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COMPARISON OF BAHM AND CONTRA COSTA APPROACHES TO HYDROMODIFICATION MANAGEMENT PLAN REQUIREMENTS

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The San Francisco Bay Regional Water Quality Control Board (Water Board) has issued municipal stormwater permit amendments that contain requirements for stormwater programs to implement Hydromodification Management (HM) requirements. The control standards established in the municipal permits generally require that post-project runoff shall not exceed pre-project rates or durations over a defined range of storm event sizes from one-tenth of the 2-year recurrence flow to the 10-year flow. The change in hydrology associated with development must be evaluated over a long timeframe using a continuous simulation hydrologic model. The results of the modeling are used to size control measures to match the pre-project flow duration patterns.

Several counties in the Bay area (Alameda, Santa Clara, San Mateo) have developed the Bay Area Hydrologic Model (BAHM) as a tool to meet HM requirements. Contra Costa County has developed sizing charts for Integrated Management Practices (IMP) to meet the requirements. BAHM is a version of the Western Washington Hydrology Model, which in turn is an implementation of the continuous simulation HSPF model with pre- and post-processing software to address sizing of stormwater controls to meet HMP requirements. The Contra Costa approach consists of pre-computed hydrographs that can be used for analysis of sizing requirements. Computation of the hydrographs in the Contra Costa approach was also done using HSPF. Thus, the BAHM and Contra Costa approaches both have applications of HSPF at their core; however, the results obtained by the two approaches differ. This memorandum summarizes investigations into the causes and implications of differences between the two methods.

1 DIFFERENCES IN PHILOSOPHY

The BAHM and Contra Costa IMP differ in their focus. Contra Costa's approach emphasizes meeting HM requirements using only onsite LID-type controls, such as bioretention and planters. Sizing factors are offered for these devices, rather than site-specific simulation. BAHM focuses on meeting HM requirements with detention ponds in combination with onsite LID devices. Performance is evaluated through direct simulation. The simulation has the ability to estimate pond size to achieve HM requirements. BAHM does not size LID devices directly. Rather, the LID devices are taken into account when auto-sizing of a pond is undertaken in a site-specific simulation.

The BAHM was developed from the Western Washington Hydrology Model (WWHM), which focuses on meeting hydromodification control requirements with ponds. Indeed, the latest version of the WWHM manual[1] does not mention bioretention or other onsite LID controls. These were added for the BAHM version to reflect California interests and practices (and are currently being added to the WWHM), but the emphasis remains on ponds. As a result, there are a number of differences in the way in which BAHM and Contra Costa simulate onsite controls.

Another significant difference in approach is that the parameters for the BAHM model applications are based on calibration to flow records from local streams, while the Contra Costa model uses reasonable, but uncalibrated parameter values. The use of uncalibrated parameters opens the model to question; however, it does not necessarily present a problem for application as long as the assumptions can be shown to be conservative.

Both approaches are valid within their intended realms of application. One interesting possibility would be to combine the approaches. That is, the Contra Costa IMP approach, which focuses on onsite controls and is easy to apply via pre-calculated sizing factors, could be applied to smaller and infill projects. The BAHM (or some other explicit simulation of continuous hydrographs using locally calibrated parameters) could be used for larger projects where the combined effects of multiple onsite and offsite controls will typically need to be evaluated. However, to use such a combined approach there should be a better reconciliation of the approaches for simulating onsite controls, as described below.

2 FACILITY SIZING

There are significant differences in the way HSPF is implemented for the BAHM and the Contra Costa methods. Most notably, the BAHM HSPF applications have been calibrated to test watersheds in individual counties, while the IMP HSPF runs are uncalibrated. As a result, the models differ in the values assigned for many individual parameters, including infiltration and interflow inflow.

Douglas Beyerlein, PE of Clear Creek Solutions (the developers of the BAHM) compared parameter values between the models and noted differences in several HSPF parameters, as summarized in Table 1.[2]: Note that the infiltration rate parameter (INFILT) on A soils was originally set to 0.70 in/hr, as cited by Beyerlein, but was subsequently revised to 0.30 in/hr according to the memorandum from Douglas P. Freitas to the Regional Board, July 2, 2007.

The differences in the last four parameters shown in Table 1 (UZSN, IRC, CEPSC, and LZETP) are small, and unlikely to cause large differences in the simulation, while the differences in the first three (INFILT, LZSN, and INTFW) are of greater concern. Beyerlein concluded that "it is expected that IMP will compute higher predevelopment/existing peak flows than BAHM. This will produce smaller-sized HMP facilities than BAHM." This conclusion had not, however, been investigated and confirmed by side-to-side comparisons. The primary reason cited by Beyerlein for his conclusion was that the INTFW parameter is much lower for the Contra Costa model, which should shift flow from subsurface to surface pathways and increase peak flow response. The differences in INFILT and LZSN parameters are likely to have a much greater impact on sizing requirements. Contra Costa's value of INFILT on Hydrologic Group A soils is much higher than BAHM (which will tend to cause more infiltration), while the value of INFILT for D soils is slightly lower than BAHM. In addition, the LZSN value used by Contra Costa is about 50 percent higher than the value used in BAHM. As the simulated infiltration rate is a function of both INFILT and the ratio of actual to nominal lower soil zone storage, the higher value of LZSN will also cause an increase in infiltration and decrease in peak runoff. Without testing it was not clear which effect would predominate. As shown below, the analysis proposed by Beyerlein is not borne out by the models; instead, Contra Costa's approach results in slightly higher storage volume requirements.

[1] Clear Creek Solutions. 2006. Western Washington Hydrology Model, Version 3.0, User Manual. Clear Creek Solutions, Inc.

[2] Beyerlein, D. 2007. Comparison of Contra Costa IMP and BAH/WWHM3/HSPF. Memorandum to file from Clear Creek Solutions, Mill Creek, WA, 2 April 2007.

Table 1. HSPF Parameter Value Comparison

Model	IMP	BAHM
Pre-development land use	Shrub, slope not differentiated	Shrub, moderate slope
INFILT A soils (infiltration rate parameter, in//hr)	0.30	0.07
INFILT D soils	0.03	0.04
LZSN A soils (nominal lower soil zone storage parameter, in)	7.0	4.8
LZSN D soils	7.0	4.5
INTFW A soils (interflow inflow parameter)	0.4	3.2
INTFW D soils	0.4	1.2
UZSN A soils (nominal upper soil zone storage parameter, in)	0.5	0.7
UZSN D soils	0.5	0.7
IRC A soils (interflow recession coefficient)	0.30	0.45
IRC D soils	0.03	0.45
CEPSC A soils (interception capacity, in)	0.06-0.10	0.13-0.15
CEPSC D soils	0.08-0.15	0.13-0.15
LZETP A soils (lower zone evapotranspiration coefficient)	0.4-0.6	0.50-0.65
LZETP D soils	0.5-0.7	0.50-0.65

Differences in the facility sizing requirements – and thus the level of channel protection - resulting from BAHM and IMP are of concern to the Water Board, and are investigated further in this memorandum. Specifically, investigation is made of application of the two different models for hypothetical development in the area of Dublin, CA, which is on the border between Alameda and Contra Costa counties. In this area, Contra Costa would use the IMP approach. Alameda would use the BAHM model.

The Alameda version of BAHM was calibrated to Castro Valley Creek and Alameda Creek by AQUA TERRA[3]. Some minor changes were subsequently made to the calibrated parameters (personal communication from Doug Beyerlein, Clear Creek Solutions, 13 August 2007), and final model parameters were extracted from the August 8, 2007 version of the BAHM model. For the IMP, HSPF pervious land parameter values are reported in Appendix A to Attachment 2 of the Contra Costa Hydrograph Modification Management Plan (15 May 2005), while impervious land parameter values are assumed to be those reported in Attachment 3. An HSPF model was set up to provide side-by-side simulations of land segments using the parameters for pervious and impervious land segments from the Contra Costa and BAHM models. The full set of Contra Costa or BAHM parameter values are specified for a land segment; however, the meteorological series are set to a single consistent basis. Comparison runs were then undertaken for 40 years (1 October 1960 – 30 September 2000), using the meteorology data series assigned by BAHM (Livermore precipitation, Calabeza potential evapotranspiration times an adjustment factor of 1.154).

BAHM considers a full range of hydrologic soil groups (A, B, C, C/D), and different slope categories. In contrast, the IMP evaluates only A and D soils (based on the argument that remaining developable land in

[3] AQUA TERRA Consultants. 2006. Hydrologic Modeling of the Castro Valley Creek and Alameda Creek Watersheds with the U.S. EPA Hydrologic Simulation Program – FORTRAN (HSPF). Submitted to Alameda Countywide Clean Water Program by AQUA TERRA Consultants, Mountain View, CA.

the county primarily falls into these categories), and does not differentiate land use/soil combinations by slope. Both simulate impervious lands, but the IMP seems to use a single category, while the BAHM differentiates by land use and slope. The IMP does not simulate developed pervious land separately, but assumes, based on tests with the uncalibrated model, that flow from developed pervious land on A soils is equal to 0.1 times the impervious flow, while flow from developed pervious land on D soils is equal to 0.7 times the impervious flow. BAHM provides a separate simulation of urban pervious lands, by soil type and slope.

As will be seen below, the IMP approach to post-project runoff from pervious land is in reasonable agreement with BAHM, at least on moderate slopes (5 – 10% slopes). The BAHM calibration, however, decreases infiltration rates and effective surface retention with increasing slopes, resulting in greater runoff. The IMP approach will therefore deviate more from BAHM in the estimation of pervious runoff as slopes increase, and will tend to underestimate the contribution of pervious runoff to the hydrograph in high slope areas. The IMP sizing factors would need to include an adjustment to account for increased runoff if they are applied for design in situations where the contributing area contains slopes greater than about 20 percent.

To provide a basis for comparison, hourly flow series, on a per-acre basis, were generated for the following land uses (focus was placed on shrubland as the pre-development condition, as this predominates in the area):

Table 2. Land Uses Simulated for Comparative Analysis

BAHM Alameda Co.	IMP Contra Costa County
Shrub on A Soils, Moderate Slope (5-10%) INFILT = 0.07, LZSN = 4.8	Shrub, A Soils INFILT = 0.30, LZSN = 7.0
Shrub on A Soils, Steep Slope (10-20%) INFILT = 0.045, LZSN = 4.5	
Urban Pervious, A Soils, Moderate Slope (5-10%) INFILT = 0.05, LZSN = 4.6	0.1 · Impervious
Shrub on C/D Soils, Moderate Slope (5-10%) LZSN = 0.035, LZSN = 3.8	Shrub, D Soils INFILT = 0.03, LZSN = 7.0
Shrub on C/D Soils, Steep Slope (10-20%) INFILT = 0.030, LZSN = 3.6	
Urban Pervious, C/D Soils, Moderate Slope (5-10%) INFILT = 0.030, LZSN = 3.6	0.7 · Impervious
Impervious: Roads, Moderate Slope (5-10%)	Impervious

The first test examined total daily volumetric flow for pre-development (shrub) land use (Figure 1). For Hydrologic Group A soils, the BAHM produces *much* higher runoff volumes than IMP, contrary to Beyerlein's inference. This occurs primarily because the infiltration rate for A soils is set much higher in the IMP model than in BAHM (0.3 vs. 0.07)[4] and this is coupled with a greater lower soil zone nominal storage (LZSN), which together amplify the amount of water lost to infiltration. These effects outweigh any differences in interflow. The IMP estimates for runoff from D soils are also lower than those from BAHM, although the difference is not as dramatic.

