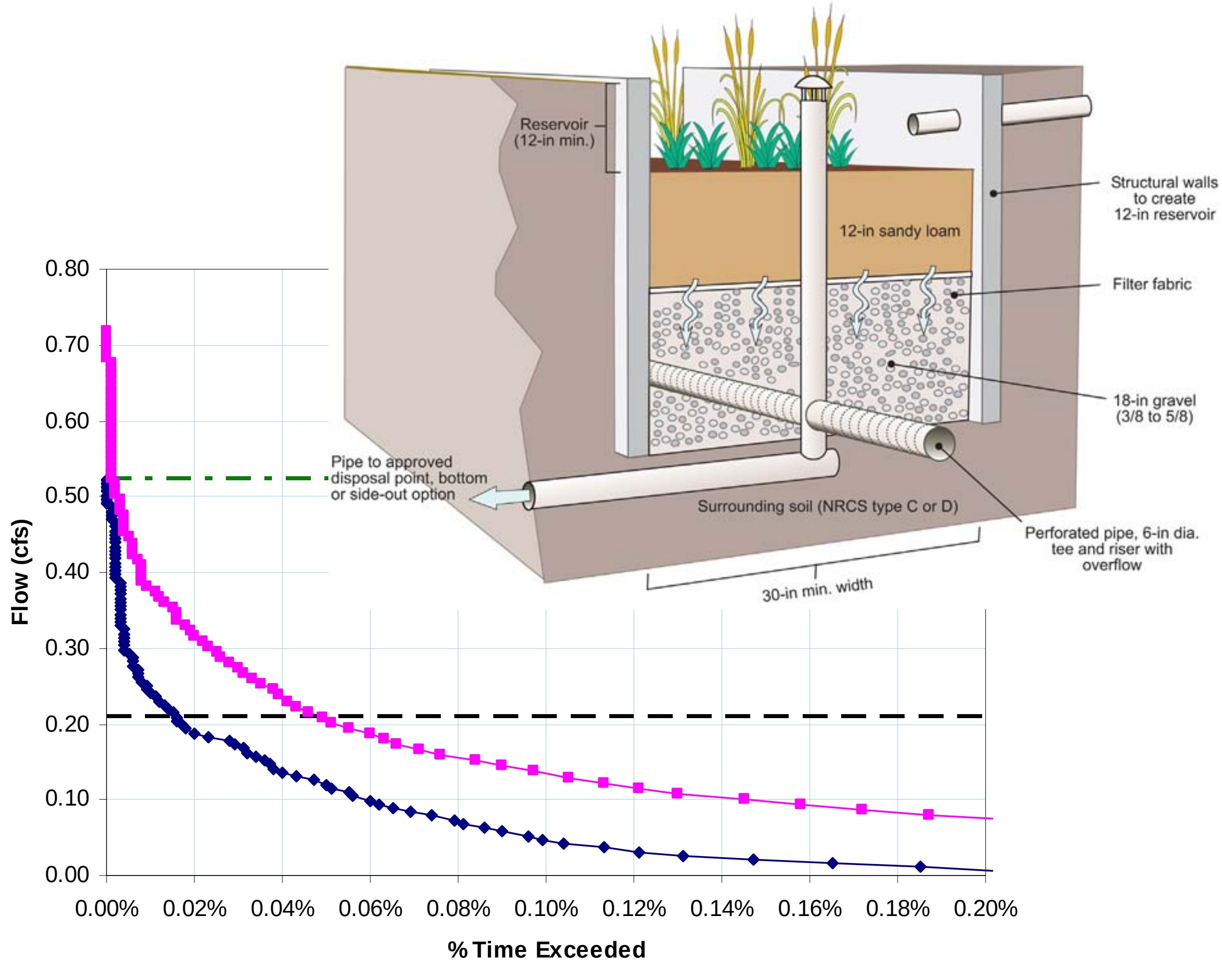
Contra Costa
Clean Water Program

Hydrograph Modification Management Plan



May 15, 2005

Contra Costa Clean Water Program
Hydrograph Modification Management Plan

Contra Costa Clean Water Program
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May 15, 2005

Corrected April 19, 2006 (corrections listed on following page)

Corrections made April 19, 2006

In Attachment 2, on page 14, label on Figure 2 has been corrected. The depth of 12 inches of sandy loam has been corrected to 18 inches, consistent with the text and the underlying modeling.

