Appendix F

Flow Duration Basin Design Guidance

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LIST OF FIGURES AND TABLES

INTRODUCTION

The flow duration (FD) basin design process is essentially an iterative process where the designer selects basin storage volumes and outlet configurations and compares the resulting discharge flow duration curve to the pre-project results. Guidelines on selecting the initial estimates are provided, as well as guidance for adjusting storage and outlet configurations. The affects of increasing or decreasing orifice diameter, invert elevation (weir elevation), and number (length); and basin storage are summarized.

DESCRIPTION OF THE FLOW DURATION BASIN

The flow duration control approach involves: 1) simulating the runoff from the project site, pre- and post-project, using a continuous rainfall record (50 years of record in this case); 2) generating flow-duration curves from the results; and 3) designing a flow control facility such that when the post-project time series of runoff is routed through the facility, the discharge pattern matches the pre-project flow-duration curve. The flow control facility is a detention basin that diverts and retains a certain portion of the runoff. The portion to be retained is essentially the increase in surface runoff volume created between the pre-project and post-project condition. This captured increase in volume is typically discharged to the ground via infiltration (and/or evapotranspiration if vegetation is present) in the basin, released at a very low rate to the receiving stream (at the critical flow for basin design, Qcp, or 10% of the "pre-project" 2-year storm), and/or diverted to a safe discharge location or other infiltration site, if feasible. For the examples presented here, the captured runoff is assumed to be infiltrated in the basin and discharged at Qcp (see next section for computation of Qcp).

The flow duration basin is designed to have two pools (Figure F-1), a low flow pool (Zone A) and a high flow pool (Zone B). The low flow pool is designed to capture small to moderate size storms, the initial portions of larger storms, and dry weather flows. The high flow pool is designed to store and release higher flows to maintain, to the extent possible, the pre-project runoff conditions. The flow duration basin can also serve as a water quality treatment facility and can be designed to treat dry and wet weather flows using a combination of extended detention and natural treatment processes. Most dry weather "nuisance flows" will also infiltrate in the basin.

The flow duration basin is sized using an iterative process of adjusting basin storage as well as selecting and adjusting orifice sizes in the outlet structure. The low flow pool within the basin is initially sized to capture the increase in runoff volume that is generated from the impervious surfaces. This capture volume is dependent on the development characteristics, the soil types, and the magnitude of change in runoff created by the proposed development. Previous analyses have shown that area requirements have less to do with the range of storms selected for management and more to do with site and development characteristics.

Once the lower pool is sized to capture the correct volume of runoff, the upper pool of the basin is sized to detain and discharge larger flows through a specific set of orifices in such a way as to reproduce the flow duration curve. The number, diameter, and elevation of these orifices are determined by a trial and error approach (King County, 1998). The combination of sizing the lower portion of the basin and the upper portion to detain and discharge high flows has the affect of capturing the correct volume of runoff and matching the pre-project distribution of hourly flows.

The outlet structure is designed to reproduce the pre-developed flow duration (runoff histogram) using orifice diameter and elevation above the bottom of the basin. Figure F-2 illustrates the outlet structure. The number, size and placement will vary from basin to basin depending on project conditions. The headwall could be constructed using steel plates in a manner that allows owner/operators to easily change the outlet configuration to improve basin performance if necessary.

