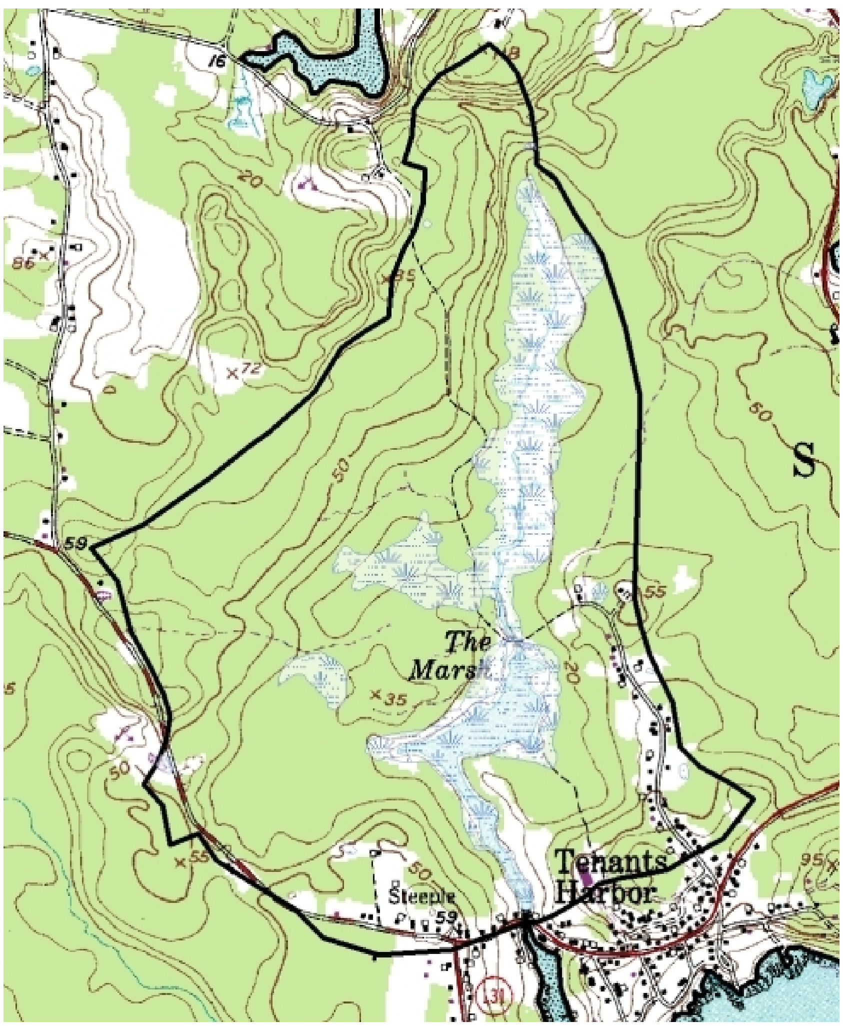
Ripley Brook – St George (Tenants Harbor) – WIN 19267

Hydrology and Fish Passage Report

Ripley Brook drains a small watershed through a culvert beneath ME131 in Tenants Harbor / St George (Figure 1). The project culvert is located at approximate head of tide. There is a small pond ("The Marsh") upstream of the culvert that has been identified as alewife habitat. Therefore, the culvert replacement will be designed to improve alewife passage. The current structure is a dilapidated 5-ft CMP. The proposed replacement is a 7-ft RCP, at least 2' lower than the current pipe, with internal weirs for hydraulic grade control. The weirs are compound trapezoidal in shape and 8' apart. The weirs are intended to widen the range of flows and tidal stage over which alewife may pass, with special attention to spring-time upstream passage. The weirs increase calculated upstream passage frequency from 22% to 37%, as compared to a maximum achievable of 40% - 47%.

Figure 1. Ripley Brook Watershed



Site Hydrology

Structure Size, Design Peak Flows and Bankfull Width

Basic watershed information and design peak flow regression estimates are shown in Figure 2. Due the small watershed size (0.94 mi^2; Figure 1) and relatively high NWI wetlands fraction (22%), the estimated peak flows are rather small. The proposed 7' RCP is more than adequate to handle flows of this magnitude. The concept of bankfull width BFW is not especially meaningful in a tidal situation, neither were there any good places to measure BFW in the vicinity of the project. That said, estimated BFW is 7.4', in accord with the proposed 7' RCP.

Figure 2. Estimated Design Peak Flows

Project Name:	St George	**PIN:**	19267
Stream Name:		**Town:**	St George
Bridge Name:		**Bridge No.**	
Route No.	ME 131	**USGS Quad:**	
Analysis by:	CSH	**Date:**	1/3/2013

Peak Flow Calculations by USGS Regression Equations **(Hodgkins, 1999)**

Enter data in blue cells only!

	km^2	mi^2	ac	*Enter data in [mi^2]*
A	2.43	0.94	600.5	Watershed Area
W	0.58	0.22	142.8	Wetlands area (by NWI)
P$_c$	482495	4869134		watershed centroid (E, N; UTM 19N; meters)
County	Knox			*choose county from drop-down menu*
pptA	46.1			mean annual precipitation (inches; by look-up)
SG	0.00			sand & gravel aquifer as decimal fraction of watershed A

Worksheet prepared by:
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A (km^2)	2.43	**Conf Lvl**	0.67
W (%)	23.79		

Ret Pd	**Peak Flow Estimate**			
T (yr)	**Lower**	**Q$_T$ (m^3/s)**	**Upper**	**Q$_T$ (ft^3/s)**
1.1		0.26		9.1
2	0.37	0.53	0.76	18.8
5	0.58	0.83	1.20	29.5
10	0.72	1.06	1.55	37.3
25	0.92	1.37	2.03	48.2
50	1.06	1.61	2.43	56.7
100	1.22	1.87	2.87	66.1
500	1.58	2.51	4.00	88.7

Reference:

Hodgkins, G., 1999.
Estimating the magnitude of peak flows for streams in Maine for selected recurrence intervals
Water-Resources Investigations Report 99-4008
US Geological Survey, Augusta, Maine

$Q_T = b \times A^a \times 10^{-wW}$

Fish Passage Hydrology – Riverine Flows and Passage Through Culvert

The critical period for passage is the spring-time upstream spawning run; delay on this end is undesirable. Juvenile out-migration in late summer and early fall is more flexible and corresponds to random but regularly occurring higher-flow events; timing is not an issue . Therefore, passage has been evaluated for the general up-migration period of April thru June. Figure 3 shows the regression estimates for monthly median flows. (April, 4.2 ft^3/s; May, 1.1 ft^3/s; June, 0.7 ft^3/s). Riverine (freshwater runoff) hydrology is important for assessing passage when tidal stage is below the culvert inlet.

Figure 3. Monthly Median Flow Estimates

Project Name:	St George (Tenants Hbr)	**PIN:**	19267
Stream Name:	Ripley Brook	**Town:**	St George
Bridge Name:	0	**Bridge No.**	
Route No.	ME 131	**USGS Quad:**	
Analysis by:	CSH	**Date:**	12/1/2008

DO NOT ENTER ANY DATA ON THIS PAGE; EVERYTHING IS CALCULATED

MAINE MONTHLY MEDIAN FLOWS BY USGS REGRESSION EQUATIONS (2004)

	Value	**Variable**	**Explanation**
	0.938	*A*	Area (mi^2)
482495	4869134	*P_c*	Watershed centroid (E,N; UTM; Zone 19; meters)
	26.56	*DIST*	Distance from Coastal reference line (mi)
	46.1	*pptA*	Mean Annual Precipitation (inches)
	0.00	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)

