Contra Costa Clean Water Program
Hydromodification Technical Report

__Submitted to:__
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List of Abbreviations

APWA	American Public Works Association
BAHM	Bay Area Hydrology Model
BASMAA	Bay Area Stormwater Management Agencies Association
CASQA	California Stormwater Quality Association
CCCWP	Contra Costa Clean Water Program
DMAs	Drainage Management Areas
Ep	Erosion potential
FDC	Flow-duration-control
FSURMP	Fairfield-Suisun Urban Runoff Management Program
HM	Hydromodification Management
HMP	Hydrograph Modification Management Plan
HSPF	Hydrologic Simulation Program – FORTRAN
IMPs	Integrated Management Practices
LARWQCB	Los Angeles Regional Water Quality Control Board
LID	Low Impact Development
MAP	Mean annual precipitation
MRLC	Multi-Resolution Land Characteristics Consortium
MRP	Municipal Regional Stormwater NPDES Permit
NRCS	Natural Resources Conservation Service
Q10	Flow rate that is equaled or exceeded an average of once per ten years
Q2	Flow rate that is equaled or exceeded an average of once per two years
Qc	Incipient motion or critical flow in a receiving stream
Qcp	Assumed lower threshold for sediment movement
SCCWRP	Southern California Coastal Water Research Project
SCVURPPP	Santa Clara Valley Urban Runoff Pollution Prevention Program
SFRWCQB	San Francisco Bay Regional Water Quality Control Board
SSQP	Sacramento Stormwater Quality Partnership
USGS	United States Geological Survey
VCSQMP	Ventura Countywide Stormwater Quality Management Program
WWHM	Western Washington Hydrologic Model

1. Introduction

Regulatory Requirements

The Contra Costa Clean Water Program (CCCWP) comprises Contra Costa County and the 19 cities and towns within the County. These Contra Costa municipalities, as well as municipalities in Alameda, Santa Clara, and San Mateo Counties, are Permittees under the Municipal Regional Stormwater NPDES Permit (MRP) issued by the San Francisco Bay Regional Water Quality Control Board (Water Board). The Cities of Vallejo, Suisun City, and Fairfield, in Solano County, are also MRP Permittees.

Pursuant to MRP Provision C.3.g., the Permittees require Hydromodification Management (HM) measures to be implemented on development projects. The requirements apply to projects that create or replace an acre or more of impervious area and increase the total amount of impervious area on the project site, subject to specified exclusions.

Provision C.3.g. states in part:

> *Stormwater discharges from HM Projects shall not cause an increase in the erosion potential [Ep] of the receiving stream over the pre-project (existing) condition. Increases in runoff flow and volume shall be managed so that post-project runoff shall not exceed estimated pre-project rates and durations, where such increased flow and/or volume is likely to cause increased potential for erosion of creek beds and banks, silt pollutant generation, or other adverse impacts on beneficial uses due to increased erosive force.*

Contra Costa Permittees' criteria for HM measures—including factors for sizing HM facilities, called Integrated Management Practices or IMPs—are incorporated in CCCWP's *Stormwater C.3 Guidebook,* now in its 7th Edition (2017). The IMPs include bioretention, bioretention combined with upstream or downstream storage, and infiltration facilities (dry wells and infiltration basins).

Provision C.e.g.vi.(2) was added when the MRP was reissued (MRP 2.0) in late 2015. It states:

> *Contra Costa Permittees shall, with the 2017 Annual Report, submit a technical report, acceptable to the Executive Officer, consisting of an HM Management Plan describing how Contra Costa will implement the Permit's HM requirements (e.g. how it will update and modify its practices to meet Permit requirements). At a minimum, the technical report shall provide additional analysis and discussion as to how existing data appropriately evaluates how existing practices available for use meet the Permit's HM requirements, including limit conditions. The report shall, as necessary, propose modifications to Contra Costa's current HM practices, or propose alternate practices that have been accepted by the Water Board, to meet the Permit's HM requirements. The report may also: provide additional data on monitored installations; provide additional analysis and discussion as to how existing and additional data appropriately evaluates existing practices, including limit conditions and the range of conditions present across Contra Costa County; and provide other information or discussion, as appropriate.*

CCCWP's approach to fulfilling these requirements is to propose a methodology and sizing factors based on direct calculation of Ep, as referenced in MRP Provision C.3.g.iii. That provision states:

> *The Permittees may, collectively, propose an additional method, using direct simulation of erosion potential, by which to meet the HM standard in Provision C.3.g.ii. Such a method shall be submitted to the Water Board for review and shall not be effective until approved by the Executive Officer. At a minimum a proposal to use this additional method shall demonstrate that stormwater discharges from HM Projects using the method will not cause an increase in the erosion potential of the*

receiving stream over the pre-project (existing) condition, and that increases in runoff flow and volume will be managed so that post-project runoff does not exceed estimated pre-project rates and durations, where such increased flow and/or volume is likely to cause increased potential for erosion of creek beds and banks, silt pollutant generation, or other adverse impacts on beneficial uses due to increased erosive force. Such demonstration shall include, but not be limited to:

(1) An appropriately detailed discussion of the theoretical approach behind the method and results for the areas to which it is proposed to be applied;

(2) Appropriate continuous simulation hydrologic modeling using Region-specific field data, including creek data (cross sections, longitudinal data, etc.), precipitation data (a record of at least 30 years of hourly data that is appropriately representative of the areas where the method is to be applied), safety factor(s), and HM control designs; and

(3) A description of how the method will be applied, including any models produced and how they will be used by the Permittees and/or project proponents. Such description shall include a listing of HM controls that may be used to comply with the HM requirements of this Permit, a description, with appropriate technical support, of how they will be sized to comply and how the Permittees will ensure appropriate implementation of the method, and all other necessary information, as appropriate.

In summary, Contra Costa Permittees are required to update their requirements for implementing HM on land development projects. They propose to meet this requirement by recalculating the *Guidebook* sizing factors for HM facilities, using direct simulation of Ep.

HM and Low Impact Development

MRP Provision C.3.c., added to the Permit in 2011, mandates the use of Low Impact Development (LID) to treat runoff from new developments.

The California Ocean Protection Council (2008) describes LID as a

> *… stormwater management strategy aimed at maintaining or restoring the natural hydrologic functions of a site to achieve natural resource protection objectives and fulfill environmental regulatory requirements; LID employs a variety of natural and built features that reduce the rate of runoff, filter pollutants out of runoff, and facilitate the infiltration of water into the ground…*
>
> *…LID design detains, treats and infiltrates runoff by minimizing impervious area, using pervious pavements and green roofs, dispersing runoff to landscaped areas, and routing runoff to rain gardens, cisterns, swales, and other small-scale facilities distributed throughout a site.*

LID was first developed as a comprehensive stormwater management strategy by Prince Georges County (1999). The hydrologic approach is described as follows:

> *The LID approach attempts to match the predevelopment condition by compensating for losses of rainfall abstraction through maintenance of infiltration potential, evapotranspiration, and surface storage, as well as increased travel time to reduce rapid concentration of excess runoff.*

LID seeks to address potential hydrologic impacts of land development by maintaining and restoring site characteristics and conditions at the smallest scale possible. Priority is placed on reducing runoff by limiting impervious surfaces, then on dispersing runoff to landscape within a site, and finally by directing runoff to small-scale facilities integrated into the landscape.

In contrast, HM addresses hydrologic impacts of land development at a watershed scale. Flow criteria are developed for streams draining the watershed, and those criteria are then translated to criteria for development of sites draining to the watershed.

LID promotes a multiplicity of approaches and promotes "green" urban development, while HM specifies that runoff discharges adhere to a specified hydraulic regime.

CCCWP committed to implementing LID beginning in 2003, and published the first edition of the *Stormwater C.3 Guidebook (Guidebook),* emphasizing LID design, in 2004.

The *Guidebook* directs applicants for development approvals to use the following LID strategies:

1. Optimize the site layout by preserving natural drainage features and designing buildings and circulation to minimize the amount of impervious surface.
2. Use pervious surfaces such as turf, gravel, or pervious pavement—or use surfaces that retain rainfall, such as "green roofs."
3. Disperse runoff from impervious surfaces on to adjacent pervious surfaces (e.g., direct a roof downspout to disperse runoff onto a lawn).
4. Drain impervious surfaces to engineered Integrated Management Practices (IMPs), which are typically bioretention facilities, sometimes augmented with additional storage. Other IMPs include flow-through planters and dry wells, which may be used in specific situations. IMPs evaporate and transpire some runoff, infiltrate runoff to groundwater, and/or percolate runoff through engineered soil and allow it to drain away slowly.

Here are some reasons Contra Costa Permittees moved toward mandating LID on new development projects, years before being required to do so (CCCWP, 2016):

- LID removes pollutants from runoff most effectively, by filtering runoff and sequestering pollutants in soil.
- Natural processes renew treatment soils and engineered soil media.
- LID facilities do not hold water or harbor mosquitoes.
- LID mimics the natural hydrology of a site, within the site and downstream.
- LID features and facilities can be attractive landscape amenities, parks, or playgrounds.
- Bioretention and other LID facilities are low-maintenance and easy to inspect.
- Placing runoff dispersal and treatment facilities in high-visibility, well-trafficked areas makes them more likely to be valued and maintained.

Under the direction of the CCCWP Development Committee, which comprises municipal planners and engineers involved in the review of applications for land development approvals, the LID guidance in the *Stormwater C.3 Guidebook* has been updated and refined over 12 years and seven editions.

The *Guidebook* includes tools and instructions, including sizing factors, to simplify sizing of bioretention facilities. This facilitates a creative, iterative design process, and encourages designers to integrate small-scale facilities into landscaped areas distributed throughout the site, which is key to achieving LID objectives (California Regional Water Quality Control Board for the Central Coast Region, 2016).

In summary, LID aims to maintain and restore hydrologic functions of a development site using bioretention and other small-scale facilities distributed throughout a site. Since 2003, Contra Costa Permittees have been working to refine design tools and procedures to facilitate implementation of LID on development sites. These tools and procedures include sizing factors for bioretention facilities and other IMPs.

Design Criteria for HM in CCCWP's Stormwater C.3 Guidebook

HM requirements were added to Contra Costa's NPDES permit in 2003. CCCWP sought a way for local developers to meet the HM criteria by using LID. This was accomplished by creating designs for LID IMPs that can also demonstrably meet HM criteria.

The *Guidebook* includes design criteria and sizing factors that land development engineers may use to determine the minimum required dimensions of a variety of IMPs. The land development engineer divides the development site into discrete Drainage Management Areas (DMAs), determines the amount of equivalent impervious area within each DMA, and uses the *Guidebook* sizing factors, embedded in an accompanying IMP Sizing Calculator, to calculate values for the following parameters for an IMP serving that DMA:

- area, ***A***
- surface storage volume, V_1
- subsurface storage volume V_2

See Figure 1-1. For treatment-only facilities, where HM requirements do not apply, only the minimum area is calculated.

The land development engineer then shows how, for each DMA, the IMP meets or exceeds minimum values for each parameter.

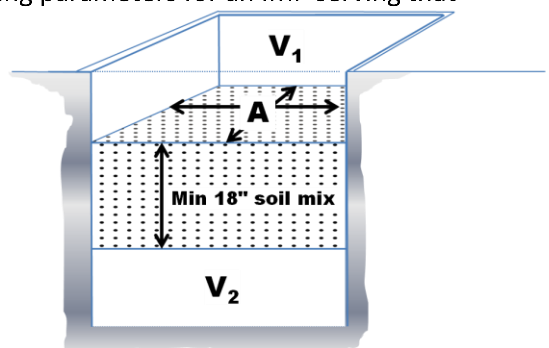


Figure 1-1. A, V_1, and V_2. Note V_2 is the free volume; gravel volume is multiplied by porosity

Bioretention facilities are the most commonly used IMPs on Contra Costa development projects. They are typically constructed for runoff treatment and to maximize retention of runoff via evapotranspiration and infiltration, but the design is adapted to also provide HM. Bioretention facilities work as follows:

Runoff enters the bioretention facility via sheet flow or pipes and is detained in a shallow surface reservoir. The reservoir also serves to spread runoff evenly across the facility surface. Runoff then percolates through an engineered soil (sand/compost mix). Some runoff is retained in soil pores and plant roots and is subsequently evaporated and transpired. Runoff that exceeds the moisture-holding capacity of the soil percolates through the soil layer and enters a subsurface storage layer (typically gravel). The treated runoff subsequently then infiltrates into the soils below the facility. If runoff enters the gravel layer more rapidly than it infiltrates, the saturation level in the gravel layer rises until it reaches the discharge elevation for a perforated pipe underdrain. When this occurs, runoff will also discharge through the perforated pipe underdrain to a discharge point (typically connected to the municipal storm drain system). In general, this discharge will occur rarely—a few times per year, or even once in many years.

In facilities constructed for HM, this perforated pipe underdrain is equipped with a flow-limiting orifice. This allows the bioretention facility to act like a flow duration control basin during the infrequent occasions when the storage layer fills, and as a LID facility at other times.

The surface reservoir is also equipped with an overflow that will become active under either of two scenarios: (1) runoff enters the surface reservoir more rapidly than it percolates through the engineered sand/compost mix, and the surface reservoir fills to its maximum volume or (2) runoff enters the facility more rapidly than it leaves via both infiltration to the soils below the facility and discharge via the underdrain, and this continues until the gravel and soil layers become fully saturated, and the surface reservoir fills to its maximum volume.

The Guidebook includes criteria and sizing factors for three design variations:

1. The Flow-through Planter, which can be built above ground or other locations where infiltration to native soils cannot be allowed.
2. Bioretention + Vault, which includes surface storage and engineered soil, but provides for subsurface storage V_2 in a separate structure rather than a subsurface gravel layer.
3. Cistern + Bioretention, which allows for upstream runoff storage V_1 in a tank or basin; runoff is then metered through an orifice to be treated in a bioretention facility.

The *Guidebook* also includes design criteria and sizing factors for "direct infiltration" facilities, that is, facilities designed to infiltrate runoff directly, without first routing it through a soil layer to remove pollutants. These design criteria and sizing factors for "direct infiltration" can be used to design infiltration basins, infiltration trenches, and dry wells.

For each type of facility, sizing factors are provided for each Hydrologic Soil Group ("A", "B", "C", and "D" soils). Differences in local rainfall are accounted for by a "Rainfall Adjustment Factor" scaled linearly with local mean annual precipitation.

In summary, a bioretention facility receives runoff from a specific delineated area, retains that runoff via infiltration and evapotranspiration, and discharges excess runoff via an underdrain and an overflow. Where HM applies, bioretention facilities are designed with a minimum facility area, a minimum surface storage volume, and a minimum subsurface storage volume, as determined by sizing factors contained in the *Guidebook* and embedded in the IMP Sizing Calculator.

Model Representation of Hydrologic Performance

A project team comprising hydrologists and engineers from Philip Williams & Associates and Brown and Caldwell developed the continuous simulation model used to determine IMP sizing factors. The work was done during 2004-2005. The modeling results formed the basis for the designs and sizing factors proposed in the CCCWP's Hydrograph Modification Management Plan (HMP), submitted to the Water Board in May 2005 and approved by the Water Board, with minor changes, in July 2006.

In 2009, Brown and Caldwell used the same continuous simulation model to create sizing factors for new IMP designs. The new IMP designs and sizing factors were incorporated into an addendum to the 4th Edition of the *Stormwater C.3 Guidebook,* and subsequently carried forward through subsequent editions of the *Guidebook*.

The model was created in HSPF (Hydrologic Simulation Program – FORTRAN). HSPF has a history going back to the 1960s, has been used and endorsed by USEPA, and has been embraced in many parts of the US for evaluation and design of the hydrologic impacts of new developments. The Western Washington Hydrologic Model (WWHM) consists of an HSPF-based simulation and a user interface, as does the Bay Area Hydrology Model (BAHM) currently used in Alameda, Santa Clara, and San Mateo Counties. Because HSPF is widely used, there is a significant body of literature and a community of practitioners to support use of the model in HSPF applications.

In HSPF, the various hydrologic processes are represented as flows and storages. Each flow is an outflow from a storage, which, at each time step, is typically a function of the storage volume at that time step and the physical characteristics of the storage. For undeveloped watersheds, HSPF models the movement of water along three paths: overland flow, interflow, and groundwater flow. A variety of storage zones are used to represent storage that occurs on the land surface and in the soil horizons.

The continuous-simulation model was developed and used to demonstrate that, with the inclusion of appropriately sized IMPs in a development project, increases in runoff flow and volume are managed so that post-project runoff does not exceed estimated pre-project rates and durations.

This requires that the model generate representation of pre-project flows at each time step over a long period, as well as post-project flows at each time step during that same period. It is then possible to make statistical cumulative comparisons of the two sets of generated data.

To develop the model, the consultant team:

- Characterized pre-project runoff peaks and durations for a range of soil groups, vegetation, and rainfall patterns characteristic of Contra Costa County development sites.
- Modeled outflow peaks and durations from several IMP designs (based on a unit area of new impervious surface draining to the IMP).
- Compared modeled pre-project flows to modeled post-project-with-IMP flows, using conservative assumptions.
- Developed calculations for sizing factors for each IMP associated with each pre-project condition.

To model the IMPs, the consultant team constructed representations of each IMP in HSPF. For example, a bioretention facility is represented in HSPF by length, cross-section geometry, layers of soil and underdrain material, and transmissivity of underlying soils.

In 2011-2012, CCCWP monitored the discharge from three bioretention facilities in Pittsburg, CA, and two bioretention + vault facilities in Walnut Creek, CA. Slotted-standpipe monitoring wells were installed in the gravel layer of each of the Pittsburg facilities. The results of this monitoring were used to calibrate the continuous-simulation model. See Topic #12 in the Literature Review below.

For the current project, the model was adapted and extended for direct simulation of erosion potential, see the discussion in Section 3.4.

In summary, a continuous simulation model, created in HSPF, was developed and used to calculate sizing factors for bioretention facilities and other IMPs. The sizing factors, along with detailed design criteria, are included in CCCWP's *Stormwater C.3 Guidebook.* The current effort extends this continuous-simulation model to recalculate the sizing factors using direct simulation of erosion potential, rather than flow duration control.

Collaborative Process

MRP 2.0 was approved by the Water Board in late November 2015. CCCWP contacted Water Board staff in January 2016 to propose a collaborative process in preparing the report.

Following up on this early discussion, CCCWP incorporated funding for this project in its FY 2016-2017 budget and, in May 2016, invited Dubin Environmental Consulting and Geosyntec Consultants to jointly prepare a proposal to do the work. The invitation included a request and specifications for a detailed Work Plan.

CCCWP contracted with Dubin/Geosyntec, who prepared a Work Plan that was discussed at an October 21, 2016 meeting with Keith Lichten and Dale Bowyer of Water Board staff.

The key outcomes of this meeting were documented in a meeting summary provided to Water Board staff on October 31, 2016:

1. In the Technical Report, CCCWP will propose a method using direct simulation of Ep to develop sizing factors for HM facilities, pursuant to MRP Provision C.3.g.iii.

2. A literature review will provide background and justification for the method.
3. The literature review will also note and describe advances and alternatives in hydromodification management, and note significant factors, such as sediment supply, and changes to the distribution of flows and seasonal timing of flows, that may not be incorporated in SF Bay Area approaches and standards.
4. A continuous simulation model of Ep, consistent with the methods used in the 2005 Santa Clara HMP, will be used.
5. A continuous simulation model of IMP performance, consistent with the methods used in the 2005 Contra Costa HMP, will be used.
6. The combined continuous simulation model will be used to examine the sensitivity of IMP sizing factors to a range of values of key parameters. The sensitivity for individual parameters will be examined. For a subset of the most sensitive parameters, the sensitivity of IMP sizing to combinations of values will be examined.
7. The sensitivity analysis will be used to promote collaboration and transparency in making decisions regarding parameter values to be used and the resulting sizing factors. During interim check-ins with Water Board staff, the consultant team will review proposed selections of parameters and ranges of parameter values.

CCCWP received further comments from Keith Lichten on December 7, 2016, requesting that the technical report address:

1. Basis for direct simulation of Ep, including experience with implementing the method, uncertainties, unknowns and approaches for applying an appropriate factor of safety.
2. Comparison with the approach used by Santa Clara and Alameda permittees, including differences in how control requirements affect facility design, construction and operation.
3. Opportunities for additional benefits that direct simulation of Ep might provide.

A meeting was held at Water Board offices on March 20, 2017 to review progress on the Technical Report. This meeting included a detailed review of the direct simulation methodology and a comparison to the flow-duration control methods used in BAHM (and currently used in Contra Costa), review of field investigations of 15 Contra Costa sites on creeks downstream from areas where development is likely to occur (conducted in early 2017), and the outcome of initial modeling runs. Conclusions from the initial modeling runs were presented as follows:

Minimum HM facility sizes are:

- Not sensitive to geomorphic parameters such as channel dimensions or slope.
- Sensitive to the assumed lower threshold for sediment movement (Qcp).
- Sensitive to the facility exfiltration rate to native soils.
- Sensitive to assumptions regarding future increases in watershed imperviousness.

Following this meeting, Dubin/Geosyntec proceeded with the modeling analysis. Results were incorporated into "read-ahead" slides provided to Water Board staff on July 13, 2017 and reviewed at a meeting with Keith Lichten, Dale Bowyer, and Selina Louie on July 20.

At this meeting, Dubin/Geosyntec showed how the three "sensitive" parameters above interact to affect minimum sizing factors. It was shown how a selected value for sizing factor could be fully protective for a broad variety of reasonable combinations of Qcp, facility exfiltration rate, and assumed future increase in watershed imperviousness. It was proposed to use this approach rather than using the "most conservative" values for all three sensitive parameters.

It was further proposed that this report would recommend an appropriate sizing factor for the "base case" of a bioretention facility in Hydrologic Soil Group "D" soils, which represents most future development in Contra Costa. Representative values for the three sensitive parameters that correspond to this selected sizing factor would then be used to generate the remaining sizing factors (for other facility types and other soil groups).

In summary, CCCWP and Water Board staff met and agreed on a Work Plan. This was followed by a midpoint meeting to review preliminary results, and a final meeting to review key project outcomes and steps for submitting the report and implementing updated and recalculated IMP sizing factors.

2. Literature Review

MRP Provision C.3.g.iii.(1) requires that a proposal to use direct simulation of erosion potential to meet the HM Standard in Provision C.3.g.ii. and include "an appropriately detailed discussion of the theoretical approach behind the method… "

The HM Technical Report Workplan (CCCWP, 2016) identified specific items to be addressed in this literature review. These items have been organized into the topics shown in Table 2-1.

For more general background in hydromodification management, the following are referenced:

- Santa Margarita Region HMP (2013), Appendix C (http://www.waterboards.ca.gov/rwqcb9/water_issues/programs/stormwater/docs/rsd_permit/hmp/SMR_HMP_App_C.pdf)
- Sacramento Stormwater Quality Partnership HMP (2013) (http://www.beriverfriendly.net/docs/files/File/HMP/HMP_Feb2013.pdf)
- Contra Costa HMP (2006) (http://www.cccleanwater.org/Publications/HMP/CCCWP_HMP_Final_051505-rev041906.pdf)

Table 2-1. Topics addressed in literature review

Topic No.	Topic Title
1	Define what Erosion Potential is and how it is calculated.
2	Summarize the technical basis for the Ep management objective written into the current MRP HM Standard and the basis for the flow-duration-control (FDC) criteria the MRP HM Standard specifies as the means to demonstrate the Ep management objective is met.
3	Summarize the strengths and limitations of using an Ep management objective to address hydromodification management.
4	Summarize the strengths and limitations of use of the flow duration control and Ep control standards as a means to meet an Ep management objective.
5	Identify and briefly analyze the assumptions inherent in extrapolating from an Ep management objective developed at the watershed scale to an FDC and Ep Control standard applicable to small catchments.
6	Describe the basis for the Ep control standard, particularly with regard to how it is similar to, and different from, flow duration control.
7	Identify and describe any instances where direct simulation of Ep has been used in the design of HM facilities. Note similarities and differences with those uses of Ep compared to the LID drainage design procedures practiced in Contra Costa County.
8	Describe generally the hydrologic objectives and effects of LID, including the maintenance of pre-development hydrology and water balance
9	Describe the hydrologic performance of bioretention with no underdrain outlet with regard to small and large runoff events
10	Summarize the Program's adaptation of bioretention to meet the Water Board's FDC criteria
11	Summarize how LID features and facilities are modeled in BAHM. Compare and contrast the approach used in the Stormwater C.3 Guidebook and IMP Sizing Calculator
12	Summarize the calibration of the Program model as detailed in the 2013 IMP Monitoring Report

Topic #1. Define Erosion Potential and how it is calculated.

Erosion Potential (Ep) is expressed as the ratio of post-project to pre-project (post/pre) long-term "work done" on the stream (SCVURPPP, 2005). The Ep ratio is a commonly used metric in the field of fluvial geomorphology to quantitatively predict hydromodification impacts while taking into account the hydrology, channel geometry, and bed and bank material of streams and how these factors change as a result of altered land use (e.g., urbanization) (BASMAA, 2015). The Ep metric calculation combines in-stream hydraulic calculations with continuous rainfall-runoff simulations for the entire range of flow events at representative reaches along a stream (CASQA, 2009). An Ep equal to one represents a post-development condition with the same transport capacity [or total work] as the pre-development condition, whereas an Ep greater than one indicates a higher transport capacity [or total effective work] in the post-development condition (CASQA, 2009).

The theory, logic, and practical application of the Ep metric to predict geomorphic impacts caused by land use changes is described in a technical paper titled Predicting Hydromodification Impacts Using a Four Factor Approach (Goodman et al, 2011). General steps to calculating Ep are also described in Appendix B of the technical report titled Hydromodification Assessment and Management in California (SCCWRP, 2012) commissioned and sponsored by the California State Water Resources Control Board. Detailed step-by-step guidance for performing Ep calculations and sizing hydromodification controls to meet the Municipal Regional Permit's HM Standard is in Appendix D of the Vallejo HMP (City of Vallejo, 2013). The approach used in this project is summarized in Section 3.4.

Direct simulation of Ep includes the following steps, as illustrated in Figure 2-1:

1. Continuous hydrologic analysis, to produce pre- and post-project flow duration histograms (see blue histogram in Figure 2-2);
2. Hydraulic analysis, to calculate stage, mid-channel flow velocity, and effective shear stress for the range of simulated flow output using channel geometry and roughness characteristics;
3. Work analysis, to produce a work rating curve or histogram using hydraulic output (Step 2) and bed/bank material strength characteristics as inputs to an effective work formula or sediment transport capacity relationship (see red histogram in Figure 2-2);
4. Cumulative work analysis, which integrates the work rating curve (Step 3) with the flow duration histogram (Step 1) to calculate long-term total work (see purple histogram in Figure 2-2); and
5. Ep analysis, which divides the total work of the post-project condition by that of the pre-project condition (post/pre).

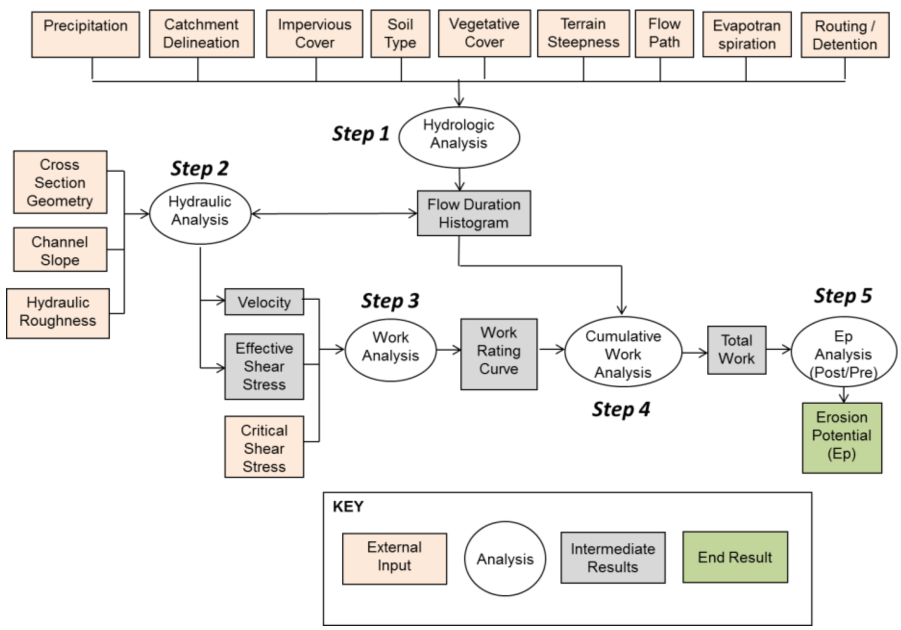


Figure 2-1. Erosion potential analysis flow chart (City of Vallejo, 2013)

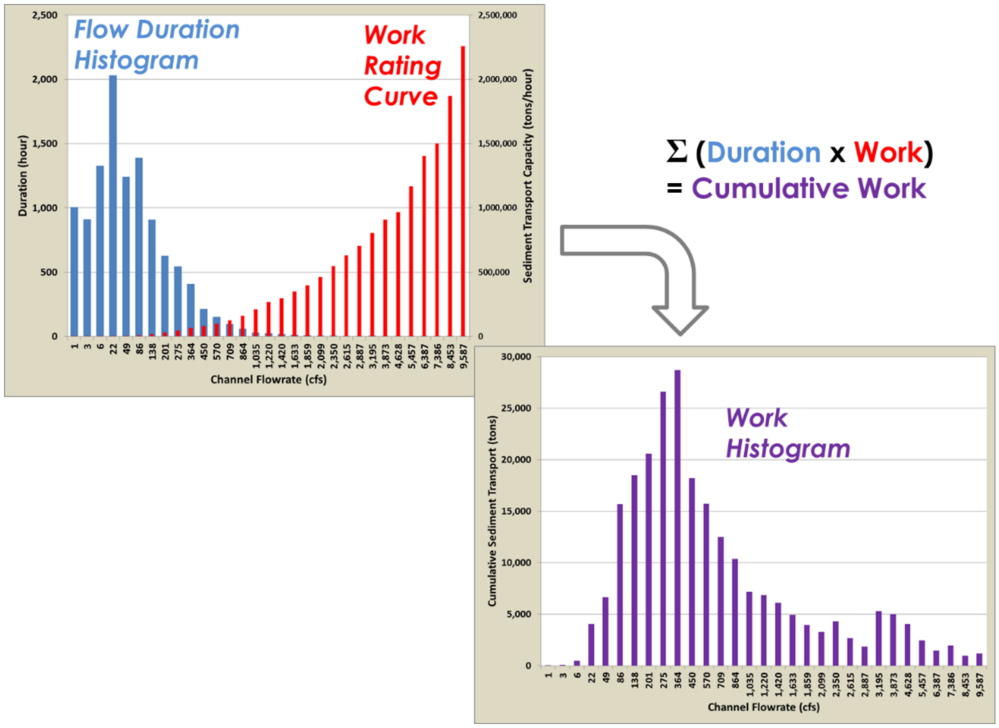


Figure 2-2. Example flow duration histogram, work rating curve, and work histogram (City of Vallejo, 2013)

Topic #2. Summarize the technical basis for the Ep management objective written into the current MRP HM Standard and the basis for the flow-duration-control (FDC) criteria the MRP HM Standard specifies as the means to demonstrate the Ep management objective is met.

Basis for the MRP Ep Management Objective. The primary technical basis for the Ep management objective written into the current HM Standard comes from a comprehensive literature review and a stream assessment completed for three watersheds in Santa Clara County, mapped in Figure 2-3, which supported the SCVURPPP HMP (2005). The literature review and stream assessment were reviewed by an Expert Panel (Professor Brian Bledsoe, Ph.D., Colorado State University; Professor Tom Dunne, Ph.D., UC Santa Barbara; and Professor Matt Kondolf, Ph.D. UC Berkeley) to help ensure that the HMP was scientifically defensible and embodies a sound approach to hydromodification management (SCVURPPP, 2005). This stream assessment, documented in Chapter 3 of the SCVURPPP HMP, included a geomorphic field assessment of erosion stability at 45 cross sections. These stability results were compared to the calculated Erosion Potential (Ep), as presented in Figure 2-4, which demonstrates a strong correlation between computed Ep and the field designated stability. Figure 2-5 shows a logistic regression curve plotting the likelihood of instability (y-axis) versus Erosion Potential (x-axis). The study concluded:

- Different creeks in the same region do not warrant a separate hydromodification standard, criteria or threshold of adjustment, and
- The transition between stable and unstable channels occurs between Ep values of 1 and 1.2, which translates to a likelihood of instability from 9 to 17 percent.

A similar and more recent study for Southern California streams (Hawley and Bledsoe, 2013) also demonstrated a strong correlation between calculated Ep and channel instability, as quantified by channel enlargement ratio. Based on a Southern California Coastal Watershed Project report on modeling and managing hydromodification effects (SCCWRP, 2013), which utilized the Hawley and Bledsoe data, Southern California channels appear to be more sensitive to hydromodification impacts than those analyzed in the

San Francisco Bay Area (i.e., the same Ep results in higher probability of instability) (VCSQMP, 2013). For comparison, purposes, the logistic regression curves for Santa Clara County (SCVURPPP, 2005) and Southern California (SCCWRP, 2013) are shown on the same plot in Figure 2-6. One caveat to this comparison is that the Southern California study used a different method of calculating Ep, one which relies on empirically derived flow duration relationships instead of hydrologic simulations (State Water Board, 2013).

In Chapter 5 of the SCVURPPP HMP (2005), the HMP's management objective was developed using the Ep index as a point of reference, as follows:

> *Stormwater discharges from a non-exempt, Group 1 development project shall not cause an increase in the erosion potential of the receiving stream over the pre-project (existing) condition, i.e., an Ep of up to 1.0 will be maintained for stream segments downstream of the project discharge point.*

This Ep management objective was incorporated directly into the HM Standard of the previous MRP (SFRWCQB, 2009) and current MRP provision C.3.g.ii as follows (SFBRWQCB, 2015):

> *Stormwater discharges from HM Projects shall not cause an increase in the erosion potential of the receiving stream over the pre-project (existing) condition.*

<u>*Basis for Flow Duration Control Criteria.*</u> The primary technical basis for the flow duration control (FCD) criteria is presented in Chapter 4 of the SCVURPPP HMP (2005). Based on a technical analysis of hydromodification controls, BMPs designed for discrete event volume control or hydrograph matching do not provide adequate protection of the erosion potential of streams, but FCD does. In Ontario, Canada, MacRae (1996) observed that attempts to control peak flow (i.e., the 2-year discharge) with no consideration for duration of flows resulted in equally degraded streams as implementing no BMP at all (SCVURPPP, 2005). This fact was also recognized in western Washington (state), where the Department of Ecology (2000) adopted a flow duration control standard in which pre- and post-project flow duration curves must be matched (SCVURPPP, 2005).

In Chapter 5 of the SCVURPPP HMP (2005), on-site FDC is one of five approved performance criteria for meeting the erosion potential management objective. The first sentence of the FDC performance criteria states the following (SCVURPPP, 2005):

> *On-site controls that are designed to provide flow duration control to the pre-project condition are considered to meet the erosion potential management objective and comply with the HMP.*

This FDC performance criteria was adopted into the HM Standard of the previous MRP (SFRWCQB, 2009) and current MRP provision C.3.g.ii as follows (SFBRWQCB, 2015):

> *Increases in runoff flow and volume shall be managed so that post-project runoff shall not exceed estimated pre-project rates and durations, where such increased flow and/or volume is likely to cause increased potential for erosion of creek beds and banks, silt pollution generation, or other adverse impacts on beneficial uses due to increased erosive force.*

MRP provision C.3.g.ii provides criteria for the range of flows to control, goodness of fit criteria, and standard HM modeling to demonstrate that post-project stormwater runoff does not exceed estimated pre-project runoff rates and durations (SFBRWQCB, 2015).

With regard to the range of flows to control, the lower flow threshold for incipient motion or critical flow in a receiving stream (Qc) is defined as the smallest flow that begins movement of the bed material or erosion of the bank (SCVURPPP, 2005). Flows less than this value do not substantially move bed material or erode the bank. Qc can be normalized as a percentage of the receiving stream's pre-development 2-year peak flowrate. The allowable low flow threshold from a flow control structure on a project site (Qcp) can be estimated as the same percentage, but of the pre-development 2-year peak flow from the project catchment. Guidance on calculating the low flow threshold is provided in Appendix A, Section 2 of the State Water Board's technical report on hydromodification (SCCWRP, 2012). Step-by-step guidance for calculating the low flow threshold to meet the MRP HM Standard is provided in Appendix C, Section 4.1 of the Vallejo HMP (City of Vallejo, 2013).

Although FDC is considered to be only one of five acceptable performance criteria stated in the SCVURPPP HMP, it has become the most popular criteria for complying with MS4 hydromodification management requirements in the Bay Area and California. In some MS4 permits and HMPs in California, FDC has been adopted as the sole performance standard for hydromodification management, even though FDC was intended to be one option for meeting the overarching erosion potential management objective. A comparison of hydromodification management performance standards in California is in Table 2-2 (CASQA, 2013 and APWA, 2013).

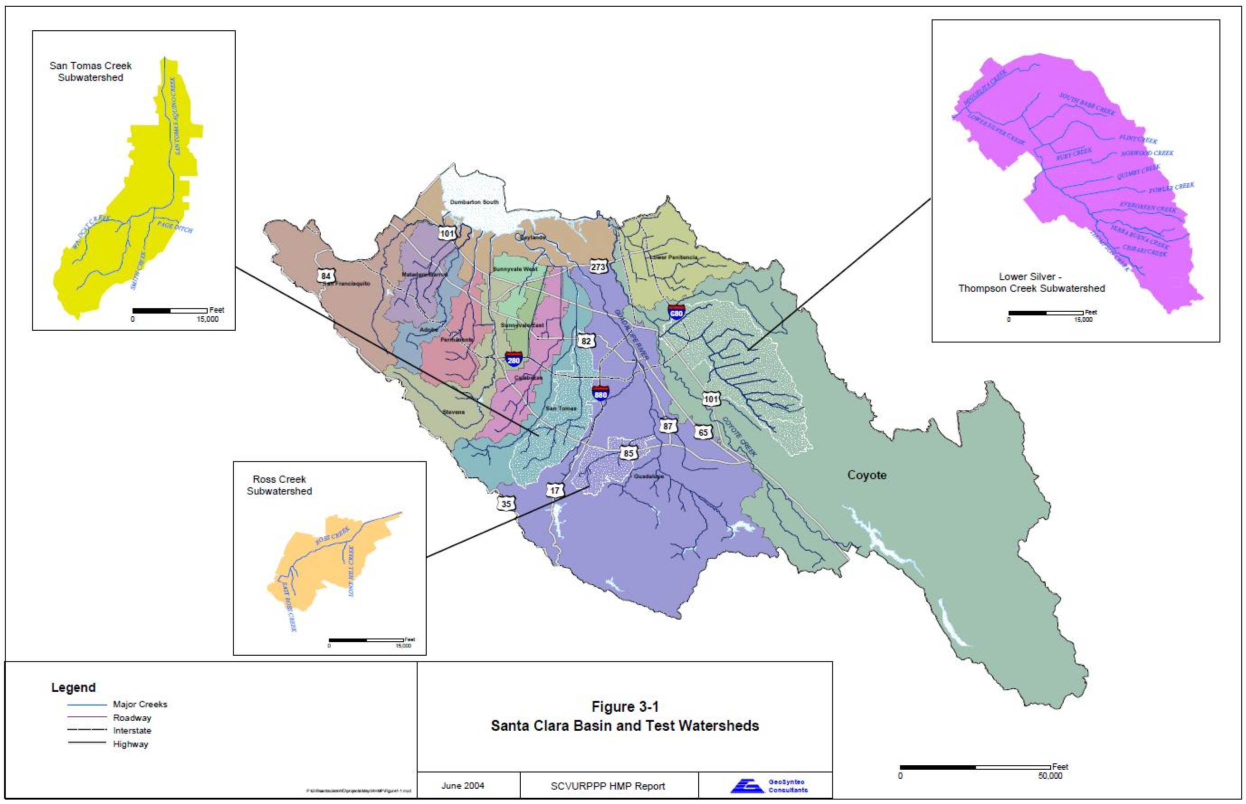


Figure 2-3. Santa Clara Basin and test watersheds (SCVURPPP, 2005)

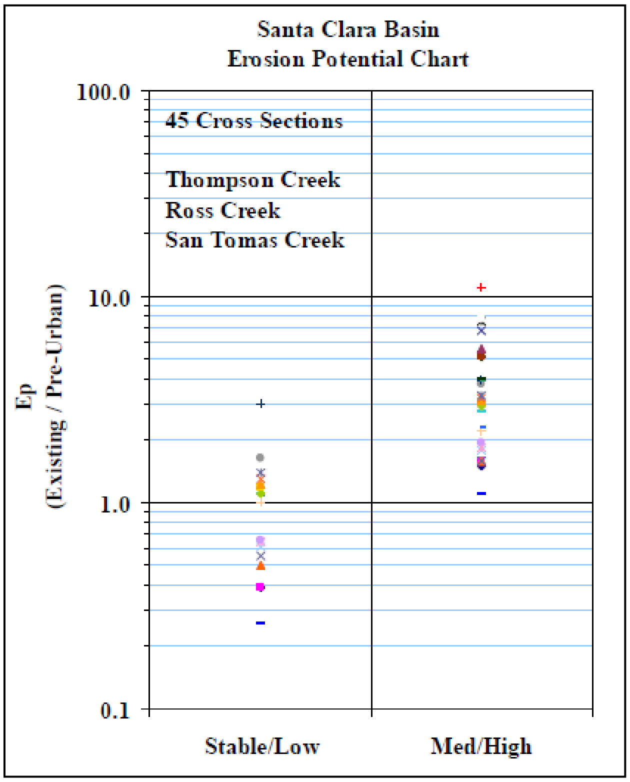


Figure 2-4. Erosion potential chart for Santa Clara Basin streams (SCVURPPP, 2005)

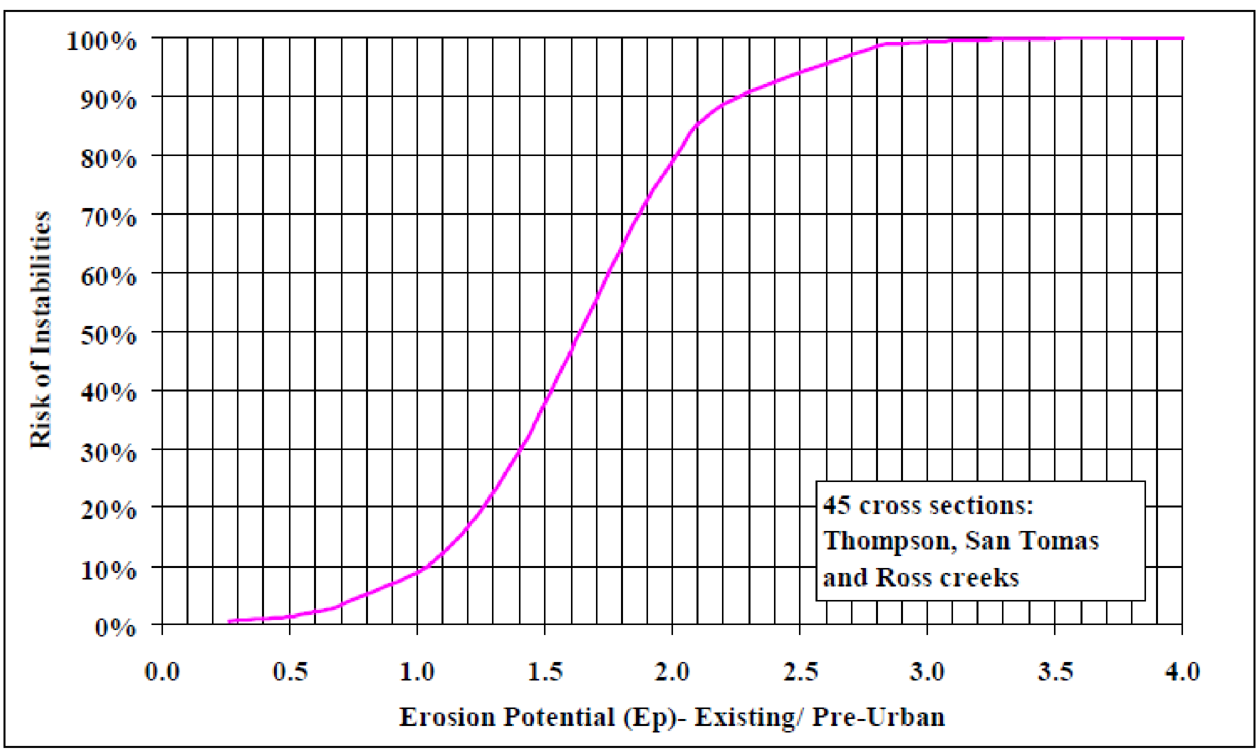


Figure 2-5. Probability of a stream segment becoming unstable, based on logistic regression of Ep values (SCVURPPP, 2005)

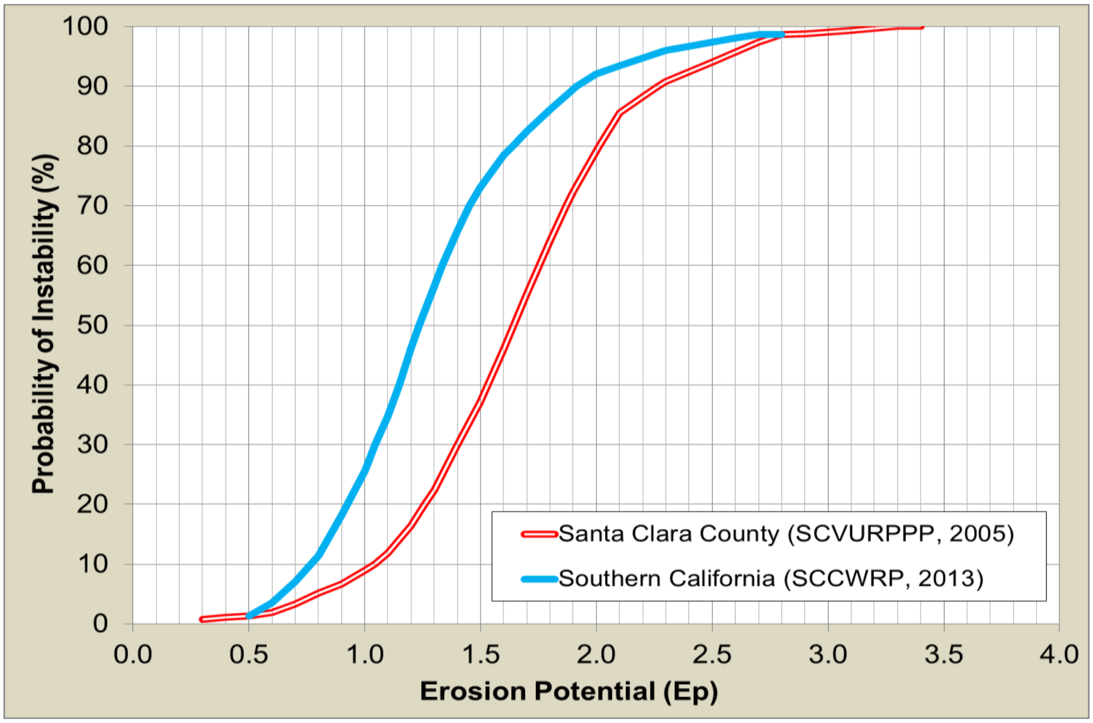


Figure 2-6. Comparison of logistic regression models of Ep and probability of instability for Santa Clara County (San Francisco Bay Area) and Southern California (VCSQMP, 2013)

Table 2-2. Comparison of hydromodification management performance standards in California (CASQA, 2013 and APWA, 2013)

Elements of Hydromodification Management		**Santa Clara County**	**Alameda County**	**San Mateo County**	**Contra Costa County**	**Fairfield-Suisun**	**Vallejo**	Sacramento County	Santa Rosa	San Diego County	South Orange County	North Orange County	Riverside County - Santa Ana Region	Riverside County - Santa Margarita Region	San Bernardino County	Los Angeles County	Ventura County	Central Coast - Phase II	Phase II
Performance Standard	Match Design Event Hydrology											x	x		x	x		x	x
	Retain Runoff from XXth Percentile 24-hr Storm								x							x		x	
	Flow Duration Control	x	x	x	x	x	x	x		x	x			x			x		
	Erosion Potential	x		x		x	x			x						x	x		
	Account for Sediment Supply Changes									x	x		x	x	x				
	Impracticability Provision	x	x	x		x					x	x	x	x	x				

Topic #3: Summarize the strengths and limitations of using an Ep management objective to address hydromodification management.

Strengths and limitation of using an Ep management objective to manage for hydromodification are provided in the technical report *Hydromodification Assessment and Management in California*, commissioned and sponsored by the California State Water Resources Control Board Stormwater Program (SCCWRP, 2012). The reports states the following:

The Erosion Potential approach combines a sound physical basis with probabilistic outputs and requires a substantial modeling effort. Such an effort is necessary to adequately characterize the effects of hydromodification on the stability of streams that are not armored with very coarse material such as large cobbles and boulders. Although policies based on this approach should reduce impacts to channel morphology, they may still fail to protect stream functions and biota. Key simplifying assumptions and prediction uncertainty in the inputs (hydrologic modeling, assumptions of static channel geometry in developing long term series of shear stresses or stream powers, assumptions of stationarity in sediment supply, etc.) have not been rigorously addressed. Its effectiveness also depends on careful stratification of streams in a region such that fundamentally different stream types are not lumped together (e.g. labile sand channels vs. armored threshold channels with grade control) in developing general relationships for instability risk. Endpoints to date have been rather coarse, e.g. stable vs. unstable; as such, they do not provide sufficient resolution for envisioning future stream states. However, the Erosion Potential approach provides promise as an important tool for hydromodification management; it is recommended that it be refined to address sediment supply changes and to provide more finely resolved endpoints for improved predictive capabilities.

Impacts to stream habitat can occur from multiple, interrelated causes (CASQA, 2009). Habitat changes are not only a result of stream instability and erosion, but can also result directly from changes in streamflow regime (CASQA, 2009). For example, increases or decreases in base flow or changes to the seasonal availability of water will directly determine the extent and type of riparian vegetation capable of thriving in that environment (CASQA, 2009). Furthermore, an increase in the availability of water in a naturally intermittent or ephemeral system may allow invasive vegetation to become established and out-compete native plants (CASQA, 2009). As alluded to in the report commissioned by the State Water Board (SCCWRP, 2012), Ep aggregates flows/work without regard to timing, whereas analysis to support biota and its habitat might focus on the seasonality of runoff and perhaps a water balance. Thus, while Ep is a scientifically defended means for quantifying potential impact on channel morphology and does address one factor for biological health, it is not a metric that addresses all factors that contribute to biological and ecological impact.

Reduction in sediment supply is an inseparable component of hydromodification for alluvial live-bed streams, as it can have similar impacts as increased flows and durations (CASQA, 2009). If severe enough, sediment supply reductions due to urbanization or in-stream dams can starve downstream reaches of the bed load it naturally transports and thus the water flowing in the channel becomes “hungry water”, meaning the water is more prone to eroding in-stream bed and bank material (Kondolf 1997). Hungry water is more erosive because if the supply of bed sediment stops while the stream flow continues conveying bed load, then the only source of sediment available for transport is from the material that forms the channel itself. Changes in bed sediment supply, for alluvial live-bed stream channels, can be accounted for by reducing the post-project total work below that of the pre-project (i.e., Target Ep less than 1.0) in proportion to the reduction in bed sediment supply (post/pre). This represents the best current understanding of how to quantitatively account for sediment supply changes (Palhegyi and Rathfelder, 2007) without conducting complicated sediment budget analysis and sediment transport modeling. However, the approach of reducing the Target Ep is considered a conservative rule-of-thumb and does not have a strong scientific basis because the studies performed to date (SCVURPPP, 2005 and SCCWRP, 2013) have focused solely on correlating Ep to channel instability, not Ep and sediment supply reductions.

Design of stormwater BMPs to achieve the Ep management objective (i.e., via FDC or Ep control) requires continuous long-term hydrologic modeling of the project site. If different portions of the project site discharge to different receiving channels downstream, then a separate hydrologic analysis is needed for each associated outlet and tributary area onsite. Theoretically, changes in flow durations and erosive work associated with climate change could be modeled by adjusting the precipitation record used for hydrologic simulation. However, scientific understanding of how climate change will alter precipitation records for a specific region is not conclusive. The San Francisco Bay Area Climate-Smart Watershed Analyst Tool (USGS, 2017a), which is in beta version, provides predicted adjustments to monthly precipitation based on fourteen different climate models; however, these different climate models yield drastically different results. While some climate models predict dramatic increases in precipitation depth, others predict decreases, as shown in Figure 2-7.

Channel shape and material type are integral factors to stream stability and define a channel's morphology. Direct alterations to receiving streams can have positive or negative hydromodification consequences. While geomorphically an Ep management objective can take into account such direct alterations, it cannot from an ecological or biological perspective. Urbanization has historically affected channel geometry by narrowing stream corridors (e.g., with constructed levees) so that the floodplain can be developed for residential, commercial, industrial, or agricultural uses. This confinement not only destroys sensitive floodplain ecosystems, limits suspended sediment deposition overbank, and reduces in-stream infiltration, but also creates a more energetic stream system that is more prone to in-stream erosion. As the impacts of increasing discharge (magnitudes and durations) and reducing sediment supply have become better documented and understood, the water engineering practice has begun to implement channel geometry alterations in order to reduce the energy of streams and maintain stable equilibrium. These compensations include increasing channel width to depth ratio and reducing longitudinal slope by installing in-stream grade control or drop structures. In-stream construction is a disturbance to the riparian ecosystem itself, so often these alterations of channel geometry are implemented only after a stream reach is already unstable. Urbanization can impact bed and bank material strength if natural channels are physically modified or replaced with constructed channels. Obviously, constructing a new channel in place of a natural one carries a tremendous geomorphic impact in itself and has ramifications to the native riparian habitat being removed while also impacting the longitudinal connectivity of a stream system. However, coarsening the bed and banks by carefully placing larger rock or riprap or logs in-stream can increase the channel's resistance to movement and has been used to help stabilize channels in the urban setting. Lining channels in concrete was a popular method of stabilizing urban surface water channels in the 20th century because concrete channels eliminate in-stream erosion and reduce hydraulic roughness. A hydraulically smooth channel can convey greater discharge in a narrower corridor, thus maximizing the area that can be developed. Constructing concrete lined channels is no longer a common practice because of the impact it has on the natural riparian ecosystem, but more ecologically sensitive methods of strengthening bed and banks are being used such as vegetation reinforcement, mechanically stabilized earth, root wad structures, and buried grade control. Although these practices are considered more environmentally sound, they do impact channel form and should be taken into account when predicting channel form adjustment.

