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Preliminary Hydraulic Design of a Step-Pool-Type, Nature-Like Fishway

This preliminary hydraulic design method and the companion example, “Nature-Like Fishway at the Obex Mills dam site”, are presented as an instructional exercise illustrating how one may bridge the gap between a conceptual layout and subsequent design phases. During the preliminary phase, the target species design criteria, hydrologic information, and basic hydraulic relationships are used to size the step-pools and develop an initial configuration of the fishway. Typically, during the subsequent (more detailed) phases, the design is iteratively refined through numerical modeling and modified in response to site boundaries and property ownership, capital and recurring costs, technical feasibility, and other project constraints.

The following information is required to begin the preliminary hydraulic design:

- Topographic survey of project river reach and other features (e.g., dam structure)
- Bathymetry of project river reach (or estimates of river bed geometry)
- Hydrologic data for the river and project site in watershed (e.g., stream gage recordings)
- Design criteria for the target species

A preliminary hydraulic design can be developed through a step-by-step procedure. As presented in the “Federal Interagency Nature-like Fish Passage Guidelines for Atlantic Coast Diadromous Fishes” (unpublished; forthcoming 2015), design criteria have been established for selected diadromous fish species. These criteria represent minimum or maximum values of specific parameters. It is important to note that the design process involves iterative adjustment to these parameters in response to various constraints characterizing a project site. The basic steps in a preliminary hydraulic design of a step-pool-type, nature-like fishway include:

1. Establish Input Design Parameters
2. Characterize Site Hydrology
3. Define Size and Shape of Pools
4. Calculate Weir Hydraulics, Initial High Flow Calculations
5. Calculate Weir Hydraulics, Low Flow Calculations
6. Calculate Compound Weir Hydraulics, Median Flow Calculations
7. Calculate Compound Weir Hydraulics, High Flow Calculations

These seven steps are described in Figure 1, in the example below, and in the attached SMath Studio file. The reader is cautioned not to confuse a variable in these calculations (e.g., calculated channel slope) with the design criterion (e.g., recommended maximum slope) which may be represented by the same variable symbol (e.g., S_0).

SMath Studio

SMath Studio, as stated by the author, Andrey Ivashov, is a "powerful, free mathematical program with WYSIWYG editor and complete units of measurements support. It provides numerous computing features and rich user interface". The freeware may be downloaded at: http://en.smath.info/. The end user license information is available at http://smath.info/wiki/Lincense.ashx.

The example below has been solved in SMath Studio 0.97 which allows the user to view inputs, functions, and calculated results in one sequential development; in this way, it approximates an engineering calculation package. The SMath Studio example file is intended for educational purposes only.

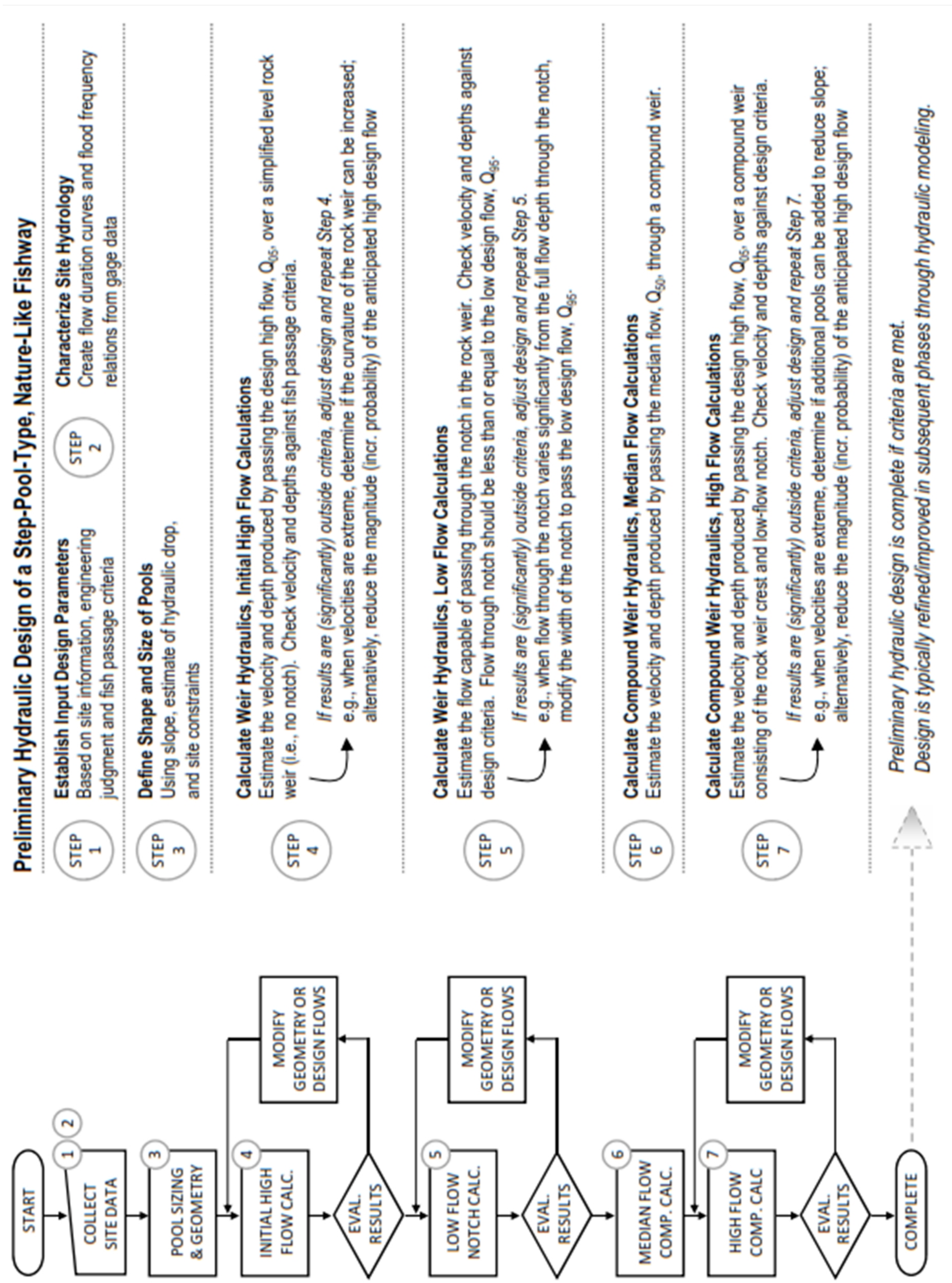

Figure 1. Flow chart illustrating the steps in the preliminary hydraulic design.

