Maine Department of Transportation
Fish Passage Policy and Design Guide

Design Guide for Fish Passage Through Culverts

Introduction

This manual is intended for the design of new and replacement culverts, as well as culvert rehabilitations, that will not block passage of identified fish species at specified design flows. Engineers will find these design guidelines useful in the implementation of Maine Department of Transportation (Maine DOT) fish passage policy as documented in the companion volume to this work (Maine DOT, 2007a). The manual is intended for use by Maine DOT engineers and designers as well as other engineers designing stream crossings in a fisheries environment. At this time stream crossing design for fish passage should continue to be performed by or under the direct supervision of an experienced hydraulic engineer working with a fisheries biologist.

This manual is limited to culverts; it does not address dedicated fishway passage structures and stream (geomorphic) simulation. Culverts are often the most desirable road crossing for small and medium sized streams from an engineering standpoint. However, it is recognized that from the larger perspective of aquatic organism passage (AOP) culverts are in fact less desirable than bridges and bottomless arches that preserve or simulate a natural stream bottom. It is recommended that the upcoming U.S. Forest Service design manual be used for stream simulation applications.

Culvert Barriers to Fish Passage

There are several common conditions at culverts that can create barriers to fish movement:

- excess drop at culvert outlet
- high velocity within culvert barrel
- inadequate depth within culvert barrel
- turbulence within culvert barrel
- debris accumulation at culvert inlet

Barriers are created by several conditions. Culverts are usually uniform and sized to pass peak design flows, e.g., the 50-year flood Q_{50}. They do not have the roughness and variability of natural stream channels and therefore do not dissipate kinetic energy effectively. Thus, velocities tend to be higher in a culvert than in the stream. This effect is amplified by the fact that existing culverts are often narrower than the stream, constricting flow at the inlet. This may have the effect of increasing velocity in the pipe, creating turbulence at the inlet, and creating velocity-induced scour holes at the outlet. Outlet scour may induce a significant drop at the outlet. The last barrier condition, debris accumulation, is due to inadequate maintenance.

New and replacement stream crossings can be designed to avoid the first four, hydraulics related, barrier conditions. The last condition, even in a well-designed culvert, depends on good maintenance attuned to the specific fish passage requirements of a culvert. Fish passage can be difficult to restore in rehabilitated and retrofit culverts. Mitigating design elements in addition to the basic culvert lining are usually needed in order to establish passage under specified conditions.

Design Objectives

General Objectives

In designing for fish passage through culverts, two objectives are paramount:

- maintain depth equal to or greater than the necessary minimum
- keep velocity less than or equal to limiting maximum sustainable fish swimming speed

The issue of uninterrupted pipe length is related to flow velocity and the ability of a fish to transit a culvert. Culverts with interior grade control structures will generally offer adequate in-pipe resting areas for fish. Culverts longer than 75 ft (23 m) and without interior structures should be referred to Maine DOT Environmental Office for determination of species-dependent length requirements.

Strictly speaking, these limiting values are determined by the target species of interest, the time of year they are moving, and the direction they are moving in. This information is summarized in Table 2 of the Fish Passage Policy. These factors, combined with watershed hydrology and channel geomorphology, provide the information necessary for estimating an appropriate passage design flow.

Generic Design Standards

While species-specific design is always appropriate, the design process can be simplified by employing generic parameters that produce robust designs suitable for most species of interest in Maine. Therefore, Maine DOT recommends as a starting point the following generic design standards:

- design for passage during September/October low flow period
- design flows shall be determined by regression equations and also field-based measurement whenever possible
- maintain at least 8 inch water depth throughout the length of the culvert at design low flows
- limit flow velocity to no more than 2 ft/s (0.6 m/s) (not including weir notches)
- limit drop in water surface elevation at outlet to 2 in
- use average of median September and October flows as design flow
- limit water level drop across grade control structures to 8 in (200 mm)
- when weirs are employed, weir notches should be at least 8 in (200 mm) wide by 8 in (200 mm) deep. Calculated dimensions should be rounded to the nearest 2 in (50 mm) increment.

The design report shall include

- calculated water surface profiles through the culvert
- calculated Energy Dissipation Factors (EDF)
- passage hydraulic performance results for other months of passage

These generic standards constitute a starting point for design. This design is likely to be overly conservative and therefore may be difficult to realize in particular situations. Consideration should then be given to a species-specific design. The final design should satisfy any particular species requirements, for example as documented in Table 2 of the Fish Passage Policy. Final design may also deviate from these general objectives, depending on site-specific factors. Species-specific factors may allow for some relaxation of these generic standards. For example, many Maine fish species can actually pass over pool drops greater than 8 in (200 mm), and designing for larger drops (e.g., 12 in (300 mm)) permits a wider inter-weir spacing and therefore fewer weirs. Reducing or eliminating the weir notch invert submergence has a similar effect.

<u>Atlantic Salmon</u>

Atlantic salmon are of special interest. The design low flow for salmon will be based on August median flow. Since salmon are strong swimmers and can jump, water level drops across grade control structures can be as large as 12 in (300 mm) and velocities as large as 8.5 ft/s (2.6 m/s) can be tolerated by adults.

General Steps in Design for Culvert Fish Passage

The following steps are generally followed when addressing fish passage through culverts.

1) identification of valuable habitat for specific species and need for passage by fisheries biologists in Maine DOT, resource agencies, and regulatory agencies
2) determination of calendar periods when passage must be provided
3) estimation of design flows during passage periods
4) culvert design
    a) new pipe: size pipe for peak flow (50-yr or similar low-frequency event) capacity and passage performance by hydraulic analysis; check flow surface width for $Q_{1.5}$ in culvert against bankfull channel width.
    b) rehabilitated pipe: hydraulic analysis to check performance of proposed rehabilitation; design mitigation measures (e.g., weirs, baffles, outlet notch ramps) if fish passage is inadequate

Fish Habitat Considerations In and Adjacent to Culverts

There are several aspects of fish habitat that warrant consideration in passing fish through culverts. Inside the culvert, the issue is the culvert bottom. For traditional enclosed circular culverts and multi-plate pipe arches, a natural bottom can be simulated with varying degrees of success by embedding the pipe and filling it with substrate, generally to a depth of 1 ft – 3 ft (300 mm – 900 mm). Detailed recommendations are given later in this report. Open bottom provide a natural, and therefore superior, bottom habitat. However, such structures can cost significantly more than enclosed culverts.

Culvert inlets and outlets are often treated with riprap to protect the structure and prevent erosion and scour. The immediate culvert inlets and outlets usually merit extensive riprap in order to provide structural protection. With regards to stream bank stabilization, it is preferred that riprap be limited to an elevation somewhere in the the 2-year to 5-year flow event stages; above this elevation, it is desirable that banks be stabilized by vegetation. Also, it is desirable that vegetation in the vicinity of inlets and outlets provide shading.

Design Approaches: New & Rehabilitated Culverts

Two basic design approaches are employed by Maine DOT. For new and replacement culverts, the preferred approach is to match culvert dimensions and gradient to natural bankfull stream channel hydraulic geometry, subject to standard Maine DOT culvert design practices. This approach is in the spirit of simulating the hydraulic aspects of the stream at fish passage flows, but stops short of creating a natural, variable bottom. The assumption is that by eliminating perched outlets and matching hydraulic geometry in the range of critical fish passage flows, fish passage is assured at those times of year when fish are also present in the adjacent natural channel. The validity of this assumption should be checked in each design. This approach simplifies design and construction and minimizes the hydraulic and hydrologic analysis necessary.

For culvert rehabilitation (e.g., by slip or invert lining), additional hydraulic analysis and design is necessary. In this case, hydraulic analysis is employed to estimate water velocities and depths under design flows. Analysis is also employed to design mitigation measures (e.g., weirs) needed to achieve velocities and depths that will pass fish.

For both new and rehabilitated pipes, grade control structures (i.e. weirs) can be used to provide both acceptable water depths and velocities. In particular, in-pipe weirs will be the preferred means for creating acceptable fish passage hydraulics in rehabilitated pipes. Culverts are typically rehabilitated by concrete invert lining and plastic slip lining. In both cases, the improved pipe is characteristically "smooth bore", creating potential problems of excessive velocity and shallow depth. Weirs eliminate the roughness/ smoothness objection by creating required backwater through the pipe. In-pipe weirs can also be constructed in new pipes; the use of pre-cast concrete pipe weir sections has been demonstrated; pre-fabricated plastic weir sections have also been introduced.

When culverts are not too steep, a single downstream weir be enough to back water through the entire pipe length, thus resolving flow depth issues as well as resolving perched outlets. Downstream weirs may also be employed to back water to the first in-pipe weir, allowing fish to enter the pipe. In some instances, though, downstream weirs may be precluded by limited right-of-way and other access issues.

Hydraulic Considerations in Culvert Fish Passage

New and replacement culverts must be designed to pass the 50-year flow event (or "flood") in accordance with Maine DOT Drainage Policy. Rehabilitated culverts should be evaluated for their ability to pass the 50-year flood, though the reduction in cross-sectional area and effects of fish passage mitigation measures may reduce the pipe capacity. Peak flows (50-year or similar low-frequency event) should be estimated according to the methods used by Maine DOT in highway and bridge design.

In addition to the traditional peak flow design standard, culverts in identified fisheries should permit fish passage during a range of low flows. Two potential hydraulic problems are addressed in designing for fish passage. Water depth in the culvert may be inadequate to permit movement. Also, the velocity in the culvert may be too high for fish to swim against in an upstream direction.

These potential barriers to passage establish two design objectives, as summarized in the Maine DOT Fish Passage Policy. Occasionally, resource and regulatory agencies may directly specify a minimum depth and/or maximum velocity to be achieved. The two design objectives relate to depth and velocity:

1) maintain adequate in-culvert water depth for identified species during low flow conditions to allow passage;
2) during periods of upstream movement, flow velocity should not exceed species swimming capacity while adequate depth is maintained

These design standards are species- and season-dependent. The depth and flow velocity should be determined by hydraulic analysis and checked against species-dependent criteria. In the case of proposed culvert rehabilitation, failure to meet standards will require mitigation measures or possibly a replacement pipe.

Energy Dissipation Factor (EDF) and Turbulence

The Energy Dissipation Factor (EDF) quantifies the capacity of a water body to dissipate the energy (potential or kinetic) of an entering flow stream. A high EDF implies high turbulence, potentially a barrier to fish passage. EDF is calculated as the rate of energy flux (i.e. power P) into the pool divided by the pool volume V,

$$EDF = P/V$$

For flow over a weir into a pool, potential energy (PE) is the appropriate measure; the kinetic energy (KE) of the water above the weir is assumed negligible. For discharge to an outlet pool, kinetic energy may be of interest. Alternatively, outlet pool EDF may be calculated as PE from the nearest upstream in-pipe weir. If there are no in-pipe weirs, then EDF can be calculated as PE from pipe inlet.

<u>EDF - Potential Energy</u>

Potential energy is calculated relative to the downstream pool elevation. For a pool drop of Δy, water above the weir has a potential energy (per unit volume) PE = $\rho g \Delta y$, where ρ is the density and g is the acceleration due to gravity. The rate at which this PE is conveyed to the pool (i.e., the power P of the water) is given by product of PE and volumetric flow rate: P = PE x Q. Then EDF is calculated as

$$EDF = \rho g(Q\Delta y/V)$$

where ρg = specific weight of water (62.4 lb/ft^3 or 9.8x10^3 N/m^3)
Q = flow (ft^3/s or m^3/s)
Δy = drop in water surface elevation (ft or m)
V = volume of receiving pool (ft^3 or m^3)

For passage of salmonids, EDF should be no greater than 5 ft-lb/ft^3/s or 250 J/m^3/s (Washington State Dept. of Fish and Wildlife, 1999; Bureau of Land Management). An example of EDF calculation in given in Appendix 2D as part of the weir notch sizing example. EDF can be controlled by decreasing Δy (drop in water surface across weir) and/or by increasing pool volume. Since pool volume depends on the distance between weirs, the culvert bottom slope ultimately imposes a critical constraint on achievable EDF.

EDF – Kinetic Energy

For discharge directly into an outlet pool, the energy to be dissipated can be taken as entirely kinetic. On a volumetric basis, KE = $\rho v^2/2$ and the energy transport rate is P = KE x Q. Then EDF is calculated as

$$EDF = \rho(v^2Q/2V)$$

where ρ = density of water (1.94 (lb.s^2/ft^3/ft or 10^3 kg/m)
v = flow velocity (ft/s or m/s)

Culvert Outlet Hydraulics: Energy Dissipation Pools

Compared to a natural stream reach of the same length, a culvert tends to dissipate less energy and therefore water exits a culvert with more kinetic energy than the stream reach. Unless properly addressed, this elevated energy may tend to dissipate by excavating an outlet scour pool. This pool may develop to such an extent that the culvert becomes perched and blocks fish passage at lower flows. The elevated exit velocities may also exceed the swimming capacity of fish and/or create turbulence that discourages fish from entering the culvert. These undesirable effects can be mitigated by constructing energy dissipation pools at culvert outlets. The pools also provide areas where fish can rest prior to their entry into culverts.

The following guidelines should be followed in pool design:

- pool outlet should be maintained by a push bar or weir at the appropriate elevation and flow capacity. The design water elevation should enable fish entry into the culvert by backing water through the pipe to adequate depth (no in-pipe weirs) or at least to the first in-pipe weir.
- pool should be stabilized to prevent scour and erosion. The pool outlet structure elevations should be secure so as to maintain desired hydraulic performance.
- use of riprap should be minimized and concentrated on protecting the culvert inlet and outlet and pond outlet structure. The banks may also be protected at the discretion of design and environmental staff, typically in the range of the Q_2 to Q_5 stages. Although riprap should generally not be placed in the pool bottom, riprap should be placed from the culvert outlet to the pool bottom.
- pool width should be at least 2 times the culvert span.
- pool length should be at least 3 times the culvert span.
- for single barrel installations only, the culvert and pool centerlines should align.
- pool should be at least 3 ft (0.9 m) deep at the design passage flow.
- Consideration should be given to placing at least three boulders in a triangular pattern in order to create fish resting areas. The boulders should be approximately 3 ft (0.9 m) in diameter (or 2.5 ft (0.75 m) diameter for culvert $D \leq 5$ ft (1.5 m))
- pool outlet structure (push bar, weir or channel) should be designed for hydraulic consistency with in-culvert weirs and to develop needed backwater at culvert exit.
- voids in outlet riprap should be filled with smaller rock to prevent underflow and throughflow.
- if pool does not back water into culvert for the design period, check that pool Energy Dissipation Factor (EDF) is no greater than 5 ft-lb/ft^3/s (= 250 J/s/m^3)
- culvert inlet and outlet should be sealed to prevent underflow.

Scour pools, either natural or constructed, will often be found at existing culverts. Maine DOT general practice will be to retain these pools when such pipes are replaced while taking measures to eliminate or reduce any outlet drops that may have developed. In the case of new culvert locations, the decision to construct outlet pools will be taken on a case-by-case basis, as they may be undesirable in particular circumstances, particularly if predation of resting fish is expected. Also, right-of-way complications may limit the space available for outlet pools.

Hydrology and Design Flows for Fish Passage

The passage design flow depends on the time of year for passage, which in turn depends on the species of interest. In general, fish are moving from April through June and September through October; the low-flow months of high summer are periods of lower activity. Final determination of design movement periods should be based on Table 2 in the Fish Passage Policy and consultation with Maine DOT Environmental Office staff and the several resource agencies. Design flows will have to be assigned on a case-by-case basis, since they are dependent on both watershed and passage period (which depends on species of interest; see Table 2 in the Fish Passage Policy.

The design flows may be determined by several different methods:

1) site inspection, channel geometry measurements, and flow measurement during periods of fish movement
2) hydraulic calculation from channel geometry measurements and specified or known flow depths for fish passage
3) estimation by USGS regression equations for monthly median flows (Dudley, 2004; Appendix 2A)
4) correlation to similar gaged watersheds

When using the equations for median monthly flows, the estimates for September and October are significantly lower than for April though June. Therefore, using the average of the September and October medians should produce a conservative design that also maintains needed depths during the late spring, higher flow months. The median flow regression equations are tabulated in Appendix 2A; easy-to-use look-up charts are also given for May, June, September, October, and the September-October average.

Method (1) is the single best method but it may not always be possible to collect data during fish passage periods. Except for winter months, data for method (2) can always be collected and therefore hydraulic estimation should be performed in most cases. Method (3), regression calculation, should always be carried out because it does not require any field work and only requires data from paper maps or available as GIS coverage from the Maine State GIS Internet web site. Ideally, at least one field-based flow estimate should be prepared along with the regression estimate. Method (4), also statistical in nature, requires a broader understanding of watershed hydrology but allows for a more comprehensive hydrology/hydraulics evaluation.

In support of establishing good measurement-based flow estimates, some sites may warrant installation of a simple staff gage as soon as possible after the need for fish passage has been established. This will allow for efficient collection of stage data during various flow conditions. Furthermore, final designs for sensitive sites may also include provision for a staff gage so that performance of the new or rehabilitated culvert can be evaluated.

Strictly speaking, the target flow for fish passage design should be species-dependent. Ultimately, the species type, age, direction of movement, and month(s) of movement should all indicate the flow or multiple flow values that will govern the design for fish passage. This information is summarized in Table 2 of the Fish Passage Policy. As a practical matter, this approach complicates a design process which invariably occurs within a context of sharply limited alternatives. Maine DOT therefore recommends that in the absence of site-specific data, it is sufficient to execute design on the basis of the average of the September and October median monthly flows. This value is close to the lowest baseflow value of the year; if adequate depth is obtained this with flow then higher depths will be obtained for the remainder of the year.

Only a handful of species move upstream to spawn during springtime higher velocities. If one of these species is known to be of interest, then the culvert should be designed for the species-specific period and flow.

Salmon are of particular interest and August has been identified as a period of salmon movement. Therefore, August is the designated low flow design period for salmon. Also, according to the regression equations, August is the lowest average monthly flow.