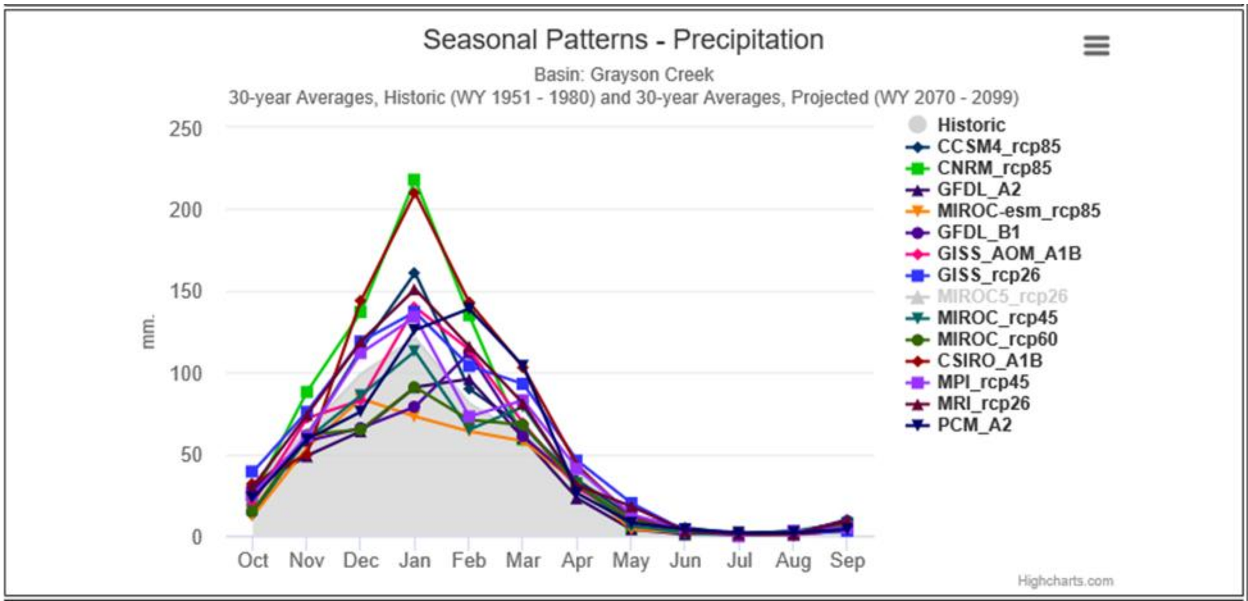


Figure 2-7. Example of predicted changes in seasonal precipitation based on available climate change models (USGS, 2017a)

Topic #4. Summarize the strengths and limitations of use of the flow duration control and Ep control standards as a means to meet an Ep management objective.

The current state of the practice for hydromodification management in California for new and redevelopment is to mimic pre-development hydrology on the project site (SSQP, 2014 and VCSQMP, 2013). The theory is that if the pre-development distribution of in-stream flows is maintained, then the baseline capacity to transport sediment, a proxy for the geomorphic condition, will be maintained as well. A popular method of mimicking the pre-development flow regime is by maintaining the pre-development distribution of runoff, known as flow duration control. This can be done onsite by routing post-development runoff through structural stormwater BMPs such that runoff is stored and slowly released to match pre-development flow duration characteristics. Applying FDC to achieve the pre-project condition is considered to be fully protective of the existing condition of the channel segment to which the project discharges (SSQP, 2014 and VCSQMP, 2013).

Flow duration matching does not require additional watershed or receiving channel analyses to ensure that Ep is being maintained in the downstream creek segments, but it does not prohibit it either (SSQP, 2014 and VCSQMP, 2013). The critical flow in a receiving stream (Qc) is defined as the smallest flow that begins movement of the bed material or erosion of the bank (SCVURPPP, 2005). Flows less than this value do not substantially move bed material or erode the bank. The allowable low flow discharge from the project site (Qcp) can be estimated as a default percentage of the pre-project 2-year peak flow from the project site if additional analyses are not performed. This default is considered appropriately protective of most receiving streams of concern for a particular region based on prior incipient motion analysis, performed at the time that an HMP was developed. A table of default low flow discharges used throughout California are presented in Table 2-3. Additional analyses needed to evaluate an alternative Qcp, expressed as a percentage of the 2-year peak flow, would require an incipient motion or bed mobilization analysis of the receiving creek segments downstream of the project discharge point and a hydrologic analysis to evaluate the 2-year peak flowrate at each creek location analyzed.

While theoretically FDC maintains the pre-project sediment transport capacity for the full distribution of erosive flows, in practice it is difficult to achieve a good match for the entire range of flows evaluated if typical onsite LID BMPs are used (VCSQMP, 2013). This is because the outlet structure configuration of a typical LID BMP consists of a simple overflow weir and a low flow orifice (if needed). However, to get a good match of the flow duration curve with passive controls such as LID BMPs, a more complicated system of intermediate weirs and/or orifices, or active controls (Goodman et al, 2015), is required. As a result, LID BMPs sized for FDC can over-mitigate site runoff and the consequence can be larger BMPs than necessary (VCSQMP, 2013). An example flow duration curve comparison showing such over-mitigation is provided in Figure 2-8.

To avoid the potential over-mitigation of FDC (as illustrated in Figure 2-8) one solution is to use direct simulation of Ep to design onsite distributed BMPs. This is termed Ep control. Using such an approach would maintain a project’s overall contribution of erosive work to its respective receiving channel, but would not attempt to directly match the distribution of flow durations (VCSQMP, 2013). An Ep numeric design approach can account for alterations to channel form (including in-stream measures), losses of bed sediment supply (by reducing the Target Ep below 1.0), and quantify benefits of green infrastructure retrofits (for which there may not be sufficient room to achieve a given HM performance standard). FDC is not able to do so because it is a hydrologic metric which yields pass/fail results instead of a non-polar metric of potential impact.

Summary

The following bulleted lists provide a summary of the strengths and limitations of the Ep control and FDC approaches.

Strengths of Ep control include:

- Ep is the most direct measure of the geomorphic processes associated with hydromodification because it accounts for changes in hydrology on channel form as well as the form itself (channel geometry and bed/bank material).
- Ep control can account for losses of bed sediment supply by using a Target Ep less than 1.0. However, this approach is considered a conservative rule-of-thumb and does not have a strong scientific basis.
- It is consistent with the erosion potential management objective included in the MRP HM Standard.
- It results in more efficient BMP sizing for on-site controls than FDC in a scientifically defensible way.
    - Due to the reduced BMP sizing, drawdown times of stormwater BMPs are reduced as well, thus reducing vector control concerns compared to FDC.
    - Less construction materials, such as gravel and sand, are necessary for BMP installation, thus reducing impact on the environment associated with manufacturing and transport of those materials.
- It provides a methodology for sizing all three types of HM Controls (onsite, regional, and in-stream) or combinations of the three.
- It can quantify benefits of green infrastructure retrofits, even if there is not enough room (e.g., within a right-of-way) for a BMP structure to achieve a given HM performance standard.

Limitations of Ep control include:

- It requires more assumptions about receiving channel geometry and material composition than FDC.

- It is not a method most civil engineers are well educated on or are accustomed to calculating, although standardized sizing tools can be developed on the front end which allow for straightforward BMP sizing.
- It does not perfectly match existing condition flow duration statistics (although neither does FDC).
- It aggregates flows/work without regard to timing, whereas analysis to support biological habitat and organisms, for example, might focus on the seasonality of flow events.

Strengths of FDC include:

- It is the most popular approach for onsite hydromodification control in California and elsewhere in the US (e.g., Western Washington).
- Its application has become standardized and accepted in the civil engineering community with the use of regional hydrology models, such as BAHM.
- It is one way to meet the overarching erosion potential management objective in the MRP HM Standard because it meets the demonstration criteria that post-project stormwater runoff does not exceed estimated pre-project runoff rates and durations.
- It does not require additional watershed or receiving channel analyses that Ep does.

Limitations of FDC include:

- It is difficult to achieve a good match for the entire range of flows evaluated if typical onsite LID BMPs are used. Thus, BMPs are typically oversized than necessary.
- It can only be used for out-of-stream (i.e., Onsite and Regional) HM Controls, and not in-stream measures.
- It is a hydrologic metric that does not account for alterations to channel form (i.e., geometry and bed/bank material) of a receiving stream nor can it account for losses of bed sediment supply.
- It does not directly quantify hydromodification benefit of green infrastructure retrofits, for which there may not be sufficient room to achieve a given HM performance standard.
- It does not typically provide a good match of existing condition flow duration statistics for stormwater BMPs that have a simple outlet configuration.
- It aggregates flows/work without regard to timing, whereas analysis to support biological habitat and organisms, for example, might focus on the seasonality of flow events.

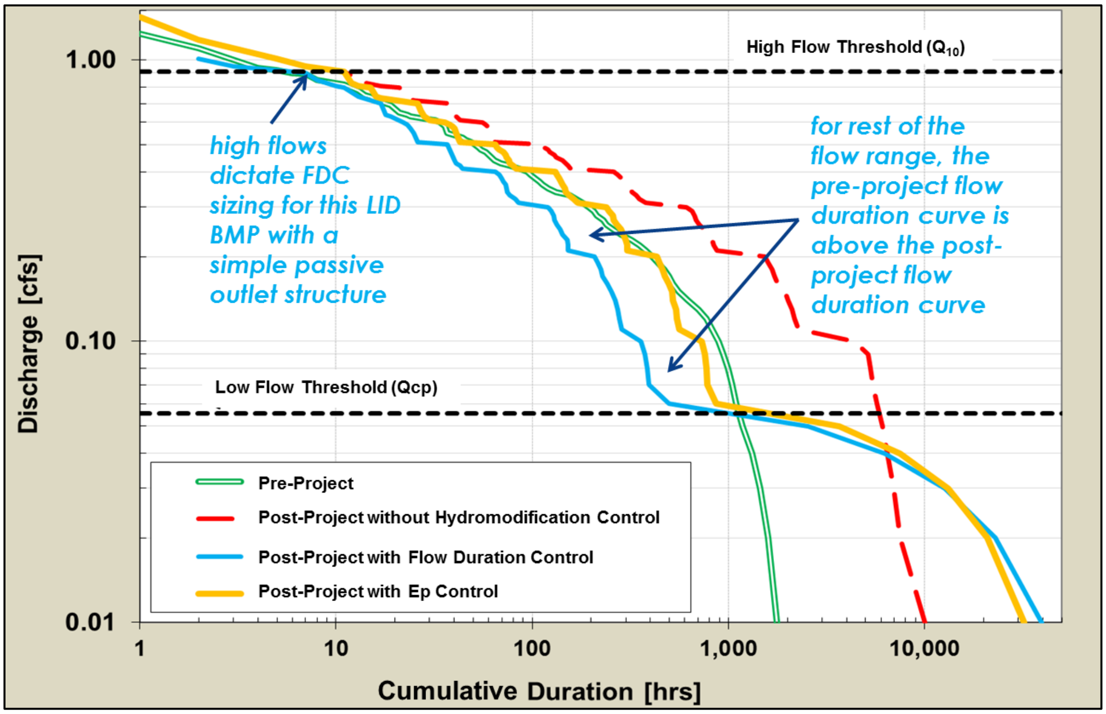


Figure 2-8. Example flow duration curve comparison for flow duration control and erosion potential control (VCSQMP, 2013)

Table 2-3. Comparison of default low flow thresholds in California (CASQA, 2013 and APWA, 2013)

Elements of Hydromodification Management		Santa Clara County	Alameda County	San Mateo County	Contra Costa County	Fairfield-Suisun	Vallejo	Sacramento County	Santa Rosa	San Diego County	South Orange County	North Orange County	Riverside County - Santa Ana Region	Riverside County - Santa Margarita Region	San Bernardino County	Los Angeles County	Ventura County	Central Coast - Phase II	Phase II
Performance Standard	Default Low Flow Threshold for FDC	10%Q2	10%Q2	10%Q2	20%Q2	20%Q2	10%Q2	25% or 45% Q2		10%, 30%, or 50% Q2	10%Q2			10%Q2			10%Q2		

Topic #5. Identify and briefly analyze the assumptions inherent in extrapolating from an Ep management objective developed at the watershed scale to an FDC and Ep Control standard applicable to small catchments.

As stated in the MRP HM Standard (Provision C.3.g.ii), the point-of-compliance for the Ep management objective is within the receiving stream, meaning that hydrologic changes in the receiving stream's tributary watershed are to be accounted for. While flow duration analysis can be performed for a particular receiving stream, such analysis requires continuous hydrologic simulation of the entire watershed tributary to it, which in many cases is not a trivial task. Instead, flow duration control is typically evaluated at the project scale and it is assumed that by achieving FDC at the project catchment level, the site would not contribute to excess erosion in the receiving stream. A conceptual illustration of the project catchment delineated in relation to the receiving stream and its tributary watershed is provided in Figure 2-9.

The underlying assumption for scaling from the project catchment level to the larger watershed scale is the critical low flow threshold. As stated in the SCVURPPP HMP (2005), "in order for the critical flow to be useful to dischargers in design of hydromodification control structures, the critical flow in the stream must be partitioned or related to an on-site project based variable". The necessity for a low flow threshold is also supported in the technical report titled *Hydromodification Assessment and Management in California* (SCCWRP, 2012), commissioned and sponsored by the California State Water Resources Control Board Stormwater Program. The report explains that the purpose of determining a low flow threshold is one of practical design consideration for stormwater BMPs. It states that:

> *If flow matching is required to be achieved for all flows down to zero, the BMP volume will be significantly larger (and therefore more costly) than if there were some low flow below which runoff could be discharged at durations longer than in the pre-project condition. A key assumption underlying the concept of a low-flow discharge is that the increase in discharge durations below this rate will not increase channel erosion because the flows are too small to initiate movement of channel materials to any significant extent.*

As described above, a low flow discharge (e.g., via an orifice) is often necessary to feasibly manage excess post-development runoff volume by discharging it at a rate below the critical discharge for incipient motion in the receiving stream (Qc). When there are several hydromodification control BMPs in one watershed, the sum of all the low flow discharges and low flow runoff contributions from undeveloped areas should be less than Qc. For ease of implementation, Qc is normalized by dividing it by the pre-development 2-year discharge (Q2) so that a project specific low flow threshold (Qcp) can be calculated (VCSQMP, 2013). Thus, the ratio of Qc / Qcp is the scaling assumption inherent in extrapolating from an Ep management objective developed at the watershed scale to an FDC standard applicable to small catchments. Because both Qc and Qcp are expressed as percentages of Q2, the scaling can also be expressed as Q2 watershed / Q2 project catchment.

When performing a direct simulation of Ep at the project catchment scale, for the purposes of Ep control, the project catchment flow rates derived from continuous hydrologic simulation (via HSPF, SWMM, or HMS) are multiplied by the ratio of the pre-development 2-year peak discharge for the watershed and project catchment (i.e., Q2 watershed / Q2 project catchment) so that hydraulic and effective work calculations can be performed for the receiving stream with a larger tributary area. This scaling translates the runoff from the project catchment to erosivity in its downgradient receiving stream (BASMAA, 2015). Scaling by this ratio also ensures that Qc and Qcp are the same percentage of Q2, similar to FDC.

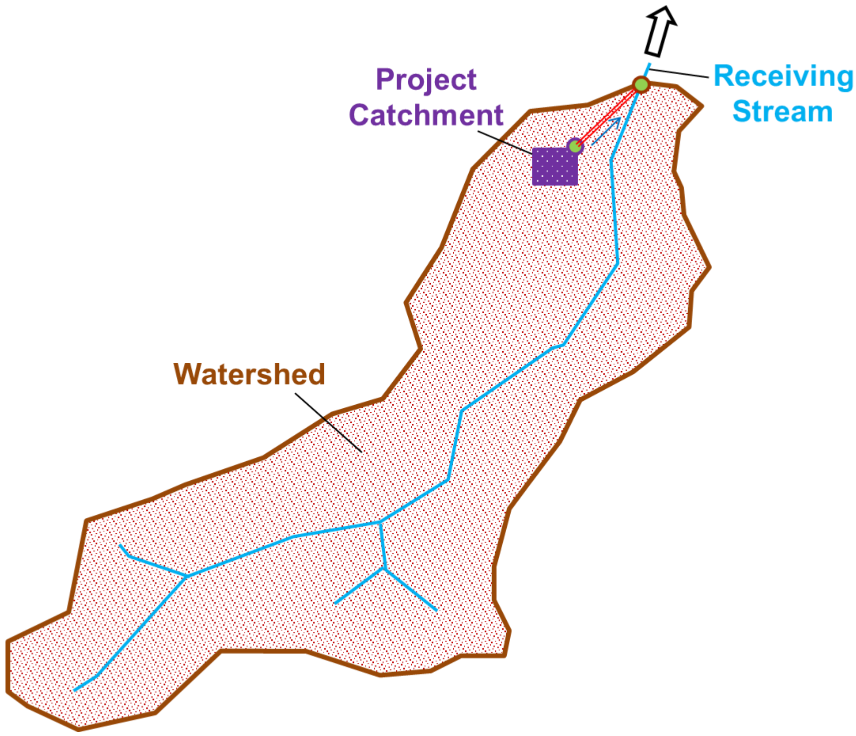


Figure 2-9. Illustration of project catchment in relation to the receiving stream and watershed (BASMAA, 2015)

Topic #6. Describe the basis for the Ep control standard, particularly with regard to how it is similar to, and different from, flow duration control.

The basis for the Ep control standard relies on regulatory, technical, and practical design considerations, as described below.

Regulatory Basis

Using the Ep metric for sizing on-site LID-type hydromodification controls is directly consistent with the wording of the HM Standard written in MRP provision C.3.g.ii (SFBRWQCB, 2015), particularly the first two sentences which identifies "erosion potential" as a management objective. The HM Standard is as follows:

> *Stormwater discharges from HM Projects shall not cause an increase in the erosion potential of the receiving stream over the pre-project (existing) condition. Increases in runoff flow and volume shall be managed so that post-project runoff shall not exceed estimated pre-project rates and durations, where such increased flow and/or volume is likely to cause increased potential for erosion of creek beds and banks, silt pollutant generation, or other adverse impacts on beneficial uses due to increased erosive force.*

While the remainder of the HM Standard goes on to describe the demonstration requirements for flow duration control (i.e., range of flows to control, goodness of fit criteria, and standard HM modeling), FDC is but one acceptable performance criteria for achieving the overarching Erosion Potential management objective, as documented in the SCVURPPP HMP (2005) and described previously in this literature review. With the addition of a provision in the current MRP for direct simulation of Erosion Potential (C.3.g.iii), there is now a regulatory pathway for this method.

Additionally, direct simulation of Ep provides a numerical approach for designing in-stream measures, per provision C.3.g.iv, where there was no method explicitly stated in the previous MRP (SFRWCQB, 2009). FDC can be used to demonstrate HM Standard compliance for onsite and regional (out-of-stream) controls, but Ep control can be used to demonstrate compliance for all three types of HM control types, or a combination thereof.

Technical Basis

The primary scientific basis for the direct simulation of Ep is demonstrated by a strong correlation between computed Ep and observed stream stability, or instability, in the field. Results from a study of 61 southern California streams (Hawley and Bledsoe, 2013) indicate that channel enlargement is highly dependent on the Ep, which explained nearly 60% of the variance (SCCWRP, 2013). No other single variable evaluated had as high of a correlation. The logistic regression curves for Santa Clara County (SCVURPPP, 2005) and Southern California (SCCWRP, 2013) (see Figure 4), as described previously in this literature review, are the best available tools to date for predicting such geomorphic impacts associated with hydromodification. While FDC is one means of reducing a receiving channel's Ep to within acceptable levels (i.e., 1.0 to 1.2 in the San Francisco Bay Area), using Ep control directly is another technically valid approach.

Also, direct simulation of Ep can account for alterations to channel form (including in-stream measures), losses of bed sediment supply (by reducing the Target Ep below 1.0), and quantify benefits of green infrastructure retrofits (for which there may not be sufficient room to achieve a given HM performance standard). FDC is not able to do so because it is a hydrologic metric which yields pass/fail results instead of a non-polar metric of potential impact.

Additional technical basis for the direct simulation of Ep is provided in Appendix B, Section 3 of the State Water Board's report on hydromodification (SCCWRP, 2012), which states the following.

> *The underlying premise of the erosion potential approach advances the concept of flow duration control (discussed in Chapters 2 and 3) by addressing in-stream processes related to sediment transport. An erosion potential calculation combines flow parameters with stream geometry to assess long term (decadal) changes in the sediment transport capacity. The cumulative distribution of shear stress, specific stream power and sediment transport capacity across the entire range of relevant flows can be calculated and expressed using an erosion potential metric, Ep (e.g., Bledsoe, 2002). This erosion potential metric is a simple ratio of post- vs. pre-development sediment transport capacity over a period of many years. The calculated capacity to transport sediment can be based on the channel bed material or the bank material, depending on which one is more erodible.*

Design Practicality Basis

As discussed previously, while theoretically FDC maintains the pre-project sediment transport capacity for the full distribution of erosive flows, in practice it is difficult to achieve a good match for the entire range of flows evaluated if typical onsite LID BMPs are used (VCSQMP, 2013). This is because the outlet structure configuration of a typical LID BMP consists of a simple overflow weir and a low flow orifice (if needed). However, to get a good match of the flow duration curve with passive controls such as LID BMPs, a more complicated system of intermediate weirs and/or orifices, or active controls (Goodman et al, 2015), is required. As a result, LID BMPs sized for FDC can over-mitigate site runoff and the consequence can be larger BMPs than necessary (VCSQMP, 2013), with calculated Ep much less than 1.0. To avoid the potential over-mitigation of FDC, one solution is to directly simulate Ep to design onsite distributed BMPs. Using such an approach maintains a project's overall contribution of erosive work to its respective receiving channel, but would not attempt to directly match the distribution of flow durations (VCSQMP, 2013).

Direct simulation of Ep thus results in reduced BMP sizes for on-site LID controls than FDC in a scientifically defensible way (see Section 5). As a result of this reduction in sizing, other practical benefits include: (1) drawdown times of stormwater BMPs are reduced as well, thus reducing vector control concerns compared to FDC; (2) less construction materials, such as gravel and sand, are necessary for BMP installation, thus reducing impact on the environment associated with manufacturing and transport of those materials; and (3) project proponents can more readily situate LID HM controls at their sites.

Topic #7. Identify and describe any instances where direct simulation of Ep has been used in the design of HM facilities. Note similarities and differences with those uses of Ep compared to the LID drainage design procedures practiced in Contra Costa County.

Direct simulation of Ep has been used in the design of HM controls for large-scale master planned developments in California. These developments include the Newhall Ranch, Centennial, and Northlake projects in Los Angeles County, all of which have performed watershed-scale Ep analysis to demonstrate conformance with a project-specific HM performance standard. None of these HM facilities have been constructed to date. The hydromodification management approaches for the Centennial and Northlake projects primarily rely on regional HM controls, with some onsite HM controls where regional detention/retention basins could not feasibly be situated. The hydromodification management approach for the Newhall Ranch project is unique because its development areas are proposed to drain to onsite LID-type controls, sized for surface water quality treatment, which would then drain to receiving channels rehabilitated with in-stream measures, primarily in the form of grade controls. The Yokohl Ranch development project in Tulare County is another proposed large-scale master planned development, which has used direct simulation of Ep. The purpose of this Ep analysis was to develop planning-level sizing nomographs for onsite and regional HM controls.

Historically, direct simulation of Ep at the watershed-scale has been most feasible for large master planned developments because: (1) these projects make up a significant proportion of their watershed's buildout development; (2) the land being developed is typically owned by one entity, thus eliminating the need for coordination of several project proponents; and (3) the size of these projects and regulatory climate in California (i.e., CEQA) warrants the cost for such comprehensive land planning. Contra Costa County's LID drainage design procedures differ from the design examples for large master planned communities because the remaining urban development in the County are anticipated to be more piece-meal with smaller projects. The CCCWP and the Contra Costa MS4 co-permittees take the position that a simple and straight-forward sizing factor approach that promotes LID treatment at the source is most effective for such development situations.

Although not considered to be design for a specific project, there are other precedents for direct simulation of Ep. The Ventura County Watershed Protection District (VCWPD) commissioned a study for an example hydromodification control nomograph for a bioretention facility (Geosyntec, 2014). The intent of this effort was to compare the BMP storage requirements for hydromodification control using direct simulation of Ep, with those for FDC (derived from the California Hydrology Model (CAHM), equivalent to BAHM) and surface water quality control. BASMAA commissioned a modeling analysis to evaluate the suitability of alternative hydromodification control standards, including Ep control, compared to the flow duration curve matching criteria included in the MRP (Geosyntec, 2015). As a result of this work, the SFRWQCB added to the hydromodification management standard in the updated MRP (Provision C3.g.iii of Order Number R2-2015-0049), which allows for direct simulation of Ep. As indicated in Table 2-2 (see Topic #2 above), there are several HMPs in California that allow for the direct simulation of Ep as an appropriate means for demonstrating compliance with a respective HM performance standard. It is worth noting that Ep analysis has also served as a basis for hydromodification management exemptions, specifically for receiving channels which have a negligible risk for hydromodification impact based on limited future buildout relative to tributary watershed size (San Diego County, 2017, Orange County, 2017, and VCSQMP, 2013).

Topic #8. Describe generally the hydrologic objectives and effects of LID, including the maintenance of pre-development hydrology and water balance.

US EPA (EPA 2017) defines LID as follows:

> *The term low impact development (LID) refers to systems and practices that use or mimic natural processes that result in the infiltration, evapotranspiration or use of stormwater in order to protect water quality and associated aquatic habitat. EPA currently uses the term green infrastructure to refer to the management of wet weather flows using these processes, and to refer to the patchwork of natural areas that provide habitat, flood protection, cleaner air and cleaner water. At both the site and regional scale, LID/green infrastructure practices aim to preserve, restore and create green space using soils, vegetation, and rainwater harvest techniques. LID is an approach to land development (or re-development) that works with nature to manage stormwater as close to its source as possible. LID employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness to create functional and appealing site drainage that treat stormwater as a resource rather than a waste product.*

The California Ocean Protection Council (2008) defines LID as:

> *…a stormwater management strategy aimed at maintaining or restoring the natural hydrologic functions of a site to achieve natural resource protection objectives and fulfill environmental regulatory requirements… LID design detains, treats, and infiltrates runoff by minimizing impervious area, using pervious pavement and green roofs, dispersing runoff to landscaped areas, and routing runoff to rain gardens, cisterns swales, and other small-scale facilities throughout a site…*

The term "Low Impact Development" originated in Prince George's County Maryland in the late 1990s. *Low Impact Development Hydrologic Analysis* (1999) details an approach using Natural Resources Conservation Service (NRCS) hydrologic methods (curve numbers) to determine site design and storage requirements needed to maintain predevelopment peak flows.

Others have used a water-balance approach, or maintenance of watershed processes, as frameworks for setting criteria for LID hydrologic performance on land development sites (California Regional Water Quality Control Board for the Central Coast Region, 2013).

CCCWP's HMP (2005) was created and adopted with the purpose of requiring land developers in Contra Costa County to use LID, rather than non-LID methods, to meet both the Water Board's stormwater treatment requirements (Provisions C.3.c and C.3.d. in the current MRP) and flow duration control requirements (Provision C.3.g. in the current MRP). The HMP was initially implemented via CCCWP's *Stormwater C.3 Guidebook* (3rd Edition, 2006), and continues to be implemented via the updated *Guidebook* (7th Edition, 2017). Ordinances adopted by each of the 20 Contra Costa municipalities reference the most current edition of the *Guidebook.*

Topic #9. Describe the hydrologic performance of bioretention with no underdrain outlet with regard to small and large runoff events.

Bioretention designs have evolved over the past 15 to 20 years to address specific flow control and water quality objectives. In the simplest designs, stormwater runoff is captured in an initial ponding area, percolated through bioretention soils and into gravel or drain rock layer. Most of the pollutants are removed in the bioretention soils through filtration, adsorption and microbial processes. In fast draining native soils, water migrates from the gravel layer into the surrounding soils where it can recharge the groundwater. In slower draining soils, bioretention facilities typically include an underdrain that discharges water and prevents the bioretention soils from remaining saturated for an extended period.

CCCWP's 2005 HMP and the *Stormwater C.3 Guidebook*, beginning with the 3rd Edition (2006), specify a flow control orifice be included on the underdrain when bioretention is used for HM. This was an important innovation that enabled bioretention and other LID facilities to meet flow duration control criteria. Other communities have used bioretention facilities without a flow control orifice to meet peak flow control or flow reduction requirements.

For example, Marin County is covered by the statewide NPDES Phase II permit and its hydromodification requirement that the 2-year peak flow for post-developed conditions must not exceed the pre-project 2-year peak flows. A continuous simulation modeling analysis demonstrated that bioretention systems sized at 4 percent of tributary impervious area, with a 6-inch surface reservoir, 18 inches of soil, 12 inches of gravel and an underdrain located at the top of the gravel layer could meet the peak flow performance standard (Dubin, 2014).

Developments in Santa Barbara County in the Central Coast Region are subject to post-construction requirements that under certain conditions (based on impervious thresholds and project location) must store and infiltrate the 85th or 95th percentile 24-hour storm. A combination of event-based and continuous simulation modeling conducted for Santa Barbara County demonstrated that enlarging the plan area of the facility (to 6 percent or more of tributary area) and/or deepening the gravel layer (in some cases to several feet deep) was necessary to fully capture the storm of interest (Dubin, 2017). The same study demonstrated that installing a flow control orifice that allows for storage in the bioretention soil and surface reservoir layers could reduce the gravel storage requirement by 15 to 30 percent (Figure 2-10).

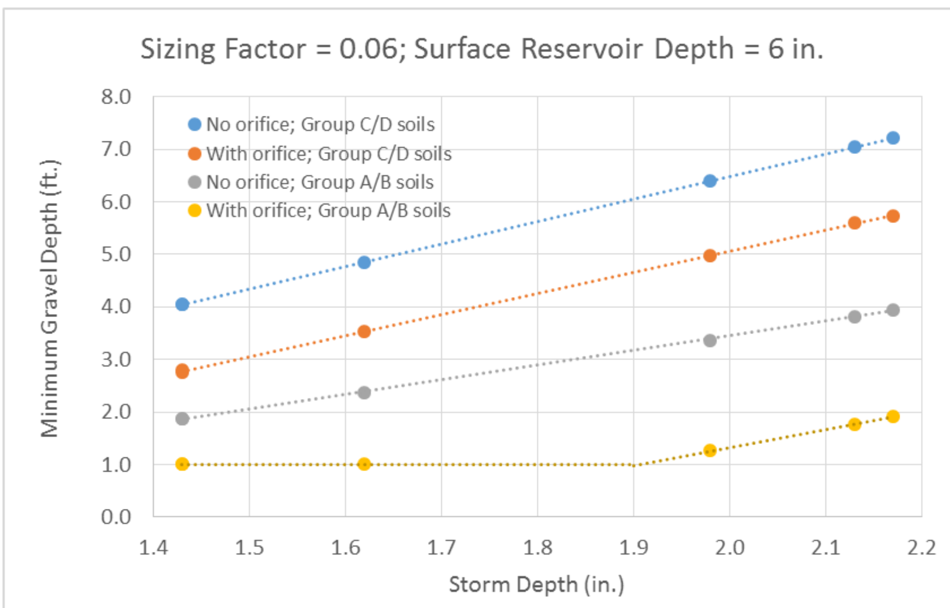


Figure 2-10. Central Coast Region bioretention storage volume requirements.

These studies illustrated the following hydraulic characteristics of bioretention facilities that contain an underdrain system and no flow control orifice:

- This configuration is effective for water quality treatment because stormwater runoff is filtered before discharge
- Stormwater discharges generally exceed pre-project flows for moderate storm events (say 0.2 to 0.5 inches)

- Raising the underdrain pipe and/or deepening the gravel layer can help reduce the discharge volume and increase the volume of water retained onsite
- For very large storms (e.g., 5-year recurrence or greater), bioretention facilities with and without a flow control orifice will produce generally similar flows; this occurs when the incoming stormwater exceeds the bioretention media's percolation rate and is discharged via an overflow relief pipe.

Topic #10. Summarize CCCWP's adaptation of bioretention to meet the Water Board's FDC criteria.

Bioretention facilities are the most commonly used IMPs on Contra Costa development projects. Bioretention facilities work as follows:

- Runoff enters the bioretention facility via sheet flow or pipes and is detained in a shallow surface reservoir. The reservoir also serves to spread runoff evenly across the facility surface. Runoff then percolates through an engineered soil (sand/compost mix). Some runoff is retained in soil pores and plant roots and is subsequently evaporated or transpired (Figure 2-11).
- Runoff that exceeds the moisture-holding capacity of the soil percolates through the soil layer and enters a subsurface storage layer (typically gravel).
- The treated runoff subsequently then infiltrates into the soils below the facility.
- If runoff enters the gravel layer more rapidly than it infiltrates, the saturation level in the gravel layer rises until it reaches the discharge elevation for a perforated pipe underdrain. When this occurs, runoff will also discharge through the perforated pipe underdrain to a discharge point (typically connected to the municipal storm drain system).

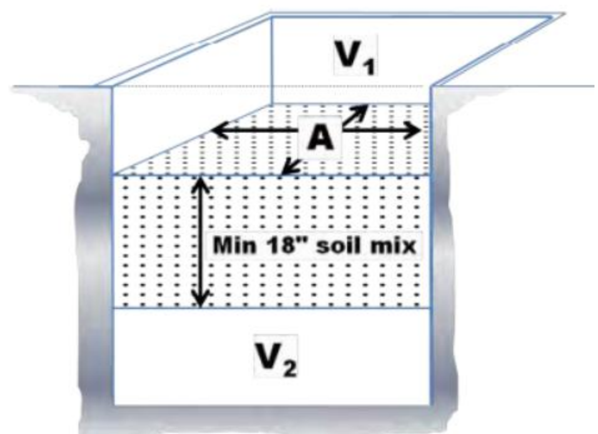


Figure 2-11. Section view of bioretention facility (source: Stormwater C.3 Guidebook, 6th Edition)

- In facilities constructed for HM, this perforated pipe underdrain is equipped with a flow-limiting orifice. This allows the bioretention facility to act like a flow duration control basin during the infrequent occasions when the storage layer fills, and as a LID facility at other times.
- The surface reservoir is also equipped with an overflow that will become active under either of two scenarios: (1) runoff enters the surface reservoir more rapidly than it percolates through the engineered sand/compost mix, and the surface reservoir fills to its maximum volume or (2) runoff enters the facility more rapidly than it leaves via *both* infiltration to the soils below the facility *and* discharge via the underdrain, and this continues until the gravel and soil layers become fully saturated, and the surface reservoir fills to its maximum volume.

Bioretention facilities have been sized from 2006 to present using the sizing factors developed during Contra Costa's 2005 HMP and included in each edition of the Stormwater C.3 Guidebook. These facilities comply with the following flow duration and peak flow standard:

- Post-project flow durations cannot exceed pre-project levels for all flow between 20 percent of the 2-year flow rate (0.2Q2) and the 10-year flow (Q10)

- Post-project peak flow discharges cannot exceed pre-project levels for all flows between 0.2Q2 and Q10

Elsewhere in the region, Phase I communities must meet a flow duration standard, but the peak flow standard is only used in Contra Costa County. Modeling experience has shown that in most instances the flow duration requirement determines the size of a bioretention facility but in locations with small facilities and/or frequent intense rainfall, the peak flow requirement can be more challenging to meet and determines the facility size. Figure 2-12 and Figure 2-13 show the performance of Contra Costa's demonstration projects at the Pittsburg Fire Prevention Bureau site meeting the peak flow and flow duration standards.

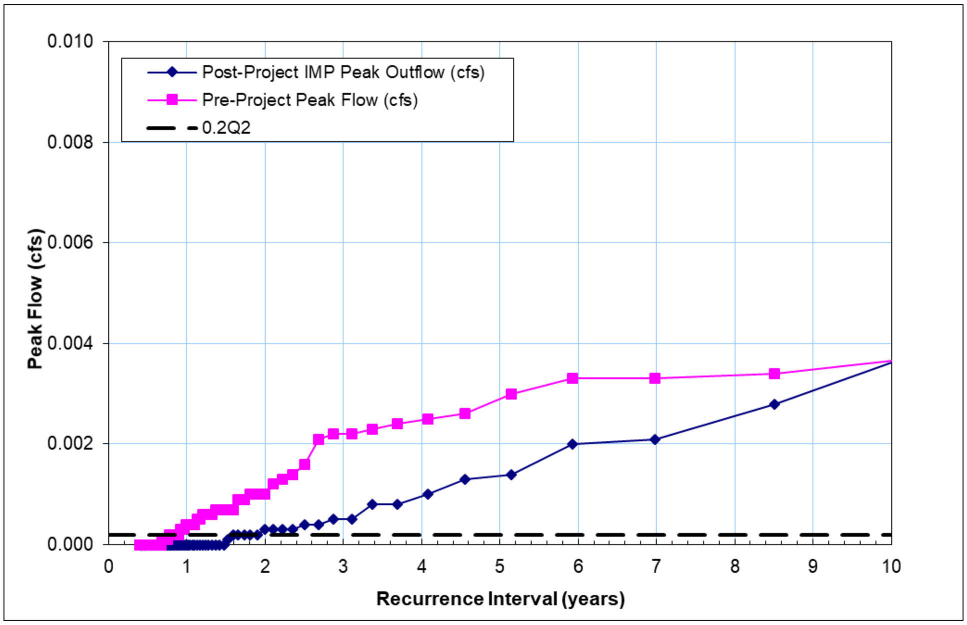


Figure 2-12. Peak flow control example for bioretention facility at Pittsburg Fire Prevention Bureau

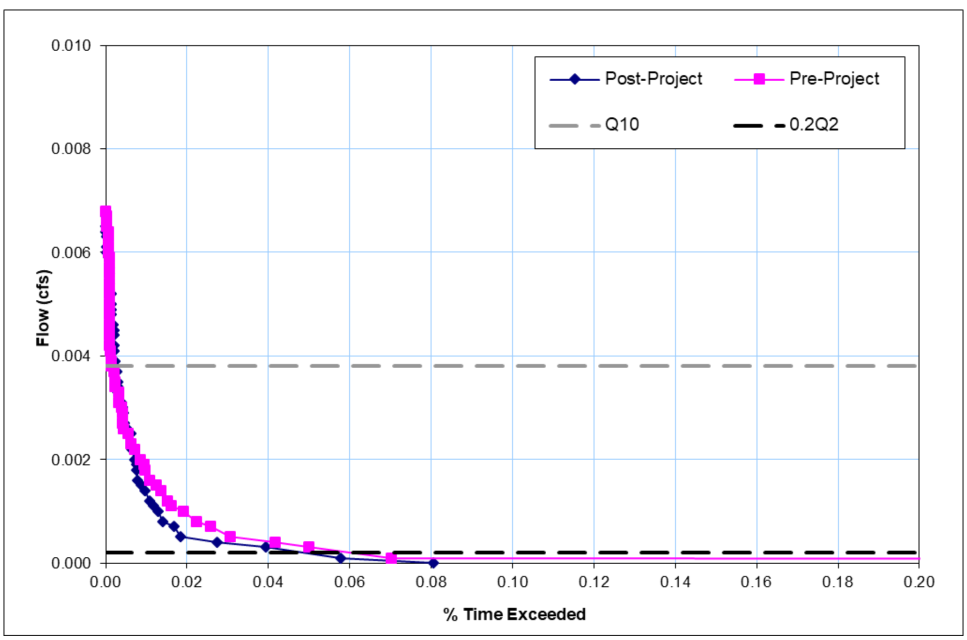


Figure 2-13. Flow duration control example for bioretention facility at Pittsburg Fire Prevention Bureau

In summary, a bioretention facility receives runoff from a specific delineated area, retains that runoff via infiltration and evapotranspiration, and discharges excess runoff via an underdrain and an overflow. The bioretention facilities installed for hydromodification meet a flow duration standard and, in Contra Costa, a peak flow control standard as well.

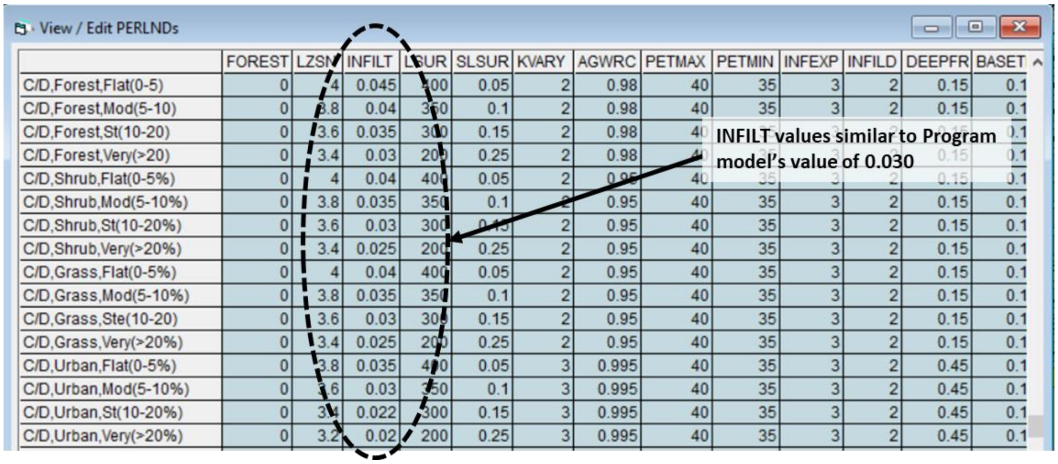
Topic #11. Summarize how LID features and facilities are modeled in the Bay Area Hydrology Model (BAHM). Compare and contrast the approach used in the Stormwater C.3 Guidebook and IMP Sizing Calculator.

This section briefly compares BAHM and the Contra Costa IMP Sizing Calculator approaches to sizing bioretention. The features, usage, LID options and relative facilities sizes are all discussed. Both tools have been successfully used to size many LID stormwater controls.

BAHM is a continuous simulation rainfall model that is based on HSPF and contains its own graphical user interface that streamlines model development and analysis of stormwater control measures. BAHM allows users to characterize catchment areas (e.g., pervious land surface types, soils), route flows, and define points of concentration for flow calculations. Each of the land surface/soil combinations has a pre-defined set of HSPF parameters. The user selects a site location and BAHM applies rainfall data from a nearby gauge and scales the rainfall data, as necessary, to account for differences between the rain gauge and project location. BAHM contains a variety of traditional stormwater facilities, such as detention ponds and storage pipes, and LID measures, including bioretention, green roofs and permeable pavements. For bioretention facilities, BAHM includes options for modifying the facility components (e.g., surface reservoir depth, bioretention soil depth) and specifying the design of the underdrain structure (height, orifice) and overflow relief structure. BAHM allows users to iteratively size stormwater facilities to meet local performance requirements.

Contra Costa has been committed to supporting LID implementation since the publication of the first edition of the Stormwater C.3 Guidebook in 2005. The IMP Sizing Calculator and C.3 Guidebook focus on simplifying the process of sizing LID facilities, including bioretention, flow-through planters, dry wells, and bioretention variations that include a downstream vault or upstream cistern. The C.3 Guidebook incorporates LID principles in site development and provides a method to size LID facilities. The IMP Sizing Calculator automates the process. The Calculator contains pre-calculated LID sizing factors (ratio of facility size to upstream impervious area) that were developed using HSPF modeling of pre-project pervious areas and impervious areas that drain to an LID facility.

Both approaches were reviewed by an independent third party in 2007 (Tetra Tech, 2007) and found to be appropriate tools for sizing stormwater measures to meet hydromodification requirements. The BAHM model has been updated since the 2007 review to incorporate the direct simulation of LID facilities and the use of LID to meet flow duration control requirements. The BAHM and Contra Costa models have generally similar approaches to a) estimating pre-project flows and b) modeling bioretention hydraulics. For example, Figure 2-14 shows that the INFILT parameter for Group C/D soils, which has the largest influence on pre-project runoff rates, is similar for the Program’s model and BAHM. Regarding bioretention hydraulics, both approaches incorporate realistic bioretention media hydraulics, gravel layers, and underdrain flow control orifices. Table 2-4 compares a broader set of model characteristics.



	FOREST	LZSN	INFILT	LSUR	SLSUR	KVARY	AGWRC	PETMAX	PETMIN	INFEXP	INFILD	DEEPFR	BASET
C/D,Forest,Flat(0-5)	0	4	0.045	400	0.05	2	0.98	40	35	3	2	0.15	0.1
C/D,Forest,Mod(5-10)	0	3.8	0.04	350	0.1	2	0.98	40	35	3	2	0.15	0.1
C/D,Forest,St(10-20)	0	3.6	0.035	300	0.15	2	0.98	40	35	3	2	0.15	0.1
C/D,Forest,Very(>20)	0	3.4	0.03	200	0.25	2	0.98	40	35	3	2	0.15	0.1
C/D,Shrub,Flat(0-5%)	0	4	0.04	400	0.05	2	0.95	40	35	3	2	0.15	0.1
C/D,Shrub,Mod(5-10%)	0	3.8	0.035	350	0.1	2	0.95	40	35	3	2	0.15	0.1
C/D,Shrub,St(10-20%)	0	3.6	0.03	300	0.15	2	0.95	40	35	3	2	0.15	0.1
C/D,Shrub,Very(>20%)	0	3.4	0.025	200	0.25	2	0.95	40	35	3	2	0.15	0.1
C/D,Grass,Flat(0-5%)	0	4	0.04	400	0.05	2	0.95	40	35	3	2	0.15	0.1
C/D,Grass,Mod(5-10%)	0	3.8	0.035	350	0.1	2	0.95	40	35	3	2	0.15	0.1
C/D,Grass,Ste(10-20)	0	3.6	0.03	300	0.15	2	0.95	40	35	3	2	0.15	0.1
C/D,Grass,Very(>20%)	0	3.4	0.025	200	0.25	2	0.95	40	35	3	2	0.15	0.1
C/D,Urban,Flat(0-5%)	0	3.8	0.035	400	0.05	3	0.995	40	35	3	2	0.45	0.1
C/D,Urban,Mod(5-10%)	0	3.6	0.03	350	0.1	3	0.995	40	35	3	2	0.45	0.1
C/D,Urban,St(10-20%)	0	3.4	0.022	300	0.15	3	0.995	40	35	3	2	0.45	0.1
C/D,Urban,Very(>20%)	0	3.2	0.02	200	0.25	3	0.995	40	35	3	2	0.45	0.1

Figure 2-14. BAHM and Contra Costa models use similar INFILT values for Group C/D soil runoff

Table 2-4. Comparison of Contra Costa IMP sizing and BAHM characteristics

Model Element	Contra Costa	BAHM
Pre-project conditions	• Limits inputs for simplicity and eliminates the ability of a user to adjust parameters for the purpose of increasing pre-project flows with the goal of reducing LID sizing • Emphasizes use of LID principles in site layout	• Enables wide range of pre-project surface and soil types • Pervious parameters vary by soil/land use • Includes catchment routing for large developments
Rainfall variability	• Uses a rainfall adjustment factor to scale IMP sizing	• Scales rain gauge data (3 sites in Alameda)
Bioretention configuration	• Dimensions of each soil and gravel layers, underdrain configuration, etc., conform to Contra Costa design standards	• Contains flexible sizing options for bioretention configuration and other LID facility types • Users and plan reviewers need to back-check to confirm model jibes with facilities as designed and constructed
Bioretention hydraulics	• 2005 HMP used Van Genuchten relations for unsaturated hydraulics • Updated in 2013 to reflect monitoring data and calibration • Detailed bioretention hydraulics are represented using two FTABLEs in HSPF	• Use Van Genuchten relations for unsaturated hydraulics • Detailed bioretention hydraulics are represented using FTABLEs in HSPF; software contains options for simplified or more complex bioretention hydraulics

To test the relative facility sizes generated by these tools, the Program model (described in Section 3) and BAHM were used to estimate bioretention size for a 1-acre paved development with an urban, moderate slope C/D soil pre-project condition. The test used a C/D soil percolation rate of 0.12 in/hr and a lower control threshold of 0.1Q2, and 22 inches per year of rainfall (for BAHM, this was the Berkeley gauge). Both models were setup to match Contra Costa's design standard: 12-inch surface reservoir (overflow 2 inches from the top), 18-inch bioretention soil layer, 30-inch gravel layer and underdrain with flow control orifice with a capacity equaling the estimated 0.1Q2 flow. Both tools showed that a sizing factor of about 0.05

would be sufficient for these conditions. The underlying modeling approaches have some similarities and differences but the facility sizes generated should be relatively similar.

In summary, both tools are appropriate for LID sizing for hydromodification projects. The Contra Costa IMP Sizing Calculator and C.3 Guidebook are closely tailored to the Program's approach to encouraging LID implementation for development projects. BAHM is more flexible and can be used to model a greater variety of development types with both traditional and LID stormwater controls. One result of this flexibility is that BAHM requires an additional level of user training, quality assurance, and municipal review to verify that, for each project, the user-selected inputs characterize pre-project conditions accurately and that the LID sizing and configuration corresponds to the project site design and facilities as designed and constructed.

Topic #12. Summarize the calibration of the Program model as detailed in the 2013 IMP Monitoring Report.

As part of its 2013 Annual Report, the Clean Water Program prepared the IMP Monitoring Report, which summarized a) the monitoring of five representative IMP installations, b) the comparison of monitoring data with the Program's HMP model outputs, and c) the calibration of the model parameters to match the monitoring results. This section summarizes the following program components:

1. Monitoring sites
2. Monitoring period
3. Monitoring data collection
4. Model calibration and results

For additional detail, see Appendix A, which contains the entire IMP Monitoring Report.

Monitoring sites. Level and flow monitoring equipment was installed for five IMP installations, including three bioretention facilities at the Pittsburg Fire Prevention building site and two bioretention + vault facilities at the Walden Park Commons development in Walnut Creek. Each of the IMPs was sized, designed and installed consistent with the C.3 Guidebook.

The Pittsburg site is located in the eastern portion of Contra Costa County, which is dryer than most parts of the County. The Walden Park Commons site was consistent with average precipitation patterns within the County. Both sites contained NRCS Hydrologic Soil Group D soils. For example, the soils report at the Pittsburg site (Kleinfelder, 2004) described the soils as "consisted predominantly of stiff to hard, moderately to highly plastic silty clays, extending to depths ranging from about 4 to 14 feet below existing site grade." At the Walden Park Commons site, the geotechnical report (Korbmacher, 2006) described the soils as "medium stiff to very stiff silty clay and sandy clay."

Monitoring period. Rain gauges and bioretention monitoring equipment were installed at the two monitoring sites for the 2011-12 and 2012-13 wet seasons. The rainfall data was compared to the long-term records available from the Contra Costa Flood Control District gauges to assess whether the captured storms were representative and sufficient for model calibration. The total rainfall recorded at the project sites was lower than the long-term average; however, the frequency of significant storm events was in line with long-term averages. For example, the two year period contained one 2-year storm, one 1-year storm and six additional storms with a recurrence of about 3 months (note: storm statistics were computed for 12-hour accumulations and compared to long-term statistics at the nearest Flood Control District rain gauge). The number of storms with 3-month or greater recurrence is important because these types of storms produce flows around the lower control threshold in the MRP. Therefore, the storm events captured during the monitoring period were sufficient to characterize IMP performance.

Monitoring data collection. At the Fire Prevention Bureau site, a slotted-standpipe monitoring well was installed within the gravel storage layer of each monitored IMP. At the Walden Park Commons site, water levels were monitored in the vaults at the downstream end of the storage pipes. Tipping bucket-style flow monitors were included at the outlet of each IMP; this technology was selected because it is particularly suitable to recording low flow rates. The sites were equipped with data loggers that recorded observations in 5-minute increments.

Model calibration and results. The monitoring observations were used to develop a calibration strategy that focused on the two main observed differences: a) the characteristics of soil percolation through the bioretention media and b) the rate of infiltration from the bioretention gravel layer to the surrounding, native soils.

The Pittsburg monitoring data showed that percolation begins after relatively modest levels of rainfall (Figure 2-15). This differed from the Program's HMP model, which simulated water movement through the bioretention media using the van Genuchten relationship for water retention. This relationship dictates that percolation rates in sandy-loamy soils would be minimal until the soil reached about three-quarters saturation. The recessional limb of the monitoring hydrographs showed that infiltration from the gravel layer to the surrounding soils also occurred much more quickly than reference infiltration values for Group D soils. For the example below, water levels dropped in excess of 1 inch per hour, which represents a percolation rate of about 0.4 inches per hour (after accounting for the soil porosity).

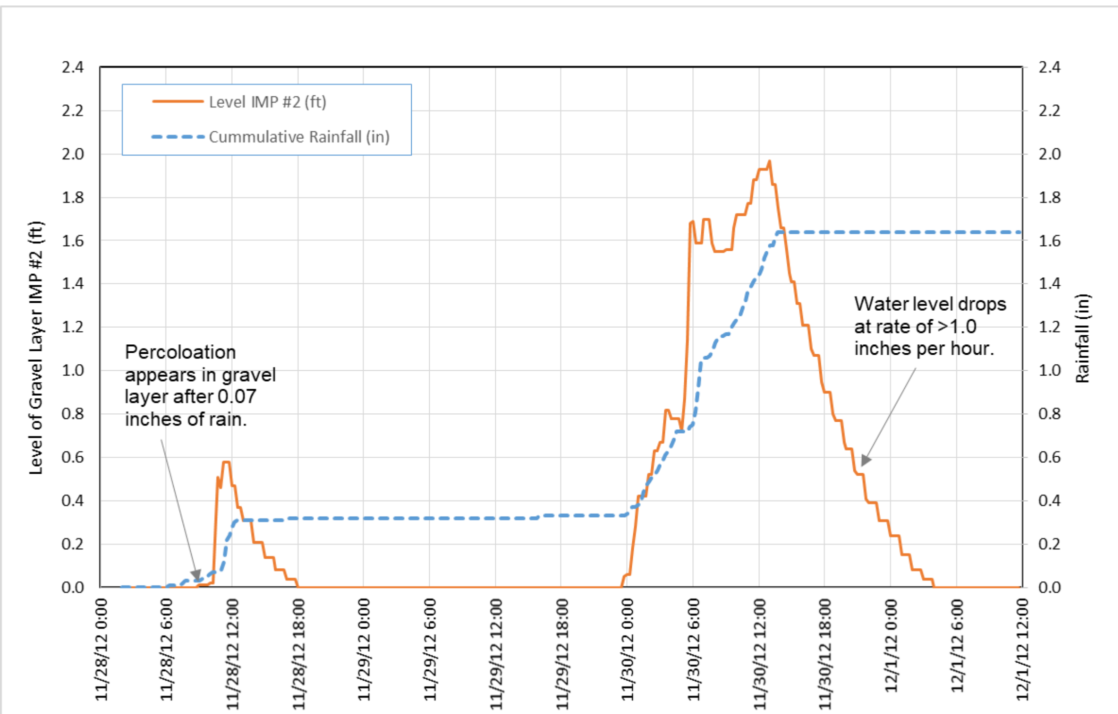


Figure 2-15. Observed water levels in Pittsburg site

The bioretention characteristics of the Program's HSPF model were calibrated to the monitoring data. Specifically, the bioretention parameters were adjusted to a) represent the capacity of the bioretention soils to hold water prior to start of percolation, b) mimic the rapid percolation that occurs once the soil moisture threshold is met, and c) approximate the rate at which water drops in the gravel layer by adjusting the infiltration rate to surrounding soils. Figure 2-16 shows the calibrated bioretention media response to soil moisture. The best estimate from the monitoring data is that percolation begins when water is about half saturated and then reaches full percolation rate when the media is between 60 and 65 percent saturated.