Example: Nature-Like Fishway at the Obex Mills dam site

Problem Description:

Obex Mills, a former textile mill site dating back to the late 1800s, is located on a major tributary of the Connecticut River in New England. In combination with the removal of an old timber crib dam and concrete abutments, the dam owner and project partners plan to construct a nature-like fishway (NLF) to provide passage and protect upstream structures from channel bed adjustments. Based on the site constraints and target species, blueback herring (*Alosa aestivalis*) and American shad (*A. sapidissima*), a notched, full-width, step-pool, rock weir NLF has been selected as the preferred fishway type. The following seven steps, as illustrated in Figure 1 and developed in the SMath file in Appendix A, describe the preliminary hydraulic design of a step-pool NLF at the Obex Mills site. Variables used to describe a step-pool NLF are illustrated in Figure 2.

Step 1. Establish Initial Design Parameters:

As with the design of any hydraulic structure, flow, velocity and other variables are influenced by the energy head and width of the conveyance. Accurate measurements of the headpond and the tailwater elevations are necessary.

The relic Obex Mills run-of-the-river dam maintained normal headpond and tailwater elevations of 100 ft NAVD 88 and 92.5 ft NAVD 88, respectively. The bankfull width of the channel establishes an estimated lateral width over which the fish passage structure is proposed. This is typically based on a site survey; alternatively, it may be estimated from a correlation to the 1.5-year flow (as developed in Step 2). For the Obex Mills dam site, the bankfull width and fishway pool width, W_P, is determined to be 120 feet.

Drop Per Pool

Based on the well-known Energy Equation, one foot of overtopping over a broad-crested weir roughly correlates to an average velocity of three feet per second. One may assume these values are appropriate for an estimate of the drop-per-pool for fishways designed to pass river herring. For the purposes of a preliminary hydraulic design, one should divide the gross head by a one-foot drop-per-pool to estimate the number of pools in the fishway.

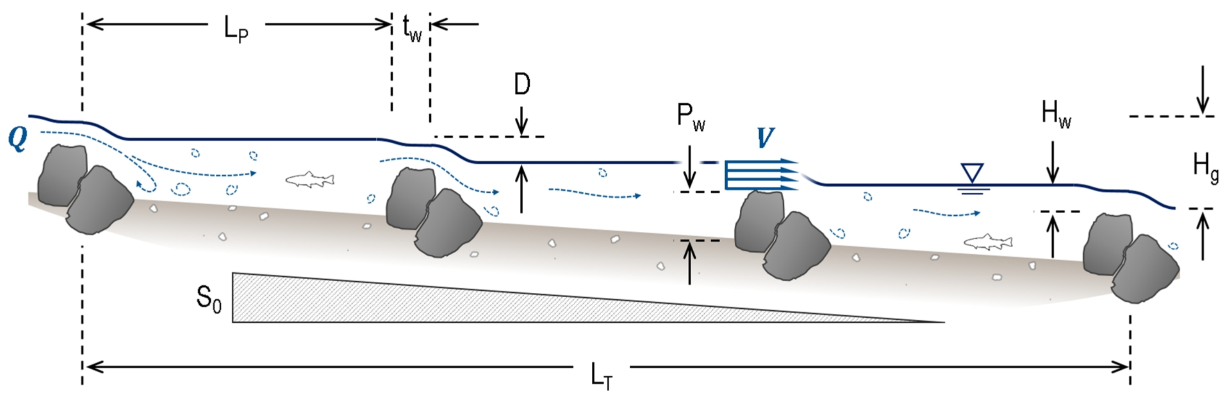

Figure 2. Variables describing the geometry of a step-pool, nature-like fishway.

The designer should estimate the number of pools, N_P, and total length of fishway, L_T, based on site conditions and project constraints. Site constraints may include slope of the natural river channel, interference with existing upstream or downstream in-stream structures, access, and site boundaries and property ownership. To address the 7.5-foot head drop and target species, the engineer assumes an initial conceptual design based on an 8-pool fishway reaching 250 feet downstream. As a general practice, the initial design should start with a 1-foot drop per pool and round up as needed to the next whole number for the initial determination of pool number.

The rock weir geometry is another important initial parameter. Typically, the longitudinal thickness or breadth of the rock weir crests, t_W, will be equal to the diameter of the rock specified by the engineer to construct the weir; the size of the rock weir stone is based on stability calculations and beyond the scope of this example. For the Obex Mills example, t_W is set at 4.0 ft for the stone length dimension. Weir structures are generally constructed in a curved arch. The curvature serves two primary purposes: 1) to provide stability against hydrodynamic forces, debris and ice loadings, and 2) to increase the effective weir length for discharge purposes. For this example, assume the curvature of the rock weir may be expressed as a multiple of the pool width, k_W=1.5. This simplified example will assume a constant-elevation rock weir crest and include a low (design) flow notch. The notch width, W_N, is based on design criteria for the target species requiring the widest opening due to body size and behavioral requirements. In this example, the notch width is set at 4.25 feet to accommodate American shad, the larger of the two target species. The height of the notch and the rock weir are 1.5 ft and 4.0 ft, respectively. A summary of the initial design criteria from this step is presented in Table 1.

L_T (ft)	N_P	W_P (ft)	P_W (ft)	t_W (ft)	k_W	W_N (ft)	P_N (ft)
250	8	120	4	4	1.5	4.25	1.5

Table 1. Initial design criteria and inputs for the Obex Mills example.

Step 2. Characterize Site Hydrology:

Flood frequency analyses are often necessary to develop the bankfull width of the channel. The bankfull width is generally established by the 1.5-year return interval flood. Many common techniques (e.g., Log Pearson Type III) employ coefficients established for the 1- and 2-year floods. Accordingly, the 1.5-year event may be interpolated.

At the Obex Mills site, a Log Pearson Type III flood frequency analysis, based on the peak flows recorded at a nearby USGS stream gage located on the river with continuous flow data from 1990 through 2009, yielded the following:

p	**RI (yrs)**	**K**	**Y_{LP3}**	**Q (cfs)**
0.99	1.01	-1.66	3.313	2050
0.5	2	-0.148	3.460	2900
0.2	5	0.769	3.550	3550
0.1	10	1.339	3.605	4000
0.04	25	2.018	3.671	4700
0.02	50	2.498	3.718	5200
0.01	100	2.957	3.763	5800

Table 2. Flood frequency analysis for the Obex Mills site.

The bankfull-width flow was estimated at 2,475 cfs, based on the mid-point value of the 1- and 2-year return events for the site. While beyond the scope of this example, it is important to note that the flood frequency analysis plays an important role in establishing the hydrologic loadings used to design stable rock weirs and other design components.

Numerous agencies, technical guidelines and scientific literature provide methods for establishing the operating flow range of a fishway. Most of these methods are based on a flow duration curve. The flow duration curve (or table) represents the historical probability, p, that specific river flows were equaled or exceeded.

The flow duration table for the known blueback herring and American shad migration period of April 15 through July 15 is based on daily average flows continuously recorded from 1990 through 2009 at the nearby USGS stream gage:

p	0	0.01	0.05	0.1	0.3	0.5	0.7	0.9	0.95	0.99	1
Q (cfs)	4,110	2,863	2,100	1,590	713	430	268	121	84	44	33

Table 3. Flow duration table for the Obex Mills example.