In Attachment 2, on page 29, a label on Figure 12 has been corrected. The depth of 12 inches of sandy loam has been corrected to 18 inches, consistent with the text and the underlying modeling.

In Attachment 4, on page 13, a paragraph in Section 5.4.5 has been corrected to read as follows: "An evaluation should be made of active sedimentation features such as channel deposits, multiple channels, mid channel bars, as well as sediment sources adjacent to the channel, such as bare or unvegetated soils and actively widening banks. Assessment of active sedimentation should include observations of the channel bed, and all banks adjacent to the channel, as well as other proximate sediment sources. Classes for active sedimentation include:" This replaces text inadvertently repeated from Section 5.4.4.

CONTRA COSTA CLEAN WATER PROGRAM

Donald P. Freitas
Program Manager

May 15, 2005

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Re: Hydrograph Modification Management Plan (HMP)

Dear Messrs. Wolfe and Pinkos:

This Hydrograph Modification Management Plan is submitted pursuant to Provision C.3.f of San Francisco Bay Board Order R2-2003-0022. The HMP, once approved by the Water Board, will be implemented so that "… post-project runoff shall not exceed estimated pre-project rates and/or durations, where the increased stormwater discharge rates and/or durations will result in increased potential for erosion …."

Contents of this Submittal

Our HMP includes this cover letter and Attachments 1-6.

This cover letter includes:

- Summary of HMP Features
- Background

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- Approach and Rationale
- Implementation Plan and Schedule
- Continuous Improvement

The attachments are as follows:

- Attachment 1 summarizes the Program's standards and criteria for implementing the HMP. All Co-permittees will require new and re-development projects subject to Provision C.3.f to meet these standards and criteria.
- Attachment 2 is a technical memorandum detailing development of sizing factors for hydrograph modification integrated management practices (IMPs). Applicants for development approvals in Contra Costa County may include these IMPs in their development projects as a means of attaining compliance with the Program's HMP standards.
- Attachment 3 is a technical memorandum specifying how applicants may use computer models and long-term hourly rainfall records to simulate runoff peaks and durations and demonstrate HMP compliance.
- Attachment 4 is a technical memorandum with methods and criteria for assessing and classifying stream reaches according to risk of accelerated erosion due to increases in runoff.
- Attachment 5 is a chronology of previous submittals pursuant to Provision C.3.f, correspondence with Water Board staff, and public outreach related to the Program's development of the HMP.
- Attachment 6 is a response to Water Board staff comments on the Program's November 15, 2004 draft HMP.

Summary of HMP Features

The distinguishing features of the Program's HMP are:

- Our HMP is ready to be implemented now. The Program will incorporate HMP requirements into a new, third edition of the Program's *Stormwater C.3 Guidebook* ninety days following the Water Board's final approval of the HMP. All Contra Costa municipalities would begin implementing the new requirements for project applications deemed complete after that date.
- To facilitate review by the Water Board and Water Board staff, our HMP standards are stated succinctly in Attachment 1. These standards will be incorporated into the

Stormwater C.3 Guidebook. The supporting information and guidance in the accompanying technical memoranda (Attachments 2, 3, and 4) will be adapted to the *Guidebook's* user-friendly format.

- Our HMP standards for control of runoff peaks and durations are based on continuous simulation of runoff using local rainfall data and locally derived parameters for initial infiltration. Our consultants used USEPA's Hydrologic Simulation Program—Fortran (HSPF), which is the same program and method that underlies the Western Washington Hydrology Model. The standards we propose are adapted from the Western Washington standards.

- Our HMP promotes the use of IMPs to provide both stormwater treatment and flow control. The HMP includes standard designs and details for IMPs. The Program's IMP sizing factors were derived by analyzing continuous simulations of runoff in HSPF.