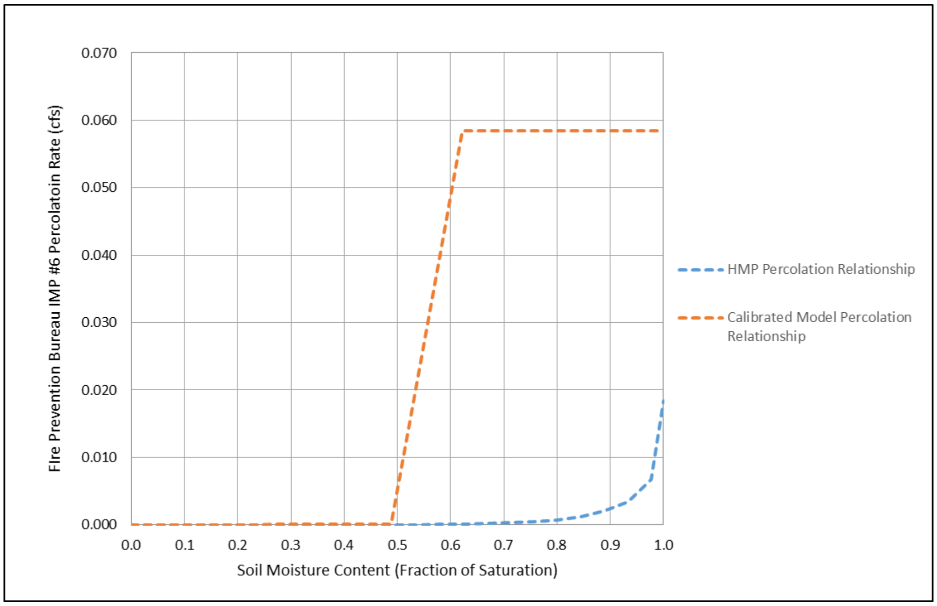


Figure 2-16. Calibrated bioretention media soil percolation

The model's percolation rate from the gravel layer into the surrounding, native Group D soils was calibrated by varying the percolation rate in the HSPF model's gravel layer FTABLE to match the falling limb of the water level hydrograph across several storms. Figure 2-17 shows two examples. The best fit was achieved with a percolation rate of 0.24 in/hr.

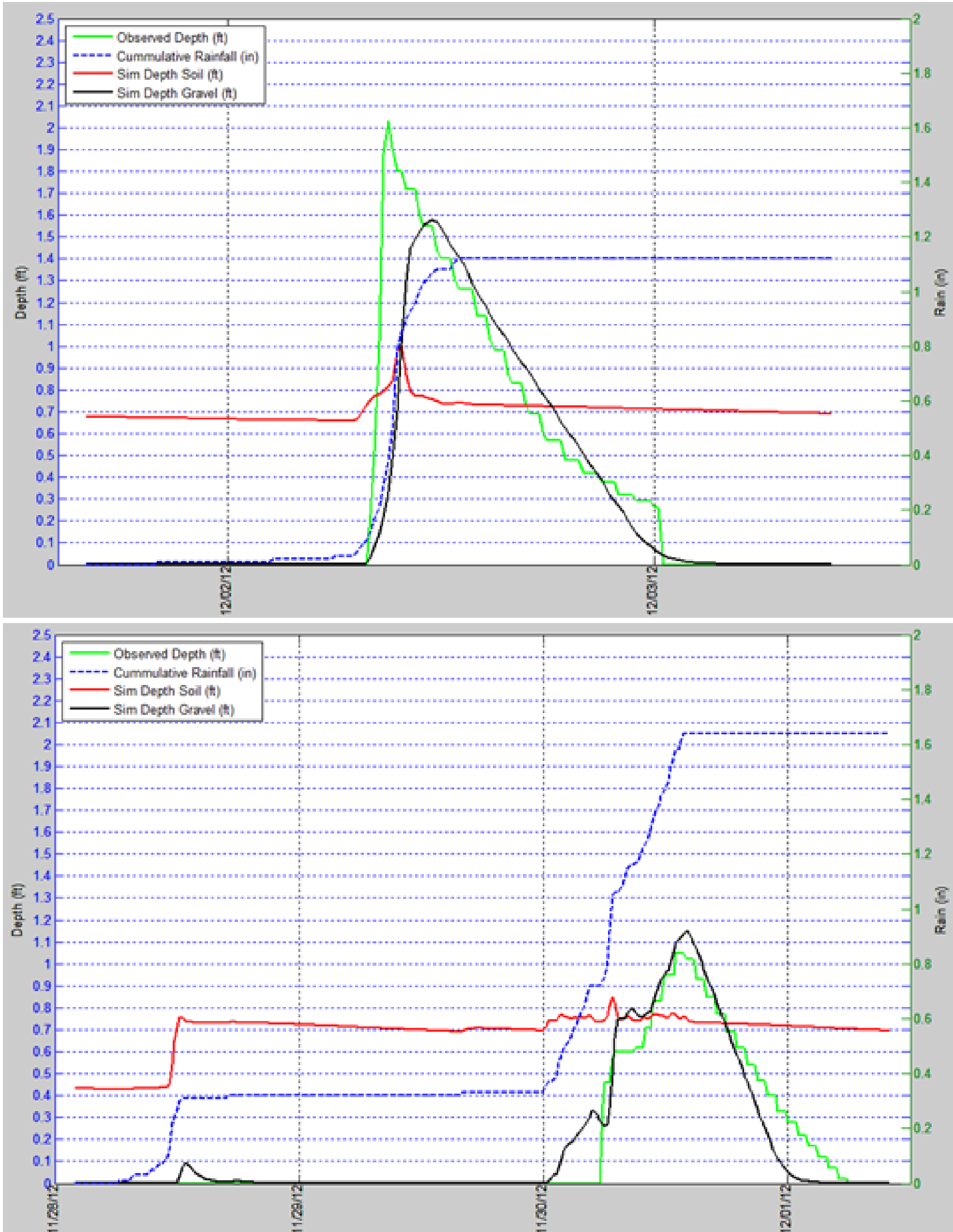


Figure 2-17. Model calibration to estimate percolation to native Group D soils (0.24 in/hr)

After the model calibration was complete, long-term HSPF simulations were run for the IMPs at both project sites to more fully test the IMP performance against the NPDES permit's flow control standard. The Fire Prevention Bureau simulations used hourly rainfall data collected at the Los Medanos gauge from 1972 through May 2013. The Walden Park Commons simulations used hourly data from the FCD 11 gauge in Martinez gauge from 1969 through May 2013. The following statistical analyses were then performed on the model outputs:

- Flow frequency statistics. The model outflow time series was divided into discrete flow events (i.e., a partial-duration series) using a 24-hour period of no flow to indicate the end of an event. The resulting table of events was sorted and ranked based on the peak flow rate. Each event was assigned a recurrence interval (sometimes referred to as a return period) using the Cunnane plotting position method. Partial duration series statistics were computed for the pre-project runoff and the post-project IMP outflows.

- Flow duration statistics. The model outflow time series was divided discrete bins (flow ranges). The number of hours – or duration – for which outflow occurred in each bin's flow range was then counted. These durations were computed for the pre-project runoff and the post-project IMP outflows.

Figure 2-18 shows the peak flow frequencies for the pre-project runoff and post-project (i.e., existing) outflow for Fire Prevention Bureau IMP #2. Figure 2-19 compares flow durations for the pre-project and existing conditions. In both figures, the IMP outflows are below the pre-project flows between 0.2Q2 and Q10. Additionally, IMP #2 outflows are below the pre-project site flows down to the 0.1Q2 threshold. Because IMP #2 was constructed with dimensions that are very similar to the minimum required dimensions included in the HMP, this suggests IMP #2 would comply with a stricter lower control threshold of 0.1Q2 due to the infiltration rates at the Fire Prevention Bureau site. The Walden Park Commons IMP also met Contra Costa's peak flow frequency and flow duration thresholds. These sites could probably also meet the 0.1Q2 threshold with a smaller flow control orifice.

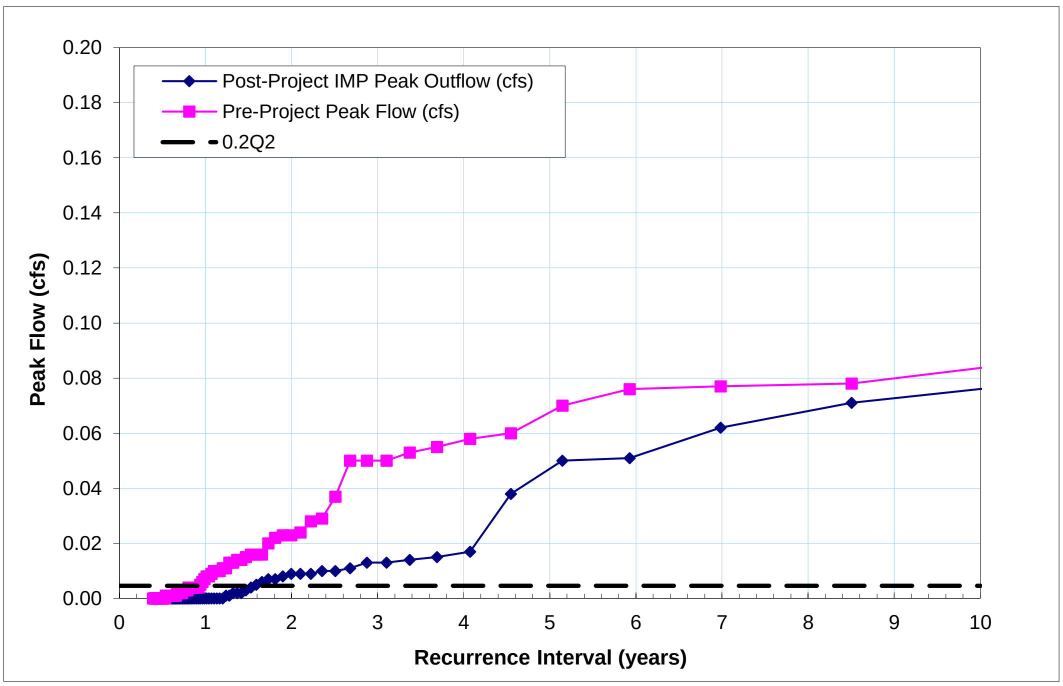


Figure 2-18. Peak flow frequency comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #2

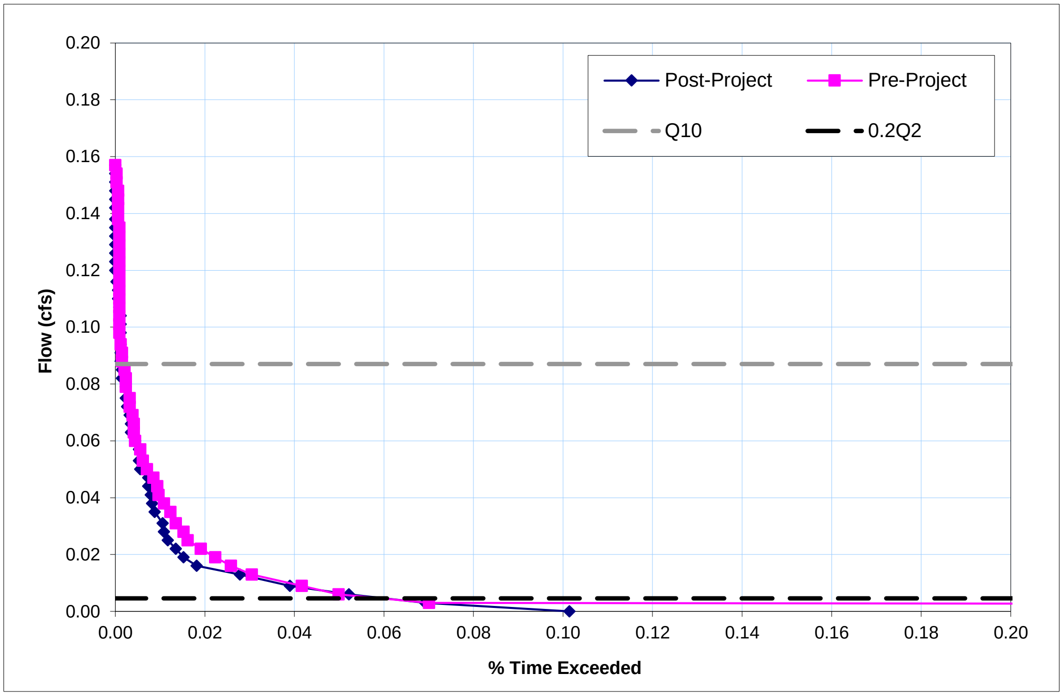


Figure 2-19. Flow duration comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #2

In 2017, Contra Costa revisited the Pittsburg Fire Prevention Bureau monitoring site to conduct additional flow and infiltration monitoring. Contra Costa also conducted infiltration monitoring of three new bioretention IMPs at an Arco gas station in Pittsburg (CCCWP, 2017a). At the Fire Prevention Bureau site, *the 2017 monitored infiltration rates exceeded the infiltration rates measured in 2011-2013*. At the Arco site, two of the three IMPs showed average infiltration rates of about 1 inch per hour while the third location showed infiltration of less than 0.2 inches per hour. During construction standing water was observed in the excavation pit for the third bioretention facility and it is likely that infiltration rates in this area are constrained by the presence of high groundwater.

Two significant conclusions can be gathered from the 2017 monitoring data. First, the Fire Prevention Bureau IMPs have not experienced any diminished performance – in fact, the bioretention infiltration capacity has improved. Second, the Arco gas station site supports the observations at the Fire Prevention Bureau, which suggests that actual infiltration rates at installed bioretention facilities are substantially greater than reference infiltration values based on soil texture.

3. Modeling and Sensitivity Analysis Approach

This section describes the modeling and technical analysis process in detail. The following topics are covered:

1. Overview of modeling approach
2. Hydrologic modeling methods
3. Geomorphic fieldwork and hydraulic modeling methods
4. Integration of modeling processes and Erosion Potential calculation

The range of parameter variability is discussed in Section 4. The modeling and sensitivity analysis results are described in Section 5.

3.1 Modeling Approach Overview

Erosion potential calculations were performed for a range of hydrologic, hydraulic, and geomorphic conditions to comprehensively evaluate sensitivity of IMP sizing factors to the variability of modeling input parameters. Long-term continuous hydrologic simulations were performed by Dubin Environmental using the HSPF program, consistent with modeling previously completed as part of the Program's HMP (CCCWP, 2006). The hydrologic modeling outputs were integrated with hydraulic and geomorphic calculations performed by Geosyntec using HEC-RAS, customized Python code, and spreadsheets. Part of the hydraulic modeling was the calculation of *effective work* for the pre- and post-project conditions. Ep was calculated as the ratio of effective work for the post- and pre-project conditions.

The performance of bioretention and other IMPs is determined for a combination of watershed variables that define inflow conditions and limits of facility outflows as well as facility design variables that affect the management of flow by the IMP (Table 3-1). We examined each of the factors affecting IMP sizing and determined how to incorporate their variability into the modeling analysis.

Table 3-1. Watershed and facility design variables that affect IMP sizing

Variable	Strategy/Rationale
Watershed Variables	
Rainfall	• Use an adjustment factor for MAP derived from linear regression
Hydrology	• Use current values for each of four hydrologic soil groups (A,B,C/D)
Geomorphic	• ***Evaluated and shown to have no impact on IMP sizing***
Incipient motion	• ***Consider variability and uncertainty when selecting sizing factors***
Build-out percent imperviousness	• ***Consider variability and uncertainty when selecting sizing factors***
IMP Variables	
Infiltration area	• Function of IMP minimum area
Storage	• Function of IMP minimum area (cross-section is specified)
Percolation rate to surrounding soils	• ***Consider variability and uncertainty when selecting sizing factors***
Orifice max. flow	• Set equal to incipient motion

The project team established reasonable upper and lower bounds for a) percolation rate for NRCS Group D soils, b) incipient motion flow threshold, and c) build-out percent imperviousness through a combination of fieldwork, GIS analysis, and published values. These parameters were then varied individually and in a compounded manner to determine how they affect IMP sizing. Finally, a recommended sizing factor was determined that would be protective during the vast majority of development scenarios. This sensitivity analysis modeling focused on bioretention facilities because these are by far the most common IMP type among projects in Contra Costa County. Figure 3-1 shows the Ep process.

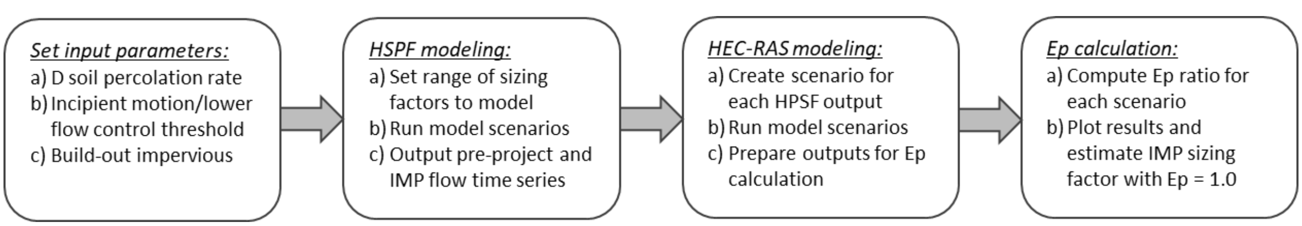


Figure 3-1. Overview of modeling process used to calculate Erosion Potential

The sensitivity analysis was developed for a “base case” that includes NRCS Group D soils and the Martinez rain gauge. Group D soils apply to more than three-quarters of development projects in Contra Costa County and the Martinez rain gauge is both centrally located and reflective of average conditions within the County. The results for this “base case” are reviewed in Sections 5 and 6 and will form a basis for the follow-up evaluation of other soil types, rain gauges and IMP types that will be included in the C.3 Guidebook.

3.2 Continuous Hydrologic Modeling Approach

Pre-project site hydrology, post-project site hydrology and bioretention hydraulics were simulated using HSPF, which is a physically-based, continuous hydrologic model that is maintained and distributed by the US EPA. The model parameters and approach to simulating bioretention hydraulics were discussed in detail in Contra Costa’s 2005 HMP. The same approach was used here with the following updates:

1. The rainfall datasets were extended through September 2016.
2. The bioretention media hydraulic characteristics were updated based on the monitoring data and modeling results described in Contra Costa’s 2013 IMP Monitoring Report. The main difference is that percolation begins at a lower moisture content than predicted by the Van Genuchten relations (see Section 2, topic 12, Figure 2-16 for more detail). This modification has a negligible effect on bioretention sizing.
3. The percolation rate to surrounding soils and bioretention outlet orifice size (used to match the lower flow control threshold/incipient motion flow) were both varied as part of the sensitivity analysis.

The bioretention modeling was setup with a 1-acre, fully paved area discharging to a bioretention facility a variable footprint area. HSPF was setup so that rain would fall on the upstream catchment and the bioretention and evapotranspiration could occur within the catchment and bioretention. The pre-project area was modeled as fully pervious, scrub land with NRCS hydrologic group D soils with an area equal to 1-acre plus the footprint of the bioretention. This matches the approach used to develop Contra Costa’s 2005 HMP. Figure 3-2 shows the pre- and post-project catchment setup.

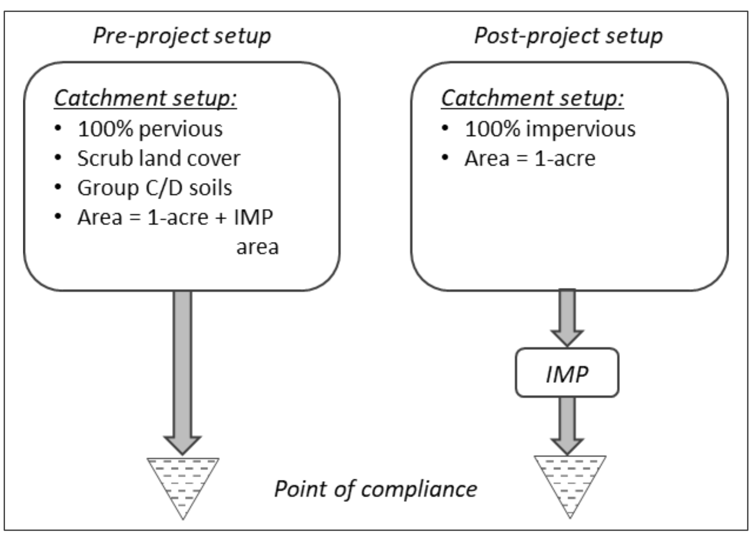


Figure 3-2. Model configuration for pre- and post-project catchment and bioretention setup

3.2.1 Rainfall Data Sources

The Contra Costa Flood Control District maintains a network of rain gauges throughout the County. Hourly data for five gauges that were included in the Contra Costa's HMP was extended through September 2016 and used for this project (Table 3-2).

Table 3-2. Contra Costa Flood Control District rain gauges used in project

FCD Station No.	Location	Period	Elev. (ft)	Mean Annual Rain (in)
11	Flood Control District, Martinez	9/1971 to present	160	16.7
12	St. Mary's College, Moraga	9/1972 to present	620	28.3
18	Orinda Fire Station, Orinda	9/1973 to present	700	28.2
19	Los Medanos, Pittsburg	7/1974 to present	130	11.9
20	Dublin Fire Station, San Ramon	9/1973 to present	355	17.0

The Martinez rain gauge was selected for the sensitivity analysis and sizing factor calculations because its period of record is the longest of any continuously operating precipitation station in the area, and storm volume and mean annual precipitation measured at Martinez lie roughly in the middle of the measurements from other gauges. The remaining gauges were used for modeling simulations to establish a regression relationship between sizing factor and mean annual rainfall (see Section 5.3).

3.2.2 Bioretention Configuration

The bioretention facility configuration was generally described in the literature review, topic 8. Table 3-3 lists the dimensions for the bioretention layers.

Table 3-3. Bioretention characteristics in HSPF model

Component	Characteristics
Surface reservoir	• Area = bioretention area • Depth = 12 inches with overflow relief set 2 inches from top of reservoir
Bioretention soil media	• Area = bioretention area • Depth = 18 inches • Percolation >= 5 inches per hour
Storage (gravel) layer	• Area = bioretention area • Depth = 30 inches • Percolation to surrounding soils = variable as part of sensitivity analysis
Underdrain	• Located at top of gravel layer • Flow restrictor orifice capacity set equal to lower flow control threshold (variable part of sensitivity analysis)

For this project, bioretention is modeled using process described in Contra Costa's 2005 HMP. These facilities are described using two separate FTABLEs, which are used in HSPF to characterize stage-storage-discharge relationships.

- The first FTABLE describes the water levels in the surface reservoir and bioretention media, percolation from the bioretention media to the gravel layer, and overflows from the surface reservoir to the downstream system.
- The second FTABLE describes water levels in the gravel layer, percolation from the gravel layer to surrounding soils, and discharge from the gravel layer via the underdrain.

3.2.3 Coordinating HSPF and HEC-RAS Modeling

Dubin Environmental and Geosyntec developed a streamlined process for integrating the results of the hydrologic modeling into the HEC-RAS hydraulic modeling and Ep calculation. The HSPF pre-project flows and bioretention outflows normalized into units of cubic feet per second per acre (cfs/ac) and written as a comma separated variable (CSV) file with columns for year, month, day, hour, pre-project flow, and bioretention outflow. A file naming convention was also developed that combined the IMP type, pre- and post-project impervious percentage, soil percolation rate, rain gauge, lower flow control threshold, and IMP sizing factor.

3.3 Geomorphology and Hydraulic Modeling Approach

This section describes the development of geomorphic characteristics that were used to create a representative HEC-RAS model for effective work and Ep calculations. The following processes are described: a) scaling the HSPF model results to the stream level, b) developing channel geometry characteristics, and c) developing bed and bank material characteristics for the model.

3.3.1 Scaling Project-Level HPSF Outputs

The hydrologic modeling was developed for tributary areas of about 1 acre. While this is representative of many development projects in Contra Costa County, the hydraulic modeling and stream channel analysis required these results to be scaled up to represent an area that is more consistent with fully formed stream channels. The ratio of pre-project flows at the stream level (1 square mile) and project level (1 acre) was computed (i.e., Q_2 watershed / Q_2 project catchment) to help form an input time series for the HEC-RAS model (Table 3-4). This scaling translated the runoff from the project catchment to erosivity in its downgradient receiving stream and supported hydraulic and effective work calculations for a larger tributary area.

Table 3-4. Scaling model results from catchment to stream level

Component	Characteristics
Receiving channel tributary area	• 1 square mile note: scaling was necessary to produce flows that are more typical formed streams and geomorphic field research
Scaling equation	• $Q_2 = 1.82 * \text{Tributary Area}^{0.904} * \text{MAP}^{0.983} = 28.94$ cfs o Tributary Area is in square miles o MAP = mean annual rainfall in inches o Empirical equation source = USGS SIR 2012-5113
Ratio of channel Q2 to project Q2	• Channel Q_2 / Project Q_2 = 28.94 cfs / 0.296 cfs = 97.8

3.3.2 Channel Geometry Assumptions

Empirical curves developed by Dunn and Leopold (1978) for the San Francisco Bay Region were used to express representative receiving channel bankfull discharge, cross-sectional area, width, and depth for the watershed area tributary to the receiving stream (Table 3-5).

Table 3-5. Dunn and Leopold channel geometry characteristics

Component	Characteristics
Bankfull cross-section area	23.45 ft2
Bankfull width	16.34 ft
Bankfull depth	1.52 ft
Bankfull discharge	51.45 cfs

Manning's equation was used to iteratively find the longitudinal slope for the assumed receiving channel, such that the wetted cross sectional area at bankfull conveys the bankfull discharge, per the Dunn and Leopold (1978) regional curve. The hydraulic analysis assumed a Manning roughness value for the main channel, corresponding to a non-vegetated, straight channel with no riffles and pools. A separate Manning's roughness was used for the over bank floodplain with an assumed side slope (Table 3-6). The "n" values used are consistent with those requested of the San Diego Regional Water Board in the development of the San Diego HMP (County of San Diego, 2011).

Table 3-6. Manning's roughness and channel slopes

Component	Characteristics
Mid-channel Manning's roughness, n	0.035
Overbank Manning's roughness, n	0.070
Overbank side slope	10 horizontal to 1 vertical
Longitudinal and channel slope	0.19 percent

3.3.3 Bed and Bank Material Assumptions

A critical low flow threshold (% of the pre-development Q_2) was used to represent the resistance to movement of the materials lining the receiving channel (Table 3-7). Flow rates below this value were assumed to contribute no geomorphic work on the channel.

Table 3-7. Critical low flow threshold calculations

Component	Characteristics
Critical low flow threshold	LCT = Variable (0.1Q2, 0.2Q2, 0.4Q2)
Project-specific low flow threshold	Qcp = {0.1, 0.2 or 0.4} * Project Q_2 Qcp = 0.0296 to 0.118 cfs
Channel-specific low flow threshold	Qcp = {0.1, 0.2 or 0.4} * Channel Q_2 Qcp = 2.89 to 11.58 cfs
Resulting critical shear stress	τ_c = 0.045 lb/ft^2 (for LCT = 0.2Q2)

3.4 Erosion Potential Calculation

Ep was calculated using the factors characterized above as inputs to the following methodology. This process is described in the following steps, consistent with Appendix D of the Vallejo HMP (2013):

1. Continuous hydrologic modeling analysis
2. Hydraulic modeling analysis
3. Binned work analysis
4. Cumulative work analysis
5. Erosion potential analysis
6. Computing IMP sizing factor

Step 1: Continuous Hydrologic Modeling Analysis

The project-scale continuous HSPF simulations, described in Section 3.2 above, were used to develop long-term simulated flow records for the pre-project condition and post-project conditions. Project flow rates derived from HSPF for the one-acre catchment were organized into a flow duration histogram which differentiates “flow bins” so that the duration of flow for each bin could be tabulated. The minimum and maximum bounds for each flow bin were multiplied by the ratio of the pre-development two-year peak discharge (i.e., Q_2 watershed / Q_2 project catchment = 28.94 cfs / 0.296 cfs = 97.8) to create a flow histogram for a representative receiving stream with tributary area of one-square mile. This scaling translated the runoff from the project catchment to that of a downgradient receiving stream with a larger tributary area.

In total, 166 flow bins were used to bound the range of simulated flowrates. The distribution for these flow bins was established according to increments of flow stage using a hydraulic analysis based on the normal depth equation. Pre- and post-project flow duration curves and flow duration histograms are provided on Figure 3-3 and Figure 3-4, respectively.

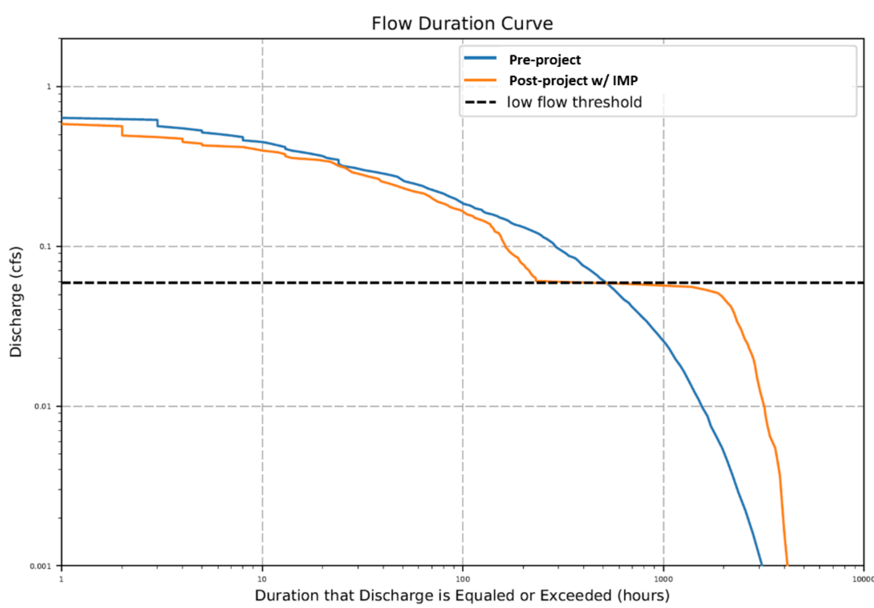


Figure 3-3. Flow duration matching example

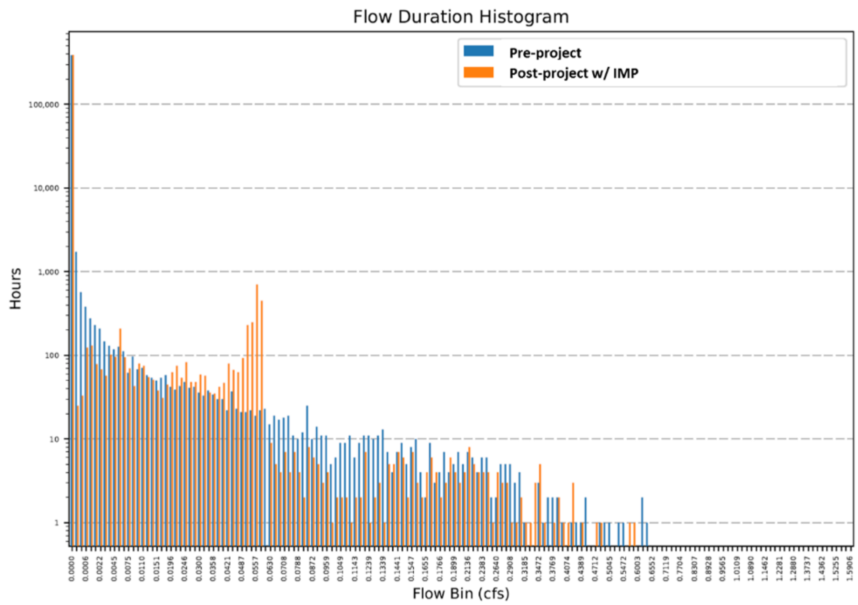


Figure 3-4. Flow durations grouped into bins and column-plotted helps illustrate differences in pre- and post-conditions

Step 2: Hydraulic Modeling Analysis

Hydraulic analysis was performed using HEC-RAS to calculate stage, mid-channel flow velocity, and effective shear stress for the range of simulated flow output (i.e., one hydraulic calculation performed for each of the 166 flow bins) using the channel geometry and roughness parameters provided in Section 3.3.2 above.

Step 3: Binned Work Analysis

The hydraulic output from the HEC-RAS model (step 2) and critical low flow threshold, provided in Section 3.3.3 above, were used to produce a work rating curve using the following effective work equation. This effective work equation was used for the SCVURPPP HMP (2005) and is also cited in the MS4 permit for the Los Angeles region (LARWQCB, 2012).

$$W = (\tau - \tau_c)^{1.5}\, V$$

Where:

W = Work;

τ = Effective Shear Stress [lb/ft^2];

τ_c = Critical Shear Stress [lb/ft^2];

V = Mid-Channel Flow Velocity [ft/s]

An example pre- and post-project work rating curve, or histogram, is provided in Figure 3-5. The same work rating curve was assumed for both pre- and post-project conditions.

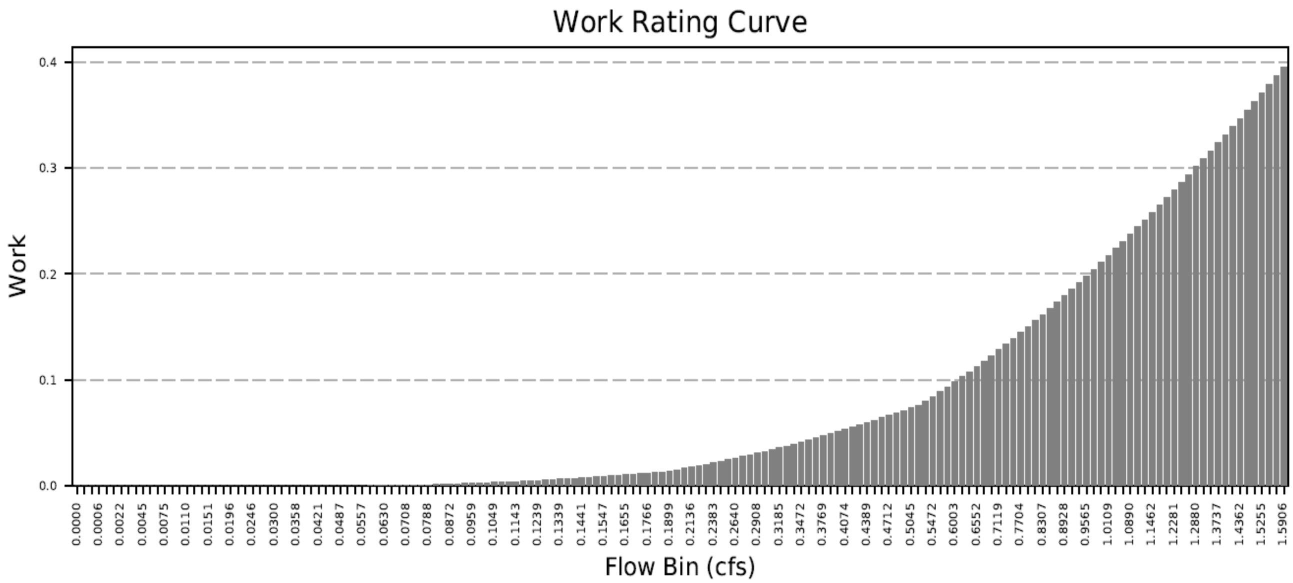


Figure 3-5. Measure of geomorphic work performed on the stream at different flow rates

Step 4: Cumulative Work Analysis

The work rating curve (Step 3) and the flow histograms (Step 1) were integrated to calculate long term total work for the pre-development condition and each post-development scenario. This analysis can be expressed as:

$$W_t = \sum_{i=1}^{n} W_i \, \Delta t_i$$

Where:

W_t = Total Work

W_i = Work per flow bin

Δt = Duration per flow bin [hours]

n = number of flow bins

The distribution of total work, represented by a work histogram, is helpful in understanding which flow rates are doing the most work in the channel of interest. A cumulative work histogram is also helpful in understanding this distribution, particularly when comparing the pre- and post-project results to one another. Example pre- and post-project work histograms and cumulative work histograms are shown in Figure 3-6 and Figure 3-7, respectively. The height of the right-most flow bin in the cumulative work histogram, Figure 3-7, represents the total work.

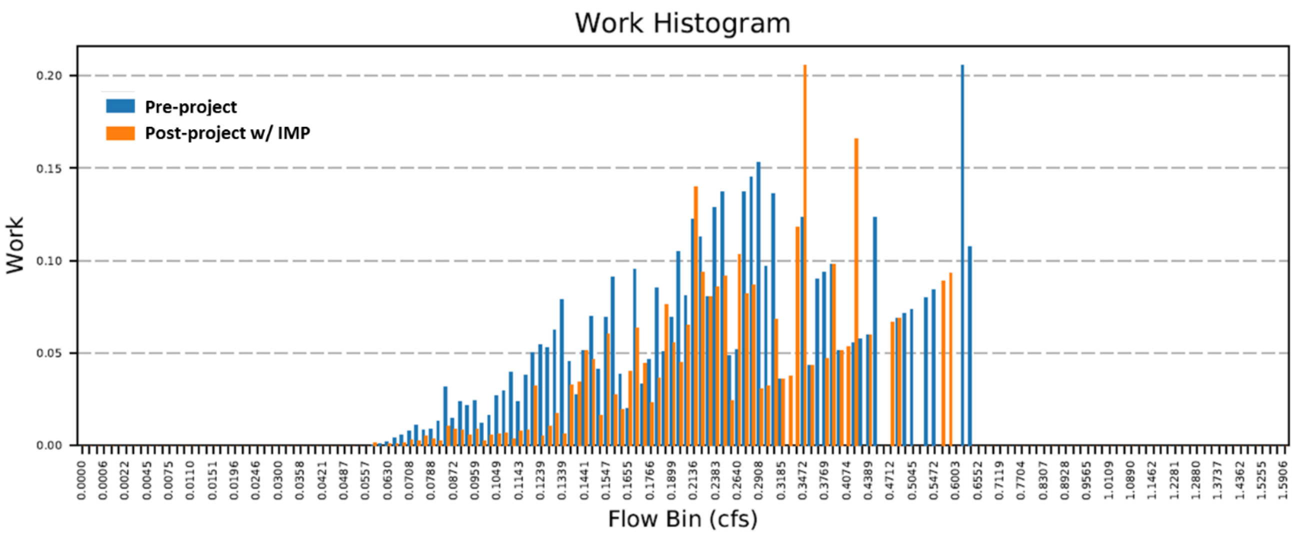


Figure 3-6. Example work histogram shows flows where pre-project work exceeds post-project work and vice versa

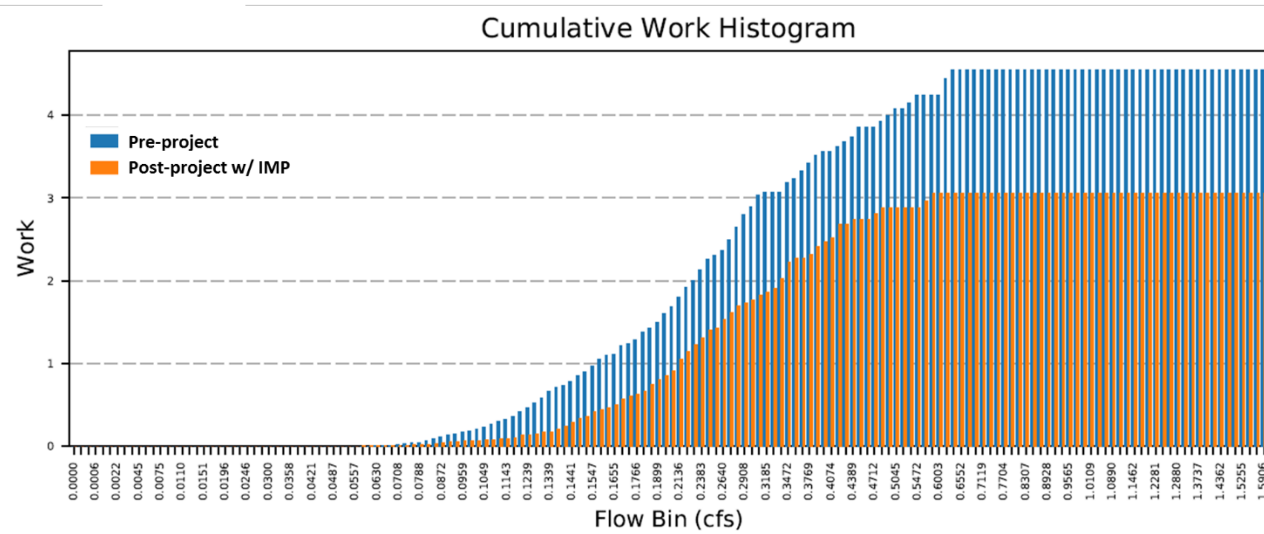


Figure 3-7. Corresponding cumulative work histogram shows IMP reduces work below pre-project conditions

Step 5: Erosion Potential Analysis

Ep was calculated by dividing the total work of the post-project condition by that of the pre-project condition (post/pre). Ep is expressed as:

$$E_p = W_{t,post} / W_{t,pre}$$

Where:

E_p = Erosion Potential

$W_{t,post}$ = Total Work associated with the post-project condition

$W_{t,pre}$ = Total Work associated with the pre-project condition

Step 6: Computing IMP Sizing Factor

IMPs effectively reduce the post-project work and Ep by providing flow control mitigation, via attenuation of runoff. In other words, onsite controls are incorporated in the Ep modeling framework at Step 1, the hydrologic analysis. Figure 3-8 shows the results for an example Ep calculations for an example scenario with the following parameters:

- Bioretention sizing factors varying from 0.020 to 0.060 in increments of 0.002 (21 in total)
- Percolation to surrounding soils = 0.24 in/hr
- Lower flow control threshold = 0.2Q2

Ep was estimated for each of the 21 sizing factors and the results were plotted. Next, a polynomial regression was fit to the data and the equation was used to compute the sizing factor corresponding to Ep = 1.0. For this example, Ep = 1.0 occurs for a bioretention sizing factor of 0.0387.

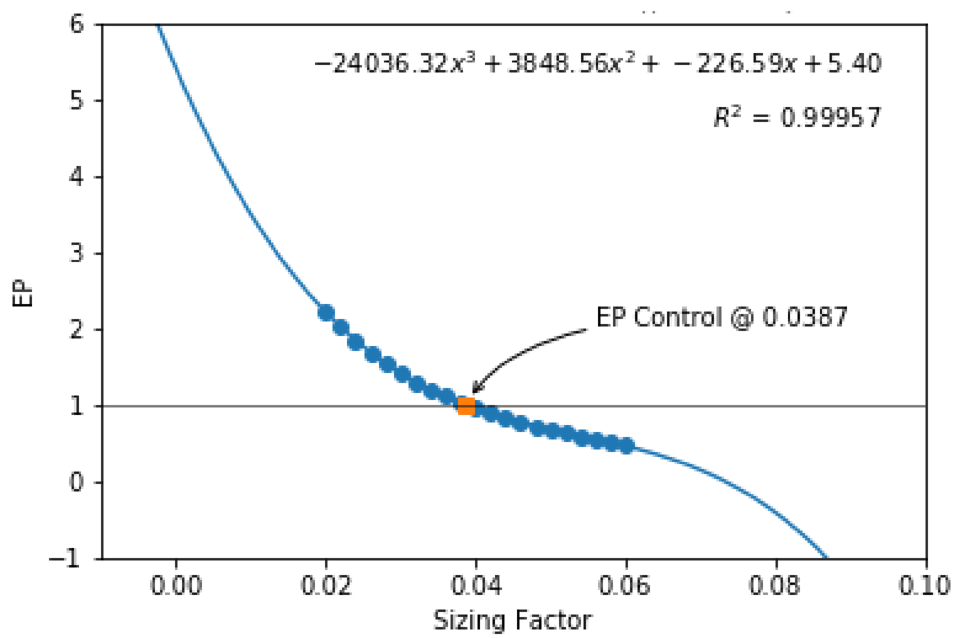


Figure 3-8. Example plot of Ep versus IMP Sizing Factor shows how Ep = 1.0 condition is computed

4. Range of Parameter Variability

This section describes the parameter bounds for each component of the sensitivity analysis, based on research/published values, fieldwork observations and GIS analysis. The variability of these parameters was applied to the "base case" scenario of Group D soils and Martinez rain gauge.

4.1 Soil Percolation Rate

The percolation rate from bioretention facilities to surrounding soils affects system performance. Higher percolation rates will reduce the hydraulic head over the underdrain and corresponding flow rates. Lower percolation rates will increase the amount of time when the gravel layer and bioretention media is saturated and discharging to the local storm drain system. About two-third of Contra Costa County has NRCS Group D soils (Contra Costa, 2005), which generally consist of clays and silty clays. There are numerous published values for Group D soils that are based on soil characteristics and/or actually monitoring of field sites (Table 4-1).

Table 4-1. Range of values for D soil percolation rate to surrounding soils

Source	Value (in/hr)	Notes
Contra Costa's 2005 HMP	0.024	• Value computed from soil characteristics in Handbook of Hydrology, Chapter 5 (Maidment, 1994). • During the preparation of Contra Costa's 2005 HMP, using LID to manage for stream bank erosion control was an innovative and controversial topic; the project team selected a conservative value due to the newness of the approach and lack of monitored installations.
Contra Costa's 2013 IMP Monitoring Study	0.24	• This value was estimated from two years of IMP monitoring at three facilities in Pittsburg and corroborated by observations at two sites in Walnut Creek. • The percolation rate in an HSPF model was calibrated to the observed gravel layer water levels during several storm events.
NRCS, Part 630 Hydrology National Engineering Handbook, Chapter 7	0.06 to 0.14	• Range based on depth to impermeable layer or groundwater table. • Statement on disturbed soils (page 7-5): As a result of construction and other disturbances, the soil profile can be altered from its natural state and the listed group assignments generally no longer apply, nor can any supposition based on the natural soil be made that will accurately describe the hydrologic properties of the disturbed soil.
Bioretention monitoring sites in Ohio (Winston, 2016)	0.067 to 0.17	• Monitoring data from two bioretention field sites in Ohio • Soils characterized as silty clay loam and clay/fill • Pre-installation measured saturated conductivity = 0.02 to 0.03 in/hr • Actual measured loss rates from facilities = 0.067 to 0.17 in/hr • Researchers suggested difference due to a combination of lateral exfiltration, higher driving head for exfiltration, minor ET
Central Coast Region SCM Sizing Calculator	0.25	• Design infiltration rate from bioretention facilities to surrounding C/D soils • Approved by Central Coast Regional Water Quality Control Board • Results incorporated into Sizing Calculator software that is used by project proponents and municipal staff throughout Central Coast Region
State of Minnesota Stormwater Manual	0.06	• Design rate for stormwater projects.
AVERAGE	0.14	• Average value computed from 5 sources

The range of published values suggests that D soil percolation rates of 0.024 in/hr to 0.24 in/hr are within the bounds are likely conditions for construction projects in the County. The average value among the six sources is 0.14 in/hr. Additional sources could help develop a distribution for percolation rates but these examples clearly show that percolation values that exceed the 2005 HMP value are likely. In particular, the Ohio monitoring site shared a key observation with the 2013 IMP Monitoring Project: installed bioretention facilities may exfiltrate water more rapidly than expected from soil characteristics due to hydraulic factors within the facility and soil disturbance on the project site.

4.2 Incipient Motion Threshold

Geosyntec performed geomorphic field reconnaissance on February 13th, 15th, and 23rd, 2017 for 15 stream channels within Contra Costa County (Figure 4-1). These channels are susceptible to hydromodification impacts (i.e., erodible bed and/or banks), are downgradient of anticipated future development, and were readily accessible. Areas anticipated for development in the near future were identified based on input from the Contra Costa municipalities and from the Greenbelt Alliance's At-Risk

Map (2017). The At-risk Map identifies protected open space, existing urban areas, and areas at various levels of risk (i.e., high, medium, and low) for future development. Appendix B summarizes the fieldwork.

Geomorphic field work performed previously for 13 other stream channels was used to supplement the 2017 field observations where appropriate. These channels were selected by Philip Williams and Associates to typify high, medium, and low vulnerability to erosion in natural channels; most are located in less urbanized areas than those selected for the February 2017 field work. For each channel visited in the field, the channel geometry and bed and bank material was noted. Measured bankfull widths and depths were compared to empirically derived values for the San Francisco Bay Area (Dunne and Leopold, 1978).

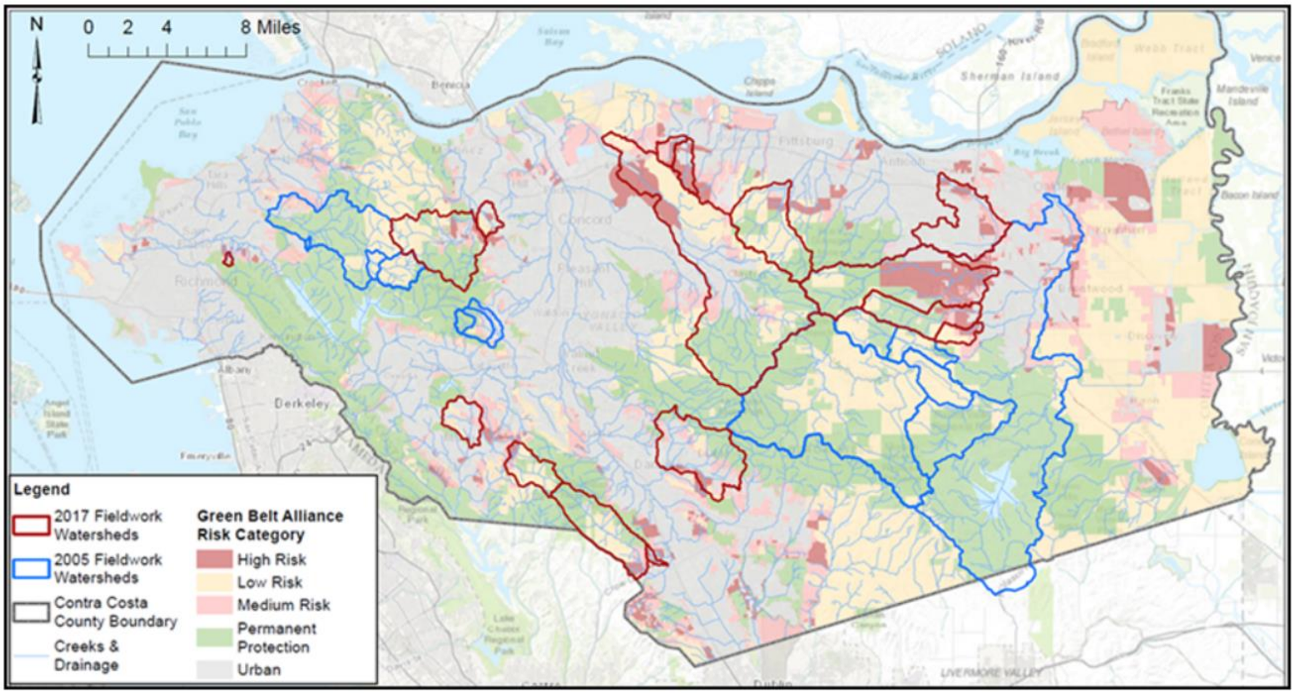


Figure 4-1. Location of fieldwork watersheds in this study and 2005 HMP

The initial goal of the fieldwork was to collect the data necessary to determine the sensitivity of IMP sizing factors to the variability of geomorphic parameters. However, the sizing factors were not sensitive to geomorphic variations because Ep is based on the ratio of post-project and pre-project work in the channel and the same channel conditions apply for the pre- and post-conditions.

Later, the geomorphic data was used to estimate the incipient motion threshold for stream bank material, which is one of the parameters included in the sensitivity analysis. (Note: the flow control orifice in bioretention facilities or other IMPs is sized to the incipient motion threshold, adjusted by the ratio of predevelopment Q2 flows for the area tributary to the IMP to Q2 flows from the watershed upstream of the stream location where Ep is determined.) The incipient flow threshold was estimated for the 15 cross-sections studied in this project and the 13 cross-sections included in Contra Costa's 2005 HMP. Upper and lower bounds for the threshold were computed using critical shear stress values of 0.26 lb/ft2 and 0.67 lb/ft2, which correspond to alluvial silt (colloidal) and shales and hardpan bank material, respectively. Vegetated banks are also consistent with the higher critical shear stress value. The incipient motion threshold was also computed for a critical shear stress of 0.35 lb/ft2, which Appendix B refers to as "a reasonable value that is generally reflective of the cohesive clay and silt banks observed in the field" and is also within the range of critical shear stress values observed in the SCVURPPP HMP (2005) and Fairfield-

Suisun HMP (FSURMP, 2009). Figure 4-2 shows the range of incipient motion thresholds for these 28 locations (note: the vertical axis is plotted as a log scale).

