APPENDIX B: HYDRAULIC AND HYDROLOGIC CONSIDERATIONS

Smelt Brook Renovations

Smelt Brook drains approximately 2.1 square miles (5.4 square kilometers) into the Fore River at Weymouth Landing in Weymouth, Massachusetts. The Smelt Brook drainage basin includes Pond Meadow Dam, which discharges into the Brook, which then flows approximately 0. 5 miles to Stetson Street. Stetson Street passes over the Smelt Brook. Downstream of the Stetson Street crossing, the river channel was redirected in the early 1970’s to flow essentially parallel to Brookside Road for approximately 0.25 mile, part of which is enclosed in an arched-section approximately 8-ft diameter bitumen-coated corrugated metal pipe (CMP). This CMP discharges at invert elevation 22.5 feet NGVD to a stilling basin at elevation 18 ft NGVD.

The elevation change from the CMP at elevation 22.5 ft NGVD to the stilling basin floor at 18 ft NGVD is an obstacle to the upward migration of fish.

The brook continues a further 650 feet before entering two culverts at a gated section, one gated and one permanently open. Normal flows pass through the gated culvert; storm flows that overwhelm the gated entrance rate pass into the ungated culvert. Over the next approximately 1,000 feet, the two culverts convey water northward, under Commercial Street and through the Weymouth Landing commercial center (ground level approximately 15 to 20 ft NGVD); and there are two separate outfall points for these culverts, discharging to the Fore River (water levels tidally influenced around 0 ft NGVD). One reach of the “lower” (normal-flow) culvert has recently been daylighted during renovations in 2019-2022 in the Weymouth Landing region.

The obstacle to migrating fish from elevation 22.5 ft NGVD to 18 ft NGVD is the focus of this report.

Figure 1 shows the basic dimensions of the existing system, based on drawings from the 1974 construction. Recent site visits have shown the baffles to be hidden under sediment. The simplified designs in this report (Alternatives 1 and 2) are based on a stilling basin 31 feet long and 14 feet wide. (Actual dimensions are slightly bigger at 33x15). Where pool boundaries are assumed to be vertical walls 1-foot-thick, it may be possible to fill the Alternative 1 and 2 pools themselves with rocks selected so as to keep the water surface close to an artificial pool-bed invert level, as might be better suited to fish that prefer to be in, and lay eggs in, shallower waters. This may be subject to refinement as part of an adaptive management process.

The objective is to make it possible for smelt to swim through a series of ponds which begin in the stream at elevation 18 ft NGVD and end up six inches above the elevation 22.5 ft NGVD invert of the outlet of the CMP. The stilling basin, which is part of a flood risk management Local Protection Plan (LPP) is close to the nearest property and so there is a concern to keep a 100-year (600 cfs) flow through the system from rising above 24 ft NGVD at the site of the stilling basin. Currently, this is assured by the stilling basin, which dissipates energy from the outfall before passing water downstream at 18 ft NGVD with a 3-to-4-ft depth.

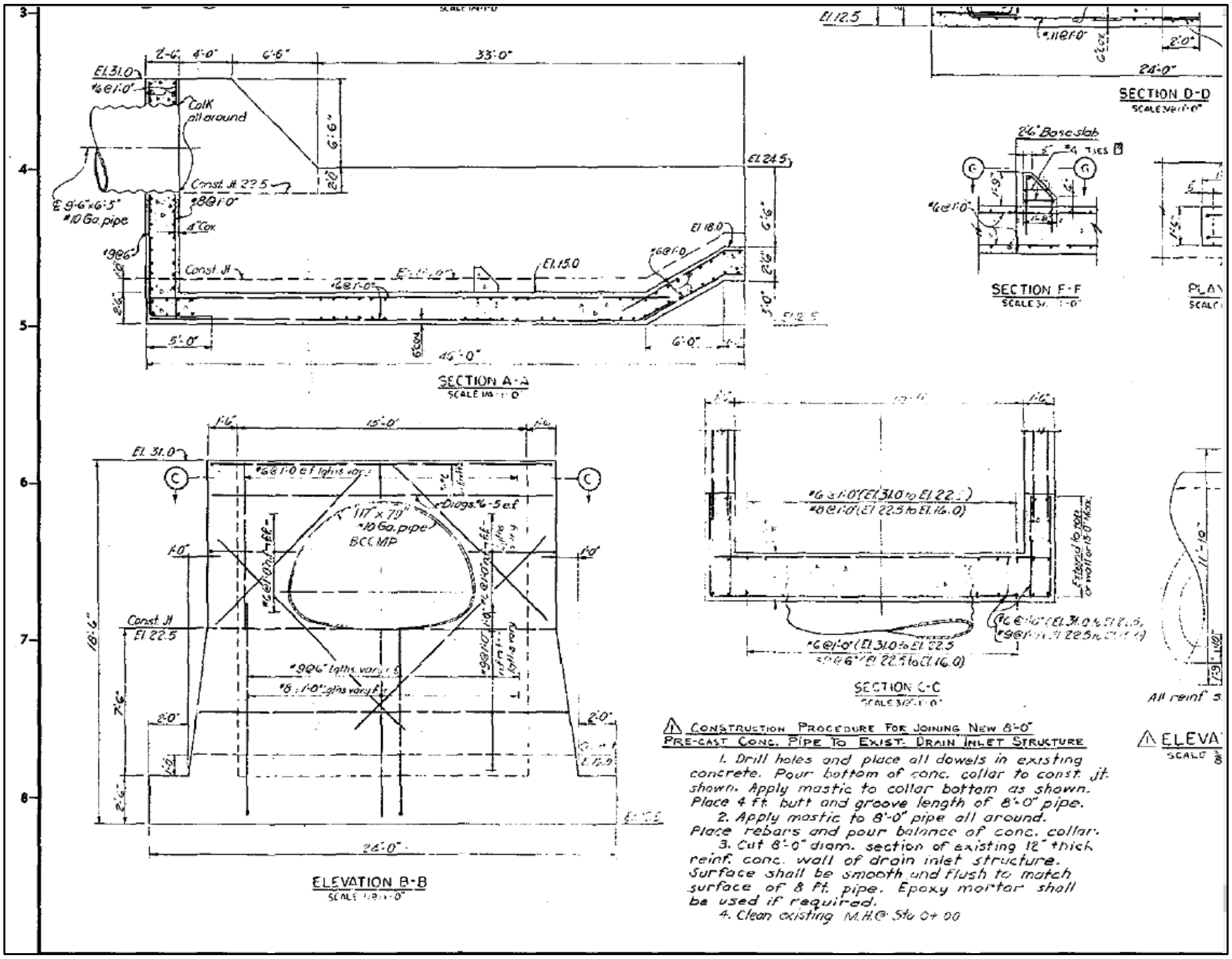

Figure 1: Existing Stilling Basin Dimensions (No Action alternative)

The objective is to make it possible for smelt to swim through a series of ponds which begin in the stream at elevation 18 ft NGVD and end up six inches above the elevation 22.5 ft NGVD invert of the outlet of the CMP.

