WORKING DOCUMENT

INTRODUCTION TO FISHWAY DESIGN

Chris Katopodis, P.Eng.

Freshwater Institute
Central and Arctic Region
Department of Fisheries and Oceans
501 University Crescent
Winnipeg, Manitoba
Canada, R3T 2N6

Ph:(204) 983-5181
FAX:(204) 984-2402

(403) 492-5554

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1 INTRODUCTION

In lakes, rivers and streams fish migrations involve completing a cycle of upstream and downstream movements. This sequence depends on the fish's life stage, its location, and the type of migration. Generally, downstream migration is a feature of early life stages, while upstream migration is a feature of adult life. Fish migrate to spawn, to feed, and to seek refuge from predators or harmful environmental conditions, such as the complete freeze-up of a stream or lake. Should a natural (e.g. a waterfall) or man-made (e.g. a dam, weir, or culvert) obstruction block the stream, fish migration may be slowed or stopped altogether. A fishway is a waterway designed to allow the passage of a species or a number of different species of fish past a particular obstruction. While in most cases fishways are built for adult spawners in some cases migrating juveniles are the target species. For adult fish spawning migrations are usually involved and delays are critical to reproductive success. For juveniles feeding migrations are usually involved and delays are not as critical.

Fish of all ages require freedom of movement to fulfil needs (e.g. reproduction, growth) which cannot be satisfied where they are. Obstructions such as dams or hanging culverts can have long-term effects, while temporary activities (construction of stream crossings) result in short-term stoppages of movement. Spawning migrations are undertaken typically by mature fish, although they are accompanied periodically by immature fish. Some migrations are extensive particularly for catadromous and anadromous species which involve movement to and from the sea. Even within freshwater systems potamodromous species may move more than 100 km. The migration period may take several weeks. Within this time frame there is a relatively short period when most fish migrate. Spawning migration occurs in the spring and fall depending on the species. Obstacles can prevent the passage of fish.

Fish also move from one area to another to feed. These movements may be upstream or downstream and occur over an extended period of time. Before winter freeze-up, fish move downstream to deeper pools for overwintering. This movement is triggered by a reduction in stream discharge. Fry and juvenile fish also show movement in seeking rearing habitat. As they grow older, they require access up and down the stream and into side channels and tributaries to find food and escape predators.

Fish passage over dams and weirs or through culverts is an important consideration in fish bearing streams. Just as adequate design and construction is required for safety, adequate provision for fish passage is required to maintain healthy fish populations. Well designed and constructed fishways provide a path that allow fish to continue migrating past dams, weirs or through culverts without unacceptable delays. Biological requirements such as fish behaviour, motivation, preferences, migration timing and swimming ability drive design and construction criteria for fishways. Although some requirements such as migration timing and the corresponding hydrological conditions in rivers and streams, or swimming performance and fishway hydraulics can be harmonized through rational approaches, other requirements such as species preferences, motivation and behaviour rely heavily on experience and judgement.

Swimming ability is a key component in the successful completion of fish migrations. Fish travelling upstream need to navigate through a variety of flows and water velocities. These range from areas of slow currents, such as pools, wide river sections or reaches of mild stream gradients, to areas of fast currents, such as rapids, narrow sections or reaches with steep gradients. Fish are able to negotiate these conditions by using different levels of swimming performance. Fish swimming performance has been classified into burst speed (highest speed attainable and maintained for less than 15 seconds), prolonged speed (a moderate speed that can be maintained for up to 200 minutes), and sustained speed (a speed maintained indefinitely). In natural waterways, fish mainly use sustained and prolonged speeds when migrating upstream and occasionally use burst speeds to overcome high velocity areas such as rapids.

Fishways allow fish to a) maintain migrations past new hydraulic structures, b) re-establish migrations after years of blockage at man-made barriers, or c) extend migrations upstream of natural barriers. Fishways continue to be a key factor in maintaining salmon stocks in the Columbia River (U.S.A.) by providing access over several hydroelectric dams. Fishways played a vital role in rebuilding the salmon runs in the Fraser River, British Columbia (Canada), after decades of severe population declines attributed to obstruction of spawning migrations. The obstruction was caused by a large rock slide at Hell's Gate Canyon which occurred during railway construction and constricted the river channel. Fishways opened a path over natural falls at the outlet of Frazser Lake, Kodiak Island, Alaska, helped develop and are perpetuating a major salmon run there. These are just a few examples illustrating the usefulness of fishways as mitigation and enhancement measures. A renaissance in fishway research and development has occurred in the last two decades, particularly in North America and Europe. This culminated in the organizing of the first International Symposium on Fishways in Gifu, Japan, in October 1990.

2 FISHWAY TYPES

Fishways usually consist of a sloping channel partitioned by weirs, baffles, or vanes with openings for fish to swim through. The in-channel devices act hydraulically together to produce flow conditions that fish can navigate. Several types of fishways have been developed and are usually distinguished by the arrangement of in-channel devices. Although several variations of each fishway type exist, fishways are classified into vertical slot, Denil, weir and culvert fishways. Excavated channels utilizing rocks, sills or weirs are also used as fishways. The different physical and hydraulic characteristics of each fishway type may make them suitable for some fish species and not suitable for others. Several types of fishways have been developed and the most common are described in sections 2.1-2.4. An effective fishway attracts fish readily and allows them to enter, pass through, and exit safely with minimum cost to the fish in time and energy.

2.1 Vertical Slot Fishways

In the vertical slot fishway, baffles are installed at regular intervals along the length to create a series of pools (Fig. 2.1) Fish easily maintain their position within each pool. Travel between pools, however, requires a burst effort through each slot. Water velocities at the slots remain almost the same from top to bottom. The main advantage of the vertical slot fishway is in its ability to handle large variations in water levels. Usually the difference between water levels in successive pools is 300 mm for adult salmon and 200 mm for adult freshwater fish. Vertical slot fishways usually have a slope of 10%.

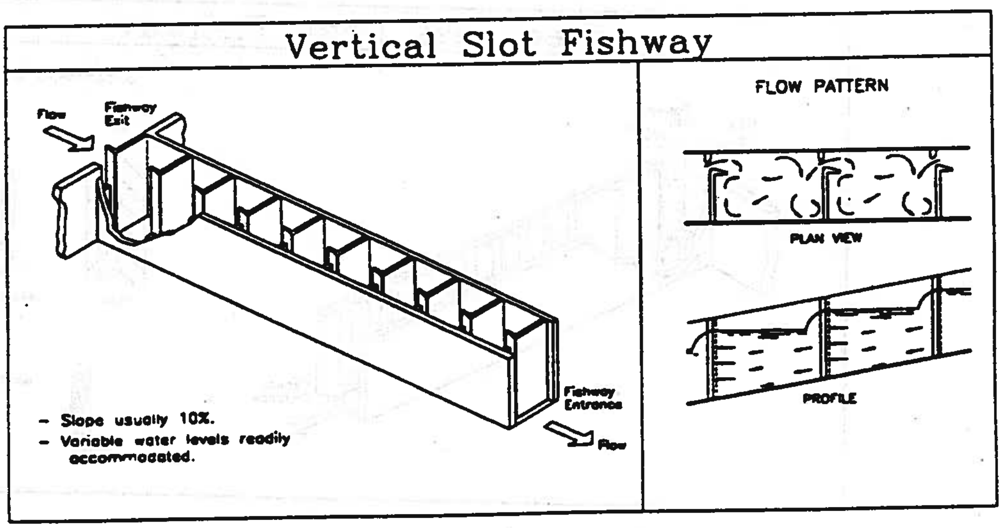


Figure 2.1 Vertical Slot Fishway

2.2 Denil Fishways

Named after its inventor, the Denil fishway consists of a rectangular chute with closely spaced baffles or vanes located along the sides and bottom. Over the years various versions of the Denil fishway have been developed and used for fish passage. Two of the more common Denil fishway types used today are shown in Figure 2.2. The plain Denil contains a series of planar baffles pointing upstream, at an angle of 45 degrees with the fishway floor. Baffles in the steeppass Denil also point in the upstream direction but are angled away from the walls of the chute.

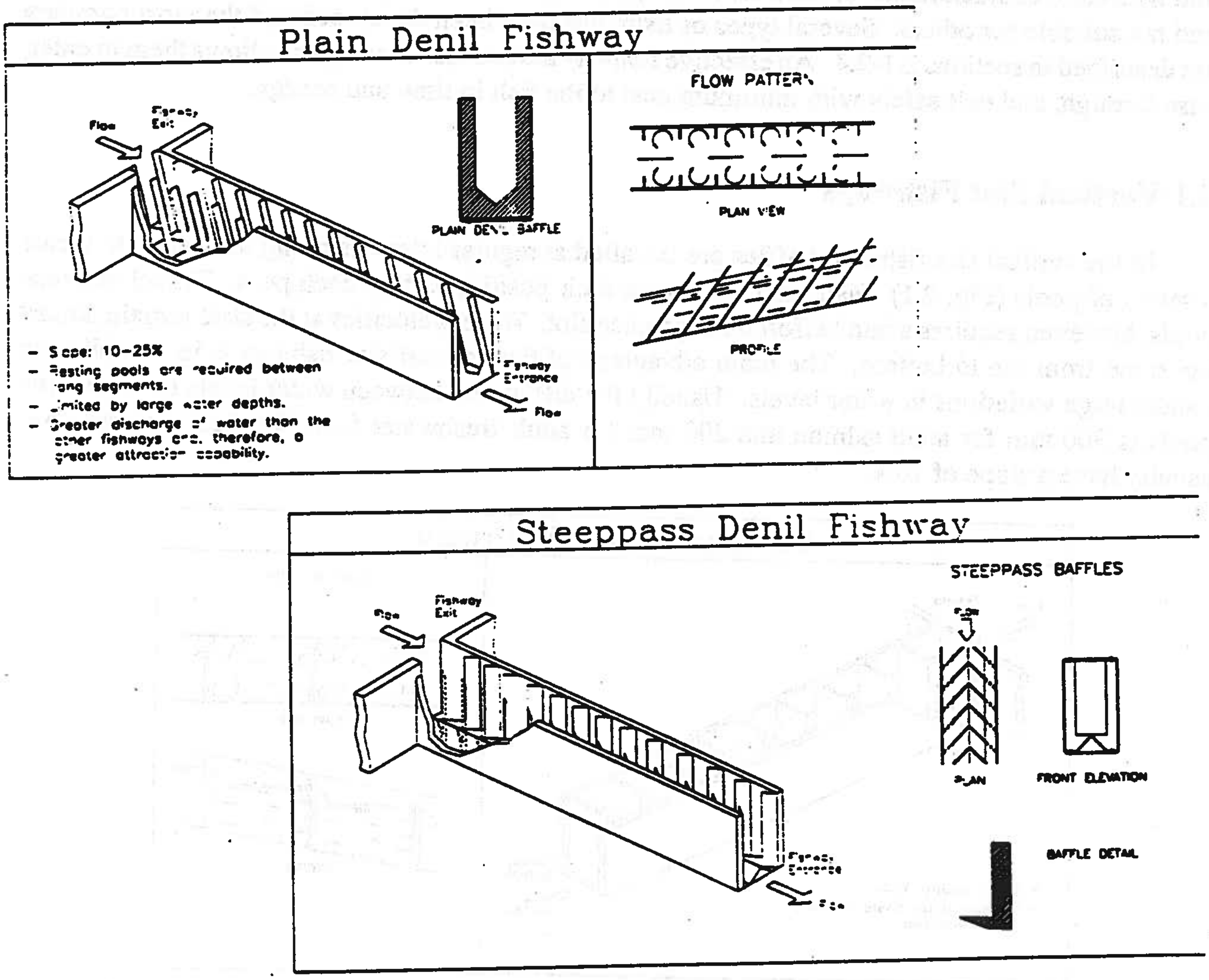


Figure 2.2 Denil Fishways

Flow through Denil fishways is highly turbulent, with large momentum exchange and high energy dissipation. For the plain Denil the water in the chute flows at a relatively low velocity near the bottom with a faster velocity near the top. For the steeppass, at low depths velocities tend to be higher near the bottom of the fishway and decrease towards the water surface. At high depths, flow divides into an upper and a lower layer, and velocity profiles become roughly symmetrical with maximum velocities at mid-depth. The large flow associated with the Denil designs, reduces the deposition of sediment within the fishway and also provides good attraction capability, assisting the fish in finding the fishway. Since fish need to constantly swim while in the chute, resting pools are placed along the fishway every 10 to 15 m for adult salmon and 5 to 10 m for adult freshwater species. Slopes for Denil fishways usually range from 10% to 15% for adult freshwater fish and 15% to 25% for adult salmon.

2.3 Weir Fishways

The weir fishway consists of a number of pools arranged in a stepped pattern separated by weirs, each of which is slightly higher than the one immediately downstream (Fig. 2.3). The fish, attracted by the flowing water, move from pool to pool by jumping or swimming (depending on the water depth) until they have cleared the obstruction. Movement between pools usually involves burst speeds. Fish can rest in the pools, if necessary as they move through the fishway. An orifice may also be added to the submerged portion of the weir allowing the fish to pass through the orifice rather than over the weir. While simple to construct, the pool and weir is sensitive to fluctuating water levels and requires adjustments. The water level drop between pools is usually set at 300 mm for adult salmon and 200 mm for adult freshwater fish. ~~Weir fishways usually have a slope of 10%.~~

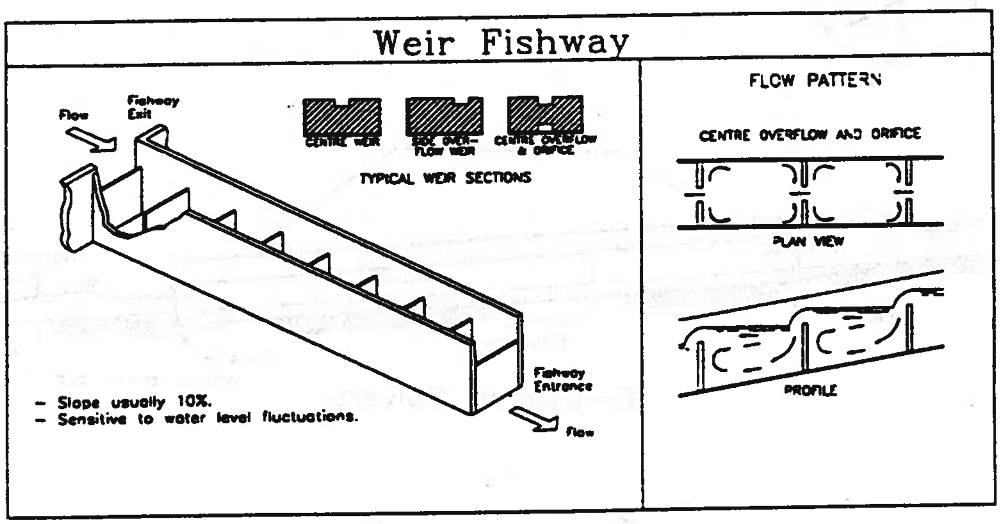


Figure 2.3 Weir Fishway

2.4 Culvert Fishways

Culverts are used to convey water from one side of a roadway embankment to the other. Culverts are built with circular, elliptic, pipe-arch, rectangular or square cross-sections. If a culvert is required to pass fish, special considerations are needed to ensure that fish can enter, pass through and exit the culvert without undue or harmful delay. In many cases culverts are placed below the stream bed and special devices such as riprap, baffles, weirs, blocks or plates are used to form a culvert fishway (Figs. 2.4a & 2.4b). Mainly associated with roadway construction, culvert fishways usually have slopes of between 0.5 and 5%.

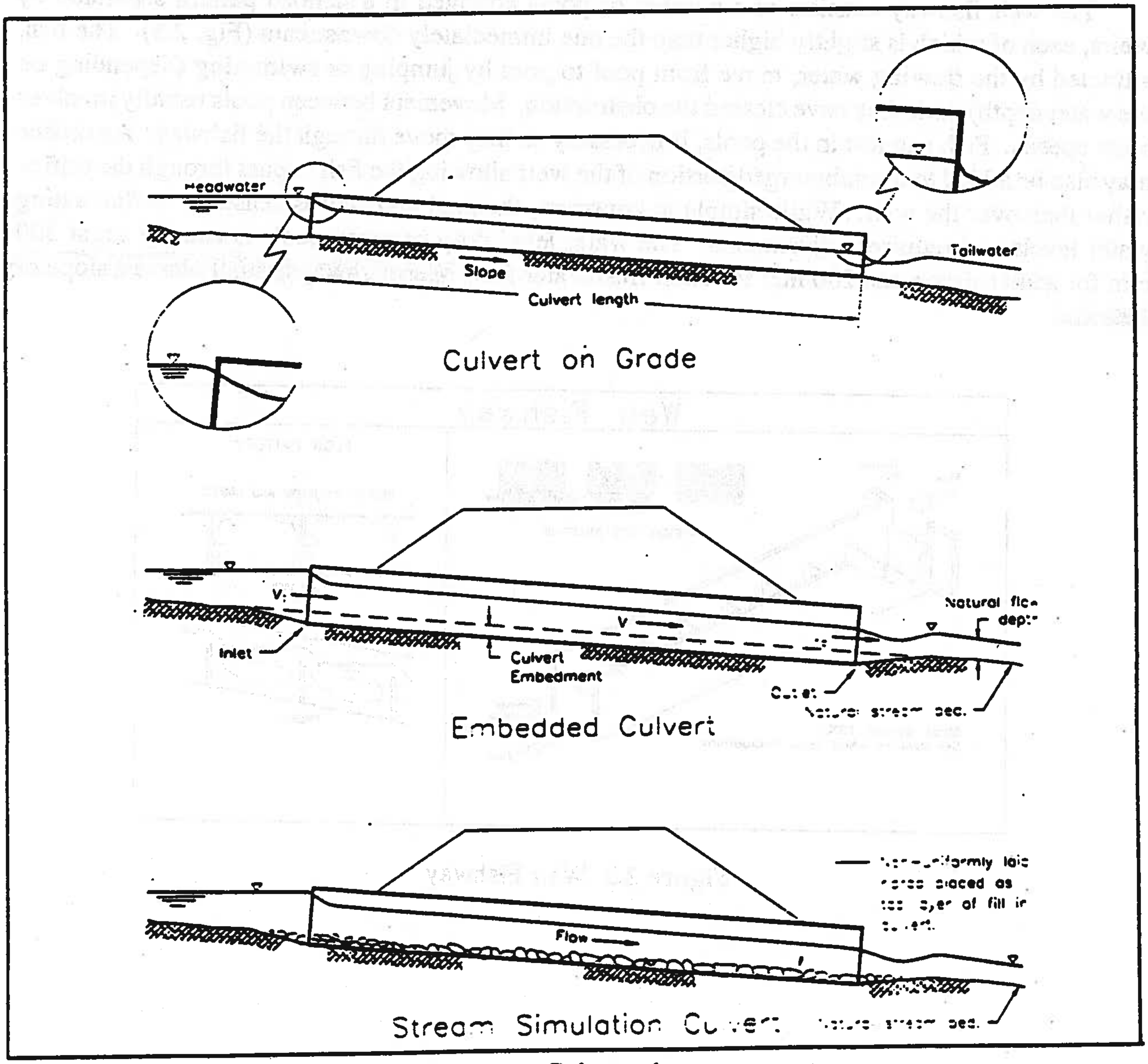


Figure 2.4a Culvert placement.

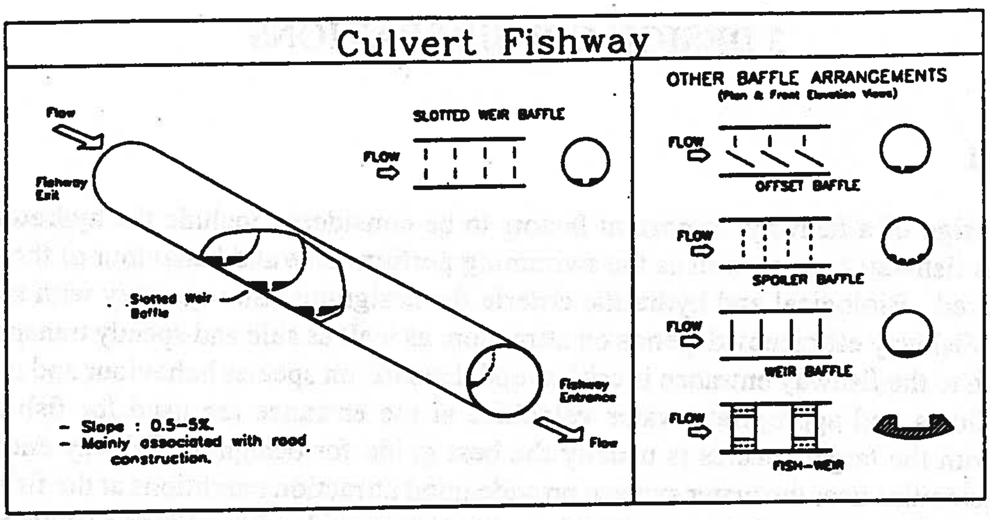


Figure 2.4b Culvert fishway.

3 DESIGN CONSIDERATIONS

3.1 General

In the design of a fishway, important factors to be considered include the hydraulic characteristics of the fishway type, as well as the swimming performance and behaviour of the species of fish to be passed. Biological and hydraulic criteria for designing fishways vary with species and sizes of fish. Fishway efficiency depends on attraction, as well as safe and speedy transport of fish. Attracting fish to the fishway entrance is critical and depends on species behaviour and motivation. Commonly, flows and appropriate water velocities at the entrance are used for fish attraction. Experience with the target species is usually the best guide for designing fishway entrances. In Denils, fast velocities near the water surface provide good attraction conditions at the fish entrance. Backwater conditions reduce fishway velocities, although considerable tailwater levels are usually needed to drown out Denil fishway flows. In vertical slot fishways, slot velocities and fish attraction conditions are affected by backwater when tailwater level exceeds some critical value (generally half of the critical depth in the slot). With slot flows drowned, the entrance pool provides little attraction for fish. Weir fishways are very sensitive to changes in water levels.

The most important factor in selecting the type of fishway to be used is the record of experience with the species of fish it is desired to pass. The Denil and vertical slot fishways have been successfully used by a wide variety of anadromous and freshwater fish. Culvert fishways have also been successful in passing various species. The weir, orifice and orifice-weir fishways have been used successfully by anadromous salmonids, but not readily by alewife, shad and probably other fish that rarely leap over obstacles or swim through submerged orifices. Both the vertical slot and the Denil allow fish to swim at their preferred depth. The Denil provides the most direct route of ascent while in the vertical slot fish use a "burst-rest" pattern to move between pools. Fish move through Denil fishways faster than through vertical slot or weir fishways.

In fishway channels, fish transport relies on water velocities not exceeding the swimming abilities of the migrating species. Swimming ability varies with species, size, as well as water temperature, oxygen, pH, and salinity. Water velocities depend on fishway type, channel slope and water depth. Velocities and depths are functions of fishway discharge and slope. Scale models of various types of fishways have provided velocities and depths for a range of discharges and slopes, as well as the functional relationship between these variables. Field studies with various fish species have tested fishway designs and demonstrated successful applications of fish passage technology.

Weir fishways are frequently the least expensive, while Denil fishways are usually less costly than vertical slot fishways. In Denil fishways effectiveness in water velocity control decreases as water depth increases. Since water velocities in Denils increase with depth, a limit is reached when water velocities start to exceed fish swimming speeds. If larger depths are required a second Denil fishway is needed. Vertical slot fishways maintain water velocities at the slots for very large water depths. This means that vertical slots can be built as deep as required to cover the entire range of water levels. Water level range and economics play a decisive role on which type of fishway is used.

The main problem with improperly designed and installed culverts is that they form velocity barriers to fish migrants at the outlet, inlet or within the culvert barrel. If water depths are too low or water velocities at any of these three culvert locations exceed fish swimming ability, fish may be prevented from reaching their spawning grounds. Since hydraulic efficiency and optimum fish passage requirements are mutually exclusive objectives, compromises must be effected that permit adequate fish protection with maximum economy. Such compromises involve the matching of water velocities with fish swimming performance at design discharges that allow limited, if any, delay in fish migrations.

Water velocities in plain culverts are usually much higher than those in natural channels. In addition, culverts provide fairly uniform velocities throughout their length, while streams provide a diverse pattern of slow to fast velocities both longitudinally and laterally. Sustained speeds are generally exceeded by culvert velocities, while fish cannot maintain burst speeds long enough to navigate the entire length of most culverts. Prolonged speeds are used for continuous passage through culverts when no resting areas are available. However, fish use a burst and rest pattern to take advantage of low water velocities that are created by the placement of rip-rap, baffles, weirs or other forms of culvert fishways. Consequently, considerable emphasis must be placed on retaining as many qualities of the original stream channel as possible at each crossing.

Migrating fish must negotiate the culvert outlet, the culvert barrel and the culvert inlet before successfully passing upstream. Hydraulic conditions, such as water velocities and depths, at each one of these three locations must be suitable for passage at the highest and lowest stream flows expected during fish migration. Fish need to swim continuously for the entire culvert length when no resting opportunities are available. Culvert length and velocities, as well as maximum distance that fish are able to swim, determine whether fish can pass through a culvert once they enter it.

For culverts, the following three approaches need to be assessed in arriving at a culvert design that satisfies engineering, economic, and fish passage requirements.

a) plain culvert that meets fish passage velocity (usually 1.2 m/s or less) and minimum water depth criteria (usually 0.2 m at inlet, barrel, and outlet).

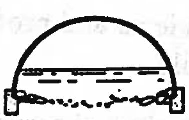
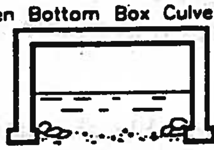
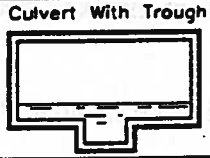
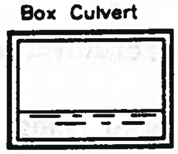
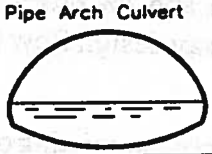
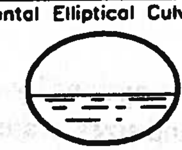
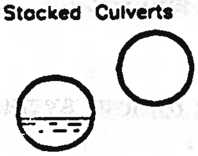
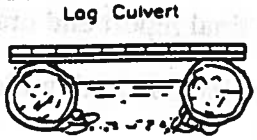
b) stream simulation approach where the status quo in the stream is preserved, i.e. average stream width and slope are maintained up to the fish passage design flow, and stream substrate is kept from washing out either by supports fixed at the culvert bottom or by large stable riprap.

c) culvert with fish passage devices.

The plain culvert rarely meets fish passage criteria particularly for small fish. Stream simulation or fish passage devices are usually needed. During the design of the Liard Highway, Northwest Territories, Canada, the "stream simulation" concept was advanced for fish passage at culvert crossings. The stream simulation concept uses stream dimensions to size the culvert and uses large riprap within the culvert to resemble passable natural rapids. In practice, a culvert is selected which is of sufficient size to maintain average stream width and cross-sectional area for the stream discharge during fish migrations. The culvert is (a) set at the average stream slope for the site, (b) placed below streambed, and (c) filled to stream grade. The top layer of the fill is non-uniformly laid riprap, large enough to be stable during the culvert design discharge.

Culverts are the most popular stream crossing structure over other alternatives for economic reasons. The final stream crossing alternative is based on a need for a crossing structure, hydrological conditions, economic factors related to installation and maintenance of the structure, and the natural resource value of the stream. Table 3.1 shows the many types of culverts that are commonly installed along with hydraulic and fisheries design considerations. Designing a culvert that is both economical and allows for the successful movement of fish is not always successful. From an environmental point of view the preferred stream crossing structure is a bridge, especially if there is a known fisheries resource. However, if a culvert is properly designed and installed, it is an acceptable stream crossing structure from both an environmental and economic point of view. Field studies with various fish species have tested culvert fishway designs and demonstrated successful applications of fish passage technology.

Table 3.1 Fisheries and hydraulic considerations for various types of culverts.

TYPE OF CULVERT	FISHERIES CONSIDERATIONS	HYDRAULIC CONSIDERATIONS
Open Bottom Pipe Arch 	- Retains natural stream bed and gradient. - Water velocities not significantly changed. - Can be designed to maintain normal stream width up to fish passage design flow.	- Wide bottom area enables passage of high flows while minimizing increases in flow depth. - Large waterway opening for low clearance installations.
Open Bottom Box Culvert 	- Retains natural stream bed and gradient. - Water velocities not significantly changed. - Can be designed to maintain normal stream width up to fish passage design flow.	- Can be designed to maintain normal width of the stream channel. - When a large end area is required in low fill, box culverts can be put side by side.
Box Culvert With Trough 	- Trough concentrates water maintaining fish passage even in low flows. - Baffles or culvert fishway can be installed inside. - Can be embedded and designed to simulate stream up to fish passage design flow.	- Can be designed to maintain normal width of the stream channel. - When a large end area is required in low fill, box culverts can be put side by side. - Note: Trough can fill with bed load material and create maintenance problems.
Box Culvert 	- Limits fish passage during low flow due to decreased flow depths. - Baffles or culvert fishway can be installed inside. - Can be embedded and designed to simulate stream up to fish passage design flow.	- Can be designed to maintain normal width of the stream channel. - When a large end area is required in low fill, box culverts can be put side by side.
Pipe Arch Culvert 	- Wide, flat profile allows improved fish passage by back watering the structure. - Baffles or culvert fishway can be installed inside. - Can be embedded and designed to simulate stream up to fish passage design flow.	- Wide bottom area of culvert enables passage of high flows while minimizing increases in the flow depth. - Large waterway opening for low clearance installations.
Horizontal Elliptical Culvert 	- Represents a compromise between pipe arch and round culvert cross-section. - Baffles or culvert fishway can be installed inside. - Can be embedded and designed to simulate stream up to fish passage design flow.	- Squat profile useful in low fill elevations. - Shape results in deeper water depth than pipe arch, but does not offer as broad a bottom area.
Stacked Culverts 	- Allows for fish passage during wider range of flows than single culvert. - Baffles or culvert fishway can be installed inside. - Can be embedded and designed to simulate stream up to fish passage design flow.	- Same hydraulic properties as type of single culvert used (Round, Box, etc.)
Round Metal Culvert 	- Baffles or culvert fishway can be installed inside. - Can be embedded and designed to simulate stream up to fish passage design flow.	- Generally constricts stream width and creates high flow velocities with increased chance of scour. - Concentrates water during low flows.
Log Culvert 	- Ensure proper decking to avoid introduction of sediment and debris into the stream. - Maintains natural stream bed and gradient.	- Exercise care not to restrict stream width. - Normally only used as a temporary facility.

3.2 Design process

The information required and the design steps needed to design a fishway for dams, weirs or culverts are outlined below:

1. Obtain a) maps of the project location and drainage basin, b) plan views and profiles of the proposed or existing dam, weir or culvert, c) aerial photos, if available.
2. List fish species which require access to habitat upstream of the project site and the main purpose for such access (e.g. spawning); provide population estimates if available, minimum and maximum length of the species considered for passage.
3. Describe the migration period for each species by giving, where possible, the dates for the start, peak, and end of migration, associated water temperatures, and estimates of peak migrant numbers.
4. Show, whenever possible, locations of spawning, rearing and feeding areas upstream, downstream and at the project site.
5. Perform a flow frequency analysis for the existing or proposed dam, weir or culvert and estimate the following:
   a) low, average, and high flows (e.g. flows at 98-95% probability of being equalled or exceeded, mean annual flood, bankfull discharge, flows at 10% and 2% probability),
   b) dam, weir or culvert design flow (e.g. 1:50 year flood) and fishway design flow (e.g. 3-day delay for 1:10 year flood).
6. Prepare stage-discharge relationships for the headwater and tailwater of the existing or proposed dam, weir or culvert.
7. Examine various design alternatives and prepare a short list of feasible options by considering site conditions and dam, weir or culvert characteristics, fish species and sizes, water levels and flows, fish behaviour and stamina, debris and ice, bank protection and stream scour or sedimentation.
8. Prepare a discharge rating curve and characteristic velocity profiles for low, average and high flows for each feasible option.
9. Prepare preliminary engineering report, drawings, and estimate costs. Show fishway dimensions, inverts and elevations, provide plan, side and cross-sectional views, stream bed and bank protection measures and fish passage devices.
10. Ensure review of the preliminary report and drawings. Prepare final report and drawings.
11. Develop a monitoring and evaluation program where desirable; include both biological and hydraulic parameters.
12. Provide a regular maintenance program, particularly to alleviate ice and debris problems.

4 FISHWAY DESIGN FLOW

One of the important tasks in designing a hydraulic structure is the estimation of the design flow through flood frequency analysis. Design flows through fishways are estimated in similar ways except that stream flows during the fish migration period are of primary interest. Another factor that affects the choice of stream flows for analysis is the biological effect of migration delay. Some spawning fish may be able to tolerate short delays in migration. Depending on the species involved excess delay may lead to spawning in marginal areas, reabsorption of spawn, depletion of energy reserves or even mortality. In many cases, particularly with Pacific salmon no delay is required by regulatory agencies. A delay period of less than three days in annual spawning migrations is usually accepted for several freshwater species. Delays longer than three days may be acceptable with 1:10 year frequency. These two criteria are used whenever sufficient data exist to estimate the maximum flow that is likely to prevail at the time of fish migration. This flow, may be used as fishway design flow, and can be estimated directly from existing or reconstructed daily flow records for each species and migration period. Design flows for other delay periods may be estimated in a similar manner.

To create a three day delay discharge frequency curve, first find the three day delay discharge value, Q_{3d}, for each year. Q_{3d} is the largest discharge value which is equalled or exceeded three times in three consecutive days over the fish migration period during a particular year. Set the initial Q_{3d} value equal to the lowest discharge value from the first three daily discharge values for the migration period. Next, determine the lowest discharge for the next three day period, i.e. the lowest discharge from the second, third and fourth days. Compare this discharge with the initial Q_{3d} value, the larger of the two becomes the new Q_{3d} value. Repeat this process for next three day period. This process of comparing values for 3 consecutive days is repeated for the entire migration period.

The Q_{3d} values for each year are then arranged in order of descending magnitude, the largest ranked as number one and the smallest ranked as number "n". The return period, T, for each Q_{3d} value is calculated by dividing the total number of Q_{3d} values plus one (n+1) by the rank number. For example, the return period for the fourth largest Q_{3d} value based on a 32 year record would be equal to 8.25 years, (32+1)/4. Return period, T, is then plotted against the corresponding value Q_{3d} on a log-log plot. The points usually plot in a straight line. This line is the frequency curve and is used to estimate other Q_{3d} values. The 1:10 year (T = 10), 3 day delay discharge may then be estimated from this frequency curve. Other more sophisticated methods of estimating return period or probability may also be used in constructing the frequency curve.

The following example illustrates the process of estimating the fishway design flow. The Water Survey of Canada hydrometric record for Redearth Creek was examined for daily flows from September 15 to October 31, corresponding to the fall spawning migration period and from May 1 to June 30 corresponding to the spring migration period. For each year of record (1974-1986) the 3-day delay discharge was selected as shown in Table 4.1. Flows for the 13 year record were ranked and the return period calculated (Table 4.1). Values of flows and return periods were then plotted in log-log format and straight lines were fitted through the data (using the power curve) for each migration period (Fig. 4.1). The fish passage design flows were then projected as illustrated in Fig. 4.1 and values of 5.3 m^3/s and 32.5 m^3/s were estimated for the fall and spring spawners respectively. If daily flow records are not available for the stream of interest, the record of another hydrologically similar stream may be used. Flows may then be extrapolated from one stream to the other using methods such as the ratio of drainage areas.

Table 4.1 Redearth Creek three day delay discharge calculation.

Redearth Creek discharge equalled or exceeded once for three consecutive days during the fall and spring fish migration periods (Q_{fall} corresponds to September 15 to October 31; Q_{spring} corresponds to May 1 to June 30). Data from Water Survey of Canada Surface Water Data for Alberta 1974-86.

Year	Q_{fall} (m^3/s)	Q_{spring} (m^3/s)	Rank #	T (years)	Q_{fall} (m^3/s)	Q_{spring} (m^3/s)
1986	2.07	30.1	1	14.00	5.29	36.2
85	4.32	14.8	2	7.00	4.32	30.1
84	3.34	17.5	3	4.67	4.14	23.3
83	1.89	17.8	4	3.50	3.65	22.1
82	3.63	22.1	5	2.80	3.63	17.8
81	2.81	23.3	6	2.33	3.34	17.5
80	4.14	16.9	7	2.00	3.34	16.9
79	1.92	13.8	8	1.75	2.94	15.7
78	3.65	15.7	9	1.56	2.81	14.9
77	3.34	14.0	10	1.40	2.39	14.8
76	5.29	14.9	11	1.27	2.07	14.0
75	2.94	12.7	12	1.17	1.92	13.8
74	2.39	36.2	13	1.08	1.89	12.7

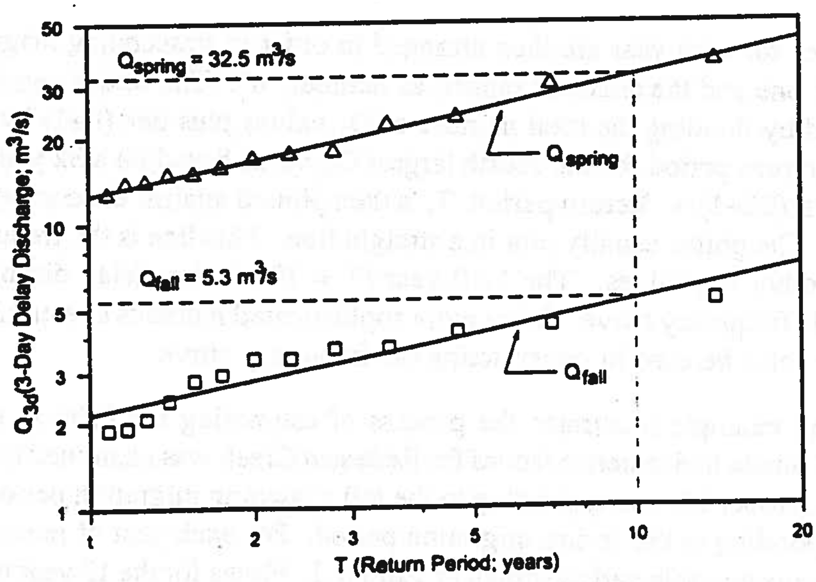


Figure 4.1 Redearth Creek frequency curves (1974-86)

5 FISHWAY HYDRAULICS

The hydraulic characteristics of various types of fishways were studied using geometrically similar scale models. Hydraulic modelling was performed on several variations of vertical slot (18 designs), Denil (6 designs), weir (2 kinds) and culvert (6 kinds) fishways. Discharge rating curves and characteristic velocity profiles for these fishways are available for a wide range of slopes and water depths. Froudian similitude laws were found to reproduce flow phenomena well, and were used for all models to transfer values between model and prototype. In Froudian models gravitational forces predominate, the velocity and time scales are represented by the square root of the geometric scale and the discharge scale is provided by the geometric scale raised to the 5/2 power. Fluid turbulent shear stresses between water jets and recirculating water seem to dominate in fishway flows providing large momentum exchange and high energy dissipation. Neglecting wall shear stresses provides a good approximation for flow analysis. Discharge rating curves were derived using a simple force balance on the predominant flow stream in each fishway type. Applicable to different fishway sizes or scales, dimensionless variables were used to summarize experimental results. For fishway discharge, the corresponding dimensionless variable is usually expressed by:

$$Q_{\bullet} = \frac{Q}{\sqrt{gS_o b_o^5}} \tag{5.1}$$

where Q is fishway discharge, S_o is slope of the fishway bed, b_o is a characteristic width (e.g. fish passage opening, slot width, orifice width, culvert diameter) and g is gravitational acceleration (constant). Dimensionless discharge $Q_{\bullet}$ is a linear or a power function of dimensionless depth, y_o/b_o. For most fishway designs tested velocity profiles along a vertical line exhibit similar geometrical shapes. Velocity profile similarity is a property manifested by a large number of turbulent jet flows. Similarity allows the analysis of velocity profiles using dimensionless variables applicable to various fishway sizes or scales. In a typical velocity profile, dimensionless local velocity, u/u_m, is commonly a linear or power function of dimensionless local depth, y/y_o or y/z_o. Here u is the local velocity at a depth y, u_m is the velocity scale representing the maximum values of u, y_o is the total depth and z_o is the height of baffle or weir in culvert fishways. In plain Denil fishways because u_m is not well defined in the profile it is substituted by u_m', the velocity at 75% of the depth. In vertical slot fishways u and u_m are approximately the same throughout the profile except near the fishway bed. Analogous to the dimensionless discharge $Q_{\bullet}$ defined above, a dimensionless velocity scale was expressed as:

$$U_{\bullet} = \frac{u_m \text{ or } u_m'}{\sqrt{gS_o b_o}} \tag{5.2}$$

The dimensionless velocity scale $U_{\bullet}$ is a linear or power function of y_o/b_o or $Q_{\bullet}$ and provides an estimate of the maximum velocities in a fishway. Velocity profiles in a fishway may be derived from the similarity analysis and the dimensionless velocity scale.

5.1 Vertical Slot Fishways

A vertical slot fishway (Fig. 2.1) consists of a sloping (or stepped) rectangular channel which is partitioned into pools. Water flows down the channel from pool to pool through slots oriented vertically. A water jet is formed at each slot and energy dissipation by jet mixing occurs in each pool. The hydraulic characteristics of several variations of the vertical slot fishway (Fig. 5.1) were studied by scale models. Both "uniform flow", where depth of flow in each pool (y_o) is approximately the same, and "gradually varied" flow, where M1 or M2 - type backwater curves may occur, were studied. Shear stress between the jet and the recirculating mass predominates, while bed or wall shear stress on the jet is negligible in comparison.

Dimensionless discharge (Q_*) varies linearly with relative depth of flow (y_o/b_o) for the 18 designs tested (Table 5.1; Figure 5.2):

$$Q_* = \frac{Q}{\sqrt{gS_o b_o^5}} = \alpha(y_o/b_o) + \beta \qquad (5.3)$$

The maximum velocity in each slot, u_m, is a function of the head drop between pools, h, and is approximated by $\sqrt{2gh}$, if the velocity in the upstream pool is neglected:

$$u_m = \sqrt{2gh} \qquad (5.4)$$

Analysis of "gradually varied" flow conditions is important, particularly at the fishway entrance, where fish attraction velocities are reduced by backwater.

Many of the vertical slot designs tested were selected in order to evaluate how hydraulic characteristics change with pool dimensions and baffle geometry. For example, designs 8-13 were tested primarily to find out how sensitive the standard pool length and width are for satisfactory performance. Designs 14-18 are modified versions of Design 1. From the results summarized in Figure 5.2 and Table 5.1, it appears that a pool width of $8b_o$ and a pool length of $10b_o$ are generally satisfactory. Minor variations in these pool dimensions would not seriously affect fishway hydraulic performance. In addition to the widely used designs 1 and 2, designs 6, 16 and 18 are recommended for practical use.

Figure 5.1 Vertical slot fishway design layouts including circulation patterns in pools.

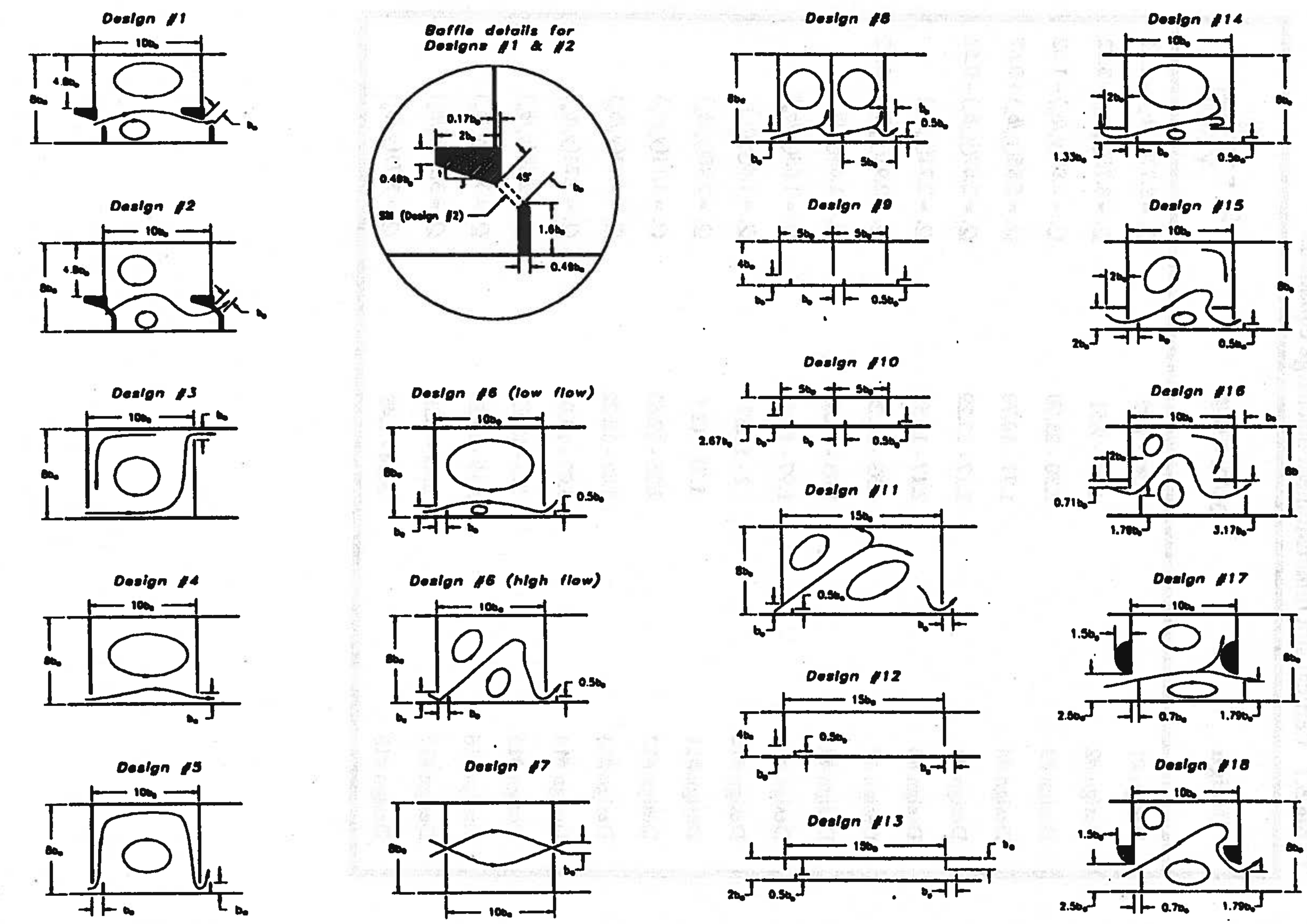
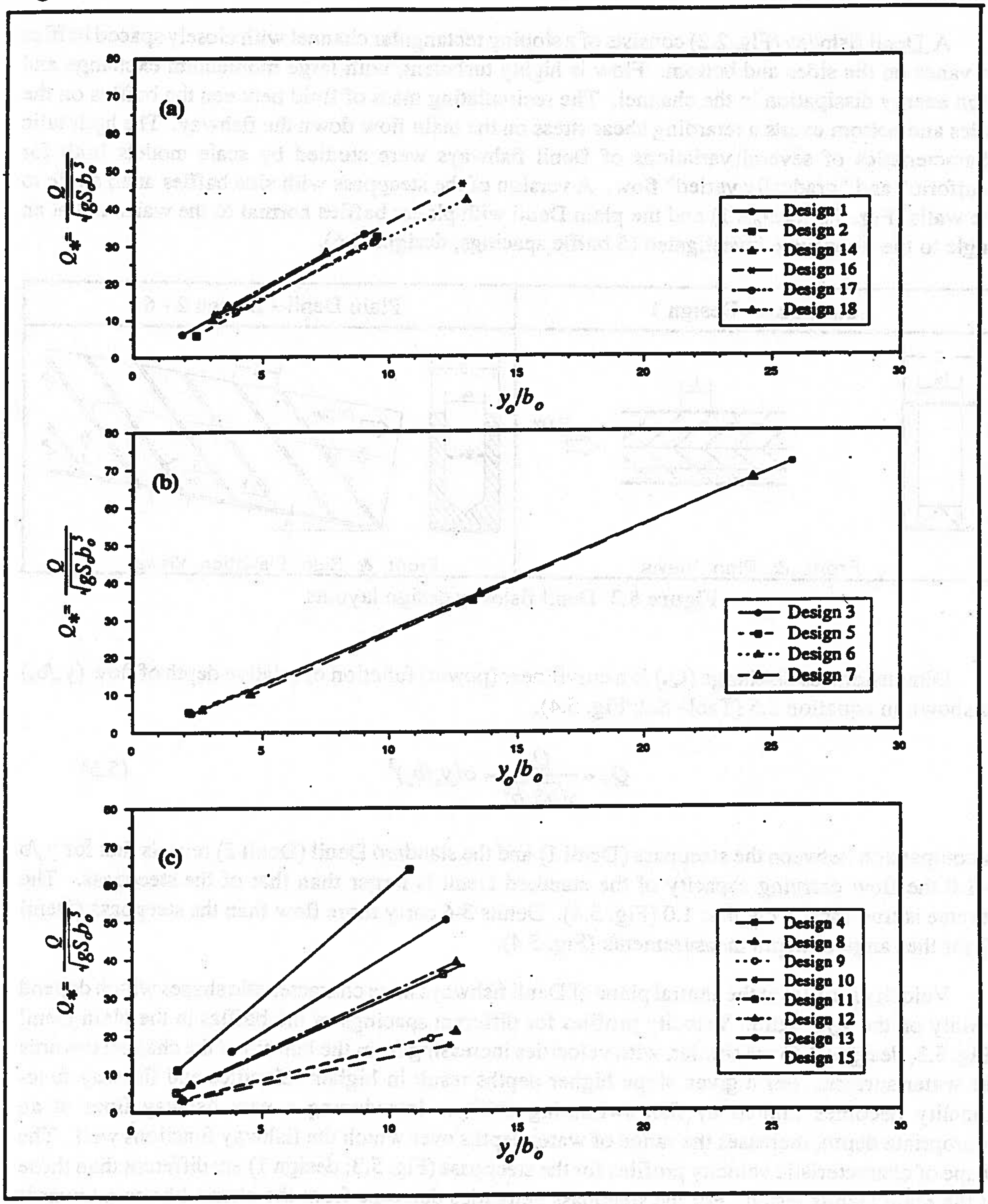


Table 5.1 Vertical slot dimensionless discharge equations.

Design	y_o/b_o range	$Q_* = \frac{Q}{\sqrt{g S_o b_o^5}}$
Design #1	1.90 - 9.02	$Q_* = 3.77(y_o/b_o) - 1.11$
Design #2	2.46 - 9.51	$Q_* = 3.75(y_o/b_o) - 3.52$
Design #3	2.30 - 25.79	$Q_* = 2.84(y_o/b_o) - 1.62$
Design #4	1.77 - 10.79	$Q_* = 5.85(y_o/b_o) + 0.67$
Design #5	2.17 - 13.29	$Q_* = 2.67(y_o/b_o) - 0.52$
Design #6	2.17 - 13.55	$Q_* = 2.71(y_o/b_o)$
Design #7	4.53 - 24.28	$Q_* = 2.91(y_o/b_o) - 3.22$
Design #8	1.93 - 12.62	$Q_* = 1.66(y_o/b_o)$
Design #9	1.97 - 11.61	$Q_* = 1.65(y_o/b_o)$
Design #10	2 - 12.37	$Q_* = 1.4(y_o/b_o)$
Design #11	1.71 - 12.1	$Q_* = 2.98(y_o/b_o)$
Design #12	2.26 - 12.63	$Q_* = 3.11(y_o/b_o)$
Design #13	3.85 - 12.22	$Q_* = 4.13(y_o/b_o)$
Design #14	3.07 - 13.04	$Q_* = 3.21(y_o/b_o)$
Design #15	3.3 - 12.83	$Q_* = 2.89(y_o/b_o)$
Design #16	3.19 - 12.87	$Q_* = 3.59(y_o/b_o)$
Design #17	3.69 - 9.38	$Q_* = 3.27(y_o/b_o)$
Design #18	3.64 - 7.48	$Q_* = 3.71(y_o/b_o)$

Figure 5.2 Variation of dimensionless discharge with relative depth for vertical slot fishways.



5.2 Denil Fishways

A Denil fishway (Fig. 2.2) consists of a sloping rectangular channel with closely spaced baffles or vanes on the sides and bottom. Flow is highly turbulent, with large momentum exchange and high energy dissipation in the channel. The recirculating mass of fluid between the baffles on the sides and bottom exerts a retarding shear stress on the main flow down the fishway. The hydraulic characteristics of several variations of Denil fishways were studied by scale models both for "uniform" and "gradually varied" flow. A version of the steeppass with side baffles at an angle to the walls (Fig. 5.3; design 1) and the plain Denil with planar baffles normal to the walls and at an angle to the floor were investigated (5 baffle spacings; designs 2-6).

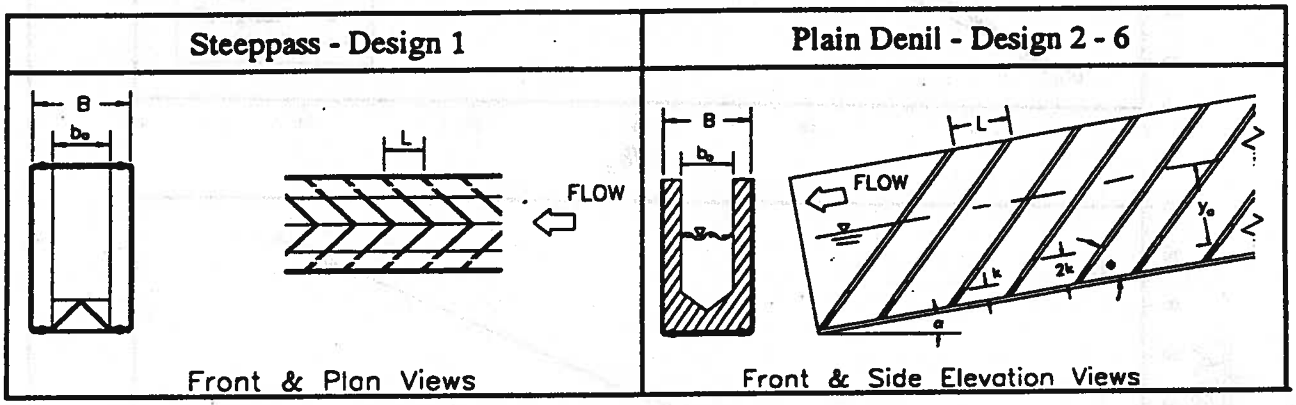


Figure 5.3 Denil fishway design layouts.

Dimensionless discharge (Q_*) is a curvilinear (power) function of relative depth of flow (y_o/b_o) as shown in equation 5.5 (Table 5.2; Fig. 5.4).

$$Q_* = \frac{Q}{\sqrt{g S_o b_o^5}} = \alpha (y_o/b_o)^{\beta} \qquad (5.5)$$

A comparison between the steeppass (Denil 1) and the standard Denil (Denil 2) reveals that for y_o/b > 1.0 the flow carrying capacity of the standard Denil is larger than that of the steeppass. The reverse is true for $0.5 < y_o/b < 1.0$ (Fig. 5.4). Denils 3-6 carry more flow than the steeppass (Denil 1) for the range of depth measurements (Fig. 5.4).

Velocity profiles at the central plane of Denil fishways have characteristic shapes which depend mainly on the y_o/b_o ratio. Velocity profiles for different spacings of the baffles in the plain Denil (Fig. 5.3; designs 2-6) are similar, with velocities increasing from the bottom of the channel towards the water surface. For a given slope higher depths result in higher velocities and fishway functionality becomes limited by fish swimming ability. Introducing a new fishway floor at an appropriate depth, increases the range of water depths over which the fishway functions well. The shape of characteristic velocity profiles for the steeppass (Fig. 5.3; design 1) are different than those of the plain Denils tested. For the steeppass velocities decrease from the channel bottom towards the surface for $y_o/b_o \le 1.2$ (and $Q_* \le 1.2$). For higher y_o/b_o ratios, flow is divided into lower and upper regions, with velocity profiles becoming roughly symmetrical and the maximum velocity at mid-depth.

Table 5.2 Denil fishway dimensionless discharge equations and velocity scales. Note that Denil 1 is the same as steeppass (Model A); u_m applies only to Denil 1 and u_m' to Denil 2-6.

Design	B/b_o	L/b_o	y_o/b_o range	$Q_* = \frac{Q}{\sqrt{gS_ob_o^5}}$	$U_* = \frac{u_m \text{ or } u_m'}{\sqrt{gS_ob_o}}$
Denil 1	1.58	0.715	0.1 - 4.0	$Q_* = 0.97(y_o/b_o)^{1.55}$	$U_* = 1.43(Q_*)^{0.48}$
Denil 2	1.58	0.715	0.5 - 5.8	$Q_* = 0.94(y_o/b_o)^2$	$U_* = 0.76(Q_*)^{0.61}$
Denil 3	1.58	1.07	0.5 - 1.2	$Q_* = 1.12(y_o/b_o)^{1.16}$	
Denil 4	2	0.91	1 - 5	$Q_* = 1.01(y_o/b_o)^{1.92}$	$U_* = 0.84(Q_*)^{0.58}$
Denil 5	2	1.82	1.3 - 4.6	$Q_* = 1.35(y_o/b_o)^{1.57}$	$U_* = 0.67(Q_*)^{0.57}$
Denil 6	2	2.58	0.8 - 4.3	$Q_* = 1.61(y_o/b_o)^{1.43}$	$U_* = 1.37(Q_*)^{0.25}$

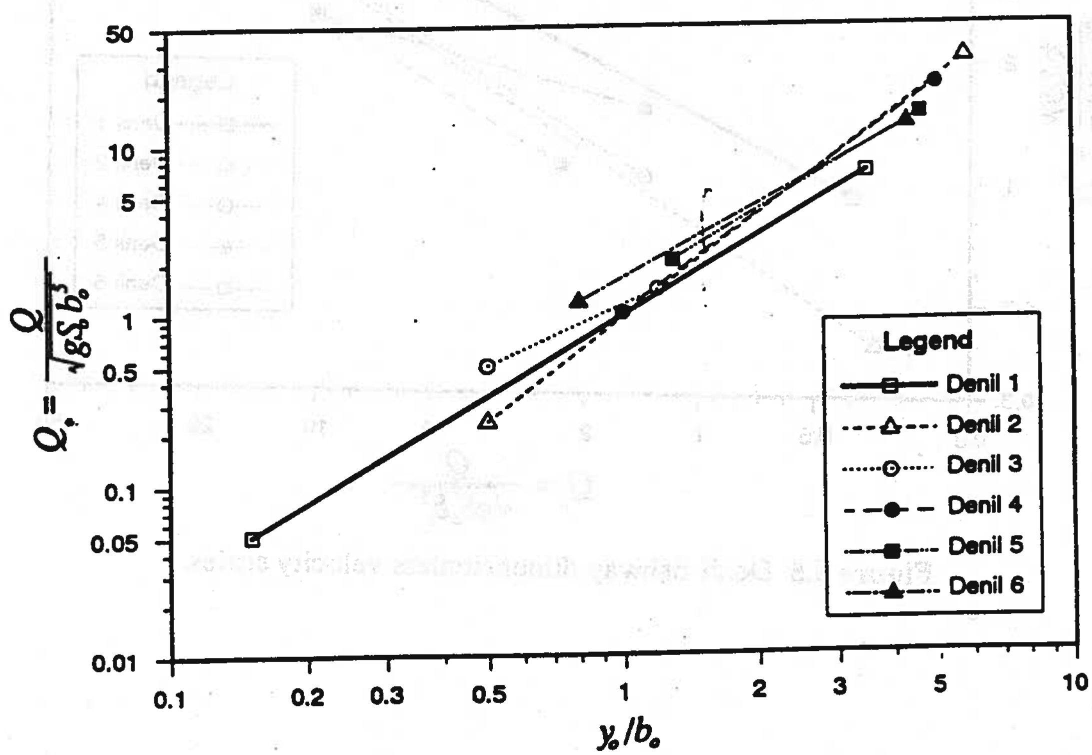


Figure 5.4 Denil fishway dimensionless discharge curves.