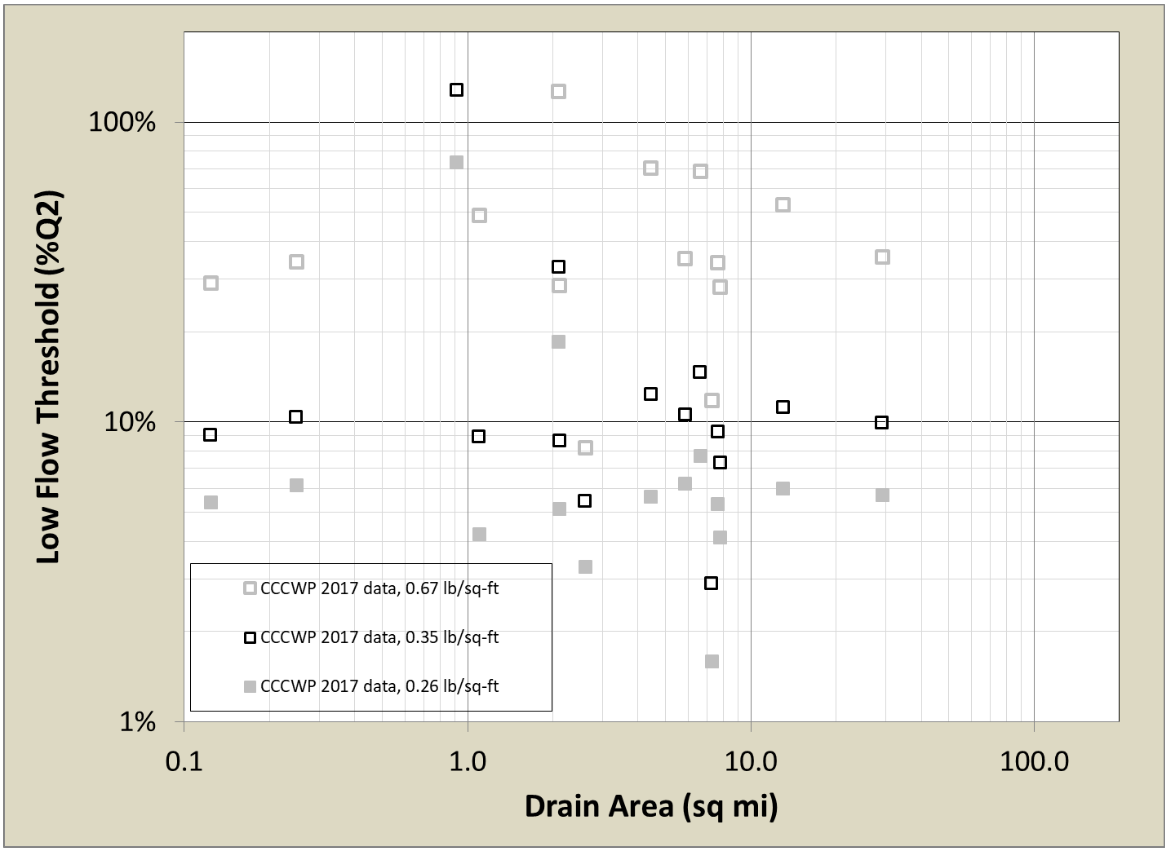


Figure 4-2. Incipient motion analysis shows variability of flow that causes erosive flows in the streams

Table 4-2 summarizes the range of incipient motion values for the three shear stress values. For the Contra Costa stream cross-sections visited in the field show, the estimated incipient motion threshold ranges from <0.1Q2 to more than 0.5Q2. The median of the low bounds and high bounds are 0.06Q2 and 0.35Q2, respectively, and several sections exceed 0.4Q2. This analysis shows the range of lower control thresholds included in the sensitivity analysis (0.1Q2, 0.2Q2, 0.4Q2) is a good representation of the variability in the field.

Table 4-2. Range of values for incipient motion for 28 Contra Costa stream sections

Statistic	Low Bound of Shear Stress (0.26 lb/ft2)	Middle Value of Shear Stress (0.35 lb/ft2	High Bound of Shear Stress (0.67 lb/ft2
Median	0.06Q2	0.10Q2	0.35Q2
Distribution of Incipient Motion Thresholds Among Stream Sections			
<0.1Q2	22	14	2
0.1Q2 to 0.2Q2	3	9	4
0.2Q2 to 0.3Q2	1	1	5
0.3Q2 to 0.4Q2	0	1	5
>0.4Q2	2	2	12

4.3 Watershed Build-Out Impervious Percentage

Geosyntec also performed geospatial analysis of the watersheds tributary to the channels where geomorphic field work was performed. The key attributes compiled for these watersheds include tributary area, existing imperviousness, and protected open space. The StreamStats program (USGS, 2017b) was used to delineate and calculate watershed area and existing imperviousness. StreamStats uses the National Land Cover Dataset (MRLC, 2011) clipped to the watershed as a basis for the imperviousness calculation. Protected open space was calculated based on GIS geospatial analysis of the Greenbelt Alliance At-Risk Map (2017) clipped to each field watershed.

The current development levels, current impervious levels, and future development levels (as defined by the Greenbelt Alliance's estimate of protected open space) were evaluated for Contra Costa watersheds to estimate the range of build-out impervious levels. The National Land Cover Dataset lists the development and impervious percentage, as estimated in 2011. The developed percentage was divided by the impervious percentage to estimate the typical imperviousness for developed areas in the watershed. The imperviousness was then multiplied by the Greenbelt Alliance's estimated build-out development percentage to estimate the build-out imperviousness percentage.

$$BuildOut\ Imp.\% = \frac{NLCD2011\ Imp\ \%}{NLCD2011\ Dev\ \%}{}^{\circ} \times GBA\ BuildOut\ \%$$

Table 4-3 lists the upstream watershed development parameters for the 15 field sites visited by Geosyntec in February 2017, including the estimated build-out impervious percentage. The far right column in the table included a 20 percent safety factor in case future development occurs at a higher density than historical development in these watersheds. Figure 4-3 shows the estimated build-out impervious values.

Table 4-3. Development levels for Contra Costa watersheds tributary to 2017 field sites

Site ID	Drainage Area (square miles)	NLCD2011 Developed	NLCD2011 Impervious	Greenbelt Alliance Build-Out	Estimated Build-Out Impervious	Est. Build-Out Impervious (x1.2 density)
El Sobrante 15	0.12	2.8%	0.7%	74%	18.6%	22.3%
Martinez 08	7.63	13.3%	1.6%	54%	6.3%	7.6%
Martinez 09	0.91	57.4%	12.5%	100%	21.7%	26.1%
Concord 06	29.01	28.2%	9.0%	62%	19.9%	23.9%
Bay Point 07	2.10	86.0%	39.5%	100%	45.9%	55.1%
Pittsburg 11	7.26	9.5%	2.7%	69%	19.9%	23.9%
Moraga 04	2.09	58.0%	13.9%	98%	23.6%	28.3%
Moraga 03	2.60	7.5%	0.4%	69%	3.5%	4.2%
Danville 01	7.77	43.8%	7.8%	71%	12.7%	15.2%
San Ramon 10	5.84	7.3%	0.9%	52%	6.8%	8.1%
San Ramon 02	0.25	17.8%	7.3%	88%	36.1%	43.4%
Antioch 12	6.62	88.5%	38.5%	100%	43.5%	52.2%
Brentwood 05	12.98	13.9%	5.0%	63%	22.9%	27.4%
Brentwood 14	1.09	8.1%	1.6%	100%	20.5%	24.6%
Brentwood 13	4.42	8.1%	2.5%	48%	14.6%	17.5%
	AVERAGE	**30.0%**	**9.6%**	**76.6%**	**21.1%**	**25.3%**
	MEDIAN	**13.9%**	**5.0%**	**71.2%**	**19.9%**	**23.9%**
	MAXIMUM	**88.5%**	**39.5%**	**100.0%**	**45.9%**	**55.1%**

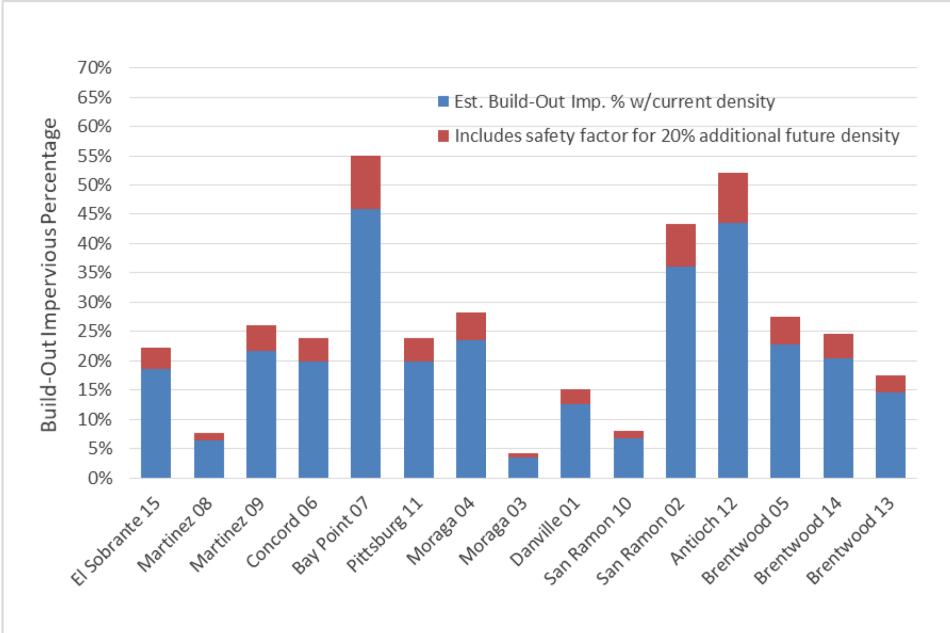


Figure 4-3. Estimated build-out impervious percentages for watersheds included in fieldwork

5. Modeling and Sensitivity Analysis Results

The sensitivity analysis recognizes there are several factors that will affect a bioretention facility's performance and the level of protection bioretention facilities provide to downstream surface water bodies. Table 3-1 in Section 3 listed the variable parameters to include in the sensitivity analysis:

1. Percolation rate from bioretention facilities into surrounding soils
2. Threshold for incipient motion of bed material in local streams
3. Build-out impervious percentage in the watershed

This section first describes the sensitivity analysis results for the parameters listed above for the "base case" scenario with Group D soils and Martinez rainfall data, and then describes how local rainfall affects IMP sizing. Section 4 examined the range of variability for D soil percolation, incipient motion flow threshold, and watershed build-out impervious percentage. Table 5-1 summarizes these results and the range of mean annual rainfall. Figure 5-1 shows the process for developing a recommended sizing factor.

Table 5-1. Characteristics of sensitivity analysis components

Sensitivity Analysis Factor	Characteristics and Range of Values
Percolation rate to surrounding soils	• Range = 0.024, 0.08, 0.12, 0.24 in/hr • Average value from reference material = 0.14 in/hr
Threshold for incipient motion in stream	• Median lower and upper bounds for surveyed streams = 0.06Q2 to 0.35Q2 • Average of median lower and upper bounds = 0.21Q2
Level of development in watershed	• Median build-out impervious = 21% • Maximum build-out impervious = 45%

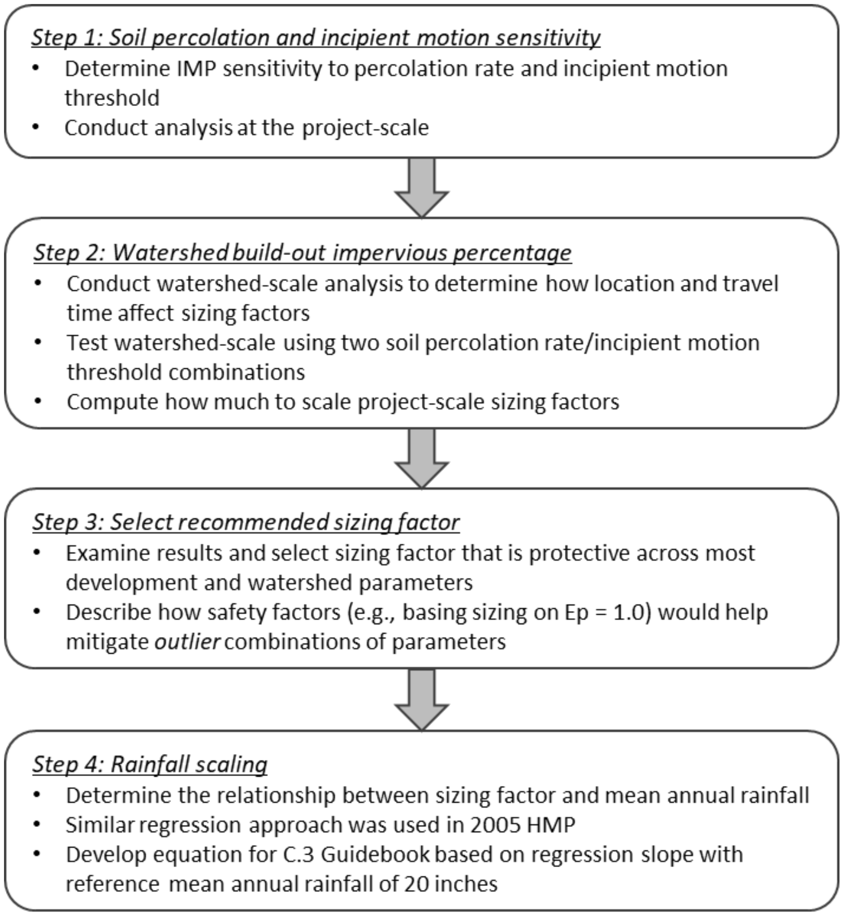


Figure 5-1. Sensitivity analysis process for computing recommended sizing factor

5.1 Percolation Rate and Incipient Motion Threshold Results

The first set of results examines the effect of percolation rate and incipient motion threshold (considered the lower flow control threshold for bioretention orifice design) on bioretention sizing. These parameters were selected because they are difficult to measure directly and were predicted to have a significant impact on bioretention sizing. Table 5-2 graphically shows the constant and variable parameters in this portion of the sensitivity analysis (note: this table is repeated for each component of analysis). The parameter value ranges are based on the discussion in Sections 4.1 and 4.2.

Table 5-2. Percolation and incipient motion compounded sensitivity

Percolation rate	Varies: 0.024 to 0.24 in/hr
Incipient motion threshold	Varies: 0.1Q2 to 0.4Q2
Build-out impervious	Constant (100% impervious)
Rainfall	Constant (Martinez rain)

For each combination of percolation rate and incipient motion threshold (12 combinations in total), Ep was computed across a range of bioretention sizing factors to determine the sizing factor that yields an Ep = 1.0 (see Figure 3-1 for process schematic and Section 3.4 for description of Ep calculation). Table 5-3 lists the Ep-based sizing factors that were computed using the Flood Control District's Martinez rain gauge.

Table 5-3. Ep-based sizing factors for D soils and lower control threshold variability

Lower Control Threshold	Percolation Rate to Surrounding NRCS Group D Soils			
	0.024 in/hr	**0.080 in/hr**	**0.120 in/hr**	**0.240 in/hr**
0.1Q2	0.067	0.059	0.056	0.048
0.2Q2	0.048	0.044	0.042	0.039
0.4Q2	0.035	0.033	0.032	0.030
Scenario conditions: NRCS D soils, Martinez gauge, Build-Out = 100% impervious				

The bioretention sizing factors for the combined percolation rate and incipient motion threshold sensitivity analysis range from 0.030 to 0.067. Reading across a specific row or specific column in the table helps show how the parameters affect bioretention sizing individually. Higher incipient motion thresholds (e.g., 0.2Q2 instead of 0.1Q2) result in smaller facilities because water can be discharged more quickly. Higher percolation rates result in smaller facilities because more of the stormwater entering the bioretention facility is managed onsite and not discharged to the downstream system.

Figure 5-2 and Figure 5-3 show the modeling results graphically. The first figure shows the effect of soil percolation rate on the bioretention sizing factor. For lower control thresholds of 0.2Q2 and 0.4Q2 the slope of the line is relatively flat and steeper for the 0.1Q2 scenario. Looking at the table above, the sizing factor for the slowest percolating soils is only 20 percent larger than the sizing factor for the fastest percolating soils for the 0.2Q2 and 0.4Q2 scenarios. For the 0.1Q2 scenario, the soil percolation has a larger effect with the facility in the slowest percolating soils about 20 percent larger than with median percolating soils and 40 percent largest than a facility in the fastest percolating soils.

The second figure shows similar lessons but from a different perspective. Comparing the vertical spread among the trend lines shows that percolation rate has a larger influence on bioretention sizing factors for small incipient motion/lower flow control thresholds. Also, the influence of the lower flow control threshold is greater for smaller values, as indicated by the steeper slope between 0.1Q2 and 0.2Q2 and relatively flatter slope between 0.2Q2 and 0.4Q2.

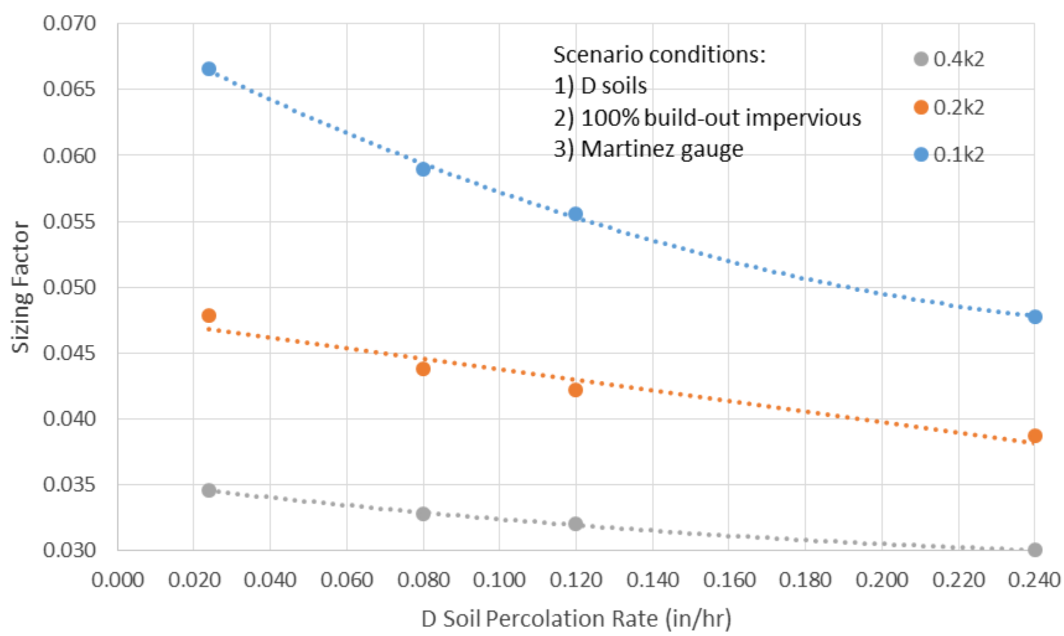


Figure 5-2. Effect of soil percolation rate on bioretention sizing factor

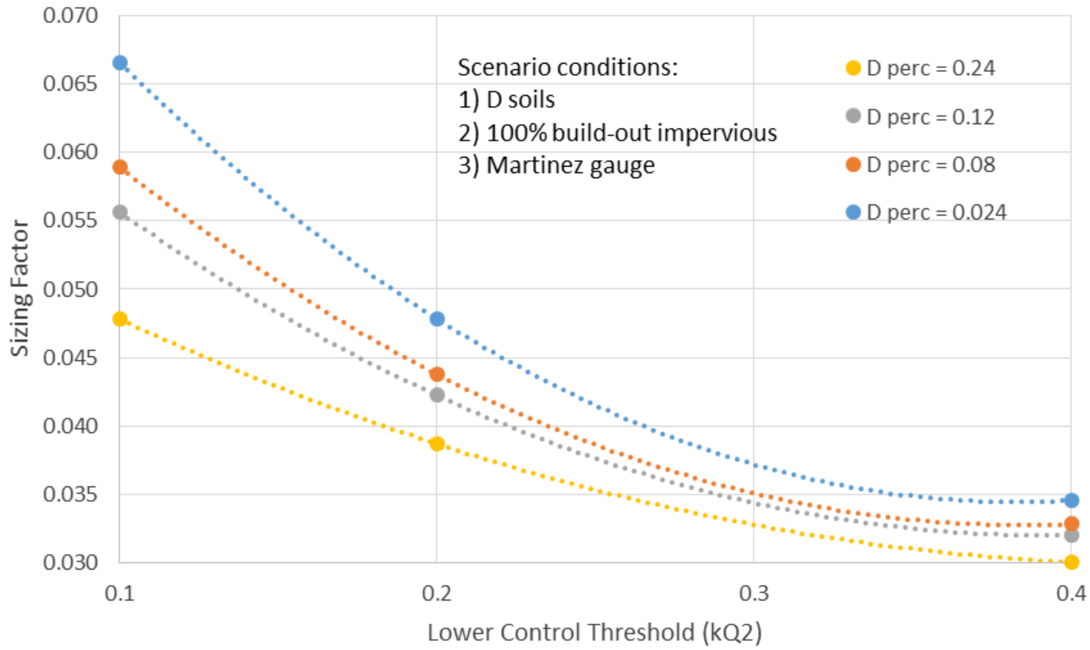


Figure 5-3. Effect of lower flow control threshold on bioretention sizing factor

The table and figures above clearly illustrate the individual and compounded influence of soil percolation rate and incipient motion threshold. But to help provide a broader picture of how to apply these results, the combined percolation and incipient motion threshold sensitivity results were fitted to a set of regression equations that help illustrate the sizing factors that would apply for different combinations of parameters. Figure 5-4 is a nomograph that shows that shows sizing factors of 0.040, 0.045, 0.050 and 0.055 and the combinations of parameters where these sizing factors apply. Please note, this analysis was conducted at the project-scale with 100 percent imperviousness. These results will be scaled in the next section when we assess the impacts of LID at the watershed scale.

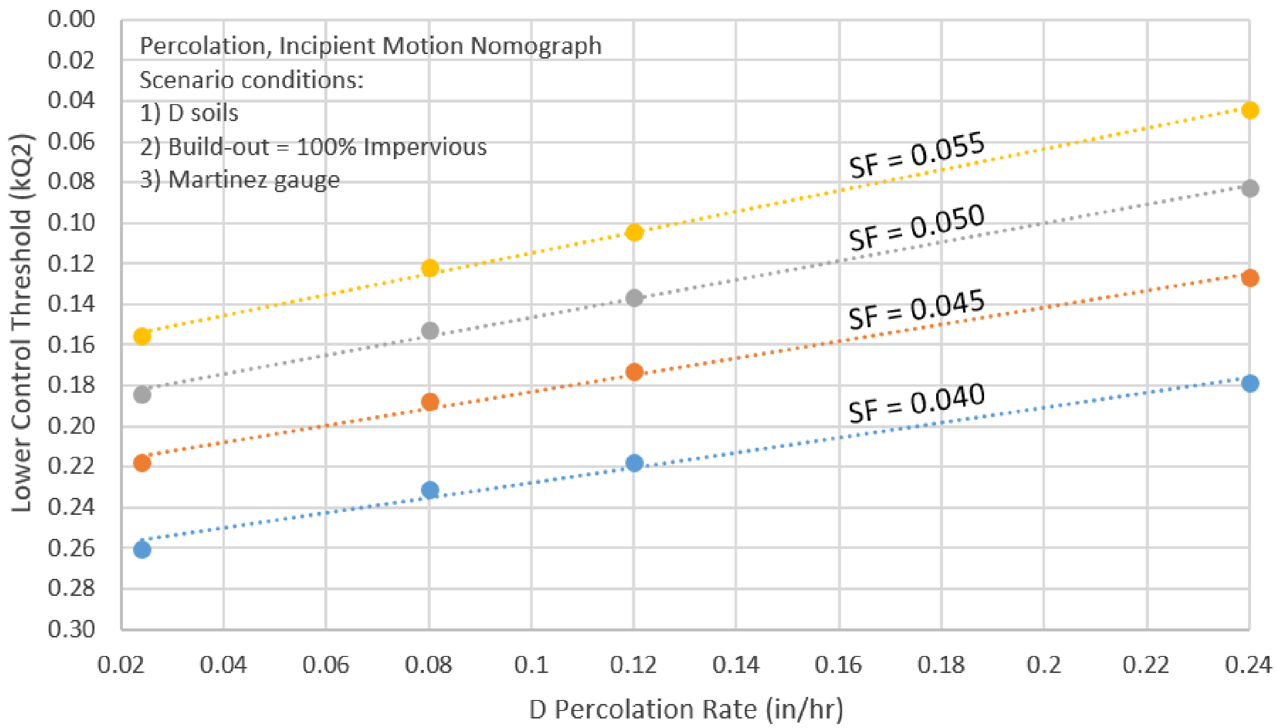


Figure 5-4. Nomograph describing the relationship among lower control threshold, soil percolation rate and sizing factor

5.2 Watershed Build-Out Impervious Percentage Results

As described in Section 3.2, the hydrologic modeling analysis computes runoff for a fully paved, 1-acre area and routes these flows through a bioretention facility. This sets the point of compliance at the outlet of the bioretention facility. This approach makes sense when designing large stormwater detention or flood control facilities but may not be appropriate for distributed LID where numerous facilities through a watershed combine to protect creeks. At the watershed scale, the travel time from the bioretention to the stream and the attenuation associated with multiple distributed bioretention facilities may reduce the necessary sizing factors.

This concept was tested by preparing a watershed-scale scale HSPF model, based on the Alhambra Creek watershed, near Martinez. The watershed boundaries and subcatchment areas were computed using watershed tools within ArcGIS and then comparing the results to the watershed boundaries and stream network data available from Contra Costa County's GIS data. The 4,900 acre watershed was divided into 24 different subcatchments and a similar number of stream segments (RCHRES elements in HSPF).

For simplicity and to test the effects of urbanization, the model was considered fully pervious with NRCS group D soils. These assumptions are close to the present reality because less than 2 percent of the watershed is impervious and all soils are Group C/D. The model was run for 40+ years and the results were compared to limited USGS flow monitoring data in the area. The largest flow event on record occurred on 1/4/1982. The USGS daily data for recorded 982 cfs. The peak hourly data from the model was about 2,000 cfs and the daily average was about 800 cfs. Without knowing for certain how the USGS data was collected, the relative agreement between the model and recorded data suggests the watershed model was an appropriate tool to use. Table 5-4 summarizes the model characteristics and Figure 5-5 shows the distribution of subcatchments.

Table 5-4. Watershed-scale model characteristics

Model Element	Characteristic
Total drainage area	4,896 acres
Number of subcatchments	24
Number of stream reaches	24
Length of stream reaches	72,850 feet

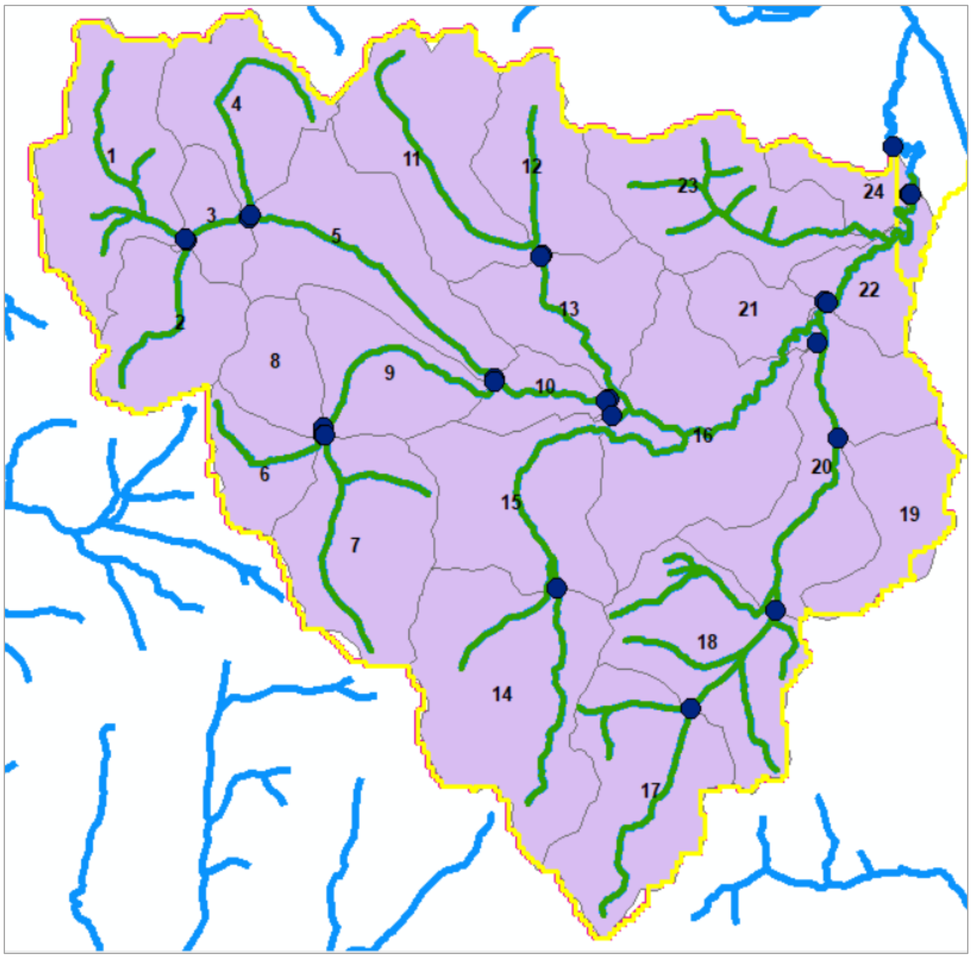


Figure 5-5. Alhambra Creek used to create model testing effect of development level on bioretention sizing

The build-out impervious value for the watershed was varied from 20 percent to 50 percent, based on the analysis presented in Section 0. For each subcatchment area, the impervious surfaces were routed to bioretention facilities while the pervious land surfaces were allowed to flow directly to the stream segments. The bioretention underdrain and overflows were also routed to the stream segments. This approach is conservative because it does not account for the C.3 Guidebook's onsite measures that mitigate runoff from pervious areas, such as soil amendments and site grading to produce self-retaining areas. Table 5-5 shows the constant and variable parameters in this portion of the sensitivity analysis.

Table 5-5. Watershed build-out impervious sensitivity analysis

Percolation rate	Two scenarios: 0.024 or 0.24 in/hr
Incipient motion threshold	Constant: 0.1Q2
Build-out impervious	Varies: 20 to 50% impervious
Rainfall	Constant (Martinez rain)

The model was run and Ep values (Ep = 1.0) were computed for a range of sizing factors. These sizing factors were then compared to the corresponding project-scale sizing factors in the previous section to determine how much the project-scale sizing factors could be adjusted based on the watershed analysis. Recognizing that development can occur at different locations in the watershed, the analysis looked at two separate locations: a) the bottom of the watershed and b) an upper watershed location at the outlet of subcatchment 3 (see Figure 5-5), which is the smallest subcatchment in the upper watershed. Table 5-6 and Table 5-7 shows the results for the lower and upper watershed, respectively.

Table 5-6. Lower watershed: effect of build-out impervious percentage

Build-Out Impervious Percent	Bioretention Sizing Factor and Scaling (Lower Control Threshold = 0.1Q2)			
	Percolation = 0.024 in/hr		Percolation = 0.24 in/hr	
	Sizing Factor	Reduction	Sizing Factor	Reduction
Project-scale (100%)	0.067	N/A	0.048	N/A
50%	0.067	0%	0.046	5%
40%	0.063	6%	0.042	12%
30%	0.059	12%	0.040	17%
20%	0.055	18%	0.036	24%

Table 5-7. Upper watershed: effect of build-out impervious percentage

Build-Out Impervious Percent	Bioretention Sizing Factor and Scaling (Lower Control Threshold = 0.1Q2)			
	Percolation = 0.024 in/hr		Percolation = 0.24 in/hr	
	Sizing Factor	Reduction	Sizing Factor	Reduction
Project-scale (100%)	0.067	N/A	0.048	N/A
50%	0.067	0%	0.048	1%
40%	0.066	1%	0.045	6%
30%	0.061	8%	0.043	11%
20%	0.058	13%	0.039	18%

The GIS analysis in Section 0 indicates that most watersheds can expect a build-out impervious percentage of 20 to 30 percent. This is based on the current patterns of development, protected open space estimates from the Greenbelt Alliance and an allowance that future develop could occur at higher densities (assumed 20 percent more dense). For projects located in the upper part of a watershed, the shaded rows in the table above indicate that bioretention retention sizing factors could be *reduced 10 percent* relative to sizing factors computed at the project-scale (see Table 5-3 in previous section). Projects located in the lower portion of a watershed could reduce sizing factors more.

Figure 5-6 is a nomograph that shows that shows sizing factors of 0.040, 0.045, 0.050 and 0.055 and the combinations of soil percolation and incipient motion thresholds where these sizing factors apply. This figure is an updated version of Figure 5-4, which has been modified to show the 10 percent sizing reduction described in the paragraphs and table above.

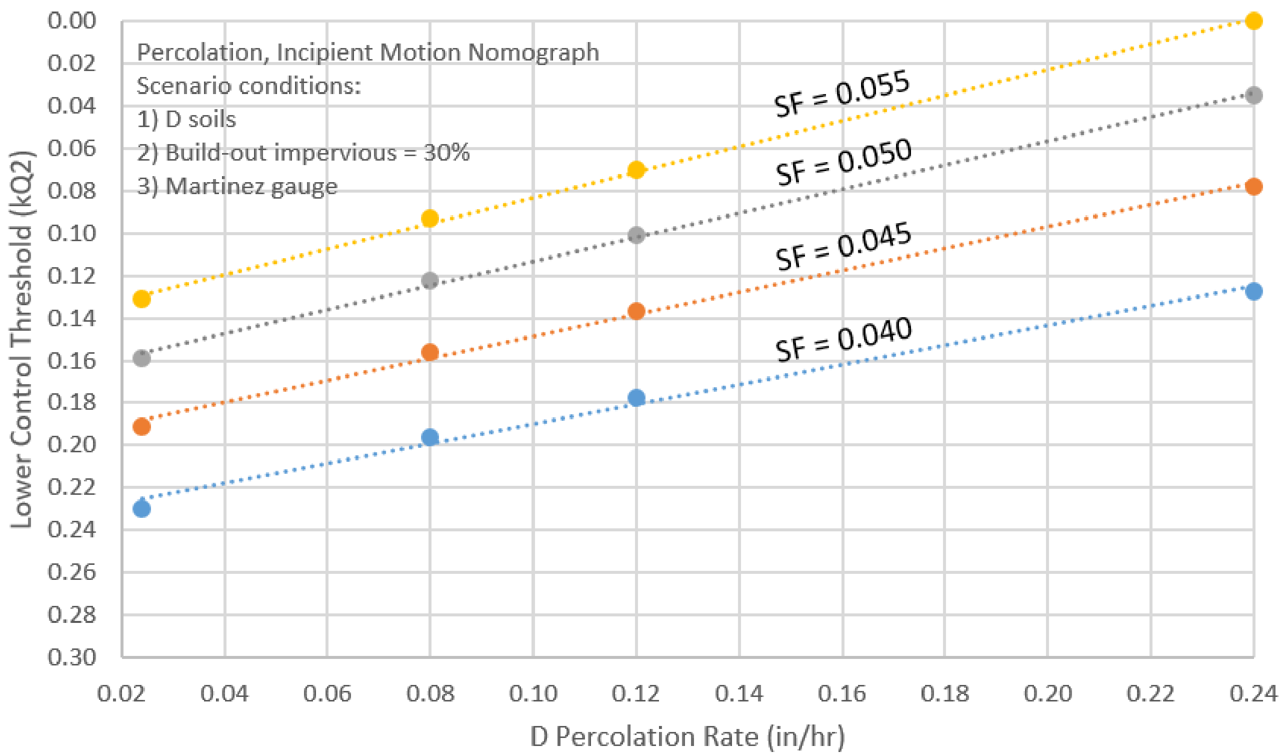


Figure 5-6. Nomograph of lower control threshold, soil percolation rate and sizing factor for 30% watershed impervious

5.3 Rainfall Variability Results

Similar to Contra Costa's 2005 HMP, bioretention facilities were modeled using different rain gauges to determine the influence of mean annual rainfall on IMP sizing factors. Ep calculations were performed for the five rain gauges described in Section 3.2.1 (see Table 3-2) for the purpose of developing a regression equation that will scale the sizing factors included in the C.3 Guidebook to reflect rainfall patterns at any project's location. Table 5-8 shows the constant and variable model parameters for this analysis.

Table 5-8. Rainfall variability model parameter setup

Percolation rate	Constant: 0.24 in/hr
Incipient motion threshold	Constant: 0.2Q2
Build-out impervious	Constant (100% impervious)
Rainfall	Varies: MAP = 11.9 to 28.3 in

Figure 5-7 shows the model results for Group D soils. As expected, the IMP sizing factors are smaller for wetter areas and larger in dryer areas. Bioretention facilities located in wetter areas receive more stormwater inflow but can also release water more quickly because these areas have higher Q2 values. The larger Q2 value has a larger influence than the additional inflow and the result in a smaller facility. This result was expected and mirrors the results from the HMP.

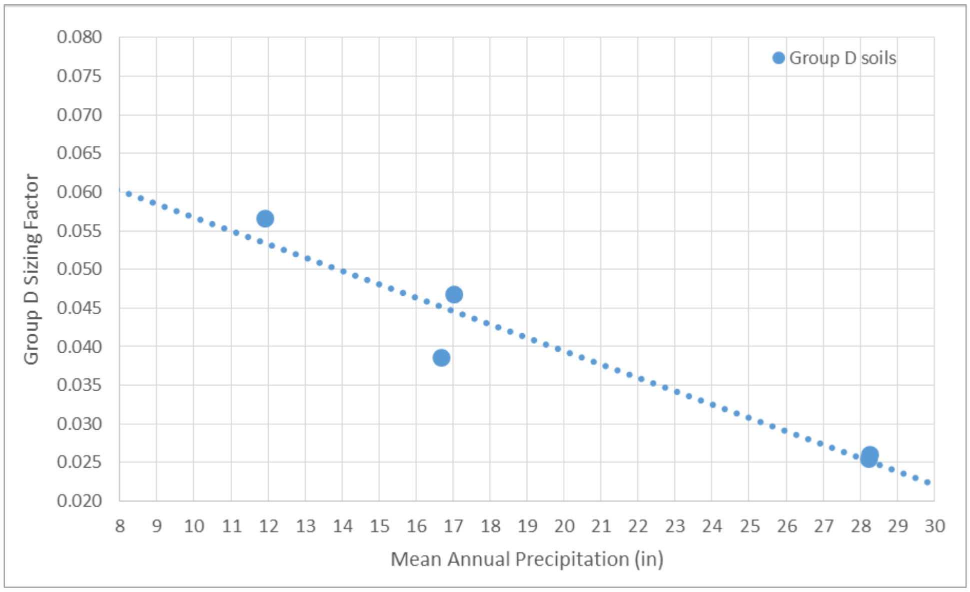


Figure 5-7. Relationship between sizing factors and mean annual rainfall

The rainfall variability results will be incorporated into the C.3 Guidebook. Similar to the previous editions, the 7th Edition will include a regression equation, based on the graphic above, will be used to scale the reference sizing factor:

$$SizingFactor_{project} = SizingFactor_{ref} + Slope * \ (MAP_{project} - MAP_{ref})$$

6. Selection of a "Base Case" Sizing Factor

The selection of a "base case" sizing factor requires integrating several issues that affect the performance of individual IMPs and their aggregated influence within a watershed. This section builds upon the discussion in Sections 4 and 5 and explores specific examples to develop an appropriate sizing factor. The following issues are examined:

1. Considerations for sizing distributed stormwater facilities and how this differs from sizing large, regional facilities
2. Factors of safety inherent in the modeling sensitivity analysis and how to incorporate them into IMP sizing decisions

6.1 Approach for Sizing Distributed Stormwater Facilities

Bioretention and other IMPs are typically distributed within a watershed and together they protect local streams from accelerated erosion due to increased development. Multiple facilities working together generate resiliency within the system as the over-performance of some facilities can balance out the under-performance of others. This stands in distinct contrast to the design requirements for large, regional stormwater facilities, such as detention or flood control ponds. These large systems must meet performance targets to protect people and downstream properties from flooding. As a result, traditional stormwater measures must be designed with conservative assumptions and contingencies that minimize the risk of failure. Oversizing is often necessary because the consequences of failure are substantial.

For bioretention and other IMP types, the consequences of under-performance or even failure among a small portion of facilities are far less significant. An under-performing bioretention facility will discharge water via an overflow structure more often than expected. The peak flows to local creeks will be higher than expected but these situations should not result in property damage or public health emergencies. Further, the under-performance of any subset of facilities can be balanced by facilities that exceed the designer's expectations. The Pittsburg and Walnut Creek sites monitored by Contra Costa Flood Control District staff (Contra Costa, 2013) are two examples of facilities exceeding the designer's expectations.

Engineering design standards and practices for facility sizing occur along a spectrum ranging from the most conservative (e.g., "let's combine layers of conservative assumptions and consider all the things that could go wrong") to the least protective (e.g., "let's design for what's likely to happen and adapt if things occur differently"). For distributed LID facilities, neither of these bookends is appropriate. The conservative approach would drive facility sizes larger while generating little additional benefit and in doing so would discourage developers from adhering to CCCWP's goals for incorporating LID principles into development projects. The least conservative approach would not sufficiently protect local streams.

For Contra Costa, we recommend an approach that falls between these bookends:

a) Selecting a sizing factor that applies across most combinations D soil percolation rates, incipient motion thresholds, and build-out impervious percentages

b) Estimating safety factors inherent in the sensitivity analysis and demonstrating that all or almost all combinations of site conditions (even the most conservative assumptions) fit within the "base case" IMP size + safety factor

c) Allowing that the small set of conditions not covered by the recommended sizing factor would be balanced by over-performance of most other facilities (e.g., a small percentage of under-sized facilities will be balanced by a larger number of facilities producing an effective Ep < 1.0)

6.2 Safety Factors Inherent in Sensitivity Analysis

This section describes the safety factors inherent in the application of erosion potential for IMP sizing, the method used to adjust property-scale modeling results to the watershed scale, and runoff assumptions for landscaped portions of a development project.

6.2.1 Application of Erosion Potential

Consistent with the MRP HM standard, IMP sizing for Ep control was performed such that, "stormwater discharges from HM projects shall not cause an increase in the erosion potential of the receiving stream over the pre-project (existing) condition." This means that a target Ep equal to 1.0 was used as the basis for meeting the Ep control standard.

Sizing the IMPs to a target Ep of 1.0 has an inherent safety factor because, based on the scientific basis for the Ep management objective, an Ep of less than 1.2 is considered protective. One of the conclusions of the SCVURPPP HMP (2005) stream assessment was that the transition between stable and unstable channels occurs between Ep values of 1.0 and 1.2, which translates to a likelihood of instability from 9 to 17 percent, respectively. Or conversely, that 91 percent of channels with an Ep value of 1.0 would be stable and 83 percent of channels with an Ep value of 1.2 would be stable.

Based on modeling analysis performed as part of this study, IMPs sized to a target Ep equal to 1.2 are 9 to 20 percent smaller than IMPs sized for a target Ep equal to 1.0, with an average sizing reduction of 15 percent. *These results suggest IMP sizes could be reduced by 15 percent while still providing a five-sixths probability of the downstream channel remaining stable.* Later in this section, we illustrate how this safety factor can help extend the combinations of D soil percolation rates and incipient motion thresholds for which the recommended sizing factor will apply.

6.2.2 Application of Watershed Scale Build-Out Impervious

The watershed-scale modeling results presented in Section 5.2 demonstrated that in watersheds with protected open space and limited build-out imperviousness, IMP sizing can be reduced. By examining the current and estimated future build-out imperviousness in Contra Costa watersheds, Section 5.2 estimated that IMPs sizing could be reduced by 10 percent, as shown in Figure 5-6. This estimate was based on a 30 percent build-out imperviousness in an upper watershed location, *which is itself a conservative assumption that incorporates a factor of safety*. The projected mean and median build-out percentages among the 15 watersheds examined is 21 and 20 percent, respectively. Only three of the 15 watersheds are expected to surpass 30 percent impervious at build-out.

If we had used a less conservative approach in the watershed-scale analysis and based our IMP scaling on 20 percent build-out impervious for an upper watershed, then the IMP size reduction would have been 15 percent (average columns three and five in Table 5-7). If we had selected 20 percent build-out impervious and lower watershed location, the reduction would have been about 21 percent (see Table 5-6). *These results suggest IMP sizes could be reduced 5 to 11 percent beyond the 10 percent reduction included in Section 5.2*.

6.2.3 Effect of LID Principles on Pervious Project Areas

Contra Costa's C.3 Guidebook encourages site development practices that can reduce site runoff from pervious areas relative to pre-project conditions. These LID measures for site development include soil amendments, self-treating areas, self-retaining areas and grading techniques such as terracing and undulation that slow down water and direct it away from paved areas. No hydrologic modeling has occurred to quantify the aggregate effect of these measures – and they will vary depending on development location, type and density – but incorporating LID principles into development and redevelopment projects is likely to produce some beneficial effects within the watershed, reducing runoff from pervious areas, increasing infiltration losses, and increasing the delivery of base flow to streams.

6.2.4 Safety Factor Summary

The application of Ep and watershed scaling that was incorporated into the IMP sizing sensitivity analysis has an inherent factor of safety. Table 6-1 summarizes the safety factors, as described in the section above.

Table 6-1. Safety Factors and Potential IMP Size Reduction

Safety Factor Component	Potential IMP Size Reduction	Notes
Basing IMP size on Ep = 1.0	15%	SCVURPPP HMP (2005) assessment showed that 83% of streams with Ep = 1.2 are stable
Basing IMP scaling on upper watershed location, 30 percent build-out impervious	5 to 11%	IMP scaling was based on 30% watershed build-out impervious; median value for Contra Costa is only 20%
Effect of LID measures on developed pervious areas	Not explicitly modeled	LID measures: soil amendments, self-treating and self-retaining areas and grading techniques can reduce pervious runoff
TOTAL	**20 to 26%**	

Factors of safety are important in engineering design, because they help ensure a facility will function as required in an environment subject to variability. However, for distributed bioretention and other IMPs, the use of safety factors in decision making is different, because many facilities work together to mitigate site runoff and protect streams. The analysis above suggests IMPs could be reduced in size by 20 to 26 percent and still would be protective of local streams. However, rather than simply reducing IMP sizes, we suggest setting a base sizing factor that is protective under most conditions and then use the safety factor to extend the range of site and watershed conditions for which the base sizing factor is protective.

6.3 Recommended "Base Case" Sizing Factor

Based on the review of the sensitivity analysis and range of values for soil percolation and incipient motion threshold, and the safety factor considerations listed above, *we recommend a base sizing factor of 0.050 for inclusion in the Stormwater C.3 Guidebook*.

Figure 6-1 is a nomograph that shows combinations of soil percolation rates and incipient motion flow thresholds for which Ep < 1.0, along with combinations of lower percolation rates and smaller incipient motion thresholds that should also be protective, based on the safety factor analysis. The figure corresponds to a base sizing factor of 0.050. Any combination of site and watershed conditions that falls below the thick dashed line would result in Ep < 1.0 (shown in darker green shading). This sizing factor should also be protective for any combination of site and watershed conditions above the dashed line and within the lighter green shaded areas that indicate the zone covered by safety factors inherent in the modeling analysis. Examining the graphic clearly shows that items a) and b) in the "approach for sizing distributed stormwater facilities" above are met. Following the graphic, we look at different combinations of percolation rates, incipient motion thresholds and build-out impervious values to verify that Ep < 1.0 for a sufficient variety of conditions so that the aggregate performance of distributed IMPs will produce Ep < 1.0.

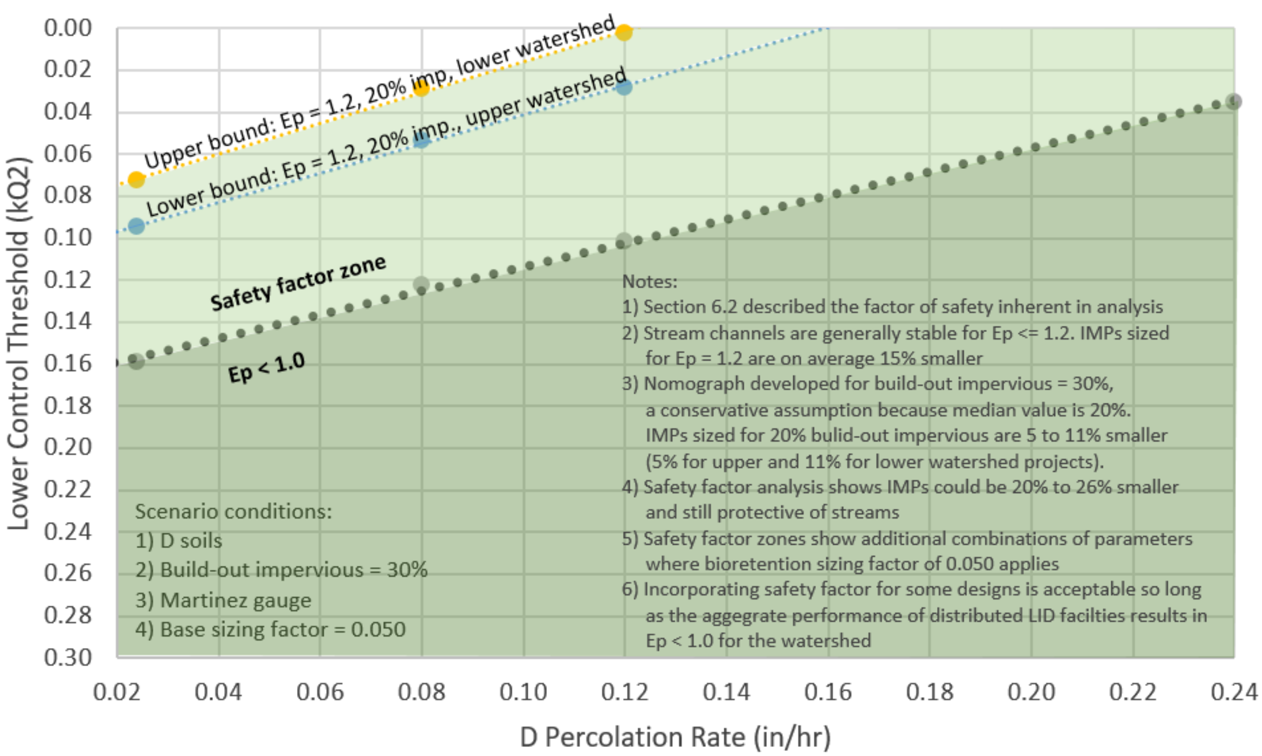


Figure 6-1. Nomograph showing conditions covered by base sizing factor and safety factor

The figure above demonstrates that most and likely all development projects will lie within the Ep < 1.0 zone or within the safety factor zone (meaning base sizing factor of 0.050 + safety factor of 20 to 26 percent). The next question is whether most projects will fall with the Ep < 1.0 zone so that the aggregate performance of IMPs within a watershed will be meet the performance objective.

To develop “real world” examples, we examined each of the 15 fieldwork sites and estimated a) the incipient motion threshold and b) build-out impervious percentage for the watershed. For the incipient motion threshold, we used a critical shear stress value of 0.35 lb/ft2, which Appendix B refers to as “a reasonable value that is generally reflective of the cohesive clay and silt banks observed in the field” and within the range of values observed in the SCVURPPP HMP (2005) and Fairfield-Suisun HMP (FSURMP, 2009). Next, we developed three different D soil percolation rates for each example, corresponding to a) the NRCS design infiltration rate for Group D soils, b) the Ohio bioretention monitoring project average value (Winston, 2016) and c) the 2013 Contra Costa IMP monitoring project value. Table 6-2 summarizes these datasets.

Table 6-2. Site and Watershed Parameters for 15 Fieldwork Sites

Item	Range for Analysis	Notes
Incipient motion threshold	0.03Q2 to 1.28Q2 avg. = 0.11Q2	Based on middle value of shear stress for bank material (0.35 lb/ft2); see Figure 4-2
Watershed build-out impervious percentage	3.5% to 45.9% avg. = 21%	Based on GIS analysis presented in Section 5.2, see Figure 4-3
Soil percolation rates	0.06 to 0.24 in/hr	Based on NRCS design value (0.06 in/hr), Ohio monitoring study results (0.12 in/hr), and 2013 Contra Costa IMP monitoring study results (0.24 in/hr)

Table 6-3 lists the 15 fieldwork sites and indicates whether this combination of site and watershed conditions would produce a) Ep < 1.0, b) Ep within the safety factor zone or c) Ep outside the safety factor zone for the base sizing factor of 0.050 and three D soil percolation rates. Please note, because the build-out impervious also varies in the table the corresponding Ep cannot be directly estimated from the nomograph (unless the build-out impervious for the watershed ~ 30 percent).

Table 6-3. Example Site and Watershed Conditions and Corresponding Ep

Location	Incipient Motion Threshold	Build-Out Impervious	D Soil Percolation Rate	Ep
El Sobrante 15	0.09Q2	18.6%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Martinez 08	0.09Q2	6.3%	0.06 in/hr	Ep < 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Martinez 09	1.28Q2	21.7%	0.06 in/hr	Ep < 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Concord 06	0.10Q2	19.9%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Bay Point 07	0.09Q2	45.9%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Pittsburg 11	0.03Q2	19.9%	0.06 in/hr	Ep > 1.2
			0.12 in/hr	Ep < 1.2 (safety factor zone)
			0.24 in/hr	Ep < 1.2 (safety factor zone)
Moraga 04	0.33Q2	23.6%	0.06 in/hr	Ep < 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Moraga 03	0.05Q2	3.5%	0.06 in/hr	Ep approx. 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0

Table 6-3. Example Site and Watershed Conditions and Corresponding Ep

Location	Incipient Motion Threshold	Build-Out Impervious	D Soil Percolation Rate	Ep
Danville 01	0.07Q2	12.7%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
San Ramon 10	0.11Q2	6.8%	0.06 in/hr	Ep approx. 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
San Ramon 02	0.10Q2	36.1%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.2 (safety factor zone)
			0.24 in/hr	Ep < 1.0
Antioch 12	0.15Q2	43.5%	0.06 in/hr	Ep approx. 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Brentwood 05	0.11Q2	22.9%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Brentwood 14	0.09Q2	20.5%	0.06 in/hr	Ep < 1.2 (safety factor zone)
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0
Brentwood 13	0.12Q2	14.6%	0.06 in/hr	Ep approx. 1.0
			0.12 in/hr	Ep < 1.0
			0.24 in/hr	Ep < 1.0

The table above indicates that distributed IMPs with a sizing factor of 0.050 will in aggregate meet the performance objective. Of the 45 combinations of site conditions (D soil percolation) and watershed conditions (incipient motion threshold, build-out impervious percentage), 34 combinations (76 percent) would produce an Ep value that is approximately equal to or less than 1.0. In many cases, the resulting Ep would be substantially less than 1.0 (although not directly estimated here). An additional 10 combinations (22 percent) would produce an Ep < 1.2, which is within the safety factor zone and would result in stable channels (see Section 6.2.1). Only one of the 45 examples would result in conditions beyond the safety factor and this is due an uncommon combination of geomorphic parameters that resulted in an estimated incipient motion threshold of 0.03Q2 and only applies for the lowest of the D soil percolation rates (and the only rate that was not directly measured in the field).