Weirs with notches feature in two of the layouts that have been reviewed. The notches have been designed to permit a range of velocities during migration flows of 1.8 to 3.2 feet per second (fps), for upstream depths of 2 to 6 inches.

Design was based on 2.4 square miles of basin contributing to the watershed. The area at the site of interest is 1.85 square miles according to USGS (StreamStats), and the design discharge for a 100-year design event at the upstream Pond Meadow Brook Dam was estimated at 320 cfs, but the flow along Brookside Road, only 650 feet downstream of the CMP, was stipulated in the O&M manual to be 600 cfs.

The larger estimate for a **100-year flow of 600 cfs** has been taken as a requirement for the system.

It is noted that StreamStats estimated a 2-year flow of 54 cfs. Median flow estimated by StreamStats was 1.79 cfs. This indicates that the spring flow is between these values. A value of 20 cfs has been assumed in this report for a typical spring flow.

Six alternatives have been reviewed:

1. Ladder on Entire Stilling Basin;
2. Ladder on One Side of the Stilling Basin;
3. Combination (nature-like bypass with weirs at designed intervals);
4. Naturelike Bypass with a “Switchback”;
5. Engineered Weirs at Intervals Along the 600-ft length of the River Reach;
6. Keyhole Slot at Base of Existing Culvert Exit.

Of these, Alternatives 1, 2, and 6 all keep the new civil works within the existing project footprint. **Alternative 2** appears to be the most feasible in terms of improving fish migration while also avoiding flood damages during the design flood of 600 cfs. The options are examined in the sections that follow.

Alternative 1: Ladder on Entire Stilling Basin

There is adequate area in the existing stilling basin for 9 pools, each 6 inches higher than its downstream neighbor, with adequate volume for the pools. The analysis assumes 1-ft thick vertical walls.

Design was based on 2.4 square miles of basin contributing to the watershed. The area at the site of interest is 1.85 square miles according to USGS (StreamStats), and the design discharge for a 100-year design event at the upstream Pond Meadow Brook Dam was estimated at 320 cfs, but the flow along Brookside Road only 650 feet downstream of the CMP was stipulated in the O&M manual to be 600 cfs.

Water falls from the CMP culvert into Pool 1, which is 7 ft wide and the entire width of the channel (taken as 14 ft wide). This pool falls into Pool 2, which is 8 ft x 7 ft, covering one-half of the width of the current stilling basin. The pool falls into Pool 3, which lies on the other side of the basin. The layout of pools is demonstrated in Table 1.

Table 1: Layout of Pools (entire stilling basin)

Culvert Exit. Invert level is 22.5 ft NGVD.	
Pool 1: 5 feet wide, 14 feet long. Invert elevation of weir to next pool: 22.5 ft NGVD (no drop yet). Depth: 7.5 feet. Minimum pool depth of 1.5 feet requires that the invert of the pool be no higher than 21.0 ft NGVD (i.e., do not fill the pool with rocks etc. to elevations above 21.0 ft NGVD).	
Pool 2. 8 ft wide, 7 ft long. Elevation of weir to next pool: 22.0 ft NGVD. EDF = 0.80 Depth: 7.0 ft >1.5 => OK.	**Pool 3**. 8 ft wide, 7 ft long. Elevation of weir to next pool: 21.5 ft NGVD. EDF = 0.91 Depth: 6.5 ft >1.5 => OK.
Pool 5. 8 ft wide, 7 ft long. Elevation of weir to next pool: 20.5 ft NGVD. EDF = 1.14 Depth: 5.5 ft >1.5 => OK.	**Pool 4**. 8 ft wide, 7 ft long. Elevation of weir to next pool: 21.0 ft NGVD. EDF = 1.02 Depth: 6.0 ft >1.5 => OK.
Pool 6. 8 ft wide, 7 ft long. Elevation of weir to next pool: 20.0 ft NGVD. EDF = 1.25 Depth: 5.0 ft >1.5 => OK.	**Pool 7**. 8 ft wide, 7 ft long. Elevation of weir to next pool: 19.5 ft NGVD. EDF = 1.37 Depth: 4.5 ft >1.5 => OK.
Pool 9. 8 ft wide, 7 ft long. Elevation of weir to next pool: 18.5 ft NGVD. EDF = 1.59 Depth: 3.5 ft >1.5 => OK.	**Pool 8**. 8 ft wide, 7 ft long. Elevation of weir to next pool: 19.0 ft NGVD. EDF = 1.48 Depth: 4.0 ft >1.5 => OK.
18 ft NGVD target exit pool WSEL	

Weir dimensions

Weir width should be set at three levels: the minimum values noted in the table, 2 inches higher, and 4 inches higher (for example: 20 ft NGVD, 20.167 ft NGVD, and 20.333 ft NGVD). Assuming the widths at each level are 3 feet (lowest) 2 feet (middle level), and 1 foot (highest weir level), and if the wall is at height 20.333 + 0.5 = 20.833 ft NGVD, then before the wall is overtopped the combination can pass:

$Q = 2.5 \times [(3*(0.833)^{1.5} + 2 (0.667)^{1.5} + 2 (0.5)^{1.5}]$

$= 2.5 \times [2.282 + 1.089 + 0.707] = 10.2$ cfs. For greater flows, the system of pools passes the water, but the flows are too strong for the migrating smelt.

If the headloss from one pool to the next is 6 inches then the velocity at the deepest part of a weir notch is 3.28 ft per second (fps) with smaller velocities over the shallower notch sections (2.67 fps over a 4-inch-deep section; 1.89 fps over a 2-inch-deep section.

Alternatively, 11 pools would be 5 inches apart vertically, and this could be accommodated on the existing stilling basin. There would be more pools, with smaller volumes in each.

