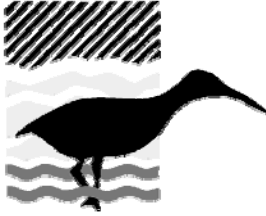


E

Hydromodification Management Requirements

TABLE OF CONTENTS

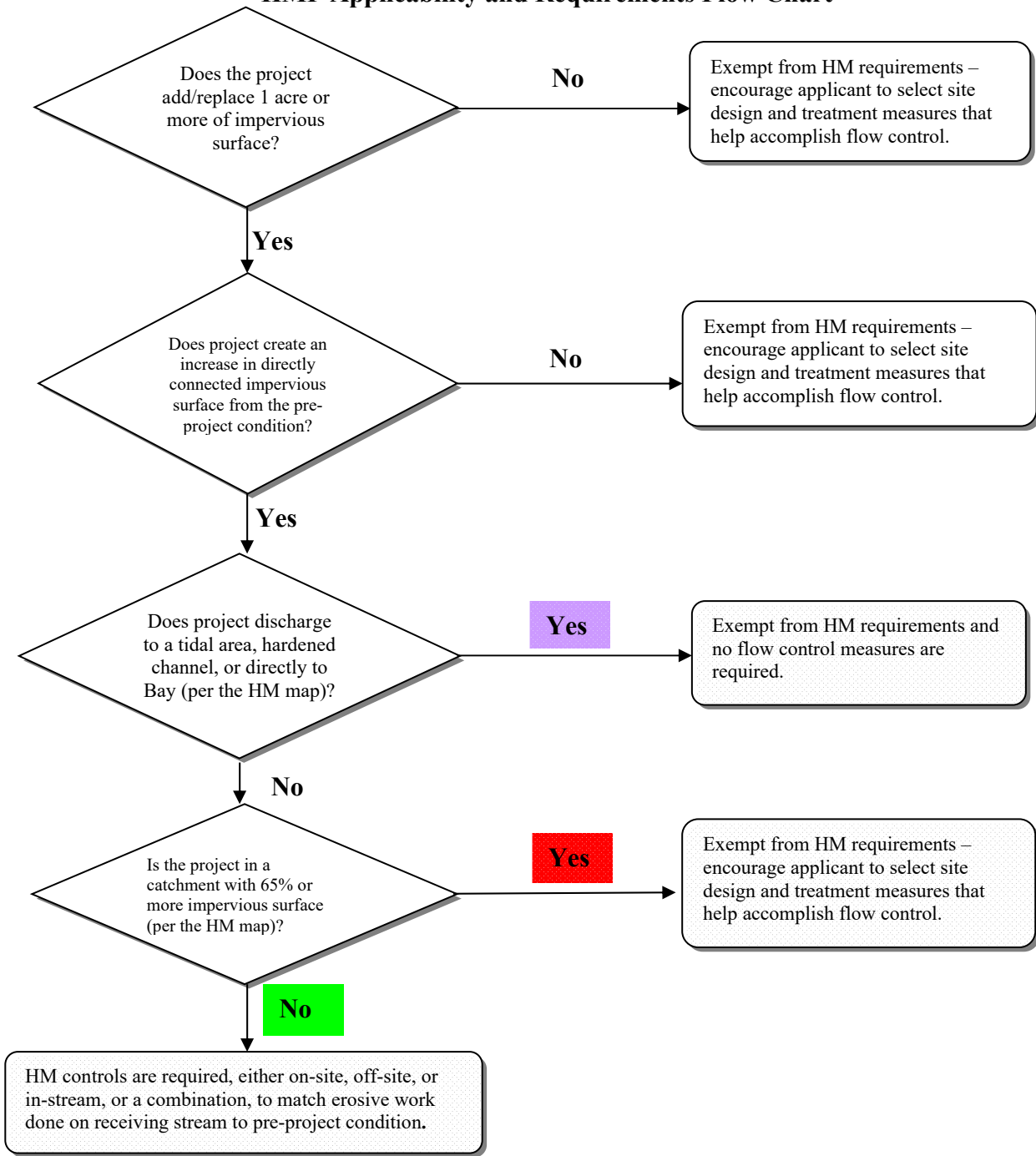
- HMP Applicability and Requirements Flow Chart
- HMP Applicability Map
- Introduction to Flow Duration Basin Design and Flow Duration Basin Design Guidance Technical Memorandum
- The Bay Area Hydrology Model – A Tool for Analyzing Hydromodification Effects of Development Projects and Sizing Solutions



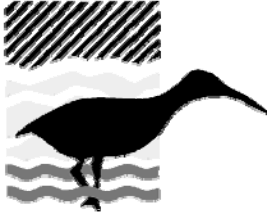
APPENDIX E-1

HMP Applicability and Requirements Flow Chart

**Appendix E-1
HMP Applicability and Requirements Flow Chart**



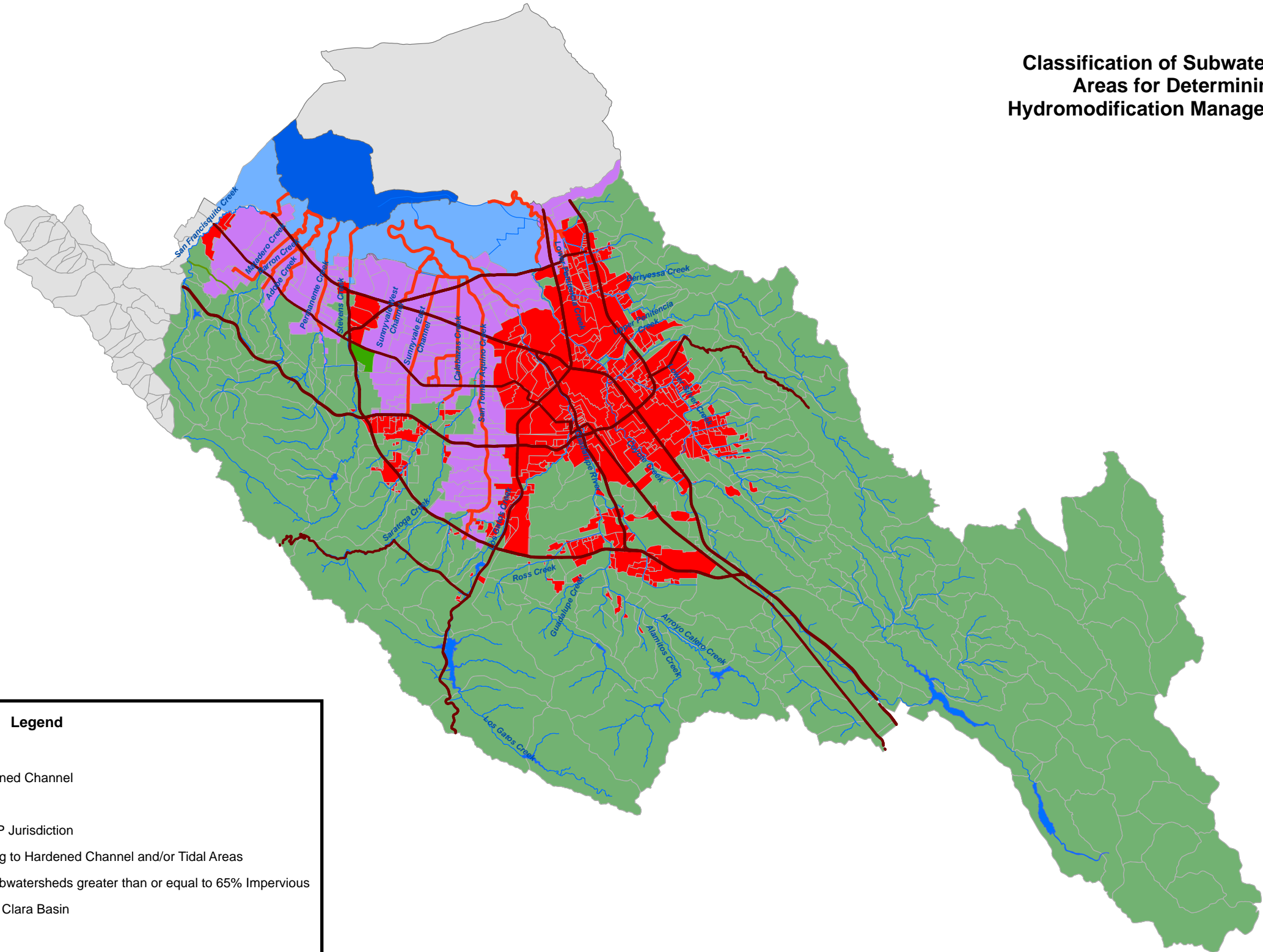
Notes:
Colors correspond to the HM Applicability Map in Appendix E-2.



APPENDIX E-2

HMP Applicability Map

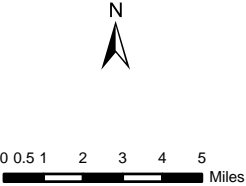
Classification of Subwatersheds and Catchment Areas for Determining Applicability of Hydromodification Management (HM) Requirements



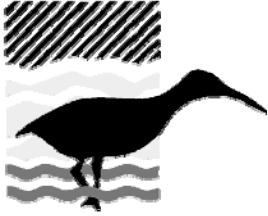
Legend

- Major Roads
- Continuously Hardened Channel
- Major Creeks
- Outside SCVURPPP Jurisdiction
- Catchments Draining to Hardened Channel and/or Tidal Areas
- Catchments and Subwatersheds greater than or equal to 65% Impervious
- Reservoirs in Santa Clara Basin
- Baylands
- Subwatersheds less than 65% Impervious

Revision Date: November 2010



This map contains revisions to the March 2009 version to reflect updated impervious surface data and/or catchment boundaries in the Cities of San Jose, Sunnyvale, Mountain View, and Milpitas, as described in the report to the Water Board dated October 14, 2010, consistent with the HM applicability criteria set forth in Attachment F, Section 4 of the MRP.



APPENDIX E-3

**Introduction to Flow Duration Basin Design
and
Flow Duration Basin Design Guidance Technical Memorandum**

Appendix E-3

Introduction to Flow Duration Basin Design

This introduction to flow duration basin design is excerpted from a technical memorandum entitled “Flow Duration Basin Design Guidance” (GeoSyntec Consultants and EOA, Inc., March 2005). The technical memorandum is included in its entirety following this introductory section, on page 5, and is also presented in Appendix F of the Hydromodification Management Plan (HMP) – Final Report (April 2005) available at www.scvurppp.org. The following is a brief overview of the flow duration basin design approach for meeting hydromodification management (HM) requirements¹.

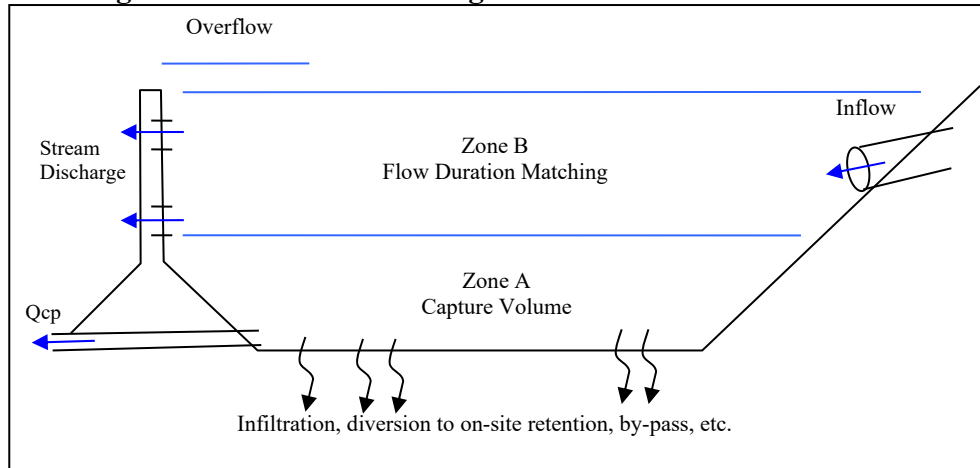
On-site controls designed to provide flow duration control to the pre-project condition are considered to comply with the HM requirements. The flow duration control approach involves: 1) simulating the runoff from the project site, pre- and post-project, using a continuous rainfall record; 2) generating 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. See the following section on hydrologic models for a discussion on generating flow-duration curves.

