Weymouth Herring Passage
& Smelt Habitat Restoration Project

Herring Brook, Weymouth, MA

FINAL DESIGN REPORT

MAY 2016

Prepared for:

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List of Abbreviations

ACEC	Area of Critical Environmental Concern
CFD	computational fluid dynamics
cfs	cubic feet per second
CY	cubic yards
DCR	Massachusetts Department of Conservation and Recreation
DFG	Massachusetts Department of Fish and Game
DFW	Massachusetts Division of Fisheries and Wildlife
DOT	Massachusetts Department of Transportation
ea	each
elev	elevation
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
ft	feet
ft/s	feet per second
Gomez and Sullivan	Gomez and Sullivan Engineers, DPC
lb	pounds
LF	linear feet
LS	lump sum
MarineFisheries	Massachusetts Division of Marine Fisheries
MassDEP	Massachusetts Department of Environmental Protection
MEPA	Massachusetts Environmental Policy Act
MHC	Massachusetts Historical Commission
MLW	mean low water
mo	month
msl	mean sea level (equal to NGVD 29)
NAVD 88	North American Vertical Datum of 1988
NGVD 29	National Geodetic Vertical Datum of 1929 (all elevations given in NGVD 29)
NOAA	National Oceanic and Atmospheric Administration
ODS	Office of Dam Safety
OPCC	opinion of probable construction cost
ORW	Outstanding Resource Water
PMF	Probable Maximum Flood
qty	quantity
SY	square yards
TOW	Town of Weymouth
TOY	time of year [restriction]
USGS	US Geological Survey
yr/yrs	year/years

1. Introduction

1.1 Project Overview & Goals

The Weymouth Back River (or Back River), located in Hingham and Weymouth, Massachusetts, supports one of the largest river herring runs in Massachusetts Bay. From the tidal waters in Hingham Bay, river herring ascend a total of six fishways on the Back River and Herring Brook to reach their spawning habitat in Whitmans Pond.

A flood control conduit was constructed in the 1960s in the upper portion of the Back River watershed to bypass storm flows past Jackson Square in Weymouth. The tunnel inlet is located just below Whitmans Pond Dam at Iron Hill Dam, with the outlet discharging adjacent to the lowermost fishway in Jackson Square. An existing fish diversion swing gate at the tunnel outlet has been ineffective at preventing upstream migrating river herring from entering the conduit, where they may become trapped and perish.

The Town of Weymouth secured funding from the Massachusetts Division of Marine Fisheries (*MarineFisheries*) to contract Gomez and Sullivan Engineers, DPC (Gomez and Sullivan) to prepare design plans, bid documents, and permit applications for an alternative solution to the problem of fish accessing the flood control tunnel. Project goals include implementing the following fish passage improvements in Herring Brook at the flood control conduit outlet near Jackson Square:

- Replace the existing fish diversion gate at the tunnel outlet with a more effective design that will prevent fish from entering the tunnel.

- Reestablish substrate suitable for smelt spawning on the concrete pad downstream of the tunnel outlet and fish ladder.

- Restore a resting pool for river herring immediately downstream of the concrete pad that has filled in with sediment primarily washed off roadways.

- Regrade an unauthorized rock weir downstream of the concrete channel to restore flow depths and velocities suitable for smelt spawning.

A project location map is shown in **Figure 1.1-1** and an aerial image of the project area is shown in **Figure 1.1-2**.

Figure 1.1-1: Project Location Map

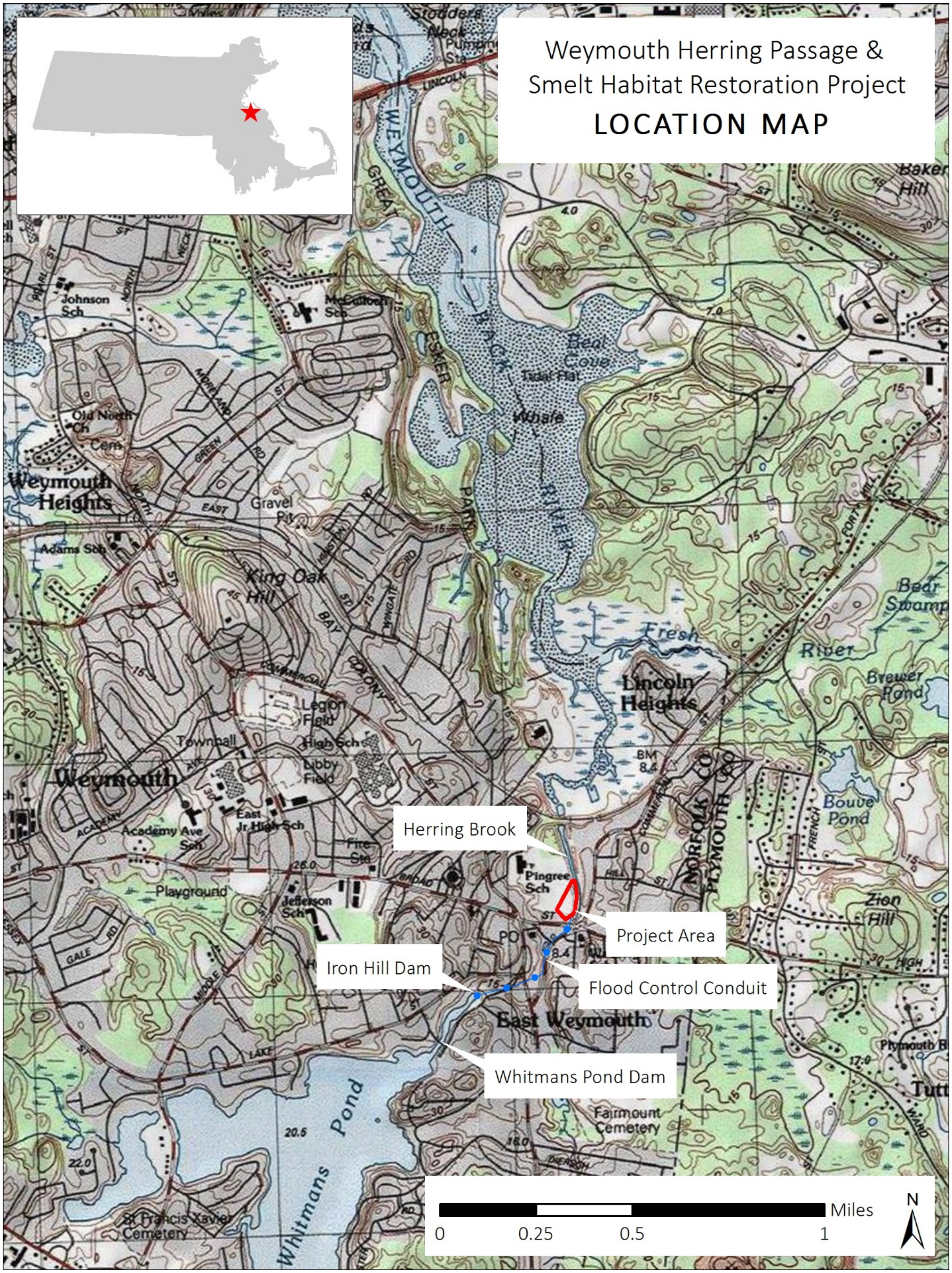
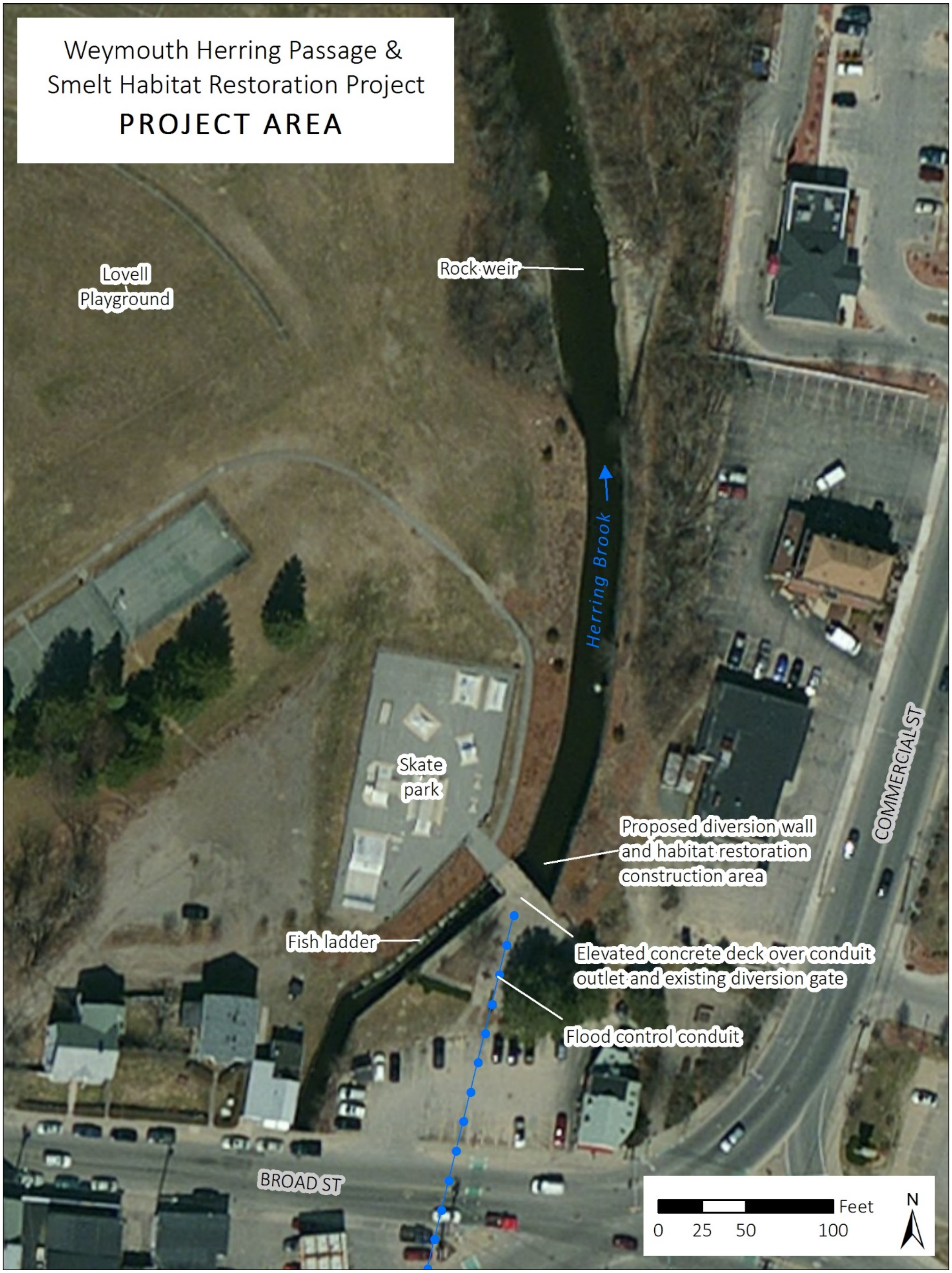


Figure 1.1-2: Project Area Map



1.2 Background

The existing fish diversion gate was constructed in the early 1980s. It is approximately 6.5 feet high by 23 feet wide and is situated on a concrete slab between two vertical concrete walls. An elevated concrete deck with a bottom elevation approximately 13.5 feet above the concrete slab supports a walkway above. The gate is constructed of metal grating framed by 8-inch-diameter horizontal metal pipes on the top and bottom and 8-inch by 12-inch by approximately 11-foot high vertical metal tubes at each side, the upper half of which are filled with lead. The entire gate rotates on a hinge attached to the channel wall.

The gate was designed to swing open during periods of high flows. However, the gate would swing open under moderate flows, which had the unintended consequence of allowing river herring to enter the tunnel. As there is no way for fish to gain access through to Whitmans Pond from the tunnel, the only exit for herring is at the outlet where they entered. Normally this would not be a significant issue, as fish would recede with the flow out of the tunnel following a high flow event; however, during two known events (2000 and 2010), a steady period of moderate to high flow occurred (i.e., flow was not decreasing; therefore river herring were not receding). The fish remained in the tunnel long enough to deplete the available dissolved oxygen, which led to the suffocation and eventual death of thousands of river herring.

Even when in the closed position, the original swing gate was insufficient at preventing river herring from entering the system. In 2004, a cooperative effort was made by *MarineFisheries* and the Town to repair and improve the functionality of the gate. The repairs included adding a fine stainless steel mesh to the gate surface, installing stop logs, and performing concrete and steel repairs to the gate and superstructure. Since these modifications, the Town has observed that the gate now opens under even more moderate flows, not just flood events, resulting in river herring entering the flood control tunnel much more frequently under a wide range of spring flows.

The gate is also experiencing corrosion, as it is now over 30 years old, and does not seal well and can remain stuck open and not return to a closed position when flows recede.

Regarding the channel downstream of the diversion, *MarineFisheries* has indicated that the existing concrete pad was previously covered with stone substrate. This material washed out during a flood event around 2005. It is thought that this material washed downstream and filled in a former river herring resting pool that had been located immediately downstream of the concrete pad. The dimensions of this former pool were observed to be about 3 to 5 feet deep and on the order of 15 to 20 feet wide. Throughout the project area, the channel has also filled in with sediment washed off roadways, impacting fish habitat and passage.

Additionally, at the downstream end of the concrete-walled channel (about 350 feet downstream of the tunnel outlet), an unauthorized rock weir has been built up, likely by people seeking to cross the stream. It backwaters Herring Brook up to the fish ladder, which has nearly eliminated spawning riffles for smelt at a location that MarineFisheries has considered for decades as one of the three largest smelt runs in Massachusetts. Restoring the channel slope by grading the rock weir is an important goal for improving migratory fish habitat at this location, and is also related to restoring a stable resting pool.

The project site is a public open space park adjacent to Lovell Playground and the Pingree Elementary School.

1.3 Design Criteria

Due to poor design and functioning of the gate, the Town was not interested in a gate rehabilitation alternative to deal with the declining condition of the gate. Through discussions with project partners, the following attributes were identified as design criteria for a replacement fish diversion:

- Provide the ability to be fully closed such that herring cannot access the tunnel via gaps or other openings
- Be of sufficient height to exclude herring from gaining access to the tunnel over the top of the diversion as close to 100% of the time as possible
- Provide sufficient open area (above or through the diversion) to safely pass anticipated flow conditions
- Provide the ability to fully drain the tunnel, such that water behind the diversion structure does not become stagnant
- Provide an opening of sufficient size and geometry to allow any herring that may become trapped to exit the tunnel with limited stress
- Include a structure to exclude American eel from moving over the diversion

Alternatives that could potentially meet these criteria were identified as either a replacement gate or a wall with a gated opening located at the floor elevation. A concrete wall with a gated opening became the selected design concept when it became apparent that a full gate replacement had notably higher construction costs with less expected longevity than a diversion wall.

1.4 Target Species

The primary target species for the redesign of the fish diversion are the anadromous alewife (*Alosa pseudoharengus*) and blueback herring (*Alosa aestivalis*), known collectively as river herring. The diversion redesign also considered the catadromous American river eel (*Anguilla rostrata*). Additional project goals include establishing spawning substrate for rainbow smelt (*Osmerus mordax*) on the concrete pad downstream of the diversion, as well as a resting pool for river herring below the concrete pad.

Table 1.4-1 outlines the timing of important life cycle events for target species throughout the year, based on discussions with *MarineFisheries*.

Table 1.4-1: Timing of important life cycle events for target diadromous species

Species	Event	Month								
		MAR	*APR*	*MAY*	*JUN*	*JUL*	*AUG*	*SEP*	*OCT*	*NOV*
Rainbow smelt	*Spawning & egg incubation*									
River herring	*Upstream migration*									
	Downstream migration									
American eel	*Upstream migration*									
	Downstream migration									

For the primary project goal of redesigning the fish diversion, the main hydraulic design consideration (from a fisheries perspective) is ensuring that the diversion is high enough to exclude river herring from passing over it during the range of flows expected for the migration period. Because the site is tidally influenced, this parameter is more a factor of the site hydrology than river herring life history; see **Section 2.4** for further discussion.

However, life history is an important consideration for the design of the rainbow smelt spawning habitat. Rainbow smelt eggs will adhere to the channel substrate and the eggs must remain inundated until fry emerge. If water levels drop, exposed eggs will suffer mortality. Based on discussions with *MarineFisheries*, water depth should be at least 0.5 feet in the smelt spawning habitat area. Additionally, the target water velocity to support smelt spawning should be 2.6 feet per second (ft/s), and velocities outside the range of 1 to 4 ft/s are considered unsuitable. These values are acceptable for river herring migrations as well, although glass eels prefer somewhat slower velocities.