One drawback with this scheme is that the pools are **smaller than 10 feet long by 5 feet wide**, which is a recommended minimum spacing for a schooling species that prefers at least 10 feet between obstacles and requires frequent rest areas. If chosen for further development, then the geometrical changes outlined below might be considered instead. It should be noted in mitigation that the pools are 3 to 7 feet deep, so that there is significant room for energy to be dissipated within each pool.

The Energy Dissipation Factor (EDF) is a measure of head loss in a given pool. A small loss in a large pool leads to a small EDF, while a larger head loss in a smaller pool leads to a larger EDF. The EDF is a value that has been compiled for many species. Stronger swimming species are more capable of swimming through systems with large EDF values; weaker swimmers require smaller EDF values.

The formula used is:

EDF = γ Q D / V where:

- γ = 62.4 lb/cubic foot of water
- Q = flow in cubic feet per second
- D = head drop across the pool in feet
- V = pool volume in cubic feet.

For this option, Q = 10 cfs; D = 0.5 ft drop per pool; V = pool volume.

With a full 10 cfs of flow, the EDF ranges from 0.8 to 1.5 ft-lb/s/ft^3.

The Energy Dissipation Factor is estimated for this case, based on a flow of 10 cfs in the system of pools. Greater flows would prove overwhelming for migrating smelt, and the fish would need to wait in the ponds, or further downstream, until the storm had subsided).

The same basic fish ladder could be extended downstream of the stilling basin, into the wider valley to allow for wider pools. A survey in March 2020 indicated that, following the end of the stilling basin, the next surveyed stream invert (50 feet beyond the end of the basin) was at elevation 17.86 ft NGVD, while the water depth at that location was approximately 1.5 feet during the March 2020 survey inspection. A series of pools 10 feet wide would extend the system 13 feet (plus wall thicknesses) beyond the stilling basin, but would require one further pool for a comfortable transition from the stream at the lowest end of the structure to the start of the ladder. The extra single pool is accommodated at the location of pools 2 and 3. This promotes a calming of the March flows at the upstream end of the structure, and would lead to a symmetrical downstream "end" with two pools straddling the streambed.

High-flow ("100-year storm") behavior

The initial (farthest upstream) pool has walls at elevation at least 20.5+0.833 = 21.33 feet (10 inches higher than the lowest weir notch elevation).

Treating this as a weir with Manning flows along the unaffected banks then the design storm passes approximately 170 cfs over the weir and 120 cfs over the banks for a total flow of only 290 cfs before the water level reaches 24 ft NGVD in the downstream channel.

Alternative 2: Ladder on One Side of the Stilling Basin

An alternative layout with pools on one side only of the stilling basin could allow for excessive streamflows to bypass the system, leaving a more constant flow in the ladder pools. The floor of the current basin is opened up for fish that are not attracted to the fish ladder structure. The pool volumes are smaller; the range of weir sizes (two levels and overflow) limits the effective flow to a maximum of:

$Q = 2.5 \times [(1*(0.833)^{1.5} + 1\ (0.667)^{1.5}]$

$= 2.5 \times [0.761 + 0.544] = 3.3$ cfs.

This choice keeps the 5-foot minimum separation between successive weirs in a series of weirs that are on walls only four clear feet apart (alternating the position of the weirs from left side to right side). This layout is shown in Table 2.

Table 2: Layout of Pools (one-sided ladder)

Culvert Exit. Invert level is 22.5 ft NGVD.	
Pool 1. 4 ft long, 7 ft wide. Elevation of weir to next pool: 22.5 ft NGVD. EDF (no fill): 0.53 ft-lb/s/ft^3. Depth: 7.0 ft > 1.5 ft => OK.	Open side of stilling basin.
Pool 2. 4 ft long, 7 ft wide. Elevation of weir to next pool: 22.0 ft NGVD. EDF (no fill): 0.61 ft- lb/s/ft^3. Depth: 6.5 ft > 1.5 ft => OK.	
Pool 3. 4 ft long, 7 ft wide. Elevation of weir to next pool: 21.5 ft NGVD. EDF (no fill): 0.70 ft- lb/s/ft^3. Depth: 6.0 ft > 1.5 ft => OK.	
Pool 4. 4 ft long, 7 ft wide. Elevation of weir to next pool: 21.0 ft NGVD. EDF (no fill): 0.79 ft- lb/s/ft^3. Depth: 5.5 ft> 1.5 ft => OK.	
Pool 5. 4 ft long, 7 ft wide. Elevation of weir to next pool: 20.5 ft NGVD. EDF (no fill): 0.88 ft-lb/s/ft^3. Depth: 5.0 ft> 1.5 ft => OK.	
Pool 6. 4 ft long, 7 ft wide. Elevation of weir to next pool: 20.0 ft NGVD. EDF (no fill): 0. 96 ft-lb/s/ft^3. Depth: 4.5 ft> 1.5 ft => OK.	
Pool 7. 4 ft long, 7 ft wide. Elevation of weir to next pool: 19.5 ft NGVD. EDF (no fill): 1.05 ft-lb/s/ft^3. Depth: 4.0 ft> 1.5 ft => OK.	
Pool 8. 4 ft long, 7 ft wide. Elevation of weir to next pool: 19.0 ft NGVD. EDF (no fill): 1.14 ft-lb/s/ft^3. Depth: 3.5 ft> 1.5 ft => OK.	
Pool 9. 4 ft long, 7 ft wide. Elevation of weir to next pool: 18.5 ft NGVD. EDF (no fill): 1.23 ft-lb/s/ft^3. Depth: 3.0 ft> 1.5 ft => OK.	
18 ft NGVD target exit pool WSEL	18 ft NGVD assumed WSEL at end of stilling basin

The Energy Dissipation Factor EDF is estimated for this case, based on a flow of 3.3 cfs in the system of pools (greater flows would overtop the sides of the pools and fall into the open side of the stilling basin area.

Variations on this design might extend the ladder design farther downstream, but the objective to reach a pool exit level of 18.5 feet would change. For example, by extending the design 100 feet, the target exit level would be approximately 17.8 feet, requiring at least one more pool.

One drawback with this scheme is that the pools are **smaller than 10 feet long by 5 feet wide**, which is a recommended minimum spacing for a schooling species that prefers at least 10 feet between obstacles and requires frequent rest areas. If chosen for further development, then the geometrical changes outlined below might be considered instead. It should be noted in mitigation that the pools are 3 to 7 feet deep, so that there is significant room for energy to be dissipated within each pool.