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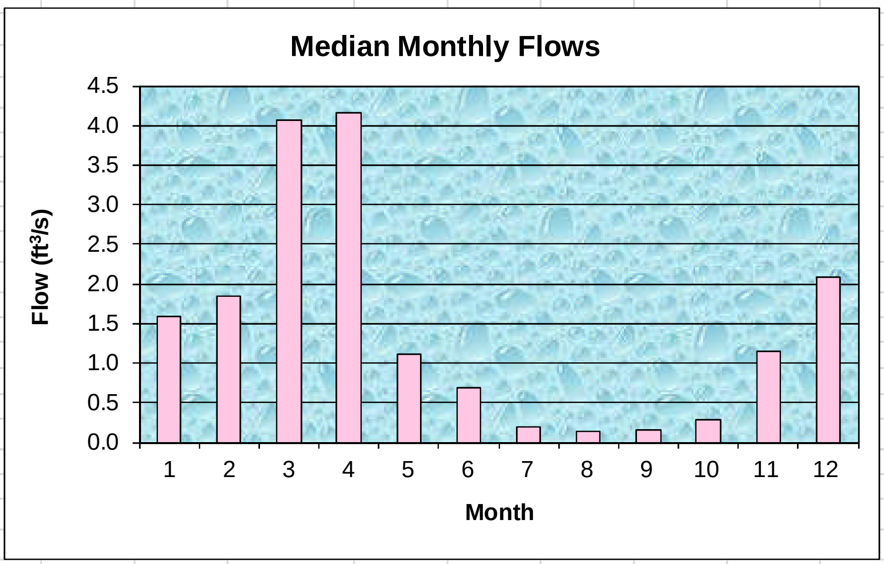
Month	**Q_{median}** (ft3/s)	(m^3/s)
Jan	**1.59**	0.0451
Feb	**1.85**	0.0524
Mar	**4.07**	0.1154
Apr	**4.16**	0.1178
May	**1.11**	0.0315
Jun	**0.69**	0.0194
Jul	**0.20**	0.0055
Aug	**0.14**	0.0040
Sep	**0.16**	0.0045
Oct	**0.29**	0.0081
Nov	**1.15**	0.0325
Dec	**2.09**	0.0593

Q_{bf}	**4.9**	
ann avg	**2.0**	
ann med	**1.0**	
$Q_{1.002}$	**4.0**	
$Q_{1.01}$	5.3	
$Q_{1.05}$	7.6	
W_{bf}	7.4	
d_{bf}	0.6	
Q_{bf}	16.9	assume v = 4ft/s

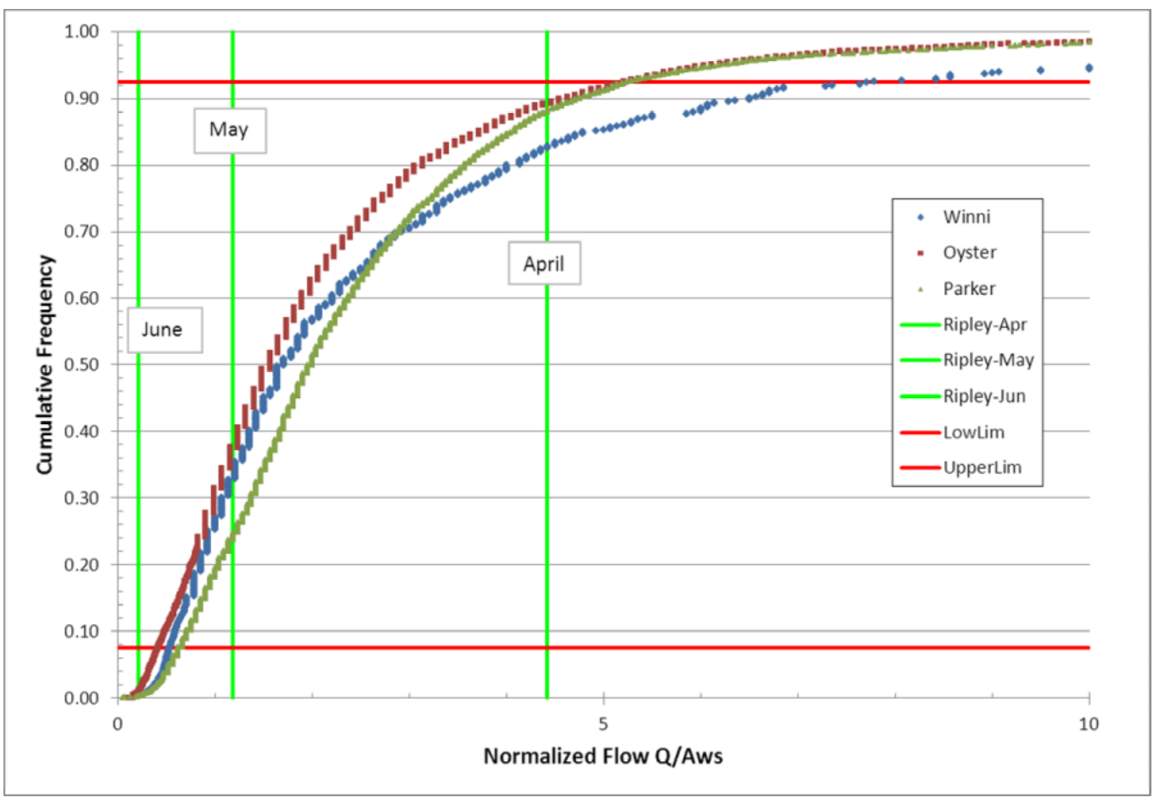


These estimates, while helpful, do not capture the frequency of flows that occurs during this period and this site (like the vast majority of small watersheds in Maine) is not gaged. In order to get a rough understanding of the range of flows, records were evaluated for three (3) small gaged coastal watersheds in northeastern Massachusetts and southern New Hampshire. The gages were

- USGS 01073785 WINNICUT RIVER AT GREENLAND, NH (A_{ws} = 14 mi^2; gaged since 2002)
- USGS 01073000 OYSTER RIVER NEAR DURHAM, NH (A_{ws} = 12.1 mi^2; gaged since 1935)
- USGS 01101000 PARKER RIVER AT BYFIELD, MA (A_{ws} = 21 mi^2; gaged since 1945)

The up-migration period was further refined to April 15 to June 14 as being representative for mid-coast Maine. Figure 4 shows the distribution curve for (4/15 – 6/14) daily average flows at the watersheds, with flows normalized by watershed area to make comparisons possible. The normalized Ripley Brook median flows are shown as vertical lines. The normalized flow frequency curves are generally consistent, though the Winnicut flows tend to be higher for a given frequency. Furthermore, the Ripley normalized medians are consistent with the chosen gage records, and so the frequency curves for the Winnicut, Oyster, and Parker Rivers were taken as reasonably representative of Ripley Brook.

Figure 4. Frequency distribution of daily average flows, 15 April – 14 June



Design hydrology for fish passage requires identification of flows for which passage can be demonstrated by analysis and calculation. It almost always involves choosing a range of flows for passage, realizing that passage is not possible (even in natural systems) at extreme highs or lows. In this project we have elected to determine the hydrologic limits based on daily flow frequency, as this gives some indication of the proportion of time over which passage (ignoring tides) can expected. (This frequency approach is an improvement over simply using monthly medians as limits; a further refinement would be to take a flow duration approach.) The goal is to provide passage over 85% of average daily flows; this corresponds to passage in 6 out of 7 days. This yields an upper frequency limit of 0.925 and a lower limit of 0.075; the corresponding normalized flows are 0.53 $ft^3/s/mi^2$ and 5.3 $ft^3/s/mi^2$. Scaled up by A_{ws} = 0.94 mi^2 for Ripley Brook, this yields the objective of providing passage at flows between 0.5 ft^3/s and 5 ft^3/s for 85% passage from April 15 thru June 14. These flow limits are combined with tidal analysis to get an overall passage efficiency.

