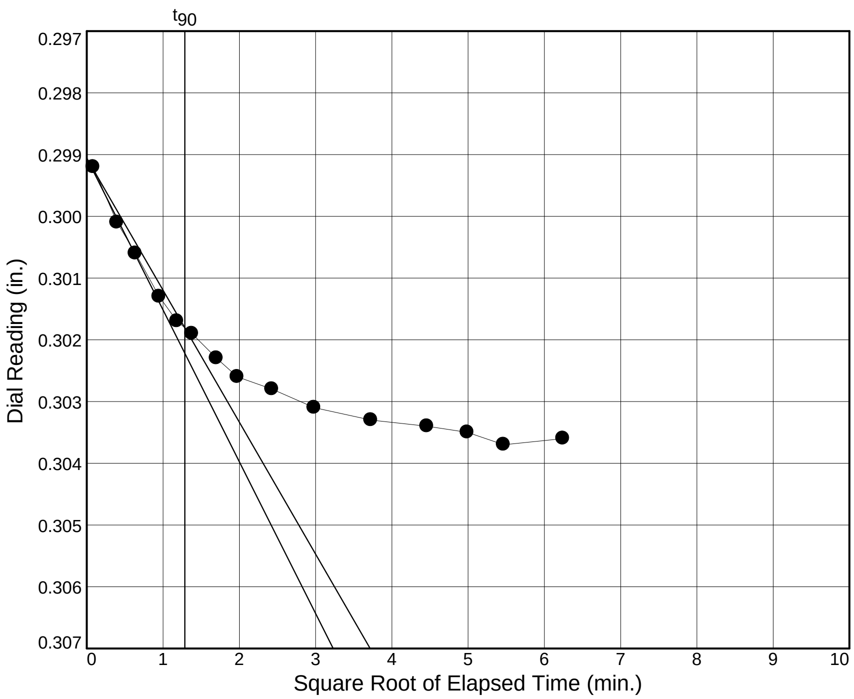
C_V @ T_{90}
0.0044 cm.^{2}/sec.

Figure C-735-5

Thielsch Engineering Inc.

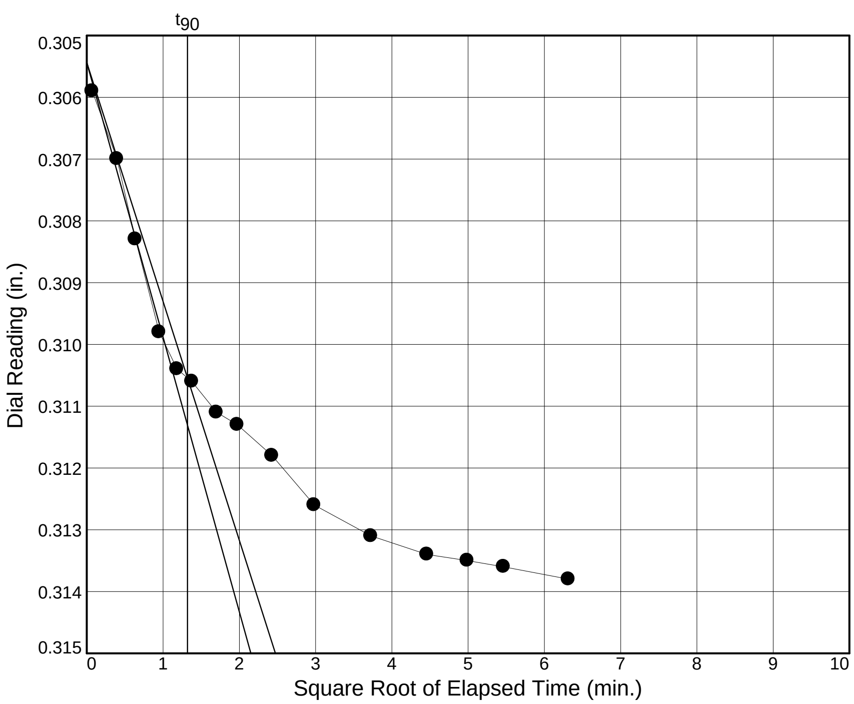
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 10
Load= 0.50 tsf
D_0 = 0.2991
D_{90} = 0.3018
D_{100} = 0.3021
T_{90} = 1.65 min.

C_v @ T_{90}
0.0068 cm.^{2}/sec.



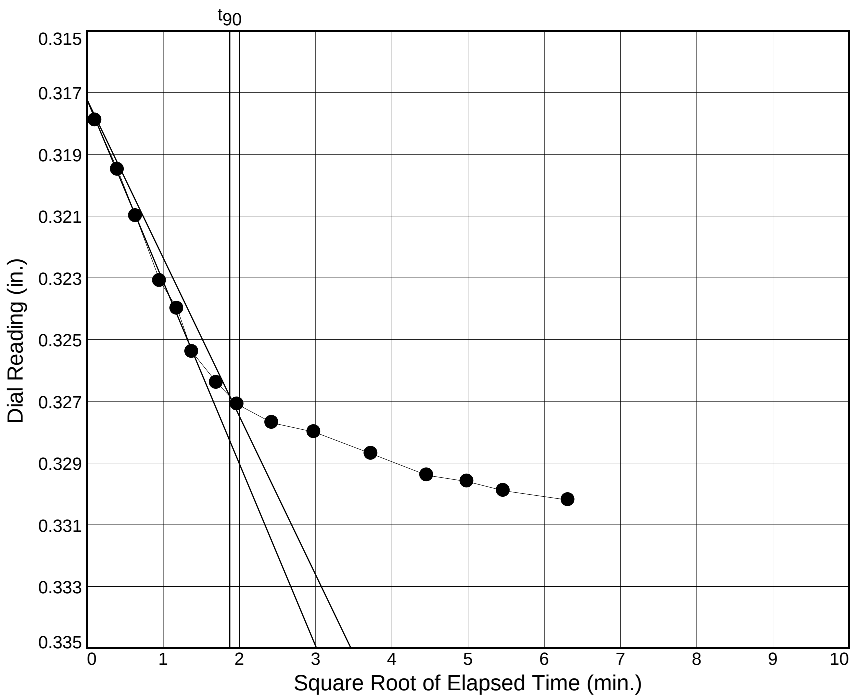
Load No.= 11
Load= 1.00 tsf
D_0 = 0.3054
D_{90} = 0.3105
D_{100} = 0.3111
T_{90} = 1.74 min.

C_v @ T_{90}
0.0063 cm.^{2}/sec.

Figure C-735-6

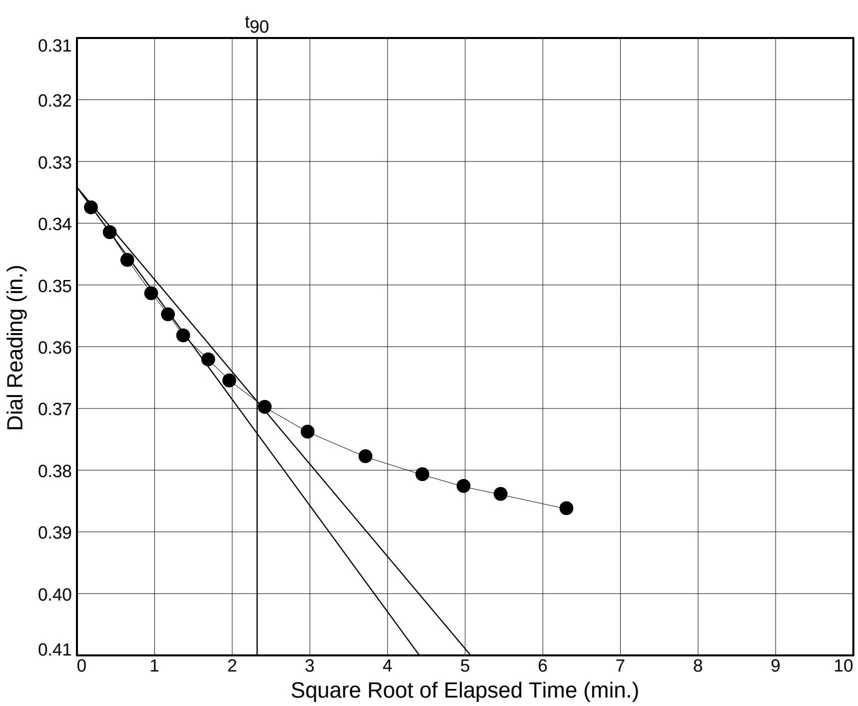
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 8-10'   Sample Number: US-1



Load No.= 12
Load= 2.00 tsf
D_0 = 0.3172
D_{90} = 0.3268
D_{100} = 0.3279
T_{90} = 3.51 min.

C_v @ T_{90}
0.0030 cm.^{2}/sec.



Load No.= 13
Load= 4.00 tsf
D_0 = 0.3342
D_{90} = 0.3689
D_{100} = 0.3727
T_{90} = 5.39 min.

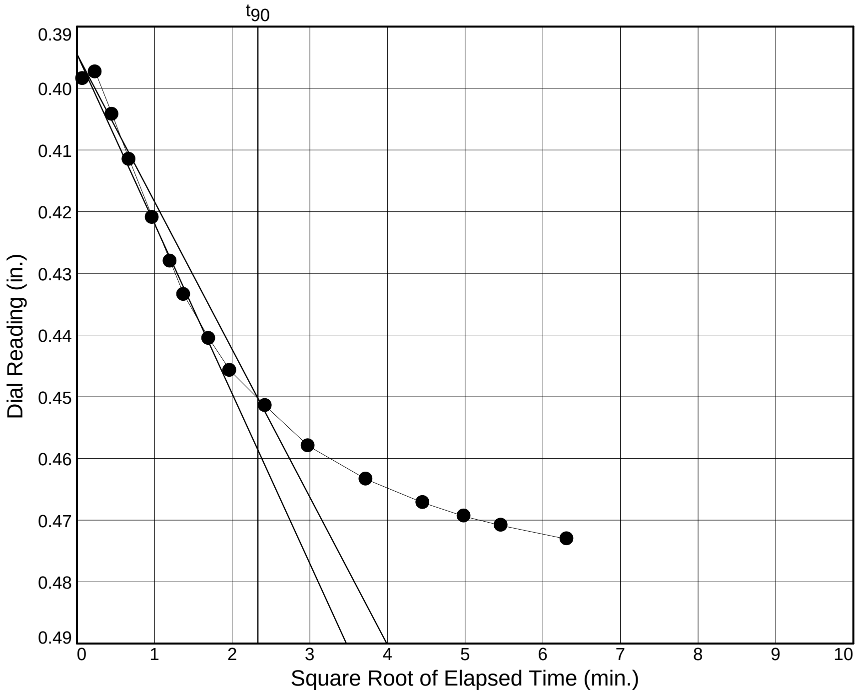
C_v @ T_{90}
0.0018 cm.^{2}/sec.

Figure C-735-7

Thielsch Engineering Inc.

Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 8-10'   Sample Number: US-1



Load No.= 14

Load= 8.00 tsf

D_0 = 0.3945

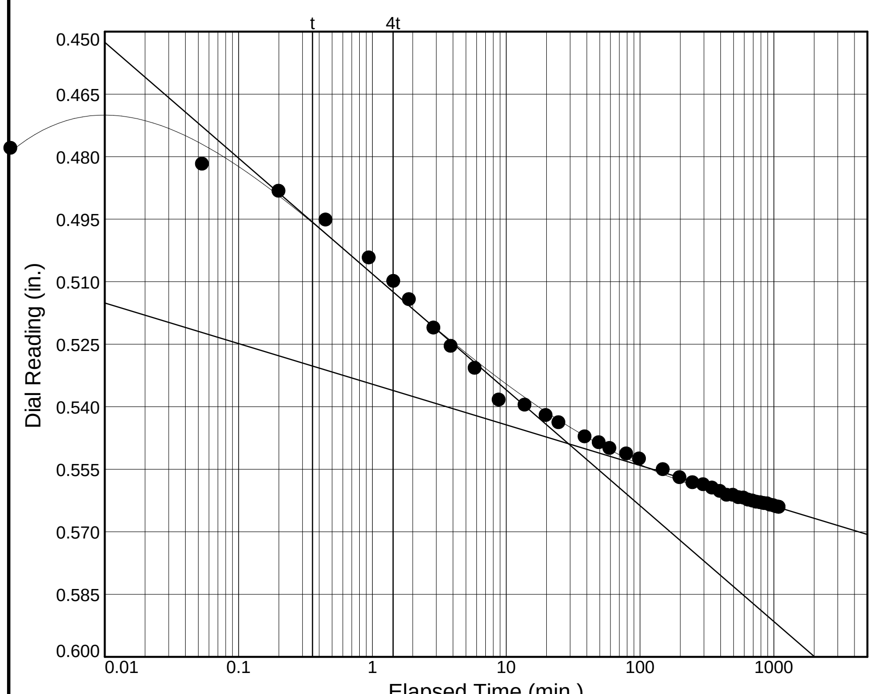
D_{90} = 0.4503

D_{100} = 0.4565

T_{90} = 5.43 min.

C_v @ T_{90}

0.0014 cm.^{2}/sec.



Load No.= 15

Load= 16.00 tsf

D_0 = 0.4793

D_{50} = 0.5141

D_{100} = 0.5489

T_{50} = 1.64 min.

C_v @ T_{50}

0.0008 cm.^{2}/sec.

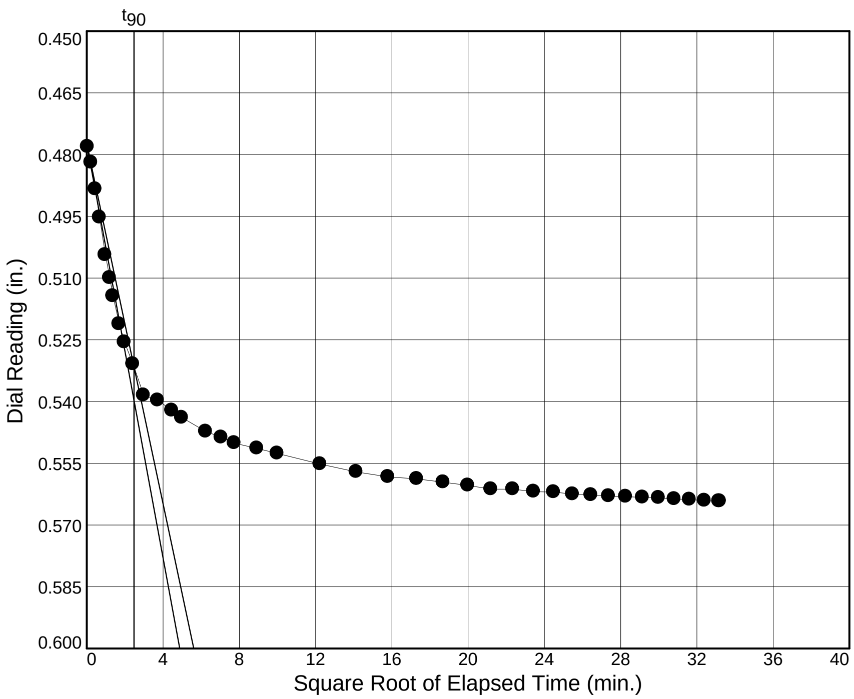
C_α = 0.010

Figure C-735-8

Thielsch Engineering Inc.

Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 8-10' Sample Number: US-1



Load No.= 15

Load= 16.00 tsf

D_0 = 0.4775

D_{90} = 0.5315

D_{100} = 0.5375

T_{90} = 6.13 min.

C_v @ T_{90}
0.0009 cm.^{2}/sec.

Figure C-735-9

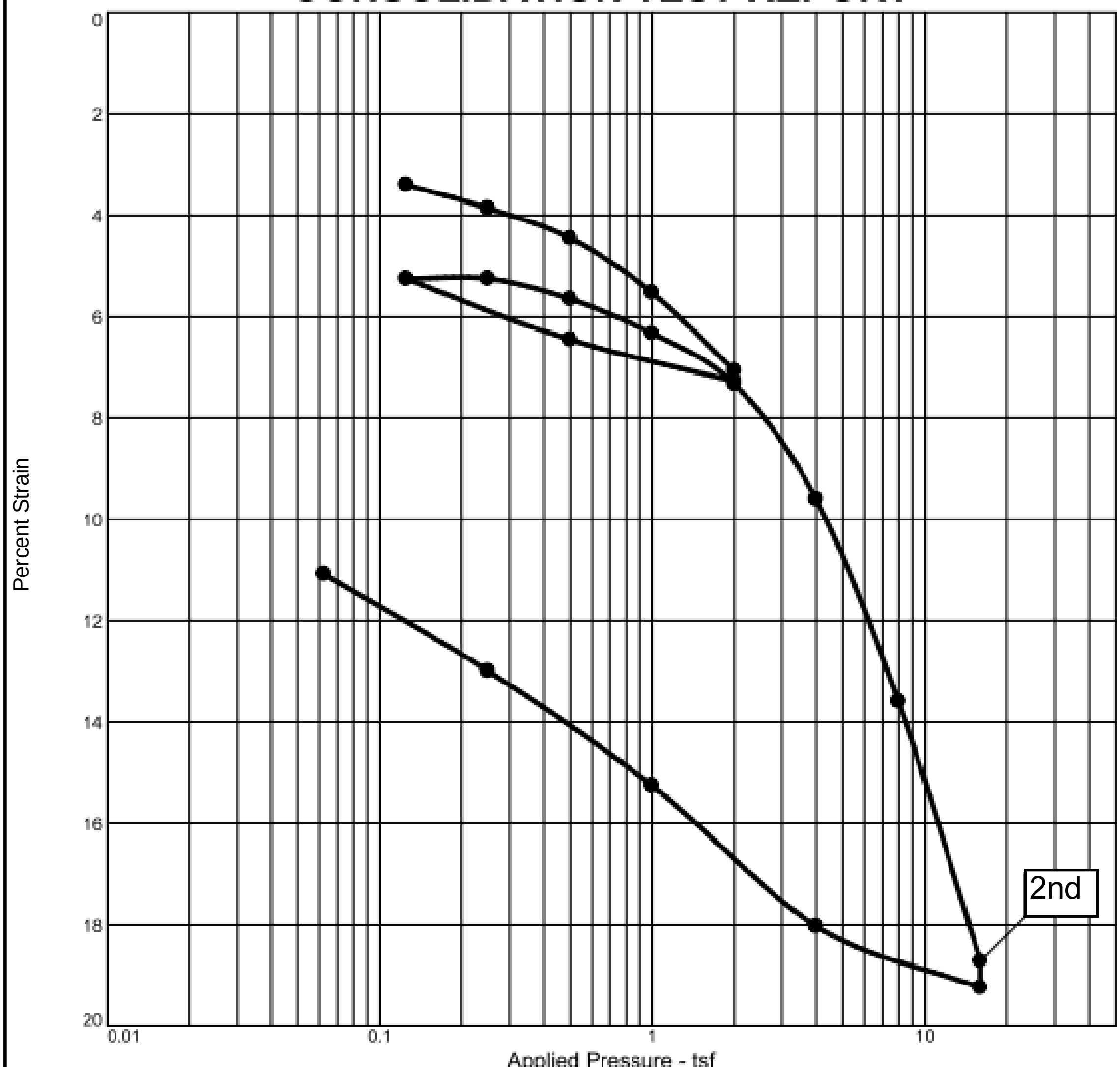
THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 http://www.thielsch.com *Let's Build a Solid Foundation*	Client Information: Simpson Gumpertz & Heger, Inc Waltham, MA PM: Steve Keppel Assigned By: Steve Keppel Collected By: SK and ZB	Project Information: Ipswitch- Supplemental Limited Subsurface Investigation Ipswitch, MA SGH Project Number: 160630.01 Summary Page: 3 of 3 Report Date: 06.14.18

LABORATORY TESTING DATA SHEET

				Identification Tests								Shear / Consolidation Tests								
Boring ID	Sample No.	Depth (ft)	Laboratory No.	Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Torvane or Type Test	$\bar{\sigma}_c$ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	EST. Internal Friction Angle	CR / RR	Laboratory Log and Soil Description
SGH-2018-2A	US-3	13.5-15.5	18-T-736		Average Total Unit Weight (13.5-15.5') = 117.1 pcf															Grey Varved Soil
		13.6-13.7		33.0									Tv = 0.40 tsf							Grading from lean clay to silty sand. Varves vary in thickness from 0.13" to 0.5"
		13.7-14.2		(SAVED)																Clay layers vary in thickness from 0.13" to over 1"
		14.5-14.7		32.3								85.8	Cons						0.19/ 0.026	Gray lean clay; Medium to stiff consistency
		14.7-15.2		(SAVED)																
		15.2-15.3		30.7									Tv = 0.55 tsf							
		15.3-15.5																		Light brown silt

Reviewed By ____________ Date Reviewed *06.15.2018*

CONSOLIDATION TEST REPORT



MATERIAL DESCRIPTION	USCS	AASHTO
Grey lean clay		

LL	PI	Sp. Gr.	Overburden (tsf)	Dry Dens. (pcf)		Moisture		Saturation		Void Ratio		P_c (tsf)	C_c
				Init.	Final	Init.	Final	Init.	Final	Init.	Final		
		2.6		85.5	96.4	32.2 %	35.6	93.4 %	100.0	0.897	0.687	4.3	0.36

Preparation Process: Trimmed using cutting ring

Condition of Test: Saturated at 2 tsf

D2435 Method	C_r	Swell Press. (tsf)	%
B	0.05		

Project No. 160630.01 **Client:** Simpson Gumpertz & Heger

Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street

Source: SGH-2018-2A **Depth:** 13.5-15.5' **Sample No.:** US-3

Thielsch Engineering Inc.

Cranston, RI

Remarks:
End of Primary Test at 14.25-14.35'.
Assumed specific gravity to be 2.6.

Checked By: sa

Title: Laboratory Manager

Figure C-736-1

Tested By: RR

CONSOLIDATION TEST DATA

6/14/2018

Client: Simpson Gumpertz & Heger
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Ipswich, MA
Project Number: 160630.01
Location: SGH-2018-2A
Depth: 13.5-15.5' **Sample Number:** US-3
Material Description: Grey lean clay
Preparation Process: Trimmed using cutting ring
Condition of Test: Saturated at 2 tsf
Test Method: B **Final Density:** 96.4 **Figure No.:** C-736-1
Testing Remarks: End of Primary Test specimen taken at 14.25-14.35'. Assumed specific gravity to be 2.6.
Tested By: RR **Checked by:** sa **Title:** Laboratory Manager

Test Specimen Data

NATURAL MOISTURE		VOID RATIO		AFTER TEST	
Wet w+t =	251.83 g.	**Spec. Gr.**	= 2.6	**Wet w+t =**	192.42 g.
Dry w+t =	202.65 g.	**Est. Ht. Solids**	= 0.527 in.	**Dry w+t =**	154.97 g.
Tare Wt. =	50.08 g.	**Init. V.R.**	= 0.897	**Tare Wt. =**	49.73 g.
Moisture =	32.2 %	**Init. Sat.**	= 93.4 %	**Moisture =**	35.6 %
UNIT WEIGHT		**TEST START**		**Dry Wt. =**	105.24 g.
Height =	1.000 in.	**Height**	= 1.000 in.		
Diameter =	2.500 in.	**Diameter**	= 2.500 in.		
Weight =	145.76 g.				
Dry Dens. =	85.5 pcf				

End-Of-Load Summary

Pressure (tsf)	Final Dial (in.)	Deformation (in.)	C_v ($cm.^2$/sec.)	C_α	Void Ratio	% Strain
start	0.00287	0.00000			0.897	
0.13	0.03686	0.03399	0.0285		0.833	3.4 Comprs.
0.25	0.04150	0.03863	0.0174		0.824	3.9 Comprs.
0.50	0.04746	0.04459	0.0103		0.813	4.5 Comprs.
1.00	0.05816	0.05529	0.0106		0.792	5.5 Comprs.
2.00	0.07562	0.07275	0.0070		0.759	7.3 Comprs.
0.50	0.06743	0.06456			0.775	6.5 Comprs.
0.13	0.05545	0.05258			0.798	5.3 Comprs.
0.25	0.05541	0.05254	0.0825		0.798	5.3 Comprs.
0.50	0.05940	0.05653	0.0337		0.790	5.7 Comprs.
1.00	0.06618	0.06331	0.0348		0.777	6.3 Comprs.
2.00	0.07626	0.07339	0.0101		0.758	7.3 Comprs.
4.00	0.09894	0.09607	0.0063		0.715	9.6 Comprs.
8.00	0.13885	0.13598	0.0046		0.639	13.6 Comprs.
16.00	0.19532	0.19245	0.0026		0.532	19.2 Comprs.
4.00	0.18312	0.18025			0.555	18.0 Comprs.
1.00	0.15543	0.15256			0.608	15.3 Comprs.
0.25	0.13282	0.12995			0.651	13.0 Comprs.
0.06	0.11371	0.11084			0.687	11.1 Comprs.

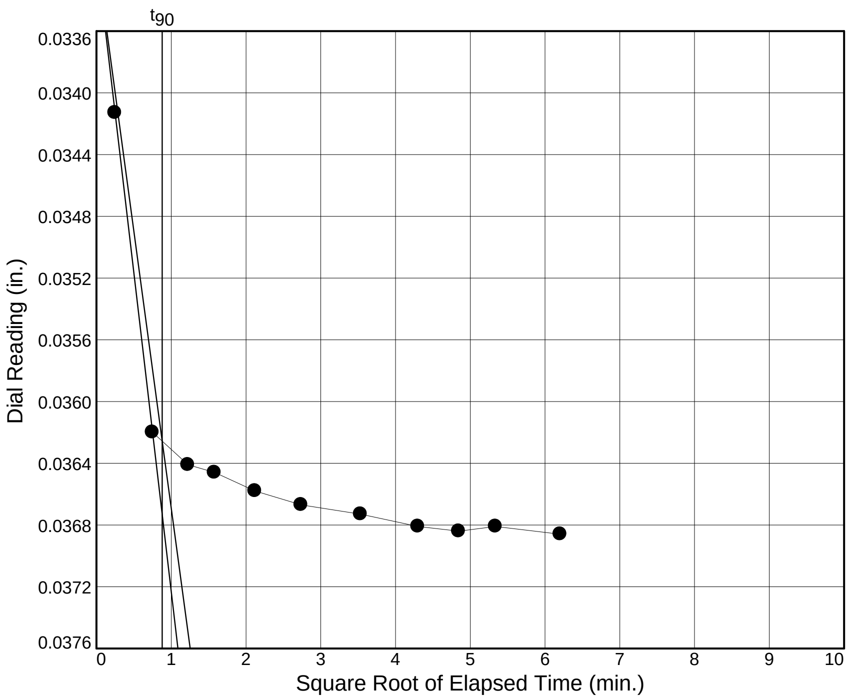
TEST RESULTS SUMMARY

Compression index (C_c), tsf = 0.36 **Preconsolidation pressure (P_p), tsf =** 4.3 **Void ratio at P_p (e_m) =** 0.709

Recompression index (C_r) = 0.05

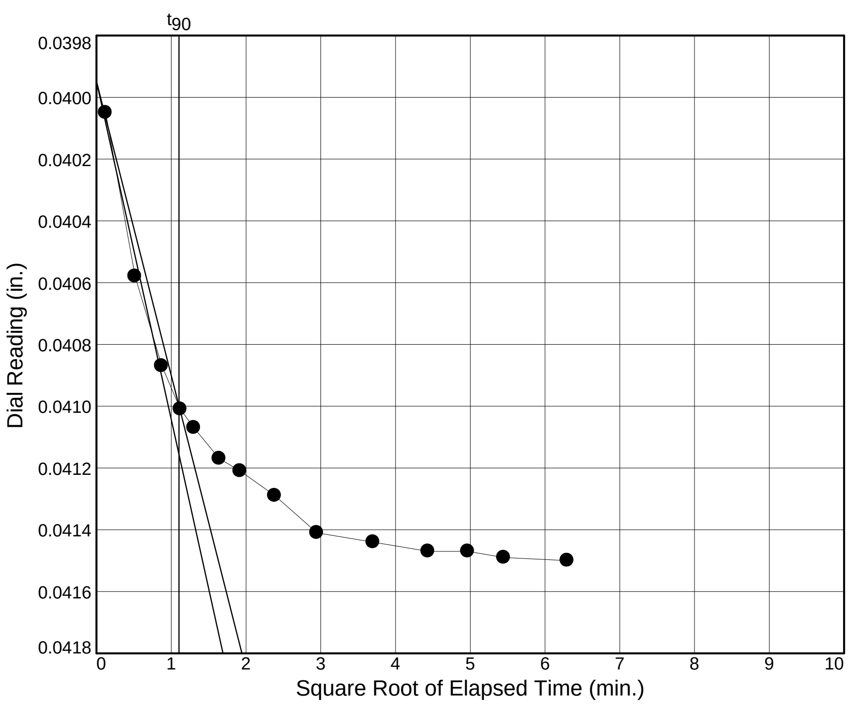
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 13.5-15.5'   Sample Number: US-3



Load No.= 1
Load= 0.13 tsf
D_0 = 0.0331
D_{90} = 0.0363
D_{100} = 0.0366
T_{90} = 0.77 min.

C_v @ T_{90}
0.0285 cm.^{2}/sec.



Load No.= 2
Load= 0.25 tsf
D_0 = 0.0399
D_{90} = 0.0410
D_{100} = 0.0411
T_{90} = 1.22 min.

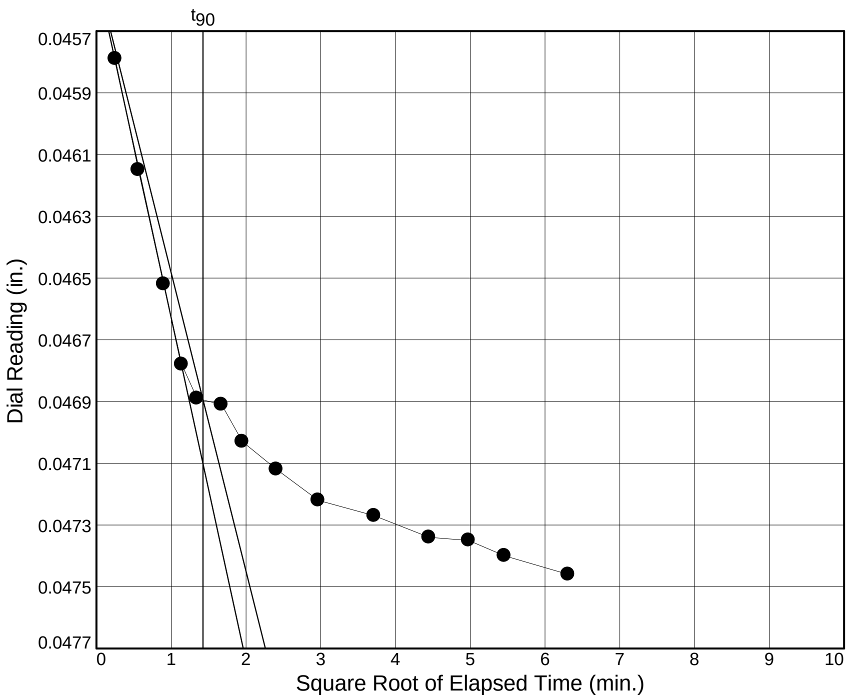
C_v @ T_{90}
0.0174 cm.^{2}/sec.

Figure C-736-2

Thielsch Engineering Inc.

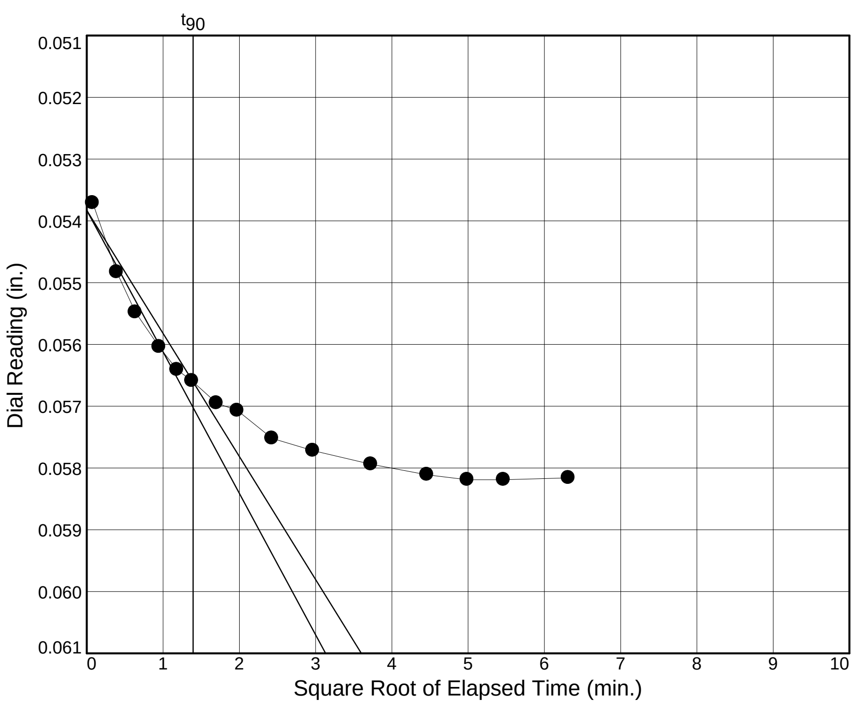
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 13.5-15.5'   Sample Number: US-3



Load No.= 3
Load= 0.50 tsf
D_0 = 0.0455
D_{90} = 0.0469
D_{100} = 0.0470
T_{90} = 2.03 min.

C_v @ T_{90}
0.0103 cm.^{2}/sec.



Load No.= 4
Load= 1.00 tsf
D_0 = 0.0538
D_{90} = 0.0566
D_{100} = 0.0569
T_{90} = 1.94 min.

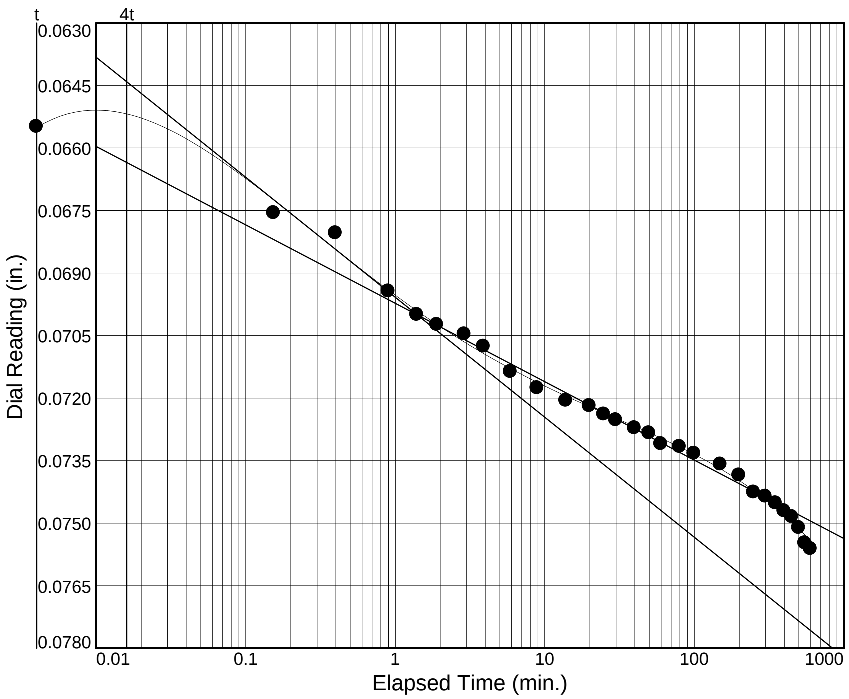
C_v @ T_{90}
0.0106 cm.^{2}/sec.

Figure C-736-3

Thielsch Engineering Inc.

Dial Reading vs. Time

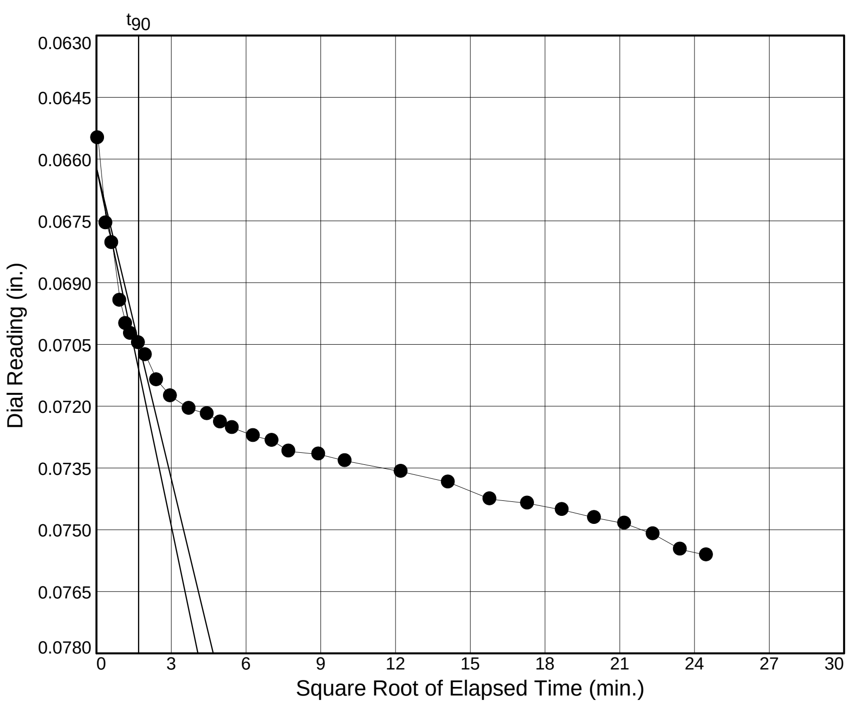
Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 5
Load= 2.00 tsf
D_0 = 0.0658
D_{50} = 0.0679
D_{100} = 0.0700
T_{50} = 0.26 min.

C_v @ T_{50}
0.0175 cm.^{2}/sec.

C_α = 0.002



Load No.= 5
Load= 2.00 tsf
D_0 = 0.0662
D_{90} = 0.0705
D_{100} = 0.0709
T_{90} = 2.85 min.

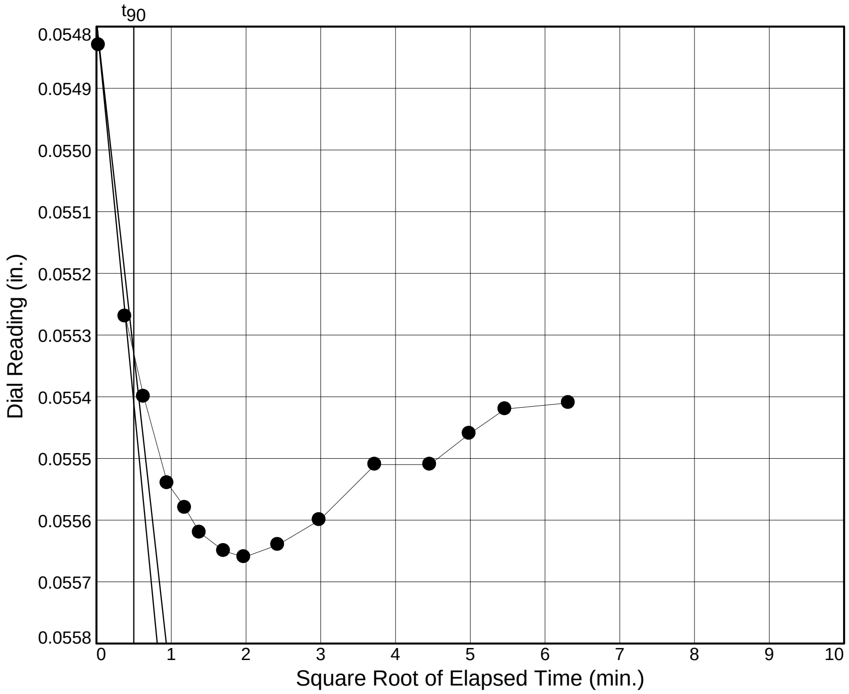
C_v @ T_{90}
0.0070 cm.^{2}/sec.

Figure C-736-4

Thielsch Engineering Inc.

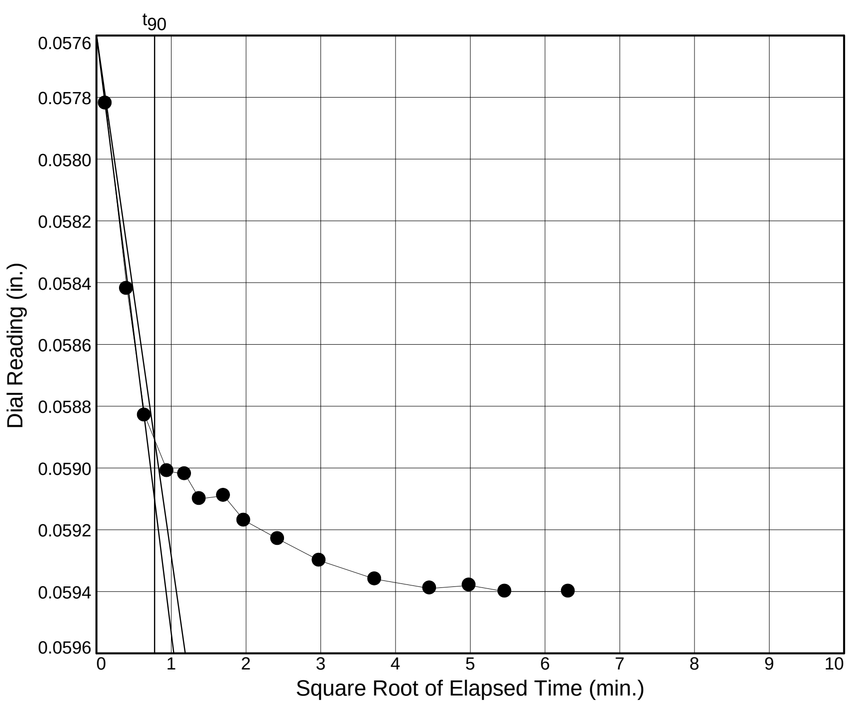
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 8
Load= 0.25 tsf
D_0 = 0.0548
D_{90} = 0.0553
D_{100} = 0.0554
T_{90} = 0.25 min.

C_v @ T_{90}
0.0825 cm.^{2}/sec.



Load No.= 9
Load= 0.50 tsf
D_0 = 0.0576
D_{90} = 0.0589
D_{100} = 0.0591
T_{90} = 0.60 min.

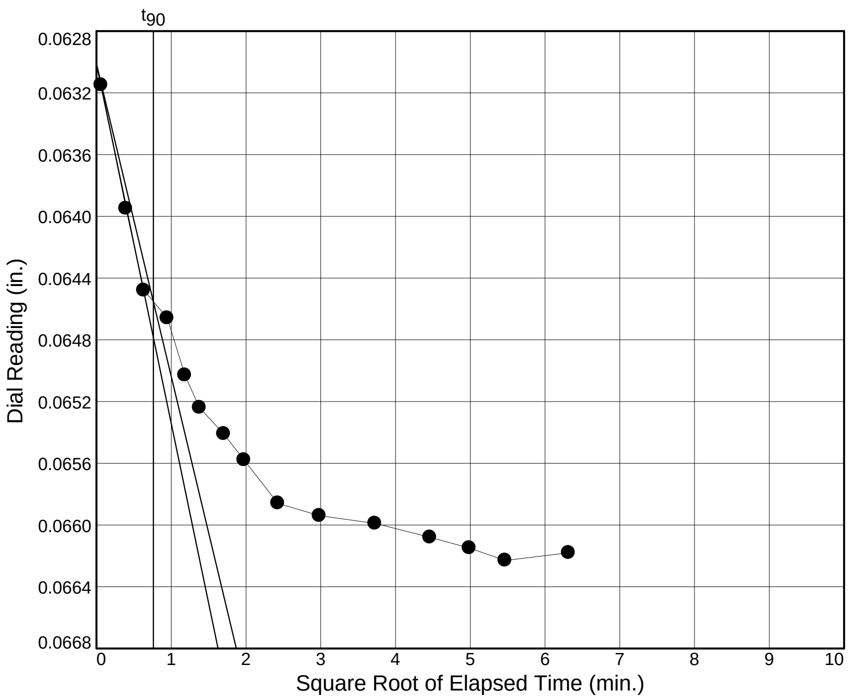
C_v @ T_{90}
0.0337 cm.^{2}/sec.

Figure C-736-5

Thielsch Engineering Inc.

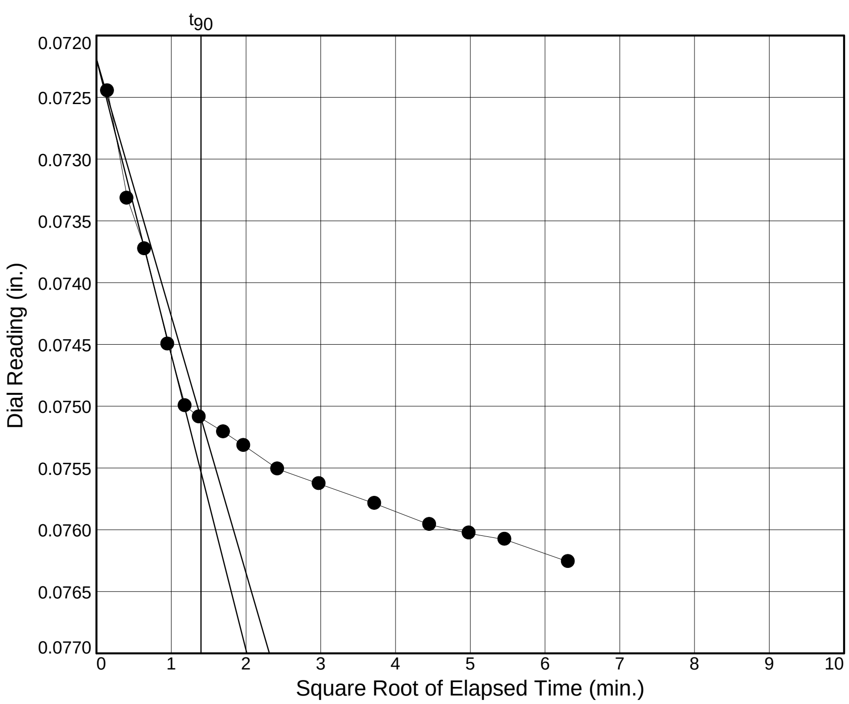
Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 10
Load= 1.00 tsf
D_0 = 0.0630
D_{90} = 0.0646
D_{100} = 0.0647
T_{90} = 0.58 min.