Design flows, or operating flows, represent the maximum and minimum river flows under which one may expect the target species to be migrating in the river. The design flows are often translated into stage fluctuations or used to design the operating range of fish passage structures. For simplicity in this example, it is assumed that the Obex Mills site will maintain a constant headpond and tailwater elevation; however, a real-world application should incorporate changing water surface elevations associated with flow in the river. Here, design flows are defined as the 5% and 95% exceedance flows. This is consistent with guidance provided by federal resource agencies (see for example, NMFS 2011). The average daily river flow during the migration season from 1990-2009 was 657 cfs. Based on the 5% and 95% exceedance levels, the nominal high, Q_{05}, and low, Q_{95}, design flows are 2,100 cfs and 84 cfs, respectively. The median flow, Q_{50}, during this period was 430 cfs.

Step 3. Define Size and Shape of Pools:

Based the initial design criteria established in Step 1 and the site hydrology developed in Step 2, one may now determine the physical and hydraulic geometry of the NLF step pools (Figure 2). The (interior) longitudinal length of the pools, L_P, can be calculated using the total fishway or project channel length, L_T, the number of pools, N_P, and the weir stone/boulder thickness, t_W. For the Obex Mills project site, L_P is:

$$L_P = \frac{L_T}{N_P} - t_W \qquad L_P = \frac{250\,ft}{8} - 4.0\,ft = 27.25\,ft \tag{1}$$

The total lift or gross head, H_g, is the difference between normal water surface elevations in the headpond and the tailwater, and for Obex Mills the following H_g is derived:

$$H_g = EL_{HW} - EL_{TW} \qquad H_g = 100\,ft - 92.5\,ft = 7.5\,ft \tag{2}$$

The hydraulic drop, D, is an important parameter that influences both water velocities and the effective slope of the fishway. D is set by the gross head at the barrier and the number of pools. At Obex Mills, D is:

$$D = \frac{H_g}{N_P+1} \qquad D = \frac{7.5\,ft}{8} = 8.33\,ft \tag{3}$$

Effective Slope

Many conveyance structures (e.g., stormwater channels) are designed under the assumption that conditions do not change with distance along the channel. In other words, the channel bottom and water surface remain parallel. Mathematically, this requires that the channel slope remain equal to the friction slope. This is called uniform or normal flow. While step-pool fishways are not truly normal flow, flow development is typically rapid and they may be regarded as uniform-in-the-mean. Conceptually, this allows one to describe the NLF average slope or "effective slope" as the ratio of gross head over the total length of the fishway.

If one assumes that the friction slope of the fishway is equivalent to the channel slope (i.e., a uniform flow assumption), then the effective slope, S_0, is the ratio of gross head to fishway length:

$$S_0 = \frac{H_g}{L_T} \qquad S_0 = \frac{7.5\,ft}{250\,ft} = 0.03 \tag{4}$$

The curved rock weir results in greater discharge per unit hydraulic drop by providing an effective length greater than the pool width. The curved or effective width of the rock weir crest for Obex Mills is:

$$W_W = k_W W_P \qquad W_W = (1.5)120\,ft = 180\,ft \tag{5}$$

Results of the Obex Mills pool size calculations are:

L_P (ft)	H_g (ft)	D (ft)	S_0 (ft/ft)	W_W (ft)
27.25	7.5	0.833	0.03	180

Table 4. Results of the pool size calculations, including physical and hydraulic geometry.

Step 4. Calculate Weir Hydraulics, Initial High Flow:

By establishing the basic geometry and hydraulic design parameters, the weir hydraulics can then be examined. It is convenient to initially evaluate a level rock weir under a high design flow condition; the goal is to determine the degree of overtopping and to evaluate the velocity created by this overtopping that will influence fish passage.

It is important to note that if the estimated velocity over the crest exceeds the maximum velocity criterion for blueback herring (the weaker of the two target species), then the preliminary hydraulic design will not provide requisite passage at the high design flow. In this situation, the designer should try to reduce the overtopping by altering the initial parameters (as set in Step 1), or recognize the infeasibility of providing passage at the standard high design flow, Q_{05}, and reduce the high design flow and re-calculate the velocities.

Hydraulically, NLF rock weirs approximate submerged broad-crested weirs. Depth over a broad-crested weir transitions from the upstream hydraulic head (nearly equal to the full energy head) to the critical depth (which theoretically occurs over the crest). However, since alosine fishways should operate under slightly submerged conditions that allow fish to move up through the nappe (i.e., the sheet of water flowing over the weir, barrage or dam), free overfall conditions are avoided and, thus, depth of flow may never drop to the critical depth. For design purposes, the water depth over the weir can be bracketed between the hydraulic head, H_W, and the critical depth, y_C.

The well-known weir equation can be used to estimate the level of overtopping during the high design flow. It takes the form:

$$Q = C_s C_w W_w {H_w}^{1.5} \tag{6}$$

where C_s is the submergence coefficient or drowned flow reduction factor, and C_W is the weir coefficient. C_s reduces the discharge based on the water level on the downstream side of the weir. Numerous studies have investigated the influence of submergence on sharp-crested weirs; the authors are not aware of a specific investigation on rock weir submergence. Plate 3-5 of EM 1110-2-1603 *Hydraulic Design of Spillways* (USACOE 1990) is used in the companion SMath file and may serve as a surrogate for the submergence coefficient in Equation 6.

The weir coefficient, C_w, for broad-crested weirs is largely a function of the thickness (or breadth) of the weir crest and the head on the weir. The coefficients in Table 5 are recommended for rock weirs:

Head	Thickness or Breadth of Rock Weir Crest, t_w (ft)								
(ft)	0.5	0.75	1	1.5	2	2.5	3	4	5
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66

Table 5. Recommended Coefficients for Rock Weirs. Values are based on broad-crested weir coefficients reported in Brater and King (1976).

The goal of this step is to determine H_W that results in a Q equal to Q_{05}. However, since C_s and C_W are also dependent on the hydraulic head, the resulting expression is implicit with respect to H_W. A mathematical software package (e.g., SMath Studio) or a simple numerical technique such as the Secant Method (Chapra and Canale, 2009) may be used to find the root of:

$$F(H_w) = \left|C_s C_w W_w {H_w}^{1.5} - Q_{05}\right| = 0 \qquad H_w = 2.78\,ft \tag{7}$$

The average velocity over the crest can then be estimated using conservation of mass (i.e., continuity):

$$V = \frac{Q}{H_w W_w} \qquad V = \frac{2100\, cfs}{(2.78\, ft)(180\, ft)} = 4.2\ \frac{ft}{s} \qquad (8)$$

This average velocity under initial high flow conditions is less than recommended maximum velocity. In the event that the (initial high flow) velocity exceeds the weakest swimming target species criterion, one may alter the design to reduce this velocity (e.g., increase the curvature and effective weir length), or recognize that the high design flow is less than the initial target exceedance of 5%. Since this criterion is met for the Obex Mills site, one proceeds to the next step.