New and Replacement Culverts: Hydraulic Geometry Matching

Designing new and replacement culverts for fish passage is generally simpler than retrofitting existing pipes. The following guidelines should be followed:

1) Employ corrugated elliptical pipe arches with the largest feasible corrugations whenever possible to maximize roughness
2) Embed pipe: for nominal diameter (or rise) $D < 48$ in (1200 mm), embed pipe invert 6 in (150 mm) in stream bed; $D \geq 48$ in (1200 mm), embed pipe invert 12 in (300 mm); allow embedded pipe to fill with natural substrate
3) If outlet pool is present, check that pool push bar creates at least 6 inch (150 mm) water depth through pipe.
4) Match pipe and stream flow geometry: flow depth and width in the pipe at bankfull flow (approximately 1.5-year return period) should approximate depth and width in the stream
5) Place pipe with zero slope, or as nearly flat as possible, not to exceed 3%
6) Size pipe for peak flow: pass the 50-year flood (100-year for $D \geq 10$ ft (3000 mm)), accounting for the capacity lost to embedding
7) Check fish passage performance: perform hydraulic analysis for depth and velocity during fish passage flows; irregular cross-section flow area (due to embedding and elliptical section) should be accounted for.

The new culvert should not constrict flow at the inlet over the range of design flows, as this will increase flow velocity and attendant kinetic energy complications. If a constriction cannot be avoided, then in-culvert weirs for water level control should be investigated.

Figure 2.1 shows an embedded circular pipe along with equations in Table 2.1 for calculating basic geometric quantities. Table 2.2 gives equations for embedded pipe arches; Table 2.3 gives corresponding tabulated values. Note that current practice does require that embedded pipes be filled with substrate to the stream channel bottom elevation; it is assumed that the embedded pipe will fill / empty naturally over time.

Figure 2.1: Embedded Circular Pipe

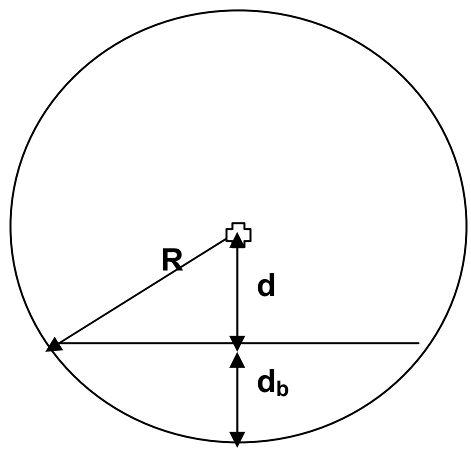


Table 2.1: Equations for Embedded Circular Pipe Geometry

Radius; diameter; embedded depth	R; $D = 2R$; d_b
Distance from bed to pipe center	$d = R - d_b$
Bottom embedded width	$w_b = 2\{d_b(D-d_b)\}^{1/2}$
Embedded Area	$A_b = R^2\cos^{-1}[(R-d_b)/R] - dw_b/2$
Open Area	$A_o = \pi R^2 - A_b$
Embedded Perimeter	$P_b = D\cos^{-1}[(R-d_b)/R]$
Open Perimeter	$P_o = \pi D - P_b$

These equations can be used to approximate elliptical pipes, with pipe rise substituted for diameter. More exact results for elliptical pipes can be calculated with the following equation:

$$A = b\,(\text{pipe rise})^a$$

The coefficients a and b are given in Table 2.2. Note that two sets of coefficients are given, for corner radii of 18 in (457 mm) and 31 in (787 mm). These coefficients were developed by regression analysis from the exact tabulated areas in Tables 3a and 3b, respectively. The tables can be used in place of the equations.

Table 2.2: Function Coefficients for Open Area in Embedded Pipe Arch

Corner Radius		Depth of Embedment			
		0 in (0 mm)	6 in (150 mm)	9 in (225 mm)	12 in (300 mm)
18 in	a	2.246	2.316	2.371	2.428
	b	0.743	0.613	0.530	0.453
31 in	a	2.260	2.291	2.320	2.351
	b	0.631	0.571	0.524	0.475
457 mm	a	2.246	2.316	2.371	2.428
	b	0.995	0.893	0.823	0.752
787 mm	a	2.260	2.291	2.320	2.351
	b	0.859	0.807	0.766	0.721
Equation: open area $A = b \times (\text{pipe rise})^a$, in (ft, ft^2) and (m, m^2)					

Table 2.3a
OPEN AREA IN EMBEDDED PIPE ARCH (U.S. Customary)

	Span (ft)	Rise (ft)	Open Area (ft^2)			
			Depth of Embedding (in)			
			0 in	6 in	9 in	12 in
Corner Radius = 18 in	6.08	4.58	22.03	19.95	18.64	17.24
	6.33	4.75	24.00	22.17	20.83	19.37
	6.75	4.92	26.17	24.47	23.06	21.54
	7.00	5.08	28.29	26.36	24.88	23.29
	7.25	5.25	30.53	28.38	26.82	25.15
	7.67	5.42	32.94	30.94	29.34	27.60
	7.92	5.58	35.23	33.01	31.32	29.51
	8.17	5.75	37.70	35.20	33.41	31.51
	8.58	5.92	40.27	38.01	36.27	34.27
	8.83	6.08	42.87	40.34	38.44	36.40
	9.33	6.25	45.78	43.48	41.59	39.50
	9.50	6.42	48.44	46.02	43.89	41.72
	9.75	6.58	51.29	48.42	46.29	44.02
	10.25	6.75	54.32	51.82	49.74	47.43
	10.67	6.92	57.48	55.11	52.96	51.00
	10.92	7.08	60.61	58.04	55.90	53.49
	11.42	7.25	64.01	61.61	59.61	57.25
	11.58	7.42	67.08	64.49	62.24	59.83
	11.83	7.58	70.40	67.59	65.24	62.61
	12.33	7.75	74.09	71.47	69.30	66.73
	12.50	7.92	77.40	74.58	72.15	69.51
	12.67	8.08	80.93	77.85	75.59	72.39
	12.83	8.33	85.48	82.07	79.33	76.38
	13.42	8.42	88.44	85.39	82.84	79.89
	13.92	8.58	92.52	89.67	87.30	84.50
	14.08	8.75	96.25	93.19	90.55	87.65
	14.25	8.92	100.07	96.76	84.16	90.84
	14.83	9.08	104.57	101.50	98.95	96.21
	15.33	9.25	108.90	106.02	103.61	100.77

	Span (m)	Rise (m)	Open Area (ft^2)			
			Depth of Embedding (in)			
			0 in	6 in	9 in	12 in
	15.50	9.42	112.93	109.86	107.30	104.28
	15.67	9.58	117.09	113.81	111.08	105.54
	15.83	9.83	122.64	119.11	116.17	112.73
	16.42	9.92	126.19	122.91	120.18	116.96
	16.58	10.08	130.55	127.05	124.13	120.68
Corner Radius = 31 in	13.25	9.33	97.69	95.03	92.68	90.27
	13.50	9.50	101.79	98.94	96.58	93.90
	14.00	9.67	106.29	103.59	101.34	98.70
	14.17	9.83	110.24	107.38	104.96	102.24
	14.42	10.00	114.53	111.46	108.91	106.01
	14.92	10.17	119.28	116.39	113.98	111.14
	15.33	10.33	123.84	121.07	118.76	116.05
	15.58	10.50	128.39	125.47	123.03	120.17
	15.83	10.67	133.08	129.89	127.23	124.10
	16.25	10.83	137.80	134.85	132.39	129.51
	16.50	11.00	142.60	139.49	136.89	133.86
	17.00	11.17	147.81	144.67	142.06	138.99
	17.17	11.33	150.80	147.65	145.03	141.94
	17.42	11.50	157.56	154.24	151.47	148.22
	17.92	11.67	163.02	159.86	157.23	154.12
	18.08	11.83	167.92	164.60	161.83	158.56
	18.58	12.00	173.54	170.36	167.71	164.58
	18.75	12.17	178.64	175.30	172.52	169.23
	19.25	12.33	184.47	181.25	178.57	175.42
	19.50	12.50	190.01	186.63	183.83	180.52
	19.67	12.67	195.37	191.82	188.91	185.44
	19.92	12.83	201.11	197.39	194.29	190.63
	20.42	13.00	207.17	203.64	200.69	197.21
	20.58	13.17	212.72	209.00	205.91	202.25

Table 2.3b
OPEN AREA IN EMBEDDED PIPE ARCH (metric)

Corner Radius	Span (m)	Rise (m)	Open Area (m^2) Depth of Embedding (mm)			
			0 mm	150 mm	225 mm	300 mm
Corner Radius = 457 mm	1.855	1.397	2.048	1.854	1.733	1.602
	1.931	1.448	2.231	2.061	1.936	1.800
	2.058	1.499	2.433	2.275	2.143	2.002
	2.134	1.550	2.630	2.450	2.313	2.165
	2.210	1.601	2.838	2.638	2.493	2.338
	2.337	1.651	3.062	2.876	2.727	2.565
	2.414	1.702	3.275	3.068	2.911	2.743
	2.490	1.753	3.504	3.272	3.105	2.929
	2.617	1.804	3.743	3.533	3.371	3.185
	2.693	1.855	3.985	3.750	3.573	3.383
	2.846	1.905	4.255	4.041	3.866	3.672
	2.896	1.956	4.503	4.278	4.080	3.878
	2.973	2.007	4.767	4.501	4.303	4.092
	3.125	2.058	5.049	4.817	4.623	4.409
	3.252	2.109	5.343	5.123	4.923	4.740
	3.328	2.160	5.634	5.395	5.196	4.972
	3.481	2.210	5.950	5.727	5.541	5.321
	3.532	2.261	6.235	5.994	5.785	5.561
	3.608	2.312	6.544	6.283	6.064	5.820
	3.760	2.363	6.887	6.643	6.441	6.203
	3.811	2.414	7.194	6.932	6.706	6.461
	3.862	2.464	7.522	7.236	7.026	6.729
	3.913	2.541	7.945	7.628	7.374	7.100
	4.090	2.566	8.221	7.937	7.700	7.426
	4.243	2.617	8.600	8.335	8.115	7.854
	4.294	2.668	8.946	8.662	8.417	8.147
	4.345	2.718	9.302	8.994	7.823	8.444
	4.522	2.769	9.720	9.434	9.197	8.943
	4.675	2.820	10.122	9.855	9.631	9.367

Corner Radius	Span (m)	Rise (m)	Open Area (m^2) Depth of Embedding (mm)			
			0 mm	150 mm	225 mm	300 mm
	4.726	2.871	10.497	10.212	9.974	9.693
	4.776	2.922	10.884	10.579	10.325	9.810
	4.827	2.998	11.399	11.071	10.798	10.478
	5.005	3.023	11.729	11.425	11.171	10.872
	5.056	3.074	12.135	11.809	11.538	11.217
Corner Radius = 787 mm	4.040	2.846	9.080	8.833	8.615	8.391
	4.116	2.896	9.461	9.197	8.977	8.728
	4.268	2.947	9.880	9.629	9.420	9.174
	4.319	2.998	10.247	9.981	9.756	9.503
	4.395	3.049	10.646	10.360	10.123	9.854
	4.548	3.100	11.087	10.819	10.595	10.331
	4.675	3.150	11.511	11.254	11.039	10.787
	4.751	3.201	11.934	11.663	11.436	11.170
	4.827	3.252	12.370	12.073	11.826	11.535
	4.954	3.303	12.809	12.534	12.306	12.038
	5.030	3.354	13.255	12.966	12.724	12.442
	5.183	3.404	13.739	13.447	13.205	12.919
	5.234	3.455	14.017	13.724	13.481	13.193
	5.310	3.506	14.645	14.337	14.079	13.777
	5.462	3.557	15.153	14.859	14.615	14.326
	5.513	3.608	15.608	15.300	15.042	14.738
	5.666	3.659	16.131	15.835	15.589	15.298
	5.716	3.709	16.605	16.294	16.036	15.730
	5.869	3.760	17.147	16.847	16.598	16.305
	5.945	3.811	17.662	17.347	17.087	16.779
	5.996	3.862	18.160	17.830	17.559	17.237
	6.072	3.913	18.693	18.348	18.059	17.719
	6.225	3.963	19.257	18.928	18.654	18.331
	6.275	4.014	19.772	19.427	19.139	18.799

Steeply Sloped Streams

This approach of matching pipe flow and depth to the natural stream works best with gentle slopes. Steeply sloped streams (slope S > 3%) require extra care and will likely require mitigation (e.g., weirs or baffles). Embedding pipes to below natural stream bed elevation may inadvertently allow headcutting to propagate upstream of the culvert inlet. Therefore, pipes should be placed on the natural stream bottom when slope exceeds 3%. Hydraulic analysis will likely indicate the need for in-pipe grade control in order to maintain adequate water depths. Downstream control may also be needed.

Rehabilitated Culverts - Corrective Measures

Existing culverts can be rehabilitated by slip lining and by invert lining. However, linings may reduce both cross-sectional flow area and surface roughness, with a possible net effect of decreasing flow depth and/or increasing flow velocity. (Corrugated aluminum structures used to line larger (typically > 10 ft diameter) culverts have essentially the same roughness as the original corrugated steel and thus do not markedly increase velocity.) The simplest approach to maintaining fish passage is to install a new culvert designed for consistency with the prevailing stream hydraulic geometry, though budgetary and other constraints may argue against replacement. If the culvert is on an identified fishery, then design measures may need to be taken in order to insure fish passage under specified conditions.

When selecting a passage mitigation measure, the first step is to determine if the lined culvert will be a barrier to passage by appropriate hydraulic and hydrologic analysis. Target design flows are chosen according to guidelines presented here and in the companion Maine DOT Fish Passage Policy volume (2007a). Then the lined pipe is evaluated for acceptable depth and velocity, according to the target species. In general, if downstream control on shallow water depths does not previously exist, then mitigation measures are likely necessary.

When a pipe is lined, the invert is raised by approximately 5 in (125 mm) due to the concrete or plastic lining. This may create a slightly hanging invert or a drop too great for fish to pass over. This effect is separate from the hydraulic aspects of depth and velocity. A sluice channel in the outlet, combined with one or more in-pipe weirs, can be employed to eliminate this drop. Alternatively, downstream external weirs can also be used, though right-of-way complications may eliminate this option.

Culvert hydraulic analysis can be performed with software such as HY8, FishXing or equivalent proprietary software for the design flows and incorporating tailwater conditions as determined by site inspection. If flow depth is too shallow or velocity too high, the following general measures suggest themselves for increasing depth. Useful countermeasures include

- tailwater control structures (weirs) installed downstream
- weirs installed in the culvert

When considering corrective measures, the first choice should be simple downstream weirs. Downstream weirs are particularly useful if a perched outlet is the major problem. Depending on the severity of the perch, more than one weir may be needed. As noted, right-of-way limitations may rule out this option or require an elaborate outlet fishway. Downstream weirs may also be useful for maintaining adequate water depths in culverts that are not too steep. External weirs offer advantages in construction and maintenance over other available measures. Plunge pools should follow the guidelines given above.

When the lining-induced drop is not too great, and if downstream weirs are not an option, a simple cutout notched sluice channel in the bottom of the culvert and extending up into the culvert may provide adequate water depth. However, by itself, this cutout channel is usually not adequate. Some potential problems include high velocity within the channel and inadequate depth above the termination of the cutout. In most cases such an outlet will need to be combined with grade control, within the pipe, downstream, or both.

In steeper pipes, in-culvert grade control achieved with simple pool-and-weir sequences should be considered. This approach is limited to larger pipes (D > 5 ft (1500 mm) minimum, and preferably D > 6 ft (1800 mm)). These measures will now be discussed in more detail.

Culvert End Treatments for Fish Passage – Cutouts or Notched Outlets

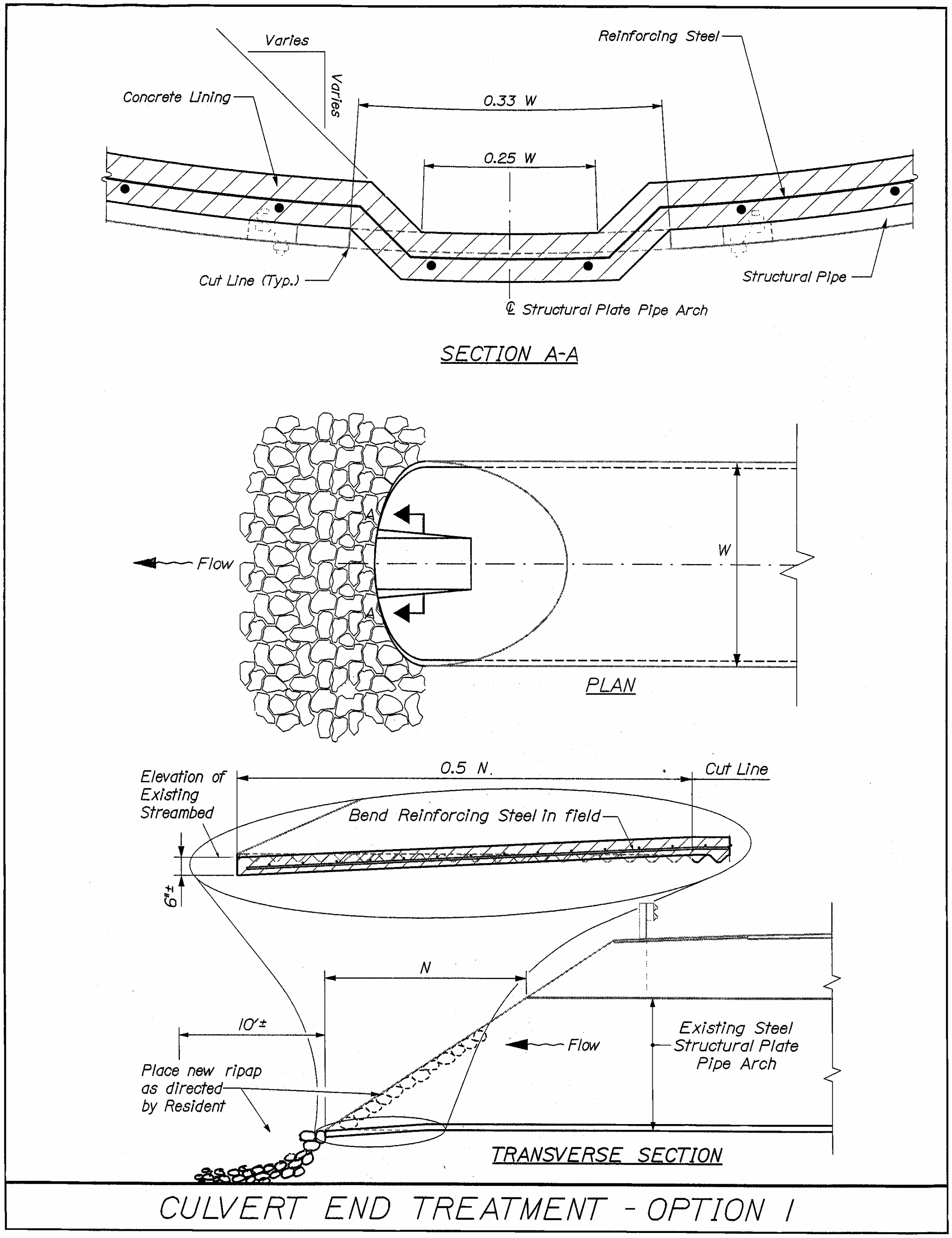
A culvert lining raises the outlet invert. If the induced jump is modest, it can be mitigated by building a ramped notch (cutout or sluice channel) into the culvert bottom. The outlet notch invert is at stream grade, providing a continuous stream/culvert bottom elevation. The channel returns to the prevailing culvert invert elevation some distance into the culvert.