The flow control facility described here 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 must be discharged to the ground via infiltration (and/or evapotranspiration if vegetation present) from the basin, released at a very low rate to the receiving stream (at the critical flow for basin design, Q_{cp} , or 10% of the pre-project 2-year peak flow), and/or diverted to a safe discharge location or other infiltration site, if feasible.

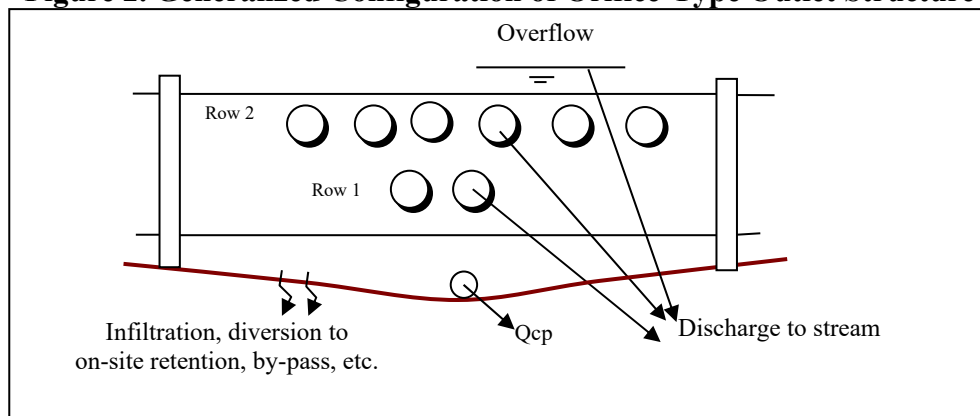
As shown in Figure 1, the flow duration basin can be conceptualized as having two pools, a low flow pool (Zone A) and a high flow pool (Zone B). The low flow pool is designed to capture the difference in volume of runoff between the pre- and post-project conditions. It will also capture small to moderate 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 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 development project. This capture volume is dependent on the development characteristics, the soil types, and the magnitude of increase in impervious surfaces. The post-project runoff calculations should take into account planned stormwater treatment or LID treatment measures that may detain or infiltrate runoff. Project proponents should see local municipality guidance for standard design criteria and policies (e.g. municipality may encourage basins with at least a 4:1 side slope integrated into the landscape rather than a deeper basin).

Project proponents and municipal staff may choose to use the Bay Area Hydrology Model (BAHM) to design and/or review flow duration control facilities. The basic design approach described here is utilized in the BAHM. The BAHM and User Manual are available at <https://www.clearcreeksolutions.info/bahm-download-page>.

Figure 1. Generalized Configuration of Flow Duration Basin

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 an outlet structure in such a way as to reproduce the flow duration curve (hydrograph). Figure 2 illustrates an outlet structure consisting of a series of orifices at set elevations above the basin bottom. The number, size and placement will vary from basin to basin depending on project conditions. The combination of sizing the lower portion of the basin to contain the increased volume and the upper portion to detain and discharge high flows has the effect of capturing the correct volume of runoff and matching the pre-project distribution of hourly flows. A weir and orifice combination could also be designed to accomplish the same level of control. A detailed description of the process for sizing a flow duration basin is provided in the technical memorandum.

Figure 2. Generalized Configuration of Orifice-Type Outlet Structure

The hydromodification control standards were applied to several case studies in the HMP. From these examples several conclusions were drawn.

- The flow duration basin sizing approach can be applied to development sites of different sizes and land use types. The area required for flow duration basins seems to be between 2 to 7 percent of the contributing catchment area, depending on the infiltration capacity of the soil

and the basin depth. Where projects have good soil infiltration rates and utilize low-impact development strategies, less land area may be required (1.5 to 2 percent).

- Basin size can be reduced by applying volume reduction or low impact design strategies. For example, disconnecting impervious area such as roof drains and discharging runoff to landscaping, bioretention areas and/or infiltration trenches reduces the difference in volume between pre- and post-project runoff that need to be controlled in the basin.
- Qcp should be incorporated in every basin design but its relative impact on basin sizing is dependent on project size and soil infiltration rates. The Qcp rates for small basins may require small orifice diameters as small as 2 inches.

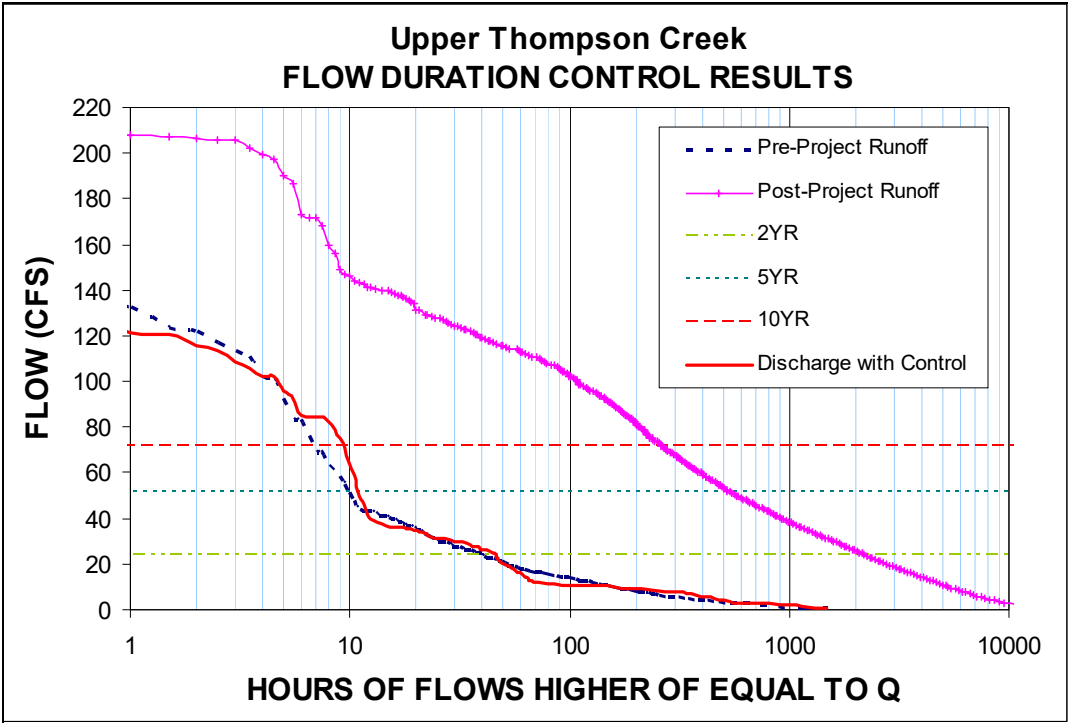
Hydrologic Models

The first two steps in designing a flow control facility are simulating the runoff from the project site, pre- and post-project, using a continuous rainfall record and generating flow-duration curves (see Figure 3 for example flow duration curves from continuous simulation modeling). Both of these steps are done using a hydrologic model. The HMP studies concluded that hydromodification controls designed for a discrete or design storm event do not provide adequate protection of the erosion potential of streams. The recommended method for hydromodification control is to maintain the pre-project flow duration curve via a flow duration control structure.

There are several hydrologic models available as public domain software that can be used². The Program, in cooperation with the Alameda, San Mateo, and Contra Costa Countywide stormwater programs, funded the adaptation of the Western Washington Hydrology Model, an automated modeling and flow control facility sizing tool, for use in the Bay Area. The completed Bay Area Hydrology Model (BAHM) is helping developers design, and municipal staff review, flow duration control facilities for HM compliance. To obtain more information and to download the BAHM and User Manual, go to <https://www.clearcreeksolutions.info/bahm-download-page>.

Army Corps of Engineers' Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), Environmental Protection Agency's (EPA's) Hydrologic Simulation Program-Fortran (HSPF), and EPA's Stormwater Management Model (SWMM).

Figure 3. Example Flow Duration Curves from Continuous Simulation Modeling



Flow Duration Basin Design Guidance

Prepared by GeoSyntec Consultants and EOA, Inc.³

LIST OF FIGURES AND TABLES

Figure F-1	Generalized Configuration of Flow Duration Basin
Figure F-2	Generalized Configuration of Orifice-Type Outlet Structure
Figure F-3	San Jose Small 12-Lot Residential Example
Figure F-4	Difference Between Pre- And Post-Project Cumulative Histograms
Figure F-5	Histogram of Discharge from the 716 Acre Test Subcatchment
Figure F-6	Example Flow Duration Curves
Figure F-7	3-Tier Sharp-Crested Rectangular Weir (A) and Circular Orifice (B) Outlet Designs
Figure F-8	Flow Duration Control Results
Figure F-9	Flow Duration Curve Results – Log-Log Scale
Figure F-10	Stage-Discharge Curves
Figure F-11	Continuous Curve Outlet Design
Figure F-12	Alternative Outlet Designs
Table F-1	Resulting Flow Duration Basin Characteristics

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

³ From SCVURPPP Hydromodification Management Plan – Final Report, Appendix F, April 2005. Available at www.sevurppp.org.

receiving stream (at the critical flow for basin design, Q_{cp} , 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 Q_{cp} (see next section for computation of Q_{cp}).

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.

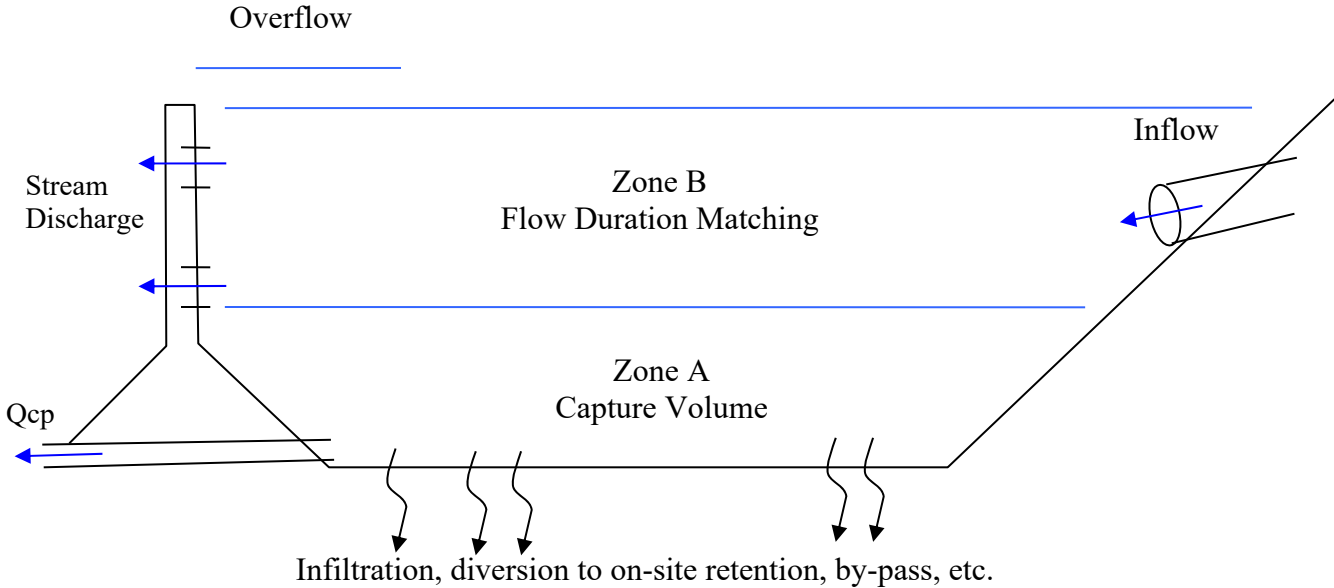


Figure F-1. Generalized Configuration of Flow Duration Basin

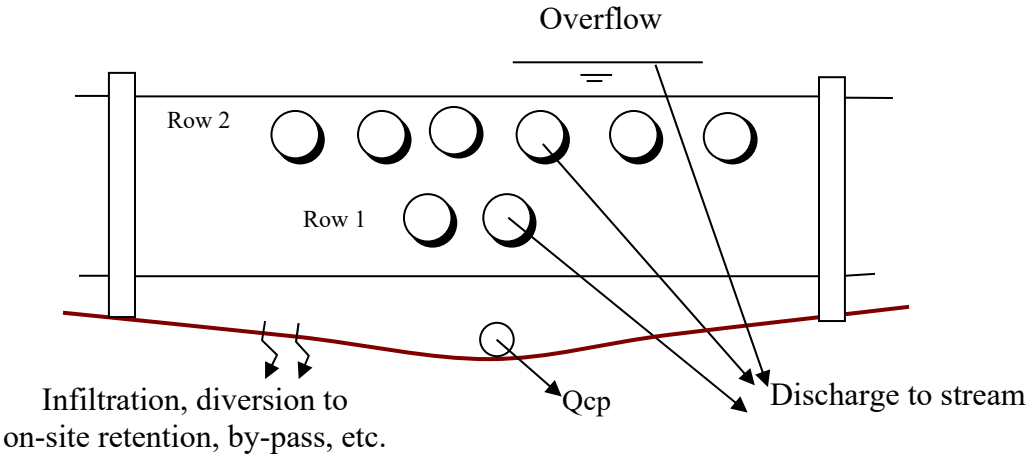


Figure F-2. Generalized Configuration of Orifice-Type Outlet Structure

DETERMINATION OF ALLOWABLE LOW FLOW DISCHARGE, Q_{CP}

The critical flow of a stream (Q_c) 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 Q_c, 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, Q_{cp}, can be calculated as 10% of the pre-project 2-year peak flow from the site. In computing Q_{cp}, 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.