Dimensionless velocity scale (U_*) is a curvilinear (power) function of dimensionless discharge (Q_*) as shown in Fig. 5.5 and Table 5.2:

$$U_* = \frac{u_m \text{ or } u_m'}{\sqrt{g b_o S_o}} = \alpha Q_*^{\beta} \tag{5.6}$$

It is important to note that both u_m and u_m' were estimated from velocity profiles at a centreline vertical. But u_m is the maximum velocity in the profile and applies only to Denil 1 (steeppass), while u_m' is the maximum velocity at 75% of the water depth in the fishway and applies to Denil 2-6. Because, the two velocity scales u_m and u_m' are defined differently, values of the dimensionless velocity scales (U_*) from Fig. 5.5 and Table 5.2 can only be compared directly for Denil 2-6. For example, U_* for Denil 6 appears higher than U_* for Denil 2 and 4 for $Q_*<5$ and lower for $Q_*>5$.

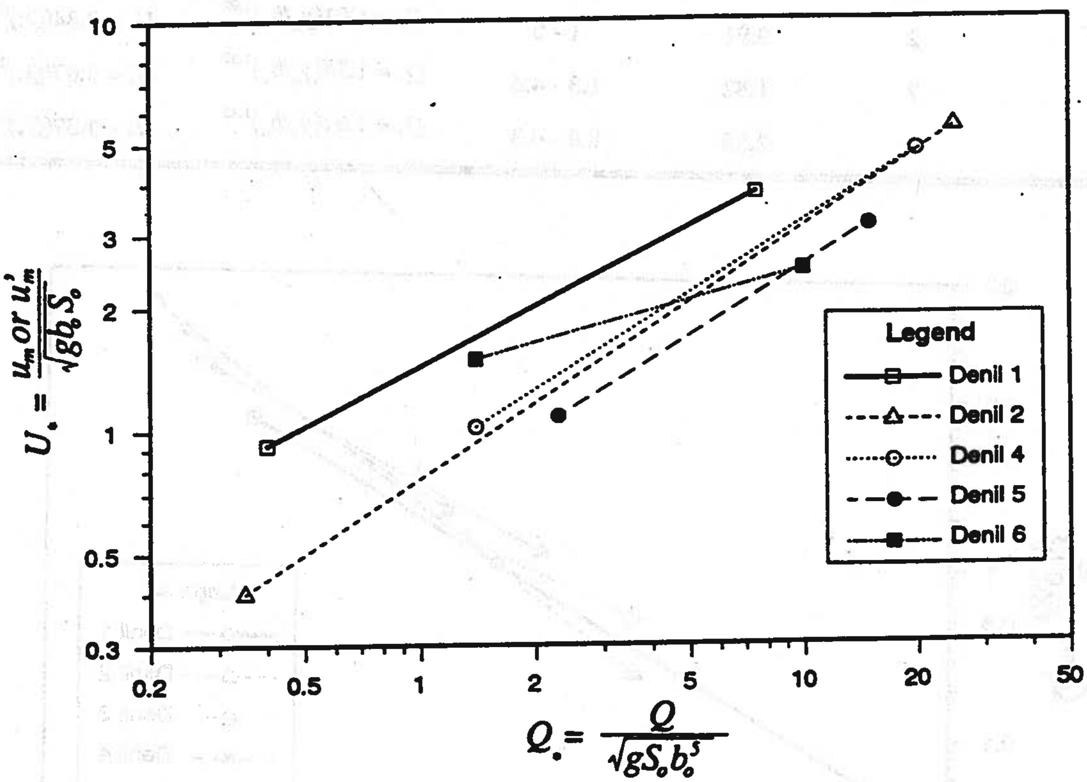


Figure 5.5 Denil fishway dimensionless velocity scales.

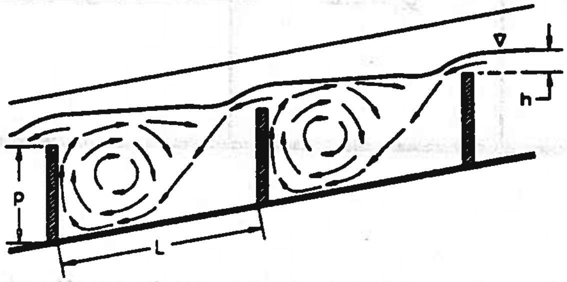
5.3 Weir Fishways

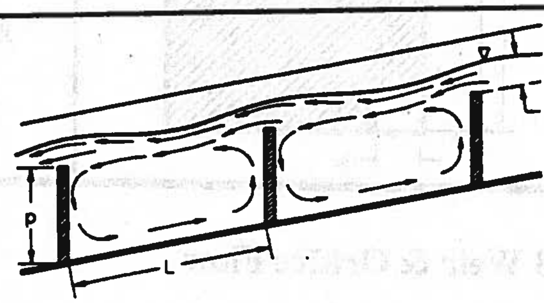
A weir fishway (Fig. 2.3) consists of a sloping (or stepped) rectangular channel partitioned into pools by weirs. Water flows a) over the weirs, b) through orifices placed at the bottom of the weirs, or c) both over the weirs and through the orifices. Weir design modifications to stimulate maximum leaping ability by fish or to swim through chutes have also been reported.

5.3.1 Weir Flow

Flow over the weirs is either "plunging" or "streaming", depending on the depth of flow for a given slope and pool length. In the plunging mode hydraulic head, h, above each weir produces a water jet, dissipating energy by turbulent mixing and diffusion. The water level below each weir is generally lower than the weir crest and the weir resembles the classical free-flow case. Limited experimental results indicate that the discharge rating curve is similar to the one for sharp-crested weirs. In the streaming mode a surface jet with approximately uniform depth, d, flows over recirculating water in the pools. The turbulent shear stress between the surface jet and the recirculating mass in the pool dominates while side wall shear stress may be neglected. The dimensionless discharge for the plunging (Q_p) and streaming (Q_s) modes are given in Table 5.3a where B is the width of the weir. Maximum velocity in the plunging mode occurs near the top of the weir and decreases to about half at the water surface. In the streaming mode the average velocity in the jet is given as V in Table 5.3a.

Table 5.3a Weir fishways - dimensionless discharge equations for flow over the weir.

<table>
<tr><th>Plunging - Weir Flow</th><th>Streaming - Weir Flow</th></tr>
<tr><td>

$$Q_p = \frac{Q_w}{Bh^{1.5}\sqrt{g}} = 0.61$$
$$u_m = \sqrt{2gh}$$</td><td>

$$Q_s = \frac{Q_w}{Bd^{1.5}\sqrt{gS_o}} = 1.5\sqrt{L/d}$$
$$V = \frac{Q_w}{Bd}$$</td></tr>
</table>

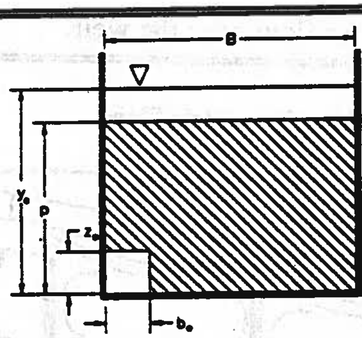
The transition between plunging and streaming flow is characterized by acceleration over each weir crest, a standing wave below each weir, and a surface jet depth that varies cyclically along the fishway. The dimensionless discharge (Q_t) during the transition from plunging to streaming flow can be calculated using equation (5.7).

Transitional - Weir Flow: $$Q_t = \frac{Q_w}{B S_o L^{1.5}\sqrt{g}} = 0.25 \qquad (5.7)$$

5.3.2 Orifice Flow

Orifices at each weir, close to the fishway bed, are frequently used. Fishway discharge for orifice flow may be analyzed as: a) a vertical slot for $y_o < z_o$, b) a submerged jet for $y_o > 2z_o$ and c) as an unsubmerged jet for in between depths where the orifice is submerged only on the upstream side. Table 5.3b summarizes dimensionless discharge (Q_d) rating curves for these cases with a standard pool and orifice configuration of $z_o = b_o$, $L = (6 \text{ to } 10)\, b_o$, $B = (5 \text{ to } 10)\, b_o$, $p = (3.5 \text{ to } 4) b_o$, with a small ($0.5 b_o$ wide) deflecting baffle a short distance ($1.0 b_o$) downstream of the orifice, similar to vertical slots.

Table 5.3b Weir fishways - dimensionless discharge equations for flow through the orifice.

Orifice Flow	Water depth	$Q_i = \frac{Q_o}{\sqrt{g S_o b_o^3}}$	Flow
	$y_o < z_o$	$Q_i = 1.94\left(\frac{y_o}{b_o}\right)$	vertical slot
	$y_o > 2z_o$	$Q_i = 2.25$	submerged jet

5.3.3 Weir & Orifice Flow

For a weir fishway with both weir and orifice flow, interaction between the hydraulic characteristics of the orifice and the weir can be neglected. The weir discharge (Q_w) can be calculated using the plunging, streaming, or transitional flow equations, and the orifice discharge (Q_o) is calculated using the dimensionless discharge equation for submerged jet flow (Q_i=2.25: Table 5.3b). The total discharge through the fishway is the sum of the weir and orifice discharges.

$$Q = Q_w + Q_o \qquad (5.8)$$

5.4 Culvert Fishways

A culvert fishway (Fig. 2.4) consists of a sloping pipe flowing partly full with regularly spaced baffles or weirs on the bottom. Several baffle and weir arrangements were studied and are illustrated in Fig. 5.6.

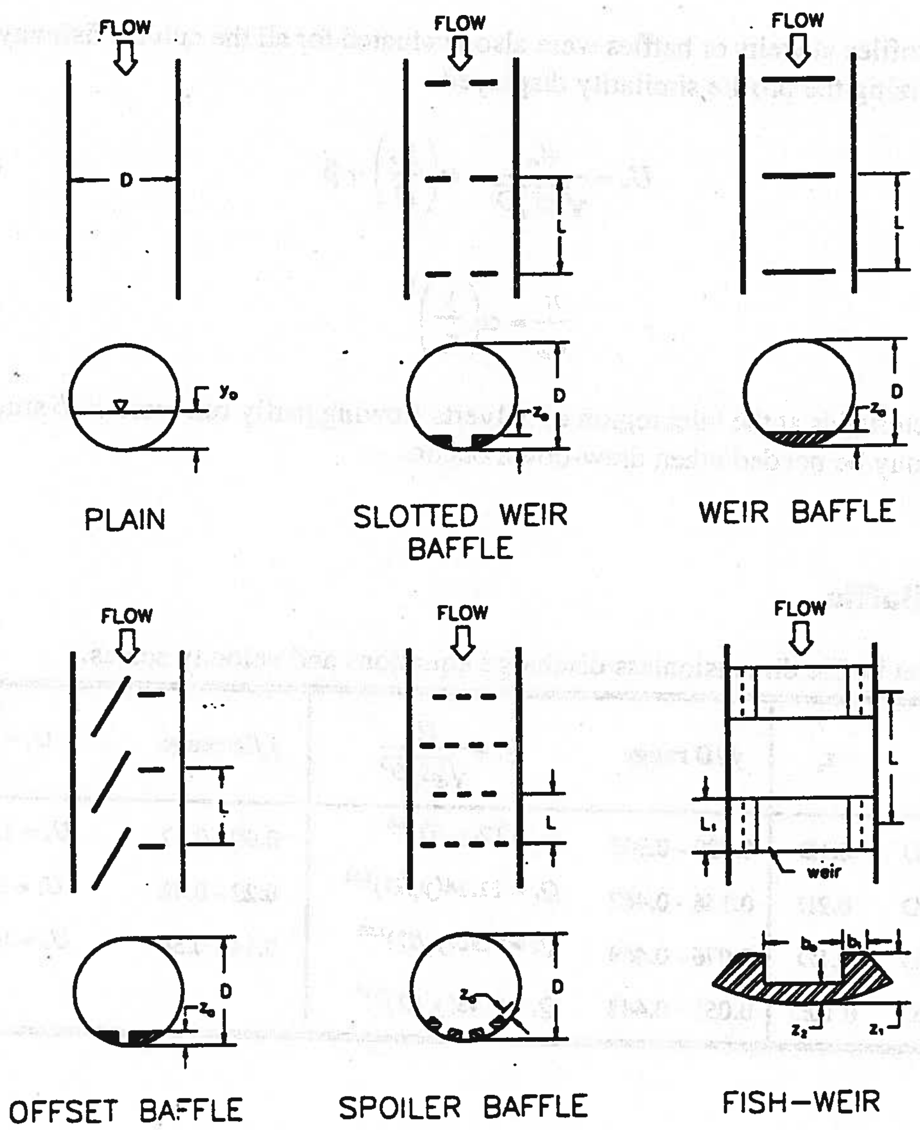


Figure 5.6 Culvert fishways - baffle and weir arrangements.

Flow analysis for culvert fishways is similar to weir fishways for depths higher than the baffle or weir height (z_o). Streaming flow occurs for all but the low depths, since z_o is 0.1 to 0.15 of the culvert diameter (D). Dimensionless discharge (Q_*) vs relative depth (y_o/D) takes the form of a power curve. Discharge rating curves and dimensionless velocity scales for the various designs are presented in sections 5.4.1 - 5.4.5.

$$Q_* = \frac{Q}{\sqrt{gS_oD^5}} = \alpha\left(\frac{y_o}{D}\right)^{\beta} \tag{5.10}$$

Velocity profiles at weirs or baffles were also evaluated for all the culvert fishways tested and analyzed by utilizing the profile similarity displayed.

$$U_* = \frac{u_m}{\sqrt{gS_oD}} = \alpha\left(\frac{y_o}{D}\right) + \beta \tag{5.11}$$

$$\frac{u}{u_m} = \alpha\left(\frac{y}{z_o}\right)^{\beta} \tag{5.12}$$

The flow characteristics at the inlet region of culverts flowing partly full were also studied. Closer baffle spacing may be needed when draw-down occurs.

5.4.1 Offset Baffle

Table 5.4 Offset baffle dimensionless discharge equations and velocity scales.

Design	L	z_o	y_o/D range	$Q_* = \frac{Q}{\sqrt{gS_oD^5}}$	y_o/D range	$U_* = \frac{u_m}{\sqrt{gS_oD}}$
D-1	0.67D	0.1D	0.029 - 0.565	$Q_* = 12(y_o/D)^{2.6}$	0.09 - 0.37	$U_* = 12.8(y_o/D)$
D-2	0.67D	0.2D	0.146 - 0.462	$Q_* = 11.14(y_o/D)^{3.63}$	0.22 - 0.42	$U_* = 5.6(y_o/D)$
D-3	0.33D	0.1D	0.076 - 0.469	$Q_* = 9.38(y_o/D)^{2.62}$	0.14 - 0.34	$U_* = 10.2(y_o/D)$
D-4	1.01D	0.10D	0.055 - 0.448	$Q_* = 9.48(y_o/D)^{2.57}$		

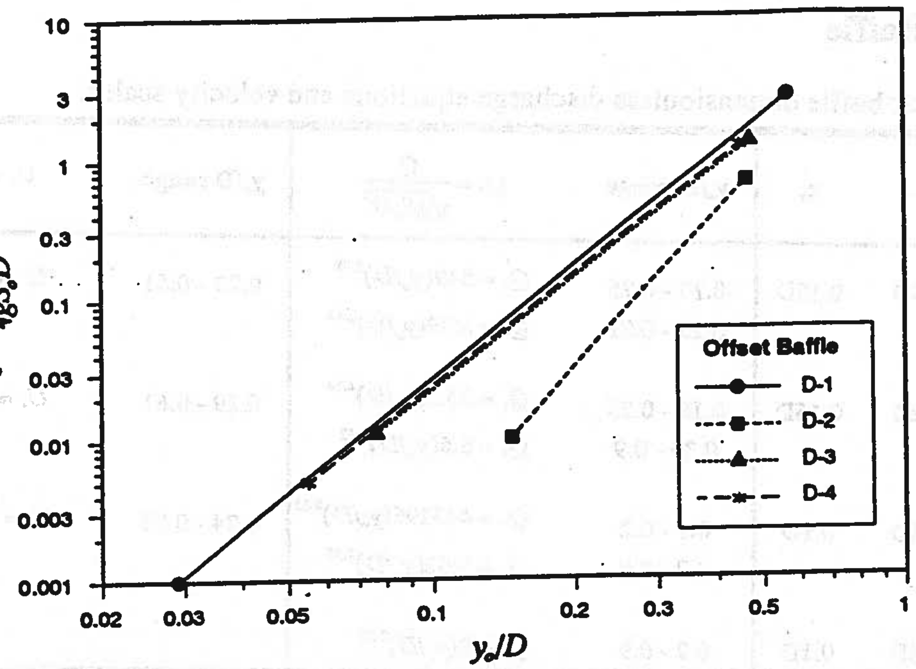


Figure 5.7 Offset baffle dimensionless discharge curves.

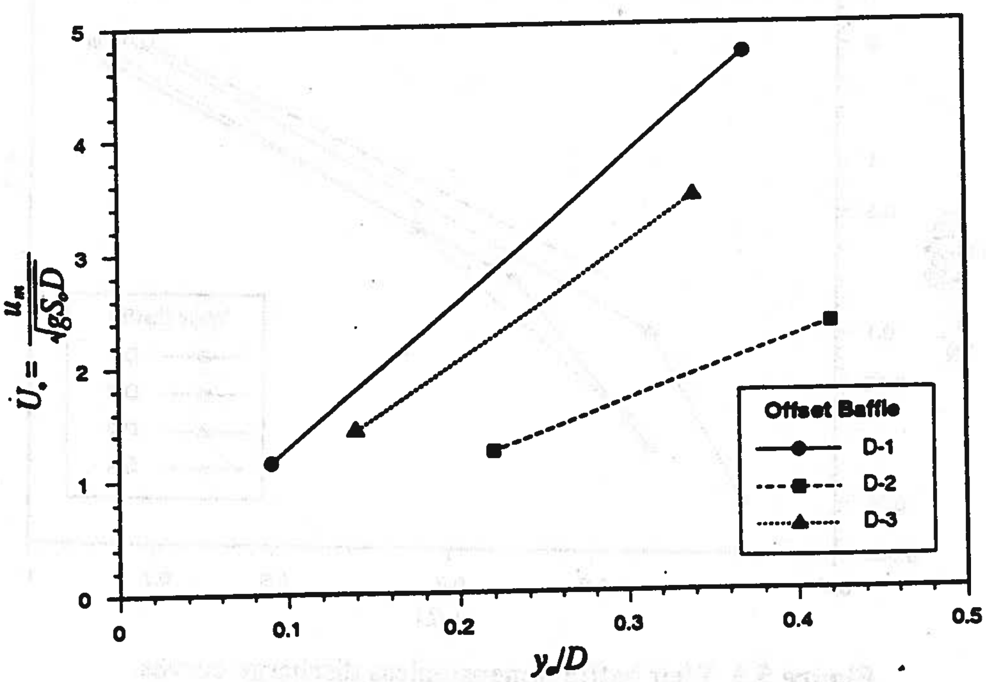


Figure 5.8 Offset baffle dimensionless velocity scales.

5.4.2 Weir Baffle

Table 5.5 Weir baffle dimensionless discharge equations and velocity scales.

Design	L	z_o	y_o/D range	$Q_* = \frac{Q}{\sqrt{gS_oD^3}}$	y_o/D range	$U_* = \frac{u_m}{\sqrt{gS_oD}}$
D-1	0.6D	0.15D	0.17 - 0.25 0.25 - 0.81	$Q_* = 549(y_o/D)^{5.78}$ $Q_* = 5.39(y_o/D)^{2.43}$	0.23 - 0.61	$U_* = 8.6(y_o/D)$
D-2	1.2D	0.15D	0.18 - 0.35 0.35 - 0.9	$Q_* = 35.3(y_o/D)^{4.14}$ $Q_* = 6.6(y_o/D)^{2.62}$	0.29 - 0.61	$U_* = 8.6(y_o/D)$
D-3	0.6D	0.1D	0.1 - 0.2 0.2 - 0.9	$Q_* = 443196(y_o/D)^{8.63}$ $Q_* = 8.62(y_o/D)^{2.53}$	0.24 - 0.53	$U_* = 10.9(y_o/D)$
D-4	1.2D	0.1D	0.2 - 0.9	$Q_* = 9(y_o/D)^{2.36}$		

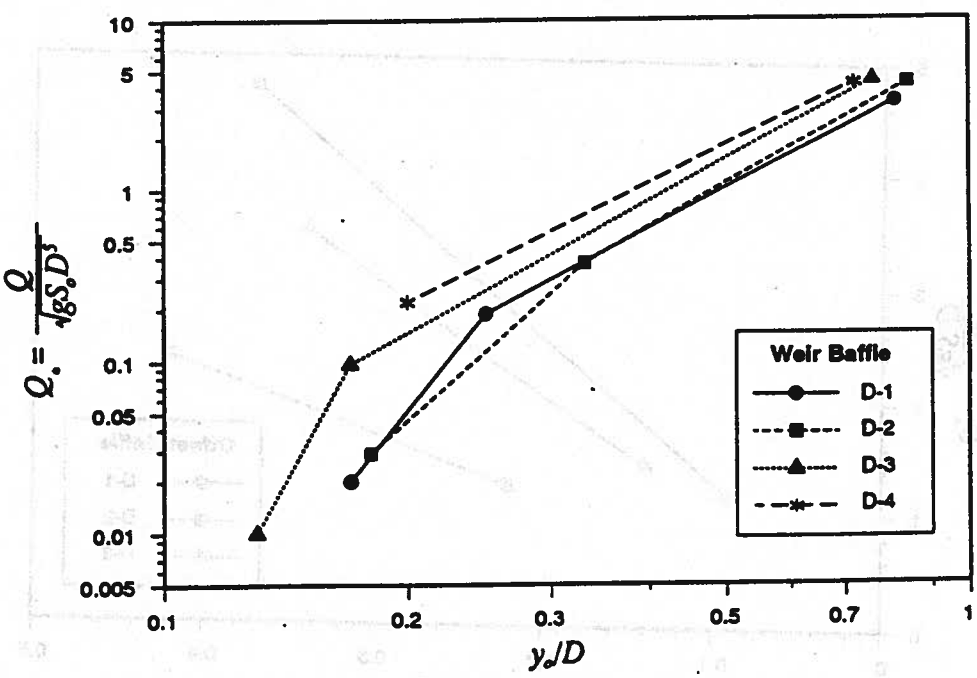


Figure 5.9 Weir baffle dimensionless discharge curves.

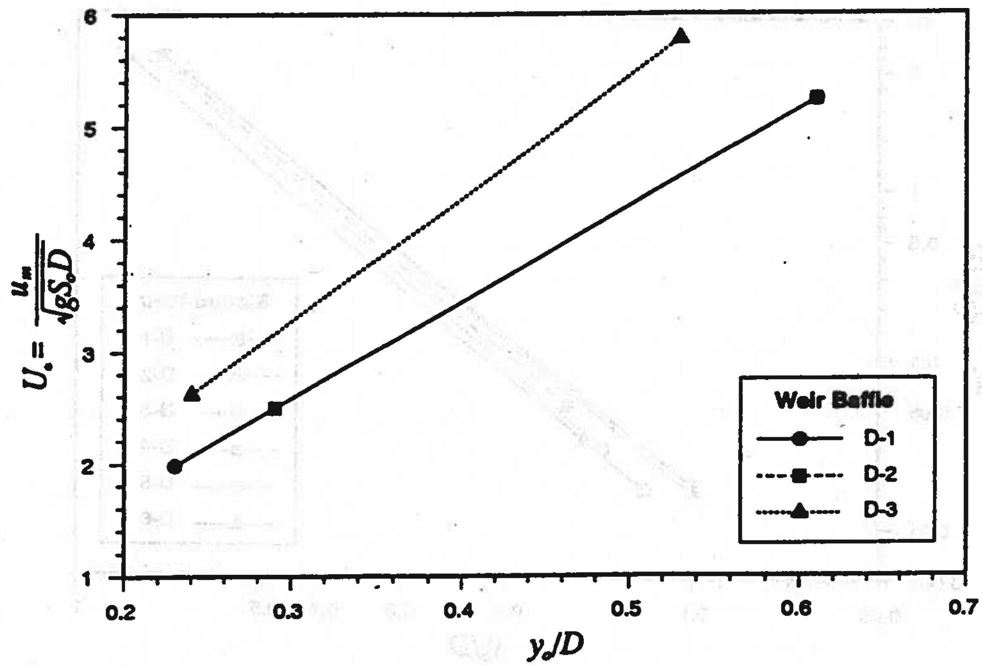


Figure 5.10 Weir baffle dimensionless velocity scales.

5.4.3 Slotted Weir Baffle

Table 5.6 Slotted weir baffle dimensionless discharge equations and velocity scales.

Design	L	z_o	y_o/D range	$Q_* = \frac{Q}{\sqrt{gS_oD^5}}$	y_o/D range	$U_* = \frac{u_m}{\sqrt{gS_oD}}$
D-1	0.6D	0.15D	0.12 - 0.85	$Q_* = 9.2(y_o/D)^{3.0}$	0.15 - 0.79	$U_* = 9.2(y_o/D)$
D-2	0.3D	0.15D	0.15 - 0.84	$Q_* = 9.2(y_o/D)^{3.0}$	0.18 - 0.78	$U_* = 9.2(y_o/D)$
D-3	1.2D	0.15D	0.14 - 0.76	$Q_* = 12.4(y_o/D)^{3.1}$	0.13 - 0.72	$U_* = 10.9(y_o/D)$
D-4	2.4D	0.15D	0.16 - 0.68	$Q_* = 13.8(y_o/D)^{3.1}$	0.14 - 0.67	$U_* = 12.7(y_o/D)$
D-5	0.6D	0.10D	0.10 - 0.73	$Q_* = 13.7(y_o/D)^{2.9}$	0.12 - 0.68	$U_* = 11.4(y_o/D)$
D-6	1.2D	0.10D	0.10 - 0.67	$Q_* = 14.9(y_o/D)^{3.0}$	0.13 - 0.68	$U_* = 12.4(y_o/D)$

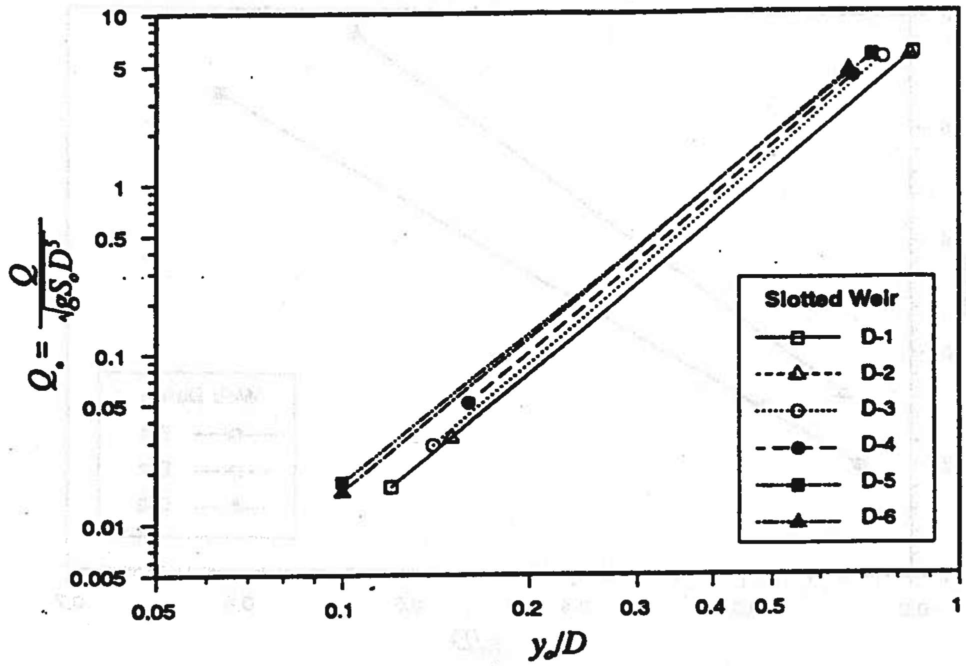


Figure 5.11 Slotted weir baffle dimensionless discharge curves.

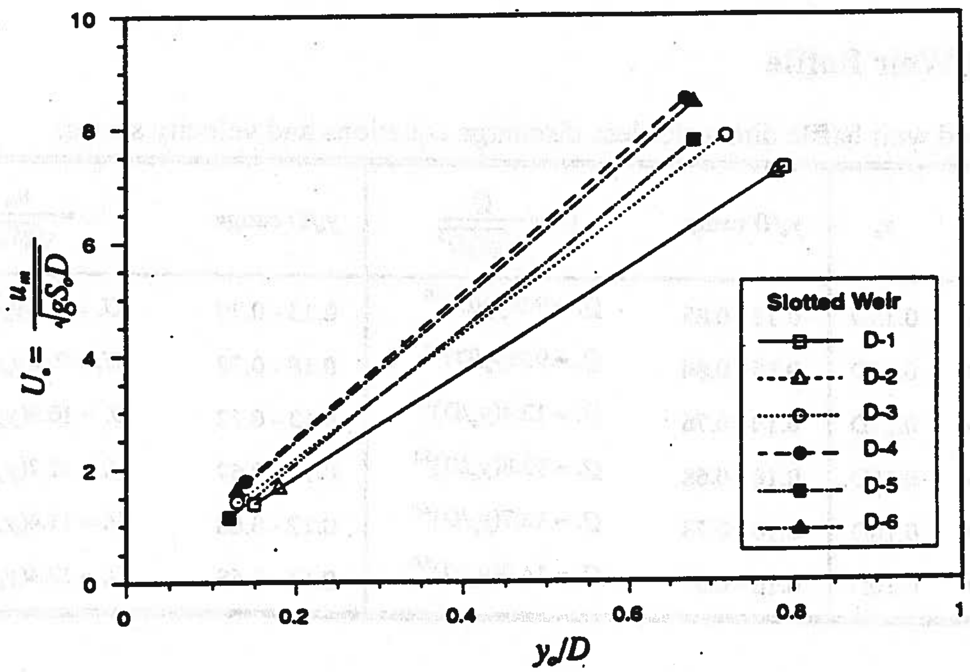


Figure 5.12 Slotted weir baffle dimensionless velocity scales.

5.4.4 Fish-Weir

For each design: $L_1 = 0.16$ D, $z_1 = 0.14D$, $z_2 = 0.06D$, $b_o = 0.22D$, $b_1 = 0.069D$ (see Fig. 5.6).

Table 5.7 Fish-weir dimensionless discharge equations and velocity scales.

Design	L	y_o/D range	$Q_* = \frac{Q}{\sqrt{gS_oD^5}}$	$U_* = \frac{u_m}{\sqrt{gS_oD}}$
D-1	2.39D	0.06 - 0.4	$Q_* = 17.63(y_o/D)^{2.88}$	$U_* = 4.8Q_*^{0.25}$
D-2	1.2D	0.09 - 0.4	$Q_* = 38.99(y_o/D)^{3.57}$	$U_* = 3.5Q_*^{0.25}$
D-3	0.6D	0.08 - 0.4	$Q_* = 21.2(y_o/D)^{3.2}$	$U_* = 3.5Q_*^{0.25}$
D-4	1.79D	0.045 - 0.4	$Q_* = 11.57(y_o/D)^{2.53}$	

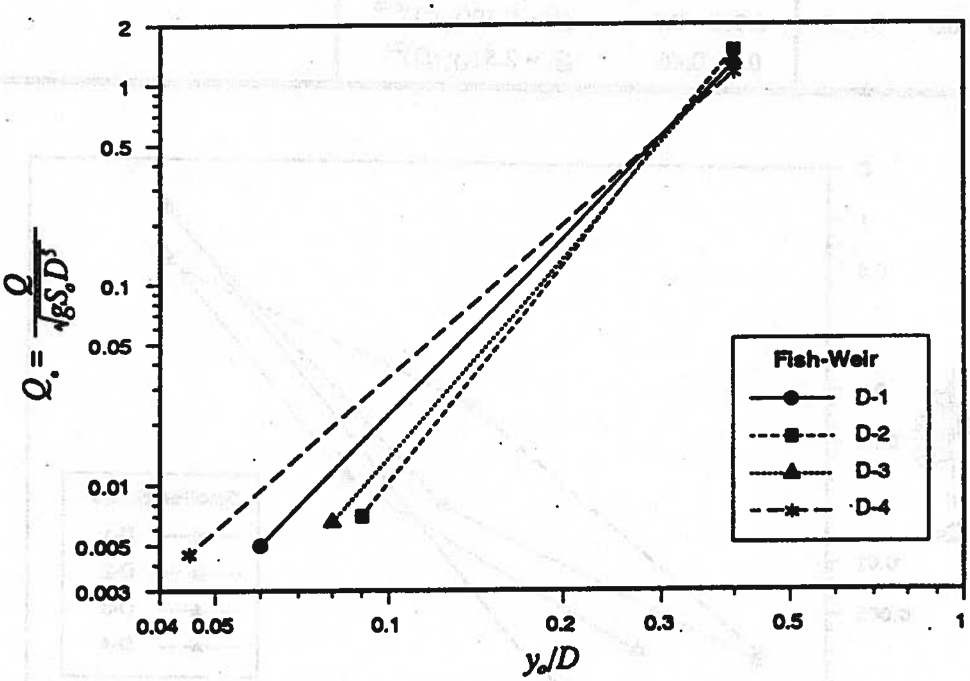


Figure 5.13 Fish-weir dimensionless discharge curves.

5.4.5 Spoiler Baffle

Table 5.8 Spoiler baffle dimensionless discharge equations and velocity scales.

Design	L	z_o	y_o/D range	$Q_* = \frac{Q}{\sqrt{gS_oD^5}}$	y_o/D range	$U_* = \frac{u_m}{\sqrt{gS_oD}}$
D-1	0.53D	0.09D	0.045 - 0.1 0.1 - 0.46	$Q_* = 0.85(y_o/D)^{2.02}$ $Q_* = 9.06(y_o/D)^{2.83}$	0.09 - 0.37	$U_* = 8.56(y_o/D)$
D-2	1.06D	0.09D	0.015 - 0.09 0.09 - 0.46	$Q_* = 0.29(y_o/D)^{1.12}$ $Q_* = 6.73(y_o/D)^{2.44}$	0.08 - 0.45	$U_* = 10.3(y_o/D)$
D-3	0.53D	0.15D	0.03 - 0.15 0.15 - 0.46	$Q_* = 0.33(y_o/D)^{1.35}$ $Q_* = 5.01(y_o/D)^{2.84}$	0.18 - 0.48	$U_* = 6.52(y_o/D)$
D-4	1.06D	0.15D	0.015 - 0.1 0.1 - 0.46	$Q_* = 0.19(y_o/D)^{0.98}$ $Q_* = 2.51(y_o/D)^{2.1}$		

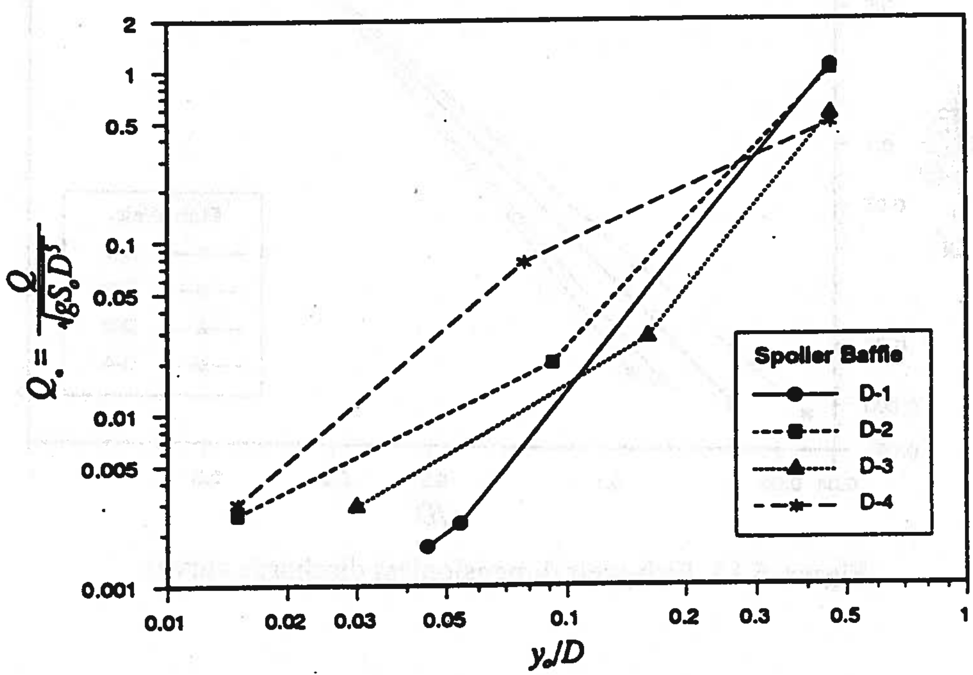


Figure 5.14 Spoiler baffle dimensionless discharge curves.

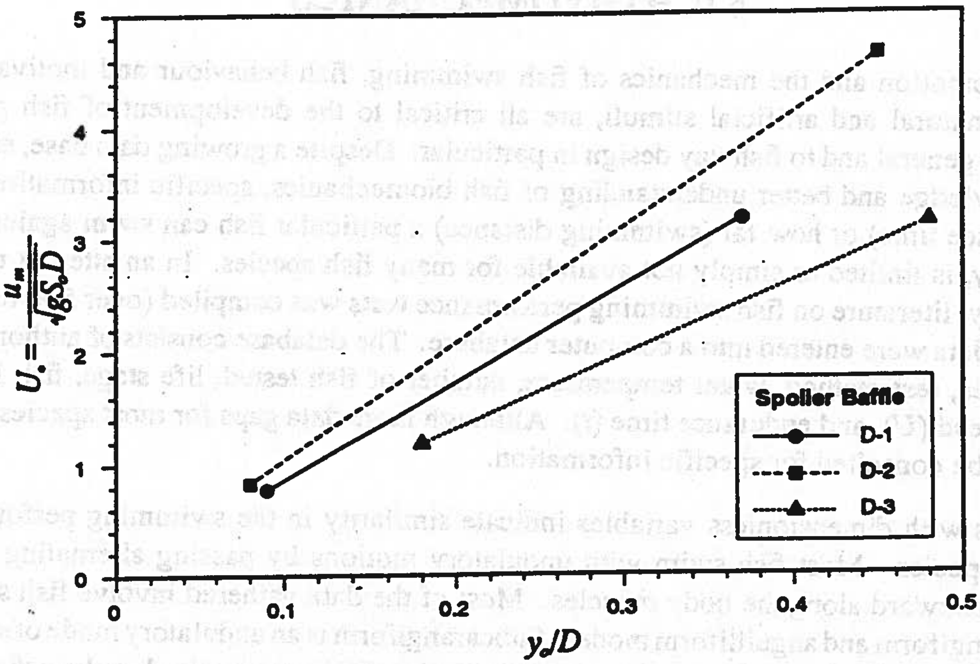


Figure 5.15 Spoiler baffle dimensionless velocity scales.

6 ICHTHYOMECHANICS

Fish locomotion and the mechanics of fish swimming, fish behaviour and motivation, fish responses to natural and artificial stimuli, are all critical to the development of fish protection technology in general and to fishway design in particular. Despite a growing data base, significant gains in knowledge and better understanding of fish biomechanics, specific information on how long (endurance time) or how far (swimming distance) a particular fish can swim against a given water velocity, is limited or simply not available for many fish species. In an attempt to address this deficiency, literature on fish swimming performance tests was compiled (over 500 references) and reported data were entered into a computer database. The database consists of author and date, genera, species, test method, water temperature, number of fish tested, life stage, fish length (l), swimming speed (U), and endurance time (t). Although large data gaps for most species exist, the database can be consulted for specific information.

Analyses with dimensionless variables indicate similarity in the swimming performance of several fish species. Most fish swim with undulatory motions by passing alternating waves of contraction backward along the body muscles. Most of the data gathered involve fish swimming in the subcarangiform and anguilliform modes. Subcarangiform is an undulatory mode of swimming characterized by small side-to-side amplitude at the anterior and large amplitude only in the posterior half or one-third of the body. The characteristic body shape is fusiform, the caudal peduncle is fairly deep and the caudal fin has a rather low aspect ratio. In the anguilliform mode most or all of the length of the body participates in propulsion. The body is long and thin, the anterior cylindrical, the posterior compressed and caudal fin is usually small. Similar hydrodynamic analysis may be applicable to fish swimming in the same mode, regardless of phyletic origin. Figure 6.1 presents the data available in the database using dimensionless variables for species swimming in the subcarangiform and anguilliform modes. Figure 6.1 indicates that for each swimming mode data tend to collapse within a relatively small region of the graph even though diverse species, data sources, and test methods are involved in the subcarangiform mode. In the burst speed range ($t \leq 20$ s) points from both swimming modes are well represented by a single line. In the prolonged speed range (20s < $t \leq$ 30 min) the anguilliform mode is well represented by the same line, while the slope of the line for the subcarangiform mode is significantly milder, indicating higher endurance for these species. The inferred relationship between the dimensionless fish speed F_f and the dimensionless endurance t_* is of the form:

$$F_f = K t_*^{-\eta} \qquad (6.1)$$

For the subcarangiform mode the species involved include: a) 10 anadromous species: Arctic charr (*Salvelinus alpinus*), Atlantic salmon (*Salmo salar*), brook trout (*Salvelinus fontinalis*), chum salmon (*Oncorhynchus keta*), cisco (*Coregonus artedii*), coho salmon (*Oncorhynchus kisutch*), humpback whitefish (*Coregonus clupeauformis*), pink salmon (*Oncorhynchus gorbuscha*), rainbow trout (*Oncorhynchus mykiss*; formerly *Salmo gairdneri*), sockeye salmon (*Oncorhynchus nerka*), b) 10 freshwater species: Arctic grayling (*Thymallus arcticus*), dace (*Leuciscus leuciscus*), flathead chub (*Platygobio gracilus*), goldfish (*Carassius auratus*), humpback whitefish (*Coregonus clu-

peauformis), largemouth bass, (*Micropterus salmoides*) longnose sucker (*Catostomus catostomus*), rainbow trout (*Oncorhynchus mykiss*; formerly *Salmo irideus*), walleye (*Stizostedion vitreum*), white sucker (*Catostomus commersoni*). The anguilliform mode includes two species, lamprey (*Petromyzon marinus*) and burbot (*Lota lota*). Data in this analysis were restricted to temperatures which did not appear to affect swimming performance.

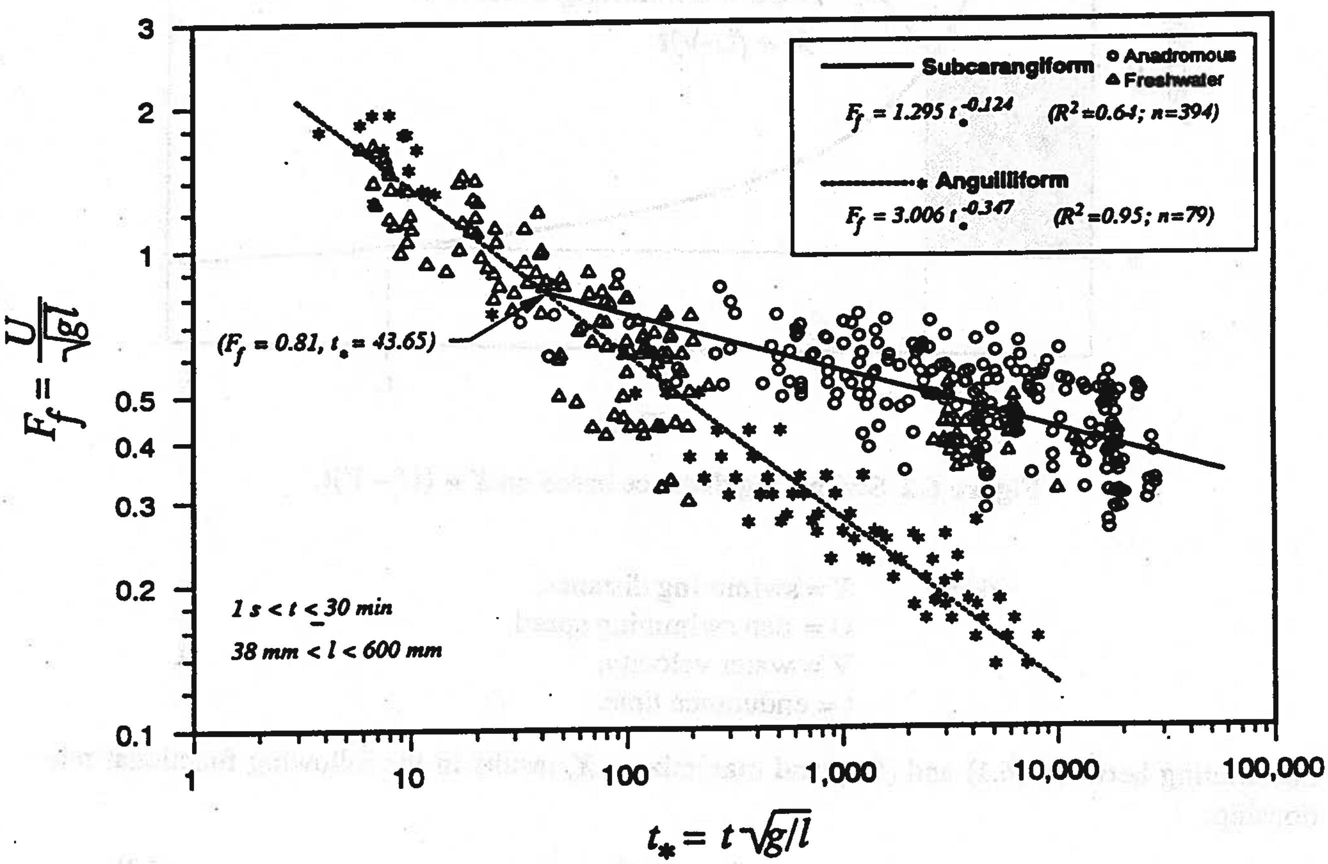


Figure 6.1 Fish endurance curves.

Figure 6.1 provides a guide to swimming speeds of several fish particularly when endurance time is of primary concern. This relationship can also be transformed into a water velocity vs swimming distance relationship. Considering the distance, X, that a fish travels by maintaining a speed, U, for a time (endurance), t, against water velocity, V, the following relationship is assumed:

$$X = (U - V)t \tag{6.2}$$

As can be seen from Figure 6.2 the swimming distance (X) is represented by the shaded area on a velocity vs time graph.

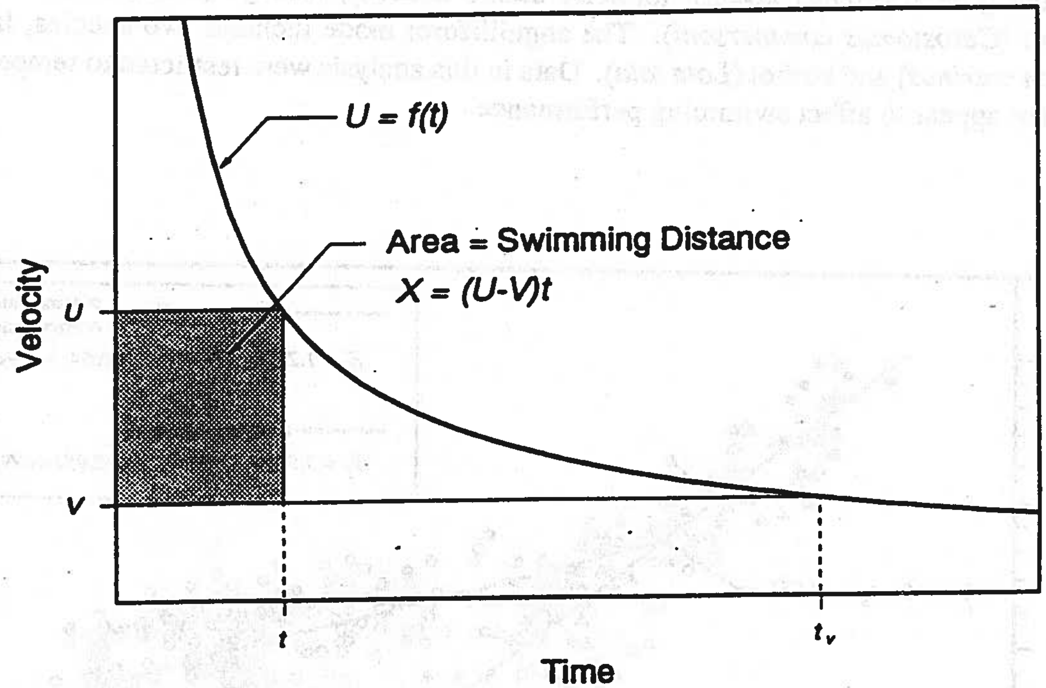


Figure 6.2 Swimming distance based on $X = (U - V)t$.

where: X = swimming distance,
U = fish swimming speed,
V = water velocity,
t = endurance time.

Substituting between (6.1) and (6.2) and maximizing X, results in the following functional relationship:

$$\xi = CF^{-\lambda} \tag{6.3}$$

where: $\xi = X_{max}/l$; $F = V/\sqrt{gl}$; $\lambda = (1-\eta)/\eta$; $C = \eta(1-\eta)^{\lambda}K^{1/\eta}$ (6.4)

Figure 6.3 illustrates equation (6.3) while Figs. 6.4 and 6.5 are derived from (6.3) once specific water velocities and fish lengths are applied. Table 6.1 summarizes the range of data for the species and variables used in the analysis. Table 6.2 provides a list of fish species and their swimming modes, while Table 6.3 summarizes regression equations for swimming speed vs fish length for several species reported in the literature.

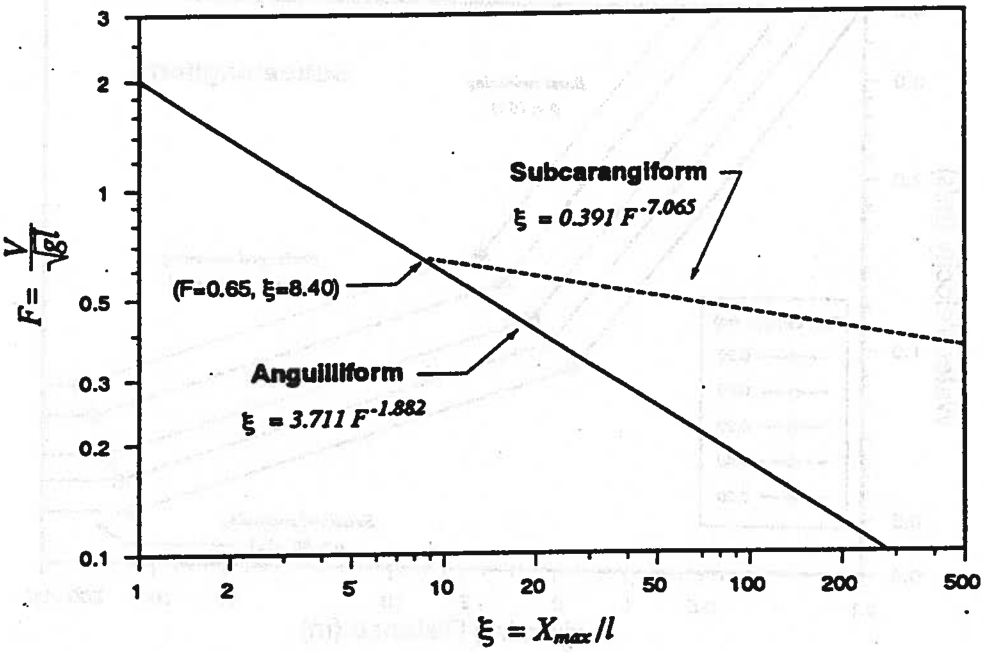


Figure 6.3 Dimensionless swimming distance curves.

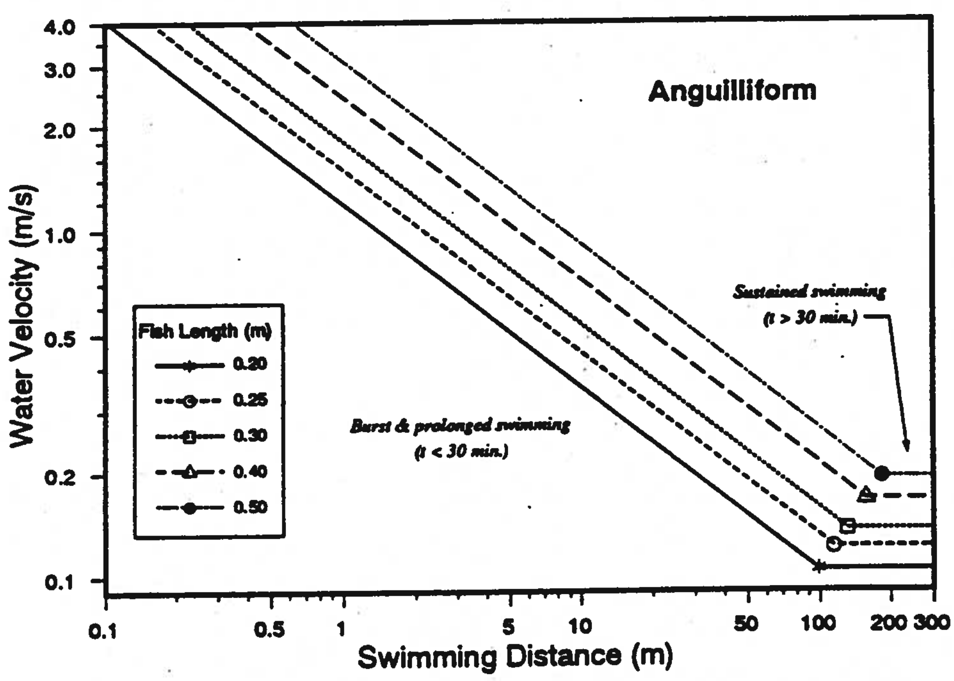


Figure 6.4 Swimming distance curves for several fish lengths (Anguilliform mode).

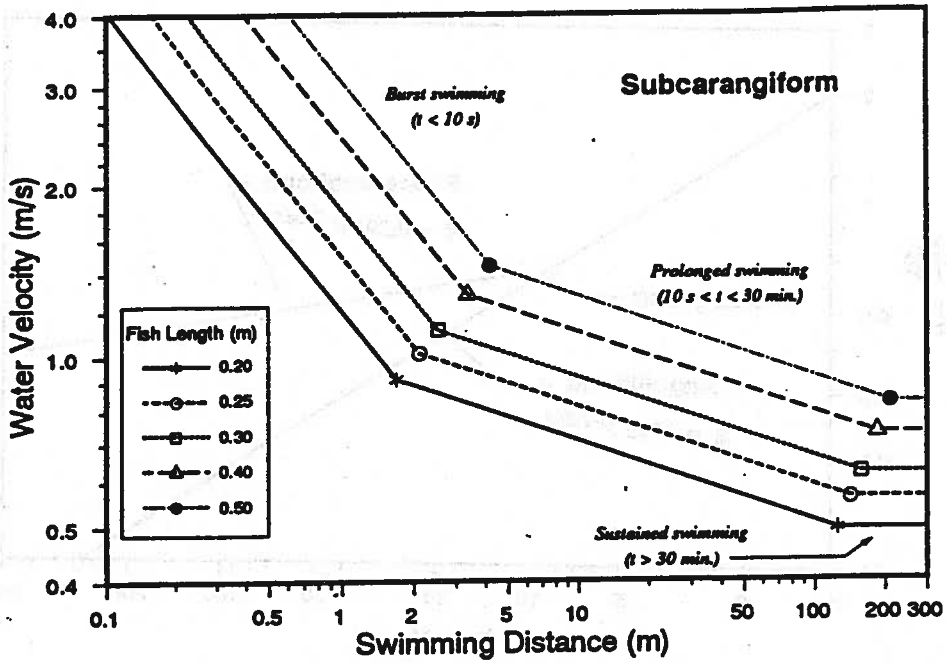


Figure 6.5 Swimming distance curves for several fish lengths (Subcarangiform mode).

Table 6.1 Variables and ranges of swimming performance data used in analysis

Common Name	Scientific Name	Length Range (mm)	Endurance Time (s)	Swimming Speed (m/s)	Temp. (°C)	No. of Fish	No. of Sources
Anguilliform Swimming Mode							
Burbot	*Lota lota*	120 - 620	600	0.360 - 0.410	7 - 12	56	1
Lamprey	*Petromyzon marinus*	145 - 508	0.8 - 1635	0.300 - 3.960	5 - 23	>75	2
Subcarangiform Swimming Mode							
Arctic char	*Salvelinus alpinus* [A]	80 - 420	6 - 1089	0.411 - 1.300	10 - 13.5	64	3
Arctic grayling	*Thymallus arcticus* [F]	70 - 370	600	0.520 - 0.720	12 - 19	94	1
Atlantic salmon	*Salmo salar* [A]	231	300	0.516	7.0	55	1
Brook trout	*Salvelinus fontinalis* [A]	41 - 172	10 - 1800	0.202 - 0.930	11.5 - 15	42	3
Chum salmon	*Oncorhynchus keta* [A]	38 - 48	300	0.181 - 0.342	10	17	1
Cisco	*Coregonus artedii* [A]	135	433 - 1800	0.458 - 0.630	12	20	1
Coho salmon	*Oncorhynchus kisutch* [A]	51 - 133	534 - 1746	0.343 - 0.701	10 - 20	>100	2
Dace	*Leuciscus leuciscus* [F]	100 - 200	1 - 20	0.430 - 2.400	15	7	1
Flathead chub	*Platygobio gracilus* [F]	170 - 300	600	0.429 - 0.627	12 - 19	28	1
Goldfish	*Carassius auratus* [F]	67 - 213	1 - 20	0.420 - 2.000	15	8	1
Humpback whitefish	*Coregonus clupeaformis* [A/F]	60 -510	72 - 1278	0.341 - 1.021	5 - 19	>200	2
Largemouth bass	*Micropterus salmoides* [F]	81 - 224	300 - 1800	0.340 - 0.589	20 - 30	190	3
Longnose sucker	*Catostomus catostomus* [F]	40 - 530	600	0.230 - 0.910	7 - 19	169	1
Pink salmon	*Oncorhynchus gorbuscha* [A]	465 - 596	72 - 1278	0.780 - 1.740	12 - 20	212	2
Rainbow trout[1]	*Oncorhynchus mykiss* [A/F]	82 - 310	1 - 1800	0.257 - 2.700	7 - 15	78	4
Sockeye salmon	*Oncorhynchus nerka* [A]	126 -621	6 - 1350	0.554 - 1.700	10 - 18	47	3
Walleye	*Stizostedian vitreum* [F]	80 - 380	600	0.380 - 0.840	19	54	1
White sucker	*Catostomus commersoni* [F]	170 - 370	600	0.480 - 0.730	12 - 19	20	1

[1] Former scientific names: Salmo gairdneri; Salmo irideus.