Typical details for end treatment options are shown in Figure 2.2. This treatment includes a riprap apron to provide a smooth transition from stream bed to the pipe edge. The notched channel should be sized to run full at low flow.

This treatment is used primarily to eliminate hanging inverts. End treatments by themselves will not correct excessive velocities or inadequate depths farther up the culvert. Therefore, they will probably be used with in-culvert grade control. Hydraulic analysis should be performed to check that:

1) adequate flow depth is achieved throughout the pipe
2) velocity standard is not exceeded in pipe and notch channel

Figure 2.2: End Treatment to Eliminate Drop



Downstream Grade Control Structures (Weirs)

Downstream weirs are used to establish grade control, i.e., to back water up into the culvert to the needed depth. It may be possible to maintain adequate depth and velocity solely with external weirs. In a sloping culvert, the minimum depth must be achieved at the culvert inlet. This depth and location helps to fix the design parameters of the downstream weirs; the design flow completes the determination of the weir parameters. Specific weir dimensions and their calculation are discussed in detail for in-culvert weirs.

Drops in water level are created at weirs and this drop may itself constitute a barrier to passage. The drop at any particular weir should ordinarily be limited to 8 in (200 mm) or a species-specific value in order to allow for passage over the weir, and the weir notch should generally be submerged 4 in (100 mm) on the downstream side. Thus, several weirs in series may be needed to create the needed tailwater elevation. The distance between weirs should be about 150% of the stream width in smaller streams, with a target minimum spacing of 16.5 ft (5 m), up to 33 ft (10 m) in larger streams. Actual spacing depends on stream slope. For reasons of cost and downstream impact, the number of structures should be kept to a minimum.

A cost-effective approach to weir construction is to employ standard concrete barrier (e.g., Jersey barrier) sections. Standard Maine DOT weir dimensions are used and notch width is calculated as detailed elsewhere in this report.

When esthetic considerations are important, weirs can be constructed of natural materials, e.g., logs on a stone foundation in smaller streams; weirs on larger streams may be constructed of rock. The simplest weir extends straight across the stream; an alternative plan form is V-shaped, pointing upstream. The log ends should be anchored to stone or block on the stream bank and keyed into the bank. The banks in the vicinity of the log ends should be riprapped to prevent scour and channel migration at higher flow. The foundation stones should be sized to withstand the 100-year flood and wrapped in geotextile so that they stand as a unit. The wrap also seals the log structure and forces more of the water over the weir or through the spillway, rather than between the logs. The weir face can be stacked vertically or angled downstream; angling creates quiescent water beneath the crest where fish can rest. The weir should be square-notched, according to the idea that fish will be attracted to and pass through the water spilling through the notch. The notch should be sized to flow full at the design passage flow using methods described below. Details for a log weir (grade control) structure (i.e., weir) are shown in Figures 2.3 and 2.4.

The use of downstream grade control will require stream bank protection and anticipation of flow around the ends of the structure. The natural stream banks should be at least 6" – 12" above the top of the weir. It is essential that any grade control structure be keyed into the stream bed and banks so as to prevent flow around and under the structure.

External weirs can create access and right-of-way issues, especially when a series of weirs is needed to obtain the necessary tailwater. With typical inter-weir spacing of 10 ft – 16.5 ft (3 m – 5m), several weirs will probably extend beyond existing right-of-way and thus may not be a practical solution. If additional drainage easement cannot be obtained, in-culvert weirs should be considered for larger pipes ($D \geq 5$ ft (1500 mm)). Alernatively, a compact fishway can be constructed at the outlet, permitting fish to surmount the overhang and enter the culvert. These outlet structures can be prefabricated (e.g., Steeppass fishway) or custom designed. Another advantage of this approach is that the pipe hang does not have to be eliminated, allowing for the pipe itself to be less steep and avoiding the need for additional excavation and possibly blasting. Even so, in-pipe weirs may still be needed. A simple Jersey barrier structure is much easier to design and build, though it can overcome only the smaller pipe hangs.

Figure 2.3: Log Drop Control Structure

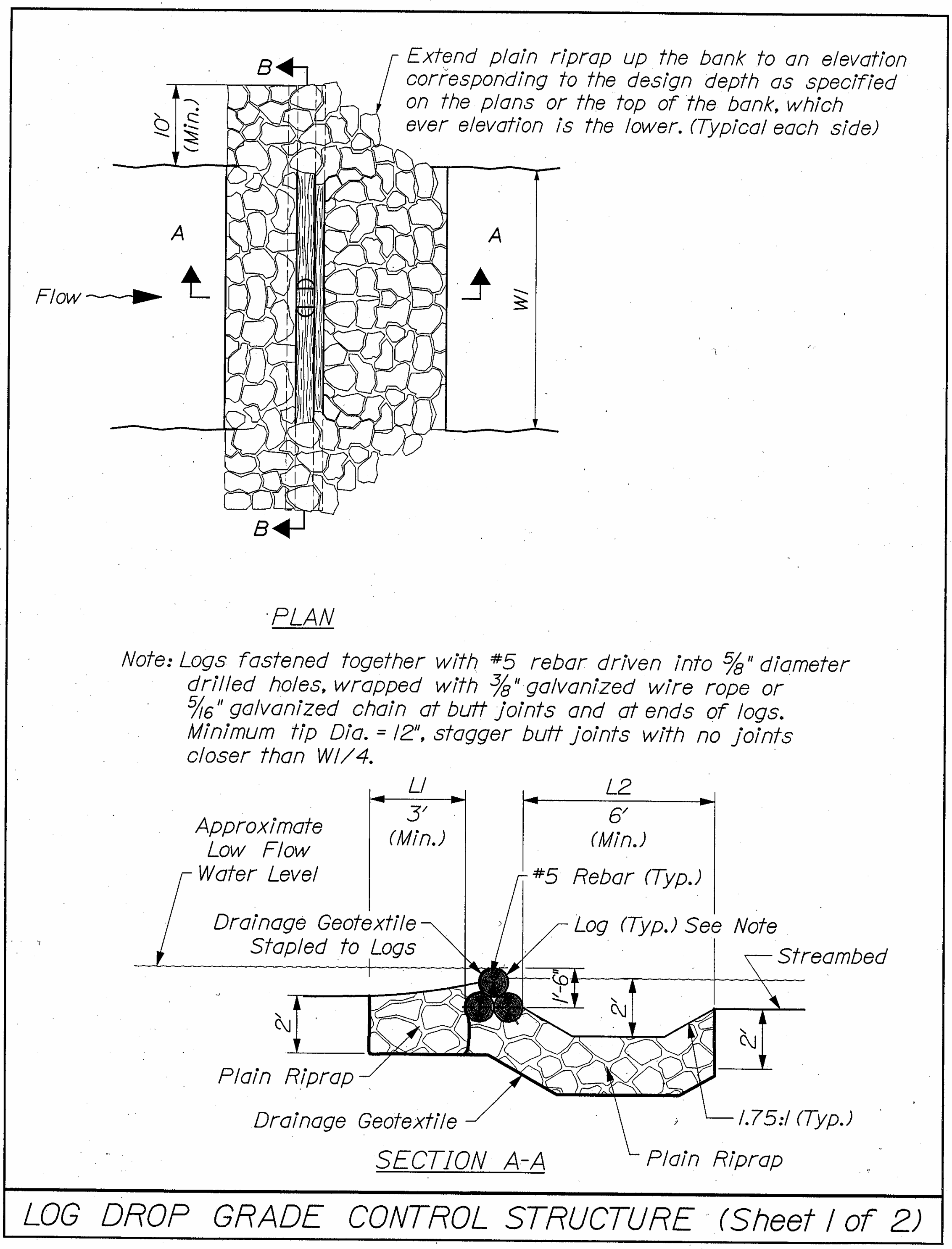
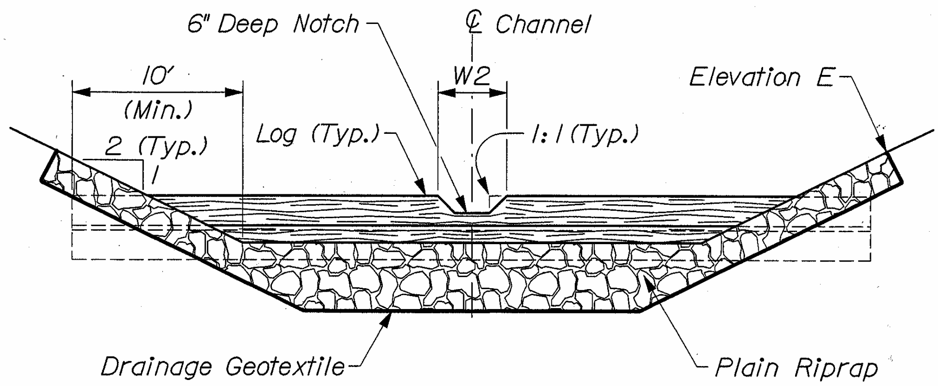


Figure 2.4: Log Drop Control Structure (cont.)



SECTION B-B

NOTE: 1.) Channel Width (W1) = as specified on the Plans

2.) Notch Width (W2) = as specified on the Plans

3.) Upstream Length (L1) = as specified on the Plans

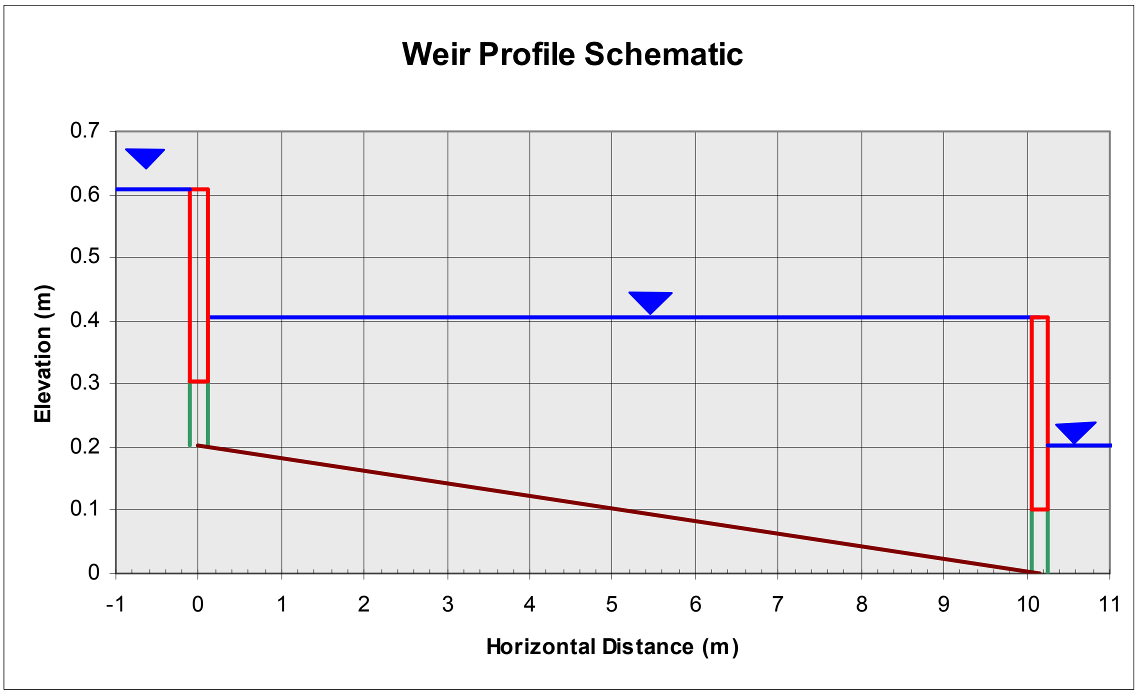
4.) Downstream Length (L2) = as specified on the Plans

5.) Top of Riprap Elevation (E) = as specified on the Plans

LOG DROP GRADE CONTROL STRUCTURE (Sheet 2 of 2)

In-Culvert Grade Control: Culverts with Weirs

Weirs are added to the interior of a culvert to create adequate water depths at low flows and limit regions of high velocity. They create a series of pools inside the culvert, the effect being increased water depth and reduced velocity to permit fish to move up through the pipe. These pools also have the effect of providing resting areas in long culverts. Such a modified culvert constitutes a type of "weir and pool" fishway. Maine DOT will use rectangular notched weirs in these situations. Due to constructability issues, in-culvert weirs are limited to larger culverts (D generally ≥ 5 ft or 1500 mm).



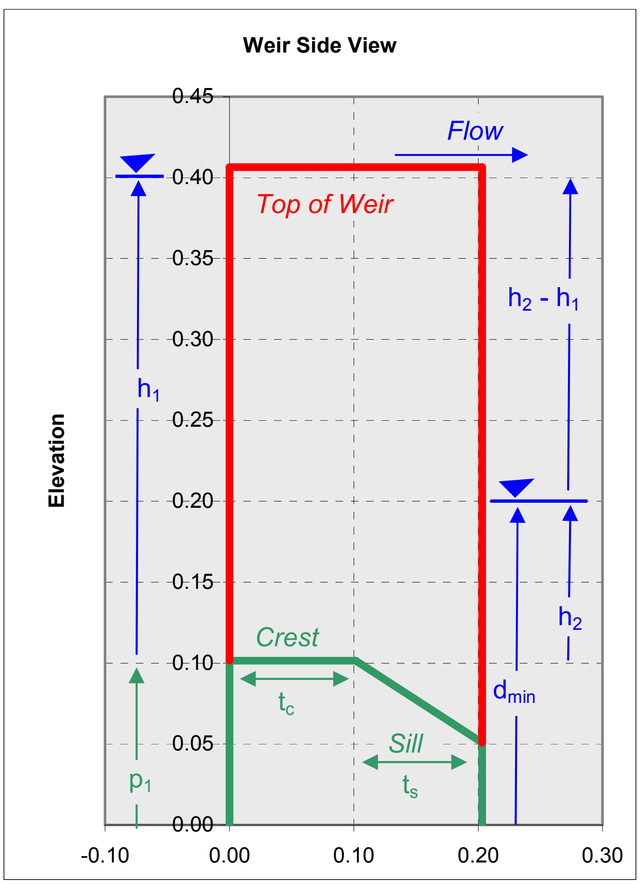
Weir Design

The objective in weir design is to pass the specified design flow while maintaining the necessary depth of water behind the weir. The shallowest depth in a weir-pool sequence in a culvert of simple uniform slope is at the downstream base of a weir. Most weir dimensions will be specified as design standards, leaving the inter-weir spacing and weir notch width as the principal parameters to be determined according to specific site topographic and hydrologic conditions and species requirements. The inter-weir spacing will typically be determined by the culvert slope and the specified drop in pool elevation. The notch width is a function of the design flow and the other specified weir dimensions.

Weir Specifications

A schematic of a section across the weir is shown below with dimensions indicated; a frontal view is given on the following page. The "invert" is synonymous with the "notch invert" or "notch crest". Most weir dimensions will be standardized as listed here. The following specifications should be observed, unless the design flow, pipe size, or construction issues indicate otherwise.

- Notch shall be at least 12 in (300 mm) deep (h_1), from top (crest) of weir to notch invert
- Notch shall be submerged by 4 in (100 mm) in the downstream pool to enable passage by non-jumping fish (h_2)
- Drop between pool elevation across weir shall be 8 in (200 mm) ($h_1 – h_2$)
- Total weir thickness ($t_c + t_s$) shall be at least 12 in (300 mm)
- Notch invert shall be at least 4 in (100 mm) thick (t_c)
- Beveled sill shall be at least 4 in (100 mm) thick (t_s)
- Notch shall be rectangular, beveled in the downstream direction with a sill slope (H:V) = (2:1)
- Distance from notch invert to culvert invert be at least 4 in (100 mm) (p_1)

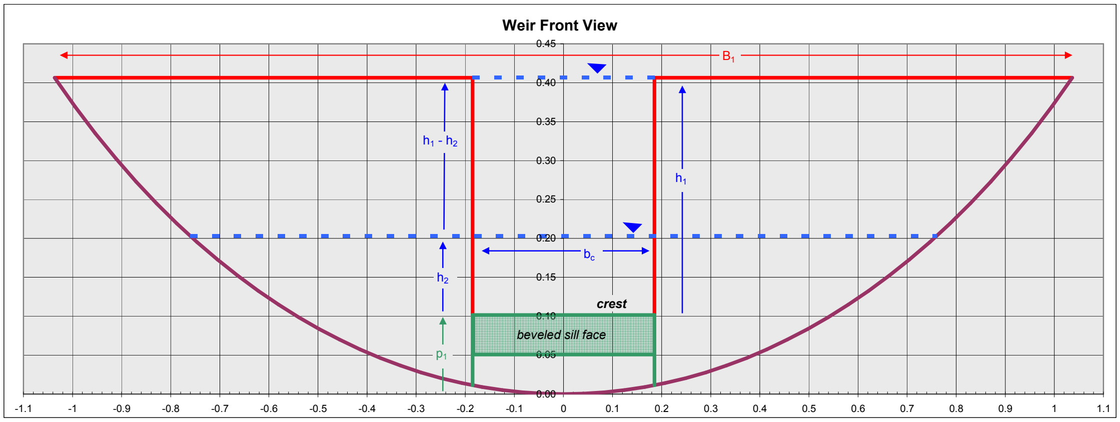


Required Depth of Water

Strictly speaking, the required depth of water depends on the species of interest and time of movement. In the interest of simplifying the design process, Maine DOT will generally use a design depth of 8 in (200 mm) at the shallowest point in a pool between weirs. A particular situation may warrant using a different value, based on the fish data in Table 2 of the Fish Passage Policy.

Drop Between Pools

The drop ($h_1 – h_2$) in water surface elevation between pools should be set according to the species of interest, depending on the ability of a fish to jump between pools. In the interests of developing a robust design suitable for a variety of species, Maine DOT will design for an 8 in (200 mm) drop between pool elevations with the notch submerged, unless particular circumstances suggest otherwise (salmon are capable of navigating 12 in drops). Since the weirs are dimensioned to be partially submerged at the design flow, both jumping and non-jumping species should be able to navigate the weir notch. Table 2 of the Fish Passage Policy provides the detailed information useful for alternative individual design standards.