[4] The IMP documentation initially specified an infiltration for A soils of 0.7; however, this was subsequently revised to 0.3 according to the memorandum from Douglas P. Freitas to the Regional board, July 2, 2007. The change makes little difference to the results presented here.

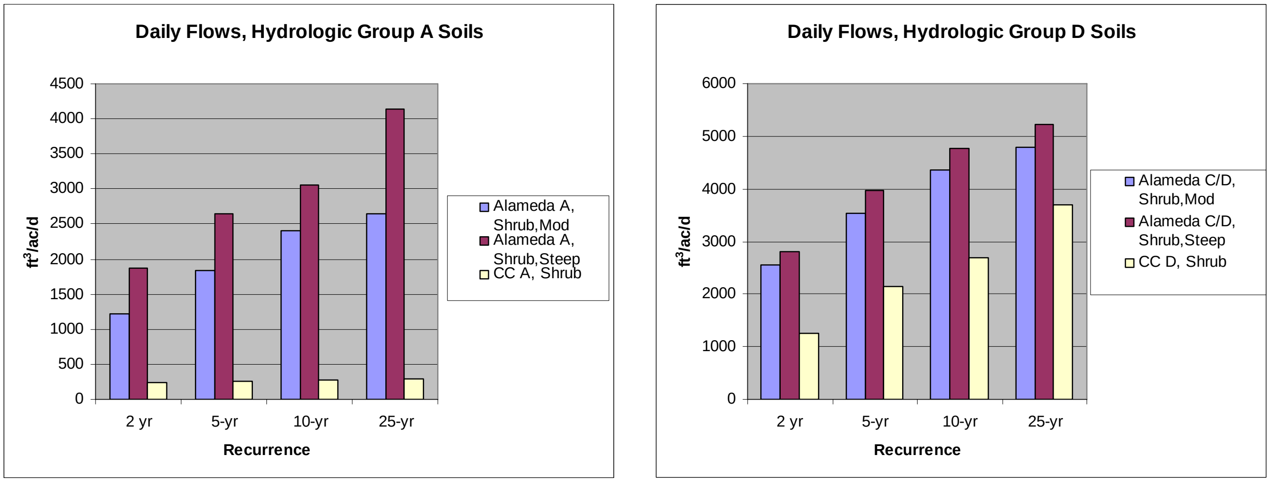


Figure 1. Comparison of Alameda BAHM and Contra Costa IMP Flow Estimates for Pre-Development Conditions

Further details on model performance can be seen by looking at hourly results for individual events (Figure 2). On A soils, the Alameda model produces a sharp response, with a trailing limb of groundwater discharge; the Contra Costa model produces almost no response (groundwater discharge is also muted, because there is an assumption that 45 percent of groundwater inflow is lost to deep aquifer storage). On D soils, the peak responses are more similar, but vary by event, while the post-peak flow remains higher for the Alameda model.

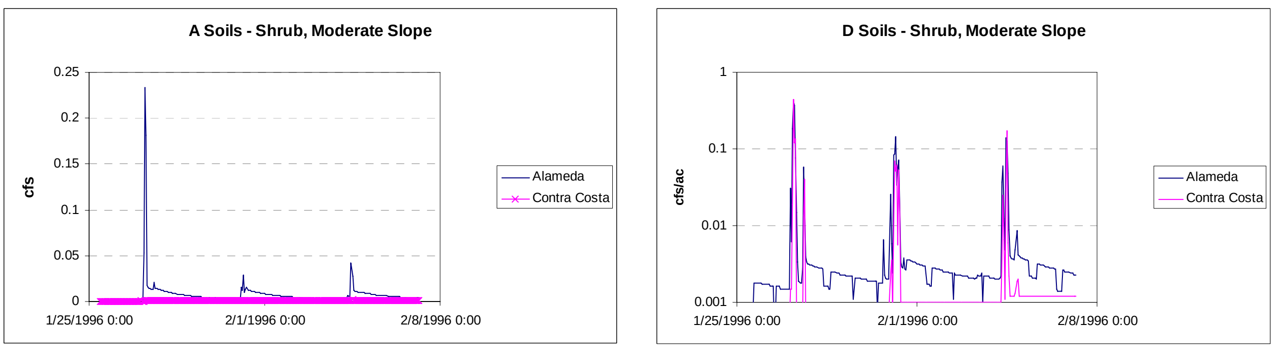


Figure 2. Comparison of Alameda BAHM and Contra Costa IMP Flow Hydrographs

Next, a hypothetical project was evaluated, assumed to be 70 percent impervious, with the remainder in urban grass. The model implementation of the BAHM and Contra Costa approaches can then be used to evaluate the post-project runoff, and the control volume needed to match the pre-development hydrograph. The post-project results for the BAHM and Contra Costa approaches are similar, despite the different parameter assumptions (Figure 3). Finally, subtracting the pre-project flows from the post-project flows yields an estimated control volume to preserve the pre-development hydrograph (Figure 4). Because of the difference in pre-development flow estimates, the Contra Costa IMP method yields a much *higher* estimate of needed control volumes. These differences at the daily scale occur mainly because of larger post-peak subsurface contributions to event flow in the Alameda model.

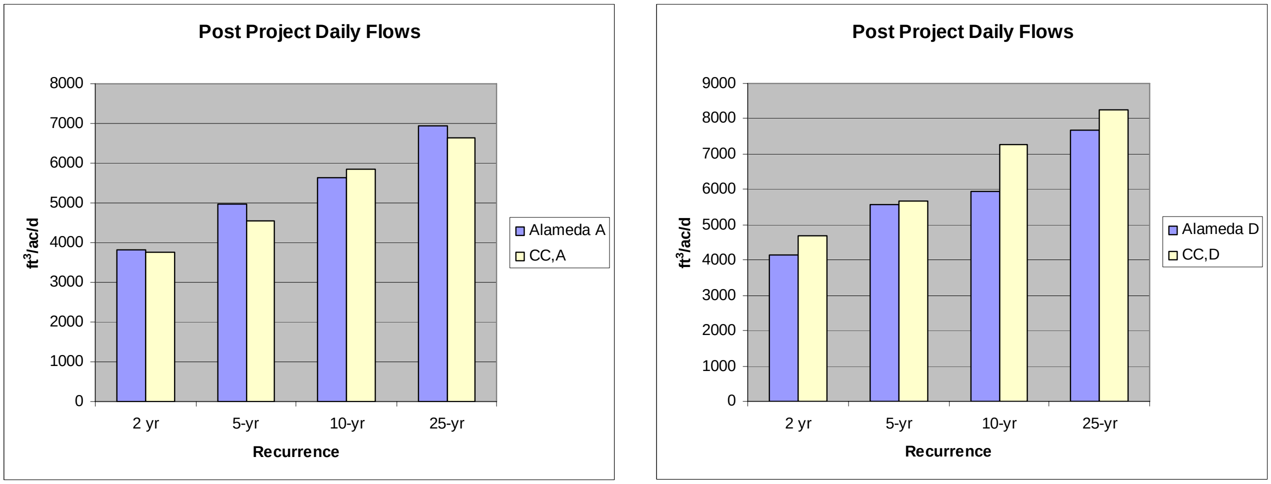


Figure 3. Comparison of Post-Project Flows for Hypothetical Development (70 Percent Impervious, Moderate Slopes)

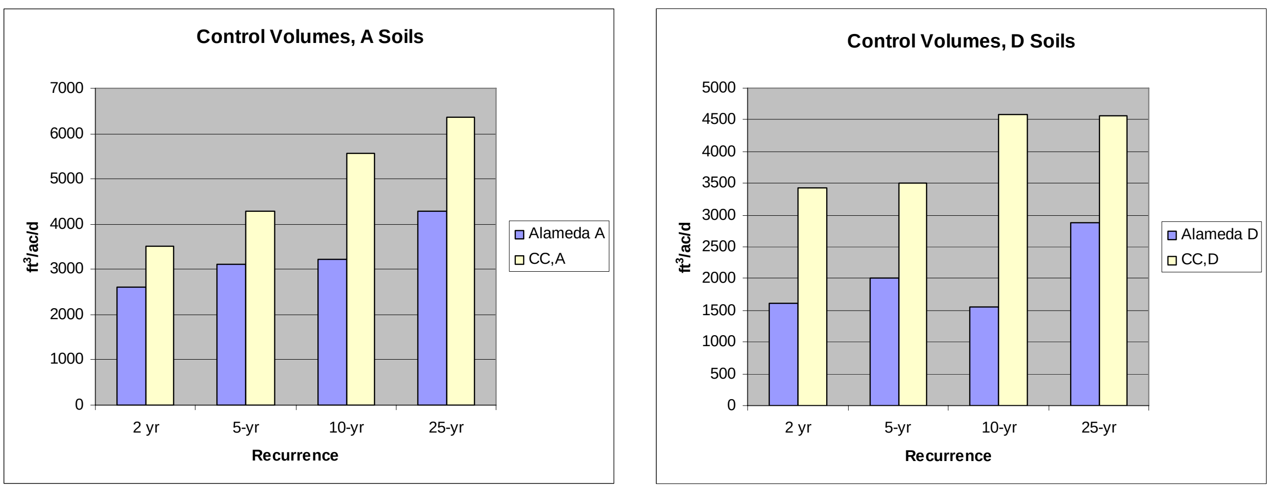
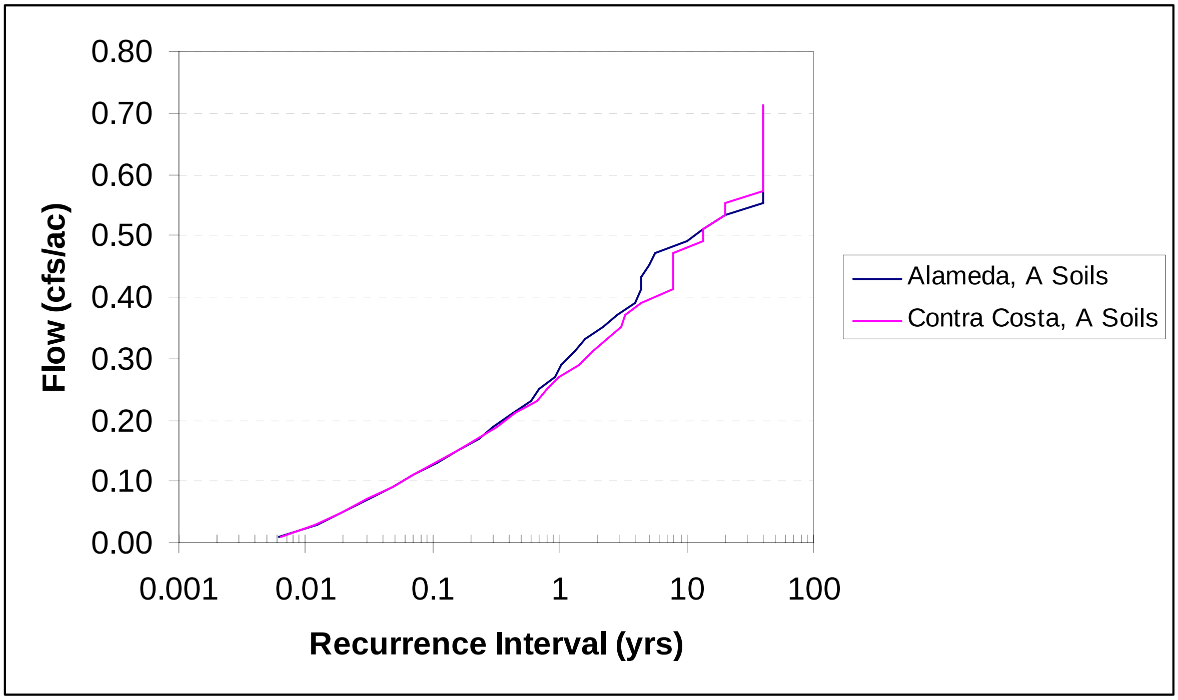