The same basic fish ladder could be extended downstream of the stilling basin, into the wider valley to allow for longer pools. A 10-foot length of pools in 9 pools implies a total length of 90 feet. This is approximately 50 feet beyond the end of the current stilling basin. The stream invert at this location is known to be at 17.9 ft NGVD (lower than 18 ft NGVD). A tenth pool might therefore be necessary, leading to a total length of 100 feet.

High-flow (“100-year storm”) behavior

> The initial (farthest upstream) pool has walls at elevation at least 20.5+0.833 = 21.33 feet (10 inches higher than the lowest weir notch elevation).
>
> Treating this as a weir with Manning flows along the unaffected banks then the design storm passes approximately 80 cfs over the weir and 1940 cfs over the banks and through the “undeveloped” half of the stilling basin. This leads to a total flow of 2020 cfs before the water level reaches 24 ft NGVD in a 42-ft-wide total channel. **This flow requirement of 600 cfs is therefore satisfied.**

The assumption of 10 cfs being a typical flow during the migration season is supported by the brief, with gaps, Smelt Brook site-specific, record of daily water levels since 2020. The record is too brief to be considered definitive. It is unclear if the elevation data accurately reflect flows, although there was an immediate downstream response to stop-log/gate-controlled changes in the upstream pool level. The gage has not been calibrated.

To assess the likely range of flow at Smelt Brook, the climate change assessment reviewed USGS records of 4 nearby sites including Old Swamp River near South Weymouth, MA (basin area 4.5 square miles) and Town Brook at Quincy, MA (basin area 4.11 square miles). Of these comparison sites, Town Brook is more like Smelt Brook in that it is downstream of a substantial flood risk management dam. The annual peak flows at these two comparison sites appear to have been decreasing over time, although the trend was not statistically significant in either case. The range of annual peak flows at these sites ranged from 48.9 cfs/square mile (cfm) at Old Swamp Brook to 83 cfm at Town Brook. The other two nearby comparison sites were downstream of larger drainage basin areas: Monatiquot River at East Braintree, MA (28.7 square miles) and Whitman’s Pond Fish Ladder at East Weymouth, MA (12.5 square miles).

The precise geometry of the notches in the weirs will need to be subject to change as might be required if typical spring flows prove to be smaller than the 10 cfs assumption. If this should be the case, then a narrower notch width, or possibly a two-level notch-invert, might need to be considered in the PED design. Flow through a notch can be manipulated with stoplog structures to obtain the required depths or velocities through the notch. The flow can be augmented for a few days at a time by releasing water from the upstream dam if necessary. These possible design and operational tweaks to the design will need greater definition at the PED phase of the project.

Flow in the channel downstream of the stilling basin has been estimated at 3.1 feet deep when 600 cfs is passing. Assuming that there is a final weir at elevation 18 feet, with its own downstream toe at 17.5 ft NGVD, then the headwater levels at each weir in the step-pool side of the stilling basin are:

- 27.20 ft upstream of the weir at elevation 22.5 ft
- 26.69 ft upstream of the weir at elevation 22.0 ft
- 26.18 ft upstream of the weir at elevation 21.5 ft
- 25.66 ft upstream of the weir at elevation 21.0 ft
- 25.13 ft upstream of the weir at elevation 20.5 ft
- 24.59 ft upstream of the weir at elevation 20.0 ft
- 24.03 ft upstream of the weir at elevation 19.5 ft
- 23.44 ft upstream of the weir at elevation 19.0 ft
- 22.79 ft upstream of the weir at elevation 18.5 ft
- 22.05 ft upstream of the weir at elevation 18.0 ft

On the open side of the basin, the same calculations apply for the theoretical headwater level of the last level (22.05 ft immediately upstream of the 18-ft weir) but the elevations at intermediate locations closer to the headwall of the stilling basin are less easily calculated for two reasons: firstly, there is a hydraulic jump occurring over the 42-ft length of the stilling basin, creating dynamic conditions that are not amenable to an exact theoretical solution; and secondly, there is no barrier to prevent water from the step-pool side of the weir from passing laterally from the "pool" side to the "open" side. Theoretically, this means that the values listed above are conservative (high), but the exact water levels cannot be stated with certainty.

Practically, therefore, the initial level of the training walls needs to be 4.7 ft higher than the invert level of the CMP In order to contain a flow of 600 cfs. The training walls are configured with a horizontal crest, followed by a sloped section, and then a longer horizontal crest. The upper portion (roughly the upper half) of the sloped section would support the design; the lower half (below the point where a fence post has been affixed) would need to be horizontal (See Figure 1).

The computations to reach this result were based on a series of weirs, the first of which is at elevation 22.5 ft NGVD. The weir discharge coefficient is 2.80. There is submergence at each weir, so the weir equation was modified to be

$$Q_{\text{submerged}} = Q_{\text{free flow}} \times [1 - \{H_{\text{downstream}}/H_{\text{upstream}}\}^{1.5}]^{0.385}$$

The last (most downstream) weir in the set is at elevation 18.0 ft NGVD; below it, the tailwater is calculated to be 3.1 ft deep using the Manning Equation for a Manning n value of 0.035 with a symmetrical trapezoidal section with base width 10 feet and sideslopes 1-on-2. The channel slope was approximately 0.0317 (1-in-31).

Figure 1: Expected Water Level in the Stilling Basin Under Alternative 2, with a Flow of 600 cfs.

The clear space in the CMP above the 27.2-ft NGVD level indicates a worst-case for a 600-cfs flow. The CMP is therefore unlikely to cause a back-up at its entrance, and so would not lead to a requirement for greater upstream structural changes in order to avoid inundation while passing 600 cfs.

The perched culvert discharging to the stilling basin is a bitumen-coated corrugated metal pipe. Although for hydraulic calculations it is usefully approximated as a circular pipe of diameter 8 feet (96 inches, or approximately 2.4 meters), the cross-section is in fact wider than it is tall: it is, however, flatter at the base (its invert) and there is an arch-shape to its “ceiling” (its soffit). The wider bottom and the corrugations in the material serve to promote fish passage up the pipe by providing rest areas in the pipe during normal flows. Required depth minima are to be confirmed during the PED phase of the project, in concert with operating and maintenance procedures for the upstream pond outlets and the downstream gate outlets and monitoring and adaptive management requirements.

This design for 9 or 10 pools leads to a total flow of 600 cfs when the water is contained in the stilling basin, with the wall heights raised to allow for water levels up to 27.2 ft at the upstream end of the stilling basin. The design is conservative in that it estimates elevations based on hydraulics on the side of the basin with the step pools, without regard to the open side of the stilling basin, where approximately 50% of the flow would pass at realistically lower depths (initial estimate would have the flow passing at current or FWOP depths, which are below the 24-ft NGVD target depth).