<u>Inter-Weir Spacing</u>

Spacing between weirs depends on the culvert slope and the specified drop between water pools across weirs. In general, the maximum spacing is calculated according to the simple geometric relationship

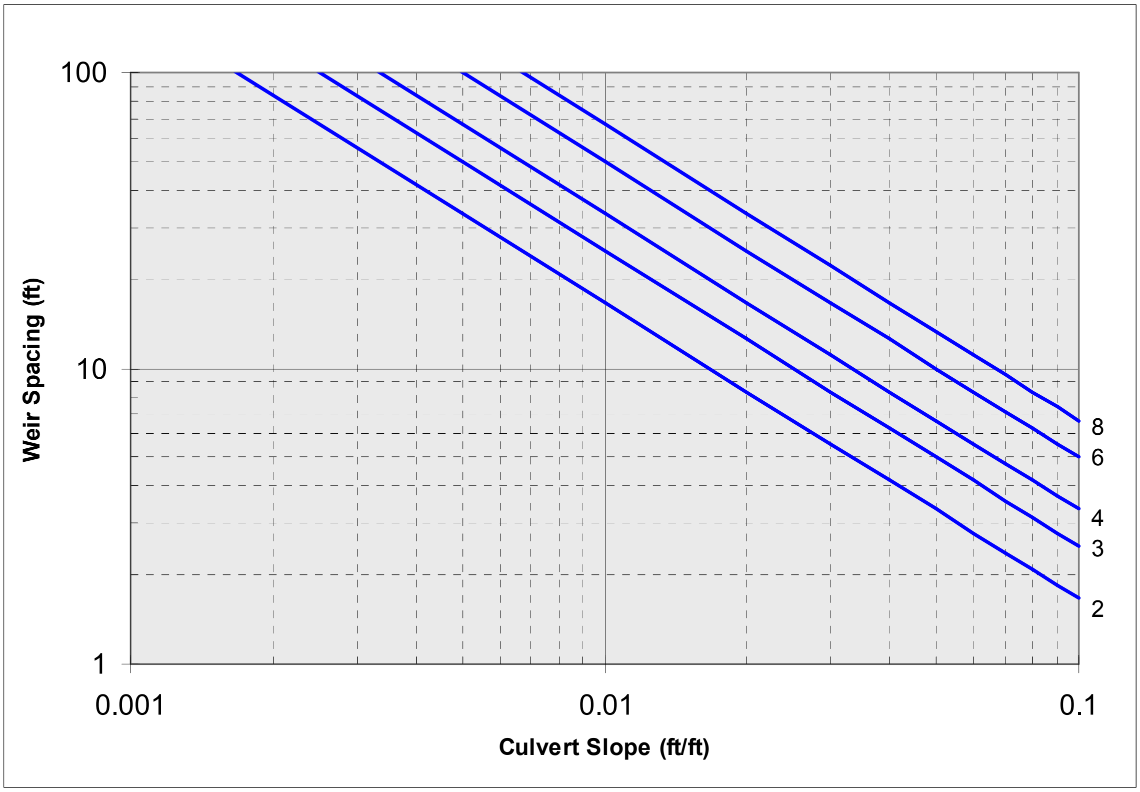
$$L_W = \Delta h / S$$

where L_W = nominal spacing between weirs = pool length
Δh = drop in water surface elevation between pools
S = culvert slope

This simple function is presented graphically for several commonly used pool drops Δh. The calculated inter-weir spacing should be interpreted as the maximum allowable spacing. The actual final design spacing may be something less than the nominal calculated value; other design and habitat issues may indicate a smaller value as being more appropriate. When concrete pipe sections with prefabricated weir units are used, select a combination of sections that will give the largest weir spacing that does not exceed the calculated value. The weir and crest elevations should be checked when something other than the initial calculated spacing is elected. The first weir should be placed at the culvert outlet.

For steeper culverts, more weirs are required at closer spacing as illustrated in the figure below. The minimum in-culvert inter-weir spacing acceptable for construction is 6 ft (1.8 m), though spacings this small indicate that alternative approaches may be more appropriate. At close spacings, the weirs function more as baffles and roughness elements as opposed to impoundment structures. The pool volumes are correspondingly smaller and EDF limitations may not be satisfied. Therefore, on steeper culverts that require closely spaced weirs, consideration should be given to using alternative approaches such as true baffle designs (preferably vertical slot weirs) instead of nominal pool-and-weir configurations.

Inter-Weir Spacing for Selected Pool Drops (in inches)



Weir Notch Width Calculation

The weir notch depth h_1 is fixed by the specified crest submergence h_2 (usually 4 in or 100 mm) and the pool drop ($h_1 – h_2$; usually 8 in or 200 mm). This leaves the notch width b_c as the weir parameter designed to accommodate the fish passage flow. The notch width is calculated using the Kindsvater-Carter (K-C) sharp-crested weir equation (in dimensionally consistent form):

$$b_c = \{Q/r_s\}/\{C_e(2/3)(2g)^{1/2}h_1^{3/2}\}$$

where

Q = flow passed by freely flowing (i.e., not submerged) weir (ft^3/s or m^3/s)
b_c = notch width (ft or m)
C_e = effective discharge coefficient

(dimensionless; 0.6 as a first approximation)

g = acceleration due to gravity (32.2 ft/s^2 or 9.8 m/s^2)

h_1 = upstream water surface elevation referenced to crest elevation (ft or m)

This version omits several correction factors but is acceptable given the numerous uncertainties in real applications. A full development of the K-C equation, including corrections, is given in Appendix 2B. Computation worksheets for the complete K-C equation are provided in Appendix 2C.

The fish pass weirs will generally be designed to flow partially submerged at design discharges, in order to pass both jumping and non-jumping species. A submerged weir will pass less water than a freely flowing weir, all other things being equal. Therefore, a weir designed for submerged flow must have a larger notch opening to accomodate the design passage flow. The submergence correction factor r_s is determined following the method of Villemonte:

$$r_s = \{1 - (h_2/h_1)^{3/2}\}^{0.385} = (Q/Q_{free}) \qquad \leq 1$$

where h_1 and h_2 are the respective upstream and downstream pool elevations above the weir crest, Q is the actual flow expected (by hydrology/hydraulics analysis), and Q_{free} is the flow through a freely discharging weir of the same dimensions. Maine DOT in-culvert weirs will usually be designed with 4 inch submergence (h_2 = 4 in or 100 mm). The effect of partial submergence is to reduce the flow over the weir. Therefore, the nominal design free flow must be increased over the actual hydrologic flow needed over the weir:

$$Q_{free} = Q/r_s$$

The weir is sized according to Q_{free} (= Q/r_s); the actual flow Q is chosen according to watershed hydrology and the flows prevailing during periods of fish movement.

<u>Design Procedure</u>

The design procedure for in-culvert weirs is fairly simple and consists of five steps:

1. estimate a design flow Q according to watershed hydrology and/or channel hydraulics and target species period of movement. If not performing a detailed channel-specific or species-specific analysis, use the average of the September and October median flows (see Appendix 2A).
2. calculate the nominal distance between weirs based on culvert slope and drop in water surface elevation between weirs. Set final spacing according to constructability requirements so as not to exceed nominal calculated value.
3. assign weir dimensions and auxiliary hydraulic design parameters. Use the values given under "Weir Specifications" above as starting values; they may have to be revised in the process of developing a final design.

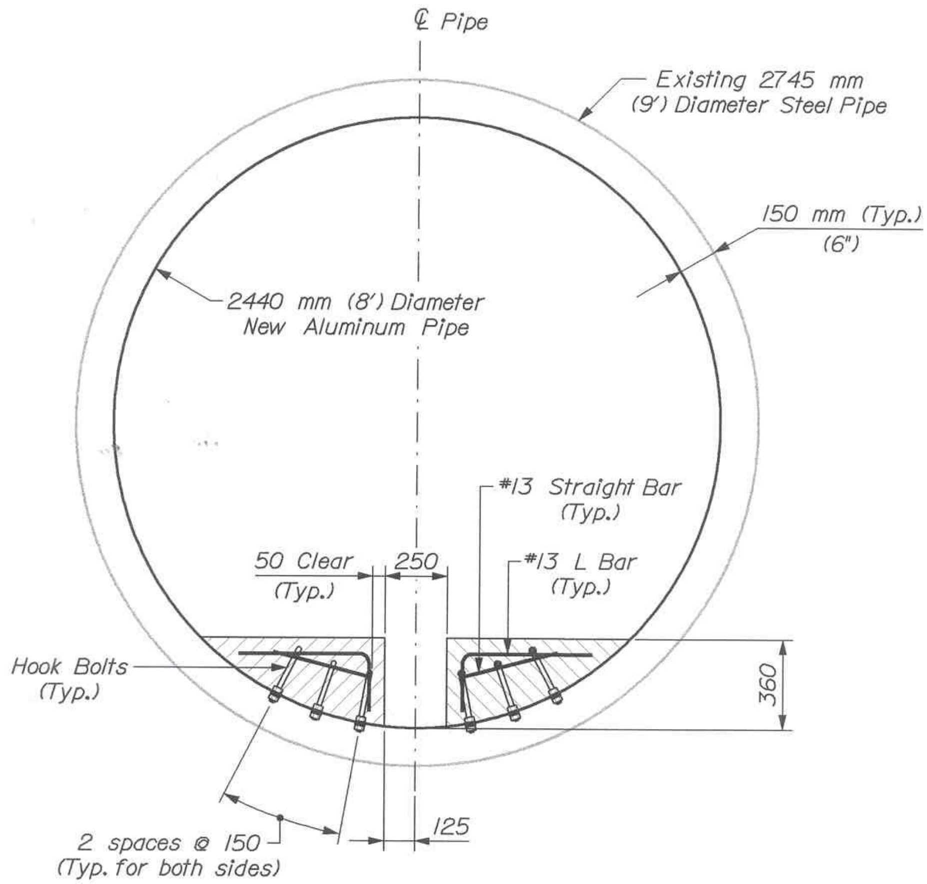
4. calculate nominal weir notch (crest) width according to K-C sharp-crested weir equation.
5. set final notch width according to constructability requirements.
6. check final design value for compliance with needed minimum pool depth.

An example illustrating the notch design calculations is given in Appendix 2D.

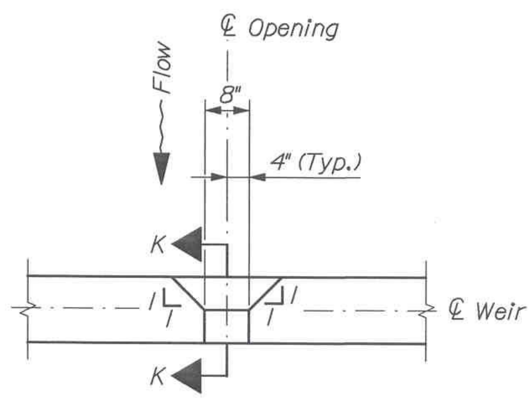
Slotted Weirs (Full-Depth Notch)

While notched weir-and-pool arrangements are attractive for maintaining water levels and velocities in relatively flat culverts, they can present construction and durability issues, particularly if the notch is not very high. Problems of weir spacing in steep culverts have already been noted. Therefore, slotted weirs (i.e., full-depth notches) may also be considered. Typical details follow in Figure 2.5; specific dimension values will vary, depending on the site. The design procedure is significantly different than for the notched weir-and-pool approach and is not covered in this edition of the Fish Passage Design Guide. Most significantly, slotted weirs are more properly classified as baffles and tend to be closely spaced. Environmental Office and/or Bridge Program engineering staff should be consulted for further information.

Figure 2.5 Slotted Weir Detail



SECTION A-A
SLOTTED WEIR



PLAN

Downstream Weirs (Grade Control Structures)

When a culvert outlet is excessively perched, downstream grade control may be needed to allow fish entry into the culvert. As a practical matter, right-of-way considerations may limit such options. That said, two types of weirs should be considered: rectangular notch weir as described previously for in-culvert applications; and full channel-width broad-crested weir. Different methods of construction will be used, though. As previously noted, concrete barrier (e.g., Jersey) makes for a simple and cost-effective weir. Rock and boulder weirs have also been used.

Rectangular Notch Weir

The rectangular notch weir is sized in the same way as for in-pipe weirs.

Broad-Crested Weir

The broad-crested weir is in many cases the pre-existing gravel/ cobble push-bar at the exit of the culvert outlet pool. The bar extends fully across the channel. The length (in direction of flow) of the bar is long compared to the depth of water on the bar. The effect is to induce critical flow over the bar. A conservative approach is to adjust the bar elevation and culvert inlet to achieve the nominal desired water depth at the culvert outlet. For example, if 8 inches (200 mm) is needed at the outlet, set the bar elevation (lowest point on the bar) 8 inches above the culvert inlet. However, this will actually produce a water surface elevation somewhat higher than nominal design, since it ignores the depth of flow over the bar. If the bar cannot be set at this relative elevation, then hydraulic design accounting for flow depth on the weir should be developed,

The bar flow depth can be accounted for by using the broad-crested weir equation:

$$Q = C_d(2/3)(2g/3)^{1/2}b_c h_1^{3/2} = \{C_d(2/3)^{3/2}g^{1/2}\}b_c h_1^{3/2}$$

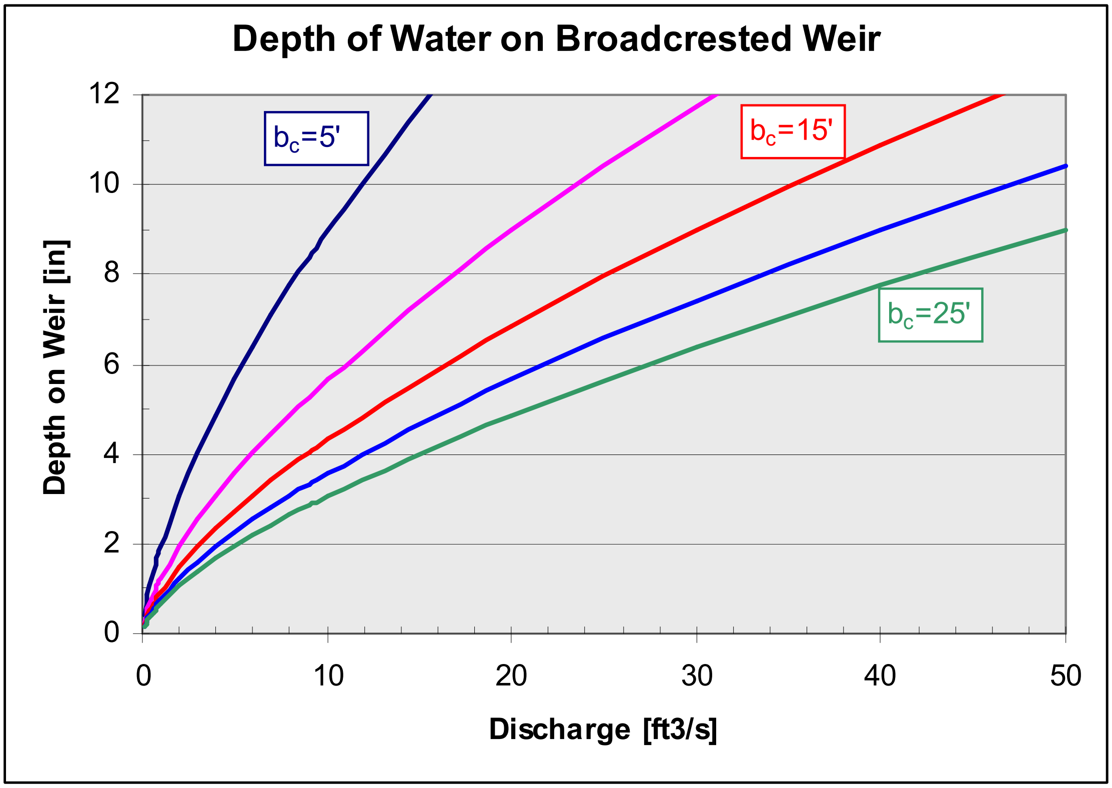
Where C_d = discharge coefficient (0.9 assumed)
b_c = channel width across the bar
h_1 = water elevation upstream of bar (referenced to bar elevation)

There are a variety of equations and charts available for determining C_d. However, in view of the uncertainty and variability inherent in the weirs contemplated here, it suffices to use a standard value of 0.9. Solving for h_1 gives the necessary depth of the bar below the desired water surface elevation:

$$h_1 = [Q/\{C_d(2/3)(2g/3)^{1/2}b_c\}]^{2/3}$$

This function is illustrated below in Figure 2.5 for a range of weir widths. Situations where this refinement might be considered include weaker swimming fish who require a minimum water depth on the weir and cannot jump the weir. Wider weirs in lower discharge environments maybe prone to such complications.

Figure 2.6: Depth of Water on Broadcrested Weir



Also, since flow over the weir is critical and therefore swifter than tranquil flow, the critical velocity over the weir should be checked for weaker-swimming fish:

$$v_c = (gh_1)^{1/2}$$

As previously noted, downstream weirs may succeed in creating the needed backwater, but they may present barriers to fish movement. Several weirs may be needed to raise the backwater while permitting fish passage over smaller incremental water level jumps.

Alternatives to Weirs: Engineered Fishways

While in-culvert and downstream grade control structures are the preferred approaches to creating the necessary hydraulic conditions for fish passage, there will occasionally be situations where they are not feasible or will not deliver the needed hydraulics. In these cases, Steeppass and Denil fishways should be considered. They are particularly suited to the following situations:

- excessive outlet drop that cannot be mitigated by downstream grade control
- right-of-way unavailable for developing downstream grade control
- steep culvert slope that would require numerous closely spaced internal weir

A drawback of these structures is that they create a long-term maintenance obligation above that of simple weirs.

An alternative to manufactured fishways such Denil or Steeppass is to build a pool-weir sequence at the culvert outlet. This enables fish to negotiate outlet hangs that cannot otherwise be corrected and also maintain minimum water depths in the culvert. A sample is shown below.

References

Bos, M.G., 1989. Discharge Measurement Structures (Third revised edition), Publication 20, International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands.

Bureau of Land Management, Coos Bay, Oregon. *Designing for Stream Simulation at Road Crossings*, Chapter 5, Pipe Arch Culverts with Sloping Fishways, http://www.or.blm.gov/coosbay/Engineering/chapter.htm

Dudley, R.W., 2004, Estimating monthly, annual, and low 7-day, 10-year streamflows for ungaged rivers in Maine: U.S. Geological Survey Scientific Investigations Report 2004-5026, U.S. Geological Survey, Augusta, Maine.