Figure 4. Comparison of Control Volumes for Hypothetical Development (70 Percent Impervious, Moderate Slopes)

The post-project flows are examined in greater detail in Figure 5, which compares the recurrence interval of flows (per acre) at the model time step of 1 hour. For A soils, the flow duration curves produced by BAHM and the Contra Costa IMP approach are in reasonable agreement, consistent with Figure 3. For D soils, the Contra Costa IMP approach consistently over-estimates the magnitude of flows of a given recurrence interval relative to the BAHM application.



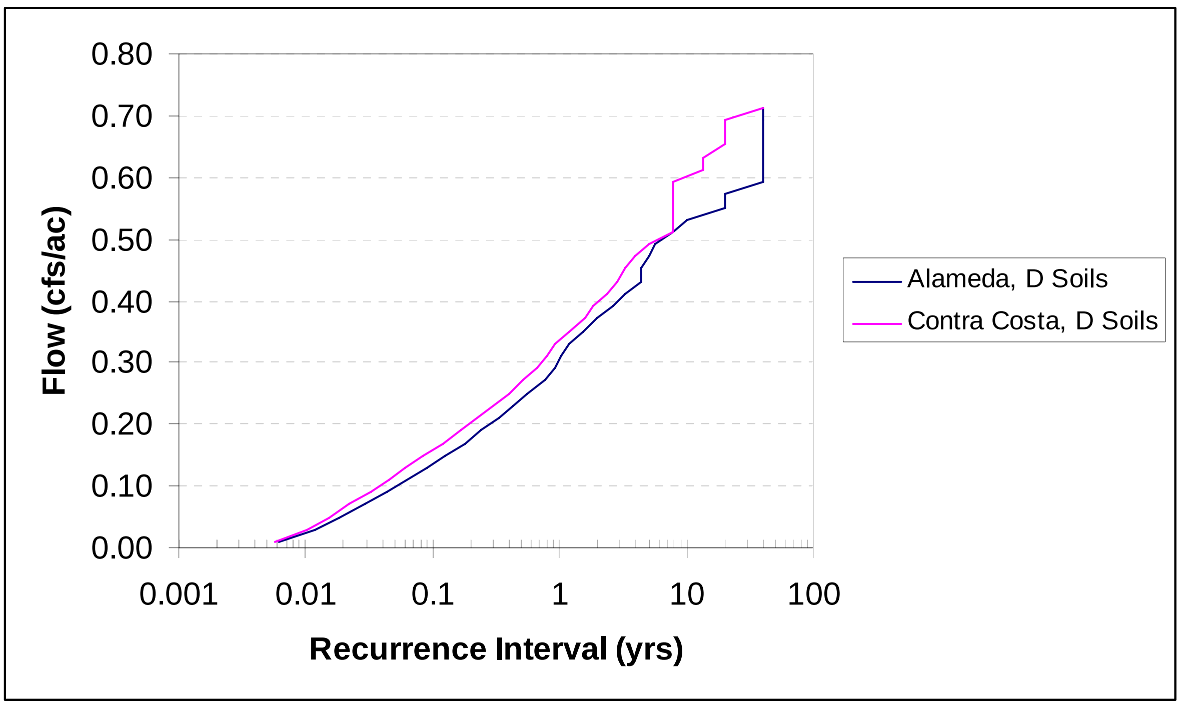


Figure 5. Flow-Duration Curves for Post-Project Hourly Flows (70 Percent Impervious, D Soils, Moderate Slopes)

The predicted event peak is also of interest. Returning to the raw hourly results, the annual maxima at various return intervals can be computed. These are shown in Figure 6 (the A soils are shown twice, once on a logarithmic scale). The difference in A soils predictions for pre-development conditions is again drastic, due to the large difference in infiltration rates in the two models. The predicted peak flows from D soils are similar, while those from impervious land are essentially identical.

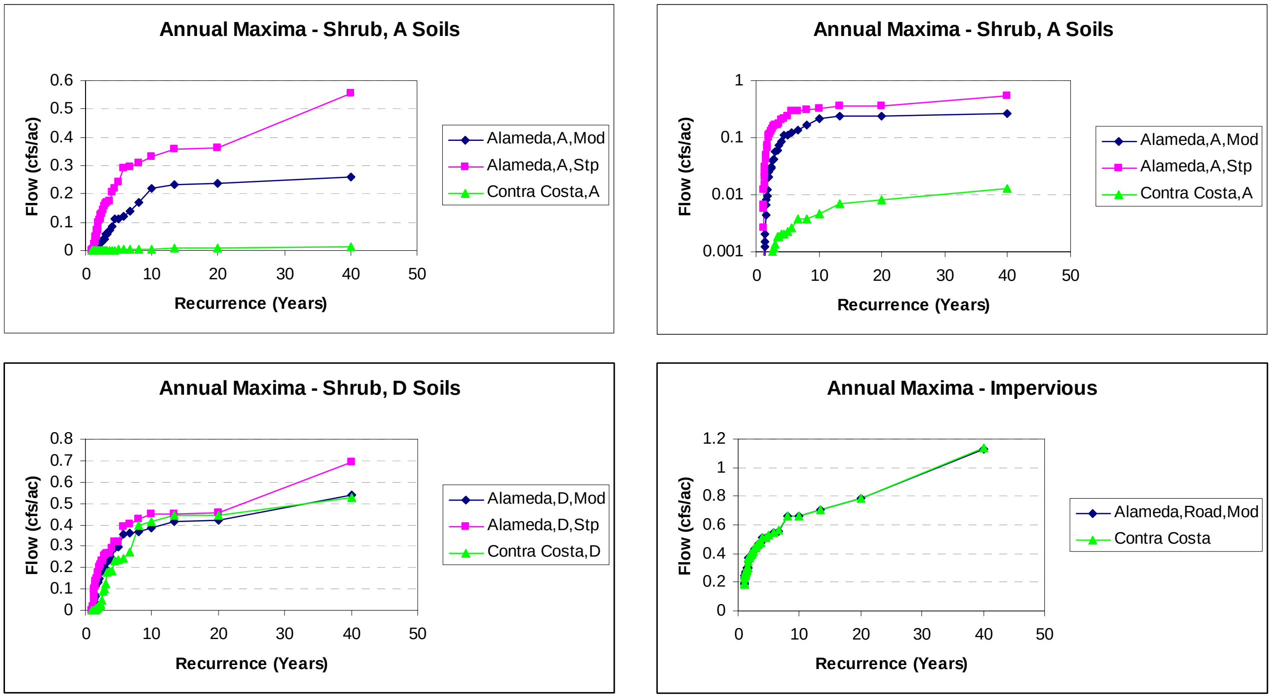


Figure 6. Comparison of Instantaneous Flow Peaks from BAHM and IMP (Annual Duration Series)

Another way to look at storage requirements is to compare the Contra Costa sizing factors to BAHM automatic calculation of required pond volume ("AutoPond" function). This was done for the sample conversion of 1 acre of shrub on D soils to 1 acre of imperviousness, as described in greater detail in Section 4. For this site, Contra Costa's IMP Sizing Tool calculates a sizing factor for control with a flow-through planter of 0.05 ac/ac, based on local rainfall. The resulting planter has a total storage volume (on the surface and in the media pore space) at the overflow riser of 0.1275 AF/ac. For the same situation, BAHM set up a pond with a default depth of 3 ft at the riser and an effective depth of 4 ft. The pond has a total storage volume at the overflow riser of 0.120 AF/ac, less than the Contra Costa IMP. The bottom of the pond created by BAHM was a square with side length of 32.263 ft. Based on the bottom area, this gives a sizing factor of 0.024 ac/ac, which is significantly less than the CC sizing factor for the flow-through planter of 0.05 ac/ac (base value of 0.04 adjusted for local rainfall to 0.05). The difference in area is consistent with the fact that a portion of the planter is occupied by soil and gravel.

A pond, however, does not have vertical sides, while a planter does. The default assumption for BAHM is that the pond will have side slopes of 3 (H/V). This adds significant area to the total pond footprint - indeed it nearly triples the total surface area at an effective depth of 4 feet, yielding a footprint of 0.073 ac/ac. With these assumptions, the footprint required by a BAHM pond is greater than the footprint required by a CC flow-through planter - but only because the pond cannot have vertical sides (for safety reasons) and thus provides a less-efficient use of space to achieve the same storage volume (the area added by the shallow side slopes of the pond (factor of about 3) is greater than the additional area required to account for the volume occupied by media in the planter (factor of about 2).

In summary, it does not appear that the Contra Costa IMP approach underestimates storage requirements relative to the BAHM approach. Instead, the IMP approach estimates much greater storage requirements – largely because the estimated storm runoff from pre-development conditions is less using the IMP. The results suggest that Contra Costa would do well to calibrate their IMP to local conditions. However, it is not the case that the Contra Costa approach will result in smaller sizing of HMP facilities. This occurs because the net effect of the differences in all parameters – and, in particular, the differences in INFILT and LZSN – results in lower estimated pre-development flows using the IMP approach.

3 Range of Flows to be Controlled

Alameda, Santa Clara and other permittees using the BAHM approach to HMPs attempt to match the pre-development hydrograph (within 10 percent) for flows between one-tenth of the two year peak (0.1Q2) and the 10-year peak flow (Q10). The lower end (0.1Q2) is based on studies and data specific to Bay Area creeks, as discussed further below.