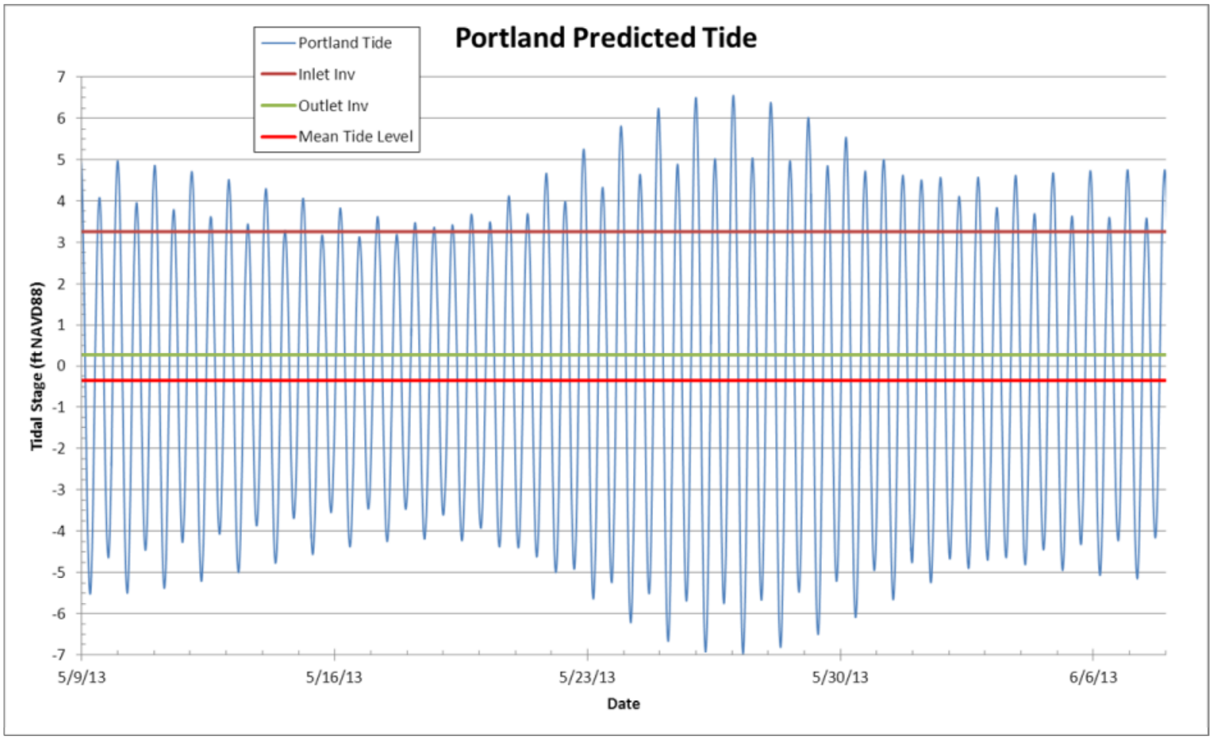
It is important to note the small magnitude of the range of flows under consideration. This is directly attributable to the small watershed size (0.94 mi^2). Fish passage at such low flows is problematic, even in natural systems. In fish passage design, flows less than 1 ft^3/s are rarely considered. In aiming for a specific passage efficiency, we will formally evaluate for Q = 0.5 ft^3/s but it is not a practical limit for passage.

<u>Fish Passage Hydrology – Tidal</u>

Tidal predictions (astronomical; not including storm effects) are available from NOAA for Tenants Harbor. However, this station is “subordinate” to the Portland gage (8418150). Thus, all tidal stage predictions for Tenants Harbor are with respect to Mean Lower Low Water (MLLW) and not an absolute datum like NAVD88, in which land surface and structural design elevations are reported. The Tenants Harbor predictions are not much different from Portland: highs and lows are 2% bigger and 11 minutes earlier. For design purposes, then, it has been assumed that Portland tidal elevations with respect to NAVD88 are representative of Tenants Harbor. Portland tidal datums are summarized in Table 1.

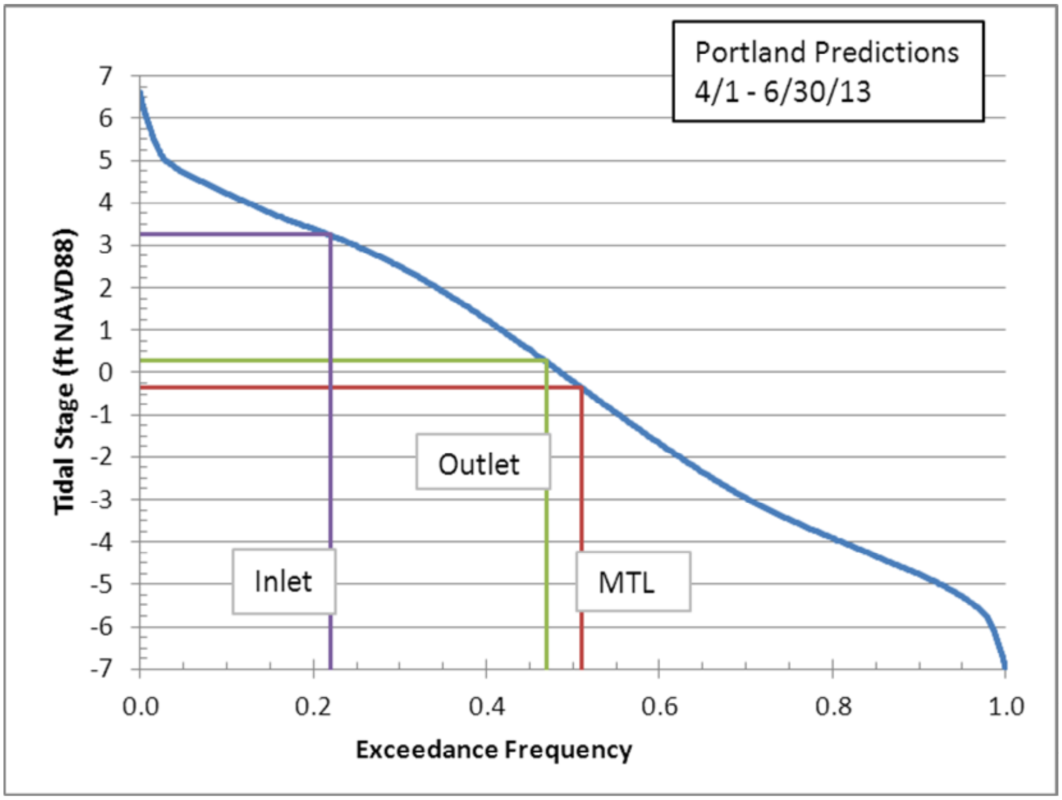
Tides follow a lunar monthly period, with the highest sequence of tides called the “spring tides” (those tides with the biggest differences between high and low values) and the lowest sequence the “neap tides” (with the smallest differences between high and low values). Figure 5 shows a 30-day period (9 May 9 – 8 June 2013) for predicted Portland tides. This period captures the largest and smallest tides during the 2013 up-migration period. The relation of tide elevations to culvert invert elevations will govern the effectiveness of alewife passage.

Figure 5. Portland predicted tides, May-Jun 2013.



The frequency distribution of tidal stage for this same period, expressed as exceedance frequency, is shown in Figure 6. Critical elevation datums (Mean Tide Level (MTL), Culvert Outlet, Culvert Inlet) are also indicated; see Table 2 for a summary of culvert data. MTL is exceeded 51% of the time. Up-migration is assumed to be possible only for tides at or above the outlet invert (0.27 ft) since the channel falls away sharply, presenting a natural barrier to passage. Furthermore, at low tides flow depth in the downstream tidal channel between the culvert and open water is exceedingly low, presenting another natural barrier.