2. Hydrologic & Hydraulic Analysis

The following types of hydrologic and hydraulic data were important for this project:

Channel Improvements

- **Flood Flows –** To check that the stone size to be used for the smelt spawning substrate and river herring resting pool can withstand the design flood

- **Typical Fish Migration Period Flows** – To check whether target flow depths and velocities (identified in **Section 1.4**) are achieved the majority of the time in the smelt spawning area

Fish Diversion

- **Tidal Surge Depths –** To determine the maximum water surface elevation at the downstream face of the proposed fish diversion in order to set the minimum diversion height to exclude herring

- **Flood Flows –** To ensure that the proposed fish diversion can pass certain flood flows without impacting the concrete beam supporting the elevated concrete deck above (separated by a distance of about 13.5 feet)

- **Flow Capacity of Existing Flood Control Conduit –** To ensure that the proposed fish diversion can pass the maximum flow that could be conveyed by the flood control conduit upstream without impacting the concrete beam supporting the elevated concrete deck above

These parameters are discussed in the following sections.

2.1 Stream Flow Gages

The Back River is a short, primarily tidal river in the towns of Hingham and Weymouth, Massachusetts that flows northward into Hingham Bay. According to the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS), the Back River technically begins at a point approximately 1,000 feet downstream of the railroad bridge located below the project site (FEMA, 2015). From this point upstream to the base of Whitmans Pond Dam (including the project site), the stream is known as Herring Brook. Whitmans Pond is fed primarily by Old Swamp River (which is considered the source of the Back River) and Mill River (which drains Weymouth Great Pond).

Four United States Geological Survey (USGS) stream gages are located in the Back River watershed near the site[1]. A summary of the gages is presented in **Table 2.1-1** on the following page. In the table, flows are given in cubic feet per second (cfs).

[1] Note that the names and descriptions of the Whitmans Pond gages are not entirely clear on the USGS website. In fact, there appears to be an error in the "LOCATION" field for Gage No. 01105607. The "LOCATION" field for Gage No. 01105606 gives an identical description except for the latitude and longitude. However, the gages appear to be mapped correctly on the "Location Map" pages. Gage No. 01105606 is at Whitmans Pond Dam, Gage No. 01105607 is at Iron Hill Dam, and Gage No. 01105608 is below the Iron Hill Dam fish ladder.

Table 2.1-1 shows that there are three gages in the vicinity of the Whitmans Pond and Iron Hill Dams. The Whitmans Pond Dam gage is located just upstream of the inlet to the flood bypass tunnel and thus represents the total flow at the upstream extent of Herring Brook. The two gages located just downstream near Iron Hill Dam—one at the inlet of the flood bypass tunnel and one at the fish ladder—could theoretically be summed to equal flow at the Whitmans Pond Dam gage. However, these three gages have relatively short periods of record (12 to 13 years), and limited peak discharge data[2].

In contrast, the Old Swamp River gage upstream of Whitmans Pond has a relatively long period of record (48 years) and is above points of water withdrawals/diversions. To evaluate the appropriateness of using the Old Swamp River gage instead of the Whitmans Pond Dam gage to estimate flows at the project site, a regression analysis was performed for average daily flows at the two sites during their common period of record (2002 to present). The results, shown in **Figure 2.1-1**, do not indicate a very strong correlation (R^2 value of 0.70).

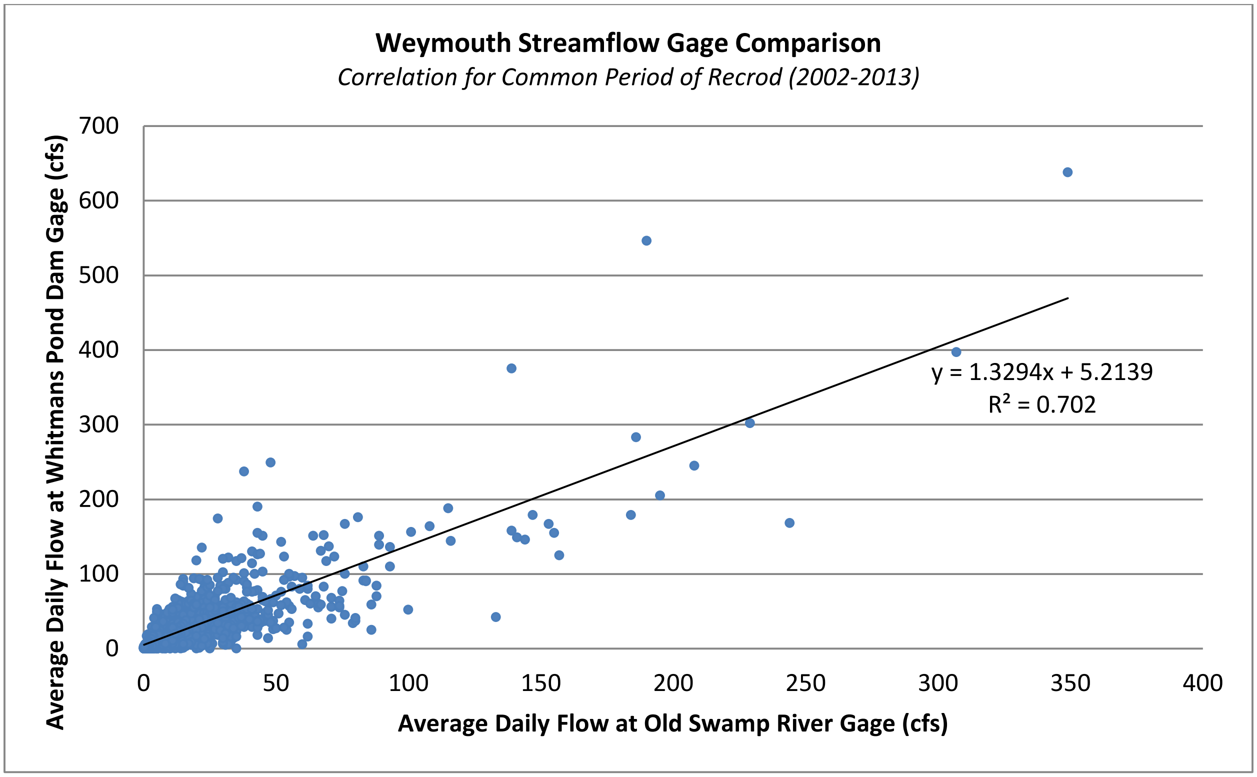
Therefore, the Whitmans Pond Dam gage represents the best available data that should be used to estimate average daily flows at the project site. Because the record for this gage has limited data on peak discharges, the FEMA FIS is the best available data for flood flow estimates at the site, as discussed in the following sections. However, for both cases, flows based on the Old Swamp River gage are also provided in this report for comparison purposes.

[2] At least 10 years of peak discharge values are recommended to perform a log-Pearson Type III flood frequency analysis according to USGS Bulletin 17B. No published peak discharge values were located for the Whitmans Pond Dam gage, and the flood bypass tunnel gage has recorded only 5 peak values. The fish ladder gage has recorded 12 peak values, but they do not represent the combined flow in Herring Brook and thus cannot be used.

Table 2.1-1: Summary of USGS Gages in the Vicinity of the Project Site

Water Body	Old Swamp River	Whitmans Pond		
Location	0.4 mi upstream of Whitmans Pond (at State Route 3 southbound lane)	Whitmans Pond Dam	Flood Bypass at Iron Hill Dam (~850 ft downstream of Whitmans Pond Dam)	Fish Ladder (~1450 ft downstream of Whitmans Pond Dam)
Gage No.	1105600	1105606	1105607	1105608
Drainage Area (mi^2)	4.5	12.4	12.4	12.5
Daily Flow Data	1966-present (48 yrs)	2001-present (13 yrs)	2002-present (12 yrs)	2001-present (13 yrs)
Peak Flow Data	1967-2013 (47 yrs)	None	2002-2005 (5 yrs)	2002-2013 (12 yrs)
Annual Mean Flow (cfs)	9.18	18.1	11.5	6.61
Max Peak Flow (cfs)	590 (5/31/84)	811 (3/15/10)	632 (3/15/10)	94 (10/15/05)
Accuracy	Records good except those for estimated daily discharges, which are poor. Gage is upstream of points of water withdrawals and diversions.	Records fair except those for flows less than 5 cfs and those for estimated daily discharge, which are poor. Periods of missing gage height record are not estimated. Flow affected by diversions for municipal use.	Records poor. Discharge affected by board changes in fish ladders at Whitmans Pond Dam and Iron Hill Dam, and by diversions from Whitmans Pond for municipal supply of Weymouth.	Records good except estimated daily discharges and discharges less than 0.2 cfs, which are fair. Includes flow through fish-ladder system. Discharge affected by gate changes at Whitmans Pond Dam, board changes at fish ladders, and diversions from Whitmans Pond for municipal supply of Weymouth. High flows affected by diversions to flood bypass system.
Notes	Closest FIS location: "At State Route 3 Northbound lane" (drainage area of 4.7 mi^2).	Daily mean discharge records previously published under Station No. 011056081, "Whitmans Pond Combined By-Pass and Fish-Ladder Flow," from water years 2002 through 2009, are now included in the daily and historical statistics for this streamgage.	Sum of these two gages is approximately equal to gage at Whitmans Pond Dam. Flow rejoins at project site.	

Figure 2.1-1: Weymouth Streamflow Gage Comparison



2.2 Flood Flows

For this project, it was important to have an estimate of flood flows (i.e., the 100-year and 500-year floods) for the design of the proposed fish diversion and channel improvements. The proposed diversion was evaluated to ensure that it could pass flood flows without impacting the support beam for the elevated concrete deck above (**Section 2.7**)[3]. Additionally, flow velocities associated with estimated peak discharges were used to ensure that the smelt spawning substrate and river herring resting pool can withstand flood flows (i.e., to determine the minimum stone size needed for these improvements).

FIS reports provide one source of information on local flood flows. The effective FEMA FIS for the Town of Weymouth (No. 25021CV001) was published on July 16, 2015 (FEMA, 2015). The hydrologic analysis for the Back River and Herring Brook in the FIS was initially conducted in 1980. A multiple regression analysis, developed by Johnson and Tasker, was applied to find runoff discharges for riverine flooding in Weymouth. Standard USGS topographic maps were used to determine watershed areas and local topography. An annual precipitation value of 3.67 feet per year, representative of the southeastern Massachusetts region, was obtained from the US Weather Bureau Technical Paper 40 (TP-40). By determining values for slope and area and using them in conjunction with the precipitation value in the Johnson-Tasker formulas, values for runoff from 10-, 2-, and 1-percent-annual-chance exceedance (i.e., 10-, 50-, and 100-year) storms were predicted. Exponents for the 0.2-percent-annual-chance (500-year) storm frequency equation were extrapolated. A check with a log-Pearson Type III analysis of the Old Swamp River gage data (using 10 years of record available at the time) found the discharge values

[3] Since flows reaching the fish diversion are regulated by the flood control conduit upstream, the flow capacity of the existing conduit was also considered in this analysis (**Section 2.5**).

computed using the Johnson and Tasker method to be within expected ranges. No new hydrologic analyses were conducted for the revised 2015 FIS.

The National Oceanic and Atmospheric Administration (NOAA) Fisheries Service has published guidance for considering climate change when developing flood frequency estimates for river restoration projects (Collins, 2011). The publication recommends extending the flood record beyond dated FEMA studies and recalculating flood flows. Thus, an updated flood frequency analysis was conducted to compare with the FIS estimates for Herring Brook. Annual peak flows at the Old Swamp River gage for the period of record (published data available for water years 1967-2013) were entered into the USGS's PeakFQ program to estimate storm events for various recurrence intervals using the Bulletin 17B methodology, which creates a Log Pearson Type III statistical evaluation of the data. The results were prorated to Herring Brook at the project site based on a ratio of drainage areas (4.5 square miles at Old Swamp River Gage vs. 14.1 square miles at project site).

A summary of flood discharges from updated flood frequency analysis as well as the effective FIS for Herring Brook at Broad Street[4] is given in **Table 2.2-1** and **Figure 2.2-1** below. Note that these values represent the total flow in both the bypass tunnel and surface channel (i.e., fishway and adjacent spillway). This is appropriate for the design of the smelt spawning substrate and herring resting pool, which would experience the combined flow. Based on the three common peak flow events on record for the gages at the inlet of the bypass tunnel and fish ladder upstream near Whitmans Pond Dam, approximately 84% of flood flows are diverted through the tunnel.

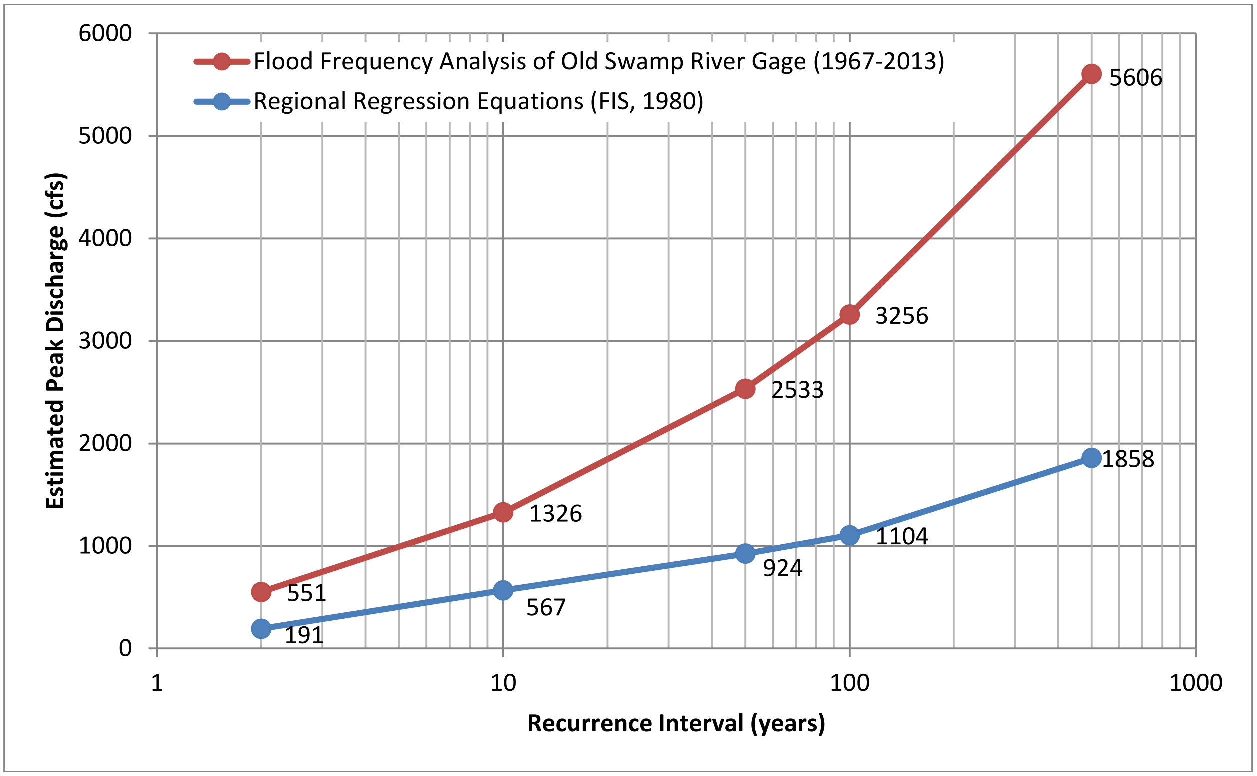
Table 2.2-1: Summary of Flood Frequency Estimates for Herring Brook at Broad Street

Annual Exceedance Probability	**Recurrence Interval** (yrs)	**Estimated Peak Discharge** (cfs)	
		Regional Regression Equations (FIS, 1980)	**Log Pearson Type III Analysis of Old Swamp Gage** (1967-2013)
50%	2	191[1]	551
10%	10	567	1326
2%	50	924	2533
1%	100	1104	3256
0.2%	500	1858	5606

[1] *2-year flood flow for FIS series extrapolated from natural log best fit line of 10-, 50-, and 100-year flows.*

[4] Broad Street is just upstream of the project site with a published drainage area of 14.1 square miles in the FIS. The difference in drainage area between Broad Street and the tunnel outlet is negligible; thus the published FIS data was used. The FIS peak discharges at this location consider the combined flow of both the flood bypass tunnel and Herring Brook.

Figure 2.2-1: Summary of Flood Frequency Estimates for Herring Brook at Broad Street



Note: 2-year flood flow for FIS series extrapolated from natural log best fit line of 10-, 50-, and 100-year flows.