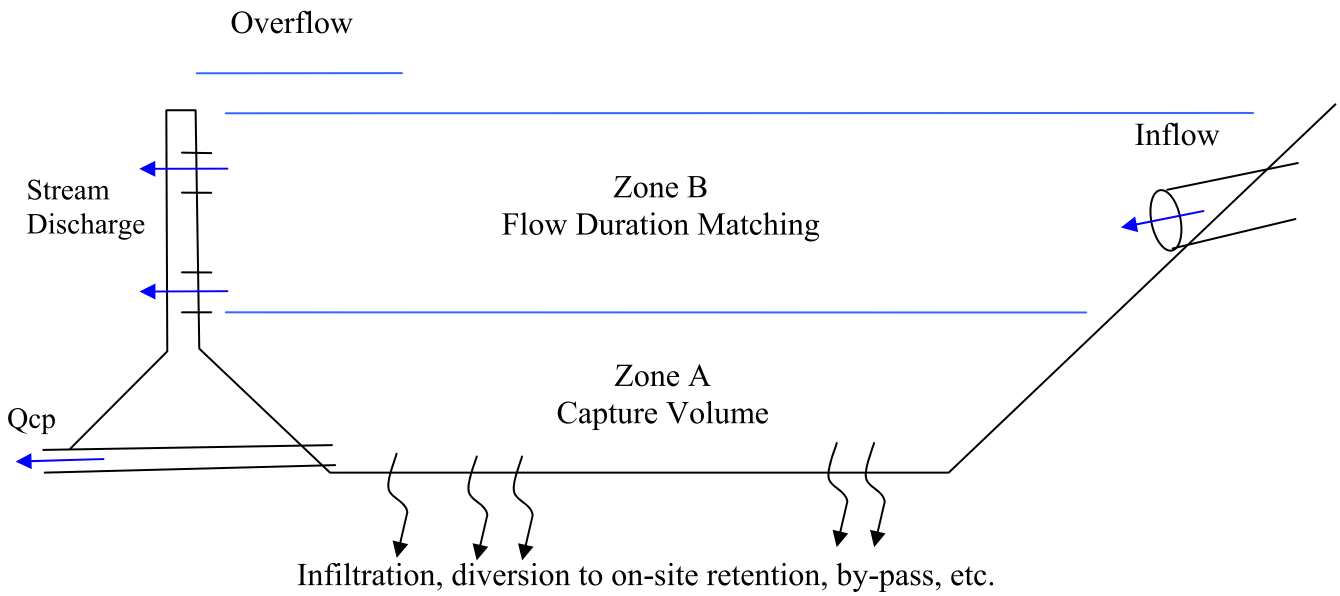


Figure F-1. Generalized Configuration of Flow Duration Basin

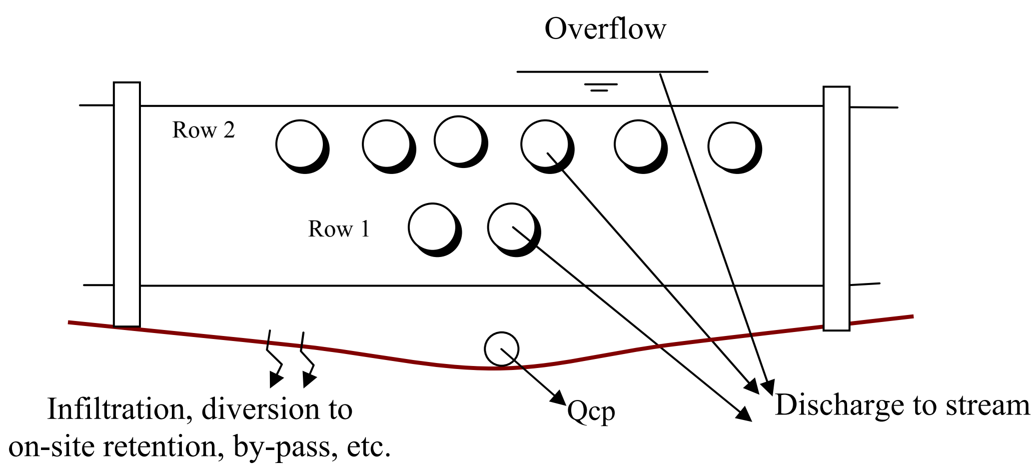


Figure F-2. Generalized Configuration of Orifice-Type Outlet Structure

DETERMINATION OF ALLOWABLE LOW FLOW DISCHARGE, QCP

The critical flow of a stream (Qc) is defined as the flow that produces the critical shear stress that initiates bed movement or erodes the toe of stream banks. A goal of hydromodification management is to control the discharge of the increased flow and volume created by development to below Qc, to minimize the potential for increased erosion. In order for the critical flow to be useful to dischargers in design of on-site hydromodification control structures, the critical flow in the stream must be partitioned or related to an on-site project-based variable. For this purpose, the in-stream critical flow was related to the pre-urban 2-year peak flow in the stream. Based on the hydrologic studies of Thompson and Ross Creeks (Chapter 3 of the HMP Report), the critical flow was generalized as being approximately 10% of the 2-year peak flow under undeveloped land use conditions.

Using this relationship, the allowable low flow discharge from a flow control structure on a project site, Qcp, can be calculated as 10% of the pre-project 2-year peak flow from the site. In computing Qcp, the original condition of the site before development must be considered. This does not imply that the developer is being required to provide flow controls to match pre-development conditions; rather, it is a means of apportioning the critical flow in a stream to individual projects that discharge to that stream, such that cumulative discharges do not exceed the critical flow in the stream.

Qcp can be computed using any standard engineering method for calculating the peak flow for a 2-year return period storm event. These include the Rational Method, synthetic design storm hydrograph approaches, and continuous simulation model records. In the Rational Method, the equation Q = CiA is used, where discharge (Q) is a function of the drainage area (A), rainfall intensity (i), and a runoff coefficient (C). The rainfall intensity can be obtained from local agency intensity-duration-frequency curves, using an estimated time of concentration for the undeveloped site. The runoff coefficient should also be selected to represent the undeveloped site and can be found in a number of standard engineering references.

As an example, the allowable low flow discharge for the San Jose Small Site Example (see Chapter 6 of the HMP Report and Technical Memorandum #8) was calculated using the Rational Method. The project drainage area (A) for the small site example is equal to 3.6 acres. The runoff coefficient (C) selected for this analysis is equal to 0.36, and is based on the undeveloped, pre-urban condition of the project site. Using a time of concentration of roughly 6 minutes for the undeveloped project site, the rainfall intensity (i) for the 2-year event was determined from Figure 6 of the County of Santa Clara Drainage Manual (March 1966) to equal 1.5 inches/hour. Therefore, the 2-year design discharge for the undeveloped project site is equal to:

$Q_{2\text{-yr}} = CiA = (0.36 * 1.5 * 3.6) = 1.94$ cfs

and

10% of $Q_{2\text{-yr}} = 0.19$ cfs

Therefore, the design Qcp for the flow-duration-basin sizing analysis of the small site example is equal to 0.19 cfs.

PROCEDURES FOR SIZING A FLOW DURATION BASIN

1. Data file preparation
   a. Need long-term (~25-50 years) stormwater runoff records for pre- and post-development conditions. These are generated using hydrologic programs, such as HEC-HMS, SWMM, and HSPF. Input to these programs is a long-term precipitation record (generally in hours although 15 minute data could be used), project area and development information, and soils information, to produce a long-term continuous runoff record.

2. Compute Pre- and Post- Flow Duration Curves
   a. For each of the runoff records, develop a histogram[1] and cumulative frequency distribution of the hourly runoff values. Use the post-project record to select histogram flow range and bin (interval) increments. Use consistent increments for the pre-project flow histogram and the post-project with control measures in place histogram.
   b. When generating the cumulative frequency distribution it is preferable to begin the count with the largest flow bin proceeding downwards to the smallest value. <u>The cumulative frequency distribution is the flow duration curve</u>.

3. Select Initial Estimates for Basin
   a. Area: set the starting area at ~2% to 7% of the catchment area. FD basins in catchments with clay soils are about 2%, while basin collecting runoff from sandy soils can be up to 7%. This seems to be a reasonable starting point.
   b. Depth: range from 2 to 10 feet. The storage of the basin will be determined from the iterative analysis; however, local jurisdictions may have limitations on depth of a basin. Shallow depths may be preferred for multi-purpose facilities.