Contra Costa's HM requirements allow four alternatives for demonstrating compliance with the standard:

1. No increase in impervious area.
2. Implementation of infiltration-based integrated management practices (IMPs) based on sizing factors described in the HMP.
3. Site-specific modeling to show that post-project runoff durations and peak flows do not exceed pre-project runoff durations and peak flows, using a continuous simulation model such as HSPF.
4. Detailed site-specific study to demonstrate that the project will not result in accelerated erosion of receiving stream reaches.

Option 3 in the Contra Costa HMP is similar to the BAHM approach; however Contra Costa does not specify use of a specific modeling package, such as BAHM. When this option is used, post-project runoff durations are to be controlled over the range of 0.1Q2 to Q10. According to Steve Anderson and Tony Dubin of Brown and Caldwell, Contra Costa anticipates that options 3 and 4 will be used infrequently for major developments, for which site-specific modeling should be used.

Option 2 is expected to be employed for most smaller projects in Contra Costa County, and, as noted above, focuses on integrated onsite IMP controls. The sizing factors for the design of IMPs to control post-development hydrographs are calculated on flows ranging from 0.5Q2 up to Q10, and thus do not directly address the range from 0.1Q2 to 0.5Q2. The rationale is stated as follows in the cover letter to the 15 May 2005 submittal: "IMPs could be designed to provide even more control of outflows in the range of flows below 0.5Q2. This would be accomplished by reducing allowable underdrain outflow and increasing the sizing factors. The Program rejected this idea because (1) we believe the current sizing factors achieve the HMP standard, as evidenced by a comparison of the resulting runoff curves, and (2) it would make the IMPs less attractive to applicants, thereby undermining the advantages to be had by promoting the use of IMPs."

A review of the flow-duration curves provided in the Contra Costa HMP shows that it was *not* always the case that the proposed sizing factors provided protection down to 0.1Q2. For this to be true, the post-development flow-duration curve calculated at the proposed sizing factor would need to remain at or below the pre-development flow-duration curve out to the 0.1Q2 flow. This appears to be true for some of the management devices analyzed (in-ground planter, infiltration trench, dry well, infiltration basin), but is clearly not true for the flow-through planter (and unclear for several others).

It is worth commenting on the original specification of the range of flows to be controlled (0.1Q2 – Q10), particularly as it differs from recommendations for Western Washington. The origin of this range is GeoSyntec's 2004 analysis of Thompson Creek in Santa Clara County[5], including calculation of effective work curves. Subsequent analyses were developed for Ross and San Tomas Creek, with "similar" results. The final justification in the Santa Clara HMP sets the upper limit at Q10 because 90-95 percent of the work on the stream is accomplished at flows less than Q10. The lower limit is set at 0.1Q2 based on an analysis of critical flow that initiates erosion of the bed or bank (individual cross sections in the three

[5] Evaluation of the Range of Storms for HMP Performance Criteria (April 1, 2004). Technical Memorandum 4, attached to Santa Clara Valley Urban Runoff Pollution Prevention Program, Hydromodification Management Plan, Final Report (April 21, 2005).

study sites had estimated critical flows that ranged from 2 to 18 percent of the Q2 peak). Critical flow is defined as the flow corresponding to the critical shear stress, τ_C, that initiates erosion. The calculation of effective work also depends on the critical shear stress. Both the selection of the value of τ_C and the form used in the effective work calculation affect the range of flows that should be controlled.

To establish the lower limit for control, separate calculations were made for the bed and bank materials. For bed materials, GeoSyntec estimated τ_C from Shields' criterion, which establishes the inertial resistance that must be overcome to initiate movement of a particle of a given diameter. The method performs well for sand grains and gravel, but deviates from observations for clay and silt particles because cohesion between particles, which increases resistance to movement, is ignored[6]. For bank materials, τ_C was estimated from literature values listed in the ASCE Manual of Engineering Practice, No. 77. Testing of shear stress measurement by a jet test device in Alameda and Sacramento Counties in 2006 confirmed that the bank τ_C values provided by ASCE were appropriate to the region[7].

According to GeoSyntec (personal communication from Gary Palhegyi, 7 December 2007), the minimum value of τ_C is usually determined by the bed materials in Bay Area streams.

The upper limit for control is calculated from an effective work index – which integrates an estimate of bedload movement as a function of shear stress (and thus of flow) with the frequency distribution of flows. Work has units of mass transport rate times velocity. The effective work index (W) is given as follows:

$$W = C \cdot \sum_{i=1}^{n} \left(\tau_{bi} - \tau_c\right)^{1.5} \cdot V \cdot \Delta t ,$$

where C is a constant coefficient, τ_{bi} is the effective shear stress at the boundary dependent on the boundary materials, τ_c is the critical shear stress for the material, V is the mid-channel velocity, t is time, and the summation is over all observed flows. The erosion potential, E_P, is then calculated as the ratio of W for post-development conditions to W for pre-development conditions. The goal cited in the BAHM development is to maintain E_P less than 1.0.

Although not cited in the document, the mass transport part of the effective work index is the generalized Meyer-Peter and Mueller equation for bedload transport. It is only one among several empirical relationships that have been developed for bedload transport. For example, the GeoTools[8] suite provides five different sediment transport options for calculating W. However, many of these have forms similar to Meyer-Peter and Mueller, differing primarily in the coefficient (which cancels out when calculating E_P.). These types of formulations are most applicable to stream systems with relatively large width to depth ratios and a limited amount of fine-grained cohesive material (clay and silt)[9].

Other formulations for non-cohesive bedload transport give results that are generally similar to Meyer-Peter and Mueller. Rates may be very different for cohesive sediments. For instance, Figure 7 compares the relative rates of sediment mobilization implied by the Meyer-Peter and Mueller formula, as well as the frequently encountered Bagnold formula for non-cohesive sediments, to the fine-grained sediment resuspension rate in the Gailani model with exponent of 3 (a value often found appropriate for river deposits). Substituting the Bagnold relationship gives a smaller increase in transport per unit increase in shear stress, implying that even less effective work would be done above the Q10. The cohesive sediment relationship has a much faster rate of increase with excess shear stress, which could imply a larger fraction of work being done above the Q10 and a smaller fraction below the Q2.

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[6] Hsu, K.J. 1989. *Physical Principles of Sedimentology*. Springer-Verlag, Berlin.

[7] Palhegyi, G.E. 2006. Evaluation of the Jet Test Device for Measuring the Critical Shear Stress and Erodibility of Cohesive Soils. GeoSyntec Consultants, Oakland, CA.

[8] Bledsoe, B.P., M.C. Brown, and D.A. Raff. 2007. GeoTools: a toolkit for fluvial system analysis. *Journal of the American Water Resources Association,* 43(3): 757-772.

[9] Simons, D. B. and F. Sentürk. 1992. *Sediment Transport Technology; Water and Sediment Dynamics*. Water Resources Publications, Highlands Ranch, CO.

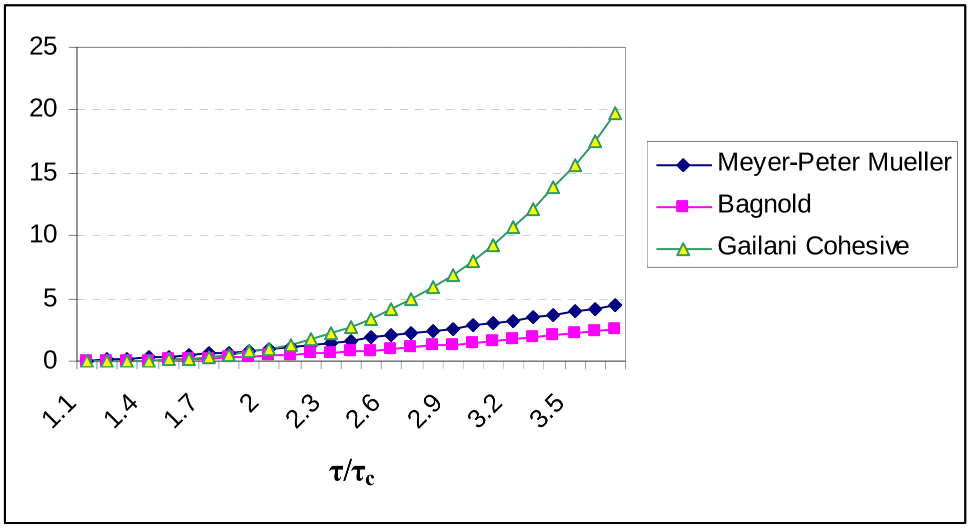


Figure 7. Comparison of Sediment Transport Equations

All of these bedload transport equations are essentially empirical, and thus dependent on the data sets for which they were developed. Calculation of effective work also depends on the estimation of critical shear stress for the bank and bed material: Because the calculation of the effective work index has a non-linear relationship to τ_c, errors in estimating τ_c will also affect the calculation of how much work is done at and above the Q10.

The estimate of τ_c directly determines the specification of the lower boundary of effective flows as the flow at which bedload motion starts. Selection of the 0.1Q2 to Q10 range for hydromodification control appears likely to be protective of most stream channels in the Bay Area with sand and gravel beds. For streams where significant amounts of fine-grained material are present in the bed, however, the Shields approach may underestimate the lower range of flows that need to be controlled, and the upper range of flows that should be controlled might need to be higher than the Q10.

4 SIMULATION OF OUTFLOW FROM IMPS

Both BAHM and the Contra Costa IMP sizing factors address onsite management practices (LID or IMPs). Both use the HSPF model as the engine for analysis, but there are differences in both the focus and technical details of the approach. The general focus differences have been mentioned above: BAHM runs simulations to size detention ponds, but can include onsite practices; the Contra Costa approach used HSPF runs to size various types of onsite practices. As in the previous section, investigation is made of application of the two different models for hypothetical development in the vicinity of Dublin, CA, which is on the border between Alameda and Contra Costa counties. In this area, Contra Costa would use the IMP approach. Alameda would use the BAHM model.

Within BAHM, the onsite practices are simulated from a set of generic building blocks: The gravel bed trench element is used (with different parameters) to simulate porous pavement, dry wells, and infiltration trenches; the lateral flow basin element is used to simulate dispersion of runoff onto pervious surfaces; and the bioretention swale element is used to simulate green roofs, rain gardens, in-ground planters, flow-through planters, bioretention basins, and dry swales. This recycling of code elements makes practical sense; however, it can lead to unexpected results if not implemented carefully. The large number of practices simulated using the bioretention swale element may be particularly problematic. These have in common an upper (planted) soil layer, a lower soil or gravel layer, and an overflow device, and may or may not have an underdrain. The details of individual practices may differ greatly, however, and can present challenges for a generic setup. In contrast, explicit models were created for each of the IMPs for development of the Contra Costa sizing factors.

Both BAHM and the Contra Costa applications simulate the hydraulic performance of these practices using HSPF Functional Tables (FTables), expressing volume-stage-discharge relationships. For many of the practices, such as bioretention and planters, two FTables are linked, representing the upper and lower zones of the device. In BAHM, the FTables are constructed based on a simple interpretation of soil properties including a fixed infiltration rate, with the addition of a Special Actions control that ensures that the infiltration rate does not exceed the available effective pore space[10]. Percolation in the Contra Costa applications is handled in a much more sophisticated way through application of Darcy's law and the Van Genuchten relationships that account for soil water retention characteristics, including suction or matric head within the soil pores. This results in infiltration rates that increase with head, rather than remaining constant, as in BAHM.