The 100-year flood flow is generally adequate for the design of channel improvements such as those proposed for the project site. As indicated in the table and figure, the regulatory (FIS) 100-year flood flow for Herring Brook at the project site is 1,104 cfs.

Based on recommendations by *MarineFisheries*, the recommended stone size to meet the needs for smelt spawning habitat as well as withstand flood flows is 6-12 inches. The closest Massachusetts Department of Transportation (DOT) standard size meeting these requirements is “modified rockfill”, with a median size of approximately 5 inches and a range of about 2.5 to 9 inches. Using the Manning’s equation, the depth and velocity of the FIS 100-year flood flow (1,104 cfs) in the 24-foot-wide channel downstream of the fish diversion were estimated as 8.1 feet and 4 ft/s, respectively[5]. The US Department of Transportation’s HEC-11 – Design of Riprap Revetment (1989) was used to verify that the modified rockfill stone size proposed for the smelt spawning substrate is anticipated to withstand the 100-year flood.

2.3 Typical Fish Migration Period Flows

The range of flows experienced at the project site during fish migration period were important for the design of both the diversion and the channel improvements. These flows were estimated using nearby stream flow gages. Average daily discharges from both the Whitmans Pond Dam (drainage area 12.4

[5] To be conservative, this analysis did not consider backwater effects from the railroad crossing downstream of the project site, which would increase water depth and decrease water velocity during high flows.

square miles) and Old Swamp River (drainage area 4.5 square miles) gages were adjusted to the project site (Herring Brook at Jackson Square, drainage area 14.1 square miles) by ratio of drainage areas. Annual average daily flow duration curves are shown in **Figure 2.3-1**. **Figures 2.3-2** and **2.3-3** show flow duration curves for the period of March 1 to June 30 only, which covers the typical river herring migration and smelt spawning seasons. (**Figure 2.3-3** is a close-up of the high flow range for the fish passage period.) Monthly and annual flow statistics are shown in **Table 2.3-1** at the end of this section.

Based on the Whitmans Pond Dam gage, the median flow at the project site during the river herring migration period is approximately 18 cfs, and typically ranges from 5 to 55 cfs (90 and 10 percent exceedance values, respectively). These values are similar to those for the smelt spawning period (March through May) and were used for the smelt spawning habitat and river herring resting pool hydraulic design targets.

Figure 2.3-1: Avg. Daily Flow Duration Curves for Herring Brook at Broad St (Annual)

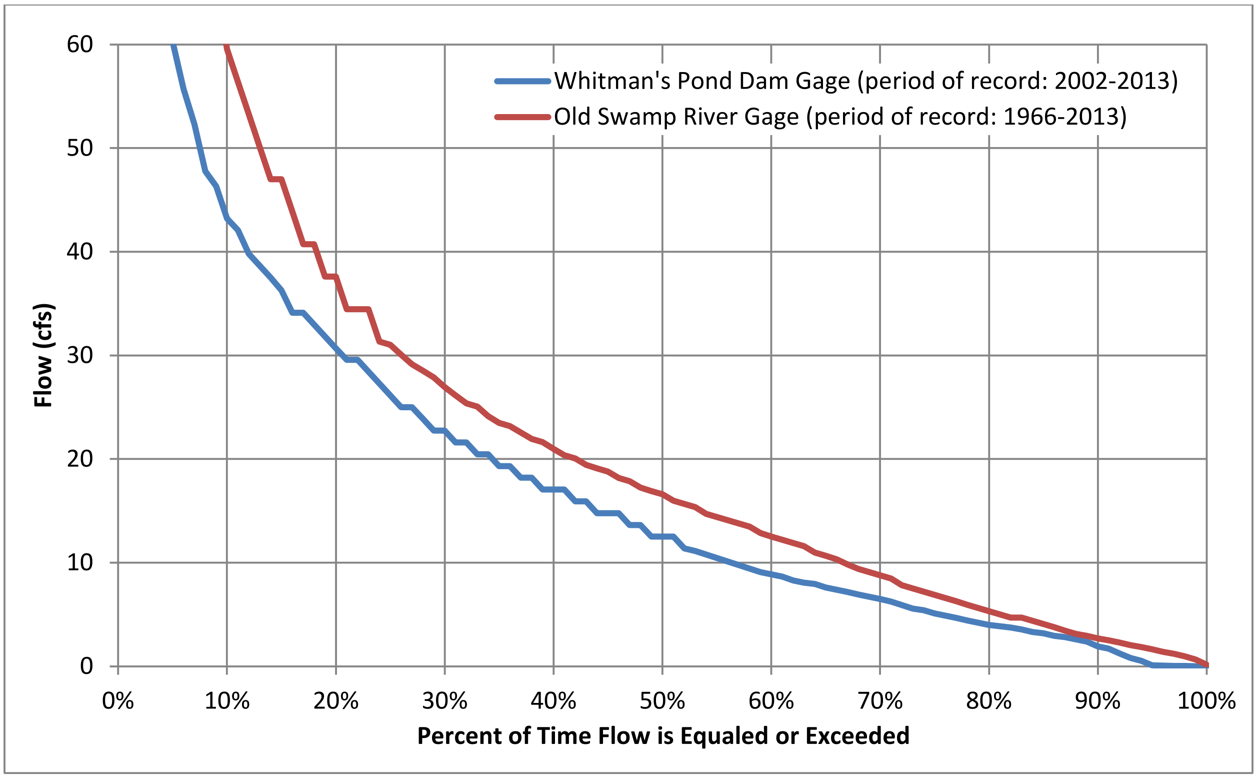


Figure 2.3-2: Avg. Daily Flow Duration Curves for Herring Brook at Broad St (Mar – Jun)

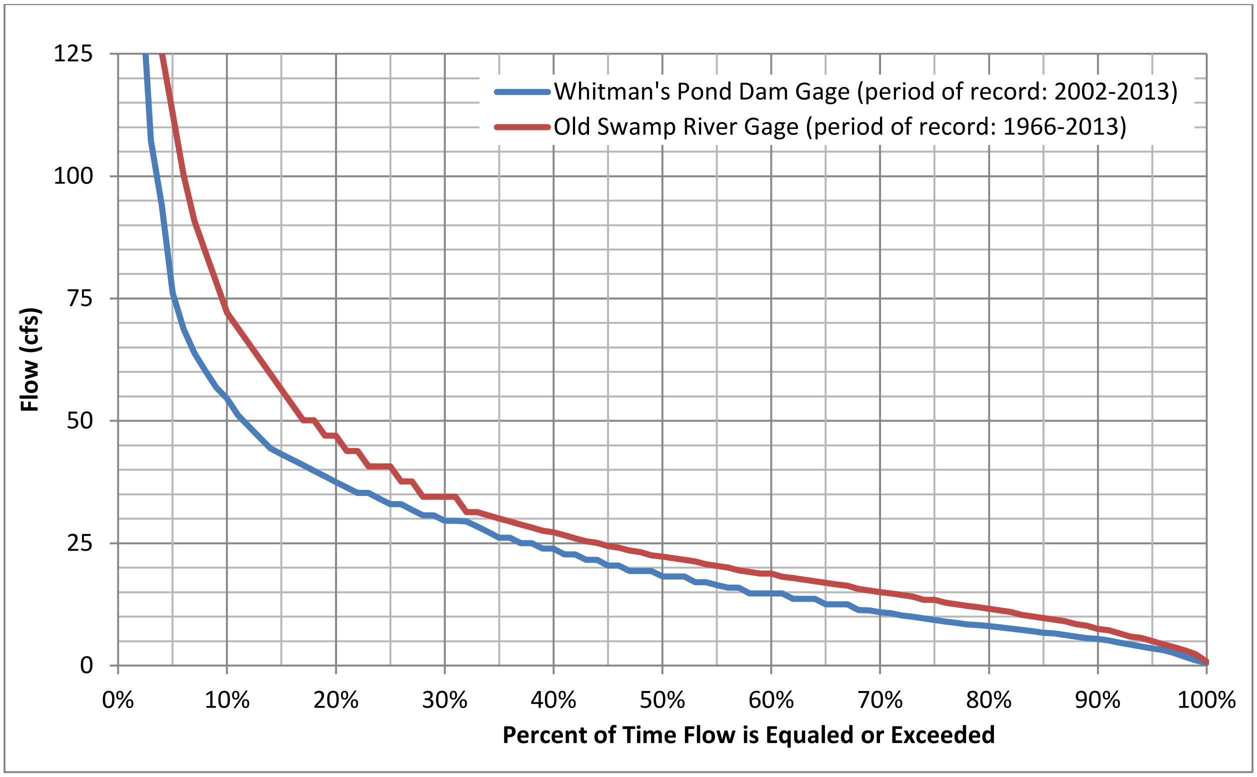


Figure 2.3-3: Avg. Daily Flow Duration Curves for Herring Brook at Broad St (Mar – Jun, High Flows)

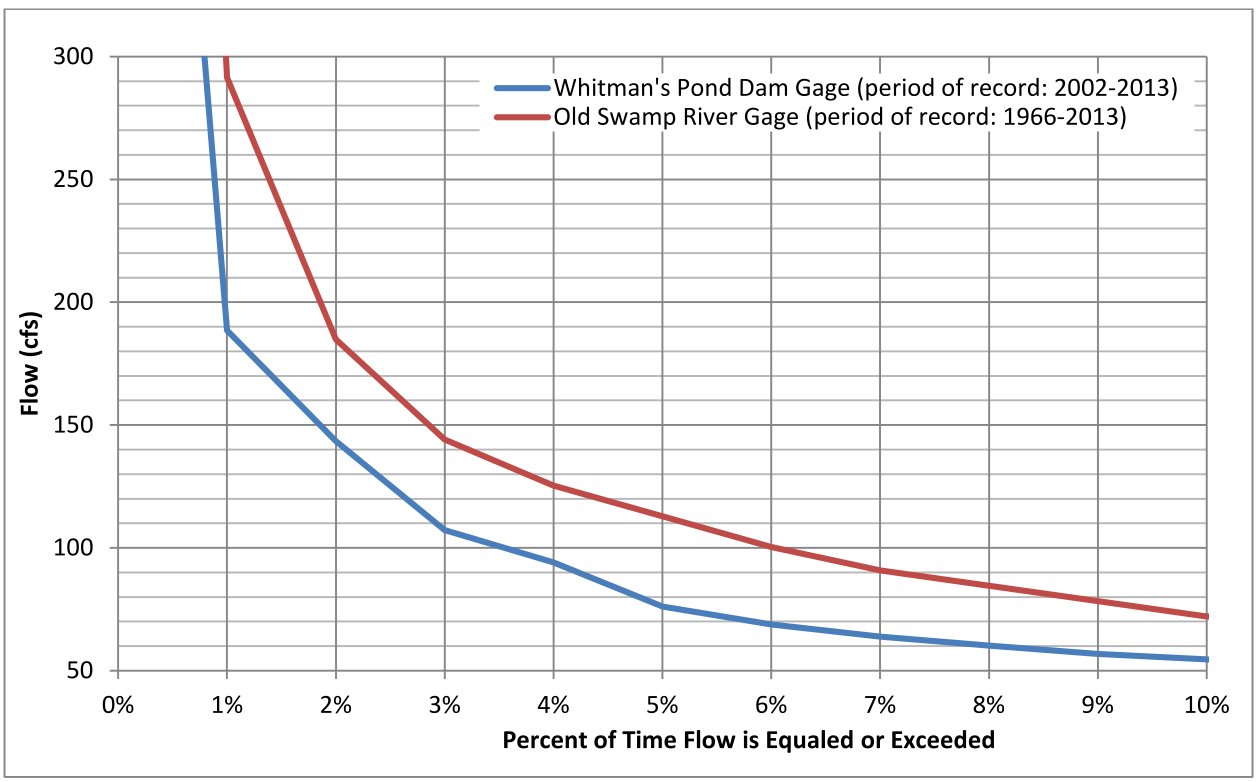


Table 2.3-1: Summary of Average Daily Flow Statistics for Herring Brook at Broad St

	Flow (cfs) for Time Period														
	JAN	*FEB*	*MAR*	*APR*	*MAY*	*JUN*	*JUL*	*AUG*	*SEP*	*OCT*	*NOV*	*DEC*	*ANNUAL*	*River Herring migration (MAR-JUN)*	*Smelt Spawning (MAR-MAY)*
Data Source: Whitmans Pond Dam at USGS Gage No. 01105606 *(adjusted to project site based on drainage area ratio)*															
Mean	23	27	42	30	23	19	8	8	8	16	19	26	21	29	32
Minimum	3	3	3	2	4	0.5	0.05	0.02	0.01	0.01	0.01	0.03	0.01	0.5	2
90% exceeds	7	10	8	6	8	2	0.3	0.06	0.1	0.03	3	3	2	5	8
50% exceeds (median)	18	19	30	25	16	8	4	5	4	6	11	23	13	18	22
10% exceeds	43	51	69	53	39	39	19	15	17	40	39	52	43	55	56
Maximum	125	190	725	269	204	279	102	125	81	451	725	173	725	725	725
Data Source: Old Swamp River at USGS Gage No. 01105600 *(adjusted to project site based on drainage area ratio)*															
Mean	37	41	55	42	29	24	9	11	11	19	30	39	29	38	42
Minimum	5	5	8	4	5	0.8	0.4	0.2	0.2	0.4	0.2	3	0.2	0.8	4
90% exceeds	11	13	16	13	10	3	1	1	1	3	7	10	3	8	12
50% exceeds (median)	23	26	34	27	19	10	4	4	4	8	15	23	17	22	26
10% exceeds	72	81	103	85	53	41	19	22	22	38	60	78	60	72	81
Maximum	655	479	1131	624	849	1009	291	260	470	962	1131	965	1131	1131	1131

The Manning's equation was used to estimate flow depths and velocities associated with the typical flows experienced at the project site during herring migration period with the proposed channel improvements in place[6]. **Figures 2.3-4** and **2.3-5** show the proposed depths and velocities, respectively. Two curves are shown—one for the narrow section of channel immediately downstream of the existing fish ladder and adjacent to the proposed fish diversion (with a width of 11 feet), and another for the wider section of channel downstream of the proposed diversion (24 feet).

Figure 2.3-4 shows that the minimum flow depth of 0.5 feet recommended for smelt spawning or river herring migration is met at flows of about 5 cfs below the fish ladder or 10 cfs below the fish diversion. These flows are exceeded approximately 90% and 73% of the time during herring migration period, respectively.

Figure 2.3-5 shows that the minimum flow velocity of 1 ft/s recommended for smelt spawning is met at flows of about 8 cfs below the fish ladder or 16 cfs below the fish diversion. These flows are exceeded approximately 80% and 56% of the time during herring migration period, respectively.

In summary, the median herring migration period flow of 18 cfs will meet all flow depth and velocity targets for smelt spawning and river herring passage. As the project progresses, *MarineFisheries* plans to work with the Town of Weymouth to optimize the design of the proposed channel modifications (i.e., rock weir grading, smelt habitat restoration) to further enhance the potential for depth and velocity improvements for smelt spawning.

[6] These curves do not include the effects of tidal surges, but rather were intentionally based only on upstream inflow and channel dimensions to allow for estimation of conservatively low water depths and high water velocities.