- Our HMP allows applicants to propose in-stream restoration, in lieu of flow control, where the benefits of a proposed restoration project substantially outweigh the potential impacts of additional runoff of the proposed development project. In stream reaches where accelerated erosion due to increased watershed imperviousness is not likely but is possible ("medium risk"), applicants may use a somewhat streamlined analysis. In stream reaches where increases in runoff flows are likely to accelerate bed and bank erosion, applicants must conduct a comprehensive geomorphic watershed analysis before proposing an in-stream project in lieu of flow control.

- We propose no exemptions, other than those stated in the permit, for project size. However, should the Water Board provide such exemptions in other counties, we will add the exclusion to our HMP standard.

- We propose no exemptions for "infill projects in highly developed watersheds." Many projects in dense urban areas may not need to fully control runoff peaks and durations because they will drain to hardened channels or will replace existing impervious area. We feel it would be difficult to create and implement workable, consensus, quantitative criteria for "infill projects in highly developed watersheds." Instead, the Program has focused on developing the technical means to assist applicants to achieve peak flow and duration control on small, urban sites using IMPs. Should the Water Board accept a workable definition of "infill projects in highly developed watersheds" in another county, we will add the definition and exemption to our HMP standard.

- We propose no exemption based on project cost, because we believe the IMPs and procedures we have developed will make it possible for applicants to achieve HMP compliance at reasonable cost. We also note the difficulty inherent in assessing, categorizing, and monitoring project expenditures, particularly after a project receives planning approval. Should the Water Board accept exemptions from permit requirements based on project cost in another county, the Program will add these exemptions to our HMP standard.

Background

As noted in Water Board staff's fact sheet accompanying Order R2-2003-0022, increases in runoff flows associated with urbanization may accelerate erosion and sediment transport in downstream watercourses, cause more frequent flooding, and widen and downcut stream channels. These impacts, in turn, can degrade stream aesthetics, habitat, and other beneficial uses.

Provision C.3.f requires the Co-permittees to implement an HMP to "…manage increases in peak runoff flow and increased runoff volume, for all Group I projects, where such increased flow and/or volume is likely to cause increased erosion of creek beds and banks, silt pollutant generation, or other waterbody impacts to beneficial uses due to increased erosive force."

The Water Board staff fact sheet describes the HMP as "an analytical method, with the inclusion of available relevant data, which a developer employs to demonstrate to the Permittees that the eventual design for the project will not lead to damaging flow impacts, when mitigative measures are included in the project."

To put the Program's approach to developing the analytical method in perspective, we note:

- Analytical methods used to evaluate hydrologic and geomorphic processes are subject to considerable uncertainty. These uncertainties are related both to the variability inherent in natural systems and to ongoing changes in scientific understanding of those systems. Mechanistic models currently available can provide insight into stream processes, but cannot provide precise predictions of stream response to hydrologic changes.
- Implementation is the key to a viable and effective HMP. Whatever analytical method or design standards are chosen, they must support the design of facilities that can be practically built and maintained in the high-value, high-density development sites typical of Contra Costa County and the Bay Area. Because the HMP will be implemented through the municipal development review process, it must specify design criteria that can be incorporated into conditions of approval.

To develop a practical HMP analytical method, Program staff and consultants have drawn on the scientific and professional disciplines of hydrology (including computer simulation of rainfall, infiltration, storage, and runoff), and stream geomorphology. Each of these disciplines employs accumulated empirical evidence, professional experience, and analysis within a generalized scientific framework. Some members of the consultant team have long-term experience in the development and application of the Western Washington Hydrology Model. Other team members have many years' experience analyzing the impacts of development on Contra Costa streams and designing stream restoration projects to mitigate those impacts. In addition, program staff and consultants have followed closely the joint effort by the Santa Clara Valley Urban Runoff Pollution Prevention Program and the Santa Clara Valley Water District to develop an HMP for municipalities in northern Santa Clara County.

This HMP provides straightforward compliance standards and tools (including design procedures, criteria, and standard details for IMPs) to assist applicants to demonstrate compliance with the Program's standards. The design criteria are conservative but reasonable. The IMPs have been selected and designed to perform reliably and consistently in the variety of different locations, soil types, and modes of development found in Contra Costa County.