C_V @ T_{90}
0.0348 cm.^{2}/sec.



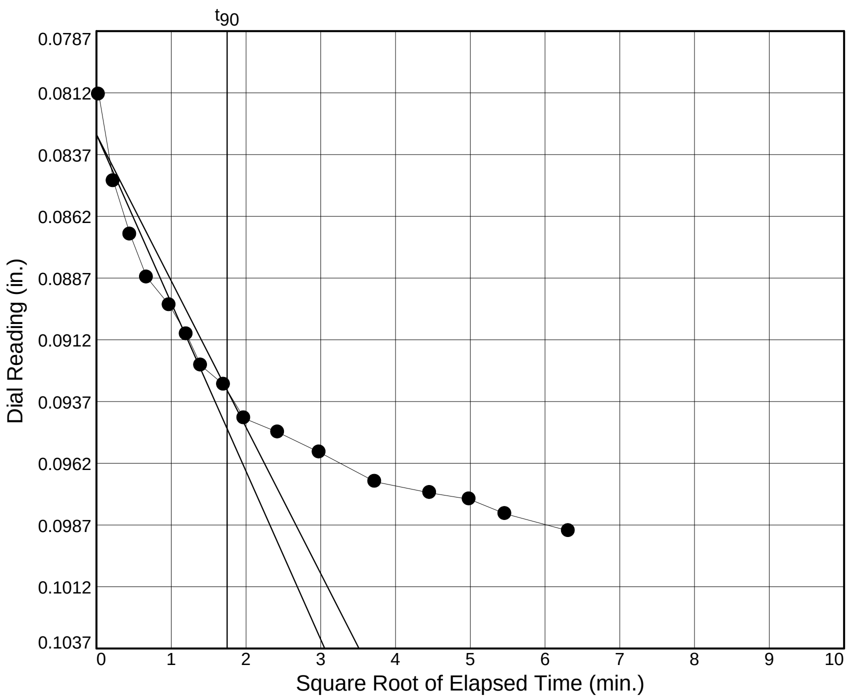
Load No.= 11
Load= 2.00 tsf
D_0 = 0.0722
D_{90} = 0.0751
D_{100} = 0.0754
T_{90} = 1.95 min.

C_V @ T_{90}
0.0101 cm.^{2}/sec.

Figure C-736-6

Dial Reading vs. Time

Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A Depth: 13.5-15.5' Sample Number: US-3



Load No.= 12

Load= 4.00 tsf

D_0 = 0.0829

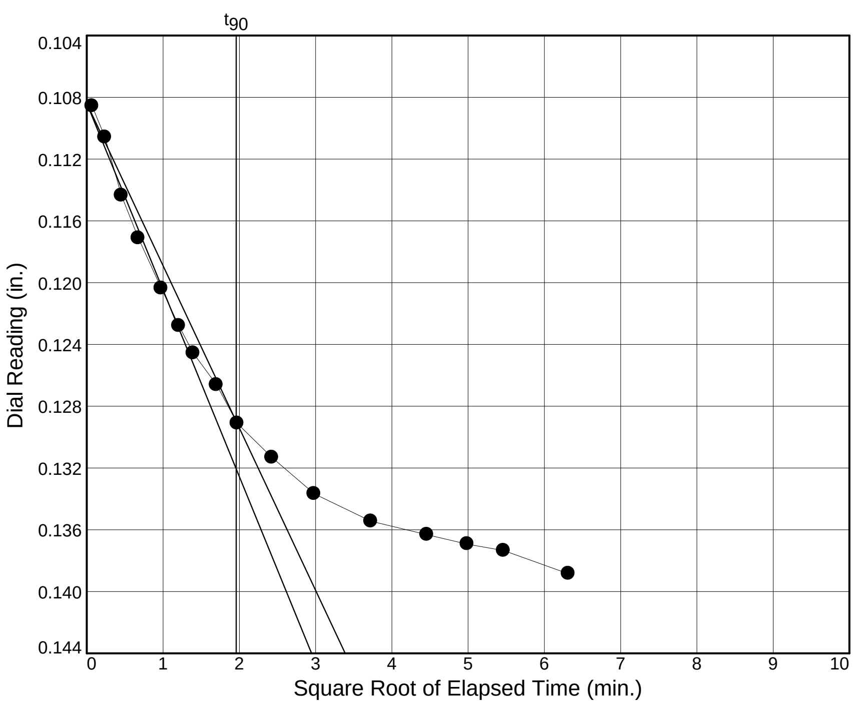
D_{90} = 0.0932

D_{100} = 0.0944

T_{90} = 3.05 min.

C_v @ T_{90}

0.0063 cm.^{2}/sec.



Load No.= 13

Load= 8.00 tsf

D_0 = 0.1084

D_{90} = 0.1290

D_{100} = 0.1313

T_{90} = 3.84 min.

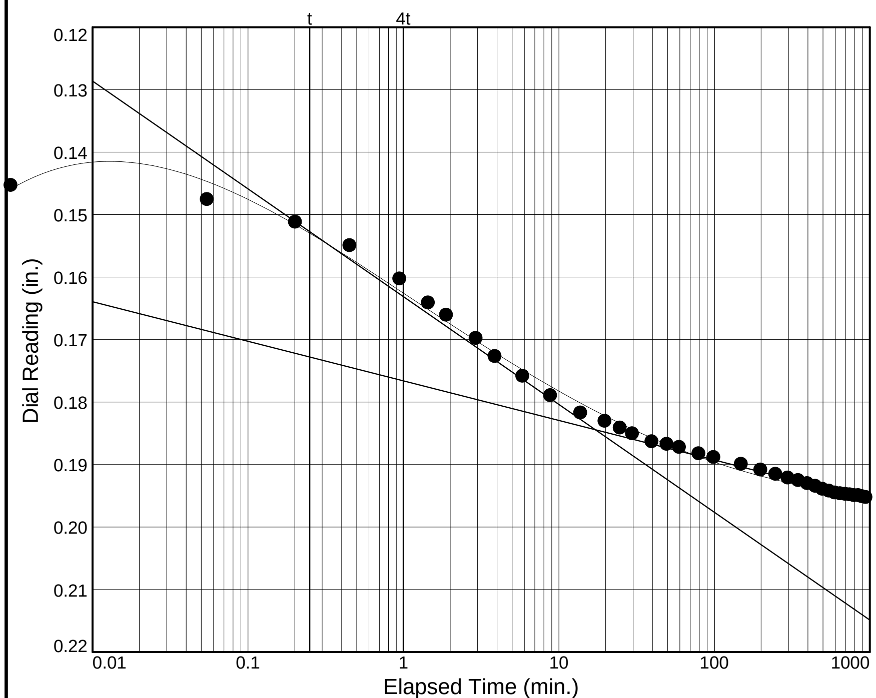
C_v @ T_{90}

0.0046 cm.^{2}/sec.

Figure C-736-7

Dial Reading vs. Time

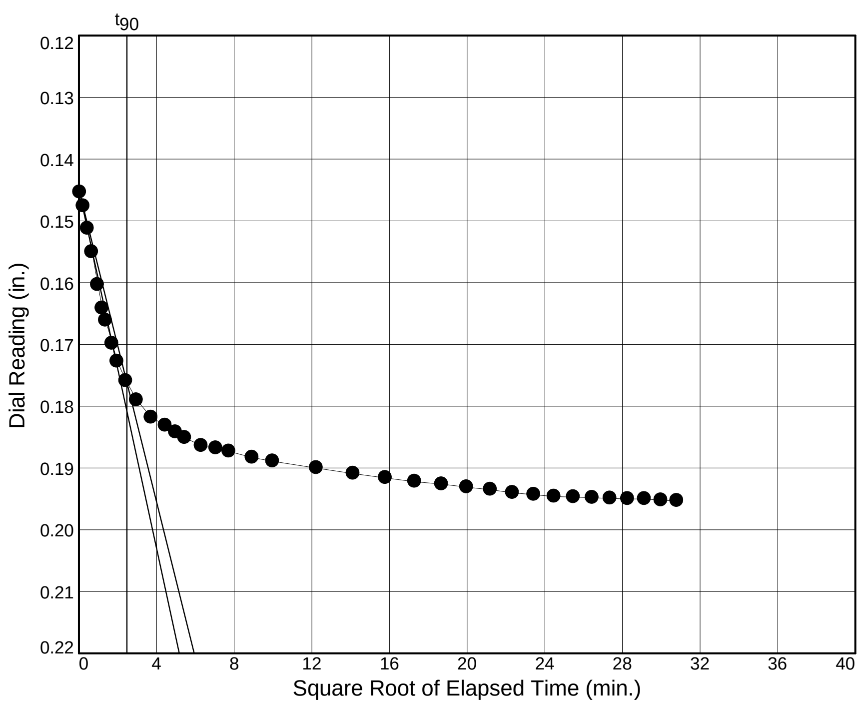
Project No.: 160630.01
Project: Ipswitch - Suppplemental Limitied Subsurface Investigation
10 Estes Street
Source of Sample: SGH-2018-2A   Depth: 13.5-15.5'   Sample Number: US-3



Load No.= 14
Load= 16.00 tsf
D_0 = 0.1434
D_{50} = 0.1639
D_{100} = 0.1844
T_{50} = 1.21 min.

C_v @ T_{50}
0.0031 cm.^{2}/sec.

C_α = 0.006



Load No.= 14
Load= 16.00 tsf
D_0 = 0.1449
D_{90} = 0.1761
D_{100} = 0.1796
T_{90} = 6.10 min.

C_v @ T_{90}
0.0026 cm.^{2}/sec.

Figure C-736-8

Thielsch Engineering Inc.

ATTACHMENT 7

Horsley Witten Group
Sustainable Environmental Solutions
90 Route 6A • Unit 1 • Sandwich, MA 02563
508-833-6600 • horsleywitten.com

MEMORANDUM

To: Kristopher Houle, DER; Wayne Castonguay, IRWA

Cc: Ethan Parsons, Town of Ipswich

From: Neal Price

Date: December 4, 2018; Revised January 25, 2019

Re: Ipswich Mills Dam Removal Feasibility Study – Task 4 (Potential Infrastructure Impacts) Summary

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1.0 Introduction

The Horsley Witten Group, Inc. (HW) is pleased to submit to the Massachusetts Division of Ecological Restoration (DER) and the Ipswich River Watershed Association (IRWA) the following memorandum summarizing Task 4 work completed as part of the Ipswich Mills Dam Removal Feasibility Project (the Project), located in Ipswich, Massachusetts approximately 700-feet south (upstream) of the Route 133/South Main Street/Choate Bridge crossing (Figure 1). Task 4 is an evaluation of the potential impacts from dam removal on other structures besides the EBSCO publishing facility, located immediately adjacent and upstream of the dam on the left bank of the river heading downstream (river left). An assessment of potential impacts to the EBSCO facility was previously submitted by Simpson, Gumpertz and Heger, Inc. (SGH) as a Task 3 summary memorandum on February 20, 2018, and supplemented by a follow-up memorandum on June 29, 2018.

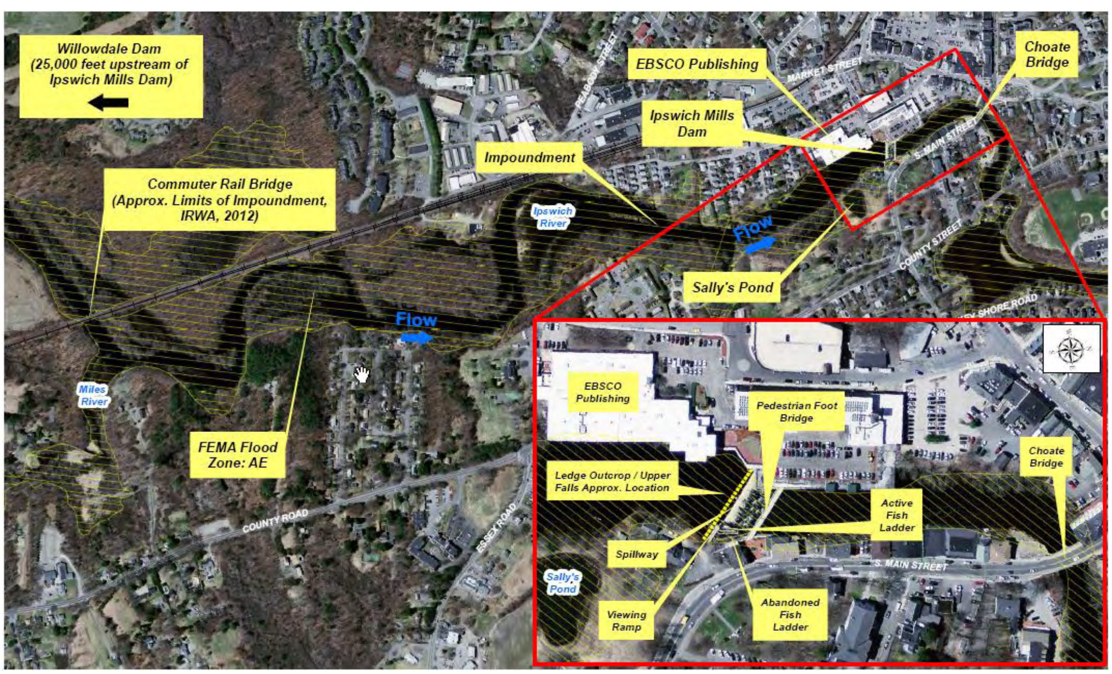


Figure 1. Key Project Area Features

This memorandum builds off information discussed in prior task summary memorandums. Please refer to the Task1 summary memorandum for more detail about the project and site background, and the Task 2 summary memorandum for more detail about river hydrology and hydraulics. As mentioned above, the EBSCO facility is discussed in the Task 3 summary memorandum. Of particular interest to this Task 4 memorandum is the potential for bedrock located at the dam site. As more fully discussed in the Task 1 memorandum, the elevation of competent bedrock ledge at the dam site is not yet accurately known. Therefore, the hydraulic modeling conducted for this project (refer to Task 2 summary memorandum) takes a conservative approach by assuming that bedrock is not present higher than the observed river bottom elevation upstream and downstream of the dam. This assumption leads to the conservative prediction of faster and more erosive river flows that would tend to cause more sediment migration and greater impacts to adjacent structures than would be anticipated if there were competent bedrock at a higher elevation than assumed to date for this project.

This Task 4 assessment of potential impacts to other infrastructure besides the EBSCO facility was intended as a high-level, preliminary evaluation based on visual observation and a comparison to modeled dam-out river flow conditions, as presented in the previously submitted Task 2 Hydrologic and Hydraulic (H&H) Analysis memorandum. The H&H memorandum describes modeling conducted using the U.S. Army Corps of Engineers (USACE) Hydraulic Engineering Centering River Analysis Systems (HEC-RAS) software. HEC-RAS model simulations were run for existing and dam-out conditions under high and low tide and various river flow scenarios, including 2-year storm, 10-year storm, 25-year storm, 50-year storm, 100-year storm, 500-year storm, 5% exceedance, 50% exceedance, and 95% exceedance.

No subsurface investigations or structural analyses for specific infrastructure items were conducted as part of this Task 4 assessment. However, the analyses described in this memorandum were informed by prior work including an initial dam removal feasibility study completed in 2014, the three prior tasks of this current feasibility study, field observations, and the Town's evaluations of protection strategies for sewer line infrastructure located within the river's channel. In this memorandum, where applicable, observations of structures deemed to be at potential risk for negative impact from dam removal are noted and recommendations are made for further, more detailed evaluation, and/or for mitigation options that could be considered. Please note that the assessments conducted under this task focused on structures that could potentially be negatively impacted by dam removal. Structures that are simply in currently poor or deteriorating condition, but are not anticipated to have their condition further negatively impacted as a result of dam removal, are mentioned, but no mitigation or further study options are presented for them.

The following process was followed to conduct the Task 4 assessment discussed in this memorandum:

1. Review aerial photography to identify potential structures in the project vicinity to evaluate;
2. Discuss with IRWA and other Tech Team members potential structures to evaluate to take advantage of its in-depth, local knowledge;

3. Field observe the stretch of river from the railroad bridge down to the lower falls (downstream from the County Road Bridge) from either the water side, the land side, or both to further vet those potential structures identified in steps 1 and 2, and to search for additional structures with potential to be impacted;
4. Visit structures with potential to be impacted to visually observe and photograph their conditions;
5. Compare the locations of identified structures with modeled changes in river level, velocity and erosive shear stress under dam-out conditions to evaluate if potential hydraulic changes might impact those structures; and
6. Make recommendations to protect potentially at-risk structures and/or mitigate against potential damages.

Figures 2A (downstream) and 2B (upstream) depict the locations of the structures evaluated and discussed in this memorandum. River stationing for the hydraulic model and the design plans is also shown on these figures. Stationing begins with zero at the Green Street Bridge (approximately 1,400 feet downstream below the lower falls) and runs approximately 2.25 miles upstream to the most upstream H&H model transect above the railroad bridge at Station 118+10. It would be expected that hydraulic impacts from potential dam removal would extend further upstream (into the currently impounded area) than downstream, and the Task 2 hydraulic modeling bears that out. Potential hydraulic impacts from dam removal are modeled to dissipate rapidly downstream of the dam under all modeled scenarios. The modeled extent of significant impact downstream of the dam for either river stage or flow velocity is approximately 100 feet (Station 2,934), shortly downstream of the pedestrian bridge.

In contrast, some potential hydraulic impacts under at least some flow scenarios are modeled to extend more than a mile and a half upstream, at least 1,000 feet upstream of the railroad bridge at the upper limits of currently impounded conditions. Water levels at the most upstream model transect (Station 118+10) are predicted to drop by approximately 0.9 feet under 50% exceedance flow conditions. No water level changes at this most upstream transect are modeled to occur for any other flow scenarios, and no velocity changes are modeled to occur for any scenarios. More significant water level and velocity changes are modeled to occur beginning just below the railroad bridge under all modeled flow scenarios. The river bed beneath the railroad bridge has piled up rocks for scour protection of the bridges support piers that create a small hydraulic drop that appears to prevent significant hydraulic impacts from extending much further upstream. H&H model results by river stationing location for all modeled flow scenarios are included herein as Appendix A.

Based on steps one through four of the above process, 21 structures (or groups of structures along a contiguous river stretch) were identified for comparison to hydraulic modeling results for further evaluation. Seven of those structures are downstream of the dam and 14 are upstream. Those structures, or groups of structures, are shown on Figure 2, and listed in Tables 2 and 3, below.

Outside of the longer-term potential impact assessment discussed in this memorandum, dam removal would also precipitate short term changes in sediment mobilization as the river reacts to the removal of the flow restriction currently created by the dam and seeks to obtain a new equilibrium of sediment dynamics in line with the new flow regime. Currently the dam tends to retain behind it some of the coarse sediment migrating downstream, resulting in downstream areas that are sediment deprived. During this transitional period, softer/more mobile sediments currently retained behind the dam will migrate downstream, begin to fill in voids in currently sediment deprived locations, and continue to migrate downstream until they are deposited in locations where the flow energy regime is supportive of deposition. This process will continue over the transitional period until the river's sediment dynamics approach equilibrium with the post-dam flow energy dynamics.

2.0 Downstream Structures

Despite the fact that hydraulic impacts from potential dam removal are modeled to dissipate within approximately 100 feet downstream of the dam, in this memorandum potential impacts to structures are evaluated and discussed down to below the Choate Bridge, approximately 1,000 feet downstream. Discussion is extended over this longer downstream area due to the high density of infrastructure in the area (such as the Town's main sewer interceptor and siphon) and the historic significance of the Choate Bridge and other structures. Table 1 lists the downstream structures evaluated for potential impacts from dam removal, their likelihood of potential impact, and whether or not mitigation is proposed. All distances are approximate river channel distances and left and right directions are relative to downstream river flow.

Table 1. Downstream Structures Evaluated for Potential Impacts

ID	Description	~ Feet from Dam	Nearest Model Station	Potential Impact	Further Action
DS-1	Old Fishway Wall	0-50 Right	3,020 & 3,041	Moderate	Reinforcement
DS-2	Pedestrian Platform Piers	0-300 Left	3,020 & 3,041	Moderate	Reinforcement
DS-3	Pedestrian Bridge	60	2,998	No	No
DS-4	Building Foundations/ Walls Downstream of Pedestrian Bridge	90-700 Right	2,387; 2,522; 2,701; 2,717; & 2,934	No	Monitoring
DS-5	Farley Brook Outfall River Left	440	2,522 & 2,701	No	No
DS-6	Sewer Interceptor and Siphon	450-1,000	2,387	No	No
DS-7	Choate Bridge	750	2,306	No	Monitoring

2.1 Old Fishway and Wall River Right (DS-1):

In-use and Abandoned Fishways Views Downstream (left) and Upstream (right)

Two Fishladders currently exist shortly downstream from the dam on river right. The one located closer to the river channel center is newer and still in use. The one flush with the retaining wall at the river's right edge has been abandoned since the newer one was built in 1996 (Haley and Aldrich, 2009). Under the proposed draft Task 2 Conceptual Design for dam removal the newer fish ladder will be removed. The outside edge of the older fish ladder is integral with the retaining wall on the river right edge of the river and so cannot be fully removed.

H&H modeling indicates that water levels in the vicinity of the fish ladder will stay largely similar for the larger flow scenarios and drop by as much as 1.5 feet for the 95% exceedance/ low tide flow scenario. River flow velocities are simulated to either drop, or marginally increase by approximately 0.2 feet per second (fps), depending upon the flow scenario. Given that both water levels and river flow velocities along this stretch of river are generally anticipated to stay similar or drop under modeled dam-out conditions, no significant impact to the old fish ladder and its adjacent wall is anticipated as a result of dam-removal. Nevertheless, because the exact alignment of the river channel in this area under a dam removal scenario for all flow conditions is uncertain, because of the importance of the retaining wall adjacent to the old fish ladder, and because of the fact that the fish ladder will be in the heart of the construction zone if the dam is removed so that mitigation work can be accomplished relatively easily, the draft design proposes to fill the void space in the old fish ladder with large rocks salvaged from the dam removal area to protect the adjacent retaining wall. Concrete filling may also be considered for the old fish ladder in subsequent design phases. The filled and reinforced older fish ladder will serve as mitigative protection for the adjacent retaining wall in the event more erosive river flows are ultimately experienced at this location following potential dam removal, despite current modeling indications to the contrary.

2.2 Pedestrian Platform Piers River Left (DS-2):

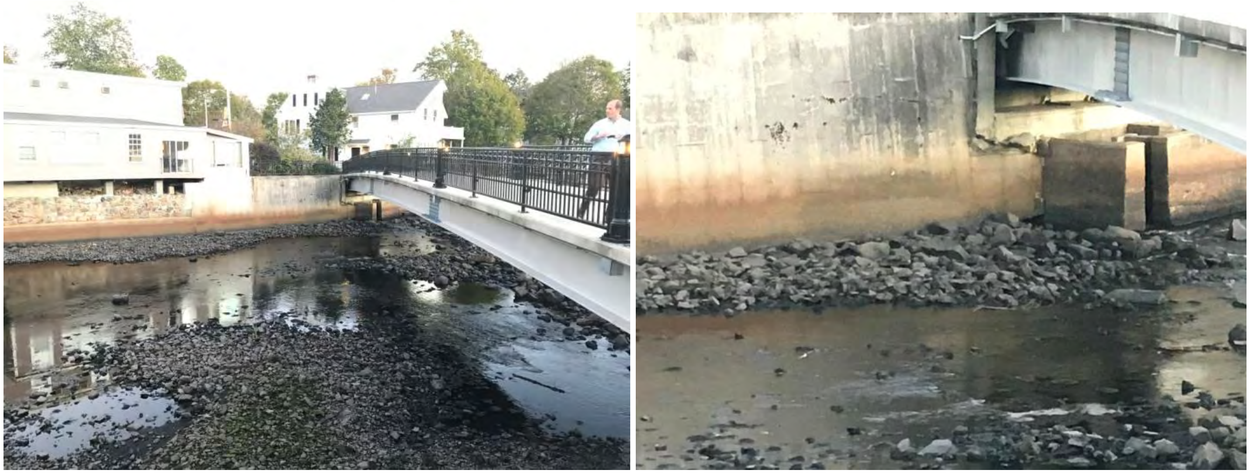
Pedestrian Platform Support Piers Far-field (left), Close-up (right), and Underneath (bottom)

A concrete pedestrian platform supported on stone block piers is located immediately downstream of the dam on river left. The platform forms the left end of the pedestrian bridge that spans the river shortly downstream from the dam and provides a means for pedestrians to access either the EBSCO facility or a parking lot further downstream on the left river bank as part of the Town's Riverwalk.

H&H modeling indicates that water levels in the vicinity of the pedestrian platform will remain similar to current conditions under dam-out conditions. River flow velocities are simulated to either drop, or increase slightly by approximately 0.2 fps, depending upon the flow scenario. Given that both water levels and river flow velocities along this stretch of river are generally anticipated to drop under modeled dam-out conditions, no significant impact to the pedestrian platform is anticipated as a result of dam-removal.

Nevertheless, because the exact alignment of the river channel in this area under a dam removal scenario for all flow conditions is uncertain, because of the importance of the pedestrian platform, and because of the fact that the platform will be in the heart of the construction zone if the dam is removed so that mitigation work can be accomplished relatively easily, the draft design proposes to place salvaged large rocks as scour protection around and in front of the support piers for the pedestrian platform. The reinforcement will serve as mitigative protection for the platform in the event more erosive river flows are ultimately experienced at this location following potential dam removal, despite current modeling indications to the contrary.

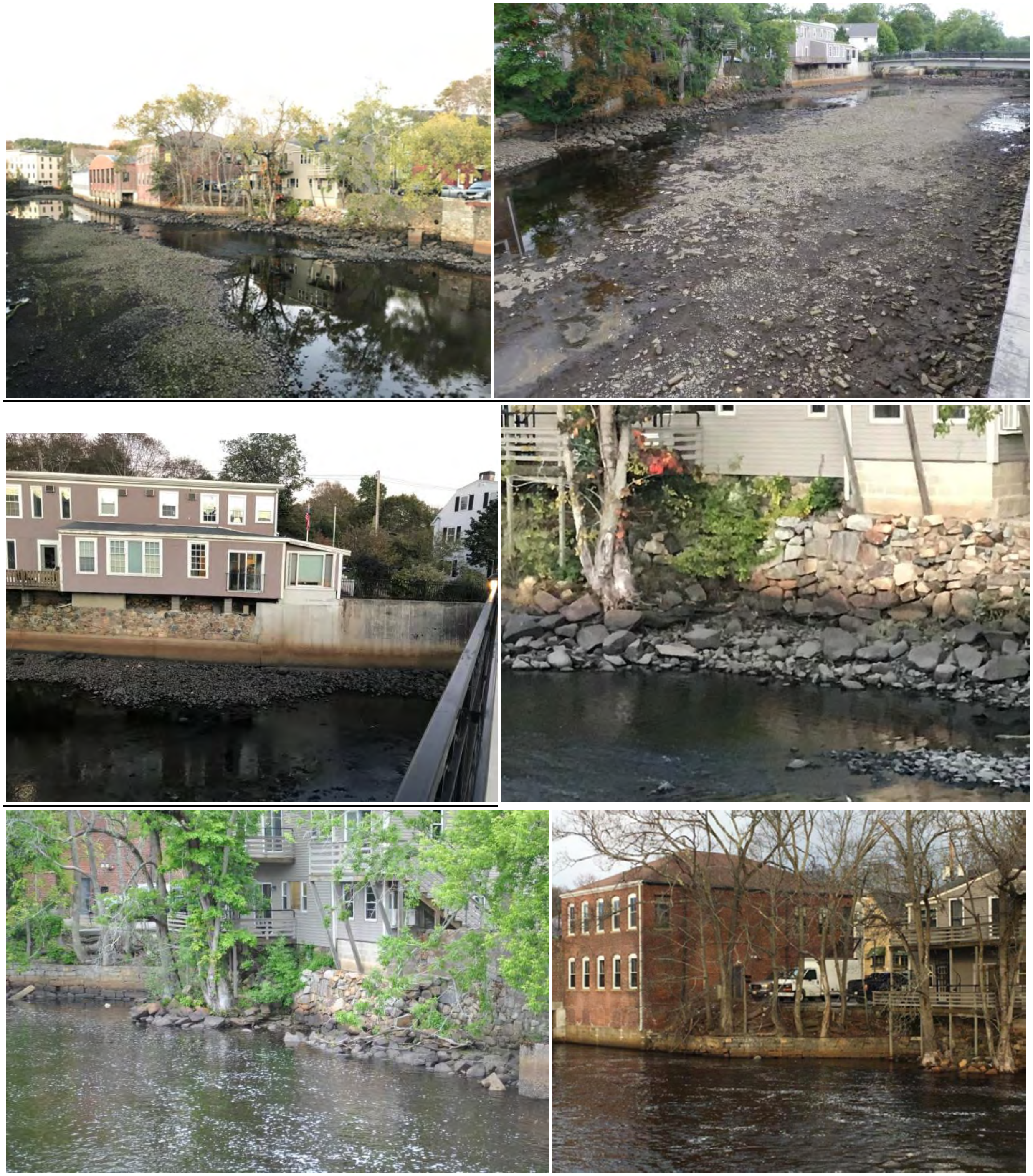
2.3 Pedestrian Bridge (DS-3):



Pedestrian Bridge View to River Right Far-field (left) and Close-up (right)

An I-beam supported, concrete-platform pedestrian bridge spans the river approximately 60 feet downstream from the dam. The bridge was built in approximately 2001 (bridge design plans are dated January 2001) and is supported on the right end by a retaining wall and on the left end by the pedestrian platform. The bridge remains above the river level in all but the largest flood events (e.g. The Mother's Day Storm of 2006). Modeling indicates that water levels will stay largely similar at this location under dam-out conditions. River flow velocities are simulated to either stay the same or decrease under all modeled flow scenarios. Because the bridge is generally above the river level now and would continue to be above the river under dam-out conditions, and because river flow velocities are not simulated to increase, dam removal is not expected to impact this structure.

2.4 Foundations and Walls Below Pedestrian Bridge River Right (DS-4):



Foundations/Walls Below Pedestrian Bridge River Right Far Views Downstream (top left) and Upstream (top right), and Close-up Views Moving Downstream (next three rows)

The right bank of the river from the dam down to the Choate Bridge is lined by either retaining walls or concrete building foundations that act, effectively, as retaining walls. As indicated by the above photographs the walls range from relatively new concrete that visually appear to be in good condition to loose stone that visually appear to be in moderate to somewhat degraded condition. The length of loose stone wall that visually appears to be in the relatively worst condition along the stretch is located from approximately 200–300 feet downstream of the dam in front of a building located at 47-51 South Main Street and a parking lot immediately downstream of that (see photographs above second row right and third row left and right). Another loose stone wall in similar moderate condition is located a little further downstream (approximately 550-600 feet downstream from the dam) in front of 27-37 South Main Street (see photographs above bottom row).

Modeling indicates that water levels and flow velocities along this stretch of the river will be essentially unchanged under dam-out conditions and, therefore, these structures are unlikely to be impacted by dam removal. The overall condition of some sections of these loose stone walls may not be optimal but dam removal is not anticipated to exacerbate hydraulic conditions that may continue to impact those structures over time.

2.5 Farley Brook Outfall River Left (DS-5):

Farley Brook and Stormwater Outfall Entering River Beneath Parking Lot River Left

Farley Brook (culverted along most of its length) joins the Ipswich River via a culvert under the public parking lot river left approximately 440 feet downstream from the dam. A separate stormwater pipe also enters the river at this location at a higher elevation (see photograph above). The stormwater pipe is above all but the highest river stages under either existing or dam-out conditions and will, therefore, not be impacted by dam removal. The Farley Brook culvert is within the range of normal river stage fluctuations under current conditions. Hydraulic modeling indicates no significant changes to river stage or flow velocity at this location under dam-out conditions and, therefore, it too is unlikely to be impacted by dam removal.

2.6 Sewer Interceptor and Siphon (DS-6):

Exposed Sewer Force Main Crossing Beneath Choate Bridge River Left

An exposed sewer Interceptor runs in the river channel along the left bank from the area of the public parking lot, under the Choate Bridge, and then further downstream (approximately 400 – 1,000 feet downstream of the dam total run length). There is also a gravity sewer line “siphon” extending under the river roughly half way in between Choate and County Street Bridges (approximately 1,000 feet downstream of the dam). The interceptor line is supported by concrete piers that rest on the river bed. Both the line and the piers are partially exposed and subject to scour. Due to its importance to the Town, contamination risk to the environment, and exposed condition within the river channel, these sewer lines are vulnerable pieces of infrastructure under existing conditions. We understand that the town has initiated resiliency-related work on the sewer interceptor and siphon and has budgeted to implement its recommendations. Reportedly, the siphon is due to be replaced and then protected with rip rap in 2019, and the currently exposed interceptor is due to be protected with rip rap in 2020. Thus, the potential issues identified in this report relative to the interceptor and siphon should be addressed prior to any potential dam removal timeframe.

With regards to the potential impacts from dam removal, hydraulic modeling indicates no significant changes to river stage or flow velocity at this location under dam-out conditions and, therefore, it is unlikely to be impacted by dam removal. The currently exposed condition of the main interceptor, and to a lesser degree, the siphon, does place them at risk for damage from river debris such as large trees or logs. The existing dam currently catches a percentage of this debris, at least temporarily, that might otherwise continue downstream and risk damaging the exposed pipes. However, according to IRWA, nearly all floating debris currently caught by the dam eventually work free and continues downstream and the Town on occasion removes debris from the dam of particular concern. The Town’s proposed mitigation plan for the sewer lines should protect the pipes from this impact risk so long as dam removal occurs following the completion of that mitigation project.

2.7 Choate Bridge (DS-7):

Choate Bridge Views from Upstream (top and lower right) and from Downstream (lower left)

According to the Town of Ipswich, the Choate Bridge was built in 1764 and is reportedly the oldest, surviving, double-arched, stone bridge in America. It is, therefore, of immense historical significance. Since it carries South Main Street into downtown Ipswich, it is also of high importance as a piece of transportation infrastructure for the region. Hydraulic modeling indicates no significant changes to river stage or flow velocity at this location under dam-out conditions and, therefore, the bridge is unlikely to be impacted by dam removal. However, since the U.S. Army Corps of Engineers (USACE) and Federal Emergency Management Agency (FEMA) may require a more detailed flood analysis during permitting, and since the bridge holds such practical and historical significance for the Town, flood elevations should be further evaluated during future design stages.

While no significant changes in river stage or velocity are modeled to occur as a result of potential dam removal, the bridge is modeled to be a flow restriction during larger flow events under current conditions. Under a potential dam-out scenario sediment currently retained behind the dam will migrate downstream. Model results indicate that bed shear stresses beneath the bridge will be sufficient to transport sediment sizes up to cobbles, so no long-term sedimentation impacts at the bridge are anticipated. However, it's possible that some sediment may be temporarily retained beneath the bridge, depending upon the flow conditions under which it is mobilized, until it is remobilized and transported past the bridge during subsequent high flow events. Monitoring of sediment transport past the bridge is recommended following potential dam removal.

3.0 Upstream Structures

Due to the relatively long upstream extent of potential hydraulic impact from dam removal, potential impacts to structures are evaluated and discussed up to and including the railroad bridge, approximately 7,500 feet upstream. While modeled hydraulic impacts from dam removal extend further upstream of the railroad bridge under some scenarios, this upstream stretch of the river is undeveloped along its banks and no potentially-impacted infrastructure exists upstream of the railroad bridge for over a mile. Table 2 lists the upstream structures evaluated for potential impacts from dam removal, their likelihood of potential impact, and if mitigation is proposed. All distances are approximate river channel distances and left and right directions are relative to downstream river flow.

Table 2. Upstream Structures Evaluated for Potential Impacts

ID	Description	~ Feet from Dam	Nearest Model Station	Potential Impact	Further Action
US-1	Retaining Wall River Right	0-150 Right	3,072	Low	Reinforcement
US-2	Retaining Wall River Left	0-100 Left	3,072	Low	Reinforcement
US-3	EBSCO Foundation	100-440 Left	3,260	Low	Reinforcement
US-4	Sally's Pond Outfall	250-450*	3,496	Unknown	Monitoring
US-5	Sally's Pond Canoe Launch	300	3,496	Low	Not Needed Post Dam Removal
US-6	Peatsfield St. Canoe Launch	920 Left	3,900	Low	Adaptive Management
US-7	Saltonstall Brook	1,200 Right	3,900 & 5,359	Low	Monitoring
US-8	Kimball Brook	1,400 Left	3,900 & 5,359	Low	Monitoring
US-9	Railroad Bridge Bank	2,500-2,800 Left	5,359	Low	Further Study
US-10	Shady Brook Culvert	5,200 left	7,408	Low	Monitoring
US-11	Railroad Bridge Bank	5,300-5,600 Left	7,408 & 9,283	Low	Further Study
US-12	IRWA Dock	6,300 Right	9,283	Low	Monitoring
US-13	Miles River	7,200 Right	10,513	Low	Monitoring
US-14	Railroad Bridge	7,500	10,625 &10,689	Low	Further Study

*Outfall not observed and not on record plans. Existence hypothetical.

Under a potential dam removal scenario, the greatest changes in river hydraulics and geometry are expected at and shortly upstream of the dam site. As such, this area would also be expected to experience the greatest potential risks to infrastructure. Because the exact depth to bedrock or other hard bottom controlling river bed elevation upstream of the current dam site is unknown, the modeled dam-out geometry assumes that bed levels immediately upstream of the dam would evolve or be regraded such that the gradient of the channel through the former dam location would approximate that of upstream and downstream reaches. Under this conservative assumption, bed levels at and immediately upstream of the current spillway could be reduced by as much as 7 feet in elevation (from approximately 8 feet to approximately 1 foot) with a similar magnitude drop in water surface elevation during low flows. It is likely that the greatest changes would occur at and around the thalweg.

It should also be noted that under potential dam-out conditions, tidal hydraulic influence (though not necessarily actual saline water) is anticipated to extend approximately 4,350 feet upstream of the dam to the vicinity of Upper River Road (Station 74+08 on Figure 2B). Therefore, while the text below discusses maximum potential water level declines, actual water level conditions will vary two times per day with the rising and falling tides and with seasonal river flows. The magnitude of the difference between modeled high tide versus low tides is greatest furthest downstream and declines to a minimal amount by Station 74+08. Tidal influence is also more evident for low flow events than for high ones. For larger storm events in particular, the tidal influence is overwhelmed by the downstream river flow.

Average channel velocities at cross sections bounding the former dam site (Station 3041 at the toe of the existing dam and Station 3072 located 21 feet upstream of the existing spillway) are predicted to decrease during the 100-year flood event and increase during the 2-year flood event. River bed shear stresses are predicted to increase along the channel sides at both cross sections as a result of dam removal. Values predicted for pre- and post-removal scenarios remain within the range of shear stresses for mobilizing gravel smaller than 0.8 inch in diameter. Upstream along the margins of the lower impoundment, sediment has accumulated adjacent to existing river retaining walls and the EBSCO building and has been colonized by wetland vegetation. Predicted changes in water surface elevations suggest that these areas where ground levels are 7 and 8 feet in elevation will be above the 2-year flood water surface after the dam is removed. Provided this material remains in place and continues to support vegetation growth following dam removal, it will define a new bankfull cross section width approximately 40 feet narrower than the former impoundment. It may also help to buffer adjacent infrastructure, including retaining walls and some areas of the EBSCO building foundations, from direct hydraulic forces and undermining. However, with the simulated decrease in base level adjacent to the accumulated sediment, some sloughing of the material could occur with evacuation of impounded sediment elsewhere. As discussed further below, the conceptual design for dam removal includes protective measures intended to minimize the risks from erosion or sediment sloughing to infrastructure adjacent to the river’s edges in the lower impoundment.

3.1 Retaining Wall River Right (US-1):

Stone Retaining Wall River Right Far-field View (left) and Close-up (right)

A stone retaining wall creates the riverbank and protects the yard of a private residence immediately upstream (approximately 0-150 feet) of the dam on river right. Hydraulic modeling suggests that, under dam-out conditions, water levels will remain the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 0.6 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 3 fps with shear stresses below 0.3 pounds per square foot (psf). Table 3 below, from the Vermont Agency of Natural Resources Stream Geomorphic Assessment Handbook (VNR, April 2004) lists the critical shear stresses for various particle sizes. Table 4, from the USACE Ecosystem Management and Restoration Research Program (EMRRP, May 2001), graphically depicts the stability of channel linings for different stream velocity ranges.

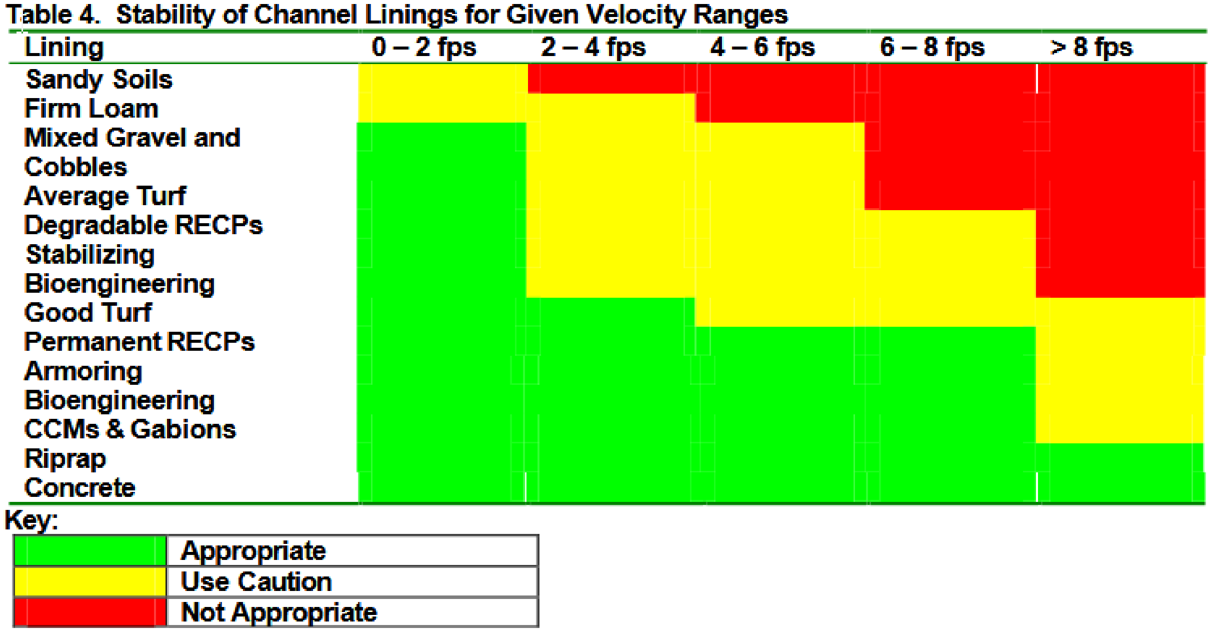
As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to coarse gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Also of note, those scenarios with the greatest simulated increases in river velocity are also simulated to have water levels several feet below the bottom of the retaining wall (and therefore also laterally away from the wall), further limiting the potential for impact to the wall. Therefore, dam removal is not considered likely to significantly impact the retaining wall.

Table 3. Limiting Shear Stresses for Uniform Non-cohesive Sediments

Particle Size	***d_s (in)***	***τ_c(psf)***	***Particle Size***	***d_s (in)***	***τ_c(psf)***
Boulder			**Sands**		
Very large	>80	37.4	Very coarse	>0.04	0.01
Large	>40	18.7	Coarse	>0.02	0.006
Medium	>20	9.3	Medium	>0.01	0.004
Small	>10	4.7	Fine	>0.005	0.003
Cobble			Very fine	>0.003	0.002
Large	>5	2.3			
Small	>2.5	1.1			
Gravel			**Silts**		
Very coarse	>1.3	0.54	Coarse	>0.002	0.001
Coarse	>0.6	0.25	Medium	>0.001	0.001
Medium	>0.3	0.12			
Fine	>0.16	0.06			
Very fine	>0.08	0.03			

d_s – diameter
τ_c – critical shear stress

Table 4. Stability of Channel Linings for Given Velocity Ranges



Another factor to consider is that, since the depth to bedrock or other hard bottom upstream of the current dam site is unknown, the amount of channel incisement that might occur as soft sediments migrate downstream under a potential dam-out scenario is also unknown. The conservative assumptions utilized in the Task 2 H&H modeling analysis allow for a significant reduction of river bed levels under potential dam-out conditions. While the majority of that incision would likely occur in the center of the channel thalweg, it is unclear whether or not such a reduction of bed levels could potentially impact the stability of retaining walls at the river's edges. It is recommended that the additional sediment probing and depth to bedrock investigation be conducted immediately upstream of the current dam location to better inform the potential depth to which the river bed might decline under potential dam-out conditions. This information would improve the accuracy of the H&H modeling and better inform the potential for impacts to all retaining walls, foundations, or other structures shortly upstream of the dam.

Due to the unknown depth to bedrock or other hard-bottom controlling river bed elevation, because the exact alignment of the river channel in this area under a dam removal scenario for all flow conditions is uncertain, because of the importance of the retaining wall, and because of the fact that the wall will be within the heart of the construction zone if the dam is removed so that protective work can be accomplished relatively easily, the draft concept design for dam removal proposes to construct fabric encapsulated soil (FES) lifts with large rock toe protection to provide scour protection in front of the wall. The FES lifts will be planted and seeded to provide a soft, vegetative protective barrier at higher elevations, while the large stone toe provides hard protection at lower elevations. The reinforcement will serve as protection for the wall in the event more erosive river flows are experienced at this location following potential dam removal, despite current modeling indications to the contrary. Further modeling and analyses are recommended, once the elevation of bedrock or other hard-bottom controlling river bed elevation is better defined, in order to better evaluate the potential for channel incision to impact the stability of retaining walls at the river's edges.