Step 5. Calculate Weir Hydraulics, Low Design Flow:

In an approach similar to the previous step, the discharge through the weir notch, Q_N, is examined. Based on the appropriate design criteria, we assume the following for the Obex Mills site: a notch depth of 2.5 feet (below the rock weir crest) to accommodate the body size of American shad, the larger of the two target species, and a lateral notch width, W_N, of 4.25 feet. During low design flows, one anticipates the flow to pass solely through the notch. Therefore, the notch depth is equivalent to the head on the notch, H_N, and a direct application of weir equation is used to estimate the discharge:

$$Q_N = C_s C_N W_N {H_N}^{1.5} \qquad Q_N = 42.76\, cfs \qquad (9)$$

where and C_N is the notched weir coefficient. The average velocity through the notch can be estimated using the conservation of mass equation:

$$V = \frac{Q}{H_N W_N} \qquad V = \frac{42.76\, cfs}{(2.5\, ft)(4.25\, ft)} = 4.02\ \frac{ft}{s} \qquad (10)$$

These results confirm that the notch will pass the low design flow (i.e., $Q_N < Q_{low}$) and that the velocities through the notch do not exceed the blueback herring design criterion recommended for Obex Mills. If $Q_N > Q_{low}$ the designer may consider increasing the lateral width of the notch. Since this design criterion is met at the Obex Mills site, we proceed to the next step.

Step 6. Calculate Weir Hydraulics, Median Flow:

Building on Steps 4 and 5, one now refines the preliminary design by evaluating the hydraulics under median flow conditions through a compound weir that includes the low flow notch. The flow through a compound weir is described by:

$$Q = C_s C_N W_N {H_N}^{1.5} + C_s C_w W_w {H_w}^{1.5} \tag{11}$$

Similar to Step 4, Equation 11 allows us to determine both H_W and H_N by setting Q equal to the median flow, Q_{50}. To solve (numerically or otherwise), an additional equation is required. Recall:

$$H_N = H_w + (P_w - P_N) \qquad\qquad H_N = H_w + 2.5\,ft \tag{12}$$

H_W can be determined by solving for the root of:

$$F(H_w) = \left| C_s C_N W_N [H_w + (P_w - P_N)]^{1.5} + C_s C_w W_w {H_w}^{1.5} - Q_{50} \right| = 0 \tag{13}$$

For the Obex Mills step-pool NLF, the hydraulic head at the median flow of 430 cfs is:

$$H_w = 0.85\,ft \qquad\qquad H_N = H_w + 2.5\,ft = 3.35\,ft \tag{14}$$

Under median flow conditions, the average velocity over both the crest and through the notch can be estimated using continuity:

$$V_w = 2.46\,ft \qquad\qquad V_N = 4.52\,\frac{ft}{s} \tag{15}$$

The velocity and depths corresponding to the median flow are for comparison purposes only; therefore, the iteration as shown in Steps 4, 5 and 7 in Figure 1, is generally not recommended for this step.

Step 7. Calculate Compound Weir Hydraulics, High Design Flow

Repeating Step 6 for the high design flow, we again refine the preliminary design by evaluating the hydraulics under high flow conditions through a compound weir that includes the low flow notch. The flow through a compound weir is described by Equations 11 and 12. In this application, we determine both H_W and H_N by setting Q equal to the high design flow, Q_{95}. To solve, an additional equation is required. H_W can be determined by solving for the root of:

$$F(H_w) = \left|C_s C_N W_N [H_w + (P_w - P_N)]^{1.5} + C_s C_w W_w {H_w}^{1.5} - Q_{95}\right| = 0 \tag{16}$$

For the Obex Mills step-pool NLF, the hydraulic head at the high design flow is:

$$H_w = 2.7\, ft \qquad H_N = H_w + 2.5\, ft = 5.2\, ft \tag{17}$$

The average velocity over the crest and through the notch can be estimated using continuity:

$$V_w = 4.15\, ft \qquad V_N = 5.82\, \frac{ft}{s} \tag{18}$$

As illustrated in preliminary design flow chart (Figure 1), if the (high design flow) velocity or any other parameter exceeds its design criterion, one may alter the design to reduce this velocity (e.g., increase the curvature and effective weir length), or recognize that the high design flow is less than the initial target exceedance of 5%.

The preliminary hydraulic design is complete once it has been verified that the results meet the target species design criteria. This preliminary design approach allows the designer to complete other preliminary tasks such as project layout, basic cost estimates, and feasibility assessments. Subsequent design phases may then incorporate these results as inputs to a computational hydraulic model (e.g., HEC-RAS or more refined model) in order to further refine the design.

Disclaimer

This technical report was developed by the authors with input from other subject-matter experts. It is intended for use by persons who have the appropriate degree of experience and expertise. The recommendations contained in this report are not universally applicable and should not replace site-specific recommendations, limitations, or protocols.

The authors have made considerable effort to ensure the information upon which these guidelines are based is accurate. Users of this information are strongly encouraged to independently confirm the information and recommendations contained within this document. The authors accept no responsibility for any inaccuracies or information perceived as misleading. The findings and conclusions in this report are those of the authors and do not necessarily represent the views of the University of Massachusetts Amherst, the U.S. Fish and Wildlife Service, the National Oceanic and Atmospheric Administration, or the United States Geological Survey.

Any use of trade, product, or firm names is for descriptive purposes only and does not imply endorsement by the U.S. Government or the University of Massachusetts Amherst.

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APPENDIX

Example: Obex Mills

Brett Towler, USFWS, 3/27/2015

Preliminary Hydraulic Design of a Step-Pool, Nature-Like Fishway

This worksheet solves an example problem for the preliminary hydraulic design of a step-pool type nature-like fishway (NLF).

Problem Description: Obex Mills, a former textile mill site dating back to the late 1800s, is located on a major tributary of the Acme River in New England. Following removal of the timber crib dam and concrete abutments, the owner and stakeholders have planned to construct a nature-like fishway to improve passage over the relatively aggressive channel slope. Based on the site constraints and target species (river herring and American shad), a notched, full-width, step-pool NLF has been selected as the preferred fishway type.

Given recommend design criteria, estimate the limiting velocities over rock weirs and estimate the size and dimensions of the NLF step-pool fishway.

Data Files, Custom Functions, and Constants Used in the Solution:

i. The following commands import hydrologic and hydraulic data from CSV files. CSV files must be located in the same folder as this SMath worksheet.

```
FLD:= importData("FloodFreq.csv", ".", 0, ",", 0, 0, 0, 0, 0)        Flood Return Intervals
FDC:= importData("FlowDuration.csv", ".", 0, ",", 0, 0, 0, 0, 0)     Flow Duration Curve
WEIR:= importData("WeirCoeff.csv", ".", 0, ",", 0, 0, 0, 0, 0)       Weir Coefficients Table
SUB:= importData("DrownedFlow.csv", ".", 0, ",", 0, 0, 0, 0, 0)      Submergence Coefficients
```

ii. The following custom user functions are used in this worksheet. The functions are defined here and called throughout the solution (below).

```
minterp(x, y, M):= | A:= submatrix(M, 1, 1, 2, cols(M))
                   | B:= submatrix(M, 2, rows(M), 1, 1)
                   | C:= submatrix(M, 2, rows(M), 2, cols(M))
                   | for j ∈ 1, 2 .. length(A)
                   |    CC_j:= linterp(B, submatrix(C, 1, rows(C), j, j), y)
                   | linterp(A^T, CC, x)
```

Bi-linear interpolation of a matrix; assumes independent variables are in first row and first column. Note that function (and its arguments) is unitless.