4. Select Initial Estimate for Outlet Structure
   a. Start with ONLY a bottom orifice, which is sized to discharge at a maximum rate equal to the critical flow rate (Qcp = 10% of pre-project 2-year peak flow) when the basin is full. The volume of the initial FD basin can be approximated by routing post-project flows through this basin with the bottom orifice and weir overflow, and then comparing the total number of hours of the resulting FD curve at Qcp to the pre-project curve at this flow magnitude. Adjust the volume of the initial FD basin so that these curves match in total number of flow hours at Qcp. Increasing the basin storage volume moves the FD curve to the left. Decreasing storage volume moves the curve to the right.
   b. After adjusting the basin storage volume, then add one orifice at ¾ of the effective depth of the basin. Set the orifice diameter at 6 inches. The lowest orifice corresponds to the lowest arc of the flow duration curve.
   c. After adjusting the basin storage volume and adding the first orifice, then add a second orifice at ⅞ of the effective depth of the basin. The combined first and second orifice

[1] A histogram is a graphical representation of the frequency distribution of a series of data. The histogram provides a visual impression of the shape of the distribution as well as the amount of scatter. A histogram is developed by dividing the range of values in the data set into equal intervals. The procedure is to count the number of data points that fall into each interval, thereby determining the frequency of occurrence of flows with similar magnitudes for each interval.

corresponds to the second arc of the flow duration curve, and represents the combined flows.

d. Increasing the lower orifice diameter will adjust the slope and curvature of the lowest arc of the flow duration curve. Increasing orifice diameter increases the range of flow magnitude that can be discharged through this orifice, which shifts the arc upwards. Decreasing orifice diameter reduces the lowest arc.
e. Increasing or decreasing orifice elevation shifts the transition point between arcs along the FD curve. Increasing the elevation moves the transition point left and upwards, while decreasing the elevation moves the point right and downwards.
f. Increasing storage volume also helps match the curve in the upper high flow range. In most cases, the facility can be sized so that a small amount of overflow occurs during infrequent large flows.
g. Refinements should be made in small increments and performing one change at a time. It is best to begin with sizing the storage volume and then adjusting the number/size of the lowest orifice to match the lowest part of the FD curve first. Then proceed upwards by adding and adjusting the next highest orifice discharges to match the remaining portion of the FD curve.

5. The range of discharge capacity should approximately match the range of pre-project discharge.
   a. Orifice diameters should be selected such that the range of flows, given the range of hydraulic head on the orifice, approximates the range of flows discharging from the site under pre-project conditions.
6. Stage-Discharge Relationship
   a. The stage-discharge relationship is defined by the sum of all the outflows from the basin: 1) discharge by infiltration through the wetted bottom of the basin; 2) discharge through a small orifice discharging at the critical flow rate (Qcp); and 3) discharge through the outlet structure designed to match the pre-project flow duration curve.

TEST FOR GOODNESS-OF-FIT

Matching flow duration curves is the preferred method of hydromodification management to protect the beneficial uses of streams. The question addressed in this section is, how close do these curves need to match?

Figure F-3 shows the flow duration curves for the small 12-lot subdivision in San Jose described in Section 5.3 of the HMP Report. This figure includes the pre-project, post-project, and post-project with BMP flow duration curves. Based on this figure, the post-project with BMP curve closely matches the pre-project curve for small frequent flows up to 1.5 cfs, than deviates for less frequent high flows. Visually this looks like a pretty good match. However, the HMP needs a consistent and accurate means to measure the difference and limit deviations above the pre-project conditions.

According to the Western Washington flow duration basin sizing guidelines (Washington Department of Ecology, 2000), the post-project curve cannot exceed the pre-project curve by more than 10%, over no more than 10% of the length of the curve. Deviations less than the pre-project condition are allowed and unlimited[2]. Basins designed with large over control will require larger land areas.

Figure F-4 plots the difference between the pre- and post- cumulative volume, which is simply the magnitude of flow for each bin in the histogram times the frequency of that bin, and then summed. Flows less than Qcp are not included. The difference is plotted as a percent of the cumulative pre-project volume. The figure shows, or expresses, the definition of the goodness-of-fit in terms of runoff volume. The cumulative post-project runoff volume cannot exceed the cumulative pre-project volume by more than 10%, over no more than 10% of the length of the curve. Also, the total cumulative runoff volume over the full histogram cannot exceed the pre-project condition.