A - Anadromous, F - Freshwater

Table 6.2 Summary of species and swimming modes; all species from Osteichthyes class except lamprey which are from the Agnatha class.

Common Name	Scientific Name	Family / Subfamily	Order	Swimming Mode
Alewife	Alosa pseudoharengus	Clupeidae	Clupeiformes	Subcarangiform
Arctic charr	Salvelinus alpinus	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Arctic grayling	Thymallus arcticus	Salmonidae / Thymalinae	Salmoniformes	Subcarangiform
Atlantic salmon	Salmo salar	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Bonytail chub	Gila elegans	Cypriuidae	Cypriniformes	Subcarangiform
Broad whitefish	Coregonus nasus	Salmonidae / Coregoninae	Salmoniformes	Subcarangiform
Brook trout	Salvelinus fontinalis	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Burbot	Lota lota	Gadidae	Gadiformes	Anguilliform
Carp	Cyprinus carpio	Cyprinidae	Cypriniformes	Subcarangiform
Chinook salmon	Oncorhynchus tschawytscha	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Chum salmon	Oncorhynchus keta	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Cisco	Coregonus artedii	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Coho salmon	Oncorhynchus kisutch	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Colorado squawfish	Ptychocheilus lucius	Cyprinidae	Cypriniformes	Subcarangiform
Dace	Leuciscus leuciscus	Cyprinidae	Cypriniformes	Subcarangiform
Flathead chub	Platygobio gracilis	Cyprinidae	Cypriniformes	Subcarangiform
Goldfish	Carassius auratus	Cyprinidae	Cypriniformes	Subcarangiform
Humpback chub	Gila cypha	Cyprinidae	Cypriniformes	Subcarangiform
Humpback whitefish	Coregonus clupeaformis	Salmonidae / Coregoninae	Salmoniformes	Subcarangiform
Inconnu	Stenodus leucichthys	Salmonidae	Salmoniformes	Subcarangiform
Lake sturgeon	Acipenser fulvescens	Acipenseridae	Acipenseriformes	Subcarangiform/Carangiform
Lake trout	Salvelinus namaycush	Salmonidae	Salmoniformes	Subcarangiform
Lamprey	Petromyzon marinus	Petromyzontidae	Petromyzontiformes	Anguilliform
Largemouth bass	Micropterus salmoides	Centrarchidae	Perciformes	Subcarangiform
Longnose sucker	Catostomus catostomus	Catostomidae	Cypriniformes	Subcarangiform
Northern pike	Esox lucius	Esocidae	Salmoniformes	Subcarangiform/Labriform[1]
Pink salmon	Oncorhynchus gorbuscha	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Rainbow smelt	Osmerus mordax	Osmeridae	Perciformes	Subcarangiform
Rainbow trout	Oncorhynchus mykiss[2]	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Sockeye salmon	Oncorhynchus nerka	Salmonidae / Salmoninae	Salmoniformes	Subcarangiform
Threespine stickleback	Gasterosteus aculeatus	Gasterosteidae	Gasterosteiformes	Diodontiform/Ostraciform
Walleye	Stizostedion vitreum	Percidae	Perciformes	Subcarangiform
White perch	Morone americana	Percichthyidae	Perciformes	Subcarangiform/Carangiform
White sucker	Catostomus commersoni	Catostomidae	Cypriniformes	Subcarangiform
Yellow perch	Perca flavescens	Percidae	Perciformes	Subcarangiform

1 Labriform swimming mode as a predator.
2 Former scientific name Salmo gairdneri.

Table 6.3 Fish swimming performance data - regression equations. Note: all data are from increasing velocity tests.

Scientific Name	Common Name	Temp. T(°C)	Length Range l (m)	LM[1]	F/A[2]	Time t (s)	#[3]	Regression Equation	Reference
Alosa pseudoharengus	Alewife	15	0.046 - 0.150	FL	FW	3600	31	$U = 0.771\ L^{0.193}$	Griffiths (1979)
Catostomus catostomus	Longnose sucker	13	0.040 - 0.530	FL	FW	600	179	$U = 1.261\ L^{0.529}$	Jones et al (1973)
Catostomus commersoni	White sucker	16	0.170 - 0.370	FL	FW	600	20	$U = 1.309\ L^{0.552}$	Jones et al (1973)
Coregonus clupeaformis	Humpback whitefish	13	0.060 - 0.510	FL	FW	600	168	$U = 0.912\ L^{0.350}$	Jones et al (1973)
Coregonus nasus	Broad whitefish	12.5	0.060 - 0.330	FL	FW	600	24	$U = 0.770\ L^{0.450}$	Jones et al (1973)
Esox lucius	Northern pike	13	0.120 - 0.620	FL	FW	600	192	$U = 0.617\ L^{0.550}$	Jones et al (1973)
Micropterus salmoides	Largemouth bass	10	0.216	TL	FW	1800	15	$U = 0.086 \cdot 1072^{L}$	Beamish (1970)
		15	0.225	TL	FW	1800	15	$U = 0.161 \cdot 125.9^{L}$	
		20	0.197	TL	FW	1800	15	$U = 0.280 \cdot 23.44^{L}$	
		25	0.200	TL	FW	1800	15	$U = 0.318 \cdot 14.79^{L}$	
		30	0.224	TL	FW	1800	15	$U = 0.317 \cdot 15.85^{L}$	
		34	0.212	TL	FW	1800	15	$U = 0.249 \cdot 24.55^{L}$	
Morone americana	White perch	10	0.076 - 0.248	FL	FW	3600	52	$U = 0.897\ L^{0.333}$	Griffiths (1979)
Lota lota	Burbot	13	0.120 - 0.620	FL	FW	600	53	$U = 0.442\ L^{0.070}$	Jones et al (1973)
Oncorhynchus nerka	Sockeye salmon	2	0.092	TL	AN	3600	?	$U = 1.459\ L^{0.0294}$	Brett & Glass (1973)
		5	0.092	TL	AN	3600	?	$U = 1.600\ L^{0.0243}$	
		10	0.092	TL	AN	3600	?	$U = 1.965\ L^{0.0294}$	
		15	0.092	TL	AN	3600	?	$U = 2.500\ L^{0.0345}$	
		20	0.092	TL	AN	3600	?	$U = 2.300\ L^{0.0250}$	
Osmerus mordax	Rainbow smelt	10	0.070 - 0.163	FL	FW	3600	31	$U = 1.148\ L^{0.504}$	Griffiths (1979)
Perca flavescens	Yellow perch	10	0.096 - 0.245	FL	FW	3600	55	$U = 0.703\ L^{0.307}$	Griffiths (1979)
		20	0.096 - 0.245	FL	FW	3600	60	$U = 0.579\ L^{0.114}$	
Platygobio gracilis	Flathead chub	16	0.170 - 0.300	FL	FW	600	28	$U = 1.450\ L^{0.670}$	Jones et al (1973)
Salmo salar	Atlantic salmon	7-12	0.197 - 0.256	TL	AN	300	55	$U = 0.173 + 1.57\ L$	McCleave & Stred (1975)
Salvelinus alpinus	Arctic charr	10	0.080 - 0.420	FL	AN	600	26	$U = 1.660\ L^{0.606}$	Welch (1979)
Stenodus leucichthys	Inconnu	16	0.080 - 0.410	FL	FW	600	22	$U = 0.678\ L^{0.175}$	Jones et al (1973)
Stizostedion vitreum	Walleye	16	0.040 - 0.500	FL	FW	600	54	$U = 1.369\ L^{0.510}$	Jones et al (1973)
Thymallus arcticus	Arctic grayling	13	0.070 - 0.370	FL	FW	600	105	$U = 0.880\ L^{0.193}$	Jones et al (1973)

1 Length measurement: FL=fork length, TL=total length. 2 FW=freshwater, AN=anadromous. 3 Number of fish tested.

7 FISHWAY EFFECTIVENESS

7.1 General

There is a large body of literature documenting the successes and failures of fishway installations around the world. Generally, fish passage effectiveness varies with fishway design practice, species and site conditions. Fishways for the highly motivated salmon spawners are commonly successful, several design options are available, and numerous facilities exist as examples. Fishways for other species and juvenile fish are more recent and not as well documented. In the last decade several fishways were monitored in the Canadian provinces of Alberta, Saskatchewan, Manitoba and Ontario (Fig. 7.1). These fishways were used mostly by spawning fish which migrate entirely within a freshwater system of rivers and lakes.

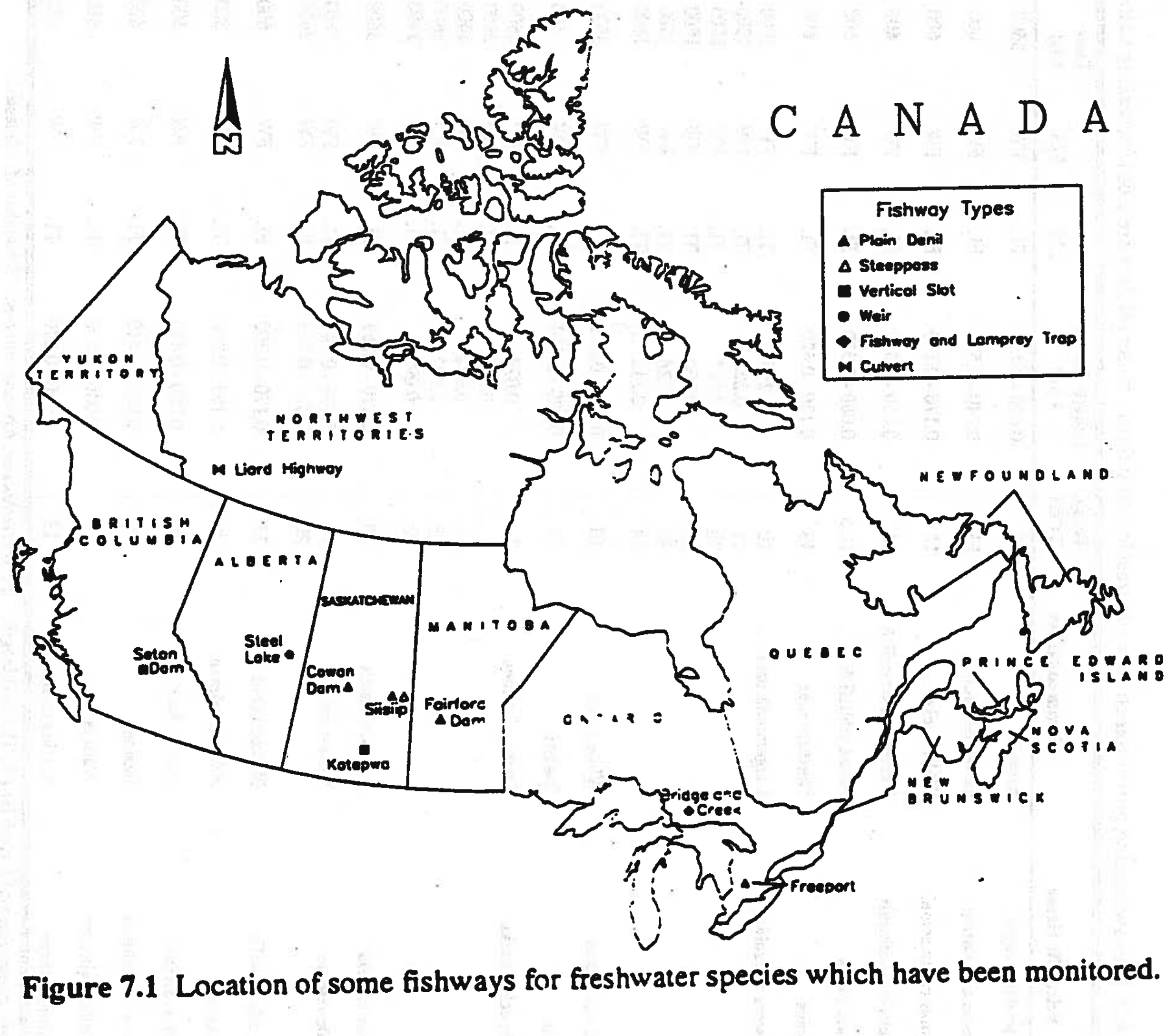


Figure 7.1 Location of some fishways for freshwater species which have been monitored.

Field studies have provided assessments of several Denil, vertical slot, weir and culvert fishways. Difficulties with some installations, particularly poorly designed weir fishways were overcome. Adult species which used such fishways include Arctic grayling, mountain whitefish (*Prosopium williamsoni*), lake whitefish (*Coregonus clupeaformis*), cisco (*Coregonus artedii*), northern pike, walleye, sauger (*Stizostedian canadense*), yellow perch (*Perca flavescens*), trout-perch (*Percopsis omiscomaycus*), white sucker, longnose sucker, carp (*Cyprinus carpio*), and burbot. Juveniles of some of the above species as well as spottail shiners (*Notropis hudsonius*) have also been reported using fishways. Several Liard Highway culverts were constructed using the stream simulation approach. At four of these the performance of culvert fish passage was assessed. This field study indicated that the culverts presented no difficulty to the spring migrations of Arctic grayling, longnose sucker and northern pike. Culvert velocities were comparable to the natural stream and no spawning migration delays were apparent. The presence of riprap, at least on one occasion, assisted the establishment of flow under the culvert ice and allowed fish to pass through the culvert without delay. With stream simulation, culvert construction has the potential for preserving or enhancing fish habitat since gravel, placed or deposited naturally in the culvert, may provide habitat suitable for fish spawning.

7.2 Assessment of Denil Fishways for freshwater species

Fish movements through Denil fishways in the Grand River Weir near Freeport, Ontario, the Fairford Dam near Fairford, Manitoba and the Cowan Dam in Saskatchewan were assessed using traps at the fish exit (upstream end) of each facility. The Freeport fishways were assessed daily from April 20 to May 11, 1990, the Fairford fishway was assessed daily from May 6-28 and June 2-12, 1987; and the Cowan fishway was assessed daily from April 27 to May 11, 1985, and weekly thereafter until June 10, 1985. At Cowan and Fairford the trap was lifted and emptied at least three times per day; in the morning, afternoon and evening. At Freeport the trap's were lifted and emptied twice per day. The data collected during the assessment program consisted of counting and identifying all of the species captured in the traps, as well as determining the fork lengths and other biological data (sex, spawning condition, weight) for key species. Water levels upstream, downstream and throughout each fishway were recorded as well as water temperatures.

The Fairford and Cowan plain Denil fishways have a similar layout, consisting of three flumes equipped with planar baffles, two resting pools and two vertical lift control gates (Fig. 7.2). The Grand River Weir at Freeport contains two plain Denil fishways with identical cross-sectional dimensions. The east bank fishway consists of a single flume at a 20% slope, the west bank fishway consists of three flumes each at a 10% slope and two resting pools. Figure 7.2 shows an isometric and plan view of the Fairford fishway as well as the plan views of the east and west bank fishways at Freeport. The plan view of the Cowan fishway is a mirror image of the Fairford fishway. Table 7.1 lists the dimensions for each fishway. The control gates at the outlet of each fishway allow for the operation of either all three fishway flumes when tailwater is low, or only the upper flume when tailwater is high.

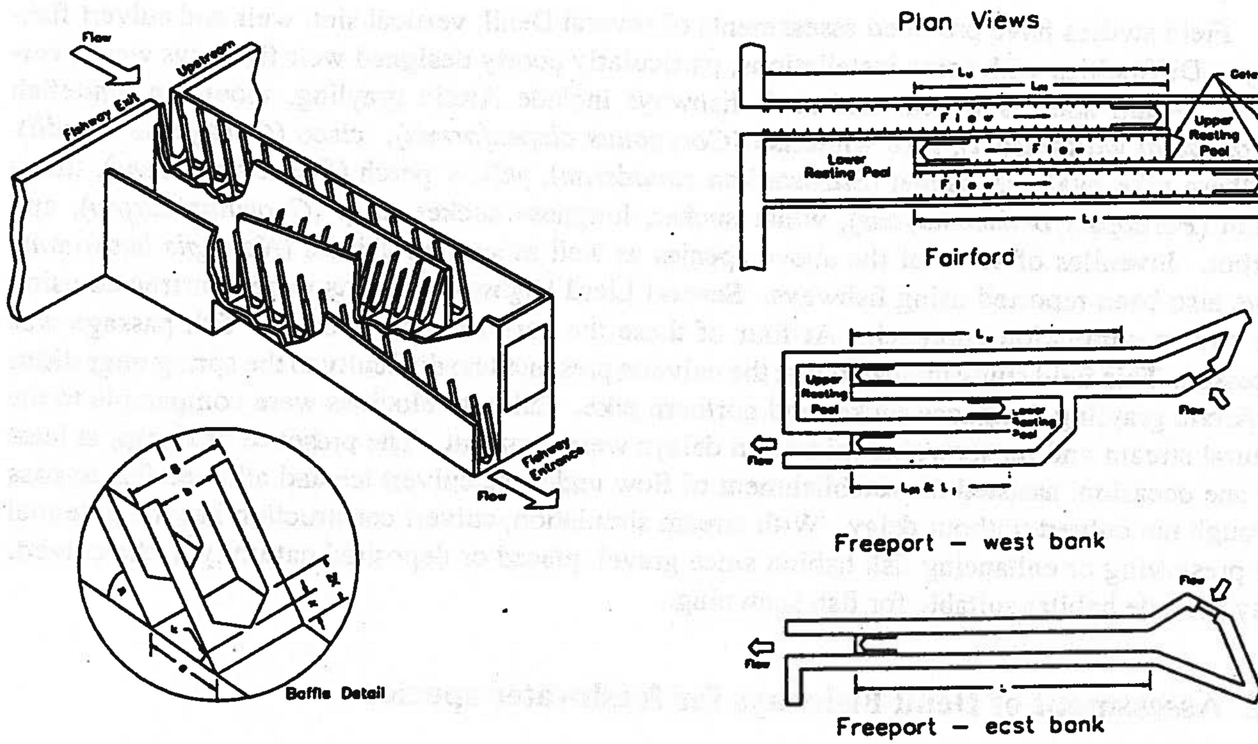


Figure 7.2 Isometric and plan views of Denil fishways at Fairford and Freeport.

Table 7.1 Dimensions of the Freeport, Fairford and Cowan fishways. Symbols are defined in Figure 7.2.

Dimensions	Fairford	Cowan	Freeport	
B (mm)	500	634	596	
b (mm)	300	400	360	
a (mm)	300	300	250	
k (mm)	88.4	106.1	127.3	
K (mm)	125	150	180	
ψ	45°	45°	45°	
Total drop (m)	2.9	2.20	1.67	
Fishway Section (upper, middle, lower)			**West fishway**	**East fishway**
Length (L, m)	6.3, 5.0, 6.6	9.5, 6.0, 8.5	7.7, 4.5, 4.5	8.4
Slope (S, %)	12.9, 12.8, 12.6	12.6, 10.0, 10.0	10, 10, 10	20
Resting pools (length x width x depth)				
Upper (m)	1.45 x 1.18 x 2.00	2.35 x 1.45 x 2.50	1.5 x 1.6 x 2.2	no resting pools
Lower (m)	1.39 x 1.13 x 1.61	3.50 x 1.45 x 2.50	1.5 x 1.6 x 2.2	

Fishway depths, discharges, velocities and water surface profiles are interdependent and relationships between them for Denil fishways are provided in section 5.2. Water surface profiles within each fishway were derived from the measurements obtained during the assessment. Water depths in the fishway were calculated as the distance from the baffle crest ("V") to the water surface. Water depths near the fishway exit were free of backwater effects and were selected to estimate fishway discharge and velocity.

The Fairford, Cowan and Freeport fishways correspond to the standard or Denil 2 design. The dimensionless discharge relationship from Table 5.2 reduces to the following discharge rating curves which were used to estimate the discharge through each fishway from the measured water depths (Figs. 7.3 and 7.4):

Fairford fishway: $$Q = 0.58y_o^{2.0} \quad (7.1)$$

Cowan fishway: $$Q = 0.66y_o^{2.0} \quad (7.2)$$

Freeport, west bank (10%): $$Q = 0.56y_o^{2.0} \quad (7.3)$$

Freeport, east bank (20%): $$Q = 0.79y_o^{2.0} \quad (7.4)$$

where y_o is in m and Q in m^3/s.

Results from hydraulic model studies were used to estimate water velocities along the center line of each fishway at a location near the fish exit, where no backwater effect was detected. Water velocities in plain Denil fishways are low at the bottom of the flume, and increase upwards to the water surface. A layer of fast water exists near the water surface. This implies that fish ascending the fishway face varying water velocities dependent on their swimming depth. A representative range of velocities that fish may have had to negotiate at Cowan, Fairford and Freeport were estimated by calculating velocities corresponding to $0.2y_o$, $0.4y_o$ and $0.6y_o$, where y_o is the water depth in the fishway (Figs. 7.3 and 7.4).

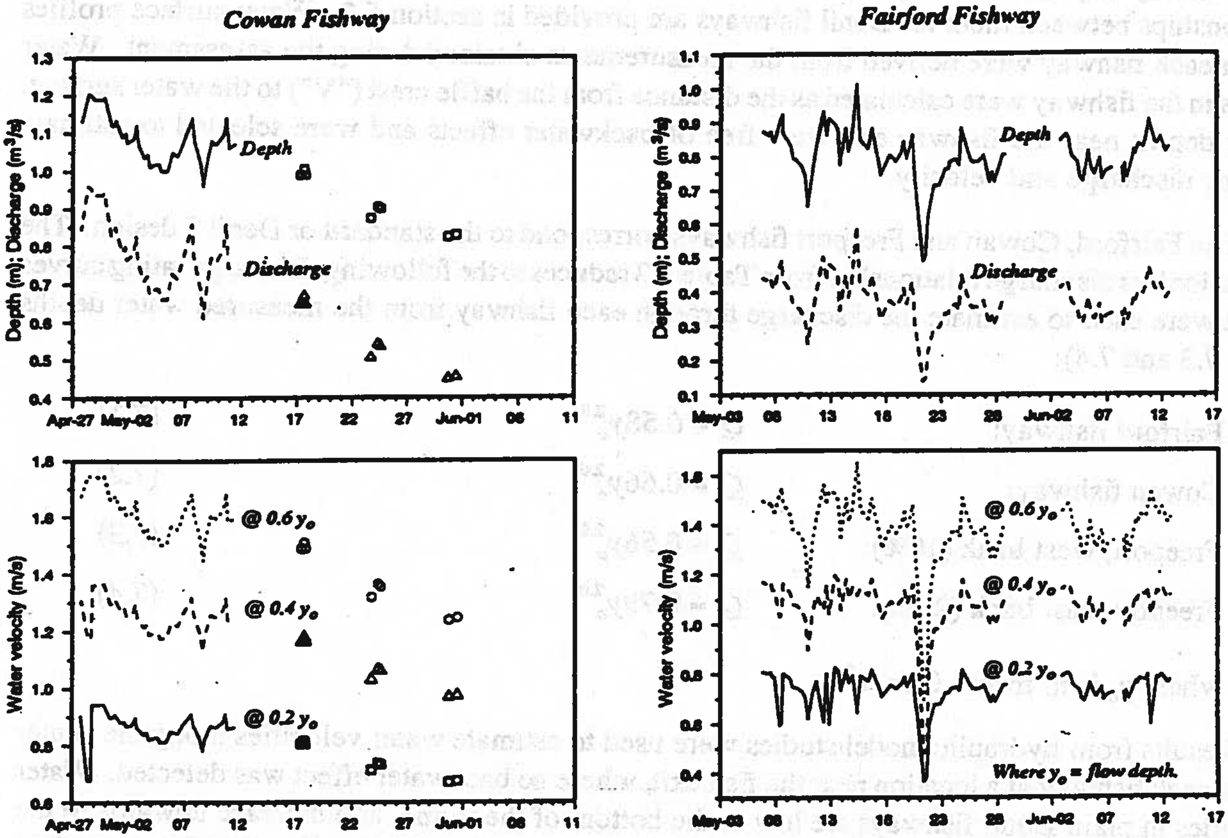


Figure 7.3 Depth, discharge and velocity profiles for Cowan (1985) and Fairford (1987) fishways.

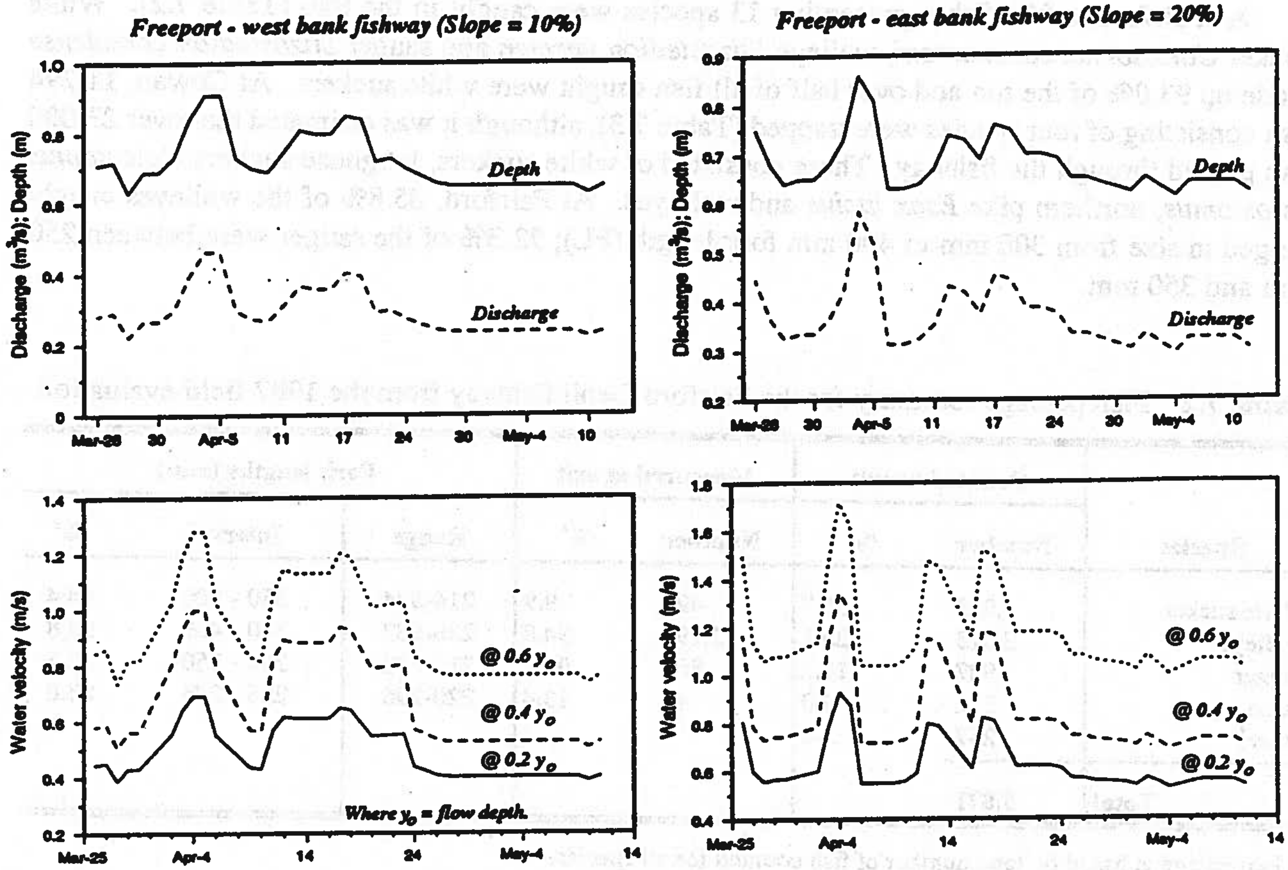


Figure 7.4 Depth, discharge and velocity profiles for the fishways in the Grand River Weir at Freeport (1990).

At Fairford, 8,871 fish representing 13 species were caught in the trap (Table 7.2). White sucker *Catostomus commersoni*, walleye *Stizostedion vitreum* and sauger *Stizostedion canadense* made up 93.0% of the run and over half of all fish caught were white suckers. At Cowan, 11,294 fish consisting of four species were trapped (Table 7.3), although it was estimated that over 23,000 fish passed through the fishway. These consisted of white suckers, longnose suckers *Catostomus catostomus*, northern pike *Esox lucius* and walleyes. At Fairford, 85.8% of the walleyes caught ranged in size from 300 mm to 400 mm fork length (FL); 92.3% of the sauger were between 250 mm and 350 mm.

Table 7.2 Fish passage summary for the Fairford Denil fishway from the 1987 field evaluation.

	Counted at exit		Measured at exit		Fork lengths (mm)		
Species	Number	%[a]	Number	%[b]	Range	Interval	%[c]
White sucker	5,032	56.7	496	9.9	214-524	350 - 500	94.4
Walleye	2,313	26.1	2,193	94.8	236-682	300 - 400	85.8
Sauger	907	10.2	854	94.2	212-398	250 - 350	92.3
Cisco	352	4.0	47	13.4	220-296	225 - 275	83.0
Other[d]	267	3.0					
Total	8,871						

a) Percentage is based on total number of fish counted for all species.
b) Percentage is based on number of fish counted for each species.
c) Percentage is based on number of fish measured for length.
d) 175 (2.0%) shorthead redhorse; 79 (0.9%) carp; 4 burbot; 3 lake whitefish; 2 freshwater drum; 1 longnose sucker; 1 silver redhorse; 1 quillback; 1 channel catfish.

Table 7.3 Fish passage summary for the Cowan Denil fishway from the 1985 field evaluation.

	Counted at exit		Measured at exit		Fork lengths (mm)		
Species	Number	%[a]	Number	%[b]	Range	Interval	%[c]
White sucker	5,054	44.8	1,229	24.3	250-498	350-500	96.6
Longnose sucker	4,803	42.5	746	15.5	347-532	350-500	93.7
Northern pike	1,095	9.7	853	77.9	324-800	350-500	96.8
Walleye	342	3.0	341	99.7	265-480	350-450	90.0
Total	11,294						

Note: a, b, c same as in Table 7.2.

At Cowan, 96.8% of the northern pike ranged from 350 mm to 500 mm, and 90.0% of the walleyes from 350 mm to 450 mm. Headwater levels at Fairford were fairly constant, but decreased at Cowan over the study period. This was reflected in the water depths measured in each fishway, and in the estimated fishway discharges and velocities. At the upstream end of the fishway at Fairford, water depths remained fairly constant at approximately 0.8 m, although they ranged from 0.5 m to 1.0 m. At Cowan, depths at the fishway decreased over time from about 1.2 m to 0.8 m. Estimated discharges through each fishway ranged from 0.14 to 0.59 m^3/s at Fairford and from 0.45 to 0.96 m^3/s at Cowan. Estimated water velocities were low near the bottom of each fishway (0.7 - 0.9 m/s) and high near the water surface ($\geq$ 1.5 m/s). At specific water depths, velocities fluctuated around average or trend lines. Over the respective evaluation periods, these trend lines remained fairly constant at Fairford, while continuously decreased at Cowan. Although all species caught were able to ascend both fishways, northern pike waited 2 to 3 weeks before using the fishway at Cowan. Long residence time by northern pike below this and other dams may be a reflection of behaviour in relation to foraging, spawning, or passing through Denil fishways. More comprehensive studies with northern pike are under way.

At Freeport, 1,590 fish representing 8 species were trapped at the west bank fishway while only 314 fish representing 7 species were caught at the east bank fishway (Table 7.4 and 7.5). Fish strongly preferred the west bank fishway (10% slope) as 82.5% of the total number of fish used this fishway compared to only 17.5% which used the east bank fishway (20% slope; Fig. 7.5)

Table 7.4 Fish passage summary for the Freeport Denil fishway on the west bank (10% slope) from the 1990 field evaluation.

	Counted @ exit		Measured @ exit		Fork lengths (mm)		
Species	Number	%[a]	Number	%[b]	Range	Interval	%[c]
Carp	12	0.8	12	100	400 - 645	500 - 600	83.3
Chub	4	0.3	4	100	175 - 196	150 - 200	100
Common shiner	1100	69.2	9	0.8	95 - 165	150 - 190	55.5
Hognose sucker	149	9.4	149	100	152 - 337	250 - 300	43
Moxostoma species	110	6.9	110	100	162 - 523	250 - 300	30
Rock species	118	7.4	29	24.6	120 - 214	150 - 200	72.5
Smallmouth bass	19	1.2	19	100	232 - 377	250 - 300	53
White sucker	78	4.9	78	100	186 - 366	200 - 250	41
Total	1590						

Note: a, b, c same as in Table 7.2.

Table 7.5 Fish passage summary for the Freeport Denil fishway on the east bank (20% slope) from the 1990 field evaluation.

Species	Counted @ exit		Measured @ exit		Fork lengths (mm)		
	Number	%[a]	Number	%[b]	Range	Interval	%[c]
Carp	1	0.3	1	100	551		
Chub	4	1.3	4	100	115 -142	100 - 150	100
Common shiner	123	39.2	6	4.9	86 - 170	150 - 200	67
Hognose sucker	98	31.2	98	100	164 - 329	300 - 350	55
Moxostoma species	34	10.8	34	100	229 - 492	300 - 350	56
Smallmouth bass	18	5.7	18	100	258 - 413	250 - 300	56
White sucker	36	11.5	36	100	115 - 349	200 - 350	88
Total	314						

Note: a, b, c same as in Table 7.2.

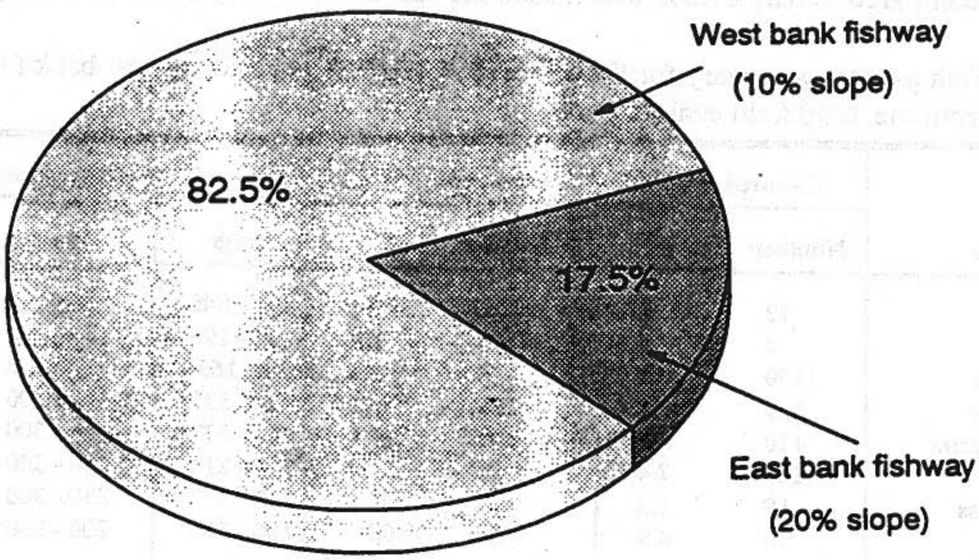


Figure 7.5 Percentages of the total number of fish using the two fishways in the Grand River Weir at Freeport (1990).

8 DESIGN EXAMPLES

8.1 Altrude Creek Culvert

Public Works Canada (PWC) used three culverts at the Trans-Canada Highway (TCH) crossing of Altrude Creek in Banff National Park (Table 7.1, Fig. 8.1). While all three culverts would assist with flood flows, the culvert arrangement is intended to allow fish passage through the first culvert and ice passage through the third culvert. Altrude Creek flows north to the Bow River and has been identified as the best fish-producing stream that will be intersected by the TCH.

Table 8.1 Altrude Creek culvert dimensions.

Culvert	Size & Type	Length (m)	Slope (%)	Inlet Elev.(m)	Outlet Elev. (m)
1-F Fish	3100 mm x 1980 mm S.P.C.S.P.A.	73	0.5	1430.0	1429.64
2-N Normal	3100 mm x 1980 mm S.P.C.S.P.A.	71	1.0	1430.3	1429.79
3-I Ice	1600 mm diameter C.S.P	69	1.0	1431.2	1430.51

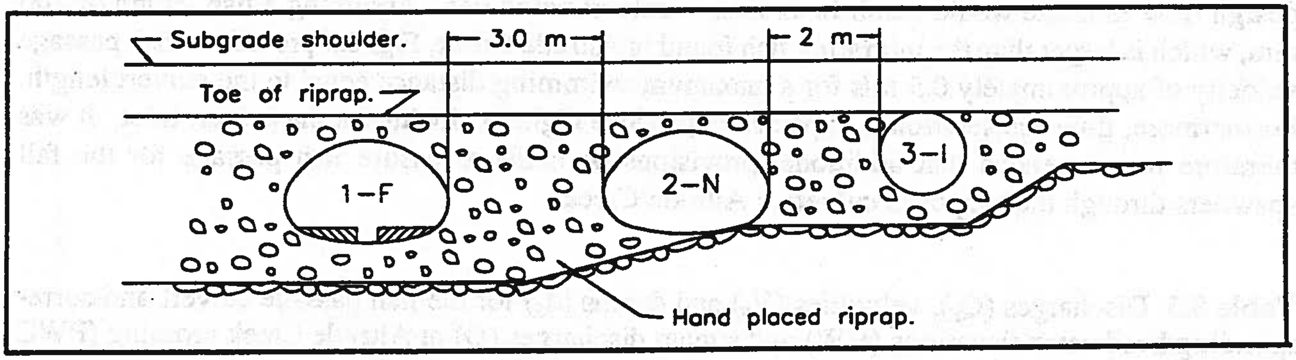


Figure 8.1 Altrude Creek culverts - end view.

8.1.1 Fish migration discharge

Field studies determined that late summer or fall spawning migrations of mountain whitefish, brook and bull trout occurred in Altrude Creek. A fall fish migration period of September 15 to October 31 was assumed and Redearth Creek records were used to estimate Altrude Creek flows (Table 8.2). From frequency analysis (Table 4.1; Fig. 4.1) a value of 5.3 m^3/s was found for the fall fish migration discharge of Redearth Creek. This corresponds to a value of 2.9 m^3/s for Altrude Creek.

Table 8.2 Altrude Creek discharges (Q_A) estimated from recorded flows of Redearth Creek (Q_R) obtained from Water Survey of Canada, Historical Sreamflow Summary for Alberta to 1986.

Discharge description	$Q_R(m^3/s)$	$Q_A(m^3/s)$
October mean monthly flow	1.77	0.96
September mean monthly flow	3.16	1.72
average of Sept. and Oct. means	2.47	1.34
fall fish migration discharge (Sept.15-Oct.31)	5.3	2.9

Note: Drainage areas of Altrude and Redearth creeks were estimated as 80 km^2 and 147 km^2, respectively; $Q_A = (80/147)\ Q_R = 0.54\ Q_R$

8.1.2 Fish passage design

PWC estimated water velocities in the culvert for several discharges and these are summarized in Table 8.3. Figure 8.2 presents graphically the PWC estimates from Table 8.3. Two points are highlighted because they appear inconsistent. Uncertainties existed as to the discharge estimates and the extent of the fish migration period. According to PWC calculations even the lowest fishway design flow estimate would result in excessive culvert velocities. Assuming a fish length of 200 mm, which is larger than the migrating fish found in Altrude Creek, Fig. 6.5 provides a fish passage velocity of approximately 0.5 m/s for a maximum swimming distance equal to the culvert length. Furthermore, flow acceleration is expected to produce higher velocities at the culvert inlet. It was therefore recommended that additional provisions be made to ensure fish passage for the fall spawners through the proposed culvert at Altrude Creek.

Table 8.3 Discharges (Q_F), velocities (V_F) and depths (d_F) for the fish passage culvert and corresponding headwater elevations (HW) and stream discharges (Q) at Altrude Creek crossing (PWC estimates).

HW (m)	Q (m^3/s)	Q_F (m^3/s)	V_F (m/s)	d_F (m)
1432.14	19.55	9.80	2.02	1.98
1431.45	9.18	5.20	1.30	1.45
Projected from Fig. 8.2		2.9	0.8	1.1
1431.17	2.93	1.60	0.48	1.17
1430.97	2.57	1.84	0.58	0.97
1430.91	1.38	0.84	0.33	0.91

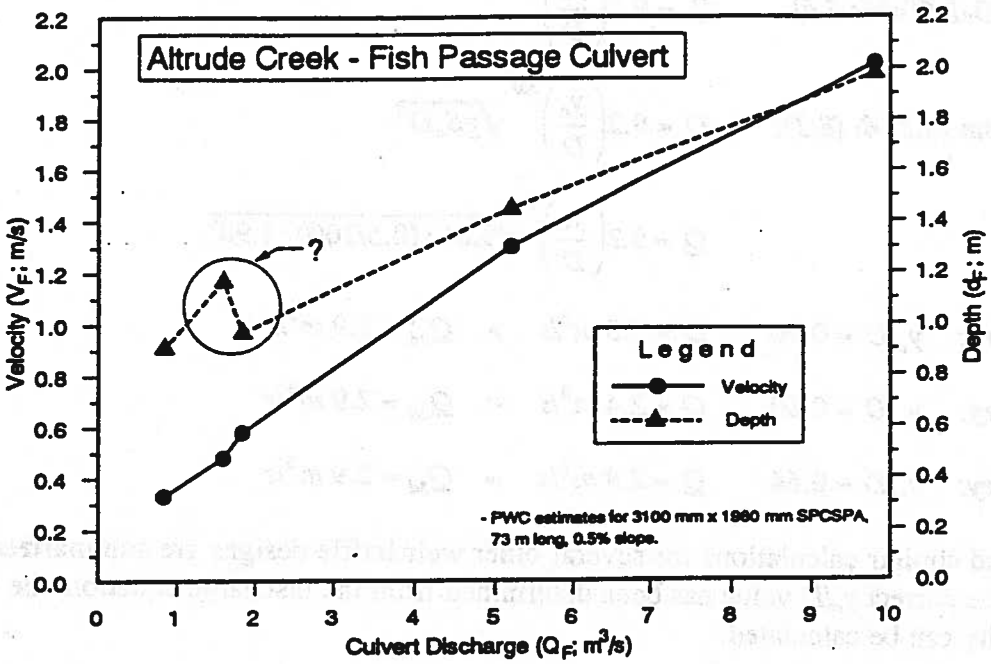


Figure 8.2 Depth and velocity vs discharge for fish passage culvert at Altrude Creek.

The proposed arrangement of the three culverts at the Altrude crossing did not allow for stream width and slope to be maintained so the stream simulation technique was unsuitable. Various fish passage devices were considered and the slotted-weir was selected for detailed design (Fig. 5.6).

Highest water velocity in the slotted weir baffle culvert fishway occurs through the slots in the weirs. The six slotted weir baffle fishway designs presented in Table 5.6 were evaluated using the dimensionless discharge equations and velocity scales to determine maximum water velocities at the slots, based on the fish passage flow. The slotted weir baffled culvert design is acceptable when the water velocity through the slots is within the burst swimming ability of the design fish species. Although the equations in Table 5.6 were developed for circular culverts, they were used to approximate discharge and velocity in the Altrude Creek pipe arch culvert. For the Altrude Creek pipe arch culvert the vertical height of the culvert (1980 mm) was selected in place of the diameter (D). The steps used to calculate velocity for design D-1 are presented below.

The dimensionless discharge equation for design D-1 was selected from Table 5.6 and used to determine the y_o/D value which produced a discharge which was equal to the fish passage flow of 2.9 m^3/s (Table 8.2) at Altrude Creek. The Altrude Creek fish passage culvert had dimensions D=1.98 m and S=0.5%.

For design D-1 (Table 5.6): $$Q_* = 9.2\left(\frac{y_o}{D}\right)^{3.0} \tag{8.1}$$

from (5.1) & (8.1): $$Q = 9.2\left(\frac{y_o}{D}\right)^{3.0}\sqrt{gS_oD^5} \tag{8.2}$$

$$Q = 9.2\left(\frac{y_o}{D}\right)^{3}\sqrt{9.81\cdot(0.5/100)\cdot 1.98^5} \tag{8.3}$$

try: $y_o/D = 0.70$ $\quad Q = 3.8\ m^3/s \quad > \quad Q_{3d} = 2.9\ m^3/s$

try: $y_o/D = 0.60$ $\quad Q = 2.4\ m^3/s \quad < \quad Q_{3d} = 2.9\ m^3/s$

try: $y_o/D = 0.64$ $\quad Q = 2.9\ m^3/s \quad = \quad Q_{3d} = 2.9\ m^3/s$

This and similar calculations for several other weir baffle designs are summarized in Table 8.4. Once the correct y_o/D value has been determined from the discharge equation, the maximum water velocity can be calculated:

For design D-1 (Table 5.6): $$U_* = 9.2\left(\frac{y_o}{D}\right) \tag{8.4}$$

from (5.2) & (8.4): $$U = 9.2\left(\frac{y_o}{D}\right)\sqrt{gS_oD} \tag{8.5}$$

when $\frac{y_o}{D} = 0.64 \qquad U = 1.8\ m/s$

Table 8.4 Summary of discharge and water velocity calculations based on the slotted weir baffle designs in Table 5.7 for the fish passage culvert at Altrude Creek.

Design	L (m)	z_s (m)	y_o/D	Q_*	Q (m^3/s)	U_*	U (m/s)
D-1	1.2	0.3	0.70	3.2	3.8	6.4	2.0
			0.60	2.0	2.4	5.5	1.7
			0.64	2.4	2.9	5.9	1.8
D-2	0.6	0.3	0.64	2.4	2.9	5.9	1.8
D-3	2.4	0.3	0.59	2.4	2.9	6.4	2.0
D-4	4.8	0.3	0.57	2.4	2.9	7.2	2.2
D-5	1.2	0.2	0.55	2.4	2.9	6.2	1.9
D-6	2.4	0.2	0.54	2.4	2.9	6.7	2.1

Based on the velocity of 1.8 m/s calculated above a 200 mm trout would be able to travel approximately 0.5 m before fatiguing (Fig. 6.5) which is acceptable in allowing the fish to pass through the high velocity areas at the slot. Given the uncertainties on Altrude Creek flow estimates and the approximation of the arch culvert hydraulics, D-1 was selected as a conservative design. Slotted-weirs, 0.3 m in height, adapted to the pipe arch cross-section and spaced at 1.2 m intervals at the bottom of the proposed culvert (Fig. 8.3), would provide fish with the opportunity to pass through the culvert using a burst and rest swimming pattern.

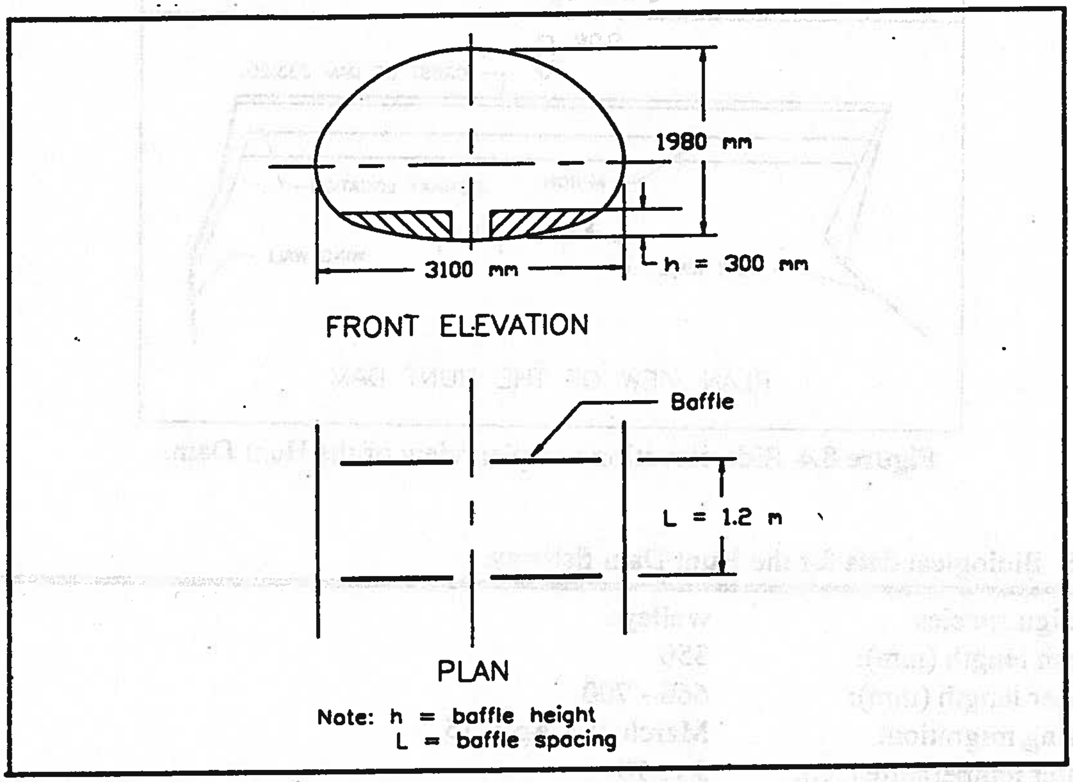


Figure 8.3 Slotted weir culvert fishway for Altrude Creek.

8.2 Hunt Dam Vertical Slot Fishway

The Hunt Dam (Fig 8.4), located on the Thames River in London, Ontario was identified as an obstacle which prevented fish from gaining access to approximately 100 km of upstream habitat. Observations of fish runs at the base of the dam noted the following species: walleye, rainbow trout, chinook salmon, northern pike, yellow perch, mooneye, sucker and carp. The purpose of the Hunt Dam fishway was to expand seasonal spawning migrations of walleye, rainbow trout and Chinook salmon in order to increase angling opportunities upstream of the dam.

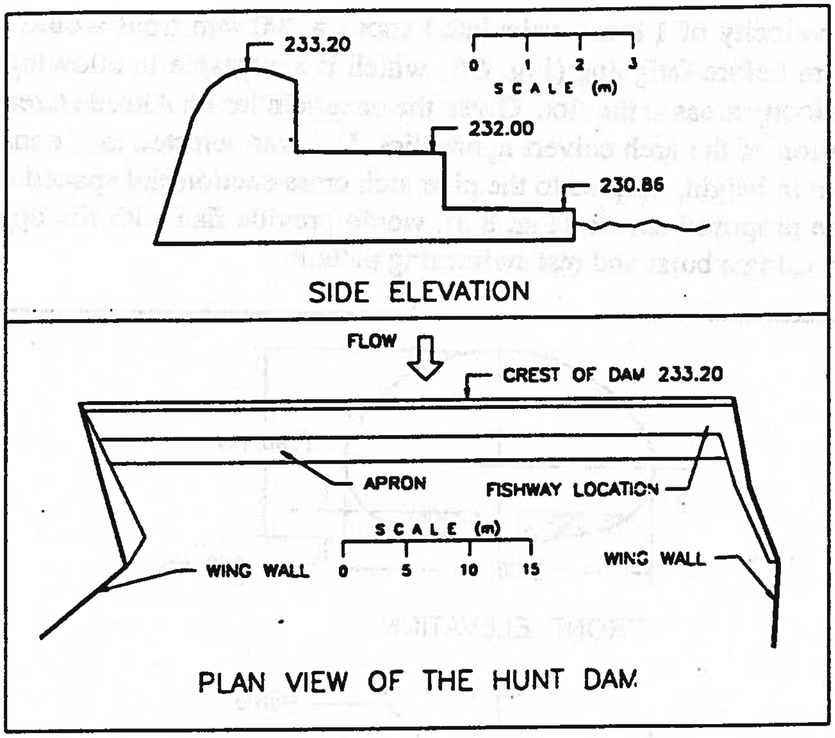


Figure 8.4 Side elevation and plan view of the Hunt Dam.

Table 8.5 Biological data for the Hunt Dam fishway.

Design species:	walleye
Mean length (mm):	350
Upper length (mm):	660 - 700
Spring migration:	March 15 - April 15
Water temperature (°C):	5 to 10

Using the method shown in Section 4 the fishway design flow during the migration period at Hunt Dam, based on the 1:10 year, 3 day delay discharge (Q_{3d}) was calculated to be:

max flow: $Q_{3d} = 155\ m^3/s$ (1:10 year, 3 day delay)
min flow: $Q_{min} = 5\ m^3/s$.

The discharge rating curves for the headwater and tailwater of the Hunt Dam are shown in Figure 8.5. Water levels at the dam during migration were determined from the discharge rating curves based on the maximum and minimum fishway design flows.

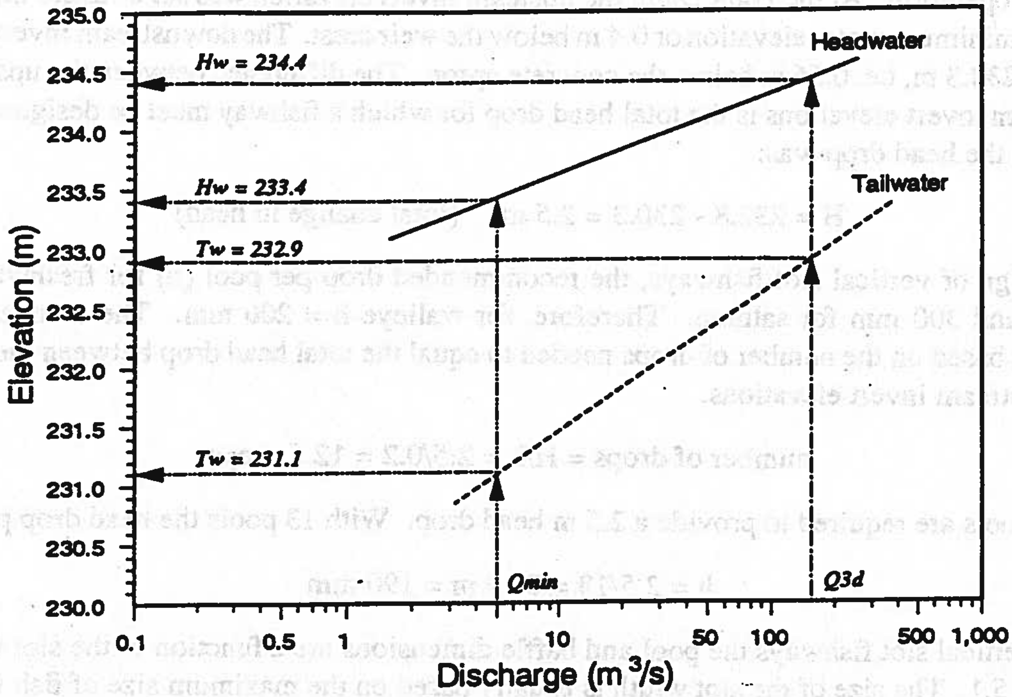


Figure 8.5 Hunt Dam discharge rating curves.

From Figure 8.5 the maximum and minimum upstream (headwater, H_w) and downstream (tailwater, T_w) water elevations were estimated as:

	H_w (m)	T_w (m)
maximum	234.4	232.9
minimum	233.4	231.1

Dam configuration from Fig. 8.4: crest of weir = 233.2 m
d/s concrete floor = 230.86 m

8.2.1 Design Calculations

Due to the range of tailwater levels expected at the Hunt Dam during the fish migration period, experience indicated that the vertical slot fishway would probably be the most effective for this location. The vertical slot fishway design #18 (Fig. 5.1) was selected for the Hunt Dam fishway, based on its good hydraulic charactistics and ease of construction.

Invert elevations for the upstream and downstream ends of the fishway were established from the minimum headwater and tailwater elevations calculated previously. Fishway inverts were set slightly lower than the minimum water level elevations in order to ensure adequate flow depth in the fishway during low flow periods. For vertical slot fishways 0.6 m is a common minimum depth for proper operation. At the Hunt Dam, the upsteam invert elevation was set at 232.8 m, i.e. 0.6 m below the minimum water elevation or 0.4 m below the weir crest. The downstream invert elevation was set at 230.3 m, i.e. 0.56 m below the concrete apron. The difference between the upstream and downstream invert elevations is the total head drop for which a fishway must be designed. For the Hunt Dam the head drop was:

$$H = 232.8 - 230.3 = 2.5 \text{ m} \quad \text{(total change in head)}$$

In the design of vertical slot fishways, the recommended drop per pool (h) for freshwater fish is 200 mm, and 300 mm for salmon. Therefore, for walleye h = 200 mm. The number of pools required is based on the number of drops needed to equal the total head drop between the upstream and downstream invert elevations.

$$\text{number of drops} = H/h = 2.5/0.2 = 12.5 \text{ drops}$$

Thirteen pools are required to provide a 2.5 m head drop.. With 13 pools the head drop per pool is:

$$h = 2.5/13 = 0.19 \text{ m} = 190 \text{ mm}$$

For vertical slot fishways the pool and baffle dimensions are a function of the slot width (b_o), see Figure 5.1. The size of the slot width is usually based on the maximum size of fish which will be using the fishway. Chinook salmon with an upper length range of 750 to 900 mm had been identified as one species which would use the fishway. In order to provide these large fish with adequate room to maneuver through the slots a slot width of 300 mm was selected. Pool and baffle dimensions are shown in Figure 8.6, note the pool width was rounded up to 2.5 m. The height of the pool walls in vertical slot fishways is based the maximum water elevations at the upstream and downstream ends of the fishway expected during the migration period.