Figure 2.3-4: Estimated Flow Depth in Channel Downstream of Proposed Diversion

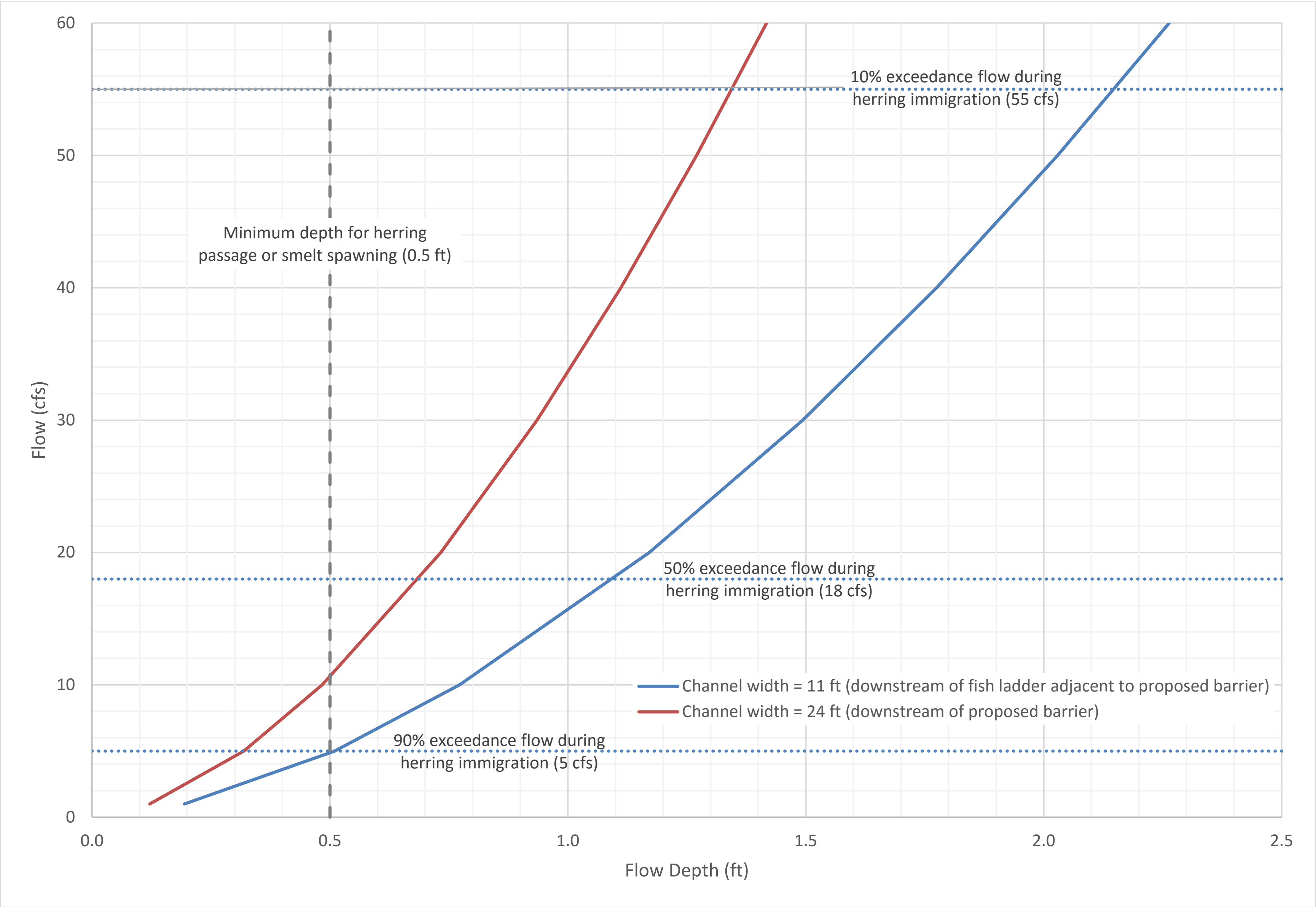
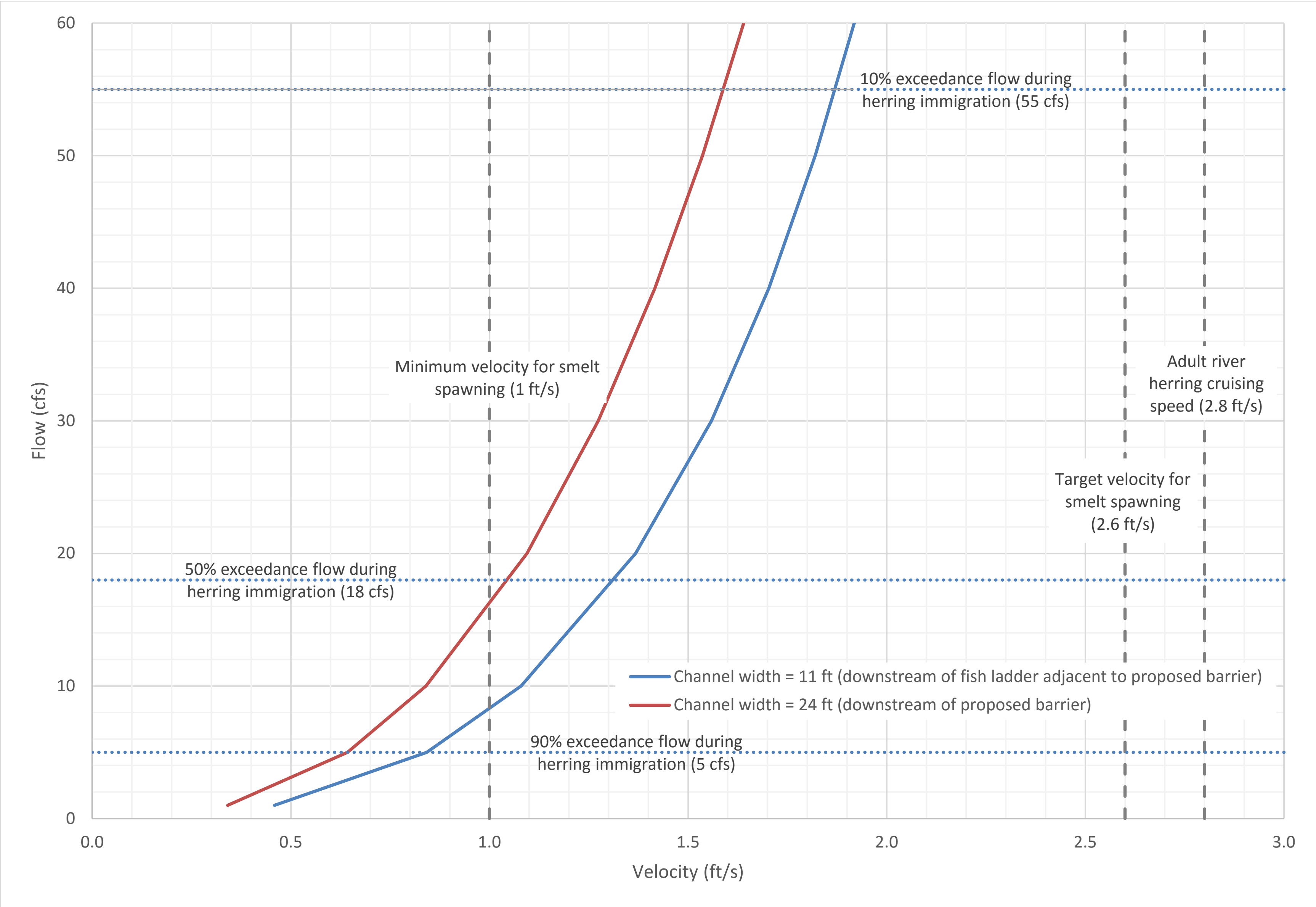


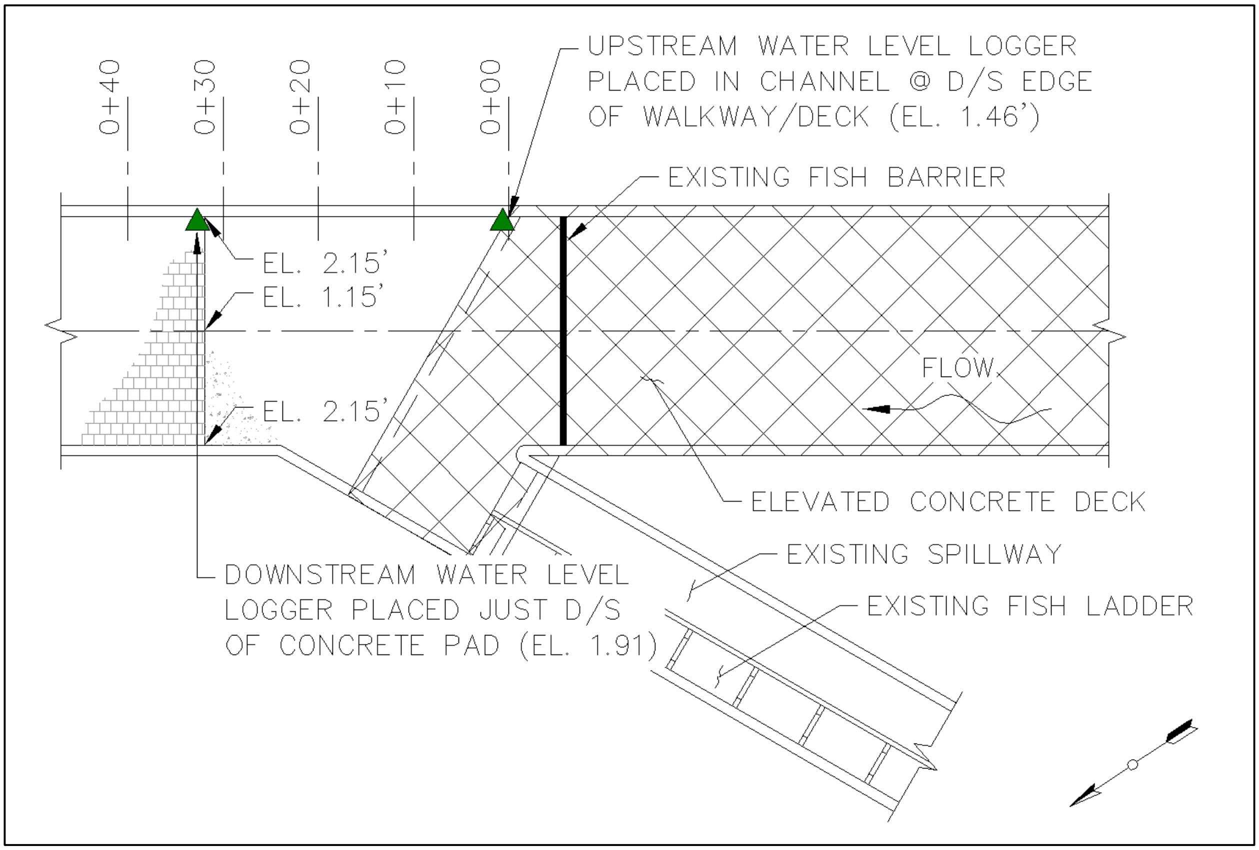
Figure 2.3-5: Estimated Flow Velocity in Channel Downstream of Proposed Diversion



2.4 Tidal Surge Depths

To gain a better understanding of the relationship between flow, tidal surges, and water surface elevations at the project site, two water level loggers were installed in the vicinity of the existing fish diversion for the period of February 28, 2014 through April 8, 2014. The locations of the loggers are shown in **Figure 2.4-1**. The logger referred to as "Upstream Water Level Logger" was placed just below the existing fish diversion gate at the downstream edge of the walkway/deck. The "Downstream Water Level Logger" was installed approximately 35 feet downstream, just below the extent of the concrete pad.

Figure 2.4-1: Location of Installed Water Level Loggers



Note: All elevations given in feet in National Geodetic Vertical Datum of 1929 (NGVD 29), also referred to as Mean Sea Level (MSL). Conversion factors for other vertical datums are given in ***Table 2.4-1****.*

Table 2.4-1: Vertical Datum Conversion Factors for the Project Area

Starting Vertical Datum	Datum Conversion Factor (feet)			
	NGVD 29 /MSL	NAVD 88	MLW	TOW
National Geodetic Vertical Datum of 1929 (NGVD 29) or Mean Sea Level (msl)	-	-0.08	4.37	5.83
North American Vertical Datum of 1988 (NAVD 88)	0.08	-	5.17	6.63
Mean Low Water (MLW)	-4.37	-5.17	-	1.46
Town of Weymouth (TOW)	-5.83	-6.63	-1.46	-

A summary of water depth statistics at the two loggers is provided in **Table 2.4-2**. **Figures 2.4-2** and **2.4-3** provide the raw time series water depth and flow data for the upstream and downstream loggers, respectively. **Figure 2.4-4** provides water depth duration curves for both loggers.

Because the tides are influenced by both the moon and the sun, when these two gravitational bodies are aligned, as during a new moon or full moon, the tidal effect is increased (i.e., high tides are higher). These are known as spring tides, named not for the season, but for the fact that the water "springs" higher than normal. Conversely, when the sun and moon are 90 degrees apart, as during the first and third quarter moons, high tides are at their lowest point, known as a neap tide. For reference, moon phases are shown on **Figures 2.4-2** and **2.4-3**.

Table 2.4-2: Summary of Water Depth Statistics for Water Level Loggers

		Water Depth Statistics *(ft, during 2/28/14 through 4/8/14)*		
		Daily Maximum (baseflow + tide)	*Daily Minimum (baseflow only)*	*Daily Surge (tide only)**
Upstream Water Level Logger	MIN	2.9	1.5	1.3
	MEDIAN	4.6	1.7	2.9
	MEAN	4.6	1.8	2.9
	MAX	6.7	2.6	4.8
Downstream Water Level Logger	MIN	3.9	2.1	1.3
	MEDIAN	5.1	2.3	2.7
	MEAN	5.2	2.4	2.8
	MAX	7.3	3.6	4.8

**Daily surge was calculated by subtracting the minimum (baseflow) depth from the maximum (high tide) depth for each day, not for the overall min/median/mean/max values.*

The water level logging period captured a high flow event on March 31, 2014 with a peak discharge of 207 cfs[7] at approximately 12:45 PM. According to the flow duration analysis (see **Figure 2.3-3**), this flow is exceeded about 1% of the time during the river herring migration period. Therefore, the maximum water depth recorded by the water level loggers during this high flow event (6.7 feet at the upstream

[7] Based on the Whitmans Pond Dam gage adjusted by ratio of drainage area to the project site.

logger or 7.3 at the downstream logger, or the average of 7 feet) would be a conservative height for the redesigned fish diversion to avoid overtopping 99% of the time.

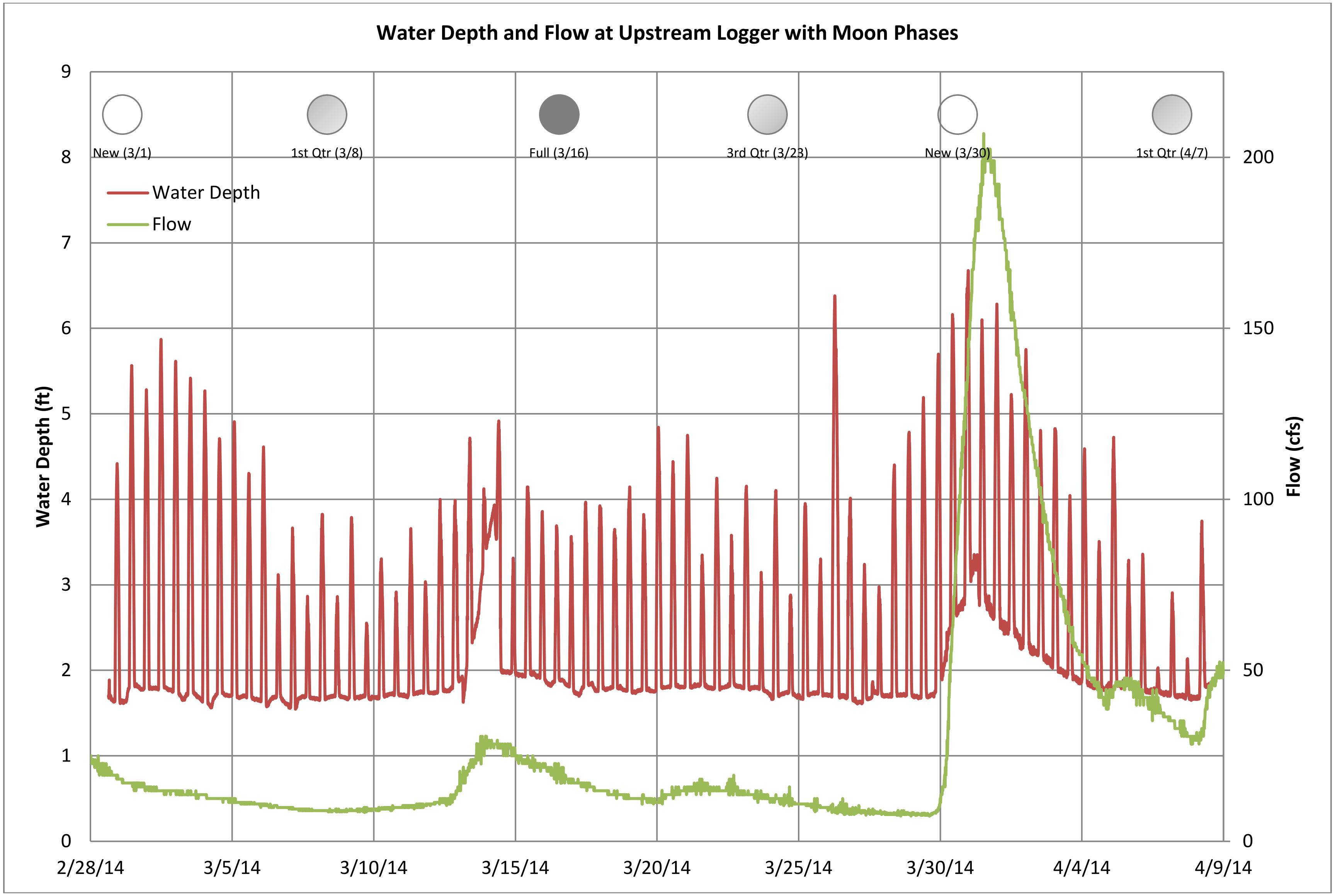
A buffer of 2 feet of separation between the maximum water depth and the top of the wall is recommended as a factor of safety to avoid the potential for fish overtopping the wall. Therefore, a fish diversion wall height on the order of 9 feet should exclude river herring for about 99% of the flows during river herring migration period.