6.4 Appropriateness of the Selected Sizing, including Factors of Safety

The recommended "base case" sizing factor of 0.050 is selected to explicitly meet the standard in MRP Provision C.3.g. and to achieve reasonable protection of beneficial uses by protecting against downstream erosion.

Reviewing Figure 6-1, both smaller and larger "base case" sizing factors were considered and rejected.

Smaller sizing factors (0.040 or 0.045) were considered and found defensible, but were rejected because:

- These sizing factors would protect a smaller range of potential development site scenarios, particularly those where exceptionally low percolation rates and especially vulnerable downstream channels might coincide (although it is not known if potential development sites where these conditions coincide actually exist).
- Requirements that HM facilities be designed with larger sizing factors (roughly equal to the recommended "base case" sizing factor, as adjusted for rainfall) have been in effect in Contra Costa

for over a decade, indicating that implementing facilities of this size is practically and economically feasible.

A larger sizing factor (0.055) was considered, but rejected because:

- There would be little incremental benefit to using a larger sizing factor in place of the recommended factor. The proportion of development sites where exceptionally low percolation rates and especially vulnerable downstream channels coincide in a way to justify the larger factor is very small—so small that it is possible there will be no future development sites where this would actually occur.
- There are significant environmental impacts associated with construction of bioretention facilities and other IMPs, and these impacts scale with facility size. Impacts include those from sand and gravel mining and from the transport of gravel, sand, and compost to development project sites. Additional impacts are associated with excavation of HM facilities and the placement of imported material.
- Contra Costa's successful implementation of LID and HM depends in large part on a consensus among municipal staff and land development professionals that the requirements and guidance in the Stormwater C.3 Guidebook can be practically implemented on all or nearly all development projects at reasonable cost, and that fulfillment of the requirements enhances project value. Municipal experience shows that a sizing factor of 0.04 (used for treatment-only bioretention facilities) is widely accepted, but that acceptance declines (and resistance increases) as the required facility size increases much beyond that threshold.
- Consideration of uncertainties, and the balance of risks and factors of safety, suggest that actual erosion potential is likely to be less than that represented by the model output, as discussed below.

The following risks were considered in selecting the recommended sizing factor:

- There are uncertainties associated with the distribution of stream incipient motion thresholds, soil percolation rates, and watershed imperviousness at build-out.
- Flaws in design and construction could result in IMPs that perform less effectively than the model indicates.
- Some constructed IMPs could be intentionally or unintentionally removed following construction, and IMPs may not be adequately maintained.
- IMPs could exhibit a systematic decline in percolation rates over future years.

These risks are mitigated, and in some cases outweighed, by implicit factors of safety.

Although there is both natural variation and engineering uncertainty associated with each of the three sensitive variables, the coincidence of extreme values for two or more of these variables will be rare. Further, because LID practice directs the distribution of many small bioretention facilities or other IMPs throughout a development site, the potential underperformance of any one facility presents less risk than would exist in a more centralized HM design.

The hydrologic benefits of LID drainage design are significant and are not fully considered in the calculation of runoff flows and the resulting Ep. Contra Costa permittees have been directing the implementation of LID on development projects for over 12 years. By systematically applying this experience in a process of continuous improvement, they have developed, in the Stormwater C.3 Guidebook, detailed and effective guidance for the planning, design, and construction of LID features and facilities. This guidance includes LID practices such as minimizing site imperviousness, dispersing runoff to vegetated (self-retaining) areas, and terracing slopes, all of which complement the use of IMPs to manage runoff from roofs and pavement. The multiplicity of LID practices and facilities mimics natural hydrology in a more robust and reliable way than can be achieved with non-LID controls.

Among the hydrologic benefits of LID is the slowing of runoff flows, and losses to infiltration, that occur as runoff moves through a complex and varied drainage system toward the point or points of discharge from the site. This is likely to be particularly significant during periods when LID facilities are emptying via orifices that limit flows to a relative trickle.

Contra Costa permittees have opted to standardize designs for LID/HM controls. This facilitates effective plan-checking and construction inspection, greatly reducing errors in design and construction. As required by Provision C.3.h., permittees maintain a database of constructed LID/HM facilities and conduct prioritized inspections of these facilities, with follow-up and adequate authority to ensure any deficiencies in operation and maintenance are addressed timely.

Contra Costa permittees also direct that the bioretention facilities and other LID/HM controls be placed in high-visibility, well-trafficked areas, enhancing the likelihood that they will be well-maintained. Current knowledge of the long-term performance of LID facilities indicates that, as these natural systems mature, their performance—including percolation rates to native soils—does not decline.

7. Conclusion and Next Steps

A “base case” sizing factor of 0.050, applicable to bioretention facilities in HSG “C/D” soils at the Martinez gauge, is recommended for inclusion in the *Stormwater C.3 Guidebook.*

Following acceptance of this recommendation, next steps will include:

- Calculation of sizing factors for the remaining IMPs and other hydrologic soil groups.
- Development of a regression-based equation for determining rainfall adjustment factors.
- Publication of an update of the sizing factors shown and equations referenced in Table 3-6 in the Stormwater C.3 Guidebook, 7th Edition.

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Appendix A: 2013 IMP Monitoring Report

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Contra Costa Clean Water Program

IMP Monitoring Report

IMP Model Calibration and Validation Project

Municipal Regional Permit Attachment C

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IMP Modeling Analysis and Results

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0 · Executive Summary

The Contra Costa Clean Water Program (CCCWP) comprises Contra Costa County and the 19 cities and towns within the County, all of which are Permittees under an NPDES permit issued by the San Francisco Bay Regional Water Quality Control Board (Water Board).

Pursuant to permit provision C.3.g., the Permittees require Hydromodification Management (HM) measures to be implemented on development projects. HM measures are intended to control runoff flows so that they do not exceed pre-project flow rates and durations for a specified range of flows. The requirements apply to projects that create or replace an acre or more of impervious area and increase the total amount of impervious area on the project site.

Criteria for HM measures—including factors for sizing HM facilities, called Integrated Management Practices or IMPs—are incorporated in CCCWP's *Stormwater C.3 Guidebook*. The IMPs include bioretention and variations consisting of bioretention combined with upstream or downstream storage.

The sizing factors were developed using a continuous-simulation computer model. The model uses 30 or more years of hourly local rainfall data and generates corresponding estimates of hourly runoff. Model output is used to compare estimated runoff in the site's pre-development condition to runoff post-development, including incorporation of HM measures. Sizing factors represent the minimum IMP areas and volumes required to fully control runoff flows to match the pre-development condition.

The permit requires CCCWP to implement a model calibration and verification project, which is the subject of this report. The purpose of the project is to determine the flow-control effectiveness of the IMPs. The permit specifies that IMPs at a minimum of five locations be monitored for a minimum of two years and that the observed flows be compared to flows that would be estimated by the model.

Three IMPs (bioretention facilities) at an office building in Pittsburg, and two IMPs (bioretention + downstream vault facilities) at a townhouse development in Walnut Creek, were monitored during the 2011-2012 and 2012-2013 water years. Rainfall data was collected at each location. For the IMPs at the Pittsburg site, the water level in the subsurface storage layer was also continuously monitored.

Results of the comparison show that the IMPs provide considerably greater flow-control effectiveness than predicted by the model. The primary reason is that model inputs underestimated the amount of runoff that would be infiltrated by the IMPs. In addition, it was found that runoff percolated through the IMPs soil/compost planting mix more readily than the model predicted. Following changes to input parameters, including the infiltration rate of underlying soils, the model outputs closely matched observed IMP flows and storage.

Local long-term rainfall records were then input to the calibrated model to analyze how IMPs would perform in comparison to current and potential future permit requirements. The simulation indicates that the IMPs fully control runoff flows between the thresholds specified in the current permit (two-tenths of the 2-year pre-project peak flow, or 0.2Q2, and the 10-year pre-project peak flow, or Q10). The Pittsburg bioretention IMPs also control runoff flows within a range extended to the potential future threshold of one-tenth of the 2-year pre-project peak flow, or 0.1Q2. The Walnut Creek bioretention + vault facilities could control flows within the extended range with minor modifications.

In next steps, CCCWP will work with other Bay Area Permittees, through the Bay Area Stormwater Management Agencies Association (BASMAA), to propose appropriate flow-control criteria and sizing factors to be used during the term of a reissued Regional Municipal Stormwater NPDES permit. Lessons learned with regard to facility design details have already been incorporated into the current 6th edition of the *Stormwater C.3 Guidebook.*

1 · Background: Hydrograph Modification Management

1.1 Permit Definitions and Requirements

Provision C.3.g. in the San Francisco Bay Regional Water Quality Control Board's Municipal Regional Stormwater Permit (MRP), titled "Hydromodification Management" (HM), defines HM projects as those creating or replacing an acre or more of impervious area, subject to various exclusions. Provision C.3.g. requires that:

> The stormwater discharges from HM Projects shall not cause an increase in the erosion potential of the receiving stream over the pre-project (existing) condition. Increases in runoff flow and volume shall be managed so that post-project runoff shall not exceed estimated pre-project rates and durations, where such increased flow and/or volume is likely to cause increased potential for erosion of creek beds and banks, silt pollutant generation, or other adverse impacts on beneficial uses due to increased erosive force.

Specific requirements for design of HM controls are:

> For Alameda, Contra Costa, San Mateo, and Santa Clara Permittees, HM 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. For Fairfield-Suisun Permittees, HM controls shall be designed such that post-project stormwater discharge rates and durations shall match from 20 percent of the 2-year peak flow up to the pre-project 10-year peak flow. Contra Costa Permittees, when using pre-sized and pre-designed Integrated Management Practices (IMPs) per Attachment C of this Order, are not required to meet the low-flow criterion of 10% of the 2-year peak flow. These IMPs are designed to control 20% of the 2-year peak flow. After the Contra Costa Permittees conduct the required monitoring specified in Attachment C, the design of these IMPs will be reviewed.

Nearly identical requirements for new development projects appear in the 2010 East Contra Costa County Municipal NPDES Permit issued by the California Regional Water Quality Control Board for the Central Valley Region.

In the MRP, the referenced Attachment C specifies:

> The Program shall monitor flow from Hydrograph Modification Integrated Management Practices (IMPs) to determine the accuracy of its model inputs and assumptions. Monitoring shall be conducted with the aim of evaluating flow control effectiveness of the IMPs. The Program shall implement monitoring where feasible at future new development projects to gain insight into actual versus predicted rates and durations of flow from IMP overflows and underdrains.
>
> At a minimum, Permittees shall monitor five locations for a minimum of two rainy seasons. If two rainy seasons are not sufficient to collect enough data to determine the accuracy of model inputs and assumptions, monitoring shall continue until such time as adequate data are collected.
>
> Permittees shall conduct the IMP monitoring as described in the IMP Model Calibration and Validation Plan in Section 5 of this Attachment. Monitoring results shall be submitted to the Executive Officer by June 15 of each year following collection of monitoring data. If the first year's data indicate IMPs are not effectively controlling flows as modeled in the HMP, the Executive Officer may require the Program to make adjustments to the IMP sizing factors or design, or otherwise take appropriate corrective action. The Permittees shall submit an IMP Monitoring Report by August 30 of the second year of monitoring. The IMP Monitoring Report shall contain, at a minimum, all the data, graphic output from model runs, and a listing of all model outputs to be adjusted, with full explanation for each. Board staff will review the IMP Monitoring Report and require the Program to make any appropriate changes to the model within a 3-month time frame.

Section 4 of MRP Attachment C states in part:

> Monitoring shall be conducted with the aim of evaluating flow control effectiveness of the IMPs. The IMPs were redesigned in 2008 to meet a low flow criterion of 0.2Q2, not 0.1Q2, which is current HMP standard for Contra Costa County. The Program shall implement monitoring at future new development projects at a minimum of five locations and for a minimum of two rainy seasons to gain insight into actual versus predicted rates and durations of flow from IMP overflows and underdrains. If two rainy seasons are not sufficient to collect enough data to

> determine the accuracy of model inputs and assumptions, monitoring shall continue until such time as adequate data are collected….
>
> ….The principal use of the monitoring data shall be a comparison of predicted to actual flows. The Dischargers shall ensure that the HSPF model is set up as it was to prepare the curves in Attachment 2 of the HMP, with appropriate adjustments for the drainage area of the IMP to be monitored and for the actual sizing and configuration of the IMP. Hourly rainfall data from observed storms shall be input to the model, and the resulting hourly predicted output recorded. Where sub-hourly rainfall data are available, the model shall be run with, and output recorded for, 15-minute time steps.
>
> The Dischargers shall compare predicted hourly outflows to the actual hourly outflows. As more data are gathered, the Dischargers may examine aggregated data to characterize deviations from predicted performance at various storm intensities and durations.
>
> Because high-intensity storms are rare, it will take many years to obtain a suitable number of events to evaluate IMP performance under overflow conditions. Underdrain flows will occur more frequently, but possibly only a few times a year, depending on rainfall and IMP characteristics (e.g., extent to which the IMP is oversized, and actual, rather than predicted, permeability of native soils). However, evaluating a range of rainfall events that do not produce underflow will help demonstrate the effectiveness of the IMP.

Similar, but less detailed, requirements were incorporated into RWQCB Order R2-2006-0050, whereby the San Francisco Bay Regional Water Quality Control Board (Water Board) adopted Contra Costa's HMP in 2006. That Order was superseded by the MRP.

1.2 Hydromodification, Control Methods, and Measurements

1.2.1 Hydromodification and Stream Erosion

The following brief summary of factors affecting stream erosion was included in the HMP Work Plan submitted in November 2004. Subsequent research has upheld these points.

> Contra Costa streams are subject to a myriad of influences, and it is typically difficult, if not impossible,

to generalize regarding causes and effects across the entire County. Further, it is often difficult to attribute any particular observed condition in a specific stream to only one proximate cause. In general, it is necessary to consider many potential causes and to consider their relative significance. For example, Riley (2002) attributes the incision of stream channels in the Bay Area over the past 100 years primarily to climate changes and earth movement, while noting that incision may be induced accelerated by land use change as well.

As an illustration of the interaction of these influences, consider the stream equilibrium equation identified by Lane (1955).

$$(\text{Sediment load} \times \text{sediment size}) \ \alpha \ (\text{slope} \times \text{discharge})$$

A change in any one of these four factors may contribute to disequilibrium (net erosion or deposition stream sediments) and consequent changes in channel width and depth.

- Sediment load may increased by earth movement (e.g., geologic uplift and mass wasting), land disturbance (e.g., agriculture, road construction), or loss of vegetation, or may be decreased by land development (e.g., paving, terracing), by dams, or by dredging.
- Sediment size may be affected by changed balance among different sediment loads (and the erosion of different geologic strata), by dams, or by in-stream mining.
- Stream slopes are often increased by straightening (removal of meanders), or may be increased or decreased by the placement of downstream culverts or grade controls.
- Finally, stream discharge, and particularly rainfall/runoff relationships, may be increased by deforestation, agriculture, and other land use changes, prior to and including urbanization, or may be decreased by dams and diversions.

The above considerations address only system-wide instabilities, those that are in effect over a long reach or series of reaches. Bank erosion at specific sites may be related to the presence or absence of vegetation and to localized channel conditions (e.g., placement or removal of woody debris or riprap upstream or downstream).

1.2.2 Criteria for Control of Runoff Flows from Development Projects

Notwithstanding the complexity of factors affecting stream erosion, and the watershed scale at which those factors interact, California's nine Regional Water Quality Control Boards (Water Boards) have focused on controlling increased flows and durations from individual development sites.

The nine Water Boards have adopted a variety of criteria, using a mix of methodologies and engineering methods, to regulate land development.

Some Water Boards use the estimated peak flow or volume resulting from a specific storm event ("design storm") as a criterion. Examples of "design-storm" based criteria follow:

- No increase in the predevelopment 2-year peak flow (Orange County and the statewide Phase II permit for small municipalities)
- No increase in runoff volume resulting from the 85th percentile storm or 95th percentile storm, depending on development project location (Central Coast Region)
- No increase in 2-year peak flow or peak duration or increase in runoff volume from the 85th percentile storm (North Coast Region)

Criteria required by other Water Boards involve an analysis of rainfall and runoff over 30 years or more. This continuous simulation approach is discussed in Section 2 below. To determine whether the criteria are met, an hourly rainfall record of 30 years or more is used. Hourly runoff volumes are estimated using a continuous-simulation model applicable to the development site. Runoff is simulated in the pre-project condition and in the post-project condition with proposed IMPs or other flow-control facilities.

The pre-project and post-project runoff statistics are compiled to compare the duration of simulated flow at each flow rate, from rare high flows to more frequent low flows.

The post-project flow durations must be equal to or less than the pre-project flow durations for flows within a specified range.

The Water Boards have required different ranges to be used. The basis for setting different ranges is, ostensibly, that different streams have different thresholds of flow at which their beds or banks may be eroded and the resulting sediment transported downstream. However, in fact, the ranges are often applied to all the stream segments on all the streams in a whole city or even an entire county.

The lower limit of the range is more critical to facility design. The lower limit is commonly expressed as a fraction of the 2-year pre-project peak runoff flow (Q2). Here are some low-flow thresholds currently mandated by the various Water Boards:

- Sacramento-area municipalities: 0.25Q2 or 0.45Q2
- San Diego County municipalities: 0.1Q2, 0.3Q2, or 0.5Q2, depending on receiving channel material and dimensions.
- Cities of Fairfield and Suisun City: 0.2Q2
- Santa Clara, Alameda, and San Mateo Counties: 0.1Q2
- Contra Costa County: 0.2Q2 when applied to specified IMPs.

1.3 LID and HM

The California Ocean Protection Council describes Low Impact Development (LID) as a

> … stormwater management strategy aimed at maintaining or restoring the natural hydrologic functions of a site to achieve natural resource protection objectives and fulfill environmental regulatory requirements; LID employs a variety of natural and built features that reduce the rate of runoff, filter pollutants out of runoff, and facilitate the infiltration of water into the ground…
>
> …LID design detains, treats and infiltrates runoff by minimizing impervious area, using pervious pavements and green roofs, dispersing runoff to landscaped areas, and routing runoff to rain gardens, cisterns, swales, and other small-scale facilities distributed throughout a site.

LID was first developed as a comprehensive stormwater management strategy by Prince Georges County (1999). The hydrologic approach is described as follows:

> The LID approach attempts to match the predevelopment condition by compensating for losses of rainfall abstraction through maintenance of infiltration potential, evapotranspiration, and surface storage, as well as increased travel time to reduce rapid concentration of excess runoff.

In essence, LID seeks to address potential hydrologic impacts of land development by maintaining and restoring site characteristics and conditions at the smallest scale possible. Priority is placed on reducing runoff by limiting impervious surfaces, then on dispersing runoff to landscape within a site, and finally by directing runoff to small-scale facilities integrated into the landscape.

In contrast, HM attempts to address hydrologic impacts of land development at a watershed scale. Flow criteria are developed for streams draining the watershed, and those criteria are then translated to criteria for development of sites draining to the watershed. (In the case of the San Francisco Bay Water Board's approach, criteria developed for flows within selected reaches of three streams in Santa Clara County were applied to all Bay Area development sites directly and without further analysis.)

LID promotes a multiplicity of approaches and promotes "green" urban development, while HM specifies that runoff discharges adhere to a specified hydraulic regime.

The HM criteria adopted by the San Francisco Bay Water Board specify the use of flow duration control basins, and require "HM controls shall be designed such that post-project stormwater rates and durations match pre-project discharge rates and durations…." In flow duration control basins, this "match" is achieved through the sizing and placement of orifices draining a basin. Cost-effectiveness and operational considerations favor larger basins (the opposite of LID's small-scale approach). Indeed, the MRP allows compliance through the use of regional-scale flow-duration control basins.

1.4 CCCWP Approach to HM

CCCWP committed to implementing LID beginning in 2003, and published the first edition of the *Stormwater C.3 Guidebook (Guidebook),* emphasizing LID design, in 2004. Faced with the San Francisco Bay Water Board's subsequent emphasis on HM, as opposed to LID, CCCWP sought a way for local developers to meet the HM criteria by using LID. This was accomplished by creating designs for LID IMPs that can also demonstrably meet HM criteria.

CCCWP guidance for HM compliance is incorporated in the *Guidebook.* The *Guidebook* is referenced in stormwater ordinances adopted by each Contra Costa municipality.

The *Guidebook* provides applicants for HM projects the following options for HM compliance. The options also appear in MRP Attachment C:

1. Demonstrate there is no increase in impervious area.
2. Use the HM IMPs in the Guidebook.
3. Use a continuous simulation model and a rainfall record of at least 30 years to show estimated post-project runoff durations and peak flows do not exceed pre-project durations and peak flows.
4. Show that there is a low risk of downstream erosion because all downstream channels are pipes, hardened channels, subject to tidal action, or aggrading, or that a channel restoration project will be constructed that takes the post-project flows into account.

For Option 2, the *Guidebook* incorporates sizing factors that land development engineers may use to determine the minimum required dimensions of a variety of IMPs. The land development engineer divides the development site into discrete Drainage Management Areas (DMAs), determines the amount of equivalent impervious area within each DMA, and uses the *Guidebook* sizing factors to calculate minimum values for the following parameters for an IMP serving that DMA:

- area, **A**
- surface storage volume, **V_1**
- subsurface storage volume **V_2**

See Figure 1-1. The land development engineer then shows how, for each DMA, the IMP meets or exceeds minimum values for each parameter.

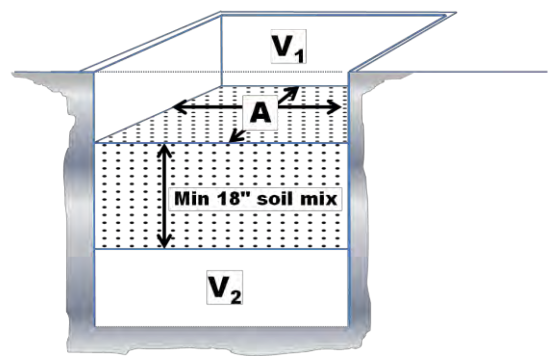

Figure 1-1. A, V_1, and V_2. Note V_2 is the free volume; gravel volume is multiplied by porosity

1.4.1 Bioretention HM Facilities

Bioretention facilities are the most commonly used IMPs on Contra Costa development projects. They are typically constructed for runoff treatment and to maximize retention of runoff via evapotranspiration and infiltration, but the design is adapted to also provide HM. Bioretention facilities work as follows:

Runoff enters the bioretention facility via sheet flow or pipes and is detained in a shallow surface reservoir. The reservoir also serves to spread runoff evenly across the facility surface. Runoff then percolates through an engineered soil (sand/compost mix). Some runoff is retained in soil pores and plant roots and is subsequently evapotranspirated. Runoff that exceeds the moisture-holding capacity of the soil percolates through the soil layer and enters a subsurface storage layer (typically gravel). The treated runoff subsequently then infiltrates into the soils below the facility. If runoff enters the gravel layer more rapidly than it infiltrates, the saturation level in the gravel layer rises until it reaches the discharge elevation for a perforated pipe underdrain. When this occurs, runoff will also discharge through the perforated pipe underdrain to a discharge point (typically connected to the municipal storm drain system). In general, this discharge will occur rarely—a few times per year, or even once in many years.

In facilities constructed for HM, this perforated pipe underdrain is equipped with a flow-limiting orifice. This allows the bioretention facility to act like a flow duration control basin during the infrequent occasions when the storage layer fills, and as a LID facility at other times.

The surface reservoir is also equipped with an overflow that will become active under either of two scenarios: (1) runoff enters the surface reservoir more rapidly than it percolates through the engineered sand/compost mix, and the surface reservoir fills to its maximum volume or (2) runoff enters the facility more rapidly than it leaves via *both* infiltration to the soils below the facility *and* discharge via the underdrain, and this continues until the gravel and soil layers become fully saturated, and the surface reservoir fills to its maximum volume.

In summary, a bioretention facility receives runoff from a specific delineated area, retains that runoff via infiltration and evapotranspiration, and discharges excess runoff via an underdrain and an overflow.

1.4.2 Variations of Bioretention Facilities for HM

The *Guidebook* includes criteria and sizing factors for three design variations:

1. The Flow-through Planter, which can be built above ground or other locations where infiltration to native soils cannot be allowed.

2. Bioretention + Vault, which includes surface storage and engineered soil, but provides for subsurface storage $\mathbf{V_2}$ in a separate structure rather than a subsurface gravel layer.
3. Cistern + Bioretention, which allows for upstream runoff storage $\mathbf{V_1}$ in a tank or basin; runoff is then metered through an orifice to be treated in a bioretention facility.

As described in Section 4, this model calibration and validation project included monitoring of Bioretention + Vault facilities as well as bioretention facilities.

The *Guidebook* also includes design criteria and sizing factors for "direct infiltration" facilities, that is, facilities designed to infiltrate runoff directly, without first routing it through a soil layer to remove pollutants. These design criteria and sizing factors for "direct infiltration" can be used to design infiltration basins, infiltration trenches, and dry wells. This model calibration and validation project did not include "direct infiltration" facilities.

2 · Model Representation of Hydrologic Performance

A project team comprising hydrologists and engineers from Philip Williams & Associates and Brown & Caldwell developed the continuous simulation model that is the subject of this model verification and calibration project. The work was done during 2004-2005. The modeling results formed the basis for the designs and sizing factors proposed in the CCCWP's Hydrograph Modification Management Plan (HMP), submitted to the Water Board in May 2005 and approved by the Water Board, with minor changes, in July 2006.

In 2009, Brown and Caldwell used the same continuous simulation model—with the same input parameters and assumptions—to create sizing factors for new IMP designs. The new IMP designs and sizing factors were incorporated into an addendum to the 4th Edition of the *Stormwater C.3 Guidebook,* and subsequently carried forward through the 5th and 6th (most recent) *Guidebook* edition.

The model was created in HSPF (Hydrologic Simulation Program – Fortran). HSPF has a history going back to the 1960s, has been used and endorsed by USEPA, and has been embraced in many parts of the US for evaluation and design of the hydrologic impacts of new developments. The Western Washington Hydrologic Model (WWHM) consists of an HSPF-based simulation and a user interface, as does the Bay Area Hydrology Model

(BAHM) currently used in Alameda, Santa Clara, and San Mateo Counties. Because HSPF is widely used, there is a significant body of literature and a community of practitioners to support use of the model in HSPF applications.

In HSPF, the various hydrologic processes are represented as flows and storages. Each flow is an outflow from a storage, which, at each time step, is typically a function of the storage volume at that time step and the physical characteristics of the storage. For undeveloped watersheds, HSPF models the movement of water along three paths: overland flow, interflow, and groundwater flow. A variety of storage zones are used to represent storage that occurs on the land surface and in the soil horizons.

The continuous-simulation model was developed and used to demonstrate that, with the inclusion of appropriately sized IMPs in a development project, increases in runoff flow and volume are managed so that post-project runoff does not exceed estimated pre-project rates and durations.

This requires that the model generate representation of pre-project flows at each time step over a long period, as well as post-project flows at each time step during that same period. It is then possible to make statistical cumulative comparisons of the two sets of generated data.

To develop the model, the consultant team:

- Characterized pre-project runoff peaks and durations for a range of soil groups, vegetation, and rainfall patterns characteristic of Contra Costa County development sites.
- Modeled outflow peaks and durations from several IMP designs (based on a unit area of new impervious surface draining to the IMP).
- Compared modeled pre-project flows to modeled post-project-with-IMP flows, using conservative assumptions.
- Developed calculations for sizing factors for each IMP associated with each pre-project condition.

To model the IMPs, the project team constructed representations of each IMP in HSPF. For example, a bioretention facility is represented in HSPF by length, cross-section geometry, layers of soil and underdrain material, and transmissivity of underlying soils.

3 · Model Verification and Calibration Project Design

This project compared model-predicted hydrologic performance to actual hydrologic performance for five facilities at two test sites.

3.1 Steps for Model Verification and Calibration

The experimental design of this project can be summarized as follows:

1. Create a customized version of the HSPF model for each test facility and its corresponding tributary area to continuously simulate inflow, infiltration, evapotranspiration, and underdrain discharge for that test facility. The customized versions use the same values as the 2004-2005 model for soil permeability and bioretention planting soil characteristics, and facility-specific values for the tributary drainage area size and runoff factors and for facility dimensions.
2. Measure rainfall at each test site at each time increment.
3. Input site rainfall data, and use the model to predict, for each time increment, the rates and volumes of inflow, infiltration, evapotranspiration and underdrain discharge for each test facility, as well as storage within each component of the facility.
4. Directly measure the underdrain discharge for each facility at each time increment. (Also, for three of the test facilities, the saturation level in the gravel layer was measured at each time increment.)
5. Compare predicted to measured flows and storage.
6. Adjust the previously assumed model parameter values so that predicted flows and storage more closely approximate measured flows and storage at each time increment (that is, calibrate the model).

3.2 Evaluation of Sizing Factors

The procedure for calculating sizing factors, previously implemented in 2004-2005 and again in 2009, was used with the now-calibrated model to evaluate whether the current sizing factors for bioretention and bioretention + vault facilities are adequate.

Long-term hourly rainfall records from two of the same rain gauges previously used for calculating the sizing factors were input into one of the calibrated site-specific models to examine whether the facility met regulatory criteria.

This procedure was completed for two regulatory scenarios:

1. For a low-flow criterion of 0.2Q2, as specified under the MRP adopted in 2009.
2. For a low-flow criterion of 0.1Q2.

Results are in Section 6.

4 · Project Test Facility Characteristics and Parameters

The CCCWP sought to identify development projects with the following characteristics (Cloak, 2009):

- One or more facilities (bioretention, flow-through planter, bioretention + vault, or cistern + bioretention).
- Facilities must include an underdrain (as required on sites where native soils are in Hydrologic Soil Groups "C" or "D").
- Clearly defined and accurately sized Drainage Management Areas.
- Facilities designed according to the criteria in the Guidebook 4th Edition, including documentation and calculations of minimum and provided bioretention surface area, surface storage volume, diameter of circular orifice, and subsurface storage volume.
- Arrangements/permissions to work with the project contractor and inspector to document and verify construction of the facilities.
- 24-hour access and permission from site owner to access facilities to maintain monitoring equipment.
- Above-ground location to mount a datalogger, rain gauge, and telemetry.

There were five test facilities at two test sites. Three bioretention facilities were monitored at the Pittsburg Fire Prevention Bureau Building, and two bioretention + vault facilities were monitored at Walden Park Commons, a 65-unit townhouse development in Walnut Creek.

4.1 Pittsburg Fire Protection Bureau Building

4.1.1 Site Description

The Pittsburg Fire Protection Bureau Building is located at 2329 Loveridge Road in Pittsburg. Total project site area is 1.09 acres. The site is nearly flat. A single-story building of about 19,000 square feet houses offices of the Contra Costa Fire Protection District. There is an accompanying parking lot with 35 spaces and a trash enclosure. The site includes landscaping around the building, around the perimeter of the site adjacent to Loveridge Road and Loveridge Circle, and in parking medians. The project was constructed during 2011.

As originally designed, the project included a paved overflow parking area. With this area, the total new impervious surface exceeded one acre. The City of Pittsburg required HM compliance for the project. In later revisions to the project scope, the overflow parking area was left graveled rather than paved and the total new impervious area was reduced to 26,457 square feet.

Figure 4-1.
Pittsburg site pre-project

4.1.2 Pre-Project Condition and Site Soils

Figure 4-1 shows the site in its pre-project condition. As can be seen in the photo, the site was previously undeveloped; however, it had been used for parking and perhaps as a construction staging area.

Figure 4-2. Excavation of IMP #2 at Pittsburg Site.

Borings on the site were taken in 2004. According to the report by Kleinfelder (2004), subsurface soils "consisted predominantly of stiff to hard, moderately to highly plastic silty clays, extending to depths ranging from about 4 to 14 feet below existing site grade." This covers the range of depths at the bottom of the bioretention facilities. Surface soils were found to have high shear strength and be highly plastic, as indicated by Atterberg Limits: a Liquid Limit of 59% and a corresponding Plasticity Index of 37. This indicates high expansion potential. The shear strength of the soils is apparent in Figure 4-2.

Boring depths extended as deep as 31 feet, and groundwater was not encountered.

4.1.3 Drainage Management Areas

The Pittsburg Fire Protection Bureau Building design for treatment and HM compliance incorporates eight DMAs.

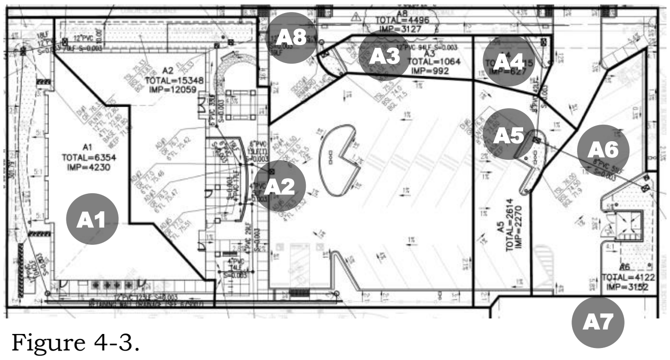

Figure 4-3.
Pittsburg site Drainage Management Areas.

For the model verification project, the completed site was inspected to verify that DMA delineation corresponded to site drainage as built. This included visual verification of the location of rain gutters and downspouts. In addition, the parking lot and grounds were inspected to verify that grade breaks correspond to the DMA boundaries shown in the project plans.

See Figure 4-3 and Table 4-1.

DMA 7 is a self-treating pervious graveled area. DMA 8 consists of driveway and sidewalk areas that could not be made to drain to treatment facilities. The remaining six DMAs each drain to a bioretention facility. Three of these six bioretention facilities were selected to be monitored as part of this project; these are designated as A2, A4, and A6 in Table 4-1.

Table 4-1. Pittsburg Fire Protection Bureau Building Facility Dimensions.

	Tributary Area		**Bioretention Facility Dimensions**						
	Landscaped (SF)	Impervious (SF)	A (SF)	A* (gravel layer)	V_1 (CF)	Surface Depth (in.)	V_2 (CF)	Gravel Depth (in.)	Orifice diameter (In.)
A1	1582	4230	558	558	316	7	379	21	0.51
A2	2415	12059	886	886	874	12	961	33	0.81
A3	0	992	60	72	72	12	72	30	0.21
A4	0	627	67.5	82.5	44	6	44	15	0.17
A5	180	2270	170	195	130	9.5	170	31	0.32
A6	562	3152	340	340	204	6	258	19	0.41

*The gravel layer on some facilities extended beyond the surface dimension due to installation of a curb that extended only to top of the gravel layer.

4.1.4 Design of Bioretention Facilities

Each of the three test bioretention facilities was constructed using the cross section and key features specified in the 4th Edition of the *Guidebook*. Some specifications that were new for the 5th ("MRP") Edition were incorporated. All three facilities have:

- Surface reservoir depth as required for V_1
- 18-inch depth sand/compost mix
- Subsurface reservoir of Class 2 permeable (Caltrans Specification 68-1.025), as required for V_2
- Underdrain of PVC SDR 35 perforated pipe
- Underdrain discharge orifice
- Curb inlets; these are constructed somewhat differently from the standard 12-inch-wide curb cut and consisted of pipe sections in the curb face.
- Outlet structures consisting of 24" × 36" precast catch basins; this larger size was to ensure the instrument technician would be able to enter and access the tipping buckets located where the underdrain discharges to the outlet structure.
- Monitoring wells, composed of a section of 6-inch PVC pipe extending vertically through the soil and gravel layers.

Bioretention facilities A2 and A4 were designed with perimeter walls. Bioretention facility A6 was designed without perimeter walls.

A discharge orifice design was developed for this project; the design was subsequently included in the 5th Edition of the *Guidebook*. The design incorporates a solid PVC pipe extending through the wall of the outlet structure; the pipe is fitted with a threaded cap. The orifice is drilled into the cap. This allows the cap to be removed so that the orifice and pipe can be cleaned if necessary; it also allows the cap to be

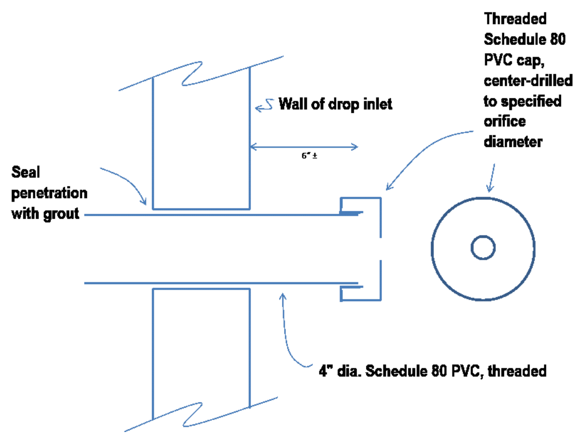

Figure 4-4. Underdrain Orifice Detail.

replaced if the orifice size needs to be adjusted. See Figure 4-4.

As is typical on development sites, the area for some of the IMPs substantially exceeds the minimum. See Table 4-1. This is done for constructability. It is often easier and more cost-effective to build a facility with dimensions that coincide with the available space (such as a parking median) than to build the additional walls and other structure necessary to minimize the size of the IMP.

4.1.5 Construction of Bioretention Facilities

The bioretention facilities were constructed consequent with the construction of the Fire Protection Bureau Building during 2011.

The facilities were constructed generally as designed. The following issues were encountered during construction:

The outfall structures had to be constructed deep enough to fit tipping buckets beneath the underdrain discharge elevation. Because the site is flat, and because the municipal storm drain in Loveridge Road is shallow, there was concern that during storm events flow from the municipal storm drain would back up into the site storm drains and flood the tipping buckets. To address this concern, the most downstream on-site drainage structure (not a bioretention outfall) was fitted with a weir wall and a pump placed on the upstream side with discharge to the downstream (municipal storm drain) side. The pump operated successfully to maintain drainage over the weir wall.

The addition of curbs and widening of curbs for structural stability resulted in reductions to the surface area of each test facility. The reduced areas were noted in updated drawings (and in Table 1) and incorporated into the customized model for each facility.

Following excavation, the native clay soils at the bottom of each bioretention facility were “ripped” using the toothed bucket of the excavator.

4.1.6 Instrumentation

A rain gauge was located on the roof of the trash enclosure.

Each of the three bioretention facilities was equipped with the following measuring devices:

- A tipping bucket, Model TB1L made by Hydrological Services Ltd., located in the facility overflow structure to measure flows discharged through the underdrain orifice

- A piezometer, located in a monitoring well

The instruments were connected to a datalogger on the site via wired connections. Some of the wired connections were strung through the site storm drains—a notable convenience. The datalogger was connected via telemetry to the County Flood Control District's data system.

4.2 Walden Park Commons

Walden Park Commons is a 65-unit multi-family development on a 4.59-acre site fronting Oak Road in Walnut Creek. The site is flat, sloping less than 0.5% away from Oak Road.

4.2.1 Pre-Project Conditions and Site Soils

The site was previously occupied by ten single-family homes with pools, sheds, and associated driveways. These accounted for 74,000 square feet (1.7 acres) of pre-project impervious area.

A geotechnical study of the site (Korbmacher Engineering, 2006) found site soils were native to the site (that is, not fill), and that soils "consisted of a medium stiff to very stiff silty clay and sandy clay." The near-surface soils have moderate expansion potential.

The Korbmacher report indicates groundwater was encountered in borings at a range of 7 to 11 feet below existing grade.

4.2.2 Drainage Management Areas

The applicant was required to ensure all site impervious surfaces drain to LID treatment. The applicant was allowed to size and design bioretention facilities for "treatment only" for new impervious areas equivalent to the pre-project impervious area. For the remainder of the site (corresponding to the increase in impervious area as a result of the project), the applicant was required to provide both treatment and HM control. See the CCCWP's "Guidance on Flow Control for Development Projects on Sites that are Already Partially Developed," (March 2009).

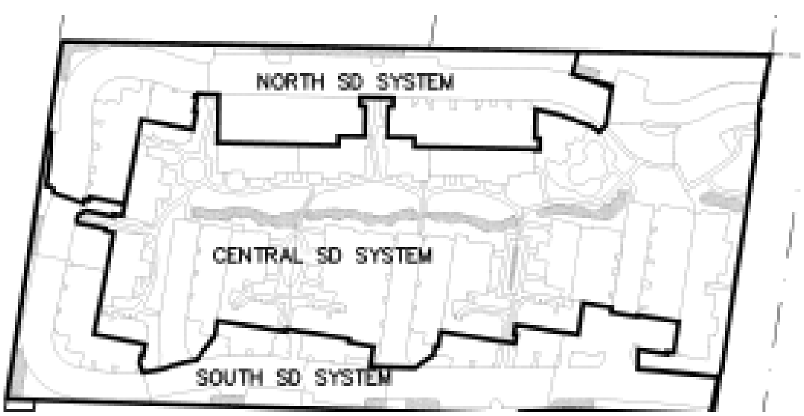

Figure 4-5.
Walden Park Commons Storm Drainage Areas

The site was divided into North, Central, and South areas, with the Central area being routed to treatment-only bioretention facilities. See Figure 4-5. The Central area DMAs and treatment facilities are not considered further in this report.

The North Area is divided into eight DMAs. There are six impervious DMAs totaling 33,301 square feet of impervious roof and driveway, and two landscaped DMAs with 5,948 square feet of pervious area.

The South Area is divided into 19 DMAs. There are 14 impervious DMAs with 36,257 square feet of impervious area, and five landscaped DMAs with 7,495 square feet of pervious area.

All DMAs in the North and South Areas were drained to bioretention facilities. Landscaped DMAs were assigned a runoff factor of 0.7 as specified in the 2005 HMP; that is, landscaped areas were assumed to be 70% impervious. Roofs and paved areas were assumed to be 100% impervious.

4.2.3 Design of Bioretention Facilities

A sizing factor of 0.04 was applied to the resulting equivalent impervious area. Bioretention facilities were sized to exceed this minimum.

Key characteristics of the bioretention facilities are:

- 18 inches of sand/compost mix
- Class 2 permeable drainage layer
- Overflow constructed of vertical ADS pipe, cut to design height
- 6-inch perforated pipe underdrain
- Overflow and underdrain connected to large-diameter storage pipe

The bioretention facilities are located between the site's loop road and the site perimeter fence and are generally configured as linear swales. According to construction drawings, the bottom of the excavation was sloped toward a central line running the length of the swale. The gravel (Class 2 permeable) layer is likewise sloped. The upstream sections do not have underdrains; the most downstream section of the bioretention facilities (near the rear of the development) includes a perforated pipe underdrain See Figure 4-6.

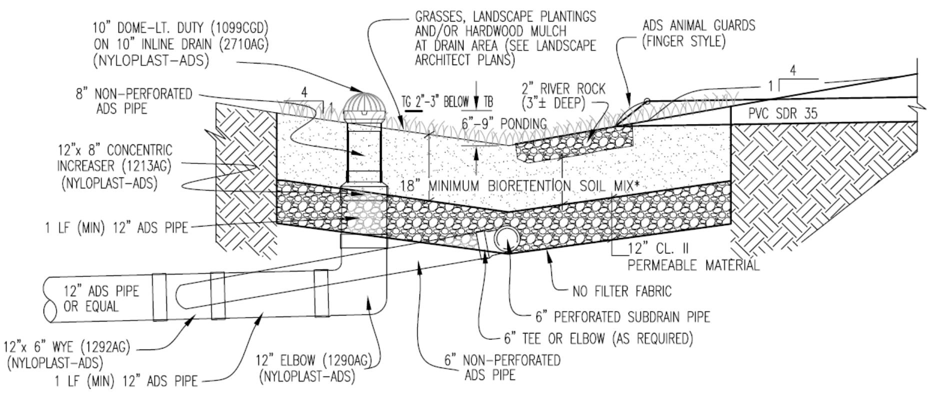

Figure 4-6. Configuration of Walden Park Commons Bioretention Facilities

This configuration allows runoff to infiltrate over much of the bioretention facility area; however, runoff pooling in the gravel layer of the most downstream section will tend to enter the underdrain pipe rather than infiltrate.

4.2.4 Design of Downstream Storage

The underdrain/overflow from the bioretention facilities is routed to common storage facilities—one for the North Area and one for the South Area. The storage consists of reinforced concrete pipe of 30" and 42" diameter set at a slope of 0.005. This information was used to establish stage-storage relationships within the model (See Section 6.)

The concrete pipe storage facility is sealed, preventing exfiltration to the Class A/B backfill material around the pipes and eliminating the opportunity for subsequent infiltration to the native soils around and beneath the storage pipe. This is a significant variance from the design intent for Bioretention + Vault facilities. The *Guidebook* design detail for Bioretention + Vault shows a chamber with an open bottom.

The storage pipes for the North Area and South Area each discharge into concrete vaults at the rear of the development. Each vault is equipped with a weir wall. A pipe through each weir wall conveys metered flows. Each of these pipes is equipped with a PVC pipe and threaded cap. An orifice drilled into the cap meters flows.

Should either of the storage pipes become full, flows would overtop the corresponding weir wall. Downstream of the weir walls, the vaults discharge to the City of Walnut Creek storm drain system.

4.2.5 Construction of Facilities

Drainage facilities were constructed, along with most of the townhouses, during 2011. The following were noted following construction:

Because the bioretention facilities were designed without a hard delineation of their perimeter (that is, they slope seamlessly to surrounding landscaping), it is difficult to visually discern their areal extent. The facilities were surveyed post-construction to confirm the floodable area (that is, the area that lies below the overflow height) corresponded to the areas shown in the descriptions and calculations submitted by the applicant.

Data from the initial storm showed vault outflows began soon after the beginning of a rain event, the facilities were inspected for construction errors that might cause short-circuiting. It was found that the overflow pipe risers had been constructed with perforated pipe, which could have allowed ponded runoff to enter the overflow rather than percolating through the soil/compost mix layer. This was corrected on March 6, 2012.

4.2.6 Instrumentation

Because the bioretention areas were routed to common detention vaults, the total area tributary to the vault is relatively large, and the allowable discharge rate is correspondingly large. To illustrate, the 0.1Q2 discharge from the North and South Areas at Walden Park Commons is 0.07 and 0.08 cfs, respectively, compared with 0.02 cfs for the largest of the bioretention facilities (Facility A2) at the Pittsburg Fire Protection Bureau Building. The larger flow rates allowed the use of electromagnetic flow meters ("magmeters") rather than tipping buckets. Model #EX 81P-40 by Seametrics was selected. The correspondingly larger orifice sizes (over an inch) also helped alleviate concerns about potential orifice clogging.

The magmeters were installed in 1.5" diameter sections of pipe extending upstream of the orifice discharge and through the weir.

The selected magmeter sensors generate a frequency range from 0 – 550Hz over a velocity range of 0.28 – 20 feet per second respectively. This frequency was sampled by the data logger every 15 minutes and velocity was calculated from frequency.

A rain gauge was located in the central courtyard of the development.

The data was transmitted every half hour to a County mountain top repeater and to the office base station where the entire data base is maintained.

5 · Data Collection and Review

Instrumentation and telemetry were established in September 2011 and maintained through May 2013. The instrumentation was operating for all storms during this period. Following is a list of occurrences that affected data collection.

5.1 Exceptions Affecting Data Collection—Pittsburg

5.1.1 Tips When Piezometer Levels Show No Outflow

During each sizable storm, tipping buckets recorded a single tip although piezometer levels indicated the saturation level in the gravel layer had not reached the height of the underdrain. These tips could have been caused by small amounts of runoff entering the underdrain rather than percolating through the unsaturated gravel layer, or by rain falling directly into the tipping bucket.

5.1.2 Data Loss on October 22, 2012

Data for a storm on this date showed very high flows entering the tipping bucket for IMP #2. On examination of the data, it was determined that the recorded flows were outside the range of the tipping buckets ability to record. On further investigation, it was determined that moisture had caused wired connections between the tipping buckets and the datalogger to short-circuit. The wired connections were insulated with silicone rubber sealer. The erroneous data was taken out of the data base at that time.

5.2 Exceptions Affecting Data Collection—Walden Park

5.2.1 Construction Error on Overflow Risers

As noted above, a construction error may have allowed short-circuiting of flows during storms prior to March 6, 2012.

5.2.2 Cut-out at High Flows

It was noted that data for some events showed flows rising following the onset of rain, suddenly dropping to zero, and then resuming with a falling limb as the storage pipe drained. On investigation it was determined the most likely cause was turbulent flow within the discharge pipe.

As a backup method of measuring flows, on January 17, 2013 level sensors were installed in the discharge vault. Also at this time two feet of linear pipe was installed upstream of the magmeters. It was planned to correlate the water levels and measured flows to establish a rating curve and to use the rating curve to estimate flows during intervals when the flow sensor was not registering. However, there were not enough subsequent storms to establish the rating curve, and no subsequent flows were high enough to cause recurrence of the problem.

5.3 Data Review and Consistency Check

Data were reviewed for internal consistency and consistency with expectations and visual observations. The following were noted:

- Rainfall data was consistent with observed events and other rain gauge data collected by the District.
- Saturation levels in the Pittsburg bioretention facilities rose to relative levels consistent with rainfall depths and with facility sizing.
- Discharge measured at the Walden Park facilities was recorded at relative flows consistent with rainfall intensity and depths.

In summary, the data collected covered most but not all storm events during the monitoring period. In addition, the 2-year monitoring period corresponded to a time of relatively few rainfall events, and smaller rainfall events, compared with long-term averages. There were no events intense enough to cause overflow of bioretention facility surface reservoirs at either site, or with enough intensity and volume to cause underdrain discharge at the Pittsburg facilities.

However, the data collected are sufficient for comparison of facility performance with the performance predicted by the model. See Section 6.

6 · Analysis and Results

This section describes the modeling and data analysis methods that were used together to characterize the performance of the Pittsburg and Walden Park Commons IMPs. This section contains the following details:

- Evaluation of rain gauge data for the monitoring period and a comparison of monitored storm events to long-term rainfall statistics for the area.
- Evaluation of IMP monitoring data and the potential implications of the hydraulic characteristics on long-term IMP performance.
- Comparison of HMP model results and IMP monitoring data.
- Description of model parameter adjustments to produce closer agreement between the model outputs and IMP monitoring data.
- Discussion of the current IMP sizing factors and their adequacy for meeting the NPDES permit's flow duration control standard.

Additional modeling and analysis details are contained in Appendix A.

6.1 Comparison of Simulated and Recorded Data

6.1.1 Storm Characteristics

Rainfall accumulations for the 2011-12 and 2012-13 monitoring periods were examined to determine how the monitoring period compares to long-term trends in the Pittsburg and Walnut Creek areas. The purpose of this analysis was to assess whether the monitored storms are representative for the area and whether the storms produced enough rain to adequately characterize the long-term performance of the IMPs at the Fire Prevention Bureau Building in Pittsburg and Walden Park Commons in Walnut Creek.

For the Pittsburg site, the closest rain gauge with a long-term record is Los Medanos, which is located between Pittsburg and Antioch. For Walnut Creek, the closest representative rainfall gauge with a long-term record is the FCD11 gauge located in Martinez.

Table 6-1 shows the seasonal rainfall totals at each project rain gauge and the long-term seasonal averages at the Los Medanos and Martinez gauges. At Pittsburg, the total rainfall was 13 percent below average for the first monitoring season and about average for the second season. At Walden Park Commons, the rainfall was 5 percent below average for the first monitoring season and 24 percent below average for the second season.

Table 6-1. Seasonal Rainfall Totals				
Pittsburg Fire Prevention Bureau				
Season	Dates	Project Site Rainfall (in)	Los Medanos Avg. Rainfall (in)	Difference
1	Oct-2011 – Apr-2012	6.84	7.85	-13%
2	Sept-2012 – May-2013	8.14	8.20	-1%
Walden Park Commons				
Season	Dates	Project Site Rainfall (in)	Martinez Avg. Rainfall (in)	Difference
1	Nov-2011 – Apr-2012	17.19	18.05	-5%
2	Sept-2012 – May-2013	14.69	19.31	-24%

Even though the total rainfall was less than average over the monitoring period, there were several significant events during each season. Table 6-2 and Table 6-3 list the 10 and 13 largest rainfall events that were recorded during the monitoring period at the Fire Prevention Bureau and Walden Park Commons, respectively. The Walden Park Commons list was expanded to capture three events for which both outflow rates and storage pipe levels were recorded. The "recurrence" column in the two tables refers to how often a storm of similar magnitude would be expected to occur, based on the long-term rainfall data. Depth-duration-frequency curves were developed for the Los Medanos and Martinez sites for this analysis.

Table 6-2. Pittsburg Fire Prevention Bureau Site Storm Events			
Start Date	Duration (hours)	Total (in)	Recurrence (12-hr)
1/19/2012	90	1.45	3-month
3/15/2012	49	0.66	3-month
3/24/2012	13	0.65	3-month
4/12/2012	40	1.20	3-month
10/22/2012	26	0.51	<3-month
11/21/2012	9	0.45	<3-month
11/28/2012	56	1.64	2-year
12/1/2012	17	1.12	1-year
12/21/2012	46	1.00	3-month
12/25/2012	14	0.50	3-month

Table 6-3. Walden Park Commons Site Storm Events			
Start Date	Duration (hours)	Total (in)	Recurrence (12-hr)
1/19/2012	95	3.51	1-year
2/29/2012	36	1.01	<3-month
3/13/2012	109	2.59	3-month
3/24/2012	17	1.03	3-month
3/27/2012	16	0.89	<3-month
4/10/2012	79	2.81	3-month
11/20/2012	11	0.92	3-month
11/29/2012	69	4.64	2-year
12/21/2012	69	2.32	3-month
12/25/2012	24	0.79	<3-month
2/19/2013	9	0.34	<3-month
3/30/2013	36	0.76	<3-month
4/4/2013	8	0.29	<3-month

The number of significant storm events during the monitoring period is very consistent with the long-term local rainfall record. For example, there were 8 events that exceeded the 3-month recurrence (for 12-hour rainfall accumulations) at the Fire Prevention Bureau site and 7 events surpassing this threshold at the Walden Park Commons site. This is important, because 3-month storm events would be expected to produce flow rates that approach the lower control threshold flow rate in the County's current NPDES permit (two-tenths of the two-year flow rate, or 0.2Q2). Additionally, the Fire Prevention Bureau and Walden Park Commons sites both experienced 2 rainfall events that were larger than the 1-year (12-hour) storm. In conclusion, the monitoring period included enough storms across a range of intensities and total accumulations to adequately demonstrate how the IMPs perform.