Q_{cp} 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:

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

and

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

Therefore, the design Q_{cp} 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 histogram⁴ 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. The cumulative frequency distribution is the flow duration curve.
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 ($Q_{cp} = 10\%$ 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 Q_{cp} 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 Q_{cp} . 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 $\frac{3}{4}$ 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 $\frac{7}{8}$ of the effective depth of the basin. The combined first and second orifice

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 (Q_{cp}); 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, then 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 unlimited⁵. 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 Q_{cp} 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

San Jose Small 12-lot Residential Example

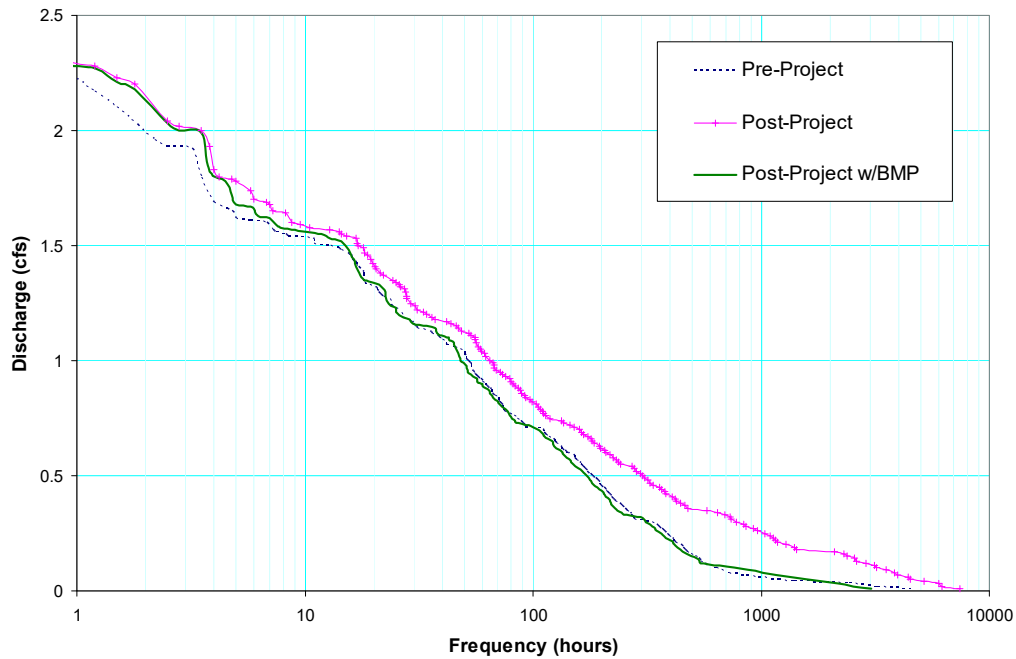
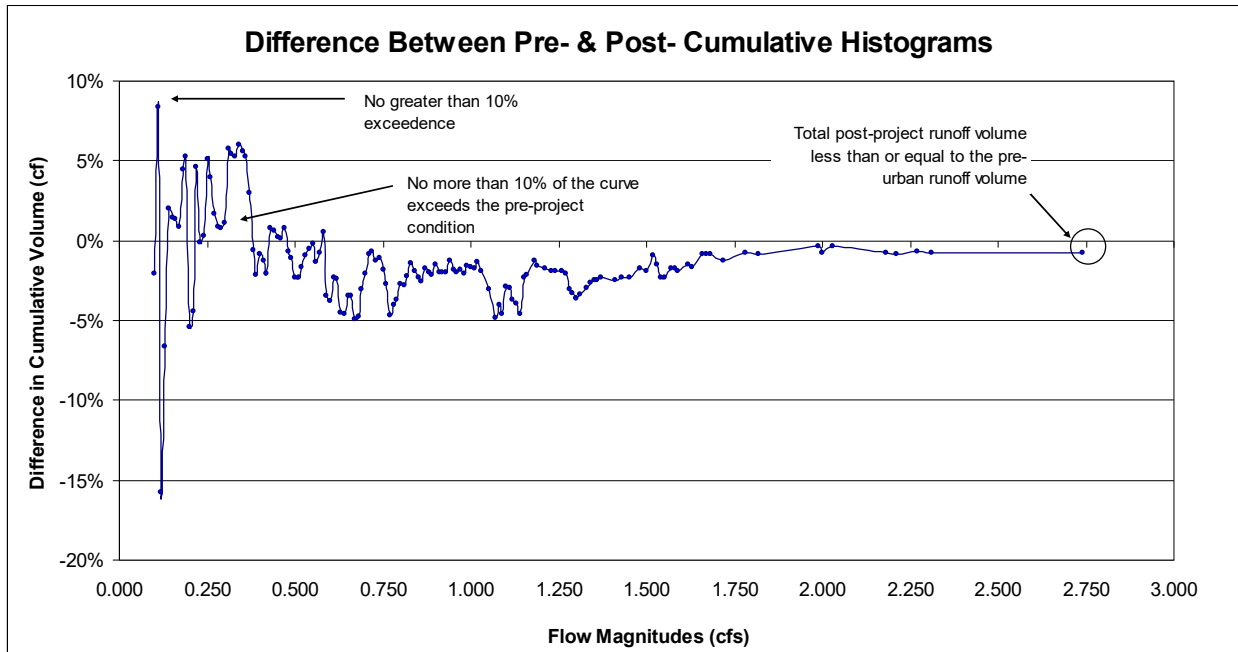


Figure F-4

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 Q_c	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	<math><1</math> day	3.6 days
Q_c (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.

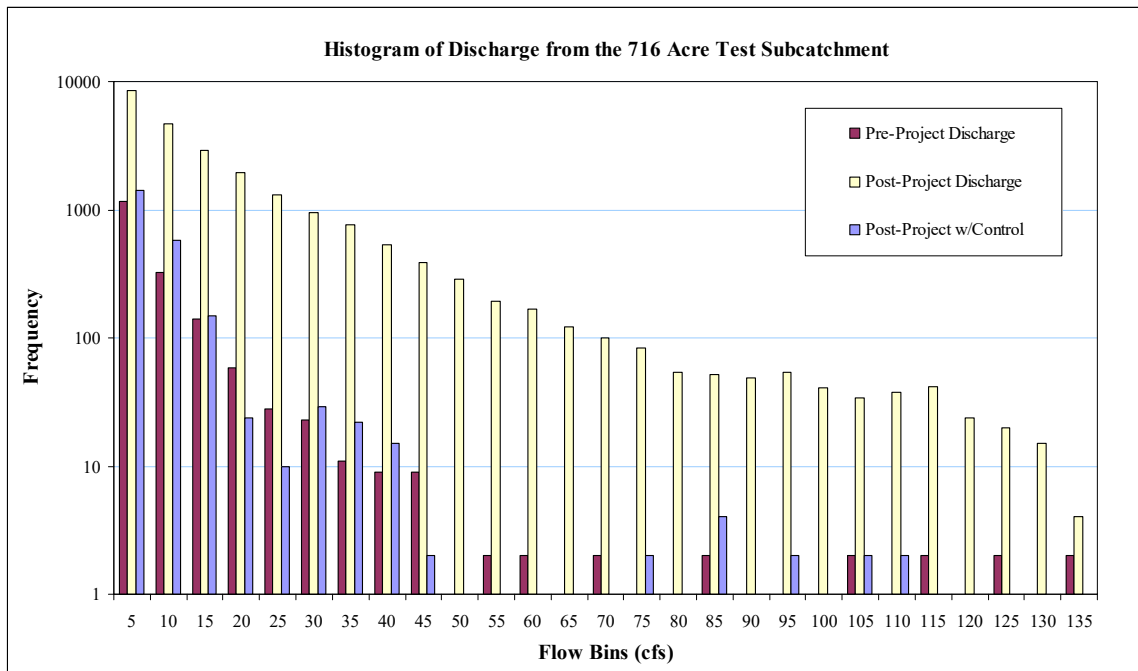
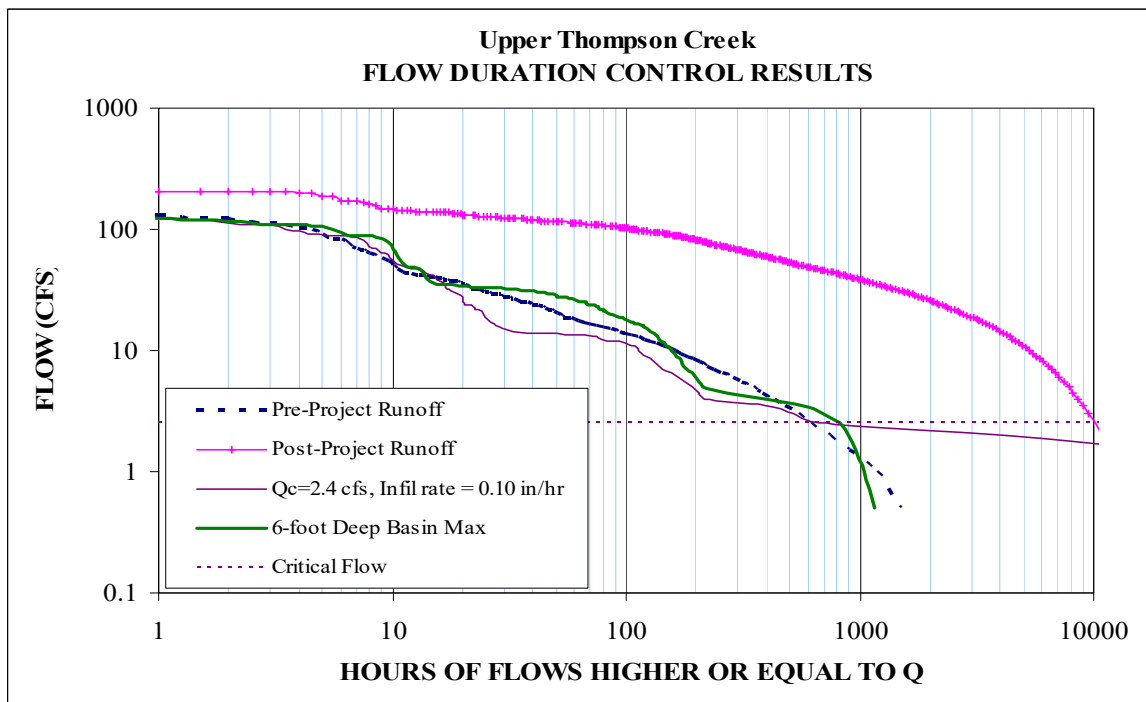


Figure F-5. Histogram of Discharge from the 716 Acre Test Subcatchment

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.