In general, maximum daily (high tide) water surface elevations at the site seem to be more influenced by the moon phase than by the base flow. It appears that base flows below about 50 cfs do not have a significant impact on the high tide elevation. Based on the gage data, a flow of 50 cfs is exceeded about 3-10%[8] of the time during the river herring migration period (March through June). Looking at the water depth duration curve (**Figure 2.4-4**), it can be seen that water depths due to tidal surge are generally below 5 feet most of the time[9]. Therefore, a diversion wall height of 7 feet (5 feet to avoid overtopping plus 2 feet of separation buffer) would be expected to exclude river herring approximately 90-97% of the time during their migration period.

A 9-foot-high wall would provide about 2-9% of additional river herring exclusion, while a 7-foot-high wall would provide greater flow capacity. With a lower wall, the crest elevation could be adjusted through downward opening gates, flashboards, or other operable systems to accommodate higher heights to restrict fish passage, but lower heights to allow for increased flood passage. Conversely, if a higher wall is selected for design, additional flow capacity could be achieved through sluice gates and/or by extending the length of the wall (discussed in **Section 2.6**).

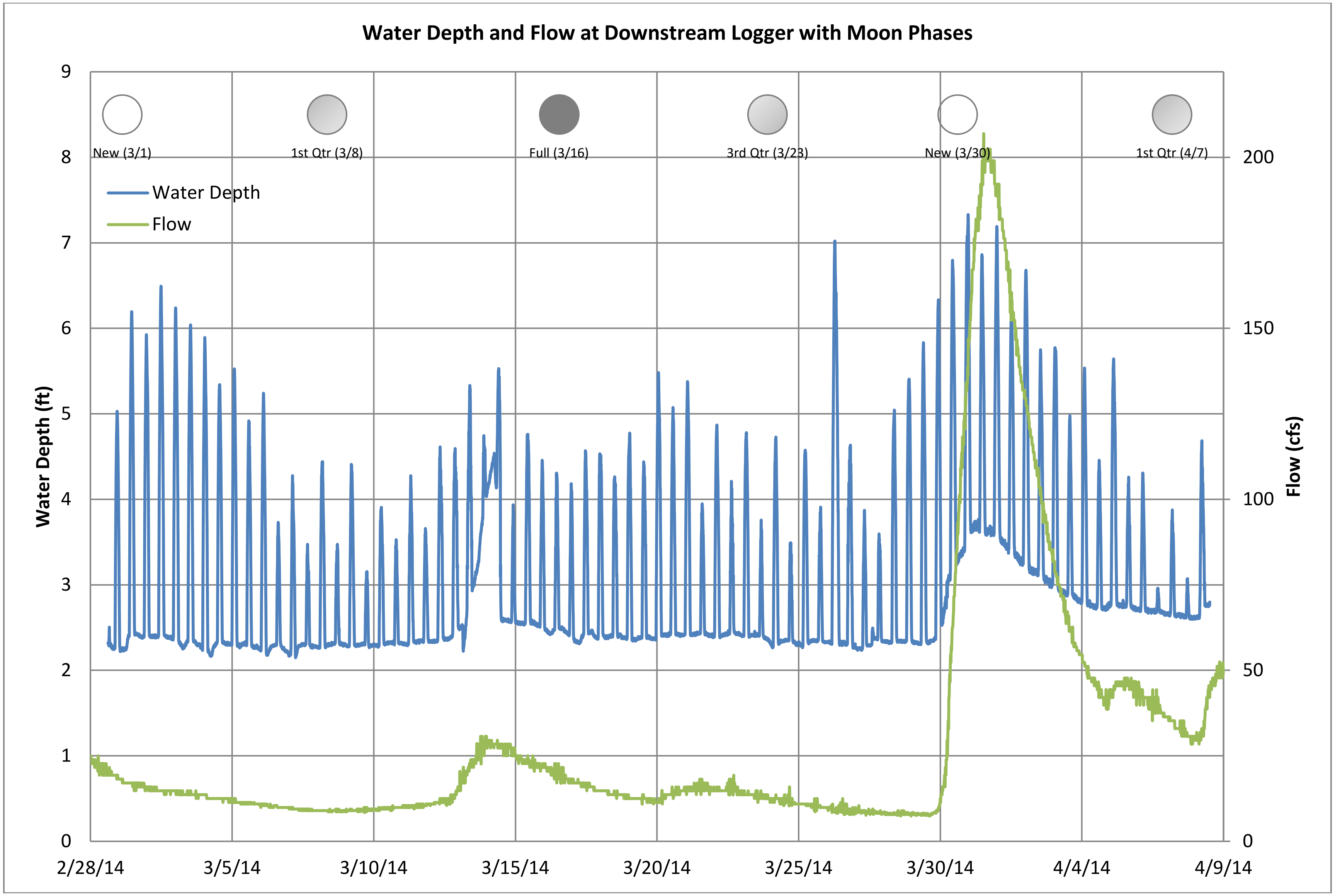
[8] Range of values provided for both the Whitmans Pond Dam gage (10%) and Old Swamp River gage (3%).
[9] About 95% of the time (averaging the upstream and downstream loggers).

Figure 2.4-2: Water Depth and Flow at Upstream Water Level Logger



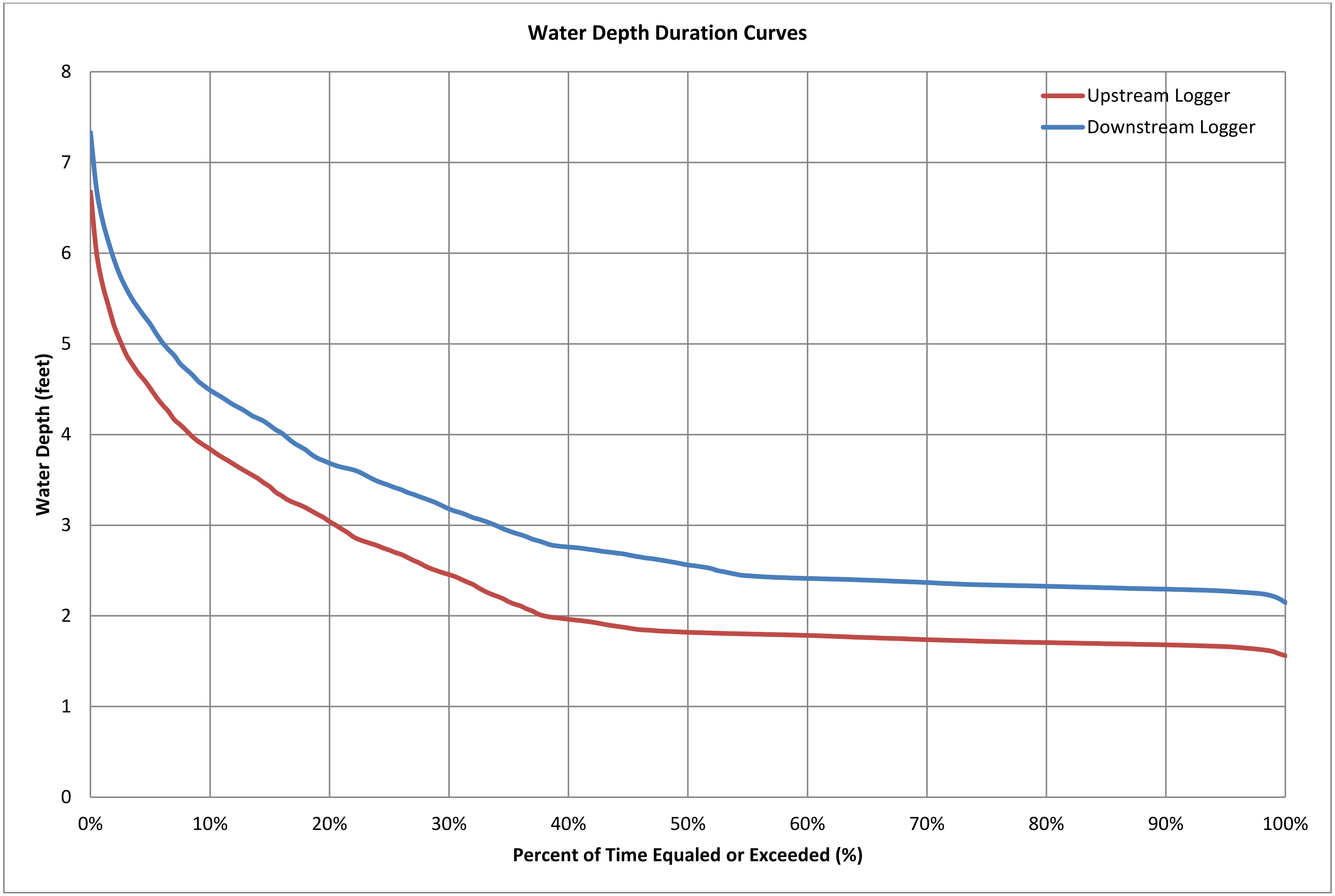
Flow recorded by USGS Gage No. 01105606 at Whitmans Pond Dam, adjusted by ratio of drainage area to the project site (14.1 mi^2 at site / 12.4 mi^2 at gage).

Figure 2.4-3: Water Depth and Flow at Downstream Water Level Logger



Flow recorded by USGS Gage No. 01105606 at Whitmans Pond Dam, adjusted by ratio of drainage area to the project site (14.1 mi^2 at site / 12.4 mi^2 at gage).

Figure 2.4-4: Water Depth Duration Curves at Upstream and Downstream Water Level Loggers



2.5 Flow Capacity of Existing Flood Control Conduit

Due to the uncertain nature of flood frequency estimates for the project site, as well as the fact that flows at the proposed fish diversion location are regulated by the flood control conduit upstream, it was important to estimate the flow capacity of the existing conduit. Various existing sources of information about the conduit as well as a new hydraulic analysis were considered to arrive at a capacity estimate.

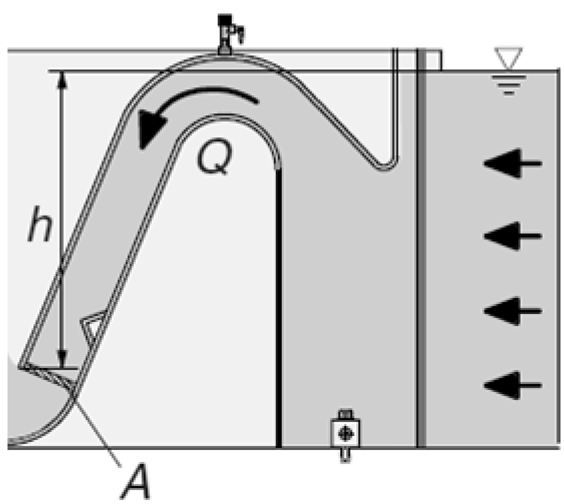
The Herring Brook Flood Control Conduit was constructed by the Massachusetts Department of Public Works, Division of Waterways in two phases—a downstream section (Contract No. 2163) completed sometime in the 1960s, and an upstream section (Contract No. 2664) completed around 1971. The upstream inlet structure is part of Iron Hill Dam (MA 02492, located about 850 feet below Whitmans Pond Dam). It appears that this portion of the conduit was designed/constructed concurrently with Whitmans Pond Dam (MA 00775) just upstream as part of Contract No. 2664. Relevant design plans and reports for the flood control conduit are included in the references in **Section 4**.

Flow to the flood control conduit is regulated by a siphon spillway system at the upstream inlet at Iron Hill Dam. The inlet consists of a set of four rectangular siphon spillways, each approximately 5.5 feet wide by 4.9 feet high, for a total cross-sectional area of 108 feet.

Design Discharge

According to the design report for the structure (Metcalf & Eddy, 1969), the capacity of each siphon is 575 cfs, for a total capacity of 2,300 cfs. However, the report does not provide any design calculations or information about how the capacity was determined. It does note that the maximum reservoir elevation is 62.6 feet msl.[10] Using this information, an attempt was made to verify the reported capacity. It was determined that a flow of 2,300 cfs could be reasonably obtained using the equation for flow through an orifice:

$$Q = C\,A\,(2\,g\,H)^{1/2}$$



[10] Note that a maximum reservoir elevation of 62.4 feet msl is given in the text of the design report, which conflicts with the value of 62.6 feet msl indicated in Figure 5 of the report, so the more conservative (higher) value was assumed.

where:

Q = discharge through an orifice (2,300 cfs)
C = discharge coefficient
A = orifice area (5.5 ft wide x 4.9 ft high x 4 siphons = 108 ft^2)
g = gravitational acceleration (32.2 ft/s^2)
H = head (62.6 ft max reservoir elevation – 47 ft average top of outlet orifice elevation = 15.6 ft)

Back-calculating for the discharge coefficient using these assumptions, a coefficient of 0.67 is obtained. Given that discharge coefficients for siphon spillways typically range from 0.6 to 0.8 (Stickney, 1922), this seems reasonable.

Theoretical Maximum Discharge

However, the theoretical maximum discharge of a siphon spillway is not governed by the orifice equation, but rather the free vortex equation:

$$Q_{max} = V_{crest,\,max}\, R_{crest}\, b\, [ln\,(R_{crown}/R_{crest})]$$

where:

Q_{max} = maximum discharge through a siphon (cfs)
$V_{crest,\,max}$ = maximum velocity of flow over siphon spillway crest (ft/s)
R_{crest} = radius of curvature at crest of siphon (1)
R_{crown} = radius of curvature at crown of siphon (6)
b = width of siphon throat section (5.5 ft x 4 siphons = 22 ft)

It is known that the maximum pressure at the spillway crest is theoretically 34 feet of water at sea level. Allowing for the vapor pressure of water, loss due to turbulence, etc., the maximum net effective head is rarely more than about 25 feet, which corresponds to a maximum velocity of 40 feet per second (Khatsuria, 2004). Using this information in the free vortex equation yields a maximum discharge of about 1,700 cfs, which was assumed to be the limiting capacity of the existing flood control conduit for this analysis.

Dam Safety Inspection Information

The Massachusetts Department of Conservation and Recreation (DCR), Office of Dam Safety (ODS) requires periodic dam safety inspections for jurisdictional dams. Recent dam safety inspection reports for Iron Hill Dam (Pare, 2009 and 2013) also contain flow information about the siphon spillway. However, there are discrepancies in the reported capacities and design flows for the structure:

- 1,195 cfs – reported siphon spillway capacity (Pare, 2013, page 7)
- 2,100 cfs – reported siphon spillway flow for spillway design flood (Pare, 2013, page 12)
- 600 cfs – reported siphon spillway capacity (Pare, 2009, page 7)

Likewise, there are discrepancies in the reported spillway design flood for the dam, which is one half the Probable Maximum Flood (½ PMF):

- 3,544 cfs – Reported ½ PMF (Pare, 2013, page 7 and multiple locations in text)
- 3,520 cfs – Reported ½ PMF (Pare, 2013, page 12; Pare, 2009, page 7)

For this study, Pare was consulted about the discrepancies and their calculation methods. Pare responded that the discrepancies in reported siphon spillway discharges are likely errors due to tables not updating properly, and that the correct value is 2,100 cfs. This corresponds to the design flow for the dam, which is the ½ PMF, or 3,544 cfs. It was calculated using the orifice flow equation assuming a head of 18.5 feet (from the top of the dam crest at elevation 65.5 feet to the top of the outlet orifice at average elevation 47 feet). Back-calculating from those assumptions using the orifice flow equation provided above, it appears that Pare used an orifice discharge coefficient around 0.56 (A. Orsi, personal communication, July 14, 2014).

Pare indicated that the siphon spillway discharge was also calculated for the 100-year flood flow. The FIS 100-year flood flow of 1,040 cfs (approximately 300 feet upstream of Ironhill Street) was used, resulting in a siphon flow of 643 cfs. This calculation was based on the assumption of ogee weir flow (i.e., assuming that siphon flow is not activated during the 100-year flood) (A. Orsi, personal communication, July 14, 2014). Pare's reasoning for this assumption is unclear and does not seem appropriate, given that the siphons were designed to pass flood flows and would likely fulfill that function using the relatively more efficient siphonic action.