HMP Approach and Rationale

Integration with other C.3 Provisions and with the development review process

Each Co-permittee has adopted an updated stormwater ordinance, based on a model provided by the Program, mandating implementation of the Provision C.3 requirements. The ordinances require development permit applications be accompanied by a Stormwater Control Plan that meets the criteria in the most recent version of the Program's *Stormwater C.3 Guidebook.*

The *Guidebook* includes step-by-step instructions to assist applicants to prepare Stormwater Control Plans. Each project's Stormwater Control Plan documents—in a consistently organized, easy-to-check format—that the project has been planned and designed to comply with each of the C.3 provisions.

The current *Guidebook* (second edition, March 2005) already contains instructions for minimizing impervious area, creating "self-retaining" areas that are disconnected from the drainage system, and for selecting and sizing integrated management practices (IMPs) to treat runoff in compliance with the requirements of Provision C.3.d.

The ordinances and *Guidebook* also already require each site with treatment facilities to have a Stormwater Facilities Operation and Maintenance Plan. These same operation and maintenance requirements will be extended to hydrograph modification management facilities.

Implementation of the other C.3 provisions began February 15, 2005. The Co-permittees' initial experience working with the first applications for projects subject to the C.3 requirements tends to validate the *Guidebook* approach.

Following approval of this HMP by the Water Board, the Program will incorporate HMP requirements into a third edition of the *Guidebook.* Applicants' documentation of compliance with the HMP will be incorporated into their Stormwater Control Plans.

HMP Standard and Methods of Demonstrating Compliance

Attachment 1 is the Program's HMP Standard. Applicants may demonstrate compliance by any of the following methods:

1. Show there will be no increase in directly connected impervious area.
2. Use IMPs that meet Program design requirements to control all runoff from new impervious areas.
3. Model and compare post-project to pre-project runoff peaks and durations.
4. Show projected increases in runoff peaks and/or durations will not accelerate erosion of receiving stream reaches.

The discussion below articulates the background and rationale for each method and addresses how the Co-permittees will implement each method.

Method #1: Demonstrate there is no increase in directly connected impervious area.

Hydrograph modification impacts of development are caused by the replacement of pervious soil with impervious surfaces such as rooftops and paving. Therefore, the *Guidebook* will include instructions for applicants to document, measure, and compare pre-project to post-project directly connected impervious area (DCIA). If there is no change in DCIA, it is assumed that the major hydrograph modification impacts are avoided. Other provisions, described below, address potential impacts not related to DCIA.

Development projects may also alter the infiltration characteristics of pervious areas, either decreasing or increasing the amount of runoff as compared with pre-project conditions. In the case of “D” soils characteristic of much of the county, soil amendments needed to support landscaping will increase infiltration rates and moisture retention, reducing runoff rates. In addition, the Program’s *Guidebook* requires applicants implement, to the maximum extent practicable, landscape features that minimize runoff including (for example) concave medians. Using the *Guidebook* step-by-step instructions for drainage design, applicants are strongly encouraged to make landscaped areas “self-retaining,” and thereby avoid the cost and trouble of treating runoff from these areas.

A related concern expressed by Water Board staff is that in some developments, a new drainage system could convey runoff more efficiently, even where the impervious area decreases or remains the same. Under such a scenario, volumes would remain the same or decrease, durations would decrease, but peak runoff rates would increase. This scenario is addressed by the Program’s approach to implementing stormwater treatment requirements. Following the instructions in the *Stormwater C.3 Guidebook*,[1] applicants’ designs must direct runoff to self-retaining areas (at a maximum 2:1 ratio of impervious:pervious area) or route runoff from impervious areas to treatment IMPs such as swales, planter boxes, and bioretention areas. These treatment IMPs detain runoff in a surface reservoir, filter it through 18 inches of soil, collect the filtrate in a subsurface trench filled with aggregate, and then allow the treated runoff to either seep into the ground or into perforated pipes (to be collected and transported to the storm drain system).

Although not specifically designed for flow control, these facilities extend the time of concentration, particularly for small storms. As an additional method to ensure against increases in peak flow, the *Guidebook* will require applicants to include, in their stormwater control plans, a qualitative comparison of pre-project and post-project drainage efficiency.

This method allows applicants for projects on previously developed sites, where an increase in impervious area is not proposed, to easily and simply demonstrate HMP compliance. Where applicable, this option will obviate any need to document an HMP exemption based on impracticability, proportion of watershed “build-out,” proximity to transit, or characteristics of the receiving stream.