The critical zone in terms of fish passage through verical slot fishways is at the slot. It is in this area that fish are confronted with the highest water velocities. Fish usually use burst swimming to pass the high velocity zone at the slot. The water velocity in the slot is relatively constant from top to bottom and can be calculated using equation 5.4. Slot velocity for the Hunt Dam vertical slot fishway:

$$u_m = \sqrt{2 \cdot 9.81 \cdot 0.19} = 1.93 \text{ m/s}$$

For successful fish passage the fish swimming speed (V) must be greater than water velocity in slot (u_m). The swimming distance vs water velocity plot (Figs. 6.4 and 6.5) can be used to determine if the slot velocity exceeds the swimming ability of the design fish species. With vertical slot fishways the swimming distance (X_{max}) is based on length of the water jet at the slot. For the Hunt Dam fishway this distance was estimated to be 0.5 m. From Figure 6.5 a 350 mm walleye can swim 0.5 m against a water velocity of 3.0 m/s. Since this velocity is greater than the slot velocity, a 350 mm walleye will be able to move through the slot and therefore this design is acceptable.

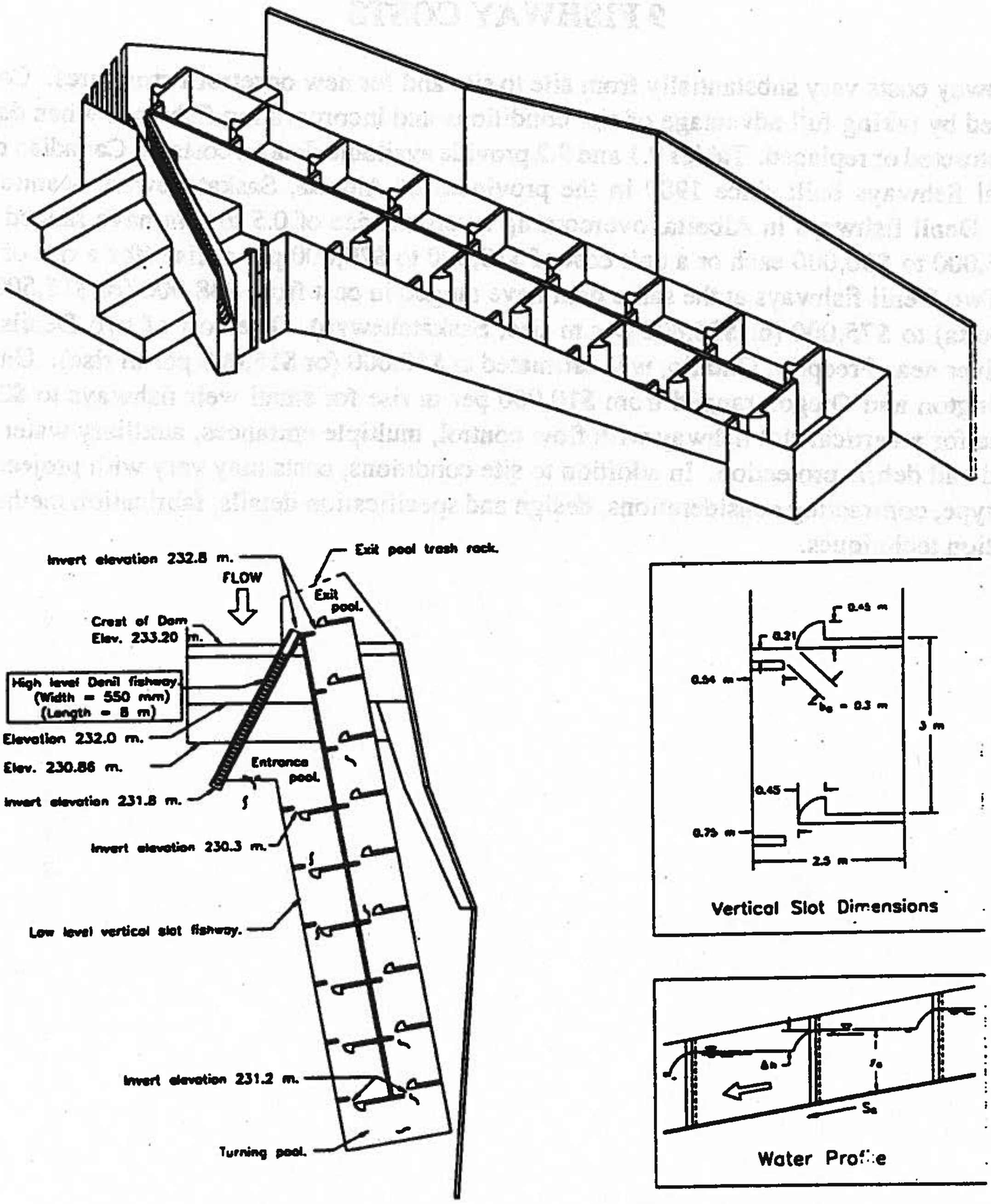


Figure 8.6. Isometric and plan view of the Hunt Dam fishways.

The recommended layout for the Hunt Dam fishway is shown in Figure 8.6. The use of two fishways was recommended in order to provide more effective fish passage. A turning pool was incorporated into the vertical slot fishway in order to locate the fishway entrance near the base of the dam where fish are known to congregate. A plain Denil fishway located adjacent to the vertical slot fishway was designed to provide attraction water near the entrance of the vertical slot fishway, as well as providing an alternate, more direct route upstream during high tailwater levels.

9 FISHWAY COSTS

Fishway costs vary substantially from site to site and for new or retrofit structures. Costs can be reduced by taking full advantage of site conditions and incorporating fishways when dams are first constructed or replaced. Tables 9.1 and 9.2 provide available data on costs (in Canadian dollars) of several fishways built since 1980 in the provinces of Alberta, Saskatchewan, Manitoba and Ontario. Denil fishways in Alberta, overcoming a vertical rise of 0.5 to 2 m have ranged in cost from \$15,000 to \$30,000 each or a unit cost of \$10,000 to \$20,000 per m rise (for a rise of 1 m or more). Two Denil fishways at the same dam have ranged in cost from \$38,000 (or \$15,500 per m rise; Alberta) to \$75,000 (or \$26,800 per m rise; Saskatchewan). The cost of two Denils on the Grand River near Freeport, Ontario, was estimated at \$50,000 (or \$15,000 per m rise). Unit costs in Washington and Oregon ranged from \$10,000 per m rise for small weir fishways to \$200,000 per m rise for a vertical slot fishway with flow control, multiple entrances, auxiliary water supply and flood and debris protection. In addition to site conditions, costs may vary with project scope, fishway type, contracting considerations, design and specification details, fabrication methods and construction techniques.

Table 9.1 Unit costs ($ per m of vertical rise) for several fish passage facilities built between 1980 and 1989 in Ontario, Manitoba, and Saskatchewan.

FACILITY	SOURCE	CONSTRUCTION INFORMATION	DESIGN SPECIES	TOTAL RISE(m)	TOTAL COST($)	UNIT COST($/m)
ONTARIO						
Thornbury Fishlock	MNR	1980-Retrofit	Chinook salmon	7.02	272 000	37 700
Walkerton fish bypass channel	MNR	1980-Retrofit	Rainbow trout	2.18	74 000	33 900
Haines fish bypass channel	MNR	1987-Retrofit	Rainbow trout	1.67	55 000	32 900
Grand River at Freeport[1]	AE	1989-New Concrete; metal baffles	Walleye	3.34	50 000	15 000
MANITOBA						
Fairford Denil fishway	DNR	1984-Retrofit; Timber Attraction water flume	Walleye	2.49	113 734	45 676
SASKATCHEWAN						
Cowan Denil fishway	PRW	1985-Retrofit Concrete; metal baffles	Northern pike	3.20	124 686	38 964
Kampsack (2 Denil fishways with a vertical rise of 1.40m each)	PFRA	1988-New Concrete; metal baffles	Walleye	2.80	75 000	26 786

1. Two fishways rising 1.67 m each.

Table 9.2 Unit costs ($ per m of vertical rise) for fabrication and installation of standard Denil fishways built between 1983 and 1988 in Alberta (Source Alberta Environment).

FACILITY	CONSTRUCTION INFORMATION	DESIGN SPECIES	TOTAL RISE (m)	COST($)			UNIT COST($)		
				FABRI-CATE	INSTALL	TOTAL	FABRI-CATE	INSTALL	TOTAL
Lesser Slave Lake[1]	1983-Retrofit; Timber	N. Pike Goldeye	1.84	10 000	13 000	23 000	5 435	7 065	12 500
Beaverlodge[2]	1984-Retrofit; Steel	Arctic grayling N. Pike	1.63	12 500	2 600	15 100	7 669	1 595	9 264
Ethel Lake	1986-New; Steel	N. Pike Walleye	0.50	15 000	5 000	20 000			
Cadotte Lake	1988-Retrofit; Steel	N. Pike Walleye	1.29	23 000	3 600	26 600	17 829	2 791	20 620
Parlby Creek:									
Spotted Lake	1988-Retrofit; Steel	N. Pike	1.20	12 500	7 500	20 000	10 417	6 250	16 667
Carlyle	1988-Retrofit; Steel	N. Pike	1.25	10 500	7 500	18 000	8 400	6 000	14 400
		TOTAL	2.45	23 000	15 000	38 000	9 388	6 122	15 510

1. Two fishways rising 0.94m and 0.90m respectively.
2. Two fishways rising 0.74m and 0.89m respectively.

10 NOTATION

B width of fishway
b_o width of fish passage opening
D diameter of culvert
d depth of surface jet for streaming weir flow
F_f dimensionless fish speed
F dimensionless water velocity
g gravitational acceleration 32.2
h hydraulic head
k baffle notch height
l fish length
L pool length, baffle spacing
Q discharge through fishway
Q_o discharge through orifice
Q_w discharge over weir
Q_* dimensionless fishway discharge
Q_j dimensionless discharge for submerged jet flow through the orifice
Q_p dimensionless discharge for plunging weir flow
Q_s dimensionless discharge for streaming weir flow
Q_t dimensionless discharge for transitional weir flow
S_o slope of fishway bed
t fish endurance time
t_* dimensionless fish endurance
u time averaged water velocity
u_m maximum value of u
u_m' maximum value of u at 75% of depth
U fish speed
U_* dimensionless velocity scale
V average water velocity
X fish swimming distance
y_o characteristic depth of flow
z_o height of baffle, weir, sill
$\alpha,\beta,\eta,\lambda,C,K$ coefficients
ξ relative maximum fish swimming distance
ν kinematic viscosity of water

11 SELECTED REFERENCES

11.1 Fishway Design

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INTRODUCTION TO FISHWAY DESIGN

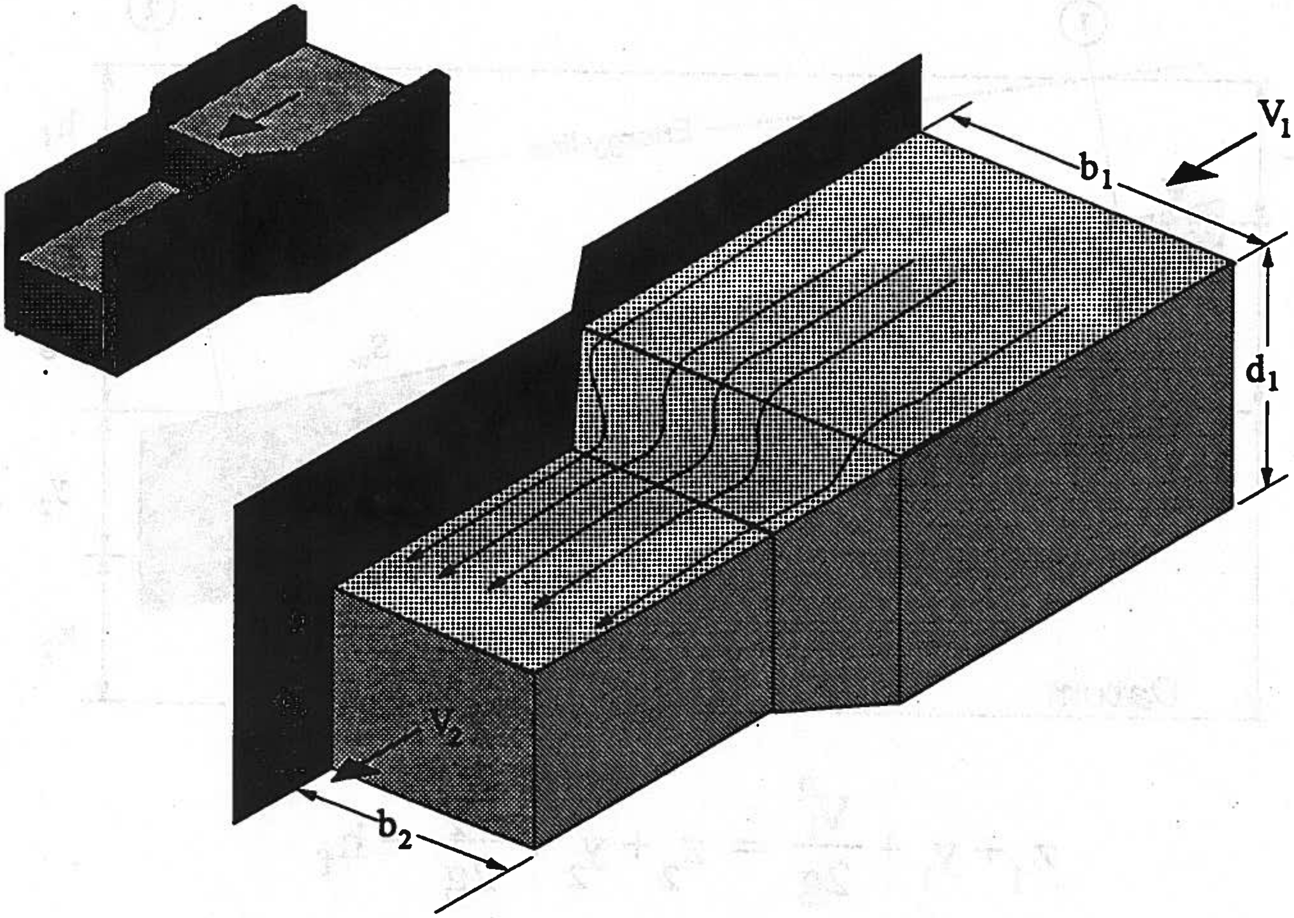
ADDENDUM

by

Chris Katopodis, P.Eng.

NOVEMBER 1994

Conservation of Mass - Continuity



$$Q = V A$$

$$\rho V_1 A_1 = \rho V_2 A_2$$

where: $A_1 = b_1 d_1$
and $A_2 = b_2 d_2$

b = channel width (m of ft).

d = water depth perpendicular to channel bottom (m or ft).

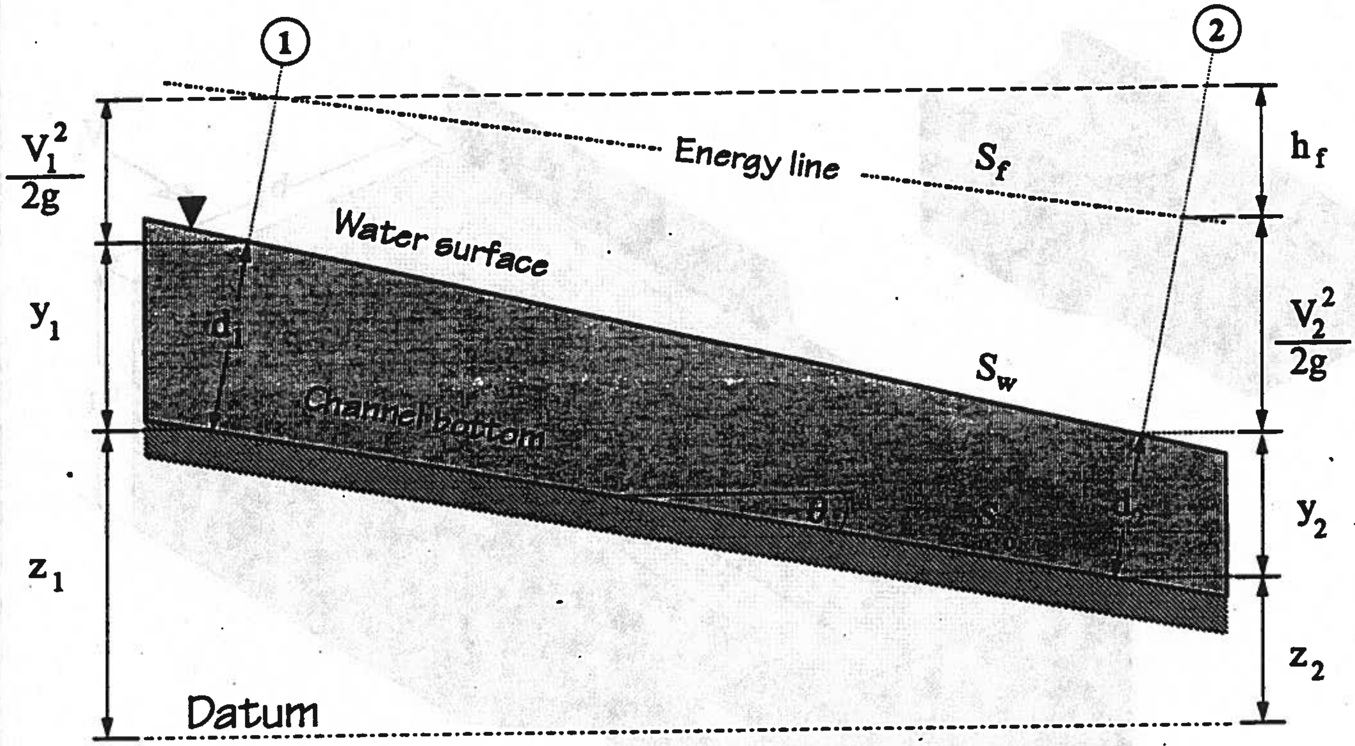
Q = discharge (m^3/s or ft^3/s).

V = average velocity (m/s or ft/s).

ρ = fluid density (for water 1000 kg/m^3 or 62.4 lbm/ft^3).

NOTE: the rapidly varied part of the flow is caused by the sudden change in the cross-sectional area of the channel.

Conservation of Energy



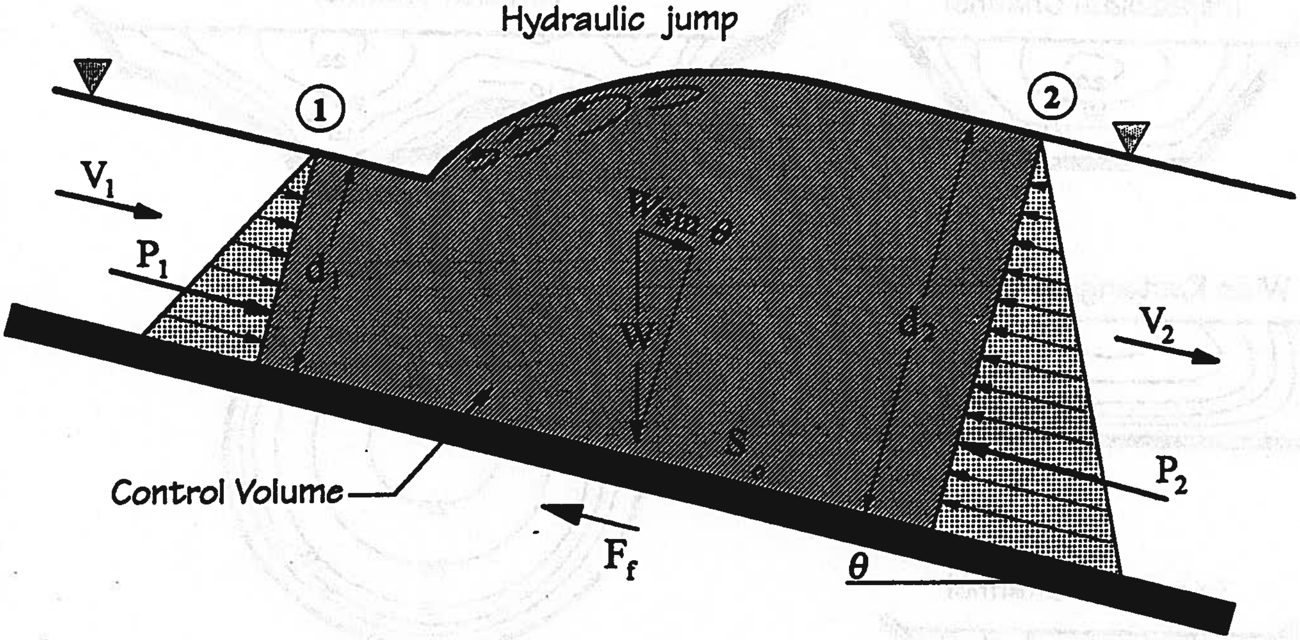
$$z_1 + y_1 + \frac{V_1^2}{2g} = z_2 + y_2 + \frac{V_2^2}{2g} + h_f$$

$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2} = \text{specific energy}$$

NOTE: $y = d\cos\theta$, for $0° < \theta < 10°$ or $0 < S_o < 20\%$ (or 1:5), $y \cong d$ to $0.98\,d$

- d = water depth perpendicular to channel bottom (m or ft).
- g = gravitational acceleration ($9.81\ m/s^2$ or $32.2\ ft/s^2$).
- h_f = energy (head) loss (m or ft).
- z = elevation above datum (m or ft).
- A = cross-sectional area (m^2 or ft^2).
- S_f = slope of energy line.
- $S_o = \tan\theta$ = slope of channel bottom.
- S_w = slope of water surface.
- V = average velocity (m/s or ft/s).
- θ = angle of channel bottom with the horizontal (radians or °).

Conservation of Momentum



$$\text{momentum} = (\rho Q)V \ = \text{mass x velocity}$$

$$\rho Q\,(V_2 - V_1) = P_1 - P_2 + W\sin\theta - F_f$$

d = water depth perpendicular to channel bottom (m or ft).

F_f = friction force (N or lbf).

P = pressure force (N or lbf).

Q = discharge (m^3/s or ft^3/s).

S_o = tan θ = slope of channel bottom.

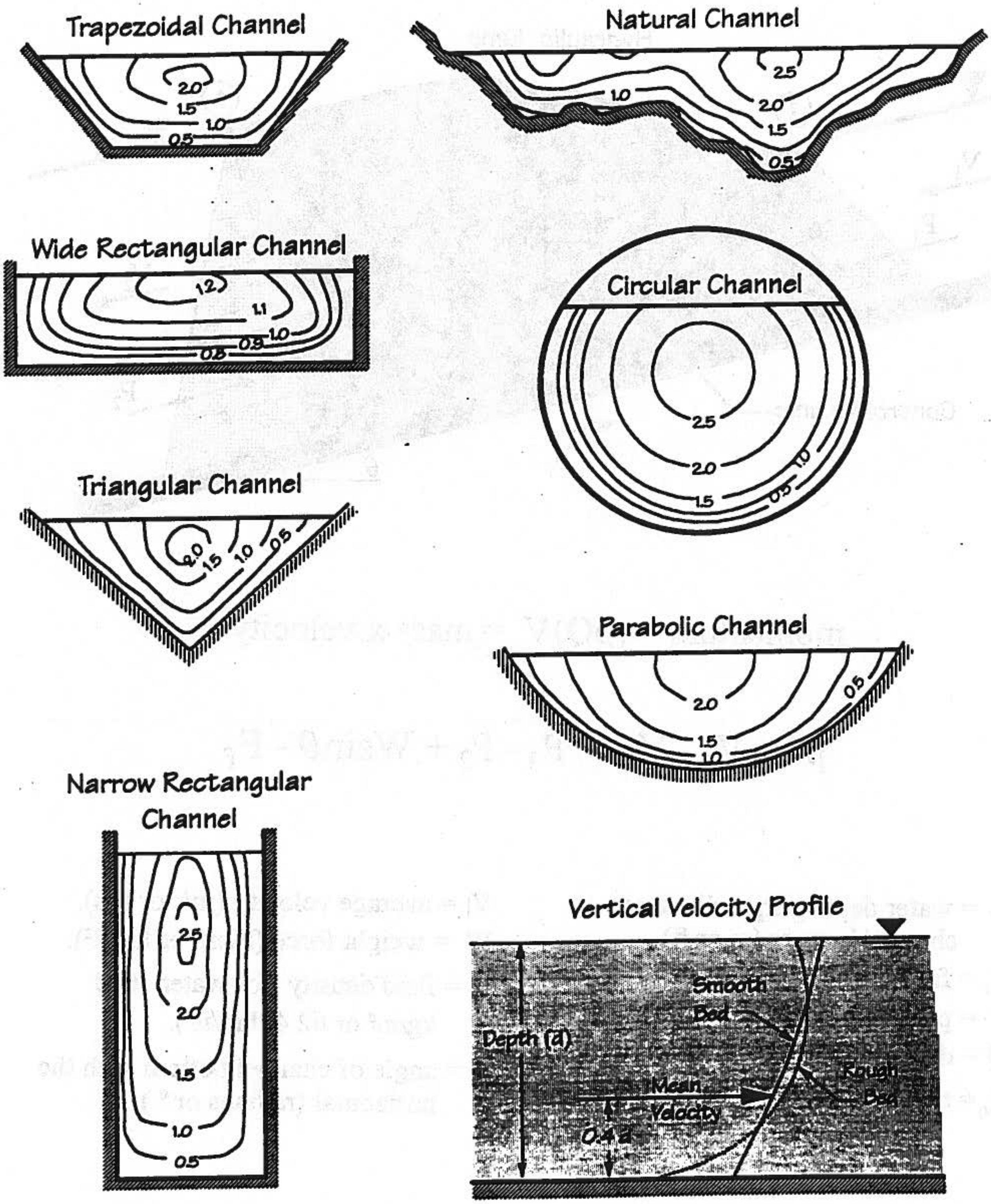
V = average velocity (m/s or ft/s).

W = weight force (N/m^3 or lbf/ft^3).

ρ = fluid density (for water 1000 kg/m^3 or 62.4 lbm/ft^3).

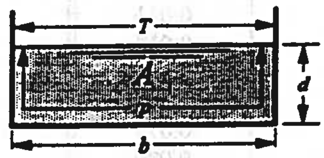
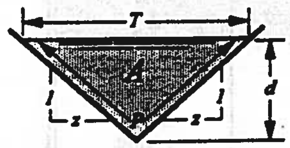
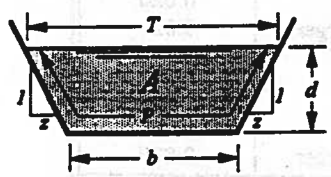
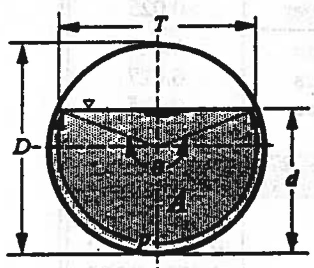
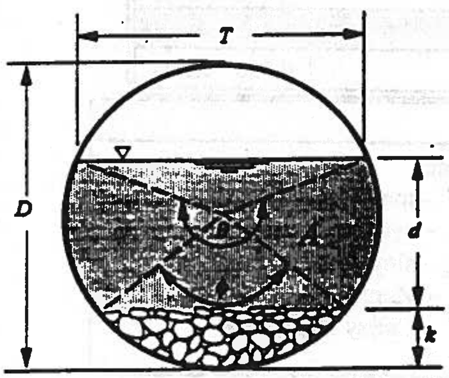
θ = angle of channel bottom with the horizontal (radians or °).

Velocity Distribution Characteristics



Note: contour values are used to illustrate relative velocity intervals and therefore do not correspond to a particular discharge.

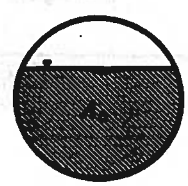
Channel Cross-section Geometry

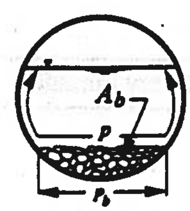
Cross-section	Area (A)	Wetted Perimeter (P)	Hydraulic Radius (R = A/P)	Top Width (T)
 Rectangular	bd	$b+2d$	$\frac{bd}{b+2d}$	b
 Triangular	zd^2	$2d\sqrt{1+z^2}$	$\frac{zd}{2\sqrt{1+z^2}}$	$2zd$
 Trapezoidal	$(b+zd)d$	$b+2d\sqrt{1+z^2}$	$\frac{(b+zd)d}{b+2d\sqrt{1+z^2}}$	$b+2zd$
 Circular	$1/8(\theta-\sin\theta)D^2$	$1/2\,\theta D$	$1/4\{1-(\sin\theta/\theta)\}D$	$\sin(\theta/2)D$
	where: $\theta = 2\cos^{-1}(1-2d/D)$			
 Circular with gravel bed	$A = A_c - A_b$ $A_c = 1/8(\theta-\sin\theta)D^2$ $A_b = 1/8(\phi-\sin\phi)D^2$	$P = P_c + P_b$ $P_c = 1/2(\theta-\phi)D$ $P_b = \sin(\phi/2)D$	$\frac{A_c - A_b}{P_c + P_b}$	$\sin(\theta/2)D$

$\theta = 2\cos^{-1}\{1-2(d+k)/D\}$

$\phi = 2\cos^{-1}(1-2k/D)$

ϕ and θ in radians.





Typical Manning's Roughness Coefficient (n) Values.

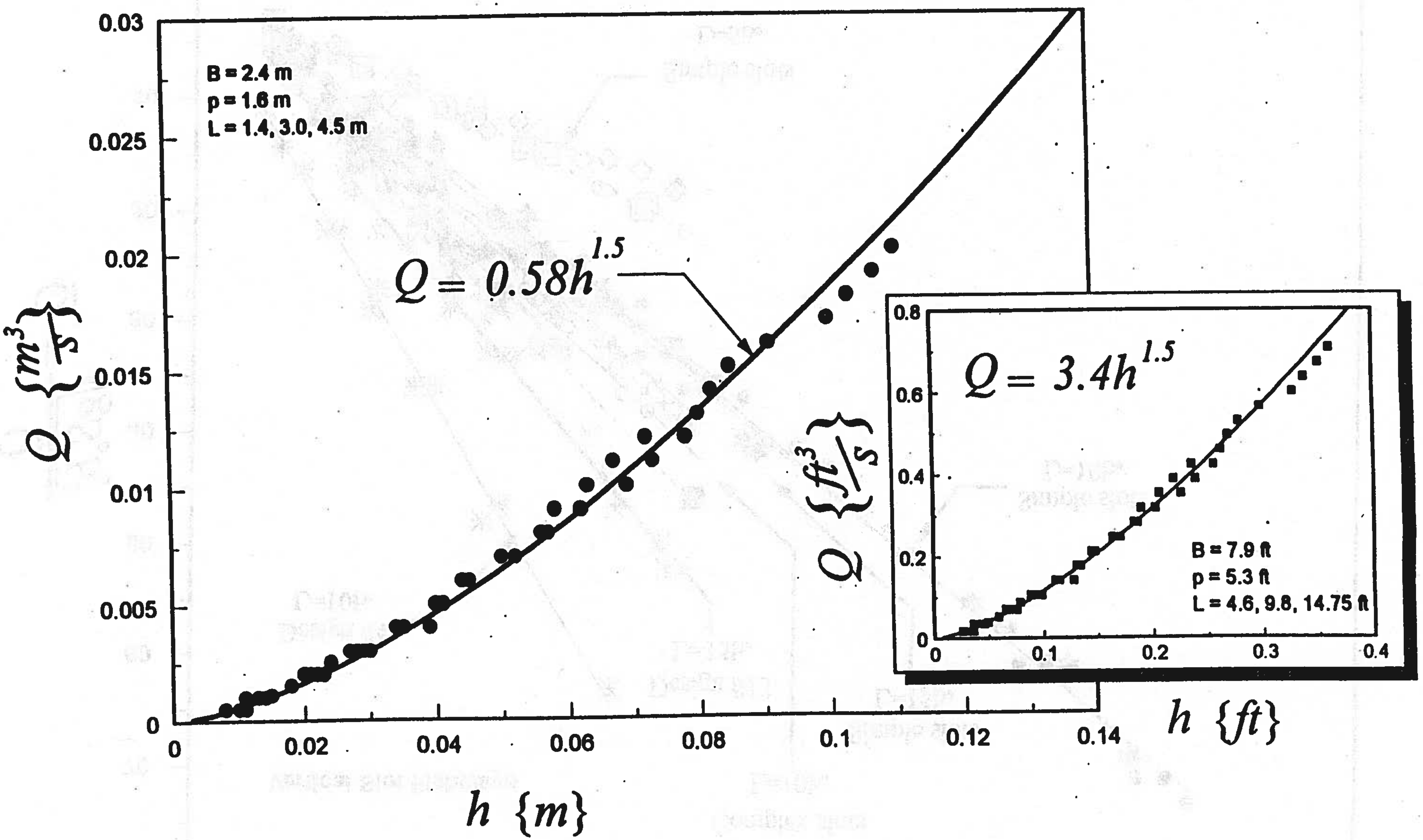
RELATIVELY SMOOTH SURFACES			n
Glass, plastic, machined metal			0.010
Dressed timber, joints flush			0.011
Sawn timber, joints uneven			0.014
Cement plaster			0.011
Concrete, steel troweled			0.012
Concrete, timber forms, unfinished			0.014
Brickwork or dressed masonry			0.014
Rubble set in cement			0.017
Earth, smooth, no weeds			0.020
Earth, some stones and weeds			0.025
CULVERTS - CORRUGATED PIPE	**Corrugations, pitch x depth (mm)**	**Diameter (mm)**	**n**
Structural plate pipe	152 x 51	1500	0.033
		2120	0.032
		3050	0.030
		4610	0.028
Standard Corrugated Steel Pipe - Helical	68 x 13	500	0.015
		900	0.018
		1200	0.020
		1400 & larger	0.021
	76 x 25	1200	0.023
		1600	0.024
		2000	0.026
		2200 & larger	0.027
	125 x 26	1400	0.022
		1800	0.024
		2000 & larger	0.025
Standard Corrugated Steel Pipe - Annular	68 x 13	all diameters	0.024
	76 x 25	all diameters	0.027
	125 x 26	all diameters	0.025
NATURAL RIVER CHANNELS			**n**
Clean and straight			0.025 - 0.030
Winding with pools and shoals			0.033 - 0.040
Very weedy, winding and overgrown			0.075 - 0.150
Mountain stream, containing gravel, cobbles and few large boulders			0.030 - 0.050
Mountain stream, containing cobbles and large boulders			0.040 - 0.070
FISHWAYS			**n**
Denil 2			0.030 - 0.160

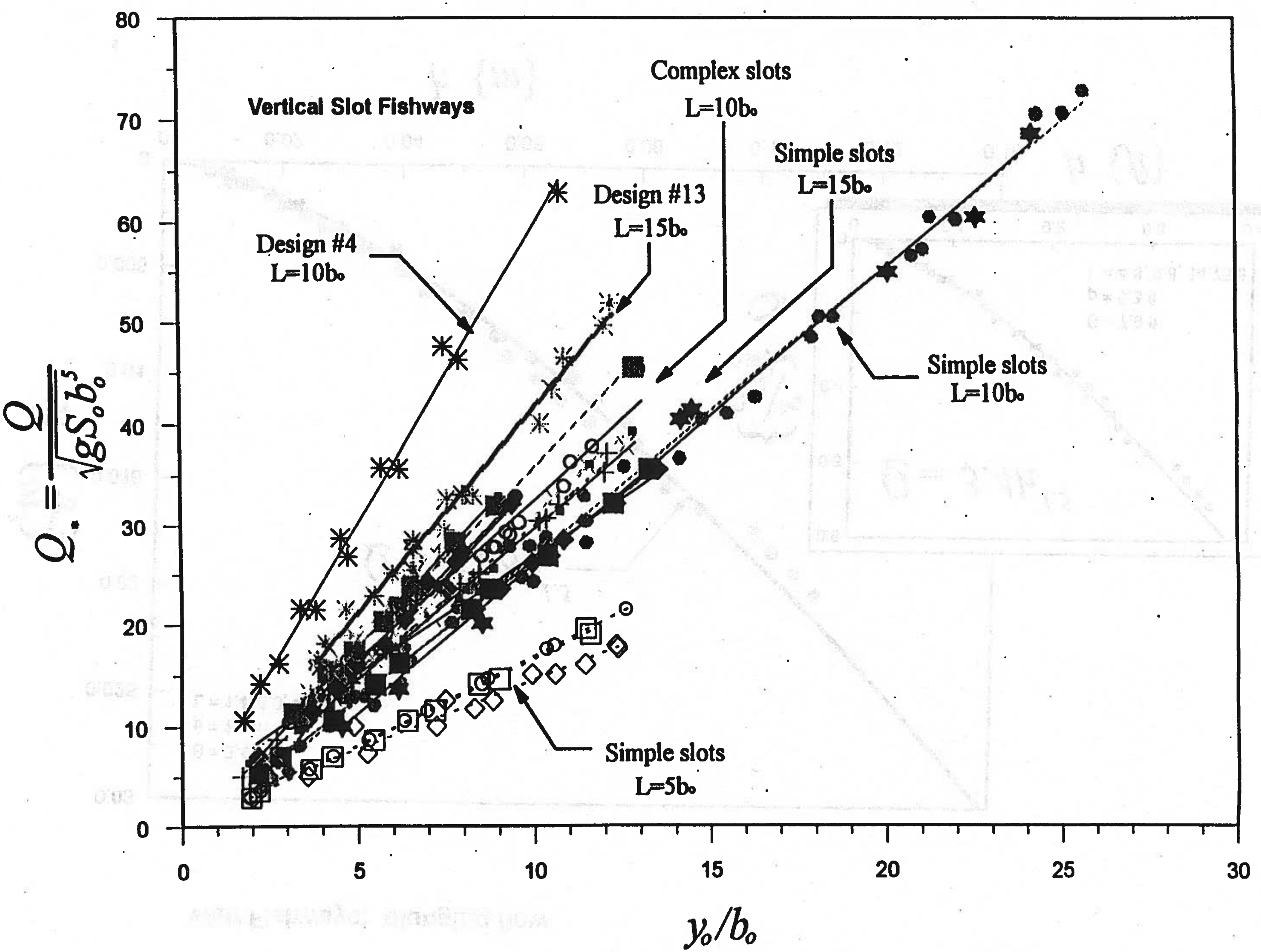
VELOCITY EQUATIONS - UNIFORM FLOW		
Manning's formula:	$V=\frac{1}{n}R^{2/3}S_o^{1/2}$	m, s units
	$V=\frac{1.486}{n}R^{2/3}S_o^{1/2}$	ft, s units
Chézy formula:	$V=C\sqrt{RS_o}$	

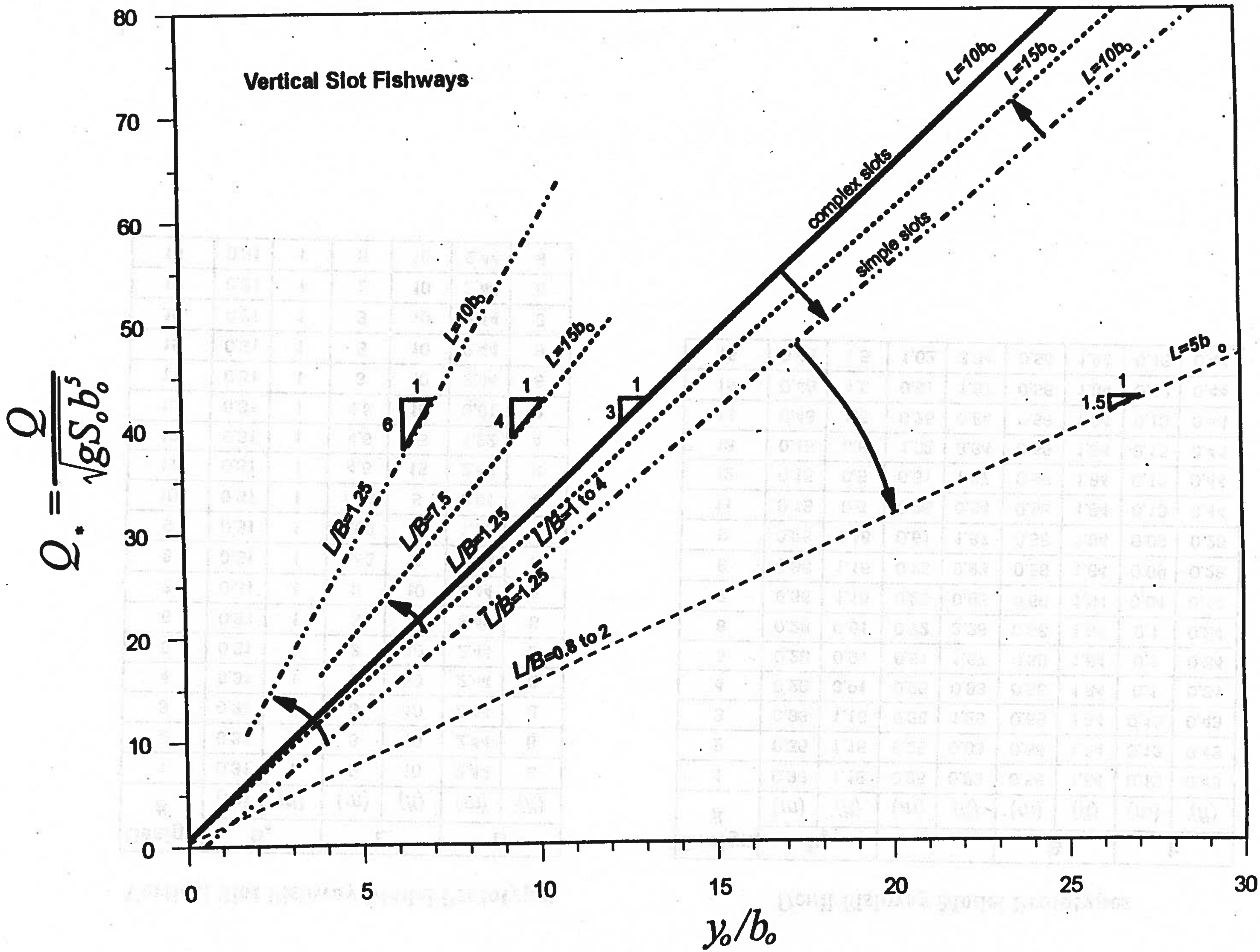
where:

- V - mean velocity (m/s or ft/s)
- R - hydraulic radius (m or ft)
- S_o - slope of the energy line
- n - Manning's coefficient
- C - Chézy coefficient

Weir Fishways: plunging flow




Vertical Slot Fishways
Complex slots
L=10b₀
Simple slots
L=15b₀
Design #13
L=15b₀
Design #4
L=10b₀
Simple slots
L=10b₀
Simple slots
L=5b₀
$Q_* = \frac{Q}{\sqrt{g S_o b_o^5}}$
y_o/b_o
0
5
10
15
20
25
30
0
10
20
30
40
50
60
70
80


Vertical Slot Fishways
complex slots
simple slots
L=10b₀
L=15b₀
L=10b₀
L=5b₀
L/B=1.25
L/B=1.5
L/B=1.25
L/B=1 to 4
L/B=1.25
L/B=0.8 to 2
$Q_* = \frac{Q}{\sqrt{g S_o b_o^5}}$
y_o/b_o

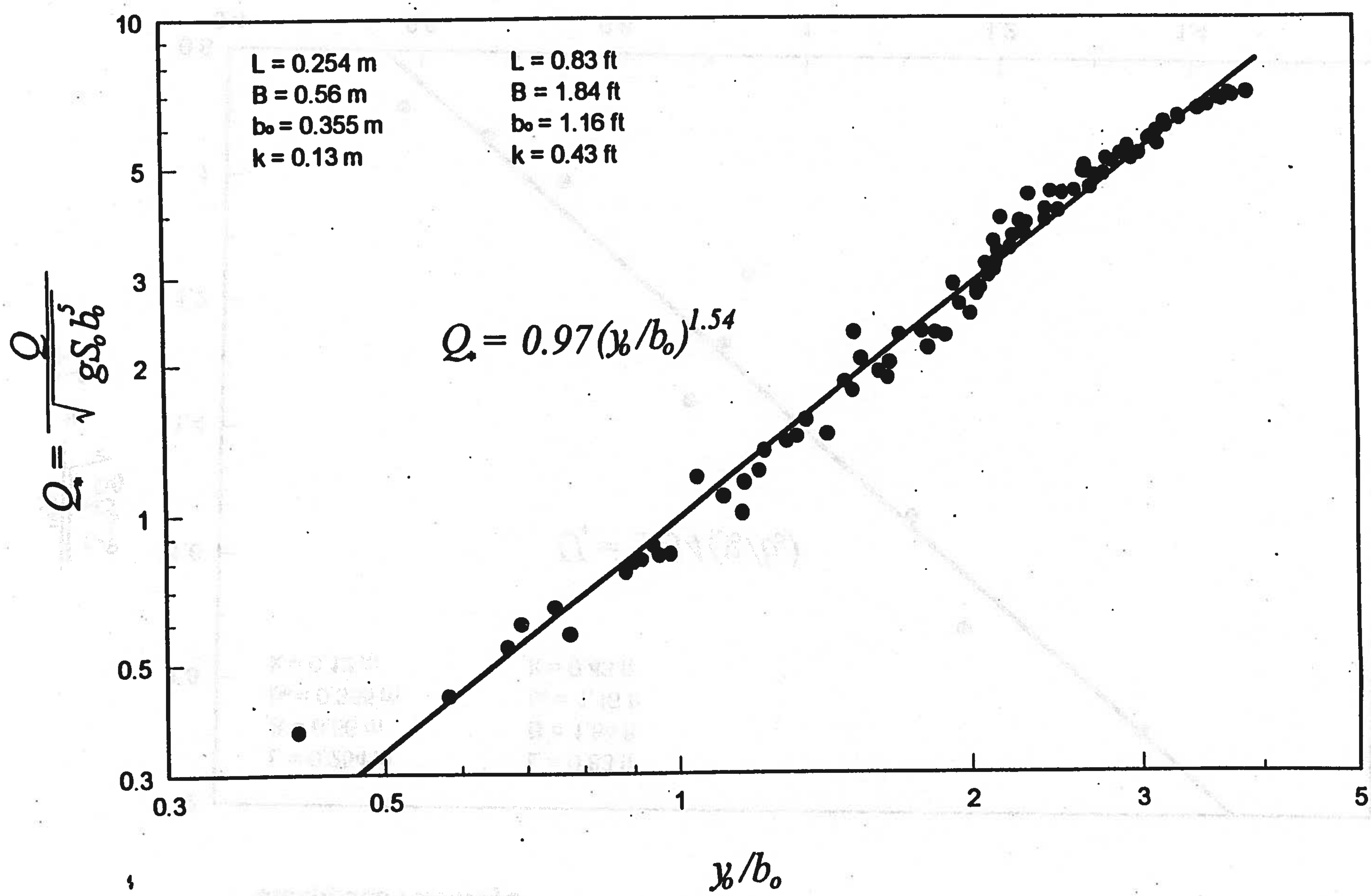
Vertical Slot Fishway Model Prototypes

Design #	b_o		L		B	
	(m)	(ft)	(m)	(ft)	(m)	(ft)
1	0.31	1	3	10	2.44	8
2	0.31	1	3	10	2.44	8
3	0.31	1	3	10	2.44	8
4	0.31	1	3	10	2.44	8
5	0.31	1	3	10	2.44	8
6	0.31	1	3	10	2.44	8
7	0.31	1	3	10	2.44	8
8	0.31	1	1.53	5	2.44	8
9	0.31	1	1.53	5	1.22	4
10	0.31	1	1.53	5	0.61	2
11	0.31	1	4.6	15	2.44	8
12	0.31	1	4.6	15	1.22	4
13	0.31	1	4.6	15	0.61	2
14	0.31	1	3	10	2.44	8
15	0.31	1	3	10	2.44	8
16	0.31	1	3	10	2.44	8
17	0.31	1	3	10	2.44	8
18	0.31	1	3	10	2.44	8

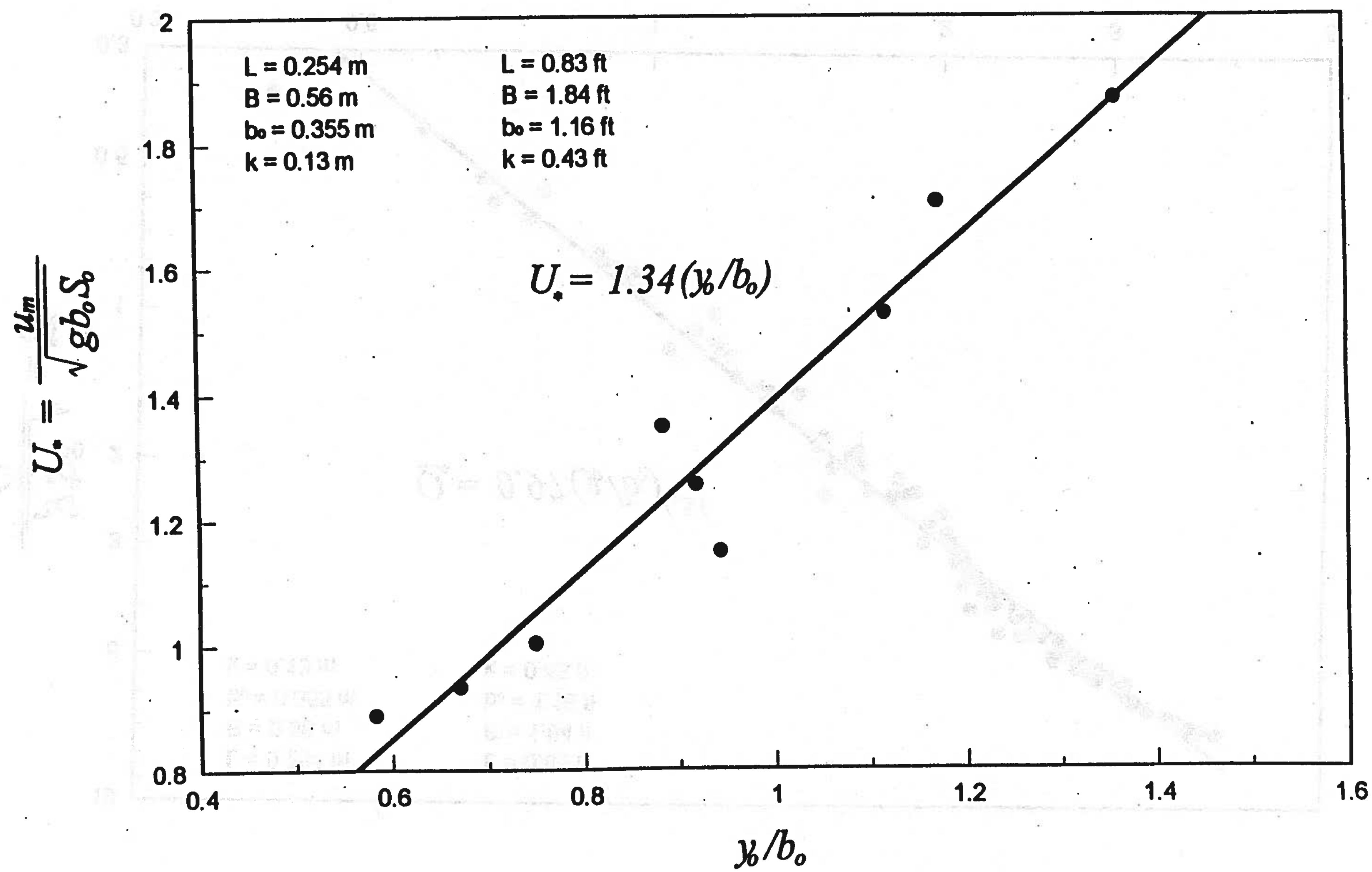
Denil Fishway Model Prototypes

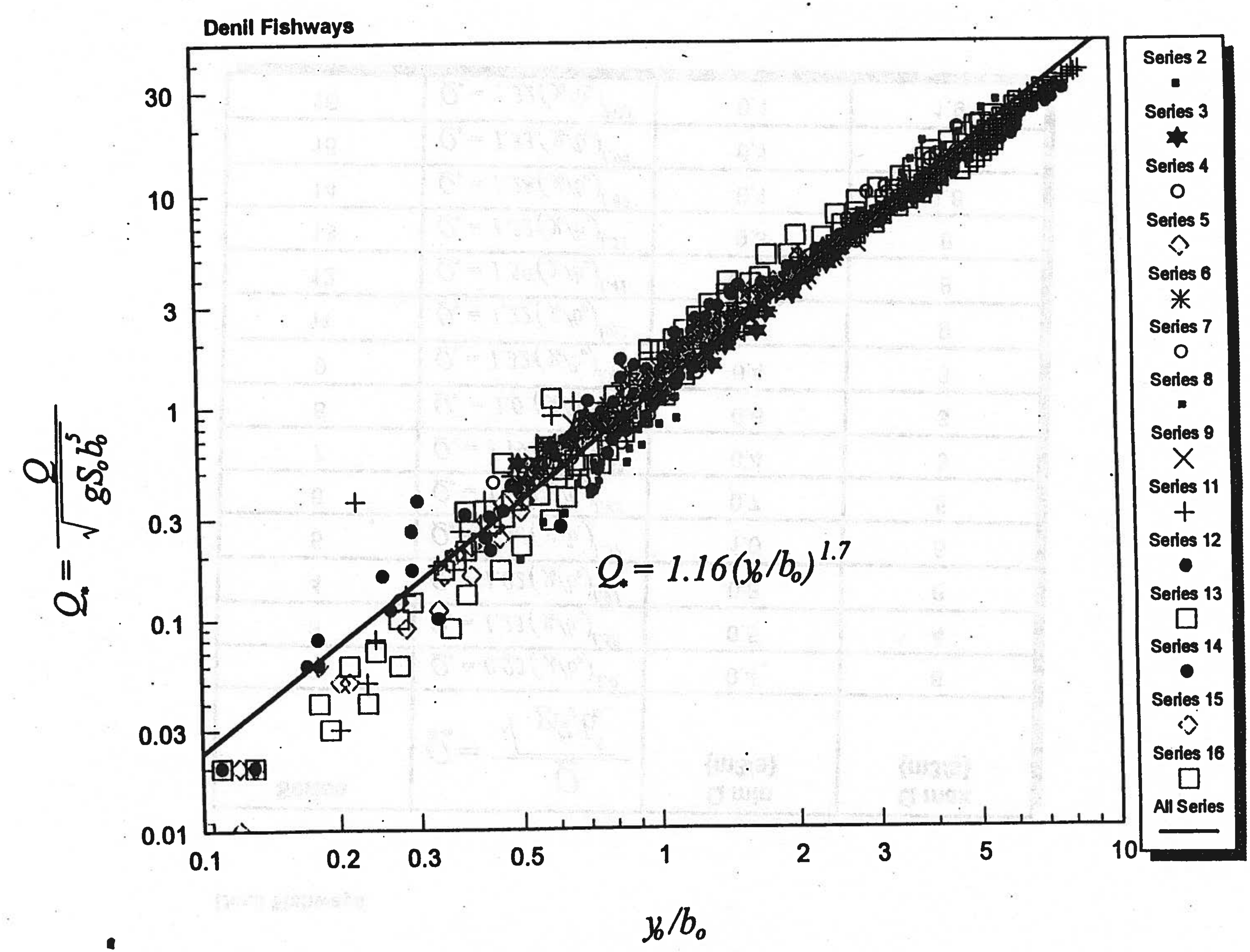
Design #	b_o		L		B		k	
	(m)	(ft)	(m)	(ft)	(m)	(ft)	(m)	(ft)
1	0.36	1.18	0.25	0.83	0.56	1.84	0.13	0.43
2	0.36	1.16	0.25	0.83	0.56	1.84	0.13	0.43
3	0.36	1.16	0.38	1.25	0.56	1.84	0.13	0.43
4	0.28	0.91	0.25	0.83	0.56	1.84	0.1	0.34
5	0.28	0.91	0.51	1.67	0.56	1.84	0.1	0.34
6	0.28	0.91	0.72	2.36	0.56	1.84	0.1	0.34
7	0.36	1.16	0.25	0.83	0.56	1.84	0.04	0.14
8	0.36	1.16	0.25	0.83	0.56	1.84	0.09	0.28
9	0.36	1.16	0.51	1.67	0.56	1.84	0.09	0.28
11	0.15	0.5	0.26	0.84	0.56	1.84	0.13	0.44
12	0.15	0.5	0.51	1.67	0.56	1.84	0.13	0.44
13	0.15	0.5	1.02	3.34	0.56	1.84	0.13	0.44
14	0.46	1.5	0.26	0.84	0.56	1.84	0.13	0.44
15	0.46	1.5	0.51	1.67	0.56	1.84	0.13	0.44
16	0.46	1.5	1.02	3.34	0.56	1.84	0.13	0.44

Steeppass Fishways



Steeppass Fishways



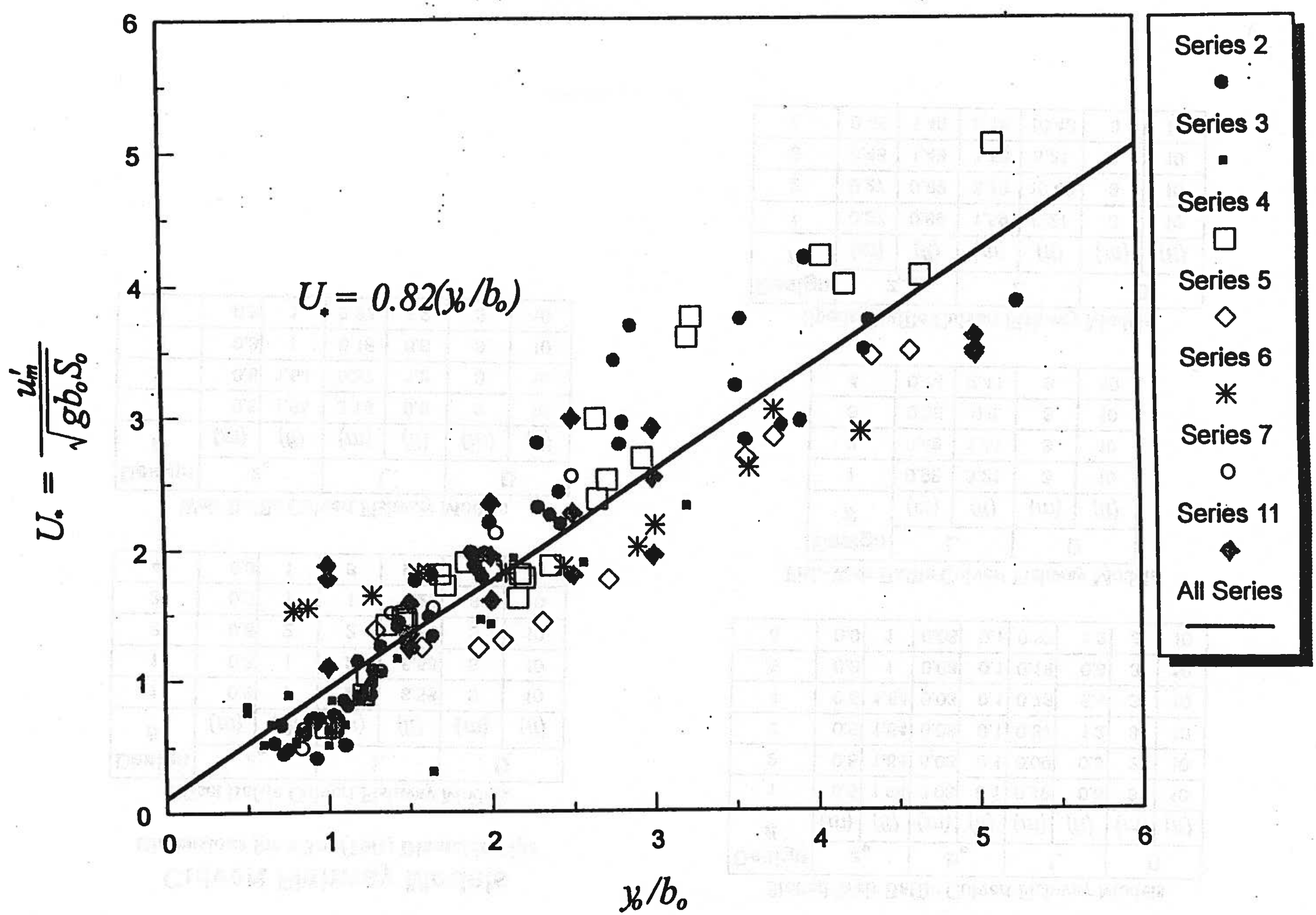


Denil Fishways
$Q_* = \frac{Q}{\sqrt{g S_o b_o^5}}$
30
10
3
1
0.3
0.1
0.03
0.01
0.1
0.2
0.3
0.5
1
2
3
5
10
y_o/b_o
$Q_* = 1.16(y_o/b_o)^{1.7}$
Series 2
Series 3
Series 4
Series 5
Series 6
Series 7
Series 8
Series 9
Series 11
Series 12
Series 13
Series 14
Series 15
Series 16
All Series