The Contra Costa HMP describes the setup for the flow-through and in-ground planters in detail. The default sizing factor for the flow-through planter is specified at 0.05 ac/ac for D soils. The rainfall along the Alameda-Contra Costa border is slightly higher than the Martinez rainfall used to set up the default sizing factors. Contra Costa provides an IMP Sizing Tool program (v. 0.9), which varies sizing factors according to rainfall regime. Use of this tool for the 1-acre (unit) test site yielded a sizing factor that still rounds to 0.05 ac/ac and a maximum underdrain flow of 0.104385 cfs/ac. (This flow rate is equivalent to Contra Costa's estimate of 0.5Q2, adjusted for local rainfall. BAHM would estimate a higher value of the 0.5Q2 of 0.117 cfs/ac for this site; however, the underdrain flow rate is still greater than the BAHM estimate of 0.1Q2, which is 0.0234 cfs/ac.) A device of similar design was set up via the BAHM interface (see Figure 8), which then generates an HSPF UCI file that can be compared to the Contra Costa model. The resulting HSPF files differ first as a result of differing technical assumptions. In addition, there appear to be some errors in the way that BAHM generates the model representation, as will be described below.

[10] Both BAHM and the Contra Costa IMP models work with the effective porosity, which is the portion of total soil porosity that can actively store and convey water, using literature values by soil type.

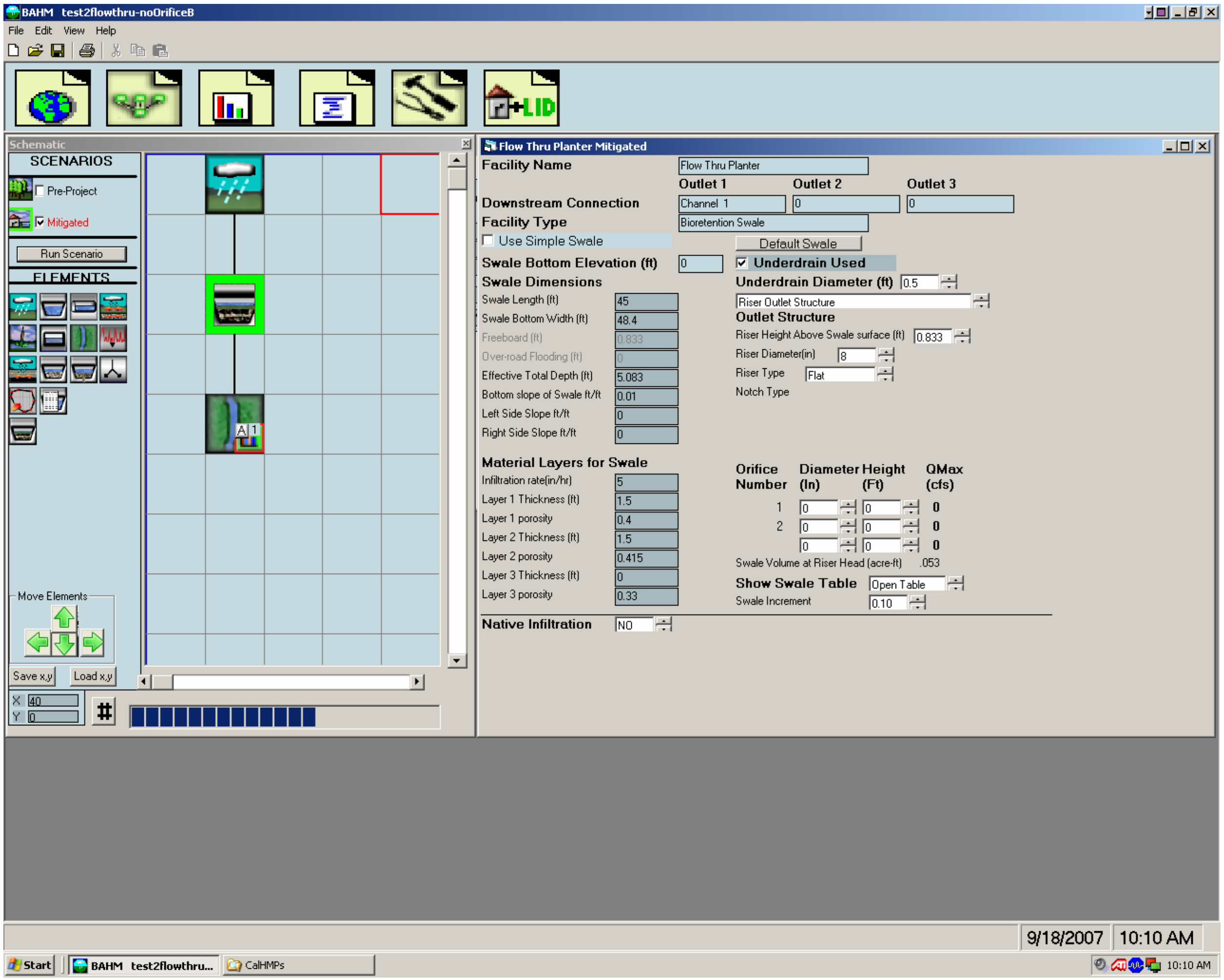


Figure 8. BAHM Setup Screen for Flow-Through Planter

The key technical differences between the BAHM and Contra Costa assumptions are summarized below:

	BAHM	Contra Costa
Percolation	Constant soil percolation rate, capped at limit of available pore space in lower layer.	Head-variable, based on Darcy's equation and consideration of matric head and bubbling pressure
Underdrain	Based on pipe size, no provision for orifice control on underdrain outlet[11]	Includes outlet orifice control establishing maximum discharge rate at 0.5Q2
Evapotranspiration	Applied to both surface and gravel media layers, with factors of 0.5 on the surface and 1.0 on gravel	Applied to surface layer only, with factor of 0.7

[11] As shown in Figure 8, the BAHM interface includes an option to specify an orifice for the bioretention swale element. It would seem that this should be used to establish flow control on the underdrain. However, when this option is selected, BAHM creates an FTable that includes an orifice entry into the riser in the upper soil layer. The orifice option thus cannot currently be used to control rate of discharge from the underdrain in the lower (gravel) layer. Presumably, BAHM could be used to better match the Contra Costa design by specifying a smaller underdrain pipe diameter (rather than orifice control), thereby limiting the maximum outflow to the desired level. That approach is not, however, discussed or recommended in the BAHM documentation and so was not implemented in the tests described here.

For evapotranspiration (ET), Contra Costa used potential evapotranspiration (PET) reduced by a crop factor of 0.7, based on guidance from AquaTerra. BAHM applies a factor of 0.5 to the surface soils, citing (but not referencing) information that amended soils typically exhibit a lower rate of ET than native soils. BAHM applies the full PET rate to the subsurface gravel layer, while Contra Costa assumes no evapotranspiration from this layer. Application of full PET to the gravel layer appears to be an error in the BAHM setup for the flow-through planter configuration, as this is certainly not a free water surface, and root penetration should be minimal.

The HSPF UCI file generated by BAHM for this scenario appears to have other errors as well. In both BAHM and the Contra Costa approach, the planter is represented by two connected reach elements (RCHRES), each with a corresponding FTable describing volume-flux relationships. The upper RCHRES represents the amended soil layer (1.5') plus freeboard in the planter above the soil surface and the riser. The lower RCHRES represents the gravel layer (1.5') and underdrain. (The setup used assumes D soils, where an underdrain would be needed and infiltration out of the bottom of the planter would be minimal.) For the upper layer, BAHM generated the FTable shown in Table 3 in Appendix A, in which Outflow 1 represents discharge through the overflow riser and Outflow 2 represents percolation to the lower (gravel) layer, assuming sufficient pore space availability.

Examination of Table 3 shows that BAHM simulates a constant infiltration rate, independent of head or pore suction effects. Overflow through the riser begins at 1.5', which is the depth of the surface layer. It should start at 1.5 + 0.833 = 2.333'.

The lower layer is also specified as 1.5' in thickness. BAHM creates the FTable shown in Table 4 of Appendix A for this layer. The representation of this layer created in the BAHM FTable is just less than 3' in thickness, instead of 1.5', as intended. The total effective depth calculated by BAHM is 5.083' (bottom layer + surface layer + freeboard + over road depth). It will be noted that the sum of the maximum depth in FTable 2 and the next to last depth in FTable 1 is approximately equal to the total effective depth. It would appear that BAHM has allocated the total effective depth incorrectly between the two FTables.

The outflow from the underdrain specified in FTable 2 is also suspect. This begins at a depth between 0.79 and 0.84 feet, which is equal to the height of the riser pipe above the amended soil layer. The outflow rate rises to a maximum at a depth of 2.5', then drops suddenly.

The reason for this confusion arises from the BAHM process of simulating the flow-through planter through modifications of the bioretention swale element. The BAHM description of implementation of the flow-through planter shows the underdrain at the bottom of the gravel layer, and states that "stormwater enters the planter above ground and then infiltrate[s] through the soil and gravel storage layers before exiting through a discharge pipe," as would be expected. However, the information on the basic bioretention/rain garden element provides conflicting assumptions. Specifically, "the bottom of the underdrain pipe is assumed to be at the bottom of the ***amended*** soil layer." As a result, the interface seems to have set up the model with the underdrain 1.5 ft above the bottom of the planter, and continues FTable 2 through the depth of the gravel and amended soil. However, FTable 1 is not set up to represent only storage above the surface of the amended soil as (1) flow into the riser starts at a depth of 1.5', not 0.833', and (2) infiltration from the upper to the lower layer is only a function of depth in FTable 1. Similar problems appear to affect BAHM simulation of other LID components. For example, a bioretention area is also supposed to be simulated with an underdrain at the bottom of a subsurface gravel layer, but BAHM will place the underdrain at the bottom of the upper, amended soil layer.

These apparent errors in the BAHM interface will have different types of impacts on the flow control simulated by the flow-through planter. First, there is dead storage at the bottom of the lower layer. As evapotranspiration is applied to this layer, some flow that should exit through the underdrain will be converted to evapotranspiration. Second, the height of the riser inlet above the bottom of the underdrain, as represented in the FTable, is 3.667' (given that the underdrain is represented as 0.833' above the bottom of the lower layer), rather than the intended 3.833'. This would result in overestimation of bypass flow through the riser during large storm events. Together these effects should result in an underestimate of the frequency of very low flows and an overestimate of the frequency of very high flows. However, both effects are expected to be small.