$C_w(t, h, M) := \text{minterp}(t, h, M)$ Weir coefficient

$$C_{df}(h, d, M) := \left| \begin{array}{l} xx := \text{submatrix}(M, 1, 23, 1, 1) \\ yy := \text{submatrix}(M, 1, 23, 2, 2) \\ 1 - \dfrac{\text{linterp}\left(xx, yy, \dfrac{d}{h}\right)}{100} \end{array} \right.$$

Drowned Flow Reducution factor

$$q\left(h, d, w, t, C_1, C_2\right) := C_{df}\left(h, d, C_2\right) \cdot C_w\left(t, h, C_1\right) \cdot w \cdot h^{\frac{3}{2}}$$

Weir equation. Note the function (and its arguments) is unitless.

iii. The following constants, units, and coefficients are used in the solution:

$g := 32.17 \dfrac{ft}{sec^2}$ acceleration due to gravity

$\rho := 1.94 \dfrac{slug}{ft^3}$ density of water

$cfs := \dfrac{ft^3}{sec}$ custom unit "cfs" built from standard units

1. Establish Initial Design Parameters - These inputs set basic geometry of the fishway.

Figure 1

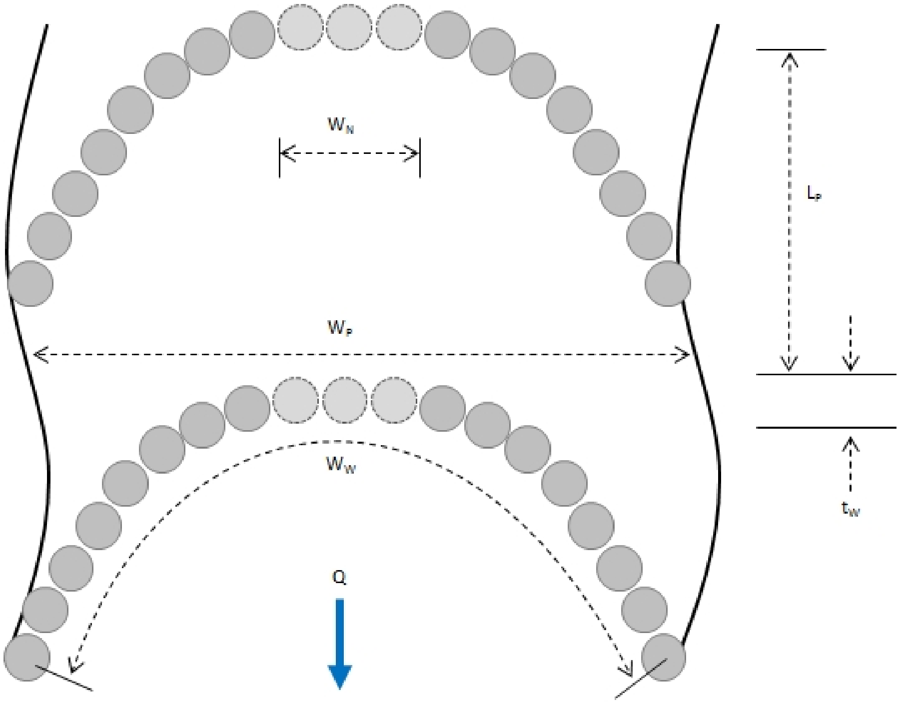
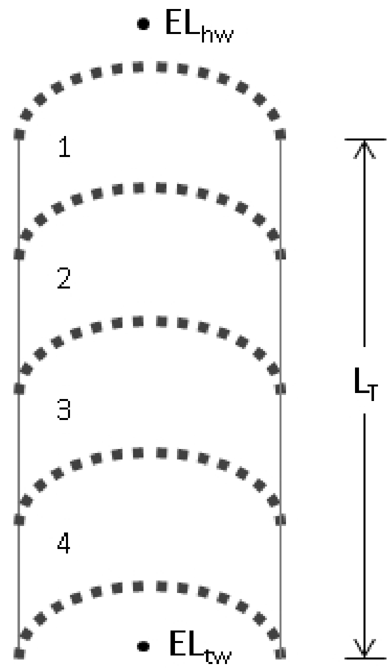


Figure 2



USER INPUT OF INITIAL DESIGN PARAMETERS

$EL_{hw} := 100\ ft$	Elevation of headwater
$EL_{tw} := 92.5\ ft$	Elevation of tailwater
$L_T := 250\ ft$	Total length of fishway
$N_P := 8$	Number of pools in fishway
$W_P := 120\ ft$	Width of the pool and lateral width of river
$P_W := 4.0\ ft$	Height of rock weir crest measured from channel bottom
$t_W := 4.0\ ft$	Longitudinal thickness (breadth) of the rock weir
$k_W := 1.5$	Ratio of curved rock weir width to pool width
$W_N := 4.25\ ft$	Lateral width of notch; est. based on criteria for
$P_N := 1.5\ ft$	Height of notch invert measured from channel bottom

These 10 variables (highlighted in yellow) are the only user inputs to the model.

2. Characterize Site Hydrology - Flow Duration and flood frequency developed.

$Q_{bw} := \frac{FLD_{1,2}\, cfs + FLD_{2,2}\, cfs}{2}$	$Q_{bw} = 2475\, cfs$	Bank width of river; typically 1.5 year flood
$Q_{100yr} := FLD_{7,2}\, cfs$	$Q_{100yr} = 5800\, cfs$	100-year flood is used in rock sizing and stability

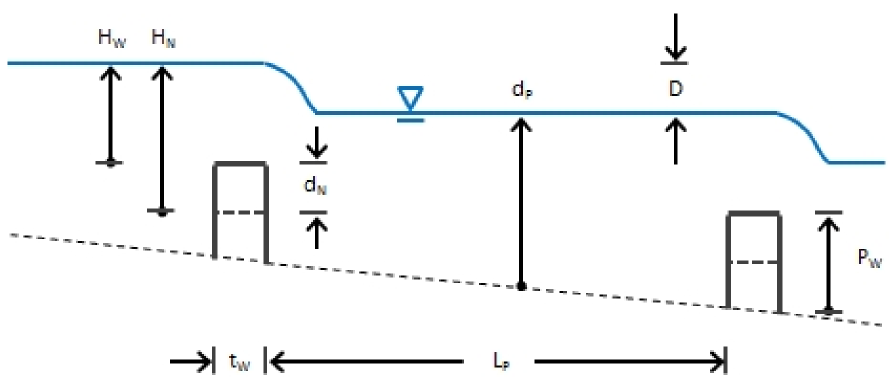
Note: Neither the bankwidth flow, Qbw, nor the 100-year flood, Q100yr, are used directly in this worksheet. Qbw serves as a hydrologic vertification of the appropriate pool width, Wp, for a full-width NLF. Q100yr is used in stability and sizing of both base material and stone sizes in the rock weir (which influences the weir thickness, tw).