Upstream of the stilling basin, the flow is conveyed in a bitumen-coated (BC) corrugated metal pipe (CMP). Give that the CMP is not circular in section, the wider base promotes a smoother, but shallower, outflow during normal or migration-season flows. Although there is a requirement for acceptable depths at weir notches, the full six-inch (150 mm) depth requirement, as cited for predator avoidance, is less critical in the closed CMP.

In the event of required depth-changes inside the CMP, pipe walls can be coated to adjust flow depths and effective diameter as a means to promote a desired depth of flow. For a circular pipe (a useful approximation), the following equation is introduced:

$$n_1 / n_2 = [d_1 / d_2]^{(8/3)}$$

where n and d are the Manning roughness and pipe diameter values of the two pipes.

Given that a typical CMP roughness is typically in excess of n=0.022, an equivalent flow of 600 cfs in the pipe should be feasible with a liner of smoother material. It is possible that a partial change for only the bottom of the pipe would increase flow depths inside the pipe under low flows (below 20 cfs) because of the non-circular shape of the existing pipe, while maintaining the 600 cfs design capacity. Similar Manning n and cross-section adjustments could be applied to allow for a partial lining of only a portion of the pipe, to enhance the flow depths during low profiles.

Liner could be inserted as a measure to address poor performance as part of adaptive management plan. These details will be expanded in a more detailed design phase. It is noted that an 8-ft diameter pipe with Manning n=0.024, being lined to create a 7.5-ft pipe, would need to have a Manning n of 0.020 (which is still rougher than a typical n-value of 0.015 for a rough concrete finish).

For the current layout of the CMP, a flow of 734 cfs passes through a circular 8-ft pipe of Manning n=0.024 when the pipe is 95% full. That this flow exceeds the target 600 cfs by 22% indicates that the structural changes downstream of the CMP will not lead to increased flooding upstream of the CMP.

This alternative meets the planning objectives and avoids the inundation constraints, and therefore was carried forward.

Alternative 3: Combination (nature-like bypass with weirs at designed intervals): extend a side-channel along the side of the steep "valley" so as to avoid the constraint of having the whole structure inside the stilling basin. This allows for longer pools, and smaller depths.

A design objective is to funnel 15% of the spring flow through a side channel, so that it provides an attractive flow for any migrating fish. By adjusting the weir locations in the dividing walls between the separate pools, the total distance is increased to result in fish swimming in a channel with an effective 1% grade. See Figures 2 and 3.

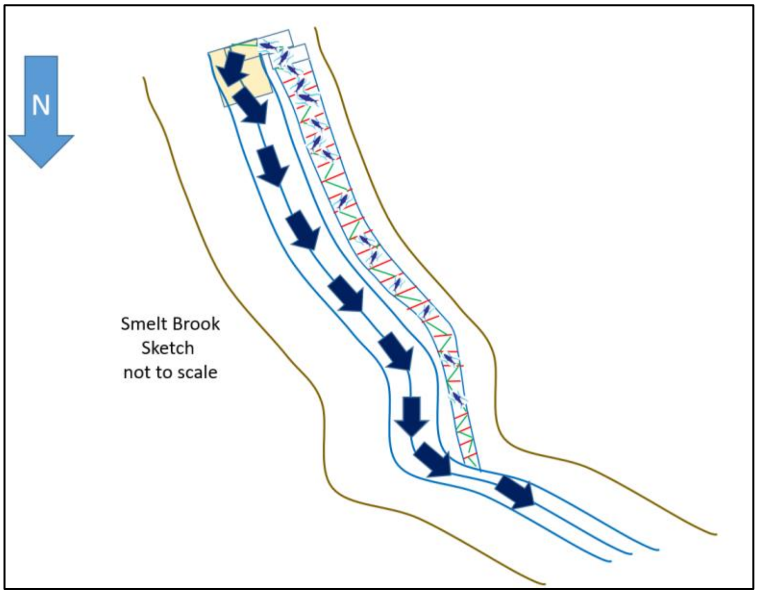

Figure 2: Smelt Brook: alternative canal flow for fish passage

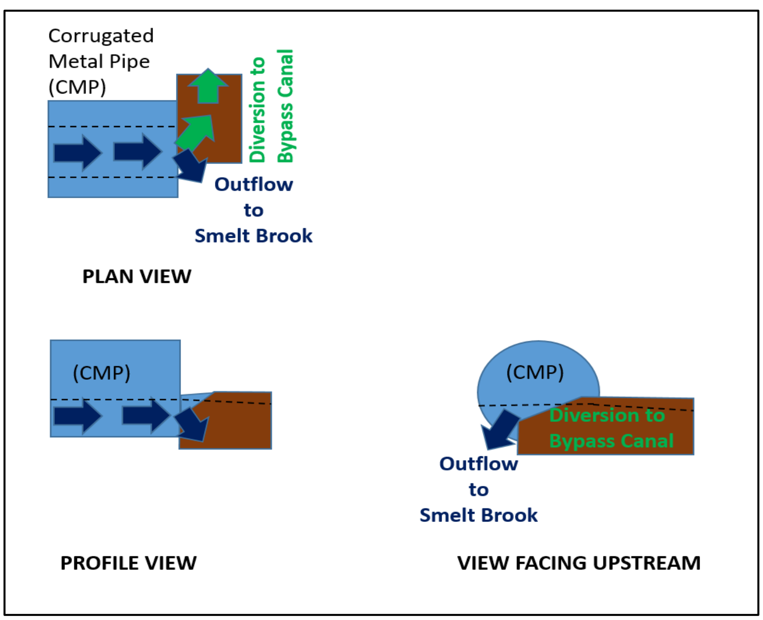

Figure 3: Smelt Brook: alternative canal intake for fish passage

600 feet distance along the stream profile has grade 1.6%, falling from 22.5 ft NGVD (CMP invert at exit) to 12.96 ft NGVD. Difference in elevation is 9.5 feet. Need an effective profile distance of 950 feet. With square pools, the extra distance directly across diagonals is 1.4x600=840 ft, too small. Rectangular pools are needed for an appropriate minimum factor (950/600=1.6). The pools need to be wider than this minimum requirement because the notches should not be at the edges of the pools.