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.

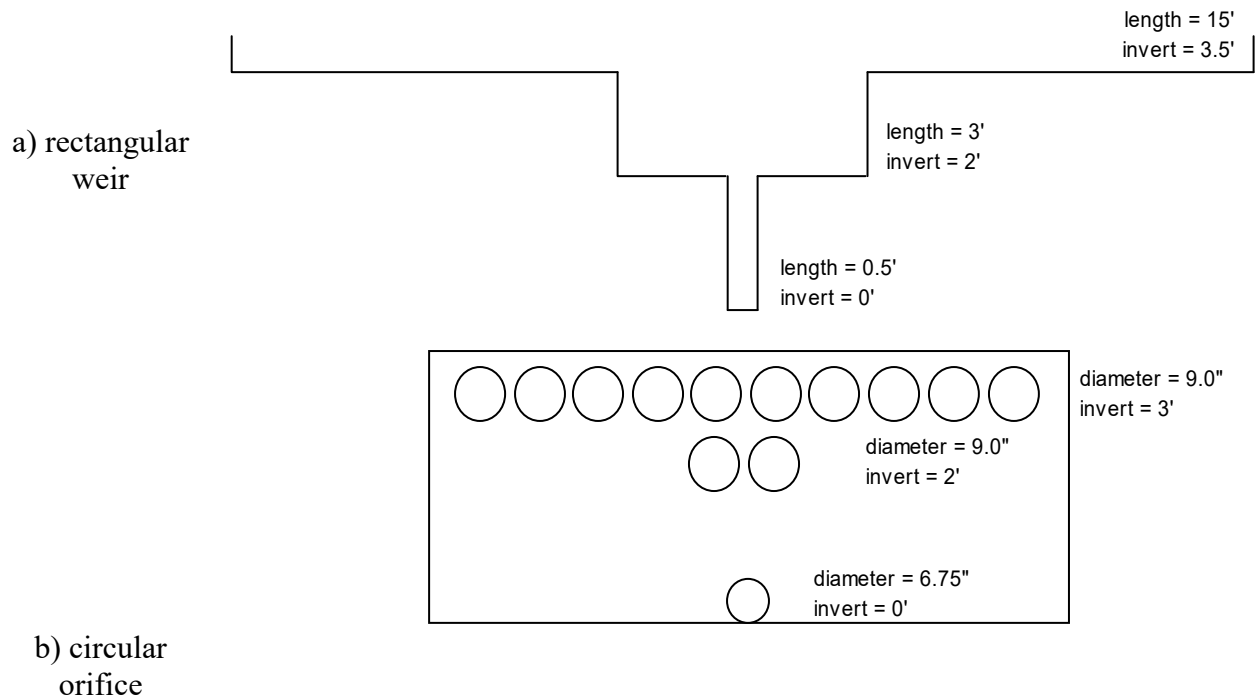


Figure F-7: 3-Tier Sharp-Crested Rectangular Weir (A) and Circular Orifice (B) Outlet Designs

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 (Q_{cp}) (2.4 cfs in this simulation).

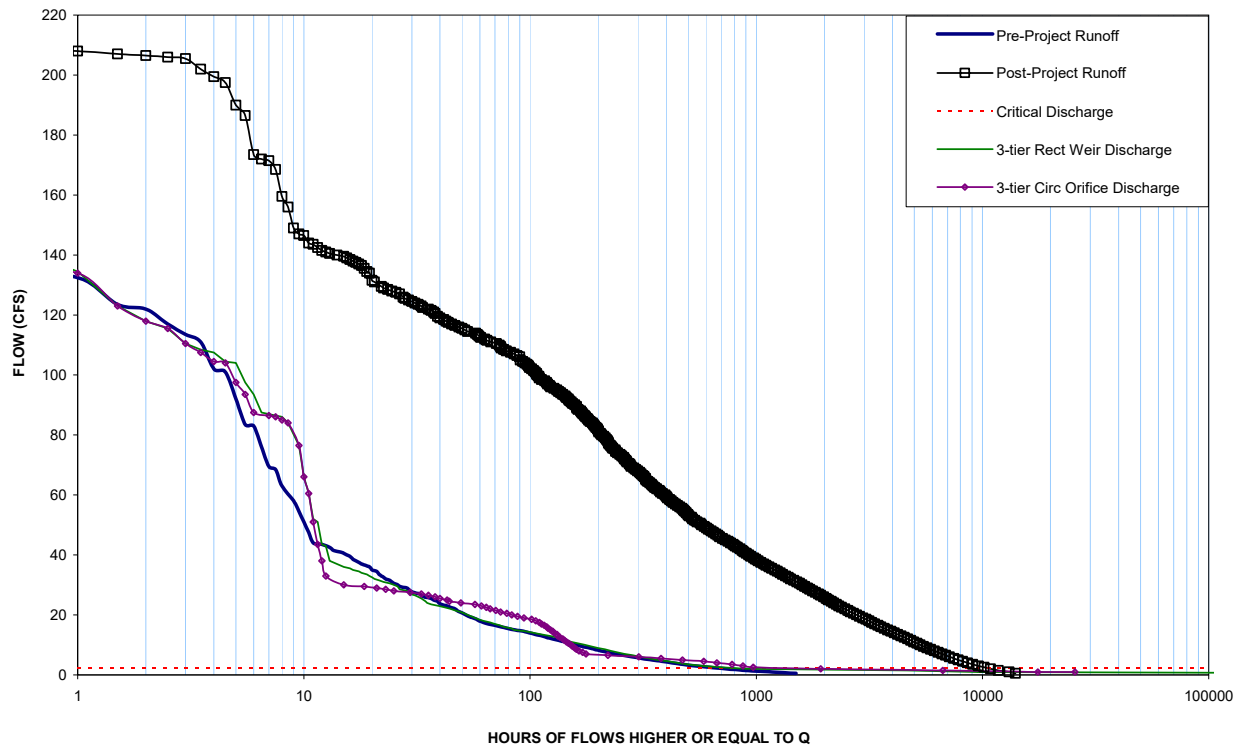


Figure F-8. Flow Duration Control Results

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.

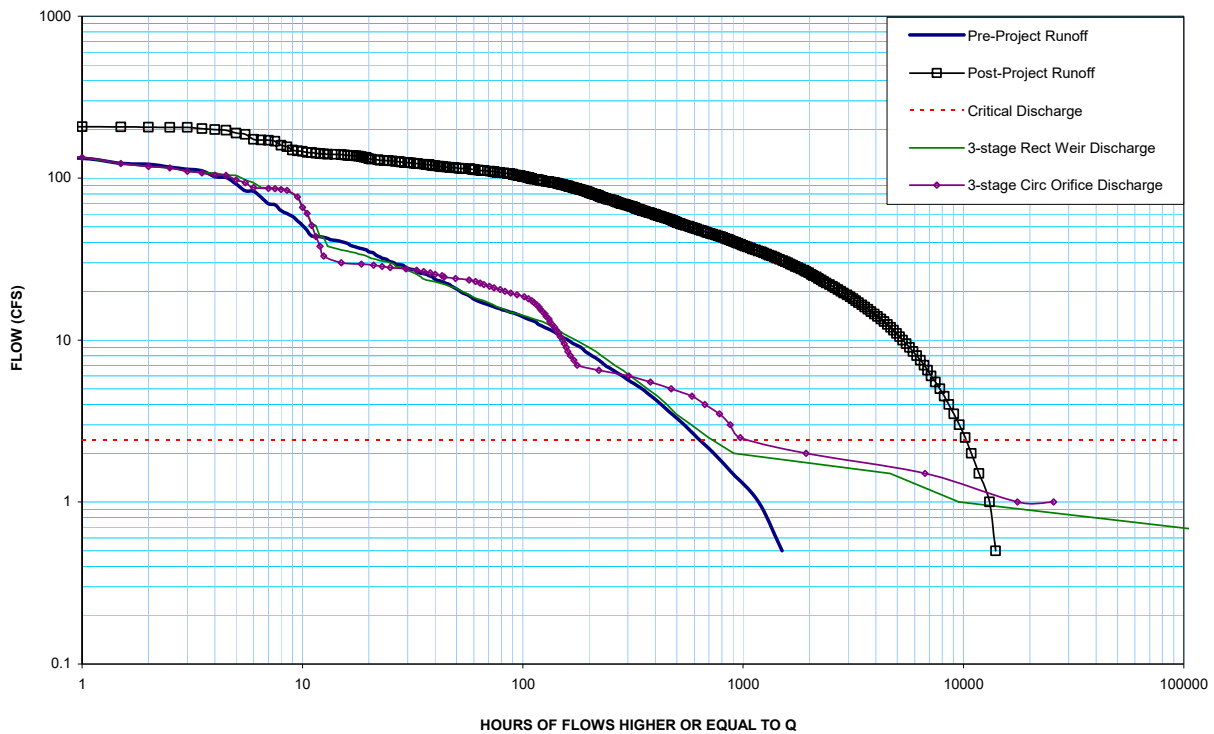


Figure F-9. Flow Duration Curve Results – Log-Log Scale

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.

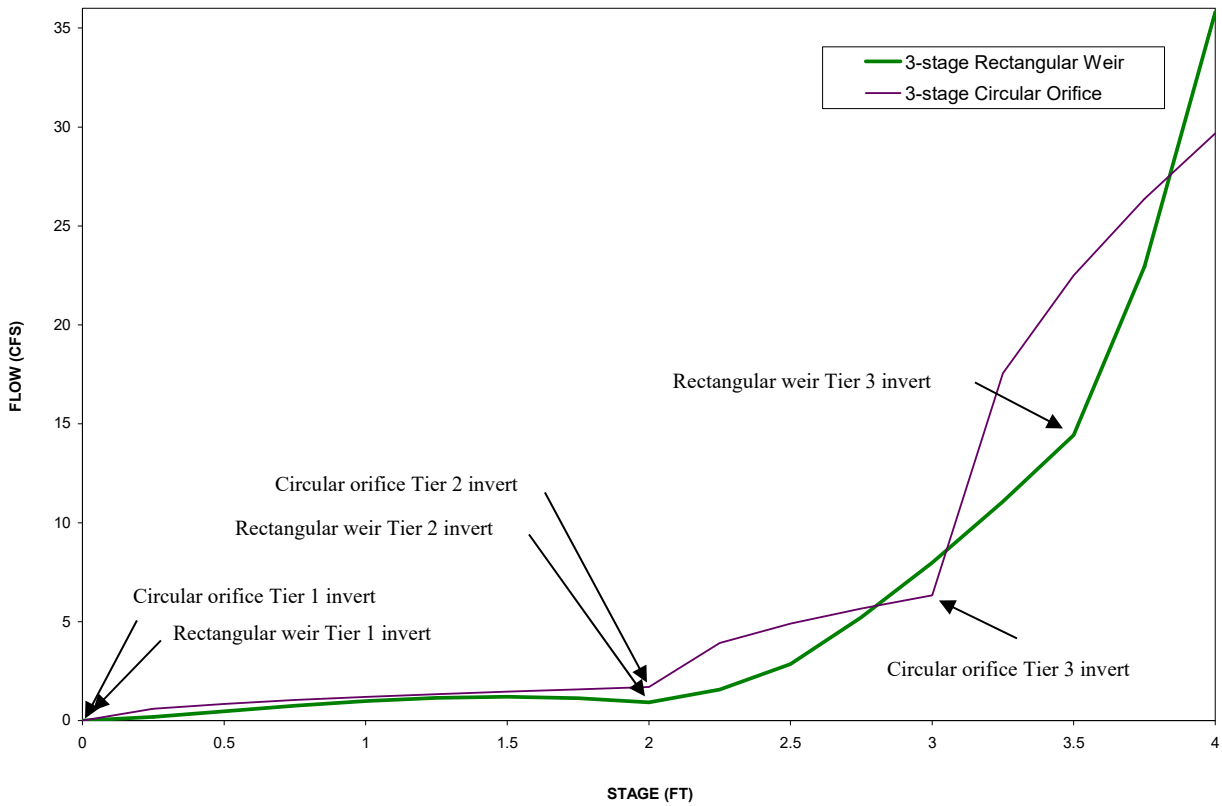


Figure F-10. Stage-Discharge Curves

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.