In contrast, the setup of the flow-through planter by Contra Costa County is more in line with expectations. The FTable for the upper soil layer (created by Tetra Tech for the example application based on scaling to local precipitation of a sample FTable provided by Tony Dubin of Brown and Caldwell; see Table 5 in Appendix A) shows percolation rates that increase non-linearly with moisture storage, while inflow to the riser begins when the appropriate ponding depth is reached. The FTable for the lower soil layer (Table 6 in Appendix A) shows outflow through the underdrain beginning as the depth and volume of water increase above zero. In the Contra Costa approach, the sizes of pipe perforations and/or flow control orifices are assumed to be sized so as to limit the underdrain outlet flow to the pre-development 0.5Q2 flow. (As noted in Section 2, the magnitude of the pre-development Q2 flow will be estimated differently by the BAHM and Contra Costa models). The maximum possible rate of percolation from the upper soil layer to the lower soil layer is then equal to the estimated 0.5Q2 flow by continuity principles. (For an in-ground planter with an open bottom, the maximum rate of percolation would be equal to the 0.5Q2 flow plus the rate of deep percolation out of the bottom of the planter.) The overflow riser is sized such that the sum of the maximum overflow rate and the maximum underdrain outflow is equal to the estimated pre-development Q10 flow, as shown in Section 4.4.1 of the Contra Costa HM submittal. (It is noted that while this design assumption is made for the development of sizing factors, Appendix I of the Contra Costa C.3 Guidebook does not provide design criteria for the overflow riser.)

Neither BAHM nor the Contra Costa models simulate direct overflowing of the planter. Instead, water in excess of the height of the planter walls is assumed to pond on adjacent surfaces and eventually discharge through the riser or planter medium. The planter is sized sufficiently large, however, that overtopping of the planter walls will be extremely rare (for a 10-year simulation from Oct. 1979 – Sept. 1989, neither the BAHM nor Contra Costa simulations resulted in overtopping of the planter).

The Contra Costa simulations used to develop the IMP sizing factors were undertaken on a unit (per-acre) basis, so that the area of the IMP specified in the FTable is equivalent to the sizing factor. Flows through both the overflow riser and the underdrain were assumed to be controlled by the depth of water in the individual layer and the capacity of the pipe, and thus do not change with area of the device, while the percolation rate scales directly with the area. Iterative evaluations were then used to determine the sizing factor that meets the hydrograph matching criteria.

As noted above, a key difference in the representations of the flow-through planter is Contra Costa's inclusion of outflow control on the underdrain. The Contra Costa HMP, Attachment 2, p. 19 says "When an underdrain is included in the configuration...[flow] rate is calculated using the orifice equation...so that the underdrain flow will match 0.5Q2 when the lower gravel layer is fully saturated. The Stormwater C.3 Guidebook will specify criteria for sizing pipe perforations and/or flow control orifices to ensure that the underdrain flow is limited to 0.5Q2." The BAHM approach assumes little limitation on underdrain discharge, with the result that the gravel layer rarely fills above the top of the underdrain, which seems unreasonable. In the test simulations, the lower layer represented by BAHM never filled to a depth greater than 1.6 ft below the top of the lower layer.

The performance of the various representations can be demonstrated through a head-to-head comparison. To do this, an HSPF model was set up representing a 1-acre (unit) conversion from Scrub/Shrub on D soils with moderate slopes to impervious area (rooftop) in the Dublin area. The BAHM interface was allowed to set up the pre-development, post-development, and mitigated scenarios, using the flow-through planter. The Contra Costa representation of the same IMP was then added to the UCI file. Total outflows from the IMP were routed to nominal reaches for comparison of resulting flow durations. Runs were analyzed for the period of 10/1/1979 – 3/22/1987, representing the maximum amount of hourly data that will fit on one Excel worksheet.

In making the comparison, it is important to keep several facts in mind:

- The BAHM setup of the IMP appears to have errors in the way the FTable is specified.
- BAHM models IMPs as they have typically been built in the past, with no outlet flow control on the underdrain.
- The pre-development flows, as generated by BAHM, differ from those that would be simulated by Contra Costa.

Despite these issues, the comparisons reveal a number of interesting aspects of the different simulation methods.

Prior to cross-comparison of BAHM and the Contra Costa approach to the flow-through planter, the effect of assigning ET to the lower soil layer in BAHM was investigated. Over the period of the simulation, removing ET from this layer results in an increase in total flow through the underdrain of only 0.70 percent. The impact on the flow duration curve is minimal, as shown in Figure 9.

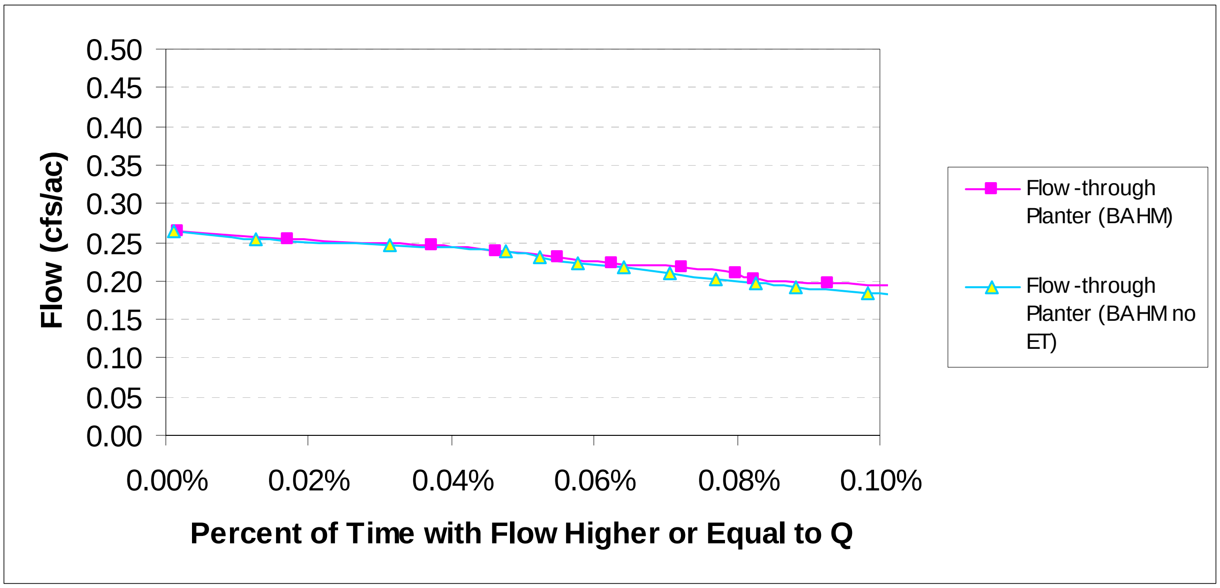


Figure 9. Flow Duration Curve for BAHM Implementation of Flow-Through Planter, with and without ET Applied to Lower Layer

Experiments with different size IMPs in BAHM revealed conditions in which, at the highest (least frequent) flows, there is a greater frequency of exceedance predicted when the simulation is done with ET on the lower layer than when it is done without ET on the lower layer. This seems counterintuitive, but does have an explanation. The divergence only occurs when the total flow is greater than the maximum infiltration rate of 0.202 cfs/ac. When ET is assigned to the lower layer, there is typically more empty pore space available in the lower layer prior to a runoff event, which in turn can result in smaller volume stored in the surface layer, which in turn results in less infiltration at the start of a runoff event when the antecedent volume is below the first step in the surface FTable. On the other hand, the antecedent volume stored in the lower layer tends to equalize between the simulations with and without ET as soon as infiltration begins. BAHM is set up so that outflow demand is estimated with a KS weighting factor of 0.5 (the HSPF default). This parameter weights the outflow between the outflow demand present at the beginning of the time step and that present at the end of the time step. Therefore, the higher infiltration rates simulated when ET is applied to the lower layer result in a greater calculation of outflow demand at the end of the time step because the total head (antecedent volume plus infiltration) is greater. This problem could be fixed by setting the KS factor to zero (so that outflow demand is dependent only on the volume at the start of the time step) or by using a shorter time step.

It was also noted that the BAHM simulation without ET produces many more days with small but non-zero discharges (13.5 percent) than does the simulation with ET on the lower layer (9.0 percent). This is an anomaly due to the way in which BAHM constructed the FTable, which (incorrectly) places the underdrain 1.5' above the bottom of the planter. When ET is applied to the lower layer, water declines below the underdrain; with no ET it remains filled to just below the underdrain and thus responds to small amounts of infiltration.

The full BAHM run covers the default period of 1960-2004. Over this period, the 0.1Q2 flow is estimated by BAHM as 0.02339 cfs/ac (from Q2 of 0.2339), while Q10 is estimated as 0.59 cfs/ac.

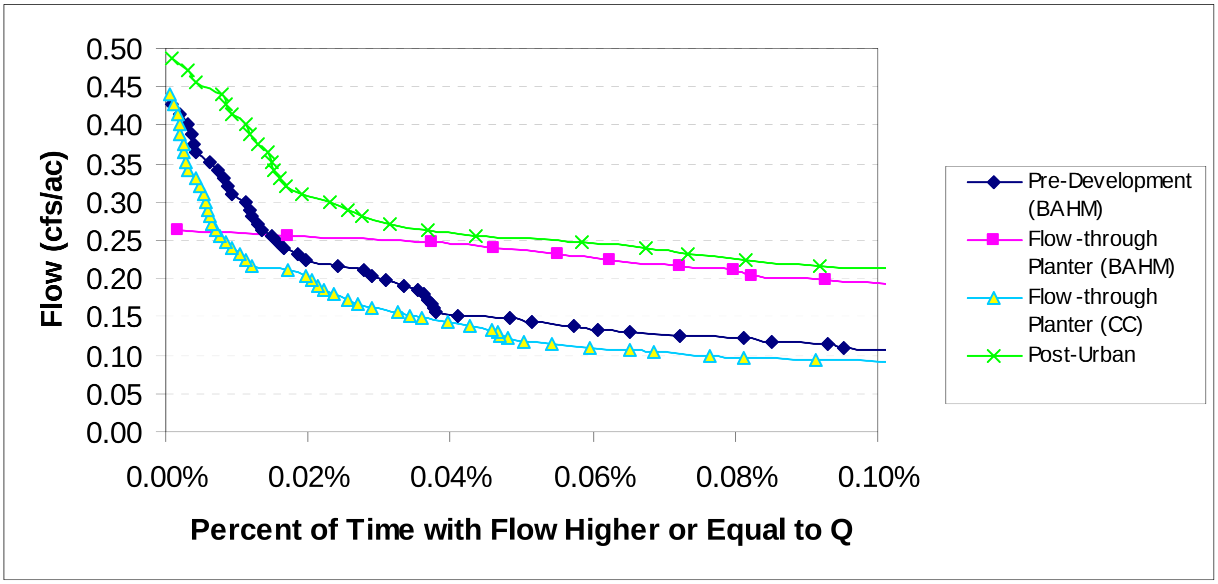
A direct comparison of the BAHM (with ET applied to the lower layer) and Contra Costa simulations of the outflow from the flow-through planter was also conducted. While the results reveal some interesting points, a direct comparison is not fully valid for two reasons:

- BAHM does not simulate outflow control on the underdrain (and the flow-through planter cannot alone achieve the desired control of the post-development flow duration), while the Contra Costa approach does allow such control.
- As noted above, there appear to be errors in the FTables created by BAHM.