Structural details: The canals might need to be one long rectangular canal with sets of stoplog slots for weirs to be inserted with geometry designed to facilitate maintenance and later fine-tuning of the design. Designed like an aqueduct, the canal could be above or at or slightly below ground grade depending on location. Might need piers in places. Design should be for the canals filled with water. If the water should rise above this level then there would be an overflow to back into the stream.

Assuming the pools in the canal are 10 feet long.
Pool width is taken as (1.6 x 10 feet) + 1 foot weir width + 1 foot edge allowance = 18 feet.
Number of pools = 600/10 = 60.
Drop in water level per pool (Delta WL) = 9.5/60 = 0.16 ft = 1.9 inches.
This solution takes up an 18-ft width which is almost half of the available channel “easement” width of roughly 45 feet. If implemented, the designer might prefer to place the canal on the right side of the channel (away from the houses at the beginning).

Assuming the pools in the canal should be 5 feet long. This might need refinement because a typical wall thickness between pools might need to be taken into account.
Pool width is (1.6x5) feet + 1 foot weir width + 1 foot edge allowance = 10 feet.
Number of pools = 600/5=120.
Drop in water level per pool = 9.5/120 = 0.08 ft = 0.95 inch.

High-flow (“100-year storm”) behavior

The initial (upstream end) of the structure has walls at elevation at least 20.5+0.833 = 21.33 feet (10 inches higher than the lowest weir notch elevation). It is assumed that the structure protrudes part-way across the CMP outlet and directs only a small amount (less than 10 cfs) along the canal system. The structure partly obstructs extreme flows. The blockage is taken for this purpose as being as for Alternative 2, although the structure does also limit the available space on the banks of the river.

Taking an extreme case, in which the channel base is only 7 feet wide, with a grade 1:1 (as opposed to the 1-on-2 gradient) the combined flow for weir plus channel is 80+1290 = 1370 cfs. The alternative keeps the 600 cfs flow below 24 ft NGVD.

Alternative 4: Naturelike Bypass with a "Switchback"

Allow for one narrow route approximately 5 feet wide, with a switchback distance of 175 feet. At a suitable location, there is a weir connection feeding the flow into a section that flows back towards the conduit for 180 feet. After this 180-ft detour, the flow is again reversed and set of pools takes the flow downstream to the stream channel. The total distance of 600 feet is extended by 2 lengths of 175 feet for a total extra length of 350 feet. The channelized flow reaches the stream channel after a diversion of 950 feet. Pool distances along this route are set to drop progressively by increments (decrements) of no more than 6 inches. Assuming that 4 inches is an acceptable minimum then the 9.5-foot total is achieved with a total of 9.5/0.333=28.5 (say 28) steps. The average gradient along this channel is 1%.

A general sketch is shown on Figure 4.

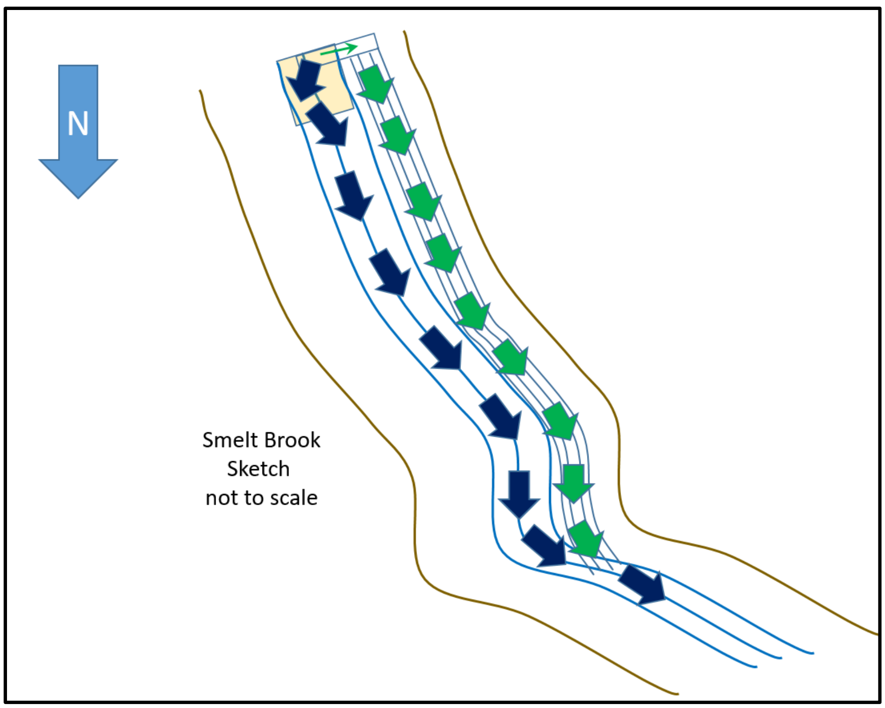

Figure 4: Alternative 4:

Flow is estimated using the Manning equation assuming a rectangular section 5 ft wide and 4 ft tall with Manning n=0.03 (would depend on the material used to fill the canal, could be as little as 0.013 initially (smooth cement finish) but after rocks are added and vegetation begins to grow then 0.03 would be expected). The many forced "breaks" at dividing walls would limit the flows.

Q = A x (1.49/n) (s^0.5) [(A/WP)^0.6667]
Q = 20 x (1.49/0.03) x (0.01^0.5) x [20/(4+5+4)^0.6667]
Q = 20 x 49.7 x 0.1 x 1.333 = 132 cfs. This is about one-third of the peak spring flow.
The velocity of 6.62 fps is likely excessive for smelt.

Try 2 ft tall and repeat:
Q = 10 x 49.7 x 0.1 x [10/(2+5+2)^0.6667]

Q = 10 x 49.7 x 0.1 x 1.073 = 53 cfs. This is about 15% of the peak spring flow.
Velocity = 53/10 ~ 5.3 fps.

Try 1 ft tall and repeat:
Q=5 x 49.7 x 0.1 x 0.799 = 19.9 cfs. This is about 40% of the peak spring flow.
Velocity = 19.9/5=3.97 fps.

Try 0.5 ft tall and repeat:
Q = 2.5 x 49.7 x 0.1 x 1.32 = 16.4
Velocity = 16.4/2.5=6.56 fps

The pools are about 1 ft deep (flow area 0.5 to 1.5 feet) under this alternative, in order to avoid excessive velocities.

To avoid any greater flows/velocities, the wall height on each side of the canal pools should be chosen to be no higher than 2 feet above the tops of the "weir" walls that divide the pools.