Figure 6. Tidal Stage Frequency



Tides exceed the outlet elevation 47% of the time. This represents a theoretical maximum achievable passage frequency, assuming that the unknown natural channel under the culvert is passable over all runoff flow conditions. As noted above, though, it is likely that passage is unlikely over the lower range of flows. Even though these flows are small in magnitude, they are estimated to occupy a significant portion of the frequency curve. For example, approximately 30% of flows are less than 1 ft^3/s. Thus, while we will refer to 47% as a theoretical maximum efficiency in a natural channel system with the same inverts as the proposed culvert, we suspect that the true maximum that might be obtainable by natural or engineered means is something less than 47%. The inlet (upstream) elevation is exceeded 22% of the time. The tides are in the pipe 25% of the time (= 47% – 22%). Thus, providing passage through the pipe could significantly increase passage efficiency over relying only on tidal submergence of the inlet. Passage through the pipe is critical to providing alewife passage over a large range of tidal stage.

Table 1. Portland Tidal Datums (ft NAVD88)

Datum	Value	Description
MHHW	4.65	Mean Higher-High Water
MHW	4.21	Mean High Water
Culvert Inlet Elevation	*3.26*	
Culvert Outlet Elevation	*0.27*	
MTL	-0.35	Mean Tide Level
MSL	-0.32	Mean Sea Level
DTL	-0.3	Mean Diurnal Tide Level
MLW	-4.91	Mean Low Water
MLLW	-5.26	Mean Lower-Low Water
NAVD88	0	North American Vertical Datum of 1988
STND	-13.81	Station Datum
GT	-3.9	Great Diurnal Range
MN	-4.69	Mean Range of Tide
DHQ	-13.37	Mean Diurnal High Water Inequality
DLQ	-13.47	Mean Diurnal Low Water Inequality
HWI	-10.22	Greenwich High Water Interval (in hours)
LWI	-4.06	Greenwich Low Water Interval (in hours)
Maximum	8.87	Highest Observed Water Level
Max Date & Time	2/7/1978 10:30	Highest Observed Water Level Date and Time
Minimum	-8.71	Lowest Observed Water Level
Min Date & Time	11/30/1955 17:18	Lowest Observed Water Level Date and Time
HAT	6.69	Highest Astronomical Tide
HAT Date & Time	5/17/1999 4:42	HAT Date and Time
LAT	-7.38	Lowest Astronomical Tide
LAT Date & Time	1/21/1996 22:36	LAT Date and Time

Design for Up-Migration Fish Passage

The design challenge is to provide passage through the replacement 7'D concrete pipe. It can be assumed that passage is always possible whenever tides are at or above the inlet invert (3.26'), about 22% of the time. Tides are in the pipe another 25% of the time. Passage is not possible when tides are below the outlet invert. Invert elevations of the new pipe are significantly lower than existing. The outlet invert gives access to an additional 3' of tidal stage. By itself, this will greatly increase access to upstream. However, passage cannot be demonstrated in a simple smoothbore concrete pipe at the design slope when tides are below the (upstream) inlet and flow is presumed to run without backwater effects in at least some of the pipe. Additional measures are needed.

The proposed replacement is a 7'D RCP. Relevant structural design details are summarized in Table 2; geometrical relations for the 7'D pipe are shown in Figures 7 and 8. A 7' diameter was chosen primarily to improve passage capacity; the existing 5'D CMP is already adequate from the perspective of hydraulic capacity and the new pipe has nearly double the capacity by section area.

The new inverts have been lowered as much as possible so as to improve alewife access. Further lowering the inlet would present problems grading into the stream bed. The outlet invert elevation is limited by site conditions. The downstream channel drops off sharply from the outlet; this outlet channel segment is effectively too steep for passage when it is not flooded by the tide. Further lowering the outlet would make the pipe too steep. Also, it is doubtful that even an open bottom structure could significantly improve on the effective outlet elevation. Thus, the outlet elevation is essentially fixed by natural conditions. These natural conditions therefore impose a theoretical limit on achievable passage efficiency of 47%; passage is not possible when stage is below the outlet invert.

Table 2. Culvert design dimensions and details.

	Proposed	**Existing**
Material	concrete	corrugated metal
Shape	round	round
Diameter (inner)	7'	5'
Length	72'	62'
Upstream invert elevation (NAVD88)	3.26'	5.3'
Downstream invert elevation (NAVD88)	0.27'	3.5' (approx.)
Slope	0.042	0.029
Invert drop per 8' section	0.33' (4")	NA

Figure 7. Circular geometry – area as a function of depth (D = 7')

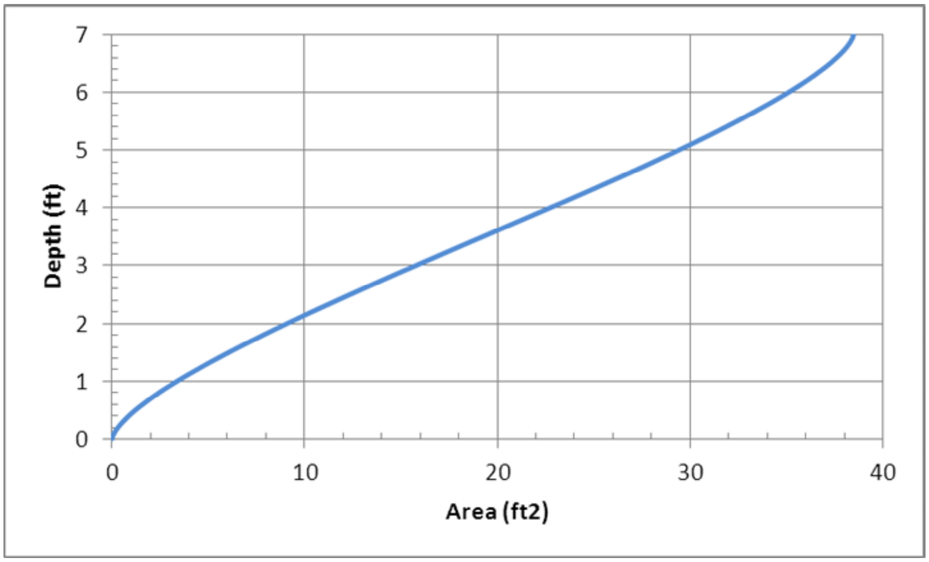
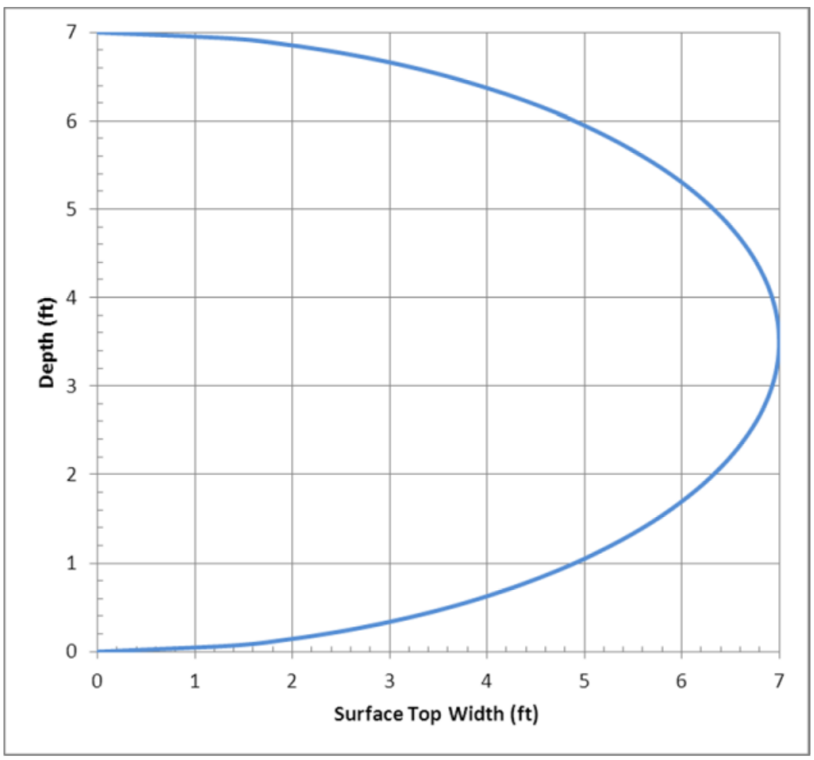


Figure 8. Circular geometry – surface top width as a function of depth (D = 7')



The combination of steep pipe (S = 0.042) and smoothbore material (concrete; n = 0.012) would suggest that alewife can only pass when tidal stage submerges the inlet invert; in other words, when the pipe is completely backwatered. Otherwise flow depths can be too shallow (< 4" = 0.33') and/or velocities too fast (> 5 ft/s). This is confirmed by calculations with the HY-8 culvert analysis software. Results for when the tide is 4" above the outlet invert are shown in Figures 9 – 10.