Denil Fishways

Series	$Q_* = \frac{Q}{\sqrt{gS_o b_o^5}}$	Q min {m3/s}	Q max {m3/s}
2	$Q_* = 0.93(y_o/b_o)^{2.0}$	0.4	6
3	$Q_* = 1.23(y_o/b_o)^{1.58}$	0.5	4
4	$Q_* = 1.02(y_o/b_o)^{1.91}$	0.8	6
5	$Q_* = 1.36(y_o/b_o)^{1.57}$	1.0	5
6	$Q_* = 1.61(y_o/b_o)^{1.43}$	0.7	5
7	$Q_* = 1.19(y_o/b_o)^{1.62}$	0.4	3
8	$Q_* = 1.0(y_o/b_o)^{1.89}$	0.6	3
9	$Q_* = 1.32(y_o/b_o)^{1.43}$	0.4	3
11	$Q_* = 1.22(y_o/b_o)^{1.62}$	0.2	9
12	$Q_* = 1.59(y_o/b_o)^{1.41}$	0.27	8
13	$Q_* = 1.52(y_o/b_o)^{1.51}$	0.3	8
14	$Q_* = 1.28(y_o/b_o)^{1.82}$	0.1	1.8
15	$Q_* = 1.33(y_o/b_o)^{1.99}$	0.1	1.9
16	$Q_* = 1.32(y_o/b_o)^{2.09}$	0.1	1.9

Denil Fishways



Culvert Fishway Models

Dimensions for a 3m (10ft) Diameter Pipe

Offset Baffle Culvert Fishway Models

Design #	z_o		L		D	
	(m)	(ft)	(m)	(ft)	(m)	(ft)
1	0.3	1	2	6.56	3	10
1	0.3	1	2	6.56	3	10
2	0.6	2	2	6.56	3	10
3	0.3	1	1	3.28	3	10
4	0.3	1	3	9.84	3	10

Weir Baffle Culvert Fishway Models

Design #	z_o		L		D	
	(m)	(ft)	(m)	(ft)	(m)	(ft)
1	0.5	1.64	0.18	0.6	3	10
2	0.5	1.64	0.37	1.2	3	10
3	0.3	1	0.18	0.6	3	10
4	0.3	1	0.37	1.2	3	10

Slotted Weir Baffle Culvert Fishway Models

Design #	z_o		b_o		L		D	
	(m)	(ft)	(m)	(ft)	(m)	(ft)	(m)	(ft)
1	0.5	1.64	0.03	0.1	0.18	0.6	3	10
2	0.5	1.64	0.03	0.1	0.09	0.3	3	10
3	0.5	1.64	0.03	0.1	0.37	1.2	3	10
4	0.5	1.64	0.03	0.1	0.73	2.4	3	10
5	0.3	1	0.03	0.1	0.18	0.6	3	10
6	0.3	1	0.03	0.1	0.37	1.2	3	10

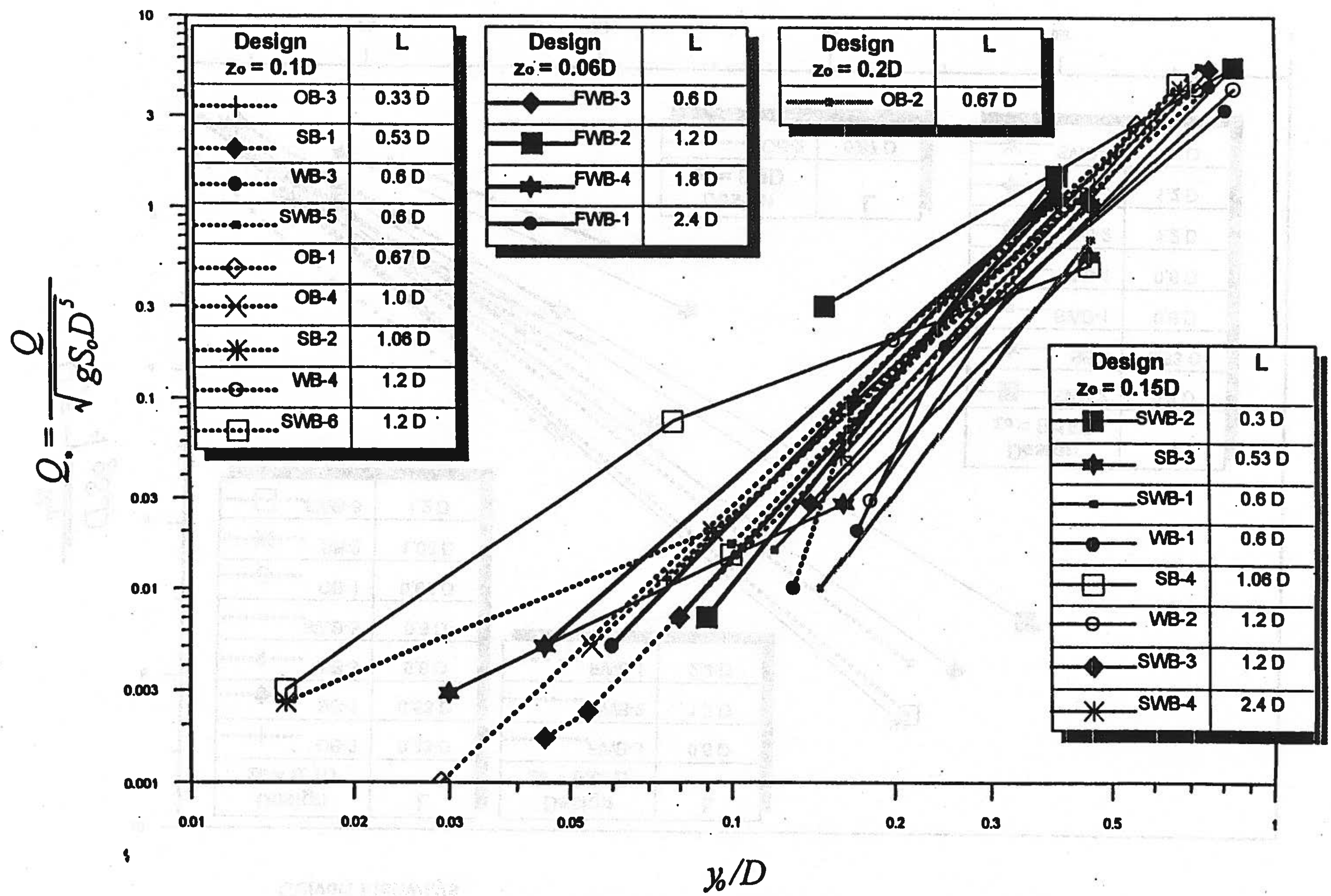
Fish-Weir Baffle Culvert Fishway Models

Design #	L		D	
	(m)	(ft)	(m)	(ft)
1	0.98	3.21	3	10
2	0.49	1.61	3	10
3	0.25	0.8	3	10
4	0.74	2.41	3	10

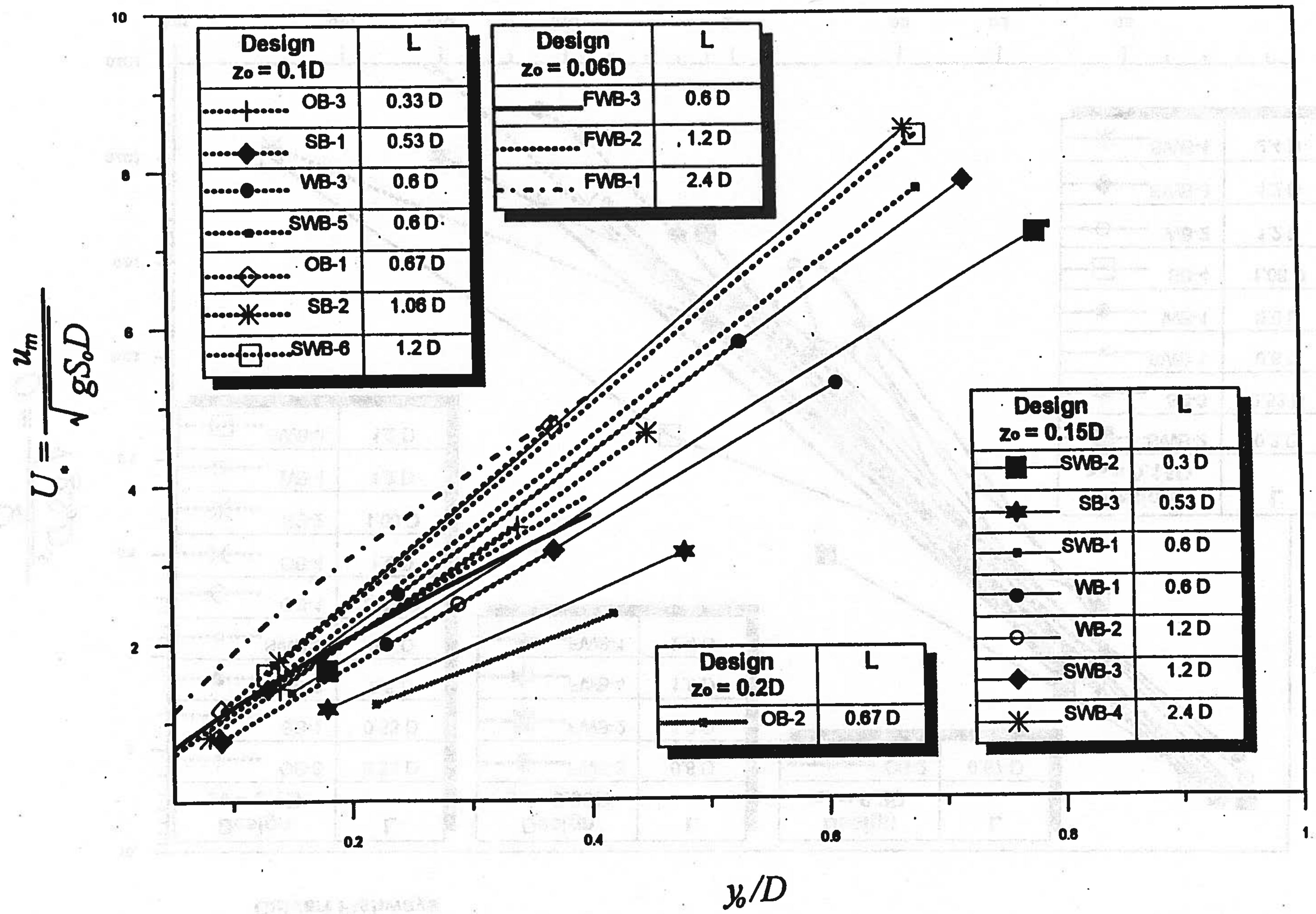
Spoiler Baffle Culvert Fishway Models

Design #	z_o		L		D	
	(m)	(ft)	(m)	(ft)	(m)	(ft)
1	0.27	0.89	1.59	5.21	3	10
2	0.27	0.89	3.18	10.43	3	10
3	0.45	1.48	1.59	5.21	3	10
4	0.45	1.48	3.18	10.43	3	10

Culvert Fishways



Culvert Fishways



HYDRAULIC SIMILITUDE - PHYSICAL HYDRAULIC MODELING

A model, commonly a small replica of a prototype, reproduces physical phenomena and allows measurements made in the model to be transfered to the prototype. The most important conditions for similarity between model and prototype are:

- **Geometric similarity:** the shape of the model is the same as that of the prototype (similar flow boundaries). In making the model, all ptototype dimensions (lengths, widths, heights) are reduced by the same factor called the scale.
- **Dynamic similarity:** the ratio of any two forces acting in the model must be the same as the ratio of the corresponding forces in the prototype.

Geometric and dynamic similarity commonly ensure kinematic similarity as well. Kinematic similarity means that the shape of streamlines and flow patterns at any particular time are the same in model and prototype.

In **Hydraulics** the most common forces involved in fluid motion are:

- **Inertia (F_I):** the resistance or "reluctance" of a particle or fluid volume to accelerate. This force is always present and is proportional to mass x acceleration or $(\rho L^3)\{V/(L/V)\}$.

  F_I prop. to $\rho L^2 V^2$ (1)

- **Gravity (F_g):** weight of particle or fluid volume. This force is usually dominant in free surface flows and is proportional to mass x gravitational acceleration or

  F_g prop. to $\rho L^3 g$ (2)

- **Viscosity (F_v):** generated primarily by intermolecular cohesive forces and offers resistance to shear deformation and is important when flow is not fully turbulent or for a submerged body moving through a fluid. This force is proportional to shear stress x area or

  F_v prop. to $\mu V L$ (3)

- **Pressure (F_p):** difference in hydrostatic pressure. This force is proportional to pressure change x area or

  F_p prop. to $\Delta p L^2$ (4)

NOTE:

L is a linear dimension (e.g. length; m or ft)

V is fluid velocity (m/s or ft/s)

Δp is the fluid pressure on a surface (N/m^2 or lbf/ft^2)

g is the gravitational acceleration (9.81 m/s^2 or 32.2 ft/s^2)

ρ is the density of the fluid (for water at 4°C 1000 kg/m^3; or 62.4 lbm/ft^3 or 1.94 $slugs/ft^3$)

μ is the dynamic viscosity of the fluid (varies with fluid temperature)

(for water at 20°C 0.001 kg/ms or Ns/m^2; or 2×10^{-5} lbf-s/ft^2)

ν is the kinematic viscosity of the fluid (for water at 20°C 1×10^{-6} m^2/s or 1×10^{-5} ft^2/s)

γ is the specific weight of the fluid (for water 9810 N/m^{3}or 62.4 lbf/ft^3)

$$\gamma = \rho g \tag{5}$$

$$\nu = \mu/\rho \tag{6}$$

Common Dimensionless Parameters (force ratios):

- **Froude number**

$$Fr = \left[\frac{\text{inertia force}}{\text{gravity force}}\right]^{1/2} = \frac{V}{\sqrt{gL}} \tag{7}$$

- **Reynolds Number**

$$Re = \left[\frac{\text{inertia force}}{\text{viscous force}}\right] = \frac{VL}{\nu} \tag{8}$$

- **Euler Number**

$$Eu = \left[\frac{\text{pressure force}}{\text{inertia force}}\right] = \frac{\Delta p}{\rho V^2} \tag{9}$$

Also known as pressure coefficient (e.g. pitot tube) or drag coefficient (e.g. submerged body moving through water) and written as:

$$C_p = \frac{\Delta p}{\frac{1}{2}\rho V^2} \tag{10}$$

The sqare root of the inverse of (10) is known as the velocity coefficient (e.g. orifice, nozzle) and written as:

$$C_v = \frac{V}{\sqrt{2\Delta p/\rho}} \tag{11}$$

In open channel flow Froudian similitude (i.e. Fr for model and prototype are the same) is common. In this case, if the subscript r denotes the ratio of a prototype (subscript p) : model (subscript m) quantity, the following apply:

$$\left.\begin{array}{c} \text{Length } L_r = L_p/L_m \text{ ; Velocity } V_r = V_p/V_m = L_r^{1/2} \\ \text{Discharge } Q_r = Q_p/Q_m = L_r^{5/2} \text{ ; Time } t_r = t_p/t_m = L_r^{1/2} \\ \text{Mass } M_r = M_p/M_m = \rho_r L_r^3\text{; Force } F_r = F_p/F_m = \rho_r L_r^3 \end{array}\right\} \tag{12}$$

Pressure $p_r = p_p/p_m = \rho_r L_r$

Note: ρ_r is 1 if the same fluid is used in model and prototype

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ANALYSIS OF ICHTHYOMECHANICAL DATA FOR FISH PASSAGE OR EXCLUSION SYSTEM DESIGN

Chris Katopodis
Freshwater Institute
Central and Arctic Region
Department of Fisheries and Oceans
501 University Crescent
Winnipeg, MB., CANADA R3T 2N6
Phone: (204) 983-5181 Fax: (204) 984-2402

Ichthyomechanical data base

Fish speed and stamina, locomotion and the mechanics of fish swimming, are key to the development and design of passage, exclusion, and guidance systems such fishways (including culverts), fish screens, fish barriers (including sea lamprey velocity barriers), and fish louvers. The large amount of data available, although primarily from laboratory respirometer studies and unevenly distributed between species, offered an opportunity to consider its systematic application to the development, design and testing of such devices. With this motive, comprehensive searches were made and literature on fish swimming performance tests was compiled, and published data were entered on spreadsheets. This data base includes the following information: scientific and common fish species name, swimming mode, fish length (*l* in m), swimming speed (*U* in m/s), endurance or time to fatigue (*t* in s), water temperature during testing, life stage (e.g. juvenile or adult), test method (e.g. constant or increasing velocity), number of fish tested, regressions of swimming speed versus fish length for specific endurance times as reported in the literature, publication reference (author and date), and relevant comments (Katopodis and Gervais 1991). The data base may be consulted for information on specific species, although many species either have very limited data or are not represented at all. The data base, which is available on request, is presently been revised and updated. Investigators with additional data which do not appear in primary publications, are encouraged to provide it for inclusion.

Analytical framework

Figure 1. This graph illustrates the widely reported trend that larger fish can maintain higher swimming speeds. It also shows that, except for some outliers, most data seem to group by swimming mode. Note that available data represent almost exclusively the anguilliform and subcarangiform swimming modes.

Figure 2. If the points on the upper left are considered outliers, this graph provides a hint that larger fish may start fatiguing later and may last longer at a specific speed than small fish.

Figure 3. The reduction of swimming speed with endurance time is indicated by this graph, which is analogous to exponential decay. Data points appear to group by swimming mode in the prolonged endurance range.

Figure 4. This graph is a reflection of Figure 3, except that normalized dimensionless variables are used for speed and endurance. The anguilliform and subcarangiform swimming modes appear quite distinct in the prolonged endurance range.

Figure 5. Using the same dimensionless variables as in Figure 4 for the two predominant swimming modes only and by screening the data as indicated on the graph, regression analysis for the anguilliform data set and the subcarangiform data set in the prolonged range was performed. This figure indicates that for each of the two swimming modes, data tend to collapse within relatively small regions of the graph even though diverse species, data sources, and test methods are involved, particularly in the subcarangiform data set (Katopodis 1991).

Figure 6. Mean regression lines and regression equations are presented. Lines that envelope each data set are also illustrated, along with the corresponding equations. Mean lines may be used as an estimate when it is desirable to pass or exclude 50% of a group of fish. Lower envelope lines may be used as an estimate when it is desirable to pass or protect almost all fish (e.g. fish screens). Upper envelope lines may be used as an estimate when it is desirable to block the passage (e.g. sea lamprey in the Great Lakes) or exclude almost all fish. Northern pike data, although very limited are included for comparison.

Figure 7. This figure is derived mathematically from the equations in Figure 6 by using the relationship $X=(U-V)t$, where X is the distance fish travel against a water velocity V by swimming for a time t. X_{max} refers to the assumption that fish will choose to maximize swimming distance when faced with a specific water velocity. Mean and envelope lines were calculated from the corresponding equations from Figure 6. These curves are useful when spacing resting areas for fish, designing culverts, fishways or velocity barriers, where the distance travelled is a key parameter.

Figure 8. This figure presents mean swimming distances only, for selected fish lengths and is a direct application of the corresponding relationships in Figure 7. It is a more straight forward graph to use since variables are presented in actual units, but of course it is limited to the fish lengths illustrated and to mean values only. The insert on the lower left corner provides a three dimensional representation of the surfaces of mean swimming performance for the anguilliform and subcarangiform modes. Performance in the burst range for both swimming modes is defined by the surface labelled "burst". Performance in the prolonged and sustained ranges are defined by the top surfaces for the subcarangiform group and the bottom surfaces for the anguilliform group. Performance similarities in the burst range and performance differences in the prolonged and sustained ranges are evident for fish of the same length.

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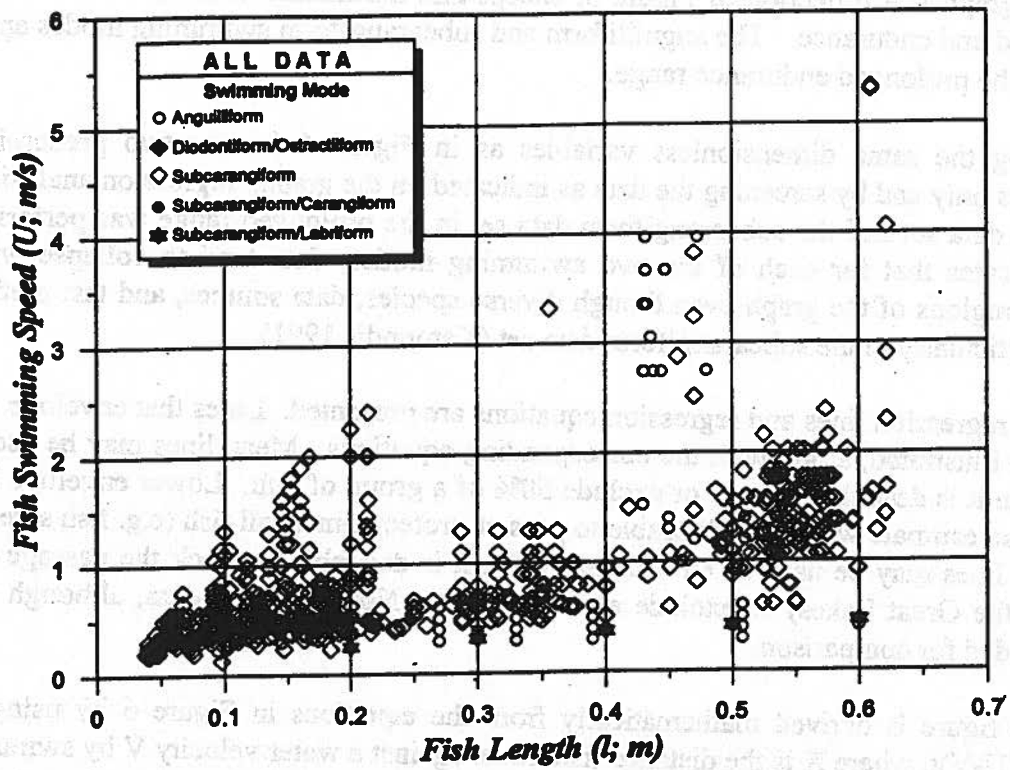


Fig. 1. Swimming speed vs fish length by swimming mode.

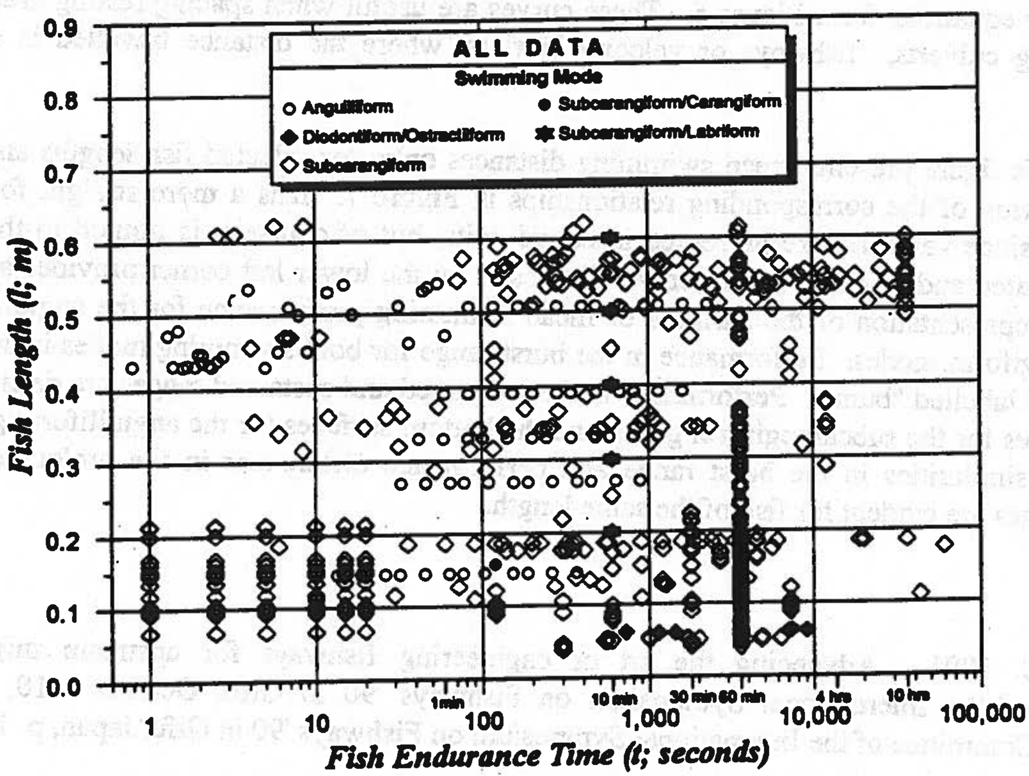


Fig. 2. Fish length vs endurance by swimming mode.

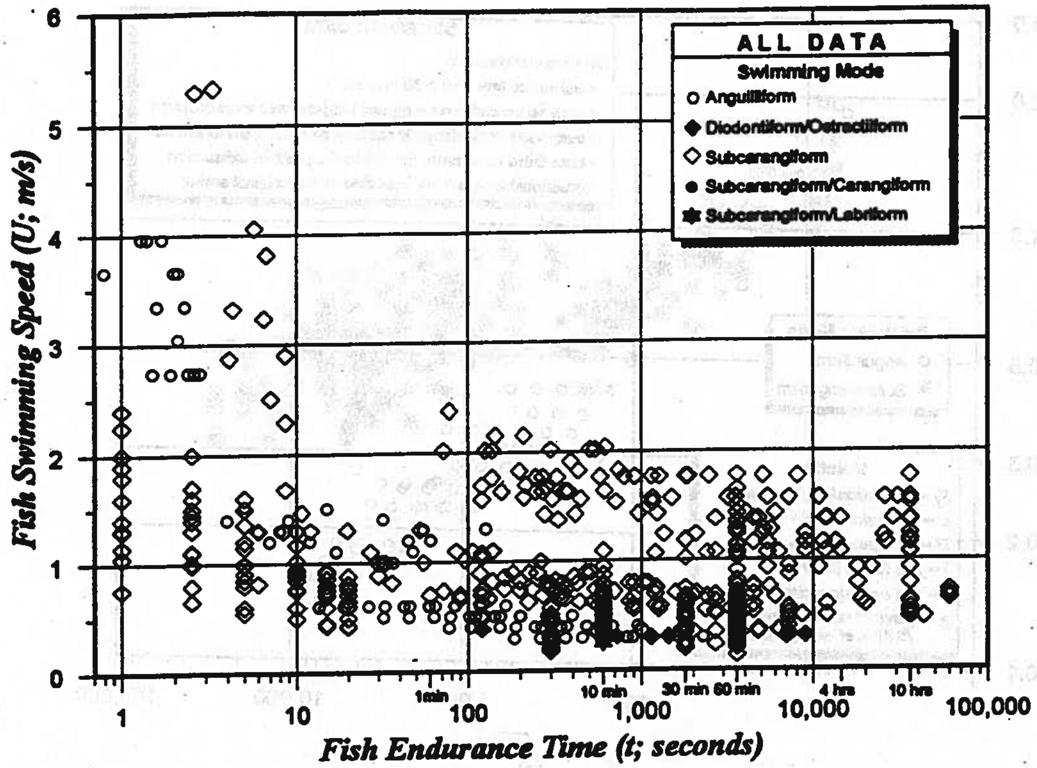


Fig. 3. Fish swimming speed vs endurance by swimming mode.

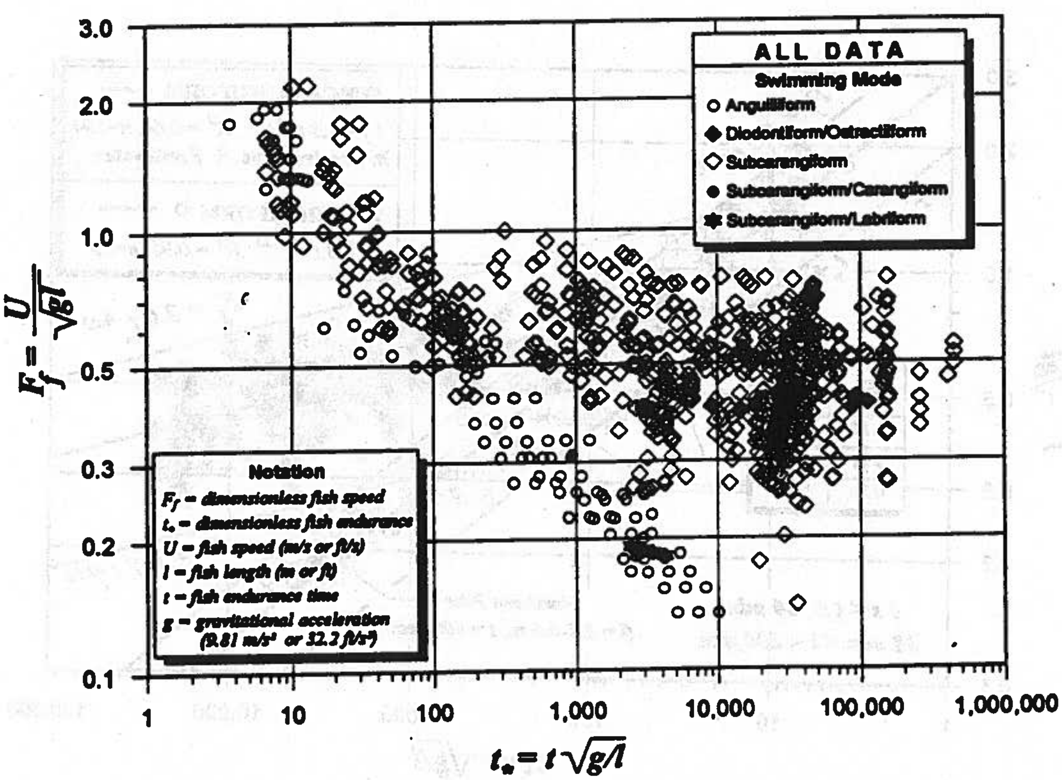


Fig. 4. Dimensionless form of fish speed vs endurance by swimming mode.

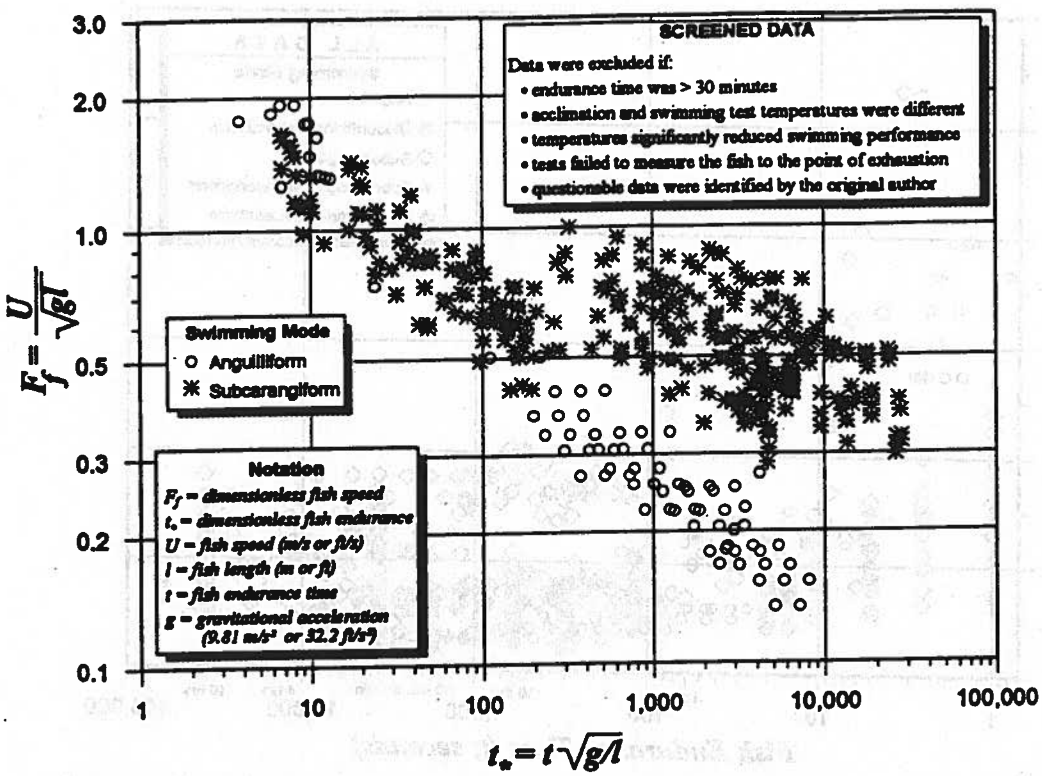
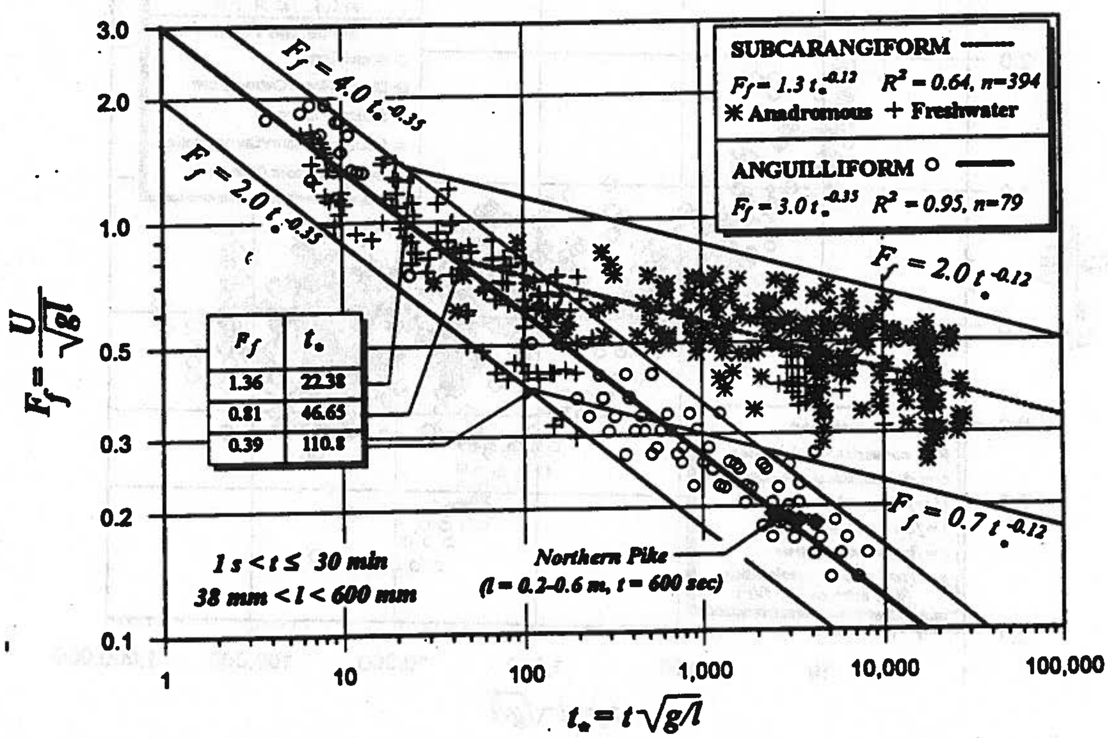


Fig. 5. Dimensionless form of fish speed vs endurance by swimming mode.



F_f	t_*
1.36	22.38
0.81	46.65
0.39	110.8

Fig. 6. Dimensionless endurance: mean and envelope lines.

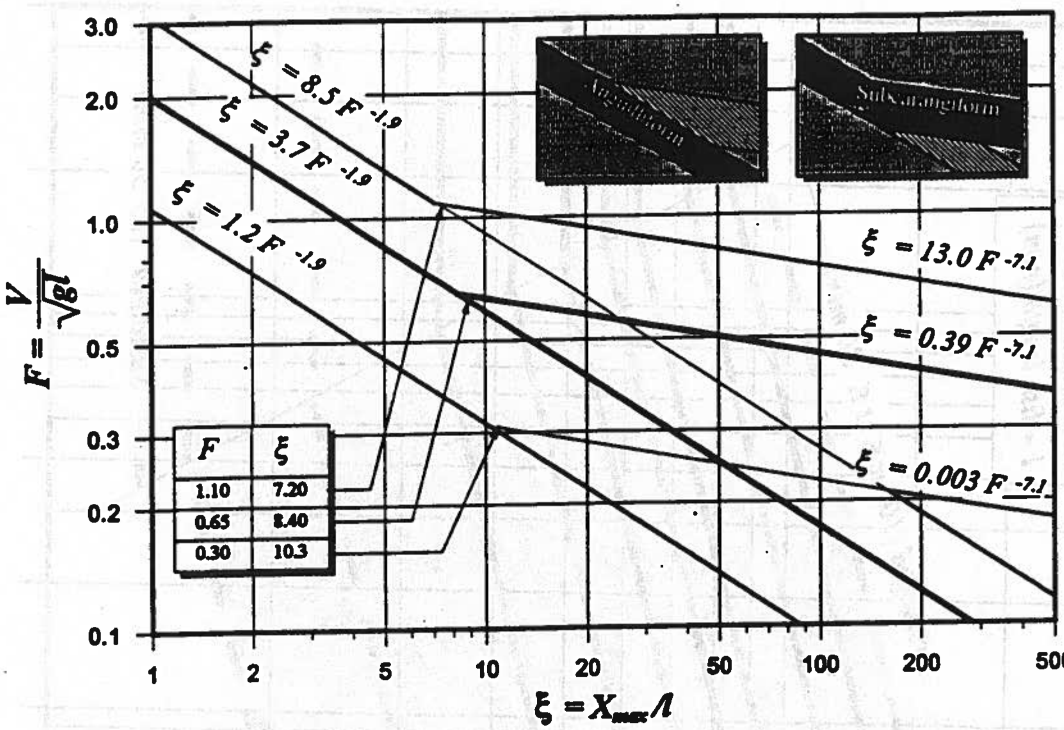


Fig. 7. Dimensionless swimming distance: mean and envelope lines.

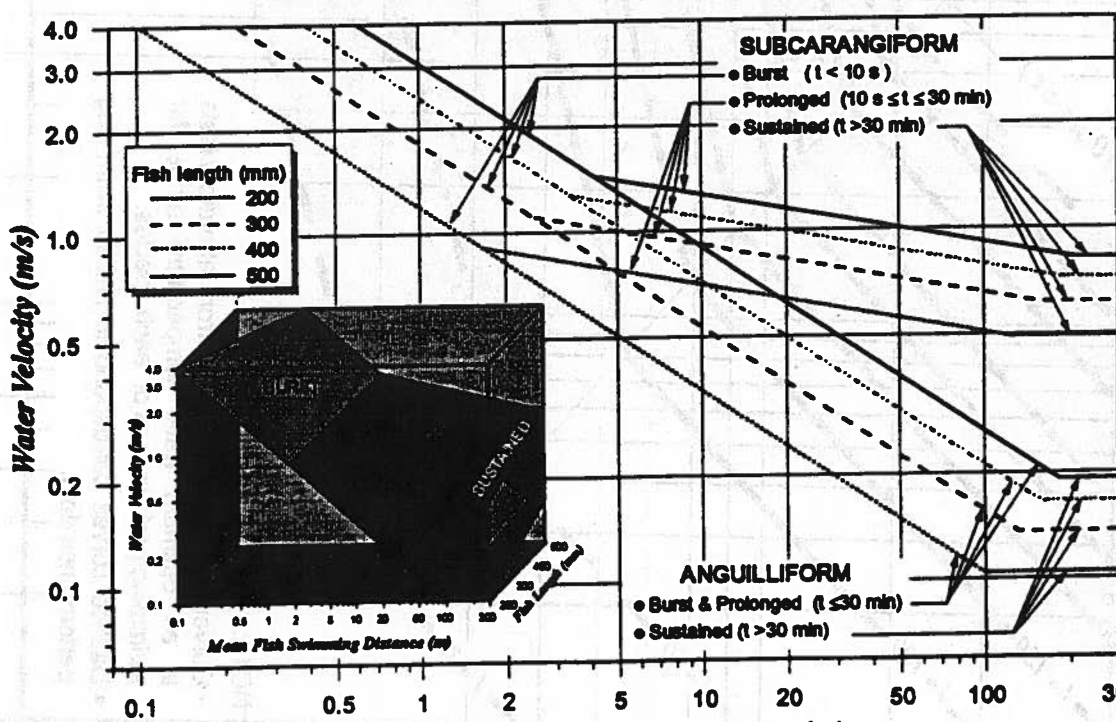
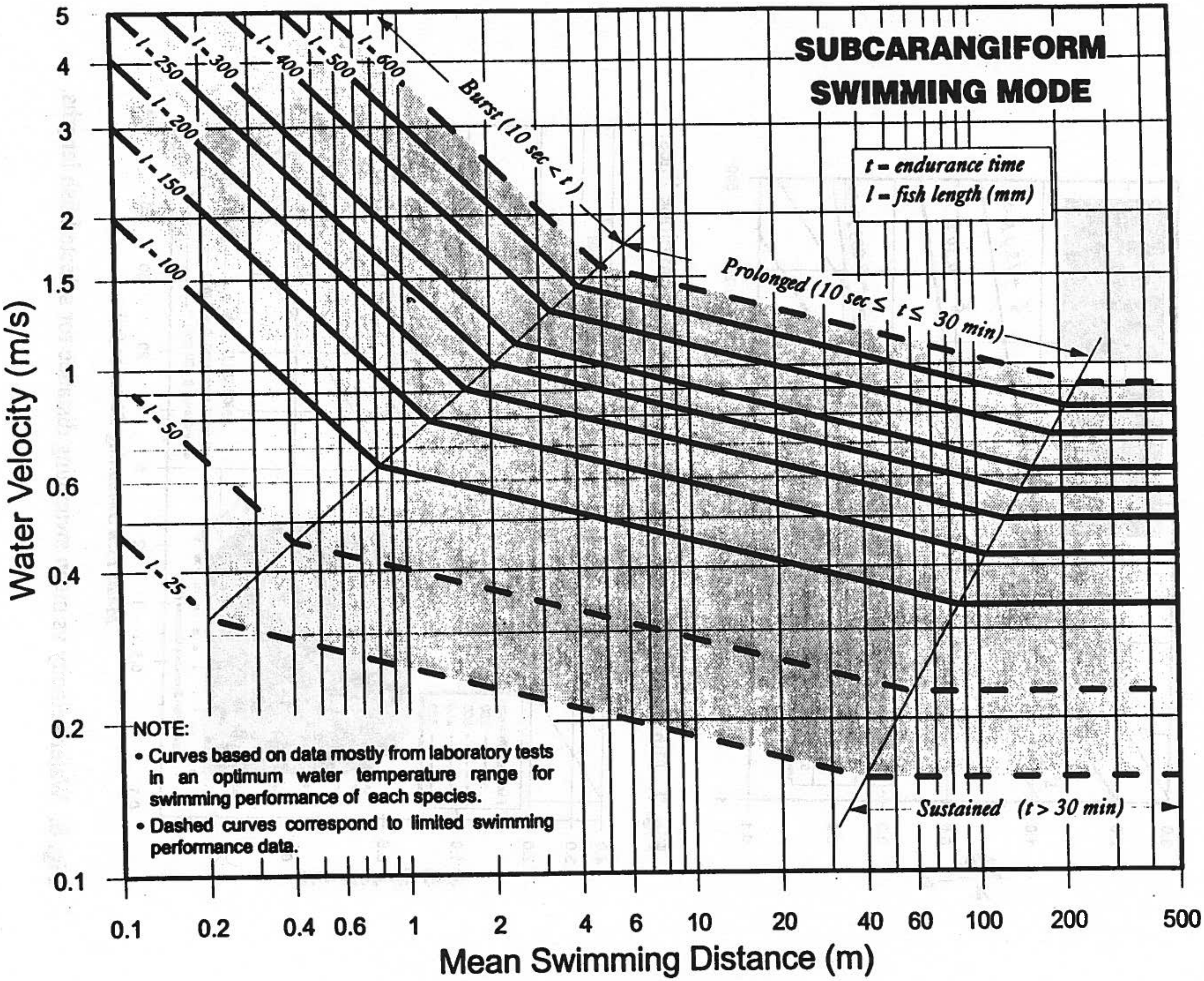
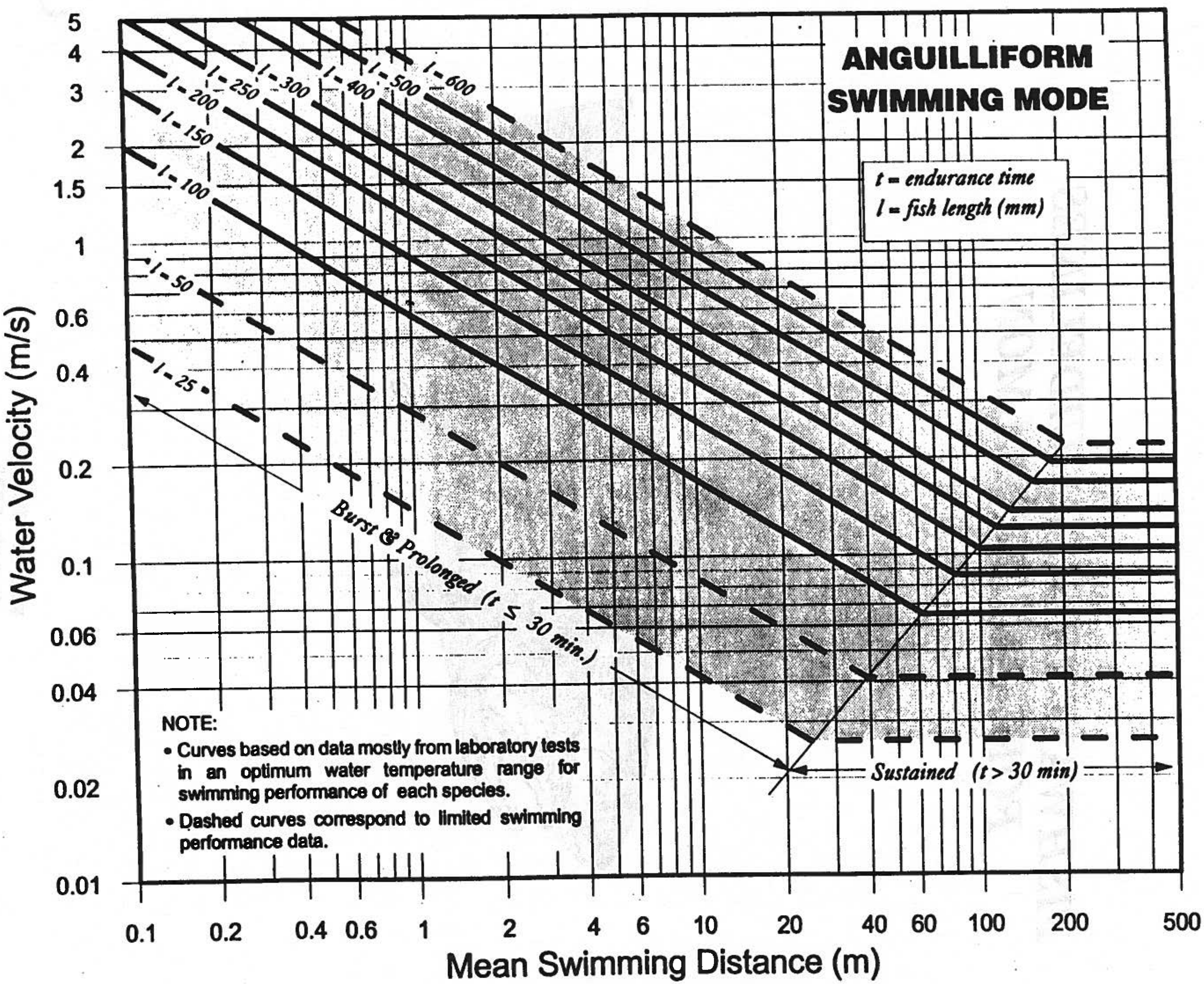
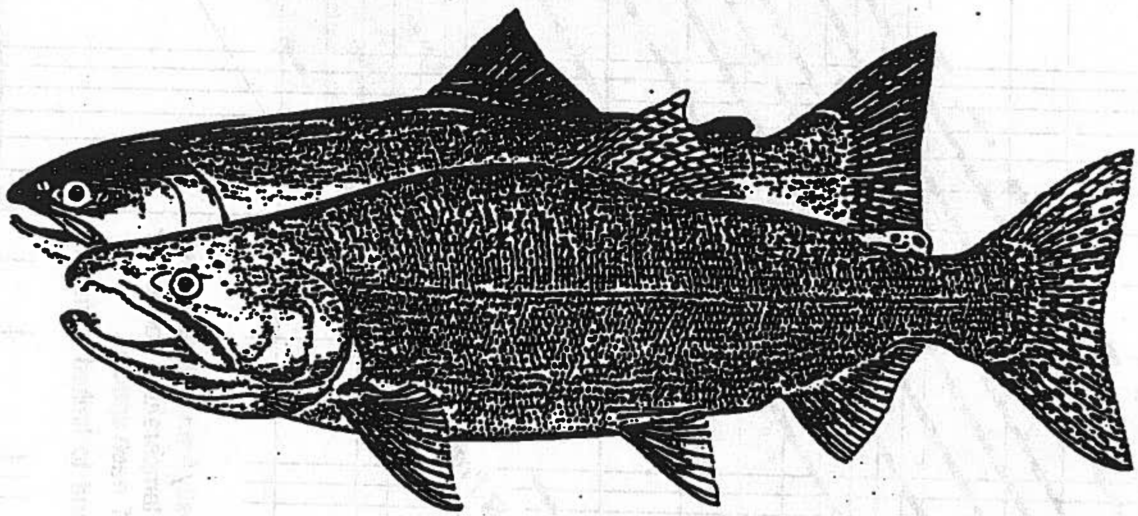


Fig. 8. Water velocity vs mean swimming distance for selected fish lengths.

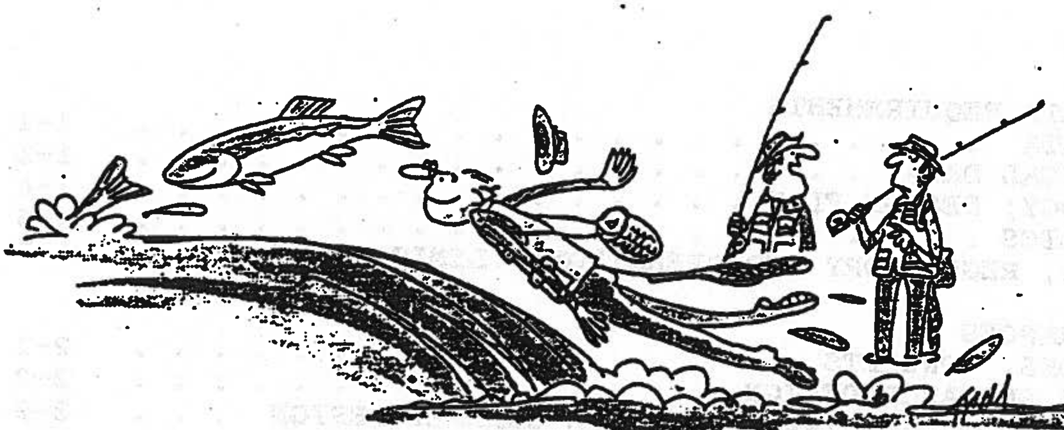

SUBCARANGIFORM
SWIMMING MODE
t = endurance time
l = fish length (mm)
Burst (10 sec < t)
Prolonged (10 sec ≤ t ≤ 30 min)
Sustained (t > 30 min)
l = 600
l = 500
l = 400
l = 300
l = 250
l = 200
l = 150
l = 100
l = 50
l = 25
NOTE:
• Curves based on data mostly from laboratory tests in an optimum water temperature range for swimming performance of each species.
• Dashed curves correspond to limited swimming performance data.
Water Velocity (m/s)
5
4
3
2
1.5
1
0.6
0.4
0.2
0.1
Mean Swimming Distance (m)
0.1
0.2
0.4
0.6
1
2
4
6
10
20
40
60
100
200
500


ANGUILLIFORM
SWIMMING MODE
t = endurance time
l = fish length (mm)
l = 600
l = 500
l = 400
l = 300
l = 250
l = 200
l = 150
l = 100
l = 50
l = 25
Burst & Prolonged (t ≤ 30 min.)
Sustained (t > 30 min)
NOTE:
• Curves based on data mostly from laboratory tests in an optimum water temperature range for swimming performance of each species.
• Dashed curves correspond to limited swimming performance data.
Water Velocity (m/s)
5
4
3
2
1.5
1
0.6
0.4
0.2
0.1
0.06
0.04
0.02
0.01
Mean Swimming Distance (m)
0.1
0.2
0.4
0.6
1
2
4
6
10
20
40
60
100
200
500

FISHWAY DESIGN GUIDELINES FOR PACIFIC SALMON



Ken M. Bates
March 1992



"That guy thinks like a fish."

FISHWAY DESIGN GUIDELINES FOR PACIFIC SALMON

Working Paper 1.5, 11/94

Ken Bates, Habitat Program Chief Engineer
Washington Department of Fish and Wildlife
600 Capitol Way North
Olympia, WA 98501-1091

(360) 902-2545
Fax (360) 902-2946
bateskmb@dfw.wa.gov
Compuserve 70531,3542

The fine print: Most of the information in this paper comes from my fish passage design work with Washington State Department of Fish and Wildlife. The suggestions and recommendations in this paper however are my own and do not necessarily reflect WDFW policies or standards.

FISHWAY DESIGN GUIDELINES
FOR PACIFIC SALMON

INTRODUCTION

This publication describes practical guidelines for the design of fish passage facilities for upstream migrating anadromous fish. Though the standards listed most specifically apply to Pacific salmon and steelhead, the considerations certainly apply to many anadromous and resident species. I hope to expand the scope of this paper to include resident species. The design standards I suggest here are a result of formal studies and practical experience. The practical experience includes failures; learning experiences that define the limits. This paper does not intend to be a treatise on hydraulics, hydrology or biology. Those are the foundation of fish passage technology but I assume the reader is already somewhat familiar with those disciplines.

The scale of systems to which these guidelines apply spans from mainstem passage in rivers such as the Columbia to small culverts under county roads. My definition of fishway is any structure or modification to a natural or artificial structure for the purpose of fish passage. The fishway is a system that may include attraction features, barrier dam, entrances, auxiliary water system, collection and transportation channels, the fish ladder itself, an exit and operating and maintenance standards. It can be a formal concrete structure, pools blasted in the rock of a waterfall or log controls in the bed of a channel. The water and fish of course are necessary parts of the system. A brief glossary of fishway terms used here is included at the end of this paper.

I hope that this paper is a useful tool. This is a working paper; it won't be finished until it is done. It will be updated periodically. I appreciate comments and criticisms. This paper in no way replaces the professional hydraulic engineering and biological expertise that are necessary for successful design of fish passage facilities.

PREDESIGN DATA REQUIREMENTS

A variety of physical, hydrologic, and biologic considerations will determine whether a given obstruction is passable. Data gathered prior to the design forms the basis for the technical analysis and design. Time taken to collect and understand these data and develop a consensus on relevant design criteria before investing in technical design is a valuable investment.

SITE DATA

Site data are the physical description of the barrier and river channel and uplands associated with the barrier and design and operation of the fish passage resolution. Getting a good site description is most essential for small projects where extra field trips can become a substantial burden to the project.

The best way to describe topography is by a stadia survey or a series of cross sections if dealing with a length of channel. The bathymetry of plunge pools should be included in the survey. Channel and bar configurations should be analyzed to help understand high flow hydraulics. Ordinary and high water marks should be recorded. Sufficient channel cross sections should be surveyed to develop a hydraulic model and a tailwater rating curve. Aerial and ground photos are very valuable for larger sites.

<u>Geology, soils</u> Soil conditions relate primarily to construction and structural design considerations. They may dictate the basic fishway concept selected.

<u>Access</u> Access includes considerations of equipment access for construction. Be aware of utilities that must be relocated, road detours required, bank slopes and soil conditions. Operation and maintenance considerations should begin with predesign; safe access during bad weather conditions is essential for good operation.

<u>Flood Protection</u> Record flood information including forebay and tailwater high water marks, bed load information and debris quantity and character.

<u>Check lists</u> Check lists remind the designer of the many details of information needed at a site. These details are especially important on small projects.

BIOLOGICAL DATA

Biological data are the fish passage design criteria. What species are targeted for passage? When are they present? What are their swimming abilities? What behaviors can be used to enhance their passage success?

Fish passage design is normally based on the weakest species requiring passage and should accommodate the weakest individual within that species. Management objectives may however direct blockage of certain species or age classes. The following biological variables should be considered in designing passage.

Species Species of fish is of course the most basic variable in passage design. The swimming and leaping capabilities of species can determine design criteria; design criteria among species of salmon and steelhead vary little.

Species should also be considered that are not the primary intended target but are present and may or may not require passage. Fishways on the Columbia River were not originally intended for shad passage because they were not a commercially valuable species. Passage research focused on salmon and steelhead. American Shad populations in the Columbia River have expanded from about two hundred thousand in the early 60's to as many as four million passing The Dalles Dam in 1990. Fishways were initially a blockage or at least a hinderance to shad passage; they accumulated within the Bonneville fishway at times to the extent that they blocked the passage of salmon.