3.2 Retaining Wall River Left (US-2):

EBSCO Retaining Wall River Left Far-field View (left) and Close-up (right)

A concrete and stone retaining wall creates the riverbank and protects a patio area for the EBSCO facility immediately upstream (approximately 0-100 feet) of the dam on river left. . Hydraulic modeling suggests that, under dam-out conditions, water levels will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 0.6 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 3 fps with shear stresses below 0.3 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to coarse gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Also of note, those scenarios with the greatest simulated increases in river velocity are also simulated to have water levels several feet below the bottom of the retaining wall (and therefore also laterally away from the wall), further limiting the potential for impact to the wall. Therefore, dam removal is not considered likely to significantly impact the retaining wall.

As discussed above, since the depth to bedrock or other hard bottom upstream of the current dam site is unknown, the amount of channel incisement that might occur as soft sediments migrate downstream under a potential dam-out scenario is also unknown. Therefore, it is also uncertain whether or not such a reduction of bed levels could potentially impact the stability of retaining walls at the river's edges. It is recommended that the additional sediment probing and depth to bedrock investigation be conducted immediately upstream of the current dam location to better inform the potential depth to which the river bed might decline under potential dam-out conditions. This information would improve the accuracy of the H&H modeling and better inform the potential for impacts to all retaining walls, foundations, or other structures shortly upstream of the dam.

Due to the unknown depth to bedrock or other hard-bottom controlling river bed elevation, because the exact alignment of the river channel in this area under a dam removal scenario for all flow conditions is uncertain, because of the importance of the retaining wall, and because of the fact that the wall will be within the heart of the construction zone if the dam is removed so that protective work can be accomplished relatively easily, the draft concept design for dam removal proposes to construct fabric encapsulated soil (FES) lifts with large rock toe protection to provide scour protection in front of the wall. The FES lifts will be planted and seeded to provide a soft, vegetative protective barrier at higher elevations, while the large stone toe provides hard protection at lower elevations. The reinforcement will serve as protection for the wall in the event more erosive river flows are experienced at this location following potential dam removal, despite current modeling indications to the contrary. Further modeling and analyses are recommended, once the elevation of bedrock or other hard-bottom controlling river bed elevation is better defined, in order to better evaluate the potential for channel incision to impact the stability of retaining walls at the river's edges.

3.3 EBSCO Foundation (US-3):

EBSCO Foundation River Left Far-field View (left) and Close-up (right)

Slightly further upstream from the concrete and stone retaining wall on river left (approximately 100-440 feet from the dam), the foundation of the EBSCO facility creates the left river bank under existing conditions. Underwater test pits dug during the Task 3 Structural assessment of the EBSCO facility during this current feasibility study revealed that this foundation wall extends below the depth to which the river might potentially drop following dam removal, and appears to have rip rap stone protection placed in front of it that has become covered with river sediment over time.

Hydraulic modeling suggests that, under dam-out conditions, water levels will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 1.5 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 3 fps with shear stresses below 0.3 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to coarse gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. A portion of the EBSCO building foundation is located further out into the river and has existing river bed bathymetry at lower elevation than occurs at the retaining wall mentioned above. Therefore, the river's water level is more likely to remain at or near the foundation under dam-out condition for more flow scenarios than is the case for the retaining wall mentioned above.

Despite the generally low risk to the EBSCO foundation indicated by hydraulic modeling and despite the fact that test-pits suggest that at least some rip rap protection may already be in place in front of the foundation, the draft concept design for dam removal does include some reinforcement protection for the foundation wall. This protection is recommended because of the fact that foundation is located further out in the river channel than the walls mentioned above, because the exact alignment of the river channel in this area under a dam removal scenario is uncertain for all flow conditions, because of the importance of the foundation wall, and because of the fact that the foundation is relatively close to the construction zone if the dam is removed so that mitigation work can be accomplished relatively easily. The reinforcement proposed for the foundation wall in the draft concept design for dam removal is to construct FES lifts with large rock toe protection to provide scour protection in front of the foundation. The FES lifts will be planted and seeded to provide a soft, vegetative protective barrier at higher elevations, while the large stone toe provides hard protection at lower elevations. The reinforcement will serve as mitigative protection for the foundation in the event more erosive river flows are experienced at this location following potential dam removal, despite current modeling indications to the contrary.

For a more detailed discussion of the potential impacts to the EBSCO facility beyond the potential hydraulic impacts discussed here, please refer to the Task 3 EBSCO Structural Assessment memorandum.

3.4 Sally's Pond Outfall (US-4):

Sally Pond Outlet Control Structure Far-field View (left) and Close-up (right)

According to Town Conservation Commission records, Sally's Pond was constructed at its location shortly upstream of the dam above the right bank in the 1970s. An outfall control structure was visually observed adjacent to the pond shore, between the pond and the river. Inside the outlet structure is a removable board, weir structure that appears to have been built to allow the pond to drain to the river at varying pond elevations that could be set with the weir. The structure appeared to be disused and at least partially clogged with debris (IRWA observation). No outfall to the river was observed even during the impoundment drawdown period in September 2016, suggesting that the river end of the outfall must be below the water level observed during the drawdown, or even potentially buried in sediment. If an outfall exists to the river, visual observation of the likely connection line between the outlet structure on the river bank and the river suggests that it would likely be located somewhere between approximately 250-450 feet upstream from the dam.

Hydraulic modeling indicates that water levels at this location will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 2.3 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 4 fps with shear stresses below 0.35 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to coarse gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Therefore, some channel bottom erosion and sediment migration may occur as the river adjusts to new hydraulic conditions following dam removal.

The concern here is that if the new river level were significantly below the invert of the outfall for extended periods when the outfall was actively draining significant water from the pond to the river, and the exposed sediments beneath the culvert invert were sufficiently soft and mobile, a headcutting concern could be realized between the culvert and the new river's edge. However, even if the pond's outlet pipe proves to be above the river's water level under dam-out conditions and most flow scenarios, it is unclear if the outlet is currently active. According to IRWA, and based on visual observation of the outlet structure at the pond's edge, it does not appear that the Town actively uses the outlet structure to release water from the pond to the river. Therefore, potential headcutting concerns at this location do not appear likely as a result of potential dam removal. We recommend that conditions be monitored following dam removal to see if an outlet pipe is observed that actively drains pond water.

Another potential impact from dam removal to consider is the impact to Sally's Pond itself. The water level in Sally's Pond is likely a relatively close approximation of the surrounding groundwater elevation, though the pond's outlet control structure likely serves to maintain pond levels somewhat higher than the surrounding groundwater elevations. Lowered river levels following potential dam removal would result in lowered groundwater levels for areas in close proximity to the river. Because groundwater responds much slower to prevailing hydrologic conditions than does surface water, groundwater impacts from river level changes will respond more closely to average river levels than to any time-specific river level condition. H&H modeling shows an approximately 5.5-foot decrease in river levels for the area in front of Sally's Pond for the 50% exceedance flow condition, averaged between low and high tide conditions. Due to the resistance to flow posed by the aquifer sediments, changes in groundwater levels will quickly dissipate with distance from the river.

No hydrogeologic study was undertaken of the characteristics of the aquifer between the pond and the river as part of this feasibility study so an accurate estimation of the changes in groundwater levels cannot be made at this time. However, assuming some plausible aquifer characteristics (transmissivity of 3,000 feet squared per day, and a storage coefficient of .5 (averaging aquifer storage and river storage)), then using the Theis equation to calculate the pumping rate necessary to produce a 5.5-foot drawdown adjacent to a hypothetical pumping well, and then also using the Theis equation to estimate the drawdown 100 feet and 250 feet away from the hypothetical well (the approximate distance between the near and far ends of pond from the river), groundwater drawdowns are estimated of approximately 1.5 feet and 0.8 feet, respectively, after a month of pumping (approximation of steady state conditions).

This is likely a conservative over estimate of the potential drawdown due to the use of simplified pumping test analytical equations to estimate groundwater drawdown, when we are actually looking at potential pond level changes resulting from river level changes. Both the pond and the river will respond to natural climatic and hydrologic variations from local precipitation and evapotranspiration that are not included in the simplified drawdown estimates. The pond's outlet structure will also retain pond levels above the local groundwater to some extent. Assuming a groundwater level decline around Sally's Pond somewhere in the range of one foot following potential dam removal, the wetland community of the pond would likely be expected to transition to more bordering vegetated wetland (BVW) and less open water.

3.5 Sally's Pond Canoe Launch (US-5):

Sally's Pond Public Canoe Launch River Right Far-field View (left) and Close-up (right)

A public canoe launch is located on the river right bank below Sally's Pond. Modeled hydraulic river changes at this location following potential dam removal are as described above for the Sally's Pond outfall. Since the river stage is modeled to drop at this location under all but larger storm flow conditions, the distance between the current canoe launch site and the water's edge will increase under most conditions when the canoe launch would likely be used. The potential risk here is that canoe launch access to the river may be impacted if the newly exposed shoreline is too soft or muddy to allow for equivalent access as currently exists. According to IRWA, the river bottom at this location is relatively firm so that potential impacts to canoe access are expected to be minimal. According to IRWA, this canoe launch site is used primarily as a portage location for paddlers to get around the dam. Therefore, under a dam-out scenario usage of the portage would be significantly reduced. An adaptive management approach is recommended for this site. Conditions should be monitored following dam removal and improvements made to the canoe launch site, as necessary, to preserve public access. According to IRWA, such improvements, if needed, are anticipated to be relatively minor. They might include providing supportive matting, or similar solid walking surface, between the current and post dam-removal river's edges to allow for continued canoe launch access.

3.6 Peatfield Street Canoe Launch (US-6):

Peatfield Street Public Canoe Launch River Left (during September 2016 drawdown)

A, public river access/ canoe launch site is located at the end of Peatfield Street, approximately 920 feet upstream from the dam on river left. Hydraulic modeling indicates that water levels at this location will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 1.8 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 4 fps with shear stresses below 0.35 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to coarse gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Therefore, some channel bottom erosion and sediment migration may occur as the river adjusts to new hydraulic conditions following dam removal.

Since the river stage is modeled to drop at this location under all but larger storm flow conditions, the distance between the current canoe launch site and the water's edge will increase under most conditions when the canoe launch would likely to be used. However, for average to lower flow events (those conditions under which canoe launch access might be expected to be greatest impacted), twice daily high tides will raise modeled river elevations by between 0.5 and 1.5 feet above those modeled for low tides. The potential concern here is if canoe launch access to the river may be impacted if the newly exposed shoreline is too soft or muddy to allow for equivalent access as currently exists. According to IRWA, the river bottom at this location is relatively firm so that potential impacts to canoe access are expected to be minimal. An adaptive management approach is recommended for this site. Conditions should be monitored following dam removal and improvements made to the canoe launch site, as necessary, to preserve public access. According to IRWA, such improvements, if needed, are anticipated to be relatively minor. They might include providing supportive matting, or similar solid walking surface, between the current and post dam-removal river's edges to allow for continued canoe launch access.

3.7 Saltonstall and Kimball Brooks (US-7&8):

Saltonstall Brook River Left (left) and Kimball Brook River Right (right) Confluences

Approximately 1,200 and 1,400 feet upstream from the dam on river right and left, respectively, two tributaries (Saltonstall and Kimball Brooks) join the river. Both confluences are natural channels with no culverts or other hard, engineered structures. Hydraulic modeling indicates that water levels at this location will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 1.8 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 4 fps with shear stresses below 0.35 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to coarse gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Therefore, some channel bottom erosion and sediment migration may occur as the river adjusts to new hydraulic conditions following dam removal.

Since the river stage is modeled to drop at these river confluences under all but larger storm flow conditions, and shear stresses and velocities are modeled to be capable of mobilizing smaller sediment sizes, some degree of sediment migration and channel headcutting is expected to occur as the tributary channel seeks to adjust to the new river level. This type of channel reconfiguration is expected as the river and its tributaries reconfigure themselves to the natural conditions that prevailed before the influence of the dam. According to IRWA, and as observed from aerial photography, there are no culverts, bridges, or other infrastructure in the vicinity of these confluences likely to be impacted by channel reconfiguration. We recommend that conditions be monitored following dam removal to document the extent of headcutting that occurs.

<u>3.8 Railroad Embankments Near Hayward Street and Across From IRWA Building (US-9&11):</u>


Railroad Embankment River Left Near Hayward Street Far-field View (left) and Close-up (right)


Railroad Embankment River Left Across from IRWA Office Far-field View (left) and Close-up (right)

The railroad line crossing and adjacent to the river is owned by the Massachusetts Bay Transportation Authority (MBTA) and operations, including maintenance and repairs, are undertaken by Keolis, a private company contracted by the MBTA to run the commuter rail service. Currently the river touches the embankment for the MBTA railroad line at two locations upstream from the dam. The closest, approximately a half mile upstream from the dam, is near Hayward Street and shortly upstream from Sixth Street, where an approximately 265-foot long length of the embankment is in close proximity to the river's edge. The second, approximately a mile upstream from the dam, is across the river from the IRWA offices in a largely wooded, undeveloped area of the river's left bank, where an approximately 285-foot long length of the embankment is in close proximity to the river's edge. Both locations are characterized by relatively steep banks that are moderately well vegetated with some indications of minor ongoing erosion (see second row photo right, above). The concern at these two locations is if, under a dam-out scenario, the river was to hit these banks at higher velocities that might exacerbate erosion.

At the Hayward Street embankment, hydraulic modeling indicates that water levels at this location will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 6 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 1 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 2.5 fps with shear stresses below 0.15 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to medium gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Also of note, those scenarios with the greatest simulated increases in river velocity are also simulated to have water levels several feet lower in the river channel, and thus laterally further away from the embankment, further limiting the potential for impact to the embankment.

At the embankment across the river from the IRWA building, hydraulic modeling indicates that water levels at this location will remain essentially the same or slightly lower for the larger storm flows and drop moderately under the lower flow scenarios (up to approximately 1.7 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase slightly under the moderate and low flow scenarios (up to an approximately 0.6 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 2.5 fps with shear stresses below 0.15 psf. These shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils (Pennsylvania DEP Erosion and Sediment Control Manual, 2012). Also of note, those scenarios with the greatest simulated increases in river velocity are also simulated to have water levels a foot or so lower in the river channel, and thus laterally further away from the embankment, further limiting the potential for impact to the embankment.

Because modeled increases in river velocity are moderate, and because those flow scenarios where velocity is modeled to increase also are simulated to have lower river elevations, the potential for significant erosive impacts to these embankments appears relatively low. Nonetheless, due to the importance of the railroad infrastructure and because some potential increase of erosive river flows is possible, further study of this concern is recommended. Such study should include field evaluation of the topography and bathymetry of the embankments and adjacent river channel, field evaluation of the sediment characteristics relevant to their potential for erosion, scour analysis of the potential for erosion at these locations (U.S.DOT, April 2012), and more detailed design to specifically address any identified erosion concerns. The MBTA and Keolis should also be consulted to solicit their input on the types of additional analyses and design factors it would require to satisfy any of its potential concerns.

3.9 Shady Brook Culvert (US-10):

Shady Brook Culvert Downstream End (left), Shortly Downstream (right), and Upstream (bottom)

Approximately a mile upstream from the dam, and just downstream from the more upstream railroad embankment discussed above, on river left, the Shady Brook tributary joins the river. Shady Brook drains a small wetland upstream (northwest) of the railroad tracks and is conveyed beneath the railroad bed by a culvert. According to IRWA, the Ipswich River may flood into the Shady Brook area during high flow events and then drain back to the river downstream through the culvert beneath the railroad bed. According to 2015 culvert modification plans obtained from the Ipswich Conservation Commission by IRWA, the culvert is an old, 3-foot X 5-foot granite-block, box culvert. The 2015 modifications appear to have been the addition of two -1-foot diameter concrete pipes inserted into the upstream end of the granite box culvert to extend the culvert inlet approximately 20 feet upstream and away from the railroad embankment, presumably to allow for the addition of stone erosion control protection to that upstream side of the embankment.

IRWA and HW staff visited the site In September of 2018 to observe conditions and noted that the upstream work had been completed approximately as shown on the 2015 plans, that the downstream invert was dry at the time of the visits, and that no significant erosion was currently evident downstream of the culvert. According to IRWA, a berm at the river's edge separates the river from Shady Brook during drier, lower-flow conditions.

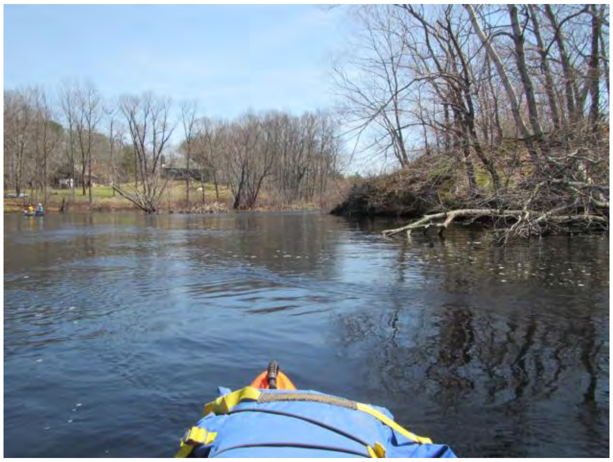
The Shady Brook culvert is offset approximately 400 feet from the main river channel and is therefore not included in the H&H model. Hydraulic modeling for the main river channel indicates that water levels in the vicinity of the Shady brook confluence will remain essentially the same or slightly lower for the larger storm flows and drop significantly under the lower flow scenarios (up to approximately 5 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 0.8 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 2.5 fps with shear stresses below 0.2 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to medium gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Therefore, some channel bottom erosion and sediment migration may occur as the river adjusts to new hydraulic conditions following dam removal.

Since the river stage is modeled to drop at these river confluences under all but larger storm flow conditions, and shear stresses and velocities are modeled to be capable of mobilizing smaller sediment sizes, the potential for some degree of sediment migration and channel headcutting exists as the tributary channel seeks to adjust to the new river level. This type of channel reconfiguration is expected as the river and its tributaries reconfigure themselves to the natural conditions that prevailed before the influence of the dam. Because the culvert is offset so far from the main river channel and is separated from the main channel by a berm, the potential that lowered water levels in the main channel as a result of dam removal would encourage significantly increased headcutting below the culvert may be less than would otherwise be the case. We recommend that conditions be monitored following dam removal to document the extent of headcutting that occurs.

3.10 IRWA Dock (US-12):

IRWA maintains a dock and canoe launch site approximately 6,300 feet upstream from the dam on river right. Hydraulic modeling indicates that water levels at this location will remain essentially the same or slightly lower for the larger storm flows and drop moderately under the lower flow scenarios (up to approximately 1.7 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase slightly under the moderate and low flow scenarios (up to an approximately 0.6 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 2.5 fps with shear stresses below 0.15 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to medium gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Therefore, some channel bottom erosion and sediment migration may occur as the river adjusts to new hydraulic conditions following dam removal. According to IRWA, since the depth of water under the existing dock exceeds 1.7 feet during current low flow periods, dam removal is not expected to limit access significantly.

3.11 Miles River (US-13):



Miles River Confluence River Right (left) and Further Upstream on the Miles River (right)

The Miles River is the largest tributary entering the Ipswich River in the stretch of the lower river assessed in this memorandum. It joins the Ipswich approximately 7,200 feet upstream from the dam on river right, shortly downstream from the railroad bridge crossing. Hydraulic modeling at this location indicates that water levels in the vicinity of the Miles River confluence will remain essentially the same or slightly lower for the larger storm flows and drop moderately under the lower flow scenarios (up to approximately 1.7 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 0.4 fps increase under the 2-year storm and 5% exceedance scenarios). However, even those increased flow velocities are simulated to remain under 2 fps with shear stresses below 0.1 psf. As can be seen in Tables 3 and 4, those modeled shear stresses and velocities are capable of mobilizing smaller sediment sizes from silts up to fine gravel, but not larger materials or any vegetated or stabilized surfaces. The Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), also indicates that these shear stresses are adequate to mobilize bare, erodible soils, but not sufficient to erode vegetated or reinforced soils. Therefore, some channel bottom erosion and sediment migration may occur as the river adjusts to new hydraulic conditions following dam removal.

Since the river stage is modeled to drop at these river confluences under all but larger storm flow conditions, and shear stresses and velocities are modeled to be capable of mobilizing smaller sediment sizes, some degree of sediment migration and channel headcutting is expected to occur as the tributary channel seeks to adjust to the new river level. This type of channel reconfiguration is expected as the river and its tributaries reconfigure themselves to the natural conditions that prevailed before the influence of the dam. Because the Miles River is the largest tributary in the potentially impacted stretch of the Ipswich River assessed herein, the concern for headcuttting is perhaps greater here than at the smaller tributaries due to its greater flow rate. Contradicting the potentially greater risk for headcutting here due to higher flows is the fact that the modeled declines in water level here are less than at locations further downstream.

According to IRWA, and as observed from aerial photography, there are no culverts, bridges, or other infrastructure in the vicinity of this confluence likely to be impacted by channel reconfiguration. There is a large wetland complex in the area of the confluence that would be expected to transition its vegetation community type towards less land under water and more BVW under the influence of lower average water levels under potential dam-out conditions. Emergent marsh would also be expected to shift to lower elevations. We recommend that conditions be monitored following dam removal to document the extent of headcutting that occurs.   Therefore, an adaptive management approach is recommended here, the same as at the other confluences. We recommend that conditions be monitored following dam removal to document the extent of headcutting that occurs, as well as the changes in wetlands community types.

3.12 Railroad Bridge (US-14):

Railroad Bridge Views (Photos Taken During September 2016 Drawdown)
from Upstream (top two rows) and Downstream (bottom two rows)

Located approximately a mile and a half upstream from the dam, the railroad bridge crossing is the most upstream structure with the potential to be impacted by dam removal. According to IRWA, the bridge is owned by the MBTA and operations are undertaken by Keolis. The crossing is actually two separate bridges adjacent to each other. The railroad bridge is the more downstream of the two, newer, and supported by newer concrete piers. The older, more upstream bridge, supported by timber piers, is for the railroad's service road.

Due to the elevated, stone-reinforced river bed beneath the bridges, a hydraulic drop is created at the bridge during lower river water levels under current conditions (as evident from the above photos taken during the September 2016 drawdown and with drought conditions). Visual observations during the drawdown also indicated that there is a tendency for logs and other large debris flowing down river to become trapped against the upstream bridge support piers, exacerbating the potential for a hydraulic drop at this location. Under a potential dam-out scenario with lowered water levels, both the magnitude of that hydraulic drop and the frequency with which it occurs are expected to be increased. According to IRWA, MBTA conducts routine clearing of debris beneath the bridge. That maintenance program would likely become more important to sustain fish and recreational passage beneath the bridges with lowered water levels under potential dam-out conditions.

Hydraulic modeling immediately downstream from the railroad bridge indicates that water levels will remain essentially the same or slightly lower for the larger storm flows and drop moderately for all other flow scenarios (up to an approximately 1.1 feet for the 95% exceedance/low tide scenario). River flow velocities are modeled to remain essentially the same or drop slightly for the higher flow scenarios, and to increase significantly under the moderate and low flow scenarios (up to an approximately 7 fps increase under the 2-year storm and 5% exceedance scenarios). These increased flow velocities are simulated to reach up to 12 fps with shear stresses up to 5.5 psf. As can be seen in Tables 3 and 4, and as shown in the Pennsylvania DEP Erosion and Sediment Control Manual (PADEP, 2012), these shear stresses and velocities require significant reinforcement with heavy stone, rip rap, gabions, or other engineered protection. Therefore, the potential for increased erosion at this location will need to be fully evaluated in subsequent stages of potential dam removal design and discussed with the bridge's owners (MBTA) and operators (Keolis).

Despite these significant increases in erosive river flow for certain flow scenarios, the simulated velocity and shear stress numbers for those scenarios with the greatest modeled increases resulting from potential dam removal are still lower than are modeled to currently occur under larger storm events. For example, the dam-out modeled velocity and shear stress for the 2-year storm of 12 fps and 5.5 psf, respectively, are less than the modeled existing conditions values for the 10-year storm event of 15.3 fps and 8.1 psf. No significant changes to river level, velocity, or shear stress are simulated to occur as a result of dam removal for flow scenarios of the 10-year storm and greater.

Hydraulic modeling immediately upstream from the railroad bridge indicates significantly different results than for the downstream side. Upstream of the bridge, modeling still indicates that water levels will remain essentially the same or slightly lower for the larger storm flows, but water levels are only simulated to drop for the lowest of flow scenarios (up to an approximately 1 foot for the 50% exceedance/low tide scenario and approximately 0.6 feet for the 95% exceedance/low tide scenario). No significant change in water level is simulated to occur for any storm flow scenario or the 5% exceedance scenario. Modeled changes to river flow velocities follow a similar trend. Velocities are modeled to remain essentially the same or drop slightly for all storm and higher flow scenarios, and to increase moderately under the moderate and low flow scenarios (up to an approximately 1.5 fps increase under the 50% exceedance/low tide scenario). However, these increased flow velocities under the 50% and 95% exceedance scenarios are simulated to remain below 4 fps with shear stresses below 0.6 psf, both below values that currently exist under all modeled storm and higher flow scenarios.

These modeling results suggest that the existing stone scour protection beneath the bridge will become the new elevation control holding back upstream river flows and increasing river velocity beneath the bridge for the lower flow and non-storm scenarios. Higher modeled river flow scenarios and all storm flow scenarios are not modeled to be significantly impacted by dam removal at this location. Because the existing scour protection apparently adequately serves the existing conditions, and the existing conditions for higher flow scenarios are simulated to represent more erosive velocities and shear stresses than the increased velocity, lower flow scenarios simulated to occur under dam-out conditions, the existing scour protection may be adequate for potential dam-out conditions.

Nevertheless, because of the importance of the railroad bridge and because more erosive conditions under lower flow scenarios may occur more frequently than those erosive conditions that currently occur under higher flow scenarios, further study of this concern is recommended. Such study should include field evaluation of the sediment characteristics relevant to their potential for erosion, scour analysis of the potential for erosion at these locations (U.S.DOT, April 2012), and more detailed design to specifically address any identified erosion concerns. The MBTA and Keolis should also be consulted to solicit their input on the types of additional analyses and design factors it would require to satisfy any of its potential concerns. Access to the railroad line for work would be via the existing gates and maintenance roads. One is located at Hayward Street from the North, and the other at Waldingfield Road from the south (Figure 2B).

H&H modeling indicates that the area beneath the railroad crossing may become more problematic for fish passage under potential dam-out conditions due to the hydraulic jump created by the existing rock-bed scour protection under low flow conditions. As was observed during the 2016 drawdown, irregularities in the rock bed may provide diverse flow conditions and opportunities for fish passage over this short distance that would allow functional fish passage to continue under potential dam-out conditions, but this situation should be further evaluated in subsequent project design phases to ensure that appropriate fish passage is maintained (Turek et al, 2016, and USFW, 2017). Fish passage design should be conducted in concert with scour protection design to ensure that both goals can be achieved.

3.0 Drinking Water Wells

The potential concerns regarding dam removal on drinking water wells are that groundwater levels might drop sufficiently to reduce well yields, and that, if saline water were to migrate further upstream than currently occurs, such saline water might potentially impact water quality in the aquifer surrounding the river. Regarding potential declines in groundwater level from dam removal the following should be considered:

- The river represents a boundary condition “hinging” one end of a groundwater table transect line running perpendicular away from the river. Groundwater flows from inland locations towards the river boundary. The slope of the water table transect line is determined by the elevation of the river boundary condition at one end, and by precipitation-derived inputs to groundwater (and any withdrawals by wells) along the remainder of the transect. Because hydraulic changes in groundwater occur much slower than in surface water due to the restrictive nature of the solid aquifer matrix through which groundwater must move, groundwater levels tend to respond more to longer-term, average, surface water boundary condition levels than to shorter term fluctuations. Therefore, the average tidal condition in the river influences neighboring groundwater levels more significantly than does either low or high tide conditions. Similarly, the average, climatically-influenced river level over periods of weeks or months is more significant than hourly or daily fluctuations.
- The restrictive nature of the aquifer also dampens the influence of boundary condition elevation changes as you move landward away from the river boundary. As discussed above in Section 3.4 above regarding Sally’s Pond, estimated groundwater declines from changes in river level dissipate rapidly on the order of hundreds of feet away from

the river, even for areas of the river proximal to the dam where river level declines would be greatest. And as one moves further upstream from the dam, the estimated declines in river level decrease so that the corresponding distance laterally away from the river in which significant declines in groundwater might occur also decreases.

Regarding potential salinity impacts the following should be considered:

- While we know that tidal influence currently extends up to the dam, and would extend upstream beyond the dam under dam-out conditions, we do not know how saline the actual water chemistry is at the dam site (or the vertical distribution of salinity within the water column), and we therefore do not how far upstream of the dam saline water might reach under dam-out conditions. According to IRWA, salt water is rarely detected above the lower falls in the dozens of water samples collected by the Division of Marine Fisheries (DMF) over the years, and only reaches to the dam site for spring high tides that occur during periods of low river flow. In addition, also according to IRWA, the existence of a population of Brookweed (a tidal freshwater species sensitive to salt) near County Road and the existence of Rainbow Smelt spawning area between the dam and County Road further suggests that the likelihood for significant and regular salt water contributions at and above the dam site are unlikely.

- Salt water is more dense than fresh water and thus tends to pool at the bottom of a water column. This is true both for surface waters and for groundwater. For example, portions of the Cape Cod Aquifer float as a freshwater lens above denser salt water below. Such a situation does not occur for the Ipswich River because the aquifer beneath and around the Ipswich River does not have sufficient depth of permeable materials connected to the ocean to allow for such a salt water conenctino through groundwater. The lower falls on the Ipswich River is one example of a bedrock boundary impeding the ability of ocean-based salt water to infiltrate into the base of the aquifer. Any salt water influence from the river on the aquifer is limited to the quantity that may infiltrate from the river to the aquifer during periods when the pressure head in the river is greater than the underlying aquifer (e.g. high tides). Since the prevalent gradient is from the aquifer into the river (the river is a discharge boundary for the aquifer), opportunities for significant salt water infiltration from the river into the aquifer are limited. At a position as upstream along the river as the dam, the significance of salt water influence from the river on the aquifer is likely minimal in terms of both the actual salinity and the horizontal extent of any such influence away from the river. Further upstream, the likely significance diminishes still further.

IRWA researched wells located from the dam site upstream to the identified limit of potential water level impact from dam removal shortly upstream of the railroad bridge (as identified by the Task 2 H&H analysis) and extending out 1,000 feet to either side of the river. IRWA research included Board of Health (BOH) records for direct evidence of private wells and public drinking water connection records for indirect evidence. Any developed property not recorded to be receiving public water supply was assumed to have a private well. This research did not include the possibility of irrigation wells on properties connected to the public water system. IRWA research revealed the following:

- There are no public water supply sources within the potential dam-removal impact area described above. The closest active sources operated by the Town of Ipswich are three gravel-packed wells. One is the Winthrop Well located in close proximity to the river at

200 Topsfield Road, about three miles upstream form the dam and well above the limit of potential hydraulic impact from dam removal. The other two are the Fellows Road Well and the Essex Road well located in close proximity to each other about ¾ mile east of the river, at approximately the same river-length distance upstream from the dam as the railroad bridge crossing. All three of these wells are located far outside of the zone of potential negative impacts from dam removal.

- Regarding potential future public water supply sources, according to IRWA, there is no potential for increased public water withdrawals since the regulatory safe yield for the Ipswich River basin has been exceeded, thus prohibiting the permitting of additional withdrawals within the basin over what is allowed currently. In terms of opportunities to develop new replacement sources, dam removal would not affect those since, according to IRWA, any new wells would be developed upstream of the potential influence of dam removal (due to the developed land use closer to the dam) and any new surface water withdrawals would not be practical due to the marginal amount of storage provided by the current dam and the need to provide advanced treatment for a river water source.
- Ipswich BOH private well records only go back to the year 2000. There has been one new well known to have been installed since that time within the area of potential impact (shown on Figure 2B as Private Well 1). This well is located over a mile upstream from the dam along the west bank of the Miles River near its confluence with the Ipswich. it is a deep bedrock wells whose hydraulic and water quality influences from the river would be even less than discussed above for the surficial sand and gravel aquifer.
- While not recorded in BOH records, IRWA is aware of three other known private wells within the zone of potential influence from dam removal. Private Well 1 (as shown on Figure 2B) was installed around 1990 and is located across the Miles River from Private Well 1 at the confluence with the Ipswich. It is also a deep bedrock well and therefore also has diminished potential impact from dam removal. Private Wells 3 & 4 (as shown on Figure 2B) are located approximately 1,300 up the Miles River from its confluence with the Ipswich and are, therefore, even further removed from any potential impact from dam removal.
- IRWA is also aware of another potential, but unconfirmed well at a landscape company not on town water (Unconfirmed Well on Figure 2B). If present, its depth or even what aquifer it is screened in is unknown.
- According to IRWA, the entirety of the remaining properties within the potential area of influence dam removal are connected to or have access to the public water supply. In addition, any potential impacts felt by private wells (known or unknown) as a result of dam removal could be readily mitigated by connecting to town water.

While the potential for significant impact from dam removal to any private wells along the river is low based on available information, it is recommended that additional efforts be made to identify any additional private wells (e.g., irrigation wells) beyond those discussed herein. Any wells identified within the zone of potential influence from dam removal should have their baseline depths to water and salinity documented. That would allow for a comparison of future well conditions to baseline in the event that those well owners believe that their wells have been impacted following potential dam removal.

4.0 Summary

Based on examination of aerial photography, visual field observations, and discussions with IRWA, seven structures downstream of the dam and 14 upstream of the dam were identified as locations with the potential to be impacted by the hydraulic effects of dam removal. Comparing the locations of those 21 structures to modeled changes in river level, velocity, and erosive shear stress under various river flow scenarios revealed that three of those structures may potentially experience some potentially negative hydraulic impact from dam removal. Those three structures are the retaining wall immediately downstream of the dam on river right integrated into the abandoned fishway, the support piers for the pedestrian platform immediately downstream from the dam on river left, and the railroad bridge crossing at the upstream limits of the river impoundment. The two structures adjacent to the dam have erosion protection mitigation measures proposed as part of the conceptual design for dam removal to protect them from the potential of increased erosion under a dam-out scenario. For the railroad bridge, it is recommended to conduct further scour and fish passage analyses, and to work with the bridge owners to discuss modifications to the scour protection there. In addition, it is recommended that additional field study and scour analyses be conducted for the two areas where the railroad embankment currently touch the river's edge. While H&H modeling does not currently indicate a significant risk to these structures, further study is recommended due to the importance of the infrastructure and its close proximity to the river.

Three other structures appear less likely to experience significant negative impacts but are still recommended to receive protective actions as part of any potential dam removal plans. Those three structures are the retaining walls immediately upstream of the dam on river right and left, and the EBSCO building foundation shortly upstream from the dam on river left. Since the depth to bedrock or other hard bottom control of the river bed upstream of the current dam site is unknown, the amount of channel incisement that might occur as soft sediments migrate downstream under a potential dam-out scenario is also unknown. Therefore, it is also uncertain whether or not such a reduction of bed levels could potentially impact the stability of retaining walls at the river's edges. It is recommended that the additional sediment probing and depth to bedrock investigation be conducted upstream of the current dam location to better inform the potential depth to which the river bed might decline under potential dam-out conditions. This information would improve the accuracy of the H&H modeling and better inform the potential for impacts to all retaining walls, foundations, or other structures shortly upstream of the dam. Further modeling and analyses are recommended, once the elevation of bedrock or other hard-bottom controlling river bed elevation is better defined, in order to better evaluate the potential for channel incision to impact the stability of retaining walls at the river's edges.

Four tributary confluences, two canoe launches, and the IRWA dock area were evaluated to have low potential for headcutting of soft sediments beneath them that might lead to erosion or gullying. That head cutting is essentially a natural process where the river and its tributaries seek to regain a geometry representative of dam-out hydraulic conditions that would have previously existed. No significant concerns for infrastructure were identified relative to these potential headcutting areas but monitoring is recommended. In addition to the hydraulic conditions, the potential for headcutting is also influenced by the composition of the river bed materials at those locations. Loose, fine grained materials are more susceptible to erosion than coarse and compacted materials. The material composition of the river bed at all locations is not currently known, particularly those river bed locations that are currently under relatively deep water. Additional probing of sediment thickness and analyses of sediment composition are recommended throughout the current impoundment area to better inform the potential for headcutting and sediment migration.

While the potential for significant impact from dam removal to any private wells along the river is low based on available information, it is recommended that additional efforts be made to identify any additional private wells (e.g. irrigation wells) beyond those discussed herein. Any wells identified within the zone of potential influence from dam removal should have their baseline depths to water and salinity documented. That would allow for a comparison of future well conditions to baseline in the event that those well owners believe that their wells have been impacted following potential dam removal.

5.0 References

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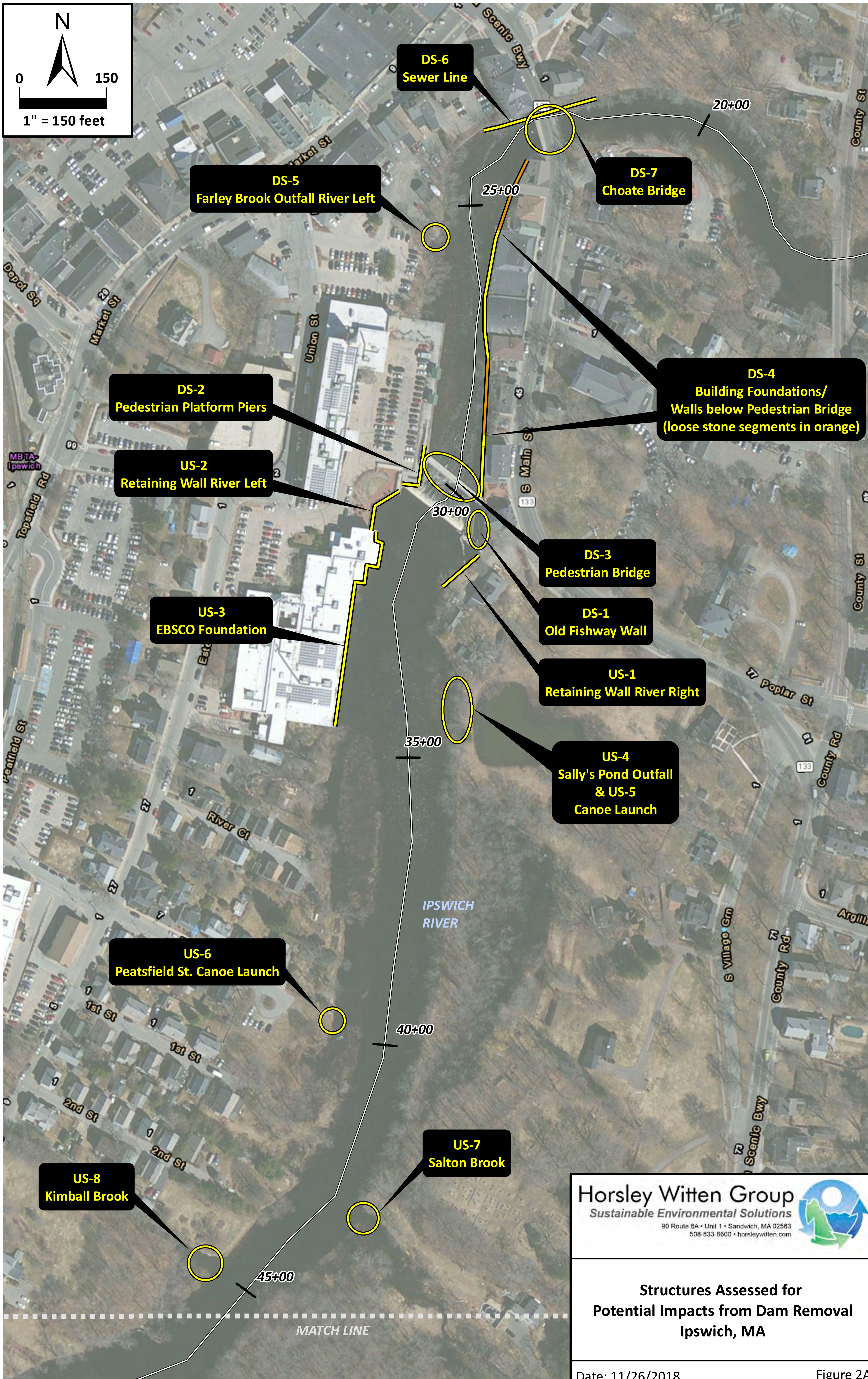

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Structures Assessed for Potential Impacts from Dam Removal
Ipswich, MA

Date: 11/26/2018

Figure 2A

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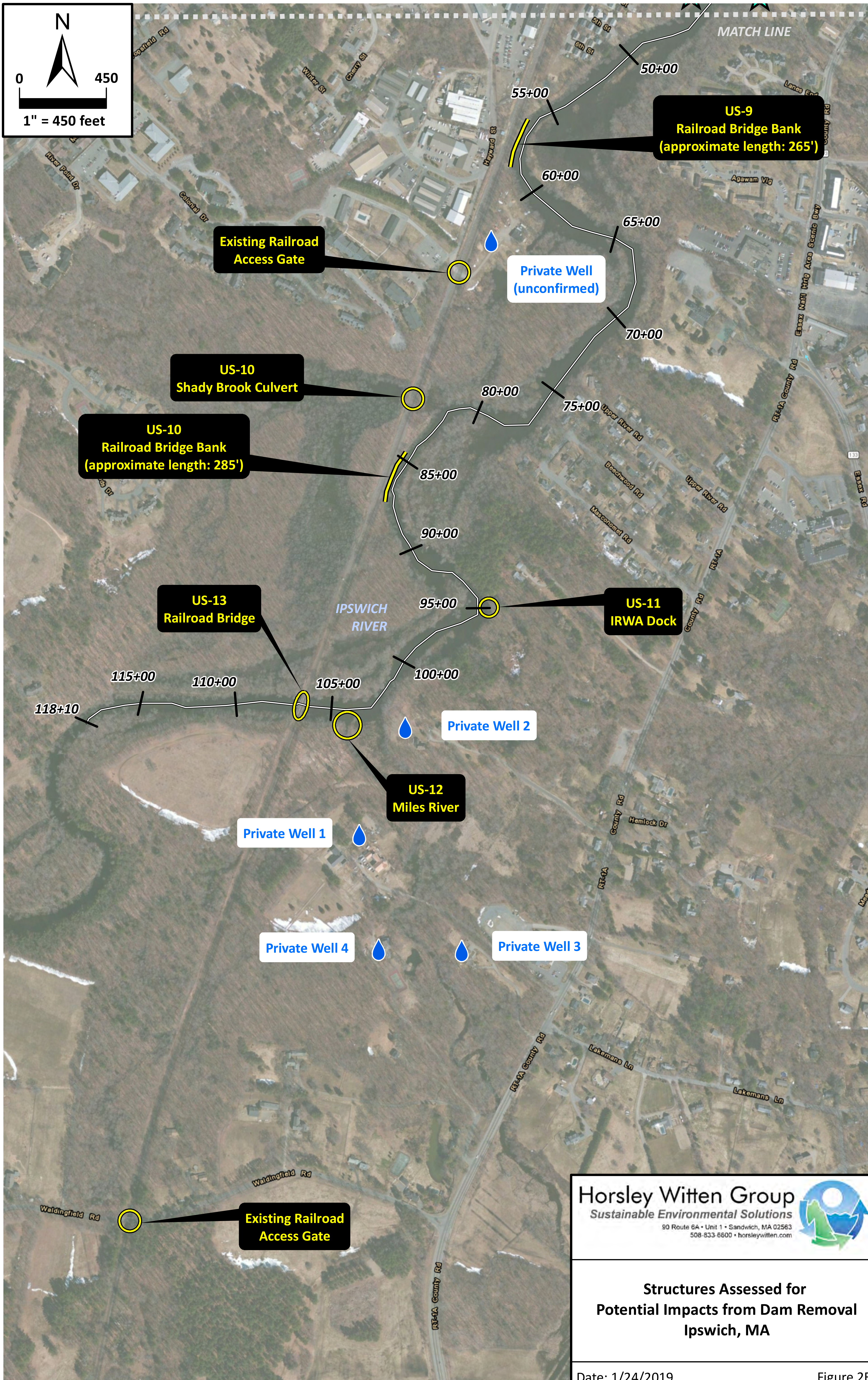

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Structures Assessed for Potential Impacts from Dam Removal Ipswich, MA

Date: 1/24/2019

Figure 2B

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APPENDIX A: HEC-RAS Model Results

Appendix A. Ipswich River HEC-RAS Model Results

(Ordered from Upstream to Downstream)