$Q_{95} := FDC_{14,2}\, cfs$	$Q_{95} = 83.95\, cfs$	The flow qualled or exceeded 95% of the time
$Q_{50} := FDC_{9,2}\, cfs$	$Q_{50} = 430\, cfs$	The median flow
$Q_{05} := FDC_{4,2}\, cfs$	$Q_{05} = 2100\, cfs$	The flow qualled or exceeded 5% of the time

Note: Q95 and Q05 are used in establishing the low and high design flows, respectively. The median flow is a convienent indicator of moderate flows and helps establish the trend between low and high flows (and dependent variables such as velocity).

3. Define Pool Shape and Size - Calculations of pool dimensions and fishway weir length.

Figure 3



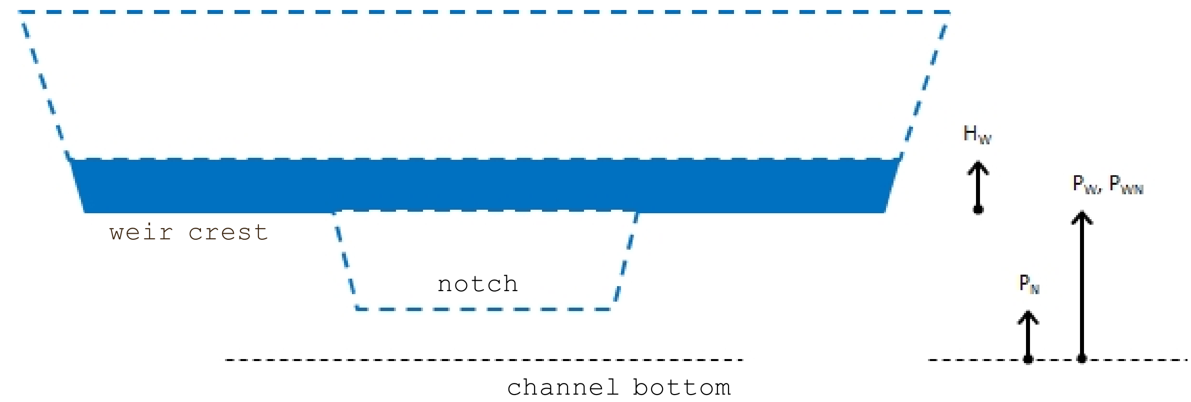
$L_P := \frac{L_T}{N_P} - t_W$	$L_P = 27.25\, ft$	Estimate of longitudinal length of fishway pools
$H_g := EL_{hw} - EL_{tw}$		Gross head (or lift) across fishway
$D := \frac{H_g}{N_P + 1}$	$D = 0.8333\, ft$	Hydraulic drop per pool; influences velocity and slope
$S_0 := \frac{H_g}{L_T}$	$S_0 = 0.03$	NLF slope
$W_W := k_W \cdot W_P$	$W_W = 180\, ft$	Curved length or effective width of rock weir arches.

$W_{WN} := W_W - W_N$ $\quad W_{WN} = 175.75\,ft$ $\quad$ Rock weir length less notch length

4. Calculate Weir Hydraulics, Initial High Flow

For this step, the depth of flow and velocity over the rock weir is estimated for the high flow. All flow is assumed to pass over the crest (i.e., no notch) for this initial step. Assume the drowned flow reduction factor (due to submergence) is approximately 1.0.

Figure 4



Hydraulically, fishway rock weirs approximate submerged broad-crested weirs. Depth over a broad-crested weir transitions from the depth corresponding to the energy head (driving the flow) and the critical depth (which theoretically occurs over the crest). However, since most fishways weirs operate under slightly submerged conditions, depth of flow may never drop to the critical depth. For design purposes the depth over the weir can be bracketed between the hydraulic head and the critical depth.

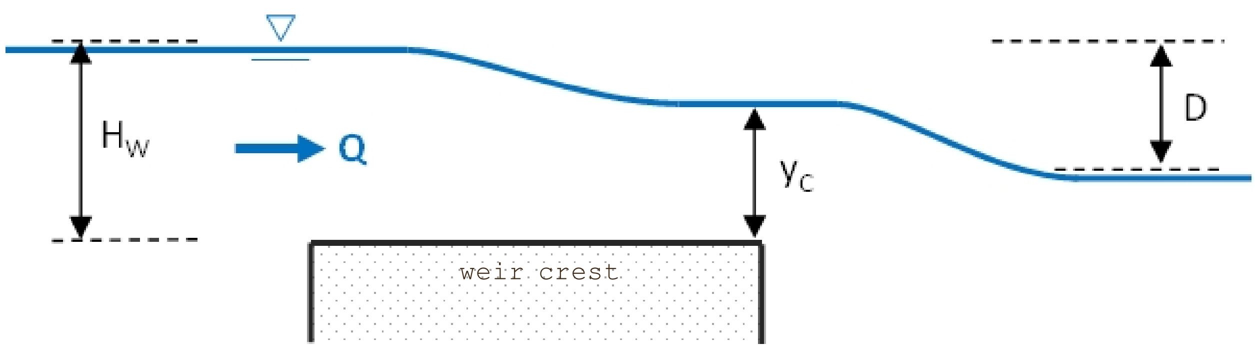
$$H_W := \text{solve}\left(q\left(hh, \frac{D}{ft}, \frac{W_W}{ft}, \frac{t_W}{ft}, \text{WEIR}, \text{SUB}\right) - \frac{Q_{05}}{cfs} = 0, hh\right) ft$$

The weir equation is solved implicitly for the head over weir under high flow. The "solve" function is used to numerically solve implicity for Hw.

$H_W = 2.78\,ft$ $\quad$ (Energy) head over weir crest

$y_C := \frac{2}{3} \cdot H_W$ $\quad y_C = 1.85\,ft$ $\quad$ Critical depth over weir crest

Figure 5



Similarly, the velocity over the crest can be bracketed by the approach velocity driven by the energy head and the critical velocity.

$V_W := \dfrac{Q_{05}}{H_W \cdot W_W}$ $\qquad V_W = 4.2 \dfrac{ft}{sec}$ Approach velocity at upstream edge of rock weir crest

$V_C := \left(\dfrac{1}{3} \cdot H_W \cdot 2 \cdot g\right)^{\frac{1}{2}}$ $\qquad V_C = 7.72 \dfrac{ft}{sec}$ Critical velocity; upper limit not likely to be realized under submerged conditions

Turbulence and air entrainment can hamper fish movement. The energy dissipation factor (EDF) is a commonly used metric of bulk turbulence and air entrainment in fishways pools. While no specific criteria for EDF is offered, the designer is cautioned to ensure this value remains relatively small.