Passage design for Pacific salmon and steelhead is a relatively simple task compared to Asian and South Pacific fish passage designs that must accommodate crabs, eels and small weak fish as target species.

Design of fish passage for resident species in North America is also difficult. Migrations for adfluvial spawning, feeding, redistribution due to density and water quality are common among resident fish. They often migrate at younger life stages; their migration timing and motivation is usually unknown. Resident species tend to be swimmers rather than leapers. To their benefit, they may move at lower stream flows and delay may not be as significant as it is for anadromous fish.

Species such as chum salmon that may have a stream residency of only a few days may be more greatly impacted by a minor delay than other species. The impact associated with chum delay is unsuccessful spawning. For coho a delay can result in a poor distribution of spawners through a watershed. Delay of any salmon species can result in a loss of production.

It should be acknowledged that all obstructions, whether mitigated with fishways or not, cause migration delay. A well designed culvert will cause minor delay; the change in hydraulics and light conditions are enough to cause a fish to hesitate. The larger the river, the greater the likely delay. It is not uncommon to experience delays up to a day at fishways in large rivers. The fish passage design criteria and the design hydrology described below are conservative in an attempt to mitigate the inevitable delay.

Behavior of fish is critical to fish passage design and is often a function of species. Shore and depth orientation during migration, where they hold, how they respond to hydraulic, light and enclosure conditions may be important factors. Though chum salmon are powerful swimmers, they refuse to leap. A minor plunging drop of less than a foot can be a barrier whereas a steep chute four feet high is easily swum through. Pink salmon behave similar to chum.

The condition of the fish may influence the design criteria. Swimming capabilities of anadromous fish generally decline as fish migrate upstream.

Timing Understanding the seasonal as well as diurnal behavior of the fish is important in setting the period of operation and the range of flows through which the fishway will operate. An understanding of the timing will also help define the impact of delay of a species.

Most adult salmon migrate during daylight hours. Figure 1-1 shows the timing of steelhead passage at Lower Granite Dam in 1992. The timing shown is typical for salmon though it can vary considerably.

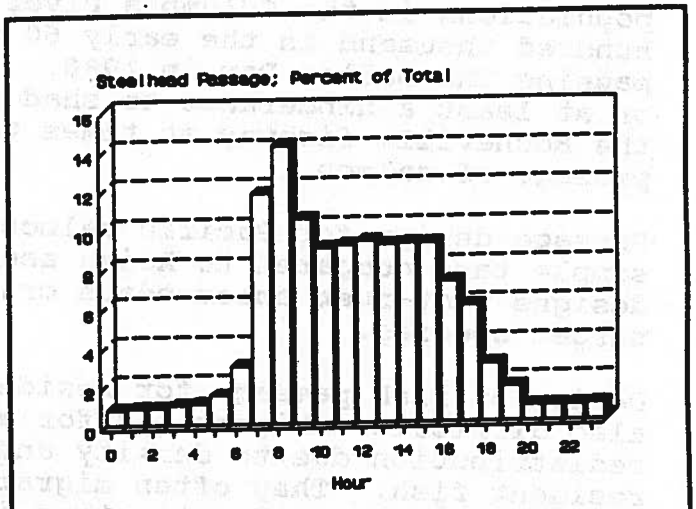


Figure 1-1. Steelhead Passage by Hour, Lower Granite Dam; 1992.

Sockeye passage at Zosel Dam on the Okanogan River for the same year was concentrated at night; 94.9% of the fish moved through the ladder between the hours of 8:00 PM to 4:00 AM. The passage timing of these fish may have been influenced by water temperature. They had been blocked for possible more than a month downstream by high water temperature in the Okanogan River.

Age Fish passage programs tend to concentrate on upstream passage improvements for adult fish. The productivity of some stocks however depend greatly on the ability of juvenile fish to redistribute both upstream and downstream into favorable rearing habitats. There is information documenting the value of over-wintering habitat and on the swimming speeds and stamina of juvenile fish.

Though much effort has gone into protecting and enhancing rearing habitats, little data are available regarding the requirements for upstream passage of juvenile fish. Juvenile anadromous fish that remain in fresh water substantial periods of time before migrating downsteam are particularly vulnerable to blockages in small streams.

Fish passage criteria typically ignore the need for upstream juvenile passage. Though not specifically protected, regulations are interpreted to include juvenile fish passage as well as adults. Washington State Department of Fish and Wildlife (WDFW) culvert design criteria assumes that adequate juvenile passage is provided if the hydraulic characteristics for adult passage are achieved during peak flows. It is assumed that the adult passage hydraulic conditions will result in a roughened channel through the culvert and that juvenile fish can tolerate some delay and will be subjected to less severe hydraulic conditions than adult migrants. Alaska Department of Fish and Game (ADF&G) recognizes juvenile migrants by grouping them with other weak fish and providing maximum allowable average culvert velocities for them.

<u>Size</u> The minimum and maximum sizes of fish of each species expecting to pass may determine maximum velocities and drops and minimum depths within a fishway. Swimming capabilities are a function of size of fish; they are a consideration in design of culverts and modifications of falls. Fish passage is designed for the smallest fish of the species requiring passage.

<u>Run size</u> The ultimate size of the peak of the run may control the size of fishway or collection pools. Hell's Gate fishways on the Fraser River were are the earliest example that I am aware of in which the size of the fishway was regulated by the size of the run expected. This is rarely the case; hydraulic considerations within the fishway pools normally control the design.

For examples of sizing fishways for the size of run, see Section 5.

HYDROLOGY; FISH PASSAGE DESIGN FLOW

There are few situations in which fish passage can be maintained during all flood flows. It is expected that adult migrants do not move during highest river flows. Fishway observations have verified this in locations where fish were blocked or chose not to move during high flows in high gradient channels. Keep in mind however that adult migrations of many species are induced by freshets; fish passage during moderate flood events is critical.

A high passage design flow, Q_{hp}, must be selected to begin design. Q_{hp} is defined as the highest stream flow at which specified fish passage criteria are satisfied. Fish passage will likely still occur at higher flows but hydraulic conditions diverge from the design criteria. Biological limitations of delay of fish can affect selection of the high passage design flow; different stocks of fish may require more or less strict criteria. Also consider that the barrier itself may become passable at some high flow. The analysis of barriers is not addressed here.

A variety of design flow criteria have been suggested or used. Gebhards and Fisher (1972) suggested an allowable migration delay of 6 consecutive days for salmon and trout. Dryden and Stein

(1975) recommend that a 7 day impassable period should not be exceeded more than once in the design period of 50 years, and that a 3 day impassable period should not be exceeded during the average annual flood. The States of California and Washington suggest that passage should be provided 90% of the migration period of the target species (Kay and Lewis, 1970), (Bates, 1988). The Alaska design flow is a mean annual flood event with a two day duration. A design discharge equivalent to 30% of the average annual flood has been suggested as a general guide in British Columbia (Dane, 1978).

WDFW has presented design flow criteria for fish passage at instream stormwater detention basins; "fish passage criteria shall not be violated more than 100 hours during the migration season and for no longer than 24 hours at any one time" (Bates, 1981). For culverts, Alaska Department of Fish and Game (ADFG) uses the mean annual two-day flood as the high passage design flow.

The selection of hydrologic design criteria should consider the type of runoff expected during fish passage. Storm and rainfall runoff is event oriented and leads to frequency analysis. Snow melt runoff peaks are typically not as high but last longer and therefore lead to a event duration analysis.

Hydrologic criteria may also depend on whether the objective of fish passage is mitigation or enhancement; the design criteria listed here are intended as mitigation criteria. Enhancement opportunities should not be lost only because they impractical by the application of mitigation criteria.

These criteria require a hydrologic analysis of gauging records, correlation to other streams or other hydrologic models.

To build a simple Q_{hp} model, WDFW performed a regression analysis on 32 streams with drainage basins less than 20 square miles and minimum gauging records of 10 years. All of the streams are in Western Washington so the model is limited to use in that area. This model is presented here only as an example; it should not be used in other regions without first being verified. The model is given in Equation (1-1).

$$Q_{hp} = CA^{1.11}P^{1.40} \tag{1-1}$$

A is the basin in square miles and P is the mean annual precipitation at the gauging station. C is a coefficient ranging from 0.019 to 0.030 depending on rainfall-runoff characteristics of the basin and define the 95% confidence interval of the estimate. Values of C at the lower end of the range are applied to basins with good natural flood storage and attenuation and low to moderate slopes.

Structural design will depend on an analysis of higher flows than the high passage design flow.

The other end of the fish passage flow range is the low design flow, Q_{lp}. There are no specific agency requirements in the Pacific Northwest for the low passage design flow that I am aware of. The low fishway design flow should be considered in the negotiation of instream flows. Fish passage considerations that should be given to instream flows include adequate flow for fish attraction and adequate flow for operation of screen bypasses.

Third rule for naive engineers: Constants defined in technical manuals are treated as variables.

A necessary tool for any river engineering work is a set of flow exceedance curves by month for, at least in this case, the extent of the migration season.

HYDRAULICS

Hydraulics is the science of the static and dynamic behavior of fluids and can be observed, analyzed or modeled. Hydraulic principles are applied to the river channel and passage barrier to appropriately locate the fishway entrances and exit and to determine the required scale of the facility and entrance flows. Hydraulics at the fishway entrance and within the fishway are the basis of successful fish passage.

Flow patterns within the tailwater should be understood at various depths and stream flows. Both tailwater and forebay rating curves are required for fishway design. They should be based on observations through a range of low to high flows. They should be extrapolated by an acceptable hydraulic technique to include higher and lower flows as necessary.

The location of the passage barrier often is a function of stream flow. The barrier often moves a substantial distance between low and high flows. Velocities, turbulence, upwells, reverse currents and aeration can all affect attraction and access to fishways.

Model studies can be a valuable tool to help the designer understand the fishway setting. Models are especially valuable when trying to locate fishway entrances at a proposed dam where the flow patterns in an energy dissipation structure are not well understood or at an existing barrier where hydraulic conditions cannot be observed. A simple two-dimensional model is often adequate to establish fishway entrance locations in conjunction with hydraulic jumps or heavy turbulence below an energy dissipator.

A good example of use of a model to determine fishway and entrance locations is the Hell's Gate fishway in the Fraser River. The Hell's Gate barrier is a high flow velocity through a narrow gorge; surface velocities are about 23 feet per second (fps). The location of the high velocity barrier moves up and down the channel several hundred feet with changing river flow. The water surface varies vertically 90 feet during the salmon migration season. A 1:50 scale model of the canyon was built to study the barrier at different flows and to locate the fishway entrances and exits. The data for this type of analysis would be nearly impossible to measure in the field. Six fishways have been built at Hell's Gate; they are located both vertically at different elevations on both river banks and horizontally along the channel to provide fish passage at a wide range of flows.

Be careful in interpreting turbulence observed in a model. Turbulence is a function of viscosity. Scale models are usually governed by laws of gravity and viscous forces can not be directly scaled. Turbulence could be quantified by measuring rapid pressure fluctuations in the gravity model and scaling them to full scale. A correlation of pressure fluctuations at full scale to known passage conditions would be required.

FUNDING, REGULATORY AND OPERATIONAL LIMITATIONS

There are operational limitations that can affect the design and success of the fishway. Such limitations include dam and hydroelectric operating schedules, minimum regulated in-stream flows, maintenance schedule of the dam or related facilities and O&M limitations including personnel, access and funding. O&M funding always seems in jeopardy; consider the implications of potential reduced O&M funding on the operation of the fishway.

Proper fishway operation and maintenance is best achieved when there is a clear understanding of the intended operation and an appreciation for the importance of fish passage every hour of the day. The more complicated the operation, the more likely the fishway will not be operated as intended. Often the most critical fish passage timing coincides with the worst conditions of rain, rapidly changing stream flow, wind and debris. Crews responsible for operation of fish facilities are also often responsible for other infrastructures that are stressed and require attention at the same time. Pacific salmon often start migrations in response to freshets. Fall freshets usually come with wind that carries debris, especially fall leaves, into the stream.

The preliminary design process is a good time to start developing an operating manual. Continue its development throughout the design; make sure it is realistic and the operator of the facility agrees to it and is committed to it as it is developed.

FISHWAY ENTRANCE

The fishway entrance is often the most difficult design element and the most critical to successful fish passage.

The key to successful passage is to bring the fish to the fishway; to bring them from the uncontrolled natural river environment to the controlled fishway system. As Jack Orsborn so aptly puts it:

NFI = NFO

No fish in; No fish out. If you can't attract fish into a fishway, they won't get upstream. Once fish are in the entrance pool moving them through the fishway is relatively simple.

FUNCTIONS, CONCEPTS

Fishway entrances and entrance pools have a variety of purposes; more complicated hydraulic and hydrologic settings require more complicated entrance pool designs. The application of these functions and specific guidelines will be discussed in the entrance and entrance pool design sections that follow.

<u>Access</u> The most obvious and necessary purpose of fishway entrance pools is for fish access to the fishway. Other more subtle purposes though influence its design.

<u>Attraction</u> The entrance is the key to attraction of fish. Think of the jet of water leaving the fishway entrance as an extension of the fishway into the tailwater. That extension is a path that guides fish to the fishway. The further the entrance jet penetrates the tailwater, the further the path is carried. Penetration is a function of the three factors: jet momentum, shape and alignment.

<u>Introduce Auxiliary Water</u> Auxiliary water is often supplied within the entrance pool or transportation channel to strengthen the entrance attraction jet or to increase velocities within a channel. The flow is introduced through diffuser systems designed to not delay fish but to guide them to the fishway itself.

<u>Hydraulic Control</u> The entrance controls the hydraulic characteristics of the entrance flow. Details of the entrance control the shape, orientation, flow characteristics and stability of the entrance jet.

The geometry of the entrance and its elevation relative to the tailwater determine whether the flow that exits it either plunges or streams.

<u>Transition</u> The entrance is a transition from river to fish ladder for a range of tailwaters and hydraulic conditions. It is a transition from the natural river environment to the sterile

artificial environment of the fishway.

<u>Combine Multiple Entrances</u>   Entrance pools and collection channels may collect fish entering through several entrances into a single ladder. The Columbia River fishways have entrance and collection channels that were intended to collect fish from up to 20 entrances along a powerhouse in addition to as many as three main entrances at each bank.

FISHWAY ENTRANCE DESIGN

The design of the entrance should consider as appropriate the functions and concepts described above.

<u>Location</u>   The location of the fishway entrance should logically be at the upstream-most point of fish passage. Also take into account the locations where fish hold before attempting to pass the barrier and routes by which they will approach the barrier and fishway.

Fishways are normally constructed on banklines where construction, operation and maintenance access are simple. Conditions that lead to placing fishway entrances at each bank include:

- wide channel; a single fishway cannot attract fish from the opposite bank of a wide channel;
- holding areas far from each other or from the barrier;
- migration routes along each bankline;
- hydraulic conditions that prevent or distract fish passage from any part of the channel to the entrance.

Multiple entrances may be associated with separate fishways or connected with a transportation channel. A single entrance with an entrance flow of 100 cfs may be adequate in a 150 foot wide river channel that is uniform in cross section and flow distribution.

Fish will normally migrate along the channel banks during high flows to take advantage of the lower velocities in the bankline boundary layer. Some salmon are also shoreline oriented; they follow the shoreline for guidance. Crossing the entire channel through the turbulent tailrace of a powerhouse or other barrier during high flow may be impossible. The penetration of the entrance flow is also diminished at high flows due to high tailwater velocities and turbulence. Fish tend to not swim back downstream in search of a passage route; if there is a hydraulic barrier between them and the fishway entrance, they are not likely to find it without being delayed.

All of these hydraulic considerations will likely change through the range of passage design flows. Hydraulic models are especially helpful in fixing the fishway entrance location. Field observations and sketches of flow patterns below and above the barrier should be made especially for high flows. Observations of fish location and orientation when attempting to pass a barrier are valuable.

Multiple entrances that operate each within a specific range of flows may be necessary where changes in the tailrace hydraulic conditions are great. Low and high flow entrances are often provided.

Low flow entrances are located close to the base of the dam. They are usually operated only at low flow. When a roller bucket energy dissipator is located far downstream from the dam crest, fish can become trapped in the pool between the dam and the back roll. In that case, the low flow entrance should be located in that pool and be operated at all times. Low flow entrances should also be located beneath the nappe of the spillway when it separates a substantial distance from the dam.

Finding the best location for high flow entrances is more complex. Redundant entrances can be provided if the proper fishway entrance locations are not well identified. As many as four entrances have been provided on fishways in the Yakima River basin. It is cheaper to provide an additional unused port in a fishway wall during initial construction than it is to later extend the fishway for the sake of an additional entrance.

The hydraulic conditions of a dam tailrace depend on the flow spilled over the dam and the style of energy dissipation built into the dam. A high flow entrance must be located downstream of the hydraulic jump when that is the style of energy dissipation employed. It will be further downstream than if a roller bucket dissipator is used. Roller buckets contain the dissipation within a narrow segment just below the roller bucket.

Be aware of eddies and local flow conditions especially at high flow. "Upstream" to a migrating fish means nose into the approaching flow. Fish that must approach fishway entrances located in an eddie, may have to swim downstream or cross-current relative to the local direction of flow. A fishway that is built on a bankline can create eddies that make a high flow entrance difficult to find.

Distractions such as spilling water or jets of water can be as effective in leading fish away from entrances as the entrances can be in attracting them.

<u>Entrance Flow</u> There are no specific fishway entrance flow criteria. The entrance flow must be adequate to compete with spillway or powerhouse discharge flow for fish attraction. Site conditions and especially tailwater hydraulics and channel width help determine entrance flow requirements.

The greater the momentum of the jet, the further it reaches into the tailwater and the more successfully it can guide fish to the entrance. Momentum is defined as mass times velocity.

$$Momentum = Mass \times Velocity \qquad (2\text{-}2)$$

The units are the mass per unit time exiting the fishway and the velocity of the jet. It makes sense that the more flow that can be put through the entrance, the further the jet penetrates the tailwater.

The scale of the river setting gives some insight into entrance flows requirements. Table 2-1 shows the total entrance flow for a range of fishway scales. The fishway flow in the table is the total flow from all entrances for all fishways at each site. For example, Sunnyside Dam has three fishways and a total of four entrances with 104 cfs maximum flow at each. Anderson Creek on the other hand is a single fishway with one entrance. The design flows are the 10% mean daily exceedance flows for the migration season regardless of the flow for which specific fishways may have been designed.

Table 2-1. Entrance Flows at Various Fishways.

Fishway; Location	Entrance Flow; cfs (Q_E)	Design Flow; cfs (Q_{hp})	Q_E/Q_{hp} %
Sunnyside Dam Yakima River	416	7,400	5.6
Sunset Falls Skykomish River	234	6,000	3.9
Naches Cowiche Naches River	92	2,500	3.7
Centralia Dam Nisqually River	80	1,750	4.6
Easton Dam Yakima River	120	1,300	9.2
Anderson Creek; Nooksack trib.	9.7	112	8.7

These fishways operate effectively; they are in locations without unusual tailwater conditions.

This information should not govern a design; it is provided only to show the wide range of entrance flows selected and how they relate to river scale.

<u>Alignment</u> Low flow entrances should be aligned perpendicular to the channel alignment or parallel to the barrier to maximize their reach into the channel.

High flow entrances may be placed at a 30° angle to the high flow streamline. Ideally they would be oriented along the edge of the high flow hydraulic barrier. A benefit of the angled entrance is that the entrance jet penetrates the tailwater to a greater extent than if aligned perpendicular to a turbulent, high velocity tailrace condition. An entrance oriented at too great an angle to the high flow streamline may produce an eddy causing water to flow upstream along the fishway wall just downstream of the entrance. The protrusion into the stream of the angled entrance provides an abutment and a velocity shadow behind which fish can move upstream. Passage is then blocked by the abutment and the high velocities in the stream beyond it. Those fish are right at the alternative passage route however, the fishway entrance.

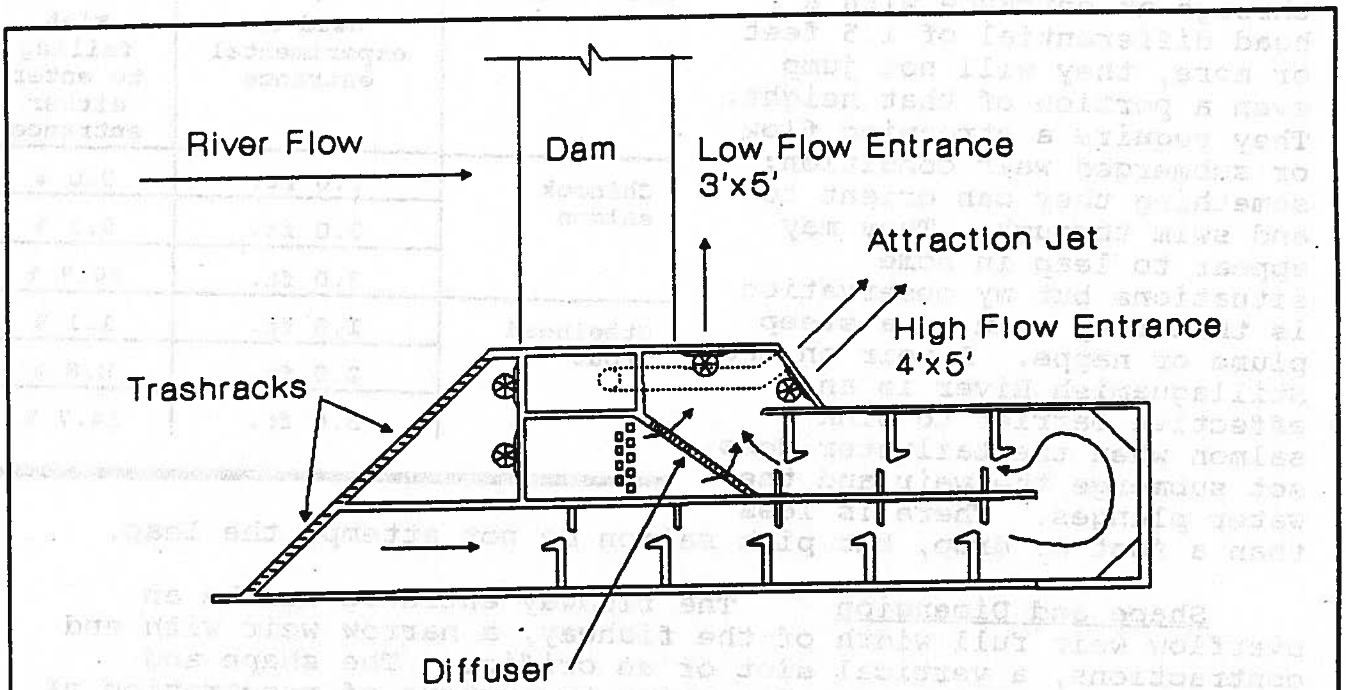


Figure 2-1. Wapato E. Branch Fishway; Yakima River

The Wapato fishway on the Yakima River has good examples of low and high flow entrances. A sketch of the fishway is shown in Figure 2-1 and a detail of the entrance pool in Section 4.

<u>Entrance Head</u> The water surface differential between the entrance pool and the tailwater is a criterion established by fish passage requirements and desired entrance flow characteristics.

For Pacific salmon and steelhead, an entrance head of about 1.2 feet is preferred for streaming flow conditions. A range of 1.0 to 1.5 feet is a normal operating range.

Gauley et al (1967) tested the preference of chinook, sockeye and steelhead for submerged fishway entrances with head differentials of 1.0, 2.0 and 3.0 feet. Theoretical velocities through the orifices are 8.0, 11.3, and 13.9 fps at these heads. They evaluated only preferences between pairs of entrances, not attraction to them. A majority of fish of each of the three species chose the 2.0 and 3.0 foot entrance heads when compared to the 1.0 foot entrance head. An increasing number of fish failed to enter any entrance, however, when the head was increased to 2.0 and 3.0 feet. This information is summarized in Table 2-2.

The fact that these fish chose to remain in the tailwater pool rather than pass through the experimental or control entrances suggests that they were attracted to the greater flow from the experimental entrance, but would not pass through it.

Chum and pink salmon have more specific requirements. Though they have no trouble swimming through an entrance with a head differential of 1.5 feet or more, they will not jump even a portion of that height. They require a streaming flow or submerged weir condition; something they can orient to and swim through. They may appear to leap in some situations but my observation is that they swim up a steep plume or nappe. A weir on the Stillaguamish River is an effective barrier to pink salmon when the tailwater does not submerge the weir and the water plunges. There is less than a foot of drop, but pink salmon do not attempt the leap.

Table 2-2. Fish failing to enter experimental entrance or control entrance with 1.0 ft head.

	Head on experimental entrance	Fish failing to enter either entrance
Chinook salmon	1.0 ft.	0.0 %
	2.0 ft.	9.1 %
	3.0 ft.	29.7 %
Steelhead trout	1.0 ft.	1.1 %
	2.0 ft.	8.8 %
	3.0 ft.	14.7 %

Shape and Dimension The fishway entrance can be an overflow weir full width of the fishway, a narrow weir with end contractions, a vertical slot or an orifice. The shape and elevation of the entrance determine the extent of penetration of the entrance jet into the tailwater. The momentum of the jet is dissipated in the tailwater by the shear forces at its boundaries; the less the surface of its boundary, the less rapidly it will be dissipated. For this reason, an ideal shape would be circular and the least desirable would be a narrow vertical slot. A more practical shape is a square or rectangular port with a width to height ratio from 0.6 to 1.25.

A jet will stream from the entrance only if it does not plunge. A plunging flow will drop nearly vertical and set up a hydraulic roll and surface counterflow downstream of the entrance. A streaming flow, on the other hand, will remain intact near the water surface or at the elevation of an orifice entrance. A weir or port sill that is broad crested, smooth and with an efficient upstream floor contraction is more likely to stream. Efficient side contractions will also enhance the jet.

Entrances vary in size from 12 feet wide by 8 feet deep on the Columbia River dams, 3 to 4 feet wide by 4 to 5 feet deep on the Yakima and Wenatchee Rivers to 18 inches wide by a foot deep on small streams. The smallest recommended entrance port dimension is 30 inches.

Slot entrances are useful where a strong shear velocity approximately parallel to the entrance jet alignment is present or is created by the protrusion of the fishway into the stream. The shear velocity must be strong enough to be a substantial passage barrier. The vertical slot is located adjacent to the high velocity and functions similar to the angled high flow fishway entrance described on Page 2-5. It creates a velocity shadow so fish can approach the entrance area and the high shear velocity beyond the entrance prevent passage past the entrance. The Hell's Gate fishways and fishways on Cedar Creek, a tributary of Lewis River, and the Klickitat River are designed with shear slot entrances.

Fish behavior may also affect entrance geometries. Thompson et al (1967) tested 371 chinook, coho and steelhead to find their preference for size and shape of fishway entrance. Flows were varied to maintain a constant entrance velocity of 8.0 fps. All species preferred a 3.9 foot square entrance by 9 to 1 over a 1.5 foot wide by 4 foot high entrance. They also tested preferences for 2 foot by 5 foot submerged rectangular orifices aligned vertically and horizontally. All three species preferred the vertical orientation to the horizontal.

At small remote fishways that are not regularly maintained, selection of an entrance width is often a trade-off between being narrow enough to produce an attractive streaming jet with the flow available and wide enough to stay clear of debris. To maintain the desired entrance head, an overflow weir is commonly used in lieu of a small orifice. A common minimum width for weir notches for debris concerns is 18 inches.

<u>Light</u> Daytime lighting requirements at fishway entrances is clear. Concurrent with the entrance dimension tests discussed above, Gauley et al (1967) tested the preference for lighted or dark entrances with 1.0 feet of head. Even when given the choice of the large (3.9 ft. by 3.9 ft.) entrance which was dark or the smallest (1.5 ft. by 2.0 ft.) entrance which was lighted, 80%, 90% and 69% of the chinook, coho and steelhead chose the lighted one.

In a similar test they found that lighted entrances were again chosen by 86% of the sockeye tested.

These tests used an array of 1000 watt mercury vapor lights suspended over the water surface which gave an average light intensity of 850 foot-candles at the water surface and 38 foot-candles at mid orifice depth. This is equivalent to a bright cloudy day.

Hiram's Law:
If you consult enough experts, you can confirm any opinion.

Other fishway studies on Columbia River dams have evaluated entrance lighting conditions with variable results. Improved passage has occurred at sites when a 150 watt submerged thallium iodide light was lit at fishway entrances as compared to either an unlit condition or use of a 500 watt quartz iodide light. The unit tested was a 150W Edo Western model 1207 and was described as "a mercury vapor lamp with thallium iodie added to the high pressure mercury discharge." This produces a blue-green spectral component that penetrated very well through water. The quartz iodide light improved fish passage but to a much smaller degree. The unlit conditions had normal ambient light within the fishway. Sensitivity of fish to various light frequencies should be accounted for. It makes sense that they would be most sensitive to green or shorter wave lengths that are not absorbed in water as are higher frequencies.

Field surveys in the Fraser River Canyon show that upstream migration slows during hours of darkness and large accumulations of salmon in resting pools downstream of difficult passage areas such as fishways and rapids has been common (Saxvik, 1990). Floodlights were installed at some of the Fraser River fishways beginning in 1989. The lighting allowed fish to continue through the fishways at night and eliminated the downstream accumulation and congestion of fish overnight. The fishway capacity was increased by the lighting but not directly by extending the daily period of passage. The capacity was increased by secondarily eliminating the congestion of fish that had reduced passage capacity.

Flexibility should be built into the control system of lights so they can provide a range of intensity through a gradual tansition section from light ambient conditions to dark fishway conditions or to mimic ambient conditions as turbidity varies.

Experimental lights within the Hell's Gate and other Canadian fishways have resulted in continued passage during the night and thereby reduced crowding and delay during the following day.

The use of lights in conduits has also been tested and is reviewed on Page 2-10 in the section on transportation channels.

<u>Elevation</u> The entrance head differential must be maintained as the stream flow increases. The rising tailwater backwaters and drowns out the entrance unless the entrance adjusts to compensate. This can be done by mechanically raising the entrance weir or by increasing the entrance flow. Both of these options are discussed later; entrance gates on Page 2-9 and auxiliary water systems in Section 4.

Entrances that behave as submerged weirs can become orifices at higher flows as they are submerged by the higher tailwater. No adjustment is needed for a port entrance; the velocity is constant as long as the entrance flow and the port area remain constant. The entrance elevation can also be automated by using a floating bulkhead entrance. An orifice within the floating bulkhead remains at the same elevation relative to the tailwater as the bulkhead follows the tailwater elevation.

Vertical slot entrances, on the other hand, are backwatered by increasing tailwaters. The effective area of the slot increases and thus the entrance head and velocity are diminished unless auxiliary water is added. An exception to this is wing gate entrances described in the section on wing gates.

If an objective is to maximize attraction at low flow, the entrance should be submerged to optimize the streaming jet flow. The flow predominantly streams when it is backwatered by the tailwater to at least 30% of its depth. With less submergence, it tends to plunge. Often the entrance can plunge at low flow when less attraction is required but should stream as a jet at higher tailwaters.

It is prudent to provide a safety factor in the entrance sill elevation by including the capability of setting the entrance lower than expected. Potential channel degrading or scour should be taken into account especially if a new dam is being constructed that will either trap sediment or scour the bed by energy dissipation. Stream bed controls are especially vulnerable; they cause the downstream channel to scour. A bed control built at bed level downstream of a sediment trap in Swan Creek, a Puyallup River tributary, was left suspended three feet above the bed after a single flood. The stream bed degraded from under the controls due to efficient sediment capture in the trap and clearwater scour.

<u>Entrance Gate</u> Entrance gates are used to control the hydraulics described above; elevation, width, and head differential.

An upward closing gate with a lifting yoke can be used as an adjustable entrance sill. Large entrance gates should be motorized or at least equipped with nut driver attachments if they are to be operated as expected. Gate stems and stem blocks commonly fail; extra sturdy components should be used. Entrance gates submerged in roller bucket energy dissipators should be extra heavy duty to withstand the vibrations and turbulence there. A rising stem gate is necessary to not have a gate stem through the middle of the entrance.

Wingates are like doors that swing horizontally and open downstream. They are normally built in tandem as double doors. They do not control the sill elevation but control the head differential through a wide range of tailwater elevations.

ENTRANCE POOL AND TRANSPORTATION CHANNEL DESIGN

The detail designs of the entrance pool and transition to fishway are critical to fish passage. Essentially all passage problems that I am aware of within fishways occur in entrance pools or collection channels. Fish pass several pools before they establish a continuous upstream movement pattern. Most dropbacks occur over the first weirs or the entrance of the fishway.

<u>Hydraulics</u> Shape the pool and locate auxiliary water diffusers and ladder entrance to provide a stable flow pattern and transportation velocity to lead fish from entrances to fish ladder. Excess space where eddies, flow separations or dead water may occur should be eliminated from entrance pools and channels. Corners should be rounded or filled; dead ends and areas should be eliminated. The Wapato fishway entrance shown in ? is a good example.

Wall deflections or channel expansions should be limited to about

1:8 to prevent flow separations.

If an entrance pool has multiple entrances to different tailwaters that are separated hydraulically from each other, a flow instability can be created. Surging can occur as one entrance draws the entrance pool down, that tailwater pool fills and backwaters the entrance pool the entrance flow decreases and then the tailwater pool drains. This sequence repeats alternating between entrances. The entrances can be isolated from each other hydraulically be inserting several fishway weirs between them; a junction pool and parallel fishway legs may be required.

<u>Velocity</u> In transportation channels, a uniform velocity from 1.0 to 4.0 fps should be maintained; 2.0 fps is a normal operating criteria. Laboratory studies have not found an optimum velocity within that range consistent for all species. It is prudent to design the system capacity for the higher velocities though they may not be used. Transportation velocities are also applied to the lower end of the fishway when it is flooded by a high backwater. When it is flooded, the increased cross section area results in a reduced velocity unless auxiliary water is provided. Auxiliary water is discussed in Section 4.

Salmon move at a relative speed, with reference to the ground, of about 2 to 4 fps whether swimming at a prolonged or burst speed.

<u>Light</u> Studies conducted at the Fisheries Engineering Research Laboratory found no statistically significant improvement in passage time for summer chinook, sockeye or steelhead through pipes when a gradual light transition zone was provided. It is generally believed that fish delay at changes in light conditions however. Considering the pronounced preference of salmon and steelhead for lighted entrances, efforts should be made to light transportation conduits and provide light transition areas at their entrances. See the section on entrances, Page 2-7 for specific light suggestions.

AUXILIARY WATER SYSTEM

The auxiliary water system is the source, control and supply of supplementary water to the lower end of the fishway. There are four general purposes of auxiliary water:

- provide additional flow at the fishway entrance for enhanced fish attraction;
- maintain desired flow and velocity in transportation channel;
- supply water for parallel fishway legs;
- supply water for fishway flow control.

DIFFUSER DESIGN

Auxiliary water is introduced to the fishway through wall or floor diffusers. Diffusers are bar grating, perforated plate or wood racks.

There are four objectives of a good diffuser design:

- introduce adequate auxiliary water with a uniform distribution;
- minimize maintenance demand; operator friendly;
- discourage attraction of fish;
- protect fish from injury.

The spaces between bars of a diffuser must be sized to prevent fish passage and injury. They should also be narrow enough that fish cannot injure their eyes as they nose into the spaces between the diffuser bars. Fish are commonly seen pushing into the gaps as if trying to force their way through. Openings should be small enough to protect the smallest fish present. Senn (1984) suggests opening dimensions of 1.5 inches for chinook, 1.0 inch for coho and steelhead and 0.75 inch for sockeye for traps. Picket traps on the Cedar River and auxiliary water diffusers on the Wenatchee River have clearances of 1.0 inch for sockeye without gilling problems.

Overall diffuser size should be enough to maintain a velocity such that fish are not attracted to it. Diffusers, like screens, are designed with a gross velocity criteria, the flow divided by the overall diffuser area. A diffuser velocity of 1.0 fps is generally applied for salmon. Studies have shown passage delays when auxiliary water is added through diffusers at velocities as low as 0.25 fps. Delays generally increase with higher diffuser velocities (Gauley et al, 1964)

To attract fish away from the diffuser and into the fish ladder, a steady attractive stream of flow should be directed from the fishway along the face of the diffuser and at an angle slightly away from it. This depends on a good entrance pool design; the Wapato fishway entrance pool shown in Figure 3-1 is an example of a good layout of entrance pool, auxiliary water diffuser and fishway.

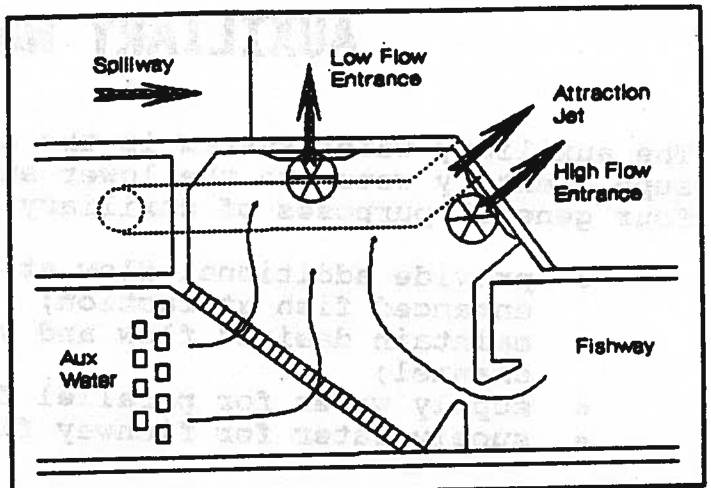


Figure 3-1. Wapato fishway entrance pool.

An energy dissipation chamber and/or baffle system is usually needed upstream of the diffuser panel to assure a tranquil and uniform flow distribution through the grating. A common flow distributor is two rows of vertical steel channels a foot apart and offset from each other and each with about 50% open area. The idea is to create a few tenths of head loss. Another good distribution system used in floor diffusers is the stepped baffle shown in Figure 3-2. This system should also work for a wall diffuser when turned vertically.

Wall Diffusers Wall diffusers have a great advantage over floor diffusers in their ease of maintenance; they can be cleaned from overhead with a rake.

Make sure the upstream side of the diffuser grating is clear of horizontal support bars so debris can be raked. An unobstructed access must be provided from overhead. If there is a grating over the auxiliary water chamber, light-weight hinged grating should be provided appropriately for access for cleaning. The entire upstream side of the rack must be accessible for raking; the grating should be mounted flush with upstream wall of the diffuser port therefore.

The entire diffuser grating should be submerged at the lowest water surface in the pool. Alternatively, insure that the diffuser flow is reduced when it is exposed. Water will spill through the exposed portion of the diffuser. Fish are often attracted to spilling water or its aeration and leap at it.

Standard commercial bar walkway grating is used for diffuser grating.

Floor Diffusers An advantage of floor diffusers is that fish are less attracted by upwelling water than flow from a wall diffuser or at least they don't jump at it. They are difficult to clean however. Several floor diffuser systems have been designed to be pivoted or hinged to allow flushing of debris. I am not aware of any of these currently in operation.

AUXILIARY WATER SUPPLY

Auxiliary water is supplied by gravity from forebay, pumped from the tailrace or combination of both. Gravity auxiliary water control gates can often be designed so they are essentially self controlling. This is done by matching the control gate flow characteristics to the auxiliary water flow requirements; a proportional weir or orifice gate is required.

<u>Chimneys</u> Chimneys are a passive device used to control flow progressively to a series of fishway pools as the tailwater rises and increasingly backwaters the fishway pools. A purpose of the auxiliary water in this situation is to provide additional flow and maintain a desired velocity in the backwatered pools for attractive transportation conditions. See the discussion in Section 3 regarding transportation channels.

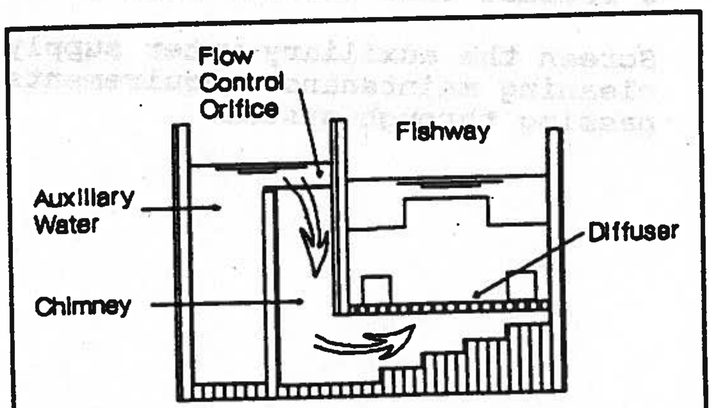


Figure 3-2. Chimney flow distributor and floor diffuser.

A chimney system is shown in Figure 3-2. A porosity panel or orifice creates a constant head loss between the diffuser pool and entrance pool. At low tailwater, the chimney supplies no flow to the diffuser. A constant head is maintained between the diffuser pool and entrance pool and between the entrance pool and the tailwater. The entrance pool and diffuser pool water surfaces therefore rise with rising tailwater. When the diffuser pool water surface rises to the level of the chimney flow control orifice, water flows through the orifice and diffuser. The supply orifice controls the quantity and should be designed with an adjustable open area. A series of progressively higher chimneys will supply water to progressively higher fishway as the tailwater continues to rise.

The chimney concept can be used to supply wall diffusers also.

<u>Source</u> Do not be supplement the fishway with water from sources other than the primary water supply. They that may alter the scent of the water and confuse the homing instinct of fish. No surface water runoff or water with human scent should be allowed to enter the fishway.

The difference between deep and shallow forebay water supplies should be considered. The reservoir at Green Peter dam on the Santiam River is stratified. The fish facilities are no longer in operation for a number of reasons. When the facilities were operated, returning adults could not be attracted into a fishway. They were primarily attracted to a 10 cfs juvenile outfall that had a reservoir surface water source rather than the fishway of 50 cfs or powerhouse of up to 5000 cfs. The fishway and powerhouse water sources were both deep in the reservoir. As juveniles, these fish had moved through the reservoir in shallow water; as adults they likely recognized it as a separate source from the deep water and tried to follow it back upstream.

It is especially important to provide some hatchery drain water to hatchery fishways. At the Cowlitz salmon hatchery, less than 1 cfs of hatchery drain water is added to 23 cfs of fishway water and 150 cfs of fishway entrance water to effectively guide fish. Another 175 cfs of hatchery drain water goes directly to the river upstream of the fishway barrier dam.

Their is some belief that turbid water tends to attract fish and motivates them to move. The turbid water may be the equivalent to a freshet that induces fish to migrate.

Screen the auxiliary water supply where practical to eliminate cleaning maintenance requirements and prevent loss of juvenile fish passing through system.

OTHER GUIDANCE AND ATTRACTION MEANS

<u>Auxiliary attraction jets</u> Attraction jets adjacent to fishway entrances improve passage at least in some situations. The intention is to provide a stronger jet that penetrates the tailwater to reinforce and buoy the entrance jet. It must have a high enough velocity to act as an effective barrier to passage. Several of the Yakima River fishways have attraction jets as shown for Wapato Dam fishway in Section 3. These have not been evaluated. Attraction jets are attached to the outer wall of Alaska steeppass fishways at several locations; see Section 5.

<u>Sloped or notched weir</u> Special weir configurations are often used to help attract fish. Be careful that a notched weir does not add so much water that it drowns out the entrance with turbulence. Sloped and notched weirs have not been evaluated as far as I know. I think they are of marginal if any benefit. At low flow they are not needed because the fishway is attractive enough. At high flow, the distribution of flow is only marginally affected; likely not enough to change overall tailrace flow patterns enough to attract fish. Notches would be helpful in a very long crest and where the range of fish passage flows is not great.

<u>Angled weir</u>

<u>Crown spill</u>

<u>Distractions</u> Distractions that might attract fish away from the fishway entrance are as important as attraction to the fishway. Any concentrated flows in the tailwater should be eliminated or diffused to eliminate their attraction.

The hydraulic conditions at dam abutments often attract fish. A contraction occurs as the flow passes abutment the wall; it concentrates a small portion of flow several feet away from the wall alignment. A low flow shadow is left immediately next to the wall alignment. Fish approach the dam in the low flow shadow and are attracted to the concentrated flow. Abutment walls could be designed as efficient contractions to eliminate this distraction.

When a ogee weir crest bends at an angle point creating a dogleg alignment, a distraction is created. If the crest angle points upstream, flow from the two doglegs combine and form a strong jet in line with the bisecting angle of the dogleg. The crests can be tapered up to a high point at the vertex to decrease the flow from both legs. Baffle blocks can also be installed on the dam apron to break up the resulting jet.

FISH LADDERS

This section considers the fish ladder itself; the actual structure through which fish climb to a high elevation. It is part of an entire fishway system.

I divide fishways into six classifications based on their hydraulic design and function. Critical design elements, dimensions and limitations of the styles that are relevant to passage of Pacific salmon and steelead are discussed in this section.

Pool and weir fishways have distinct pools in which the energy of the flow entering is entirely dissipated. The hydraulic control between the pools are overflow weirs with or without orifices.

Vertical slot fishways also have distinct steps but the hydraulic control is a narrow vertical slot open at the top.

Roughened channels are chutes or flumes with roughness designed to control the velocity to a point adequate for fish passage. A natural stream channel is a roughened channel as are Denil style fishways and culverts with or without baffles.

Hybrid fishways are a combination of weir and pool, vertical slot or roughened channel fishways.

Mechanical fishways include lifts, brails and locks. They are mechanically operated fishways that can raise fish over an obstacle or into a trap or hauling tank.

Elvir and climbing passes are fishways through which fish climb either by using their fins or suction parts. The only relevant fishway in this category for the northeast Pacific would be for lampreye.

Appropriate selection of a fishway style for a specific site depends on a number of variables:

- species and age classes to be passed,
- scale of system; channel, hydrology
- degree of flow control available,
- dependability of operation and maintenance,
- debris, bedload and ice considerations
- mitigation and enhancement goals,
- capital and O&M costs.

DESIGN CONSIDERATIONS; POOL STYLE FISHWAYS

Fish Behavior Fish behavior and swimming abilities affect design concepts and details of fish ladder design. Fish move through fishways in different patterns. Early chinook tend to use orifices and late chinook and sockeye prefer weirs. The movement of early and late steelhead is the reverse of this. Shad use weirs exclusively and are wall oriented. They follow the walls and can be trapped in corners where there is no exit. Shad require streaming flow conditions for best passage. See Page 5-6 for a description of streaming and plunging flow conditions.

The fishways on the Columbia River dams were not initially designed with the consideration of shad passage. The flow control sections had orifices with no overflow sections. Shad would not pass through the orifices. The accumulation of shad then blocked salmon passage. As a result, hundreds of shad died in a day in the fishways.

Squaw fish, suckers and carp use the orifices.

Chum and pink salmon will not leap. They are strong fish but require a slot, orifice or submerged weir for passage. This may be due to the fact that they spawn lower in the river systems than other species and have not in their evolution developed to pass over water falls or other obstruction.

Pool Volume The volume required in a fishway pool must provide both adequate hydraulic capacity and fish capacity. The hydraulic capacity nearly always governs. It is the volume for adequate energy dissipation within the pool. This prevents a carryover of energy to the next downstream pool and reduces turbulence and aeration to the point that resting area is provided. The energy dissipation criteria for salmon and steelhead is based on a maximum energy dissipation of 4 foot pounds of energy per second per cubic foot of volume.

This criteria is applied by Equation (5-1) where V_{pool} is the

$$V_{pool} = \frac{\gamma \times Q \times h}{4 ft\text{-}lb/ft^3/sec} \qquad (5\text{-}1)$$

required effective energy dissipating volume of the pool. γ is the unit weight of the fluid (water) in pounds per cubic feet, Q is the flow entering the pool in cfs and h is the head of the flow entering the pool in feet. Portions of the pool, because of its length or shape, may not contribute to energy dissipation. It is recommended that no pool length greater than ten feet be included in energy dissipation volume calculation. Europeans use the same (200 watts/m^3) for salmon and 3 ft-lb/sec/ft^3 (150 watts/m^3) for shad (Larinier, 1990).

The pool volume may also depend on required fish capacity in situations of very large and concentrated runs. It is a volume requirement based on a maximum instantaneous loading rate and the

$$V_{pool} = \frac{C}{60} \times \frac{V}{r} \qquad (5\text{-}2)$$

allowable density of fish. The pool volume for fish capacity can be derived from Equation (5-2) where v is the volume required per fish in cubic feet, c is fish passage in number of fish per hour and r is the rate of fish movement in minutes per pool.

The pool volume recommended for fish is about 0.4 cubic foot per pound of fish. Columbia River fishway capacity experiments indicated a significant delay of fish when loaded at a density of 0.15 ft^3/pound when compared to a density of 0.36 ft^3/pound (Elling and Raymond, 1959). The fish were a mixture of fall chinook, sockeye and coho. The concentration of fish caused streaming flow conditions to develop at a density of 0.86 ft^3/pound and some delay occurred as a result. The streaming flow in this test was partially due to the weir configuration however. See Page 5-6 for a discussion on streaming flow.

It is interesting though I presume just coincidental that this volume recommendation is essentially the same as the standard long term holding density for chinook, 0.5 ft^3/pound used by WDF (Bob Hager, pers. comm.).

Fish tend to accumulate in the lower pools of a fishway or where hydraulic conditions change. It takes several pools before they accommodate and resume consistent up-ladder movement. Once they enter the fishway itself, salmon spend 2.5 to 4 minutes travel time per pool in the Columbia River fishways.

To size a fishway for the size of run; the resting volume within the pools is based on the maximum expected density of fish. Lacking other site data, the maximum density of fish is estimated as a specific portion of the entire run. For salmon, 10% of the run can typically pass a site in a single day. This estimate is verified by recent records from collection and counting stations on the Wenatchee, Cedar and Yakima Rivers. At Bonneville Dam, typically 2 to 8% of the chinook and steelhead and 5 to 8% of the sockeye and coho pass in a day at the peak of each of those runs. About 12% of the Okanogan River sockeye passed Zosel Dam in a day after they had been delayed for about a month by high water temperatures.

Nearly all passage is during the day with 60% from daylight to noon and the remaining 40% from noon to darkness. A typical distribution of passage is shown in Section 1. Based on these estimates, the maximum hourly passage rate would be 1.0% of the total run. It is estimated that the peak pink and sockeye salmon passage rates at Hell's Gate in 1989 and 1990 were each more than 20,000 fish per hour (Saxvik, 1990, 1991). That rate would account for about 1.5% and 0.6% respectively of the total runs. Simultaneous runs of different species have to be considered when calculating expected loading rates.

The design of the Bonneville Dam fishways was the first time the expected density was calculated as part of a fishway design. It was expected that 100,000 fish might pass through each fishway in a single day. The closest approach to that estimate that I am aware of is 47,000 salmon through both fishways in a day in 1986. It is not uncommon for over 50,000 shad to pass in a day.

The Hell's Gate fishways were the first to actually be designed for fish capacity. The capacity is believed to have been reached in 1954 when 2.0 million sockeye passed through Hell's Gate in six days (Andrew, 1990). The average rate was estimated to be 20,000 per hour fish in a single fishway.

In 1989 a new low level fishway was constructed at Hell's Gate on the Fraser River. The design capacity of the fishway was intended to be the same as the high level fishways at the same site, 6,000 fish per hour. This low flow capacity was based on the assumptions of 4.0 cubic foot per fish and a travel time of about one pool per minute. In 1990, 3.4 million Adams River sockeye moved through the ladder in less than 30 days (Saxvik, 1991). The peak hourly rate is estimated to have been 20,000 fish per hour. Accounting for the actual volume in the fishway at the time (5.0 feet of depth; design depth was 4.0 feet) the estimated passage rate was 2.4 times the intended designed capacity.

The minimum fishway pool depth for weir & pool and vertical slot fishways varies from 3 to 8 feet. The depth required depends on the scale of river, just as entrance flows.

<u>Orifices</u> The minimum size recommended for orifices within fishway weirs is 15 inches wide by 18 inches high for salmon. The primary reason for not allowing smaller orifices is the increased risk of plugging by debris. Orifices as small as 12 inches wide by 15 inches high are used.

The Min-Max rule of Government: The minimum criteria cited will be the maximum value applied.

The best use of orifices is as used in the Ice Harbor design fishway shown in Figure 5-2. The orifice located below the overflow section of the weir enhances the plunging action of the weir by reinforcing the roller circulation. Orifices can be used alone without overflow weirs as a flow control device; see the section on flow control.

Orifices should be shaped with efficient entrance contractions to provide the most stable flow.

<u>Head Differential</u> The recommended head differential between pools is normally 1.0 foot for most salmon and trout, 0.75 foot for chum and shad and 0.25 foot for grayling.

The lesser head differential for chum insures a nappe that fish can swim through. The head can be much greater if it is through a submerged slot or contained in a chute without free falling flow.

Depth over Weirs   One foot of water depth is a normal design. Increasing the depth to 1.2 feet in studies of the Ice Harbor style design significantly decreased the median passage time. The water depth over the weirs in large Columbia River pool and weir fishways has been increased to 1.1 foot to help dampen flow instabilities. In some cases it has been increased to a depth of 1.6 feet to induce streaming flow to improve shad passage. This change increased the fishway flow from 82 to 115 cfs. See Page 5-6 for a description of plunging and streaming flows.

A minimum of three inches of depth over tributary fishway weirs without flow control is reasonable for leaping fish. Where pinks or chum are to be passed, a notch with a minimum width of 1.5 feet and submerged by at least 6 inches by the next downstream weir works well. The notch should widen toward the top to help pass debris.

Freeboard   Freeboard from the water surface to the top of the wall should be a minimum of three feet. It is not uncommon for fish to leap out of fishways with lower walls. They will often leap at the upstream weir, miss and bounce off a side wall. Efforts should be made to minimize unnecessary leaping by eliminating concentrated spills near the walls and upwelling in corners.

An easy way to extend the freeboard in smaller fishways is construct a fence on top of the fishway wall flush with the inside wall.

Fishway Bends   Long fishways are often laid out to switch back on themselves through a series of 180° bends. Weaver and Thompson (1963) reported significantly longer passage times through corner and bend pools.

Regardless of the fishway style, details of the bends should be considered carefully to eliminate upwelling in corners and to maintain a consistent flow pattern. An additional pool length at the bend is often required to realign the flow to the downstream weir or slot. The jet flow from an orifice or vertical slot entering a turning pool should be aligned to follow the outside wall of the turn. The outside walls of the turn should be shaped to carry the jet around the bend without impacting a wall and upwelling.

If the jet must follow the inside wall, the wall should be extended for a minimum of 8 feet downstream of weir or add a baffle to deflect the flow into the center of the pool. For vertical slots, the required baffle is essentially the same as the short wall forming a vertical slot.

POOL AND WEIR FISHWAYS

Pool and weir fishways are the most common style in the Pacific Northwest and are applied to all scales of fish passage. Streambed controls at culverts fall into the category of pool and weir fishways though their design criteria differ and will be discussed separately in the section on tributary fish passage.

A primary limitation of pool and weir fishways is their narrow range of operating flow when no other flow control is provided. The lower limit of flow is the depth of flow over a weir, typically 0.25 feet. The upper limit is the flow at which the energy dissipation criteria discussed on Page 5-2 is exceeded or at the point the flow enters an unstable regime as described below. If the flow regime is consistently streaming, the upper limit will be determined by turbulence (inadequate energy dissipation) or the velocity of the streaming jet.

<u>Pool and Weir Hydraulics</u> Figure 5-1 shows plunging and streaming regimes in a fishway. The normal flow circulation in a pool and weir fishway is termed plunging regime. This circulation is set up by the nappe from the upstream weir plunging toward the fishway floor, moving downstream along the floor, then rising along the face of the next weir and either dropping over the weir or rolling back upstream along the surface of the pool. Streaming flow occurs at higher flows than the plunging regime. A continuous surface jet passes over the crests of the weirs and skims along the surface of the pools. The weir is backwatered by the downstream weir. Shear forces create a circulation in the pool opposite to that in the plunging regime. Rajaratnam et al. (1988) provide a good description of these flow regimes and empirical hydraulic formulae for both regimes.

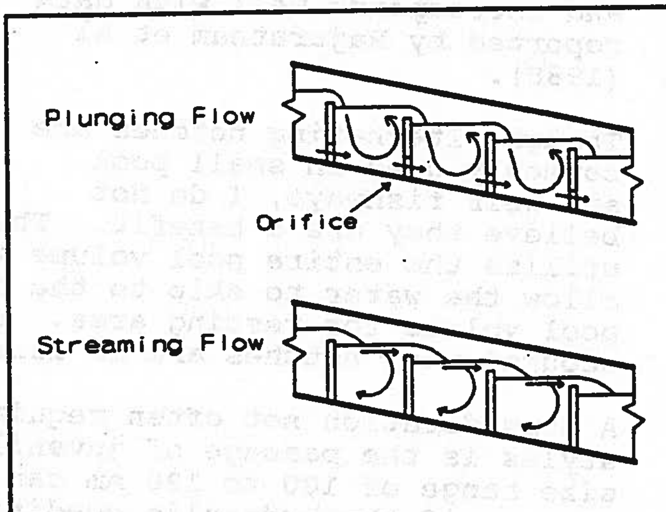


Figure 5-1. Plunging and Streaming Flow Regimes in Pool and Weir Fishway.

A hydraulic instability occurs in the transition regime between the upper range of plunging flow and the lower range of streaming flow. The transition regime and should be avoided. Passage studies have repeatedly shown that when the flow transitions, there is a delay in passage. The instability can also set up large oscillations that gallop through the fishway. The streaming regime should only be used with care. Energy is not dissipated in each pool of the fishway; the streaming jet is difficult to manage.

The shape of the weir crest and the presence and design of orifices

within the weir affect the hydraulics of the downstream pool. The orifice in Figure 5-1 supports the plunging circulation set up by the spill above it. An orifice in the streaming mode would conflict with the circulation.
The weir crest and orifice are effective in extending the plunging regime of flow. Table 5-1 shows in an eight foot pool the plunging regime extended by 36% (3.9 cfs/ft to 5.2 cfs/ft) by rounding the crest and adding an orifice. The unstable regime was essentially eliminated. This data is from Andrews (n.d.) and corresponds well with data reported by Rajaratnam et al (1988).

Table 5-1. Limits of Flow Stability; Pool and Weir Fishways. (Flow is cfs per length of weir in feet.)

	Upper Limit of Plunging Flow		Lower Limit of Streaming Flow	
	Flow	Head	Flow	Head
8-foot pool				
Square Crest	3.9 cfs/ft	1.1 ft	6.1 cfs/ft	1.1 ft
Square Crest, ports	4.2 cfs/ft	1.0 ft	±4.2 cfs/ft	
Round Crest, ports	5.2 cfs/ft	1.0 ft	±5.2 cfs/ft	
10-foot pool				
Square Crest	4.0 cfs/ft	1.1 ft		
Round Crest	4.4 cfs/ft	1.0 ft		

Though alternating notches are commonly used in small pool and weir fishways, I do not believe they are a benefit. Their intended purpose is to better utilize the entire pool volume for energy dissipation. I prefer to allow the water to skip to the next pool and preserve some of the pool volume for resting area. Bedload debris is also better scoured when notches are in line.