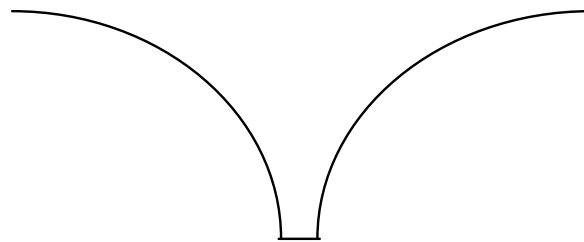
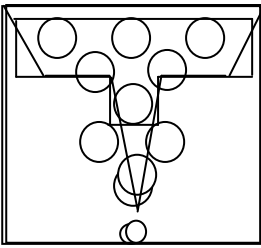
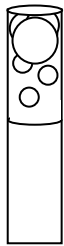


Figure F-11. Continuous curve outlet design



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)¹. 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.

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_c) is calculated as follows:

$$L_c = L - 0.1nH$$

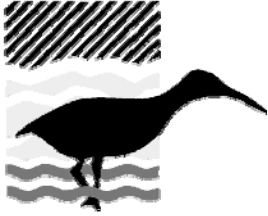
where: L_c = effective length of weir crest (ft)
 L = measured length of weir crest (ft)
 n = number of contractions (2)
 H = head above crest (ft)

From this equation, the effective length of L_c 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.

$$Q = C L_c H^{3/2}$$

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.



APPENDIX E-4

**The Bay Area Hydrology Model – A Tool for Analyzing
Hydromodification Effects of Development Projects and Sizing
Solutions**

The Bay Area Hydrology Model – A Tool for Analyzing Hydromodification Effects of Development Projects and Sizing Solutions

Jill Bicknell, P.E., EOA, Inc., Santa Clara Valley Urban Runoff Pollution Prevention Program¹

Doug Beyerlein, P.E., Clear Creek Solutions, Inc.²

Arleen Feng, P.E., Alameda Countywide Clean Water Program³

Abstract

The California Regional Water Quality Control Board, San Francisco Bay Region, is requiring stormwater programs to address the increases in stormwater runoff rate, volume, and duration created by new and redevelopment projects, known as hydromodification, where those increases could cause erosion of receiving streams. Municipal stormwater discharge permits in the San Francisco Bay area contain a requirement for stormwater programs to develop Hydro-modification Management Plans (HMPs) that describe how each program's agencies will meet this requirement.

The hydromodification control standard established in municipal permits is that post-project runoff shall not exceed pre-project rates and/or durations, over a defined range of storm event sizes. Research has shown that, to develop effective measures for control of changes in flow duration, the changes in a project site's hydrology cannot be evaluated for a single storm event with traditional design storm approaches. The change in hydrology must be evaluated over a longer time frame using a continuous simulation hydrologic model, and the results used to size control measures to match pre-project flow duration patterns. These analysis methods require specialized expertise and are difficult for many developers' engineers to perform and for municipal staff to review.

During development of their HMPs, three stormwater programs in the southern San Francisco Bay area, the Santa Clara Valley Urban Runoff Pollution Prevention Program, the Alameda Countywide Clean Water Program, and the San Mateo Countywide Storm Water Pollution Prevention Program, recognized this problem and decided to jointly fund development of a tool to simplify the analysis of hydromodification effects and to help design flow control measures. The tool, known as the Bay Area Hydrology Model (BAHM), is a Bay area version of the Western Washington Hydrology Model developed by Clear Creek Solutions for the Washington State Department of Ecology. It consists of a user-friendly graphical interface through which the user inputs information about the project and desired control measure (e.g., detention basin or underground vault); an engine that automatically loads appropriate parameters and meteorological data and runs the continuous simulation model HSPF to generate flow duration curves; a module that sizes the control measure to achieve the hydromodification control standard; and a reporting module. The tool uses parameters that have been calibrated for two watersheds in Alameda County, and is in the process of being calibrated for two watersheds in Santa Clara County.

This paper describes the background and need for the BAHM, development of the BAHM and appropriate parameters for the southern Bay Area, and examples of the application of the tool to size hydromodification control facilities for two development projects.

¹ 699 Town & Country Village, Sunnyvale, CA 94086, 408-720-8811, jcbicknell@eoainc.com

² 15800 Village Green Drive #3, Mill Creek, WA 98012, 425-892-6454, beyerlein@clearcreeksolutions.com

³ 951 Turner Court, Room 300, Hayward, CA 94545, 510-670-5575, Arleen@acpwa.org

Introduction

Urbanization of a watershed modifies natural watershed and stream processes by altering the terrain, modifying the vegetation and soil characteristics, introducing pavement and buildings, installing drainage and flood control infrastructure, and altering the condition of stream channels through straightening, deepening, and armoring. These changes affect hydrologic characteristics in the watershed (rainfall interception, infiltration, runoff and stream flows), and affect the supply and transport of sediment in the stream system. The change in runoff characteristics from a watershed caused by changes in land use conditions is called hydrograph modification, or simply hydromodification.

As the total area of impervious surfaces increases in previously undeveloped areas, infiltration of rainfall decreases, causing more water to run off the surface as overland flow at a faster rate. Storms that previously didn't produce runoff under rural conditions can produce erosive flows. The increase in the volume of runoff and the length of time that erosive flows occur ultimately intensify sediment transport, causing changes in sediment transport characteristics and the hydraulic geometry (width, depth, slope) of channels. The larger runoff durations and volumes and the intensified erosion of streams can impair the beneficial uses of the stream channels.

The California Regional Water Quality Control Board (Water Board), San Francisco Bay Region, is requiring stormwater programs to address the increases in runoff rate and volume from new and redevelopment projects where those increases could cause increased erosion of receiving streams. The Phase 1 municipal stormwater permits in the Bay Area contain requirements to develop and implement hydromodification management plans (HMPs) and to implement associated management measures.

The first Bay Area permit to include the new requirements was that of the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP)⁴. SCVURPPP conducted an assessment of hydromodification impacts on streams tributary to South San Francisco Bay and developed an HMP Report⁵ that describes how SCVURPPP agencies will meet this requirement. On July 20, 2005, the Water Board adopted key provisions of the HMP Report and required implementation of the provisions within three months.

Other Bay Area stormwater permits that contain the requirement to develop and implement HMPs include those of the Alameda County, San Mateo County, Contra Costa County, and Fairfield-Suisun area stormwater programs⁶. The Contra Costa HMP was adopted by the Water Board on July 12, 2006. The other stormwater programs have submitted final HMPs to the Water Board and are awaiting review and approval.

Permit Requirements

Provision C.3.f. of the NPDES permit, *Limitation on Increase of Peak Stormwater Runoff Discharge Rates*, describes the HMP requirements. Under Provision C.3.f., municipalities are required to develop an HMP to describe how they plan to manage increases in the magnitude, volume, and duration of runoff from new development and significant redevelopment projects in order to protect streams from increased potential for erosion or other adverse impacts. The

⁴ SCVURPPP consists of the thirteen cities of Santa Clara Valley, Santa Clara County, and the Santa Clara Valley Water District (SCVWD), all of which are Co-permittees on a joint NPDES permit to discharge stormwater to South San Francisco Bay.

⁵ SCVURPPP, *Hydromodification Management Plan, Final Report*, April 2005 (www.scvurppp.org)

⁶ These programs together comprise about 60 additional municipal or county co-permittees.

requirements apply to development projects that create and/or replace 1 acre or more of impervious surface area.

In implementing the HMP, runoff controls⁷ must be designed so that “post-project runoff shall not exceed estimated pre-project rates, durations, and volumes from the development site” (Provision C.3.f.i). Runoff controls are not required for projects that discharge stormwater runoff where the potential for erosion, or other impacts to beneficial uses, is minimal. Such situations may include: discharges into creeks that are concrete-lined or significantly hardened (e.g., with rip-rap, sack concrete, etc.) downstream to their outfall in San Francisco Bay; underground storm drains discharging to the Bay; and construction of infill projects in highly developed watersheds, where the potential for single-project and/or cumulative impacts is minimal (Provision C.3.f.ii).

The permit also requires: completion of a literature review; development of a protocol to evaluate hydromodification impacts to downstream watercourses; identification of an appropriate limiting rainfall event or range of events; a description of how municipal agencies will incorporate the HMP requirements into local approval processes; and guidance on management practices.

Technical Analysis of Hydromodification Controls

SCVURPPP and its consultant team completed a number of technical analyses to address key issues for the HMP, such as the effectiveness of various flow control techniques, the range of storm events to be considered for HMP criteria, and examples of flow duration basin sizing for local projects⁸. The key findings of these analyses, which served as the basis for developing performance criteria for the HMP, are described below.

Effective Design Approaches

It has been previously demonstrated that control of peak flows alone is not adequate for erosion control (MacCrae, 1996). SCVURPPP’s studies (GeoSyntec, 2004, TMs #5 and 7) showed that hydromodification controls designed for discrete event volume control or design storm hydrograph matching do not provide adequate protection of receiving streams. The recommended effective method for hydromodification control is *flow duration control*. This approach involves maintaining the magnitude and duration of post-project flows at the same level as the pre-project flows (i.e., matching the long term pattern of flow rates and the number of hours they occur) via a flow duration control structure, for the full distribution of flows within a significant range. The flow duration approach considers the entire multi-year discharge record, as opposed to a single event. Flow controls should be supplemented by site design measures that reduce the amount of post-project runoff generated at the site.

Range of Storms to Manage

An evaluation was performed of the range of flows that are the most important for stream channel erosion and hydromodification impacts in Santa Clara Valley (GeoSyntec, 2004, TM#4). The evaluation was based on watershed assessments conducted for three subwatersheds in the Valley. The lower limit of the range is based on the critical flow (Q_c) in each stream reach that initiates erosion of the stream bed or bank. For all three subwatersheds, Q_c could be approximated as 10% of the 2-year pre-development peak flow. To partition this allowable flow

⁷ This document uses the term runoff controls or flow controls to refer to Best Management Practices (BMPs) that reduce impacts of runoff volume, rate, and duration. Runoff controls that remove pollutants from stormwater will be referred to as treatment controls.

⁸ Technical memoranda describing these analyses are available in Appendix C of the HMP Report (www.scvurppp.org).

among contributing land areas, an on-site project design criteria of 10% of the pre-project 2-year peak flow was proposed as the allowable low flow from a flow control facility.

The upper limit on the range of storms was determined by evaluating the contribution of different flow magnitudes to the total amount of erosive “work”⁹ done on the stream bed and banks over a period of time. The low flows contribute the most work over time, whereas high flows contribute less work because they occur less frequently. Approximately 90-95% of the total work on the channel boundary is done by flows between Q_c and the pre-development 10-year peak flow magnitude. Flows greater than the 10-year peak flow contribute less than 10% of the total work. Thus, the 10-year pre-project peak flow was selected as the practical upper limit for controlling erosive flows.

Hydromodification Management Performance Criteria and Design Approach

As stated earlier, Permit Provision C.3.f. requires that post-project runoff shall not exceed pre-project rates and/or durations, where the increased rates and durations will result in increased potential for erosion in the receiving stream. All of the Bay Area stormwater program HMPs include performance and applicability criteria to meet this requirement. These criteria will be used by local agencies as part of the development plan review process to manage hydromodification impacts of development projects.