There would be a section of the design that is 3 canal pools wide (downstream, then upstream, then downstream again). If this is 3 x 5 feet then for approximately 180 feet, the total width of pool is 15 feet (19 feet if there are 1-foot-thick walls between the canals and some common walls. Assuming flow is controlled by the weirs then Flow = 2.5 b x (0.33^1.5) = 0.48 cfs. If the weir "control" is simply the entire wall length over dividing walls that are 5 feet long then the system is set up for a flow of 2.4 cfs. The flow is gentle enough that rocks, soil and plants could be placed in the channel to qualify as a "nature-like" bypass.

Theoretical maximum separation between weir walls is 950/29=32.7 feet to allow for 4-inch drops between the pools. Detailed site geometry may dictate a number between 30 and 35 feet, with more detailed design at the corners where the "switchbacks" occur.   Separation between the walls should not be less than 5 feet; wall spacing will dictate the elevations of the weir walls. These in turn will dictate the height of the channel walls parallel to flow (for example, 2 feet higher than the weir walls to allow for velocities to stay below 2.7 fps.

Alternative geometries could include the switchback being partly of completely under the highest level, with the final direction flow being partly underneath the switchback length. This more complex design might limit the footprint of the structure, but access and maintenance issues would become more difficult. The issue might need attention if the fish are known to recognize direction through shadows or observing the surrounding environment and hesitate at a 180° turn.

Although the design has pool depths (2.0 ft) in excess of 1.5 feet, there are **many obstacles and turns, with no real opportunity for the migrating fish to rest along the way**.

Wider pools with longer separating walls could be obtained if the "meander" is imposed with a series of bridges that take flow across the channel several times to obtain the 950 ft total distance for a 1% grade.

High-flow ("100-year storm") behavior

The initial (upstream end) of the structure has walls at elevation at least 20.5+0.833 = 21.33 feet (10 inches higher than the lowest weir notch elevation). It is assumed that the structure protrudes part-way across the CMP outlet and directs only a small amount (less than 10 cfs) along the canal system. The structure partly obstructs extreme flows. The blockage is taken for this purpose as being as for Alternative 3.

The combined flow for weir plus channel is 80 + 1290 = 1370 cfs. The alternative keeps the 600 cfs flow below elevation 24 ft NGVD.

Alternative 5: Engineered Weirs at Intervals Along the 600-ft length of the River Reach

A series of weirs across the entire stream section for several hundred feet. For an initial draft, the spacing has been set at 10 feet between the weirs. Weirs are designed with “notches” to allow for flow with at least three elevations. The intention is to ensure that, even at relatively low flows, the weirs will accommodate flow in adequate depths to promote the migration of smelt.

The basic layout is shown in Figure 5.

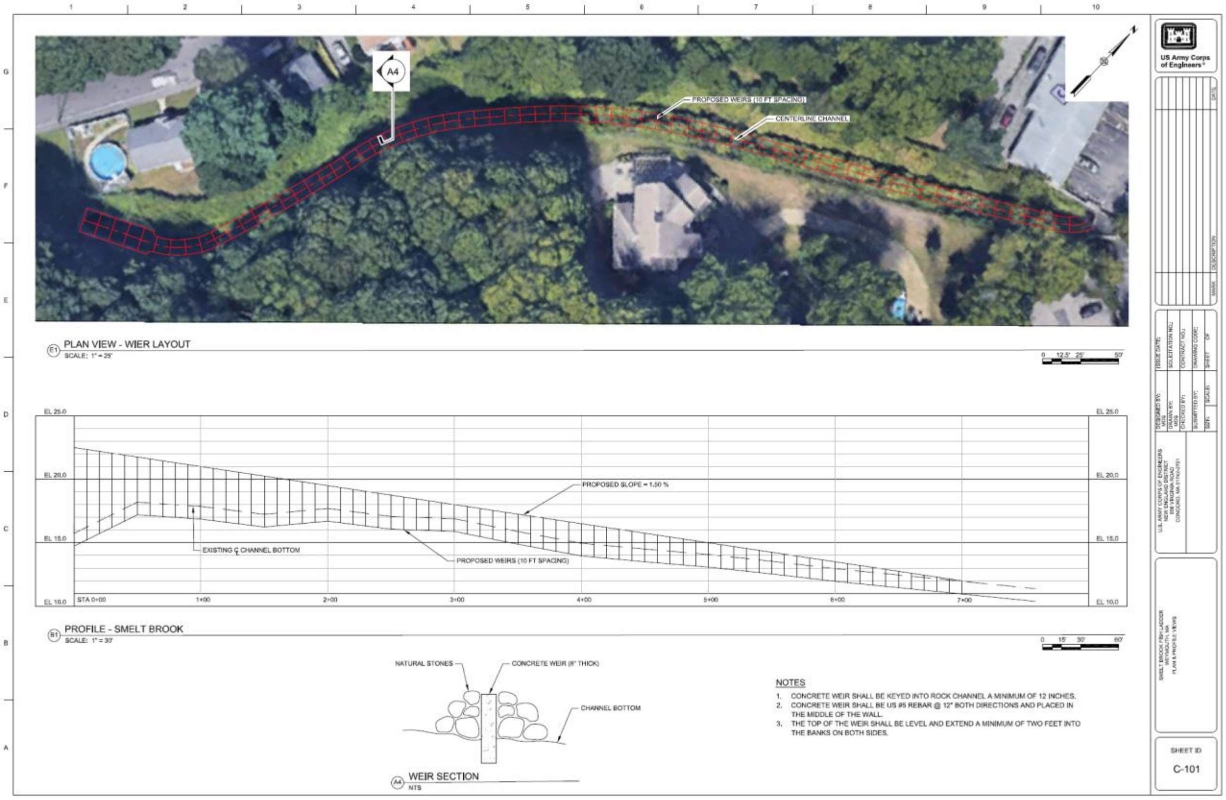

Figure 5: Alternative 5: Engineered Weirs at Intervals Along the 600-ft length of the River Reach

Given the potential for annual peak flows as high as 50 cfs, wider notches are suggested.

As an example, assume notch widths of 2 or 3 feet each at elevations that rise progressively by 2-inch increments.