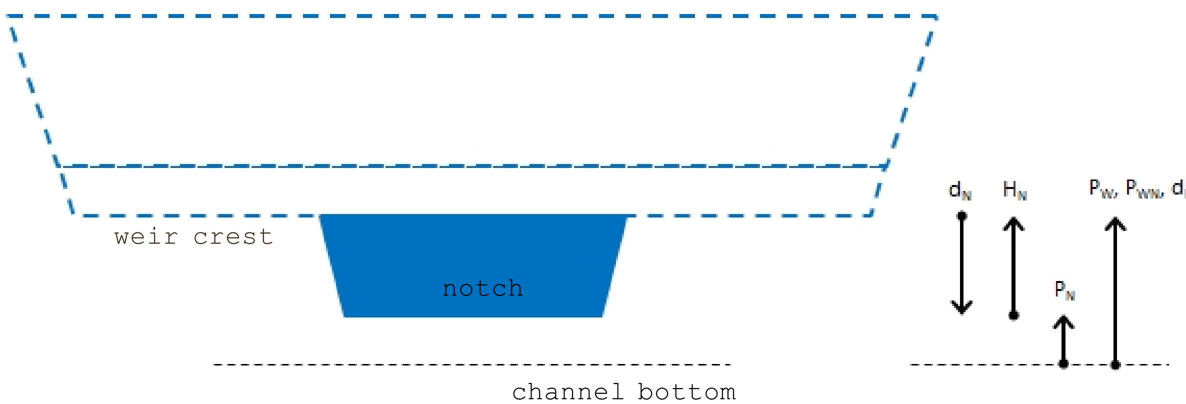
$EDF := \dfrac{Q_{05} \cdot D \cdot \rho \cdot g}{\left(P_W + H_W\right) \cdot L_P \cdot W_P}$ $\qquad EDF = 4.93 \dfrac{ft\ lbf}{sec\ ft^3}$ EDF or power dissipation rate

Note: If the velocity and depth values above are NOT within recommended ranges, then the design should be modified (e.g., reduction in high design flow) before proceeding to the next steps.

5. Calculate Weir Hydraulics, Low Design Flow

For this step, the depth of flow and velocity through the notch in the rock weir is estimated for the low flow. Here, a direct solution of the weir equation is made to check if the notch will provide passage at flows less than or equal to the low design flow.

Figure 6



$H_N := P_W - P_N$ $\qquad H_N = 2.5\ ft$ Head on notch under low flow conditions

$y_C := \dfrac{2}{3} \cdot H_N$ $\qquad y_C = 1.67\ ft$ Critical depth on notch under low flow

$Q_N := q\left(\dfrac{H_N}{ft}, \dfrac{D}{ft}, \dfrac{W_N}{ft}, \dfrac{t_W}{ft}, WEIR, SUB\right) cfs$

$Q_N = 42.76\ cfs$ Discharge through notch under low flow conditions

$Q_{95} = 83.95\ cfs$ QN is less than Q95 so design can proceed

$V_N := \dfrac{Q_N}{H_N \cdot W_N}$ $\qquad V_N = 4.02 \dfrac{ft}{sec}$ Approach velocity at upstream edge of rock weir crest

$$V_C := \left(\frac{1}{3} \cdot H_N \cdot 2 \cdot g\right)^{\frac{1}{2}} \qquad V_C = 7.32 \frac{ft}{sec}$$

Critical velocity; upper limit not likely to be realized under submerged conditions

$$EDF := \frac{Q_N \cdot D \cdot \rho \cdot g}{P_N \cdot L_P \cdot W_P} \qquad EDF = 0.45 \frac{ft\ lbf}{sec\ ft^3}$$

EDF or power dissipation rate under low-flow conditions

Summary of Low Design Flow Conditions:

--> Flow in the river is: $Q_N = 42.76\,cfs$

--> Water depth in notch is: $H_N = 2.5\,ft$ and $y_C = 1.67\,ft$

--> Notch velocity is between: $V_N = 4.02 \frac{ft}{sec}$ and $V_C = 7.32 \frac{ft}{sec}$

6. Calculate Compound Weir Hydraulics, Median Flow

Now, the depth of flow and velocity over the rock weir is estimated for the median flow. Flow passes over a compound weir that included both the low-flow notch and the level rock weir. Again, assume the drowned flow reduction factor is approximately 1.0.

Again, for design purposes the depth over the compound weir can be bracketed between the hydraulic head and the critical depth.

The conditions over the rock weir are calculated first:

$$H_W := \text{solve}\left[q\left(hh, \frac{D}{ft}, \frac{W_{WN}}{ft}, \frac{t_W}{ft}, WEIR, SUB\right) + q\left(hh + \frac{P_W - P_N}{ft}, \frac{D}{ft}, \frac{W_N}{ft}, \frac{t_W}{ft}, WEIR, SUB\right) - \frac{Q_{50}}{cfs} = 0, hh\right] ft$$

$$H_W = 0.85\,ft$$

Head on weir crest under median flow

$$y_C := \frac{2}{3} \cdot H_W \qquad y_C = 0.56\,ft$$

Critical depth on rock weir crest under median flow

The velocity over the weir crest can be bracketed by the approach velocity driven by the energy head and the critical velocity.

$$V_W := \frac{q\left(\frac{H_W}{ft}, \frac{D}{ft}, \frac{W_{WN}}{ft}, \frac{t_W}{ft}, WEIR, SUB\right) cfs}{H_W \cdot W_{WN}}$$

$$V_W = 2.46 \frac{ft}{sec}$$

Approach velocity at upstream edge of rock weir crest

$$V_C := \left(\frac{1}{3} \cdot H_W \cdot 2 \cdot g\right)^{\frac{1}{2}} \qquad V_C = 4.26 \frac{ft}{sec}$$

Critical velocity; upper limit not likely to be realized under submerged conditions

The velocity and depths corresponding to the median flow are for comparison purposes only; median flow is generally not listed as a design criterion.