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	11787	2 yr	EC low tide	1439	6.54	13	13.17	487.59	187.8	3.31	0.18	0.35	0.28	0.01	0.03
1	11787	2 yr	EC high tide	1439	6.54	13	13.17	487.59	187.8	3.31	0.18	0.35	0.28	0.01	0.03
1	11787	2 yr	PC low tide	1439	6.54	13	13.17	487.51	186.22	3.31	0.18	0.35	0.28	0.01	0.03
1	11787	2 yr	PC high tide	1439	6.54	13	13.17	487.51	186.22	3.31	0.18	0.35	0.28	0.01	0.03
1	11787	10 yr	EC low tide	3316	6.54	15.64	15.91	1134.79	254.88	4.48	0.58	0.93	0.46	0.08	0.17
1	11787	10 yr	EC high tide	3316	6.54	15.64	15.91	1134.79	254.88	4.48	0.58	0.93	0.46	0.08	0.17
1	11787	10 yr	PC low tide	3316	6.54	15.64	15.91	1134.79	254.88	4.48	0.58	0.93	0.46	0.08	0.17
1	11787	10 yr	PC high tide	3316	6.54	15.64	15.91	1134.79	254.88	4.48	0.58	0.93	0.46	0.08	0.17
1	11787	25 yr	EC low tide	4569	6.54	17.07	17.38	1505.29	264.87	4.92	0.73	1.17	0.52	0.12	0.24
1	11787	25 yr	EC high tide	4569	6.54	17.07	17.38	1505.29	264.87	4.92	0.73	1.17	0.52	0.12	0.24
1	11787	25 yr	PC low tide	4569	6.54	17.07	17.38	1505.29	264.87	4.92	0.73	1.17	0.52	0.12	0.24
1	11787	25 yr	PC high tide	4569	6.54	17.07	17.38	1505.29	264.87	4.92	0.73	1.17	0.52	0.12	0.24
1	11787	50 yr	EC low tide	5644	6.54	18.2	18.53	1808.77	273.26	5.21	0.82	1.33	0.56	0.14	0.29
1	11787	50 yr	EC high tide	5644	6.54	18.2	18.53	1808.77	273.26	5.21	0.82	1.33	0.56	0.14	0.29
1	11787	50 yr	PC low tide	5644	6.54	18.2	18.53	1808.77	273.26	5.21	0.82	1.33	0.56	0.14	0.29
1	11787	50 yr	PC high tide	5644	6.54	18.2	18.53	1808.77	273.26	5.21	0.82	1.33	0.56	0.14	0.29
1	11787	100 yr	EC low tide	6846	6.54	21.29	21.52	2693.47	299.43	4.48	0.76	1.28	0.38	0.11	0.23
1	11787	100 yr	EC high tide	6846	6.54	21.29	21.52	2693.47	299.43	4.48	0.76	1.28	0.38	0.11	0.23
1	11787	100 yr	PC low tide	6846	6.54	21.29	21.52	2693.47	299.43	4.48	0.76	1.28	0.38	0.11	0.23
1	11787	100 yr	PC high tide	6846	6.54	21.29	21.52	2693.47	299.43	4.48	0.76	1.28	0.38	0.11	0.23
1	11787	200 yr	EC low tide	8187	6.54	21.77	22.06	2835.83	303.07	5.12	0.89	1.48	0.49	0.14	0.3
1	11787	200 yr	EC high tide	8187	6.54	21.77	22.06	2835.83	303.07	5.12	0.89	1.48	0.49	0.14	0.3
1	11787	200 yr	PC low tide	8187	6.54	21.77	22.06	2836.76	303.1	5.12	0.89	1.48	0.49	0.14	0.3
1	11787	200 yr	PC high tide	8187	6.54	21.77	22.06	2836.76	303.1	5.12	0.89	1.48	0.49	0.14	0.3
1	11787	500 yr	EC low tide	10203	6.54	22.32	22.73	3006.33	307.63	6.06	1.07	1.78	0.68	0.2	0.43
1	11787	500 yr	EC high tide	10203	6.54	22.32	22.73	3006.33	307.63	6.06	1.07	1.78	0.68	0.2	0.43
1	11787	500 yr	PC low tide	10203	6.54	22.31	22.73	3003.56	307.56	6.07	1.07	1.78	0.69	0.2	0.43
1	11787	500 yr	PC high tide	10203	6.54	22.31	22.73	3003.56	307.56	6.07	1.07	1.78	0.69	0.2	0.43
1	11787	5% exceedance	EC low tide	1142	6.54	12.48	12.61	408.29	141.16	2.94	0.03	0.2	0.23		0.02
1	11787	5% exceedance	EC high tide	1142	6.54	12.48	12.61	408.29	141.16	2.94	0.03	0.2	0.23		0.02
1	11787	5% exceedance	PC low tide	1142	6.54	12.48	12.61	408.17	141.09	2.94	0.03	0.2	0.23		0.02
1	11787	5% exceedance	PC high tide	1142	6.54	12.48	12.61	408.17	141.09	2.94	0.03	0.2	0.23		0.02
1	11787	50% exceedance	EC low tide	288	6.54	10.39	10.42	226.34	74.09	1.27			0.05		
1	11787	50% exceedance	EC high tide	288	6.54	10.39	10.42	226.34	74.09	1.27			0.05		
1	11787	50% exceedance	PC low tide	288	6.54	9.51	9.56	162.81	70.91	1.77			0.11		
1	11787	50% exceedance	PC high tide	288	6.54	9.51	9.56	162.81	70.91	1.77			0.11		
1	11787	95% exceedance	EC low tide	47	6.54	9.1	9.1	133.89	69.3	0.35			0		
1	11787	95% exceedance	EC high tide	47	6.54	9.1	9.1	133.89	69.3	0.35			0		
1	11787	95% exceedance	PC low tide	47	6.54	8.47	8.47	90.98	66.57	0.52			0.01		
1	11787	95% exceedance	PC high tide	47	6.54	8.47	8.47	90.98	66.57	0.52			0.01		
1	10867.77	2 yr	EC low tide	1439	4.43	12.79	12.82	1206.74	348.12	1.46	0.26	0.14	0.06	0.02	0.01
1	10867.77	2 yr	EC high tide	1439	4.43	12.79	12.82	1206.74	348.12	1.46	0.26	0.14	0.06	0.02	0.01
1	10867.77	2 yr	PC low tide	1439	4.43	12.79	12.82	1206.55	348.11	1.46	0.26	0.14	0.06	0.02	0.01
1	10867.77	2 yr	PC high tide	1439	4.43	12.79	12.82	1206.55	348.11	1.46	0.26	0.14	0.06	0.02	0.01
1	10867.77	10 yr	EC low tide	3316	4.43	15.46	15.52	2886.68	927.05	2.01	0.32	0.3	0.09	0.02	0.02
1	10867.77	10 yr	EC high tide	3316	4.43	15.46	15.52	2886.68	927.05	2.01	0.32	0.3	0.09	0.02	0.02
1	10867.77	10 yr	PC low tide	3316	4.43	15.46	15.52	2886.68	927.05	2.01	0.32	0.3	0.09	0.02	0.02
1	10867.77	10 yr	PC high tide	3316	4.43	15.46	15.52	2886.68	927.05	2.01	0.32	0.3	0.09	0.02	0.02
1	10867.77	25 yr	EC low tide	4569	4.43	16.96	17.01	4274.1	931.18	2.06	0.43	0.36	0.09	0.03	0.03
1	10867.77	25 yr	EC high tide	4569	4.43	16.96	17.01	4274.1	931.18	2.06	0.43	0.36	0.09	0.03	0.03
1	10867.77	25 yr	PC low tide	4569	4.43	16.96	17.01	4274.1	931.18	2.06	0.43	0.36	0.09	0.03	0.03
1	10867.77	25 yr	PC high tide	4569	4.43	16.96	17.01	4274.1	931.18	2.06	0.43	0.36	0.09	0.03	0.03
1	10867.77	50 yr	EC low tide	5644	4.43	18.13	18.18	5368.15	934.61	2.1	0.49	0.38	0.09	0.04	0.03
1	10867.77	50 yr	EC high tide	5644	4.43	18.13	18.18	5368.15	934.61	2.1	0.49	0.38	0.09	0.04	0.03
1	10867.77	50 yr	PC low tide	5644	4.43	18.13	18.18	5368.14	934.61	2.1	0.49	0.38	0.09	0.04	0.03
1	10867.77	50 yr	PC high tide	5644	4.43	18.13	18.18	5368.14	934.61	2.1	0.49	0.38	0.09	0.04	0.03
1	10867.77	100 yr	EC low tide	6846	4.43	21.31	21.33	8349.31	942.43	1.68	0.46	0.34	0.05	0.03	0.02
1	10867.77	100 yr	EC high tide	6846	4.43	21.31	21.33	8349.31	942.43	1.68	0.46	0.34	0.05	0.03	0.02
1	10867.77	100 yr	PC low tide	6846	4.43	21.31	21.33	8349.31	942.43	1.68	0.46	0.34	0.05	0.03	0.02
1	10867.77	100 yr	PC high tide	6846	4.43	21.31	21.33	8349.31	942.43	1.68	0.46	0.34	0.05	0.03	0.02
1	10867.77	200 yr	EC low tide	8187	4.43	21.79	21.83	8806.06	943.7	1.91	0.53	0.39	0.07	0.04	0.03
1	10867.77	200 yr	EC high tide	8187	4.43	21.79	21.83	8806.06	943.7	1.91	0.53	0.39	0.07	0.04	0.03
1	10867.77	200 yr	PC low tide	8187	4.43	21.79	21.83	8808.98	943.71	1.91	0.53	0.39	0.07	0.04	0.03
1	10867.77	200 yr	PC high tide	8187	4.43	21.79	21.83	8808.98	943.71	1.91	0.53	0.39	0.07	0.04	0.03
1	10867.77	500 yr	EC low tide	10203	4.43	22.37	22.42	9353.98	944.92	2.25	0.64	0.47	0.09	0.06	0.04
1	10867.77	500 yr	EC high tide	10203	4.43	22.37	22.42	9353.97	944.92	2.25	0.64	0.47	0.09	0.06	0.04
1	10867.77	500 yr	PC low tide	10203	4.43	22.36	22.41	9345.36	944.9	2.25	0.64	0.47	0.09	0.06	0.04
1	10867.77	500 yr	PC high tide	10203	4.43	22.36	22.41	9345.36	944.9	2.25	0.64	0.47	0.09	0.06	0.04
1	10867.77	5% exceedance	EC low tide	1142	4.43	12.28	12.3	1028.48	342.21	1.3	0.2	0.1	0.05	0.01	0
1	10867.77	5% exceedance	EC high tide	1142	4.43	12.28	12.3	1028.48	342.21	1.3	0.2	0.1	0.05	0.01	0
1	10867.77	5% exceedance	PC low tide	1142	4.43	12.27	12.3	1028.12	342.2	1.3	0.2	0.1	0.05	0.01	0
1	10867.77	5% exceedance	PC high tide	1142	4.43	12.27	12.3	1028.12	342.2	1.3	0.2	0.1	0.05	0.01	0
1	10867.77	50% exceedance	EC low tide	288	4.43	10.33	10.34	529.75	142.73	0.54			0.01		
1	10867.77	50% exceedance	EC high tide	288	4.43	10.33	10.34	529.75	142.73	0.54			0.01		
1	10867.77	50% exceedance	PC low tide	288	4.43	9.39	9.4	409.99	118.34	0.7			0.01		
1	10867.77	50% exceedance	PC high tide	288	4.43	9.39	9.4	409.99	118.34	0.7			0.01		
1	10867.77	95% exceedance	EC low tide	47	4.43	9.1	9.1	375.89	113.3	0.13			0		
1	10867.77	95% exceedance	EC high tide	47	4.43	9.1	9.1	375.89	113.3	0.13			0		
1	10867.77	95% exceedance	PC low tide	47	4.43	8.46	8.46	307.23	103.09	0.15			0		
1	10867.77	95% exceedance	PC high tide	47	4.43	8.46	8.46	307.23	103.09	0.15			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	10689.35	2 yr	EC low tide	1439	5.98	12.33	12.7	291.77	147.4	4.93			0.78		
1	10689.35	2 yr	EC high tide	1439	5.98	12.33	12.7	291.77	147.4	4.93			0.78		
1	10689.35	2 yr	PC low tide	1439	5.98	12.32	12.7	291.7	147.4	4.93			0.78		
1	10689.35	2 yr	PC high tide	1439	5.98	12.32	12.7	291.7	147.4	4.93			0.78		
1	10689.35	10 yr	EC low tide	3316	5.98	14.71	15.37	510.35	149.57	6.5			1.13		
1	10689.35	10 yr	EC high tide	3316	5.98	14.71	15.37	510.35	149.57	6.5			1.13		
1	10689.35	10 yr	PC low tide	3316	5.98	14.71	15.37	510.35	149.57	6.5			1.13		
1	10689.35	10 yr	PC high tide	3316	5.98	14.71	15.37	510.35	149.57	6.5			1.13		
1	10689.35	25 yr	EC low tide	4569	5.98	16.04	16.85	632.92	743.74	7.22			1.3		
1	10689.35	25 yr	EC high tide	4569	5.98	16.04	16.85	632.92	743.74	7.22			1.3		
1	10689.35	25 yr	PC low tide	4569	5.98	16.04	16.85	632.92	743.74	7.22			1.3		
1	10689.35	25 yr	PC high tide	4569	5.98	16.04	16.85	632.92	743.74	7.22			1.3		
1	10689.35	50 yr	EC low tide	5644	5.98	17.09	18.02	729.33	799.73	7.74			1.43		
1	10689.35	50 yr	EC high tide	5644	5.98	17.09	18.02	729.33	799.73	7.74			1.43		
1	10689.35	50 yr	PC low tide	5644	5.98	17.09	18.02	729.33	799.73	7.74			1.43		
1	10689.35	50 yr	PC high tide	5644	5.98	17.09	18.02	729.33	799.73	7.74			1.43		
1	10689.35	100 yr	EC low tide	6846	5.98	20.58	21.24	1051.6	831.93	6.51			0.9		
1	10689.35	100 yr	EC high tide	6846	5.98	20.58	21.24	1051.6	831.93	6.51			0.9		
1	10689.35	100 yr	PC low tide	6846	5.98	20.58	21.24	1051.6	831.93	6.51			0.9		
1	10689.35	100 yr	PC high tide	6846	5.98	20.58	21.24	1051.6	831.93	6.51			0.9		
1	10689.35	200 yr	EC low tide	8187	5.98	20.79	21.7	1070.96	832.64	7.64			1.23		
1	10689.35	200 yr	EC high tide	8187	5.98	20.79	21.7	1070.96	832.64	7.64			1.23		
1	10689.35	200 yr	PC low tide	8187	5.98	20.79	21.7	1071.31	832.65	7.64			1.23		
1	10689.35	200 yr	PC high tide	8187	5.98	20.79	21.7	1071.3	832.65	7.64			1.23		
1	10689.35	500 yr	EC low tide	10203	5.98	20.83	22.23	1074.38	832.75	9.5			1.89		
1	10689.35	500 yr	EC high tide	10203	5.98	20.83	22.23	1074.38	832.75	9.5			1.89		
1	10689.35	500 yr	PC low tide	10203	5.98	20.81	22.22	1073.23	832.71	9.51			1.9		
1	10689.35	500 yr	PC high tide	10203	5.98	20.81	22.22	1073.23	832.71	9.51			1.9		
1	10689.35	5% exceedance	EC low tide	1142	5.98	11.87	12.19	249.82	147.01	4.57			0.7		
1	10689.35	5% exceedance	EC high tide	1142	5.98	11.87	12.19	249.82	147.01	4.57			0.7		
1	10689.35	5% exceedance	PC low tide	1142	5.98	11.86	12.19	249.69	147.01	4.57			0.7		
1	10689.35	5% exceedance	PC high tide	1142	5.98	11.86	12.19	249.69	147.01	4.57			0.7		
1	10689.35	50% exceedance	EC low tide	288	5.98	10.22	10.31	124.07	96.54	2.32			0.19		
1	10689.35	50% exceedance	EC high tide	288	5.98	10.22	10.31	124.07	96.54	2.32			0.19		
1	10689.35	50% exceedance	PC low tide	288	5.98	9.11	9.34	74.87	89.79	3.85			0.57		
1	10689.35	50% exceedance	PC high tide	288	5.98	9.11	9.34	74.87	89.79	3.85			0.57		
1	10689.35	95% exceedance	EC low tide	47	5.98	9.09	9.1	74.14	89.69	0.63			0.02		
1	10689.35	95% exceedance	EC high tide	47	5.98	9.09	9.1	74.14	89.69	0.63			0.02		
1	10689.35	95% exceedance	PC low tide	47	5.98	8.45	8.46	49.27	83.36	0.95			0.04		
1	10689.35	95% exceedance	PC high tide	47	5.98	8.45	8.46	49.27	83.36	0.95			0.04		
1	10657.2						Railroad Bridge								
1	10625.05	2 yr	EC low tide	1439	6.96	11.85	12.2	302.94	165.78	4.75			0.7		
1	10625.05	2 yr	EC high tide	1439	6.96	11.85	12.2	302.94	165.78	4.75			0.7		
1	10625.05	2 yr	PC low tide	1439	6.96	9.75	11.99	119.99	77.14	11.99			5.56		
1	10625.05	2 yr	PC high tide	1439	6.96	9.75	11.99	119.99	77.14	11.99			5.56		
1	10625.05	10 yr	EC low tide	3316	6.96	10.89	14.55	215.97	160.74	15.35			8.14		
1	10625.05	10 yr	EC high tide	3316	6.96	10.89	14.55	215.97	160.74	15.35			8.14		
1	10625.05	10 yr	PC low tide	3316	6.96	10.89	14.55	215.97	160.74	15.35			8.14		
1	10625.05	10 yr	PC high tide	3316	6.96	10.89	14.55	215.97	160.74	15.35			8.14		
1	10625.05	25 yr	EC low tide	4569	6.96	11.46	15.98	267.79	165.24	17.06			9.37		
1	10625.05	25 yr	EC high tide	4569	6.96	11.46	15.98	267.79	165.24	17.06			9.37		
1	10625.05	25 yr	PC low tide	4569	6.96	11.46	15.98	267.79	165.24	17.06			9.37		
1	10625.05	25 yr	PC high tide	4569	6.96	11.46	15.98	267.79	165.24	17.06			9.37		
1	10625.05	50 yr	EC low tide	5644	6.96	11.91	17.11	308.47	165.87	18.3			10.28		
1	10625.05	50 yr	EC high tide	5644	6.96	11.91	17.11	308.47	165.87	18.3			10.28		
1	10625.05	50 yr	PC low tide	5644	6.96	11.91	17.11	308.47	165.87	18.3			10.28		
1	10625.05	50 yr	PC high tide	5644	6.96	11.91	17.11	308.47	165.87	18.3			10.28		
1	10625.05	100 yr	EC low tide	6846	6.96	15.71	17.42	653.15	789.4	10.48			2.63		
1	10625.05	100 yr	EC high tide	6846	6.96	15.71	17.42	653.15	789.4	10.48			2.63		
1	10625.05	100 yr	PC low tide	6846	6.96	15.59	17.36	642.26	787.28	10.66			2.73		
1	10625.05	100 yr	PC high tide	6846	6.96	15.59	17.36	642.26	787.28	10.66			2.73		
1	10625.05	200 yr	EC low tide	8187	6.96	16.94	18.72	764.32	809.11	10.71			2.6		
1	10625.05	200 yr	EC high tide	8187	6.96	16.94	18.72	764.31	809.11	10.71			2.6		
1	10625.05	200 yr	PC low tide	8187	6.96	16.88	18.69	759.31	808.89	10.78			2.64		
1	10625.05	200 yr	PC high tide	8187	6.96	16.88	18.69	759.3	808.89	10.78			2.65		
1	10625.05	500 yr	EC low tide	10203	6.96	19.39	21.05	987.17	826.72	10.34			2.23		
1	10625.05	500 yr	EC high tide	10203	6.96	19.39	21.05	987.17	826.72	10.34			2.23		
1	10625.05	500 yr	PC low tide	10203	6.96	19.36	21.03	984.39	826.72	10.36			2.24		
1	10625.05	500 yr	PC high tide	10203	6.96	19.36	21.03	984.39	826.72	10.36			2.24		
1	10625.05	5% exceedance	EC low tide	1142	6.96	11.41	11.7	262.95	165.16	4.34			0.61		
1	10625.05	5% exceedance	EC high tide	1142	6.96	11.41	11.7	262.95	165.16	4.34			0.61		
1	10625.05	5% exceedance	PC low tide	1142	6.96	9.48	11.48	100.59	69.92	11.35			5.23		
1	10625.05	5% exceedance	PC high tide	1142	6.96	9.48	11.48	100.59	69.92	11.35			5.23		
1	10625.05	50% exceedance	EC low tide	288	6.96	9.75	9.84	119.4	76.92	2.41			0.23		
1	10625.05	50% exceedance	EC high tide	288	6.96	9.75	9.84	119.4	76.92	2.41			0.23		
1	10625.05	50% exceedance	PC low tide	288	6.96	8.34	9.38	35.25	47.63	8.17			3.38		
1	10625.05	50% exceedance	PC high tide	288	6.96	8.34	9.38	35.25	47.63	8.17			3.38		
1	10625.05	95% exceedance	EC low tide	47	6.96	9.05	9.05	73.15	59.16	0.64			0.02		
1	10625.05	95% exceedance	EC high tide	47	6.96	9.05	9.05	73.15	59.16	0.64			0.02		
1	10625.05	95% exceedance	PC low tide	47	6.96	7.87	8.04	14.02	40.37	3.35			0.73		
1	10625.05	95% exceedance	PC high tide	47	6.96	7.87	8.04	14.02	40.37	3.35			0.73		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	10513.99	2 yr	EC low tide	1439	3.45	12.04	12.07	1429.62	449.23	1.34	0.2	0.09	0.04	0.01	0
1	10513.99	2 yr	EC high tide	1439	3.45	12.04	12.07	1429.62	449.23	1.34	0.2	0.09	0.04	0.01	0
1	10513.99	2 yr	PC low tide	1439	3.45	10.94	10.99	987.74	350.39	1.68	0.19		0.07	0.01	
1	10513.99	2 yr	PC high tide	1439	3.45	10.95	10.99	987.94	350.47	1.68	0.19		0.07	0.01	
1	10513.99	10 yr	EC low tide	3316	3.45	14.21	14.27	2645.4	733.86	2.12	0.35	0.25	0.1	0.03	0.02
1	10513.99	10 yr	EC high tide	3316	3.45	14.21	14.27	2645.4	733.86	2.12	0.35	0.25	0.1	0.03	0.02
1	10513.99	10 yr	PC low tide	3316	3.45	13.36	13.44	2086.76	591.1	2.41	0.45	0.25	0.13	0.03	0.02
1	10513.99	10 yr	PC high tide	3316	3.45	13.36	13.44	2086.76	591.1	2.41	0.45	0.25	0.13	0.03	0.02
1	10513.99	25 yr	EC low tide	4569	3.45	15.2	15.27	3374.69	743.5	2.45	0.48	0.32	0.13	0.04	0.02
1	10513.99	25 yr	EC high tide	4569	3.45	15.2	15.27	3374.69	743.5	2.45	0.48	0.32	0.13	0.04	0.02
1	10513.99	25 yr	PC low tide	4569	3.45	14.62	14.72	2947.64	738.19	2.71	0.48	0.33	0.16	0.05	0.03
1	10513.99	25 yr	PC high tide	4569	3.45	14.62	14.72	2947.65	738.19	2.71	0.48	0.33	0.16	0.05	0.03
1	10513.99	50 yr	EC low tide	5644	3.45	15.9	15.99	3899.37	748.6	2.7	0.57	0.37	0.15	0.06	0.03
1	10513.99	50 yr	EC high tide	5644	3.45	15.9	15.99	3899.37	748.6	2.7	0.57	0.37	0.15	0.06	0.03
1	10513.99	50 yr	PC low tide	5644	3.45	15.63	15.73	3696.91	746.73	2.82	0.58	0.38	0.17	0.06	0.03
1	10513.99	50 yr	PC high tide	5644	3.45	15.63	15.73	3696.91	746.74	2.82	0.58	0.38	0.17	0.06	0.03
1	10513.99	100 yr	EC low tide	6846	3.45	16.76	16.85	4542.77	752.85	2.88	0.65	0.42	0.17	0.07	0.04
1	10513.99	100 yr	EC high tide	6846	3.45	16.76	16.85	4542.77	752.85	2.88	0.65	0.42	0.17	0.07	0.04
1	10513.99	100 yr	PC low tide	6846	3.45	16.67	16.77	4479.28	752.43	2.91	0.66	0.42	0.17	0.07	0.04
1	10513.99	100 yr	PC high tide	6846	3.45	16.67	16.77	4479.28	752.43	2.91	0.66	0.42	0.17	0.07	0.04
1	10513.99	200 yr	EC low tide	8187	3.45	18.05	18.14	5519.69	760.94	2.9	0.72	0.44	0.16	0.08	0.04
1	10513.99	200 yr	EC high tide	8187	3.45	18.05	18.14	5519.66	760.94	2.9	0.72	0.44	0.16	0.08	0.04
1	10513.99	200 yr	PC low tide	8187	3.45	18.01	18.1	5488.83	760.38	2.91	0.72	0.45	0.17	0.08	0.04
1	10513.99	200 yr	PC high tide	8187	3.45	18.01	18.1	5488.81	760.38	2.91	0.72	0.45	0.17	0.08	0.04
1	10513.99	500 yr	EC low tide	10203	3.45	20.45	20.53	7472.31	839.57	2.8	0.72	0.44	0.14	0.08	0.04
1	10513.99	500 yr	EC high tide	10203	3.45	20.45	20.53	7472.31	839.57	2.8	0.72	0.44	0.14	0.08	0.04
1	10513.99	500 yr	PC low tide	10203	3.45	20.42	20.5	7451.43	839.47	2.8	0.72	0.44	0.15	0.08	0.04
1	10513.99	500 yr	PC high tide	10203	3.45	20.42	20.5	7451.43	839.47	2.8	0.72	0.44	0.15	0.08	0.04
1	10513.99	5% exceedance	EC low tide	1142	3.45	11.57	11.59	1227.67	410.99	1.17	0.16	0.06	0.03	0.01	0
1	10513.99	5% exceedance	EC high tide	1142	3.45	11.57	11.59	1227.67	410.99	1.17	0.16	0.06	0.03	0.01	0
1	10513.99	5% exceedance	PC low tide	1142	3.45	10.43	10.47	819.61	311.75	1.5	0.11		0.06	0	
1	10513.99	5% exceedance	PC high tide	1142	3.45	10.43	10.47	820.43	311.9	1.49	0.11		0.06	0	
1	10513.99	50% exceedance	EC low tide	288	3.45	9.8	9.8	663.62	148.48	0.43			0.01		
1	10513.99	50% exceedance	EC high tide	288	3.45	9.8	9.8	663.62	148.48	0.43			0.01		
1	10513.99	50% exceedance	PC low tide	288	3.45	8.27	8.28	451.37	131.12	0.64			0.01		
1	10513.99	50% exceedance	PC high tide	288	3.45	8.27	8.28	451.08	131.11	0.64			0.01		
1	10513.99	95% exceedance	EC low tide	47	3.45	9.05	9.05	556.14	139.19	0.08			0		
1	10513.99	95% exceedance	EC high tide	47	3.45	9.05	9.05	556.14	139.19	0.08			0		
1	10513.99	95% exceedance	PC low tide	47	3.45	7.29	7.29	324.47	125.67	0.14			0		
1	10513.99	95% exceedance	PC high tide	47	3.45	7.24	7.24	318.74	125.37	0.15			0		
1	9865.13	2 yr	EC low tide	1439	2.57	11.99	12	1487.53	274.66	0.97			0.02		
1	9865.13	2 yr	EC high tide	1439	2.57	11.99	12	1487.53	274.66	0.97			0.02		
1	9865.13	2 yr	PC low tide	1439	2.57	10.84	10.86	1175.6	267.8	1.22			0.04		
1	9865.13	2 yr	PC high tide	1439	2.57	10.84	10.86	1175.76	267.81	1.22			0.04		
1	9865.13	10 yr	EC low tide	3316	2.57	14.13	14.17	2454.26	579.97	1.56	0.19	0.15	0.06	0.01	0.01
1	9865.13	10 yr	EC high tide	3316	2.57	14.13	14.17	2454.26	579.97	1.56	0.19	0.15	0.06	0.01	0.01
1	9865.13	10 yr	PC low tide	3316	2.57	13.23	13.29	1835.25	287.34	1.81	0.11	0.11	0.08	0	0
1	9865.13	10 yr	PC high tide	3316	2.57	13.23	13.29	1835.25	287.34	1.81	0.11	0.11	0.08	0	0
1	9865.13	25 yr	EC low tide	4569	2.57	15.1	15.15	3019.39	584.55	1.86	0.31	0.22	0.08	0.02	0.01
1	9865.13	25 yr	EC high tide	4569	2.57	15.1	15.15	3019.39	584.55	1.86	0.31	0.22	0.08	0.02	0.01
1	9865.13	25 yr	PC low tide	4569	2.57	14.49	14.55	2665.36	581.75	2.03	0.29	0.21	0.09	0.02	0.01
1	9865.13	25 yr	PC high tide	4569	2.57	14.49	14.55	2665.36	581.75	2.03	0.29	0.21	0.09	0.02	0.01
1	9865.13	50 yr	EC low tide	5644	2.57	15.79	15.85	3423.85	588	2.09	0.39	0.27	0.09	0.03	0.02
1	9865.13	50 yr	EC high tide	5644	2.57	15.79	15.85	3423.84	588	2.09	0.39	0.27	0.09	0.03	0.02
1	9865.13	50 yr	PC low tide	5644	2.57	15.5	15.57	3256.76	586.48	2.17	0.39	0.27	0.1	0.03	0.02
1	9865.13	50 yr	PC high tide	5644	2.57	15.5	15.57	3256.77	586.48	2.17	0.39	0.27	0.1	0.03	0.02
1	9865.13	100 yr	EC low tide	6846	2.57	16.64	16.71	3927.4	597.87	2.28	0.47	0.32	0.11	0.04	0.02
1	9865.13	100 yr	EC high tide	6846	2.57	16.64	16.71	3927.4	597.87	2.28	0.47	0.32	0.11	0.04	0.02
1	9865.13	100 yr	PC low tide	6846	2.57	16.55	16.63	3874.73	596.42	2.3	0.47	0.32	0.11	0.04	0.02
1	9865.13	100 yr	PC high tide	6846	2.57	16.55	16.63	3874.72	596.42	2.3	0.47	0.32	0.11	0.04	0.02
1	9865.13	200 yr	EC low tide	8187	2.57	17.94	18.02	4716.68	609.32	2.34	0.54	0.36	0.11	0.05	0.03
1	9865.13	200 yr	EC high tide	8187	2.57	17.94	18.02	4716.66	609.32	2.34	0.54	0.36	0.11	0.05	0.03
1	9865.13	200 yr	PC low tide	8187	2.57	17.9	17.98	4691.06	609.23	2.35	0.54	0.36	0.11	0.05	0.03
1	9865.13	200 yr	PC high tide	8187	2.57	17.9	17.98	4691.04	609.23	2.35	0.54	0.36	0.11	0.05	0.03
1	9865.13	500 yr	EC low tide	10203	2.57	20.37	20.44	6205.23	620.34	2.31	0.61	0.33	0.1	0.05	0.02
1	9865.13	500 yr	EC high tide	10203	2.57	20.37	20.44	6205.22	620.34	2.31	0.61	0.33	0.1	0.05	0.02
1	9865.13	500 yr	PC low tide	10203	2.57	20.34	20.41	6189.47	620.18	2.31	0.61	0.33	0.1	0.05	0.02
1	9865.13	500 yr	PC high tide	10203	2.57	20.34	20.41	6189.47	620.18	2.31	0.61	0.33	0.1	0.05	0.02
1	9865.13	5% exceedance	EC low tide	1142	2.57	11.53	11.54	1360.59	272.17	0.84			0.02		
1	9865.13	5% exceedance	EC high tide	1142	2.57	11.53	11.54	1360.59	272.17	0.84			0.02		
1	9865.13	5% exceedance	PC low tide	1142	2.57	10.33	10.35	1040.48	266.51	1.1			0.03		
1	9865.13	5% exceedance	PC high tide	1142	2.57	10.34	10.36	1041.24	266.52	1.1			0.03		
1	9865.13	50% exceedance	EC low tide	288	2.57	9.79	9.79	897.96	246.08	0.32			0		
1	9865.13	50% exceedance	EC high tide	288	2.57	9.79	9.79	897.96	246.08	0.32			0		
1	9865.13	50% exceedance	PC low tide	288	2.57	8.25	8.25	604.18	167.54	0.48			0.01		
1	9865.13	50% exceedance	PC high tide	288	2.57	8.25	8.25	603.8	167.52	0.48			0.01		
1	9865.13	95% exceedance	EC low tide	47	2.57	9.05	9.05	741.57	175.42	0.06			0		
1	9865.13	95% exceedance	EC high tide	47	2.57	9.05	9.05	741.57	175.42	0.06			0		
1	9865.13	95% exceedance	PC low tide	47	2.57	7.29	7.29	446.42	160.42	0.11			0		
1	9865.13	95% exceedance	PC high tide	47	2.57	7.24	7.24	439.08	160.02	0.11			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	9283.72	2 yr	EC low tide	1439	5.37	11.89	11.93	926.94	210.59	1.57	0.19	0.16	0.07	0.01	0.01
1	9283.72	2 yr	EC high tide	1439	5.37	11.89	11.93	926.94	210.59	1.57	0.19	0.16	0.07	0.01	0.01
1	9283.72	2 yr	PC low tide	1439	5.37	10.62	10.69	665.99	199.18	2.16	0.07		0.14	0	
1	9283.72	2 yr	PC high tide	1439	5.37	10.62	10.69	666.14	199.19	2.16	0.07		0.14	0	
1	9283.72	10 yr	EC low tide	3316	5.37	13.96	14.05	1540.04	380.18	2.45	0.3	0.38	0.15	0.03	0.04
1	9283.72	10 yr	EC high tide	3316	5.37	13.96	14.05	1540.04	380.18	2.45	0.3	0.38	0.15	0.03	0.04
1	9283.72	10 yr	PC low tide	3316	5.37	12.96	13.09	1156.82	222.68	2.93	0.41	0.4	0.22	0.05	0.04
1	9283.72	10 yr	PC high tide	3316	5.37	12.96	13.09	1156.82	222.68	2.93	0.41	0.4	0.22	0.05	0.04
1	9283.72	25 yr	EC low tide	4569	5.37	14.88	15.01	1894.48	383.49	2.9	0.49	0.49	0.19	0.05	0.05
1	9283.72	25 yr	EC high tide	4569	5.37	14.88	15.01	1894.48	383.49	2.9	0.49	0.49	0.19	0.05	0.05
1	9283.72	25 yr	PC low tide	4569	5.37	14.2	14.36	1634.79	381.09	3.24	0.45	0.51	0.25	0.05	0.06
1	9283.72	25 yr	PC high tide	4569	5.37	14.2	14.36	1634.79	381.09	3.24	0.45	0.51	0.25	0.05	0.06
1	9283.72	50 yr	EC low tide	5644	5.37	15.54	15.69	2146.05	385.73	3.25	0.62	0.57	0.24	0.08	0.07
1	9283.72	50 yr	EC high tide	5644	5.37	15.54	15.69	2146.05	385.73	3.25	0.62	0.57	0.24	0.08	0.07
1	9283.72	50 yr	PC low tide	5644	5.37	15.22	15.39	2023.84	384.65	3.41	0.62	0.59	0.26	0.08	0.07
1	9283.72	50 yr	PC high tide	5644	5.37	15.22	15.39	2023.85	384.65	3.41	0.62	0.59	0.26	0.08	0.07
1	9283.72	100 yr	EC low tide	6846	5.37	16.37	16.55	2467.57	388.65	3.52	0.75	0.64	0.27	0.11	0.08
1	9283.72	100 yr	EC high tide	6846	5.37	16.37	16.55	2467.57	388.65	3.52	0.75	0.64	0.27	0.11	0.08
1	9283.72	100 yr	PC low tide	6846	5.37	16.27	16.46	2429.68	388.3	3.57	0.75	0.65	0.28	0.11	0.09
1	9283.72	100 yr	PC high tide	6846	5.37	16.27	16.46	2429.67	388.3	3.57	0.75	0.65	0.28	0.11	0.09
1	9283.72	200 yr	EC low tide	8187	5.37	17.69	17.87	2983.93	393.31	3.59	0.86	0.69	0.27	0.12	0.09
1	9283.72	200 yr	EC high tide	8187	5.37	17.69	17.87	2983.92	393.31	3.59	0.86	0.69	0.27	0.12	0.09
1	9283.72	200 yr	PC low tide	8187	5.37	17.64	17.83	2966.04	393.15	3.61	0.86	0.69	0.27	0.13	0.09
1	9283.72	200 yr	PC high tide	8187	5.37	17.64	17.83	2966.02	393.15	3.61	0.86	0.69	0.27	0.13	0.09
1	9283.72	500 yr	EC low tide	10203	5.37	20.16	20.33	3967.1	401.07	3.49	0.95	0.72	0.24	0.13	0.09
1	9283.72	500 yr	EC high tide	10203	5.37	20.16	20.33	3967.1	401.07	3.49	0.95	0.72	0.24	0.13	0.09
1	9283.72	500 yr	PC low tide	10203	5.37	20.14	20.3	3956.4	401	3.5	0.95	0.72	0.24	0.13	0.09
1	9283.72	500 yr	PC high tide	10203	5.37	20.14	20.3	3956.39	401	3.5	0.95	0.72	0.24	0.13	0.09
1	9283.72	5% exceedance	EC low tide	1142	5.37	11.45	11.47	833.45	206.41	1.38	0.14	0.11	0.05	0.01	0.01
1	9283.72	5% exceedance	EC high tide	1142	5.37	11.45	11.47	833.45	206.41	1.38	0.14	0.11	0.05	0.01	0.01
1	9283.72	5% exceedance	PC low tide	1142	5.37	10.12	10.19	567.78	197.55	2.01			0.13		
1	9283.72	5% exceedance	PC high tide	1142	5.37	10.13	10.19	568.46	197.55	2.01			0.13		
1	9283.72	50% exceedance	EC low tide	288	5.37	9.77	9.78	498.13	197.38	0.58			0.01		
1	9283.72	50% exceedance	EC high tide	288	5.37	9.77	9.78	498.13	197.38	0.58			0.01		
1	9283.72	50% exceedance	PC low tide	288	5.37	8.17	8.2	202.39	156.28	1.42			0.08		
1	9283.72	50% exceedance	PC high tide	288	5.37	8.16	8.2	201.99	156.18	1.43			0.08		
1	9283.72	95% exceedance	EC low tide	47	5.37	9.05	9.05	356.19	197.03	0.13			0		
1	9283.72	95% exceedance	EC high tide	47	5.37	9.05	9.05	356.19	197.03	0.13			0		
1	9283.72	95% exceedance	PC low tide	47	5.37	7.28	7.28	77.59	125.79	0.61			0.02		
1	9283.72	95% exceedance	PC high tide	47	5.37	7.23	7.24	71.68	124.61	0.66			0.02		
1	7408.49	2 yr	EC low tide	1439	3.53	11.59	11.61	1234.49	338.5	1.33	0.19	0.05	0.04	0.01	0
1	7408.49	2 yr	EC high tide	1439	3.53	11.59	11.61	1234.49	338.5	1.33	0.19	0.05	0.04	0.01	0
1	7408.49	2 yr	PC low tide	1439	3.53	9.43	9.51	666.67	176.52	2.16			0.14		
1	7408.49	2 yr	PC high tide	1439	3.53	9.44	9.51	667.3	176.54	2.16			0.14		
1	7408.49	10 yr	EC low tide	3316	3.53	13.45	13.51	2324.94	728.6	2.16	0.32	0.25	0.11	0.02	0.02
1	7408.49	10 yr	EC high tide	3316	3.53	13.45	13.51	2324.94	728.6	2.16	0.32	0.25	0.11	0.02	0.02
1	7408.49	10 yr	PC low tide	3316	3.53	11.84	11.97	1321.41	354.35	2.92	0.43	0.16	0.21	0.05	0.01
1	7408.49	10 yr	PC high tide	3316	3.53	11.84	11.97	1321.41	354.35	2.92	0.43	0.16	0.21	0.05	0.01
1	7408.49	25 yr	EC low tide	4569	3.53	14.28	14.37	2938.31	737.59	2.54	0.46	0.33	0.14	0.04	0.03
1	7408.49	25 yr	EC high tide	4569	3.53	14.28	14.37	2938.31	737.59	2.54	0.46	0.33	0.14	0.04	0.03
1	7408.49	25 yr	PC low tide	4569	3.53	13.2	13.35	2148.14	725.83	3.13	0.43	0.34	0.23	0.05	0.03
1	7408.49	25 yr	PC high tide	4569	3.53	13.2	13.35	2148.14	725.83	3.13	0.43	0.34	0.23	0.05	0.03
1	7408.49	50 yr	EC low tide	5644	3.53	14.86	14.97	3367.05	743.96	2.84	0.56	0.39	0.18	0.06	0.04
1	7408.49	50 yr	EC high tide	5644	3.53	14.86	14.97	3367.05	743.96	2.84	0.56	0.39	0.18	0.06	0.04
1	7408.49	50 yr	PC low tide	5644	3.53	14.39	14.52	3015.17	738.75	3.08	0.56	0.41	0.21	0.07	0.04
1	7408.49	50 yr	PC high tide	5644	3.53	14.39	14.52	3015.18	738.75	3.08	0.56	0.41	0.21	0.07	0.04
1	7408.49	100 yr	EC low tide	6846	3.53	15.69	15.81	3989	752.69	3.01	0.65	0.45	0.19	0.08	0.04
1	7408.49	100 yr	EC high tide	6846	3.53	15.69	15.81	3989	752.69	3.01	0.65	0.45	0.19	0.08	0.04
1	7408.49	100 yr	PC low tide	6846	3.53	15.56	15.68	3886.69	751.27	3.07	0.66	0.46	0.2	0.08	0.05
1	7408.49	100 yr	PC high tide	6846	3.53	15.56	15.68	3886.68	751.27	3.07	0.66	0.46	0.2	0.08	0.05
1	7408.49	200 yr	EC low tide	8187	3.53	17.14	17.24	5108.07	837.77	2.98	0.67	0.51	0.18	0.08	0.05
1	7408.49	200 yr	EC high tide	8187	3.53	17.14	17.24	5108.03	837.77	2.98	0.67	0.51	0.18	0.08	0.05
1	7408.49	200 yr	PC low tide	8187	3.53	17.08	17.19	5059.82	837.17	3	0.67	0.51	0.18	0.08	0.05
1	7408.49	200 yr	PC high tide	8187	3.53	17.08	17.19	5059.78	837.17	3	0.67	0.51	0.18	0.08	0.05
1	7408.49	500 yr	EC low tide	10203	3.53	19.83	19.91	7411.44	870.18	2.68	0.7	0.5	0.14	0.07	0.04
1	7408.49	500 yr	EC high tide	10203	3.53	19.83	19.91	7411.44	870.18	2.68	0.7	0.5	0.14	0.07	0.04
1	7408.49	500 yr	PC low tide	10203	3.53	19.8	19.88	7385.46	869.89	2.69	0.7	0.5	0.14	0.07	0.04
1	7408.49	500 yr	PC high tide	10203	3.53	19.8	19.88	7385.46	869.89	2.69	0.7	0.5	0.14	0.07	0.04
1	7408.49	5% exceedance	EC low tide	1142	3.53	11.19	11.21	1103.32	317.31	1.14	0.14	0.03	0.03	0.01	0
1	7408.49	5% exceedance	EC high tide	1142	3.53	11.19	11.21	1103.32	317.31	1.14	0.14	0.03	0.03	0.01	0
1	7408.49	5% exceedance	PC low tide	1142	3.53	8.9	8.96	573.77	173.11	1.99			0.12		
1	7408.49	5% exceedance	PC high tide	1142	3.53	8.92	8.98	576.35	173.2	1.98			0.12		
1	7408.49	50% exceedance	EC low tide	288	3.53	9.71	9.71	716.82	194.91	0.4	0.01		0	0	
1	7408.49	50% exceedance	EC high tide	288	3.53	9.71	9.71	716.82	194.91	0.4	0.01		0	0	
1	7408.49	50% exceedance	PC low tide	288	3.53	6.16	6.21	162.44	91.34	1.77			0.12		
1	7408.49	50% exceedance	PC high tide	288	3.53	6.18	6.22	163.71	91.79	1.76			0.12		
1	7408.49	95% exceedance	EC low tide	47	3.53	9.05	9.05	598.99	174.04	0.08			0		
1	7408.49	95% exceedance	EC high tide	47	3.53	9.05	9.05	598.99	174.04	0.08			0		
1	7408.49	95% exceedance	PC low tide	47	3.53	4.05	4.23	14.01	40.9	3.35			0.73		
1	7408.49	95% exceedance	PC high tide	47	3.53	4.28	4.34	25.26	55.84	1.86			0.2		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	5359.18	2 yr	EC low tide	1439	1.05	11.44	11.45	1758.4	418.79	0.87	0.07	0.09	0.02	0	0
1	5359.18	2 yr	EC high tide	1439	1.05	11.44	11.45	1758.4	418.79	0.87	0.07	0.09	0.02	0	0
1	5359.18	2 yr	PC low tide	1439	1.05	8.26	8.31	776.1	259.79	1.85			0.11		
1	5359.18	2 yr	PC high tide	1439	1.05	8.27	8.32	779.78	260.04	1.85			0.11		
1	5359.18	10 yr	EC low tide	3316	1.05	13.15	13.19	2523.05	458.35	1.52	0.19	0.27	0.05	0.01	0.02
1	5359.18	10 yr	EC high tide	3316	1.05	13.15	13.19	2523.05	458.35	1.52	0.19	0.27	0.05	0.01	0.02
1	5359.18	10 yr	PC low tide	3316	1.05	10.96	11.03	1568.28	326.58	2.18	0.12	0.3	0.12	0.01	0.03
1	5359.18	10 yr	PC high tide	3316	1.05	10.96	11.03	1568.28	326.58	2.18	0.12	0.3	0.12	0.01	0.03
1	5359.18	25 yr	EC low tide	4569	1.05	13.9	13.95	2867.42	463.59	1.88	0.25	0.38	0.08	0.02	0.03
1	5359.18	25 yr	EC high tide	4569	1.05	13.9	13.95	2867.42	463.59	1.88	0.25	0.38	0.08	0.02	0.03
1	5359.18	25 yr	PC low tide	4569	1.05	12.48	12.56	2214.03	453.87	2.32	0.25	0.35	0.13	0.02	0.03
1	5359.18	25 yr	PC high tide	4569	1.05	12.48	12.56	2214.03	453.87	2.32	0.25	0.35	0.13	0.02	0.03
1	5359.18	50 yr	EC low tide	5644	1.05	14.4	14.47	3100.03	467.4	2.18	0.31	0.46	0.1	0.02	0.04
1	5359.18	50 yr	EC high tide	5644	1.05	14.4	14.47	3100.03	467.4	2.18	0.31	0.46	0.1	0.02	0.04
1	5359.18	50 yr	PC low tide	5644	1.05	13.8	13.88	2819.27	462.78	2.36	0.31	0.46	0.12	0.02	0.04
1	5359.18	50 yr	PC high tide	5644	1.05	13.8	13.88	2819.28	462.78	2.36	0.31	0.46	0.12	0.02	0.04
1	5359.18	100 yr	EC low tide	6846	1.05	15.21	15.29	3479.52	473.17	2.4	0.36	0.54	0.12	0.03	0.05
1	5359.18	100 yr	EC high tide	6846	1.05	15.21	15.29	3479.51	473.17	2.4	0.36	0.54	0.12	0.03	0.05
1	5359.18	100 yr	PC low tide	6846	1.05	15.04	15.13	3400.68	471.96	2.44	0.36	0.55	0.13	0.03	0.05
1	5359.18	100 yr	PC high tide	6846	1.05	15.04	15.13	3400.68	471.96	2.44	0.36	0.55	0.13	0.03	0.05
1	5359.18	200 yr	EC low tide	8187	1.05	16.73	16.82	4209.6	487.14	2.43	0.4	0.6	0.12	0.03	0.06
1	5359.18	200 yr	EC high tide	8187	1.05	16.73	16.82	4209.57	487.14	2.43	0.4	0.6	0.12	0.03	0.06
1	5359.18	200 yr	PC low tide	8187	1.05	16.66	16.75	4177.26	486.47	2.45	0.4	0.6	0.12	0.03	0.06
1	5359.18	200 yr	PC high tide	8187	1.05	16.66	16.75	4177.23	486.47	2.45	0.4	0.6	0.12	0.03	0.06
1	5359.18	500 yr	EC low tide	10203	1.05	19.55	19.63	5638.36	543.7	2.37	0.39	0.59	0.11	0.03	0.05
1	5359.18	500 yr	EC high tide	10203	1.05	19.55	19.63	5638.36	543.7	2.37	0.39	0.59	0.11	0.03	0.05
1	5359.18	500 yr	PC low tide	10203	1.05	19.52	19.6	5621.05	538.79	2.38	0.39	0.6	0.11	0.03	0.05
1	5359.18	500 yr	PC high tide	10203	1.05	19.52	19.6	5621.05	538.79	2.38	0.39	0.6	0.11	0.03	0.05
1	5359.18	5% exceedance	EC low tide	1142	1.05	11.07	11.08	1608.2	383.88	0.74	0.04	0.08	0.01	0	0
1	5359.18	5% exceedance	EC high tide	1142	1.05	11.07	11.08	1608.2	383.88	0.74	0.04	0.08	0.01	0	0
1	5359.18	5% exceedance	PC low tide	1142	1.05	7.65	7.7	623.92	229.66	1.83			0.11		
1	5359.18	5% exceedance	PC high tide	1142	1.05	7.69	7.74	632.39	234.1	1.81			0.11		
1	5359.18	50% exceedance	EC low tide	288	1.05	9.69	9.69	1173.29	297.31	0.25		0.02	0		0
1	5359.18	50% exceedance	EC high tide	288	1.05	9.69	9.69	1173.29	297.31	0.25		0.02	0		0
1	5359.18	50% exceedance	PC low tide	288	1.05	4.78	4.81	195.13	77.72	1.48			0.07		
1	5359.18	50% exceedance	PC high tide	288	1.05	5	5.03	212.75	80.55	1.35			0.06		
1	5359.18	95% exceedance	EC low tide	47	1.05	9.05	9.05	987.57	279.06	0.05		0	0		0
1	5359.18	95% exceedance	EC high tide	47	1.05	9.05	9.05	987.57	279.06	0.05		0	0		0
1	5359.18	95% exceedance	PC low tide	47	1.05	3	3	82.33	51.37	0.57			0.01		
1	5359.18	95% exceedance	PC high tide	47	1.05	4.14	4.14	149.1	66.07	0.32			0		
1	3900	2 yr	EC low tide	1439	0.62	11.34	11.36	1728.58	392.47	1.32	0.29	0.25	0.04	0.02	0.01
1	3900	2 yr	EC high tide	1439	0.62	11.34	11.36	1728.58	392.47	1.32	0.29	0.25	0.04	0.02	0.01
1	3900	2 yr	PC low tide	1439	0.62	7.07	7.22	468.31	159.79	3.1	0.08		0.27	0	
1	3900	2 yr	PC high tide	1439	0.62	7.11	7.26	474.22	160.16	3.07	0.1		0.26	0.01	
1	3900	10 yr	EC low tide	3316	0.62	12.9	12.97	2356.1	412.57	2.38	0.56	0.53	0.12	0.06	0.05
1	3900	10 yr	EC high tide	3316	0.62	12.9	12.97	2356.1	412.57	2.38	0.56	0.53	0.12	0.06	0.05
1	3900	10 yr	PC low tide	3316	0.62	10.05	10.26	1235.2	373.17	3.84	0.8	0.52	0.35	0.13	0.07
1	3900	10 yr	PC high tide	3316	0.62	10.05	10.26	1235.2	373.17	3.84	0.8	0.52	0.35	0.13	0.07
1	3900	25 yr	EC low tide	4569	0.62	13.54	13.65	2621.79	420.34	3	0.73	0.7	0.19	0.09	0.08
1	3900	25 yr	EC high tide	4569	0.62	13.54	13.65	2621.79	420.34	3	0.73	0.7	0.19	0.09	0.08
1	3900	25 yr	PC low tide	4569	0.62	11.74	11.94	1886.8	396.18	3.91	0.88	0.78	0.34	0.14	0.12
1	3900	25 yr	PC high tide	4569	0.62	11.74	11.94	1886.8	396.18	3.91	0.88	0.78	0.34	0.14	0.12
1	3900	50 yr	EC low tide	5644	0.62	13.94	14.08	2788.93	424.19	3.52	0.87	0.84	0.26	0.12	0.12
1	3900	50 yr	EC high tide	5644	0.62	13.94	14.08	2788.93	424.19	3.52	0.87	0.84	0.26	0.12	0.12
1	3900	50 yr	PC low tide	5644	0.62	13.19	13.37	2474.11	416.28	3.89	0.94	0.89	0.32	0.15	0.14
1	3900	50 yr	PC high tide	5644	0.62	13.19	13.37	2474.12	416.28	3.89	0.94	0.89	0.32	0.15	0.14
1	3900	100 yr	EC low tide	6846	0.62	14.69	14.87	3111.57	429.08	3.88	0.99	0.97	0.3	0.16	0.15
1	3900	100 yr	EC high tide	6846	0.62	14.69	14.87	3111.57	429.08	3.88	0.99	0.97	0.3	0.16	0.15
1	3900	100 yr	PC low tide	6846	0.62	14.49	14.68	3024.79	427.84	3.98	1.01	0.98	0.32	0.16	0.16
1	3900	100 yr	PC high tide	6846	0.62	14.49	14.68	3024.78	427.84	3.98	1.01	0.98	0.32	0.16	0.16
1	3900	200 yr	EC low tide	8187	0.62	16.29	16.46	3802.98	437.93	3.88	1.05	1.04	0.29	0.16	0.16
1	3900	200 yr	EC high tide	8187	0.62	16.29	16.46	3802.95	437.93	3.88	1.05	1.04	0.29	0.16	0.16
1	3900	200 yr	PC low tide	8187	0.62	16.21	16.38	3769.56	437.53	3.91	1.05	1.04	0.3	0.16	0.16
1	3900	200 yr	PC high tide	8187	0.62	16.21	16.38	3769.53	437.53	3.91	1.05	1.04	0.3	0.16	0.16
1	3900	500 yr	EC low tide	10203	0.62	19.24	19.38	5289.16	582.92	3.69	0.96	1.04	0.25	0.09	0.15
1	3900	500 yr	EC high tide	10203	0.62	19.24	19.38	5289.15	582.92	3.69	0.96	1.04	0.25	0.09	0.15
1	3900	500 yr	PC low tide	10203	0.62	19.2	19.35	5269.08	577.07	3.7	0.96	1.04	0.25	0.09	0.15
1	3900	500 yr	PC high tide	10203	0.62	19.2	19.35	5269.08	577.07	3.7	0.96	1.04	0.25	0.09	0.15
1	3900	5% exceedance	EC low tide	1142	0.62	10.99	11.01	1593.21	388.74	1.11	0.24	0.2	0.03	0.01	0.01
1	3900	5% exceedance	EC high tide	1142	0.62	10.99	11.01	1593.21	388.74	1.11	0.24	0.2	0.03	0.01	0.01
1	3900	5% exceedance	PC low tide	1142	0.62	6.45	6.57	401.82	97.77	2.84			0.23		
1	3900	5% exceedance	PC high tide	1142	0.62	6.53	6.65	409.71	98.36	2.79			0.22		
1	3900	50% exceedance	EC low tide	288	0.62	9.68	9.68	1099	368.92	0.36	0.07	0.04	0	0	0
1	3900	50% exceedance	EC high tide	288	0.62	9.68	9.68	1099	368.92	0.36	0.07	0.04	0	0	0
1	3900	50% exceedance	PC low tide	288	0.62	4.08	4.12	193.65	76.9	1.49			0.07		
1	3900	50% exceedance	PC high tide	288	0.62	4.52	4.54	228.02	80.95	1.26			0.05		
1	3900	95% exceedance	EC low tide	47	0.62	9.05	9.05	867.55	361.58	0.07	0.01	0	0	0	0
1	3900	95% exceedance	EC high tide	47	0.62	9.05	9.05	867.55	361.58	0.07	0.01	0	0	0	0
1	3900	95% exceedance	PC low tide	47	0.62	2.87	2.87	109.42	63.72	0.43			0.01		
1	3900	95% exceedance	PC high tide	47	0.62	4.11	4.11	196.17	77.23	0.24			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	3682	2 yr	EC low tide	1439	0.25	11.33	11.35	1971.47	418.98	1.11	0.28	0.19	0.03	0.01	0.01
1	3682	2 yr	EC high tide	1439	0.25	11.33	11.35	1971.47	418.98	1.11	0.28	0.19	0.03	0.01	0.01
1	3682	2 yr	PC low tide	1439	0.25	6.93	7.04	553.95	120.84	2.6			0.18		
1	3682	2 yr	PC high tide	1439	0.25	6.97	7.08	558.89	121.24	2.57			0.18		
1	3682	10 yr	EC low tide	3316	0.25	12.88	12.93	2629.96	430.47	2.03	0.55	0.42	0.09	0.05	0.03
1	3682	10 yr	EC high tide	3316	0.25	12.88	12.93	2629.96	430.47	2.03	0.55	0.42	0.09	0.05	0.03
1	3682	10 yr	PC low tide	3316	0.25	9.94	10.09	1399.25	406.57	3.25	0.71	0.37	0.25	0.1	0.04
1	3682	10 yr	PC high tide	3316	0.25	9.94	10.09	1399.26	406.57	3.25	0.71	0.37	0.25	0.1	0.04
1	3682	25 yr	EC low tide	4569	0.25	13.5	13.59	2901.12	435.67	2.58	0.71	0.57	0.14	0.08	0.06
1	3682	25 yr	EC high tide	4569	0.25	13.5	13.59	2901.12	435.67	2.58	0.71	0.57	0.14	0.08	0.06
1	3682	25 yr	PC low tide	4569	0.25	11.66	11.81	2110.12	421.84	3.34	0.85	0.6	0.25	0.12	0.07
1	3682	25 yr	PC high tide	4569	0.25	11.66	11.81	2110.13	421.84	3.34	0.85	0.6	0.25	0.12	0.07
1	3682	50 yr	EC low tide	5644	0.25	13.89	14	3069.12	438.77	3.04	0.85	0.68	0.19	0.11	0.08
1	3682	50 yr	EC high tide	5644	0.25	13.89	14	3069.11	438.77	3.04	0.85	0.68	0.19	0.11	0.08
1	3682	50 yr	PC low tide	5644	0.25	13.12	13.27	2735.69	432.42	3.35	0.91	0.71	0.23	0.13	0.09
1	3682	50 yr	PC high tide	5644	0.25	13.12	13.27	2735.7	432.42	3.35	0.91	0.71	0.23	0.13	0.09
1	3682	100 yr	EC low tide	6846	0.25	14.64	14.78	3400.57	445.46	3.38	0.95	0.8	0.23	0.14	0.1
1	3682	100 yr	EC high tide	6846	0.25	14.64	14.78	3400.56	445.46	3.38	0.95	0.8	0.23	0.14	0.1
1	3682	100 yr	PC low tide	6846	0.25	14.43	14.58	3308.77	443.61	3.46	0.97	0.81	0.24	0.14	0.11
1	3682	100 yr	PC high tide	6846	0.25	14.43	14.58	3308.77	443.61	3.46	0.97	0.81	0.24	0.14	0.11
1	3682	200 yr	EC low tide	8187	0.25	16.24	16.38	4126.58	461.72	3.43	0.98	0.86	0.22	0.14	0.11
1	3682	200 yr	EC high tide	8187	0.25	16.24	16.38	4126.55	461.72	3.43	0.98	0.86	0.22	0.14	0.11
1	3682	200 yr	PC low tide	8187	0.25	16.16	16.3	4090.87	460.93	3.45	0.99	0.86	0.23	0.14	0.11
1	3682	200 yr	PC high tide	8187	0.25	16.16	16.3	4090.83	460.93	3.45	0.99	0.86	0.23	0.14	0.11
1	3682	500 yr	EC low tide	10203	0.25	19.19	19.33	5585.41	587.06	3.41	0.69	0.91	0.21	0.08	0.11
1	3682	500 yr	EC high tide	10203	0.25	19.19	19.33	5585.41	587.06	3.41	0.69	0.91	0.21	0.08	0.11
1	3682	500 yr	PC low tide	10203	0.25	19.16	19.29	5564.88	586.02	3.42	0.69	0.91	0.21	0.08	0.11
1	3682	500 yr	PC high tide	10203	0.25	19.16	19.29	5564.88	586.01	3.42	0.69	0.91	0.21	0.08	0.11
1	3682	5% exceedance	EC low tide	1142	0.25	10.98	11	1827.78	415.88	0.93	0.23	0.15	0.02	0.01	0
1	3682	5% exceedance	EC high tide	1142	0.25	10.98	11	1827.78	415.88	0.93	0.23	0.15	0.02	0.01	0
1	3682	5% exceedance	PC low tide	1142	0.25	6.31	6.4	481.11	114.7	2.37			0.16		
1	3682	5% exceedance	PC high tide	1142	0.25	6.4	6.49	491.32	115.58	2.32			0.15		
1	3682	50% exceedance	EC low tide	288	0.25	9.68	9.68	1293.39	405.98	0.3	0.06	0.03	0	0	0
1	3682	50% exceedance	EC high tide	288	0.25	9.68	9.68	1293.39	405.98	0.3	0.06	0.03	0	0	0
1	3682	50% exceedance	PC low tide	288	0.25	4.01	4.03	242.59	93.75	1.19			0.05		
1	3682	50% exceedance	PC high tide	288	0.25	4.47	4.49	286.9	97.17	1			0.03		
1	3682	95% exceedance	EC low tide	47	0.25	9.05	9.05	1036.94	404.56	0.05	0.01	0	0	0	0
1	3682	95% exceedance	EC high tide	47	0.25	9.05	9.05	1036.94	404.56	0.05	0.01	0	0	0	0
1	3682	95% exceedance	PC low tide	47	0.25	2.86	2.86	141.26	80.94	0.33			0		
1	3682	95% exceedance	PC high tide	47	0.25	4.11	4.11	252.18	94.54	0.19			0		
1	3469.2	2 yr	EC low tide	1439	0.76	11.31	11.33	1350.79	235.9	1.19	0.77	0.1	0.03	0.02	0
1	3469.2	2 yr	EC high tide	1439	0.76	11.31	11.33	1350.79	235.9	1.19	0.77	0.1	0.03	0.02	0
1	3469.2	2 yr	PC low tide	1439	0.76	6.63	6.82	406.72	105.21	3.54			0.36		
1	3469.2	2 yr	PC high tide	1439	0.76	6.68	6.87	411.99	105.96	3.49			0.35		
1	3469.2	10 yr	EC low tide	3316	0.76	12.85	12.9	2554.38	593.82	2.05	1.08	0.37	0.09	0.04	0.03
1	3469.2	10 yr	EC high tide	3316	0.76	12.85	12.9	2554.38	593.82	2.05	1.08	0.37	0.09	0.04	0.03
1	3469.2	10 yr	PC low tide	3316	0.76	9.75	9.94	997.65	216.4	3.68	2.09		0.34	0.14	
1	3469.2	10 yr	PC high tide	3316	0.76	9.75	9.94	997.65	216.4	3.68	2.09		0.34	0.14	
1	3469.2	25 yr	EC low tide	4569	0.76	13.46	13.54	2931.52	624.46	2.53	1.34	0.49	0.14	0.05	0.05
1	3469.2	25 yr	EC high tide	4569	0.76	13.46	13.54	2931.52	624.46	2.53	1.34	0.49	0.14	0.05	0.05
1	3469.2	25 yr	PC low tide	4569	0.76	11.51	11.69	1397.07	240.2	3.66	2.35	0.36	0.31	0.16	0.04
1	3469.2	25 yr	PC high tide	4569	0.76	11.51	11.69	1397.08	240.2	3.66	2.35	0.36	0.31	0.16	0.04
1	3469.2	50 yr	EC low tide	5644	0.76	13.84	13.94	3168.42	637.18	2.93	1.57	0.59	0.18	0.07	0.07
1	3469.2	50 yr	EC high tide	5644	0.76	13.84	13.94	3168.42	637.18	2.93	1.57	0.59	0.18	0.07	0.07
1	3469.2	50 yr	PC low tide	5644	0.76	13.04	13.18	2671.3	609.82	3.38	1.75	0.63	0.25	0.09	0.08
1	3469.2	50 yr	PC high tide	5644	0.76	13.04	13.18	2671.31	609.82	3.38	1.75	0.63	0.25	0.09	0.08
1	3469.2	100 yr	EC low tide	6846	0.76	14.59	14.7	3658.22	660.24	3.13	1.75	0.67	0.2	0.08	0.08
1	3469.2	100 yr	EC high tide	6846	0.76	14.59	14.7	3658.21	660.24	3.13	1.75	0.67	0.2	0.08	0.08
1	3469.2	100 yr	PC low tide	6846	0.76	14.38	14.5	3518.1	654	3.24	1.79	0.68	0.22	0.09	0.08
1	3469.2	100 yr	PC high tide	6846	0.76	14.38	14.5	3518.1	654	3.24	1.79	0.68	0.22	0.09	0.08
1	3469.2	200 yr	EC low tide	8187	0.76	16.22	16.31	4809.11	743.5	2.98	1.61	0.7	0.17	0.07	0.08
1	3469.2	200 yr	EC high tide	8187	0.76	16.22	16.31	4809.06	743.5	2.98	1.61	0.7	0.17	0.07	0.08
1	3469.2	200 yr	PC low tide	8187	0.76	16.14	16.23	4750.51	741.09	3.01	1.63	0.7	0.18	0.07	0.08
1	3469.2	200 yr	PC high tide	8187	0.76	16.14	16.23	4750.46	741.09	3.01	1.63	0.7	0.18	0.07	0.08
1	3469.2	500 yr	EC low tide	10203	0.76	19.21	19.27	7214.39	890.64	2.59	1.35	0.68	0.12	0.05	0.06
1	3469.2	500 yr	EC high tide	10203	0.76	19.21	19.27	7214.39	890.64	2.59	1.35	0.68	0.12	0.05	0.06
1	3469.2	500 yr	PC low tide	10203	0.76	19.17	19.24	7183.12	888.96	2.6	1.36	0.68	0.12	0.05	0.07
1	3469.2	500 yr	PC high tide	10203	0.76	19.17	19.24	7183.11	888.96	2.6	1.36	0.68	0.12	0.05	0.07
1	3469.2	5% exceedance	EC low tide	1142	0.76	10.97	10.99	1271.51	231.1	0.99	0.64	0.07	0.02	0.01	0
1	3469.2	5% exceedance	EC high tide	1142	0.76	10.97	10.99	1271.51	231.1	0.99	0.64	0.07	0.02	0.01	0
1	3469.2	5% exceedance	PC low tide	1142	0.76	6.03	6.2	346.47	96.52	3.3			0.32		
1	3469.2	5% exceedance	PC high tide	1142	0.76	6.14	6.3	357.01	97.77	3.2			0.3		
1	3469.2	50% exceedance	EC low tide	288	0.76	9.68	9.68	982.68	215.74	0.32	0.18		0	0	
1	3469.2	50% exceedance	EC high tide	288	0.76	9.68	9.68	982.68	215.74	0.32	0.18		0	0	
1	3469.2	50% exceedance	PC low tide	288	0.76	3.89	3.93	164.36	73.74	1.75			0.11		
1	3469.2	50% exceedance	PC high tide	288	0.76	4.4	4.43	203.4	78.11	1.42			0.07		
1	3469.2	95% exceedance	EC low tide	47	0.76	9.05	9.05	848.4	208.8	0.06	0.03		0	0	
1	3469.2	95% exceedance	EC high tide	47	0.76	9.05	9.05	848.4	208.8	0.06	0.03		0	0	
1	3469.2	95% exceedance	PC low tide	47	0.76	2.84	2.85	91.09	66.79	0.52			0.01		
1	3469.2	95% exceedance	PC high tide	47	0.76	4.11	4.11	181.01	75.29	0.26			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	3260	2 yr	EC low tide	1439	0.08	11.3	11.32	1314.22	181.7	1.1	0.17	0.03	0.03	0	0
1	3260	2 yr	EC high tide	1439	0.08	11.3	11.32	1314.22	181.7	1.1	0.17	0.03	0.03	0	0
1	3260	2 yr	PC low tide	1439	0.08	6.48	6.58	570.24	136.92	2.52			0.18		
1	3260	2 yr	PC high tide	1439	0.08	6.54	6.63	577.96	137.14	2.49			0.18		
1	3260	10 yr	EC low tide	3316	0.08	12.8	12.86	1924.58	528.56	2.03	0.51	0.46	0.09	0.01	0.01
1	3260	10 yr	EC high tide	3316	0.08	12.8	12.86	1924.58	528.56	2.03	0.51	0.46	0.09	0.01	0.01
1	3260	10 yr	PC low tide	3316	0.08	9.61	9.77	1035.16	157.77	3.2			0.25		
1	3260	10 yr	PC high tide	3316	0.08	9.61	9.77	1035.16	157.77	3.2			0.25		
1	3260	25 yr	EC low tide	4569	0.08	13.4	13.49	2248.68	556.47	2.51	0.73	0.73	0.13	0.02	0.02
1	3260	25 yr	EC high tide	4569	0.08	13.4	13.49	2248.68	556.47	2.51	0.73	0.73	0.13	0.02	0.02
1	3260	25 yr	PC low tide	4569	0.08	11.38	11.56	1328.2	184.03	3.46	0.55	0.15	0.27	0.02	0
1	3260	25 yr	PC high tide	4569	0.08	11.38	11.56	1328.2	184.03	3.46	0.55	0.15	0.27	0.02	0
1	3260	50 yr	EC low tide	5644	0.08	13.76	13.88	2450.35	569.68	2.91	0.9	0.94	0.18	0.03	0.03
1	3260	50 yr	EC high tide	5644	0.08	13.76	13.88	2450.34	569.68	2.91	0.9	0.94	0.18	0.03	0.03
1	3260	50 yr	PC low tide	5644	0.08	12.92	13.08	1985.37	533.19	3.38	0.87	0.82	0.25	0.03	0.03
1	3260	50 yr	PC high tide	5644	0.08	12.92	13.08	1985.38	533.19	3.38	0.87	0.82	0.25	0.03	0.03
1	3260	100 yr	EC low tide	6846	0.08	14.51	14.64	2890.17	591.46	3.08	1.09	1.16	0.19	0.04	0.05
1	3260	100 yr	EC high tide	6846	0.08	14.51	14.64	2890.16	591.46	3.08	1.09	1.16	0.19	0.04	0.05
1	3260	100 yr	PC low tide	6846	0.08	14.29	14.43	2758.43	586.63	3.21	1.1	1.16	0.21	0.04	0.05
1	3260	100 yr	PC high tide	6846	0.08	14.29	14.43	2758.42	586.63	3.21	1.1	1.16	0.21	0.04	0.05
1	3260	200 yr	EC low tide	8187	0.08	16.17	16.27	3928.04	670.79	2.82	1.04	1.3	0.16	0.03	0.05
1	3260	200 yr	EC high tide	8187	0.08	16.17	16.27	3927.99	670.78	2.82	1.04	1.3	0.16	0.03	0.05
1	3260	200 yr	PC low tide	8187	0.08	16.09	16.19	3873.87	667.8	2.86	1.04	1.31	0.16	0.04	0.05
1	3260	200 yr	PC high tide	8187	0.08	16.09	16.19	3873.82	667.8	2.86	1.04	1.31	0.16	0.04	0.05
1	3260	500 yr	EC low tide	10203	0.08	19.19	19.25	6101.16	779.44	2.31	1.05	1.25	0.1	0.03	0.04
1	3260	500 yr	EC high tide	10203	0.08	19.19	19.25	6101.16	779.44	2.31	1.05	1.25	0.1	0.03	0.04
1	3260	500 yr	PC low tide	10203	0.08	19.15	19.21	6073.64	776.82	2.32	1.05	1.26	0.1	0.03	0.04
1	3260	500 yr	PC high tide	10203	0.08	19.15	19.21	6073.64	776.82	2.32	1.05	1.26	0.1	0.03	0.04
1	3260	5% exceedance	EC low tide	1142	0.08	10.96	10.98	1254.79	172.16	0.91	0.11		0.02	0	
1	3260	5% exceedance	EC high tide	1142	0.08	10.96	10.98	1254.79	172.16	0.91	0.11		0.02	0	
1	3260	5% exceedance	PC low tide	1142	0.08	5.88	5.96	487.83	134.57	2.34			0.16		
1	3260	5% exceedance	PC high tide	1142	0.08	6	6.08	504.72	135.05	2.26			0.15		
1	3260	50% exceedance	EC low tide	288	0.08	9.68	9.68	1045.94	157.94	0.28			0		
1	3260	50% exceedance	EC high tide	288	0.08	9.68	9.68	1045.94	157.94	0.28			0		
1	3260	50% exceedance	PC low tide	288	0.08	3.81	3.83	253.09	98.75	1.14			0.04		
1	3260	50% exceedance	PC high tide	288	0.08	4.37	4.38	308.64	102.22	0.93			0.03		
1	3260	95% exceedance	EC low tide	47	0.08	9.05	9.05	946.87	156.38	0.05			0		
1	3260	95% exceedance	EC high tide	47	0.08	9.05	9.05	946.87	156.38	0.05			0		
1	3260	95% exceedance	PC low tide	47	0.08	2.83	2.84	161.73	89.75	0.29			0		
1	3260	95% exceedance	PC high tide	47	0.08	4.11	4.11	282.48	100.79	0.17			0		
1	3072.26	2 yr	EC low tide	1439	4.29	11.24	11.3	801.04	206.47	1.81	0.22		0.09	0	
1	3072.26	2 yr	EC high tide	1439	4.29	11.24	11.3	801.04	206.47	1.81	0.22		0.09	0	
1	3072.26	2 yr	PC low tide	1439	1.25	6.36	6.45	604.88	147.51	2.38			0.16		
1	3072.26	2 yr	PC high tide	1439	1.25	6.43	6.51	614	148.05	2.34			0.16		
1	3072.26	10 yr	EC low tide	3316	4.29	12.64	12.8	1066.29	229.12	3.22	1.09		0.25	0.05	
1	3072.26	10 yr	EC high tide	3316	4.29	12.64	12.8	1066.29	229.12	3.22	1.09		0.25	0.05	
1	3072.26	10 yr	PC low tide	3316	1.25	9.51	9.65	1111.97	171.94	2.98			0.22		
1	3072.26	10 yr	PC high tide	3316	1.25	9.51	9.65	1111.98	171.94	2.98			0.22		
1	3072.26	25 yr	EC low tide	4569	4.29	13.15	13.39	1193.93	391.75	4.04	1.78	0.35	0.42	0.12	0.01
1	3072.26	25 yr	EC high tide	4569	4.29	13.15	13.39	1193.93	391.75	4.04	1.78	0.35	0.42	0.12	0.01
1	3072.26	25 yr	PC low tide	4569	1.25	11.3	11.46	1435.53	207.99	3.2	0.33		0.23	0.01	
1	3072.26	25 yr	PC high tide	4569	1.25	11.3	11.46	1435.54	207.99	3.2	0.33		0.23	0.01	
1	3072.26	50 yr	EC low tide	5644	4.29	13.42	13.75	1332.95	490.49	4.69	2.14	0.75	0.55	0.17	0.04
1	3072.26	50 yr	EC high tide	5644	4.29	13.42	13.75	1332.95	490.49	4.69	2.14	0.75	0.55	0.17	0.04
1	3072.26	50 yr	PC low tide	5644	1.25	12.84	13	1777.74	231.28	3.26	0.96		0.23	0.04	
1	3072.26	50 yr	PC high tide	5644	1.25	12.84	13	1777.74	231.28	3.26	0.96		0.23	0.04	
1	3072.26	100 yr	EC low tide	6846	4.29	14.24	14.53	1900.62	672.02	4.57	1.78	1.32	0.5	0.12	0.08
1	3072.26	100 yr	EC high tide	6846	4.29	14.24	14.53	1900.62	672.02	4.57	1.78	1.32	0.5	0.12	0.08
1	3072.26	100 yr	PC low tide	6846	1.25	14.21	14.36	2559.05	671.48	3.23	0.94	0.69	0.22	0.03	0.02
1	3072.26	100 yr	PC high tide	6846	1.25	14.21	14.36	2559.04	671.48	3.23	0.94	0.69	0.22	0.03	0.02
1	3072.26	200 yr	EC low tide	8187	4.29	16.07	16.21	3156.68	699.94	3.42	1.86	1.61	0.26	0.1	0.08
1	3072.26	200 yr	EC high tide	8187	4.29	16.07	16.21	3156.63	699.94	3.42	1.86	1.61	0.26	0.1	0.08
1	3072.26	200 yr	PC low tide	8187	1.25	16.05	16.15	3819.36	699.72	2.82	1.21	1.04	0.16	0.04	0.04
1	3072.26	200 yr	PC high tide	8187	1.25	16.05	16.15	3819.31	699.72	2.82	1.21	1.04	0.16	0.04	0.04
1	3072.26	500 yr	EC low tide	10203	4.29	19.16	19.22	5360.1	739	2.44	1.54	1.51	0.12	0.06	0.06
1	3072.26	500 yr	EC high tide	10203	4.29	19.16	19.22	5360.1	739	2.44	1.54	1.51	0.12	0.06	0.06
1	3072.26	500 yr	PC low tide	10203	1.25	19.14	19.19	6021.77	738.48	2.26	1.19	1.17	0.09	0.04	0.03
1	3072.26	500 yr	PC high tide	10203	1.25	19.14	19.19	6021.77	738.48	2.26	1.19	1.17	0.09	0.04	0.03
1	3072.26	5% exceedance	EC low tide	1142	4.29	10.92	10.96	746.66	182.8	1.53			0.06		
1	3072.26	5% exceedance	EC high tide	1142	4.29	10.92	10.96	746.66	182.8	1.53			0.06		
1	3072.26	5% exceedance	PC low tide	1142	1.25	5.75	5.83	516.38	142.11	2.21			0.14		
1	3072.26	5% exceedance	PC high tide	1142	1.25	5.89	5.96	536.17	143.33	2.13			0.13		
1	3072.26	50% exceedance	EC low tide	288	4.29	9.67	9.68	561.99	172.59	0.51			0.01		
1	3072.26	50% exceedance	EC high tide	288	4.29	9.67	9.68	561.99	172.59	0.51			0.01		
1	3072.26	50% exceedance	PC low tide	288	1.25	3.76	3.78	256.11	117.69	1.12			0.04		
1	3072.26	50% exceedance	PC high tide	288	1.25	4.34	4.35	326.48	125.21	0.88			0.03		
1	3072.26	95% exceedance	EC low tide	47	4.29	9.05	9.05	473.1	170.04	0.1			0		
1	3072.26	95% exceedance	EC high tide	47	4.29	9.05	9.05	473.1	170.04	0.1			0		
1	3072.26	95% exceedance	PC low tide	47	1.25	2.83	2.83	152.14	105.8	0.31			0		
1	3072.26	95% exceedance	PC high tide	47	1.25	4.11	4.11	297.92	122.22	0.16			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	3063.68	2 yr	EC low tide	1439	5.46	11.22	11.29	678.24	203.66	2.14	0.28		0.13	0.01	
1	3063.68	2 yr	EC high tide	1439	5.46	11.22	11.29	678.24	203.66	2.14	0.28		0.13	0.01	
1	3063.68	10 yr	EC low tide	3316	5.46	12.59	12.79	944.47	227.11	3.66	1.39		0.34	0.08	
1	3063.68	10 yr	EC high tide	3316	5.46	12.59	12.79	944.47	227.11	3.66	1.39		0.34	0.08	
1	3063.68	25 yr	EC low tide	4569	5.46	13.06	13.37	1054.87	490.62	4.6	2.03	0.22	0.56	0.16	0.01
1	3063.68	25 yr	EC high tide	4569	5.46	13.06	13.37	1054.87	490.62	4.6	2.03	0.22	0.56	0.16	0.01
1	3063.68	50 yr	EC low tide	5644	5.46	13.3	13.73	1183.58	526.63	5.34	2.41	0.77	0.74	0.22	0.04
1	3063.68	50 yr	EC high tide	5644	5.46	13.3	13.73	1183.58	526.63	5.34	2.41	0.77	0.74	0.22	0.04
1	3063.68	100 yr	EC low tide	6846	5.46	14.19	14.51	1809.03	670.97	4.89	2.16	1.52	0.59	0.17	0.1
1	3063.68	100 yr	EC high tide	6846	5.46	14.19	14.51	1809.03	670.97	4.89	2.16	1.52	0.59	0.17	0.1
1	3063.68	200 yr	EC low tide	8187	5.46	16.06	16.2	3091.65	699.2	3.49	2.06	1.73	0.28	0.13	0.1
1	3063.68	200 yr	EC high tide	8187	5.46	16.06	16.2	3091.6	699.2	3.49	2.06	1.73	0.28	0.13	0.1
1	3063.68	500 yr	EC low tide	10203	5.46	19.15	19.22	5303.46	750.01	2.46	1.59	1.59	0.12	0.06	0.06
1	3063.68	500 yr	EC high tide	10203	5.46	19.15	19.22	5303.45	750.01	2.46	1.59	1.59	0.12	0.06	0.06
1	3063.68	5% exceedance	EC low tide	1142	5.46	10.9	10.96	623.91	172.44	1.83			0.1		
1	3063.68	5% exceedance	EC high tide	1142	5.46	10.9	10.96	623.91	172.44	1.83			0.1		
1	3063.68	50% exceedance	EC low tide	288	5.46	9.67	9.68	441.28	162.78	0.65			0.01		
1	3063.68	50% exceedance	EC high tide	288	5.46	9.67	9.68	441.28	162.78	0.65			0.01		
1	3063.68	95% exceedance	EC low tide	47	5.46	9.05	9.05	351.4	160.24	0.13			0		
1	3063.68	95% exceedance	EC high tide	47	5.46	9.05	9.05	351.4	160.24	0.13			0		
1	3051.85						Dam								
1	3041.58	2 yr	EC low tide	1439	1.45	7.03	7.17	485.97	103.96	2.96			0.23		
1	3041.58	2 yr	EC high tide	1439	1.45	7.03	7.17	485.97	103.96	2.96			0.23		
1	3041.58	2 yr	PC low tide	1439	1.45	6.28	6.42	470.97	114.62	3.06			0.27		
1	3041.58	2 yr	PC high tide	1439	1.45	6.34	6.48	478.41	114.79	3.01			0.26		
1	3041.58	10 yr	EC low tide	3316	1.45	9.56	9.88	731.45	121.52	4.53			0.5		
1	3041.58	10 yr	EC high tide	3316	1.45	9.56	9.88	731.45	121.52	4.53			0.5		
1	3041.58	10 yr	PC low tide	3316	1.45	9.38	9.62	836.34	121.07	3.96			0.38		
1	3041.58	10 yr	PC high tide	3316	1.45	9.38	9.62	836.34	121.07	3.96			0.38		
1	3041.58	25 yr	EC low tide	4569	1.45	11.22	11.61	906.39	126.48	5.04			0.58		
1	3041.58	25 yr	EC high tide	4569	1.45	11.22	11.61	906.39	126.48	5.04			0.58		
1	3041.58	25 yr	PC low tide	4569	1.45	11.13	11.42	1052.73	126.08	4.34			0.43		
1	3041.58	25 yr	PC high tide	4569	1.45	11.13	11.42	1052.73	126.08	4.34			0.43		
1	3041.58	50 yr	EC low tide	5644	1.45	12.69	13.13	1061.43	135.04	5.32			0.61		
1	3041.58	50 yr	EC high tide	5644	1.45	12.69	13.13	1061.43	135.04	5.32			0.61		
1	3041.58	50 yr	PC low tide	5644	1.45	12.66	12.97	1251.01	134.78	4.51			0.45		
1	3041.58	50 yr	PC high tide	5644	1.45	12.66	12.97	1251.02	134.78	4.51			0.45		
1	3041.58	100 yr	EC low tide	6846	1.45	14.19	14.4	2278.86	672.41	3.99	1.79	0.9	0.35	0.1	0.04
1	3041.58	100 yr	EC high tide	6846	1.45	14.19	14.4	2278.86	672.41	3.99	1.79	0.9	0.35	0.1	0.04
1	3041.58	100 yr	PC low tide	6846	1.45	14.15	14.35	2327.09	671.81	3.9	1.63	0.8	0.32	0.09	0.03
1	3041.58	100 yr	PC high tide	6846	1.45	14.15	14.35	2327.08	671.81	3.9	1.63	0.8	0.32	0.09	0.03
1	3041.58	200 yr	EC low tide	8187	1.45	16.03	16.15	3538.88	693.05	3.2	1.8	1.28	0.21	0.09	0.05
1	3041.58	200 yr	EC high tide	8187	1.45	16.03	16.15	3538.83	693.05	3.2	1.8	1.28	0.21	0.09	0.05
1	3041.58	200 yr	PC low tide	8187	1.45	16.02	16.14	3610.45	692.99	3.18	1.68	1.2	0.2	0.08	0.05
1	3041.58	200 yr	PC high tide	8187	1.45	16.02	16.14	3610.4	692.99	3.18	1.68	1.2	0.2	0.08	0.05
1	3041.58	500 yr	EC low tide	10203	1.45	19.13	19.19	5768.39	756.74	2.42	1.48	1.34	0.11	0.05	0.05
1	3041.58	500 yr	EC high tide	10203	1.45	19.13	19.19	5768.39	756.74	2.42	1.48	1.34	0.11	0.05	0.05
1	3041.58	500 yr	PC low tide	10203	1.45	19.13	19.19	5844.34	756.69	2.44	1.42	1.29	0.11	0.05	0.04
1	3041.58	500 yr	PC high tide	10203	1.45	19.13	19.19	5844.33	756.69	2.44	1.42	1.29	0.11	0.05	0.04
1	3041.58	5% exceedance	EC low tide	1142	1.45	6.63	6.73	448.28	102.48	2.55			0.18		
1	3041.58	5% exceedance	EC high tide	1142	1.45	6.63	6.73	448.28	102.48	2.55			0.18		
1	3041.58	5% exceedance	PC low tide	1142	1.45	5.67	5.8	402.48	112.38	2.84			0.24		
1	3041.58	5% exceedance	PC high tide	1142	1.45	5.82	5.94	418.93	113.04	2.73			0.22		
1	3041.58	50% exceedance	EC low tide	288	1.45	5.09	5.1	306.26	94.65	0.94			0.03		
1	3041.58	50% exceedance	EC high tide	288	1.45	5.09	5.1	306.26	94.65	0.94			0.03		
1	3041.58	50% exceedance	PC low tide	288	1.45	3.73	3.76	194.72	98.66	1.48			0.08		
1	3041.58	50% exceedance	PC high tide	288	1.45	4.32	4.34	255.26	105.17	1.13			0.04		
1	3041.58	95% exceedance	EC low tide	47	1.45	4.27	4.27	231.31	90.86	0.2			0		
1	3041.58	95% exceedance	EC high tide	47	1.45	4.27	4.27	231.31	90.86	0.2			0		
1	3041.58	95% exceedance	PC low tide	47	1.45	2.82	2.83	109.92	88.71	0.43			0.01		
1	3041.58	95% exceedance	PC high tide	47	1.45	4.11	4.11	232.74	102.81	0.2			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	3020.17	2 yr	EC low tide	1439	3.29	6.05	7.04	180.52	99.42	7.97			2.32		
1	3020.17	2 yr	EC high tide	1439	3.29	6.05	7.04	180.52	99.42	7.97			2.32		
1	3020.17	2 yr	PC low tide	1439	1.9	6.22	6.39	426.46	114.45	3.37			0.33		
1	3020.17	2 yr	PC high tide	1439	1.9	6.28	6.45	434.28	114.64	3.31			0.32		
1	3020.17	10 yr	EC low tide	3316	3.29	8.93	9.77	450.14	120.02	7.37			1.54		
1	3020.17	10 yr	EC high tide	3316	3.29	8.93	9.77	450.14	120.02	7.37			1.54		
1	3020.17	10 yr	PC low tide	3316	1.9	9.32	9.6	792.61	120.94	4.18			0.43		
1	3020.17	10 yr	PC high tide	3316	1.9	9.32	9.6	792.61	120.94	4.18			0.43		
1	3020.17	25 yr	EC low tide	4569	3.29	10.74	11.53	640.94	124.71	7.13			1.29		
1	3020.17	25 yr	EC high tide	4569	3.29	10.74	11.53	640.94	124.71	7.13			1.29		
1	3020.17	25 yr	PC low tide	4569	1.9	11.08	11.4	1009.2	125.87	4.53			0.47		
1	3020.17	25 yr	PC high tide	4569	1.9	11.08	11.4	1009.2	125.87	4.53			0.47		
1	3020.17	50 yr	EC low tide	5644	3.29	12.3	13.06	804.69	132.24	7.01			1.17		
1	3020.17	50 yr	EC high tide	5644	3.29	12.3	13.06	804.7	132.24	7.01			1.17		
1	3020.17	50 yr	PC low tide	5644	1.9	12.61	12.95	1207.63	134.46	4.67			0.49		
1	3020.17	50 yr	PC high tide	5644	1.9	12.61	12.95	1207.63	134.46	4.67			0.49		
1	3020.17	100 yr	EC low tide	6846	3.29	14.07	14.37	1848.41	670.74	4.94	2.61	1.22	0.58	0.22	0.07
1	3020.17	100 yr	EC high tide	6846	3.29	14.07	14.37	1848.4	670.74	4.94	2.61	1.22	0.58	0.22	0.07
1	3020.17	100 yr	PC low tide	6846	1.9	14.13	14.34	2276.43	671.55	3.99	1.69	0.82	0.34	0.09	0.03
1	3020.17	100 yr	PC high tide	6846	1.9	14.13	14.34	2276.43	671.55	3.99	1.69	0.82	0.34	0.09	0.03
1	3020.17	200 yr	EC low tide	8187	3.29	16.01	16.14	3173.62	692.87	3.49	2.27	1.61	0.27	0.14	0.08
1	3020.17	200 yr	EC high tide	8187	3.29	16.01	16.14	3173.57	692.87	3.49	2.27	1.61	0.27	0.14	0.08
1	3020.17	200 yr	PC low tide	8187	1.9	16.02	16.13	3567.6	692.93	3.21	1.72	1.23	0.21	0.08	0.05
1	3020.17	200 yr	PC high tide	8187	1.9	16.02	16.13	3567.55	692.93	3.21	1.72	1.23	0.21	0.08	0.05
1	3020.17	500 yr	EC low tide	10203	3.29	19.13	19.19	5416.26	756.63	2.47	1.7	1.54	0.12	0.07	0.06
1	3020.17	500 yr	EC high tide	10203	3.29	19.13	19.19	5416.25	756.63	2.47	1.7	1.54	0.12	0.07	0.06
1	3020.17	500 yr	PC low tide	10203	1.9	19.13	19.19	5804.75	756.64	2.45	1.44	1.31	0.11	0.05	0.04
1	3020.17	500 yr	PC high tide	10203	1.9	19.13	19.19	5804.75	756.64	2.45	1.44	1.31	0.11	0.05	0.04
1	3020.17	5% exceedance	EC low tide	1142	3.29	5.76	6.61	154.52	97.81	7.39			2.1		
1	3020.17	5% exceedance	EC high tide	1142	3.29	5.76	6.61	154.52	97.81	7.39			2.1		
1	3020.17	5% exceedance	PC low tide	1142	1.9	5.61	5.77	357.76	112.07	3.19			0.31		
1	3020.17	5% exceedance	PC high tide	1142	1.9	5.76	5.91	375.15	112.79	3.04			0.28		
1	3020.17	50% exceedance	EC low tide	288	3.29	4.7	5.06	60.27	85.61	4.78			1.16		
1	3020.17	50% exceedance	EC high tide	288	3.29	4.7	5.06	60.27	85.61	4.78			1.16		
1	3020.17	50% exceedance	PC low tide	288	1.9	3.69	3.74	153.49	97.06	1.88			0.14		
1	3020.17	50% exceedance	PC high tide	288	1.9	4.31	4.33	216.07	104.8	1.33			0.06		
1	3020.17	95% exceedance	EC low tide	47	3.29	4.1	4.25	15.06	47.95	3.12			0.65		
1	3020.17	95% exceedance	EC high tide	47	3.29	4.1	4.25	15.06	47.95	3.12			0.65		
1	3020.17	95% exceedance	PC low tide	47	1.9	2.82	2.82	73.77	86.2	0.64			0.02		
1	3020.17	95% exceedance	PC high tide	47	1.9	4.11	4.11	195.3	102.3	0.24			0		
1	2998.83	2 yr	EC low tide	1439	1.92	6.09	6.34	357.57	110.16	4.02			0.51		
1	2998.83	2 yr	EC high tide	1439	1.92	6.17	6.41	366.01	110.35	3.93			0.48		
1	2998.83	2 yr	PC low tide	1439	1.92	6.09	6.34	357.57	110.16	4.02			0.51		
1	2998.83	2 yr	PC high tide	1439	1.92	6.17	6.41	366.01	110.35	3.93			0.48		
1	2998.83	10 yr	EC low tide	3316	1.92	9.22	9.56	709.48	114.41	4.67			0.56		
1	2998.83	10 yr	EC high tide	3316	1.92	9.22	9.56	709.48	114.41	4.67			0.56		
1	2998.83	10 yr	PC low tide	3316	1.92	9.22	9.56	709.48	114.41	4.67			0.56		
1	2998.83	10 yr	PC high tide	3316	1.92	9.22	9.56	709.48	114.41	4.67			0.56		
1	2998.83	25 yr	EC low tide	4569	1.92	10.98	11.37	912.33	116.45	5.01			0.59		
1	2998.83	25 yr	EC high tide	4569	1.92	10.98	11.37	912.33	116.45	5.01			0.59		
1	2998.83	25 yr	PC low tide	4569	1.92	10.98	11.37	912.33	116.45	5.01			0.59		
1	2998.83	25 yr	PC high tide	4569	1.92	10.98	11.37	912.33	116.45	5.01			0.59		
1	2998.83	50 yr	EC low tide	5644	1.92	12.51	12.92	1100.29	159.95	5.17		0.45	0.6		0.02
1	2998.83	50 yr	EC high tide	5644	1.92	12.51	12.92	1100.3	159.95	5.17		0.45	0.6		0.02
1	2998.83	50 yr	PC low tide	5644	1.92	12.51	12.92	1100.29	159.95	5.17		0.45	0.6		0.02
1	2998.83	50 yr	PC high tide	5644	1.92	12.51	12.92	1100.3	159.95	5.17		0.45	0.6		0.02
1	2998.83	100 yr	EC low tide	6846	1.92	14.11	14.32	2284.15	670.31	4.09	1.94	0.91	0.36	0.12	0.04
1	2998.83	100 yr	EC high tide	6846	1.92	14.11	14.32	2284.14	670.31	4.09	1.94	0.91	0.36	0.12	0.04
1	2998.83	100 yr	PC low tide	6846	1.92	14.11	14.32	2284.15	670.31	4.09	1.94	0.91	0.36	0.12	0.04
1	2998.83	100 yr	PC high tide	6846	1.92	14.11	14.32	2284.14	670.31	4.09	1.94	0.91	0.36	0.12	0.04
1	2998.83	200 yr	EC low tide	8187	1.92	16.01	16.13	3582.12	696.33	3.24	1.88	1.28	0.21	0.09	0.05
1	2998.83	200 yr	EC high tide	8187	1.92	16.01	16.13	3582.07	696.33	3.24	1.88	1.28	0.21	0.09	0.05
1	2998.83	200 yr	PC low tide	8187	1.92	16.01	16.13	3582.12	696.33	3.24	1.88	1.28	0.21	0.09	0.05
1	2998.83	200 yr	PC high tide	8187	1.92	16.01	16.13	3582.07	696.33	3.24	1.88	1.28	0.21	0.09	0.05
1	2998.83	500 yr	EC low tide	10203	1.92	19.13	19.18	5858.34	764.66	2.43	1.51	1.34	0.11	0.05	0.05
1	2998.83	500 yr	EC high tide	10203	1.92	19.13	19.18	5858.34	764.66	2.43	1.51	1.34	0.11	0.05	0.05
1	2998.83	500 yr	PC low tide	10203	1.92	19.13	19.18	5858.34	764.66	2.43	1.51	1.34	0.11	0.05	0.05
1	2998.83	500 yr	PC high tide	10203	1.92	19.13	19.18	5858.34	764.66	2.43	1.51	1.34	0.11	0.05	0.05
1	2998.83	5% exceedance	EC low tide	1142	1.92	5.47	5.71	290.72	106.43	3.93			0.51		
1	2998.83	5% exceedance	EC high tide	1142	1.92	5.65	5.86	309.69	107.23	3.69			0.44		
1	2998.83	5% exceedance	PC low tide	1142	1.92	5.47	5.71	290.72	106.43	3.93			0.51		
1	2998.83	5% exceedance	PC high tide	1142	1.92	5.65	5.86	309.69	107.23	3.69			0.44		
1	2998.83	50% exceedance	EC low tide	288	1.92	2.85	3.91	34.98	70.09	8.23			3.89		
1	2998.83	50% exceedance	EC high tide	288	1.92	4.27	4.32	165.25	102.03	1.74			0.12		
1	2998.83	50% exceedance	PC low tide	288	1.92	3.51	3.67	90.4	96.3	3.19			0.47		
1	2998.83	50% exceedance	PC high tide	288	1.92	4.27	4.32	165.25	102.03	1.74			0.12		
1	2998.83	95% exceedance	EC low tide	47	1.92	2.76	2.8	28.27	66.5	1.66			0.17		
1	2998.83	95% exceedance	EC high tide	47	1.92	4.1	4.11	148.57	100.71	0.32			0		
1	2998.83	95% exceedance	PC low tide	47	1.92	2.76	2.8	28.27	66.5	1.66			0.17		
1	2998.83	95% exceedance	PC high tide	47	1.92	4.1	4.11	148.57	100.71	0.32			0		
1	2990.65			Pedestrian Bridge											