A weir with lowest elevation 20 ft NGVD might therefore have elevations from one bank to the other as tabled:

Table 3: Alternative 5: Example Notch Levels for a Weir

Elevation (feet NGVD) (for this weir wall, lowest elevation is 20.00 feet NGVD)	Length of Weir (feet)
20.67 (or as required by terrain)	3 feet (or as required by terrain)
20.50	3 feet
20.33	2 feet
20.17	2 feet
20.00	6 feet
20.17	2 feet
20.33	2 feet
20.50	3 feet
20.67 (or as required by terrain)	3 feet (or as required by terrain)

The central 6-ft part of the weir flows constantly. The narrower 2-ft sections to either side of it are narrower to ensure that there is likely to be enough depth of flow at the lower flows, and the depth quickly rises to or above 0.5 feet of overflow. In free-flowing weirs, the weirs would comfortably pass 15 to 50 cfs without excessive depth over the weir. The smelt might find the central flow too rapid, but would be able to move to the higher portions of the notch, making the flows feasible for smelt migration under a wide range of spring flow conditions. The concern is further alleviated in that the tailwater is more likely retained at a known acceptable level to maintain a stream profile (both elevation level and water surface level) of 3 percent or less.

The 3% grade would meet the current streambed level after 120 feet; 2% would do so in approximately 200 feet; 1.5% would reach the streambed at approximately 300 feet; the figure shown demonstrates that the 1% grade would reach the streambed at or possibly even after the next concrete structures (the intake to the split between the 8-ft flow and the grated overflow structures). The gentle multi-weired layout is a gentle departure from the current landscape. Each pool between weirs is easily reached by migrating smelt. Although this is a departure from the 1% grade that is recommended, the figure demonstrates that the slightly greater slopes could accommodate the proposed layout without violating the 1/30 (3.33%) slope limitation that is recommended by the May 2016 interagency guidelines (Ref __) for rainbow smelt.

High-flow ("100-year storm") behavior

An original design profile for the brook along the reach from the culvert outlet to the Quincy feature has not been located for the 100-year, 600-cfs case. This alternative does not pass 600 cfs with an acceptable water level at the first affected house, and the full extent of water levels along the brook has not been investigated in detail. Elevated water levels along this region could serve to promote geotechnical movement in the valley, which would disturb the back yards of the adjacent houses.

The alternative effectively fills in most of the existing stilling basin. During an extreme event, the water leaving the culvert can be expected to be flowing at approximately 12 feet per second. 12 fps velocities can move 1-foot diameter boulders. There is a period of adjustment as the section shape changes from the shape of the culvert to the shape of the trapezoidal channel. Velocity is likely to be in the range 10 to 12 fps at that point.

The depth of flow continues to rise until the initial conditions for a hydraulic jump are reached. Since super- and sub-critical velocities are both between 7 and 8 fps, the location of the jump would be difficult to predict. Its length is likely to be 16 to 20 feet (8 to 10 times the depth), and the jump might not be obvious in a wide trapezoidal channel.

7 fps velocities can move boulders up to 10 inches. Any rock barriers would need to have been designed with reinforcement to prevent the loss or realignment of the rocks themselves.

The option of a steeper slope (3%) would pass more water, but flows of up to 515 cfs would be smaller than 600 cfs and velocities would likely be in excess of 7.5 fps (normal depth would be 3.5 feet, although the project has space to accommodate only 2 feet below the 24-ft elevation). Given the issues with control of the flow below the 24-ft NGVD level at the first house, and the diminished flow even using a 3% slope, the alternative has not been pursued as a feasible option.

Alternative 6: Keyhole Slot at Base of Existing Culvert Exit

The culvert discharges several feet above the stilling basin floor. This option would reduce the drop at the culvert exit by excavating a sloped exit over an extended distance, so that the energy is dissipated along a longer distance, both before and after the culvert exit. In this way the Options 1 and 2 solutions (pools inside the stilling basin) need to dissipate less kinetic energy.

For each foot that the exit elevation is dropped, roughly a foot of kinetic head upstream of the exit must be dissipated; and the remaining energy needs to be dissipated downstream of the exit. This is accomplished with a surface that is less smooth than the existing CMP finish (typically Manning n = 0.023; replacement would use a masonry/rock finish with a Manning roughness of about n ~ 0.04).

For simplicity, at this feasibility-type level of review, the following assumptions are made: the excavated channel would have its exit invert elevation at 20 ft NGVD, roughly midway between the current exit invert (22.5 ft) and the current floor of the stilling basin (taken as 18 ft NGVD). The excavated channel has a 2-ft-wide base, and its sloped side-walls are 6 feet wide at the level of the culvert. The excavation invert rises to meet the culvert invert at a distance of 100 ft upstream of the current exit. The excavated channel has reinforced walls along its sides, with internal walls perpendicular to flow at even intervals.

In keeping with the previously adopted design procedure for “interruptions” at equal intervals of head difference, the 100 feet of excavation would have a lateral wall every 10 to 15 feet with a narrower notch for fish passage during periods of lower flow. This would define eight artificial pools in the excavated channel, leading to a less energetic exit velocity at the stilling basin. The lateral walls would have approximate crest lengths in inches as listed:

27/33/39/45/51/57/63/69 inches.
Corresponding approximate notch inverts would be
20.0 at current exit/20.3/20.7/21.0/21.3/21.7/22.0/22.3 feet (most upstream pool in the tunnel.)

The basic layout is shown in Figure 5.

Figure 5: Alternative 6 View Shows the "Keyhole" Culvert exit

There would be a small number (3 to 6) of lower-level pools on the stilling basin, designed to ensure a smoother transition for fish from the elevation in their culvert-approach to their entrance into the culvert sections (to the first pool in the proposed keyhole).

It should be noted that, once the excavation has been made, a culvert made to order would be needed as a replacement for the existing culvert. A concrete culvert of appropriate dimensions would need to be defined and then ordered. The replacement culvert would be needed at least as far upstream as the point where the invert of the "keyhole" matches the invert of the existing culvert.

It is not clear that the need for the replacement culvert is obvious. Hydraulically, the capacity of the exposed channel would exceed the capacity of the closed-over channel. If the open "daylighted" section is extended upstream, then a new location should be chosen for the headwall shown in the figure, even if the current wall is retained.

The excavation option requires geotechnical consideration, and a review of whether the river would wander once the excavation occurred.

There should also be a review of buried utility lines, prior to selection of this option over any other options.

High-flow ("100-year storm") behavior

The flow capacity at the downstream end of the culvert would be enhanced. At the transition to the first pool on the stilling basin, the pool walls would be at or below 20 ft NAVD. This is significantly lower than the current 22.5 ft NAVD and therefore the design capacity of 600 cfs would be maintained.