Summary of Median Flow Conditions over the Rock Weir Crest:

--> Flow in the river is: $Q_{50} = 430\ cfs$

--> Water depth over crest is: $H_W = 0.85\ ft$ and $y_C = 0.56\ ft$

--> Velocity on crest is between: $V_W = 2.46\ \frac{ft}{sec}$ and $V_C = 4.26\ \frac{ft}{sec}$

The conditions through the notch are calculated next:

$H_N := H_W + P_W - P_N$ $\qquad H_N = 3.35\ ft$ $\qquad$ Head on notch invert under median flow

$y_C := \frac{2}{3} \cdot H_N$ $\qquad y_C = 2.23\ ft$ $\qquad$ Critical depth on notch under median flow

The velocity through the notch can be bracketed by the approach velocity driven by the energy head and the critical velocity.

$$V_N := \frac{q\left(\frac{H_N}{ft}, \frac{D}{ft}, \frac{W_N}{ft}, \frac{t_W}{ft}, \mathrm{WEIR}, \mathrm{SUB}\right) cfs}{H_N \cdot W_N}$$

$V_N = 4.52\ \frac{ft}{sec}$ $\qquad$ Approach velocity at upstream edge of the notch

$V_C := \left(\frac{1}{3} \cdot H_N \cdot 2 \cdot g\right)^{\frac{1}{2}}$ $\qquad V_C = 8.47\ \frac{ft}{sec}$ $\qquad$ Critical velocity; upper limit not likely to be realized under submerged conditions

The velocity and depths corresponding to the median flow are for comparison purposes only; median flow is generally not listed as a design criterion.

$$EDF := \frac{Q_{50} \cdot D \cdot \rho \cdot g}{\left(P_W + H_W\right) \cdot L_P \cdot W_P}$$

$EDF = 1.41\ \frac{ft\ lbf}{sec\ ft^3}$ $\qquad$ EDF or power dissipation rate under median flow conditions

Summary of Median Flow Conditions through the Notch in the Weir:

--> Flow in the river is: $Q_{50} = 430\ cfs$

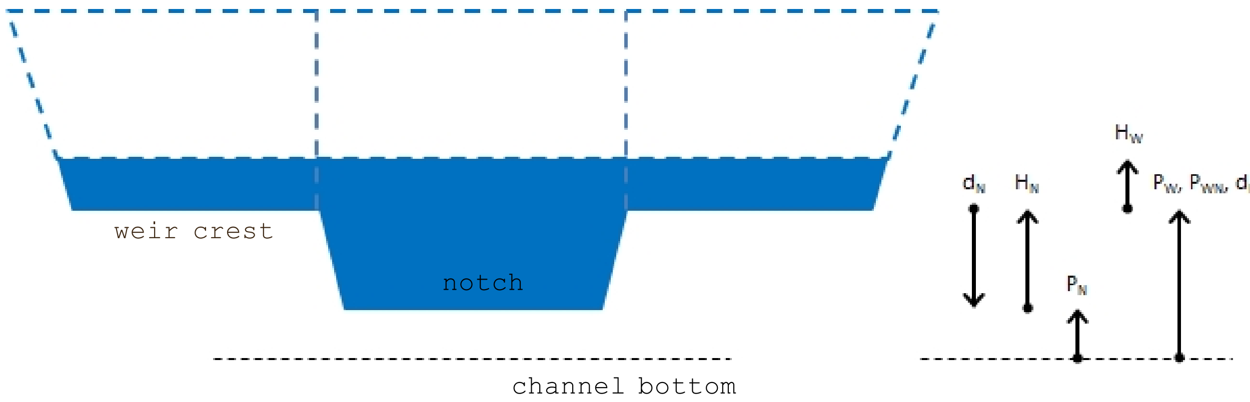
--> Water depth in notch is: $H_N = 3.3453\ ft$ and $y_C = 2.23\ ft$

--> Notch velocity is between: $V_N = 4.52\ \frac{ft}{sec}$ and $V_C = 8.47\ \frac{ft}{sec}$

7. Calculate Compound Weir Hydraulics, High Flow

Now the depth of flow and velocity over the rock weir are estimated for the high design flow. Flow passes over a compound weir that included both the low-flow notch and the level rock weir.

Figure 6



For design purposes the depth over the compound weir can be bracketed between the hydraulic head and the critical depth.

The conditions over the rock weir are calculated first (then the notch):

$$H_W := \text{solve}\left[q\left(hh, \frac{D}{ft}, \frac{W_{WN}}{ft}, \frac{t_W}{ft}, \text{WEIR}, \text{SUB}\right) + q\left(hh + \frac{P_W - P_N}{ft}, \frac{D}{ft}, \frac{W_N}{ft}, \frac{t_W}{ft}, \text{WEIR}, \text{SUB}\right) - \frac{Q_{05}}{cfs} = 0, hh\right] ft$$

$H_W = 2.7\ ft$ — Head over rock weir crest

$$y_C := \frac{2}{3} \cdot H_W$$

$y_C = 1.8\ ft$ — Critical depth on rock weir crest under high flow

The velocity over the weir crest can be bracketed by the approach velocity driven by the energy head and the critical velocity.

$$V_W := \frac{q\left(\frac{H_W}{ft}, \frac{D}{ft}, \frac{W_{WN}}{ft}, \frac{t_W}{ft}, \text{WEIR}, \text{SUB}\right) cfs}{H_W \cdot W_{WN}}$$

$V_W = 4.15\ \frac{ft}{sec}$ — Approach velocity at upstream edge of rock weir crest

$$V_C := \left(\frac{1}{3} \cdot H_W \cdot 2 \cdot g\right)^{\frac{1}{2}}$$

$V_C = 7.61\ \frac{ft}{sec}$ — Critical velocity; upper limit not likely to be realized under submerged conditions

Summary of High Flow Conditions over the Rock Weir Crest:

--> Flow in the river is: $Q_{05} = 2100\ cfs$

--> Water depth over crest is: $H_W = 2.7\ ft$ and $y_C = 1.8\ ft$

--> Velocity on crest is between: $V_W = 4.15\ \frac{ft}{sec}$ and $V_C = 7.61\ \frac{ft}{sec}$

The conditions through the notch are calculated next:

$H_N := H_W + P_W - P_N$ $\quad H_N = 5.2\ ft$ Head on notch invert under high flow

$y_C := \frac{2}{3} \cdot H_N$ $\quad y_C = 3.47\ ft$ Critical depth on notch under high flow

The velocity through the notch can be bracketed by the approach velocity driven by the energy head and the critical velocity.

$$V_N := \frac{q\left(\frac{H_N}{ft}, \frac{D}{ft}, \frac{W_N}{ft}, \frac{t_W}{ft}, \mathrm{WEIR}, \mathrm{SUB}\right) cfs}{H_N \cdot W_N}$$

$V_N = 5.82\ \frac{ft}{sec}$ Approach velocity at upstream edge of the notch

$$V_C := \left(\frac{1}{3} \cdot H_N \cdot 2 \cdot g\right)^{\frac{1}{2}}$$

$V_C = 10.56\ \frac{ft}{sec}$ Critical velocity; upper limit not likely to be realized under submerged conditions

The power dissipation under high flow conditions is:

$$EDF := \frac{Q_{05} \cdot D \cdot \rho \cdot g}{\left(P_W + H_W\right) \cdot L_P \cdot W_P}$$

$EDF = 4.98\ \frac{ft\ lbf}{sec\ ft^3}$ EDF or power dissipation rate under high flow conditions

Summary of High Flow Conditions through the Notch in the Weir:

--> Flow in the river is: $Q_{05} = 2100\ cfs$

--> Water depth in notch is: $H_N = 5.2\ ft$ and $y_C = 3.47\ ft$

--> Notch velocity is between: $V_N = 5.82\ \frac{ft}{sec}$ and $V_C = 10.56\ \frac{ft}{sec}$