A common theme among the various HMPs is that applicable projects with on-site flow control facilities that are designed to provide flow duration control to the pre-project condition are considered to comply with the HMP. Currently, most of the HMPs contain the following performance criterion: *Flow duration controls shall be designed such that post-project stormwater discharge rates and durations match pre-project discharge rates and durations from 10% of the pre-project 2-year peak flow up to the pre-project 10-year peak flow.*¹⁰

On-site flow controls include site design techniques, treatment controls that have the added effect of reducing flow (normally via infiltration), and flow control structures. Examples of site design features (also known as low impact development (LID) techniques) include minimizing impervious surface areas, preserving natural areas, limiting development especially where native soils have good infiltration characteristics, directing roof runoff to bioretention areas, and using vegetated swales in lieu of traditional underground storm drains. Flow control structures are generally detention/retention basins or underground vaults or tanks fitted with outlet structures such as weirs and/or orifices to control outflow rate and duration. Flood control and water quality treatment facilities can be combined with flow control structures; for example, water quality detention basins and wet ponds can be modified to provide hydromodification control.

The basic approach for design of flow control structures to meet hydromodification requirements involves: 1) simulating the runoff from the project site, pre- and post-project, using a continuous rainfall record; 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 curve¹¹. The flow control structure is a detention facility 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

⁹ “Work” is a measure of the erosive hydraulic forces on the stream segment in excess of what the stream bed and bank materials can withstand (critical shear stress) before sediment movement occurs.

¹⁰ The matching criterion is as follows: the post-project flow duration curve may not deviate above the pre-project flow duration curve by more than 10% over more than 10% of the length of the curve.

¹¹ See SCVURPPP, *Hydromodification Management Plan, Final Report*, April 2005, Appendix F (www.scvurppp.org) for more detailed guidance on how to design facilities for flow duration control.

post-project condition. This captured increase in volume must be discharged in one of several ways: 1) to the ground via infiltration (and/or evapotranspiration if vegetation is present) in the basin; 2) released at a very low rate to the receiving stream (at the critical flow for basin design, or 10% of the pre-project 2-year storm); and/or 3) diverted to a safe discharge location or other infiltration site, if feasible. Figure 1 shows a schematic pond facility in which the outlet structure is a standpipe riser with various openings.

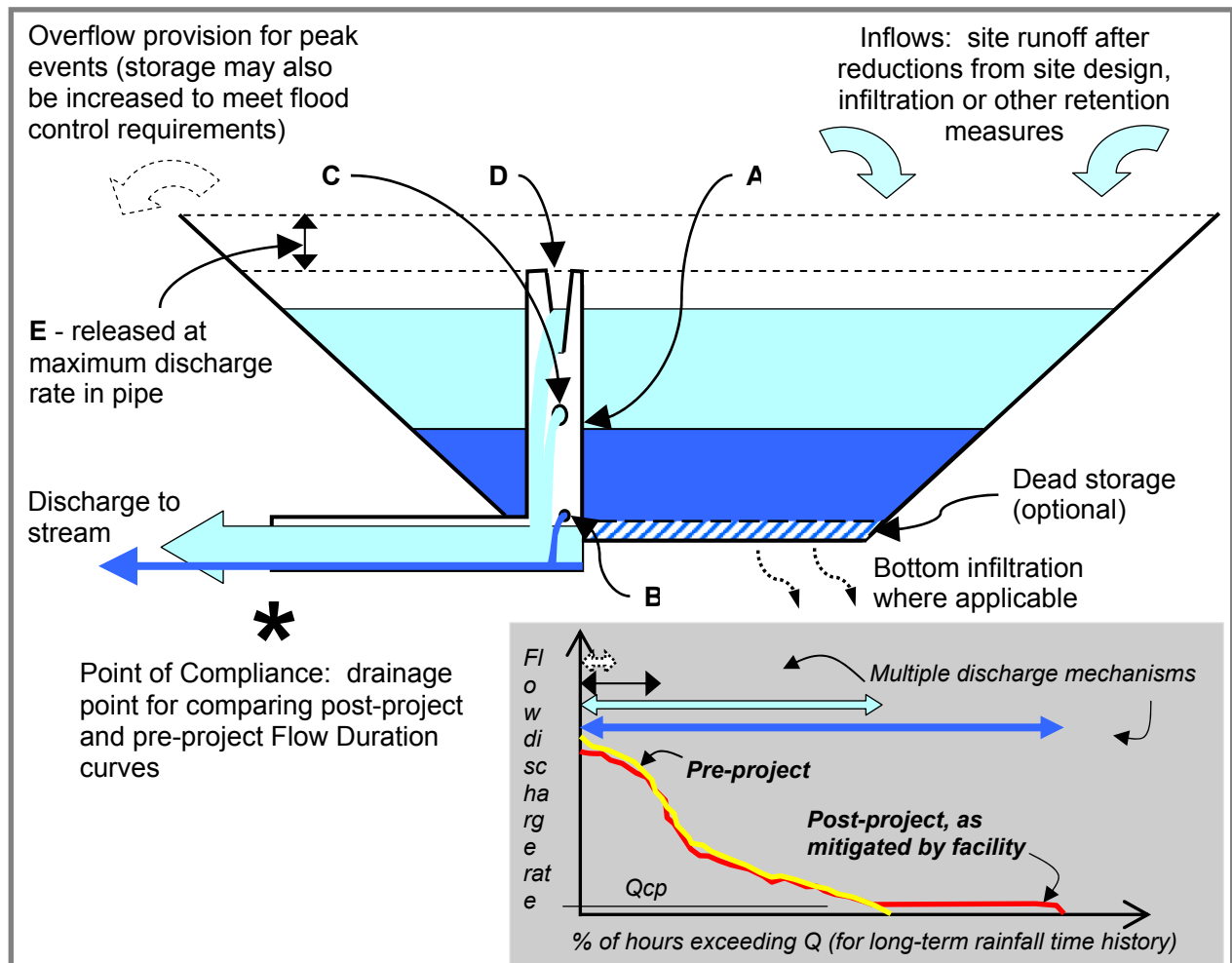


Figure 1. Schematic of flow duration pond and flow duration curves matched by varying discharge rates according to detained volume. Legend: A) outlet pipe riser; B) low flow orifice; C) intermediate orifice (1 shown); D) weir notch (V-type shown); E) freeboard above riser (typically 1 foot).

There are several public domain hydrologic models that can be used for simulating runoff for a continuous rainfall record and sizing flow control facilities. Examples are: 1) the Army Corps of Engineers' Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) which was used for the SCVURPPP analyses; 2) the U.S. Geological Survey and Environmental Protection Agency (EPA) software package called Hydrologic Simulation Program Fortran (HSPF); and 3) the EPA's Stormwater Management Model (SWMM).

Design Challenges

The concept of designing a flow duration control facility is relatively new and, as described above, requires the use of a continuous simulation hydrologic model. Development, calibration and use of such models can be data intensive and time consuming, and there is a general lack of knowledge and experience with these models among the development community and municipal staff. An additional challenge is integrating flow controls with site design and treatment controls, i.e., estimating the flow reduction benefits of site design and treatment controls and accounting for this reduction in determining the size of the flow control facility, as well as evaluating the treatment capability of the flow control facility.

To address these design challenges, SCVURPPP investigated a user-friendly, automated modeling and flow duration control facility sizing tool called the Western Washington Hydrology Model (WWHM) and decided to jointly fund the adaptation of this tool, in collaboration with the Alameda County and San Mateo County stormwater programs, for use in the Bay Area. The WWHM was developed in 2001 for the Washington State Department of Ecology to support Ecology's *Stormwater Management Manual for Western Washington* (Washington State Department of Ecology, 2001) and assist project proponents in complying with the Western Washington hydromodification control requirements. The adapted tool, known as the Bay Area Hydrology Model (BAHM), is being calibrated to southern Bay Area watersheds and enhanced to be able to size other types of control measures and LID techniques for flow reduction as well.

BAHM Overview

The BAHM software architecture and methodology is the same as that developed for the WWHM and uses HSPF as its computational engine¹². Like WWHM, BAHM is a tool that generates flow duration curves for the pre- and post-project condition and then sizes a flow duration control basin or vault and outlet structure to match the pre-project curve. The software package consists of a user-friendly graphical interface with screens for input of pre-project and post project conditions; an engine that automatically loads appropriate parameters and meteorological data and runs continuous simulations of site runoff to generate flow duration curves; a module for sizing or checking the control measure to achieve the hydromodification control standard; and a reporting module.

The HSPF hydrology parameter values used in BAHM are based on calibrated watersheds located in the San Francisco Bay Area. The initial phase of calibration for two Alameda County watersheds (AQUA TERRA Consultants, 2005) is described later in this paper. Currently work is ongoing to calibrate two watersheds in Santa Clara County.

BAHM uses one or more long-term¹³ local precipitation gages for each of the three South Bay counties and then scales the precipitation to the user's site using mean annual precipitation maps developed by local flood control districts or published as NOAA rainfall maps.

BAHM computes stormwater runoff for a site selected by the user. BAHM runs HSPF in the background to generate an hourly runoff time series from the available rain gauge data over a number of years. Stormwater runoff is computed for both pre-project and post-project land use conditions. Then, another part of the BAHM routes the post-project stormwater runoff through a stormwater control facility of the user's choice.

¹² The Department of Ecology developed the present Version 2 of the WWHM to incorporate user comments. The BAHM is based on WWHM Version 3 which is currently in development.

¹³ At least 30 years of record; 40 years or more are preferred.

BAHM uses the pre-project peak flood value for each water year to compute the pre-project 2-through 100-year flood frequency values¹⁴. The post-project runoff 2- through 100-year flood frequency values are computed at the outlet of the proposed stormwater facility. The model routes the post-project runoff through the stormwater facility. As with the pre-project peak flow values, the maximum post-project flow value for each water year is selected by the model to compute the developed 2- through 100-year flood frequency.

The pre-project two-year peak flow is multiplied by 10% to set the lower limit of the erosive flows, in accordance with the current HMP performance criteria¹⁵. The pre-project 10-year peak flow is the upper limit. A comparison of the pre-project and post-project flow duration curves is conducted for 100 flow levels between the lower erosive zone limit and the upper limit. The model counts the number of hours that pre-project flows exceed each of the flow levels during the entire simulation period. The model does the same analysis for the post-project mitigated flows.

Using the BAHM

BAHM input is relatively simple. The user must locate the project site on the appropriate county map (Figure 2). The user can zoom in or out on the map to find the exact location. BAHM uses this information to select the appropriate precipitation record and multiplier for this location.

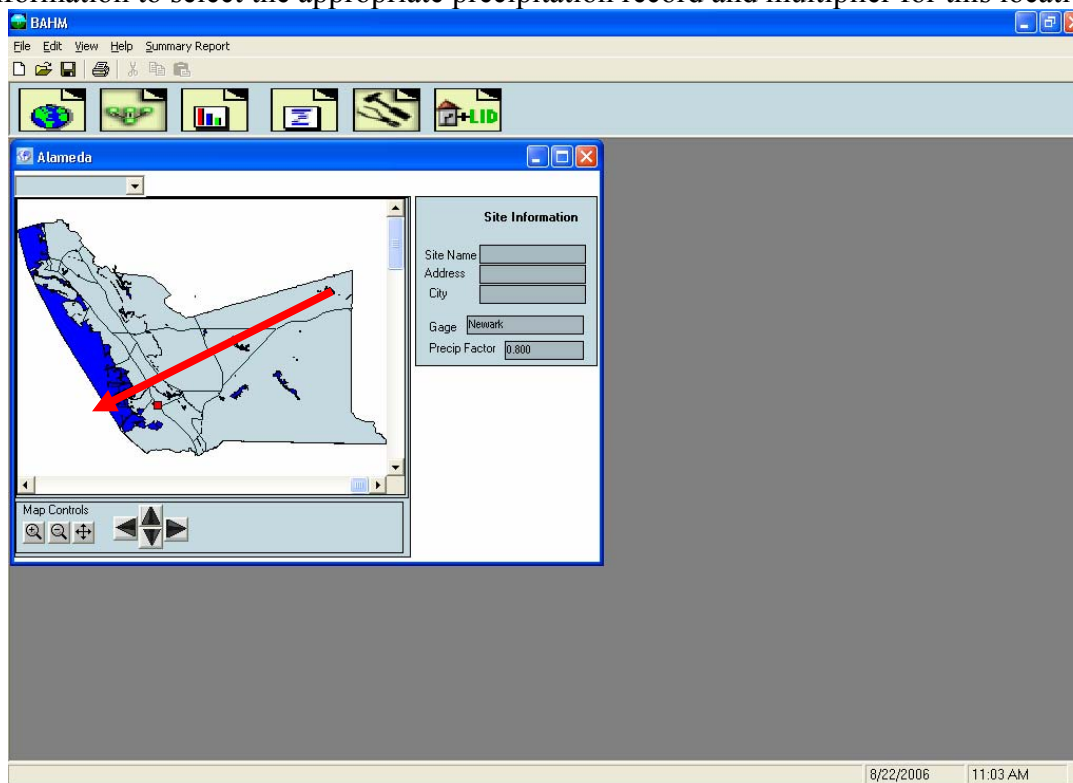


Figure 2. Example project site location.