FHWA, 1985. *Hydraulic Design Series No. 5 (HDS-5)*, "Hydraulic Design of Highway Culverts", Federal Highway Administration.

Maine DOT, 2007a. Maine DOT Fish Passage Policy, Environmental Office, Augusta, Maine.

Maine DOT, 2003. Bridge Design Manual, Bridge Program, Augusta, Maine.

Maine DOT, 2003. Highway Design Manual, Urban and Arterial Program, Augusta, Maine.

Hodgkins, 1999. Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals, *Water-Resources Investigations Report 99-4008*, U.S. Geological Survey, Augusta, ME.

R.A. Currie, Ltd., 1997. A Field Investigation of Fish Passage through Ten New Brunswick Highway Culverts, Contract 97-2950 Fish Passage Investigation, New Brunswick DOT – Structures & Materials Branch.

Savoie, R., and D. Haché, 2002. Design Criteria for Fish Passage in New or Retrofit Culverts in the Maritime Provinces, Canada. Dept. Fisheries and Oceans, Oceans & Sciences Branch, Habitat Management Section, Moncton, N.B., Canada.

Washington State Department of Fish and Wildlife, 1999. *Fish Passage Design at Road Culverts*, Appendix C, Habitat and Lands Program, Environmental Engineering Division, http://www.wsdot.wa.gov/TA/T2Center/FishPassage.pdf

Appendix 2A

Regression Equations for Monthly Median Flows in Maine Rivers and Streams

Based on

Estimating monthly, annual, and low 7-day, 10-year streamflows for ungaged rivers in Maine

U.S. Geological Survey Scientific Investigations Report 2004-5026

by

R.W. Dudley
U.S. Geological Survey
Augusta, Maine
2004

Regression equations and their accuracy for estimating monthly median streamflows for ungaged, unregulated streams in rural drainage basins in Maine.

Regression equation	Measures of Accuracy ASEP (in percent)	(PRESS/n)$^{1/2}$ (in percent)	Average EYR
$Q_{\text{jan median}} = 20.71\,(A)^{1.036}\,(DIST)^{-0.762}$	-16.1 to 19.2	-17.3 to 20.9	8.87
$Q_{\text{feb median}} = 36.54\,(A)^{1.017}\,(DIST)^{-0.890}$	-13.4 to 15.5	-14.9 to 17.5	17.5
$Q_{\text{mar median}} = 183.7\,(A)^{0.999}\,(DIST)^{-1.142}$	-16.9 to 20.4	-19.0 to 23.5	13.3
$Q_{\text{apr median}} = 0.227\,(A)^{1.010}10^{0.028(pptA)}$	-20.8 to 26.2	-22.0 to 28.3	3.75
$Q_{\text{may median}} = 0.262\,(A)^{1.070}\,(DIST)^{0.461}$	-20.4 to 25.6	-21.0 to 26.6	3.92
$Q_{\text{jun median}} = 0.734\,(A)^{1.076}$	-22.5 to 29.0	-23.6 to 30.8	4.26
$Q_{\text{jul median}} = 0.210\,(A)^{1.149}10^{1.02(SG)}$	-26.1 to 35.4	-27.3 to 37.5	3.58
$Q_{\text{aug median}} = 0.152\,(A)^{1.120}10^{1.31(SG)}$	-28.6 to 40.2	-29.6 to 42.1	3.86
$Q_{\text{sep median}} = 0.169\,(A)^{1.093}10^{1.25(SG)}$	-26.8 to 36.7	-27.8 to 38.5	5.37
$Q_{\text{oct median}} = 0.307\,(A)^{1.074}10^{1.11(SG)}$	-25.8 to 34.8	-30.0 to 43.0	8.28
$Q_{\text{nov median}} = 1.222\,(A)^{1.004}$	-28.9 to 40.6	-30.6 to 44.1	4.39
$Q_{\text{dec median}} = 12.00\,(A)^{1.000}\,(DIST)^{-0.513}$	-13.1 to 15.0	-14.6 to 17.1	21.6

ASEP — average standard error of prediction

PRESS — prediction error sum of squares

EYR — equivalent years of record

Q — streamflow statistic of interest.

A — contributing drainage area, in square miles.

SG —fraction of the drainage basin that has significant sand and gravel aquifer, on a planar area basis, expressed as a decimal. For example, if 15% of a basin's drainage area has significant sand and gravel aquifers, SG = 0.15. Based on the significant sand and gravel aquifer maps produced by the Maine Geological Survey and maintained as GIS data sets by the Maine Office of GIS.

pptA — mean annual precipitation, in (in), computed as the spatially averaged precipitation in the contributing basin drainage area. Based on non-proprietary PRISM precipitation data spanning the 30-year period 1961-1990. Data maintained as GIS data sets by the United States Department of Agriculture (1998).

DIST —distance from the coast, in miles, measured as the shortest distance from the contributing drainage basin centroid to a line in the Gulf of Maine. The line in the Gulf of Maine is defined by end points 71.0W, 42.75N and 65.5W, 45.0N, referenced to North American Datum (horizontal) 1983.

Calculation of DIST Parameter

The DIST variable in the monthly flow regression equations is calculated as distance from the coast, in miles, from the watershed centroid point P_c to a line in the Gulf of Maine. The line in the Gulf of Maine is defined by lat-long endpoints P_1 (71.0W, 42.75N) and P_2 (65.5W, 45.0N), referenced to North American Datum (horizontal) NAD 1983. The corresponding UTM (zone 19, in meters) endpoint coordinates are P_1 (336321.28E, 4734992.89N) and P_2 (775853.73E, 4988911.83N). The point P_1 is the southwest endpoint and the point P_2 is the northeast endpoint of the reference line. DIST can be calculated using the following worksheet in UTM (metric) coordinates for the endpoints.

P_c	E	N	Watershed centroid (m, UTM)
P_1	336321.28 E	4734992.89 N	SW reference line endpoint
$\lvert P_1P_c \rvert$		$\{(P_{cE}-P_{1E})^2+(P_{cN}-P_{1N})^2\}^{1/2}$	Dist bet P_c and P_1 (m)
θ		$\mathrm{Tan}^{-1}\{(P_{cE}-P_{1E})/(P_{cN}-P_{1N})\} - 30.02^o$	Angle bet lines P_1P_c & P_1P_2
DIST		$\lvert P_1P_c \rvert \sin(\theta)$ / 1610	Dist to reference line (miles)

Alternatively, DIST can be estimated using the following figure from Dudley (2004) showing the Gulf of Maine Line with the state map.

24-Hour Duration Rainfall Depths (inches) for Various Return Periods

	Return Period (years)								Annual	Comments
Location	1	2	5	10	25	50	100	500		
Androscoggin	2.5	3.0	3.9	4.6	5.4	6.0	6.5	7.8	45.3	
Aroostook C	2.1	2.1	3.2	3.6	4.2	4.6	5.0	5.9	36.1	Presque Isle
Aroostook N	2.0	2.3	3.0	3.5	4.0	4.4	4.8	5.7	36.1	Ft Kent
Aroostook S	2.2	2.5	3.3	3.8	4.4	4.9	5.3	6.4	39.0	Houlton
Cumberland NW	2.8	3.3	4.3	5.0	5.8	6.4	6.9	8.3	43.4	NW of Rt 11
Cumberland SE	2.5	3.0	4.0	4.7	5.5	6.1	6.7	8.1	44.4	SE of Rt 11
Franklin	2.4	2.9	3.7	4.2	4.9	5.4	5.9	7.0	45.6	
Hancock	2.4	2.7	3.6	4.2	4.9	5.5	6.0	7.2	45.2	
Kennebec	2.4	3.0	3.8	4.4	5.1	5.6	6.1	7.2	41.7	
Knox-Lincoln	2.5	2.9	3.8	4.4	5.1	5.7	6.2	7.4	46.1	
Oxford E	2.5	3.0	4.0	4.6	5.3	5.9	6.4	7.6	43.0	E of Rt 26
Oxford W	3.0	3.5	4.5	5.2	6.0	6.6	7.1	8.4	43.8	W of Rt 26
Penobscot N	2.2	2.5	3.3	3.8	4.4	4.9	5.4	6.4	41.5	N of Can-Atl RR
Penobscot S	2.4	2.7	3.5	4.1	4.8	5.3	5.8	6.9	39.5	S of Can-Atl RR
Piscataquis N	2.2	2.5	3.3	3.8	4.4	4.9	5.3	6.3	38.5	N of Can-Atl RR
Piscataquis S	2.3	2.6	3.4	4.0	4.6	5.1	5.5	6.6	41.0	S of Can-Atl RR
Sagadahoc	2.5	3.0	3.9	4.6	5.4	5.9	6.5	7.8	45.3	
Somerset N	2.2	2.5	3.3	3.8	4.4	4.9	5.3	6.3	37.3	N of Can-Atl RR
Somerset S	2.4	2.7	3.5	4.1	4.7	5.2	5.7	6.8	39.5	S of Can-Atl RR
Waldo	2.5	2.8	3.7	4.3	4.9	5.5	6.0	7.1	47.2	
Washington	2.4	2.5	3.4	4.0	4.8	5.4	5.9	7.1	44.2	
York	2.5	3.0	4.0	4.6	5.4	6.0	6.6	7.8	46.7	

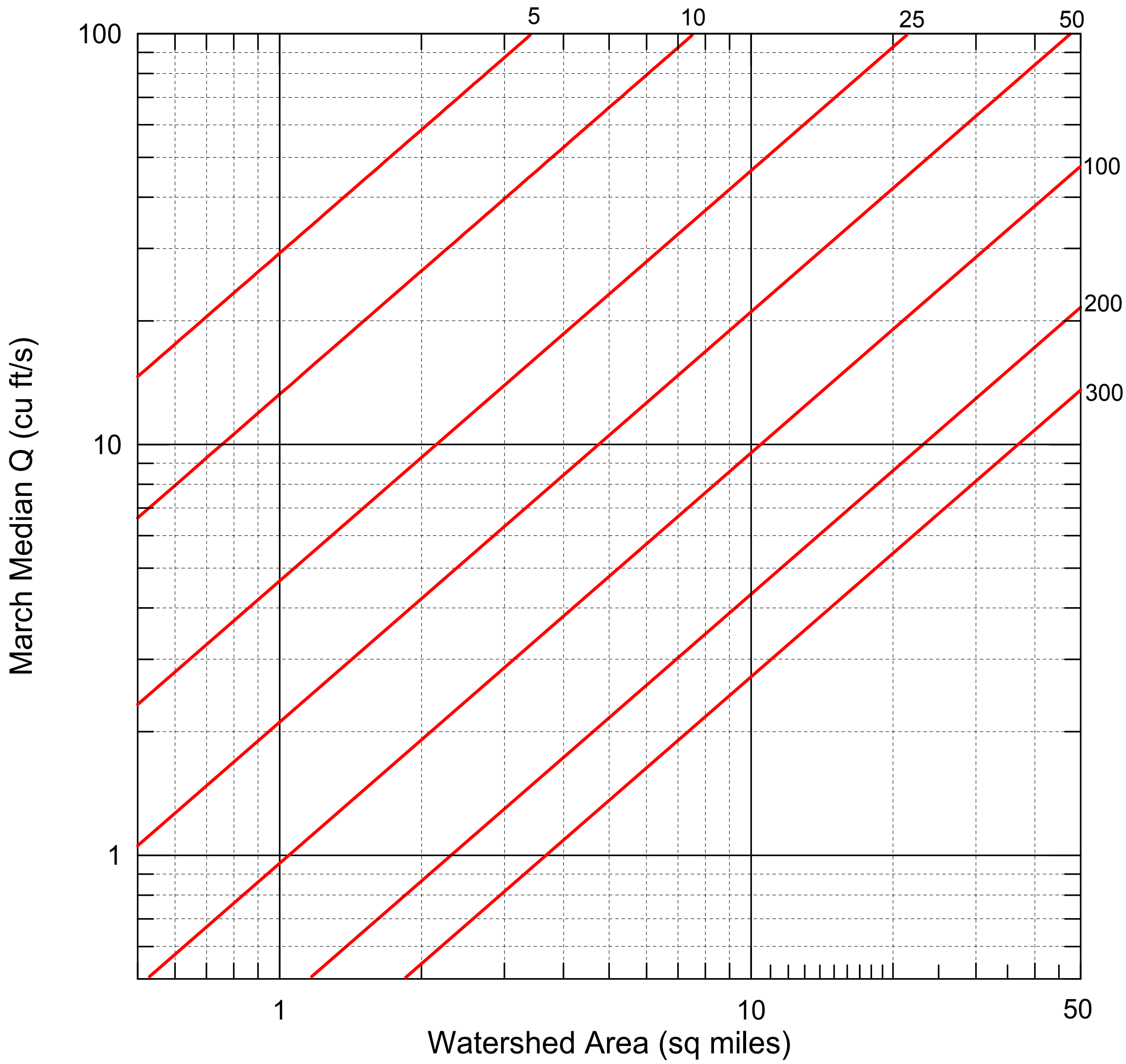
Source: Maine DEP Stormwater BMP Guide, November, 1995.

Note 1: Use Type II Storm for Oxford and Penobscot Counties, excepting towns listed below.

Note 2: Use Type III Storm for all other counties and the following towns in Oxford County (Porter, Brownfield, Hiram, Denmark, Oxford, Hebron, Buckfield, Hartford) and Penobscot County (Dixmont, Newburgh, Hampden, Bangor, Veazie, Orono, Bradley, Clifton, Eddington, Holden, Brewer, Orrington, Plymouth, Etna, Carmel, Hermon, Glenburn, Old Town, Milford, Greenfield).

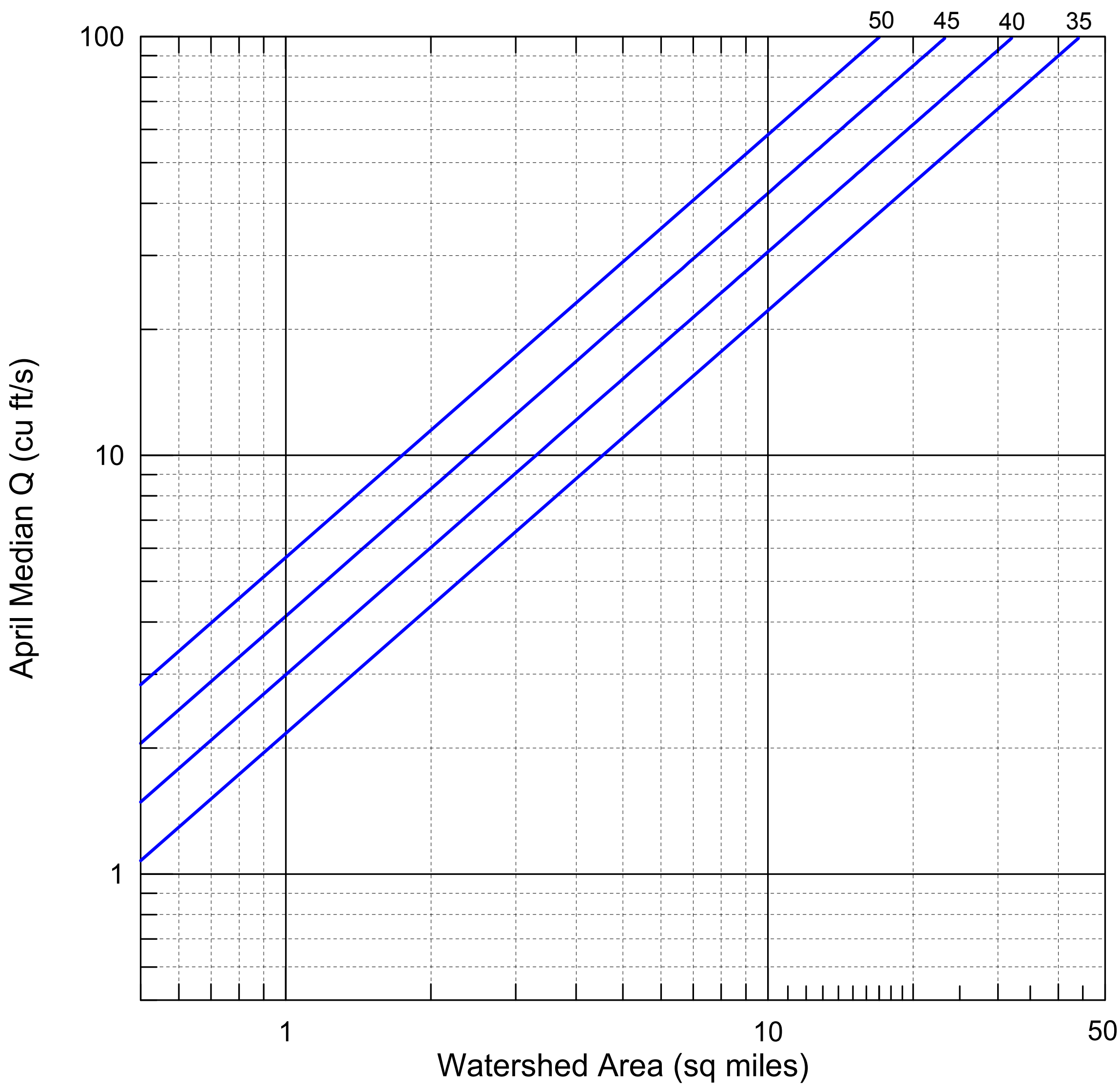
Note 3: 50-yr depths approximated as mid-point between 25- and 100-yr depths based on log-Normal probability plots.