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	2934.56	2 yr	EC low tide	1439	1	6.06	6.22	446.09	108.22	3.23			0.3		
1	2934.56	2 yr	EC high tide	1439	1	6.14	6.29	454.78	108.32	3.16			0.29		
1	2934.56	2 yr	PC low tide	1439	1	6.06	6.22	446.09	108.22	3.23			0.3		
1	2934.56	2 yr	PC high tide	1439	1	6.14	6.29	454.78	108.32	3.16			0.29		
1	2934.56	10 yr	EC low tide	3316	1	9.19	9.46	792.8	113	4.18			0.43		
1	2934.56	10 yr	EC high tide	3316	1	9.19	9.46	792.8	113	4.18			0.43		
1	2934.56	10 yr	PC low tide	3316	1	9.19	9.46	792.8	113	4.18			0.43		
1	2934.56	10 yr	PC high tide	3316	1	9.19	9.46	792.8	113	4.18			0.43		
1	2934.56	25 yr	EC low tide	4569	1	10.93	11.26	991.95	116.41	4.61			0.49		
1	2934.56	25 yr	EC high tide	4569	1	10.93	11.26	991.95	116.41	4.61			0.49		
1	2934.56	25 yr	PC low tide	4569	1	10.93	11.26	991.95	116.41	4.61			0.49		
1	2934.56	25 yr	PC high tide	4569	1	10.93	11.26	991.95	116.41	4.61			0.49		
1	2934.56	50 yr	EC low tide	5644	1	12.35	12.72	1159.97	119.9	4.87			0.52		
1	2934.56	50 yr	EC high tide	5644	1	12.35	12.72	1159.98	119.9	4.87			0.52		
1	2934.56	50 yr	PC low tide	5644	1	12.35	12.72	1159.97	119.9	4.87			0.52		
1	2934.56	50 yr	PC high tide	5644	1	12.35	12.72	1159.98	119.9	4.87			0.52		
1	2934.56	100 yr	EC low tide	6846	1	14.08	14.24	2584.31	699.68	3.58	1.94	0.77	0.27	0.11	0.03
1	2934.56	100 yr	EC high tide	6846	1	14.08	14.24	2584.3	699.67	3.58	1.94	0.77	0.27	0.11	0.03
1	2934.56	100 yr	PC low tide	6846	1	14.08	14.24	2584.31	699.68	3.58	1.94	0.77	0.27	0.11	0.03
1	2934.56	100 yr	PC high tide	6846	1	14.08	14.24	2584.3	699.67	3.58	1.94	0.77	0.27	0.11	0.03
1	2934.56	200 yr	EC low tide	8187	1	15.99	16.09	4010.96	788.22	2.96	1.71	1.1	0.17	0.08	0.04
1	2934.56	200 yr	EC high tide	8187	1	15.99	16.09	4010.91	788.22	2.96	1.71	1.1	0.17	0.08	0.04
1	2934.56	200 yr	PC low tide	8187	1	15.99	16.09	4010.96	788.22	2.96	1.71	1.1	0.17	0.08	0.04
1	2934.56	200 yr	PC high tide	8187	1	15.99	16.09	4010.91	788.22	2.96	1.71	1.1	0.17	0.08	0.04
1	2934.56	500 yr	EC low tide	10203	1	19.12	19.17	6534.85	816.48	2.18	1.43	1.17	0.09	0.05	0.03
1	2934.56	500 yr	EC high tide	10203	1	19.12	19.17	6534.85	816.48	2.18	1.43	1.17	0.09	0.05	0.03
1	2934.56	500 yr	PC low tide	10203	1	19.12	19.17	6534.85	816.48	2.18	1.43	1.17	0.09	0.05	0.03
1	2934.56	500 yr	PC high tide	10203	1	19.12	19.17	6534.85	816.48	2.18	1.43	1.17	0.09	0.05	0.03
1	2934.56	5% exceedance	EC low tide	1142	1	5.43	5.58	379.42	106.6	3.01			0.27		
1	2934.56	5% exceedance	EC high tide	1142	1	5.62	5.74	399.01	107.09	2.86			0.24		
1	2934.56	5% exceedance	PC low tide	1142	1	5.43	5.58	379.42	106.6	3.01			0.27		
1	2934.56	5% exceedance	PC high tide	1142	1	5.62	5.74	399.01	107.09	2.86			0.24		
1	2934.56	50% exceedance	EC low tide	288	1	2.95	3.04	123.88	98.49	2.32			0.23		
1	2934.56	50% exceedance	EC high tide	288	1	4.26	4.28	256.05	103.13	1.12			0.04		
1	2934.56	50% exceedance	PC low tide	288	1	2.95	3.04	123.88	98.49	2.32			0.23		
1	2934.56	50% exceedance	PC high tide	288	1	4.26	4.28	256.05	103.13	1.12			0.04		
1	2934.56	95% exceedance	EC low tide	47	1	1.6	1.77	14.48	46	3.25			0.7		
1	2934.56	95% exceedance	EC high tide	47	1	4.1	4.11	240.08	102.63	0.2			0		
1	2934.56	95% exceedance	PC low tide	47	1	1.6	1.77	14.48	46	3.25			0.7		
1	2934.56	95% exceedance	PC high tide	47	1	4.1	4.11	240.08	102.63	0.2			0		
1	2717.18	2 yr	EC low tide	1439	-2.19	5.96	6.03	650.92	139.61	2.21			0.13		
1	2717.18	2 yr	EC high tide	1439	-2.19	6.04	6.12	663.05	140.22	2.17			0.13		
1	2717.18	2 yr	PC low tide	1439	-2.19	5.96	6.03	650.92	139.61	2.21			0.13		
1	2717.18	2 yr	PC high tide	1439	-2.19	6.04	6.12	663.05	140.22	2.17			0.13		
1	2717.18	10 yr	EC low tide	3316	-2.19	9.14	9.27	1130.64	172.2	2.94	0.16	0.2	0.21	0	0
1	2717.18	10 yr	EC high tide	3316	-2.19	9.14	9.27	1130.64	172.2	2.94	0.16	0.2	0.21	0	0
1	2717.18	10 yr	PC low tide	3316	-2.19	9.14	9.27	1130.64	172.2	2.94	0.16	0.2	0.21	0	0
1	2717.18	10 yr	PC high tide	3316	-2.19	9.14	9.27	1130.64	172.2	2.94	0.16	0.2	0.21	0	0
1	2717.18	25 yr	EC low tide	4569	-2.19	10.92	11.07	1511.49	257.91	3.18	0.76	0.76	0.22	0.03	0.03
1	2717.18	25 yr	EC high tide	4569	-2.19	10.92	11.07	1511.5	257.91	3.18	0.76	0.76	0.22	0.03	0.03
1	2717.18	25 yr	PC low tide	4569	-2.19	10.92	11.07	1511.49	257.91	3.18	0.76	0.76	0.22	0.03	0.03
1	2717.18	25 yr	PC high tide	4569	-2.19	10.92	11.07	1511.5	257.91	3.18	0.76	0.76	0.22	0.03	0.03
1	2717.18	50 yr	EC low tide	5644	-2.19	12.39	12.54	2025.93	389.84	3.2	0.96	0.99	0.22	0.04	0.04
1	2717.18	50 yr	EC high tide	5644	-2.19	12.39	12.54	2025.94	389.84	3.2	0.96	0.99	0.22	0.04	0.04
1	2717.18	50 yr	PC low tide	5644	-2.19	12.39	12.54	2025.93	389.84	3.2	0.96	0.99	0.22	0.04	0.04
1	2717.18	50 yr	PC high tide	5644	-2.19	12.39	12.54	2025.94	389.84	3.2	0.96	0.99	0.22	0.04	0.04
1	2717.18	100 yr	EC low tide	6846	-2.19	14.03	14.16	2709.51	442.61	3.07	1.29	0.86	0.19	0.05	0.03
1	2717.18	100 yr	EC high tide	6846	-2.19	14.03	14.16	2709.51	442.6	3.07	1.29	0.86	0.19	0.05	0.03
1	2717.18	100 yr	PC low tide	6846	-2.19	14.03	14.16	2709.51	442.61	3.07	1.29	0.86	0.19	0.05	0.03
1	2717.18	100 yr	PC high tide	6846	-2.19	14.03	14.16	2709.51	442.6	3.07	1.29	0.86	0.19	0.05	0.03
1	2717.18	200 yr	EC low tide	8187	-2.19	15.94	16.04	3586.32	478.11	2.85	1.45	1.02	0.15	0.06	0.03
1	2717.18	200 yr	EC high tide	8187	-2.19	15.94	16.04	3586.28	478.1	2.85	1.45	1.02	0.15	0.06	0.03
1	2717.18	200 yr	PC low tide	8187	-2.19	15.94	16.04	3586.32	478.11	2.85	1.45	1.02	0.15	0.06	0.03
1	2717.18	200 yr	PC high tide	8187	-2.19	15.94	16.04	3586.28	478.1	2.85	1.45	1.02	0.15	0.06	0.03
1	2717.18	500 yr	EC low tide	10203	-2.19	19.06	19.14	5275.98	712.8	2.54	1.51	0.68	0.12	0.05	0.02
1	2717.18	500 yr	EC high tide	10203	-2.19	19.06	19.14	5275.97	712.8	2.54	1.51	0.68	0.12	0.05	0.02
1	2717.18	500 yr	PC low tide	10203	-2.19	19.06	19.14	5275.98	712.8	2.54	1.51	0.68	0.12	0.05	0.02
1	2717.18	500 yr	PC high tide	10203	-2.19	19.06	19.14	5275.97	712.8	2.54	1.51	0.68	0.12	0.05	0.02
1	2717.18	5% exceedance	EC low tide	1142	-2.19	5.33	5.4	564.93	135.31	2.02			0.12		
1	2717.18	5% exceedance	EC high tide	1142	-2.19	5.53	5.59	592.03	136.66	1.93			0.1		
1	2717.18	5% exceedance	PC low tide	1142	-2.19	5.33	5.4	564.93	135.31	2.02			0.12		
1	2717.18	5% exceedance	PC high tide	1142	-2.19	5.53	5.59	592.03	136.66	1.93			0.1		
1	2717.18	50% exceedance	EC low tide	288	-2.19	2.81	2.84	245.15	119.29	1.17			0.05		
1	2717.18	50% exceedance	EC high tide	288	-2.19	4.24	4.25	421.2	128.05	0.68			0.01		
1	2717.18	50% exceedance	PC low tide	288	-2.19	2.81	2.84	245.15	119.29	1.17			0.05		
1	2717.18	50% exceedance	PC high tide	288	-2.19	4.24	4.25	421.2	128.05	0.68			0.01		
1	2717.18	95% exceedance	EC low tide	47	-2.19	1.18	1.18	80.94	66.38	0.58			0.01		
1	2717.18	95% exceedance	EC high tide	47	-2.19	4.1	4.1	403.73	127.06	0.12			0		
1	2717.18	95% exceedance	PC low tide	47	-2.19	1.18	1.18	80.94	66.38	0.58			0.01		
1	2717.18	95% exceedance	PC high tide	47	-2.19	4.1	4.1	403.73	127.06	0.12			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	2701.64	2 yr	EC low tide	1439	-2.19	5.95	6.03	651.91	143.32	2.21			0.15		
1	2701.64	2 yr	EC high tide	1439	-2.19	6.04	6.11	664.42	143.99	2.17			0.14		
1	2701.64	2 yr	PC low tide	1439	-2.19	5.95	6.03	651.91	143.32	2.21			0.15		
1	2701.64	2 yr	PC high tide	1439	-2.19	6.04	6.11	664.42	143.99	2.17			0.14		
1	2701.64	10 yr	EC low tide	3316	-2.19	9.13	9.26	1149.38	171.12	2.89			0.25		
1	2701.64	10 yr	EC high tide	3316	-2.19	9.13	9.26	1149.38	171.12	2.89			0.25		
1	2701.64	10 yr	PC low tide	3316	-2.19	9.13	9.26	1149.38	171.12	2.89			0.25		
1	2701.64	10 yr	PC high tide	3316	-2.19	9.13	9.26	1149.38	171.12	2.89			0.25		
1	2701.64	25 yr	EC low tide	4569	-2.19	10.91	11.06	1470.21	194.72	3.11	0.05	0.33	0.32	0	0.01
1	2701.64	25 yr	EC high tide	4569	-2.19	10.91	11.06	1470.21	194.72	3.11	0.05	0.33	0.32	0	0.01
1	2701.64	25 yr	PC low tide	4569	-2.19	10.91	11.06	1470.21	194.72	3.11	0.05	0.33	0.32	0	0.01
1	2701.64	25 yr	PC high tide	4569	-2.19	10.91	11.06	1470.21	194.72	3.11	0.05	0.33	0.32	0	0.01
1	2701.64	50 yr	EC low tide	5644	-2.19	12.39	12.53	1995.53	392.62	3.09	0.95	0.82	0.3	0.04	0.03
1	2701.64	50 yr	EC high tide	5644	-2.19	12.39	12.53	1995.54	392.63	3.09	0.95	0.82	0.3	0.04	0.03
1	2701.64	50 yr	PC low tide	5644	-2.19	12.39	12.53	1995.53	392.62	3.09	0.95	0.82	0.3	0.04	0.03
1	2701.64	50 yr	PC high tide	5644	-2.19	12.39	12.53	1995.54	392.63	3.09	0.95	0.82	0.3	0.04	0.03
1	2701.64	100 yr	EC low tide	6846	-2.19	14.03	14.15	2702.58	461.52	2.92	1.34	0.82	0.25	0.06	0.03
1	2701.64	100 yr	EC high tide	6846	-2.19	14.03	14.15	2702.57	461.52	2.92	1.34	0.82	0.25	0.06	0.03
1	2701.64	100 yr	PC low tide	6846	-2.19	14.03	14.15	2702.58	461.52	2.92	1.34	0.82	0.25	0.06	0.03
1	2701.64	100 yr	PC high tide	6846	-2.19	14.03	14.15	2702.57	461.52	2.92	1.34	0.82	0.25	0.06	0.03
1	2701.64	200 yr	EC low tide	8187	-2.19	15.94	16.03	3623.09	504.57	2.66	1.51	1.07	0.2	0.07	0.04
1	2701.64	200 yr	EC high tide	8187	-2.19	15.94	16.03	3623.05	504.57	2.66	1.51	1.07	0.2	0.07	0.04
1	2701.64	200 yr	PC low tide	8187	-2.19	15.94	16.03	3623.09	504.57	2.66	1.51	1.07	0.2	0.07	0.04
1	2701.64	200 yr	PC high tide	8187	-2.19	15.94	16.03	3623.05	504.57	2.66	1.51	1.07	0.2	0.07	0.04
1	2701.64	500 yr	EC low tide	10203	-2.19	19.07	19.14	5486.7	921.31	2.39	1.36	0.67	0.15	0.04	0.02
1	2701.64	500 yr	EC high tide	10203	-2.19	19.07	19.14	5486.7	921.31	2.39	1.36	0.67	0.15	0.04	0.02
1	2701.64	500 yr	PC low tide	10203	-2.19	19.07	19.14	5486.7	921.31	2.39	1.36	0.67	0.15	0.04	0.02
1	2701.64	500 yr	PC high tide	10203	-2.19	19.07	19.14	5486.7	921.31	2.39	1.36	0.67	0.15	0.04	0.02
1	2701.64	5% exceedance	EC low tide	1142	-2.19	5.32	5.39	563.71	138.51	2.03			0.13		
1	2701.64	5% exceedance	EC high tide	1142	-2.19	5.53	5.58	591.63	140.04	1.93			0.11		
1	2701.64	5% exceedance	PC low tide	1142	-2.19	5.32	5.39	563.71	138.51	2.03			0.13		
1	2701.64	5% exceedance	PC high tide	1142	-2.19	5.53	5.58	591.63	140.04	1.93			0.11		
1	2701.64	50% exceedance	EC low tide	288	-2.19	2.81	2.83	241.28	113.52	1.19			0.05		
1	2701.64	50% exceedance	EC high tide	288	-2.19	4.24	4.25	417.88	130.24	0.69			0.02		
1	2701.64	50% exceedance	PC low tide	288	-2.19	2.81	2.83	241.28	113.52	1.19			0.05		
1	2701.64	50% exceedance	PC high tide	288	-2.19	4.24	4.25	417.88	130.24	0.69			0.02		
1	2701.64	95% exceedance	EC low tide	47	-2.19	1.17	1.18	80.75	66.34	0.58			0.01		
1	2701.64	95% exceedance	EC high tide	47	-2.19	4.1	4.1	400.27	129.17	0.12			0		
1	2701.64	95% exceedance	PC low tide	47	-2.19	1.17	1.18	80.75	66.34	0.58			0.01		
1	2701.64	95% exceedance	PC high tide	47	-2.19	4.1	4.1	400.27	129.17	0.12			0		
1	2522.86	2 yr	EC low tide	1439	-0.22	5.72	5.88	447.93	115.68	3.21			0.3		
1	2522.86	2 yr	EC high tide	1439	-0.22	5.82	5.97	459.7	116.43	3.13			0.28		
1	2522.86	2 yr	PC low tide	1439	-0.22	5.72	5.88	447.93	115.68	3.21			0.3		
1	2522.86	2 yr	PC high tide	1439	-0.22	5.82	5.97	459.7	116.43	3.13			0.28		
1	2522.86	10 yr	EC low tide	3316	-0.22	8.88	9.1	915.58	232.42	3.81	0.77	1.09	0.46	0.05	0.05
1	2522.86	10 yr	EC high tide	3316	-0.22	8.88	9.1	915.58	232.42	3.81	0.77	1.09	0.46	0.05	0.05
1	2522.86	10 yr	PC low tide	3316	-0.22	8.88	9.1	915.58	232.42	3.81	0.77	1.09	0.46	0.05	0.05
1	2522.86	10 yr	PC high tide	3316	-0.22	8.88	9.1	915.58	232.42	3.81	0.77	1.09	0.46	0.05	0.05
1	2522.86	25 yr	EC low tide	4569	-0.22	10.74	10.93	1453.19	327.88	3.66	1.43	1.47	0.39	0.09	0.08
1	2522.86	25 yr	EC high tide	4569	-0.22	10.74	10.93	1453.19	327.88	3.66	1.43	1.47	0.39	0.09	0.08
1	2522.86	25 yr	PC low tide	4569	-0.22	10.74	10.93	1453.19	327.88	3.66	1.43	1.47	0.39	0.09	0.08
1	2522.86	25 yr	PC high tide	4569	-0.22	10.74	10.93	1453.19	327.88	3.66	1.43	1.47	0.39	0.09	0.08
1	2522.86	50 yr	EC low tide	5644	-0.22	12.25	12.42	2085.87	678.6	3.59	1.15	1.28	0.36	0.05	0.05
1	2522.86	50 yr	EC high tide	5644	-0.22	12.25	12.42	2085.89	678.6	3.59	1.15	1.28	0.36	0.05	0.05
1	2522.86	50 yr	PC low tide	5644	-0.22	12.25	12.42	2085.87	678.6	3.59	1.15	1.28	0.36	0.05	0.05
1	2522.86	50 yr	PC high tide	5644	-0.22	12.25	12.42	2085.89	678.6	3.59	1.15	1.28	0.36	0.05	0.05
1	2522.86	100 yr	EC low tide	6846	-0.22	13.99	14.08	3306.2	715.21	2.88	1.35	1.32	0.22	0.06	0.05
1	2522.86	100 yr	EC high tide	6846	-0.22	13.99	14.08	3306.19	715.21	2.88	1.35	1.32	0.22	0.06	0.05
1	2522.86	100 yr	PC low tide	6846	-0.22	13.99	14.08	3306.2	715.21	2.88	1.35	1.32	0.22	0.06	0.05
1	2522.86	100 yr	PC high tide	6846	-0.22	13.99	14.08	3306.19	715.21	2.88	1.35	1.32	0.22	0.06	0.05
1	2522.86	200 yr	EC low tide	8187	-0.22	15.93	15.98	4784.05	787.72	2.38	1.28	1.36	0.14	0.05	0.05
1	2522.86	200 yr	EC high tide	8187	-0.22	15.93	15.98	4783.99	787.72	2.38	1.28	1.36	0.14	0.05	0.05
1	2522.86	200 yr	PC low tide	8187	-0.22	15.93	15.98	4784.05	787.72	2.38	1.28	1.36	0.14	0.05	0.05
1	2522.86	200 yr	PC high tide	8187	-0.22	15.93	15.98	4783.99	787.72	2.38	1.28	1.36	0.14	0.05	0.05
1	2522.86	500 yr	EC low tide	10203	-0.22	19.07	19.11	7263.55	787.72	1.82	1.21	1.24	0.08	0.04	0.04
1	2522.86	500 yr	EC high tide	10203	-0.22	19.07	19.11	7263.55	787.72	1.82	1.21	1.24	0.08	0.04	0.04
1	2522.86	500 yr	PC low tide	10203	-0.22	19.07	19.11	7263.55	787.72	1.82	1.21	1.24	0.08	0.04	0.04
1	2522.86	500 yr	PC high tide	10203	-0.22	19.07	19.11	7263.55	787.72	1.82	1.21	1.24	0.08	0.04	0.04
1	2522.86	5% exceedance	EC low tide	1142	-0.22	5.1	5.24	377.75	111.56	3.02			0.28		
1	2522.86	5% exceedance	EC high tide	1142	-0.22	5.34	5.46	404.11	113.03	2.83			0.24		
1	2522.86	5% exceedance	PC low tide	1142	-0.22	5.1	5.24	377.75	111.56	3.02			0.28		
1	2522.86	5% exceedance	PC high tide	1142	-0.22	5.34	5.46	404.11	113.03	2.83			0.24		
1	2522.86	50% exceedance	EC low tide	288	-0.22	2.61	2.69	126.63	83.47	2.27			0.2		
1	2522.86	50% exceedance	EC high tide	288	-0.22	4.21	4.22	280.38	106.6	1.03			0.03		
1	2522.86	50% exceedance	PC low tide	288	-0.22	2.61	2.69	126.63	83.47	2.27			0.2		
1	2522.86	50% exceedance	PC high tide	288	-0.22	4.21	4.22	280.38	106.6	1.03			0.03		
1	2522.86	95% exceedance	EC low tide	47	-0.22	0.99	1.07	20.99	36.25	2.24			0.27		
1	2522.86	95% exceedance	EC high tide	47	-0.22	4.1	4.1	269.18	106.02	0.17			0		
1	2522.86	95% exceedance	PC low tide	47	-0.22	0.99	1.07	20.99	36.25	2.24			0.27		
1	2522.86	95% exceedance	PC high tide	47	-0.22	4.1	4.1	269.18	106.02	0.17			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	2387.62	2 yr	EC low tide	1439	-3.27	5.69	5.77	632.38	111.67	2.28			0.13		
1	2387.62	2 yr	EC high tide	1439	-3.27	5.79	5.87	643.91	112.01	2.23			0.13		
1	2387.62	2 yr	PC low tide	1439	-3.27	5.69	5.77	632.38	111.67	2.28			0.13		
1	2387.62	2 yr	PC high tide	1439	-3.27	5.79	5.87	643.91	112.01	2.23			0.13		
1	2387.62	10 yr	EC low tide	3316	-3.27	8.82	8.99	1053.61	197.59	3.27	0.65	0.99	0.24	0.02	0.04
1	2387.62	10 yr	EC high tide	3316	-3.27	8.82	8.99	1053.61	197.59	3.27	0.65	0.99	0.24	0.02	0.04
1	2387.62	10 yr	PC low tide	3316	-3.27	8.82	8.99	1053.61	197.59	3.27	0.65	0.99	0.24	0.02	0.04
1	2387.62	10 yr	PC high tide	3316	-3.27	8.82	8.99	1053.61	197.59	3.27	0.65	0.99	0.24	0.02	0.04
1	2387.62	25 yr	EC low tide	4569	-3.27	10.66	10.85	1419.9	224.99	3.51	1.33	1.65	0.26	0.06	0.08
1	2387.62	25 yr	EC high tide	4569	-3.27	10.66	10.85	1419.9	224.99	3.51	1.33	1.65	0.26	0.06	0.08
1	2387.62	25 yr	PC low tide	4569	-3.27	10.66	10.85	1419.9	224.99	3.51	1.33	1.65	0.26	0.06	0.08
1	2387.62	25 yr	PC high tide	4569	-3.27	10.66	10.85	1419.9	224.99	3.51	1.33	1.65	0.26	0.06	0.08
1	2387.62	50 yr	EC low tide	5644	-3.27	12.17	12.36	1736	345.12	3.62	1.64	2	0.26	0.08	0.11
1	2387.62	50 yr	EC high tide	5644	-3.27	12.17	12.36	1736.01	345.12	3.62	1.64	2	0.26	0.08	0.11
1	2387.62	50 yr	PC low tide	5644	-3.27	12.17	12.36	1736	345.12	3.62	1.64	2	0.26	0.08	0.11
1	2387.62	50 yr	PC high tide	5644	-3.27	12.17	12.36	1736.01	345.12	3.62	1.64	2	0.26	0.08	0.11
1	2387.62	100 yr	EC low tide	6846	-3.27	13.84	14.03	2104.99	384.97	3.68	1.85	2.28	0.26	0.09	0.13
1	2387.62	100 yr	EC high tide	6846	-3.27	13.84	14.03	2104.98	384.97	3.68	1.85	2.28	0.26	0.09	0.13
1	2387.62	100 yr	PC low tide	6846	-3.27	13.84	14.03	2104.99	384.97	3.68	1.85	2.28	0.26	0.09	0.13
1	2387.62	100 yr	PC high tide	6846	-3.27	13.84	14.03	2104.98	384.97	3.68	1.85	2.28	0.26	0.09	0.13
1	2387.62	200 yr	EC low tide	8187	-3.27	15.76	15.94	2607.18	462.02	3.7	1.76	2.5	0.25	0.08	0.14
1	2387.62	200 yr	EC high tide	8187	-3.27	15.76	15.94	2607.16	462.01	3.7	1.76	2.5	0.25	0.08	0.14
1	2387.62	200 yr	PC low tide	8187	-3.27	15.76	15.94	2607.18	462.02	3.7	1.76	2.5	0.25	0.08	0.14
1	2387.62	200 yr	PC high tide	8187	-3.27	15.76	15.94	2607.16	462.01	3.7	1.76	2.5	0.25	0.08	0.14
1	2387.62	500 yr	EC low tide	10203	-3.27	18.93	19.08	3478.91	641.67	3.44	2.02	2.54	0.2	0.09	0.13
1	2387.62	500 yr	EC high tide	10203	-3.27	18.93	19.08	3478.91	641.67	3.44	2.02	2.54	0.2	0.09	0.13
1	2387.62	500 yr	PC low tide	10203	-3.27	18.93	19.08	3478.91	641.67	3.44	2.02	2.54	0.2	0.09	0.13
1	2387.62	500 yr	PC high tide	10203	-3.27	18.93	19.08	3478.91	641.67	3.44	2.02	2.54	0.2	0.09	0.13
1	2387.62	5% exceedance	EC low tide	1142	-3.27	5.07	5.14	564.37	109.64	2.02			0.11		
1	2387.62	5% exceedance	EC high tide	1142	-3.27	5.31	5.37	590.59	110.43	1.93			0.1		
1	2387.62	5% exceedance	PC low tide	1142	-3.27	5.07	5.14	564.37	109.64	2.02			0.11		
1	2387.62	5% exceedance	PC high tide	1142	-3.27	5.31	5.37	590.59	110.43	1.93			0.1		
1	2387.62	50% exceedance	EC low tide	288	-3.27	2.61	2.62	307.24	97.36	0.94			0.03		
1	2387.62	50% exceedance	EC high tide	288	-3.27	4.21	4.21	470.37	106.56	0.61			0.01		
1	2387.62	50% exceedance	PC low tide	288	-3.27	2.61	2.62	307.24	97.36	0.94			0.03		
1	2387.62	50% exceedance	PC high tide	288	-3.27	4.21	4.21	470.37	106.56	0.61			0.01		
1	2387.62	95% exceedance	EC low tide	47	-3.27	1.04	1.04	173.39	70.86	0.27			0		
1	2387.62	95% exceedance	EC high tide	47	-3.27	4.1	4.1	459.47	105.98	0.1			0		
1	2387.62	95% exceedance	PC low tide	47	-3.27	1.04	1.04	173.39	70.86	0.27			0		
1	2387.62	95% exceedance	PC high tide	47	-3.27	4.1	4.1	459.47	105.98	0.1			0		
1	2306.22	2 yr	EC low tide	1439	-0.56	5.51	5.71	401.4	98.53	3.58			0.33		
1	2306.22	2 yr	EC high tide	1439	-0.56	5.62	5.81	409.44	99.27	3.51			0.31		
1	2306.22	2 yr	PC low tide	1439	-0.56	5.51	5.71	401.4	98.53	3.58			0.33		
1	2306.22	2 yr	PC high tide	1439	-0.56	5.62	5.81	409.44	99.27	3.51			0.31		
1	2306.22	10 yr	EC low tide	3316	-0.56	8.45	8.9	613.54	118.39	5.4			0.65		
1	2306.22	10 yr	EC high tide	3316	-0.56	8.45	8.9	613.55	118.39	5.4			0.65		
1	2306.22	10 yr	PC low tide	3316	-0.56	8.45	8.9	613.54	118.39	5.4			0.65		
1	2306.22	10 yr	PC high tide	3316	-0.56	8.45	8.9	613.55	118.39	5.4			0.65		
1	2306.22	25 yr	EC low tide	4569	-0.56	10.15	10.75	736.91	130.27	6.2			0.8		
1	2306.22	25 yr	EC high tide	4569	-0.56	10.15	10.75	736.91	130.27	6.2			0.8		
1	2306.22	25 yr	PC low tide	4569	-0.56	10.15	10.75	736.91	130.27	6.2			0.8		
1	2306.22	25 yr	PC high tide	4569	-0.56	10.15	10.75	736.91	130.27	6.2			0.8		
1	2306.22	50 yr	EC low tide	5644	-0.56	11.55	12.25	837.84	139.8	6.74			0.91		
1	2306.22	50 yr	EC high tide	5644	-0.56	11.55	12.25	837.84	139.8	6.74			0.91		
1	2306.22	50 yr	PC low tide	5644	-0.56	11.55	12.25	837.84	139.8	6.74			0.91		
1	2306.22	50 yr	PC high tide	5644	-0.56	11.55	12.25	837.84	139.8	6.74			0.91		
1	2306.22	100 yr	EC low tide	6846	-0.56	13.12	13.92	951.13	153.76	7.2			1		
1	2306.22	100 yr	EC high tide	6846	-0.56	13.12	13.92	951.13	153.76	7.2			1		
1	2306.22	100 yr	PC low tide	6846	-0.56	13.12	13.92	951.13	153.76	7.2			1		
1	2306.22	100 yr	PC high tide	6846	-0.56	13.12	13.92	951.13	153.76	7.2			1		
1	2306.22	200 yr	EC low tide	8187	-0.56	14.94	15.83	1083.3	280.22	7.56			1.05		
1	2306.22	200 yr	EC high tide	8187	-0.56	14.94	15.83	1083.29	280.22	7.56			1.05		
1	2306.22	200 yr	PC low tide	8187	-0.56	14.94	15.83	1083.3	280.22	7.56			1.05		
1	2306.22	200 yr	PC high tide	8187	-0.56	14.94	15.83	1083.29	280.22	7.56			1.05		
1	2306.22	500 yr	EC low tide	10203	-0.56	18.02	18.97	1305.89	377.51	7.81			1.06		
1	2306.22	500 yr	EC high tide	10203	-0.56	18.02	18.97	1305.89	377.51	7.81			1.06		
1	2306.22	500 yr	PC low tide	10203	-0.56	18.02	18.97	1305.89	377.51	7.81			1.06		
1	2306.22	500 yr	PC high tide	10203	-0.56	18.02	18.97	1305.89	377.51	7.81			1.06		
1	2306.22	5% exceedance	EC low tide	1142	-0.56	4.93	5.09	359.38	94.63	3.18			0.27		
1	2306.22	5% exceedance	EC high tide	1142	-0.56	5.18	5.33	377.77	96.34	3.02			0.24		
1	2306.22	5% exceedance	PC low tide	1142	-0.56	4.93	5.09	359.38	94.63	3.18			0.27		
1	2306.22	5% exceedance	PC high tide	1142	-0.56	5.18	5.33	377.77	96.34	3.02			0.24		
1	2306.22	50% exceedance	EC low tide	288	-0.56	2.56	2.6	188.19	78.89	1.53			0.08		
1	2306.22	50% exceedance	EC high tide	288	-0.56	4.19	4.21	306.06	89.66	0.94			0.02		
1	2306.22	50% exceedance	PC low tide	288	-0.56	2.56	2.6	188.19	78.89	1.53			0.08		
1	2306.22	50% exceedance	PC high tide	288	-0.56	4.19	4.21	306.06	89.66	0.94			0.02		
1	2306.22	95% exceedance	EC low tide	47	-0.56	1.03	1.03	78.26	70.28	0.6			0.02		
1	2306.22	95% exceedance	EC high tide	47	-0.56	4.1	4.1	299.59	89.07	0.16			0		
1	2306.22	95% exceedance	PC low tide	47	-0.56	1.03	1.03	78.26	70.28	0.6			0.02		
1	2306.22	95% exceedance	PC high tide	47	-0.56	4.1	4.1	299.59	89.07	0.16			0		
1	2302.71			Choate Bridge											