[1] In the second edition, the instructions begin on page 53.

Method #2: Use Hydrograph Modification Integrated Management Practices (IMPs)

The Program has developed standard specifications and sizing criteria for seven hydrograph modification IMPs. The specifications and sizing criteria will be included in the *Stormwater C.3 Guidebook,* along with instructions for site planning for IMP implementation. The applicant will first divide the project site into discrete drainage areas, disconnecting impervious areas from the drainage system and creating "self-retaining areas" wherever possible. Drainage from remaining impervious areas is routed to IMPs.

Versions of these IMPs—suitable for meeting the runoff treatment requirements of Permit Provisions C.3.c and C.3.d—are already included in the second edition of the Program's *Stormwater C.3 Guidebook*.[2] The Program has adapted the treatment IMP standard specifications to maximize runoff storage and to meter outflow to manage hydrograph modification impacts in addition to providing runoff treatment. The Program has also developed a spreadsheet-based automated sizing tool that will allow applicants to select suitable hydrograph modification IMPs and readily obtain the required dimensions. Attachment 2 includes the designs of each IMP and details how the sizing criteria were developed. A summary follows.

Sizing factors reflect the surface area of an IMP that is required to manage runoff from a given impervious drainage area, expressed as a percentage of the drainage area. That is, if a 50 square foot IMP is required to manage runoff from a 1000 square foot impervious drainage area, the sizing factor for that IMP is 0.05. The HMP standard for managing runoff is described in Attachment 1. Program consultants used the USEPA's Hydrologic Simulation Program—Fortran (HSPF) computer model and a 35-year record of hourly rainfall data from a gauge in Martinez to develop sizing factors for the seven IMPs included in the *Guidebook*. The following steps were performed:

- Compute hourly runoff for pervious and impervious unit areas for the 35-year period.
- Establish stage-storage-discharge relationships based on the IMP designs.
- Route runoff from the impervious unit area to each IMP.
- Track and analyze infiltration and outflow from each IMP's overflow and underdrain (if any) to compute hourly discharge from the IMP.
- Compare model results for IMP discharges to those for pervious unit area runoff.
- Adjust each IMP size until the required control of peaks and durations is achieved.
- As a final step, establish correction factors for differences in rainfall patterns within the County.

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[2] See the fact sheets in Attachment C-1 to *Guidebook* Appendix C.

To analyze the effectiveness of IMPs in controlling runoff peaks and durations, Program consultants compared two sets of runoff curves representing pre-project post-project, and post-project-with-IMP conditions.

The first set of curves reflects “Flow Duration Statistics” for the various model runs. These curves are plots of runoff flow (in cubic feet per second) against the percent of time that flow is exceeded *during the entire period of rainfall record.* That is, the curves reflect the number of hours that a given flow rate is exceeded over the 35-year simulation period. The flow duration curves are the most direct and complete way of representing the duration of flows in a critical range, regardless of the storm event that generates those flows. For HMP purposes, critical flows are those most likely to move sediment (i.e., cause erosion). The consultant team compared pre-project and IMP flow duration curves to select IMP sizing factors that adequately control flow durations within the critical range of flows.

The second set of curves reflects “Peak Flow Statistics” for the various model runs. These curves are plots of the highest flow that occurs, on average, during any 1 year, 2 year, 3 year, and on up to any 10-year period within the rainfall record. The peak flow curves provide a way to evaluate peak flows associated with storm events by looking at their frequency of occurrence. The consultant team compared pre-project and IMP peak flow curves to select IMP sizing factors that control flow peaks within the critical range of events.

Visual comparison of the post-project curves to the pre-project curves (Figures 8-11, 13-14, 17-20, 22-25, 27-28, 30-31, and 33-34 in Attachment 2) shows that all seven IMPs effectively control post-project flows to pre-project conditions. Post-project flows are always below pre-project flows in the most critical portions of the curves (between one half the flow of the pre-project runoff event with an average recurrence of two years, or 0.5Q2, and the flow of the pre-project runoff event with an average recurrence of 10 years, or Q10) and are below or close to pre-project flows in the less-critical portions of the curves.