Figure F-3

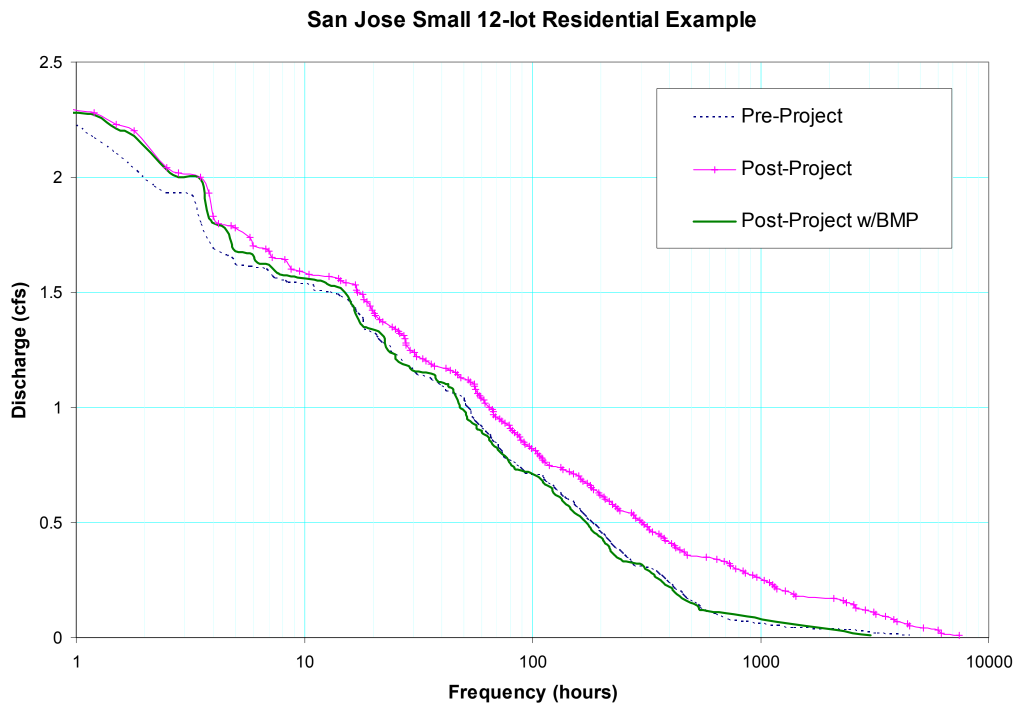
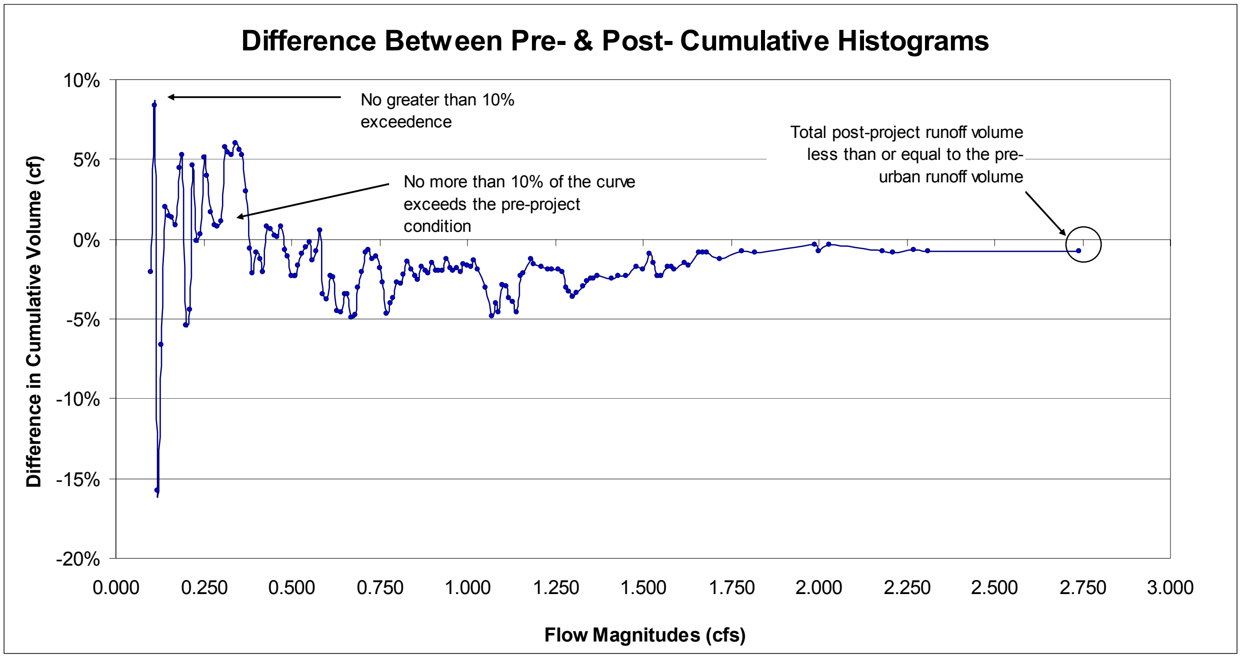


Figure F-4

[2] Deviations are unlimited with respect to erosion but habitat issues could require limits on too little runoff.



REPORTING AND GRAPHICS

This section describes the recommended reporting information and presentation graphics useful for conveying the adequacy of flow duration basin sizing to agencies plan review staff. This information includes a table of resulting basin characteristics, histograms of resulting flow characteristics, and flow duration curves of resulting flow characteristics.

TABLE OF BASIN CHARACTERISTICS

Table 1 below lists the basin characteristic information to be included and presents example information for three scenarios. The characteristics presented should include basin volume, area, depth, drain time, and discharge modes.

Table F-1
Resulting Flow Duration Basin Characteristics

Basin Characteristics	DESIGN SCENARIOS		
	Discharge at infiltration rate only	Discharge at infiltration rate plus <Qc	Basin size with roofs disconnected
Basin Volume (acre-feet)	0.11	0.10	0.08
Basin Size (% catchment)	2.1%	1.7%	1.3%
Basin Size (%DCIA)	4.6%	3.7%	2.8%
Basin Depth (feet)	1.75 ft	2.25 ft	2.5 ft
Drain time (days)	3.7 days	<1 day	3.6 days

Qc (cfs)	0	0.1 cfs	0
Infiltration Rate (loss through wetted bottom, cfs)	0.2 in/hr	0.2 in/hr (0.01 cfs)	0.2 in/hr
Outlet type and dimensions (inches)	Orifice: 3 to 6-inches	Orifice: 3 to 6-inches	Orifice: 3 to 6-inches

HISTOGRAM SHOWING PRE-PROJECT, POST-PROJECT, AND POST-PROJECT WITH BMP RESULTS

Figure F-5 presents the resulting histograms using the 716 acre Thompson Creek example. The histograms for pre-project, post-project and post-project with BMPs are shown. The frequency scale is shown as logarithmic to highlight the differences throughout the flow bin scale, otherwise the differences at the high flow end would be hard to observe.