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	2264.51	2 yr	EC low tide	1439	-0.57	5.2	5.51	323.7	77.7	4.45			0.54		
1	2264.51	2 yr	EC high tide	1439	-0.57	5.32	5.62	332.06	78.05	4.33			0.51		
1	2264.51	2 yr	PC low tide	1439	-0.57	5.2	5.51	323.7	77.7	4.45			0.54		
1	2264.51	2 yr	PC high tide	1439	-0.57	5.32	5.62	332.06	78.05	4.33			0.51		
1	2264.51	10 yr	EC low tide	3316	-0.57	7.47	8.22	476.08	83.84	6.97			1.17		
1	2264.51	10 yr	EC high tide	3316	-0.57	7.47	8.22	476.08	83.84	6.97			1.17		
1	2264.51	10 yr	PC low tide	3316	-0.57	7.47	8.22	476.08	83.84	6.97			1.17		
1	2264.51	10 yr	PC high tide	3316	-0.57	7.47	8.22	476.08	83.84	6.97			1.17		
1	2264.51	25 yr	EC low tide	4569	-0.57	8.62	9.68	553.79	92.07	8.26		0.26	1.58		0.04
1	2264.51	25 yr	EC high tide	4569	-0.57	8.62	9.68	553.79	92.07	8.26		0.26	1.58		0.04
1	2264.51	25 yr	PC low tide	4569	-0.57	8.62	9.68	553.79	92.07	8.26		0.26	1.58		0.04
1	2264.51	25 yr	PC high tide	4569	-0.57	8.62	9.68	553.79	92.07	8.26		0.26	1.58		0.04
1	2264.51	50 yr	EC low tide	5644	-0.57	9.44	10.77	614.43	99.85	9.27		0.87	1.92		0.22
1	2264.51	50 yr	EC high tide	5644	-0.57	9.44	10.77	614.43	99.85	9.27		0.87	1.92		0.22
1	2264.51	50 yr	PC low tide	5644	-0.57	9.44	10.77	614.43	99.85	9.27		0.87	1.92		0.22
1	2264.51	50 yr	PC high tide	5644	-0.57	9.44	10.77	614.43	99.85	9.27		0.87	1.92		0.22
1	2264.51	100 yr	EC low tide	6846	-0.57	10.27	11.91	675.81	105.62	10.29		1.39	2.3		0.45
1	2264.51	100 yr	EC high tide	6846	-0.57	10.27	11.91	675.8	105.62	10.29		1.39	2.3		0.45
1	2264.51	100 yr	PC low tide	6846	-0.57	10.27	11.91	675.81	105.62	10.29		1.39	2.3		0.45
1	2264.51	100 yr	PC high tide	6846	-0.57	10.27	11.91	675.8	105.62	10.29		1.39	2.3		0.45
1	2264.51	200 yr	EC low tide	8187	-0.57	11.15	13.12	741.07	112.4	11.27		1.88	2.69		0.72
1	2264.51	200 yr	EC high tide	8187	-0.57	11.15	13.12	741.06	112.39	11.27		1.88	2.69		0.72
1	2264.51	200 yr	PC low tide	8187	-0.57	11.15	13.12	741.07	112.4	11.27		1.88	2.69		0.72
1	2264.51	200 yr	PC high tide	8187	-0.57	11.15	13.12	741.06	112.39	11.27		1.88	2.69		0.72
1	2264.51	500 yr	EC low tide	10203	-0.57	12.57	14.94	846.11	126.92	12.38		2.53	3.11		1.14
1	2264.51	500 yr	EC high tide	10203	-0.57	12.57	14.94	846.11	126.92	12.38		2.53	3.11		1.14
1	2264.51	500 yr	PC low tide	10203	-0.57	12.57	14.94	846.11	126.92	12.38		2.53	3.11		1.14
1	2264.51	500 yr	PC high tide	10203	-0.57	12.57	14.94	846.11	126.92	12.38		2.53	3.11		1.14
1	2264.51	5% exceedance	EC low tide	1142	-0.57	4.69	4.93	289.31	76.33	3.95			0.44		
1	2264.51	5% exceedance	EC high tide	1142	-0.57	4.97	5.18	308.42	77.08	3.7			0.38		
1	2264.51	5% exceedance	PC low tide	1142	-0.57	4.69	4.93	289.31	76.33	3.95			0.44		
1	2264.51	5% exceedance	PC high tide	1142	-0.57	4.97	5.18	308.42	77.08	3.7			0.38		
1	2264.51	50% exceedance	EC low tide	288	-0.57	2.5	2.56	142.7	70.54	2.02			0.14		
1	2264.51	50% exceedance	EC high tide	288	-0.57	4.17	4.19	254.92	74.98	1.13			0.04		
1	2264.51	50% exceedance	PC low tide	288	-0.57	2.5	2.56	142.7	70.54	2.02			0.14		
1	2264.51	50% exceedance	PC high tide	288	-0.57	4.17	4.19	254.92	74.98	1.13			0.04		
1	2264.51	95% exceedance	EC low tide	47	-0.57	1.01	1.03	46.16	62.34	1.02			0.05		
1	2264.51	95% exceedance	EC high tide	47	-0.57	4.1	4.1	250.18	74.79	0.19			0		
1	2264.51	95% exceedance	PC low tide	47	-0.57	1.01	1.03	46.16	62.34	1.02			0.05		
1	2264.51	95% exceedance	PC high tide	47	-0.57	4.1	4.1	250.18	74.79	0.19			0		
1	2211.87	2 yr	EC low tide	1439	-0.19	5	5.38	294.03	77.15	4.89			0.7		
1	2211.87	2 yr	EC high tide	1439	-0.19	5.15	5.5	305.68	78.1	4.71	0.13		0.64	0.01	
1	2211.87	2 yr	PC low tide	1439	-0.19	5	5.38	294.03	77.15	4.89			0.7		
1	2211.87	2 yr	PC high tide	1439	-0.19	5.15	5.5	305.68	78.1	4.71	0.13		0.64	0.01	
1	2211.87	10 yr	EC low tide	3316	-0.19	7.32	8.05	489.94	92.47	6.86	0.92		1.24	0.19	
1	2211.87	10 yr	EC high tide	3316	-0.19	7.32	8.05	489.94	92.47	6.86	0.92		1.24	0.19	
1	2211.87	10 yr	PC low tide	3316	-0.19	7.32	8.05	489.94	92.47	6.86	0.92		1.24	0.19	
1	2211.87	10 yr	PC high tide	3316	-0.19	7.32	8.05	489.94	92.47	6.86	0.92		1.24	0.19	
1	2211.87	25 yr	EC low tide	4569	-0.19	8.51	9.44	605.43	102.37	7.75	1.24		1.72	0.33	
1	2211.87	25 yr	EC high tide	4569	-0.19	8.51	9.44	605.44	102.37	7.75	1.24		1.72	0.33	
1	2211.87	25 yr	PC low tide	4569	-0.19	8.51	9.44	605.43	102.37	7.75	1.24		1.72	0.33	
1	2211.87	25 yr	PC high tide	4569	-0.19	8.51	9.44	605.44	102.37	7.75	1.24		1.72	0.33	
1	2211.87	50 yr	EC low tide	5644	-0.19	9.39	10.47	698.9	108.89	8.4	1.55		2.11	0.49	
1	2211.87	50 yr	EC high tide	5644	-0.19	9.39	10.47	698.9	108.89	8.4	1.55		2.11	0.49	
1	2211.87	50 yr	PC low tide	5644	-0.19	9.39	10.47	698.9	108.89	8.4	1.55		2.11	0.49	
1	2211.87	50 yr	PC high tide	5644	-0.19	9.39	10.47	698.9	108.89	8.4	1.55		2.11	0.49	
1	2211.87	100 yr	EC low tide	6846	-0.19	10.38	11.52	854.91	262.01	8.71	1.91	3.23	2.45	0.69	0.43
1	2211.87	100 yr	EC high tide	6846	-0.19	10.38	11.52	854.9	262.01	8.71	1.91	3.23	2.46	0.69	0.43
1	2211.87	100 yr	PC low tide	6846	-0.19	10.38	11.52	854.91	262.01	8.71	1.91	3.23	2.45	0.69	0.43
1	2211.87	100 yr	PC high tide	6846	-0.19	10.38	11.52	854.9	262.01	8.71	1.91	3.23	2.46	0.69	0.43
1	2211.87	200 yr	EC low tide	8187	-0.19	11.48	12.64	1020.83	265.79	8.89	2.08	4.46	2.45	0.78	0.68
1	2211.87	200 yr	EC high tide	8187	-0.19	11.48	12.64	1020.81	265.79	8.89	2.08	4.46	2.45	0.78	0.68
1	2211.87	200 yr	PC low tide	8187	-0.19	11.48	12.64	1020.83	265.79	8.89	2.08	4.46	2.45	0.78	0.68
1	2211.87	200 yr	PC high tide	8187	-0.19	11.48	12.64	1020.81	265.79	8.89	2.08	4.46	2.45	0.78	0.68
1	2211.87	500 yr	EC low tide	10203	-0.19	13.22	14.36	1291.15	271.93	8.9	2.21	5.66	2.32	0.82	0.93
1	2211.87	500 yr	EC high tide	10203	-0.19	13.22	14.36	1291.14	271.93	8.9	2.21	5.66	2.32	0.82	0.93
1	2211.87	500 yr	PC low tide	10203	-0.19	13.22	14.36	1291.15	271.93	8.9	2.21	5.66	2.32	0.82	0.93
1	2211.87	500 yr	PC high tide	10203	-0.19	13.22	14.36	1291.14	271.93	8.9	2.21	5.66	2.32	0.82	0.93
1	2211.87	5% exceedance	EC low tide	1142	-0.19	4.49	4.81	255.18	75.33	4.48			0.61		
1	2211.87	5% exceedance	EC high tide	1142	-0.19	4.83	5.09	280.85	76.53	4.07			0.49		
1	2211.87	5% exceedance	PC low tide	1142	-0.19	4.49	4.81	255.18	75.33	4.48			0.61		
1	2211.87	5% exceedance	PC high tide	1142	-0.19	4.83	5.09	280.85	76.53	4.07			0.49		
1	2211.87	50% exceedance	EC low tide	288	-0.19	2.33	2.46	99.92	68.37	2.88			0.33		
1	2211.87	50% exceedance	EC high tide	288	-0.19	4.16	4.18	229.99	74.13	1.25			0.05		
1	2211.87	50% exceedance	PC low tide	288	-0.19	2.33	2.46	99.92	68.37	2.88			0.33		
1	2211.87	50% exceedance	PC high tide	288	-0.19	4.16	4.18	229.99	74.13	1.25			0.05		
1	2211.87	95% exceedance	EC low tide	47	-0.19	0.78	0.88	18.96	39.79	2.48			0.36		
1	2211.87	95% exceedance	EC high tide	47	-0.19	4.1	4.1	225.88	73.93	0.21			0		
1	2211.87	95% exceedance	PC low tide	47	-0.19	0.78	0.88	18.96	39.79	2.48			0.36		
1	2211.87	95% exceedance	PC high tide	47	-0.19	4.1	4.1	225.88	73.93	0.21			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	1943.54	2 yr	EC low tide	1439	-1.27	4.55	4.76	392.55	98.28	3.67			0.38		
1	1943.54	2 yr	EC high tide	1439	-1.27	4.77	4.96	414.21	100.45	3.47			0.34		
1	1943.54	2 yr	PC low tide	1439	-1.27	4.55	4.76	392.55	98.28	3.67			0.38		
1	1943.54	2 yr	PC high tide	1439	-1.27	4.77	4.96	414.21	100.45	3.47			0.34		
1	1943.54	10 yr	EC low tide	3316	-1.27	6.83	7.25	638.18	116.93	5.2			0.67		
1	1943.54	10 yr	EC high tide	3316	-1.27	6.83	7.25	638.19	116.93	5.2			0.67		
1	1943.54	10 yr	PC low tide	3316	-1.27	6.83	7.25	638.18	116.93	5.2			0.67		
1	1943.54	10 yr	PC high tide	3316	-1.27	6.83	7.25	638.19	116.93	5.2			0.67		
1	1943.54	25 yr	EC low tide	4569	-1.27	8.1	8.57	880.01	217.48	5.6		1.51	0.72		0.11
1	1943.54	25 yr	EC high tide	4569	-1.27	8.1	8.57	880.02	217.48	5.6		1.51	0.72		0.11
1	1943.54	25 yr	PC low tide	4569	-1.27	8.1	8.57	880.01	217.48	5.6		1.51	0.72		0.11
1	1943.54	25 yr	PC high tide	4569	-1.27	8.1	8.57	880.02	217.48	5.6		1.51	0.72		0.11
1	1943.54	50 yr	EC low tide	5644	-1.27	9.12	9.59	1116.19	244.4	5.69		2.07	0.73		0.17
1	1943.54	50 yr	EC high tide	5644	-1.27	9.12	9.59	1116.21	244.4	5.69		2.07	0.73		0.17
1	1943.54	50 yr	PC low tide	5644	-1.27	9.12	9.59	1116.19	244.4	5.69		2.07	0.73		0.17
1	1943.54	50 yr	PC high tide	5644	-1.27	9.12	9.59	1116.21	244.4	5.69		2.07	0.73		0.17
1	1943.54	100 yr	EC low tide	6846	-1.27	10.22	10.67	1397.34	269.49	5.67		2.46	0.72		0.21
1	1943.54	100 yr	EC high tide	6846	-1.27	10.22	10.67	1397.33	269.49	5.67		2.46	0.72		0.21
1	1943.54	100 yr	PC low tide	6846	-1.27	10.22	10.67	1397.34	269.49	5.67		2.46	0.72		0.21
1	1943.54	100 yr	PC high tide	6846	-1.27	10.22	10.67	1397.33	269.49	5.67		2.46	0.72		0.21
1	1943.54	200 yr	EC low tide	8187	-1.27	11.48	11.89	1765.81	306.66	5.53	0.21	2.62	0.66	0.02	0.22
1	1943.54	200 yr	EC high tide	8187	-1.27	11.48	11.89	1765.76	306.66	5.53	0.21	2.62	0.66	0.02	0.22
1	1943.54	200 yr	PC low tide	8187	-1.27	11.48	11.89	1765.81	306.66	5.53	0.21	2.62	0.66	0.02	0.22
1	1943.54	200 yr	PC high tide	8187	-1.27	11.48	11.89	1765.76	306.66	5.53	0.21	2.62	0.66	0.02	0.22
1	1943.54	500 yr	EC low tide	10203	-1.27	13.41	13.75	2373.57	335.61	5.14	0.3	2.98	0.55	0.03	0.24
1	1943.54	500 yr	EC high tide	10203	-1.27	13.41	13.75	2373.56	335.61	5.14	0.3	2.98	0.55	0.03	0.24
1	1943.54	500 yr	PC low tide	10203	-1.27	13.41	13.75	2373.57	335.61	5.14	0.3	2.98	0.55	0.03	0.24
1	1943.54	500 yr	PC high tide	10203	-1.27	13.41	13.75	2373.56	335.61	5.14	0.3	2.98	0.55	0.03	0.24
1	1943.54	5% exceedance	EC low tide	1142	-1.27	4.07	4.24	346.29	93.48	3.3			0.32		
1	1943.54	5% exceedance	EC high tide	1142	-1.27	4.54	4.67	391.41	98.17	2.92			0.24		
1	1943.54	5% exceedance	PC low tide	1142	-1.27	4.07	4.24	346.29	93.48	3.3			0.32		
1	1943.54	5% exceedance	PC high tide	1142	-1.27	4.54	4.67	391.41	98.17	2.92			0.24		
1	1943.54	50% exceedance	EC low tide	288	-1.27	2.07	2.11	178.07	74.3	1.62			0.09		
1	1943.54	50% exceedance	EC high tide	288	-1.27	4.13	4.14	352.09	94.1	0.82			0.02		
1	1943.54	50% exceedance	PC low tide	288	-1.27	2.07	2.11	178.07	74.3	1.62			0.09		
1	1943.54	50% exceedance	PC high tide	288	-1.27	4.13	4.14	352.09	94.1	0.82			0.02		
1	1943.54	95% exceedance	EC low tide	47	-1.27	0.74	0.75	89.48	59.7	0.53			0.01		
1	1943.54	95% exceedance	EC high tide	47	-1.27	4.1	4.1	349.34	93.81	0.13			0		
1	1943.54	95% exceedance	PC low tide	47	-1.27	0.74	0.75	89.48	59.7	0.53			0.01		
1	1943.54	95% exceedance	PC high tide	47	-1.27	4.1	4.1	349.34	93.81	0.13			0		
1	1823.46	2 yr	EC low tide	1439	-0.57	4.16	4.49	309.91	90.42	4.64			0.65		
1	1823.46	2 yr	EC high tide	1439	-0.57	4.45	4.74	336.9	91.67	4.27			0.54		
1	1823.46	2 yr	PC low tide	1439	-0.57	4.16	4.49	309.91	90.42	4.64			0.65		
1	1823.46	2 yr	PC high tide	1439	-0.57	4.45	4.74	336.9	91.67	4.27			0.54		
1	1823.46	10 yr	EC low tide	3316	-0.57	6.25	6.91	512.96	121.4	6.52		0.68	1.13		0.05
1	1823.46	10 yr	EC high tide	3316	-0.57	6.25	6.91	512.96	121.4	6.52		0.68	1.13		0.05
1	1823.46	10 yr	PC low tide	3316	-0.57	6.25	6.91	512.96	121.4	6.52		0.68	1.13		0.05
1	1823.46	10 yr	PC high tide	3316	-0.57	6.25	6.91	512.96	121.4	6.52		0.68	1.13		0.05
1	1823.46	25 yr	EC low tide	4569	-0.57	7.51	8.26	698.23	166.92	7.01		1.78	1.22		0.19
1	1823.46	25 yr	EC high tide	4569	-0.57	7.51	8.26	698.24	166.92	7.01		1.78	1.22		0.19
1	1823.46	25 yr	PC low tide	4569	-0.57	7.51	8.26	698.23	166.92	7.01		1.78	1.22		0.19
1	1823.46	25 yr	PC high tide	4569	-0.57	7.51	8.26	698.24	166.92	7.01		1.78	1.22		0.19
1	1823.46	50 yr	EC low tide	5644	-0.57	8.58	9.32	898	208.7	7.12	0.37	2.28	1.2	0.06	0.25
1	1823.46	50 yr	EC high tide	5644	-0.57	8.58	9.32	898.02	208.71	7.12	0.37	2.28	1.2	0.06	0.25
1	1823.46	50 yr	PC low tide	5644	-0.57	8.58	9.32	898	208.7	7.12	0.37	2.28	1.2	0.06	0.25
1	1823.46	50 yr	PC high tide	5644	-0.57	8.58	9.32	898.02	208.71	7.12	0.37	2.28	1.2	0.06	0.25
1	1823.46	100 yr	EC low tide	6846	-0.57	9.78	10.45	1163.05	271.28	6.95	0.77	2.83	1.09	0.16	0.32
1	1823.46	100 yr	EC high tide	6846	-0.57	9.78	10.45	1163.04	271.28	6.95	0.77	2.83	1.09	0.16	0.32
1	1823.46	100 yr	PC low tide	6846	-0.57	9.78	10.45	1163.05	271.28	6.95	0.77	2.83	1.09	0.16	0.32
1	1823.46	100 yr	PC high tide	6846	-0.57	9.78	10.45	1163.04	271.28	6.95	0.77	2.83	1.09	0.16	0.32
1	1823.46	200 yr	EC low tide	8187	-0.57	11.11	11.72	1477.94	341.36	6.77	1.06	2.99	0.99	0.24	0.32
1	1823.46	200 yr	EC high tide	8187	-0.57	11.11	11.72	1477.88	341.36	6.77	1.06	2.99	0.99	0.24	0.32
1	1823.46	200 yr	PC low tide	8187	-0.57	11.11	11.72	1477.94	341.36	6.77	1.06	2.99	0.99	0.24	0.32
1	1823.46	200 yr	PC high tide	8187	-0.57	11.11	11.72	1477.88	341.36	6.77	1.06	2.99	0.99	0.24	0.32
1	1823.46	500 yr	EC low tide	10203	-0.57	13.15	13.64	2031.34	369.62	6.24	1.24	3.27	0.79	0.28	0.32
1	1823.46	500 yr	EC high tide	10203	-0.57	13.15	13.64	2031.33	369.62	6.24	1.24	3.27	0.79	0.28	0.32
1	1823.46	500 yr	PC low tide	10203	-0.57	13.15	13.64	2031.34	369.62	6.24	1.24	3.27	0.79	0.28	0.32
1	1823.46	500 yr	PC high tide	10203	-0.57	13.15	13.64	2031.33	369.62	6.24	1.24	3.27	0.79	0.28	0.32
1	1823.46	5% exceedance	EC low tide	1142	-0.57	3.72	3.99	270.66	87.65	4.22			0.55		
1	1823.46	5% exceedance	EC high tide	1142	-0.57	4.32	4.51	324.9	91.12	3.51			0.37		
1	1823.46	5% exceedance	PC low tide	1142	-0.57	3.72	3.99	270.66	87.65	4.22			0.55		
1	1823.46	5% exceedance	PC high tide	1142	-0.57	4.32	4.51	324.9	91.12	3.51			0.37		
1	1823.46	50% exceedance	EC low tide	288	-0.57	1.9	1.98	123.32	73.03	2.34			0.21		
1	1823.46	50% exceedance	EC high tide	288	-0.57	4.11	4.13	306.11	90.24	0.94			0.03		
1	1823.46	50% exceedance	PC low tide	288	-0.57	1.9	1.98	123.32	73.03	2.34			0.21		
1	1823.46	50% exceedance	PC high tide	288	-0.57	4.11	4.13	306.11	90.24	0.94			0.03		
1	1823.46	95% exceedance	EC low tide	47	-0.57	0.7	0.71	45.08	55.4	1.04			0.05		
1	1823.46	95% exceedance	EC high tide	47	-0.57	4.1	4.1	304.87	90.18	0.15			0		
1	1823.46	95% exceedance	PC low tide	47	-0.57	0.7	0.71	45.08	55.4	1.04			0.05		
1	1823.46	95% exceedance	PC high tide	47	-0.57	4.1	4.1	304.87	90.18	0.15			0		