Figure 9 shows depth vs distance along the culvert. The depths at the lower flows are inadequate, as expected. Even at the upper flow limit (5 ft^3/s) depths are only marginally acceptable over the first 10 ft or so. It should also be noted that this calculated depth is along centerline; the depth decreases away from centerline. Thus, a concrete pipe cannot reliably produce acceptable flow depths across the target range of flows.

Figure 9. Calculated flow depth along pipe.

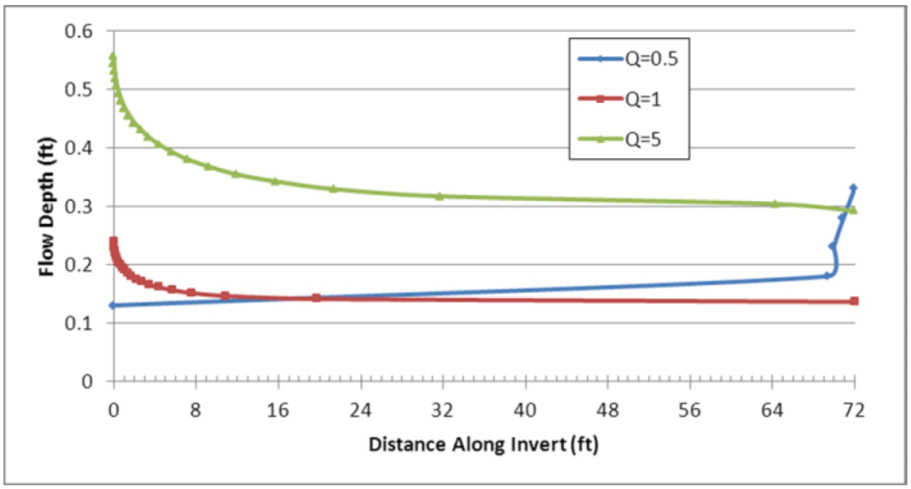
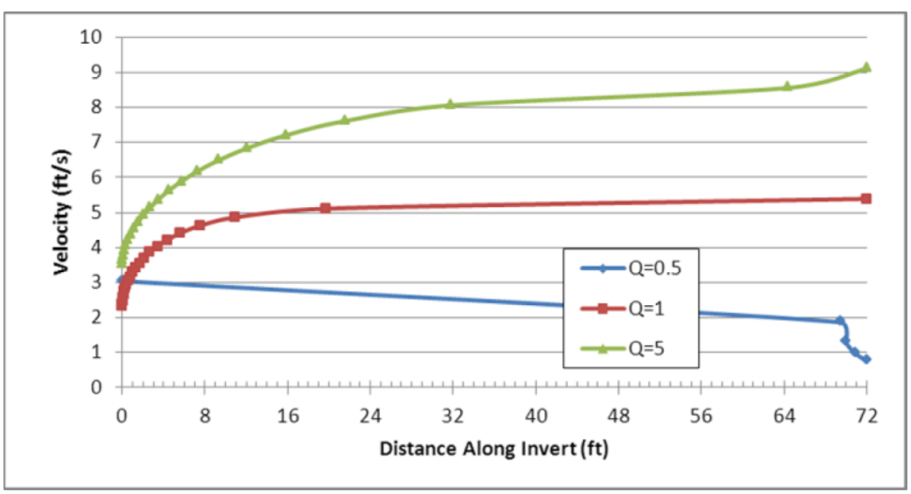


Figure 10 shows velocity as a function of distance along the pipe. Whereas flow depth appears to be marginally acceptable at the upper flow limit, the velocities are too high and pose a barrier to movement (> 7 ft/s for 75% of the length). At the lower flows, the velocities might be marginally acceptable (< 5 ft/s) but depths are too shallow.

Figure 10. Velocity along pipe.



Based on this analysis of a simple smoothbore concrete pipe, we conclude that passage can only be reliably achieved when tides are at or above the inlet (22%); hydraulic analysis indicates that passage is not likely when tides are in the pipe (25%) due to conditions depicted in Figures 9 and 10. Passage is physically impossible when tides drop below the outlet (53%) and therefore is not under consideration.

This is something of a worst-case example, with 4" tidal depth at the outlet. Some the worst effects of this configuration are mitigated when the tide is in the pipe. These mitigating effects are difficult to quantify and so in the interests of conservative design, we will conclude that, without weirs, passage will only occur when tidal stage is above the inlet invert (elevation 3.26'). The highest tides are on the order of 6.5', so this is a narrow range of tides (3.26' – 6.5') over which passage can be expected. In order to improve on this situation, weirs will be placed in the pipe.

A standard weir configuration will be evaluated for performance over the initial target range of 0.5 ft^3/s to 5 ft^3/s. The purpose of the weirs is to create flow depth ($\geq$ 4" between weirs) and velocity ($\leq$ 3-5 ft/s cruising speed, allowing for burst to 7 ft/s) conditions that alewife can navigate through the culvert. The weirs will be evaluated against a target passage efficiency of 85% of the time for when the tide is in the pipe (about 25%). Tides are above the inlet an additional 22%, during which passage is assumed. Thus, the initial target for passage is (0.85 x 0.25 + 0.22) = 43% efficiency. Weir dimensions can only be varied over a small range; spacing is essentially fixed to a minimum of one (1) weir per 8' pipe section.

The weir will have a full-depth notch (no sill), so that alewife will not have to negotiate an obstacle in the bottom of the notch. However, provision will be made for retrofit installation of low sills should the need arise. The sides of the notch are inclined instead of vertical. This has the effect of containing a somewhat wider range of flows in the notch before they go over the crest. Likewise, the weir crest is inclined for similar reasons, so as to maintain a wider range of passage flows and depths in the center of the pipe.

Figure 11 shows the weir dimensions in frontal view. The weir flow area is compound trapezoidal in shape. The purpose of the smaller centerline notch is to maintain adequate flow depths at low flows by hydraulic constriction alone with a full-depth notch (i.e. the notch invert is identically the pipe invert). A full-depth design (without sill) was chosen so that there would be no obstacles to swimming along the pipe invert. At the same time, the notch must be big enough to pass fish. The notch is 9" deep, with bottom width 6" and top width 12". The crest ties into the sides at 1.5' above centerline; the horizontal width between crest termini is 5.75'. The notch open area is similar to typical Denil fishway baffles. The crests are inclined at a fairly gentle slope; so that the flow is directed along the pipe center line, thereby avoiding shallow flow over a horizontal crest. This crest design also limits variation in flow depth over a range of passage flows while also maintaining flow depth over the crest, as reflected in the basic geometrical relationships Figure 12 (flow area) and Figure 13 (flow surface width).

Given the culvert slope (0.042), there is a drop of approximately 4" per every 8' pipe section. There will be one weir per 8' pipe section. Assuming an idealized level pool between weirs (unlikely to be achieved in practice), the pool increases in depth in the downstream direction between weirs. If the pool is at the minimum 4" (0.33') depth (0.67 ft^2 flow area) at the downstream face of a weir, then it is 8" (0.67) deep (1.87 ft^2 flow area) at the upstream face of the next weir. Thus, the ideal weir would produce 8" depth at the minimum design flow (0.5 ft^3/s). Such a weir would automatically produce acceptable flow depths for the upper limit (5 ft^3/s), after which it is just a matter of confirming acceptable velocities.