Figure 10 compares the simulation of the flow-through planter by both the BAHM and Contra Costa approaches with the flow-durations from the pre-development condition (scrub/shrub) and the unmitigated post-development condition (100 percent impervious). The same results are shown in two ways – on both arithmetic and logarithmic scales.

The figure first shows the large difference between flow durations for pre-development and unmitigated post-development conditions. Both the BAHM and Contra Costa models remain below the pre-development flow duration curve for higher flows – specifically those that exceed the infiltration rate, equivalent to 0.25 cfs/ac on the flow-through planter with sizing factor of 0.05. Below this level, the BAHM planter achieves little control and quickly converges to the unmitigated post-development line. This occurs primarily because BAHM does not provide outlet control on the underdrain – thus flows less than 0.25 cfs/ac that infiltrate to the lower layer are discharged essentially unmodified. (As discussed in Section 1, BAHM is designed to account for the presence of IMPs, but does not attempt to achieve matching of the flow-duration curve through IMPs alone.)

The Contra Costa simulation of the planter is intended to provide flow duration matching down to the 0.5Q2 level. For this site, the long-term 0.5Q2 is 0.117 cfs/ac (flows were lower during the 1979-1987 period shown on the graphs). The Contra Costa simulation actually remains below the pre-development flow duration curve down to about 0.088 cfs/ac, but is above this curve in the lower region above the 0.1Q2 flow of 0.0234 cfs/ac. Control below 0.5Q2 is (unintentionally) achieved primarily because the Contra Costa modeling underestimates pre-development flows, as discussed in Section 2. The lack of full mitigation for flows near 0.1Q2 is consistent with Contra Costa's stated intentions, and Figure 14 in the HMP shows that the flow-through planter is not capable of achieving a match to the flow duration curve down to the 0.1Q2 level. (Note that some of the other types of IMPs simulated by Contra Costa do achieve control to 0.1Q2).



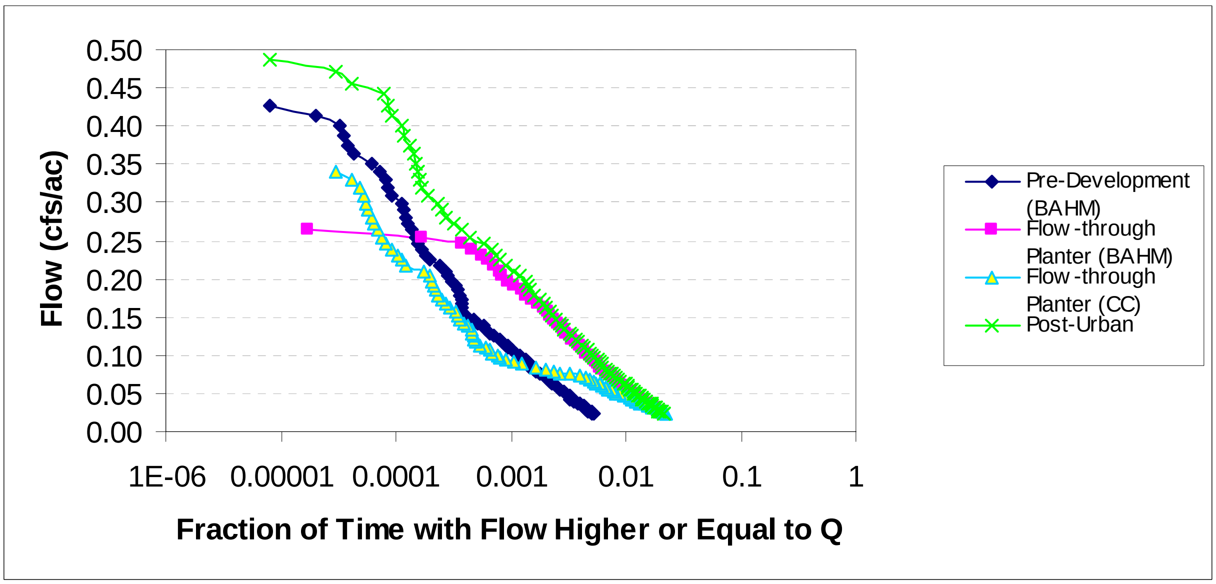
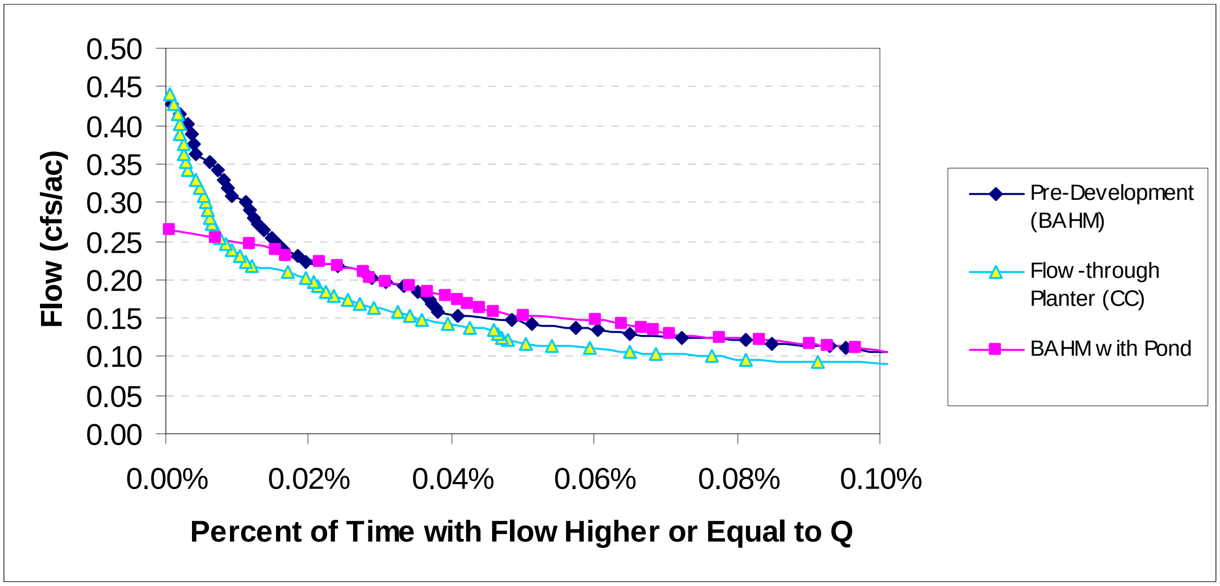


Figure 10. Simulation of Flow-Through Planter by BAHM and Contra Costa Models (Daily Results)

The BAHM model is not currently configured to achieve the necessary controls with IMPs alone, but will automatically calculate an appropriate pond size. The BAHM AutoPond option was used to estimate such a pond. Interestingly, the optimized pond sizes are almost identical whether or not a flow-through planter is included upstream. This is a result of the discrepancies in BAHM's simulation of the flow-through planter, which does not provide control in the low flow range. Therefore, comparison is made to BAHM results with control provided by a pond only. Figure 11 (similar in presentation to Figure 10) compares the results of the autosized BAHM pond and the Contra Costa flow-through planter to the pre-development flow-duration curve. The BAHM pond performs properly, providing a close match to the pre-development flow duration curve throughout most of the flow range (small deviations result because the pond was autosized on a longer precipitation record than is used in this simulation). For flows with a excursion frequency of 0.0012 to 0.00007, the flow-through planter is consistently more conservative than the optimized pond, providing slightly lower flows. However, at very low flows the flow-through planter diverges above the pre-development flow-duration curve, while the BAHM pond maintains the desired level of control.



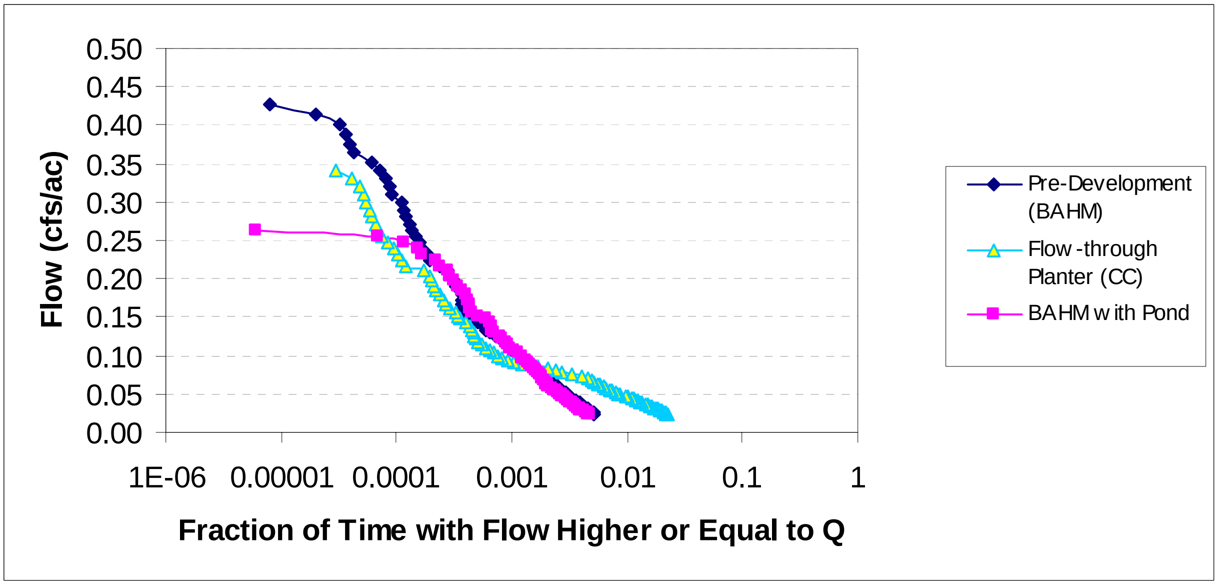


Figure 11. Comparison of Daily Flow Duration Curves for BAHM AutoPond and Contra Costa Flow Through Planter

5 SUMMARY

Both the BAHM and the Contra Costa IMP sizing factors provide valid conceptual approaches to protecting pre-development flow durations. However, the two methods have different focus and applicability. The Contra Costa IMP sizing factors are appropriate to small developments and infill, with hydrologic control onsite and primarily through LID components. They provide an easy to use approach to design, but do not examine the interaction between multiple source areas at a larger scale. The BAHM can be used on all size projects, but includes ponds in addition to LID for flow duration control. In its current version, only the sizing of ponds is automatically optimized to achieve the specified level of control. For larger developments, Contra Costa allows for the use of a site-specific hydrologic simulation model, which could be BAHM or another HSPF application.