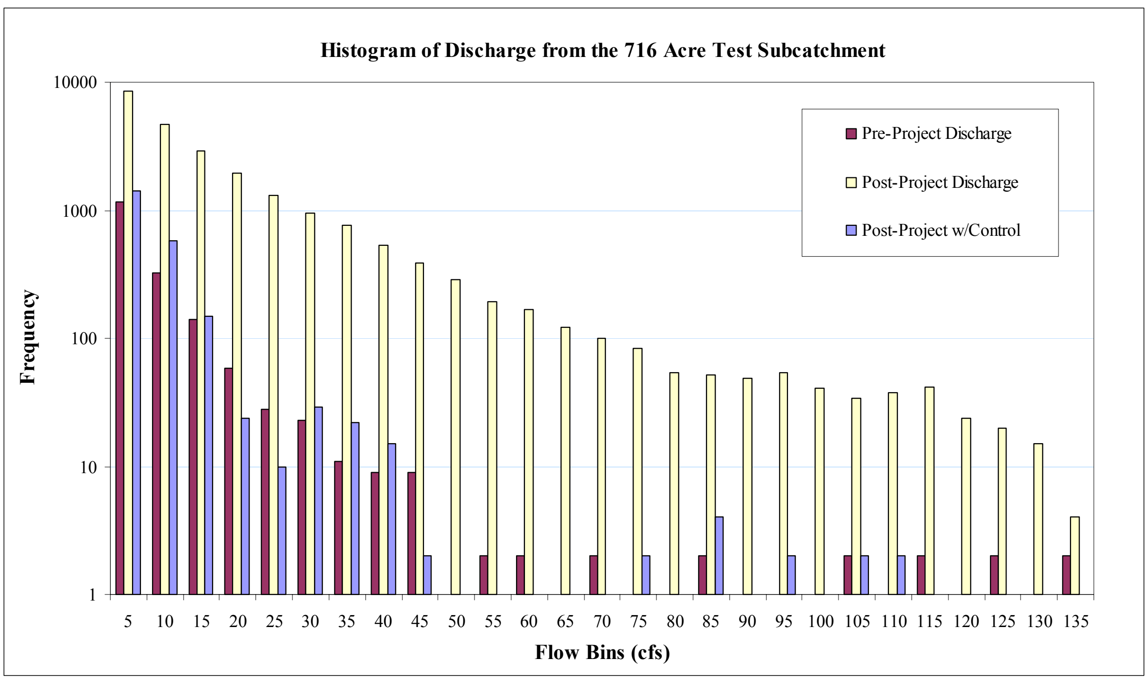


Figure F-5. Histogram of Discharge from the 716 Acre Test Subcatchment

FLOW DURATION CURVES SHOWING PRE-PROJECT, POST-PROJECT AND POST-PROJECT WITH BMP

Figure F-6 presents the resulting flow duration curves for the same Thompson Creek example. The flow duration curves for pre-project, post-project and post-project with BMPs are shown. The frequency scale is shown as logarithmic to highlight the differences throughout the flow bin scale; otherwise the differences at the high flow end would be hard to observe.

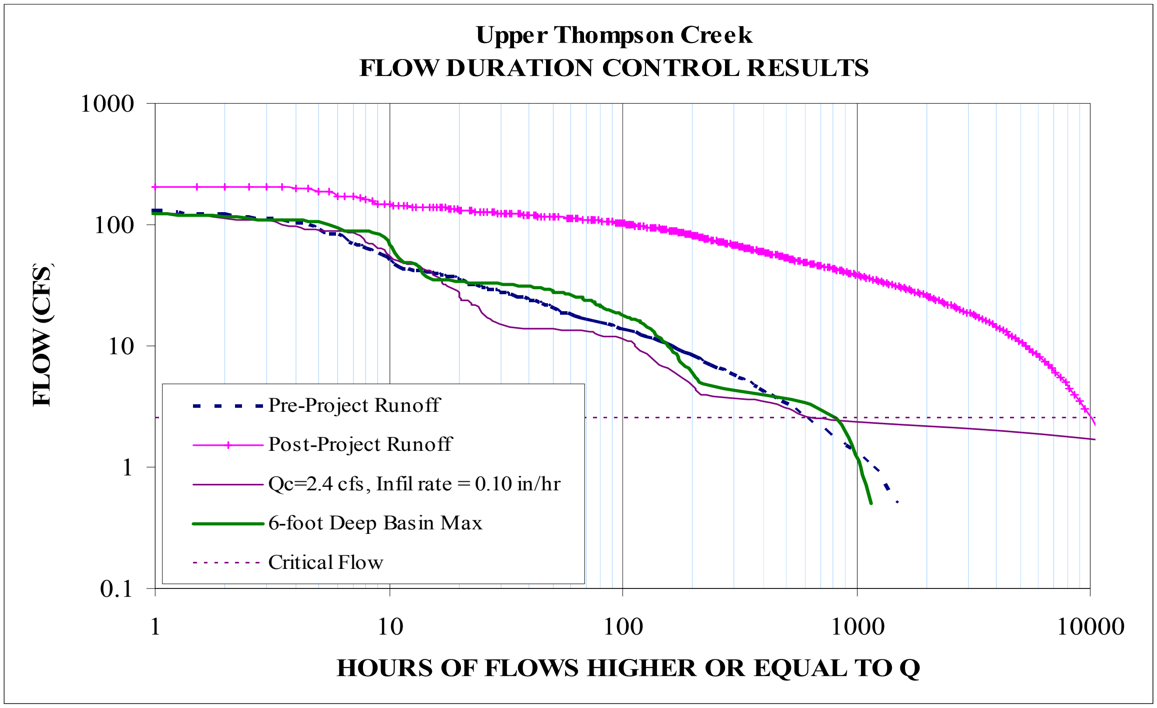


Figure F-6. Example Flow Duration Curves

OUTLET DESIGN FOR THE PURPOSE OF MATCHING FLOW-DURATION

The following addresses a number of detention basin outlet design considerations as they pertain to the goal of matching pre- and post-project flow-duration distributions.

Comparison of Multi-tier Rectangular Weir and Circular Orifice Outlet Designs

In an effort to identify significant outlet design criteria for matching pre- and post-project flow-duration, the relative performance of two outlet configurations was considered: a 3-tier rectangular, sharp-crested weir, and an outlet consisting of three tiers of circular orifices. Each outlet was assumed to discharge flows from a detention basin 1200 feet long by 1000 feet wide, with a maximum depth of 4 feet and 3:1 side slopes. Infiltration rates through the bottom of the wetted surface of the basin were assumed to be 0.2 in/hr. Downstream discharges for each outlet were calculated from a 50-year continuous rainfall time-series, input to a runoff-storage-discharge model.

Figure F-7 shows the general design of the multi-tiered rectangular weir and circular orifice outlets analyzed.