6.1.2 Observed IMP Performance Characteristics

For each significant storm event, IMP monitoring data were examined to better understand the following soil hydraulics and performance characteristics:

1. Percolation of stormwater from the ponding layer through the bioretention soils into the storage layer
2. Infiltration of treated stormwater from the storage layer to the surrounding soils (note: this applies only to the Fire Prevention Bureau bioretention IMPs)
3. Performance of storage layer and frequency of underdrain discharges
4. Any evidence of performance problems

Percolation Characteristics

At the Fire Prevention Bureau site, a slotted-standpipe monitoring well was installed within the gravel storage layer of each monitored IMP. At the Walden Park Commons site, water levels were monitored in the vaults at the downstream end of the storage pipes. The IMP percolation characteristics were examined by comparing the timing and volume of rainfall to the appearance of water within the storage layer at each IMP.

The monitoring data shows that percolation begins after relatively modest levels of rainfall. In the 2004-2005 HSPF model, bioretention soils were modeled using the van Genuchten relationship for water retention. This relationship dictates that percolation rates in sandy-loamy soils would be minimal until the soil reached about three-quarters saturation. However, water appeared in the gravel layer before that volume was reached.

Similar runoff and percolation characteristics were observed at the Fire Prevention Bureau and Walden Park Commons IMPs. The bioretention soils are faster-draining than we expected when creating HSPF models for the HMP.

Figure 6-1 shows an example percolation response for the March 16-18, 2012 storm event at IMP #2 at the Fire Prevention Bureau. The observed depths in the gravel storage layer begin to climb after the first 0.07 inches of rainfall. Based on the tributary area and our initial assumptions about the soil's water retention characteristics, we expected this initial runoff to be fully absorbed within the bioretention soils, filling the available pore spaces like water fills the void spaces in a sponge.

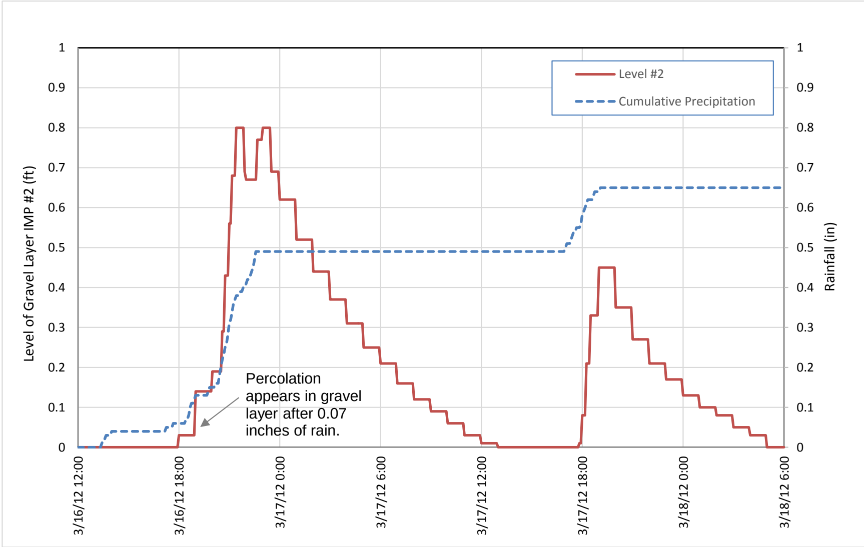

Figure 6-1. Percolation and infiltration, Fire Prevention Bureau IMP #2.

In general percolation in IMP #2 occurred after 0.07 to 0.16 inches of rain, except during an extended wet period from late-November through December 2012 when soils remained wet between storms and percolation began almost immediately after the start of a rain event. In IMP #6 percolation started later in storm events, usually after 0.3 to 0.8 inches of rain (Figure 6-2). IMP #4 is much smaller than the other IMPs and is about two-thirds larger the necessary, based on the HMP sizing factors. IMP #4 did not produce a consistent response to rainfall.

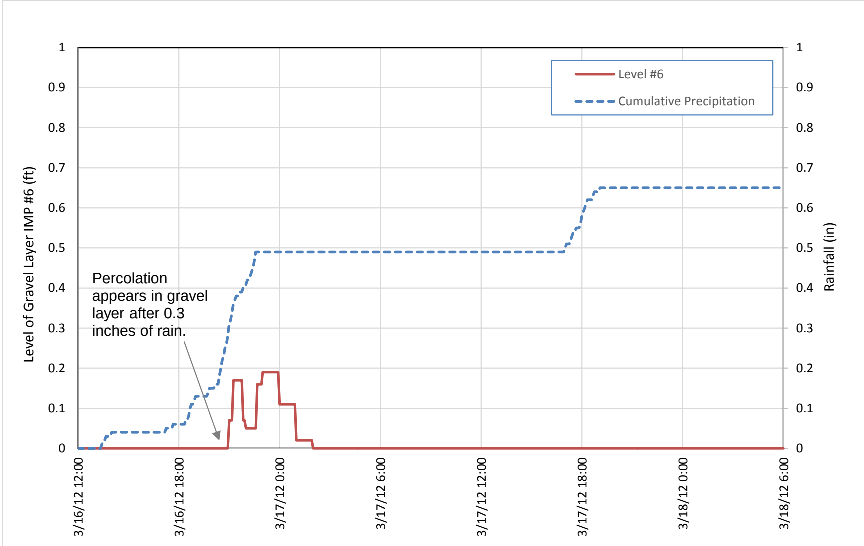

Figure 6-2. Percolation and Infiltration, Fire Prevention Bureau IMP #6.

The reasons for the different response times at IMP #2 and IMP #6 were evaluated. The large roof area adjacent to IMP #2 discharges water via three downspout connections. This water may be saturating the soils in the immediate vicinity of the downspouts and generating percolation to the gravel layer without wetting other portions of the bioretention facility.

Conversely, IMP #6 spreads inflows more broadly and provides a larger soil volume to capture stormwater runoff.

At Walden Park Commons stormwater quickly appears in the storage layer soon after rainfall begins. Figure 6-3 shows accumulated rainfall and IMP outflow for an April 2012 storm event at IMP #1 (North). The storage pipe has received enough percolation to produce outflow after 0.1 inches of rainfall is recorded.

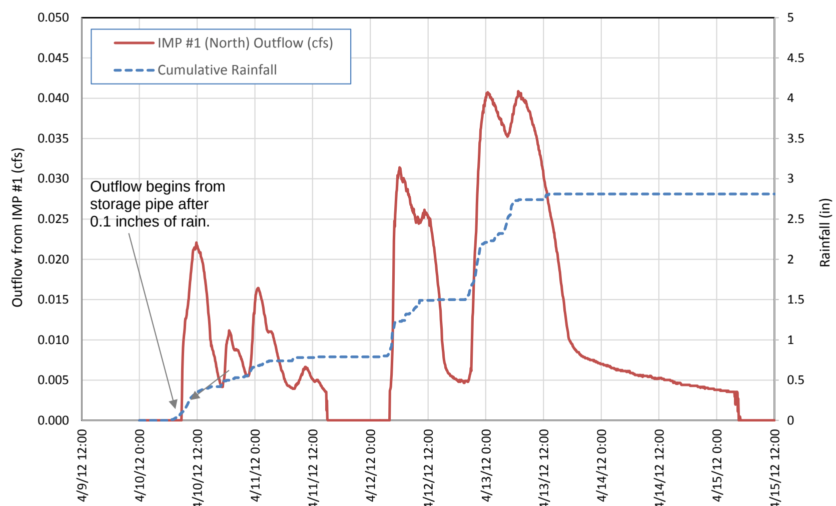

Figure 6-3. Stormwater appears in storage pipe shortly after rain begins IMP #1 (North).

Figure 6-4 shows the start of percolation at IMP #2 (South). The percolation starts later in IMP #2 (South) because a) bioretention area is larger and b) more of the tributary area contains pervious surfaces. The relative responses at IMP #1 (North) and IMP #2 (South) are similar for other storm events.

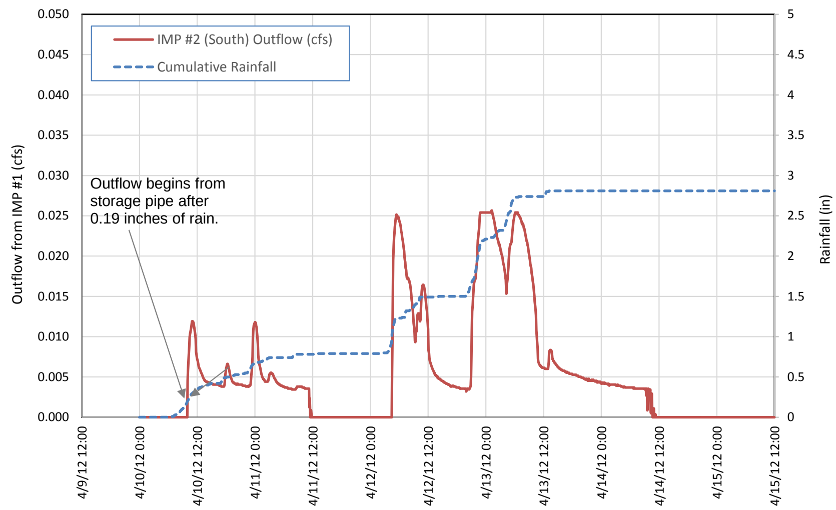

Figure 6-4. At IMP #2 (South) stormwater runoff appears in storage pipe more slowly than in IMP #1 (North)

In conclusion, the bioretention soils appear to allow percolation at lower soil moisture content levels than we expected when preparing the HMP. The effect is less pronounced in over-sized bioretention installations, such as Fire Prevention Bureau IMP #6 and Walden Park Commons IMP #2 (South). This characteristic will probably have a negligible effect on IMP performance. One potential benefit of the fast-percolating soils is the reduced likelihood stormwater building up in the ponding layer and spilling into the overflow in response to high-intensity rainfall.

Infiltration Characteristics

The infiltration characteristics of the surrounding soils were first evaluated at the Fire Prevention Bureau site, where the IMP gravel layers discharge directly to the surrounding soils. Figure 6-5 shows the recorded water levels in the storage layer at Fire Prevention Bureau IMP #2 for the November 28-30, 2012 storm event. Figure 6-6 shows the same storm event at IMP #6.

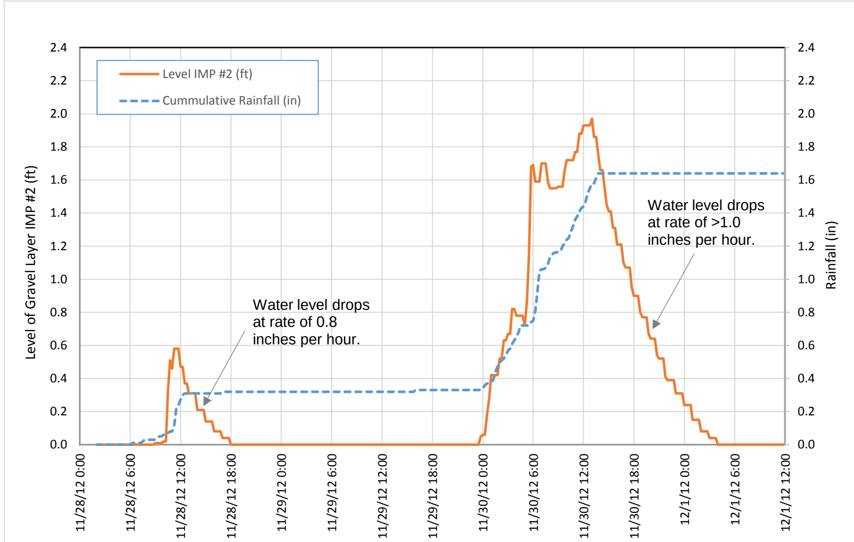

Figure 6-5. Storm recession rates at Pittsburg Site37 IMP #2

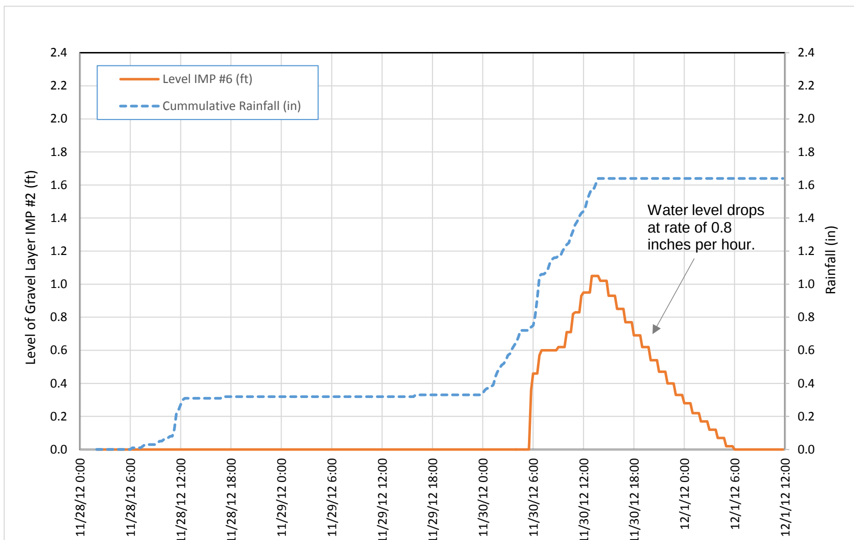

Figure 6-6. Storm recession rates at Fire Prevention Bureau IMP #6

After the rain stops, the water level in the storage layer decreases quickly—at a rate between 0.8 inches per hour and more than 1 inch per hour. Several storm events were examined and while the rate varied by storm in all cases the recession rate was higher than expected for NRCS Group D soils. Even late in the winter season, there was no noticeable groundwater mounding-related reduction in infiltration capacity. The Fire Prevention Bureau infiltration rates surpass the assumed rate of 0.024 inches per hour used in the 2004-2005 HSPF model.

In conclusion, soils at the Fire Prevention Bureau infiltrate runoff more rapidly than the reference values for NRCS Group D soils. IMPs at this site will provide a higher overall onsite stormwater capture fraction than previously expected. These IMPs should also provide a higher level of performance relative to the NPDES permit's flow duration performance standard.

The native soil characteristics for the Walden Park Commons site were indirectly evaluated using a combination of monitoring data and modeling (see Section 6.1.3).

Storage Layer and Underdrain Performance

The Fire Prevention Bureau monitoring data for IMP #2, IMP #4 and IMP #6 were also examined to determine a) how often the flow monitoring equipment registered underdrain discharge, and b) whether these discharges were caused by the filling of the gravel layer.

The items below describe the monitoring data results, which are also summarized in Table 6-4.

- IMP #2: Small underdrain discharges were recorded at 10 separate days over the 20 month monitoring period. The total volume of these discharges was less than 3 cubic feet. None of the discharges lasted more than 15 minutes and only four occurred during the 10 largest rainfall events. In all cases the corresponding water depth did not reach the level of the discharge pipe. The mostly likely reasons for the underdrain discharge are that a small amount of water migrated into the underdrain pipe as it was descending into the gravel layer, and/or that rain fell directly into the tipping bucket.
- IMP #4: Small underdrain discharges were recorded on 16 separate days with the total discharge over 20 months of 4.4 cubic feet. Similar to IMP 2, the discharge volumes are very small and not continuous. The observed water level in the gravel layer never reached the elevation of the under-drain pipe.
- IMP #6: Small underdrain discharges were recorded on 21 separate days with the total discharge over 21 months of 6.6 cubic feet. Similar to IMP 2 and IMP 4, the discharge volumes are very small and not continuous. The observed water level in the gravel layer never reached the elevation of the underdrain pipe.

Table 6-4. Pittsburg Fire Prevention Bureau Monitored Discharge Events			
IMP	Number of Underdrain Discharge Events*	Number of Events Due to Filling of Underdrain Layer	Total Volume
IMP #2	10	0	2.7 ft^3
IMP #4	16	0	4.4 ft^3
IMP #6	21	0	6.6 ft^3

*These discharge events each produced a small volume of water and were most likely due to the migration of water into the underdrain pipe as the water descended into the gravel layer, and/or rain falling directly into the tipping bucket.

<u>Evidence of IMP Performance Issues</u>

No significant or systematic IMP performance issues were evident from the monitoring data or from anecdotal observations during storm events. As noted in Section 5, the overflow risers in the bioretention facilities at Walden Park Commons were installed using perforated pipe, rather than the specified solid pipe. This allowed an unknown portion of stormwater flow to bypass the bioretention treatment. The contractor for the Walden Park Commons project corrected the problem on March 6, 2012.

<u>Summary of Observed IMP Performance</u>

The IMPs at the Pittsburg Fire Prevention Bureau Building and Walden Park Commons successfully captured, treated, detained, and slowly discharged stormwater from all storms during the two-year monitoring period. There were no overflows or significant performance issues.

The infiltration capacity of the native soils at the Pittsburg site will provide a higher level of onsite stormwater control and should allow these IMPs to surpass the flow control requirements of the NPDES permit. Additionally, the bioretention soils allow for faster percolation than was assumed when preparing the HMP. While this difference is not likely to affect the IMP sizing factors, it will protect the system from overflows during periods of very intense rainfall.

6.1.3 Comparison of Model Predictions to Measured Results

Model predictions and monitoring data (primarily water level) were compared for the 10 largest storm events during the 20-month monitoring period at the Fire Prevention Bureau (see Table 6-2 above for list of events).

Figure 6-7 shows an example comparison for Fire Prevention Bureau Building IMP #2 for the April 10-14, 2012 storm event.

Figure 6-8 shows the same storm event for IMP #6. As expected from the monitoring data review, the models do not produce early-storm percolation to the gravel storage layer that was observed in the monitoring data. The models also allow water to remain in both IMP layers for longer periods, which will make the Pittsburg site's model simulations overstate the site's sensitivity to back-to-back storms.

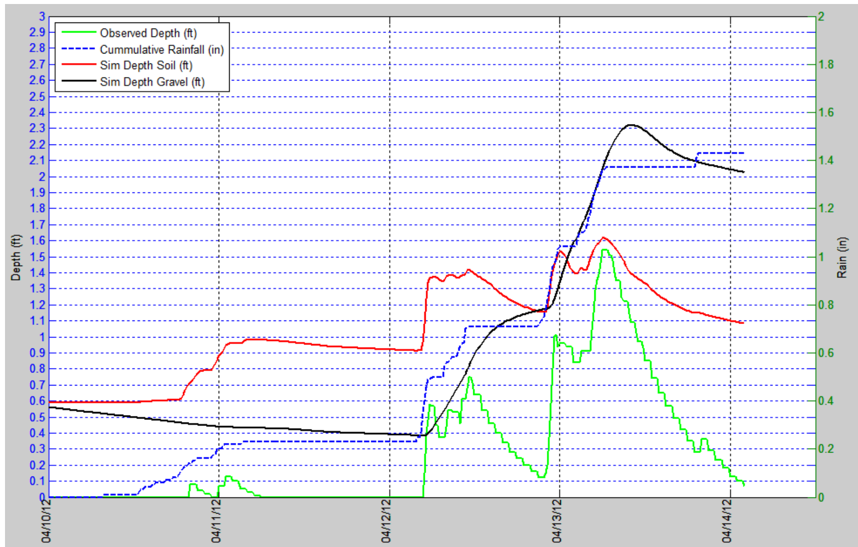

Figure 6-7. Model output and monitoring data comparison at IMP #2 from 4/10/12 to 4/14/12

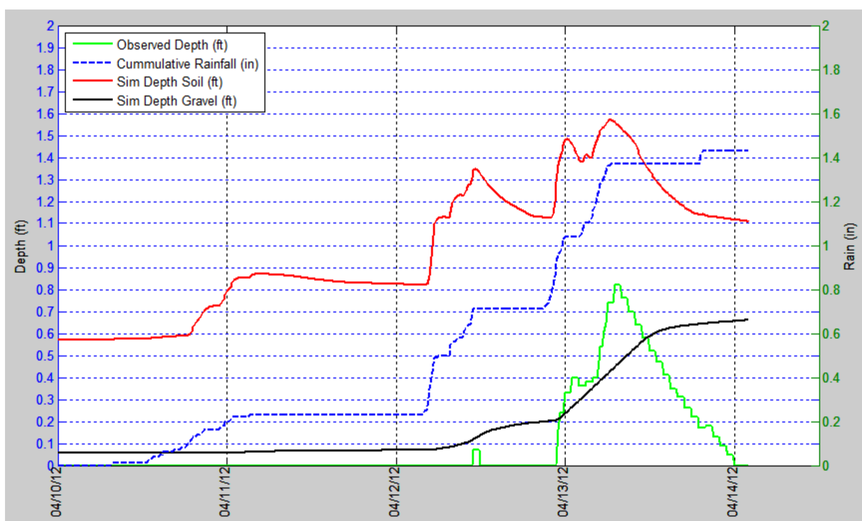

Figure 6-8. Model output and monitoring data comparison at IMP #6 from 4/10/12 to 4/14/12

The number of simulated and observed underdrain discharge events was also compared for IMP #2, IMP #4, and IMP #6. The

HSPF model predicts more frequent discharges through the underdrain pipe. Table 6-5 summarizes the model results.

Table 6-5. Pittsburg Fire Prevention Bureau Model Discharge Events			
IMP	Number of Underdrain Discharge Events	Total Volume	Notes
IMP #2	6	2,700 ft3	Each event lasts several hours
IMP #4	0	0 ft3	
IMP #6	2	87 ft3	Each event lasts several hours

At the Walden Park Commons site, there were a limited number of storms with water level data, but flow rates were recorded through both monitoring seasons. Therefore the simulated and observed outflow volumes were compared for the 13 largest rainfall events during the monitoring period. Figure 6-9 and Figure 6-10 show example results for two separate storm events for IMP #1 (North), which is located in the northwest corner of the Walden Park Commons development. Similar to the initial Fire Prevention Bureau comparison, the monitoring data shows a faster percolation response in the IMP. The model simulation produces higher outflow volumes than were measured.

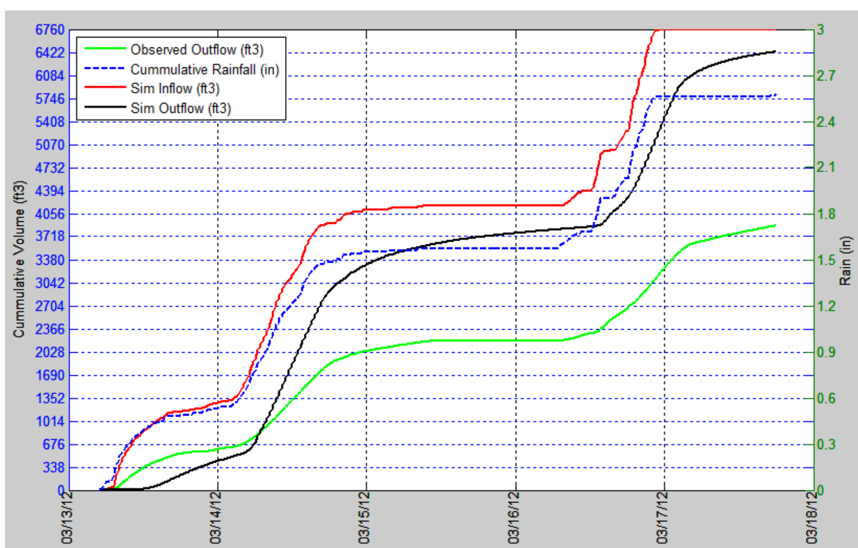

Figure 6-9. Model output and monitoring data comparison at Walden Park Commons IMP #1 (North) from 3/13/12 to 3/18/12

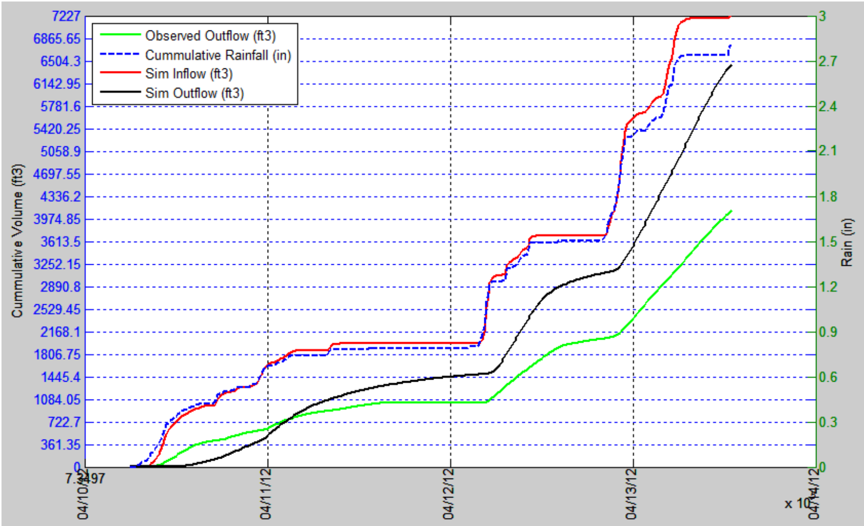

Figure 6-10. Model output and monitoring data comparison at Walden Park Commons IMP #1 (North) from 4/10/12 to 4/14/12

Figure 6-11 and Figure 6-12 compare the simulated and measured cumulative outflow volume for Walden Park Commons IMP #2 (South) for March and April 2012 storm events. The results of the comparison are similar to results for IMP #1 (North). The model simulation produces larger outflow volumes than were observed in the monitoring data.

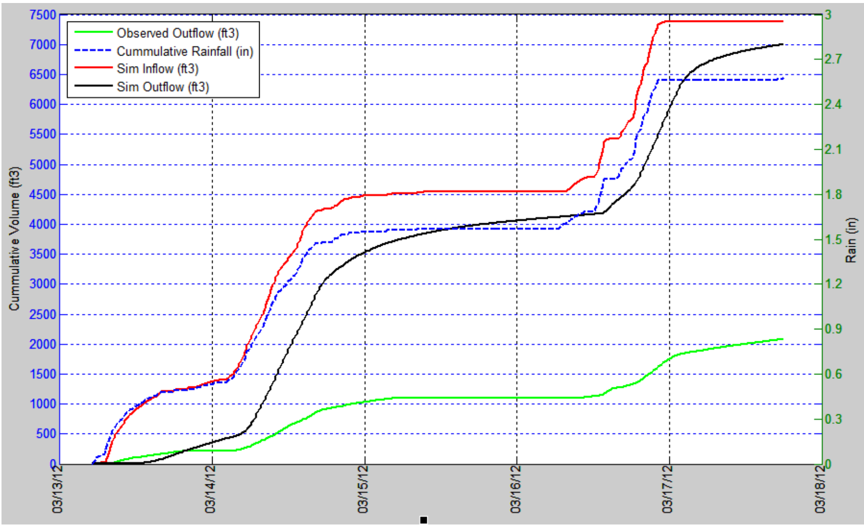

Figure 6-11. Model output and monitoring data comparison at Walden Park Commons IMP #2 (South) from 3/13/12 to 3/18/12

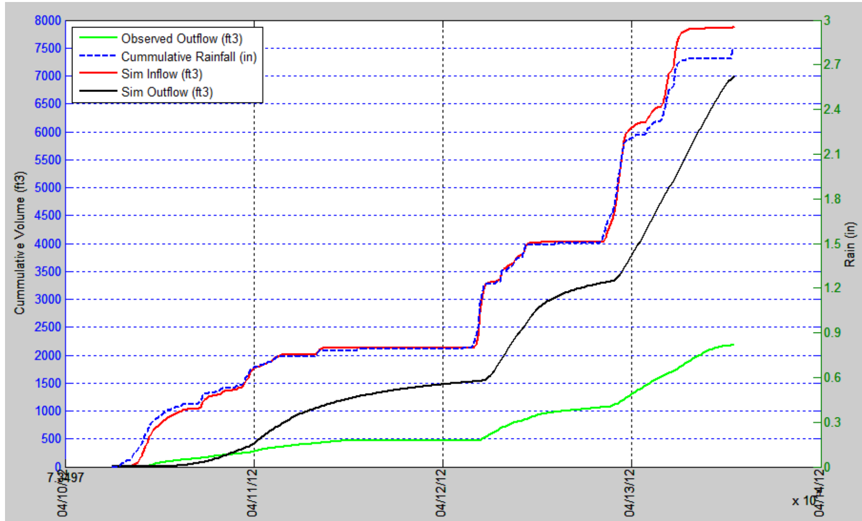

Figure 6-12. Model output and monitoring data comparison at Walden Park Commons IMP #2 (South) from 4/10/12 to 4/14/12

6.2 Adjustment of Model Parameter Values

To reduce the simulated IMP outflow and better match the monitoring data, the infiltration characteristics of each IMP were adjusted. The initial effort focused on the following revisions to Walden Park Commons IMP #1 (North):

1. The relationship between soil moisture and percolation in the bioretention soil was modified to allow percolation to begin soon after water enters the soil. The previous version of the HPSF model held back most percolation until the moisture content reached about 80 percent of saturation.
2. A zone of influence was established around the bioretention layer's underdrain. Because the monitored outflow was significantly less than the estimated inflow to the IMP, we assumed a portion of the stormwater entering the bioretention portion of IMP #1 (North) was infiltrating to surrounding soils. Similar losses to infiltration were evident in the data for IMP #2 (South).

The zone of influence value was iteratively modified until the IMP outflow volume better matched the monitoring data across a range of storm events. Figure 6-13 and Figure 6-14 show the updated results for the same two storm events included in the previous section (see Figures 6-9 and 6-10). For the zone of influence value selected, the simulated outflow volume closely matches the monitored outflow volume. For this value, 60 percent of the bioretention area drains to the underdrain and storage pipe, and the remainder infiltrates runoff to the underlying soils.

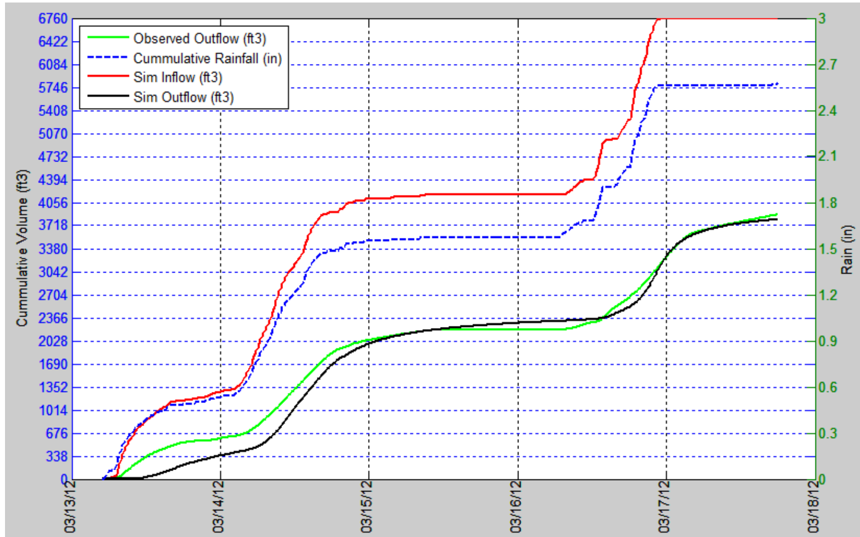

Figure 6-13. Updated model output and monitoring data comparison at Walden Park Commons IMP #1 (North) from 3/13/12 to 3/18/12

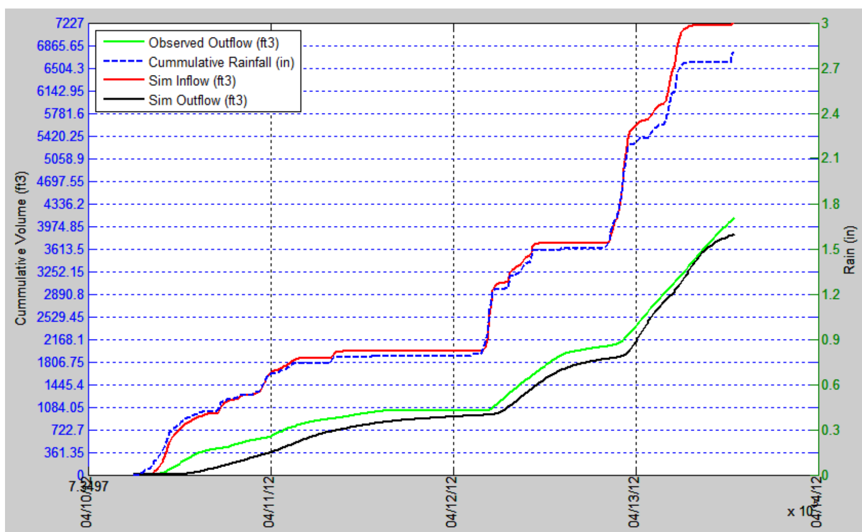

Figure 6-14. Updated model output and monitoring data comparison at Walden Park Commons IMP #1 (North) from 4/10/12 to 4/14/12

The model was also calibrated to match the response of IMP #6 at the Fire Prevention Bureau. The IMP model parameters were adjusted to a) represent the capacity of the bioretention soils to hold water prior to start of percolation, b) mimic the rapid percolation that occurs once the soil moisture threshold is met, and c) approximate the rate at which water drops in the gravel layer by adjusting the infiltration rate to surrounding soils. This parameter also affects the simulated water level in the gravel layer during storm events.

Figure 6-15 shows an example of the calibrated model's response for the November 28, 2012 storm event. This was the largest event during the monitoring period and represents about a 2-year storm for the Pittsburg area. During the initial stages of the storm the simulated water moisture content rapidly accumulates in the bioretention soil while very little water appears in the gravel layer. When the second phase of the storm occurs, percolation occurs rapidly and the gravel layer fills with more than 1 foot of water (note: the underdrain is located about 2½ feet above the bottom of the gravel layer). The simulated maximum depth matches the monitored maximum depth to within 1 inch. The simulated gravel water level recession is a little more rapid than the monitored recession. In general, the simulated and observed recession rates are similar across the range of storm events.

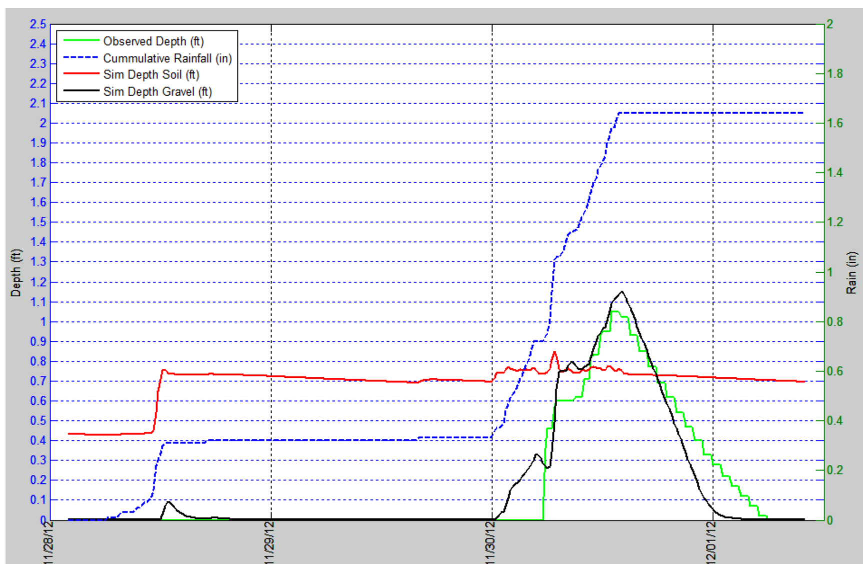

Figure 6-15. Updated model output and monitoring data comparison at Fire Prevention Bureau IMP #6 from 11/28/12 to 12/1/12

Figure 6-16 shows calibration results for a smaller storm event that occurred on March 25, 2012. This 0.65-inch event has about a 3-month (12-hour) recurrence interval. Similar to the larger event shown above, the initial rainfall is captured and held within the bioretention soils. Once the soil moisture threshold is met, stormwater percolates to the gravel layer. The simulated and monitored water levels match precisely and recession rates also agree very closely. There is an approximately one-hour offset between the simulated and monitored peak water levels, which will have no impact on the ability of the model to predict long-term IMP performance.

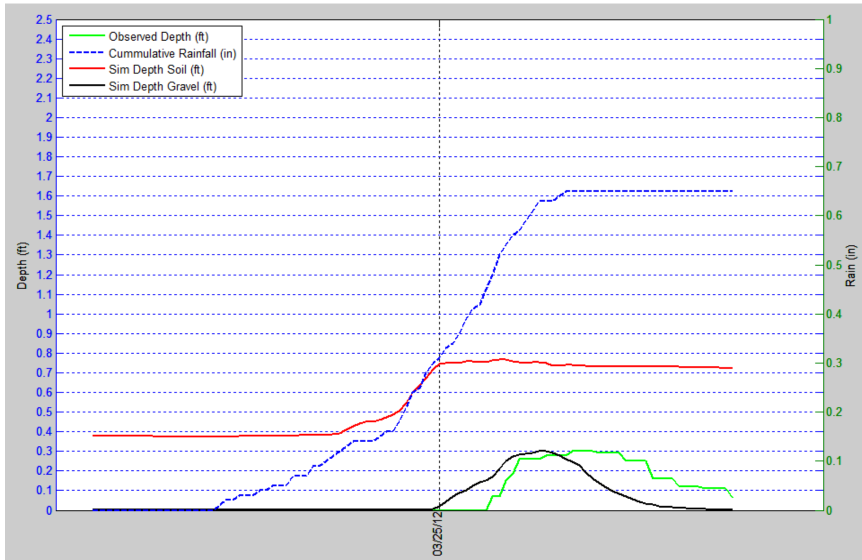

Figure 6-16. Updated model output and monitoring data comparison at Fire Prevention Bureau IMP #6 from 3/24/12 to 3/25/12

In conclusion, the bioretention characteristics were adjusted at the Walden Park Commons and Fire Prevention Bureau sites to achieve a closer agreement between the HSPF model predictions and the monitoring data. The infiltration rate to the surrounding soils was increased to 0.24 inches per hour for all the Fire Prevention Bureau IMPs.

The calibrated model adequately represents the key processing during and after storm events, specifically; a) the build-up of soil moisture, b) the percolation from bioretention soils to the storage layer and, c) the recovery of the IMP capacity through infiltration to surrounding soils (at the Fire Prevention Bureau). The calibrated model is suitable for the analysis of long-term IMP performance.

6.3 IMP Performance Compared to Flow Duration Standard

The IMP performance monitoring data review suggested the bioretention facilities at the Fire Prevention Bureau and the bioretention plus vault facilities at Walden Park Commons are likely to meet the NPDES permit requirements and may be performing in excess of these requirements by reducing flow durations below the pre-project flow durations for the specified range of flows (0.2Q2 to Q10).

Long-term HSPF simulations were run for the IMPs at both project sites to more fully test the IMP performance against the NPDES permit's flow control standard. The Fire Prevention Bureau simulations used hourly rainfall data collected at the Los Medanos gauge from 1972 through May 2013. The Walden Park Commons simulations used hourly data from the FCD 11 gauge in Martinez gauge from 1969 through May 2013. The following statistical analyses were then performed on the model outputs:

- Flow frequency statistics. The model outflow time series was divided into discrete flow events (i.e., a partial-duration series) using a 24-hour period of no flow to indicate the end of an event. The resulting table of events was sorted and ranked based on the peak flow rate. Each event was assigned a recurrence interval (sometimes referred to as a return period) using the Cunnane plotting position method. Partial duration series statistics were computed for the pre-project runoff and the post-project IMP outflows.
- Flow duration statistics. The model outflow time series was divided discrete bins (flow ranges). The number of hours – or duration – for which outflow occurred in each bin's flow range was then counted. These durations were computed for the pre-project runoff and the post-project IMP outflows.

Figure 6-17 shows the peak flow frequencies for the pre-project runoff and post-project (i.e., existing) outflow for Fire Prevention Bureau IMP #2.

Figure 6-18 compares flow durations for the pre-project and existing conditions. In both figures, the IMP outflows are below the pre-project flows between 0.2Q2 and Q10. Additionally, IMP #2 outflows are below the pre-project site flows down to the 0.1Q2 threshold. Because IMP #2 was constructed with dimensions that are very similar to the minimum required dimensions included in the HMP, this suggests IMP #2 would comply with a stricter lower control threshold of 0.1Q2. The infiltration rates at the Fire Prevention Bureau site allow for this level of performance.

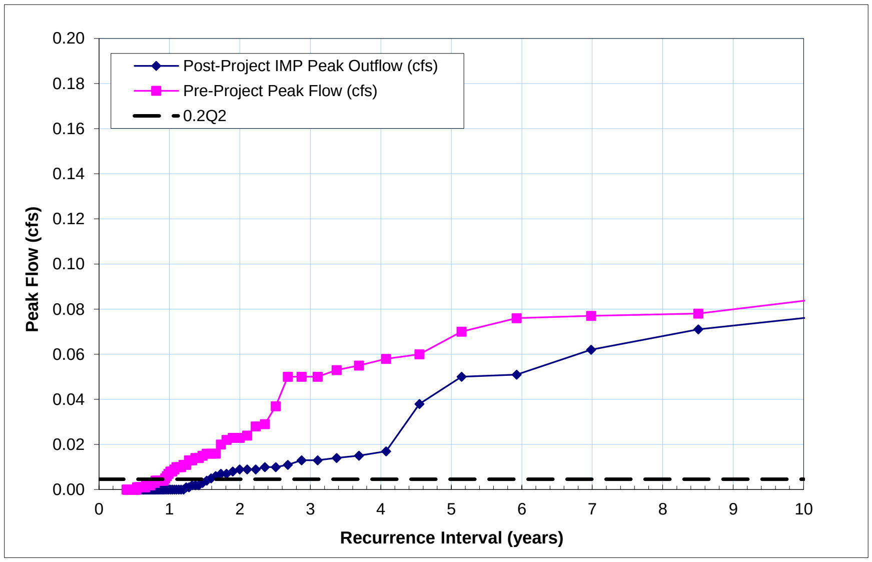

Figure 6-17. Peak flow frequency comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #2

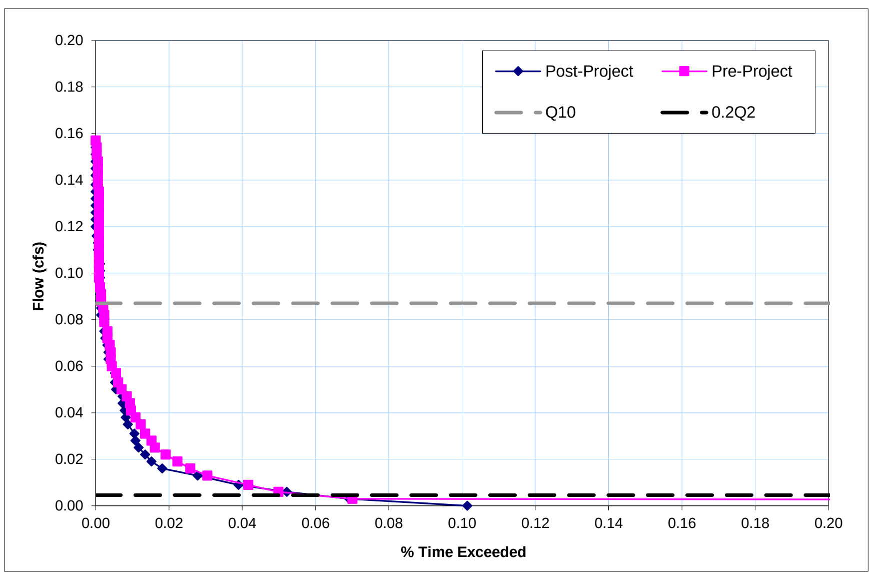

Figure 6-18. Flow duration comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #2

Figure 6-19 and Figure 6-20 compare peak flow frequencies and flow durations for Walden Park Commons IMP #1 (North), respectively. IMP #1 (North) reduces site runoff to levels below the pre-project conditions between 0.2Q2 and Q10. However, the model results indicate that IMP #1 (North) does not control flows down to the 0.1Q2 flow rate. To meet this standard, the flow control orifice diameter would need to be reduced and the storage volume potentially increased by a modest amount, and/or the storage volume would need to be allowed to infiltrate to subsurface soils—as in the *Guidebook* criteria for bioretention + vault facilities.

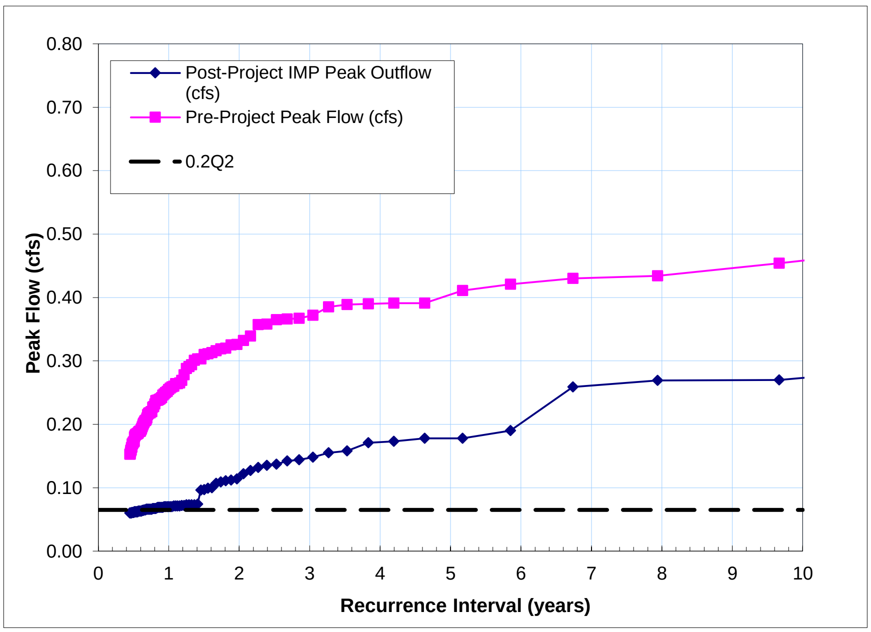

Figure 6-19. Peak flow frequency comparison for pre-project runoff and post-project outflows at IMP #1 (North)

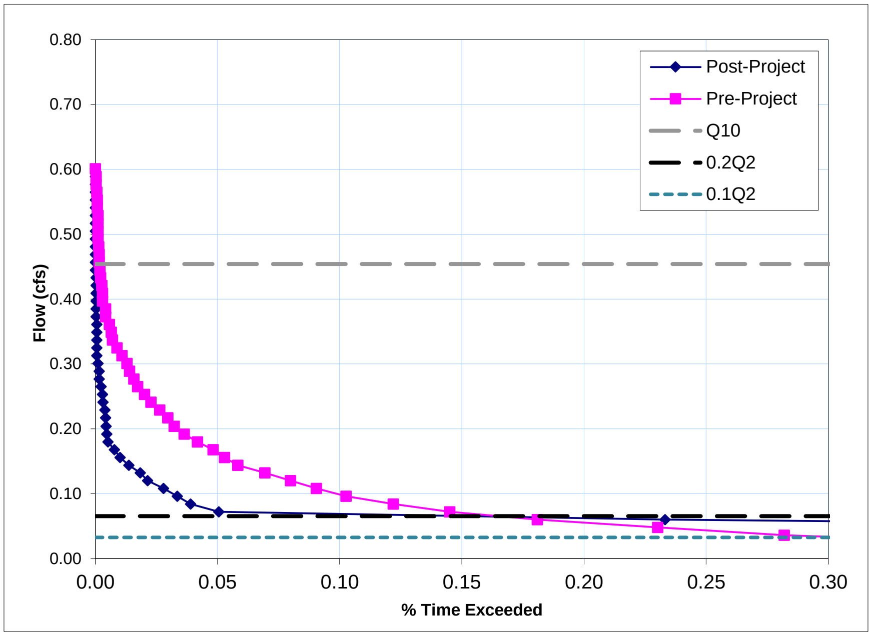

Figure 6-20. Flow duration comparison for pre-project runoff and post-project outflows IMP #1 (North)

All the IMPs successfully control outflows to their pre-project levels from 0.2Q2 to Q10. The Fire Prevention Bureau IMPs also control flows down to the 0.1Q2 threshold – benefitting from the infiltration capacity of site soil conditions. The Walden Park Commons do not control IMP outflows to the 0.1Q2 threshold, but the modeling results suggest this additional level of control could be achieved by a one or more of the following: modifying the orifice configuration, by allowing stored runoff to infiltrate to underlying soils, or by increasing the storage volume modestly.

7 · Discussion

7.1 Why These Results Are Important

The principal advantage of environmental modeling is the capability of modeling to extrapolate limited data sets to make predictions over an extended period and wide variety of conditions. However, because of limited data and the unpredictability of environmental conditions, a "garbage in, garbage out" scenario can occur, where model results are primarily a reflection of guesses and assumptions input to the model.

The 2004-2005 model used to determine CCCWP IMP sizing factors had the advantage of representing a relatively controlled system and the disadvantage of a paucity of available data representing bioretention system performance. That is, the model did, in concept, accurately represent the structure and function of bioretention facilities as they are actually built; however, there was a near-absence of data to inform the selection of values for the parameters that most strongly affect bioretention performance—most notably the rate at which treated runoff infiltrates to native soils.

Data collection for this project fills this gap, and greatly advances the CCCWP model. Previously the CCCWP model was dependent primarily on guessed and assumed values for the most important parameters; now it is based on empirically derived values. The CCCWP data may also be useful in updating similar models, such as the Bay Area Hydrology Model, that currently use guessed and assumed values for the model parameters that most strongly affect facility performance and HM compliance.

7.2 Percolation Through Bioretention Planting Media

As noted in Section 6, the model was set up with the assumption that the entire planting media layer would become mostly saturated before treated runoff proceeded to percolate into the underlying gravel layer. When modeled and measured results were compared, it was noted that runoff was measured in the gravel layer of the bioretetention facilties (at the Pittsburg Fire Protection Bureau Building) and in the storage vaults (at Walden Park Commons) much more quickly than the model predicted.

This may be occurring either because runoff percolates rapidly downward near the inlet, and much of the planting media layer did not get wet, or because the soil media exhibits less moisture-holding capacity and matric head than the model predicted, or both.

7.3 Infiltration to Native Soils

The capability of a bioretention facility to control volumes and durations of discharge is dependent on, among other factors, the rate of infiltration to native clay soils. This study demonstrated that infiltration at the five test locations is approximately 10 times faster than estimated in the 2004-2005 CCCWP model.

The estimate in the 2004-2005 CCCWP model was drawn from guidance for the use of HSPF at the watershed scale. The values selected for continuous-simulation models are typically based on calibration of models of runoff at the watershed scale—that is, to data sets consisting of local rainfall data and stream gauge data. The stream gauges represent flows collected from watersheds ranging from tens of acres to hundreds of acres.

Importantly, the resulting calibrated model values for key parameters representing losses of surface runoff to infiltration (in HSPF, “INFILT” is such a key parameter) do not necessarily correspond to results of infiltrometer tests or other direct tests of soil permeability. In fact, surface runoff losses at the watershed scale and movement of water through the pores of saturated soil are somewhat different physical processes.

The data collected by this project provide rare (perhaps unique) infiltration rate data and represent actual bioretention performance, rather than using an estimate of performance extrapolated from watershed-scale model calibrations or soil testing. Although limited to three bioretention facilities around a single 1-acre site, the data show that silty clays can, at least in some circumstances, infiltrate at rates in excess of 0.2 inches per hour—as measured by the recovery of a bioretention subsurface reservoir—and that these higher-than-expected rates are consistent throughout the season, for a range of storm sizes, and from facility to facility.

7.4 Applicability of Results Region-wide

The five IMP monitored locations are representative of typical Bay Area development patterns and conditions.

As noted in Section 5, the two bioretention + vault facilities at Walden Park Commons were constructed with some exceptions to current *Guidebook* design recommendations; these exceptions were incorporated into the customized model for the purposes of model calibration. The three facilities at the Pittsburg Fire Prevention Bureau Building were built very close to current *Guidebook* design criteria and design recommendations.

As previously noted, the rate at which runoff infiltrates to soils beneath the facility is a key factor determining overall performance. Are the infiltration rates found at the Pittsburg site representative of development sites in Contra Costa, or in the Bay Area as a whole?

There are no observed characteristics that would suggest otherwise. The site soils, described as “stiff to hard, moderately to highly plastic silty clays” in the site geotechnical report (Kleinfelder 2004) are typical of development sites throughout the Bay Area. The site is quite flat. Only the lack of near-surface groundwater would tend to suggest this site’s soils could be better-draining than similarly classified soils at another Bay Area development site.

Collection of data from bioretention facilities at additional locations would be necessary to accurately estimate the average and variance of infiltration rates that might occur in similar soils.

8 · Conclusions and Recommendations

This project demonstrated that the IMPs and sizing factors approved by the Water Board in 2006—and updated in subsequent editions of the *Guidebook*—are adequate to meet current regulatory requirements.

8.1 Next Steps for Use of the Calibrated and Validated Model

MRP Attachment C requires:

> By April 1, 2014, the Contra Costa Clean Water Program shall submit a proposal containing one or a combination of the following three options (a.-c.) for implementation after the expiration and reissuance of this permit:
>
> a. Present model verification monitoring results demonstrating that the IMPs are sufficiently overdesigned and perform to meet the 0.1Q2 low flow design criteria; or
>
> b. Present study results of Contra Costa County streams geology and other factors that support the low flow design criteria of 0.2Q2 as the limiting HMP design low flow; or
>
> c. Propose redesigns of the IMPs to meet the low flow design criteria of 0.1Q2 to be implemented during the next permit term.

CCCWP intends to work with other Permittees (through BASMAA, the Bay Area Stormwater Management Agencies Association) and with Water Board staff to develop and agree upon revised HM permit requirements applicable to all MRP Permittees that:

- Favor, rather than constrain, the implementation of LID to meet HM requirements
- Consider a potential range of low flow thresholds for streams, with the aim of revising the thresholds to provide for reasonable protection of beneficial uses
- Have a more technically defensible basis for translation of in-stream criteria to LID facility discharge criteria; this basis should include consideration of the potential future

extent of watershed development and the proportion of the watershed that the proposed development represents

- Take into account that IMPs tend to reduce flow durations to below pre-project levels for flows in the middle of the range (the most geomorphically significant range, between 0.2Q2 and Q2)
- Consider the extent of potential Bay Area development that may be subject to HM requirements vs. the effort expended so far, and that may be expended in the future, on developing and implementing HM regulations
- Apply exceptions, exclusions, and thresholds uniformly among MRP Permittees
- Incorporate design requirements and sizing factors that reflect the results of this study

8.2 Insights Concerning Bioretention Design and Construction

The CCCWP project team worked with City of Pittsburg and City of Walnut Creek staff and with the engineers and construction project managers for each of the two developments. Overall cooperation was excellent and contributed greatly to the success of the CCCWP project.