The resulting sizing factors are adjusted for different patterns of rainfall throughout the County. IMPs discharge differently in Group A or B soils vs. Group C or D soils (i.e. via infiltration vs. underdrains) so different adjustment factors are applied for IMPs associated with the different soil groups.

Use of IMPs will provide reasonable certainty that a project will not cause increased flow peaks and durations in the range most likely to increase erosion or have other significant effects on beneficial uses. The IMPs provide a cost-effective, constructible option for HMP compliance on small and large sites—and meet the requirements for stormwater treatment as well.

Because hydrograph modification IMPs also act as treatment IMPs—and because their cost and space requirements are only marginally higher than for treatment IMPs—we expect most applicants, particularly those proposing smaller developments, will select this option rather than seek compliance through site-specific runoff modeling (Method #3) or addressing characteristics of the receiving stream (Method #4).

IMPs could be designed to provide even more control of outflows in the range of flows below 0.5Q2. This would be accomplished by reducing allowable underdrain outflow and increasing the sizing factors. The Program rejected this idea because (1) we believe the current sizing factors achieve the HMP standard, as evidenced by a comparison of the resulting runoff curves, and (2) it would make the IMPs less attractive to applicants, thereby undermining the advantages to be had by promoting the use of IMPs.

The consultant team used conservative assumptions to model pre-project runoff and IMP performance. Further modeling, collection of field data, and calibration are proposed as continuous improvement tasks and could result in adjustments to sizing factors. (See discussion below under the “Continuous Improvement” heading.)

Method #3: Model and Compare Pre- and Post-Project Runoff

Although the Program’s IMP designs provide a simple and flexible means for most applicants to demonstrate compliance with the HMP standard, some applicants may wish to use other devices or strategies. Examples:

- Detention basins for flow control.
- IMPs in series, such as a flow-through planter draining to a swale, where the size of each IMP may be reduced accordingly.
- Water features, such as ponds or constructed wetlands, for hydrograph modification management.
- IMPs not included in the *Guidebook*, such as cisterns or rooftop detention.

To provide flexibility and encourage innovation, Co-permittees may allow applicants to demonstrate compliance with the HMP by modeling and comparing post-project site runoff (with and without hydrograph modification management) to pre-project site runoff.

To use this method, applicants will need to arrange for an experienced hydrologic modeler to simulate runoff from the site in its existing (pre-project) condition. The modeler will also need to establish runoff routing and stage-storage-discharge relationships for the planned detention and infiltration facilities.

Attachment 3 is the Program's guidance for modeling pre- and post-project runoff using HSPF. The guidance includes values or ranges of the key parameters, instructions for representing detention facilities, and instructions for presenting and analyzing output data.

Output data will be evaluated using the following standard, which was adapted from the Washington Department of Ecology's standard:

1. Post-project peak flows shall not exceed pre-project peak flows for recurrence intervals up to two years (Q2). For peak flows with recurrence intervals of two years through ten years (Q2 through Q10), post-project peak flows may exceed pre-project peak flows by up to 10% within a 1-year-wide band. For example, the post-project flows could exceed the pre-project flows by up to 10% between Q9 and Q10 or from Q5.5 to Q6.5, but not from Q8 to Q10.

2. Post-project runoff durations, from one-half the pre-project 2-year runoff event (0.5Q2) to Q2 shall not exceed pre-project runoff durations. For flow rates above Q2, post-project durations shall be less than or equal to pre-project runoff durations, except that the post-project duration may exceed the pre-project durations for no more than 10% of the time.

Method #4: Demonstrate projected increases in runoff peaks and durations will not accelerate erosion of receiving stream reaches.

This HMP encourages applicants for development approvals to use IMPs to control runoff peaks and durations to pre-project levels, where possible, rather than seeking exemptions or implementing in-stream mitigation. The Co-permittees would allow increases in runoff peaks and durations only when the applicant can show, with reasonable certainty, that the increases would not accelerate erosion in receiving streams or potentially degrade beneficial uses, or that the potential for erosion or other impacts to beneficial uses is minimal.