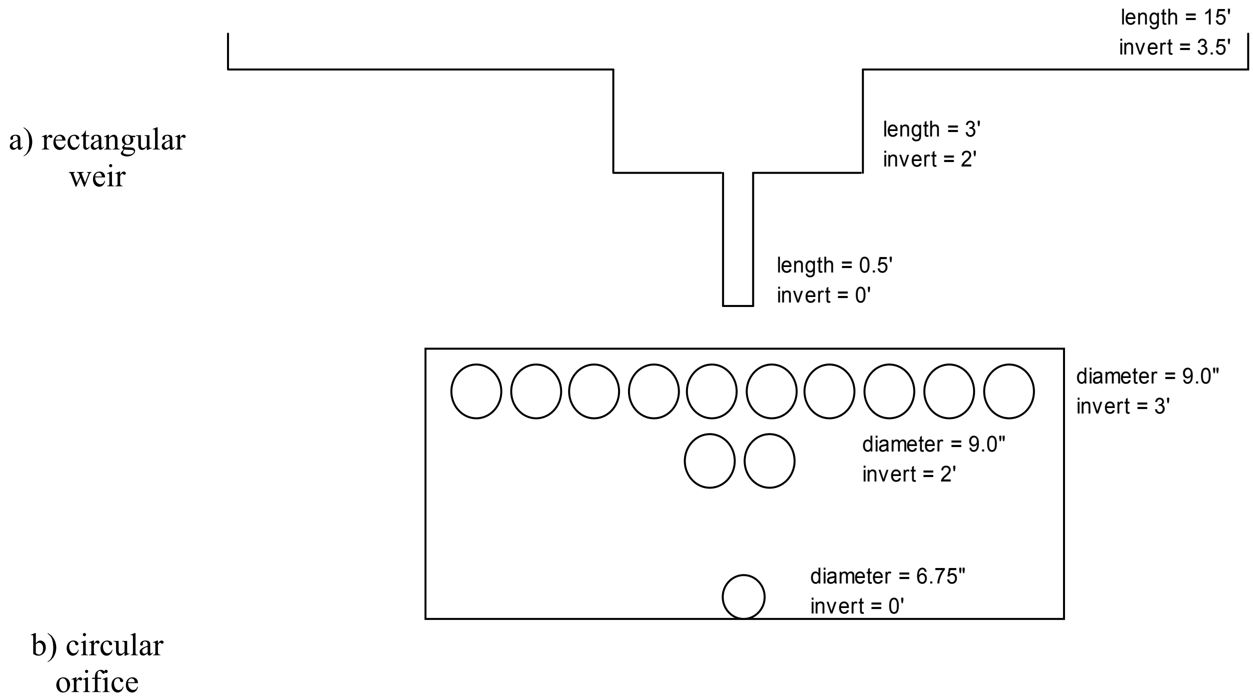


Figure F-7: 3-Tier Sharp-Crested Rectangular Weir (A) and Circular Orifice (B) Outlet Designs

The cumulative flow-duration distribution calculated for a 50-year continuous runoff-storage-discharge simulation of each outlet design is plotted in Figure F-8 alongside the flow-duration curves for the modeled pre-project catchment, post-project without flow control, and the critical discharge threshold (Qcp) (2.4 cfs in this simulation).

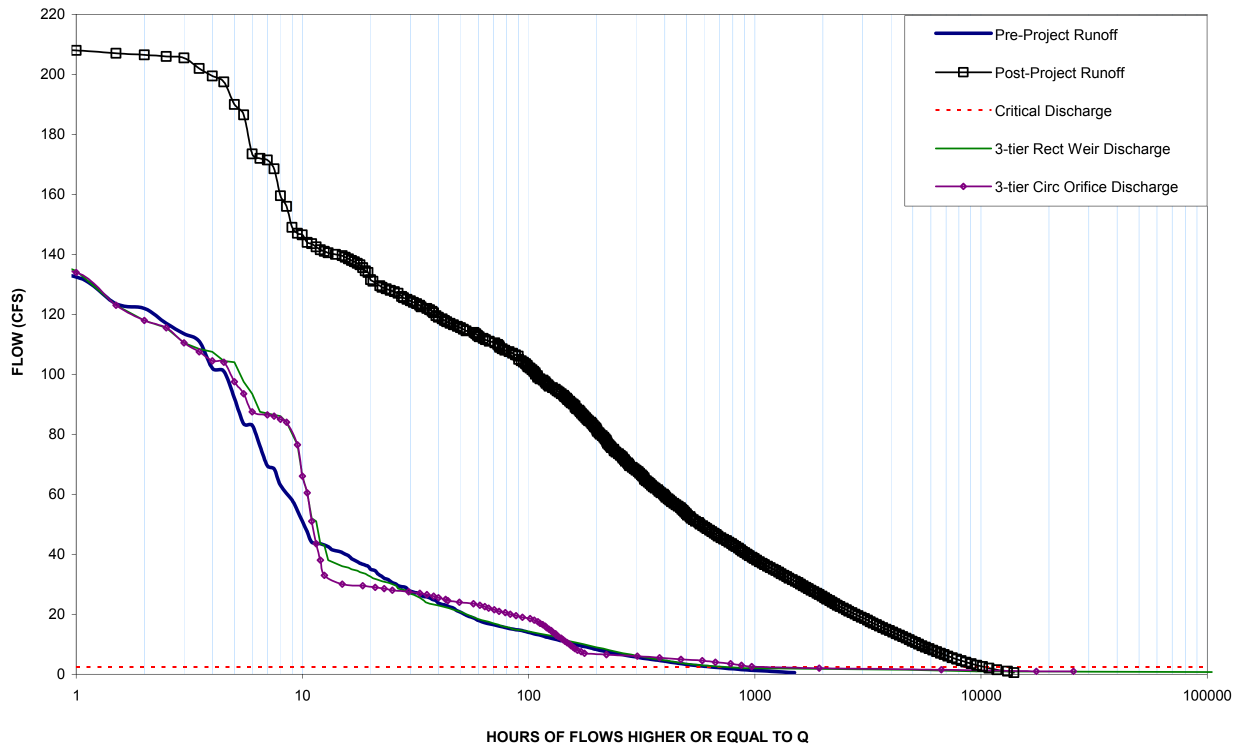


Figure F-8. Flow Duration Control Results

The 3-tier rectangular weir appears to provide a closer match to the pre-project flow-duration curve than the 3-tier circular orifice design, particularly for discharges of approximately 30 cfs or less, which constitute 96% of the pre-project flow duration. While both the rectangular and circular orifice simulations fail to match the pre-project curve above 43 cfs, flows of this magnitude represent roughly 1% of the modeled flow duration. If more time were invested, the the orifice design could be improved to achieve a closer match.