It is important to note that the two siphon flows Pare calculated (i.e., 2,100 cfs and 643 cfs) are not actual capacities (as labeled in some locations within the dam safety reports); but rather, they are estimated discharge rates corresponding to specific flood flows (i.e., the ½ PMF or FIS 100-year flood, respectively). In contrast, the 1,700 cfs value calculated using the free vortex equation above is a true capacity based on the physical dimensions of the structure, independent of inflow or head. Therefore, based on the research conducted for this study, it is assumed that flow through the siphon spillway would be limited to 1,700 cfs.

The dam safety reports also indicated that, at the time of the 2009 inspection, Iron Hill Dam could not pass the design flow, or ½ PMF (Pare, 2009). To address this, in 2012, the overflow spillway and primary outlet structure were replaced, raising the total capacity of the dam to be able to pass the design flow with no freeboard[11] (Pare, 2013).

2.6 Proposed Fish Diversion Alternatives Analysis

This section documents the various hydraulic and other factors that were considered to arrive at the selected design for the proposed fish diversion. As a quick check on the hydraulic capacity of the alternative layouts of the diversion wall, the equation for flow over a broad-crested weir was used:

$$Q = C\,L\,H^{3/2}$$

where:

Q = discharge over a broad-crested weir (cfs)
C = weir coefficient (assumed as 3.32[12])
L = effective weir length (ft)
H = head, or water depth over weir (ft)

[11] Freeboard refers to the vertical "buffer" or distance between the reservoir elevation for the given flood and the crest elevation of the dam, above which it would be overtopped by floodwaters. Typically a certain amount of freeboard, such as 1 foot, is desired for the spillway design flood.

[12] Assuming a weir breadth, b, of 2 feet. Coefficients for heads above about 4 feet remain relatively constant (3.32) for b values between 0.5 and 3 feet.

For alternatives that included one or more gates in the diversion wall, the equation for flow through an orifice (provided in **Section 2.5**) was used. In this case, the head parameter would be the difference between the upstream water surface elevation and the elevation of the centroid of the gate opening.

As noted previously, several design concepts were considered during the early project goal development phase, but were dismissed for various reasons. Due to poor design and functioning of the existing gate, the Town was not interested in a gate rehabilitation alternative to deal with the declining condition of the gate. Other alternatives included either a full gate replacement or a wall with a gated opening located at the floor elevation. A concrete wall with a gated opening became the selected design concept when it became apparent that a full gate replacement had notably higher construction costs with less expected longevity than a wall.

Preliminary Design

The preliminary design for the fish diversion included an angled wall with a total effective (centerline) length of about 40 feet. Using the weir flow equation and solving for head, it was determined that if the diversion height were fixed at 9 feet to exclude herring 99% of the time based on the results of the tidal surge analysis (**Section 2.4**), it would only be able to pass a flow of about 1,270 cfs without impacting the 3-foot-deep beam supporting the elevated concrete deck above,[13] which is less than the siphon capacity of 1,700 cfs. Furthermore, a freeboard of at least one foot between the top of the water surface over the diversion and the bottom of the beam is desired for safety.

Reducing the height of the preliminary 40-foot-long diversion wall to 7 feet would allow it to pass a flow of approximately 1,713 cfs (greater than the siphon capacity of 1,700 cfs) with a freeboard of 1 foot to the beam. Therefore, for the preliminary design, it was suggested that the fixed height of the diversion wall should be 7 feet for flood safety purposes, and that water control structures could be added to raise the height to 9 feet to exclude fish 99% of the time during herring migration period.

Various water control structures were considered, including rubber dams, slide or drum gates, and flashboards. Flashboards appeared to be the simplest and most economical option with the significant advantage of not relying on operation or intervention to pass flood flows. As such, the recommended water control structure for the preliminary design was two-foot high wooden flashboards designed to automatically trip at a head of about 2 feet. A concept plan of the preliminary design is shown in **Figure 2.6-1** at the end of this section.

However, project partners decided that the use of flashboards was not ideal. Flashboards require various components that would need to be purchased, maintained, and eventually replaced. Additionally, they could potentially fail at flows lower than intended and be difficult to replace during high spring flows, resulting in the possibility for fish to enter the flood control conduit. The most effective fish barriers have no crest operations or movable parts. The project team concluded that a slight reduction in percent of fish excluded (down to a minimum of 90%) would be acceptable in order to obtain a fixed height structure with lower operation and maintenance requirements.

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[13] The total clearance between the existing concrete pad and the bottom of the beam is about 13.5 feet.

Concept Design Alternatives

Based on this feedback, three alternative design concepts were developed to provide additional flood flow capacity with a higher fixed height. All three alternatives included a concrete diversion wall with a fixed height of 8.5 feet above the concrete pad. According to the results of the tidal surge analysis (**Section 2.4**), a wall height of 8.5 feet would still be expected to exclude herring 99% of the time, but with a lower factor of safety (1.5 feet of separation between the downstream water surface elevation and the top of the wall, instead of the recommended 2 feet). Concept plans of each alternative are shown in **Figures 2.6-2** through **2.6-4** at the end of this section. A description of the alternatives follows:

1. **Alternative 1 – One Gate:** This alternative included one gate with an overflow weir section. Above the gate would be a non-overflow section that would be the same elevation as the adjacent existing grade and allow for direct operation of the gate from that level. Hydraulically, the flow would be split between the gate and the weir during the design flood.

2. **Alternative 2 – Two Gates:** This alternative included a long weir for overflow plus two gates that would pass about 40% of the flow during the design flood. One gate would be operated from the side of the channel and the other would be operated from the elevated deck above. The entire weir would overtop with flow. The wall would be reoriented from the preliminary design to direct the gate discharges downstream.

3. **Alternative 3 – Extended Weir:** This alternative included a weir that was extended approximately 15 feet farther downstream than the two other options. A small (6 feet wide by 3.5 feet high), upward opening slide gate would be installed as a low level outlet. It would not require operation during a storm, but would be opened outside of fish migration period to allow the flood conduit to drain. This gate would be operable from the canal wall.

The first option was attractive from a structural design perspective. However, it would require gate operation during a flood event, which is not ideal due to the potential for gates to become stuck, lose power (if electric), become inaccessible due to inundated roads, and tie up emergency personnel resources. From a hydraulic standpoint, the second alternative (with two gates) provided redundancy in case one gate becomes stuck and can't be opened during a flood. However, it would be more complex to build and operate, and would likely have a higher associated cost as well.

In the end, the third option with the extended weir length (totaling approximately 55 feet) was selected as the preferred alternative for final design as it would provide passive flood capacity and does not require gate operation during most flood events. The capacity of the proposed diversion wall to pass flood flows is discussed in **Section 2.7** and additional details of the proposed design are presented in **Section 3**.