The following insights are the author's but resulted from the work of all involved.

8.2.1 Bioretention Design

To maximize the volume of runoff infiltrated, the facility must be configured so that each layer "fills up like a bathtub." The top of gravel layer should be at a consistent elevation so that all pore areas within the gravel layer are filled evenly; likewise for the soil layer and for the surface reservoir. The surface reservoir should be surrounded by concrete curbs or landscape timbers to maximize its volume (as compared to sloping sides toward the center of the facility) and to facilitate verification that the reservoir is level and will fill evenly.

The project design should be reviewed prior to construction to ensure the stability of roads, walkways, and structures adjacent to bioretention facilities has been adequately considered. Because bioretention soils cannot be compacted, bioretention walls must effectively resist lateral pressure from surrounding soils. Where necessary, bioretention walls can be made impervious as a precautionary measure to protect adjacent roads, walkways, and structures while leaving the bottom of the bioretention facility open for infiltration.

Overflow structures are best constructed from precast manholes or catch basins. Construction crews have experience setting these structures at a precise elevation. Use of an adequately sized catch basin with a grate makes it possible to verify underdrain discharge visually and to access the underdrain pipe for cleaning or maintenance. Setting the underdrain discharge elevation at the top of the gravel layer may reduce the required depth of the overflow structure.

Overflow structures can also accommodate connections to site storm drainage pipes routed through the bioretention facilities.

Orifices on underdrains may be constructed of solid PVC pipe extending a few inches into the overflow catch basin structure, threaded, and equipped with a cap. The orifice is drilled into the cap as shown in Figure 4-4.

8.2.2 Bioretention Construction

It is necessary to have an engineer familiar with the structure, function, and details of bioretention to review construction at each stage (layout, excavation, installation of underdrains and overflows, installation of gravel and soil mix, irrigation systems, and planting). In particular, elevations should be checked and it should be ensured that the soils at the bottom of the excavation are ripped.

8.3 Recommendations for Instrumentation

Success in data collection was largely attributable to the participation of an experienced instrumentation technician (Scott McQuarrie, of the Contra Costa Flood Control and Water Conservation District). Installation of rain gauges, tipping buckets, magnetic flow meters, piezometers, dataloggers, and telemetry required considerable technical ingenuity and experience to configure at each site.

For future projects monitoring the hydrologic performance of bioretention facilities, including bioretention + vault facilities, it would be possible to rely on level sensors (piezometers) rather than flow sensors or tipping buckets. Piezometers are more reliable to operate and also provide information on saturation levels. Orifice factors and/or rating curves for each fabricated orifice could be determined prior to installation. This could be done by plumbing the fabricated orifices to a small tank or reservoir and timing the falling head. Once installed, the discharge rate through the orifice, for each time interval, could be calculated from the corresponding piezometer reading.

8.4 Further Research

As noted above, it would be meaningful to obtain data from bioretention facilities installed in clay soils at additional sites. An additional 3-8 sites could be sufficient to demonstrate the regional applicability of the results found here.

This study showed the value of obtaining time-series for (1) rainfall and (2) saturation depth of the subsurface storage (gravel layer). It is recommended to select, where possible, facilities located on public development projects, as it is easier to coordinate documentation of design and construction of bioretention facilities on these projects.

As noted above, the monitoring effort could be reduced by installing only rain gauges at each site and only piezometers in each facility. As a rough estimate, instrumentation could be installed at an equipment cost of $7,000 and about 12 hours of technical labor for each facility. This does not include the cost of maintaining the instrumentation and downloading the data.

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APPENDIX A
IMP Modeling Analysis and Results

This appendix supplements the modeling and data analysis results included in Section 6 of the HMP Model Calibration and Verification report. This appendix includes a detailed description of the project site model development, rainfall analysis, model calibration and long-term simulation results.

Section 1: Project Site HPSF Model Development

HSPF models were constructed for the Fire Prevention Bureau site in Pittsburg and the Walden Park Commons site in Walnut Creek. The models were adapted from the HPSF models that were developed for the HMP by including the drainage management area characteristics, IMP configurations of each site, and time series input data for each site.

The following site-specific modifications were made:

1. Setting up subcatchment areas within HSPF to represent the project site area
2. Modifying the bioretention IMP setup to represent the actual configurations of the IMPs – the constructed areas and volumes instead of the volumes required by the HMP.
3. Incorporating local time series data, including project site rainfall data in 15-minute increments.
4. Changing the model time step from 1 hour to 15 minutes. This also necessitated changing several conversion factors within HSPF – particularly for quantities that are calculated in HPSF as volumes or depths per time step (rather than per second or per hour).

Following these modifications, various QA/QC checks (e.g., comparing IMP inflow to rainfall volumes, comparing IMP layer 1 outflow and layer 2 inflow volumes) were performed to validate the model response.

1.1 Drainage Management Areas

The HPSF model's Drainage Management Area (DMA) characteristics were derived from drainage planning information provided by the Clean Water Program. For the Fire Prevention Bureau site, the Stormwater Treatment Plan (drawing sheet C-6, dated September 2009) included the drainage areas, soil types and other information needed for the model. For the Walden Park Commons site, the C.3 Plan – Stormwater Treatment Control Plan (drawing sheet C-1, dated July 2008) were used to characterize the DMAs. Table 1 lists the Fire Prevention Bureau DMA characteristics and Table 2 lists the Walden Park Commons DMA characteristics.

Table 1. Pittsburg Fire Prevention Bureau Site DMA Characteristics[A]

DMA	Impervious Area		Pervious Area		Total Area	
	ft2	acres	ft2	acres	ft2	acres
DMA 2 (trib. to IMP 2)	12,059	0.2768	2,415	0.0554	14,474	0.3323
DMA 4 (trib. to IMP 4)	627	0.0144	0	0.0000	627	0.0144
DMA 6 (trib. to IMP 6)	3,152	0.0724	562	0.0129	3,714	0.0853

A. All pervious areas were simulated as NRCS Group D soil (PERLND 102)

Table 2. Walden Park Commons Site DMA Characteristics[A]						
DMA	Impervious Area		Pervious Area		Total Area	
	ft2	acres	ft2	acres	ft2	acres
Tributary to IMP #1 (North)						
M	11,606	0.2664	2,153	0.0494	13,759	0.3159
N	21,695	0.4980	3,795	0.0871	25,490	0.5852
Total IMP #1 (North)	33,301	0.7645	5,948	0.1365	39,249	0.9010
Tributary to IMP #2 (South)						
D	7,780	0.1786	1,381	0.0317	9,161	0.2103
E	7,574	0.1739	1,252	0.0287	8,826	0.2026
J	5,382	0.1236	2,120	0.0487	7,502	0.1722
K	8,996	0.2065	1,658	0.0381	10,654	0.2446
L	3,198	0.0734	575	0.0132	3,773	0.0866
P	3,597	0.0826	509	0.0117	4,106	0.0943
Total IMP #2 (South)	36,527	0.8385	7,495	0.1721	44,022	1.0106

A. All pervious areas were simulated as NRCS Group D soil (PERLND 102)

1.2 IMP Characteristics

The DMA source data also contained information about the site IMPs. For the Walden Park Commons site, the *SWQ and Hydrology Study for Subdivision 9147* drainage report, dated October 2010, was also reviewed to obtain the total volume included in the storage pipes. Table 3 lists the Fire Prevention Bureau IMP dimensions and Table 4 lists the Walden Park Commons IMP dimensions.

At the Fire Prevention Bureau site, the IMPs were generally constructed with dimensions that were close to the requirements of the HMP. For example, the A (area) and V2 (gravel volume) components are IMP #2 are close to the IMP requirements while the V1 (ponding layer) component was larger than required. IMP #4 and IMP #6 were constructed with larger plan areas (A) but the volume ponding layer volume and gravel volume were close to the amount required by the HMP. The underdrain piping for the Fire Prevention Bureau IMPs were located near the top of the gravel layer to provide an opportunity for more of the treated water to infiltrate to the surrounding soils.

Table 3. Pittsburg Fire Prevention Bureau Site IMP Dimensions										
IMP	Required Areas, Volumes			Constructed Areas, Volumes			Constructed Depths			Orifice Diameter (in)
	A (ft2)	V1 (ft3)	V2 (ft3)	A (ft2)	V1 (ft3)	V2 (ft3)	Ponding (in)	Soil (in)	Gravel (in)	
IMP #2	873	734	960	886	886	975	12	18	33	0.81
IMP #4	40	34	44	82.5	41	41	6	18	15	0.17
IMP #6	225	189	247	340	170	215	6	18	19	0.41

The Walden Park Commons bioretention plus vault IMPs were constructed with storage volume (V) components that approximated the HMP requirements. IMP #2 (South) was constructed with a bioretention area that is approximately 20 percent larger than required by the HMP.

Table 4. Walden Park Commons Site IMP Dimensions

IMP	Bioretention Area (ft2)	Storage Volume (ft3)	Orifice Diameter (in)
IMP #1 (North)	1,500	2,419	1.24
IMP #2 (South)	1,917	2,698	1.31

1.3 Time Series Data

Time series data were used to provide rainfall and evapotranspiration inputs to the HSPF model. Table 5 lists the time series datasets used and the periods covered by these datasets.

Table 5. HPSF Model Time Series Datasets

Dataset	Type	Source	Period	Usage
Fire Prevention Bureau Rainfall	Rainfall tipping bucket processed in 15-min increments	Contra Costa Flood Control District	Oct-2011 to May-2013	IMP hydraulic review and model calibration
Walden Park Commons Rainfall	Rainfall tipping bucket processed in 15-min increments	Contra Costa Flood Control District	Nov-2011 to May-2013	IMP hydraulic review and model calibration
Los Medanos Rainfall	Long-term rainfall in hourly increments	Contra Costa Flood Control District	Jul-1974 to Aug-2013	Long-term model simulations for Fire Prevention Bureau site
FCD11 Rainfall in Martinez	Long-term rainfall in hourly increments	Contra Costa Flood Control District	Feb-1969 to Aug-2013	Long-term model simulations for Walden Park Commons site
Brentwood Evaporation	Long-term ET data in hourly increments	CIMIS	Jan-1986 to Aug-2013	Model calibration and long-term simulations (with Los Alamitos ET data)
Los Alamitos Evaporation	Long-term ET data in hourly increments	EPA Basins software	Jul-1948 to Dec-1985	Long-term simulations combined with Brentwood. Provided pre-1986 ET data.

1.4 Model Time Step Adjustment

The HSPF models were adapted to run in either 15-minute or hourly time steps. The shorter time step provided better resolution of the IMP hydraulic processes during the model calibration process whereas hourly time steps were needed for the long-term simulations to match the available input time series data sources. Several hydrologic variables are computed by HSPF in time-dependent units (e.g., inches per time step), so conversion factors were needed to allow the model to run with different time steps. These conversions are documented within the HPSF input files (i.e., the UCI files) and listed in Table 6.

Table 6. HSPF Model Time Step Adjustments and Conversion Factors		
HSPF Block	Description	Conversion Factor Revision
NETWORK	Outflow from upper layer of IMP (HYDR) is computed in cfs whereas input to lower layer (IVOL) is computed in acre-feet per time step	For 15-minute time steps: CONVERSION = [1 FT3/S] * [1/43560 AC/FT2] * [900 S/TS] CONVERSION = 0.0207 For 1-hour time steps: CONVERSION = [1 FT3/S] * [1/43560 AC/FT2] * [3600 S/TS] CONVERSION = 0.0826
NETWORK	IMP inflows (IVOL) are computed in units of acre-foot per time step and these data are converted to cfs for reporting via the PLTGEN file	For 15-minute time steps: CONVERSION = [43560 FT3/AC-FT] * [1/900 TS/S] CONVERSION = 48.4 For 1-hour time steps: CONVERSION = [43560 FT3/AC-FT] * [1/3600 TS/S] CONVERSION = 12.1
NETWORK	Pre-project site runoff rates (PWATER SURO) are computed in units of inches per time step. These data are converted to cfs for reporting via the PLTGEN file	For 15-minute time steps: CONVERSION = [43560 FT2/AC] * [1/12 FT/IN] * [1/900 TS/S] * [AREA in AC] CONVERSION = 4.0333 * [AREA in AC] For 1-hour time steps: CONVERSION = [43560 FT2/AC] * [1/12 FT/IN] * [1/3600 TS/S] * [AREA in AC] CONVERSION = 1.0083 * [AREA in AC]

After the conversions were applied, the model outputs were tested through a QA/QC process to validate the results.

Section 2: Rainfall Characteristics

This section supplements the description included in Section 6.1.1 of the HMP Model Calibration and Verification report, specifically the estimate of recurrence intervals for the storms that were recorded during the monitoring period.

To understand the monitored storm events within the context of long-term local rainfall characteristics, depth-duration-frequency curves were developed from the long-term hourly datasets recorded at the Los Medanos gauge and the FCD11-Martinez gauge. The following method was used to develop the curves:

5. The rainfall data was parsed into discrete storm events. A dry period of 24-hours was used to separate rainfall into distinct, independent events. The resulting set of storm events is called as a partial-duration series.
6. Each rainfall event was examined to determine the maximum amount of rain that occurred within specific periods of the storm (e.g., the maximum 3-hour accumulation, 6-hour accumulation) from durations of 1-hour to 72-hours.
7. The accumulations for each duration were ranked and assigned a recurrence interval using the Cunnane plotting position method (e.g., all 12-hour accumulations were ranked, all 24-hour accumulations were ranked).
8. A logarithmic regression relationship was developed to relate rainfall depth to recurrence interval for each storm duration from 1-hour to 72-hour. The regression equations were then used to compute curves shown in Figure 1 and Figure 2. The plots only include the computed durations up to 24-hours.

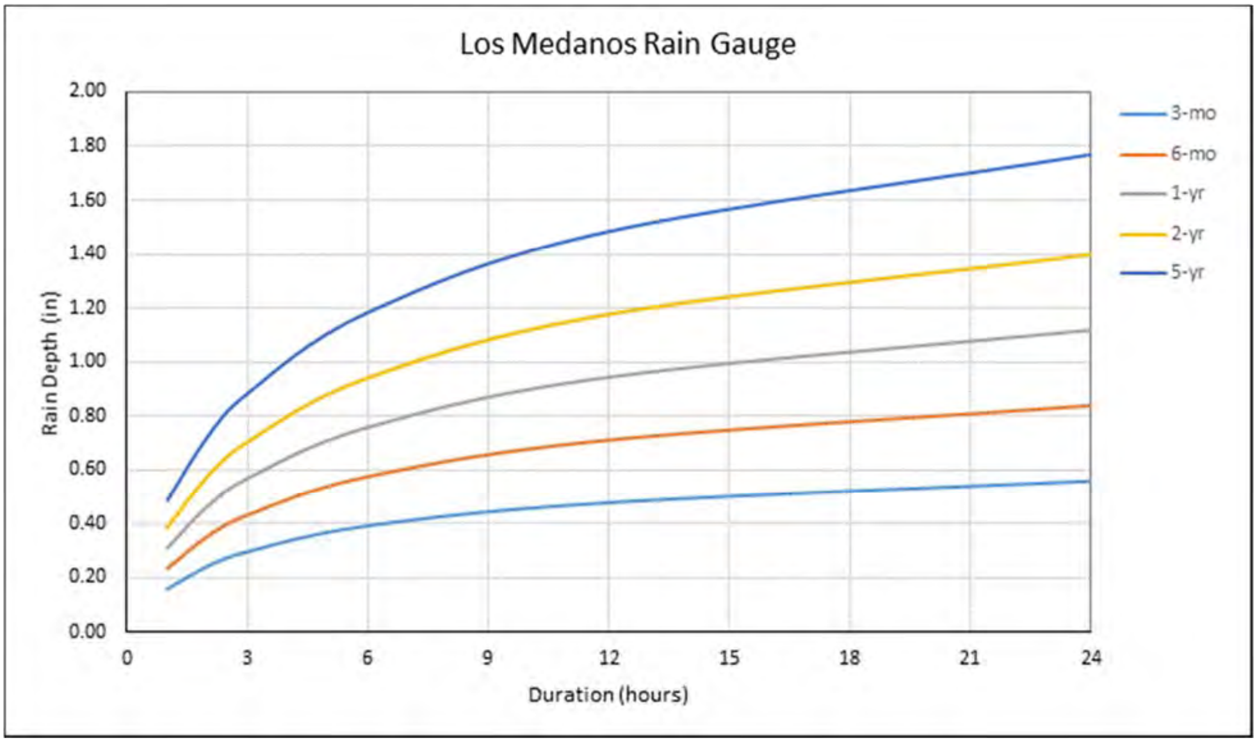

Figure 1. Depth-Duration Frequency curve for Los Medanos rain gauge. Curve was used to estimate the recurrence interval for storms monitored at the Fire Prevention Bureau site.

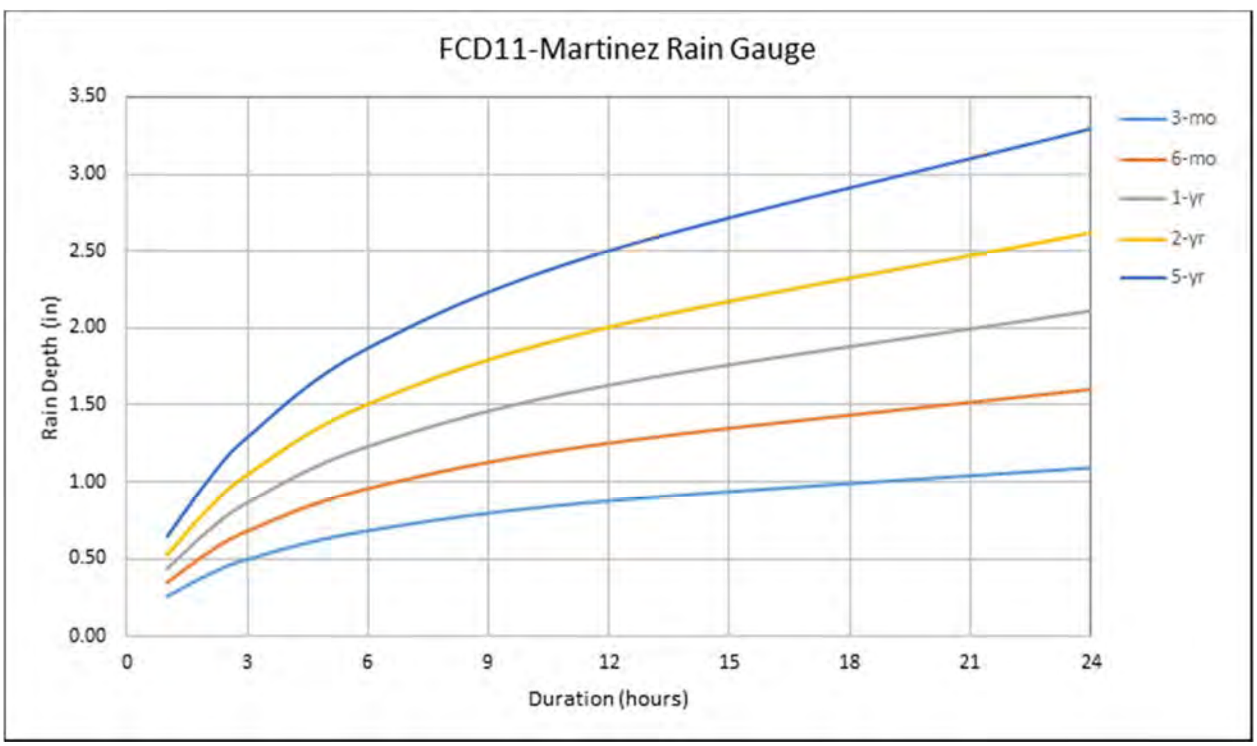

Figure 2. Depth-Duration Frequency curve for FCD11-Martinez rain gauge. Curve was used to estimate the recurrence interval for storms monitored at the Walden Park Commons site.

After the depth-duration-frequency curves were computed from the long-term rainfall datasets, similar partial-duration series rainfall accumulations were computed for the Fire Prevention Bureau and Walden Park Commons rain gauge data. The rainfall depth was computed for each significant storm for durations ranging from 1 hour to 72 hours. The accumulations were then compared to the long-term curves (either Figure 1 or Figure 2) to determine the recurrence interval for the monitored data.

Table 7 and Figure 3 provide an example of how the monitoring period storm recurrence intervals were estimated. The 11/28/2012 storm data provided a total of 1.64 inches of rain at the Fire Prevention Bureau gauge and Table 7 lists the maximum rainfall accumulation for specific periods within the storm event. These data are plotted over the long-term Los Medanos depth-duration-frequency curve in Figure 3 to provide context. The 11/28/2012 storm was approximately a 6-month to 1-year event for durations less than 6 hours. The 12-hour and 24-hour accumulations were approximately equal to a 2-year storm event.

Table 7. Rainfall Accumulations the 11/28/2012 Storm at the Fire Prevention Bureau

Duration (hour)	Rainfall (in)
1	0.31
2	0.38
3	0.44
6	0.69
12	1.20
24	1.32
48	1.33
72	1.64

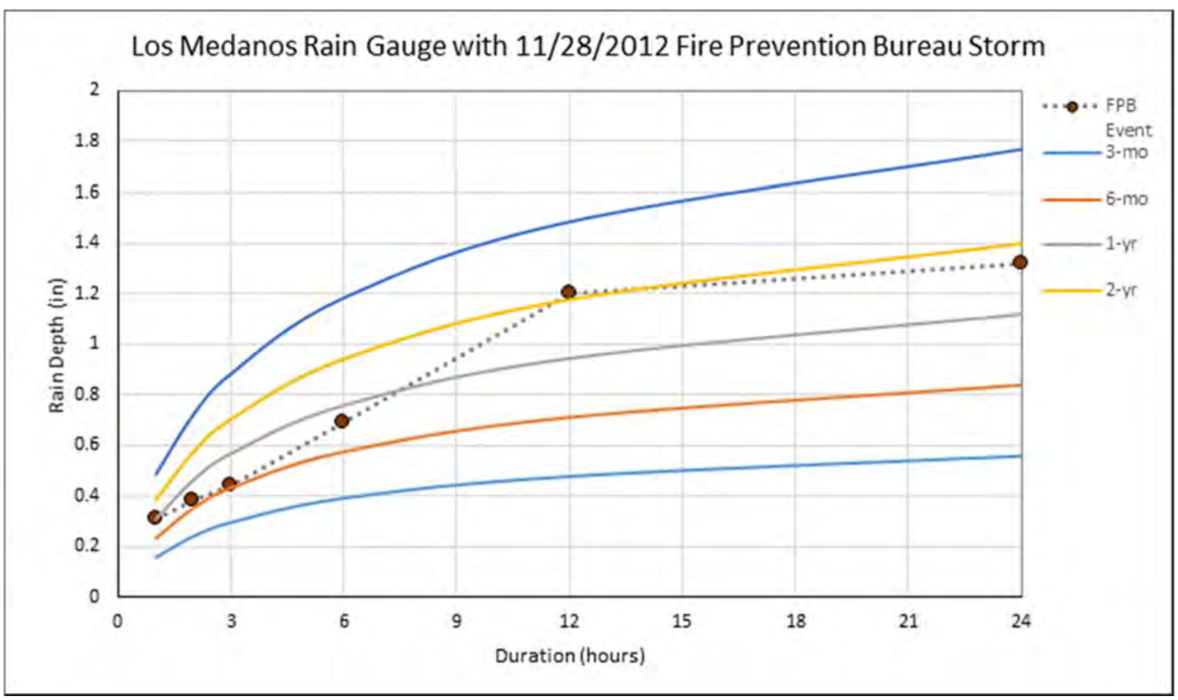

Figure 3. The 11/28/2012 storm event at the Fire Prevention Bureau was approximately a 2-year storm over a 12-hour duration.

Rainfall accumulations were compared to the depth-duration-frequency curves for all of the significant storm events listed in Table 6-2 and Table 6-3. The approximate recurrence interval was reported for 12-hour durations. This duration was selected because it balances both the short-term intensities and long-term accumulations that can affect IMP performance.

Section 3: HSPF Modeling Results

This section supplements the discussion included in Section 6.2 and 6.3 of the HMP Model Calibration and Verification report. It describes he model calibration process in greater detail and provides long-term simulation results for all IMPs.

3.1 Model Parameter Adjustments

This section describes how the model parameters were adjusted and provides additional example calibration results.

3.1.1 Bioretention Soil Characteristics

As described in Section 6.1.3, Fire Prevent Bureau bioretention soils produces faster percolation rates earlier and respond earlier in storm events than was predicted by the HPSF model used to develop the HMP. Additionally, the Fire Prevention Bureau IMPs produced significantly more infiltration to surrounding soils than the HPSF model predicted. The model calibration effort focused on these two key differences.

Rainfall and water level monitoring data and modeling results were examined to approximate a) what level of soil moisture is needed to initiative percolation from the bioretention soil to the gravel layer and b) at what rate does the percolation occur. The bioretention soils appear to produce little percolation until the soils reach about 50 percent of saturation. At this point, percolation occurs rapidly. While the precise rate was difficult to isolate, the monitoring data suggested percolation rates of up to 7.5 inches per hour could occur.

The HSPF model's representation of the bioretention soils was iteratively modified based on the percolation response of Fire Prevention Bureau IMP #6 for different storm events. The adjustments focused on a) allowing the bioretention soils to hold almost all runoff during small storm events and b) percolating the appropriate volume of stormwater to the gravel layer during large storm events.

Figure 4 illustrates how the percolation characteristics were adjusted by showing the soil moisture-percolation relationship used in the HMP models and the modified relationship that was developed by examining the Fire Prevention Bureau monitoring data. The calibrated relationship allows water to move rapidly into the gravel layer when the bioretention soils fill with water and provides the appropriate level of soil drying between storm events.

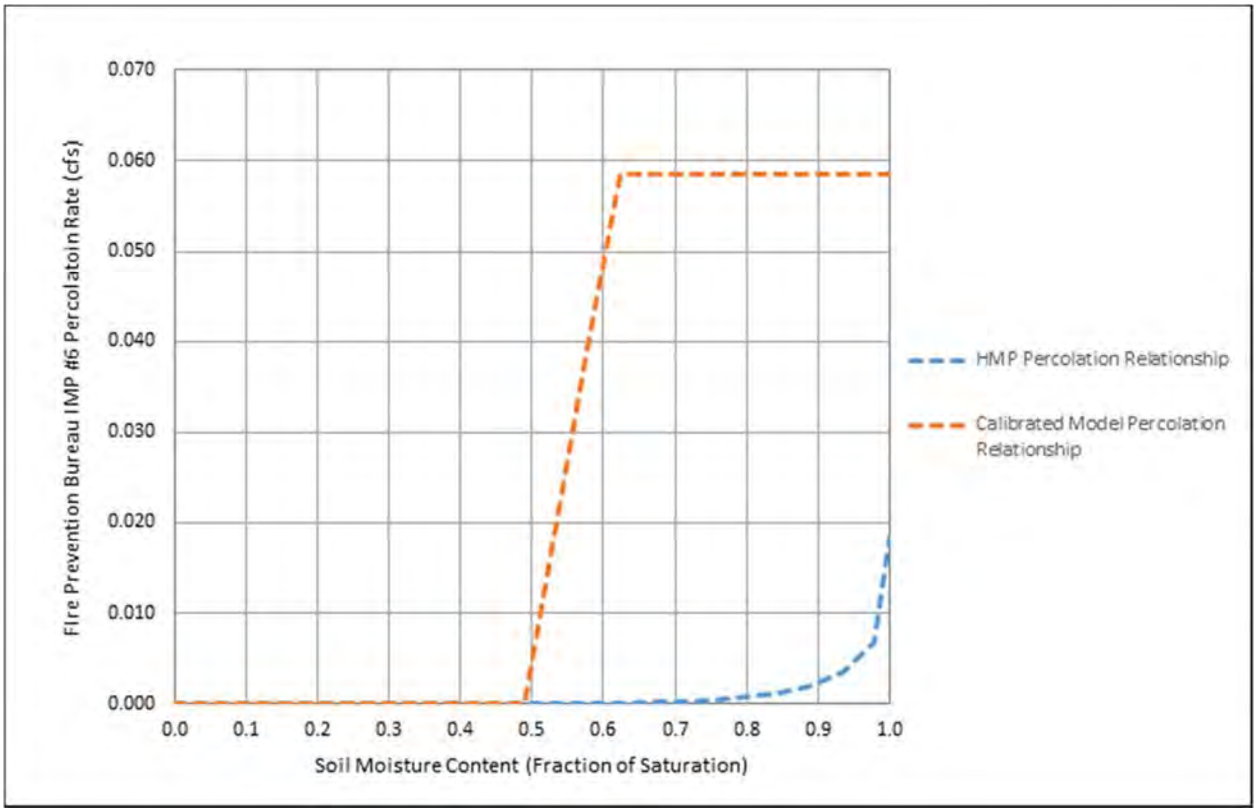

Figure 4. Soil moisture-percolation relationship for bioretention soils at the Fire Prevention Bureau

3.1.2 Infiltration to Surrounding Soils

The observed water level recession rates indicate that the NRCS Group D soils at the Fire Prevention Bureau allow for a greater level of infiltration than was expected when preparing the HMP. The HSPF model's rate of infiltration from the IMP gravel layer to the surrounding soils was adjusted iteratively until the shape of the water level curve approximated the level monitoring data across the largest storm events.

Several gravel layer-to-surrounding soils infiltration rates were tested and the best-fit rate for Fire Prevention Bureau IMP #6 was 0.24 inches per hour. Figure 5, Figure 6 and Figure 7 show the model results for the 11/28/2012 storm event with infiltration rates of 0.20 in/hr, 0.24 in/hr and 0.28 in/hr, respectively. The closest match occurs with the 0.24 in/hr simulation.

The IMP #6 calibration was then applied to the other Fire Prevention Bureau IMPs. The simulation results and monitoring data were compared for IMP #2 and the model results provided a good approximation of the monitoring data. A similar comparison was not practical at IMP #4 due to its small dimensions at IMP #4 and lack of a defined gravel layer response to rainfall.

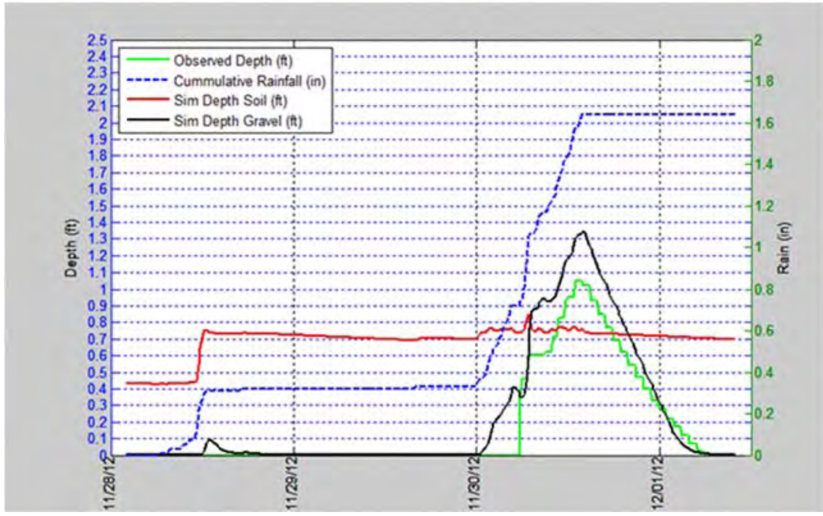

Figure 5. IMP #6 infiltration = 0.20 in/hr. Simulation > monitoring data

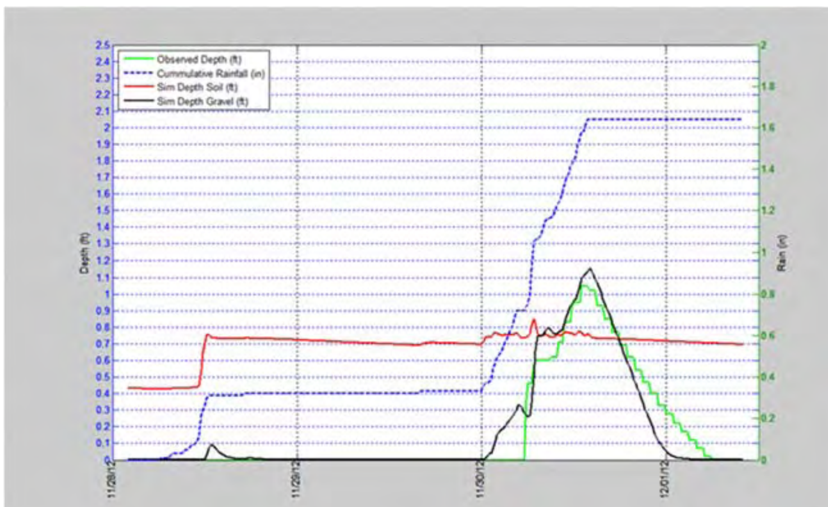

Figure 6. IMP #6 infiltration = 0.24 in/hr. Simulation ~ monitoring data

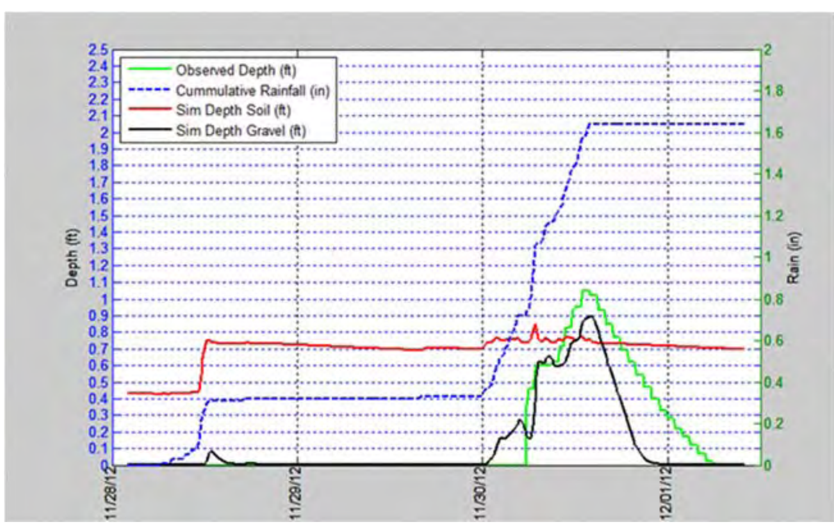

Figure 7. IMP #6 infiltration = 0.28 in/hr. Simulation < monitoring data

3.2 Long-Term Model Performance

This section describes the process for setting up the long-term model simulations and using the results to assess the performance of the Fire Prevention Bureau and Walden Park Commons IMPs in comparison to the HMP's peak flow and flow duration control standard.

3.2.1 Long-Term Simulation Setup

The calibrated models for the Fire Prevention Bureau IMPs and Walden Park Commons IMPs (see Section 6.3 for these examples) were used to prepare long-term simulations. The following steps were needed to prepare the long-term simulation models:

1. The FTABLE representations of the calibrated IMPs were copied into the HSPF long-term simulation input file.
2. The HSPF input file was linked to the long-term time series datasets described above in Table 5. The Fire Prevention Bureau simulations used hourly rainfall data collected at the Los Medanos gauge from 1974 through May 2013. The Walden Park Commons simulations used hourly data from the FCD11 gauge in Martinez from 1969 through May 2013. The evaporation time series dataset was composed of Los Alamitos data (pre-1985) and Brentwood data (1986 and later).
3. The HSPF input file unit conversions were applied as needed for the long-term simulations hourly time steps (see Table 6 for details).
4. The list of variables included model's time series output file (i.e., the PLTGEN file) were modified to allow for a comparison of pre-project and post-project conditions.

3.2.2 Long-Term Simulation Results

The long-term simulation outputs were evaluated using flow frequency statistics and flow duration statistics (see Section 6.3). Next, the IMP outflows were compared to pre-project flows to determine of the IMPs reduced peak flows and flow durations below pre-project levels. This section includes peak flow and flow duration graphics for all of the IMPs. Figure 8 through Figure 13 show results for the Fire Prevention Bureau site and Figure 14 through Figure 17 show results for the Walden Park Commons sites. All IMPs control flows to down to the current 0.2Q2 lower control threshold. Additionally, the Fire Prevention Bureau sites control flows down to the 0.1Q2 lower control threshold. The Walden Park Commons sites do not meet the stricter lower control threshold.

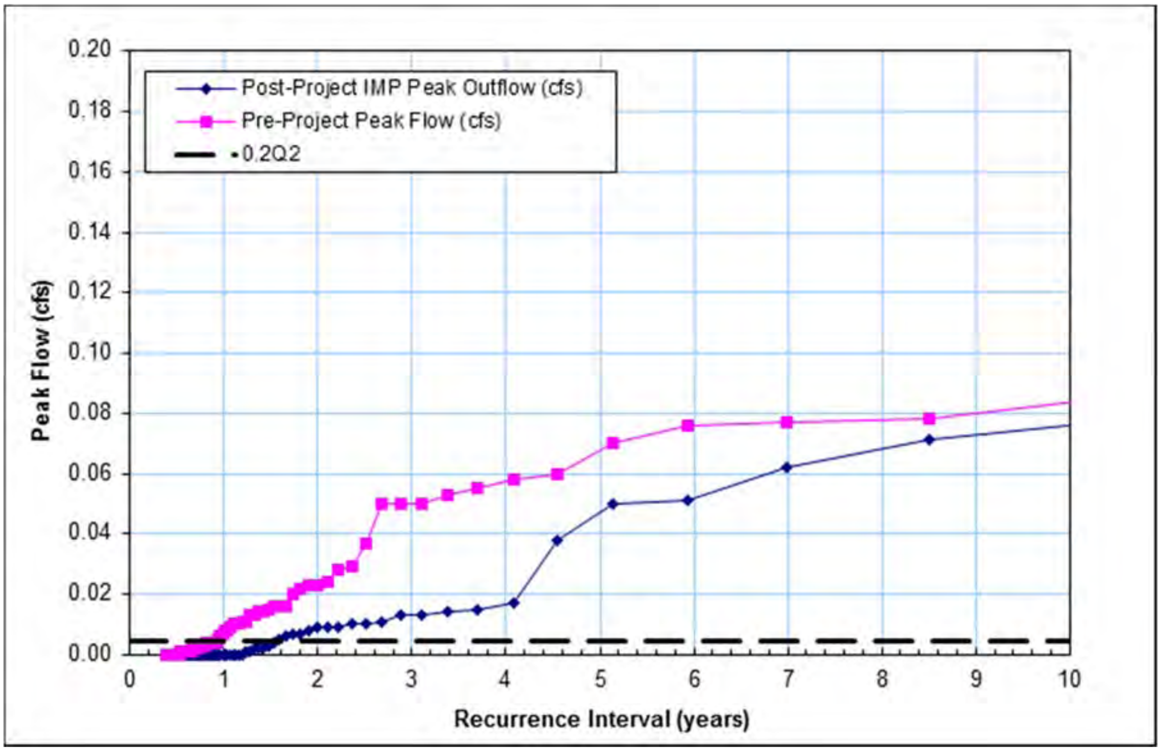

Figure 8. Peak flow frequency comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #2

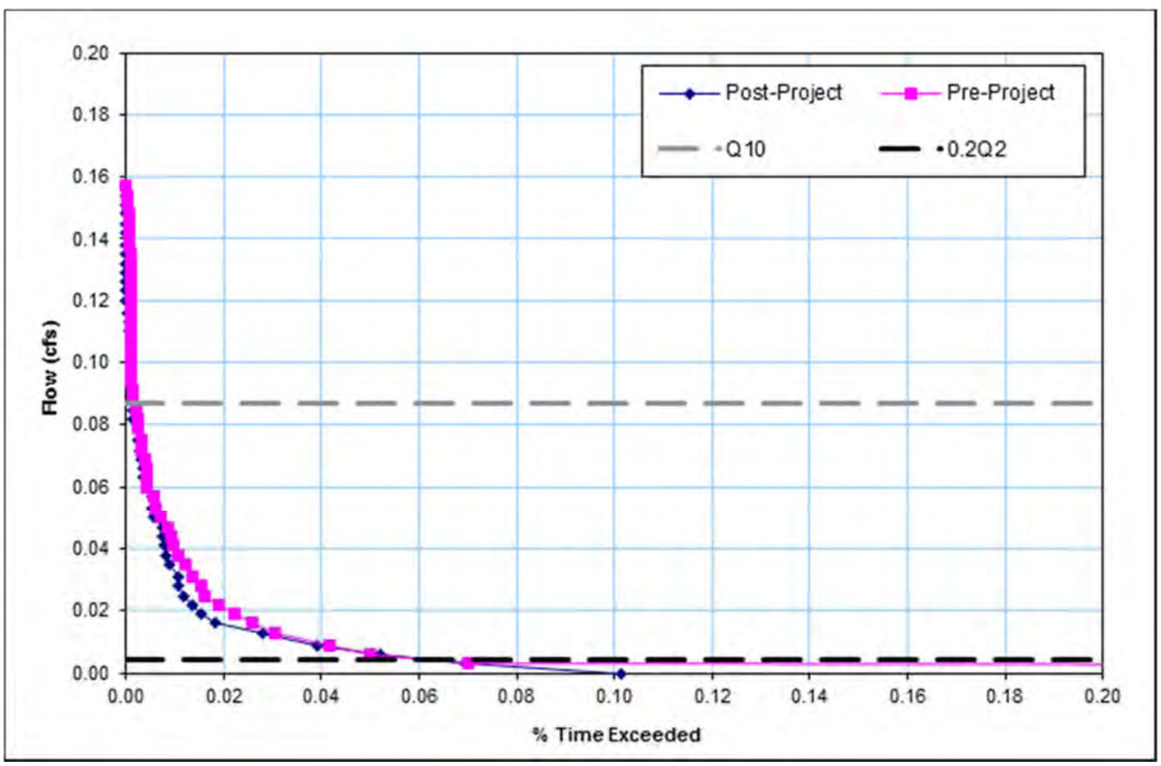

Figure 9. Flow duration comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #2

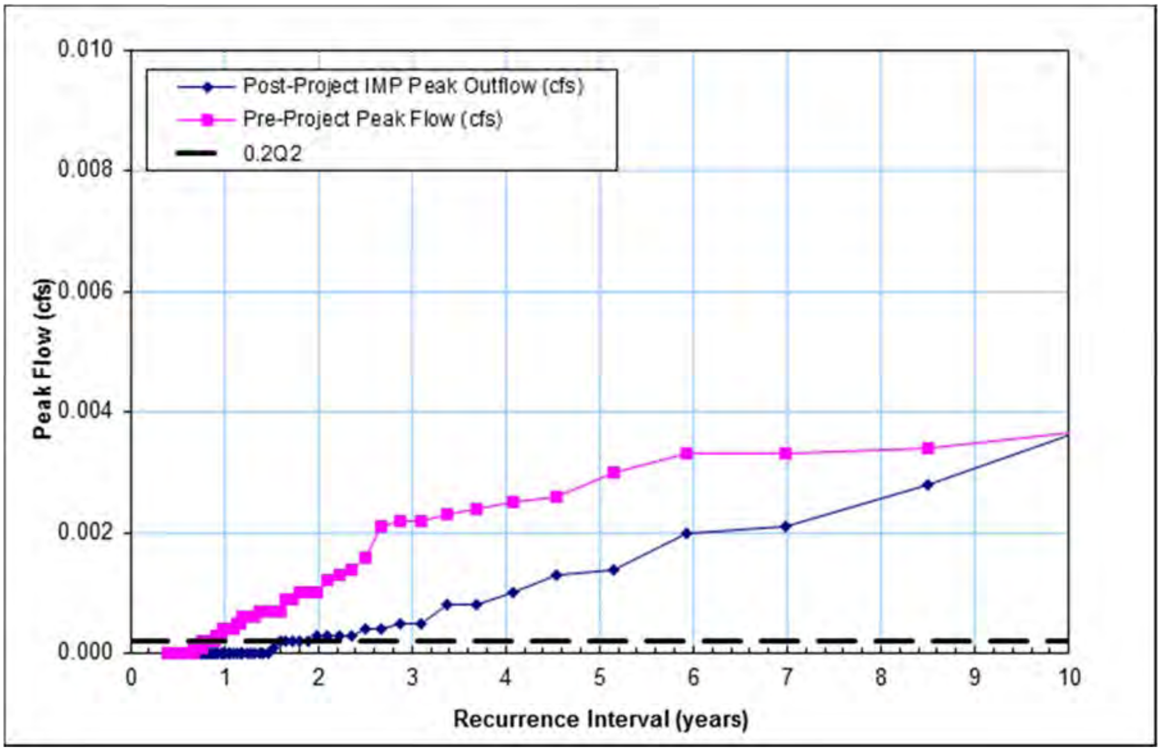

Figure 10. Peak flow frequency comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #4

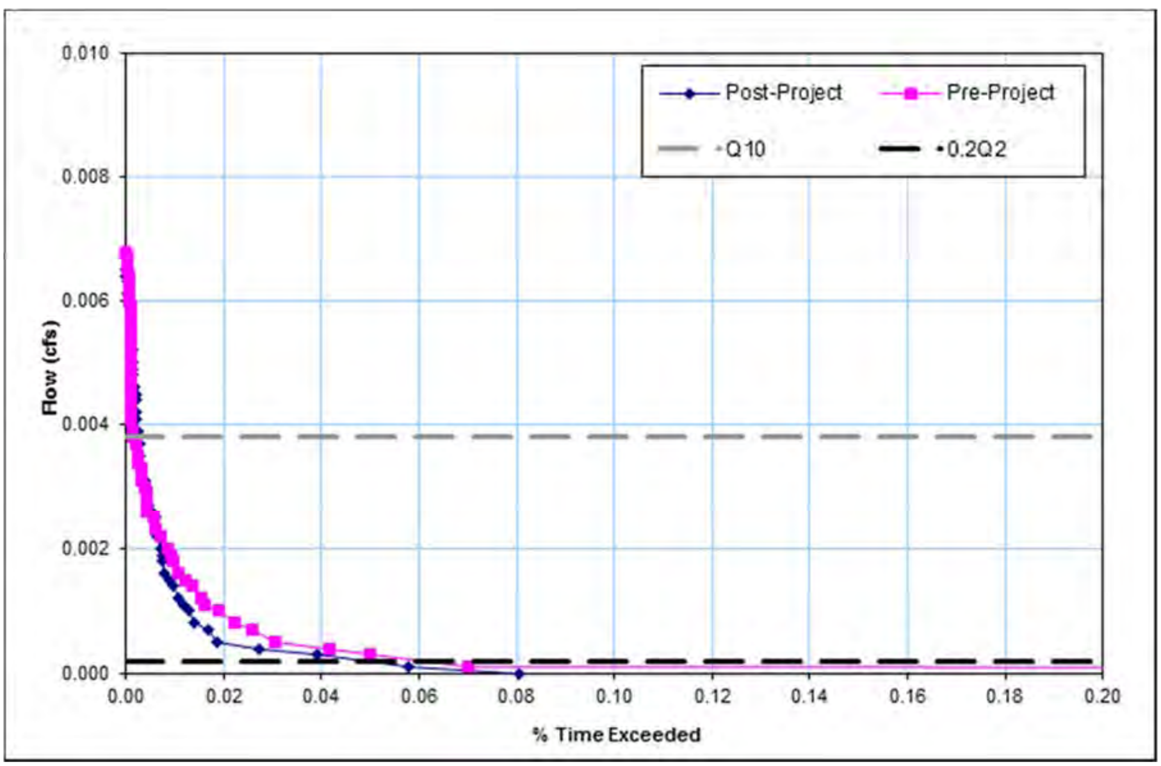

Figure 11. Flow duration comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #4

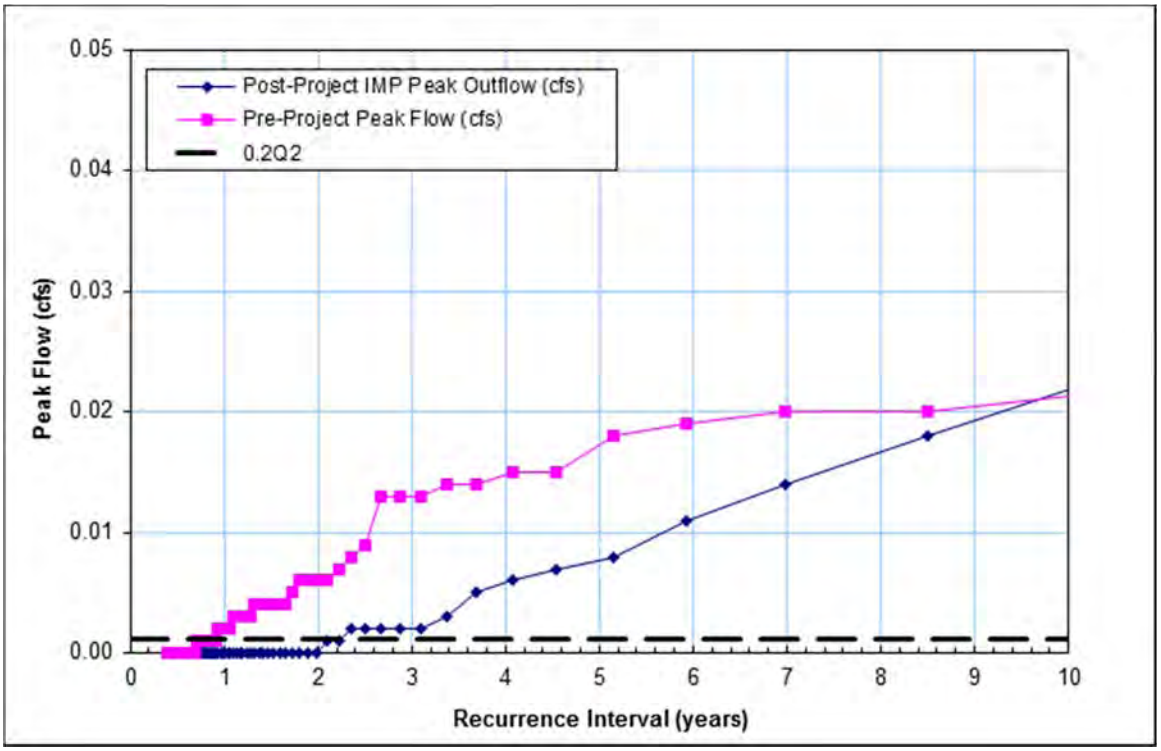

Figure 12. Peak flow frequency comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #6

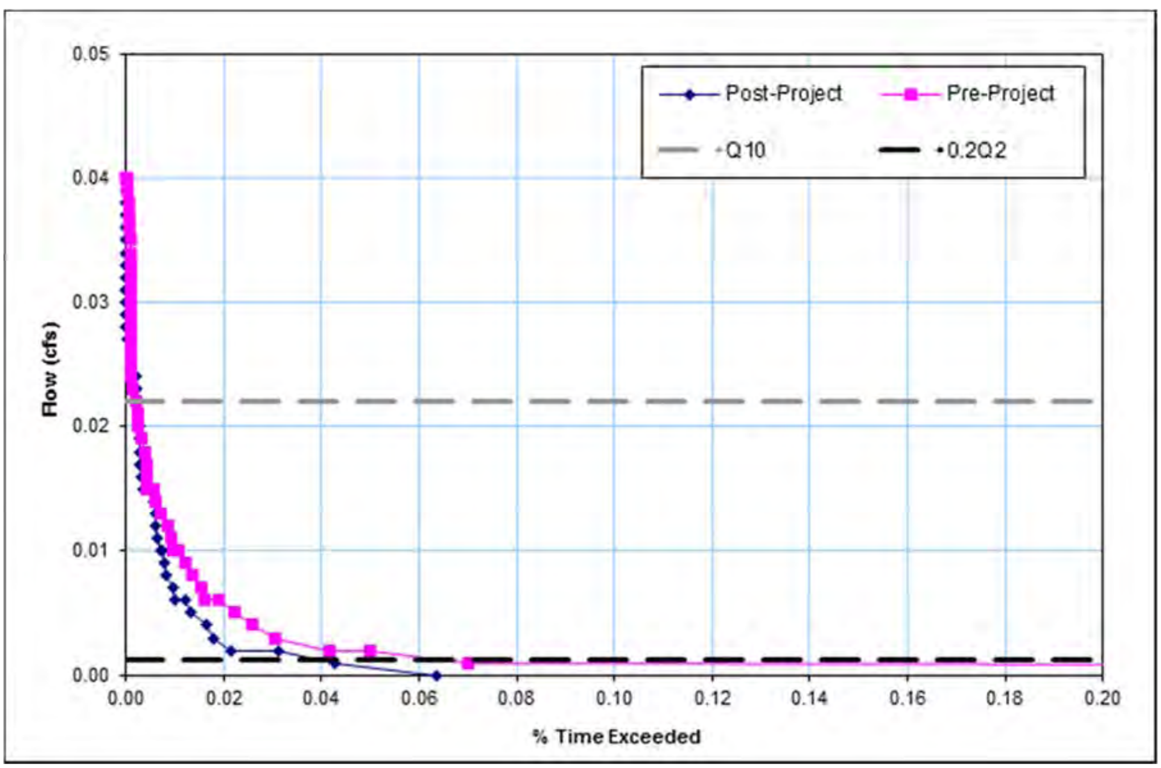

Figure 13. Flow duration comparison for pre-project runoff and post-project outflows for Fire Prevention Bureau IMP #6