The relative performance of the rectangular weir as compared to the circular orifice design is more evident when Figure F-8 is re-plotted on a log-log scale, as provided in Figure F-9.

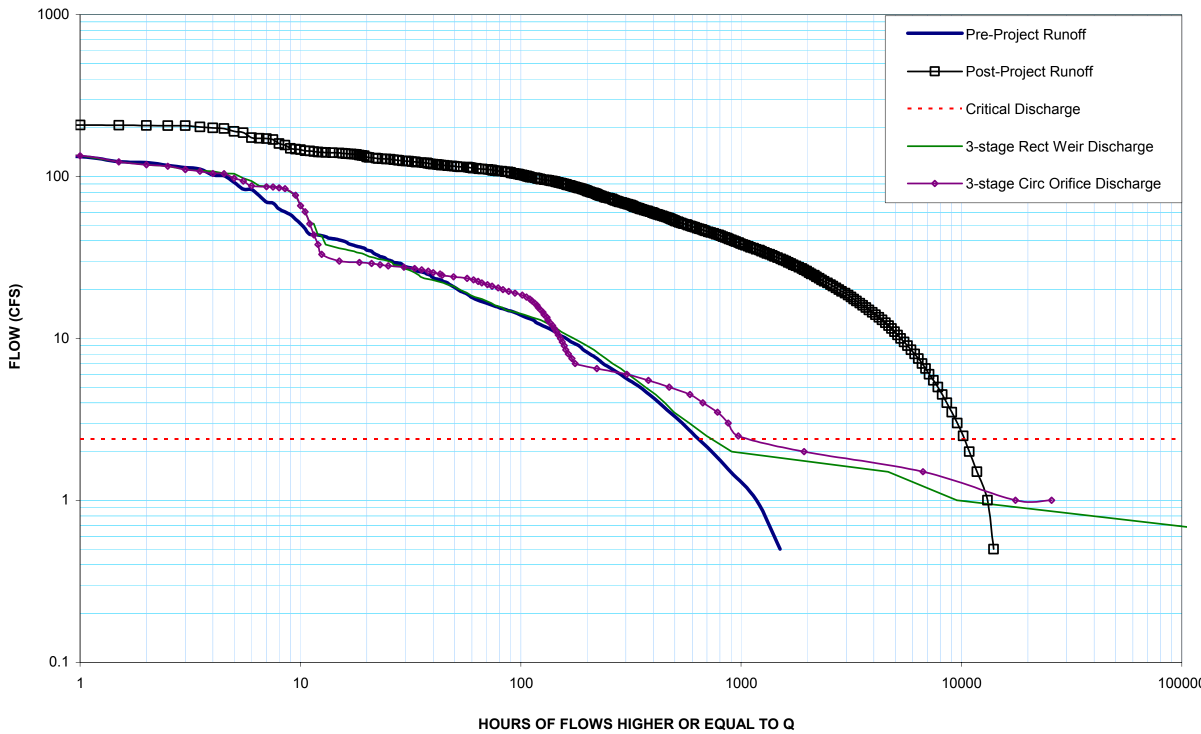


Figure F-9. Flow Duration Curve Results – Log-Log Scale

The expanded scale highlights the difference in curve shape between the rectangular and the circular orifice outlet designs. The multi-stage rectangular outlet closely follows the smooth, convex shape of the pre-project curve, with the exception of large, low-duration flows (Q > 43 cfs) and flows less than the designated "Critical Discharge". In contrast, the circular orifice outlet curve meanders about the pre-project curve, resulting in a significant proportion of duration where post-project flows are greater than those modeled for the pre-project conditions.

Comparison of the respective stage-discharge curves for each of the two designs, as shown in Figure F-10, illustrates the critical difference. For each tier of the circular orifice outlet, the stage-discharge relationship is convex, whereas the rectangular outlet yields a smoother, approximately concave curve, as is desired to match the pre-project flow-duration.