In evaluating the Contra Costa approach, it should be remembered that, as documented in Section 2, the assumptions incorporated into the uncalibrated HSPF model application for Contra Costa appear to over-estimate the difference between uncontrolled post-development and pre-development flows, resulting in an apparent overestimation of the required IMP sizing factors. Because of this, the existing sizing factors are already sufficient to achieve flow-duration control below the 0.5Q2 flow when compared to a calibrated pre-development flow-duration curve – although they do not guarantee control all the way down to the 0.1Q2 flow.

Both the BAHM and the Contra Costa IMP approaches have merit, and it would seem ideal to combine the two – allowing the use of simple sizing factors for small developments, and applying BAHM for larger developments. To achieve such an integrated approach, several improvements are recommended:

1. BAHM simulation of IMPs should be improved. There appear to be errors in the way BAHM sets up the flow-through planter representation, as noted above. In addition, the representation of infiltration is simplistic and should be refined.
2. The BAHM simulation of LID devices should consider providing for outflow limitation control on underdrains. The user would then have the option of employing such devices either as the primary control on flows (in which case, the underdrain outflow should be limited to the flow corresponding to the base of the control range), or as a secondary component (without such strict limitations on underdrain outflow) that helps to reduce the size of detention ponds.
3. Some IMPs simulated by Contra Costa do not achieve matching of flow-duration curves down to the 0.1Q2 level, and thus these IMPs do not meet current Permit requirements for hydromodification control.
4. Simulation experiments indicate that the Contra Costa IMP approach does not underestimate total storage requirements relative to the BAHM approach.
5. The Contra Costa HSPF models are uncalibrated and, as a result, appear to underestimate pre-development flows, resulting in an overestimation of the needed level of control. This deficiency could be remedied (for instance, by conducting local calibration studies or using the calibrated parameter set developed for BAHM); however, the uncalibrated model in its present form is conservative in that it provides for a higher level of control than might otherwise be required.
6. The Contra Costa C.3 Guidebook should give design criteria for overflow risers for the IMPs that employ overflow risers.

Appendix A. HSPF FTABLES

Table 3. BAHM FTable for Upper Layer of Flow-Through Planter

```
FTABLE       1
 38    6
    Depth      Area    Volume  Outflow1  Outflow2  outflow 3 ***
     (ft)   (acres) (acre-ft)   (cfs)      (cfs)     (cfs)   ***
0.000000  0.050068  0.000000  0.000000  0.000000  0.000000
0.106278  0.050069  0.005654  0.000000  0.252425  0.000000
0.162756  0.050070  0.008481  0.000000  0.252425  0.000000
0.219233  0.050072  0.011309  0.000000  0.252425  0.000000
0.275711  0.050073  0.014137  0.000000  0.252425  0.000000
0.332189  0.050074  0.016965  0.000000  0.252425  0.000000
0.388667  0.050075  0.019793  0.000000  0.252425  0.000000
0.445144  0.050077  0.022622  0.000000  0.252425  0.000000
0.501622  0.050078  0.025450  0.000000  0.252425  0.000000
0.558100  0.050079  0.028278  0.000000  0.252425  0.000000
0.614578  0.050080  0.031107  0.000000  0.252425  0.000000
0.671056  0.050082  0.033935  0.000000  0.252425  0.000000
0.727533  0.050083  0.036764  0.000000  0.252425  0.000000
0.784011  0.050084  0.039592  0.000000  0.252425  0.000000
0.840489  0.050085  0.042421  0.000000  0.252425  0.000000
0.896967  0.050087  0.045250  0.000000  0.252425  0.000000
0.953444  0.050088  0.048078  0.000000  0.252425  0.000000
1.009922  0.050089  0.050907  0.000000  0.252425  0.000000
1.066400  0.050090  0.053736  0.000000  0.252425  0.000000
1.122878  0.050092  0.056565  0.000000  0.252425  0.000000
1.179356  0.050093  0.059394  0.000000  0.252425  0.000000
1.235833  0.050094  0.062223  0.000000  0.252425  0.000000
1.292311  0.050095  0.065053  0.000000  0.252425  0.000000
1.348789  0.050097  0.067882  0.000000  0.252425  0.000000
1.405267  0.050098  0.070711  0.000000  0.252425  0.000000
1.461744  0.050099  0.073541  0.000000  0.252425  0.000000
1.518222  0.050100  0.076370  3.736699  0.252425  0.000000
1.574700  0.050102  0.079200  4.203436  0.252425  0.000000
1.631178  0.050103  0.082030  4.688134  0.252425  0.000000
1.687656  0.050104  0.084859  5.190151  0.252425  0.000000
1.744133  0.050105  0.087689  5.708908  0.252425  0.000000
1.800611  0.050107  0.090519  6.243883  0.252425  0.000000
1.857089  0.050108  0.093349  6.794596  0.252425  0.000000
1.913567  0.050109  0.096179  7.360612  0.252425  0.000000
1.970044  0.050110  0.099009  7.941526  0.252425  0.000000
2.026522  0.050112  0.101839  8.536966  0.252425  0.000000
2.083000  0.050113  0.104670  9.146586  0.252425  0.000000
2.139478  0.050114  0.107500  9.770063  0.252425  0.000000
END FTABLE  1
```

Table 4. BAHM FTable for Lower Layer of Flow-Through Planter

```
FTABLE       2

 54    4
   Depth       Area    Volume   Outflow1 ***
    (ft)    (acres) (acre-ft)    (cfs)   ***
0.000000  0.050067  0.000000  0.000000
0.056478  0.050065  0.001131  0.000000
0.112956  0.050064  0.002262  0.000000
0.169433  0.050063  0.003393  0.000000
0.225911  0.050061  0.004524  0.000000
0.282389  0.050060  0.005655  0.000000
0.338867  0.050059  0.006786  0.000000
0.395344  0.050058  0.007917  0.000000
0.451822  0.050056  0.009048  0.000000
0.508300  0.050055  0.010178  0.000000
0.564778  0.050054  0.011309  0.000000
0.621256  0.050053  0.012440  0.000000
0.677733  0.050051  0.013571  0.000000
0.734211  0.050050  0.014701  0.000000
0.790689  0.050049  0.015832  0.000000
0.847167  0.050048  0.016963  0.010948
0.903644  0.050046  0.018093  0.121910
0.960122  0.050045  0.019224  0.294276
1.016600  0.050044  0.020354  0.510778
1.073078  0.050043  0.021485  0.763750
1.129556  0.050041  0.022615  1.048533
1.186033  0.050040  0.023746  1.361905
1.242511  0.050039  0.024876  1.701460
1.298989  0.050038  0.026007  2.065316
1.355467  0.050036  0.027137  2.451946
1.411944  0.050035  0.028268  2.860082
1.468422  0.050034  0.029398  3.288643
1.524900  0.050033  0.030571  3.736699
1.581378  0.050031  0.031743  4.203436
1.637856  0.050030  0.032916  4.688134
1.694333  0.050029  0.034089  5.190151
1.750811  0.050028  0.035261  5.708908
1.807289  0.050026  0.036434  6.243883
1.863767  0.050025  0.037606  6.794596
1.920244  0.050024  0.038779  7.360612
1.976722  0.050023  0.039951  7.941526
2.033200  0.050021  0.041124  8.536966
2.089678  0.050020  0.042296  9.146586
2.146156  0.050019  0.043468  9.770063
2.202633  0.050018  0.044641  10.40710
2.259111  0.050016  0.045813  11.05740
2.315589  0.050015  0.046985  11.72072
2.372067  0.050014  0.048157  12.39679
2.428544  0.050013  0.049330  13.08538
2.485022  0.050011  0.050502  13.78627
2.541500  0.050010  0.051674  0.669000
2.597978  0.050009  0.052846  0.788000
2.654456  0.050008  0.054018  0.910000
2.710933  0.050006  0.055190  1.035000
2.767411  0.050005  0.056362  1.163000
2.823889  0.050004  0.057534  1.291000
2.880367  0.050003  0.058706  1.421000
2.936844  0.050001  0.059878  1.553000
2.993322  0.050000  0.122101  1.684000
END FTABLE  2
```

Table 5. Contra Costa IMP FTable for Upper Layer of Flow-Through Planter

```
FTABLE      11
 31    5
   Depth       Area    Volume  Qover      Qperc      Null    ***
    (ft)    (acres) (acre-ft)   (cfs)      (cfs)      (cfs) ***
    0.00       0.05    0.0000     0.000    0.0000
    0.10       0.05    0.0020     0.000    0.0000
    0.20       0.05    0.0040     0.000    0.0000
    0.30       0.05    0.0060     0.000    0.0000
    0.40       0.05    0.0080     0.000    0.0000
    0.50       0.05    0.0100     0.000    0.0000
    0.60       0.05    0.0120     0.000    0.0000
    0.70       0.05    0.0140     0.000    0.0001
    0.80       0.05    0.0160     0.000    0.0002
    0.90       0.05    0.0181     0.000    0.0005
    1.00       0.05    0.0201     0.000    0.0011
    1.10       0.05    0.0221     0.000    0.0023
    1.20       0.05    0.0241     0.000    0.0048
    1.30       0.05    0.0261     0.000    0.0101
    1.40       0.05    0.0281     0.000    0.0220
    1.50       0.05    0.0301     0.000    0.0317
    1.60       0.05    0.0351     0.000    0.0414
    1.70       0.05    0.0401     0.000    0.0512
    1.80       0.05    0.0451     0.000    0.0609
    1.90       0.05    0.0501     0.000    0.0706
    2.00       0.05    0.0551     0.000    0.0803
    2.10       0.05    0.0601     0.000    0.0900
    2.20       0.05    0.0651     0.000    0.0998
    2.30       0.05    0.0701     0.000    0.1044
    2.40       0.05    0.0751     0.100    0.1044
    2.50       0.05    0.0801     0.312    0.1044
    2.60       0.05    0.0851     0.419    0.1044
    2.70       0.05    0.0901     0.493    0.1044
    2.80       0.05    0.0951     0.561    0.1044
    2.90       0.05    0.1001     1.131    0.1044
    3.00       0.05    0.1051     1.226    0.1044
END FTABLE 11
```

Table 6. Contra Costa IMP FTable for Lower Layer of Flow-Through Planter

```
FTABLE      12
 16    4
   Depth       Area    Volume  Q outlet ***
    (ft)    (acres) (acre-ft)   (cfs)    ***
    0.00       0.05    0.0000    0.0000
    0.10       0.05    0.0021    0.0000
    0.20       0.05    0.0042    0.0011
    0.30       0.05    0.0063    0.0059
    0.40       0.05    0.0083    0.0193
    0.50       0.05    0.0104    0.0509
    0.60       0.05    0.0124    0.0658
    0.70       0.05    0.0145    0.0712
    0.80       0.05    0.0166    0.0760
    0.90       0.05    0.0187    0.0808
    1.00       0.05    0.0208    0.0851
    1.10       0.05    0.0228    0.0894
    1.20       0.05    0.0249    0.0931
    1.30       0.05    0.0270    0.0969
    1.40       0.05    0.0291    0.1006
    1.50       0.05    0.0312    0.1044
END FTABLE 12
```