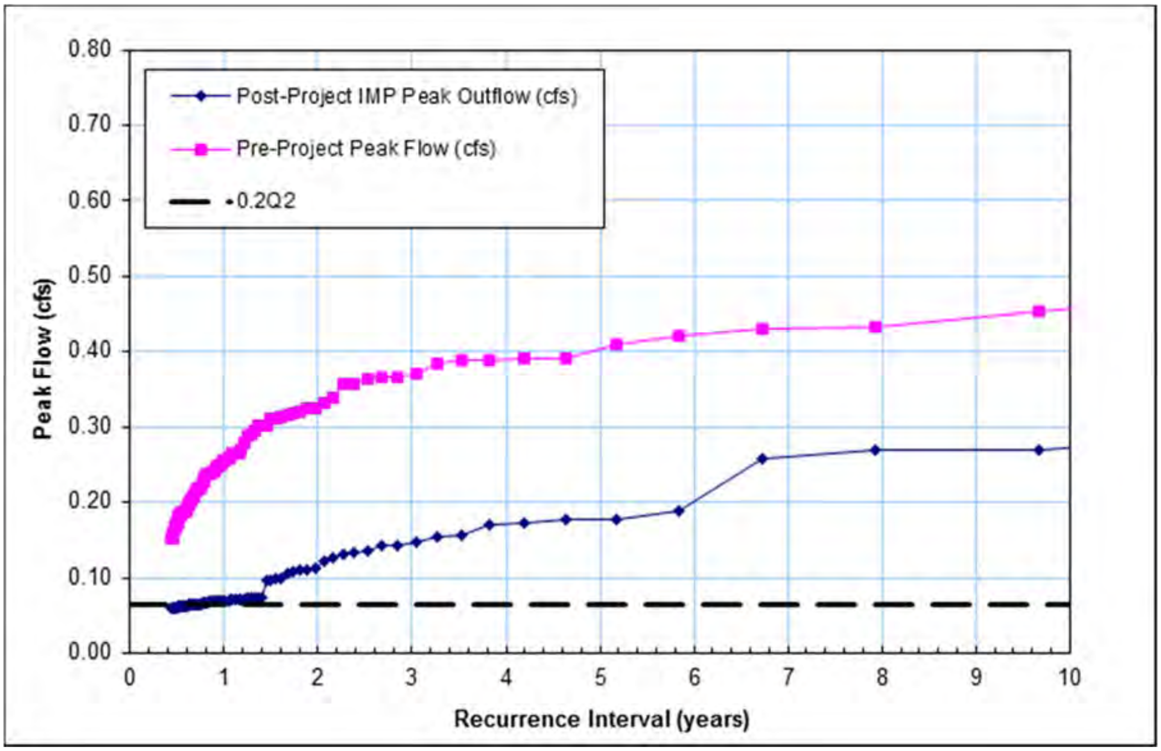

Figure 14. Peak flow frequency comparison for pre-project runoff and post-project outflows for Walden Park Commons IMP #1 (North)

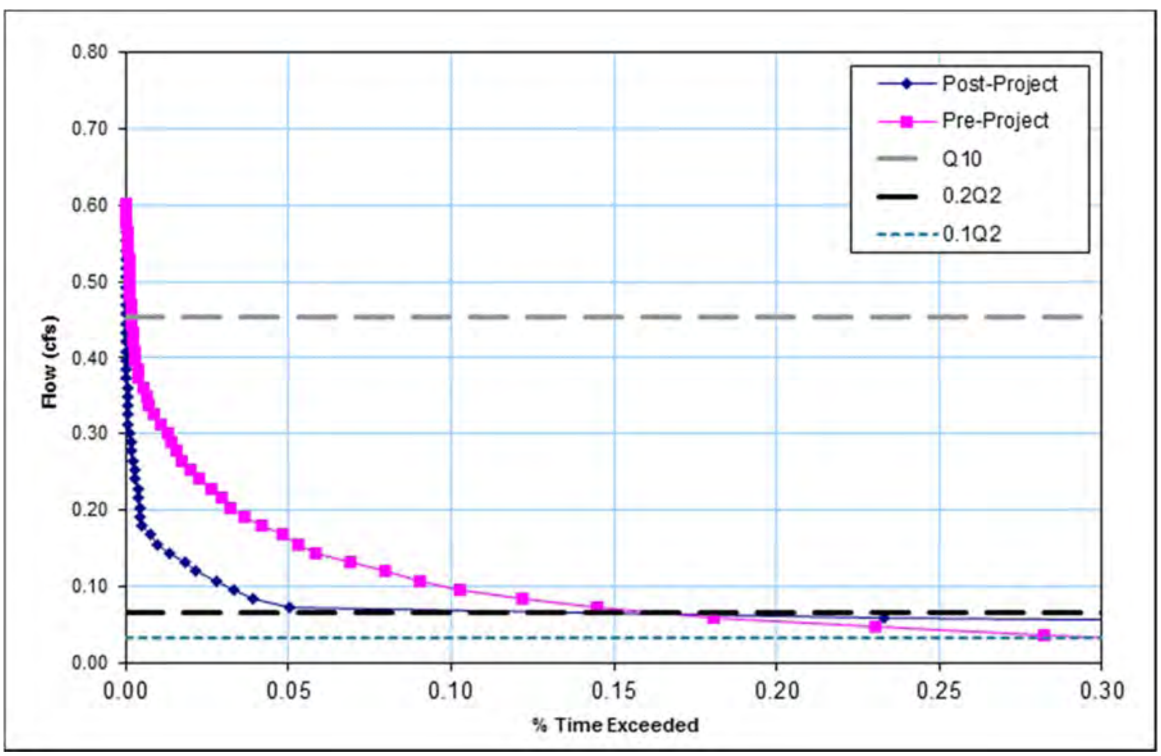

Figure 15. Flow duration comparison for pre-project runoff and post-project outflows for Walden Park Commons IMP #1 (North)

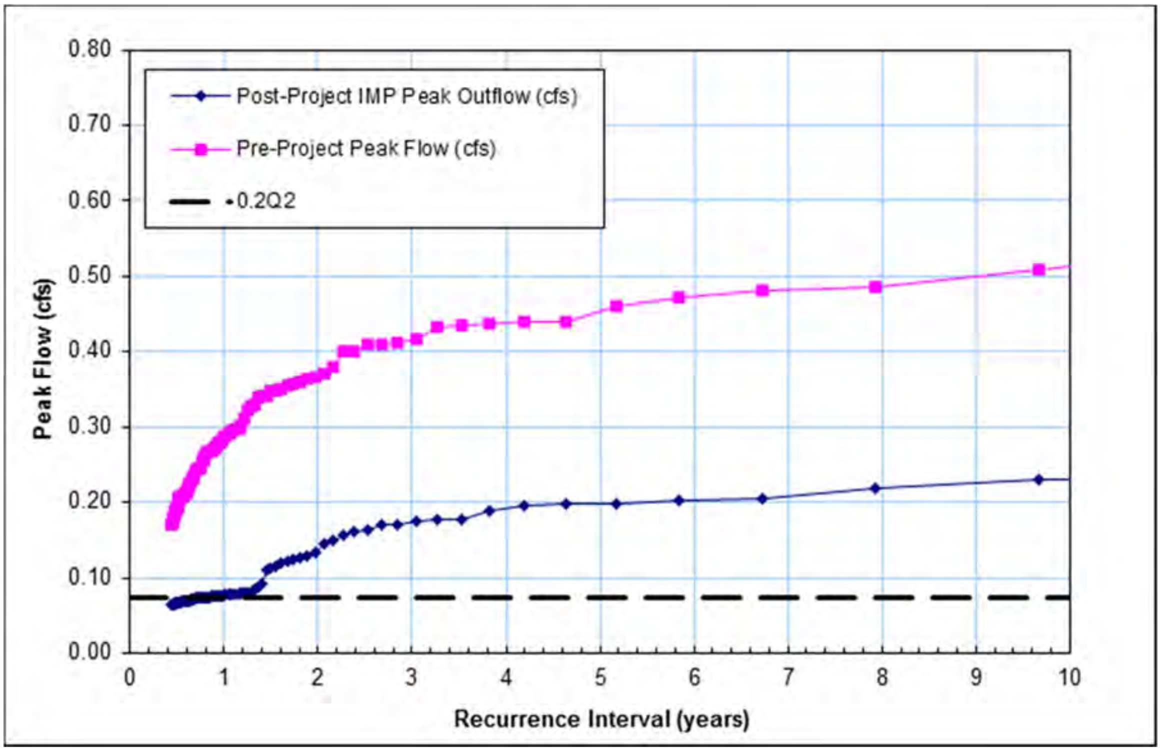

Figure 16. Peak flow frequency comparison for pre-project runoff and post-project outflows for Walden Park Commons IMP #2 (South)

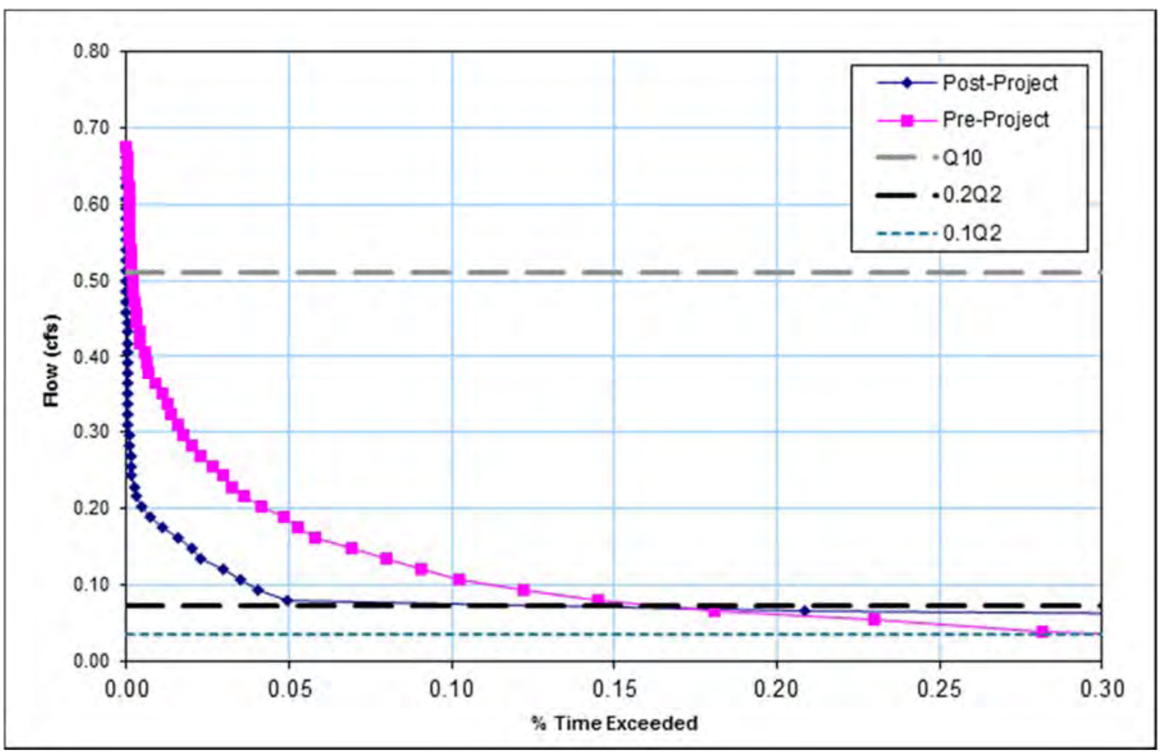

Figure 17. Flow duration comparison for pre-project runoff and post-project outflows for Walden Park Commons IMP #2 (South)

Appendix B: Geomorphic Fieldwork Summary

APPENDIX B: GEOMORPHIC FIELD WORK SUMMARY

Geosyntec performed geomorphic field reconnaissance on February 13th, 15th, and 23rd, 2017 for 15 stream channels within Contra Costa County. These channels are susceptible to hydromodification impacts (i.e., erodible bed and/or banks), are downgradient of anticipated future development, and were readily accessible. Areas anticipated for development in the near future were identified based on input from the Contra Costa municipalities and from the Greenbelt Alliance's At-Risk Map (2017). The At-Risk Map identifies protected open space, existing urban areas, and areas at various levels of risk (i.e., high, medium, and low) for future development. Geomorphic field work performed previously for 13 other stream channels in 2005, as documented in Attachment 4 of the CCCWP HMP (2006), were used to supplement the 2017 field observations where appropriate. These previously observed channels were generally located in less urbanized areas than those observed in February 2017. For each channel visited in the field, the channel geometry and bed and bank material was noted. Measured bankfull widths and depths were compared to empirically derived values for the San Francisco Bay Area (Dunne and Leopold, 1978).

Geosyntec also performed geospatial analysis of the watersheds tributary to the channels where geomorphic field work was performed. The key attributes compiled for these watersheds include tributary area, existing imperviousness (%), and protected open space (%). The StreamStats program (USGS, 2017) was used to delineate and calculate watershed area and existing imperviousness. StreamStats uses the National Land Cover Dataset (MRLC, 2011) clipped to the watershed as a basis for the imperviousness calculation. Protected open space was calculated based on GIS geospatial analysis of the Greenbelt Alliance At-Risk Map (2017) clipped to each field watershed. The geomorphic field observations and watershed attributes served as a basis for the range of receiving channel and land use-related parameter values modeled for the isolated parameter sensitivity analysis (i.e., bankfull width, bankfull depth, mid-channel roughness, longitudinal slope, overbank side slope, tributary area, low flow threshold, existing imperviousness, and protected open space). Figure B-1 shows the stream channels where field observations were made, their tributary watersheds, and the Greenbelt Alliance At-Risk Map (2017) as a backdrop.

Table B-1 provides data collected and calculated for each channel observed in the field and its tributary watershed. Figures Figure B-2 to Figure B-7 show the range of geomorphic and watershed land use parameters compiled from the 2017 field observations, geospatial analysis, and data provided in Attachment 4 of the CCCWP HMP (2006). The distribution of data is plotted with tributary area on the x-axis so that the relative dependence on watershed area can be seen for each parameter. The range of geomorphic and watershed land use parameters was used as a basis for choosing low-, middle-, and upper-bound values for the isolated parameter sensitivity analysis as provided in Table B-2.

The channels observed in February 2017 had deeper and wider bankfull dimensions than those visited in 2005. The reason for this discrepancy could be that the 2017 field sites were situated in watersheds which have experienced more legacy hydromodification impact and geomorphic adjustment due to urbanization. The existing watershed imperviousness is greater overall for the 2017 field sites than it is for the 2005 sites. Another possible reason for the discrepancy is that the field personnel may have used slightly different definitions of the bankfull channel. Geosyntec maintained consistency with the field protocol, described in the CCCWP HMP (2006), wherever possible (e.g., took bankfull observations at riffle locations). There were instances where Geosyntec noted low flow channels at the bottom of the channels observed, but considered these features to be below the active bankfull depth. Phillip Williams and Associates staff in 2005 may have considered these low flow channels to be representative of bankfull, thus resulting in smaller recorded widths and depths than in 2017.

Findings of a preliminary incipient motion analysis indicate that the low flow threshold for the County streams observed can vary considerably from channel to channel (i.e., from less than 10% Q2 to well above 40% Q2). The distribution of low flow threshold results for Contra Costa County as well as Santa Clara County and Fairfield-Suisun, for reference, are provided in Figure B-8. The following subsections explain the preliminary technical calculations performed to evaluate the low flow threshold at each channel observed in the field in February 2017. Low flow thresholds were also calculated for the 2005 field sites based on available data provided in Attachment 4 of the CCCWP HMP (2006), but emphasis was placed on incipient motion calculations in 2017 because: (1) the 2017 field sites were observed in person by the authors of the CCCWP HM technical study; (2) bottom width of the channels observed in 2005 are unknown; and (3) the 2017 field sites are receiving channels which will likely be subject to future hydromodification due to anticipated urban development in the tributary watersheds. The incipient motion calculations were performed consistently with Appendix A (Section 2) of the State Water Board's technical report on hydromodification (SCCWRP, 2012) and Appendix C (Section 4.1) of the Vallejo HMP (City of Vallejo, 2013).

Definition of Critical Flow

The critical flow for stream bed and/or bank mobility (Qc) is the threshold flow that creates an applied hydraulic shear stress equal to the defined critical shear stress for the channel boundary (the point at which the bed and/or bank material begins to mobilize). The defined critical shear stress is based on either bed material or bank material, but also varies depending on the density of vegetation. Qc is an in-stream, low-flow criteria that cannot be exceeded when all sub-areas (including all individual projects or portions of projects) are contributing flow to the stream, if the stream is to be protected from response to hydromodification. Qcp is the portion of Qc from each project and undeveloped areas within the watershed. It is important to note that Qc and Qcp represent the local conditions (i.e., the resilience of the receiving stream). Selecting a value for Qcp that is too high could concentrate cumulative stormwater discharges above the critical flow for bed mobility and exacerbate erosion problems (City of Vallejo, 2013).

Determination of Critical Shear Stress (τ_c)

The resistance of bed and bank materials is quantified by their critical shear strengths, (τ_c) that is, the value where entrainment or transport begins (SCCWRP, 2012). Critical shear stresses were assumed based on an often-cited reference by Fischenich (2001), which provides a summary (compiled from the relevant literature) for critical shear strength values for various materials.

The 2017 field observations indicated that the material that is the most sensitive to channel adjustment is primarily the side banks. This material consisted of a range of materials, from cohesive sandy silt and clay to hardpan, some of which was reinforced with bank vegetation to varying degrees. Based on the literature, three critical shear stresses were used for incipient moiton calculations. The lower-bound value assumed 0.26 lbs/sq-ft for alluvial colloidal silt (Fischenich, 2001). The upper-bound value assumed 0.67 lbs/sq-ft representative of hardpan, which is also in the range of critical shear stress for vegetated soil (Fischenich, 2001). A middle value of 0.35 lbs/sq-ft was used because this was: (1) within the range of values used for incipient motion calculations in the SCVURPPP HMP (2005) (0.14 to 0.38 lbs/sq-ft) and Fairfield-Suisun HMP (FSURMP, 2009) (0.32 lbs/sq-ft); (2) between the lower- and upper-bound values for the channels observed in Contra Costa County; and (3) considered a reasonable value that is generally reflective of the cohesive clay and silt banks observed in the field. One of the 15 channels (Las Tramps Creek in Moraga) had non-consolidated sand banks, so the values assumed for this channel were much less (i.e., 0.045, 0.06, and 0.075 lbs/sq-ft).

Determination of Critical Flow (Qc)

For a specific set of hydraulic conditions at a location (i.e., cross sectional shape, channel slope, bed and bank roughness), the flow rate at which critical shear values are reached can be calculated (SCCWRP, 2012). For this preliminary analysis, these calculations were made with a programmed spreadsheet using Manning's normal depth equation. The evaluated channel geometry assumed trapezoidal cross-section geometry based on observed bankfull dimensions (i.e., bankfull width, bankfull depth, and bottom width) and . The applied flowrate was iterated until the effective shear stress equaled the assumed critical shear stress using a goal seek function. Results for the channels observed in 2017 are provided in Table B-3.

Determination of Pre-Development 2-Year Flow (Q2)

The pre-development 2-year flow in the channel (Q2) was calculated using an empirical equation documented in the USGS Scientific Investigation Report titled "Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006" (2012). The equation used is as follows:

$$\text{Channel Q2} = 1.82 * \text{Tributary Area}^{0.904} * \text{MAP}^{0.983}$$

Where the Tributary Area is in square miles and Mean Annual Precipitation (MAP) is in inches.

Results for the channels evaluated in 2017 are provided in Table B-3.

Determination of Low Flow Threshold (Qc and Qcp)

The normalized low flow threshold (Qc and Qcp) is determined by dividing Qc by Q2 to get a percentage of Q2. This ratio can eventually be multiplied to the project Q2, calculated using partial duration series analysis of the pre-project continuous simulation, to calculate Qcp in cubic feet per second (cfs). Results for the channels observed in 2017 are provided in Table B-3 and plotted in Figure B-8.

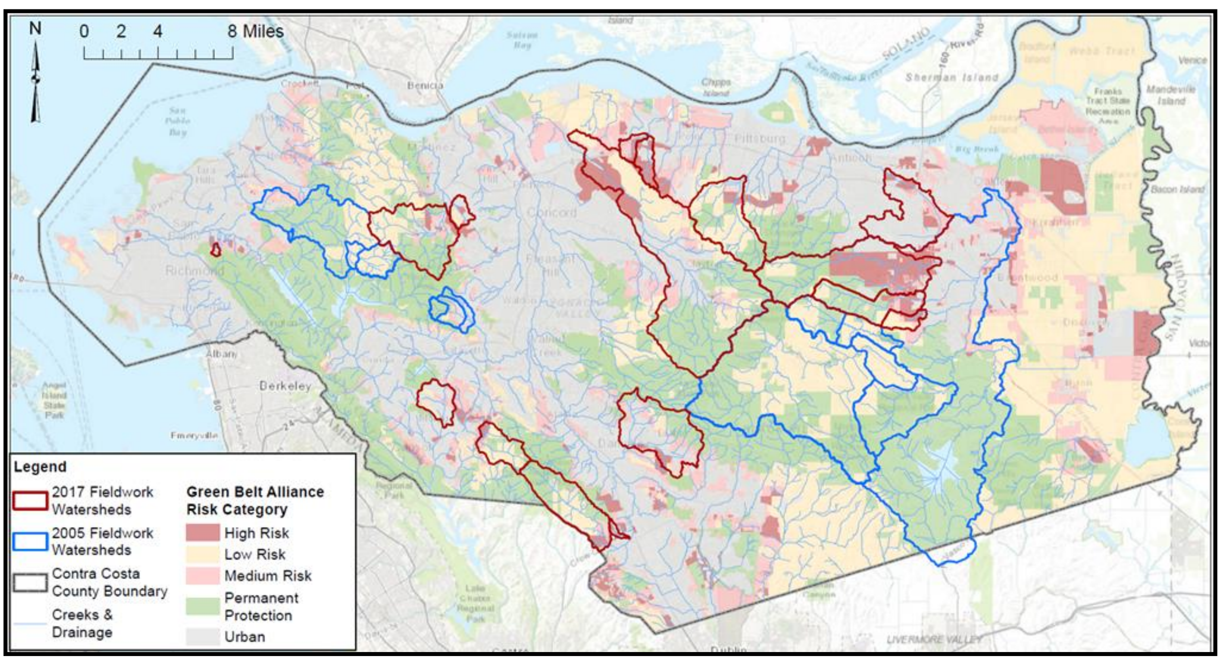

Figure B-1. Location of watersheds tributary to stream channels where geomorphic field work was performed

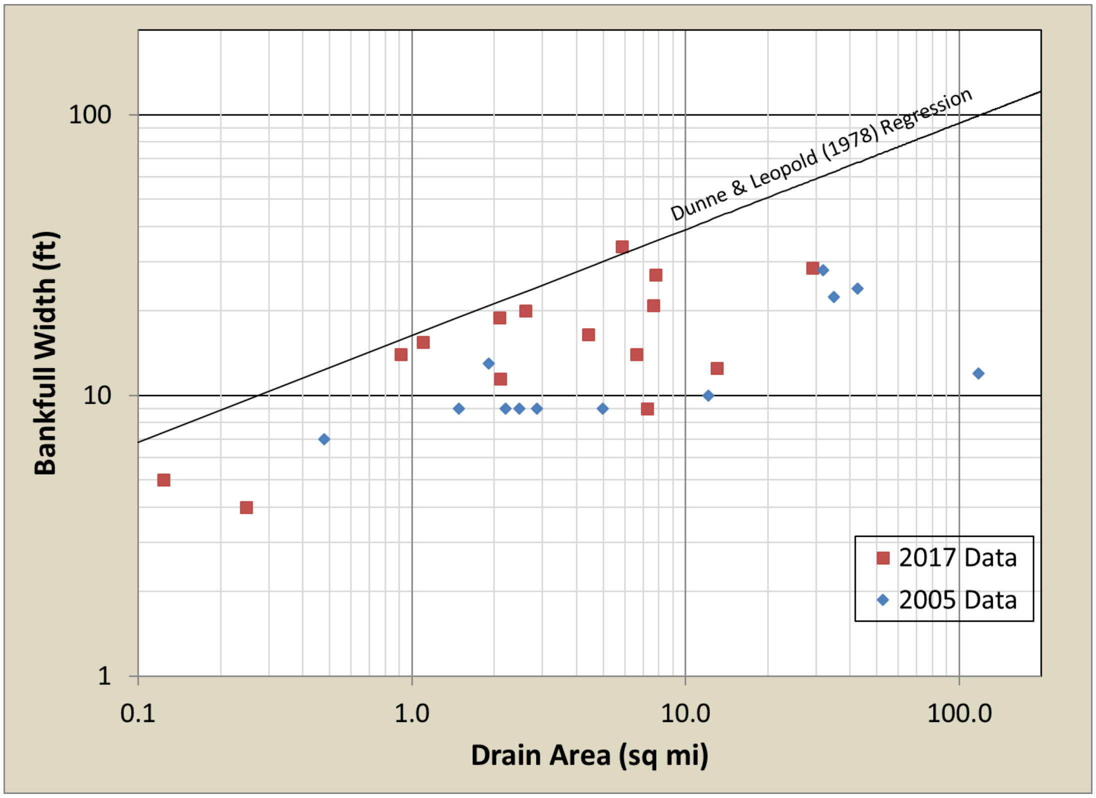

Figure B-2. Distribution plot of bankfull width

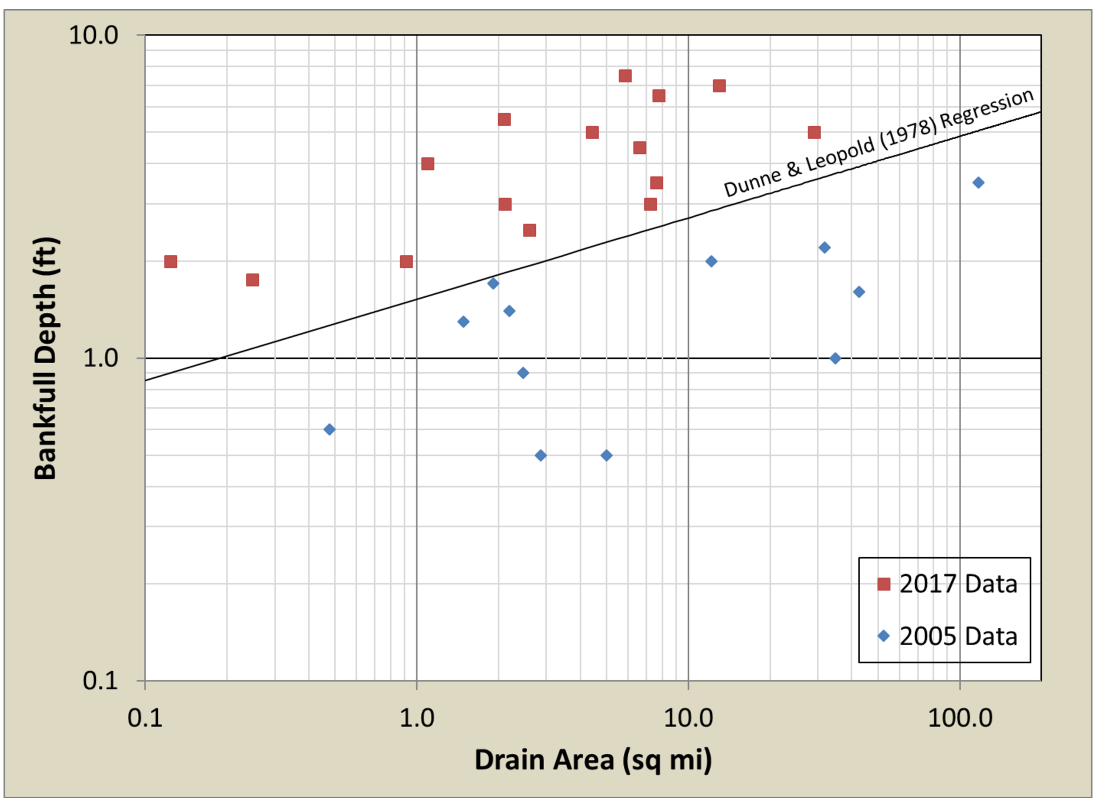

Figure B-3. Distribution plot of bankfull depth

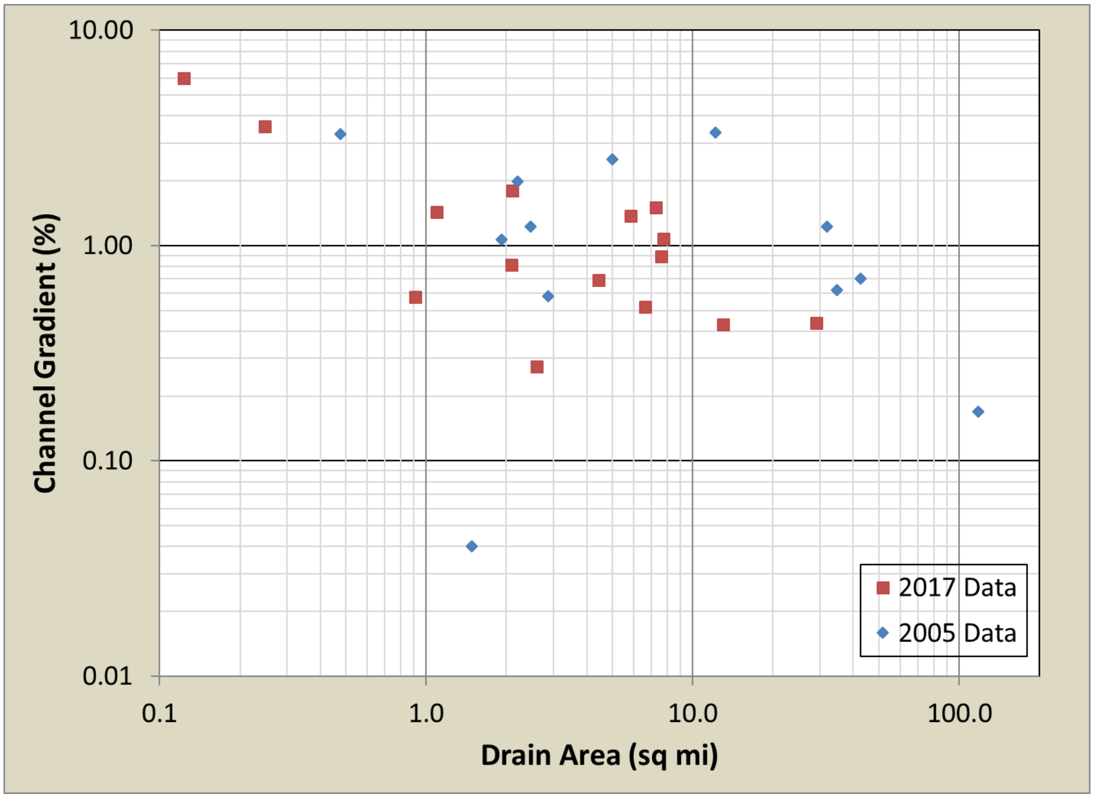

Figure B-4. Distribution plot of channel longitudinal slope

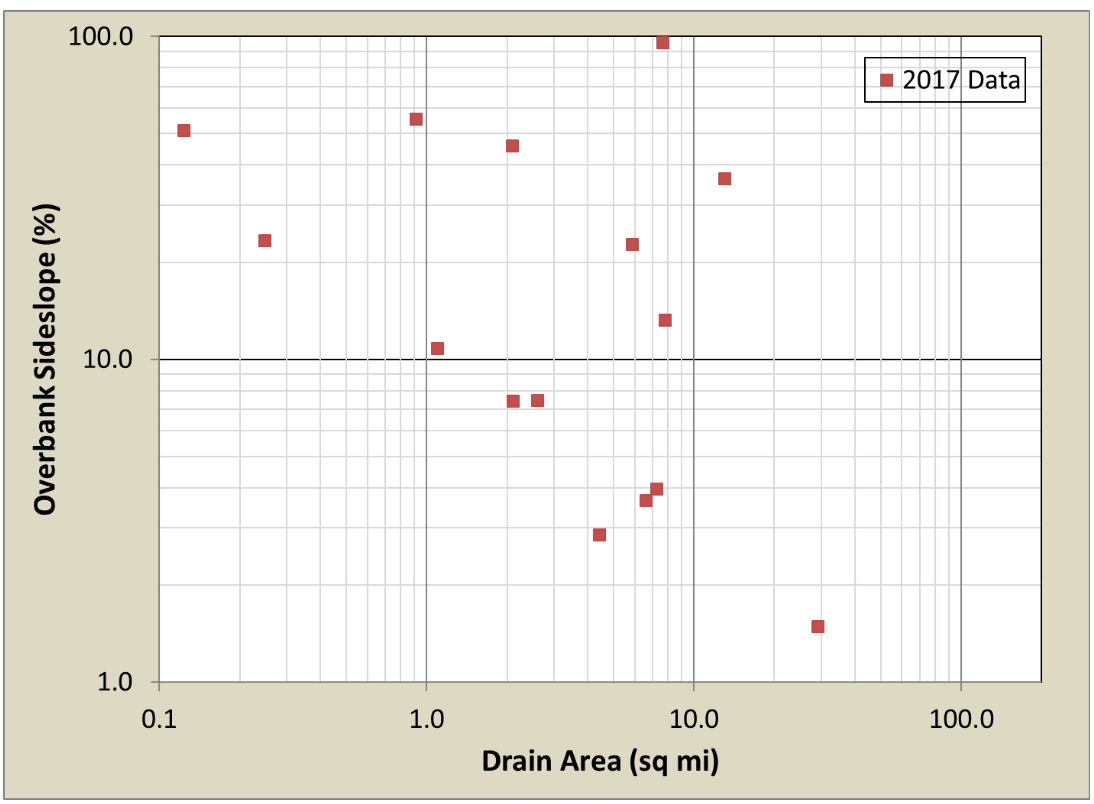

Figure B-5. Distribution plot of overbank side slope

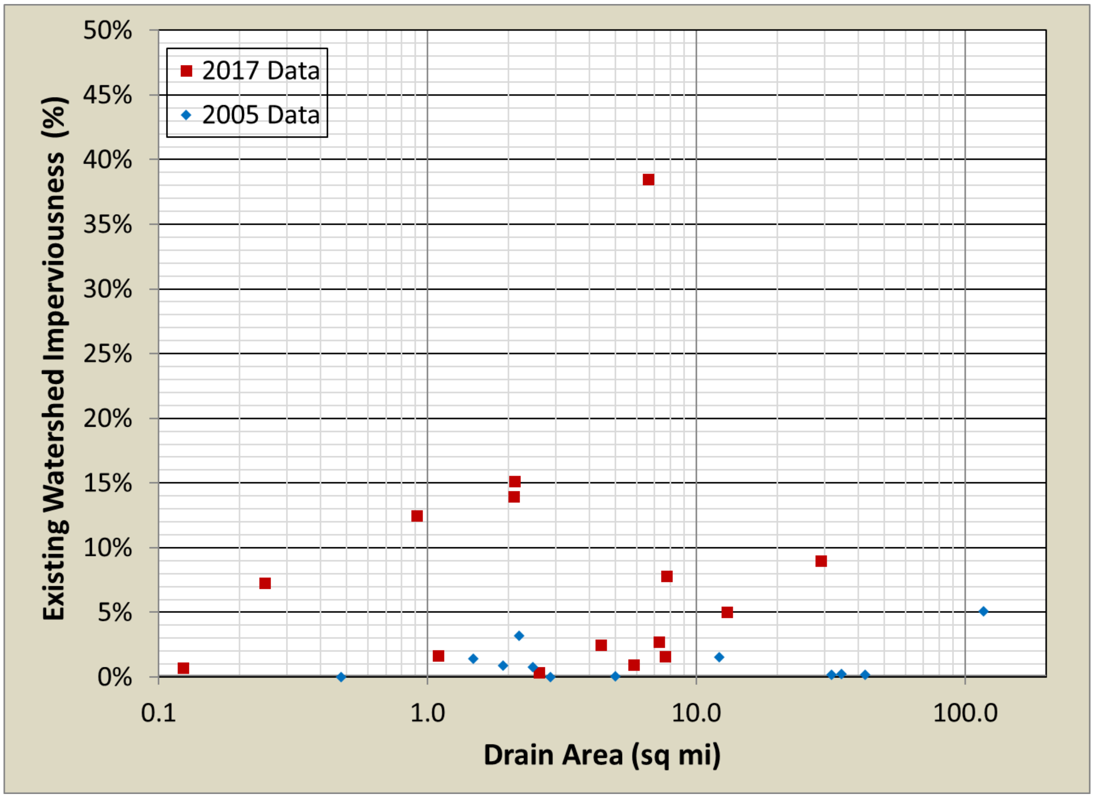

Figure B-6. Distribution plot of existing watershed imperviousness

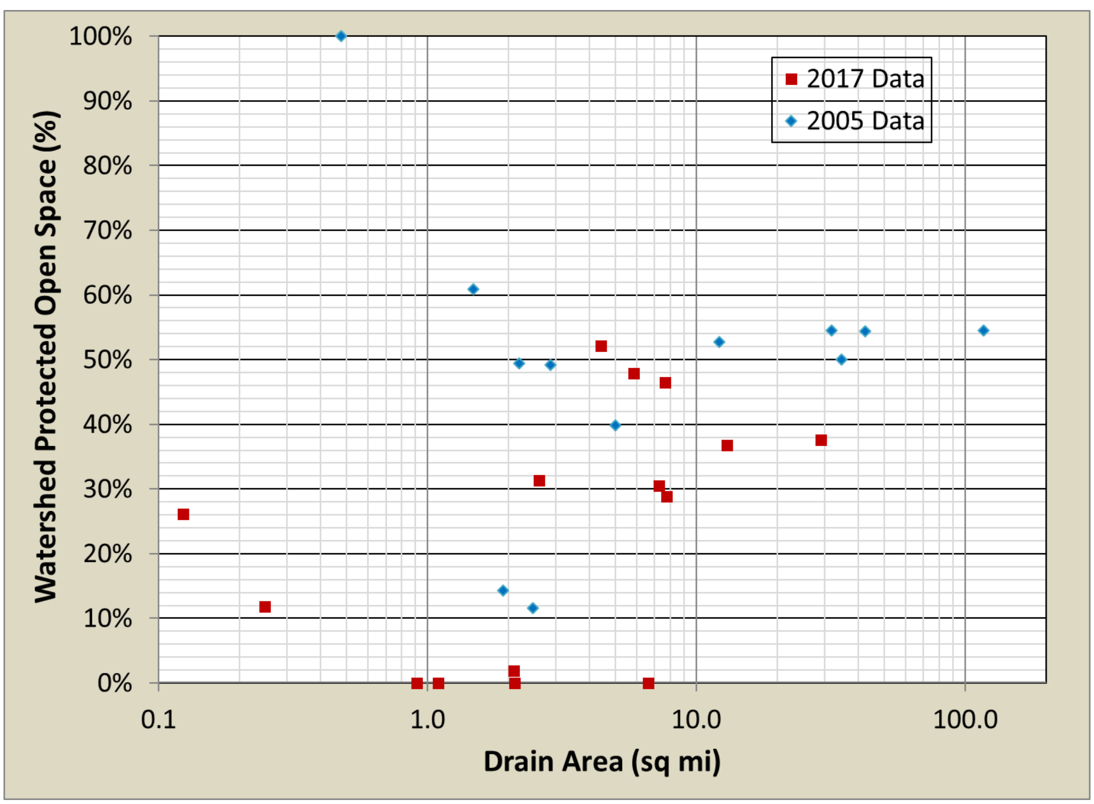

Figure B-7. Distribution plot of watershed protected open space

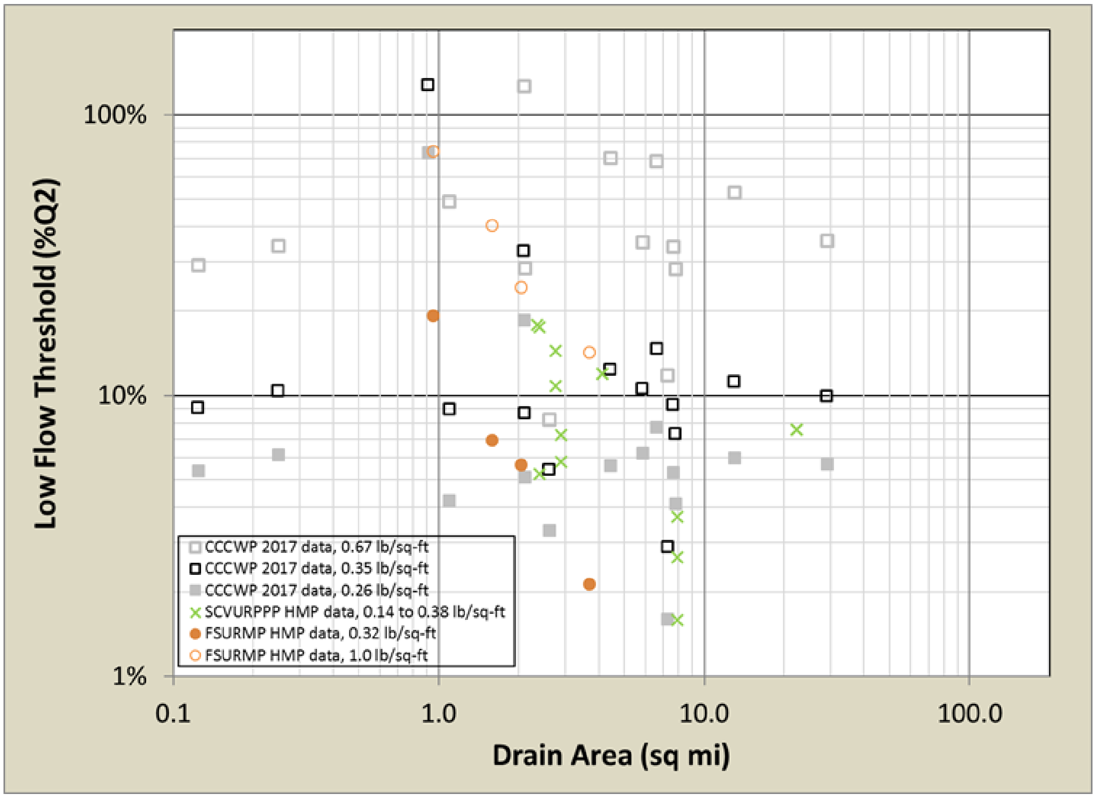

Figure B-8. Distribution plot of low flow threshold

Table B-1. Data Summary Table for Stream Channels Observed and Tributary Watersheds

Date Observed	Time Observed	Site ID	Drainage Area (sq mi)	Bankfull Depth (ft)	Bankfull Width (ft)	Estimated Overbank Slope (%)	Channel Gradient (%)	Bed Materials	Bank Materials	Existing Watershed Impervious ness (%)	Watershed Protected Open Space (%)
2/13/2017	10:08 AM	El Sobrante 15 - La Colina Creek access at Clark Boas Trail	0.1	2.0	5.0	51	5.97	cohesive silt w/ gravel & cobble. Some boulder	cohesive clay/silt, veg on top of bank w/ root reinforcement	1	26
2/13/2017	11:04 AM	Martinez 08 - Alhambra Creek access at Alhambra Ave near Phyllis Terrace	7.6	3.5	21.0	96	0.89	large cobble and gravel embedded in clay/silt, bedrock in some areas	clay/silt w tree roots sporadic. General lack of vegetation near water line	2	46
2/13/2017	12:05 PM	Martinez 09 - Nancy Boyd Creek access at Pleasant Hill Rd and Alhambra Ave	0.9	2.0	14.0	56	0.58	fine gravel with cobble riprap, sand deposit and reeds mid channel, silt underneath	cohesive silt, tall grass	12	0
2/13/2017	1:52 PM	Concord 06 - Mt Diablo Creek access at Diablo Creek Golf Course	29.0	5.0	28.5	1	0.44	silt with gravel	clay/silt with some angular boulder, tall grasses	9	38
2/13/2017	2:34 PM	Bay Point 07 - Unnamed Drainage access at Riverside Dr near Riverview Middle School	2.1	3.0	11.5	7	1.80	vegetated silt bottom	sandy silt, mostly vegetated with grass	15	0
2/13/2017	3:03 PM	Pittsburg 11 - Kirker Creek access at Buchanan Park	7.3	3.0	9.0	4	1.50	sandy silt, some gravel and sand deposits	cohesive silt, grass veg	3	31

Date Observed	Time Observed	Site ID	Drainage Area (sq mi)	Bankfull Depth (ft)	Bankfull Width (ft)	Estimated Overbank Slope (%)	Channel Gradient (%)	Bed Materials	Bank Materials	Existing Watershed Imperviousness (%)	Watershed Protected Open Space (%)
2/15/2017	10:12 AM	Moraga 04 - Laguna Creek access at Corlis Dr and Moraga Rd	2.1	5.5	19.0	46	0.82	cohesive clay/silt with sand and some gravel deposits, cobble, trash	cohesive clay/silt with root reinforcement, some large woody debris	14	2
2/15/2017	11:15 AM	Moraga 03 - Las Trampas Creek access at Bollinger Canyon Rd and Valley Hill Dr	2.6	2.5	20.0	7	0.27	gravel over silty sand	silty sand with sand deposits, cohesive clay/silt at higher elevations, some trees with scrub and brush	0	31
2/15/2017	12:20 PM	Danville 01 - Green Valley Creek access at Matadera Way and Diablo Rd	7.8	6.5	27.0	13	1.08	cobble and gravel with intermixed sand	sandy silt, reinforced with tree roots, sand deposits, riprap just d/s	8	29
2/15/2017	1:30 PM	San Ramon 10 - Bollinger Creek access at Crow Canyon Rd and Bollinger Cyn Rd	5.8	7.5	34.0	23	1.37	cobble with gravel	silty sand conglomerate, gravel embedded, roots, some friable silt stone	1	48
2/15/2017	2:43 PM	San Ramon 02 - trib access from Old Crow Canyon Road	0.2	1.8	4.0	23	3.59	gravel with coarse sand and vegetation, step pools present	clayey silt, vegetated with ivy	7	12
2/23/2017	11:30 AM	Antioch 12 - East Antioch Creek access at Trembath St	6.6	4.5	14.0	4	0.52	cohesive clay/silt with gravel and sporadic cobble	cohesive clay vegetated with grass	38	0

Date Observed	Time Observed	Site ID	Drainage Area (sq mi)	Bankfull Depth (ft)	Bankfull Width (ft)	Estimated Overbank Slope (%)	Channel Gradient (%)	Bed Materials	Bank Materials	Existing Watershed Imperviousness (%)	Watershed Protected Open Space (%)
2/23/2017	12:59 PM	Brentwood 05 - Sand Creek access via Streets of Brentwood parking lot trail access	13.0	7.0	12.5	36	0.43	likely cobble with gravel (couldn't see due to turbid flowing water)	cohesive clay/silt with vegetated grass toward top	5	37
2/23/2017	2:13 PM	Brentwood 14 - Dry Creek access at Mountain View Dr	1.1	4.0	15.5	11	1.43	cohesive clay/silt	cohesive clay/silt with reeds and brush vegetation	2	0
2/23/2017	2:43 PM	Brentwood 13 - Deer Creek access at Mountain View Dr	4.4	5.0	16.5	3	0.69	soft cohesive clay/silt	cohesive clay/silt with grass vegetation	2	52
2005	N/A	32 - Marsh Creek	117.4	3.5	12	N/A	0.17	clay silt	silt loam	5	55
2005	N/A	37 - Marsh Creek	42.6	1.6	24	N/A	0.7	gravel	gravel & silts	0	54
2005	N/A	38 - Marsh Creek	34.8	1	22.5	N/A	0.62	cobble	silt	0	50
2005	N/A	43 - Marsh Creek	5.0	0.5	9	N/A	2.51	clay silt	silty loam	0	40
2005	N/A	56 - Pinole Creek Tributary	0.1	2	2	N/A	4.04	silt/sand	silty sand	4	95
2005	N/A	58 - Releiz Creek	2.2	1.4	9	N/A	1.98	gravel	silt	3	49
2005	N/A	59 - Upper Releiz Creek	0.5	0.6	7	N/A	3.29	Silt	Silt	0	100
2005	N/A	60 - Releiz Creek	1.5	1.3	9	N/A	0.04	gravels & silt	gravel & silts	1	61
2005	N/A	62 - Pinole Creek: Amber Swartz Park	12.1	2	10	N/A	3.34	silty gravel	clay silt	2	53
2005	N/A	64 - Pinole Creek	2.5	0.9	9	N/A	1.23	gravel	silty clay	1	12
2005	N/A	65 - Pinole Creek	1.9	1.7	13	N/A	1.07	pea Gravel	silty clay	1	14
2005	N/A	74 - Briones Valley Headwaters	2.9	0.5	9	N/A	0.58	silty clay	silty clay	0	49
2005	N/A	75 - Marsh Creek Headwaters	31.9	2.2	28	N/A	1.23	cobble/gravel	gravel	0	55

Table B-2. Range of Geomorphic and Watershed Land Use Parameters Used for the Isolated Parameter Sensitivity Analysis

Parameter	# of Iterations	Range of Representative Values	Analysis Type	Basis for Range of Values
Low flow threshold	3	0.1Q2, 0.2Q2 (baseline) and 0.4Q2	Hydrology and Geomorphology	Field work and Northern CA HMPs
Existing watershed imperviousness	3	0 (baseline), 10, and 40%	Hydrology	Geospatial analysis of National Land Cover Dataset (2011) and fieldwork watersheds
Watershed Protected Open Space	3	0% (baseline), 30%, and 50%	Hydrology	Geospatial analysis of Greenbelt Alliance map and fieldwork watersheds
Receiving channel tributary area	4	0.1, 1 (baseline), 10, and 100 square miles	Geomorphology	Geospatial analysis of fieldwork watersheds
Channel bankfull width	3	5.0, 10.0, and 16.3 (baseline) feet	Geomorphology	Fieldwork channel observations
Channel bankfull depth	3	0.6, 1.5 (baseline), and 4.0 feet	Geomorphology	Fieldwork channel observations
Mid channel roughness	3	n = 0.035 (baseline), 0.045, and 0.055	Geomorphology	Fieldwork channel observations
Longitudinal slope	3	0.19 (baseline), 1.4, and 3.5 %	Geomorphology	Topographic map and fieldwork channels
Overbank side slope	3	50:1, 10:1 (baseline), and 2:1 (H:V)	Geomorphology	Fieldwork channel observations
Effective work equation	3	Effective Work Function (LARWQCB, 2012) (baseline), Competent or Limiting Velocity Function (Pemberton and Lara, 1984), Meyer Peter Muller Function (1948)	Geomorphology	Literature review

Table B-3. Data Summary Table for Low Flow Threshold

Date Observed	Time Observed	Site ID	Drainage Area (sq mi)	MAP (in/yr)	Q2 (cfs)	Lower Bound τc (lb/sq-ft)	Mid-Value τc (lb/sq-ft)	Upper Bound τc (lb/sq-ft)	Lower Bound Qc (cfs)	Mid-Value Qc (cfs)	Upper Bound Qc (cfs)	Lower Bound Qc (% of Q2)	Mid-Value Qc (% of Q2)	Upper Bound Qc (% of Q2)
2/13/2017	10:08 AM	El Sobrante 15 - La Colina Creek access at Clark Boas Trail	0.1	23.4	6.1	0.26	0.35	0.67	0.3	0.6	1.8	5%	9%	29%
2/13/2017	11:04 AM	Martinez 08 - Alhambra Creek access at Alhambra Ave near Phyllis Terrace	7.6	23.4	253.4	0.26	0.35	0.67	13.5	23.6	86.3	5%	9%	34%
2/13/2017	12:05 PM	Martinez 09 - Nancy Boyd Creek access at Pleasant Hill Rd and Alhambra Ave	0.9	21.3	33.9	0.26	0.35	0.67	24.9	43.4	67.7	74%	128%	200%
2/13/2017	1:52 PM	Concord 06 - Mt Diablo Creek access at Diablo Creek Golf Course	29.0	26.4	955.8	0.26	0.35	0.67	54.7	95.0	340.9	6%	10%	36%
2/13/2017	2:34 PM	Bay Point 07 - Unnamed Drainage access at Riverside Dr near Riverview Middle School	2.1	21.8	73.7	0.26	0.35	0.67	3.8	6.4	21.0	5%	9%	29%
2/13/2017	3:03 PM	Pittsburg 11 - Kirker Creek access at Buchanan Park	7.3	14.4	150.5	0.26	0.35	0.67	2.4	4.4	17.8	2%	3%	12%
2/15/2017	10:12 AM	Moraga 04 - Laguna Creek access at Corlis Dr and Moraga Rd	2.1	17.5	59.0	0.26	0.35	0.67	10.9	19.4	75.0	19%	33%	127%

Date Observed	Time Observed	Site ID	Drainage Area (sq mi)	MAP (in/yr)	Q2 (cfs)	Lower Bound τc (lb/sq-ft)	Mid-Value τc (lb/sq-ft)	Upper Bound τc (lb/sq-ft)	Lower Bound Qc (cfs)	Mid-Value Qc (cfs)	Upper Bound Qc (cfs)	Lower Bound Qc (% of Q2)	Mid-Value Qc (% of Q2)	Upper Bound Qc (% of Q2)
2/15/2017	11:15 AM	Moraga 03 - Las Trampas Creek access at Bollinger Canyon Rd and Valley Hill Dr	2.6	17.5	71.7	0.045	0.06	0.075	2.4	3.9	5.9	3%	5%	8%
2/15/2017	12:20 PM	Danville 01 - Green Valley Creek access at Matadera Way and Diablo Rd	7.8	16.1	178.6	0.26	0.35	0.67	7.4	13.1	50.5	4%	7%	28%
2/15/2017	1:30 PM	San Ramon 10 - Bollinger Creek access at Crow Canyon Rd and Bollinger Cyn Rd	5.8	16.2	138.3	0.26	0.35	0.67	8.6	14.7	48.7	6%	11%	35%
2/15/2017	2:43 PM	San Ramon 02 - trib access from Old Crow Canyon Road	0.2	24.2	11.8	0.26	0.35	0.67	0.7	1.2	4.1	6%	10%	34%
2/23/2017	11:30 AM	Antioch 12 - East Antioch Creek access at Trembath St	6.6	24.2	230.0	0.26	0.35	0.67	17.8	33.8	158.1	8%	15%	69%
2/23/2017	12:59 PM	Brentwood 05 - Sand Creek access via Streets of Brentwood parking lot trail access	13.0	24.1	422.3	0.26	0.35	0.67	25.4	47.5	223.9	6%	11%	53%
2/23/2017	2:13 PM	Brentwood 14 - Dry Creek access at Mountain View Dr	1.1	24.1	45.2	0.26	0.35	0.67	1.9	4.0	22.2	4%	9%	49%

Date Observed	Time Observed	Site ID	Drainage Area (sq mi)	MAP (in/yr)	Q2 (cfs)	Lower Bound τc (lb/sq-ft)	Mid-Value τc (lb/sq-ft)	Upper Bound τc (lb/sq-ft)	Lower Bound Qc (cfs)	Mid-Value Qc (cfs)	Upper Bound Qc (cfs)	Lower Bound Qc (% of Q2)	Mid-Value Qc (% of Q2)	Upper Bound Qc (% of Q2)
2/23/2017	2:43 PM	Brentwood 13 - Deer Creek access at Mountain View Dr	4.4	21.2	140.5	0.26	0.35	0.67	7.9	17.4	99.0	6%	12%	70%

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