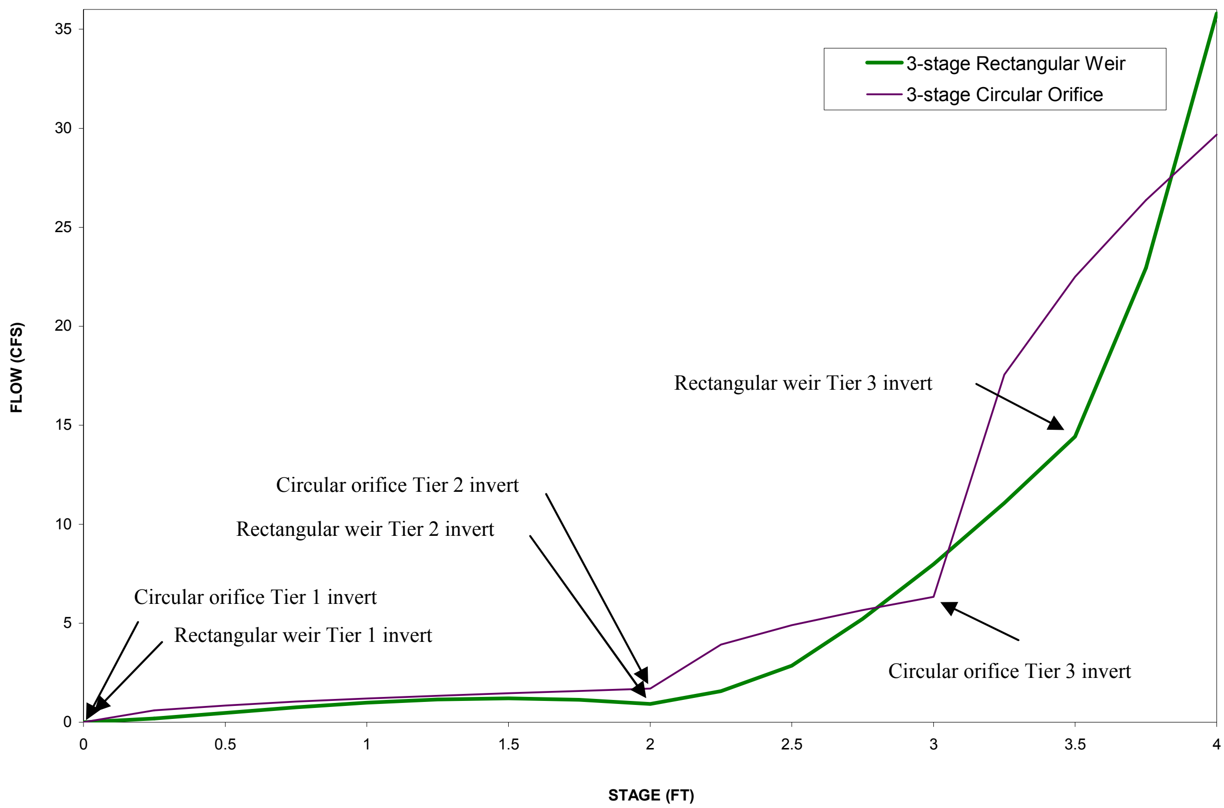


Figure F-10. Stage-Discharge Curves

In summary, these results suggest that an "ideal" outlet design in terms of matching flow-duration is similar to the multi-tier rectangular weir analyzed here, but with smooth, curved sides as shown in Figure F-11, rather than a stepped design. It is assumed that a power equation could be derived for such an outlet, thereby facilitating design and sizing calculations.

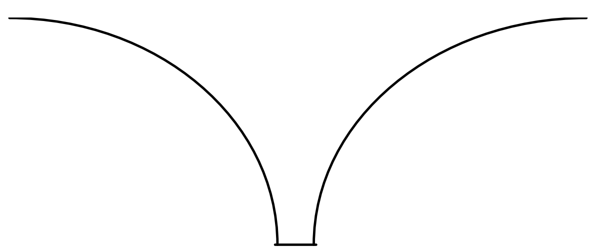
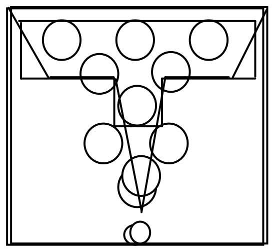
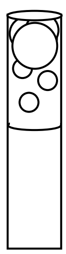


Figure F-11.Continuous curve outlet design



Alternative Outlet Designs

The outlet designs compared in this analysis represent only two possible configurations. Figure F-12 displays several additional conceptual designs. The actual performance of these configurations was not analyzed here. However, the performance of these designs can be easily evaluated by looking at stage-discharge curves for any proposed design.



a) perforated plate or pipe

or

b) circular orifice/rectangular weir combination

c) v-notch/trapezoidal weir combination

Figure F-12. Alternative outlet designs

Possible outlet configurations vary in terms of complexity of design and construction, and in suitability for matching flow-duration. Of the three above, the perforated outlets (6a) offer the greatest flexibility, as essentially any number of orifices of varying diameter may be used to achieve the desired stage-discharge relationship; however this design is difficult to construct properly. The other two designs combine different outlet shapes to develop this relationship. Based on results from the 3-tier rectangular weir analysis, it is presumed that the performance of these alternative methods is associated with how well they approximate the shape of the design in Figure F-11 and a smooth concave stage-discharge relationship.

Limitations on Three Stage Outlet Design

In order to achieve acceptable matching of the pre- and post-project flow duration curves at low, high-duration flows (e.g. < 4 cfs) in this analysis, it was necessary to significantly constrict the size of the lowermost tier (6" at most for L1 of the rectangular weir, 6.75" diameter for the circular orifice outlet)[i]. Such a small low-flow outlet size exposes the structure to a heightened risk of clogging. It is presumed that such an issue arises with any relevant outlet design – namely, that to match the very low, high duration pre-project flows, a blockage-prone low-flow outlet is required, and will be part of any design configuration.

A possible solution is to employ an outlet design that is not prone to clogging by incorporating a filtration component for low flow in order to screen out small debris. For example, flows could pass through a high flow rate (large diameter) perforated vertical riser embedded in crushed stone and filter fabric before discharging through the outlet control (e.g. multi-stage weir or series of orifices).

DATA AND RESOURCE REQUIREMENTS

The primary data requirements for flow duration basin sizing are long term flow records from the project site, representing pre-project and post-project conditions. The post-project flow record is then routed through hydraulic modeling software (e.g. SWMM, HEC-RAS), which approximates the effect of a flow duration basin, represented as a stage-storage-discharge curve, in order to match the pre-project condition.

The long term precipitation records and watershed hydrologic characteristics, used to create the necessary flow records through the application of hydrology modeling software (e.g. HEC-HMS, SWMM, HSPF), are also required.

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[i] A secondary issue which arises is that the standard sharp-crested rectangular weir discharge equations break down when applied to very narrow crest lengths under high hydraulic head. To account for minor energy losses at the contraction of the weir crest, the "effective" length of each stage crest (L) is calculated as follows:

$$L_e = L - 0.1nH$$

where:
L_e = effective length of weir crest (ft)
L = measured length of weir crest (ft)
n = number of contractions (2)
H = head above crest (ft)

From this equation, the effective length of L1 goes to zero when the head above the crest is 5 times the measured crest length, resulting in zero discharge when calculated from the standard equation for a sharp-crested rectangular weir.

$$Q = C\,L_e\,H^{3/2}$$

where:
Q = discharge (cfs)
C = discharge coefficient, $C = 3.27 + 0.4\,(H/P)$

This yields an unsatisfactory result for this stage of the weir.