Figure 2.6-1: Preliminary Design Alternative

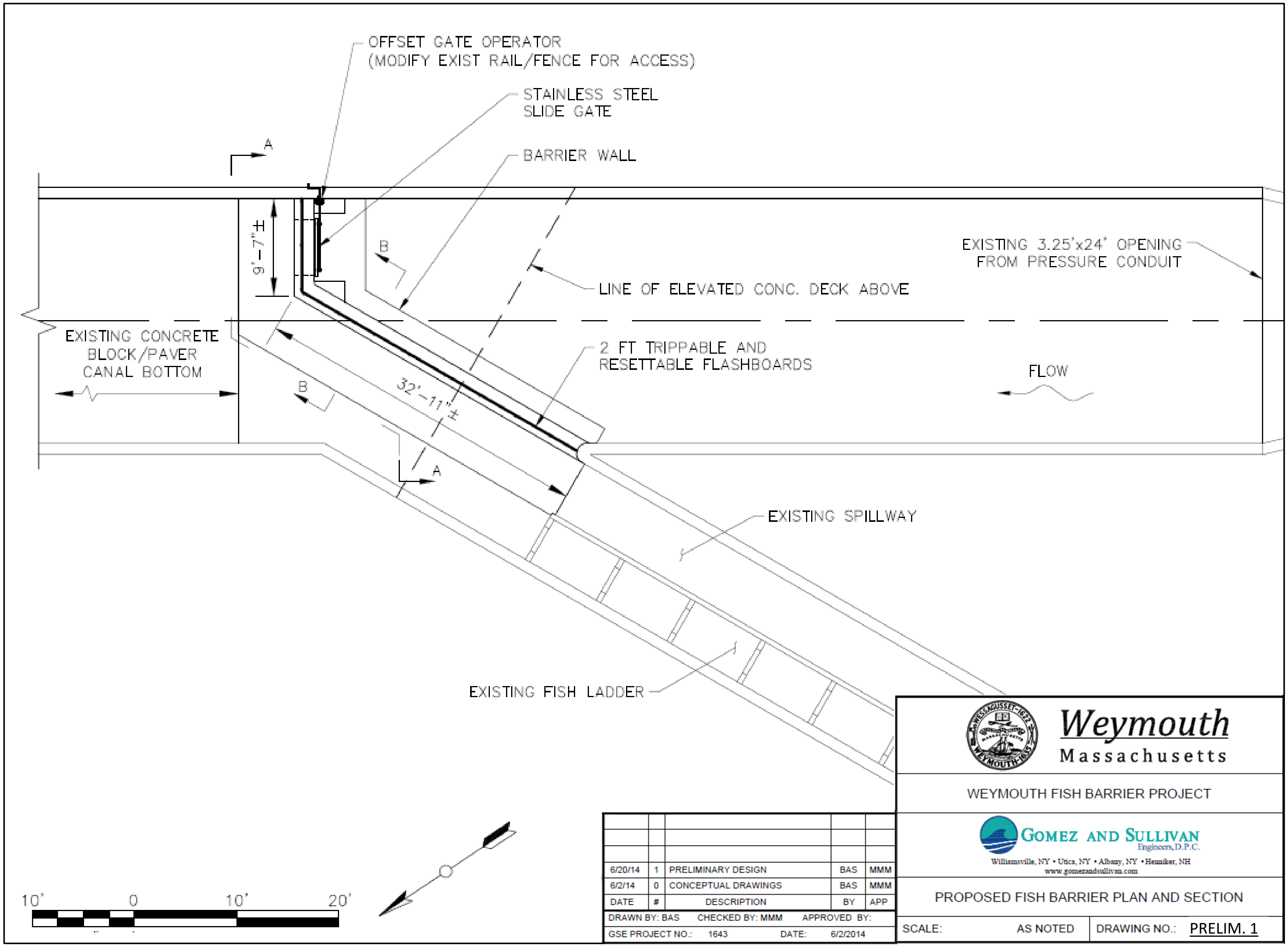


Figure 2.6-2: Concept Design Alternative #1 – One Gate

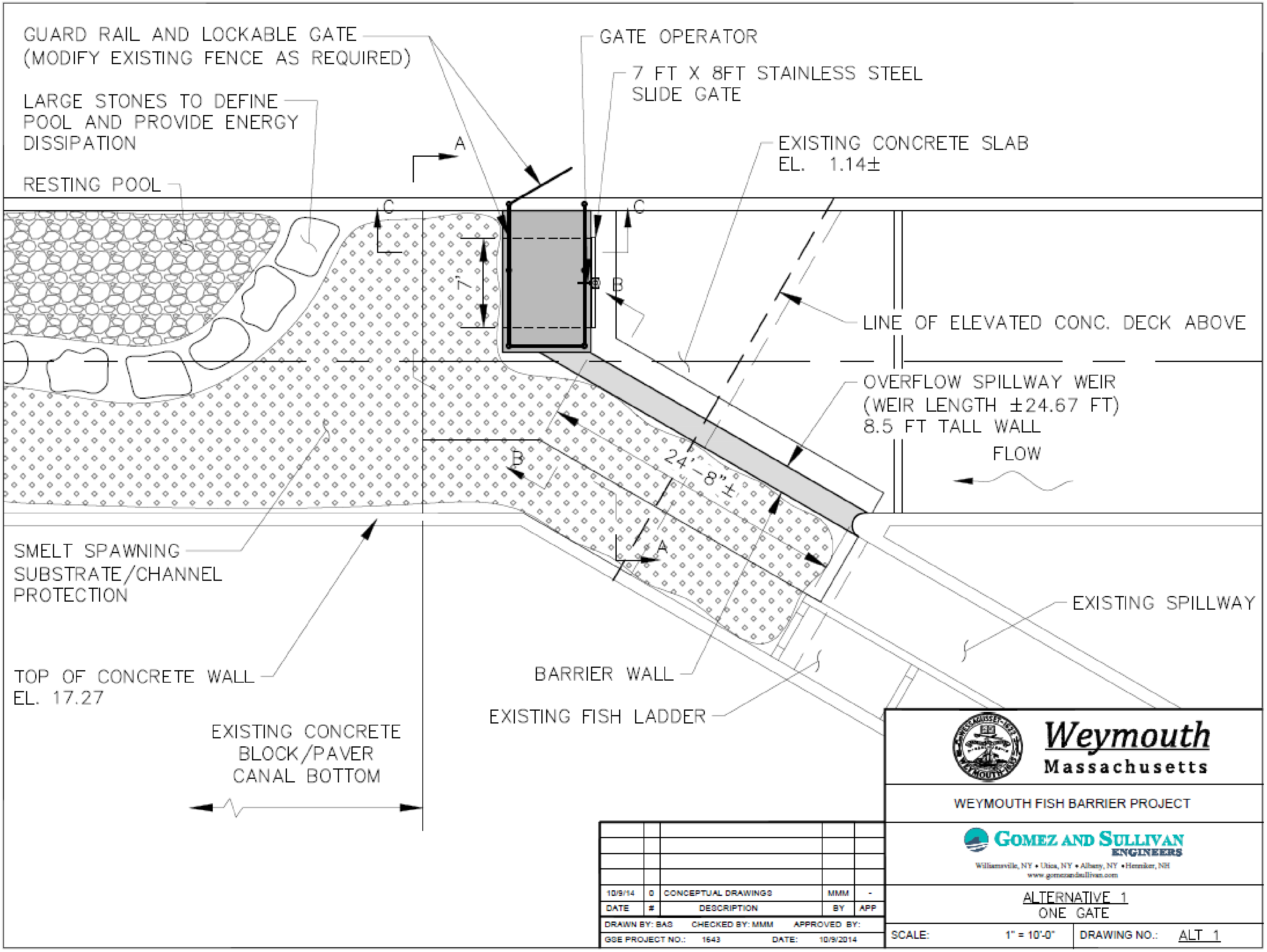


Figure 2.6-3: Concept Design Alternative #2 – Two Gates

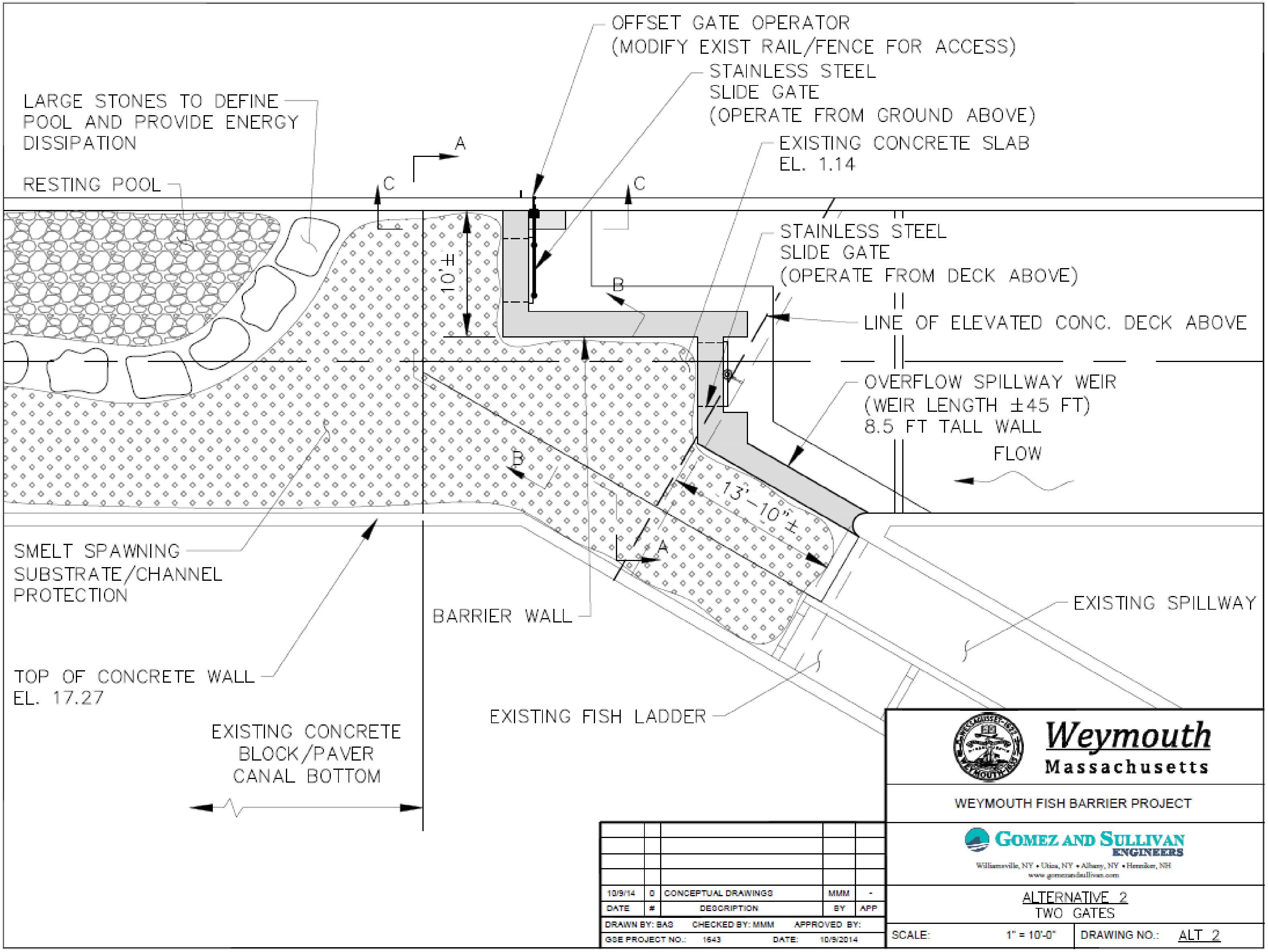
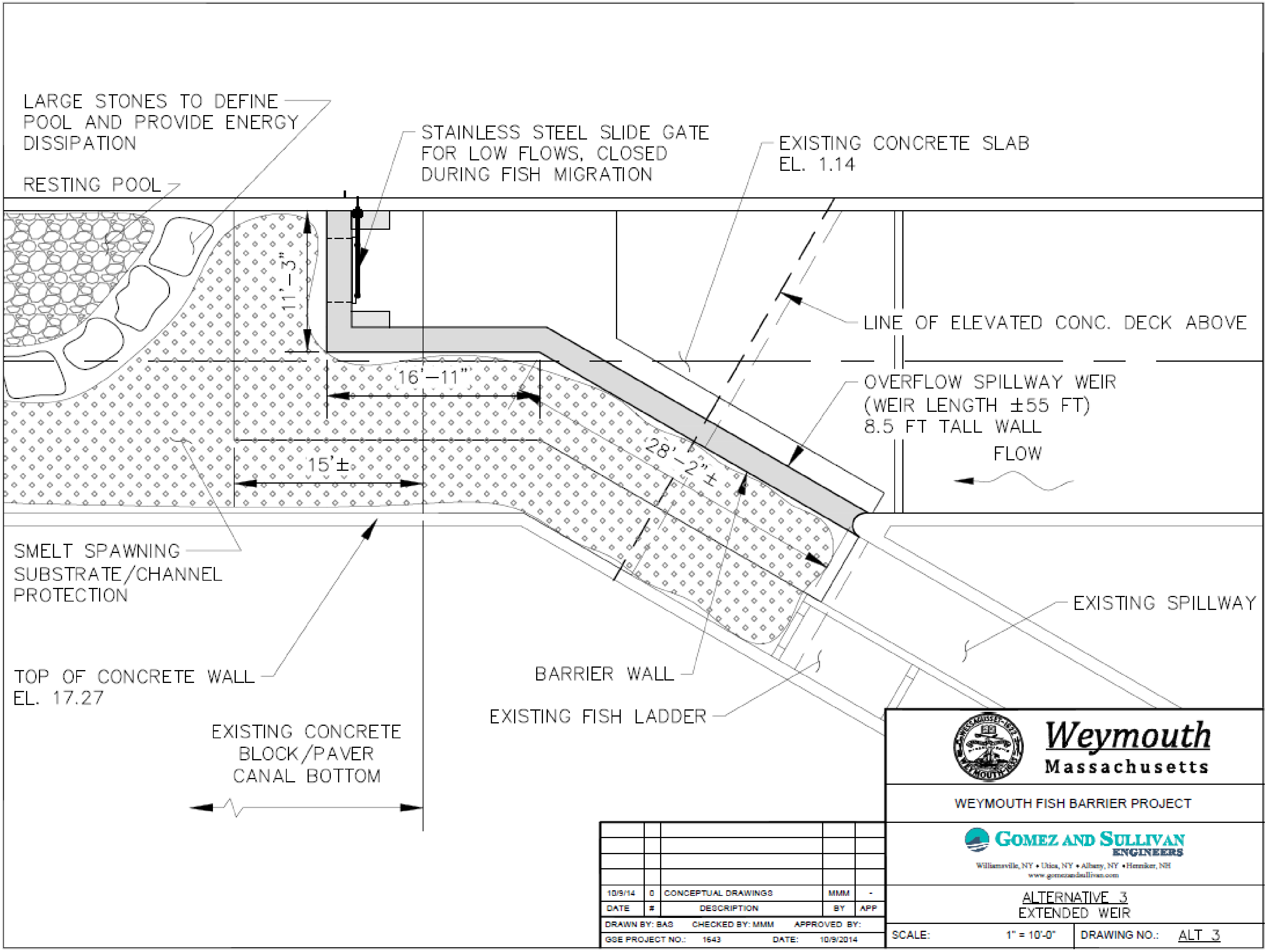


Figure 2.6-4: Concept Design Alternative #3 – Extended Weir



2.7 Hydraulic Capacity of Proposed Fish Diversion

The proposed diversion wall is approximately 8.5 feet high with an overall length of approximately 55 feet and a thickness varying from 2 to 3 feet. A 6-foot-wide by 3.5-foot-high stainless steel slide gate (upward opening) will be installed as a low level outlet. Using this information, the hydraulic capacity of the proposed wall to pass flood flows was estimated.

The weir flow equation (provided in **Section 2.6**) indicates that the proposed structure could pass approximately 1,460 cfs with the gate closed and 1 foot of freeboard to the support beam above. With the gate opened and no freeboard between the water surface and the beam, a combination of weir flow and orifice flow (provided in **Section 2.5**) equations indicate that the structure could pass up to about 2,450 cfs. For reference, the 100- and 500-year flood flows (according to the FIS) are about 1,100 and 1,860 cfs, respectively, and the capacity of the siphons at the inlet to the flood control conduit is assumed to be 1,700 cfs.

However, hydraulics in the project area are complex and are influenced by the flood conduit's siphon spillway inlet, open channel flow in Herring Brook, tidal conditions, and a downstream railroad crossing constriction. The weir and orifice flow equations do not take into account any potential reduction in the efficiency of the weir to pass flood flows due to a high "tailwater," or downstream water surface elevation. If the tailwater is high enough, the weir may become "submerged" which reduces its capacity. As such, a three-dimensional computational fluid dynamics (CFD) model was developed in order to more thoroughly analyze the capacity of the proposed diversion wall to pass flood flows without impacting the beam supporting the elevated concrete deck above.

A schematic of the model layout is shown in **Figure 2.7-1**. It encompasses approximately 500 linear feet, including the fish ladder, a portion of the existing flood control conduit, the stilling basin at the outlet of the flood control conduit, and the downstream channel. Existing drawings indicate that the elevation of the channel bottom in the area of the proposed wall vary from approximately 1.1 to 1.5 feet. The top of the new diversion was set to be a minimum of 8.5 feet above the channel bottom at elevation 10.0 feet. A separate model geometry was created to represent the gate opening[14]. Sensitivity analyses were run with each of these geometries to determine the impacts of operating the gate.

Boundary Conditions

A CFD model requires boundary conditions, which are known inputs (such as known water surface elevations or source flows) that allow the model to establish starting water surface elevations at the upstream and downstream extents.

For the downstream boundary condition, the known water surface elevation for the 500-year flood was used. The FIS indicates that the water surface elevation expected in the vicinity of the downstream channel under a 500-year flood event (1,858 cfs) is approximately 14.5 feet (i.e., 4.5 feet above the

[14] The model was developed with an earlier iteration of the gate opening width (7 feet instead of the final design width of 6 feet). However, the slight reduction in gate opening area (less than 15%) is not expected to significantly affect the modeling results, as the gate contributes a low percentage of the overall structure capacity. Additionally, as described in the results, the modeled flows are conservative and more than account for the slight reduction in flow through the gate.

proposed fish diversion)[15]. As such, a specified water surface elevation of 14.5 feet was utilized for the downstream boundary condition. Sensitivity analyses were run for each of the geometries (i.e., gate closed and gate open) with the downstream boundary at elevations 10.5 and 12.5 feet (i.e., 0.5 and 2.5 feet above the fish diversion) as well.

For the upstream boundary conditions, inflows for the flood control conduit and the surface channel of Herring Brook (i.e., to the fish ladder channel adjacent to the tunnel outlet) were needed. However, no stage versus discharge rating curve was found for Iron Hill Dam. As such, the distribution of flow between the flood control conduit and the surface channel for a given flood (e.g., the 500- year flood) was not known. The dam safety report does indicate, though, that for the Iron Hill Dam spillway design flood (i.e., the ½ PMF or 3,544 cfs), approximately 2,100 cfs (60% of the total flow) would be conveyed by the flood control conduit and 1,435 cfs would enter the surface channel. These values represent the best available information and are conservatively higher than the 100- and 500-year floods (1,100 and 1,860 cfs, respectively) and the assumed capacity of the conduit (1,700 cfs).

As such, the inflow value of 2,100 cfs was used as the upstream boundary condition for the flood control conduit in the model. However, weir calculations indicate that the reported flow for the surface channel (1,435 cfs) would not be contained by the concrete channel in the area of the fish ladder (just upstream of the tunnel outlet) without overtopping the channel walls. Therefore, a specified water surface elevation of 17.85 feet (corresponding to the top of the concrete channel walls upstream of the fish ladder) was set as the upstream boundary condition for the surface channel.

Model Results

The model indicates that the surface channel of Herring Brook in the area of the fish ladder (just upstream of the tunnel outlet) is able to pass approximately 520 cfs prior to overtopping the concrete channel walls. Therefore, the total amount of flow reaching the area downstream of the proposed diversion in the model is approximately 2,620 cfs (which is still conservatively higher than the 100- and 500-year floods). Additional results of the various model runs are presented in **Table 2.7-1** below.

The results show that the proposed fish diversion structure would be able to pass in excess of the 500-year flood without impacting the elevated concrete deck if the gate is open. Furthermore, from the data, it can reasonably be assumed that the structure would be able to pass the 100-year flood with the gate closed and still maintain at least 1 foot of freeboard between the water surface elevation and the support beam. This is based on the fact that the FIS reported tailwater drops over 3 feet from the 500-year to the 100-year flood and the model indicates that the height of water over the weir drops with the tailwater (in addition to the flow dropping from the modeled 2,620 cfs to the 100-year flood flow of 1,100 cfs). It can also be inferred that the weir should not limit the discharge from the siphons or impact the spillway capacity of Iron Hill Dam under these conditions (i.e., 100-year flood with gate closed or 500-year flood with gate open). The operation and maintenance manual for the fish diversion structure will specify that the gate should be opened if flows are anticipated to be in excess of the 100 year storm.

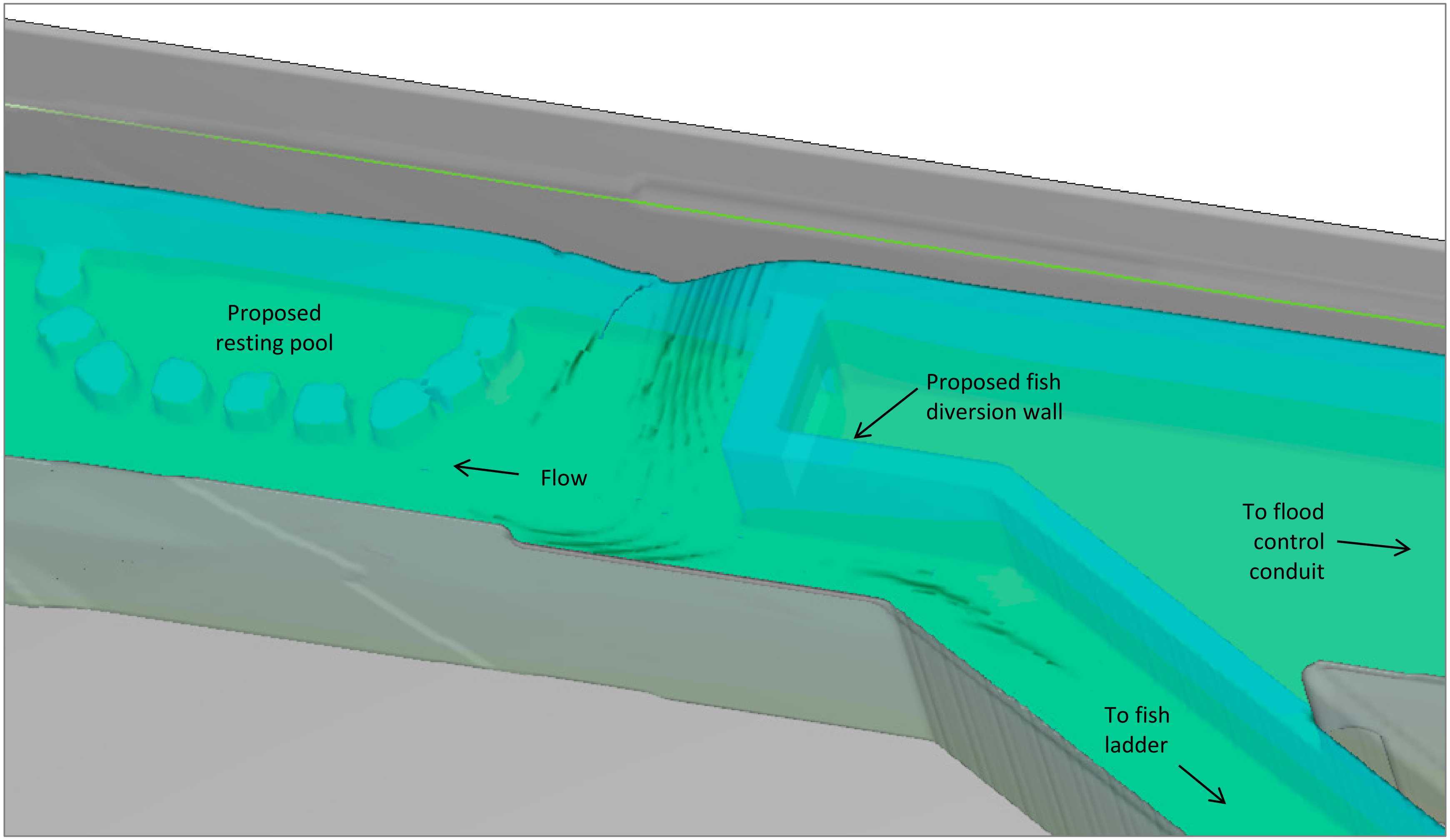
[15] Note that the FIS water surface profiles incorrectly show the Herring Brook channel bottom rising steeply between the railroad crossing and Broad Street. In reality, the elevation change does not occur until upstream of the project site at the fish ladder, around station 2,200 feet upstream of the confluence with Weymouth Back River. Consequently, the elevation of the 500-year flood was extrapolated from the flat backwater area upstream of the railroad crossing.

Table 2.7-1: Summary of CFD Model Results

Gate Position	Downstream Water Surface Elev. (ft)	Flow through Gate (cfs)	Water Surface Elev. at Fish Diversion (ft)	Beam Impacted*
Gate Closed	10.5	N/A	15.9	Yes
	12.5	N/A	16.4	Yes
	14.5	N/A	17.1	Yes
Gate Open	10.5	430	14.4	No
	12.5	390	15.0	No
	14.5	370	15.8	No

**The bottom of the concrete deck support beam is approximately at elevation 15.8 feet where it crosses over the proposed fish diversion wall.*

Figure 2.7-1: CFD Model Schematic



This screenshot from the CFD model shows the concrete channels leading from the flood control conduit and fish ladder, the proposed fish diversion wall, the proposed resting pool (bordered by large granite blocks), and the modeled water surface.

3. Proposed Design

3.1 Details of the Proposed Design

Drawings of the proposed design are provided in **Appendix B**.

Fish Diversion

The proposed fish diversion will be constructed of reinforced concrete as a cantilever type wall. The wall will be 8.5 feet high with an overall length of approximately 55 feet and a thickness varying from 2 to 3 feet. The existing metal swing gate and concrete pad will be removed. The new concrete wall stem will extend vertically from a new concrete footing and pad. The proposed wall will have an extended toe (downstream section of footing apron) and narrow heel (upstream section of footing apron) to maximize overturning resistance. A key placed below the existing apron will provide protection against potential undermining of the soil at the foundation. A metal angle will be placed on the downstream side of the wall to act as a diversion for climbing eels.