Figure 11: Weir Schematic – Front View (dimensions in [ft])

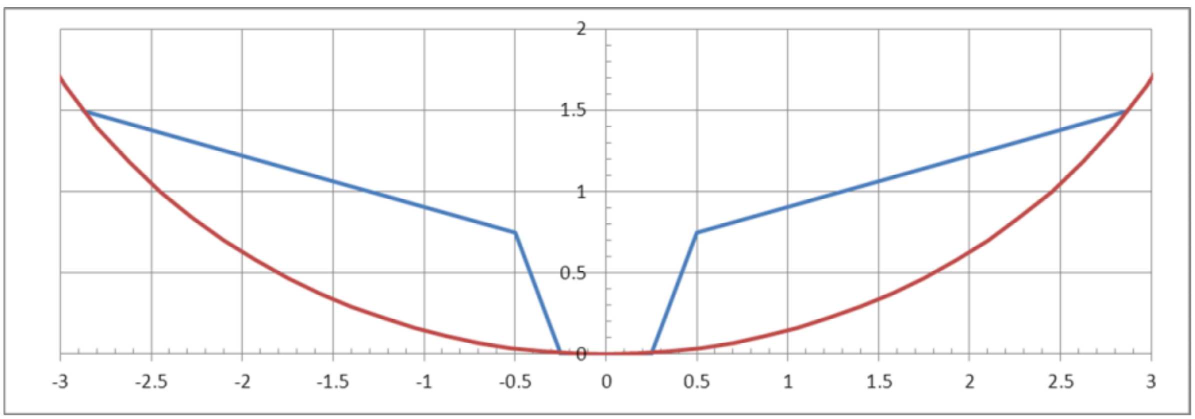


Figure 12. Weir Flow Width Function

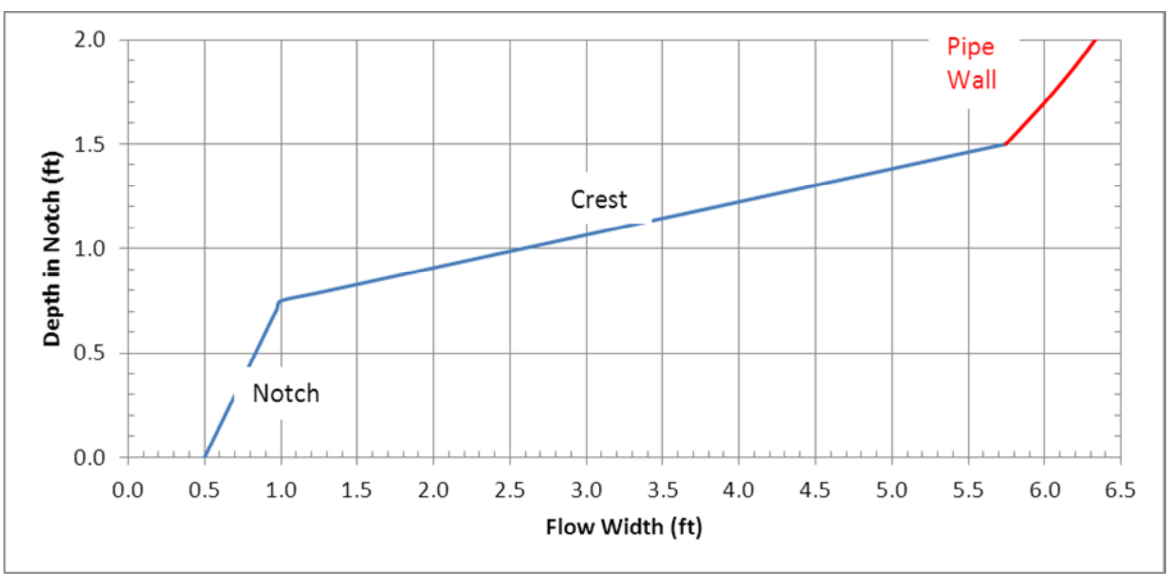
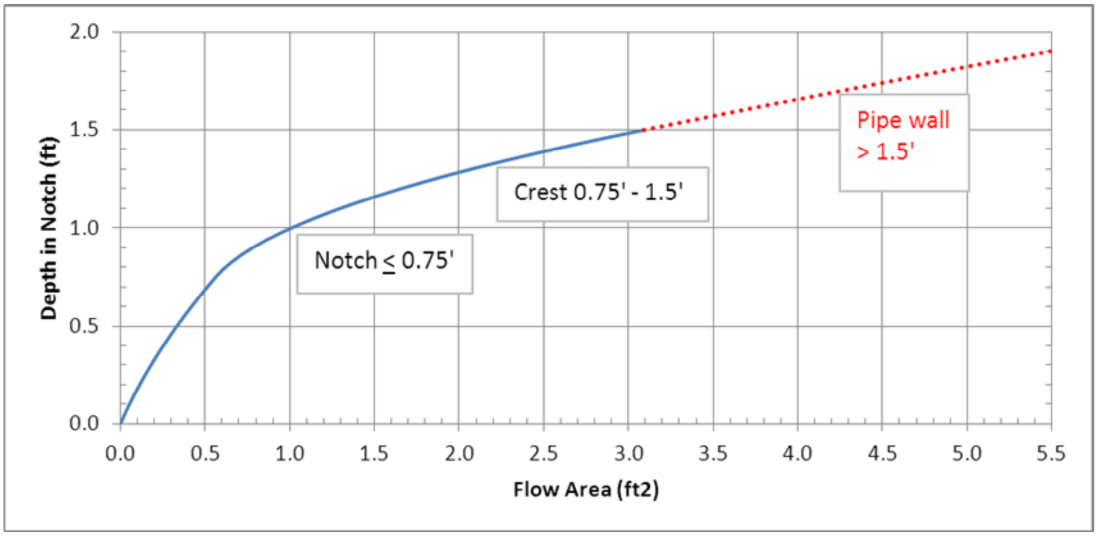


Figure 13. Weir Flow Area Function



The weir was modeled as a sharp-edged weir. The performance curve is shown in Figure 14, as well as additional calculated parameters for specific flows of interest in Table 3. A flow of 1.2 ft^3/s produces the required minimum 8” (0.67’) depth on the notch, thereby achieving the required minimum 4” (0.33’) flow depth at the downstream face of the adjacent upstream weir. This is somewhat higher than the initial lower limit target of 0.5 ft^3/s. As explained below, though, we do expect passable conditions at some flows less than this nominal calculated limit of 1.2 ft^3/s. The initial upper limit target flow of 5 ft^3/s produces a depth of 1.25’ (15”) on the pipe invert just upstream of the weir; hence a flow depth of 0.92’ (11”) at the upstream weir. In fact, we can expect acceptable hydraulic passage conditions (depth and velocity) at flows as high as 20 ft^3/s (cumulative frequency = 99.7%). Simplified pool-and-weir water surface profiles for 1.2 ft^3/s and 5 ft^3/s are shown in Figure 15. Note that the notch is flush to the pipe invert so there will not be sharp drops between pools, as suggested in the figure.