March Median Flows for Selected Distances from Coast



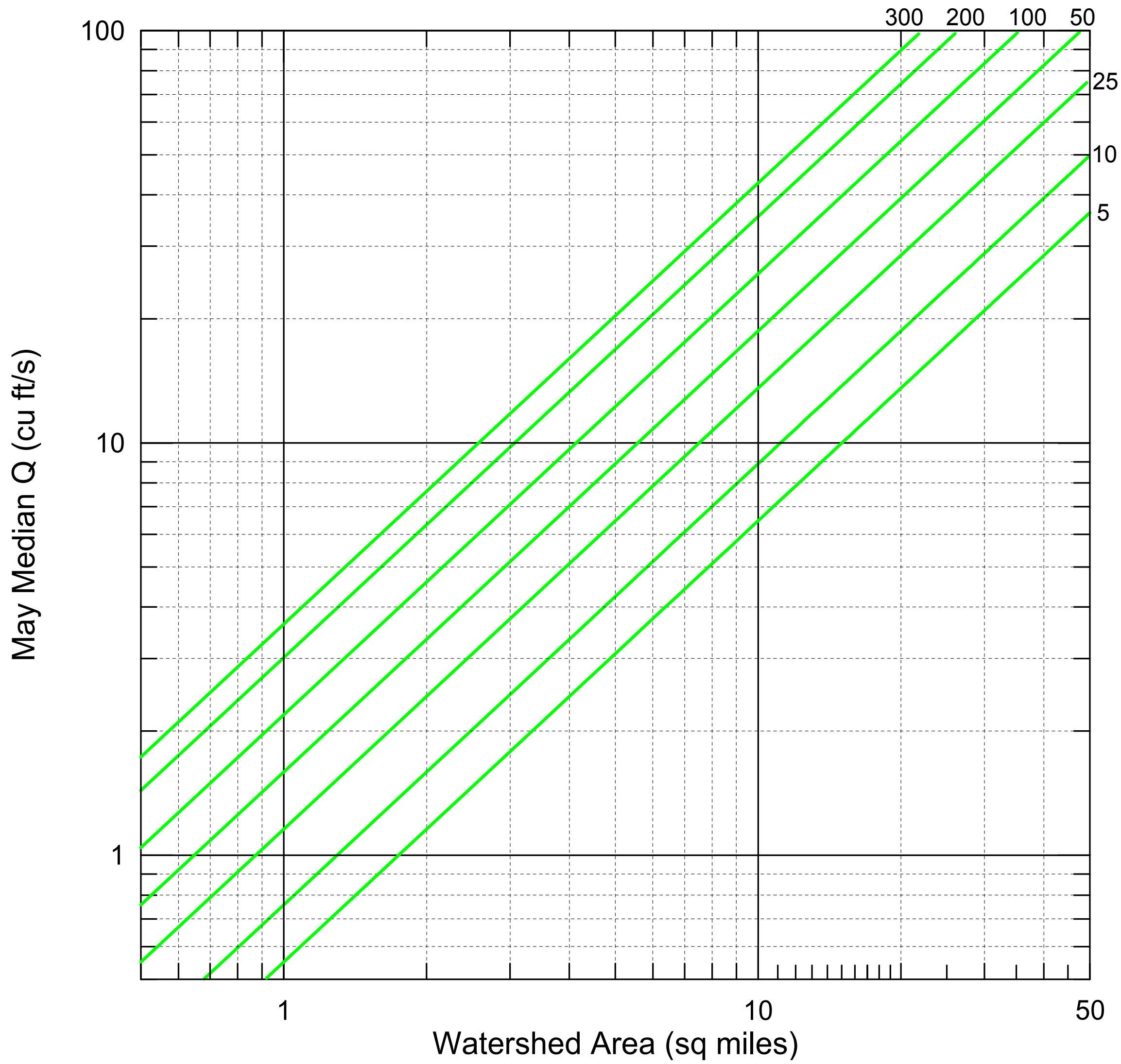
Note: Distance in miles from line in Gulf of Maine.
See flow equation page for explanation of distance determination.

April Median Flows for Selected Average Annual Precipitation



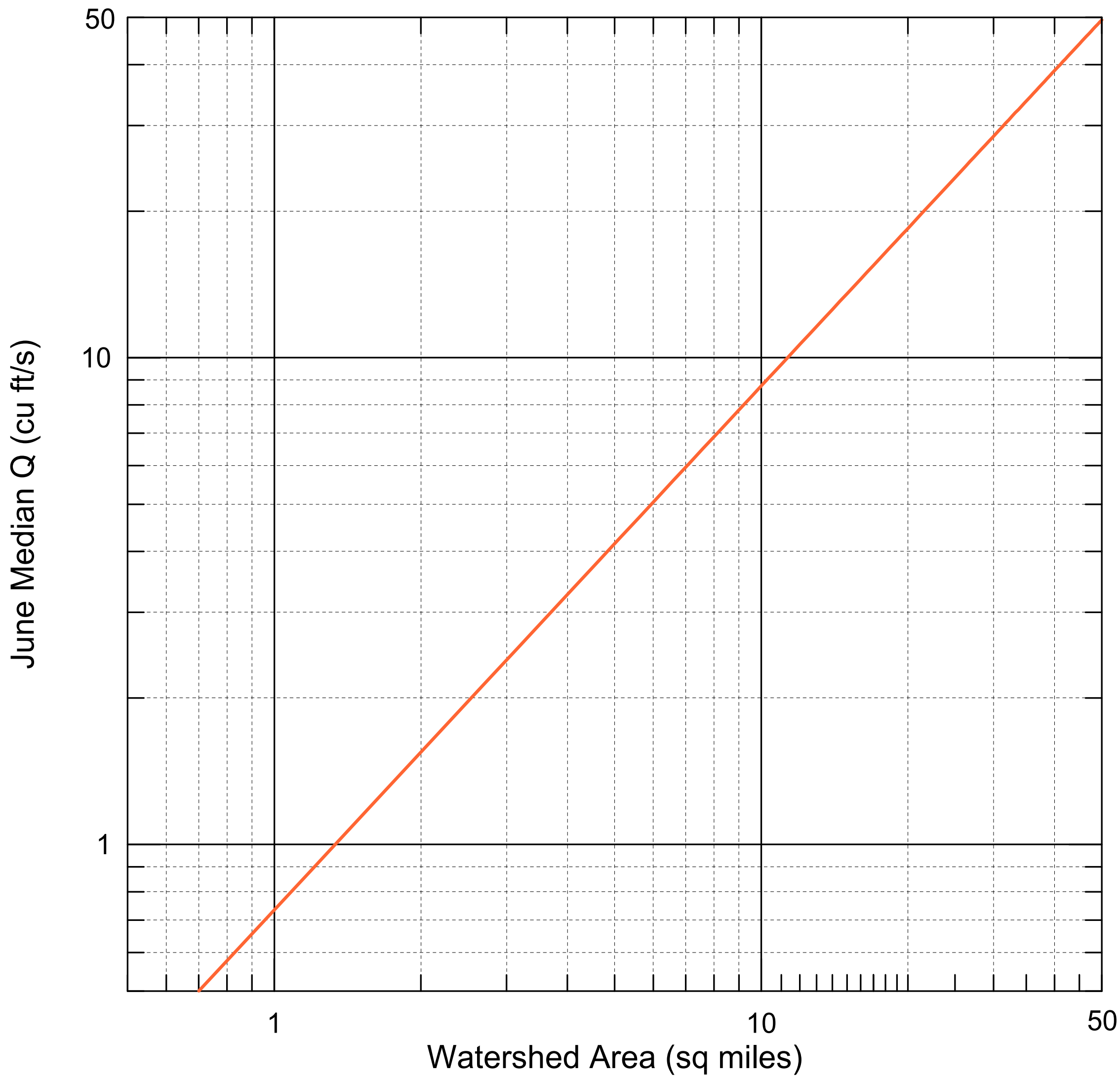
Note: Average annual precipitation in (inches).

May Median Flows for Selected Distances from Coast

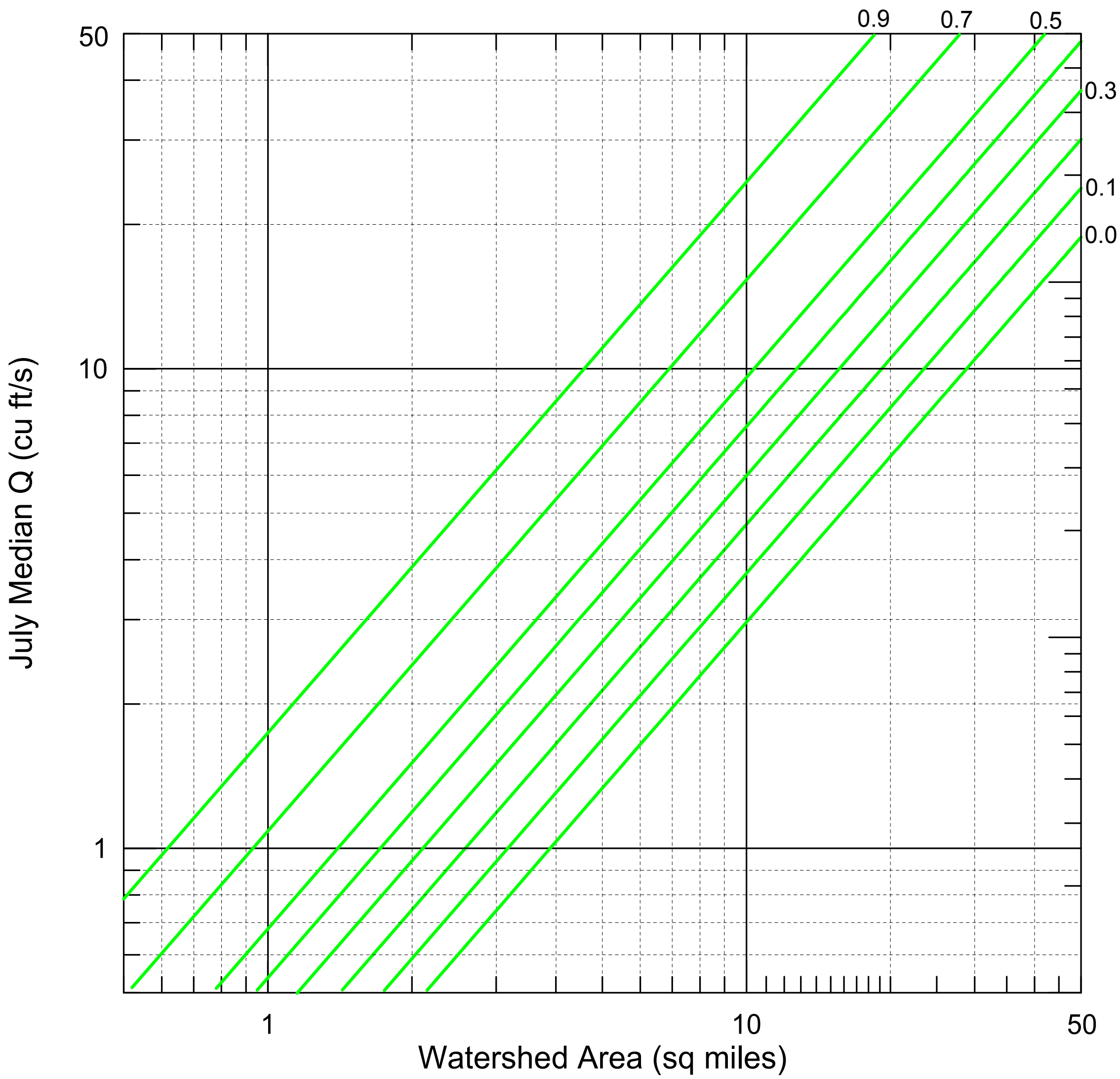


Note: Distance in miles from line in Gulf of Maine.
See flow equation page for explanation of distance determination.

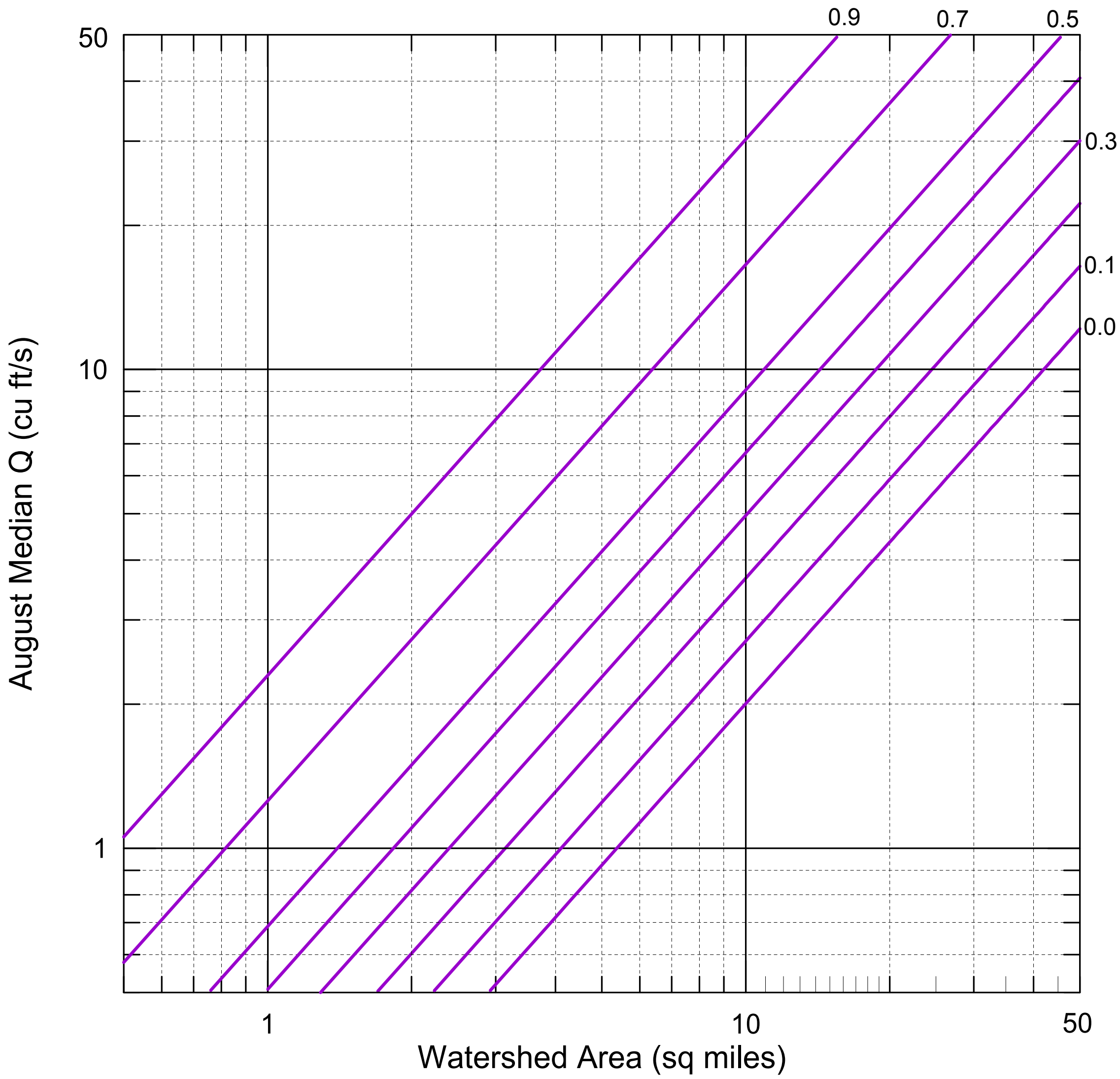
June Median Flows



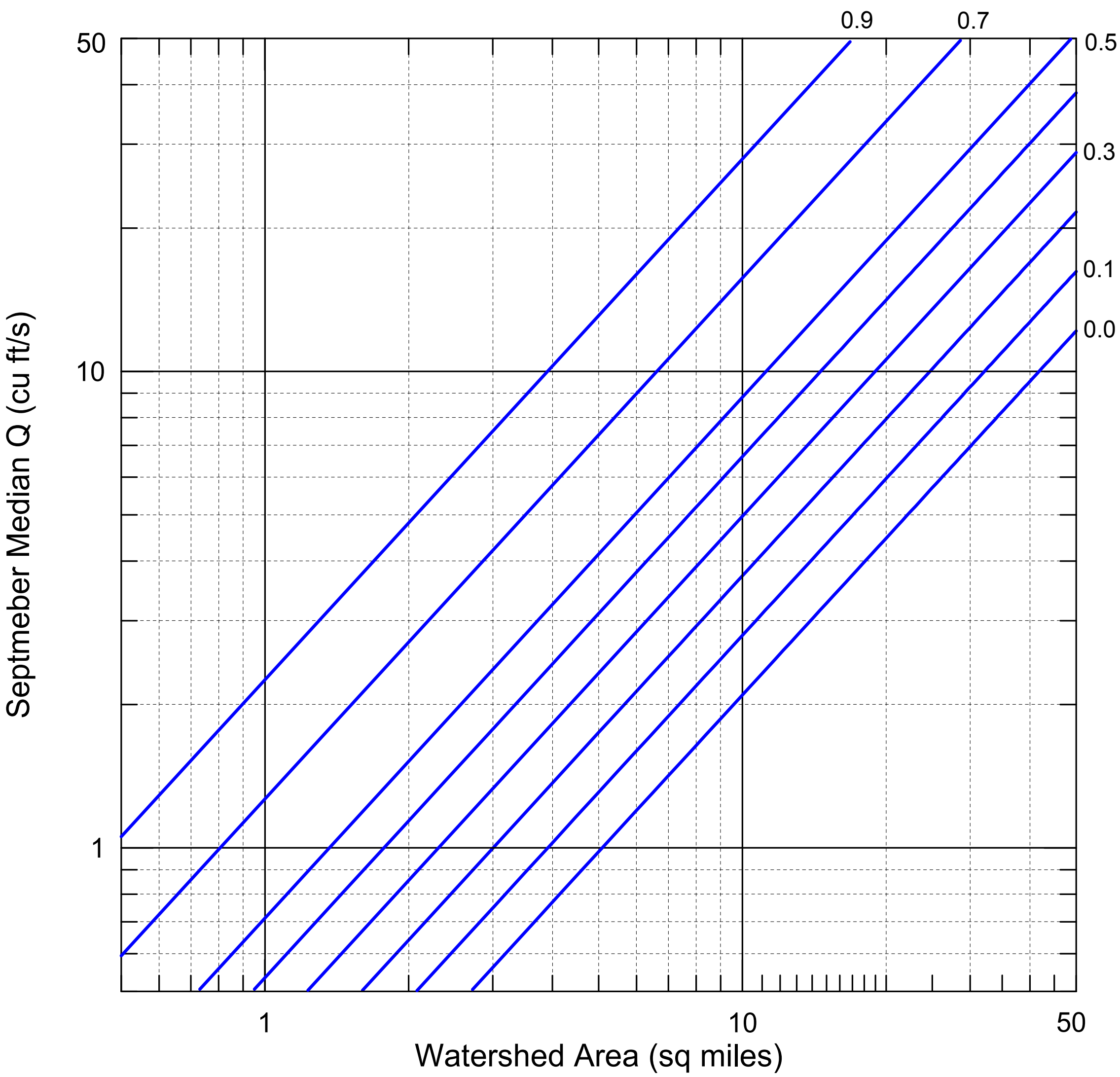
July Median Flows for Selected Sand & Gravel Fractions