A consideration not often required in the selection of fishway styles is the passage of juvenile salmon. Coho juveniles in the size range of 100 to 120 mm can easily ascend a pool and weir fishway if the hydraulic conditions are appropriate. Thousands of coho juveniles are observed every spring moving upstream en the Lake Symmington fishway at Big Beef Creek.

<u>Ice Harbor Fishway</u> The Ice Harbor fishway is a 1-on-10 pool and weir fishway with orifices, flow stabilizers and a non-overflow section in the middle of each weir. It was initially developed in the 1960's at the Fisheries Engineering Research Laboratory for use at Ice Harbor Dam.

The half Ice Harbor fishway (Figure 5-2) is just as the name implies, half of the full Ice Harbor fishway, cut along the centerline. It is the recommended weir configuration for moderate to large applications where good flow control is available.

The full Ice Harbor fishway is 16 feet wide with two 5 foot overflow weirs. A flow of about 70 cfs is required. As many as 1,371 fish have passed through a full Ice Harbor fishway in an hour without sign of delay (Weaver et al, 1966).

Chinook and steelhead made significantly faster ascents when the depth over the weirs was increased from 0.95 to 1.20 foot with the orifice open. The opposite was true when the orifice was closed . The 1.20 feet of depth was reported by Weaver et al as head differential but I believe they actually tested water depth.

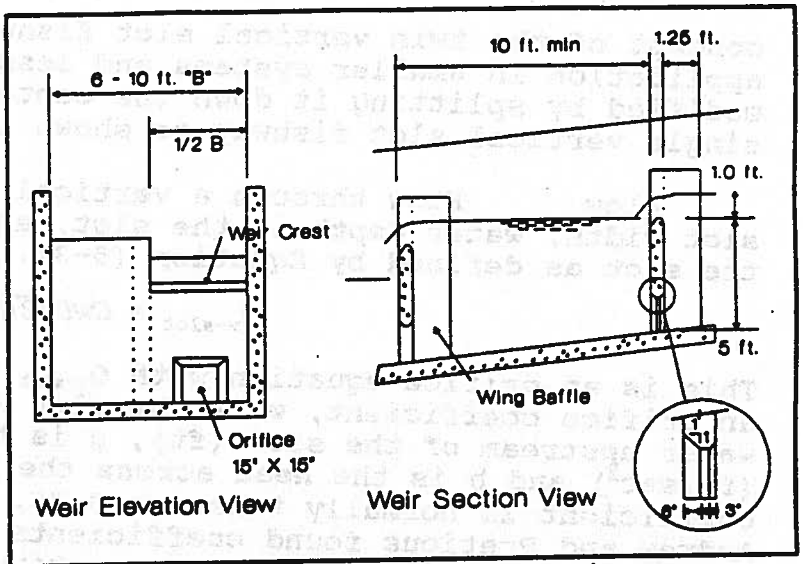


Figure 5-2. Half Ice Harbor Fishway.

VERTICAL SLOT FISHWAY

A vertical slot fishways also has distinct steps; the hydraulic control is a narrow, full height vertical slot open at the top. Its greatest advantage is that it is entirely self regulating. A vertical slot fishway is shown in Figure 5-2.

It operates without mechanical adjustment through a range of tailwater or forebay water surface elevations, except for minimum a depth requirement, equal to the depth of the vertical slots. The difference in elevation between the tailwater (or entrance pool) and forebay is nearly equally divided among all of the fishway steps. Any change in forebay and/or tailwater water surfaces is automatically compensated for by distributing the change throughout the fishway.

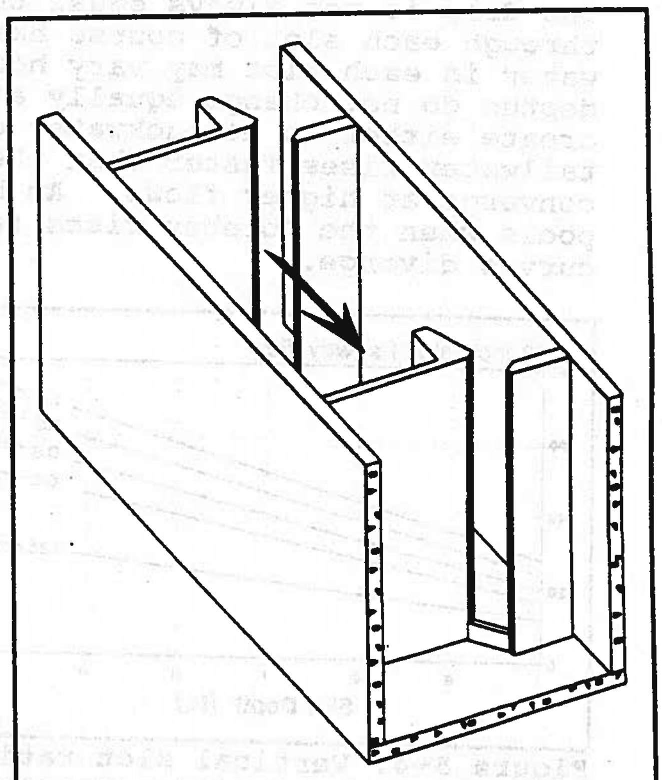


Figure 5-3. Single Vertical Slot Fishway.

Energy is dissipated in each pool by the jet cushioning and mixing with water in the portion of the pool between the larger baffles. As additional flow passes through the fishway, the pool depths increase creating additional pool volume and maintaining the appropriate energy dissipation.

The vertical slot fishway was first developed for application at Hell's Gate on the Fraser River. Model studies by Milo Bell and C.W. Harris were used to develop the concept of the twin vertical slot fishway for Hell's Gate. For application in smaller systems and less fish capacity, it was later modified by splitting it down the centerline creating the standard single vertical slot fishway as shown in Figure 5-2.

Flow Flow through a vertical slot is a function of the slot width, water depth in the slot, and head differential across the slot as defined by Equation (5-3).

$$Q_{v\text{-}slot} = CwD\sqrt{2gh} \tag{5-3}$$

This is an orifice equation with $Q_{v\text{-}slot}$ being the fishway flow, C is an orifice coefficient, w is the slot width (ft), D is the depth of water upstream of the slot (ft), g is the gravitational constant (ft/sec^2) and h is the head across the slot (ft). The orifice coefficient is normally taken as 0.75. Early model studies by Andrew and Pretious found coefficients of 0.82 to 0.62 for 12-inch slots with and without sills respectively (Andrew, n.d.). Flow measurements by the Bureau of Reclamation in the Sunnyside fishway on the Yakima River reveal coefficients of 0.91 to 1.04 for a 15-inch slots without sills (Onni Perala, pers. comm.). Figure 5-4 and Figure 5-5 show the vertical slot flow graphically for a coefficient of 0.75.

The drop is not always equal through all of the slots. The flow through each slot of course has to be identical. The depth of water in each slot may vary however if the forebay and tailwater depths do not change equally as the river flow changes. This will create either an M1 backwater curve in the lower pools when the tailwater rises faster than the forebay and the rating curves converge at higher flows. An M2 drawdown curve occurs in the upper pools when the forebay rises faster than the tailwater; the rating curves diverge.

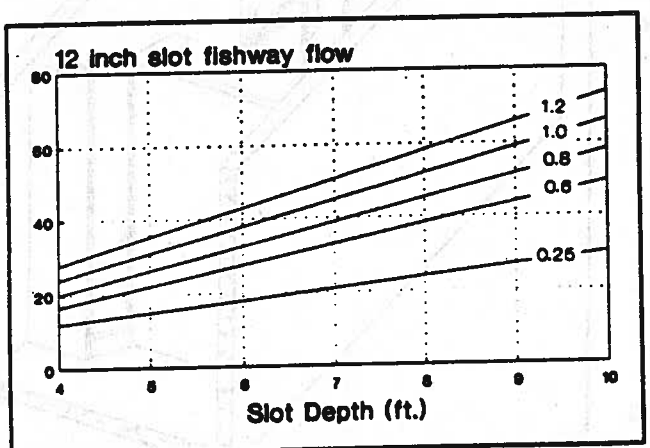


Figure 5-4. Vertical slot rating for various head differentials.

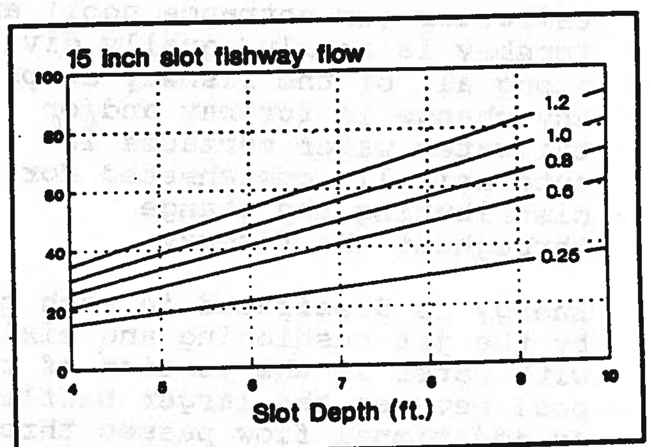


Figure 5-5. Vertical slot rating for various head differentials.

Different design processes are required for the backwater or drawdown situations. The floor elevations are based on minimum depth requirements at low flow for either case. The number of slots is determined by the maximum forebay to tailwater head differential whether it is at low or high flow. The low flow water surface profile or the high flow profile are analyzed for the backwater (converging ratings) and drawdown (diverging ratings) cases respectively to verify that the minimum head differential through the slots maintains the minimum transportation velocity. A slot velocity of 3.0 fps which is equivalent to about 0.25 feet of head is recommended as a minimum.

A normal minimum recommended depth at the upstream side of a slot is five feet; some are commonly operated to as low as three feet of depth.

<u>Dimensions</u> The dimensions of the vertical slot and pool are critical to the stability of flow. The dimensions shown in Figure 5-6 should be adhered to unless specific experience or studies indicate that other configurations work. Katopodis (1991) has tested a number of vertical slot fishway designs and suggests several minor variations in the geometry of the slot that also work well.

I learned this the hard way. Because of site constraints, the dimensions of the vertical slot ladder built at Tumwater Dam on the Wenatchee River in 1986 were modified. A 15 inch slot was used with pools that were 8 feet long by 12 feet wide. The additional pool width was intended to compensate for the reduced length. The result was very unstable flow with surge amplitudes greater than three feet. To stabilize the flow, the slot width was reduced to 12 inches and 12 inch sills were installed in the slots.

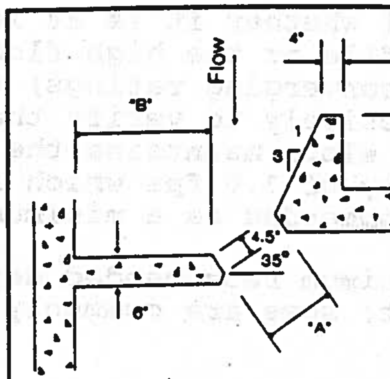
Sills across the bottom of the slots tend to stabilize the flow. Without them, especially at low depths, the flow tends to bypass the pool and move directly towards the next slot. The jet enters less of the cushioning pool and its energy may not be dissipated. The change of direction is caused by the fact that without a sill, the flow is forced to spread; it passes through the slot with a certain depth and is forced to occupy a foot less depth in the downstream pool assuming one foot of differential. The sill allows the jet to occupy the same depths above and below the slot and therefore stay more intact.

Sills should be placed across the slot at the floor if the vertical slot is operated with upstream depths less than about five feet or where the head differential may exceed the standard 1.0 feet. They offer some benefit to the pool hydraulics at any depth but also incrementally diminish the fishway flow.

Standard widths of vertical slots are 12 and 15 inches. Slots as narrow as 6 inches are suggested by Clay (1961) for smaller fish. Other dimensions of the vertical slot, in this case, would be reduced proportionately.

Hell's Gate main double vertical slot fishways have 30 inch slots and pools that are 18 feet long by 20 feet wide. The slots are up to 30 feet high. The maximum head differential through those slots is 0.6 ft. Remember that these fishways were designed for fish capacity rather than hydraulic capacity.

Passage The full depth vertical slots allow fish passage at any depth. The path of fish passage is assumed to not be tortuous; fish are able to move directly from slot to slot in nearly a straight path. This concept has not been verified. Hydraulic studies by Katopodis (1991) verified that the velocity through the slot is constant throughout the vertical profile.



Ladder Width	A	B	C
6'	12"	1'-3 3/4"	3'-1"
8'	16"	2'-3 3/4"	4'-1"

Figure 5-6. Vertical Slot Dimensions.

The vertical slot is not usually suited for species that require overflow weirs for passage or that must orient to walls. Pink salmon passage through Seton Creek vertical slot ladder has been timed at an average of 48 seconds per pool (Andrew, 1990).

ROUGHENED CHANNELS

Roughened channels are chutes or flumes with roughness designed to control the velocity to a point adequate for fish passage. A natural stream channel is a roughened channel as are Denil style fishways and culverts with or without baffles. Fish are expected to swim the length of a roughened channel in a single swimming effort whether it is in burst, prolonged or sustained swimming mode.

Roughened channels naturally have a velocity that is more attractive than a plunging overflow weir for a given flow due to the velocity of the jet exiting the fishway and possibly its aeration.

Roughened channels are typically used in smaller streams.

The exit (flow inlet) to any roughened channel should be carefully designed to minimize the water inlet head loss due to flow contraction and a sudden drawdown. The depth of water in the fishway and therefore its capacity is reduced by the extent of the drawdown.

Denil The Denil is an artificial roughened channel. It is used extensively throughout the world though is usually not the first choice of fishway style in the Northwest due to its limited operating range and vulnerability to debris blockages. Several sticks crosswise in a Denil and it can effectively be a blockage.

Primary use of Denils in this region is for temporary fish passage either until permanent facilities are constructed or during reconstruction of an existing fishway.

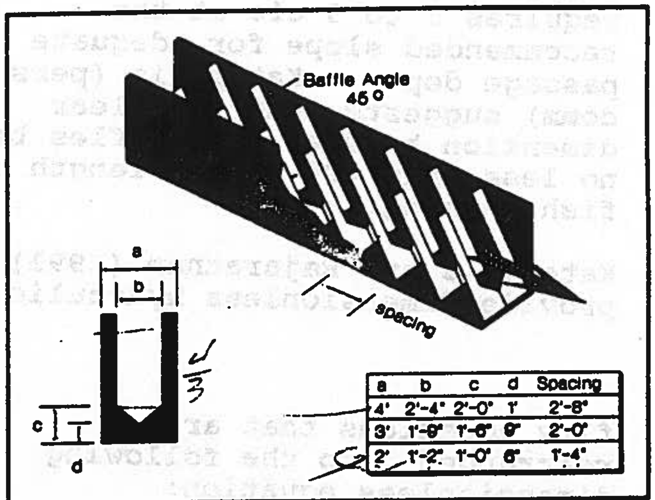


Figure 5-7. Denil Fishway.

A normal slope of 17% is recommended though they have been successfully used at slopes up to 25%. Standard dimensions are shown in Figure 5-7; the most commonly used size is the 4 foot width. A wide range of flows are possible depending on fishway size, slope and water depth. Rearranging the non-dimensional equations developed by Katopodis and Rajaratnam, the flow for a Denil design similar to that in Figure 5-7 (Design 4) is given in Equation (5-4). Q is the flow (cfs), D is the depth of flow above the vee baffle (ft), b is the open width of the fishway between the baffles (ft) and S is the fishway slope (ft/ft).

$$Q = 5.73D^2\sqrt{bS} \qquad (5\text{-}4)$$

Flow control is important though not as critical as for a weir and pool ladder. The forebay must be maintained within several feet to maintain good passage conditions in the Denil. According to the velocity profiles developed by Rajaratnam and Katopodis, centerline velocities increase towards the water surface in Denils where D/b is greater than 3.0. The height of the denil fishway has no limits; additional height adds attraction flow and operating range without additional passage capacity because of the higher velocities in the upper part of the fishway. Denils are typically constructed with depths of 4 to 8 feet.

Standard length sections are 30 feet; they can be built out of plywood or steel or concrete with steel baffles.

<u>Alaska Steeppass</u> The Alaska Steeppass (ASP) is a specific style of Denil fishway originally developed for remote installations. Steeppasses are used in the Northwest primarily in trapping and evaluation facilities and for temporary fish passage during construction of other facilities. There are also small ASP installations at small falls and dams.

The ASP is more efficient than the standard Denil in controlling velocities; it has a more complex set of baffles that are angled upstream into the flow. It requires less flow than the Denil, from about three to six cfs for the standard ASP depending on slope. The normal slope recommended is about 25%; they have been tested and used successfully up to a slope of 33%.

The concerns regarding debris and limited operating range are more critical in an ASP than a Denil because it has smaller open dimensions. A standard ASP has an open area between the baffles 22 inches high by 14 inches wide and requires 3 to 8 cfs at the recommended slope for adequate passage depth. Katopodis (pers comm) suggests that the clear dimension between the baffles be no less than 40% of the length of fish passing.

Katopodis and Rajaratnam (1991) provide dimensionless hydraulic flow equations that are rearranged into the following dimensionless equation:

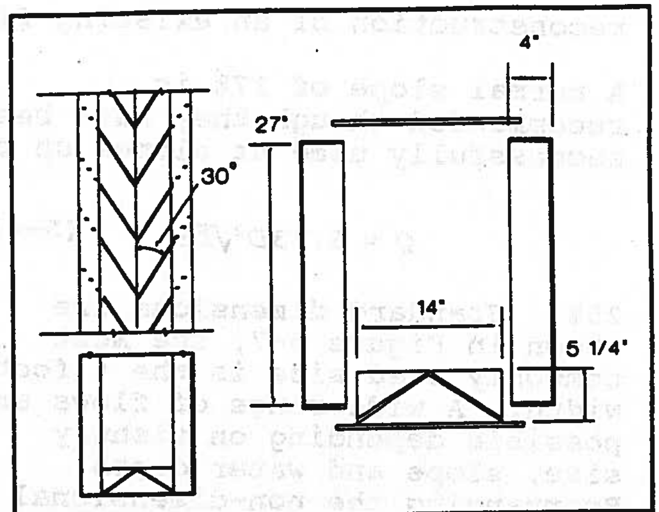


Figure 5-8. Alaska Steeppass Fishway.

$$Q_{ASP} = 0.97b^{5/2}\left(\frac{y_o}{b}\right)^{1.55}\sqrt{gS} \quad (5\text{-}6)$$

Q_{ASP} is the fishway flow, b is the open width inside the fishway, y_o is the depth of flow above the floor vanes, g is the gravitational acceleration and S is the ASP slope.

The English unit equation for the standard ASP derived from the Rajaratnam and Katopodis work is:

$$Q_{ASP} = 1.12S^{.5}y_o^{1.55}g^{.5} \quad (5\text{-}7)$$

Q_{ASP} is in cfs, y_o is in feet, g is in feet/second^2.

The average velocity in the ASP can be calculated from Equation (5-7). It is this average velocity that ASP designs are normally based on. The point velocity varies through the vertical profile. Rajaratnam and Katopodis found the the velocities to vary through the vertical profile in a full flowing standard ASP (y_o/b=1.6). The velocities varied by a factor of 2 and nearly linearly with depth with the lowest velocity at the water surface. ASP's that have a greater relative depth (y_o/b) than 2.1 have the highest velocity near the middle of the profile that increases with increasing y_o/b.

The primary advantage of the ASP is that it is prefabricated, modular and relatively light weight. ASP units are usually fabricated out of aluminum in 10 foot lengths and bolted together with end flanges. A 10-foot unit weighs 1500 pounds. The cost of a unit can be based on its weight and the current cost of aluminum fabrication. Plastic fabricators are interested in fabricating an ASP fishway and claim they can be competitive with aluminum in cost and durability.

Flow control is very important; the forebay water surface cannot vary more than a foot without creating passage difficulties. The tailwater should be maintained within about the same range to prevent a plunging flow or a backwatered condition that reduces the entrance velocity and therefore attraction. Slatick (1975) found that the median passage time for salmon increased four-fold and 25% fewer salmon entered the fishway when the downstream end was submerged by 2.5 feet.

ASP units can be hinged at either end to accommodate water surface fluctuations. The fishway flow will change, of course, with changing slope.

ASP sections are usually set in lengths of 20 to 30 feet. Resting pools are provided between sections. The design of resting pools between sections of ASP is important in order that energy is dissipated, upwelling does not distract fish and velocity does not carry over into the downstream section. Blacket (1987) concluded a delay of fish in a resting pool can delay following fish. He studied passage through ASP's 180 feet long at a 22% slope with and without resting pools. Though passage was 31 to 69% greater through the ASP with resting pools, there were no obvious passage problems for sockeye, pink, chum and chinook salmon in either case.

The maximum passage rate observed by Blacket through a single ASP was about 1850 sockeye per hour over 2½ hours and 400,000 in less than four weeks. He concluded the ASP fish passage capacity was not exceeded based on the rate of fish dropping back out of the fishway entrance pool. Slatick (1975) estimated the passage capacity of chinook in an ASP was between 650 and 1140 fish per hour. Shad will pass through a steeppass but reluctantly. Slatick found that nearly 100% of spring and summer migrant salmon entered the ASP within an hour whereas only 61% of the shad had entered the fishway after three hours.

To provide auxiliary water a box conduit or open trough can be attached to the side of the ASP. The high velocity from the conduit or trough reinforces the flow from the fishway. The open trough is used when the tailwater backwaters the fishway significantly; the flow is open to the tailwater regardless of the tailwater elevation. The French Creek, a Snohomish River tributary, and Elk Creek, a Chehalis River trib are examples of this. Trapping counts at Elk Creek show a significant increase in passage on the days the auxiliary water is on. The auxiliary water at Elk Creek actually reverses the tailrace circulation in the vicinity of the entrance as a passage improvement.

<u>Roughened Chute</u> Orsborn and Powers (1984) tested a narrow chute with 1½ inch by 1½ inch blocks across the floor at 6 inch spacing for roughness. Chum and coho had passage rates exceeding 95% at slopes of 27% and 15% respectively. The test flume was eight feet long.

<u>Engineered Steepened Stream Channel</u> Constructed channel fishways are intended to replicate steep natural channels. If

adequate land is available, a natural channel can often be constructed around a low barrier that remains as a flow control spillway. Such channels have been constructed with control sills and with rough rock linings.

Mill Creek fishway on a tributary to the Bogachiel River has 18 inch riprap imbedded in a concrete slurry. It is 95 feet long on a slope of 8.0%. It has a 10-yr design flow of 1200 cfs and a high passage design flow of 160 cfs. It was constructed in 1970 and has required one major repair to replace rock since then.

Colony Creek fishway is constructed over a deep soft clay substrate. The barrier dam remains in place as a flow control spillway and an orifice at the fishway exit controls peak flows to the fishway. The channel slope is 3.4%. The fishway channel has a four foot toe width and has a riprap liner 18 inches thick. Boulders, 24 inches in diameter, were placed on alternating sides of the channel on the riprap bed four feet on center. The boulders act as roughness elements to control the velocity. Six inches of 3 inch pit run gravel was placed over the riprap bed between the boulders. The pit run gravel seals the riprap below it and provides some channel diversity through the fishway. The fishway is below a pond so no other bed material will enter the channel.

A practical limit of slope for rock lined constructed channels is about 3.5% without specific design of roughness elements and their anchoring system. Steeper channels could be built but only with very careful design and construction; bed sills are recommended.

HYBRID FISHWAYS

<u>Pool and Chute Fishway</u> The pool and chute fishway is an alternative to pool and weir fishways that operates through a wider range of stream flows without other flow control. The pool and chute is a pool and weir fishway at low flow and cross between a pool and weir and a roughened chute at high flow. The weirs are vee shaped with a horizontal weir set into a notch at the apex of the vee.

At low flow, the fishway performs as a pool and weir fishway with the flow plunging and dissipating in each pool. At high flow, a high rate of streaming flow passes down the center of the fishway; plunging flow and good fish passage conditions are maintained on the edges of the pools. The economy of the concept is achieved by exceeding the usual criteria of fishway pool volume based on energy dissipation in each pool.

Pool and chute fishways should not be used where the total drop exceeds about six feet until it is more thoroughly tested. They can be used in culverts only when the culvert is very wide. Additional research is required to develop comprehensive design standards.

The general configuration of the pool and chute is shown in

Figure 5-9 which depicts the Town Dam fishway.
The pool widths are about twice the pool lengths. The notch and weir dimensions are determined by designing capacity for the design passage flow to not spread to the outer edges of the sloping weirs. Those areas then remain as holding areas and passage corridors.

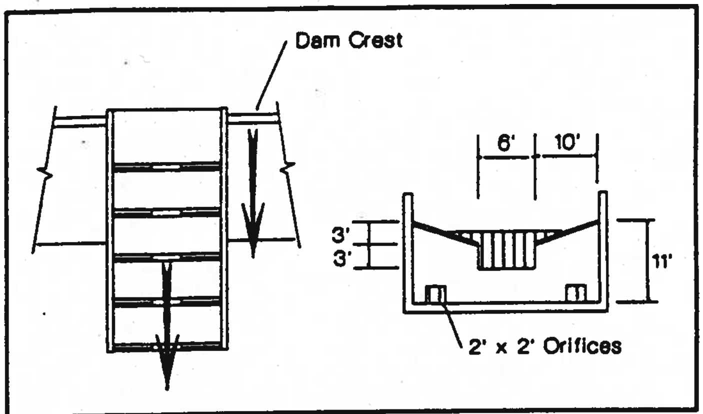


Figure 5-9. Pool and chute fishway; Town Dam, Yakima River.

Bates (1991) reported the results of model study of the pool and chute fishway at slopes of 4.9, 11.1 and 16.7%. At passage design flow, the Chezy coefficient varied from 30 for a 4.9% slope to 20 for a 16.9% slope. Streaming flow could not be created at the steeper slope. Slopes greater than 11% should therefore only be used with great caution. The upstream two weirs are lowered relative to the gradient of the other weirs to account for less velocity head entering the fishway.
Pool and chute fishways have been constructed in several situations in Washington State. The largest was constructed in 1988 by the Bureau of Reclamation on the Yakima River and is shown in Figure 5-9. It has a high design passage flow of 343 cfs when the total river flow is 3880 cfs. Based on the model data by Bates (1991), the velocity of the jet exiting the fishway is 11.5 fps and 20 feet downstream it is 7.1 fps. This highlights the need to use this concept with caution. If not sited appropriately, the high energy jet will scour the downstream channel and/or banks. The steep slope tested by Bates created a velocity of 2.2 fps 20 feet downstream because the flow was plunging instead of streaming.

Pool and chute fishways have been installed inside of short culverts. Other pool and weir fishways have been constructed with a geometry similar to this design.

The pool and chute fishway shown in Figure 5-10 is a variation constructed on Kenney Creek, a Nooksack River tributary. It is a switchback pool and chute. The entire stream passes through the center of the fishway like the standard pool and chute. Low flows follow a more circuitous route in a weir and pool configuration. It has a high passage design flow of 27 cfs and a 100 year flood flow estimated at 460 cfs.

The switchback layout has the advantage that though its fish passage hydraulic profile is 8.0%, the structure has a physical profile of 16%. The Kenney Creek fishway was fit into a short reach of channel between a road culvert and the main channel of the Nooksack River.

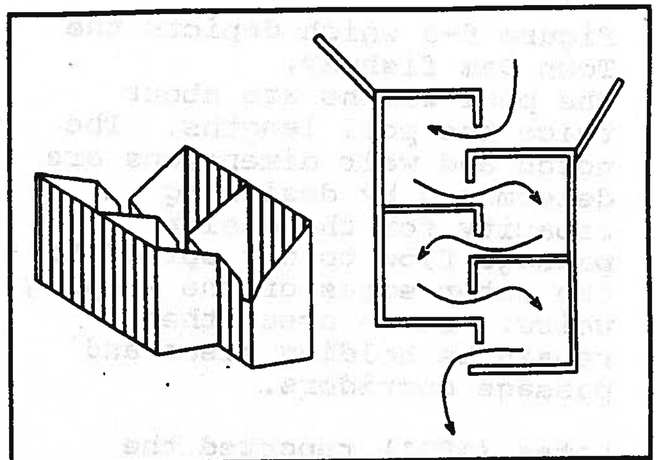


Figure 5-10. Switchback Pool and Chute Fishway.

FISHWAY FLOW CONTROL

The purpose of flow control is to extend the range of river flows through wich the fishway operates within criteria. Flow control accomodates fluctuations in the forebay water surface while maintaining acceptable ladder flow conditions.

The degree of flow control required for a fishway depends on the style of fish ladder. The recommended range of operation of different ladder styles is discussed in previous sections.

There are essentially five styles of flow control; they might be used individually or together:

- Self adjusting fishway;
- Spillway control;
- Orifice or vertical slot flow control section;
- Adjustable weirs;
- Multiple level exit.

<u>Self Adjusting Fishway</u> Vertical slot and orifice fishways are self adjusting. As long as the forebay and tailwater elevations do not exceed the height of the slots or structure, the fishway functions as intended. The Denil and pool and chute fishways are intended to also be self adjusting though their ranges of operation are much more limited.

<u>Spillway Control</u> A spillway that controls the fishway forebay provides flow control. Its effectiveness is a function of its length. Automated spillway gates can control pool elevation to within a few tenths of a foot.

<u>Orifice or Vertical Slot Control Section</u> A flow control section is a portion of the fishway upstream from the primary fish ladder specifically intended for that purpose. It contains orifices and/or vertical slots through which fish pass. The hydraulic slope through the flow control section increases as the forebay rises. Greater flow and head loss therefore occur through the section compensating for change in forebay. To accomodate the change in flow through the control seciton, either auxiliary water can be supplied or excess water bled off below the control section.

<u>Auxiliary Water Flow Control</u> Auxiliary water flow control sections are ususally designed to carry the full fish ladder flow at high forebay. The auxiliary water is suplied to the lower fish ladder at anything but the highest forebay condition.

Figure 6-1 is a schematic of an orifice flow control section including the hydraulic profiles of the high and low forebay conditions. Easton Dam fishway on the Yakima River for example has a constant ladder flow of about 27 cfs. This is supplied from two sources; the orifice control section (9 cfs at low forebay to 27 cfs at high forebay) and the auxiliary water system (from 18 cfs at low forebay to none at high forebay).

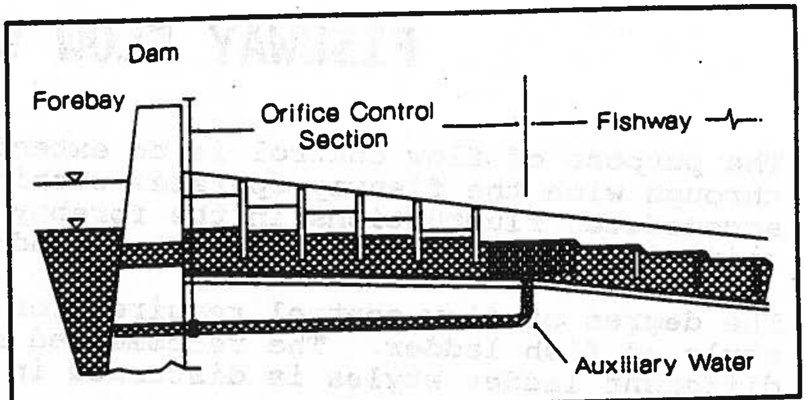


Figure 6-1. Schematic of Auxiliary Water Orifice Flow Control Section.

This type of auxiliary water system operates contrary to the forebay; less flow is needed with increasing head. Therefore, unless it can be operated very carefully and with close attention, the auxiliary water control gate should be automated and electronically tied to the forebay water surface.

Minimum orifice sizes are similar to those in fishways. Debris will plug the orficies which are difficult to maintain du to their depth. Care should be taken in design of the orifice so jets do not align and energy is not carried from one orifice to the next. They can be designed with a geometry as if they were very short slots in a vertical slot ladder.

Vertical slot flow control is preferred to orifice flow control but requires substantially more flow. It consists of simply a vertical slot ladder section ahead of the primary ladder. The flow control section can also be a combination of orifices and slots.

<u>Bleed-Off Flow Control</u> The bleed-off flow control section is hydraulically similar to the auxiliary water flow control section. Instead of auxiliary water being supplied at low forebay, however, excess water is bled off at the downstream end of the flow control section during high forebay conditions. It is used when the normal operating condition is the low forebay; the flow control accomodates peak flows.

Elk Creek fishway, on a Chehalis River tributary, has an Alaska steeppass fish ladder with an orifice flow control section above it. Part of the excess flow from the flow control section is wasted back to the stream and the remainder is used as auxiliary attraction water at the fishway entrance.

<u>Adjustable Weirs</u> Instead of bringing the forebay to the fishway, adjustable weirs and multiple level exits take the fishway to the forebay. Telescoping or tilting weirs in the upper portion of the fishway can accomodate a small variation in forebay elevation.

Adjustable weirs should be automated and tied electronically to the forebay water surface. They are usually actuated individually. As many as six tilting gates are mechanically tied together and operated as a single unit on the Takase Weir in Japan (Watanabe et al, 1990). Forebay fluctuations of up to 11.5 feet are accommodated at that fishway.

Telescoping gates are preferred over tilting weirs. They have a wider operating range than tilting weirs with the same overall length of control section. They also have better flow conditions within the pools; tilting gates disrupt normal flow patterns within the pools.

Another style of adjustable weir flow control is the Nibutani style gate describe by Watanabe (1990). The control section is hinged at the downstream end. The upstream end is raised and lowered mechanically to follow the forebay water surface. The flow control section of the Nubutani Dam in Japan is designed with 20 weirs and a length of 120 feet. It has a forebay operating range of 22 feet. There are other examples of this concept designed in Yugoslavia. As far as I am aware, none have been constructed.

<u>Multiple Level Exit</u> If a forebay is operated in more than one distinct operating range, multiple level exits may apply; Roza Dam on the Yakima River is an example. The forebay water surface at the upper range is maintained within several tenths of a foot by spillway roller gates. Because of icing conditions and for maintenance purposes the roller gates are open and the forebay is drained to a nominal elevation 15 feet below the high forebay range for about a month each year.

A lower exit simply branches off of the fishway at the appropriate elevation and exits through a gated conduit in the dam. When not in use, the lower branch is gated closed. In the case of Roza Dam, three telescoping weirs provide flow control at the lower level exit. The switch between high and low exits is manual; the fishway must be inspected each time and any stranded fish are removed.

Multiple exits can be used to accomodate a variable forebay if the fluctuations are gradual and not frequent. The Nagura Dam on the Hida-gawa River in Japan has exits from each of the top nine fishway pools. Each pool has a sluice gate exit about a foot lower than the pool above it. Only one gate is operated at a time depending on forebay level.

FISHWAY EXIT

EXIT DESIGN

Fish exiting the fishway into a forebay often tend to delay; they may be disoriented and take some time to adjust again to the new environment.

To get upstream fish must swim into current; they also tend to follow the shoreline. The exit must be located where fish can orient to a shoreline and into a consistent current that will guide them upstream. The exit should be at a depth comparable to depths within the fishway.

Forebay currents should be understood through the complete range of design flows. River flow passing the falls, powerhouse or spillway may set up eddies within the forebay that confound the problem. The classic example of this situation is the south fishway at Bonneville Dam. Figure 7-1 shows a sketch of the major and minor migration routes of fish leaving Bonneville Dam. Radio tagging studies determined that a significant number of fish moved "upstream" into the river current and along the bankline that led them to the spillway where they fell back downstream. An eddy at the northern side of the tip of Bradford Island created the counter flow that guided the fish to the spillway. The fish then returned through the fishway and were counted again. One radio tracked chinook fell back three times and was later tracked 250 miles upstream.

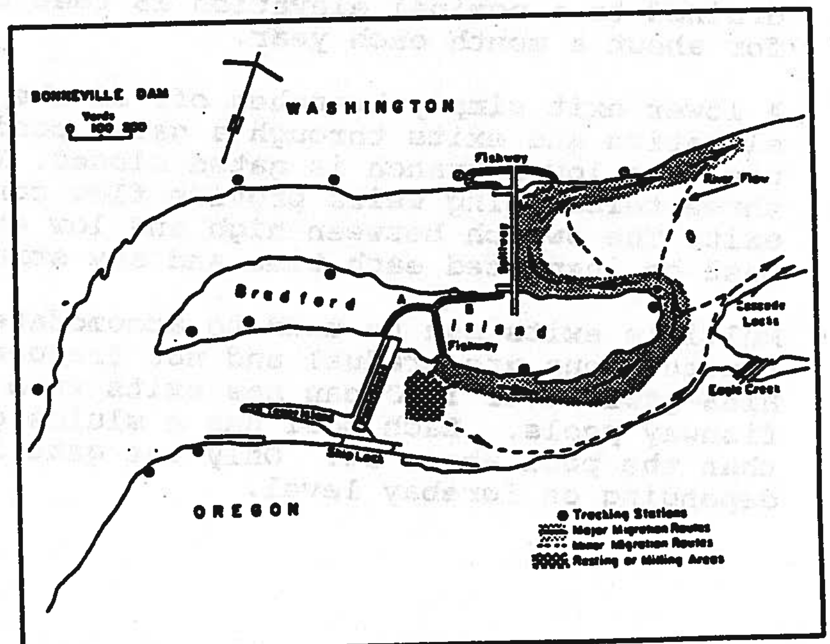


Figure 7-1. Plan view of Bonneville Dam showing upstream major and minor migration routes.

The rate of fallback depended on river flow and varied from 13.4% at flows between 140,000 and 200,000 cfs to as high as 67% at river flows over 200,000 cfs. The eddy apparently is washed out at very high flows; fish left the tip of Bradflor Island and swam directly upstream at flows greater than 400,000 cfs. An additional powerhouse is now located in a new channel north of the spillway shown in Figure 7-1.

Fallback has been studied at other dams on the Columbia River. Fallback rates appear to be generally less than 10%. Most fish that fall back survive.

Avoid exit location next to the spillway or powerhouse intakes. Try to locate the exit on a bankline that will guide fish upstream rather than in the center of the channel or on an island. Extend the exit channel upstream if necessary to locate the exit in an area of consistent positive downstream flow.

The exit location also determines the water source of the fishway. Avoid locating exit in stagnant area where water quality may be poor or where there is any risk of contamination entering the river. For attraction to the fishway, water quality in the ladder must be the same as the water from powerhouses or spillways; consider odor, temperature, and surface water runoff.

TRASH RACK AND DESIGN DETAILS

The exit should have a trash boom and/or coarse trash rack. Consider wind and current directions to help determine the rate of debris accumulation.

Debris racks that are cleaned manually with rakes should have a maximum normal velocity no greater than about 2.0 fps. Much higher velocities, and they are increased when debris accumulates, make it difficult to manage a rake by hand. There are standard mechanical trash cleaning rakes that are self operating on a timer system or manually operated similar to a back-hoe with a rake attachment.

The river velocities that sweep across the face of the fishway are important. Higher velocities will result in substantial head loss across the trash rack as the kinetic energy of the sweeping flow is lost. Excess head loss may result in a decreased fishway flow depending on the fishway flow control mechanism and it may result in sediment deposition. They can also likely cause some fallback of fish that may be disoriented when exiting the fishway.

For salmon, vertical bars should have 5 to 10" clearances. Horizontal bars should be spaced no closer than 18" apart. Horizontal bars should be inset or on the back side of the vertical bars so debris can slide up the outside face of the trash rack. A curtain wall above the trash rack and flush with its face can be helpful when there is adequate depth that the additional open rack is not needed. If it is designed at the right elevation, larger debris during high flows will pile up against the wall where it is easier to remove than when it is twined into the bars of the trash rack.

A sluice must often be provided to maintain depth of water at the fishway entrance. Consider also access for equipment to clean debris and sediment from the exit and forebay upstream of it. Where substantial large debris is likely, a system for winching heavy debris off the rack is helpful.

Slope the face of the trash rack at 1 horizontal to 4 or 5 vertical for leverage and easy manual cleaning. Provide a sturdy railing for cleaning and consider the need for lights for night maintenance.

Provide structural freeboard on the fishway exit to prevent flood damage.

Provide stoplogs or a closure gate for dewatering the fishway for maintenance.

A trash boom can be helpful. The ideal trash boom is a shear boom designed to carry debris past the fishway exit to the spillway or falls. Small fishways usually need nothing more than a single shear log chained at both ends or attached with a sliding ring to a vertical post. For larger debris and greater buildups consider a double or triple log boom for accumulating debris.

MISCELLANEOUS DESIGN CONSIDERATIONS

Provide staff gages for operation

- above and below entrances to measure entrance head and flow;
- at auxiliary water diffusers and trash racks to determine the extent of debris plugging;
- at a fishway weir to measure fishway flow;
- in the forebay as a river flow gauge.

Make sure the staff gauges will be visible for the operators. Consider even the orientation of deck grating bars so staff gauges below the grating can be seen easily.

An operation manual should include the following items:

- River flow (staff gauge elevation) which identify high flow and low flow regimes for entrance gate operation. The low and high flow operating regimes should overlap so there is a narrow range through which either gates can be operated.
- Settings of entrance gates; they are normally either entirely open or entirely shut.
- Head differentials that must be maintained at entrances and trash and diffuser racks.
- Settings of auxiliary water systems based on auxiliary water system staff gauges and river flow.

Counting, collection, sorting and loading facilities

Security, lights intrusion and flow alarms

Safety, attractive public nuisance

All surface water must drain away from the fishway. Odors and contaminants can be a deterrent to fish passage.

STRUCTURAL DESIGN

Debris, bed load, ice and flood protection

In-stream protection; scour, flotation

Abrasion (weirs, vertical slot floor); steel caps

Materials

TRIBUTARY FISH PASSAGE DESIGN

Fish passage concepts for small streams are presented in this section. More detail is provided here because it is the detail that makes the success of this scale of project.

Many miles of salmon and steelhead spawning and rearing habitat are blocked by small natural falls and man made barriers such as dams and road crossings. Several watersheds in Washington State were surveyed in 1984 to identify all human made barriers to salmon migration. The two basins have a total area of 30 square miles and a total channel length of 50 miles. Portions of the basins blocked from salmon use varied from 6% in a rural watershed to 24% in an urbanized watershed (Tom Burns, WDF, pers. com.). Since the upper reaches of these watersheds provide the best spawning habitat, likely as much as 50% of the spawning habitat was not accessible.

Tributary fishway sites are often remote, seldom inspected and have little or no flow control.

Improved access into these areas is normally provided by the construction of cast-in-place concrete fishways. In an effort to minimize construction and maintenance costs, alternative fish passage concepts and construction methods have been developed for both adult and juvenile salmon. The ideas described here are primarily intended for use in small tributaries. Some are new ideas; others are refinements of methods used previously. Construction cost savings are made by:

- prefabricating units for remote installation,
- designs that utilize the channel itself as the fishway,
- designs that do not require equipment access and
- concepts that exceed normal fishway design standards without reducing efficiency.

TRIBUTARY FISHWAY CONSTRUCTION CONCEPTS

The following fishway concepts have been designed and constructed recently by Washington Department of Fisheries. Not all of these ideas are new.

Log Sills   Log sills are control sills built into the stream bed as shown in Figure 10-1 and spanning the entire channel width. They are a low cost and durable means of fish passage for streams with natural gradients of less than about 3% and channel toe widths of less than about 30 feet. The log sills described here are intended for fish passage. Similar designs are used with the objective of enhancing rearing or spawning habitat. The design of habitat structures is often different in order to create a concentrated flow and deeper plunge pools.
Log sills are typically installed in a series with a spacing from 125% to 175% of the channel width and a minimum spacing of 15 feet.

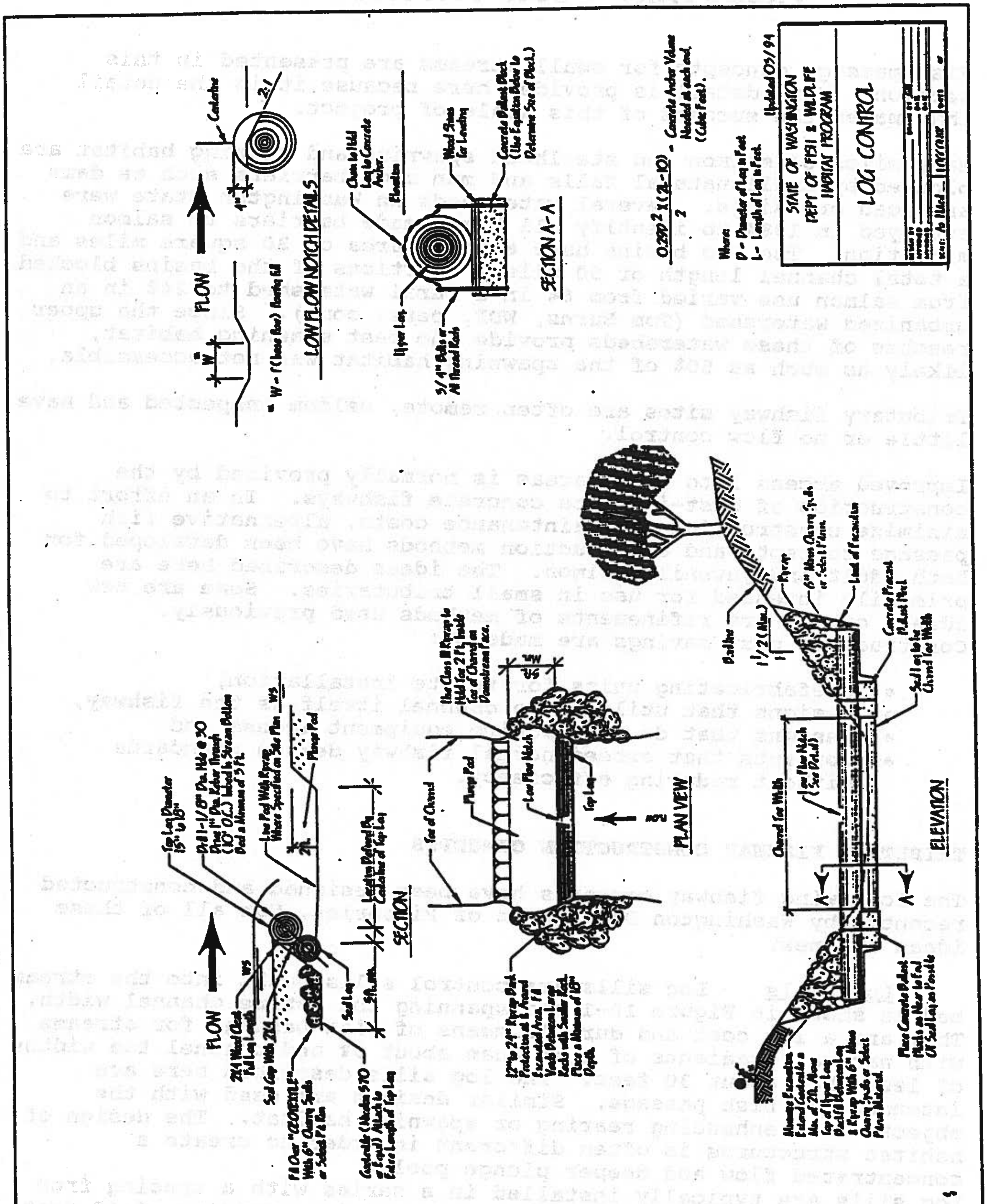


Figure 10-1. Streambed log sill design detail.

A closer spacing causes the scour pool of each log to extend to the next sill downstream and does not allow bed material to accumulate and protect the upstream face of the downstream sill. Log sills are not structurally durable in themselves. They support the streambed which protects and seals the log weirs. When used for fish passage, sills within a series should be constructed with equal lengths for uniform hydraulic conditions at high flows.

A pair of logs, each with a minimum diameter of one foot are placed into the bed; it is recommended that the sum of the diameters at any point along the structure is at least three feet. The downstream pool will scour to a depth greater than two feet below the downstream control elevation. The bottom log is offset upstream on a line about 45° from vertical to allow the scour to undercut the upper log. The top log is strapped to precast concrete blocks buried below each end of the sill and adequate to anchor the logs. A good rule of thumb to control deflection of the top log is to use a log with a diameter 1/25th of the log length.

Beach's Law
No two identical parts are alike.

Careful anchorage or ballasting of the logs is a critical to their durability. The design described here depend entirely on the concrete ballast block.

In 1953, 41 instream structures in Sequoia National Forest in Northern California were evaluated after 18 years of operation. The most common reason for structure failure of log dams and deflectors was inadequate anchorage of the ends (Ehlers, 1954). The most common factor of the successful structures was the presence of dense willow stands that also helped anchor the ends of the logs.

Double logs are used to prevent the scour pool from undermining the structure. The ends are buried into trenches excavated into the streambanks a minimum of five feet. The logs are normally douglas fir due to availability, straightness and longevity. Their longevity is determined by the amount of bed material abrasion; the sills are installed level, they are permanently submerged and resist decay. Retired mill boom logs have been a good source of logs; they are long and straight and preserved years of salt water use. After 18 years, the 9 remaining log dams of the original 15 in the Sequoia National Forest study showed no signs of rot or deterioration even though they were at least partially exposed during most of that time.

A seal is attached to the upstream face of the top log, buried 2 feet and extended upstream at least 6 feet. Geotextile fabric is used with a tensile strength of at least 600 pounds and a burst strength of at least 1200 pounds. Geotextile fabric has good longevity, availability and flexibility for ease of construction. It is easier to install than impermeable material which billows in the current during installation. The fabric must be extended into the trenches to completely seal the structure.

Riprap mixed with soil is packed over the ends of the logs within the trenches and on the banks extending to six feet downstream of the sills. The riprap is bank protection not ballast. A pool is excavated two feet deep by six feet long in the channel downstream of each log sill in anticipation of a scour pool that will develop naturally. If a pool is not initially constructed, there is a risk that the first high flow will stream over the sills, energy will not be adequately dissipated and the downstream channel will be damaged. The bank rock must extend to the floor of the pool. In installations where bed material does not pass into and through the fishway, the floor of the pool should also be lined with riprap rock.

By my observations, the maximum fish passage design flow is limited to about 9.5 cfs per foot of length of the log sill. The maximum safe high design flow has not been quantified. The highest known flow incurred in my experience by a series of log sill structures is 15 cfs per foot of length. The weir coefficient for a log weir submerged to 50% of its depth is about 2.7 based on field measurements. Heiner (1991) found a weir coefficient of about 3.8 for full scale unsubmerged smooth (PVC pipe) weirs in a laboratory.

Because of the recommended minimum spacing and maximum elevation drop, the maximum final slope of a series of log sills is 5%. It is difficult to steepen a channel with an initial natural slope greater than about 3% with this style of log sill. The number of sills required to steepen a channel is given by Equation (10-1) by simple geometry where N is the number of sills required, H is the elevation gain to be achieved, L is the spacing of the sills (15 feet minimum recommended), S_o is the initial and S_d the desired

$$N = \frac{H}{L \times (S_d - S_o)} \tag{10-1}$$

channel slope (in ft/ft).

Sills should be located in straight sections and at the entrance and exits of channel bends; they should not be installed in bends. There is a risk that if a lower sill of a series fails, those above it will be undermined and also fail; dominoes. If a number of bed sills are placed in a series, deeper sills should be placed at intervals, say, every fifth sill. The deeper sills should be designed as independent dams assuming the downstream controls do not maintain a backwater. Their purpose is to prevent the chain reaction and the failure of the entire series.

Log sills as described here can be placed with a single piece of equipment or by hand in small dewatered installations. The current (1994) total cost of construction of a log sill within a flowing stream is about $3000. Maintenance of full spanning sills is much less intensive than formal fishways because the channel is not constricted and debris freely passes. Accumulation of a moderate amount of debris does not present a risk to the structure and can provide good rearing cover in the plunge pools.

A notch is cut in the crest of the sill after it is installed. The shape and size of the notch depends on the species requiring passage and the low flow expected at the time of passage. The notch generally slopes down to form a plume that fish can swim through rather than be required to leap through a free nappe. Be careful to not make the notch so large that at low flow the top of the log is dewatered.

Of about 150 log sills of this type installed by Washington Department of Fish and Wildlife since 1983, two have failed by being undermined; they were single log structures.

Single log sills have also been cabled into bedrock channels using 9/16" galvanized steel cable and C-10 HIT Hilti dowelling cement (Espinosa and Lee, 1991).

Log sills are also good for stabilizing certain channel erosion problems, holding spawning gravel and for creating holding and rearing pools. The designs for those purposes are somewhat different than for fish passage and are not discussed here.

<u>Plank Sills</u> Rough cut milled timbers are placed across the bed of a channel to form sills similar to the log sills described above. They are intended to be constructed by hand in small or spring source streams with regular flow. They are installed with a maximum drop between pools of 8 inches. When installed in steady spring source streams, a series of plank sills can be installed at a slope up to 7%. Plank sills have an application limited to channel toe widths of about 10 feet. The maximum standard timber length available is 16 feet; each end is imbedded three feet into the bank.

Untreated fir timbers are used in perennial streams where the wood will be always be submerged. Cedar is used in ephemeral streams. The planks are trenched into the bed of the channel and anchored with U-bolts to steel pipes driven into the streambed. They are tilted about 20° downstream so the nappe spills free of the sill for better juvenile fish access. The ends are buried in the channel banks; the excavated trenches are filled with light riprap rock mixed with soil.

Plank sills are especially useful for providing upstream juvenile salmon passage. They are well suited for streams with sandy beds. In the last 5 years WDF has constructed 44 of these structures without a failure. A benefit of plank sills is they can be constructed entirely by hand. The cost of construction of a plank sill varies greatly since the primary cost is labor; the amount of labor depends on site conditions such as bank height and soils.

Plank sills have been constructed in wide channels using zig-zag and spider weir designs. They are primarily intended for juvenile fish passage and are described in the <u>Upstream Juvenile Passage</u> section.

<u>Precast Concrete Fishways</u> In areas that are remote from

concrete plants and in situations that are difficult to dewater and pour concrete, precast concrete fishways can be installed.

Three styles have been used. A series of reinforced precast concrete troughs are fitted together to form pool and weir fishways. A system of integrating the fishway with precast foundation blocks eliminates the need for cast-in-place concrete and assures accurate grade control. The capability of available lifting equipment limits the size of the troughs; units as large as 20 feet long 4 feet wide and 3.6 feet deep have been used. The joints between units are tapered to fit closely and self-align; keyways have not been successful. A commercial tar impregnated compression seal is used between the concrete units. Wood stoplogs are installed in guides in the concrete.

The second style uses separate precast wall and floor units that are bolted together to form a pool and weir fishway. This is not as an efficient means of construction as the troughs and is used only when required by lack of access of appropriate heavy equipment. The precast units are typically 4 feet by 5.8 feet and 5 inches thick. They can be skidded into sites with hand equipment. A commercial product similar to this in Japan uses this concept to build large formal fishways.

The third style of precast concrete fishway consists of drop structures that are fabricated in one piece complete with wing walls, cutoff wall and plunge pool. They have been used to control the grade and provide fish passage in small relocated streams.

These designs are obviously limited by the weight of the units to be installed. Since the fishway pool volumes are limited, they have an inherent limited use confined to small or spring fed streams with consistent flow.

__Laminated Beam Weirs__ Narrow bedrock channels often present difficult construction and maintenance problems. They are often remote, difficult to access for crews, materials and equipment and difficult to dewater for construction. Laminated beam weirs are intended for these situations. They can be delivered and installed entirely by labor crews.

Appropriate anchor points are chosen in the rock walls of a narrow ravine. The channel floor and walls are frequently shaped by minor blasting as necessary to provide a reasonably smooth face of sound rock.

Guides are attached to the channel walls with rock bolts; the guides are long enough that the lowest anchor bolts are just above the waterline during low flow. The submerged portion of the guide is cantilevered below this and supported by a single shear pin anchored vertically in the channel floor near the center of the weir. Milled, untreated, 4 by 6 inch fir timbers are each trimmed to appropriate lengths to fit into the guides and tight to each rock wall. They are stacked one on another and connected with spikes. Twelve weirs of this style have been installed in the last

5 years. Of those, 2 failed; they, and a third that survived, were just upstream of a railroad trestle that was removed in an emergency due to a massive debris jam that formed during the extreme floods of November, 1990.

"Ecology" Blocks Precast rectangular blocks and especially Jersey median barriers are not suitable for fish passage sills. They are difficult to set with enough precision, they scour, settle and roll and gaps open that do not seal. They are generally ugly.

A concept of precast has been designed that would correct the deficiencies of ecology blocks. Precast cantilevered walls could be placed to form an arch structure. The arch shape would prevent differential settling.

Gabions Gabions are not a good fish passage device. They are unstable, deteriorate and are easily damaged. A benefit often stated of gabions is the possibility of using locally available stream gravel and cobble for fill. Fill of this type is like trying to stack marbles; the gabion deforms and quickly loses its intended shape. It may also roll as it deforms. Galvanized gabion wires do not withstand the erosion of bed material wear. With only slight bedload abrasion, I have observed gabion wires to fail in three years in a stream (Chico Creek, Puget Sound trib) that is considered moderately corrosive. Debris easily snags gabions either breaking them or distorting the wire fabric.

Fish passage conditions over gabions is often poor. They are difficult to seal and keep sealed. They act as a shallow weir and require a rigid notch imbedded into the crest for fish passage.

The only good use of gabions for fish passage that I am aware of is as a foundation upon which log or timber weirs can be constructed. They should not be exposed to the bed of the stream.

FISH PASSAGE AT ROAD CULVERTS

CULVERT BARRIERS

Migrating salmon and steelhead need not be hindered by road crossing culverts. WDFW is currently (1994) evaluating fish passage at all Washington state highway culverts. They are finding that of about 1200 culverts 27% are barriers. About half of the barriers are complete, the other half are partial or temporal barriers.

This chapter describes the design process and characteristics of passable culverts and remedies for those that aren't.

Salmon and steelhead commonly pass through unlighted culverts up to a mile in length but they often delay when approaching a culvert due to the change in hydraulic and/or light conditions.

Fish passage barriers at road culverts are created by several conditions. Culverts are uniform and efficient for water passage; they do not have the roughness and variability of stream channels and therefore do not dissipate energy as readily. The concentration of energy in the form of velocity, turbulence or downstream channel scour are the most prevalent blockages.

Culverts are a rigid boundary set into a dynamic stream environment. As the natural stream channel changes, especially with changes in hydrology due to land use changes, the culvert is not able to accomodate.

Proper design for fish passage may require a bridge where a culvert would otherwise be adequate.

There are five common conditions at culverts that create migration barriers:

- excess drop at culvert outlet;
- high velocity within culvert barrel;
- inadequate depth within culvert barrel;
- high velocity and/or turbulence at culvert inlet;
- turbulence within the culvert;
- debris accumulation at culvert inlet.