¹⁴ The actual flood frequency calculations are made using the federal standard Log Pearson Type III distribution described in Bulletin 17B (United States Water Resources Council, 1981). This standard flood frequency distribution is provided in U.S. Geological Survey program J407, version 3.9A-P, revised 8/9/89. The Bulletin 17B algorithms in program J407 are included in the BAHM calculations.

¹⁵ In the BAHM, this low flow limit is a user-defined variable, to allow flexibility pending potential changes in regulatory requirements.

The user then goes to the Scenario Generator screen (Figures 3a and 3b) where the land use, vegetation, and soils information are specified. For the Bay Area counties, the vegetation categories are forest, shrub, grass, and urban landscape. Pre-project vegetation can be any of the first three categories. There are three major soil categories: SCS A, B, and C/D soils¹⁶. Post-project land use can include roofs, streets/sidewalks/parking, and pond in addition to the four vegetation categories.

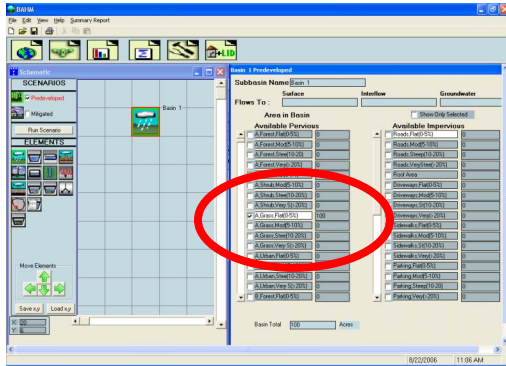


Figure 3a. Pre-project land use.

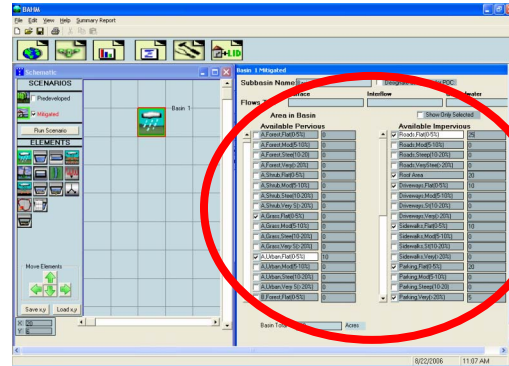


Figure 3b. Post-project land use.

The user inputs the number of acres of pre-project land use in each of the different land categories and does the same for the proposed development. For residential development there is the option to include low impact development (LID) practices such as roof runoff infiltration or dispersal and porous pavement. These LID practices reduce runoff and stormwater facility size. The user selects the type of stormwater control facility to include in the analysis. The available types are standard trapezoidal pond, tank (cylindrical, arched), vault, and irregular-shaped pond. The user can select one, two, or three orifices and a riser with a flat or notched weir (notch types include rectangular, V-shaped, and Sutro types). The facility can include infiltration, if appropriate for the site. The facility can be either manually sized to meet flow duration standards or the user can use the pond optimization feature (AutoPond) in BAHM to size the facility. An example of the BAHM pond information input form is shown in Figure 4.

¹⁶ Soil groupings based on calibration work completed to date.

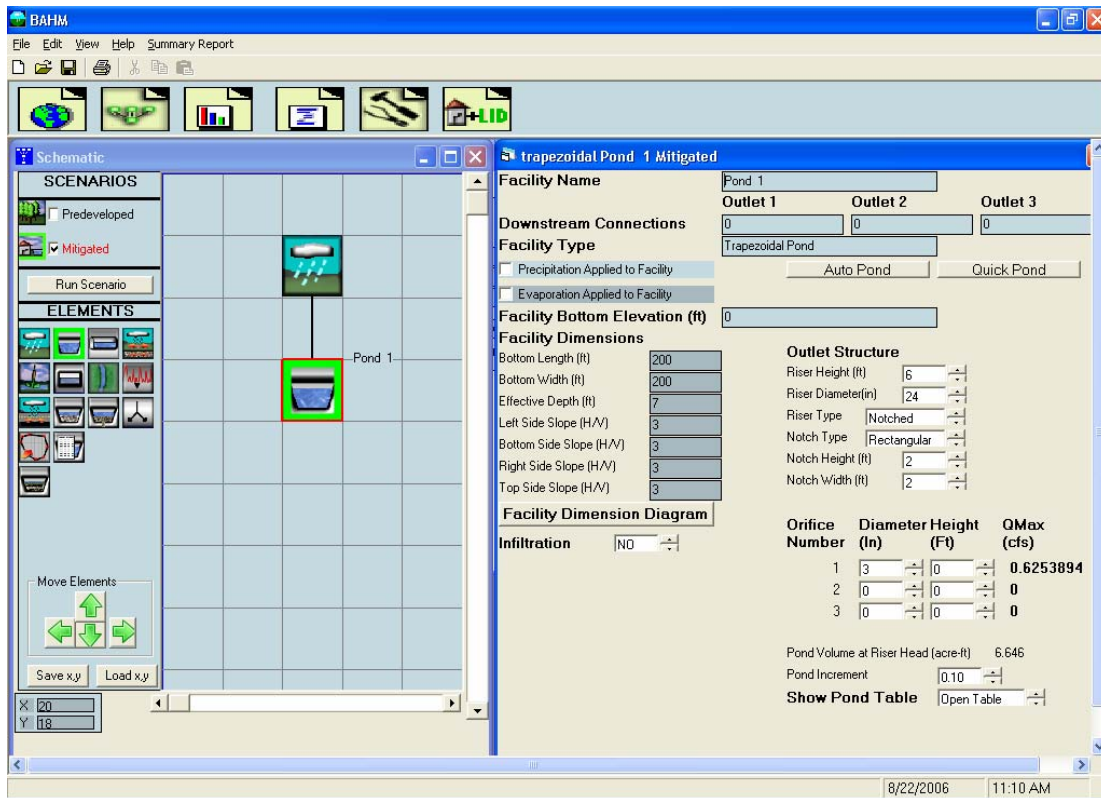


Figure 4. Pond information.

AutoPond uses a complex set of rules to select pond dimensions and outlet orifice diameters and heights. Once AutoPond has made an initial selection of pond and orifice sizes, BAHM runs HSPF to generate the long-term hourly runoff time series. The runoff is routed through the stormwater control facility and a flow duration comparison is made with the pre-project flows. If the post-project flow duration results do not pass the flow duration standard criteria then AutoPond changes dimensions and tries again. If the post-project flow duration results pass the standard then AutoPond tries to make the pond smaller. This produces the smallest (and most efficient) pond possible to meet the flow duration standard. Any time in this process the user has the option to stop AutoPond and make manual changes, if desired.

The user has the option of adding a water quality facility either upstream or downstream of the stormwater control facility. By placing the water quality facility upstream the user can take advantage of the flow moderation it provides to the control facility. This will result in a smaller stormwater control facility. Conversely, the water quality facility will have to be made larger to handle the greater variations in flows than if it is downstream of the control facility (which then moderates the flows to the water quality facility)¹⁷.

BAHM produces model output in both graphical and tabular form. The major graphical output of interest is the flow duration plot of pre-project flow and mitigated post-project flow (Figure

¹⁷ If the user wishes to design a flow control basin that will also accomplish stormwater treatment, the BAHM can be used to check the detention time in the basin to see if it meets design standards (typically 48 hours for settling fine-grained particulates).

5). All of the mitigated post-project flow values must be the same as or to the left of the pre-project values.

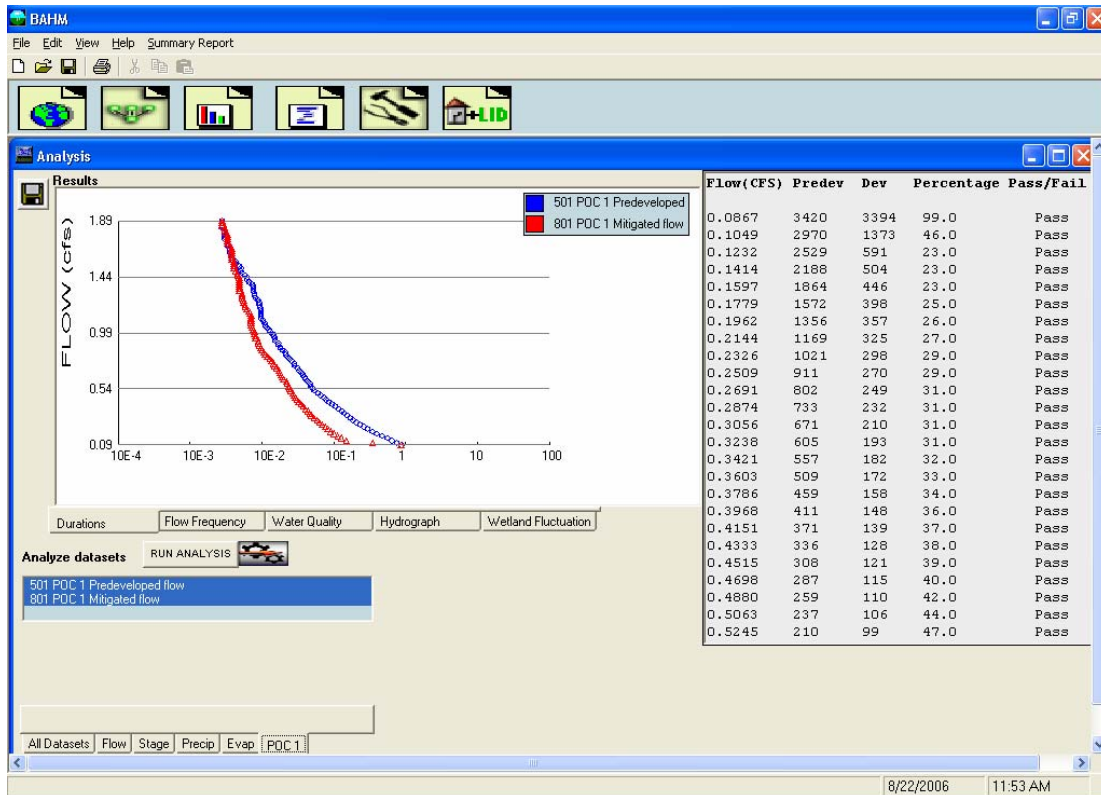


Figure 5. Flow duration comparison.

Numeric output is provided in tabular form. BAHM produces a project report that lists all of the input information. This includes the precipitation station and multiplier used, both pre- and post-project land use types and acreages, and dimensions and specifications of the flow control facility. This can be used by municipal staff to check the facility design. The user also has the option of saving the project file to disk. This project file can be later read into BAHM by the user or a reviewer to check or further modify the project.

The project report also lists the number of hours the pre- and post-project flows exceed each of the 100 flow duration levels and whether or not the flow control facility passes or fails the flow control standard for that level (Figure 5). Failure at any one of the 100 levels means the facility fails to meet the flow duration standard.

Low impact development (LID) practices have been recognized as opportunities to reduce and/or eliminate stormwater runoff at the source before it becomes a problem. They include compost-amended soils, bioretention, permeable pavement, green roofs, rain gardens, and spray irrigation. All of these approaches reduce stormwater runoff. BAHM can be used to determine the magnitude of the reduction and the amount of stormwater detention storage still required to meet HMP requirements.