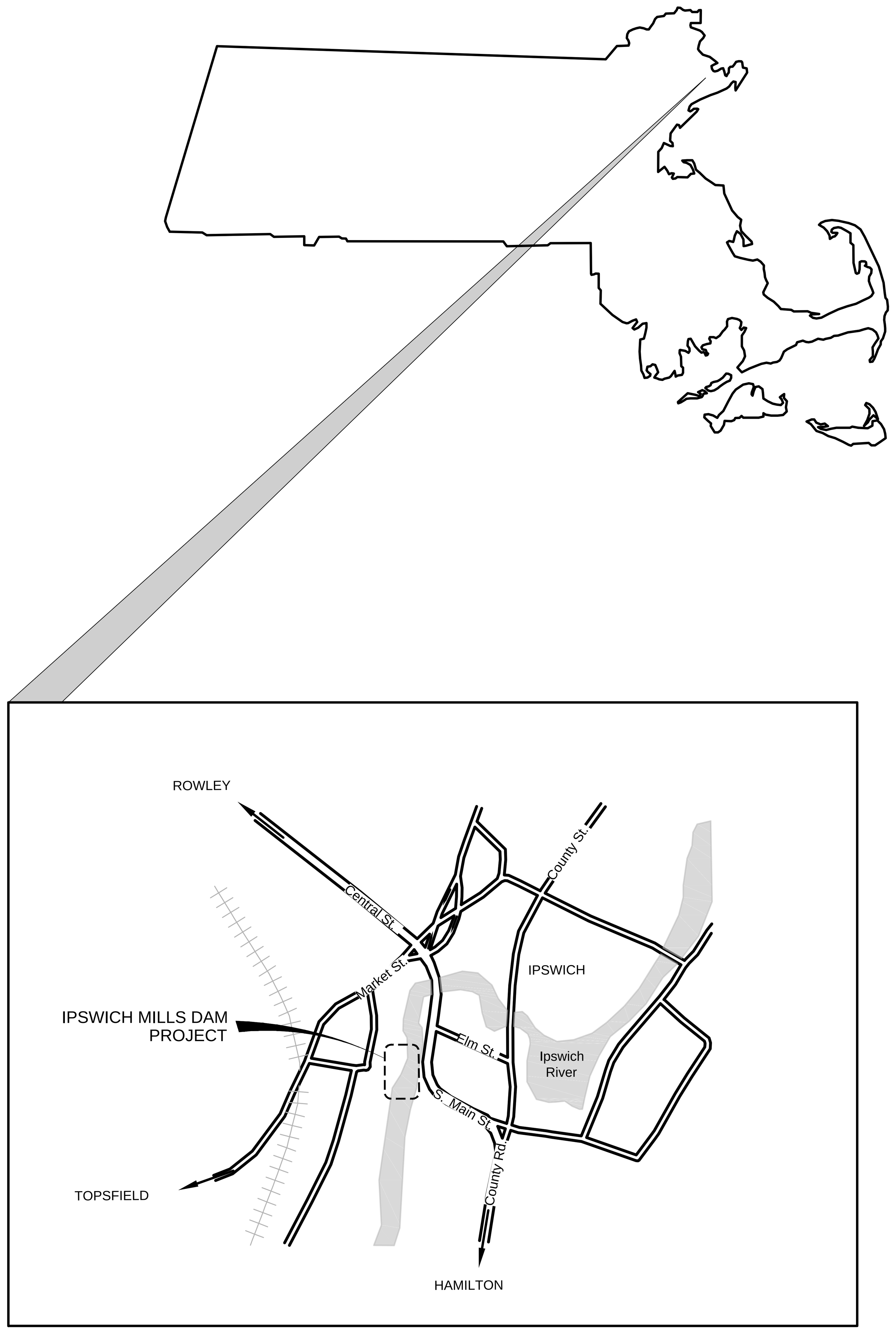
Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	1659.6	2 yr	EC low tide	1439	-3.66	4.24	4.29	755.56	172.26	1.9			0.1		
1	1659.6	2 yr	EC high tide	1439	-3.66	4.52	4.57	798.04	174.24	1.8			0.08		
1	1659.6	2 yr	PC low tide	1439	-3.66	4.24	4.29	755.56	172.26	1.9			0.1		
1	1659.6	2 yr	PC high tide	1439	-3.66	4.52	4.57	798.04	174.24	1.8			0.08		
1	1659.6	10 yr	EC low tide	3316	-3.66	6.43	6.58	1082.55	186.63	3.06			0.22		
1	1659.6	10 yr	EC high tide	3316	-3.66	6.43	6.58	1082.56	186.63	3.06			0.22		
1	1659.6	10 yr	PC low tide	3316	-3.66	6.43	6.58	1082.55	186.63	3.06			0.22		
1	1659.6	10 yr	PC high tide	3316	-3.66	6.43	6.58	1082.56	186.63	3.06			0.22		
1	1659.6	25 yr	EC low tide	4569	-3.66	7.71	7.91	1272.94	193.73	3.59			0.29		
1	1659.6	25 yr	EC high tide	4569	-3.66	7.71	7.91	1272.94	193.73	3.59			0.29		
1	1659.6	25 yr	PC low tide	4569	-3.66	7.71	7.91	1272.94	193.73	3.59			0.29		
1	1659.6	25 yr	PC high tide	4569	-3.66	7.71	7.91	1272.94	193.73	3.59			0.29		
1	1659.6	50 yr	EC low tide	5644	-3.66	8.75	8.99	1426.4	213.96	3.96			0.33		
1	1659.6	50 yr	EC high tide	5644	-3.66	8.75	8.99	1426.41	213.97	3.96			0.33		
1	1659.6	50 yr	PC low tide	5644	-3.66	8.75	8.99	1426.4	213.96	3.96			0.33		
1	1659.6	50 yr	PC high tide	5644	-3.66	8.75	8.99	1426.41	213.97	3.96			0.33		
1	1659.6	100 yr	EC low tide	6846	-3.66	9.88	10.16	1594.95	226.9	4.29			0.38		
1	1659.6	100 yr	EC high tide	6846	-3.66	9.88	10.16	1594.95	226.9	4.29			0.38		
1	1659.6	100 yr	PC low tide	6846	-3.66	9.88	10.16	1594.95	226.9	4.29			0.38		
1	1659.6	100 yr	PC high tide	6846	-3.66	9.88	10.16	1594.95	226.9	4.29			0.38		
1	1659.6	200 yr	EC low tide	8187	-3.66	11.15	11.47	1783.66	234.25	4.59			0.42		
1	1659.6	200 yr	EC high tide	8187	-3.66	11.15	11.47	1783.63	234.25	4.59			0.42		
1	1659.6	200 yr	PC low tide	8187	-3.66	11.15	11.47	1783.66	234.25	4.59			0.42		
1	1659.6	200 yr	PC high tide	8187	-3.66	11.15	11.47	1783.63	234.25	4.59			0.42		
1	1659.6	500 yr	EC low tide	10203	-3.66	13.09	13.47	2072.67	245.31	4.92			0.46		
1	1659.6	500 yr	EC high tide	10203	-3.66	13.09	13.47	2072.67	245.31	4.92			0.46		
1	1659.6	500 yr	PC low tide	10203	-3.66	13.09	13.47	2072.67	245.31	4.92			0.46		
1	1659.6	500 yr	PC high tide	10203	-3.66	13.09	13.47	2072.67	245.31	4.92			0.46		
1	1659.6	5% exceedance	EC low tide	1142	-3.66	3.78	3.82	687.43	168.88	1.66			0.08		
1	1659.6	5% exceedance	EC high tide	1142	-3.66	4.37	4.4	775.01	173.17	1.47			0.06		
1	1659.6	5% exceedance	PC low tide	1142	-3.66	3.78	3.82	687.43	168.88	1.66			0.08		
1	1659.6	5% exceedance	PC high tide	1142	-3.66	4.37	4.4	775.01	173.17	1.47			0.06		
1	1659.6	50% exceedance	EC low tide	288	-3.66	1.91	1.92	409.68	150.23	0.7			0.02		
1	1659.6	50% exceedance	EC high tide	288	-3.66	4.12	4.12	737.95	171.44	0.39			0		
1	1659.6	50% exceedance	PC low tide	288	-3.66	1.91	1.92	409.68	150.23	0.7			0.02		
1	1659.6	50% exceedance	PC high tide	288	-3.66	4.12	4.12	737.95	171.44	0.39			0		
1	1659.6	95% exceedance	EC low tide	47	-3.66	0.7	0.7	234.58	132.42	0.2			0		
1	1659.6	95% exceedance	EC high tide	47	-3.66	4.1	4.1	735.48	171.33	0.06			0		
1	1659.6	95% exceedance	PC low tide	47	-3.66	0.7	0.7	234.58	132.42	0.2			0		
1	1659.6	95% exceedance	PC high tide	47	-3.66	4.1	4.1	735.48	171.33	0.06			0		
1	1657.32			County Street Bridge											
1	1618.21	2 yr	EC low tide	1439	-0.81	4.07	4.23	452.98	190.02	3.18			0.34		
1	1618.21	2 yr	EC high tide	1439	-0.81	4.39	4.51	511.7	195.39	2.81			0.26		
1	1618.21	2 yr	PC low tide	1439	-0.81	4.07	4.23	452.98	190.02	3.18			0.34		
1	1618.21	2 yr	PC high tide	1439	-0.81	4.39	4.51	511.7	195.39	2.81			0.26		
1	1618.21	10 yr	EC low tide	3316	-0.81	5.99	6.25	807.1	207.78	4.11			0.47		
1	1618.21	10 yr	EC high tide	3316	-0.81	5.99	6.25	807.11	207.78	4.11			0.47		
1	1618.21	10 yr	PC low tide	3316	-0.81	5.99	6.25	807.1	207.78	4.11			0.47		
1	1618.21	10 yr	PC high tide	3316	-0.81	5.99	6.25	807.11	207.78	4.11			0.47		
1	1618.21	25 yr	EC low tide	4569	-0.81	6.98	7.31	988.48	214.91	4.62			0.55		
1	1618.21	25 yr	EC high tide	4569	-0.81	6.98	7.31	988.5	214.91	4.62			0.55		
1	1618.21	25 yr	PC low tide	4569	-0.81	6.98	7.31	988.48	214.91	4.62			0.55		
1	1618.21	25 yr	PC high tide	4569	-0.81	6.98	7.31	988.5	214.91	4.62			0.55		
1	1618.21	50 yr	EC low tide	5644	-0.81	7.71	8.1	1123.44	220.11	5.02			0.63		
1	1618.21	50 yr	EC high tide	5644	-0.81	7.71	8.1	1123.45	220.11	5.02			0.63		
1	1618.21	50 yr	PC low tide	5644	-0.81	7.71	8.1	1123.44	220.11	5.02			0.63		
1	1618.21	50 yr	PC high tide	5644	-0.81	7.71	8.1	1123.45	220.11	5.02			0.63		
1	1618.21	100 yr	EC low tide	6846	-0.81	8.46	8.91	1261.54	225.35	5.43			0.7		
1	1618.21	100 yr	EC high tide	6846	-0.81	8.46	8.91	1261.53	225.35	5.43			0.7		
1	1618.21	100 yr	PC low tide	6846	-0.81	8.46	8.91	1261.54	225.35	5.43			0.7		
1	1618.21	100 yr	PC high tide	6846	-0.81	8.46	8.91	1261.53	225.35	5.43			0.7		
1	1618.21	200 yr	EC low tide	8187	-0.81	9.22	9.75	1402.98	230.66	5.84			0.79		
1	1618.21	200 yr	EC high tide	8187	-0.81	9.22	9.75	1402.93	230.66	5.84			0.79		
1	1618.21	200 yr	PC low tide	8187	-0.81	9.22	9.75	1402.98	230.66	5.84			0.79		
1	1618.21	200 yr	PC high tide	8187	-0.81	9.22	9.75	1402.93	230.66	5.84			0.79		
1	1618.21	500 yr	EC low tide	10203	-0.81	10.27	10.9	1595.96	237.83	6.39			0.9		
1	1618.21	500 yr	EC high tide	10203	-0.81	10.27	10.9	1595.96	237.83	6.39			0.9		
1	1618.21	500 yr	PC low tide	10203	-0.81	10.27	10.9	1595.96	237.83	6.39			0.9		
1	1618.21	500 yr	PC high tide	10203	-0.81	10.27	10.9	1595.96	237.83	6.39			0.9		
1	1618.21	5% exceedance	EC low tide	1142	-0.81	3.63	3.77	376.35	163.43	3.03			0.32		
1	1618.21	5% exceedance	EC high tide	1142	-0.81	4.28	4.36	491.05	193.47	2.33			0.18		
1	1618.21	5% exceedance	PC low tide	1142	-0.81	3.63	3.77	376.35	163.43	3.03			0.32		
1	1618.21	5% exceedance	PC high tide	1142	-0.81	4.28	4.36	491.05	193.47	2.33			0.18		
1	1618.21	50% exceedance	EC low tide	288	-0.81	1.85	1.91	144.11	96.78	2			0.16		
1	1618.21	50% exceedance	EC high tide	288	-0.81	4.11	4.12	460.37	190.69	0.63			0.01		
1	1618.21	50% exceedance	PC low tide	288	-0.81	1.85	1.91	144.11	96.78	2			0.16		
1	1618.21	50% exceedance	PC high tide	288	-0.81	4.11	4.12	460.37	190.69	0.63			0.01		
1	1618.21	95% exceedance	EC low tide	47	-0.81	0.69	0.7	46.12	64.46	1.02			0.05		
1	1618.21	95% exceedance	EC high tide	47	-0.81	4.1	4.1	458.43	190.51	0.1			0		
1	1618.21	95% exceedance	PC low tide	47	-0.81	0.69	0.7	46.12	64.46	1.02			0.05		
1	1618.21	95% exceedance	PC high tide	47	-0.81	4.1	4.1	458.43	190.51	0.1			0		

Reach	River Station	Profile	Plan	Q Total	Min Ch El	W.S. Elev	E.G. Elev	Flow Area	Top Width	Vel Chnl	Vel Left	Vel Right	Shear Chan	Shear LOB	Shear ROB
				(cfs)	(ft)	(ft)	(ft)	(sq ft)	(ft)	(ft/s)	(ft/s)	(ft/s)	(lb/sq ft)	(lb/sq ft)	(lb/sq ft)
1	1529.18	2 yr	EC low tide	1439	-0.51	2.72	3.73	178.87	88.74	7.92		8.29	2.29		2.46
1	1529.18	2 yr	EC high tide	1439	-0.51	3.91	4.28	294.85	108.56	4.78		5.08	0.76		0.83
1	1529.18	2 yr	PC low tide	1439	-0.51	2.72	3.73	178.87	88.74	7.92		8.29	2.29		2.46
1	1529.18	2 yr	PC high tide	1439	-0.51	3.91	4.28	294.85	108.56	4.78		5.08	0.76		0.83
1	1529.18	10 yr	EC low tide	3316	-0.51	4.3	5.8	338.45	112.78	9.65		10.1	3		3.21
1	1529.18	10 yr	EC high tide	3316	-0.51	4.3	5.8	338.45	112.78	9.65		10.1	3		3.21
1	1529.18	10 yr	PC low tide	3316	-0.51	4.3	5.8	338.45	112.78	9.65		10.1	3		3.21
1	1529.18	10 yr	PC high tide	3316	-0.51	4.3	5.8	338.45	112.78	9.65		10.1	3		3.21
1	1529.18	25 yr	EC low tide	4569	-0.51	5.07	6.84	428.13	120.91	10.58		10.86	3.41		3.54
1	1529.18	25 yr	EC high tide	4569	-0.51	5.07	6.84	428.13	120.91	10.58		10.86	3.41		3.54
1	1529.18	25 yr	PC low tide	4569	-0.51	5.07	6.84	428.13	120.91	10.58		10.86	3.41		3.54
1	1529.18	25 yr	PC high tide	4569	-0.51	5.07	6.84	428.13	120.91	10.58		10.86	3.41		3.54
1	1529.18	50 yr	EC low tide	5644	-0.51	5.67	7.62	502.98	128.15	11.16		11.35	3.66		3.75
1	1529.18	50 yr	EC high tide	5644	-0.51	5.67	7.62	502.98	128.15	11.16		11.35	3.66		3.75
1	1529.18	50 yr	PC low tide	5644	-0.51	5.67	7.62	502.98	128.15	11.16		11.35	3.66		3.75
1	1529.18	50 yr	PC high tide	5644	-0.51	5.67	7.62	502.98	128.15	11.16		11.35	3.66		3.75
1	1529.18	100 yr	EC low tide	6846	-0.51	6.24	8.42	576.81	131.99	11.9		11.8	4		3.96
1	1529.18	100 yr	EC high tide	6846	-0.51	6.24	8.42	576.81	131.99	11.9		11.8	4		3.96
1	1529.18	100 yr	PC low tide	6846	-0.51	6.24	8.42	576.81	131.99	11.9		11.8	4		3.96
1	1529.18	100 yr	PC high tide	6846	-0.51	6.24	8.42	576.81	131.99	11.9		11.8	4		3.96
1	1529.18	200 yr	EC low tide	8187	-0.51	6.83	9.25	656.16	135.96	12.59		12.24	4.33		4.15
1	1529.18	200 yr	EC high tide	8187	-0.51	6.83	9.25	656.16	135.96	12.59		12.24	4.33		4.15
1	1529.18	200 yr	PC low tide	8187	-0.51	6.83	9.25	656.16	135.96	12.59		12.24	4.33		4.15
1	1529.18	200 yr	PC high tide	8187	-0.51	6.83	9.25	656.16	135.96	12.59		12.24	4.33		4.15
1	1529.18	500 yr	EC low tide	10203	-0.51	7.65	10.38	770.51	141.81	13.47		12.76	4.75		4.38
1	1529.18	500 yr	EC high tide	10203	-0.51	7.65	10.38	770.51	141.81	13.47		12.76	4.75		4.38
1	1529.18	500 yr	PC low tide	10203	-0.51	7.65	10.38	770.51	141.81	13.47		12.76	4.75		4.38
1	1529.18	500 yr	PC high tide	10203	-0.51	7.65	10.38	770.51	141.81	13.47		12.76	4.75		4.38
1	1529.18	5% exceedance	EC low tide	1142	-0.51	2.4	3.29	151.29	84.94	7.38		7.87	2.08		2.29
1	1529.18	5% exceedance	EC high tide	1142	-0.51	3.99	4.21	303.52	109.42	3.69		3.91	0.45		0.49
1	1529.18	5% exceedance	PC low tide	1142	-0.51	2.4	3.29	151.29	84.94	7.38		7.87	2.08		2.29
1	1529.18	5% exceedance	PC high tide	1142	-0.51	3.99	4.21	303.52	109.42	3.69		3.91	0.45		0.49
1	1529.18	50% exceedance	EC low tide	288	-0.51	1.05	1.51	52.87	57.51	5.42		5.49	1.38		1.41
1	1529.18	50% exceedance	EC high tide	288	-0.51	4.09	4.11	314.88	110.53	0.9		0.95	0.03		0.03
1	1529.18	50% exceedance	PC low tide	288	-0.51	1.05	1.51	52.87	57.51	5.42		5.49	1.38		1.41
1	1529.18	50% exceedance	PC high tide	288	-0.51	4.09	4.11	314.88	110.53	0.9		0.95	0.03		0.03
1	1529.18	95% exceedance	EC low tide	47	-0.51	0.23	0.41	14.1	36.2	3.66		2.56	0.78		0.46
1	1529.18	95% exceedance	EC high tide	47	-0.51	4.1	4.1	315.56	110.59	0.15		0.15	0		0
1	1529.18	95% exceedance	PC low tide	47	-0.51	0.23	0.41	14.1	36.2	3.66		2.56	0.78		0.46
1	1529.18	95% exceedance	PC high tide	47	-0.51	4.1	4.1	315.56	110.59	0.15		0.15	0		0
1	1369.76	2 yr	EC low tide	1439	-3.63	-1.28	0.14	151.47	135.7	9		10.1	3.64		4.32
1	1369.76	2 yr	EC high tide	1439	-3.63	4.1	4.13	1048.31	179.3	1.37		1.38	0.05		0.05
1	1369.76	2 yr	PC low tide	1439	-3.63	-1.28	0.14	151.47	135.7	9		10.1	3.64		4.32
1	1369.76	2 yr	PC high tide	1439	-3.63	4.1	4.13	1048.31	179.3	1.37		1.38	0.05		0.05
1	1369.76	10 yr	EC low tide	3316	-3.63	-0.59	2.13	252.34	157.74	12.31		14.25	6.07		7.56
1	1369.76	10 yr	EC high tide	3316	-3.63	4.1	4.26	1048.31	179.3	3.15		3.19	0.25		0.26
1	1369.76	10 yr	PC low tide	3316	-3.63	-0.59	2.13	252.34	157.74	12.31		14.25	6.07		7.56
1	1369.76	10 yr	PC high tide	3316	-3.63	4.1	4.26	1048.31	179.3	3.15		3.19	0.25		0.26
1	1369.76	25 yr	EC low tide	4569	-3.63	-0.23	3.19	309.69	161.31	13.96		15.86	7.32		8.88
1	1369.76	25 yr	EC high tide	4569	-3.63	4.1	4.4	1048.31	179.3	4.34		4.39	0.48		0.49
1	1369.76	25 yr	PC low tide	4569	-3.63	-0.23	3.19	309.69	161.31	13.96		15.86	7.32		8.88
1	1369.76	25 yr	PC high tide	4569	-3.63	4.1	4.4	1048.31	179.3	4.34		4.39	0.48		0.49
1	1369.76	50 yr	EC low tide	5644	-3.63	0.05	4.01	355	162.55	15.17		16.94	8.27		9.75
1	1369.76	50 yr	EC high tide	5644	-3.63	4.1	4.55	1048.31	179.3	5.36		5.43	0.74		0.75
1	1369.76	50 yr	PC low tide	5644	-3.63	0.05	4.01	355	162.55	15.17		16.94	8.27		9.75
1	1369.76	50 yr	PC high tide	5644	-3.63	4.1	4.55	1048.31	179.3	5.36		5.43	0.74		0.75
1	1369.76	100 yr	EC low tide	6846	-3.63	0.34	4.84	403.72	163.87	16.3		17.92	9.15		10.54
1	1369.76	100 yr	EC high tide	6846	-3.63	4.1	4.76	1048.31	179.3	6.5		6.58	1.09		1.11
1	1369.76	100 yr	PC low tide	6846	-3.63	0.34	4.84	403.72	163.87	16.3		17.92	9.15		10.54
1	1369.76	100 yr	PC high tide	6846	-3.63	4.1	4.76	1048.31	179.3	6.5		6.58	1.09		1.11
1	1369.76	200 yr	EC low tide	8187	-3.63	0.66	5.7	455.3	165.18	17.39		18.86	10.02		11.32
1	1369.76	200 yr	EC high tide	8187	-3.63	0.66	5.7	455.3	165.18	17.39		18.86	10.02		11.32
1	1369.76	200 yr	PC low tide	8187	-3.63	0.66	5.7	455.3	165.18	17.39		18.86	10.02		11.32
1	1369.76	200 yr	PC high tide	8187	-3.63	0.66	5.7	455.3	165.18	17.39		18.86	10.02		11.32
1	1369.76	500 yr	EC low tide	10203	-3.63	1.1	6.9	529.03	167.04	18.77		20.07	11.13		12.3
1	1369.76	500 yr	EC high tide	10203	-3.63	1.1	6.9	529.03	167.04	18.77		20.07	11.13		12.3
1	1369.76	500 yr	PC low tide	10203	-3.63	1.1	6.9	529.03	167.04	18.77		20.07	11.13		12.3
1	1369.76	500 yr	PC high tide	10203	-3.63	1.1	6.9	529.03	167.04	18.77		20.07	11.13		12.3
1	1369.76	5% exceedance	EC low tide	1142	-3.63	-1.42	-0.25	132.24	130.46	8.19		9.16	3.11		3.67
1	1369.76	5% exceedance	EC high tide	1142	-3.63	4.1	4.12	1048.31	179.3	1.08		1.1	0.03		0.03
1	1369.76	5% exceedance	PC low tide	1142	-3.63	-1.42	-0.25	132.23	130.46	8.19		9.16	3.11		3.68
1	1369.76	5% exceedance	PC high tide	1142	-3.63	4.1	4.12	1048.31	179.3	1.08		1.1	0.03		0.03
1	1369.76	50% exceedance	EC low tide	288	-3.63	-1.95	-1.69	70.07	100.89	3.97		4.26	0.82		0.91
1	1369.76	50% exceedance	EC high tide	288	-3.63	4.1	4.1	1048.31	179.3	0.27		0.28	0		0
1	1369.76	50% exceedance	PC low tide	288	-3.63	-1.95	-1.69	70.07	100.89	3.97		4.26	0.82		0.91
1	1369.76	50% exceedance	PC high tide	288	-3.63	4.1	4.1	1048.31	179.3	0.27		0.28	0		0
1	1369.76	95% exceedance	EC low tide	47	-3.63	-2.68	-2.56	16.95	44.01	2.86		2.68	0.5		0.45
1	1369.76	95% exceedance	EC high tide	47	-3.63	4.1	4.1	1048.31	179.3	0.04		0.05	0		0
1	1369.76	95% exceedance	PC low tide	47	-3.63	-2.68	-2.56	16.95	44.01	2.86		2.68	0.5		0.45
1	1369.76	95% exceedance	PC high tide	47	-3.63	4.1	4.1	1048.31	179.3	0.04		0.05	0		0

ATTACHMENT 8

IPSWICH RIVER
IPSWICH MILLS DAM REMOVAL
CONCEPT PLANS
TOWN OF IPSWICH, MASSACHUSETTS

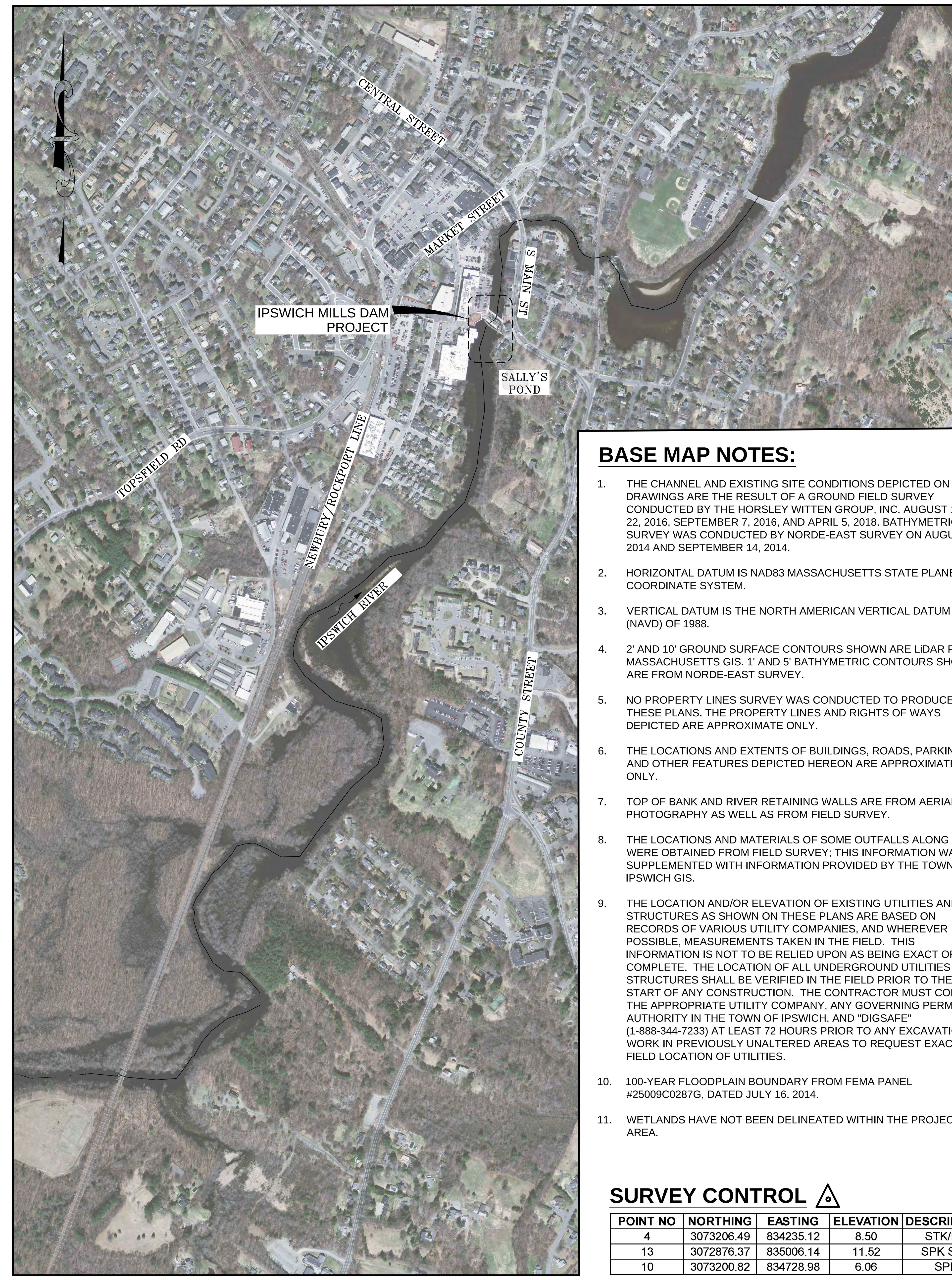
December 10, 2018



VICINITY MAP

NOT TO SCALE

SHEET INDEX



SITE MAP

SCALE: 1" = 500'

BASE MAP NOTES:

1. THE CHANNEL AND EXISTING SITE CONDITIONS DEPICTED ON THESE DRAWINGS ARE THE RESULT OF A GROUND FIELD SURVEY CONDUCTED BY THE HORSLEY WITTEN GROUP, INC. AUGUST 17 & 22, 2016, SEPTEMBER 7, 2016, AND APRIL 5, 2018. BATHYMETRIC SURVEY WAS CONDUCTED BY NORDE-EAST SURVEY ON AUGUST 26, 2014 AND SEPTEMBER 14, 2014.
2. HORIZONTAL DATUM IS NAD83 MASSACHUSETTS STATE PLANE COORDINATE SYSTEM.
3. VERTICAL DATUM IS THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988.
4. 2' AND 10' GROUND SURFACE CONTOURS SHOWN ARE LIDAR FROM MASSACHUSETTS GIS. 1' AND 5' BATHYMETRIC CONTOURS SHOWN ARE FROM NORDE-EAST SURVEY.
5. NO PROPERTY LINES SURVEY WAS CONDUCTED TO PRODUCE THESE PLANS. THE PROPERTY LINES AND RIGHTS OF WAYS DEPICTED ARE APPROXIMATE ONLY.
6. THE LOCATIONS AND EXTENTS OF BUILDINGS, ROADS, PARKING, AND OTHER FEATURES DEPICTED HEREON ARE APPROXIMATE ONLY.
7. TOP OF BANK AND RIVER RETAINING WALLS ARE FROM AERIAL PHOTOGRAPHY AS WELL AS FROM FIELD SURVEY.
8. THE LOCATIONS AND MATERIALS OF SOME OUTFALLS ALONG RIVER WERE OBTAINED FROM FIELD SURVEY; THIS INFORMATION WAS SUPPLEMENTED WITH INFORMATION PROVIDED BY THE TOWN OF IPSWICH GIS.
9. THE LOCATION AND/OR ELEVATION OF EXISTING UTILITIES AND STRUCTURES AS SHOWN ON THESE PLANS ARE BASED ON RECORDS OF VARIOUS UTILITY COMPANIES, AND WHEREVER POSSIBLE, MEASUREMENTS TAKEN IN THE FIELD. THIS INFORMATION IS NOT TO BE RELIED UPON AS BEING EXACT OR COMPLETE. THE LOCATION OF ALL UNDERGROUND UTILITIES AND STRUCTURES SHALL BE VERIFIED IN THE FIELD PRIOR TO THE START OF ANY CONSTRUCTION. THE CONTRACTOR MUST CONTACT THE APPROPRIATE UTILITY COMPANY, ANY GOVERNING PERMITTING AUTHORITY IN THE TOWN OF IPSWICH, AND "DIGSAFE" (1-888-344-7233) AT LEAST 72 HOURS PRIOR TO ANY EXCAVATION WORK IN PREVIOUSLY UNALTERED AREAS TO REQUEST EXACT FIELD LOCATION OF UTILITIES.
10. 100-YEAR FLOODPLAIN BOUNDARY FROM FEMA PANEL #25009C0287G, DATED JULY 16, 2014.
11. WETLANDS HAVE NOT BEEN DELINEATED WITHIN THE PROJECT AREA.