Provision C.3.f.(ii) states in part:

> [HMP requirements do] not apply to new development and significant redevelopment projects where the project discharges stormwater runoff into creeks or storm drains where the potential for erosion or other impacts is minimal. Such situations may include discharges into creeks that are concrete-lined or significantly hardened (e.g., with rip-rap, sackcrete, etc.) downstream to their outfall in San Francisco Bay, underground storm drains discharging to the Bay, and infill projects in highly developed watersheds, where the potential for

> single-project and/or cumulative impacts is minimal. Guidelines for such situations shall be included as part of the HMP.

To implement this requirement, the Program has prepared a definition of a "low-risk" stream. The definition is in paragraph 4.a. of Attachment 1. Projects in such situations must still reduce imperviousness to the maximum extent practicable, and will still include treatment BMPs, but need not match post-project runoff peaks and durations to pre-project peaks and durations.

In other streams, it may be possible and appropriate, in some circumstances, to allow applicants to mitigate potential effects of increased runoff by implementing in-stream restoration techniques.

Provision C.3.f.(vii) states:

> The Dischargers may develop an equivalent limitation protocol, as part of the HMP, to address impacts from changes in the volumes, velocities, and/or durations of peak flows through measures other than control of those volumes and/or durations. The protocol may allow increases in peak flow and/or durations, subject to the implementation of specified design, source control, and/or treatment control measures and land planning practices that take into account expected stream change (e.g., increase in the cross-sectional area of stream channel) resulting from changes in discharge rates and/or durations, while maintaining or improving beneficial uses of waters.

Although some Contra Costa streams are in good condition, there is a substantial backlog of needed restoration work. Some of these restoration projects would require extensive watershed analysis before proceeding to design; others (particularly on streams which have generally stable beds but also have eroding bends or bank failures) could proceed quickly following analysis of the localized problem.

The HMP standard (Attachment 1) accommodates these differing situations by distinguishing "medium risk" vs. "high risk" of accelerating stream erosion. Attachment 4 includes a detailed methodology and instructions for classifying streams and development situations as "medium risk" vs. "high risk".

In a "medium risk" stream reach, accelerated erosion due to increased watershed imperviousness is not likely but is possible, and the uncertainties can be more easily and effectively addressed by a mitigation project than by additional study.

After a preliminary report indicates a project presents a "medium risk," the applicant has the option—as an alternative to matching post-project to pre-project runoff peaks and durations—to propose a mitigation project. If a suitable project exists in the same stream reach or watershed, that project should be proposed; otherwise, a project in another watershed may be acceptable. The proposal must include a preliminary design and an opinion and supporting analysis by a qualified environmental professional that the expected environmental benefits of the restoration project substantially outweigh the potential impacts from the increase in runoff that would be produced by the development project.

By contrast, a project in a "high-risk" situation, including any project over 20 acres and any project discharging to an unstable stream (e.g., incised or confined channel, or having beds or banks composed of fine materials, as detailed in Attachment 4) could propose a mitigation project only after completing a comprehensive analysis to determine design objectives for channel restoration. (Or the applicant may choose to match post-project runoff peaks and durations to pre-project peaks and durations using IMPs (Option #2) or site-specific modeling and design (Option #3)).

Plan and Schedule for HMP Implementation

Following Water Board approval of this HMP, the steps to implementation are:

- Finalize and test the IMP sizing tool.
- Incorporate the HMP policies, IMP sizing tool, IMP designs, and other technical information into a third edition of the Program's *Stormwater C.3 Guidebook.*
- Prepare and conduct information/training sessions for municipal staff and land development professionals.
- Validate the approach by conducting a comprehensive assessment of one Contra Costa watershed.
- Initiate Continuous Improvement.
- Begin requiring HMP implementation in Stormwater Control Plans for projects deemed complete after a set date.

The Program proposes to complete these tasks and begin implementation 90 days after the Water Board's final approval of this HMP.

Continuous Improvement

The HMP incorporates the Co-permittees' initial success in implementing C.3 requirements (using the *Stormwater C.3 Guidebook*), uses current scientific understanding and technical tools, and adopts generally conservative assumptions throughout. We believe implementation will ensure estimated post-project runoff peaks and durations do not exceed estimated pre-project peaks and durations where increased stormwater runoff peaks or durations could cause erosion or other significant effects on beneficial uses.