BAHM explicitly includes the following LID practices:

- Roof runoff dispersion on adjacent pervious land
- Bioretention
- Green roofs

Other LID practices (such as pervious pavement and amended soils), can be implicitly modeled by adjusting parameters to represent these surfaces.

BAHM Parameter Development

BAHM uses HSPF calibrated parameter values to accurately compute stormwater runoff for the range of land use, soil, topographic, and climatic conditions found in the southern Bay area counties. Since it is not appropriate to use parameter values from other parts of the country, the participating stormwater programs are sponsoring calibration activities to support BAHM development.

For the ACCWP-sponsored phase, a review of Alameda County watersheds with appropriate streamflow and meteorological records was conducted, and Castro Valley Creek and upper Alameda Creek were selected as calibration watersheds (see Figure 5)¹⁸. These two watersheds encompass an appropriate range of land use, soils, vegetation, and climatic conditions that represent a significant fraction of Alameda County. The Castro Valley Creek watershed is a highly developed urban and suburban area of about 5.5 square miles with moderate precipitation averaging 20-22 inches per year. Significant and continuous base flow reflects impacts of lawn and landscape irrigation, especially during summer months. In contrast, the modeled portion of the Alameda Creek watershed is a highly rugged and almost completely undeveloped area approximately 33.5 square miles in size. Most of the drainage originates in Santa Clara County and annual precipitation averages approximately 20-24 inches though rain gauge data is sparse. The objective for deriving calibrated HSPF parameter values from these watersheds is for the model to be usable for both urban and undeveloped areas throughout the county.

To provide local calibrated parameter values, HSPF model simulations were performed for a period of 10 years for Castro Valley Creek and 7.5 years for upper Alameda Creek. The Castro Valley simulation period was divided into a 5-year calibration period and a 5-year validation period.

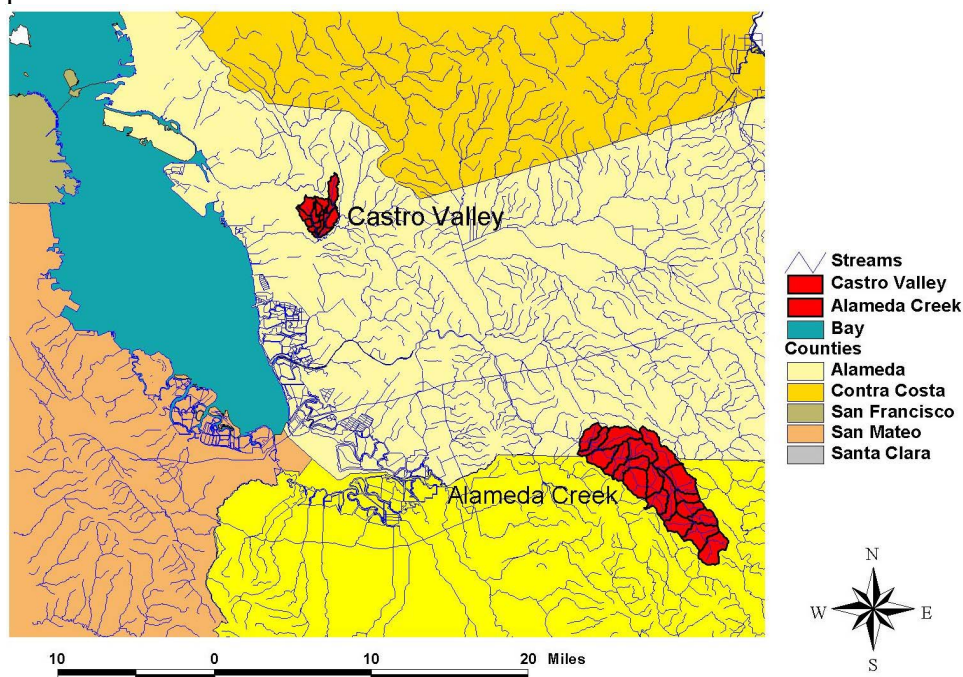


Figure 6. Castro Valley and Alameda Watersheds

¹⁸ The ACCWP-sponsored calibration report and data memorandum can be found at www.cleanwaterprogram.org.

Calibration of a watershed with HSPF is a cyclical process of making parameter changes, running the model and producing comparisons of simulated and observed values, and interpreting the results. The procedures have been well established over the past 20 years as described in the HSPF Application Guide (Donigian et al., 1984) and recently summarized by Donigian (2002).

Hydrologic simulation combines physical characteristics of a watershed and observed meteorological data to produce a simulated hydrologic response. HSPF simulates flow to the stream network from four components: surface runoff from hydraulically connected impervious areas, surface runoff from pervious areas, interflow from pervious areas, and shallow groundwater flow from pervious areas. Because historic streamflow is not divided into these four units, the relative relationship among these components must be inferred from the examination of many events over several years of continuous simulation.

Figure 7 illustrates the mean daily flow over the simulation periods in log format for Castro Valley Creek and Alameda Creek, respectively. The daily patterns shown by the model clearly reflect the observations.

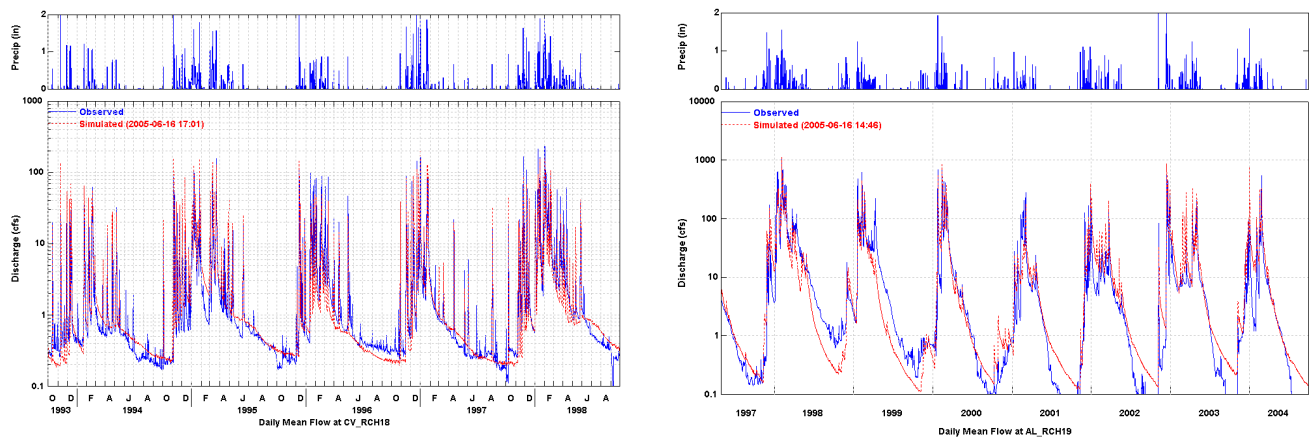


Figure 7. Calibration Daily Flow (simulated vs. observed) for Castro Valley Creek and upper Alameda Creek, with observed precipitation inputs.

Another way to look at the calibration results is to look at the statistics for different components of the streamflow record. For Castro Valley Creek, the percent differences between observed and simulated values are primarily less than 5%, with errors for some statistics in the 5-10% range but still indicating a very good calibration. Alameda Creek statistics show similar results and indicate a very good calibration for this watershed also.

Considering the quality of available data, the hydrographs and statistics indicate a very good calibration of HSPF parameter values for the Castro Valley Creek and upper Alameda Creek watersheds. The resulting model parameters were recommended for use in the Alameda County version of the BAHM. SCVURPPP has begun additional calibration modeling for two watersheds in Santa Clara County

Application of the Tool

There have not been opportunities to demonstrate BAHM applications due to its recent adoption and introduction to the local engineering community. However, in Western Washington there is now considerable experience with using BAHM's cousin, WWHM.



In the Seattle metro area WWHM was used to size the stormwater control facilities for a new Costco store. Runoff from the site drains directly to adjacent Little Bear Creek, a salmon-spawning tributary of the Sammamish River. WWHM was used to design underground stormwater storage facilities.

The Costco store site is located on 14.38 acres between SR 522 and Highway 9. An additional 2.35 acres of Highway 9 improvements were built along the store's frontage. WWHM was used to size two stormwater systems for Costco. On-site stormwater runoff (from the store and parking lot) is routed to an underground storage facility consisting of 5,240 linear feet of 96-inch diameter pipe (6.04 ac-ft of storage). The runoff from off-site Highway 9 improvements is directed to a separate underground storage system with 1,140 linear feet of 96-inch pipe (1.31 ac-ft of storage). Both systems include 6 inches of dead storage for initial water quality treatment. Additional water quality treatment was provided by Stormwater Management, Inc.'s Stormfilter units. Costco was able to meet the Washington State Department of Ecology HMP requirements on a commercial site with limited space adjacent to critical salmon habitat.



Another example is Snoqualmie Ridge, a 1,343 acre planned community in Upper Snoqualmie Valley, 30 miles east of Seattle. Over 40 percent of the community has been set aside as open space, including parks, trails, preserved wetlands and a golf course. The community includes 2,200 homes plus a business park and retail space.

Ten stormwater detention ponds were designed using WWHM to control stormwater impacts. The ponds range in size from 2 acre-feet of storage to 20 acre-feet and have been incorporated into the adjacent residential neighborhoods and golf course. The community views these ponds as visual amenities.

Conclusions

The WWHM methodology and software have been used extensively in major metropolitan areas and have been shown to be an effective tool for assisting project proponents meet regulatory requirements regarding hydromodification control. Its successor, the BAHM, will facilitate design of flow control facilities in the San Francisco Bay area, by providing a easier and more standardized way of using continuous simulation modeling and allowing computation of the benefits of site design/LID and treatment measures in reducing flows.



Furthermore, the BAHM will assist municipal agencies in their review of flow control facilities as part of development project approval.

References

- AQUA TERRA Consultants. 2005. Hydrologic Modeling of the Castro Valley Creek and Alameda Creek Watersheds with the U.S. EPA Hydrologic Simulation Program – FORTRAN (HSPF). Mountain View, CA. www.cleanwaterprogram.org
- Donigian, A.S., Jr., J.C. Imhoff, B.R. Bicknell and J.L. Kittle, Jr. 1984. Application Guide for the Hydrological Simulation Program - FORTRAN EPA 600/3-84-066, Environmental Research Laboratory, U.S. EPA, Athens, GA. 30613
- Donigian, A.S. , Jr. 2002. Watershed Model Calibration and Validation: The HSPF Experience. WEF-National TMDL Science and Policy 2002, November 13-16, 2002. Phoenix AZ, WEF-2002 Specialty Conference Proceedings on CD-ROM.
- MacRae, C.R. *Experience from Morphological Research on Canadian Streams: Is Control of the Two-Year Frequency Runoff Event the Best Basis for Stream Channel Protection. Effects of Watershed Development and Management on Aquatic Ecosystems*, ASCE Engineering Foundation Conference, Snowbird, Utah, pg 144-162. 1996.
- GeoSyntec Consultants, 2004. *Technical Memorandum #4: Evaluation of the Range of Storms for HMP Performance Criteria*. Prepared for the Santa Clara Valley Urban Runoff Pollution Prevention Program, April 1, 2004.
- GeoSyntec Consultants, 2004. *Technical Memorandum #7: Flow Duration Control Example*. Prepared for the Santa Clara Valley Urban Runoff Pollution Prevention Program, April 1, 2004.
- GeoSyntec Consultants, 2004. *Technical Memorandum #8: Sizing Flow-Duration Controls for a Small Development Project in San Jose*. Prepared for the Santa Clara Valley Urban Runoff Pollution Prevention Program, April 29, 2004.
- SCVURPPP, 2005. *Hydromodification Management Plan, Final Report*. Prepared by EOA, Inc. and GeoSyntec Consultants. April 2005. www.scvurppp.org
- United States Water Resources Council. 1981. Guidelines for Determining Flood Frequency. Bulletin #17B of the Hydrology Committee. Washington, DC.
- Washington State Department of Ecology. 2001. Stormwater Management Manual for Western Washington. Volume III: Hydrologic Analysis and Flow Control Design/BMPs. Publication No. 99-13. Olympia, WA.