SURVEY CONTROL

POINT NO	NORTHING	EASTING	ELEVATION	DESCRIPTION
4	3073206.49	834235.12	8.50	STK/PK
13	3072876.37	835006.14	11.52	SPK SET
10	3073200.82	834728.98	6.06	SPK

Funded By: MASSACHUSETTS ENVIRONMENTAL TRUST

Checked By: MB
Drawn By: SJ
Designed By: KC, CC
Date: December 10, 2018

inter-fluve
220 Concord Ave, 2nd Floor
Cambridge, MA 02138
617.714.5537
www.interfluve.com

Plan Set: IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title: PROJECT LOCATION AND SHEET INDEX

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverways Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
25 Green Street
Ipswich, MA 01938
978-356-6600

Survey Provided By:
Horsley Witten Group, Inc.
90 Route 6A
Sandwich, MA 02563
Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041
Sheet: 1 of 8
Sheet Number: C - 1

LEGEND

FLOOD	FEMA FLOODPLAIN BOUNDARY
	NORDE-EAST SURVEY 1' CONTOUR
	NORDE-EAST SURVEY 5' CONTOUR
	LiDAR 2' CONTOUR
	LiDAR 10' CONTOUR
15+00	THALWEG CENTERLINE ALIGNMENT
	HW SURVEY TRANSECT
W	WATER LINE
S	SANITARY SEWER LINE
	BROOK / STREAM
12" PVC	SURVEY-LOCATED OUTFALL
OUTFALL	GIS-LOCATED OUTFALL

GRAPHIC SCALE

90 0 45 90 180 360

(in feet)
1 INCH = 90 FEET

Funded By:
MASSACHUSETTS ENVIRONMENTAL TRUST

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508-833-3150 fax

Date: December 10, 2018
Designed By: -
Drawn By: MCL
Checked By: NP

Plan Set:
IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title:
EXISTING CONDITIONS PLAN (1)

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
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Ipswich, MA 01938
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Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041
Sheet : 2 of 8
Sheet Number: EX - 1

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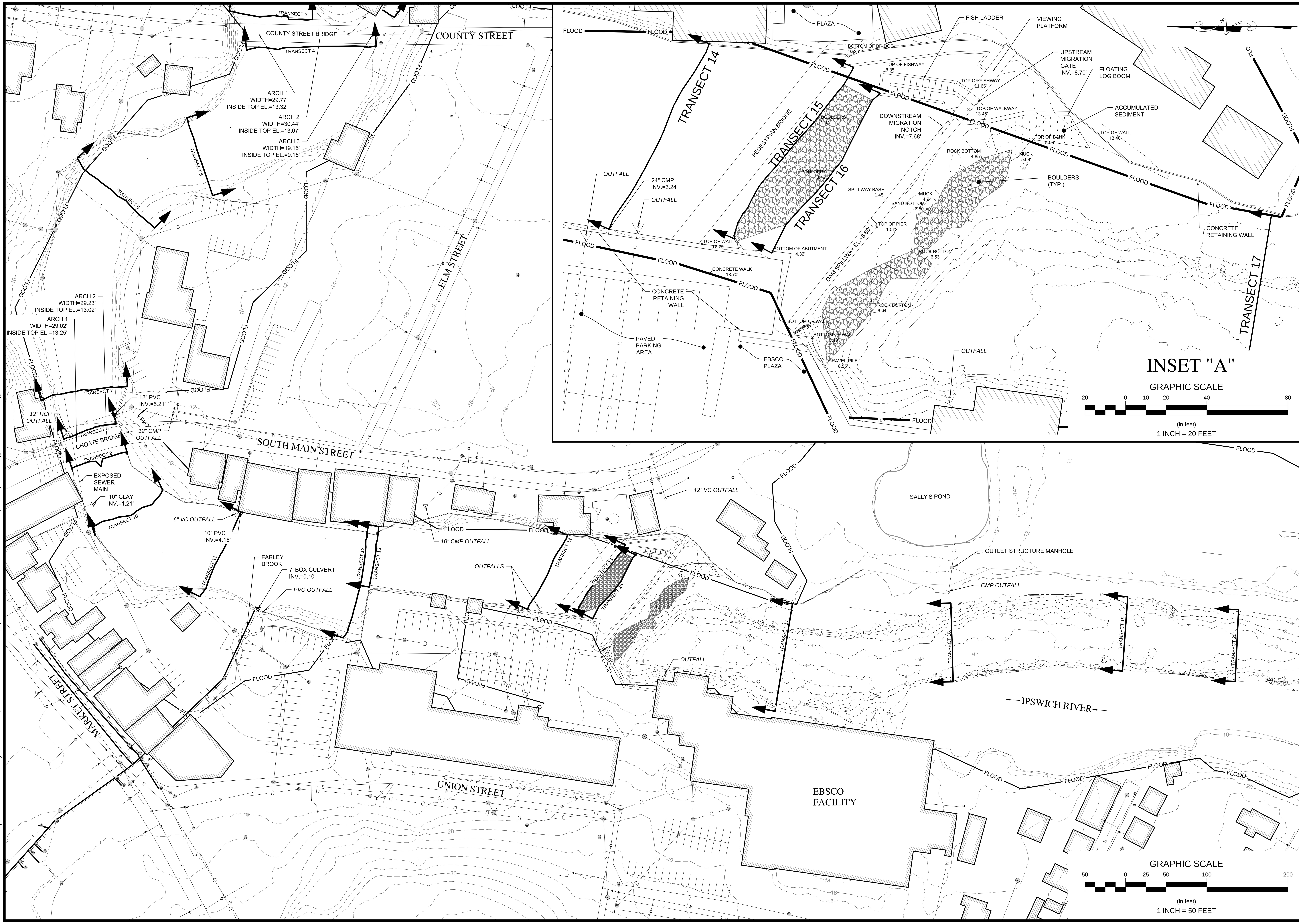

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508-833-3150 fax

Designed By: -
Drawn By: MCL
Checked By: NP
Date: December 10, 2018

Plan Set: IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title: EXISTING CONDITIONS PLAN (2)

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverways Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
25 Green Street
Ipswich, MA 01938
978-356-6600

Survey Provided By:
Horsley Witten Group, Inc.
90 Route 6A
Sandwich, MA 02563
Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041
Sheet: 3 of 8
Sheet Number: EX - 2

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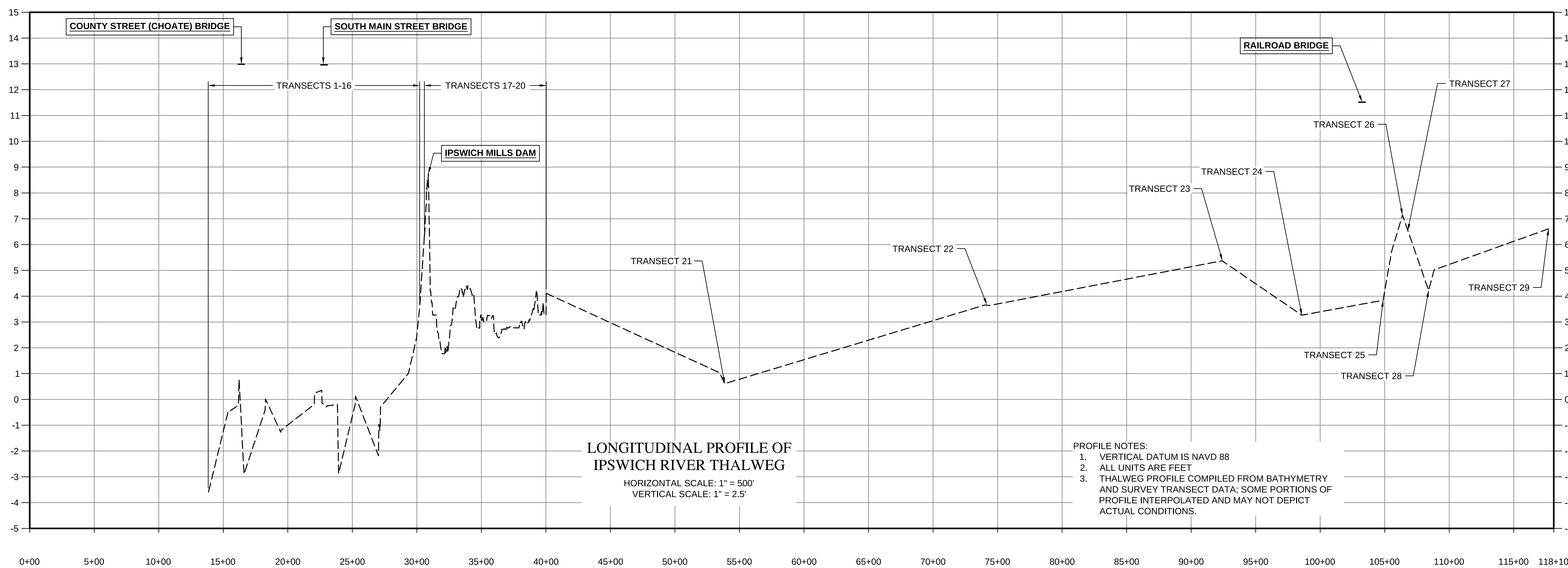

LONGITUDINAL PROFILE OF
IPSWICH RIVER THALWEG

HORIZONTAL SCALE: 1" = 500'
VERTICAL SCALE: 1" = 2.5'

PROFILE NOTES:
1. VERTICAL DATUM IS NAVD 88
2. ALL UNITS ARE FEET
3. THALWEG PROFILE COMPILED FROM BATHYMETRY AND SURVEY TRANSECT DATA; SOME PORTIONS OF PROFILE INTERPOLATED AND MAY NOT DEPICT ACTUAL CONDITIONS.

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508-833-3150 fax

Date: December 10, 2018
Designed By: -
Drawn By: MCL
Checked By: NP

Plan Set:
IPSWICH DAM REMOVAL
FEASIBILITY STUDY
IPSWICH, MASSACHUSETTS

Plan Title:
RIVER PROFILE

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
25 Green Street
Ipswich, MA 01938
978-356-6600

Survey Provided By:
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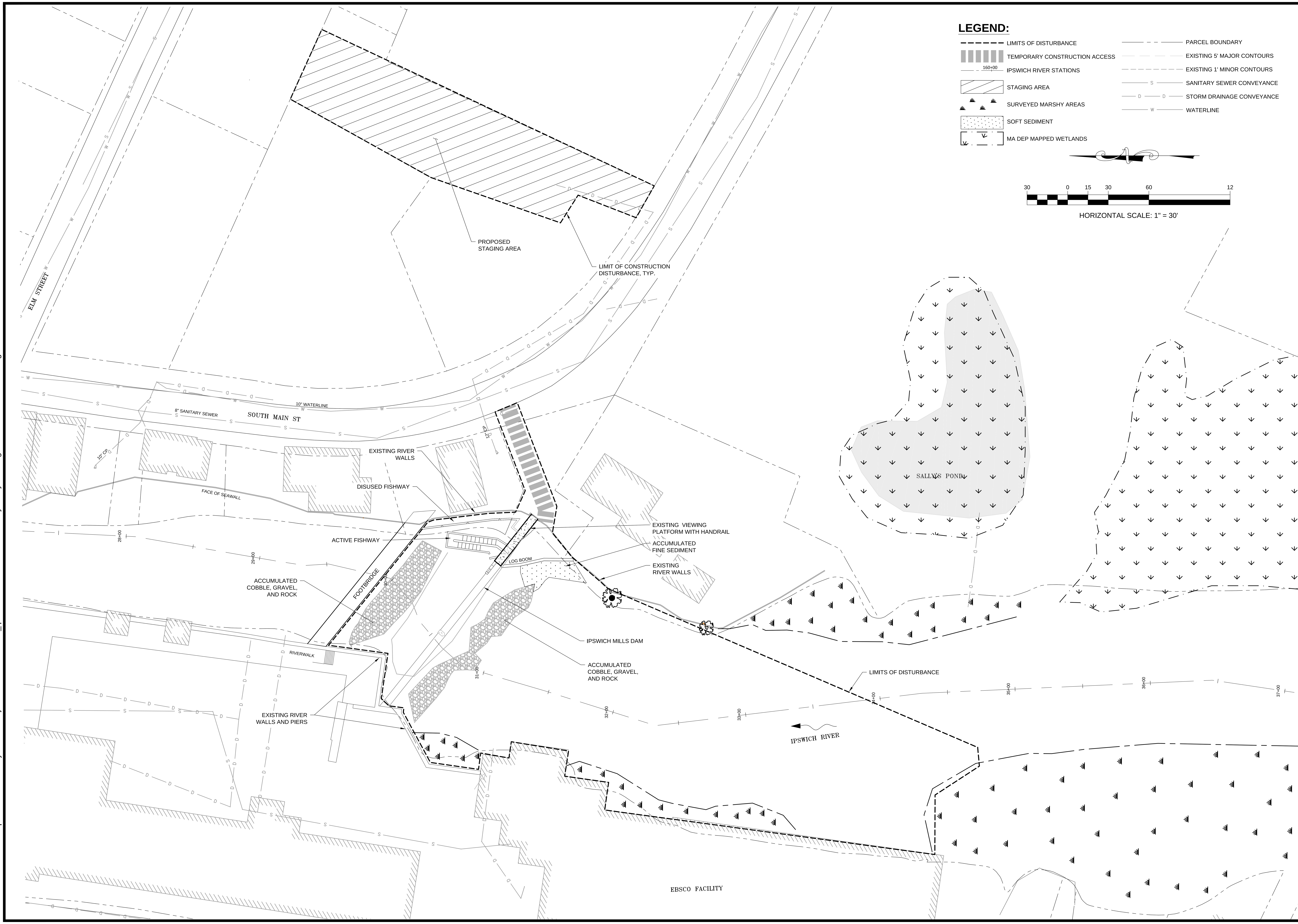
Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041
Sheet: 4 of 8

Sheet Number:
EX - 3



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Checked By: MB

Drawn By: SJ

Designed By: KC, CC

Date: December 10, 2018

Plan Set:

IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title:

DAM REMOVAL ACCESS AND STAGING

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

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Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number:	Sheet :
16041	5 of 8

Sheet Number: R - 1

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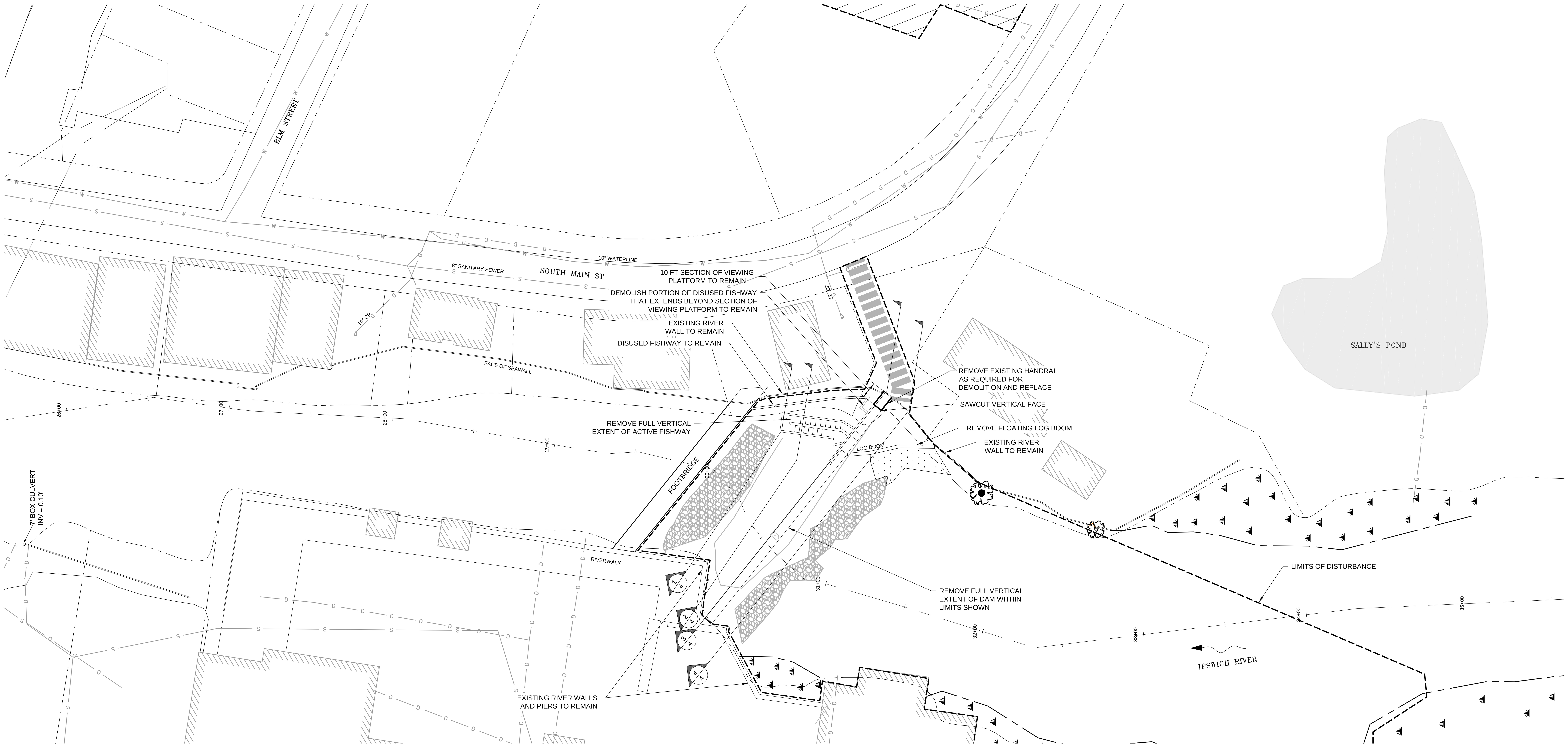

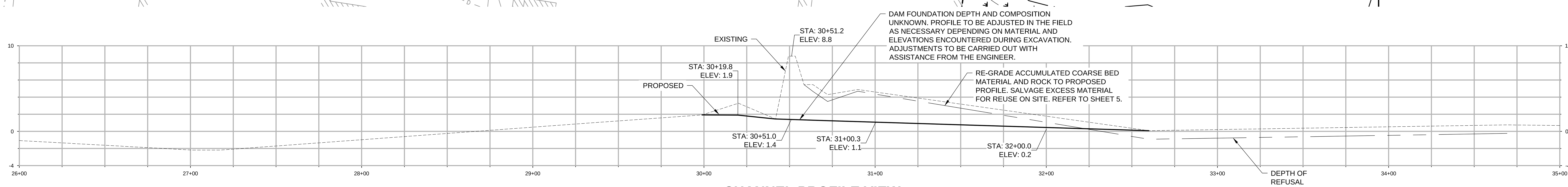

CHANNEL PROFILE VIEW

LEGEND:

- LIMITS OF DISTURBANCE
- TEMPORARY CONSTRUCTION ACCESS
- 160+00 IPSWICH RIVER STATIONS
- STAGING AREA
- SURVEYED MARSHY AREAS
- EXISTING SOFT SEDIMENT
- PARCEL BOUNDARY
- EXISTING 5' MAJOR CONTOURS
- EXISTING 1' MINOR CONTOURS
- S SANITARY SEWER CONVEYANCE
- D STORM DRAINAGE CONVEYANCE
- W WATERLINE
- ACCUMULATED COARSE BED MATERIAL AND ROCK

30 0 15 30 60 120

HORIZONTAL SCALE: 1" = 30'

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Date: December 10, 2018 | Designed By: KC, CC | Drawn By: SJ | Checked By: MB

Plan Set: IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title: DEMOLITION PLAN AND PROFILE

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
25 Green Street
Ipswich, MA 01938
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Survey Provided By:
Horsley Witten Group, Inc.
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Sandwich, MA 02563
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Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041

Sheet : 6 of 8

Sheet Number: R - 2

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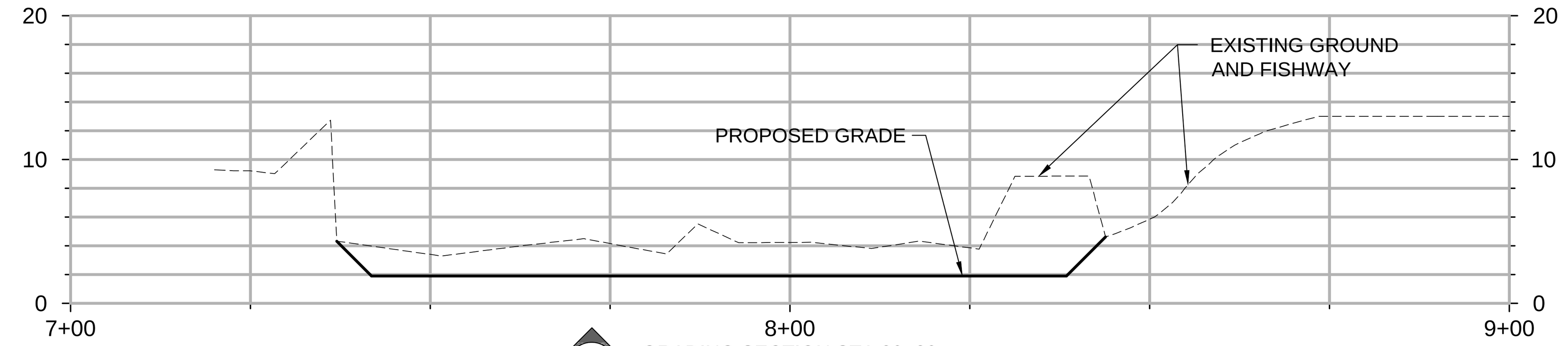
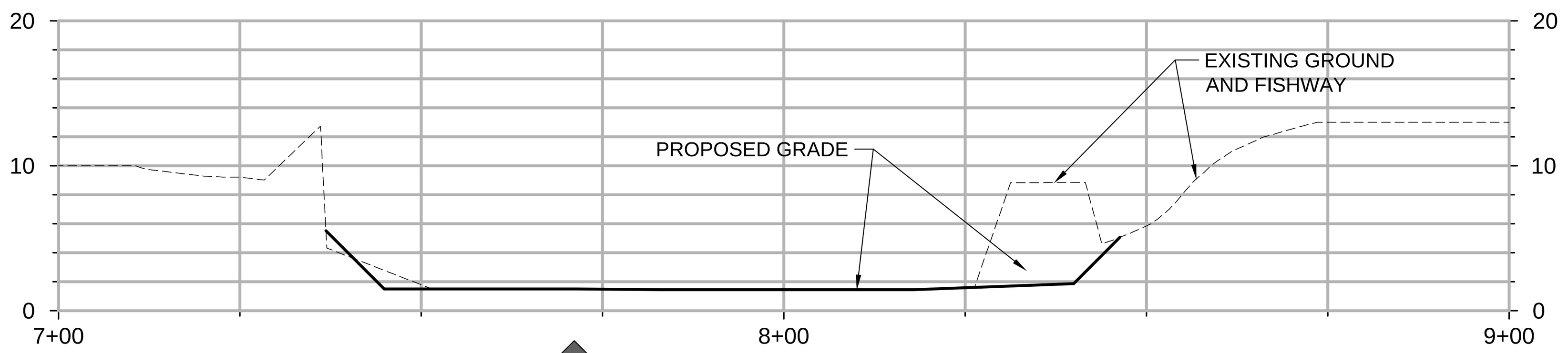


1 GRADING SECTION STA 30+20



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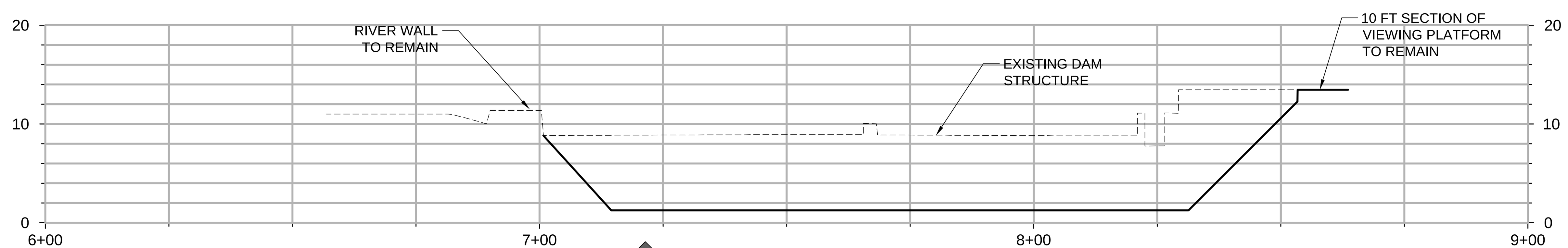

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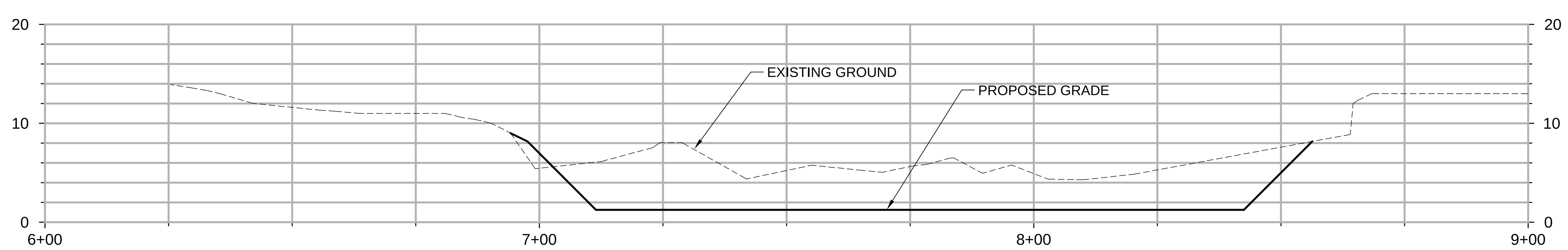
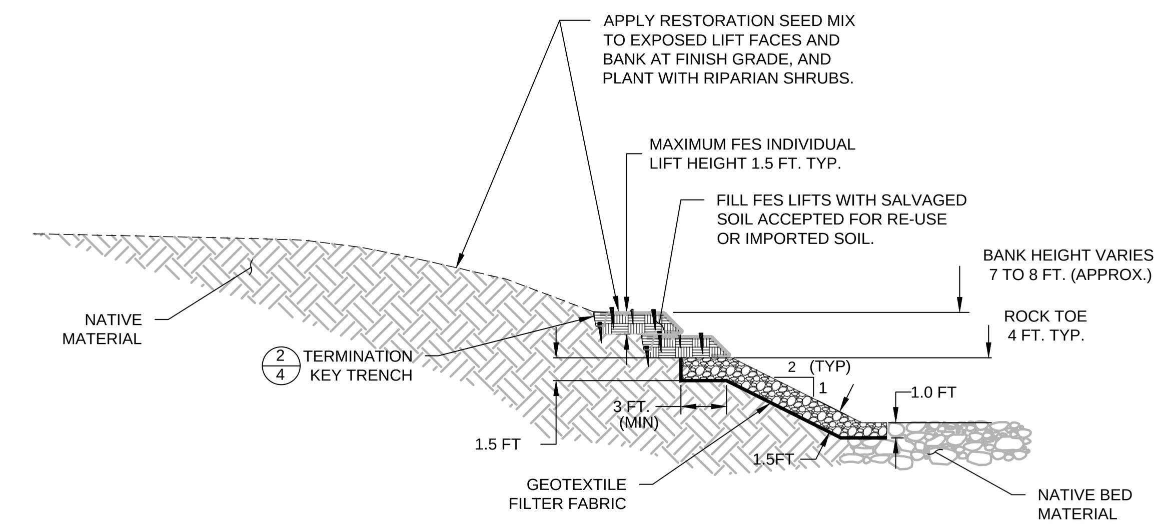


4 GRADING SECTION STA 30+72

GRADING SECTION NOTES:

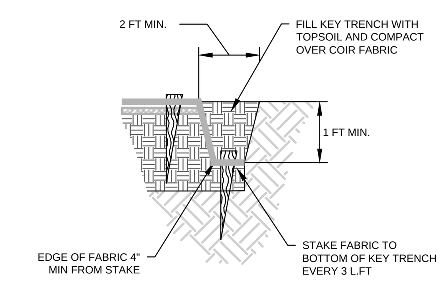
1. CROSS SECTIONS ARE ORIENTED TO FACE DOWNSTREAM.
2. CROSS SECTIONS TAKEN FROM THE IPSWICH MILLS DAM HEC-RAS MODEL DATED 05/31/18.
3. CROSS SECTION DATA AT STA 30+20 AND STA 30+41 AND THE DAM STRUCTURE ARE FROM THE 2016 HORSLEY WITTEN GROUP SURVEY.
4. CROSS SECTION DATA AT STA 30+72 WAS EXTRACTED FROM THE 2014 NORDE-EAST BATHYMETRIC SURVEY.



1 4 TYPICAL SECTION
FES LIFTS WITH ROCK TOE PROTECTION
SCALE: 1" = 10'

TYPICAL SECTION NOTES:

1. CONSTRUCT ROCK TOE WITH SALVAGED ROCK ACCEPTED FOR RE-USE BY THE ENGINEER OR WITH ROUNDED TO SUBANGULAR STONE D50 = 6 INCHES.
2. FINAL ROCK TOE CONFIGURATION AND ROCK SIZE AND GRADATION TO BE DETERMINED IN FUTURE DESIGN PHASES.



2 4 KEY TRENCH DETAIL
TERMINATION TRENCH
NOT TO SCALE

GRADING SECTIONS SCALE:

HORIZONTAL SCALE: 1" = 20'

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Date: December 10, 2018
Designed By: KC, CC
Drawn By: SJ
Checked By: MB

Plan Set: IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title: CROSS SECTIONS

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
25 Green Street
Ipswich, MA 01938
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Survey Provided By:
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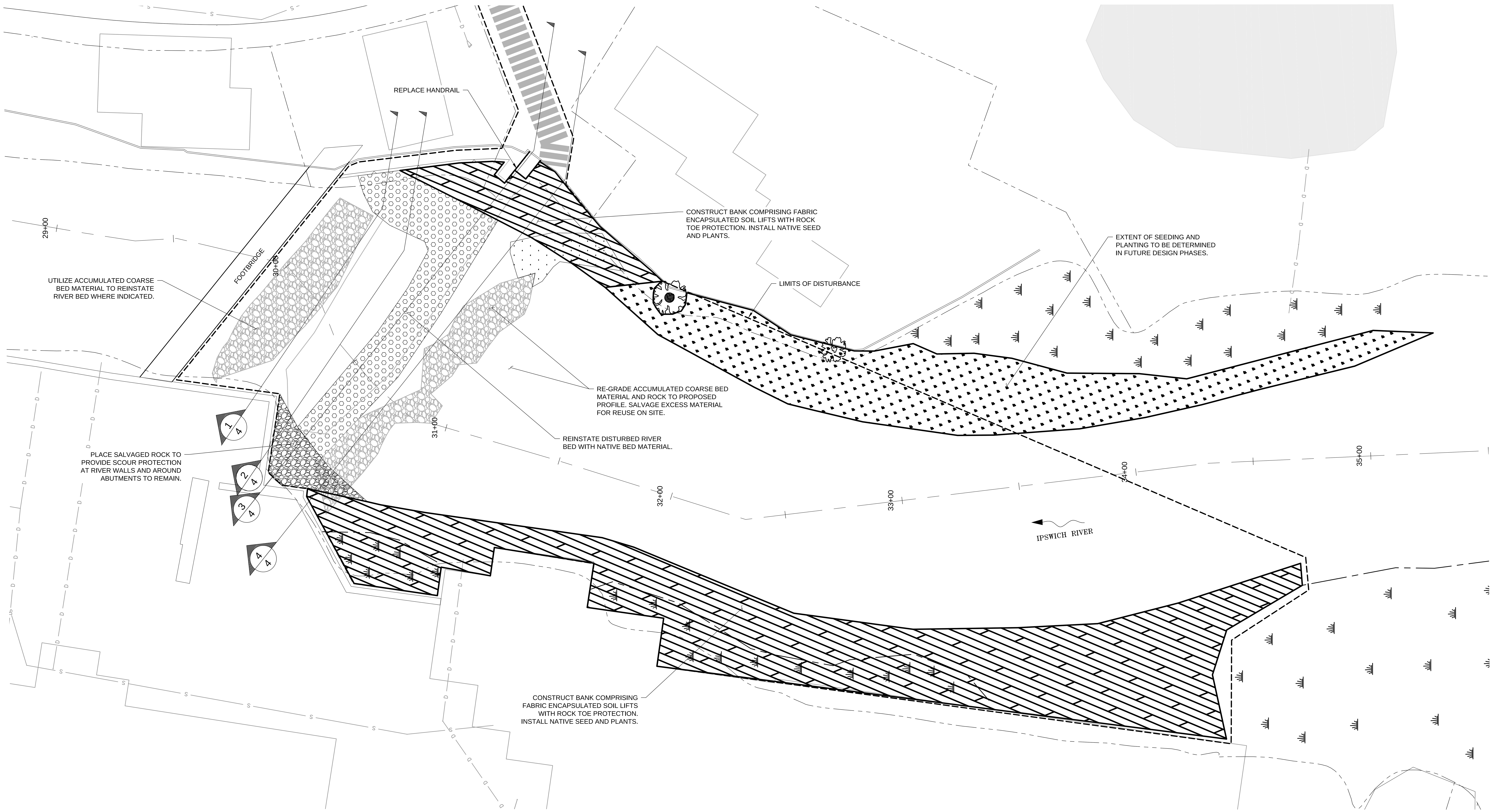
Sandwich, MA 02563
Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041
Sheet : 7 of 8
Sheet Number: R - 3



LEGEND:

- LIMITS OF DISTURBANCE
- TEMPORARY CONSTRUCTION ACCESS
- 160+00 IPSWICH RIVER STATIONS
- STAGING AREA
- SURVEYED MARSHY AREAS
- EXISTING SOFT SEDIMENT
- FES LIFTS WITH ROCK TOE PROTECTION, SEEDED AND PLANTED
- NATIVE SEEDING AND PLANTING
- PARCEL BOUNDARY
- EXISTING 5' MAJOR CONTOURS
- EXISTING 1' MINOR CONTOURS
- S SANITARY SEWER CONVEYANCE
- D STORM DRAINAGE CONVEYANCE
- W WATERLINE
- ACCUMULATED COARSE BED MATERIAL AND ROCK
- REINSTATED NATIVE BED MATERIAL

NOTES:

1. SELECTION OF NATIVE SEED AND PLANTS MUST SUIT LOCAL TIDAL CONDITIONS. SCOPE FOR SEEDING AND PLANTING TO BE DEVELOPED IN FUTURE DESIGN PHASES.

20 0 10 20 40 80

HORIZONTAL SCALE: 1" = 20'

Funded By: MASSACHUSETTS ENVIRONMENTAL TRUST

inter-fluve
220 Concord Ave, 2nd Floor
Cambridge, MA 02138
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www.interfluve.com

Date: December 10, 2018 | Designed By: KC, CC | Drawn By: SJ | Checked By: MB

Plan Set: IPSWICH DAM REMOVAL FEASIBILITY STUDY IPSWICH, MASSACHUSETTS

Plan Title: TREATMENT AND PLANTING PLAN

Prepared For:
Dept. of Fish and Game
Division of Ecological Restoration
Riverway Program
251 Causeway Street Suite 400
Boston, MA 02114

Town of Ipswich
25 Green Street
Ipswich, MA 01938
978-356-6600

Survey Provided By:
Horsley Witten Group, Inc.
90 Route 6A
Sandwich, MA 02563
Phone: (508) 833-6600
Fax: (508) 833-3150
Dated: September 7, 2016

Registration:

Project Number: 16041 | Sheet: 8 of 8

Sheet Number: R - 4

last modified: 12/14/18 printed: 12/14/18 by ml H:\Projects\2016\16041 DER_Ipswich Dam Removal Feasibility Study\Drawings\16041 INTERFLUVE.dwg

Ipswich Mills Dam Removal

Engineer's Opinion of Probable Construction Cost
Concept Design Submittal
13-Nov-18

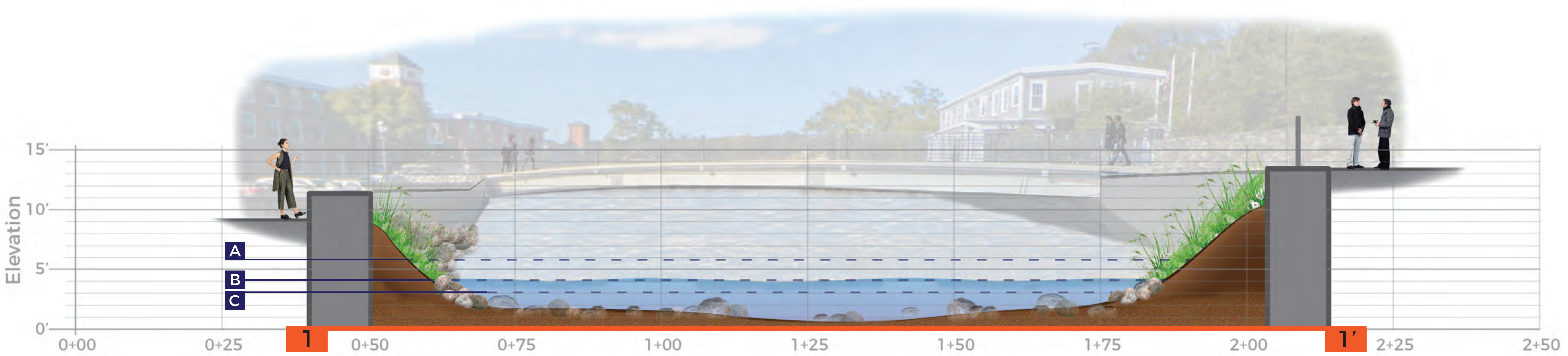
Base Bid Items

No.	Item	Quantity	Unit	Unit Cost	Total Cost	Notes
1	Mobilization & Demobilization	1	LS	$ 49,000	$ 49,000	10% of other items plus $10,000 for traffic control. Includes all access such as temporary access ramp into channel.
2	Flow Management, Erosion and Pollution Control	1	LS	$ 25,000	$ 25,000	In-stream flow management within the primary work area associated with dam removal, protection of catch basins, misc. erosion control activities
3	Dam Demolition and Disposal	1	LS	$ 50,000	$ 50,000	Removal of concrete and masonry dam, saw cutting and removal a portion of the viewing platform, and removal of fishway. Includes incidental clearing and grubbing, removal and replacement of handrail, protection of existing river walls, regrading and reinstating river bed, and off-site disposal of concrete, masonry, and miscellaneous dam materials.
4	Earthwork	1	LS	$ 70,000	$ 70,000	Excavation of rock and impounded sediment. Includes stockpiling and reuse of salvaged rock and off-site disposal of excavated material not acceptable for reuse or not incorporated into the project.
5	Rock	580	TON	$ 75	$ 43,500	Includes import and placement of rock toe beneath FES lifts and associated water control. Placement of salvaged rock for reinstating river bed at former dam location and for rock scour protection where shown adjacent to river walls is included in Earthwork.
6	FES Lifts	500	LF	$ 100	$ 50,000	Price is per linear foot assuming two lifts above rock toe. Includes water control outside the primary work area around the dam.
7	Potted Shrubs/Trees	400	EA	$ 70	$ 28,000	Assumes planting on 6-foot centers
8	Seed	1.5	AC	$ 5,000	$ 7,500	
Alternates						
A1	Rock Scour Protection - Railroad Crossing	210	TON	$ 150	$ 31,500	Optional scour protection at railroad bridge. Includes access and necessary flow control.
A2	Rock Scour Protection - Railroad Embankment	570	TON	$ 150	$ 85,500	Optional scour protection adjacent to railroad embankment. Includes access and necessary flow control.
				Subtotal	$ 440,000	
				Contingency (30%)	$ 132,000	
				Total	**$ 572,000**	

ATTACHMENT 9

IPSWICH DAM REMOVAL FEASIBILITY STUDY
RENDERING OF CONCEPTUAL DAM REMOVAL DESIGN
FEBRUARY, 2019

MASSACHUSETTS ENVIRONMENTAL TRUST
IPSWICH RIVER WATERSHED ASSOCIATION *The Voice of the River*
Division of Ecological Restoration
inter·fluve
Horsley Witten Group *Sustainable Environmental Solutions*



A Spring Flow (5% Exceedance) at High Tide. Elevation 5.8' (NAVD88)

B Average Flow (50% Exceedance) at Low Tide. Elevation 3.8' (NAVD88)

C Late Summer (95% Exceedance) at Low Tide. Elevation 2.8' (NAVD88)

Note: Flow conditions A, B, & C are also shown on perspective renderings.

IPSWICH DAM REMOVAL FEASIBILITY STUDY

ILLUSTRATIVE SECTION

JANUARY, 2019

MASSACHUSETTS ENVIRONMENTAL TRUST
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Division of Ecological Restoration
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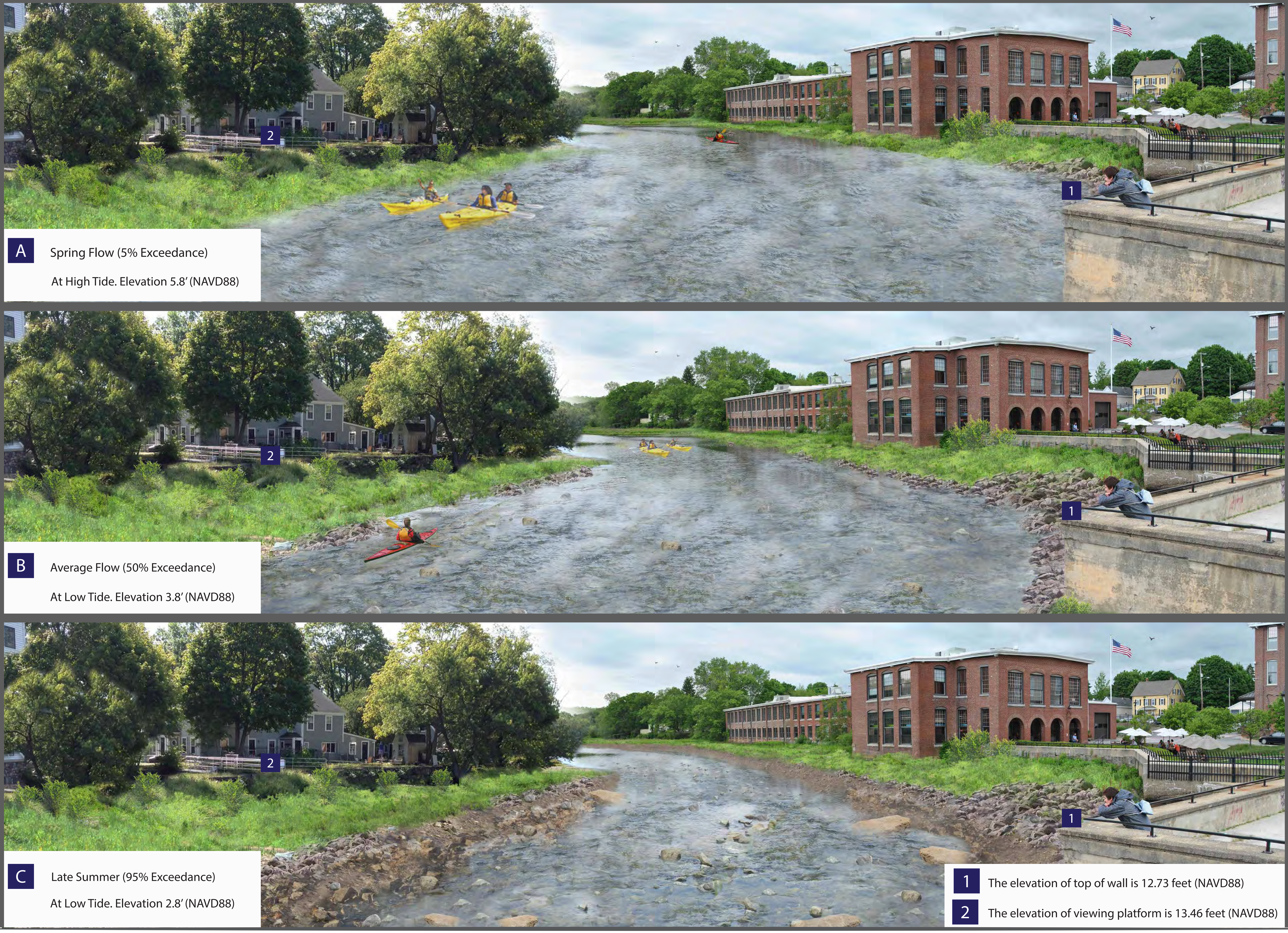

CONCEPTUAL RENDERING OF IPSWICH RIVER AT IPSWICH MILLS POST-DAM REMOVAL

JANUARY, 2019

MASSACHUSETTS ENVIRONMENTAL TRUST

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