Figure 14. Weir Performance Curve (extrapolated beyond depth = 1.5 ft)

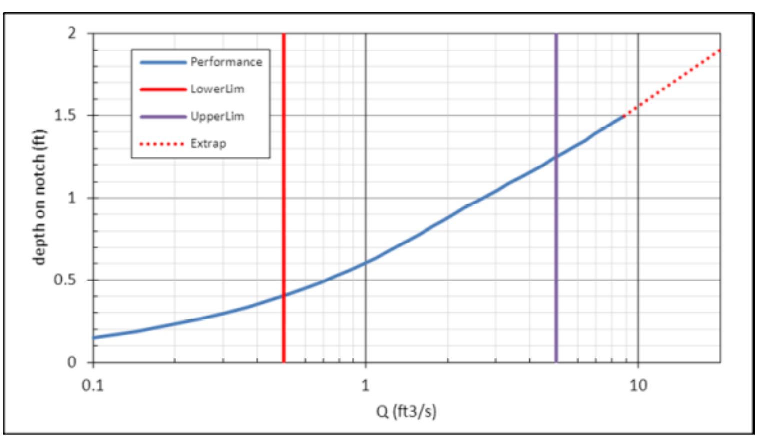
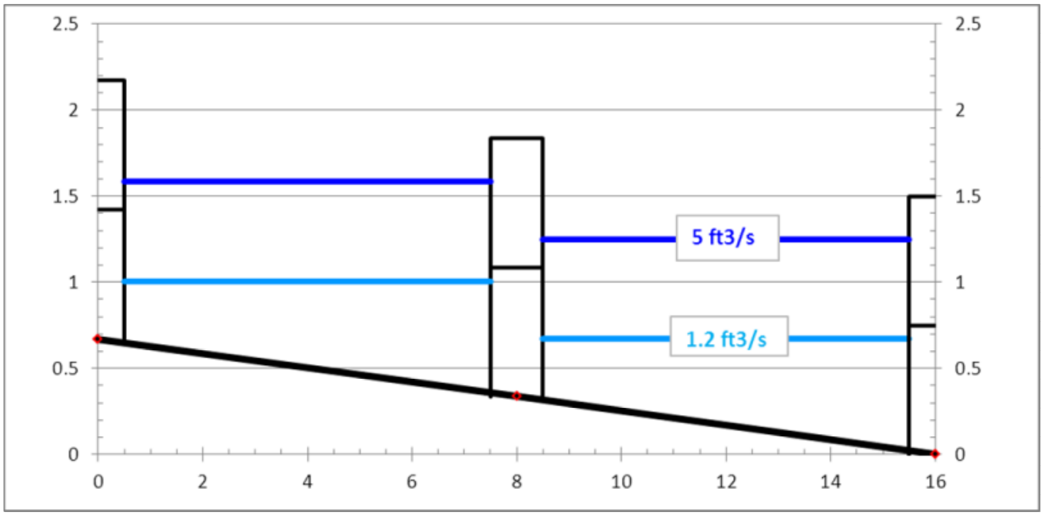


Figure 15. Simplified water surface profiles between weirs



The calculated effective lower limit of 1.2 ft^3/s is conservative for several reasons and we actually expect some passage at lower flows. Each weir will create at least some submergence of the weir invert upstream, with greater submergence at higher flows. This has the effect of raising the actual depths above the simple calculated values. This effect can be estimated by the method of Villemont (1947). So the actual flow depths will likely be somewhat greater than calculated, all other things being equal. Furthermore, passage is only possible while the tidal stage is at or above the outlet invert (i.e., when the tide is in or above pipe). This creates an additional backwater effect, also not captured. This tidal backwater effect is further enhanced on incoming tides, since the tidal inflow is against the riverine outflow. None of these effects is captured explicitly in the design analysis, and so the design is conservative.

Table 3 summarizes the calculated performance at the lower and upper flow limits. Relying only on tidal flooding of the inlet, passage is possible about 22% of the time at best. By hydrologic analysis of freshwater flows only, we aimed for a passage rate of 85%. This would give an overall passage efficiency of (0.22 + 0.85 x 0.25) = 0.43 = 43% as the calculated design goal. By actual calculated performance, we have achieved an efficiency of {0.22 + (0.99-0.38)x0.25} = 37%. Put another way, 37% of the time there will be at least 8" depth in each notch, at least 4" minimum depth elsewhere, and velocity $\leq$ 5ft/s throughout the pipe. This compares to the theoretical maximum achievable efficiency of 47%, the frequency with which the tide reaches the outlet. As noted, the "true" achievable maximum is likely something less than 47%. While passage was not realized over the range (0.5 ft^3/s – 1.2 ft^3/s; 31% "lost" efficiency), additional passage was picked up beyond the original upper target of 5 ft3/s (7% "gained").

Table 3: Weir Performance Summary at Target Flow Limits (units of [ft] and [sec])

	Target Lower Limit 0.5 ft^3/s	Calculated Lower Limit 1.2 ft^3/s	Target Upper Limit 5 ft^3/s	Calculated Upper Limit 20 ft^3/s
Cumulative Flow Frequency (%)	7.5	38	92.5	99.7
d_w - Depth at weir (from performance curve)	0.41 (5”)	0.67 (8”)	1.25 (15”)	1.9 (23”)
A_w - Flow area in weir (Fig. 9)	0.26	0.43	1.90	5.48
W_w - Surface width on weir (Fig. 10)	0.77 (9.2”)	.95 (11”)	4.15 (50”)	6.23 (75”)
V_w - Nominal velocity thru weir = Q/A_w	2.0	2.5	2.7	3.7
A_{Sw} - Flow area in pipe section at weir (Fig. 6)	0.91	1.87	4.66	8.44
W_{sw} – Surface width in pipe section at weir (Fig. 7)	3.29	4.20	5.36	6.23
V_s - Velocity in pipe section at weir = Q/A_{sw}	0.6	0.6	1.1	2.4
d_u - Depth at upstream weir = (d_w – 0.33’)	0.08 (1”)	0.33 (4”)	0.92 (11”)	1.57 (19”)
A_u - Flow area in pipe at upstream weir (Fig. 6)	0.08	0.67	2.99	6.45
W_u - Surface width in pipe at upstream weir (Fig. 7)	1.50	4.20	4.73	5.84
V_u - Velocity in pipe section at upstream weir = Q/A_u	6.3	1.8	1.7	3.1

References

Villemont, J.W., 1947. “Submerged weir discharge studies”, Engineering News Record, pp. 54-57, 25 Dec 1947.

Appendix A

Alewife Fact Sheet

Maine Department of Marine Resources

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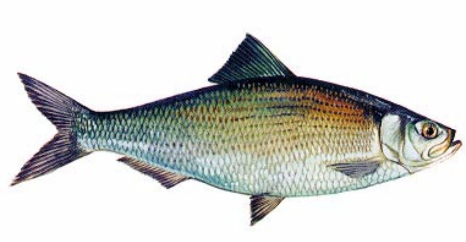
ALL ABOUT MAINE ALEWIVES...

What are alewives?



Alewives are anadromous (sea-run) fish that spend the majority of their life at sea but return to freshwater to spawn. Alewives have co-evolved and co-existed with other native fish and wildlife in Maine's streams, rivers, ponds and lakes for thousands of years. Alewives are members of the herring family; their close cousins are shad and blueback herring. Alewives have slender bodies, and they normally grow to 10 – 11" in length, and weigh about half a pound. Repeat spawners can be as large as 14" and weigh a pound or more. Alewives are grayish green on their back, and silvery on their sides and belly. They've got a single black spot just behind their eye, and their tails are forked.

Were alewives originally present in our lakes?

The bad news is that many Mainers have never seen an alewife run because Maine's historically thriving alewife population has plummeted during the last two centuries. Dams, pollution and overfishing have taken their toll. Southern Maine's Alewife Brook, for example, no longer has alewives.

But historians and scientists tell us that prior to Europeans settling this region, there was probably not a stream anywhere in the Gulf of Maine that didn't have an annual alewife migration, unless it was blocked by impassable waterfalls. One early historian said, "There can have been hardly an accessible pond in the whole State they did not visit." Of all the migratory fish that came up Maine's rivers, alewives were the most abundant. One history of Gardiner and Pittston, written in 1852, relates that "alewives were so plentiful there at the time the country was settled, that bears, and later swine, fed on them in the water. They were crowded ashore by the thousands."

Native Americans and European settlers depended on the bounty brought to inland waters by spring migrations. When one river town built a dam and blocked the fish from their spawning habitat, one early chronicler wrote that the inhabitants of the next town were outraged. "It was difficult to persuade the aggrieved people to forbear using violence to open a passage for ye fish… the cry of the poor every year for want of the fish…is enough to move the bowels of compassion in any man that hath not an heart of stone." In 1809, the selectmen in Benton ordered a mill dam to be torn down because it blocked huge runs of alewives and shad on the Sebasticook River.