Culvert barriers are the result of design, improper installation, inadequate maintenance, or subsequent channel changes. They are very often the result of degrading channels that leave the culvert perched. Changes in hydrology due to urbanization are a primary reason for degrading channels. Barriers are also caused by scour pool development at the culvert outlet. The scour pool may be good habitat in itself but it moves the backwater control of the elevation further downstream and therefore to a lower elevation. Culverts with large scour pools are also likely a velocity barrier at high flows.

Many culvert barriers are not apparent during low and normal stream flows. Culverts must be analyzed at both the low and high fish passage design flows. Definition and selection of design flows are discussed in the hydrology section.

NEW CULVERT DESIGN

When considering only culvert capacity, culverts are normally designed with respect to the direction of water flow; from the upstream going downstream. By this method, the culvert is designed with inlet control for a design flow capacity.

The design process for fish passage must be reversed from this typical engineering orientation. The designer must think like a fish. The design first proceeds in the direction of the fish passage; from downstream going upstream. Culverts designed for fish passage normally result in outlet control condition at all fish passage flows. The usual inlet control analysis must then be done to verify adequate culvert capacity for the high structural flow.

Inlet control is similar to the design of an orifice; the maximum limit of upstream backwater determines the maximum allowable culvert inlet head loss and therefore the size of the culvert (orifice). Inlet control occurs when the flow at the inlet is supercritical. Outlet control is an open channel hydraulic analysis. Start with the water surface in the channel downstream of the culvert and calculate the backwater upstream through the culvert to determine its size.

Culvert design must simultaneously consider the hydraulic effects of culvert size, slope, material and elevation to create depths, velocities and a hydraulic profile suitable for fish passage. The following sequence is suggested for culvert design with fish passage:

- **Fish passage requirements.** Determine species and sizes and swimming capabilities of fish requiring passage.
- **Length of culvert.** Find the culvert length based on geometry of the road fill.
- **Species and age determine velocity criteria.** Actual allowable velocity and depth depends on species and length of culvert.
- **Hydrology.** Fish passage design criteria must be satisfied at the fish passage design flow.
- **Control velocity and depth by size, velocity, slope.** Find size, shape, roughness and slope of culvert to satisfy velocity criteria assuming open channel flow. Verify that the flow is subcritical at all fish passage flows.

- **Channel backwater depth.** Calculate the backwater elevation at the culvert outlet for low and high fish passage design flow conditions.

- **Match culvert to backwater depth.** Set the culvert elevation so the low and high flow channel backwater elevations are at least as high as the water surface in the culvert at those flows.

- **Correct channel profile.** If necessary the upstream and/or downstream channel profiles may have to be corrected to set the culvert.

- **Check high flow capacity.** Verify that flood flow capacity of the culvert is adequate.

Several iterations of parts of the design sequence are required to get the optimum design.

The suggested design sequence uses normal depth calculations in the culvert. This is a conservative design because it does not account for a backwater that will increase the depth.

Fish passage criteria will usually control culvert design; flood passage criteria are normally less stringent. The Washington Department of Fish and Wildlife fish passage regulations and the Alaska Department of Fish and Game (ADF&G) fish passage standards for new culvert installations are attached as examples as Appendix B. These criteria were developed in conjunction with the respective states' departments of transportation.

<u>Velocity</u> Culverts designed for fish passage should allow fish to pass at their prolonged swimming speed (See the Glossary for definitions of prolonged, sustained and burst swimming speeds). To make progress relative to the ground when swimming at this speed, the water velocity must be at a lower velocity. Travel swimming speed is defined as the swimming velocity relative to a fixed reference point such as the ground. Travel swimming speeds seem to be about 2.0 feet per second for all species of salmon. This conclusion is based on timing of coho and chinook swimming through culverts at prolonged swimming speeds, coho and chum swimming through roughened chutes and pinks swimming over an exposed apron in burst swimming efforts.

Sustained, prolonged and burst swimming speeds for numerous species have been listed in many references. Bell (1986) is a good reference. A common standard used in culvert design is the ADF&G curve which I have modified in Figure 11-1. It shows the maximum culvert length allowable for salmon, steelhead and grayling for any water velocity. It is based on a passage swimming speed of 2.0 feet per second. Most agency passage criteria, including WDFW and ADF&G, are more restrictive than this figure.

Nearly all culvert fish passage design standards stipulate maximum

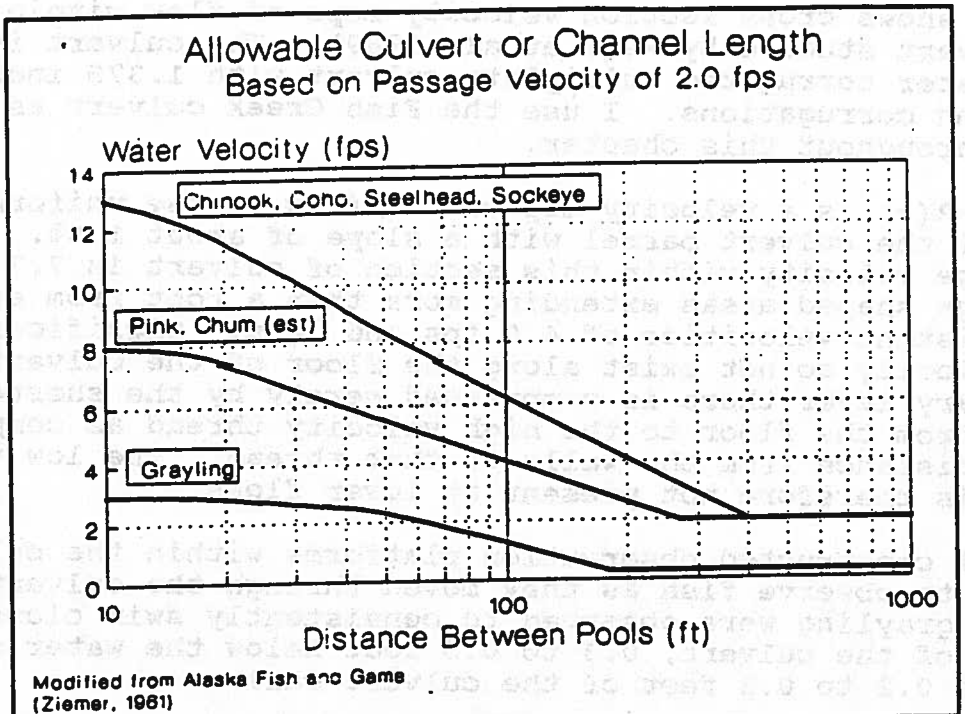


Figure 11-1. Maximum Allowable Culvert Length Based on Swimming Capabilities.

culvert velocities calculated as the average velocity in the cross section of the flow and assuming normal open channel flow throughout the culvert. Fish are very sensitive to variations in hydraulic conditions and they quickly find the lower velocity in the boundary layer of the culvert walls. ADF&G allows a waiver of the normal velocity criteria where special modifications have been made to the culvert design and accurate estimates of the velocity distribution within the culvert are available.

Figure 11-2 demonstrates the boundary layer effects in a rough wall

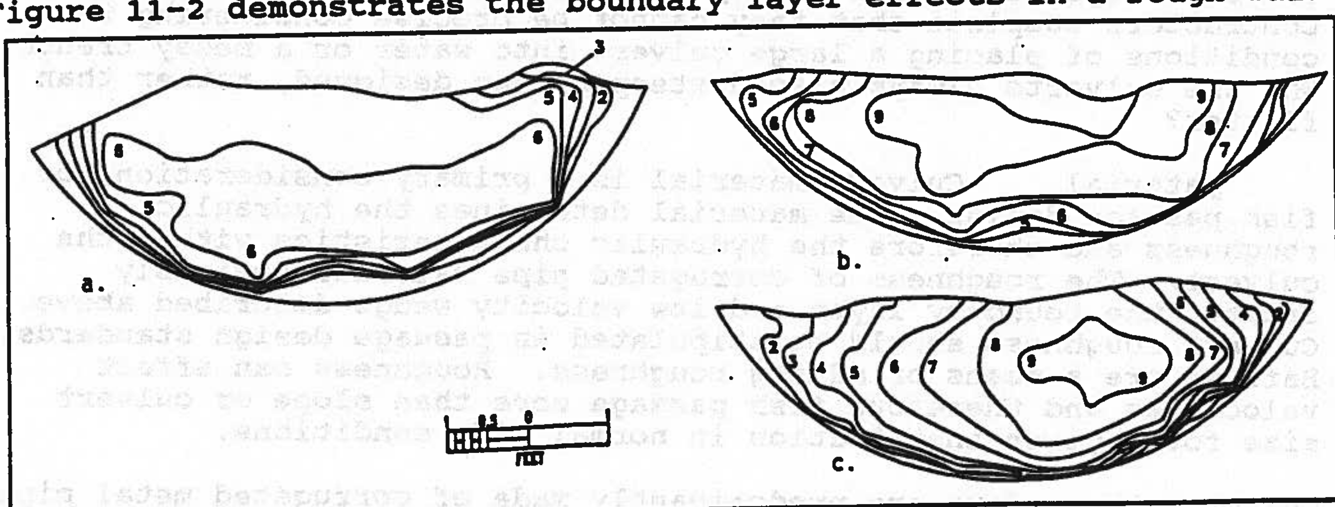


Figure 11-2. Fish Creek velocity maps; feet per second. Q = ±108 cfs; (a) at inlet, (b) 6 feet below inlet, (c) 30 feet below inlet. From Kane et al (1989).

pipe. It shows cross section velocity maps of flow within the Fish Creek culvert studied by Kane et al (1989). The culvert is a 9.6 foot diameter corrugated multiplate culvert with 1.375 inch deep by 6 inch long corrugations. I use the Fish Creek culvert as an example throughout this chapter.

Figure 11-2(c) is a velocity map of the flow in the uniform flow portion of the culvert barrel with a slope of about 1.5%. Though the average velocity within this section of culvert is 7.7 fps, large wedge shaped areas extending more than a foot from each wall have consistent velocities of 4.0 fps and less. Significant areas of low velocity do not exist along the floor of the culvert because the boundary layer there is compressed merely by the shorter distance from the floor to the high velocity thread as compared with the distance from the walls to that thread. The low velocity corridor is therefore not present at lower flows.

Kane et al constructed observation platforms within the culvert and were able to observe fish as they moved through the culvert. Migrating grayling were observed to consistently swim close to the left side of the culvert, 0.3 to 0.5 feet below the water surface and within 0.2 to 0.3 feet of the culvert wall.

<u>Size and Slope</u> Culvert size (diameter or equivalent) and slope partially control the mean velocity and velocity distribution within the culvert barrel and away from the influence of the inlet and outlet. They shouldn't be the only variables considered however; culvert roughness is discussed later.

Good design and careful construction are required to assure the intended elevation and gradient. Poorly designed culverts over compressible soils often end up bowed concave upwards due to the compression loading of the fill. The upper end of the culvert is therefore steeper than intended and the lower end may have an adverse slope and a drop to the downstream channel. The specified culvert slope is often not complied with during construction. Contractors complain that they cannot be precise considering the conditions of placing a large culvert into water or a messy trench. Why are culverts always placed steeper than designed, rather than flatter?

<u>Material</u> Culvert material is a primary consideration for fish passage design. The material determines the hydraulic roughness and therefore the hydraulic characteristics within the culvert. The roughness of corrugated pipe material obviously creates the boundary layer and low velocity wedge described above. Culvert roughness should be stipulated in passage design standards. Baffles are a means of adding roughness. Roughness can affect velocities and therefore fish passage more than slope or culvert size for a given installation in normal site conditions.

Culverts these days are predominantly made of corrugated metal pipe (CMP). The depth of the corrugation and the thickness of the material together establish the loading capacity of the culvert. Table 11-1 shows pipe materials that are commonly available and the

Table 11-1. Culvert products and roughnesses available.

Pipe Material	Corrugation Dimensions (d x w);(inches)	Roughness (Mannings n)	Range of Diameters Available (ft)
Concrete	na	.012 (ADOT)	3 - 8
Spiral Rib	3/4 @ 7-1/2"	.011 (WCHL)	1.25 - 8.5
Standard	1/2 x 2-2/3	.024 (ADOT)	1 - 8
	1 x 3	.028 (ADOT)	3 - 12
Plate	1-1/3 x 6 2 x 6	.029 .030 (ADOT)	4 - 20
Deep Core	4 x 15	?	6 - 32

size ranges and the roughness of each. Roughness is presented throughout this section as roughness coefficient "n" used in Manning formula:

$$Velocity = \frac{r^{2/3} x s^{1/2}}{n}$$

where r is the hydraulic radius in feet and s is the dimensionless slope.

There are two ways to evaluate culvert roughness relative to fish passage; average velocity and boundary layer velocity. The average velocity is simply the discharge divided by the area of the flow. This is the velocity specified in agency passage criteria. The boundary layer velocity is the velocity within the influence of the culvert wall.

Consider the boundary layer velocity at a point six inches from the wall. It is the edge of a zone that could likely be used by an adult fish and in agreement with the observations of Kane et al who observed Grayling consistently within 0.3 feet of the wall. For standard corrugated pipe (1/2 x 2-2/3" corrugations), the Corps of Engineers (1955) found the velocity at that point to be about 94% of the average velocity. For paved culverts with a Manning n of 0.013 the velocity was 98% of the average. The data presented by Kane et al on the other hand shows the velocity at that point to be about 78% of the average for a culvert with a Manning n of about .030 (1-3/8" x 6" corrugations).

As would be expected, the average as well as the boundary layer velocities of smooth pipe are substantially greater than for rough corrugated pipe. Table 11-2 demonstrates the hydraulic conditions within the Fish Creek culvert assuming roughnesses of three different corrugated pipe products. V_{ave} and V_{BL} are the average velocity and boundary layer velocity as described in the paragraph

Table 11-2. Culvert hydraulic conditions with various roughnesses; 114" culvert, 110 cfs.

Roughness (n)	V_{ave} (fps)	Depth (ft.)	V_{BL} (fps)
.012	14.7	1.6	14.4
.024	9.0	2.2	8.5
.030	7.7	2.4	6.0

above. Depth is the depth of the water. As described before, the roughness of the actual Fish Creek culvert is 0.030. The implications of the culvert roughness to fish passage are obvious. Fish passage is jeopardized by the smoother culverts whether they depend on the average velocity or the boundary layer velocity. Very little boundary layer, as measured by its variance from the average velocity, is created in the smoothest culvert.

Consider the effect of roughness compared to culvert slope. By the Manning formula, the velocity varies with the square root of the slope and inversely with the Mannning roughness coefficient. Consider the case of the Fish Creek culvert. As seen in Table 11-2, if a concrete pipe had been selected instead of the corrugated plate, the average velocity in the culvert would have been increased from 7.7 to 14.7 fps; a 91% increase. For the slope to have the same effect of increasing the average velocity by 91%, it would have to be increased from the as-built slope of 1.5% to 5.5%.

The spiral rib pipe listed in Table 11-1 is the latest and greatest from the industry and is being touted as the most efficient and cost effective culvert material design. There are many existing culverts that do not meet agency fish passage requirements but pass fish through the boundary layer corridor. If the industry reverts to smooth pipe similar to the concrete pipes used in the past, fish passage will generally be impaired.

In addition to the general hydraulic benefits of corrugated material, Kane et al (1989) observed small grayling (75 to 150 mm) swimming into and holding stationary within the recessed corrugations. These fish were able to swim through the culvert when the average pipe velocity was at least 5.75 fps.

<u>Shape</u> Elliptical and arch shapes are beneficial to fish passage. They have increasingly lower hydraulic grade lines by the fact that more area is available at a lower elevation in the culvert. The greater wetted perimeter at normal flows also produces a lower average velocity.

A depth of about 25% or more of the culvert diameter is needed to get enough of a low velocity boundary wedge to function for passage. At flows below that depth, the velocities shown at the floor of the culvert would approach those shown near the floor in Figure 11-2(c) but the low velocity wedges would not exist.

Circular, arch and elliptical pipes have increasingly less low velocity wedge corridors. Unless adequate flow to provide a low velocity wedge in a circular culvert is assured, arch, elliptical and circular culvert shapes are preferred in that order.

Minimum Depth of Flow Except for the wedge effect described above, depth of flow limitation is the minimum depth requirements at low flow that allow fish to swim. Depths of 0.8 feet for resident trout, and pink and chum salmon and 1.0 foot for chinook, coho, sockeye and steelhead are recommended. Flow at half these depths is usually passable in a burst swimming effort when the water velocity is low. The minimum design depth is calculated by the Manning equation at the low design flow.

Elevation Excess drop at the culvert outlet is caused by either a perched or an undersized culvert from which the flow must fall to the water surface of the downstream channel. The appropriate culvert elevation is identified by comparing the water surfaces of the culvert and the downstream channel at both the low and high design flows. Best fish passage design is for the two water surfaces to match at the culvert outlet assuming normal flow depth in the culvert. A drop in hydraulic grade of 1.0 foot or less may be allowed for adult salmon. The effect of the drop is to accelerate the flow near the culvert outlet to a velocity greater than the allowed average velocity within the culvert. ADF&G culvert criteria stipulate a maximum velocity at the culvert inlet and outlet as well as the normal velocity within the culvert. The allowable inlet and outlet velocities must not exceed a fish's burst swimming capability.

As a safety factor against a degrading channel or scour pool, culverts should be countersunk below their minimum required elevation. They should be placed below the natural stream grade or the grade established by the low flow hydraulic grade described above, whichever is lower, by at least 20% of the culvert diameter or rise. The natural grade for this situation is defined as the elevation of the downstream low flow hydraulic control. Use a downstream hydraulic control that is at least thirty feet downstream of the culvert. Any bed controls closer than that may be scoured out by the culvert discharge.

Flood flow capacity should be calculated using the area of the culvert above the countersunk portion since it may fill with bed material.

The potential for channel degrading during the life of the culvert should be considered. If the downstream channel degrades, the culvert may be left perched. A primary cause of channel degrading is due to changes in hydrology resulting from land use changes. The degrading may also be the natural evolution of the channel. Culverts in channels flowing north into the Strait of Juan de Fuca in Washington State are passage barriers that are universally a result of the channels naturally incising through glacial outwash material. Inspect the channel and other culverts in the vicinity to see if there are signs of channel degrading.

Bottomless culverts are corrugated metal arches mounted on steel or concrete foundations walls along each side. The floor of the culvert is a natural or constructed streambed. Bottomless culverts are therefore often stipulated for installation where fish passage and fish habitat mitigation are required.

Precast concrete bottomless culvert sections on concrete foundations are also available. Bottomless culverts are useful to fish passage because their stability depends on a rough natural bed that also insures fish passage. They are often recommended in channels that may degrade because a channel that degrades at a bottomless culvert becomes a structural issue to the owner rather than a passage barrier to fish. Structural issues seem to be resolved much sooner than passage barriers.

If sized properly a natural streambed will be maintained within the bottomless culvert. If undersized, the bed and footings will scour jeopardizing the culvert and fill. The bed material must be analyzed for stability considering the increased velocity the culvert is a channel constriction.

Substantial loads are applied to the footings of large bottomless culverts in high road fills. Conservative DOT's do not allow bottomless culverts for those reasons. There is no functional difference between a bottomless culvert and an oversized round culvert countersunk deeply into the streambed.

<u>Inlet</u> The culvert inlet is the upstream end; it is the exit for fish migrating upstream. Culvert inlet conditions are rarely given the consideration they deserve. Fish that pass through the difficult conditions of a long culvert must often exit through an hydraulic jump an accelerated contraction near the culvert inlet and an inlet drop upstream of it. Fish that expend their energy in passing the culvert can be exhausted and unable to exit the culvert. Since there is usually no resting area within the culvert, the fish will eventually drop back downstream and out of the culvert.

A culvert has one of two hydraulic characteristics at any flow; inlet control or outlet control. Inlet control occurs when the culvert barrel is able to carry more flow than the inlet will accept. Under outlet control, the capacity is determined by the culvert barrel and/or the backwater from the downstream channel. A fish passage design should be outlet control at least up to the fish passage design flow. It may become inlet control at higher flows.

Two hydraulic phenomenon occur at the culvert inlet when it is designed with inlet control. A hydraulic contraction creates a drawdown of the water surface and a plume with velocities much greater than the average velocity based on normal open channel flow. The contraction is caused by the inefficiency of the inlet Inefficient contractions often produce supercritical flow at the inlet; a hydraulic jump then occurs within the confines of the culvert. Larger fish may be able to power through jump but smaller fish are likely blocked by the turbulence of the hydraulic jump.

Kane et al (1989) did careful velocity mapping throughout the Fish Creek culvert and show the effects of the inlet contraction. At the inlet, Figure 11-2(a), looking downstream, the flow is fairly uniform with an average velocity of 5.6 fps. The water surface is distorted as a result of the flow approaching the culvert at a skew from the left side. Flow separates from the left side of the culvert at the inlet eliminating any low velocity wedge and piles up on the right side. At six feet below the inlet, Figure 11-2(b), the jet has contracted and produces an average velocity of 9.0 fps with very slight low velocity wedges and still a distribution of flow toward the right wall. This distribution of flow is shown again in Figure 11-2(c) at thirty feet downstream of the inlet where the flow approaches normal flow conditions and an average velocity of 7.5 fps. The skewed flow distribution persisted in this case the entire length of the 60 foot long culvert.

This example of Fish Creek also demonstrates that the alignment of the upstream channel to the culvert is important in marginal passage situations. A passage corridor exists the entire length of the Fish Creek culvert along the left wall but is eliminated near the inlet because of the approach alignment. Fish that pass the entire length of the culvert along the left wall may not be able to exit. The Fish Creek culvert is aligned at less than 30° to the approaching flow.

The inlet loss coefficient is a measure of the efficiency of the inlet and the extent of the flow contraction. They can be used to calculate the entrance

$$h_e = K_e \times \frac{V^2}{2 \times g} \qquad (11\text{-}2)$$

loss by Equation (11-2) where k_e is the inlet coefficient, V is the average velocity and g is the acceleration of gravity. Inlet coefficients are summarized in Table 11-3 for various culvert inlet configurations. The inlet coeefiecients are for relative depths (upstream depth divided by culvert diameter) of about 1.2. Inlet coefficients should not normally exceed 0.7 for fish passage. They should not exceed 0.5 where passage conditions within the culvert are

Table 11-3. Culvert Inlet Coefficients.

Type of Inlet	Inlet Loss Coefficient; K_e
Projecting from fill	0.9
Miter to conform to fill slope	0.7
Headwall; Square edge	0.5
No bed	0.38
50% bed	0.73
Wingwalls 10° to 30° to barrel	0.5
Rounded Headwall (R=1/12 Dia)	0.2
Wingwalls 30° to 75° to barrel	0.2

marginal and 0.2 where juvenile passage is desired. The inlet coefficient is a measure of the efficiency of the flow contraction from the stream channel to the culvert. An oversized culvert that does not contract the flow has an inlet coefficient of zero.

A thorough hydraulic analysis of inlet conditions is provided by Jordon and Carlson (1987). They used hydaulic models to develop inlet contraction coefficients for a range of relative depths and depths of streambed within the culvert. As shown in Table 11-3 the contraction coefficient is increased from .38 to .73 when the relative depth of the bed in the culvert is increased from nothing (no bed) to 50% (50% bed) of the culvert diameter. Though the bed itself creates a more efficient inlet, the change in shape of the entrance and the roughness of the bed likely cause the increase in the contraction coefficient.

An added difficulty at culvert inlets is the inlet drop. The head loss during inlet control flow conditions creates a reservoir effect upstream that promotes deposition. At lower flows the deposit creates a drop right at the inlet that can become a migration barrier. Kane and Wellen (1985) observed about 10 percent of the 200 culverts they examined in Alaska had significant drops at the inlet.

Flared and mitered wingwalls are recommended for culvert installations especially where the alignment of flow approaching the culvert is greater than 30° to the culvert alignment.

ADG&G culvert criteria acknowledges fish passage restrictions at the culvert inlet and outlet. A specific velocity is allowed at those locations.

<u>Other Design Considerations</u> Baffles are not recommended in new culvert installations. Fish passage through baffled culverts has not been rigorously and biologically tested and shown to be equal to culverts designed with standard design criteria. A number of baffle designs have been tested; average velocities have been measured that imply possible successful fish passage. Turbulence created by the baffles has not been measured however. Baffles do have an application in the design of barrier remedies because of their relative cost compared to culvert replacement.

Culvert baffles reduce the culvert capacity and snag debris. Debris can not be removed as it should be during some fish passage migration seasons because of high flow conditions. During the fish passage season, road and highway agencies usually cannot be expected to monitor and adequately maintain many baffled culverts. If appropriate monitoring and maintenance were included in a life cycle cost analysis of a baffled culvert compared to a larger culvert without baffles, the larger culvert is likely less costly.

Debris at culverts is a never-ending problem for fish passage. Fortunately it is also a problem for flood passage and therefore is usually taken care of by the culvert owner. Debris racks should not be allowed on culverts; if debris is a perceived issue, the culvert size should be increased appropriately. Where the need for debris racks prevails over common sense, they should be constructed entirely above the normal water surface. Leave an open space below the rack for normal flow conditions and fish passage. Floating debris will be caught by the rack at higher flows and water surfaces.

Avoid culvert installations in potentially degrading channels or compensate by over-sizing and counter-sinking the culvert. Locate the culvert discharge 20 to 30 feet upstream from a natural stream bed control.

A scour pool may develop at the culvert outlet due to poor energy dissipation through the culvert. The pool may be important to the protection of the downstream channel. Without it, the high energy of the flow from the culvert will be passed downstream and dissipated elsewhere. The energy may be dissipated by bed or bank scour. The pool is also beneficial as a holding area for fish approaching the culvert. In channels with rock or cemented bed, a plunge pool should be excavated to about two feet below the culvert invert and with a length of about two culvert diameters.

BAFFLES

A variety of baffle styles can be used to reduce the average velocity and/or increase the water depth within the culvert barrel. Baffles are not the preferred alternative but are the only reasonable option in extreme situations. They require substantial maintenance to keep them clean of debris and to replace them as they fail. Maintenance is usually impossible during high flow fish passage seasons so passage is lost for at least part of a season when they fail or plug.

<u>Baffle Hydraulics</u> Before you start the design of a baffle system, you must determine whether you need weirs or baffles. Most baffles act as weirs at low flow and as increased roughness at high flows.

True baffles act together as roughness elements for high flow and have a low profile and close spacing. They reduce average and boundary layer velocities. They provide a consistent rough boundary layer without creating so much turbulence that the boundary layer does not persist until the next downstream baffle. Flow over baffles is streaming. Baffles can only be designed with an understanding of the concepts of streaming and plunging flow regimes which are discussed in Chapter 5.

Baffles are often inappropriately designed as weirs. Weirs are discreet hydraulic elements where energy dissipation is concentrated. When designed as weirs, the fishway pool volume criteria, as described in the fishway section of this report, must be complied with. Otherwise the turbulence and potential hydraulic jump associated with the weir become a fish barrier instead of the velocity that is being mitigated. That fishway pool volume criteria effectively precludes the use of anything but very deep weirs inside of oversized culverts. Weirs are high and far apart. The spacing of weirs cannot be increased substantially to get the required volume required. The energy dissipation volume must be provided within about ten feet of a weir. Otherwise, regardless of the overall volume, the turbulence near a weir precludes passage to it or over it. Weirs within culverts may apply well for resident species that have a lower fish passage design flow than anadromous species.

An ideal baffle system is one that is designed as a hydraulic roughness where energy is continously dissipated. Baffles as roughness elements would be the most likely to provide passage for weak fish and juveniles, not snag debris, be virtually free of maintenance, and minimize the reduction in culvert capacity. Corrugated plates could be installed as a sleeve in a smooth-walled culvert. They would be bolted into the culvert invert and would extend up the walls far enough such that the sleeve is filled by the high passage design flow.

<u>Baffle Design</u> Three styles of baffles that are often installed in box, arch and ciruclar culverts are shown in Figure 11-3. The height of the weirs is typically 0.5 to 1.5 feet and depends on the baffle spacing, roughness required, minimum low flow depth and reduced culvert capacity allowable. Weir baffles are not recommended for slopes over 1.0% and are best used to retain bed material in the culvert.

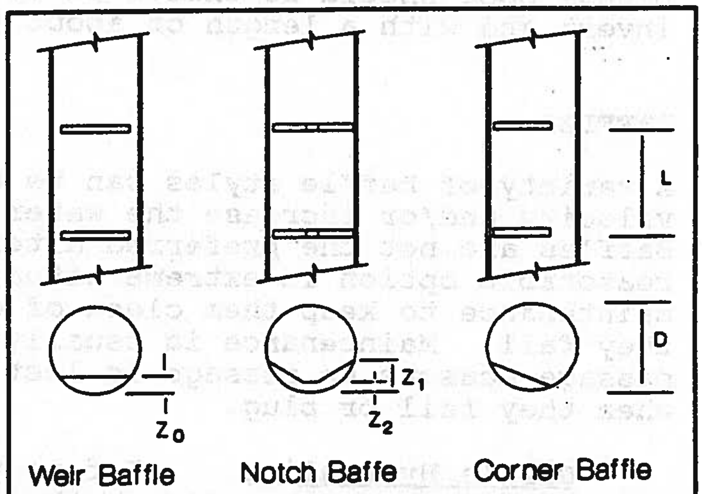


Figure 11-3. Culvert baffle styles.

The notch baffle is especially useful in arch plate culverts. The central weir height Z_2 can be from zero, in which case there are two independent corner baffles, to several feet high. The length of the central weir can be several feet. The notch baffle can be applied to culverts with slopes of 2.5 to 5.0% and have been installed successfully in short culverts as steep as 12% but act as weirs at such high slopes.

Corner baffles apply to culverts with slopes in the range of 1.0 to 2.5%. They are intended to provide wall roughness with minimum blockage of debris.

Several studies by Rajaratnam and Katopodis (with others) referenced here are good guides for determining average velocities over various baffle arrangements.

Careful consideration of flow conditions is needed for a good baffle design; the flow over baffles is often supercritical. The supercritical flow results in hydraulic jumps or standing waves between baffles. In that case, a velocity passage barrier is converted to a turbulence barrier. An objective of a good design is to maintain subcritical depth over the baffles; hence the baffles act as roughness elements rather than discreet weirs. Supercritical flow may develop at any flow depending on the culvert and baffle characteristics. Flow conditions for a range of flows should be investigated to confirm that the flow remains subcritical.

The offset baffle system was developed by Webb and McKinley (1956) through a scale model study; it is shown in Figure 11-4. The spacing of the baffles for hydraulic conditions is L=1.12B; the width of the slot is b=0.125B and the offset a=0.26B. They can be placed in box, circular or arch culverts. The dimensions given by Webb and McKinley are intended for the best hydraulic conditions.

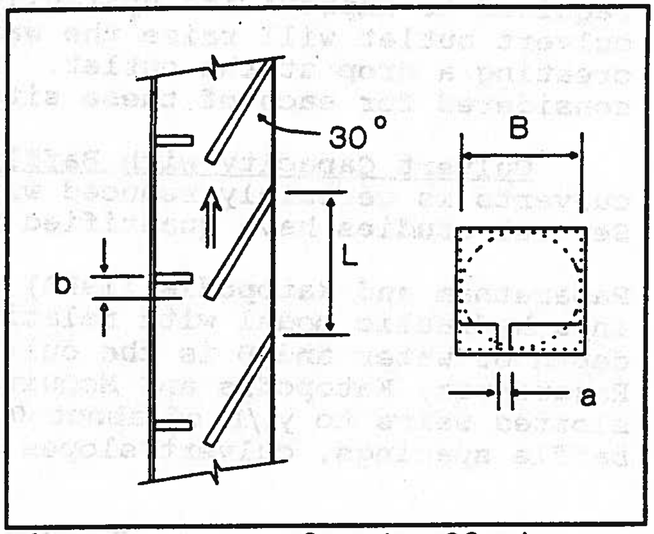


Figure 11-4. Culvert offset baffle design.

Practical considerations must prevail. Design the slot width with debris in mind; the prescribed notch width is so small, especially in culverts less than 6 feet wide, it consistently collects debris. The spacing should be set with bed material in mind; a hydraulic drop of about 0.2 feet across the offset baffles is necessary to keep gravel from filling the baffles. The offset baffle applies to culvert slopes of 2.5 to 5.0%. A recommended modification of the offset baffle system in flatter culverts, 1.0 to 2.5% slopes, is to shorten or eliminate the short stub baffle.

A hydraulic benefit of the offset baffle, with or without the stub baffle, is that it sets up a spiral circulation through the culvert. The flow is deflected by the angled baffle. The deflection carries debris and bed material away from the slot and tends to lift the material from the apex of the long baffle.

None of these baffle styles is intended to pass juvenile fish. They may in fact block juvenile passage by creating large scale turbulence relative to the size of the fish. A baffle system is less likely to block small fish if it is installed on a single wall so the boundary layer and passage corridor along the opposite wall is not disturbed.

For the same reason I suggest that baffles should not be placed on alternating sides of a culvert. If they are, fish and debris must cross the culvert at each baffle reducing their likelihood of successful passage. Alternating baffles also cause the flow to slosh from side to side at moderate to high flows leaving no stable or still water areas.

Care should be taken in locating baffles near each end of the culvert. Baffles installed in the area of the culvert inlet contraction may significantly reduce the culvert capacity when it is in inlet control condition and will likely confound fish passage in that area at lower flows. The upstream baffle should be placed about one culvert diameter downstream of the inlet and should be high enough to ensure subcritical flow at the inlet at the high design flow. A modification to the culvert inlet may also be required to improve its hydraulic efficiency. Baffles near the culvert outlet will raise the water surface within the culvert creating a drop at the outlet. Special baffle designs should be considered for each of these situations.

<u>Culvert Capacity with Baffles</u> The hydraulic capacity of culverts is certainly reduced with the addition of baffles. Several studies have quantified that loss of capacity.

Rajaratnam and Katopodis (1990) studied weir baffles in round pipes in a hydraulic model with relative depths (y_o/D, where y_o is the depth of water and D is the culvert diameter) up to 0.80. Rajaratnam, Katopodis and McQuitty (1989) did similar work with slotted weirs to y_o/D of about 0.7. A range of baffle heights, baffle spacings, culvert slopes and flows were tested. Non-

$$Q = C(y_o/D)^a \sqrt{gS_oD^5} \tag{11-3}$$

Table 11-4. Slotted weir and weir baffle hydraulic characteristics.

	z_o	L	b_o	C	a	Limits
SWB-1	0.15D	0.6D	0.1D	9.2	3.0	$y_0/D < 0.6$
SWB-2	0.15D	0.3D	0.1D	9.2	3.0	$y_0/D < 0.6$
SWB-3	0.15D	1.2D	0.1D	10.9	3.1	$y_0/D < 0.5$
SWB-4	0.15D	2.4D	0.1D	12.7	3.1	$y_0/D < 0.6$
SWB-5	0.10D	0.6D	0.1D	11.4	2.9	$y_0/D < 0.6$
SWB-6	0.10D	1.2D	0.1D	12.4	3.0	$y_0/D < 0.5$
WB-1	0.15D	0.6D	na	5.39	2.43	$0.25 \geq y_0/D < 0.8$
WB-2	0.15D	1.2D	na	6.6	2.62	$0.35 \geq y_0/D < 0.8$
WB-3	0.10D	0.6D	na	8.62	2.53	$0.35 \geq y_0/D < 0.8$
WB-4	0.10D	1.2D	na	9.0	2.36	$0.20 \geq y_0/D < 0.8$

dimensional flow equations were developed by the authors for all baffle styles tested. Those equations are simplified here to the form of Equation (11-3). In the equation, C and a are a coefficient and exponent that depend on the baffle configuration and are determined experimentally. Q is the discharge, g is the acceleration due to gravity, and S_o is the non-dimensional slope of the culvert. The dimension z_o is the height of the baffle as shown in Figure 11-3 and b_o is the width of a full-depth notch cut in the center of the weir baffle to create a slotted weir baffle.

The dimensions and their respective coefficients and exponents found are provided in Table 11-4. The first column contains the slotted weir baffle (SWB) and weir baffle (WB) style labels used by the authors. Differences in the styles are represented by the dimensions in the next three columns of the table. z_o is the relative height of the baffles and L is the distance between them as shown in Figure 11-3.

The limits shown in the table are the limits of experimental data or valid correlation for the coefficients and exponents. Other additional correlations were developed for the weir baffles at lower depths. There is no reason to doubt that the equations are reasonable for depths up to nearly full culvert flow.

Similar work has been done on offset baffles (Rajaratnam, et al, 1988), and other weir and baffle shapes in round culverts (Rajaratnam, et al, 1990).

Similar work for weir baffles in square box culverts was done by Shoemaker (1956). Internal culvert friction loss and entrance losses were calculated from model studies. Shoemaker used the

$$h_f = f\frac{L_c V^2}{D2g} \qquad \textbf{(11-4)}$$

Darcy-Weisbach friction equation, Equation (11-4), as a hypothetical model for the culverts with baffles where h_f is loss in head caused by friction, L_c the length of conduit, D the diameter of pipe (4 times the hydraulic radius of noncircular pipes), and $V^2/2g$ the gross section velocity head in the conduit.

The baffles tested were full-width weir baffles with a rounded leading edge with a radius equal to one tenth of the culvert height. Baffle heights of 0.10, 0.20 and 0.30 times the culvert height and spacings of 1.0, 2.0 and 4.0 times the culvert height were studied.

The variation of the Darcy-Weisbach friction factor is shown in Figure 11-5 from Shoemaker where Z_o is the baffle height and L is the baffle spacing.

Friction factors for close baffle spacings should be used cautiously. As would be expected, as the baffle spacing approaches zero, the baffle roughness actually decreases and the cross sectional area of the culvert becomes the area of the culvert remaining above the baffles. Shoemaker, in his calculation of velocity head, used the gross culvert area.

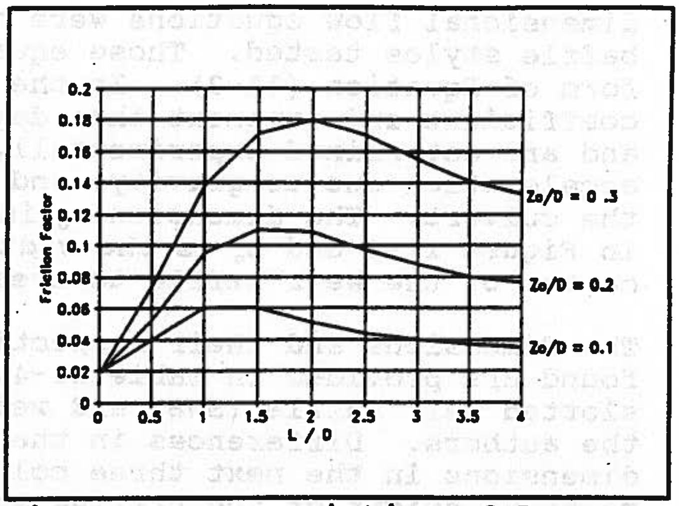


Figure 11-5. Variation of Darcy-Weisbach friction factor with baffle spacing.

The second analysis by Shoemaker (1956) provided means for evaluation of other energy components characterizing the flow through a culvert. The assumption is made that entrance and outlet losses, as well as the friction losses, are proportional to velocity head. With these assumptions the energy equation for flow through the culvert can be written as

$$HW = \left(K_e + C_e + f\frac{L_c}{D}\right)\frac{V^2}{2g} + P - S_o L_c \qquad (11\text{-}5)$$

where HW is the headwater elevation above the invert at the culvert entrance, K_e and C_e are culvert entrance and exit head loss coefficients, P is the outlet water surface elevation, and S_o is the slope of the culvert. Other parameters are as defined above. Shoemaker describes a reasonable approximation of P as the distance from the culvert invert to the center of the flow in the opening above a baffle.

Combined values of the head loss coefficients K_e and C_e were derived by Shoemaker as a single coefficient C_a and is shown in Figure 11-6 as a function of baffle spacing and height.

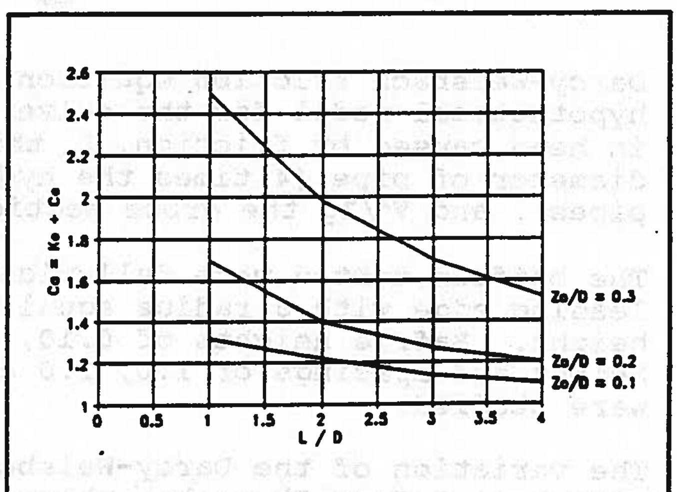


Figure 11-6. Energy coefficients for various baffle arrangements.

In Shoemaker's model, the culvert entrance and exit had aprons extending 2.5 times the culvert width, wing walls flared at 34° from the culvert line and mitered at a 2:1 slope. The upstream-most baffle was consistently placed one culvert height downstream from the culvert entrance and the downstream-most baffle was placed at the edge of the apron.

The effect of the actual culvert wall roughness should not be ignored at high flows. Sayre and Albertson (1963) conducted model studies and compiled other data on various baffle and roughness arrangements in rectangular channels. They related resistance to baffle density which accounts for both baffle spacing and size. The studies were done with baffles in smooth wall pipes though the off-set baffles were also tested in a rough walled pipe. For any given depth y_o/D above about 0.20, the flow in the rough pipe was about 20% less than that in the smooth pipe.

<u>Baffle Installation</u> Baffles in concrete culverts can be made of wood timbers, steel plate or precast concrete. Wood baffles have lasted about 15 years in Ennis Creek, a high gradient stream with a heavy rate of cobble and boulder bed load. Bent steel plates work well with one leg bolted to the floor and pointing downstream.

Expansion ring anchors work well in round pipes and can be installed without diverting flow of the culvert. Rods are rolled to the shape of the inside of the culvert interior and are attached to an anchor plate. The rod and anchor plate are fixed into the culvert by expanding the rod into the recess of a culvert wall corrugation. This is done by tightening a nut on one end of the rod against a sleeve attached to the other end of the rod. Once the rod and anchor plate are secured, the baffle is bolted to the anchor plate. This system will also work in smooth culverts. A set of four shear bolts must first be anchored into the culvert wall; the expansion ring is then installed against the upstream side of the shear bolts.

Baffles in new culverts should be welded in place before the culvert is galvanized. Bolt anchor systems for existing circular or arch culverts need further work. Anchor bolt and J-bolt systems have worked.

CULVERT BARRIER REMEDIES

How are culvert barriers fixed? The reality is that few are fixed, especially the more difficult and expensive ones. Regulatory agencies ar reluctant to pursue legal means for passage remedies. Passage remedies are costly and enforcement is essentially voluntary in the west coast states and British Columbia.

<u>Removal / Replacement</u> Culvert removal or replacement is often necessary to satisfy fish passage criteria and should not be ignored as a possibility. It is too easy to lessen passage criteria to avoid a more costly solution. Policy guidance is essential in this situation.

In the case of culverts that are lowered or replaced on a flatter slope, the upstream channel will be left perched above the culvert. That channel can either be allowed regrade naturally or in a gradual controlled process. It can also be supported to prevent regrading. Natural regrade of up to two feet is often accepted.

The impacts of the resulting channel regrade is considered site by site and depend on height of the perched bed, habitat present, bed material and subgrade soil characteristics and channel gradient. The extent of the channel regrade is predictable. Channel profiles and bed and streambank conditions for 500 to 1000 feet upstream are necessary for planning in this situation.

Impacts extend upstream in the regrade of the channel and downstream in sediment deposition in habitats and reduction of flood capacity. Channels with shallow gravel beds over deep sand or silt subgrades will regrade extensively mobilizing substantial bed material and likely bank material. Nick point passage barriers often develop in the upstream degraded bed.

To mitigate the regrade impacts, bed control sills may be constructed in the upstream channel. Arch shaped rock sills or an armored channel bed may be placed to control the rate of regrade with the intention that they will gradually fail. As they fail, the bed regrades and the placed rock partially armors the new bed in a steeper gradient. The process is more gradual and less destructive than a sudden regrade.

<u>Steepened Downstream Channel</u> The channel below the culvert can be steepened to eliminate the drop from a culvert. This is often accomplished with streambed log controls as described in the section on tributary passage. They are usually built as one foot steps and can be built as vertical log walls when placed high above the original streambed.

Multiple year projects can designed for situations in which substantial elevations gain is required. Several log dams are built the first year and the intervals between the dams are allowed to fill naturally with bed material and consolidate. The project is completed by placing additional log controls at the desired spacings and grade between the original set of controls.

Riprap bed controls have been constructed downstream of many culverts in the past. They are acceptable only in low gradient channels when a rise of less than a foot is required. Higher rock weirs tend to wash out over a period of time.

A steep roughened rock channel can be constructed below a culvert. It must start at least 30 feet downstream of the culvert. There is a discussion of roughened channel fishways in the tributary passage section of this report.

<u>Fishway</u> Several pool type fishways can be applied to culvert passage situations. Pool and weir fishways built adjacent to the channel downstream of the culvert are common. A culvert apron can function as flow control for the fishway with the fishway water supply being controlled by a slot. Either pool and chute or the offset pool and chute are good choices spanning the entire channel and in line with the culvert. Fishways can also be constructed culverts though the culvert must be greatly oversized in order for the fishway to comply with standard energy dissipation criteria.

Streams with culverts normally do not have adequate water at low flow to consider the use of orifices.

Fishways are discussed in detail in Chapter 5 of this paper.

UPSTREAM JUVENILE PASSAGE

In recent years fisheries scientists have documented the benefit of the upstream movement of coho salmon fry and fingerlings from river mainstems into sheltered off-channel ponds. The prevalent salmon species using these habitats along the Washington coast and Puget Sound is coho salmon. Fry (40 to 60 mm) migrate during May and June and fingerlings (80 to 100 mm) migrate from October through December (King, 1990 and Peterson, 1982).

Steepened rock channels, fishways and streambed controls have been constructed specifically for upstream juvenile salmon passage. Passage of juvenile fish through culverts was discussed in previous sections. Upstream juvenile fish passage is provided through most adult pool and weir fishways if adequate still water is provided in the pools. The biggest difficulty in that case is the normal lack of flow control in small tributary fishways. Flow from off-channel habitat is often supplied by spring or groundwater and is very stable compared to river mainstem flows. Construction cost for nine juvenile passage projects built by Washington Department of Fisheries recently varied from $1000 to $6000 (1993 dollars) per foot of rise. Construction costs increase substantially as the design flow increases or the total head differential exceeds about three feet (Powers, 1993).

Most structures designed to pass juvenile fish also pass adult fish but passage can often be a problem for adults because of inadequate depth.

Most of the fishway styles described in Section 10, Tributary Fish Passage Design are effective for juvenile passage.

Fryway To provide juvenile coho passage into sloughs and beaver ponds a portable, inexpensive fryway has been developed, tested and constructed in Alaska and Washington states. Providing dependable upstream passage for fry into beaver ponds must consider unique conditions. The pond water surface can vary considerably with changes in flow and beaver activity. The limited swimming ability of fry limit hydraulic conditions for passage. Continuing beaver activity can plug or block open fishways. For an effective fry passage program, fry must be distributed into many isolated ponds; a large number of remote installations are necessary.

The fryway was tested with a prototype installation in April through May, 1987 in Washington State. Two parallel fryways with different test conditions were installed to temporarily replace a pool and weir fishway through which coho fry were known to pass. Traps were placed at the upstream end of each fryway; fish successfully passing each fryway were counted daily and moved into the pond above the fryways. The more successful of the two was used in the next test as a control to evaluate other configuration changes such as baffle spacing and fryway slope. The parallel evaluation was intended to compare both attraction to the fryways and passage through them.

The fryway tested is shown schematically in Figure 12-1. The fishway portion of the fryway is a 12 inch diameter PVC pipe with a series of vee-notch weirs inside. One end of the pipe is submerged in the pond and attached with a flexible coupling to an entrance pipe that passes through the dam and is submerged in the tailwater of the dam. The upstream end is attached to flotation tanks and includes an elbow so the exit is submerged in the pond. The flotation is designed to raise and lower the fryway and exit and maintain a constant flow of 0.5 cfs through the fryway. The exit tested has successfully eliminated plugging by beavers; it consists of a PVC pipe similar to the fryway with many two inch holes distributed throughout its length and circumference. This fryway is similar to a design developed independently in British Columbia, Canada, (Smallwood, n.d.).

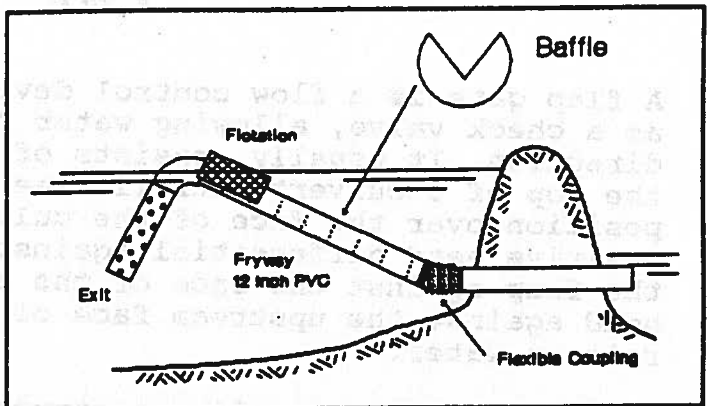


Figure 12-1. WDF fryway cross section.

Two additional fryways were installed in 1988 and 1990 in Washington State. Four fryways have been with a similar design were constructed and evaluated in Cordova Ranger District of the Chugash National Forest in Alaska (Vanessa Alao-MacLeod, Chugach National Forest, pers. comm.). These fryways are currently being evaluated by trapping fish in minnow traps in the beaver ponds.

The fryways tested in Washington successfully passed juvenile coho as small as 68 mm at slopes up to 35%. The fryway slope of 25% appeared the most practical; it passed 2.7 times as many fry as the 35% slope though only 10% less than the 15% installation.

Adult coho successfully moved upstream through the fryway during the initial testing. There is a risk of these fish being injured or trapped inside the fryway. A recent installation included an adult barrier rack installed below the fryway.

Additional hydraulic and biological testing of the fryway are needed before specific design criteria or limitations can be defined.

<u>Zig-Zag and Spider Weirs</u> Spider weirs are a special design of plank weirs normally intended for juvenile fish passage. The <u>Tributary Fish Passage Design</u> section generally describes plank weirs. Zig-zag and spider weirs are shown in Figure ?. They can span broad channels and are often used for wetland creation for juvenile rearing. The plank crest elevations for spider weirs can be designed to create a switchback channel route to elongate it and provide small steps for passage.

FLAP GATES

A flap gate is a flow control device that in principle, functions as a check valve, allowing water to flow through it in only one direction. It usually consists of a flat plate that is hinged at the top of a culvert outfall. The plate falls into a near vertical position over the face of the culvert opening to close it. A positive head differential against the downstream face will force the flap against the face of the culvert to seal it. A positive head against the upstream face of the gate will force it open to release water.

Flap gates are typically constructed of cast iron. Plastic, fiberglass and aluminum gates are also available. Larger gates are constructed of wood and are often hinged at the sides rather than the top.

They are typically attached to culverts that are placed through tide dikes river flood dikes. They allow the stream to normally drain in its normal direction but prevent high tides or river floods from backing water into the stream channel.

FISH PASSAGE

Flap gates are obviously a barrier to all fish migration when they are closed. Unless specially designed for fish passage, most are also a barrier to migration when they are open.

They can be a barrier due to the head differential across the gate or by the narrow opening available for passage when the gate is open. They are also be a barrier, like any other culvert, by being perched above the downstream channel or water surface.

There are several solutions to fish passage barriers through flap gates. A method used in Canada is to modify the gate mounting hardware so the gate is rotated 90° and is hinged on the side. The hardware has to be modified to structurally support the gate, to keep it from opening too far, and to provide a thrust bearing for the weight of the gate. The gate should be mounted at an angle less than 90°. If it is rotated a full 90°, the weight of the gate will not help close it.

A second method is to use a light weight gate such as plastic or aluminum. These lighter weight materials are formed into a thin dome shaped gate by several manufacturers. The gate, being considerable lighter, opens much wider with less head differential. They also therefore have greater outflow capacity.

Figure 13-1 and Figure 13-2 show as an example the difference in hydraulic characteristics of four-foot diameter cast iron and aluminum flap gates. The figures show the maximum opening of the gates (gap) and flow for a range of submergences and head differentials. The gap is the distance the bottom of the flap gate swings away from the frame. The submergence is the depth of the downstream water surface above the bottom of the gate when it is closed. The curves are derived from a theoretical static, hydraulic model that accounts for specific weight and submergence of the gate and pressure head. It does not account for velocity head.

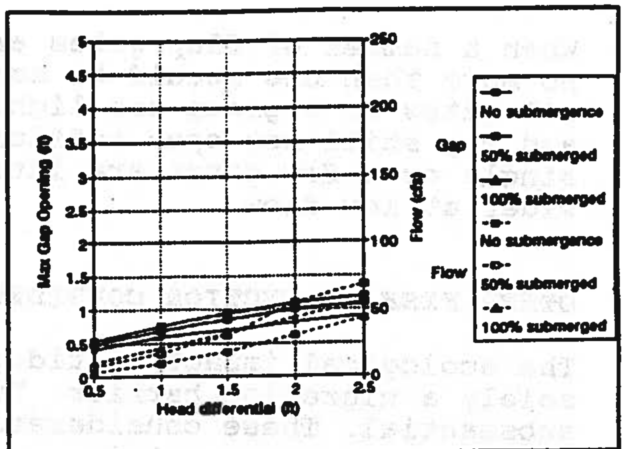


Figure 13-1. Gap and flow for 4-foot cast iron flap gate.

Conditions for upstream fish passage should be less than a foot of head differential and at least a foot of gap opening. Realize that the gap is the opening at the invert of the gate; at the mid point of the culvert, the opening is half the gap measurement. This one foot criteria is the same as standard fishway entrance criteria.

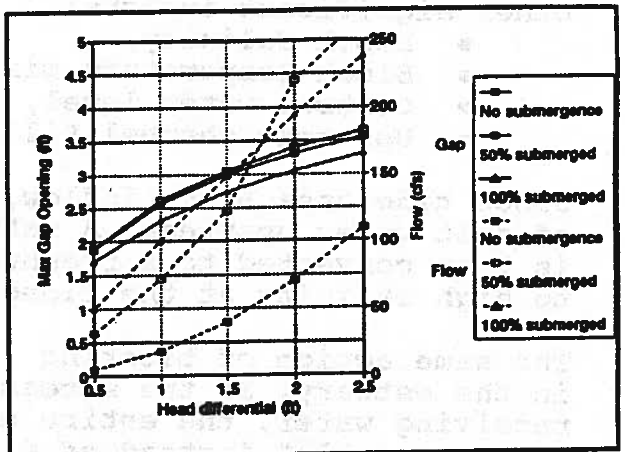


Figure 13-2. Gap and flow for 4-ft aluminum flap gate.

The benefit of lighter weight gates is obvious from the figures. There are no conditions for the cast iron gate in Figure 13-1 that comply with fish passage standards. To create an opening of 1.0 foot, a head differential of 2.0 feet and a submergence of 2.0 feet are required.

For any given gate submergence (downstream water level) and head differential, the aluminum gate is open at least several feet wider than the cast iron gate. For example at a head differential of one foot and no submergence, the aluminum gate is open over 2.5 feet and the cast iron gate is open about 0.5 feet. To open one foot, the cast iron gate requires a head differntial of 1.7 feet. The lighter gate also has much greater flow capacity at any submergence and head. The upstream pool will then drain more rapidly and approach optimum fish passage conditions.

A third solution to flap gate passage is the use of gate operators or latches. Operators and latches have been designed to prevent the flap gate from closing until the water surface rises to a critical elevation. One design is equipped with float mechanisms that trip a latch allowing the gate to swing shut. To my knowledge, this concept has not yet been constructed. Another design uses an electrically powered gate to close and open in response to the water surface. This concept is obviously the most expensive and has the greater risk of mechanical and electrical failure.

When a number of flap gates are placed on parallel culverts, often no more than one should be equipped with a light weight gate. If all gates in a group are light weight, they will compete for flow and may still not open sufficiently for fish passage. When just a single or a few gates are intended for fish passage, they open wider at low flow.

OTHER FISH PROTECTION CONSIDERATIONS

The ecological impact of tidegates in estuaries is much more than solely a migration barrier. Their influence on water quality may be substantial. These considerations are speculation; I am not aware of them being documented.

In addition to creating migration barriers tidegates have three other significant impacts:

- Block Salinity,
- Block temperature mixing,
- Control water level,
- Upstream channel filling.

Since tidegates block inflow, in estuaries they block the movement of salt water upstream. A natural estuary with a salinity gradient is then converted to a freshwater pond with an instantaneous change to high salinity at the tidegate outfall.

The same action of blocking inflow also prevents temperature mixing in the estuary. If the stream is any different temperature than the receiving water, the entire change becomes concentrated at the tidegate outlet instead of dispersed through the estuary.

Because of their hydraulic control, tidegates minimize the upstream water level fluctuation. In Puget Sound, the upstream water surface is controlled to within a few feet instead of the natural 5 to 15 feet tidal fluctuation.

The upstream channel is altered by the change in hydraulic conditions caused by the tidegate impoundment. A natural estuary is characterized by tidal surge channels created by the rush of tide waters in and out. A tidegate essentially eliminates the tidal flow and therefore fills with sediment.

By these four actions, the basic chemistry, tidal characteristics and ecology of the upstream area is drastically altered. These changes likely work cumulatively with the migration barrier impact to further affect fish production. The salinity and temperature impacts are concentrated at the tidegate itself. If fish cannot move upstream through the tidegate they cannot willfully select their preferred temperature and salinity conditions. When they pass through the tidegate, they are instantly dropped into a radically new water quality environment with no opportunity to move out of it.

These impacts can be mitigated to an extent. The usual objective of tidegates is to control the upstream water surface. It is important to establish the specific upstream water surface that is allowable. Tidegates, when installed, are usually intended to protect only from extreme high tide flooding. In that case they don't need to be closed more than a few times a year. They are closed during a majority of the time however. To prevent its closure except when needed a tidegate can be equipped with an automatic latching mechanism. Another option is to include an orifice in the tidegate or a small culvert be placed next to the culvert with the tidegate. Either orifice assures a controlled amount of sea water passes upstream at every tide cycle. With a good design, the volume of water that flows in during an extreme tide will not exceed the allowable storage volume and flood elevation above the culvert.

To make these mitigation options work, the surface water hydrology of the upstream contributing basin must be well understood. The hydraulics of the gate must be modelled to assure the intended upstream protection is adequate. This is not a difficult task.