A 6-foot-wide by 3.5-foot-high stainless steel slide gate (upward opening) will be installed as a low level outlet. Type 316L stainless steel was specified by the Town for the added corrosion protection in the harsh environment. The gate will be closed to prevent herring from accessing the flood control conduit during the herring migration period (approximately March 1 through June 30), but will be kept open at other times of the year to allow water to freely flow from the flood control conduit and not be impounded by the wall.

Due to the extent of overtopping and the ground conditions indicated by soil boring logs prepared for the original construction of the flood control conduit, micropile anchors were selected to resist the significant forces anticipated during flood flows. These anchors will be drilled into the ground and grouted into the subsurface soil, or rock if encountered, and will extend into the wall stem. The anchors are designed to allow the wall to remain stable at the anticipated flood loads.

The proposed wall will be angled to align with the existing fish ladder. This configuration will provide increased weir length for flood protection and will enhance attraction to the fish ladder because the majority of the water spilled over the diversion will fall at the base of the fish ladder. Presently, under some conditions of higher flows, fish can be more attracted to the flood control conduit than the fish ladder because the conduit flow is undiluted Whitmans Pond water whereas the fish ladder can receive more stormwater runoff. This alignment also allows the operable gate to be located out from underneath the existing elevated deck allowing for easier access, maintenance, and operation. For all of these reasons, a wall angled with relation to the channel was preferred to a wall perpendicular to the channel such as the existing metal gate.

Hydraulics in the channel are complex and are influenced by the flood conduit's siphon spillway inlet, open channel flow in Herring Brook, tidal conditions, and a downstream railroad crossing constriction. Considering all of these factors, the diversion wall was designed to pass the 100-year flood flow (1,100 cfs) with over 1 foot of freeboard to an existing elevated deck concrete support beam above the wall with the gate closed, and in excess of the 500-year flood flow (1,860 cfs) with no freeboard and the gate opened. At the 500-year flood flow, the structure will impound less than 5 acre-feet, contained entirely within the existing flood control conduit.

Channel Improvements

Improvements to the channel downstream of the fish diversion will be constructed to reestablish smelt spawning habitat and to restore a resting pool for herring. For the smelt spawning habitat, the concrete pad below the fish ladder and wall will be covered with a 12-inch layer of grouted rip-rap (consisting of 6- to 12-inch-diameter stone) topped by a 12-inch layer of loose 4- to 8-inch-diameter cracked stone. An additional 2 cubic yards of 4- to 8-inch cracked stone will be spread over the channel downstream of the grouted section. For the resting pool, the channel downstream of the concrete pad will be excavated to approximate the former pool dimensions of about 3 to 4 feet deep, 15 to 20 feet wide, and 30 feet long (for a total volume of 50 to 90 cubic yards (CY)).[16] Large stones with major dimensions on the order of three feet and weighing approximately one ton will be used to define the extent of the restored resting pool and act as energy dissipaters to help prevent future washouts of the substrate.[17] Additionally, an unauthorized rock weir at the downstream extent of the concrete-walled channel will be regraded to restore flow depths and velocities suitable for smelt spawning. This will involve distributing the approximately 10 CY of rocks comprising the weir up- and downstream over a length of about 150 feet and a slope of approximately 0.5%.

3.2 Construction Methods

An overview of the proposed construction plan is shown in **Figure 3.2-1**. A more detailed proposed plan for construction access and water, erosion, and sedimentation controls is shown on **Drawing C1** of the design plans in **Appendix B**. Additional notes are provided on the cover sheet and details of the proposed water control system are shown on **Drawings C2**. Note that the proposed plan only represents the recommendation of the engineer. The selected contractor for the project will be required to submit a construction sequence plan, which will include proposed means, methods, and phasing required for water, erosion, and sedimentation control. The plan will need to be approved by the project engineer and the Town and adhere to all conditions contained in relevant permits.

Access

Construction access and staging areas for the project will primarily be located on an existing parking area, open field, and paved paths on Town lands adjacent to the Lovell Playground and a skate park on the west side of Herring Brook. The total disturbance area is anticipated to be less than 1 acre. Disturbance to existing park plantings will be minimized. A crane is recommended to lift a mini-excavator or small skid-steer loader into the channel to conduct the work. The machine would be removed from the channel at the end of each work day. Temporary gravel access roads will be constructed for routes crossing vegetated areas or existing paved paths.

Water Control

Water control at the site will consist of 1) stopping inflow into the flood control conduit at the intake, 2) bypassing water from the surface channel of Herring Brook (i.e., upstream of the fish ladder) around the work area, and 3) controlling backwater from downstream (including tidal surges).

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[16] Note that the design plans indicate excavating an area only 10 feet wide and only to the depth of the existing concrete block pavers, not below. The 15 to 20 foot width and 3 to 5 foot depth (approximating the former dimensions of the resting pool) will be specified in a design addendum to be developed by the Town of Weymouth.
[17] The Town's design addendum will also specify sinking the large perimeter stones deeper so they rise only 6 to 12 inches above the surrounding substrate.

In order to address the inflow into the flood control conduit, stoplog slots in the existing siphon intakes at Iron Hill Dam can be fitted with boards to close the conduit. With the siphons closed, all flow will be diverted to the surface channel of Herring Brook.

To control surface channel flow and tidal surges at the construction area, a cofferdam and gravity bypass pipe system is recommended. This system will divert flow around the work area and safely pass any juvenile herring migrating downstream. The cofferdams will need to be on the order of five to six feet tall to effectively isolate the construction site. Because of the narrow nature of the channel and the need for a relatively tall structure, a prefabricated cofferdam such as Portadam is recommended. Dewatering of the work area will be accomplished by pumps directed to a dewatering area in an open field. After initial dewatering, only minimal maintenance pumping of runoff entering the work area is anticipated. The discharge water is not expected to be contaminated.

The rock weir grading is proposed to be completed within the wetted channel downstream of the cofferdam diversion during a period of low flow.

Erosion and Sedimentation Control

The project is not anticipated to have significant erosion and sedimentation impacts as the site and the nature of the construction activities are not particularly susceptible to erosion. The proposed construction access and staging area is essentially flat with no steep slopes. The Herring Brook channel through the project area has vertical concrete side walls and a bottom lined with either solid concrete or concrete block pavers.

Applicable soil erosion and sedimentation control notes are shown on the cover sheet and Drawing C1 of the design drawings (**Appendix B**). The selected contractor will be responsible for developing and implementing a plan to control construction-related impacts, including erosion, sedimentation, and other pollutant sources during construction and land disturbance activities. The plan will be required to comply with all conditions contained in relevant permits and must be approved by the engineer and the Town.

During construction, temporary erosion, sedimentation, water, and pollution controls will be utilized in accordance with Best Management Practice (BMP) guidelines recommended by MassDEP. To prepare the site, natural vegetation will be preserved to the extent practicable. (For this reason, a preliminary access route option passing south of the skate park was abandoned to preserve existing tree plantings.) Erosion of proposed access routes (through a mowed field and along existing paved footpaths) will be controlled by installing a stabilized construction entrance and gravel access roads. Erosion and sedimentation due to stormwater runoff will be managed with approved measures such as silt socks or entrenched silt fences installed at the limits of all work/disturbances. Disturbed and stockpile areas will receive temporary seeding/mulching/rip-rap as appropriate. Dust will be controlled as necessary. As noted, pumping will only be needed during initial dewatering and then for minimal maintenance needs thereafter. Pump discharge will be directed into filter bags to capture fine sediments. The site will be restored to its former condition following construction.

Timing

The project should be constructed during a period of relatively low flow and at a time that will have the lowest impact on marine resources (including smelt spawning and river herring migration).

Construction of the fish diversion would likely take on the order of 1 week for cofferdam installation and dewatering, 1 week for demolition, 2 weeks to form and pour the concrete, 1 week for the gate installation, and 1 week to remove the cofferdam system, totaling approximately 6 weeks. Considering additional time needed for mobilization/demobilization, construction of temporary access roads, installation of sedimentation and erosion controls, implementation of the channel improvements, and site restoration, about 2 to 3 months should be allotted for the entire construction period.

The project area is located in a coastal zone and therefore is subject to *MarineFisheries'* recommendations for seasonal or "time of year" restrictions (TOYs) on in-water construction work. The TOY date ranges were established to provide protection to marine resources during times when there is a higher risk of known or anticipated significant lethal, sublethal, or behavioral impacts. Adverse impacts to marine fisheries resources can result from suspension of fine grain sediments, lowered dissolved oxygen levels, impediments to migration, direct removal of important shelter, forage, or spawning habitat, and direct mortality. The TOY restriction for the Weymouth Back River Area of Critical Environmental Concern (ACEC; within which the project area is located) recommends avoiding in-water construction work from February 15 through November 15 (Evans et al., 2015). At least a spring TOY is likely for this project.

Table 1.4-1 provided information about the timing of important life cycle events for target diadromous species that utilize the project area seasonally. Spring construction is not recommended due to smelt spawning (March through May) and upstream migrations of river herring (March through June) and American eel (April through July), as well as typically high flows. Downstream migration of juvenile herring occurs from July through November. However, it is anticipated that fish can be safely passed downstream by the proposed gravity bypass system. Therefore, the recommended construction period is August through October to minimize impacts to marine resources and take advantage of relatively low flows, pending approval by *MarineFisheries*. Alternatively, flows are also low in the winter (December to February) and diadromous fish species are not likely to be present in the project area during this time. Construction during the winter would require freeze protection for concrete.

3.3 Opinion of Probable Construction Cost

An opinion of probable construction cost (OPCC) for the proposed fish diversion and channel improvements is provided in **Table 3.3-1**. The OPCC was developed using the DOT's published weighted average bid prices[18], R. S. Means Construction Cost Data, and available final costs from comparable projects. The OPCC itemizes costs for mobilization/ demobilization, access and water handling, erosion and sediment control, removal of the existing diversion, and construction of the new diversion, smelt spawning habitat, resting pool, and downstream rock weir grading. A contingency of 20% was included and an allowance of $25,000 was added for bidding and construction phase services.

[18] Median prices for all districts from the period of 2013 to 2014. DOT's Standard Specifications for Highways and Bridges provide more detail about methods and included services for each item.

Table 3.3-1: Cost Estimate for Weymouth Herring Passage & Smelt Habitat Restoration Project

Category	Item	Unit*	Qty	Unit Cost	Total Cost
Mobilization/ Demobilization	Mobilization/demobilization	LS	10%	$451,165	$45,116
				SUBTOTAL	**$45,116**
Site Access	Temporary fence	LF	310	$11	$3,410
	Silt fence	LF	240	$5	$1,200
	Selective clearing & thinning	SY	190	$3	$570
	Clearing & grubbing	SY	170	$4	$680
	Geotextile fabric (for separation)	SY	170	$6	$1,020
	Gravel subbase (M2.01.7)	CY	250	$56	$14,000
	Crushed stone, 1-1/4" (M2.01.3)	TON	20	$40	$800
	Chain link fence removed & reset	FT	30	$25	$750
	Crane	MO	1	$5,100	$5,100
				SUBTOTAL	**$27,530**
Water Control	Cofferdam	LS	1	$19,685	$19,685
	Sandbags	EA	125	$1	$125
	Sand borrow (M1.04.0 a)	CY	60	$40	$2,400
	Water diversion pump	LS	1	$7,400	$7,400
	Bypass pipe	LS	1	$22,300	$22,300
	Dewatering bag	EA	1	$75	$75
	Stoplogs (3 x 12 x 12' lumber)	EA	40	$20	$800
				SUBTOTAL	**$52,785**
Diversion Wall	Demolition	LS	1	$7,440	$7,440
	Concrete excavation	CY	40	$500	$20,000
	Concrete block removal/salvage	LS	1	$2,400	$2,400
	Earth excavation	CY	170	$25	$4,250
	Micropiles	LF	360	$155	$55,800
	Gravel subbase (M2.01.7)	CY	70	$56	$3,920
	Concrete (4500 psi)	CY	143	$1,370	$196,088
	Water resistant admix	LB	663	$3	$1,990
	Waterstops	LF	225	$7	$1,572
	Stainless steel slide gate, 72" x 42"	EA	1	$28,500	$28,500
	Chain link fence gate with posts, 60"	FT	3	$142	$426
				SUBTOTAL	**$322,386**
Habitat Restoration	Dredging & disposal of material	CY	5	$45	$225
	Grouted rip-rap	SY	292	$120	$35,040
	Modified rockfill (M2.02.2)	CY	26	$75	$1,975
	Granite blocks (3' x 3' x 2')	LS	1	$8,824	$8,824
	Rock weir grading	DAY	1	$2,400	$2,400
				SUBTOTAL	**$48,464**
				SUBTOTAL Direct Construction Cost	**$496,281**
				Contingency Allowance (20%)	*$99,256*
				TOTAL Direct Construction Cost *(rounded up to the nearest $1000)*	**$596,000**
				Bidding & Construction Phase Services	*$25,000*
				TOTAL OPINION OF PROBABLE CONSTRUCTION COST ($2015)	**$621,000**

**See List of Abbreviations for descriptions of unit abbreviations.*

3.4 Regulatory Review

The following regulatory submittals, reviews, and permits are anticipated to be required for this project. Applications and forms will be submitted to the appropriate agencies as part of this contract.

Table 3.4-1: List of Anticipated Required Regulatory Reviews and Permits

Permit/Review	Agency	Applicability
Environmental Notification Form (ENF)	MA Environmental Policy Act (MEPA) Office	Review thresholds exceeded include: 1) alteration of 1,000 or more sf of outstanding resource waters, 2) new fill or structure or expansion of existing fill or structure in a regulatory floodway, 3) construction, reconstruction or expansion of an existing solid fill structure of 1,000 or more sf base area occupying flowed tidelands or other waterways, and 4) any Project within a designated ACEC.
401 Water Quality Certificate (WQC)	MA Dept. of Environmental Protection (MassDEP)	Dredging or any activity resulting in the discharge of dredged or fill material (e.g., sediment release) greater than 100 CY or any amount in an Outstanding Resource Water (ORW) that is also subject to federal regulation. Major Project Certification for Fill & Excavation required due to fill in an ORW.
Chapter 91 Waterways License	MassDEP	Dredging of a navigable waterway.
Chapter 253 Jurisdictional Determination	MA Dept. of Conservation & Recreation (DCR) Office of Dam Safety (ODS)	Any project to construct, repair, materially alter, breach, or remove a dam. Proposed structure is 8.5 ft high and impounds less than 5 ac-ft (in the existing flood control conduit) at maximum pool. As such it does not meet the definition of a dam (> 25 ft or > 50 ac-ft), but does meet criteria requiring a jurisdictional determination (> 6 ft or > 15 ac-ft).
Project Notification Form (PNF)	MA Historical Commission (MHC)	Projects that require state funding, licenses, or permitting.
Section 106 Historic Review		Projects that require federal funding, licenses, or permitting. Jackson Square is a historic district and the Herring Run is a historic structure.
Coastal Zone Management Act (CZMA) Federal Consistency Review	MA Office of Coastal Zone Management (CZM)	Most projects that: 1) are in or can reasonably be expected to affect a use or resource of the MA coastal zone, and/or 2) require federal licenses or permits, receive certain federal funds, or are a direct action of a federal agency.
Clean Water Act Section 404 Programmatic General Permit	US Army Corps of Engineers (USACE)	Discharge of dredged or fill material in a water of the United States, or instream construction activities. Anticipated to require Category II review due to proposed fill.
Fishway Permit	MA Div. of Marine Fisheries (DMF)	Any activity to construct, reconstruct, rebuild, repair, or alter any anadromous fish passageway.
Wetlands Protection Act Notice of Intent (NOI) & Order of Conditions	MassDEP / Conservation Commission	Any construction in or near a wetland resource. Anticipated to qualify for a Restoration Order of Conditions general permit as a fish passage improvement project. Project is not located within Estimated or Priority Habitat of Rare Species, so is not subject to the Massachusetts Endangered Species Act (MESA) review.
National Pollutant Discharge Elimination System (NPDES) Permit	Environmental Protection Agency (EPA)	Discharges from certain construction sites, including clearing, grading, and excavation activities. Since disturbance will be < 1 acre and discharge is not anticipated to be contaminated, a Dewatering General Permit (DGP) may be required, or the project may potentially be covered as allowable non-stormwater discharge under the community's Small MS4 Permit, or there may be no NPDES permit requirement.

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