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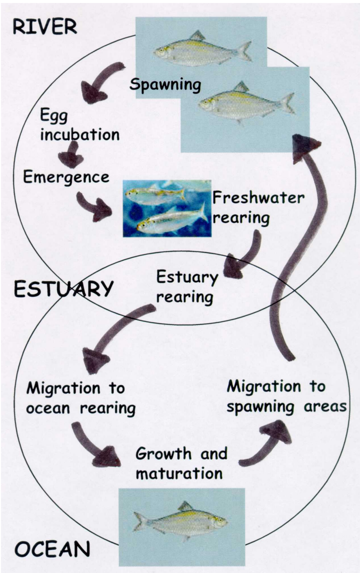
Doug Watts (migrating alewives)
Ethan Nedeau (Damariscotta Lake)
All other photos:
U.S. Fish and Wildlfe Service
NOAA -- Fisheries

December, 2003

What is the alewife's life cycle?



Every May and June, adult alewives, guided by their sense of smell, migrate upstream from the ocean to rivers, streams, ponds and lakes to spawn. Spawning occurs in ponds and lakes or the quiet backwaters of rivers and streams. Some males return to freshwater when they are three years old. Females usually return when they are four or five years old. One female alewife can produce somewhere between 60,000 to 100,000 eggs, but only a few eggs survive to the juvenile stage, and sometimes only as few as three juveniles survive to adulthood. Although some adults die after spawning, the majority of adults make their way back to the ocean shortly after spawning – and many return the following spring to spawn again. During their downstream migration, adult alewives resume feeding, primarily on zooplankton. Once hatched, juvenile alewives remain in freshwater lakes and ponds where they also feed on zooplankton. Juvenile alewives grow anywhere between one to six inches, depending on the productivity of the lake. From mid-July through October, juveniles migrate downstream to the ocean where they grow to adulthood.

zooplankton (magnified 30-35X)

Do alewives affect water quality?

Maine Dept. of Environmental Protection (DEP) studies in more than a dozen Maine lakes with natural or reintroduced runs of alewives have not shown water quality decline that can be attributed to alewives, according to Barry Mower, a fisheries biologist and water quality specialist. It is well-substantiated that the major factor causing algae blooms in our lakes is the introduction of phosphorus. There are many sources of phosphorus in our lakes -- and most are directly linked to residential development.

When adult alewives migrate into a freshwater pond or lake, there is an influx of phosphorus to the lake. However, the majority of the spawning alewives return to the ocean, taking phosphorus with them. Additionally, young alewives that grow in freshwater ponds and lakes incorporate phosphorus from lakes into their bodies. That phosphorus is removed when the young migrate to the ocean.

Damariscotta Lake

Studies coordinated by Maine DEP in the 1970s on Little Pond in Damariscotta and studies coordinated by Maine DEP, Maine Dept. of Marine Resources (DMR) and Maine Dept. of Inland Fisheries and Wildlife (DIFW) on Lake George in Canaan in the 1990s, supplemented by additional water quality studies in half a dozen other Maine lakes and ponds with restored alewives demonstrate that when alewives are restored, there is no impact or a minor net

decrease in total lake phosphorus. In fact, data from Maine points to good water quality on lakes with healthy alewife populations. To name only a few, those lakes include Nequasset Lake in Woolwich, Damariscotta Lake in Nobleboro and Jefferson, Alamoosook Lake in East Orland, and Gardiner Lake in East Machias. A little further afield in southeastern Massachusetts, the Assawompsett Ponds host the largest alewife population in New England (two million adult alewives this past spring). Most of the ponds in this complex have served as public water supplies since about 1900, and water quality and quantity in the ponds is outstanding, even though the ponds are generally very shallow. And, it must be added, the area surrounding these ponds is undeveloped.

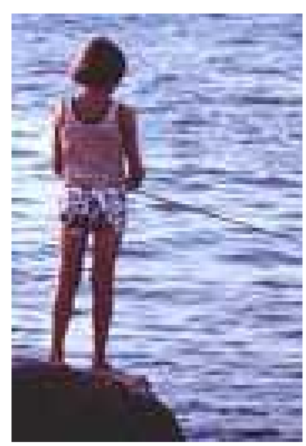
Are alewives important for recreational or commercial fishing?

Alewives are an integral part of marine and freshwater food chains. Both adult and juvenile alewives are small and are therefore eaten by many other species of native, introduced, commercially and recreationally important fish. In freshwater, alewives are food for large- and smallmouth bass, brown trout and other salmonids. In the estuaries and the ocean, striped bass, cod and haddock feed on alewives, and the recovery of these economically valuable fish depends in part, on restored populations of alewives. In addition, lobstermen depend on alewives; they are the traditional spring bait for lobsters.

smallmouth bass

Atlantic cod

The ten-year study conducted by Maine Dept. of Marine Resources, Maine Dept. of Inland Fisheries and Wildlife and Maine Dept. of Environmental Protection on Lake George in Canaan showed that alewife stocking had no detrimental effects on freshwater fish such as smallmouth bass, brown trout, chain pickerel and white perch in terms of size or abundance. Young-of-the year smelt actually grew better in the presence of alewives! Moreover, many other lakes in Maine, such as Sabattus Pond and Damariscotta Lake have thriving alewives that co-exist with healthy populations of other fish. The Assawompsett Pond complex in southeastern Massachusetts, which hosts the largest alewife population in New England (two million adult alewives this past spring), offers great fishing. According to local fisherman, the ponds support exceptionally robust populations of largemouth and smallmouth bass, crappie, white perch, yellow perch, walleye, pickerel, pike, catfish, suckers, and a variety of baitfish.



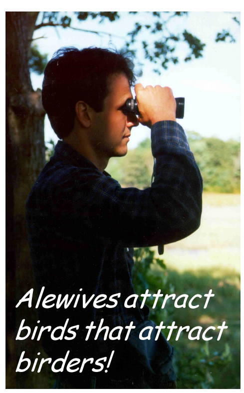
How do alewives benefit lakes, rivers, and the ocean?

While alewives present a spectacular migration every spring that's lovely for people to watch, alewives perform other vital functions in the larger ecosystem. For example, in the spring, when alewives move up our rivers, that's *precisely* the same time juvenile salmon smolts are moving downriver. If you were a sharp-eyed osprey in a riverside tree, what would you go for? One of the zillions of alewives you see down there, or the few salmon smolt hidden by alewives? Alewives provide cover for those salmon. In the same way, healthy populations of alewives also provide cover for upstream migrating adult salmon that could be preyed on by eagles or osprey, and for young salmon in the estuaries and open ocean that might be captured by seals.


Atlantic salmon

The important message is that alewives tie our ocean, rivers and lakes together, providing vital nutrients and forage needed to make healthy watersheds. Imagine huge schools of alewives that swim in the Gulf of Maine, as far as 120 miles out. Then the adults move, in huge waves, back inshore and up into freshwater.

Once they have spawned, adults migrate back downstream, followed later in the summer and fall by the juveniles. Between and within those various habitats, *everything* eats alewives: striped bass, bluefish, weakfish, tuna, cod, haddock, halibut, American eel, rainbow trout, brown trout, lake trout, landlocked salmon, smallmouth bass, largemouth bass, pickerel, pike, white and yellow perch, seabirds, bald eagle, osprey, great blue heron, gulls, terns, cormorants, seals, whales, otter, mink, fox, raccoon, skunk, weasel, fisher, and turtles.


Alewives attract birds that attract birders!

Alewives have been central to the web of life in Maine for millenia. If we give alewives a chance by helping restore them to their ancestral spawning grounds, alewives will once again play an important role in bringing our rivers, lakes, estuaries and oceans back to life. In return, we will be treated to exuberance and bounty in Maine's watersheds, in a way that none of us have fully experienced in our lifetimes.