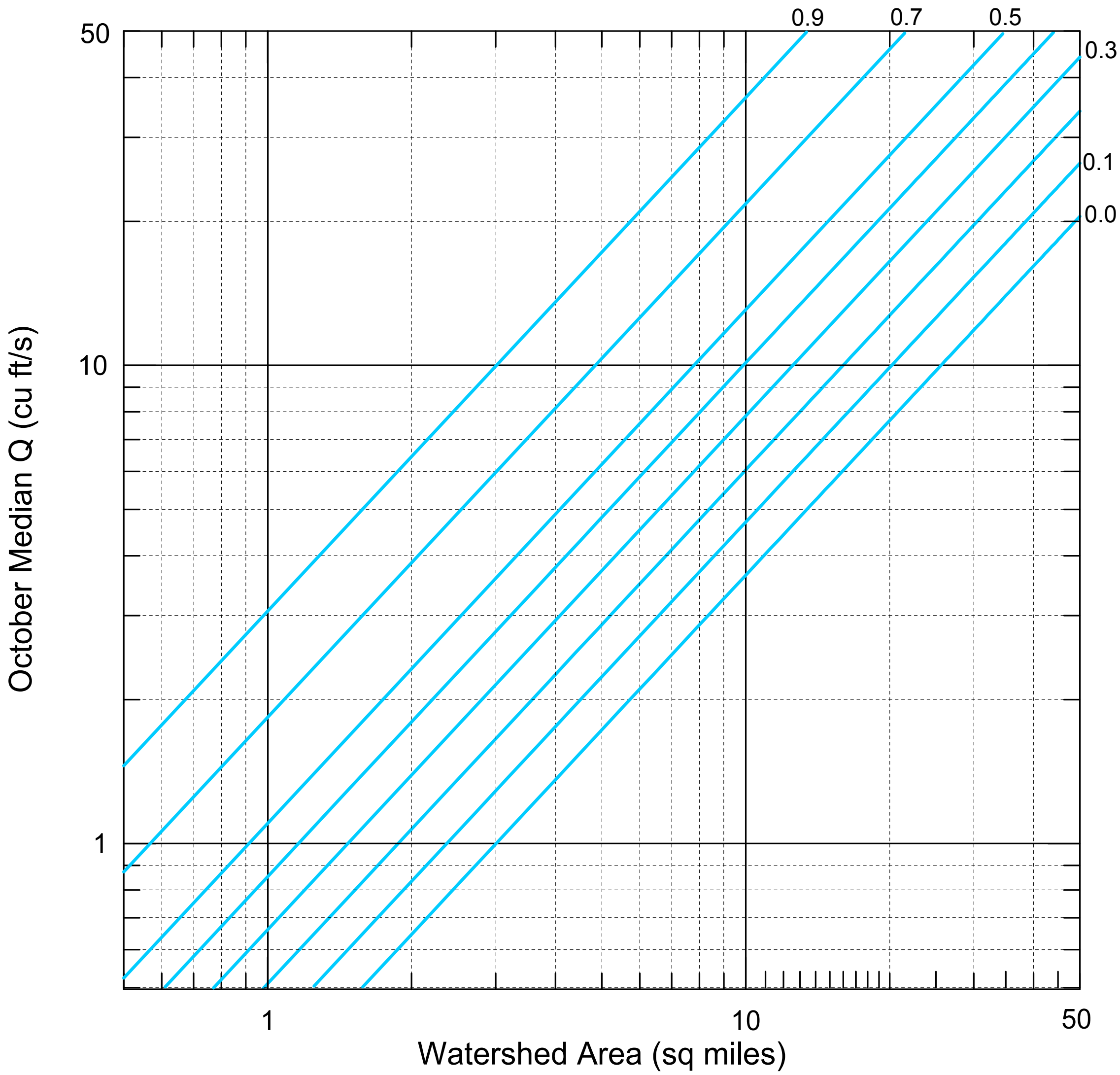
August Median Flows for Selected Sand & Gravel Fractions



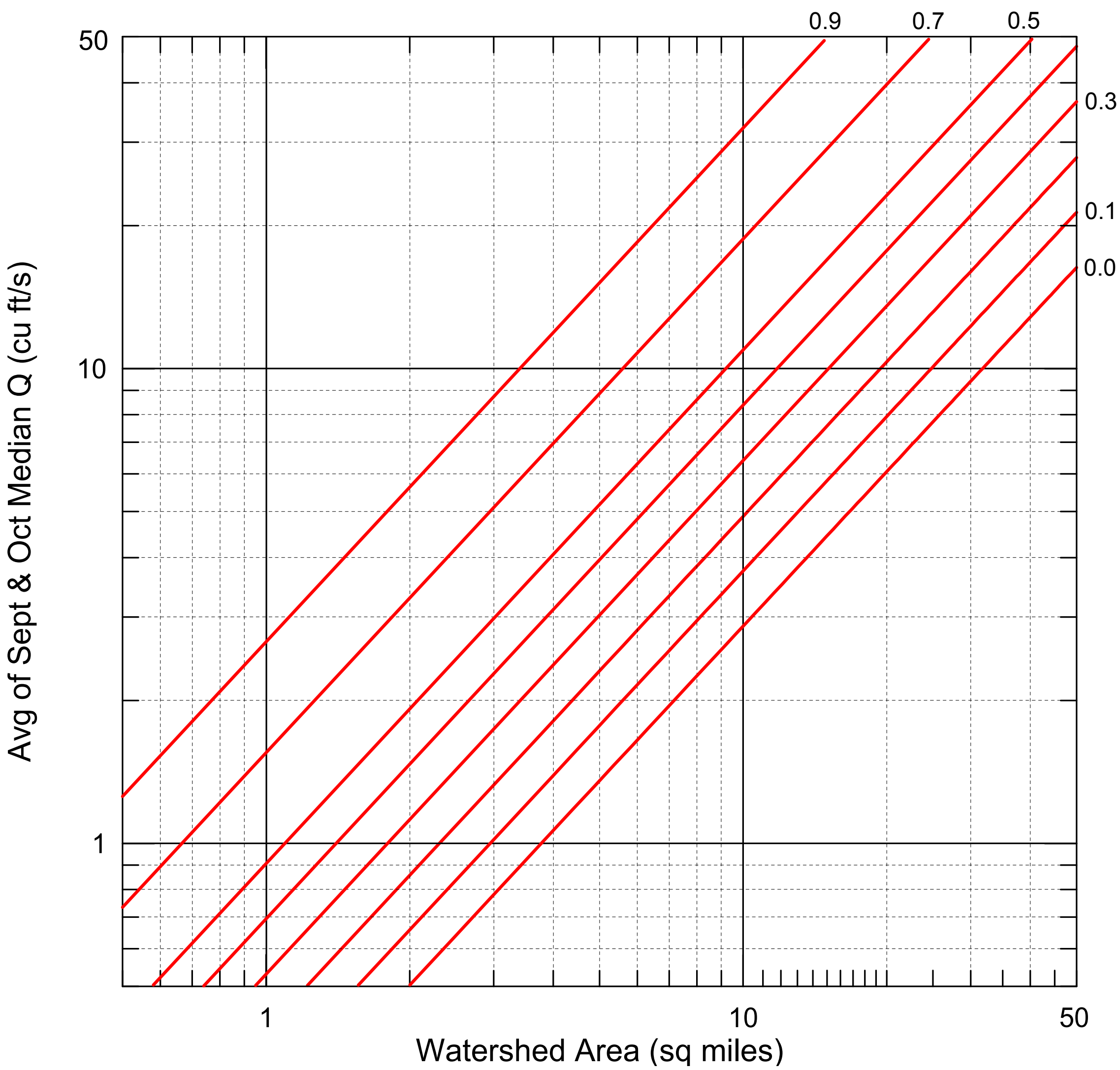
September Median Flows for Selected Sand & Gravel Fractions



October Median Flows for Selected Sand & Gravel Fractions



Average of September & October Median Flows
for Selected Sand & Gravel Fractions



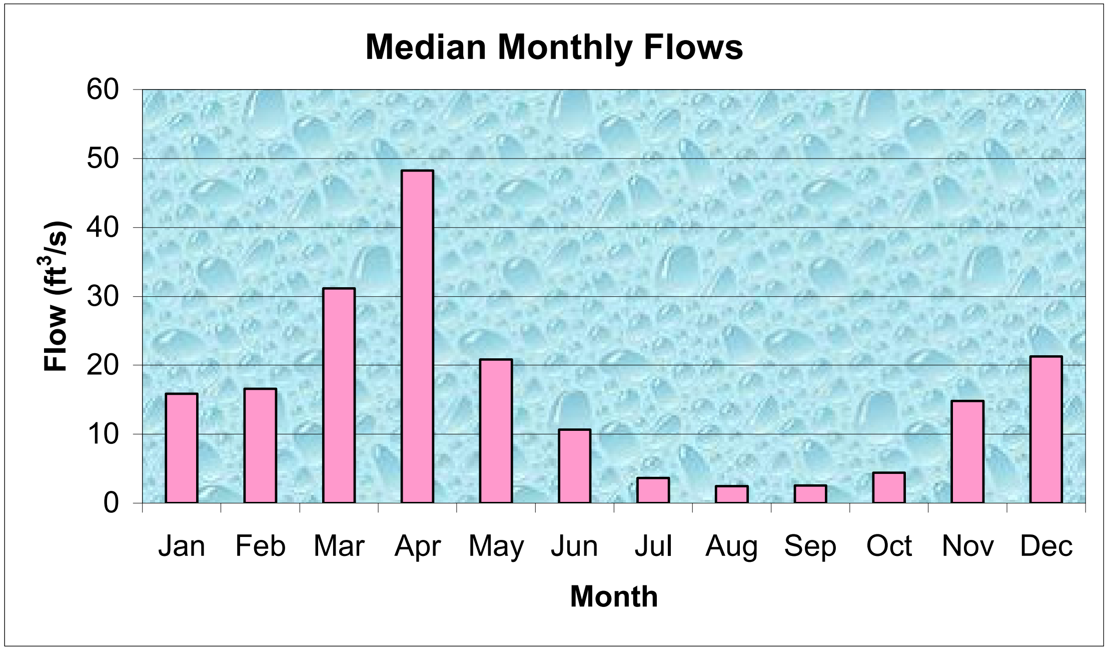
Project Name:	Example	**PIN:**	00000.00
Stream Name:	Any Stream	**Town:**	Anytown
Bridge Name:	Any Bridge	**Bridge No.**	0000
Route No.	Route 999	**USGS Quad:**	Any Quad
Analysis by:	CSH	**Date:**	2/3/2004

MAINE MONTHLY MEDIAN FLOWS BY USGS REGRESSION EQUATIONS (2004)

Worksheet prepared by:
Charles S. Hebson, PE
Chief Hydrologist
Maine Dept. Transportation
Augusta, ME 04333-0016
207-624-3073
Charles.Hebson@Maine.gov

	Value	**Variable**	**Explanation**	31.08
	12	*A*	Area (mi^2)	
625257	4979679	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)	
	41.57	*DIST*	Distance from Coastal reference line (mi)	
	44.2	*pptA*	Mean Annual Precipitation (inches)	
	0.00	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)	

Month	Q_{median} (ft3/s)
Jan	**15.88**
Feb	**16.58**
Mar	**31.16**
Apr	**48.26**
May	**20.86**
Jun	**10.64**
Jul	**3.65**
Aug	**2.46**
Sep	**2.56**
Oct	**4.43**
Nov	**14.81**
Dec	**21.28**



Appendix 2B

Calculations for Kindsvater-Carter Sharp-Crested Weir And Correction for Weir Submergence

Weir Notch Width Calculation

The weir notch depth h_1 is fixed by the specified crest submergence h_2 (usually 4 in or 100 mm) and the pool drop ($h_1 - h_2$; usually 8 in or 200 mm; 12 in when passing just salmon). This leaves the notch width b_c as the weir parameter designed to accommodate the fish passage flow. The notch width is calculated using the Kindsvater-Carter (K-C) sharp-crested weir equation:

$$Q = C_e b_e (2/3)(2g)^{1/2} h_e^{3/2}$$

where

- Q = flow passed by freely flowing (i.e., not submerged) weir (ft^3/s or m^3/s)
- b_e = effective notch width = $b_c + K_b$ (ft or m)
- K_b = notch width correction (tabulated function) (ft or m)
- b_c = actual notch width (ft or m)
- C_e = effective discharge coefficient (tabulated function)
- g = acceleration due to gravity (32.2 ft/s^2 or 9.81 m/s^2)
- h_e = effective head = h_1 + 0.003 ft (0.001 m)
- h_1 = upstream water surface elevation referenced to notch invert elevation (ft or m)

This equation can be quite accurate when calibrated for carefully constructed sharp-crested weirs used in flow-measurement situations. However, culvert weirs will not be built as "true" sharp-crested weirs and there is also significant uncertainty in the design flow estimates. Therefore, the correction for effective head (0.003 ft) can be ignored and h_1 used in place of h_e. The notch width correction K_b is a tabulated empirical function (see Appendix 2B). Again, it is a very small number (-0.003 ft (-0.04 in) < K_b < 0.016 ft (0.19 in)) compared to expected notch widths (b_c typically > 0.5 ft) and so can be ignored. The effective discharge coefficient C_e is a function of the notch width-channel width ratio (b_c/B_1) and above crest–below crest depth ratio (h_1/p_1). This functional dependence on b_c must be accounted for in the solution for b_c. This function is also tabulated in Appendix 2B. Employing the suggested approximations, the weir equation becomes

$$Q = C_e b_c (2/3)(2g)^{1/2} h_1^{3/2}$$

The fish pass weirs will be designed to flow partially submerged at design discharges, in order to pass both jumping and non-jumping species. A submerged weir will pass less water than a freely flowing weir, all other things being equal. Therefore, a weir designed for submerged flow must have a larger notch opening to accomodate the design passage flow. The submergence correction factor r_s is determined following the method of Villemonte:

$$r_s = \{1 - (h_2/h_1)^{3/2}\}^{0.385} = (Q/Q_{free}) \qquad \leq 1$$

where h_1 and h_2 are the respective upstream and downstream pool elevations above the weir crest, Q is the actual flow expected (by hydrology/hydraulics analysis), and Q_{free} is the flow through a freely discharging weir of the same dimensions. Maine DOT in-culvert weirs will usually be designed with 4 inch submergence (h_2 = 4 in or 100 mm). The effect of partial submergence is to reduce the flow over the weir. Therefore, the nominal design free flow must be increased over the actual hydrologic flow needed over the weir:

$$Q_{free} = Q/r_s$$

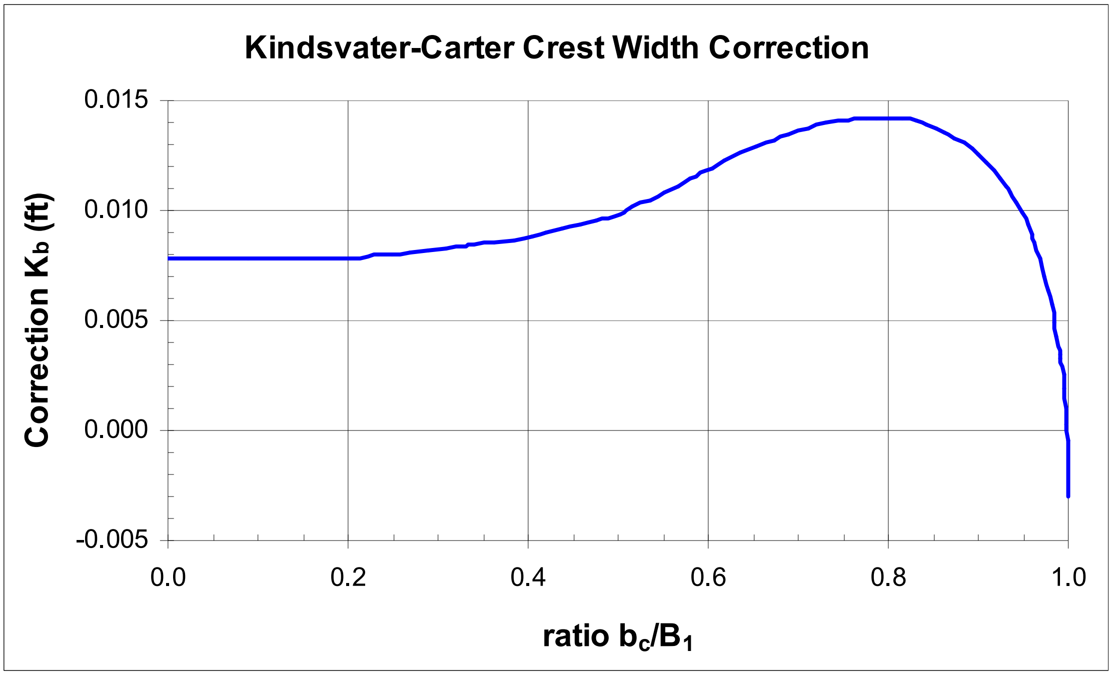
The weir is sized according to Q_{free} (= Q/r_s); the actual flow Q is chosen according to watershed hydrology and the flows prevailing during periods of fish movement.

Solving for the design notch width gives

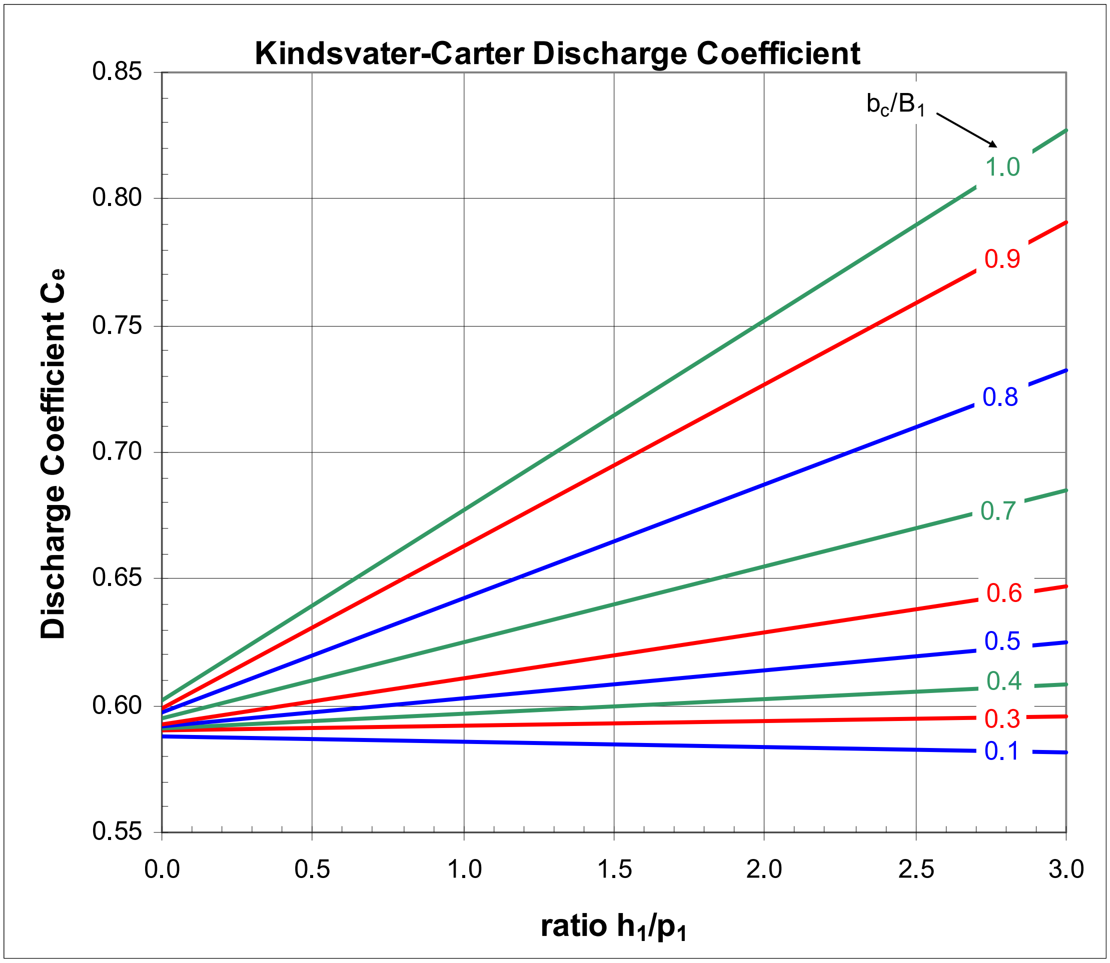
$$b_c = \{Q/r_s\}/\{C_e(2/3)(2g)^{1/2}h_1^{3/2}\}$$

This is actually a non-linear equation in b_c, since the discharge coefficient C_e is a function of b_c. Several iterations will be needed to solve for b_c, using the above equation in conjunction with the K-C charts and tables in Appendix 2B. A manual worksheet for executing the design calculations is provided in Appendix 2C. Alternatively, the calculations can be completed efficiently by computer spreadsheet.

Typical pipe and weir sizes will yield a relative notch width in te range $0.1 < b_c/B_1 < 0.5$; typical notch dimensions and water depths will produce h_1/p_1 approximately = 1. As a good approximation, then, the weir discharge coefficient C_e can be set = 0.6.



Kindsvater-Carter Crest Width Correction					
b_c/B_1	K_b (ft)	K_b (m)	b_c/B_1	K_b (ft)	K_b (m)
0.00	0.0079	0.0024	0.80	0.0141	0.0043
0.20	0.0079	0.0024	0.82	0.0141	0.0043
0.25	0.0082	0.0025	0.84	0.0141	0.0043
0.30	0.0082	0.0025	0.86	0.0135	0.0041
0.35	0.0085	0.0026	0.88	0.0131	0.0040
0.40	0.0089	0.0027	0.90	0.0125	0.0038
0.45	0.0092	0.0028	0.92	0.0118	0.0036
0.50	0.0098	0.0030	0.94	0.0105	0.0032
0.55	0.0108	0.0033	0.96	0.0089	0.0027
0.60	0.0121	0.0037	0.98	0.0056	0.0017
0.65	0.0128	0.0039	1.00	-0.0030	-0.0009
0.70	0.0135	0.0041			
0.75	0.0141	0.0043			



B_1 = weir top width (usually $\geq$ 2 ft)
p_1 = notch invert elevation above pipe invert (usually 0.25 ft – 0.5 ft)

Kindsvater-Carter Discharge Coefficient Equation Parameters

b_c/B	μ	β	b_c/B	μ	β
0.0	-0.0023	0.587			
0.1	-0.0021	0.588	0.6	0.0180	0.593
0.2	-0.0018	0.589	0.7	0.0300	0.595
0.3	0.0020	0.590	0.8	0.0450	0.597
0.4	0.0058	0.591	0.9	0.0640	0.599
0.5	0.0110	0.592	1.0	0.0750	0.602

Equation: $C_e = \mu(h_1/p_1) + \beta$

Appendix 2C

Manual Worksheet for Rectangular Weir Notch Sizing

Project Name	____________________	PIN	________________
Stream Name	____________________	Town	________________
Route No.	____________________	Culvert No.	________________
Designer:	____________________	Date	________________

Maine Department of Transportation
Culvert Fish Passage Weir-and-Pool Design Worksheet

Watershed Characteristics and Design Flow

1	Area (A)		sq miles
2	Sand & Gravel Fraction (SG)		Decimal fraction of area
3	Passage Design Flow Q		ft^3/s

Note: sand & gravel values only needed for Sep and Oct monthly median flow equations; other design flow estimation methods may be used.

Weir, Culvert and Hydraulic Specifications

(perform all calculations in consistent units of feet or meters)

1	$h_1 - h_2$		Water level drop across weir
2	h_2		Submerged depth on weir
3	d_{min}		Min pool depth (downstream base of weir)
4	D		Pipe diameter
5	S		Culvert slope
6	h_1		Upstream depth on weir $h_2 + (h_1 - h_2)$
7	p_1		Height of weir crest above invert $d_{min} - h_2$
8	d_1		Upstream pool depth at weir $h_1 + p_1$
9	r_s		Submergence ratio $\{1-(h_2/h_1)^{1.5}\}^{0.385}$
10	B_1		Pool top width at weir $2\{d_1(D - d_1)\}^{1/2}$ *for circular culverts*
11	L_w		Weir spacing $(h_1 - h_2)/S$
12	Q		Design flow adjusted for submergence Q/r_s

Calculations for Weir Rectangular Notch Width

Computation Constants

1	(Q/r_s)		from above
2	$(2/3)(2g)^{1/2}$		5.35 $ft^{1/2}/s$; 2.95 $m^{1/2}/s$
3	$h_1^{3/2}$		h_1 from above
4	$A = (Q/r_s)/\{(2/3)(2g)^{1/2}h_1^{3/2}\}$		computation constant A
5	B_1		pool width B_1 from above
6	h_1/p_1		above crest-below crest depth ratio

Iteration for Notch Crest Width b_c

Iteration	0	1	2	3	4	5	6	7
b_c/B_1								
K_b								
C_e								
$b_e = A/C_e$								
$b_c = b_e - K_b$								

Notes: always use consistent units of [feet] or [meters] in hydraulic calculations
set initial (iteration 0) b_c value = ½ of B_1;
get K_b and C_e by look-up in Appendix B;
iterate until crest width b_c stops changing

Appendix 2D

Weir Notch Sizing and EDF Calculation Example

Design Example

A 10-ft diameter culvert under a deep fill has been identified as needing attention. Whatever approach is taken, passage for trout must be provided. After evaluating several alternatives, concrete invert lining has been identified as the best choice. Design a pool-weir arrangement to pass fish.

Watershed and culvert data are summarized in the following table:

Watershed		**Culvert**	
Area	12 mi^2 (31.1 km^2)	Diameter	10 ft (3000 mm)
NWI area	24.8%	Slope	2%
Sand & gravel aquifer	0 %	Length	60 ft (18.3 m)
Avg annual precip	44.2 in (1123 mm)	Roughness n	0.024 (CMP)
Distance to coast	41.6 mi (67.4 km)		

<u>Fish Requirements</u>

Based on Table 2 in the Fish Passage Policy, trout are moving from April through November, though passage is less critical in the warm-water months of July and August. Flows in the September and October are the lowest flows in the months of interest and therefore provide the basis of a conservative design. The average of the September and October medians will be used, with the understanding that such a design will deliver the needed depths at the other, higher, flows. (Ideally, this regression estimate would be supported by a measurement-based estimate.)

Maine DOT generic design is to provide a minimum of 8 in depth when possible. Trout have a typical maximum body thickness of 4 in (100 mm), indicating a minimum depth for passage of (1.5 x 4 in) = 6 in (150 mm). In a sloping culvert, the minimum depth between weirs occurs at the base of the upper weir. Therefore, initial design will be for a depth d_{min} = 8 in (200 mm) at the downstream side of a weir. Since trout are strong swimmers, this requirement could be relaxed if engineering concerns indicate a preference for more widely spaced structures (as allowed by a bigger drop between pools).

Trout are capable of jumping, so strictly speaking, the weir does not have to be designed for submergence. However, Maine DOT general practice is to partially submerge the weir crest to facilitate passage of non-jumping species. Therefore, initial design will be for the downstream pool to be h_2 = 4 in (100 mm) above the weir crest.

<u>Design Flow</u>

Design flows can be based on field observations (actual depth/velocity measurements during the period of interest; minimum channel sections needed for movement) or median flow equations for the periods of movement. Using the watershed data, monthly median flows were estimated using the U.S. Geological Survey regression equations. The

September and October flows can be calculated using the equations in Appendix 2A, by look-up in the charts in Appendix 2A, or using the Maine DOT monthly median flow Excel worksheet.

By chart look-up, the average of the September and October medians is

$$Q = (Q_{Sep} + Q_{Oct})/2$$
$$= 3.5\ ft^3/s = 0.100\ m^3/s$$

This hydrologic design value will be adjusted for the specified submergence condition.

Weir Dimensions and Auxiliary Hydraulic Design Specifications

Recommended design values for water levels are

$h_1 – h_2 = 8$ in (0.667 ft = 200 mm) change in pool elevation across weir
$h_2 = 4$ in (0.333 ft = 100 mm) downstream submerged depth on crest

It follows that the upstream depth on the weir crest is

$$h_1 = (h_1 – h_2) + h_2 = 12 \text{ in } (1 \text{ ft} = 300 \text{ mm})$$

The height p_1 of the weir crest above the culvert invert is

$$p_1 = d_{min} – h_2 = 8 – 4 = 4 \text{ in } (0.333 \text{ ft} = 100 \text{ mm})$$

and the pool depth d_1 just upstream of the weir is

$$d_1 = h_1 + p_1 = 16 \text{ in } (1.333 \text{ ft} = 400 \text{ mm}).$$

The submergence ratio r_s is

$$r_s = \{1 – (h_2/h_1)^{3/2}\}^{0.385} = 0.921 = (Q/Q_{free})$$

The weir will actually be designed to accommodate a freely discharging flow of

$$Q_{free} = Q/r_s = (3.5\ ft^3/s)/0.921 = 3.8\ ft^3/s\ (0.108\ m^3/s)$$

Spacing Between Weirs

Spacing is calculated as

$$L_w = \Delta h/S$$

Where Δh = difference pool elevation across a weir and S is the culvert slope.

$L_w = (0.667 \text{ ft})/0.02 = 33.35 \text{ ft } (10.2 \text{ m})$

Calculate Notch Width

The notch width b_c is calculated with the K-C sharp-crested weir equation. The pipe is flowing partially full at flows characteristic of fish passage. The pool surface top width in a circular culvert just upstream of the weir is

$$B_1 = 2\{d_1(D - d_1)\}^{1/2} = 6.8 \text{ ft} = 2073 \text{ mm}$$

as calculated for a partially-flowing circular pipe. If a different culvert shape is used, then a different equation for B_1 should also be used.

The weir equation, rearranged for crest (notch) width b_c is

$$b_c = \{Q/r_s\}/\{C_e(2/3)(2g)^{1/2}h_1^{3/2}\}$$

The discharge coefficient C_e is determined using the chart in Appendix 2B. The depth ratio h_1/p_1 is (12 in/4 in) = 3. The width ratio b_c/B_1 is actually part of the solution for b_c and so an initial estimate must be made. Assume a b_c starting value ½ of the upstream pool width B_1, so initial b_c = 3.4 and b_c/B_1 = 0.5. By chart look-up, C_e = 0.63. Then

$$\begin{aligned} b_c &= \{3.8 \text{ ft}^3/\text{s}\}/\{0.63(2/3)(2 \times 32.2 \text{ ft/s}^2)^{1/2}(1 \text{ ft})^{3/2}\} \\ &= 1.13 \text{ ft} = 0.34 \text{ m} \end{aligned}$$

The assumed initial width ratio should be checked with this first iteration solution:

$$b_c/B_1 = 1.13 \text{ ft}/6.8 \text{ ft} = 0.17 \quad \text{(compare to initial value 0.5)}$$

Since this new value is so different from the initial assumption, the solution should be repeated. The new corresponding C_e value is 0.59 (for h_1/p_1 = 3, unchanged)

$$\begin{aligned} b_c &= \{3.8 \text{ ft}^3/\text{s}\}/\{0.59(2/3)(2 \times 32.2 \text{ ft/s}^2)^{1/2}(1 \text{ ft})^{3/2}\} \\ &= 1.20 \text{ ft} = 0.37 \text{ m} \end{aligned}$$

$$b_c/B_1 = 1.2/6.8 = 0.18 \text{ (compare to previous 0.17; 5\% difference)}$$

Given the uncertainty and approximation inherent in the various assumptions, this result is acceptable. Make the weir notch 1.2 ft (0.37 m) wide.

This same example is carried through in the worksheet that follows. This worksheet utilizes the additional correction K_b for the notch width. Designers can utilize the "manual" worksheet in Appendix 2C or the Maine DOT Excel worksheet for weir sizing calculations.

Design Example
Fish Passage Weir-and-Pool Design Worksheet

Watershed Characteristics and Design Flow

1	Area (A)	12	sq miles
2	Sand & Gravel Fraction (SG)	0	Decimal fraction of area
3	Design Flow Q	3.5	ft^3/s

Note: sand & gravel values only needed for monthly median flow equations; other design flow estimation methods may be used.

Weir, Culvert and Hydraulic Specifications
(perform all calculations in consistent units of feet or meters)

1	$h_1 - h_2$	8 in = 0.667 ft	W.L. drop across weir
2	h_2	4 in = 0.333 ft	Submerged depth on weir
3	d_{min}	8 in = 0.667 ft	Min pool depth (downstream base of weir)
4	D	10 ft	Pipe diameter
5	S	0.02	Culvert slope
6	h_1	*4 + 8* = 12 in = 1 ft	Upstream depth on weir $h_2 + (h_1 - h_2)$
7	p_1	*8 – 4* = 4 in = 0.333 ft	Height of weir crest above invert $d_{min} - h_2$
8	d_1	*4+12* = 16 in = 1.333 ft	Upstream pool depth $h_1 + p_1$
9	r_s	$\{1-(0.333/1)^{1.5}\}^{0.385}$ = 0.921	Submergence ratio $\{1-(h_2/h_1)^{3/2}\}^{0.385}$
10	B_1	$2\{1.333(10-1.333)\}^{1/2}$ = 6.8 ft	Pool top width at weir $2\{d_1(D - d_1)\}^{1/2}$
11	L_w	*0.667 ft/0.02 =* 33.35 ft	Weir spacing $(h_1 - h_2)/S$
12	Q/r_s	3.8 ft^3/s	Design flow adjusted for submergence Q/r_s

Calculations for Notch Width

Computation Constants

1	(Q/r_s)	3.8	from above
2	$(2/3)(2g)^{1/2}$	5.35	5.35 $ft^{1/2}/s$; 2.95 $m^{1/2}/s$
3	$h_1^{3/2}$	$1^{3/2} = 1$	h_1 from above
4	$A = (Q/r_s)/\{(2/3)(2g)^{1/2}h_1^{3/2}\}$	0.71	Computation constant A
5	B_1	6.8	Pool width B_1 from above
6	h_1/p_1	1/0.333 = 3	Above crest-below crest depth ratio

Iteration for Notch Crest Width b_c

Iteration	0	1	2	3	4	5	6	7
b_c/B_1		0.5	0.16	0.18				
K_b		0.01	0.01	0.01				
C_e		0.63	0.58	0.58				
$b_e = A/C_e$		1.13	1.22	1.22				
$b_c = b_e - K_b$	3.4	1.12	1.21	1.21				

Notes:

- *always use consistent units of [feet] or [meters] in hydraulic calculations*
- *set initial (iteration 0) b_c value = ½ of B_1;*
- *get K_b and C_e by look-up in Appendix 2B;*
- *iterate until crest width b_c stops changing*
- *blank version of this worksheet in Appendix 2C*

Energy Dissipation Factor (EDF) Calculation

The inter-weir pools should be check for acceptable EDF ($\leq$ 5 ft-lb/s/ft^3/; 250 J/s/m^3/). When designing pool-and-weir systems, it is appropriate to assume that potential energy is to be dissipated. The equation for EDF is then

$$EDF = (\rho g)(Q\Delta y/V)$$

The flow Q is the fish passage design flow, the water level drop Δy is specified in the design, and the pool volume is determined from the calculated weir spacing, the design flow depths, and channel geometry.

The pool volume is difficult to calculate for a sloped, partially full circular pipe with level water surface. An acceptable approximation is to calculate the average water depth in the pool (i.e., average of upstream and downstream depths). From this average depth, calculate a cross-sectional wetted area A_w. Then volume is (approximately) the product of this area A_w and the length L between weirs. This general approach can also be used for other cross-section geometries.

At the upstream weir, depth d_{min} =0.67 ft (8 in or 200 mm); at the downstream weir, depth ($h_1 + p_1$) = 1.33 ft (16 in or 400 mm). Water depths and areas are calculated using the equations in Table 1, with wetted area analogous to embedded area. Calculations for pool volume are given in the following table in consistent units of (ft).

		Upstr	Downstr
Radius; diam; water depth	R; D = 2R; d_b	5; 10; 0.67	5; 10; 1.33
Water surf to pipe center	$D = R - d_b$	4.33	3.67
Water surf top width	$w_b = 2\{d_b(D-d_b)\}^{1/2}$	5.0	6.8
Flow Area	$A_b = R^2\cos^{-1}(d/R) - dw_b/2$	2.18	6.19
Avg depth		1.0	
Water surf to pipe center		4.0	
Water surf top width		6.00	
Flow Area		4.09	
Length between weirs	L	33.35	
Pool Volume	V = AL	136	

Then EDF is calculated as

$$EDF = (\rho g)(Q\Delta y/V)$$
$$= (62.4 \text{ lbs/ft}^3)(3.5 \text{ ft}^3\text{/s} \times 0.67 \text{ ft/136 ft}^3) = 1.1 \text{ (ft-lb/ft}^3\text{/s)} < 5$$

Since the calculated EDF is less than the upper limit of 5 (ft-lb/s/ft^3), we conclude the pool-weir sequence provides adequate energy dissipation.