We also expect to gain, through the first years of implementation, information and insights that will help us meet the HMP's objectives more efficiently and effectively.

The initial step in this process will be to conduct an analysis of one watershed development scenario to evaluate the effectiveness of the Program's HMP standard (including use of IMPs) in controlling the cumulative effects of many development projects within a single watershed. The analysis will also help the program refine guidance for evaluating and mitigating impacts of development in situations where there is a "high risk" of accelerated erosion.

The HMP will be continuously improved through the following plan-do-check-adapt cycle:

Plan

- Adoption of this HMP by the Water Board.
- Initiation of HMP requirements according to the 90-day schedule above.

Do

- Implementation by municipalities of HMP requirements on all projects subject to the HMP.
- Documentation of each project's HMP implementation in a Stormwater Control Plan.

Check

- Tabulation, reporting, and summary of all projects countywide in the Program's Annual Report.
- Evaluation of HMP effectiveness. The evaluation will be based on a review of project Stormwater Control Plans, review of construction documents, and visits to constructed projects. The evaluation will note problems and issues encountered in implementation and anecdotal reports of IMP performance.

- Monitoring. The Program will investigate means to monitor flow from IMPs as a way of evaluating flow control effectiveness.
- Evaluation of articles, design manuals, and technical reports regarding Low Impact Development, IMP design, and IMP modeling outside Contra Costa County.
- Refinement of IMP sizing factors. The Program may refine technical assumptions used in development of the sizing factors, re-run the models, and refine sizing factors accordingly.
- Evaluation of any in-stream projects implemented in connection with the HMP.

Adapt

- Updates to the Stormwater C.3 Guidebook as needed
- Between Guidebook updates, interim Program guidance memoranda to municipal staff as needed.

In closing, we express our appreciation for the effort and contributions of Water Board staff, particularly Christine Boschen, Keith Lichten, and Jan O'Hara, as we prepared the HMP.

Should you have any questions regarding the HMP, please contact Tom Dalziel at (925) 313-2392.

Sincerely,

Donald P. Freitas
Program Manager

DPF:td
cc. C. Boschen, SFBRWQCB
C. Palisoc, CVRWQCB

Hydrograph Modification Management Standard

All projects subject to this standard[1] shall ensure estimated post-project runoff peaks and durations do not exceed estimated pre-project peaks and durations if increased stormwater runoff peaks or durations could cause erosion or other significant effects on beneficial uses.[2]

By allowing no increase or impact from any individual project (or only *de minimis* increase or impact), the standard is intended to ensure that beneficial uses are reasonably protected from the potential cumulative effects of foreseeable future development in the same watershed. In addition, each of the following methods and criteria for demonstrating compliance with the standard is defined using conservative criteria (e.g., by using an upward bias when assessing and estimating potential impacts of hydrograph modification and a downward bias when estimating the effectiveness of hydrograph modification management measures). Finally, the methods and criteria emphasize distributed, infiltration-based IMPs that mimic natural infiltration processes, minimizing the potential for cumulative impacts.

Demonstrating Compliance with the Standard

Applicants may demonstrate compliance with the standard by demonstrating any one of the following:

1. **No increase in directly connected impervious area.** The applicant may compare the project design to the pre-project condition and show the project will not increase directly connected impervious area and also will not facilitate the efficiency of drainage collection and conveyance. The comparison shall include all of the following:
    a. Assessment of site opportunities and constraints to reduce imperviousness and retain or detain site drainage.
    b. Description of proposed design features and surface treatments used to minimize imperviousness.
    c. Inventory and accounting of existing and proposed impervious areas.
    d. Design details and descriptions to show which proposed areas are "self-retaining" or drain to stormwater treatment facilities. "Self-retaining" areas do not contribute to directly connected impervious area. Impervious areas draining to stormwater treatment facilities are considered directly connected.
    e. A qualitative comparison of pre-project to post-project efficiency of drainage collection and conveyance. Stormwater treatment integrated management practices (IMPs) such as those in the *Stormwater C.3*

[1] Subject to definitions and limitations in C.3.c.i of Water Board Order R2-2003-0022.

[2] This is a restatement of Water Board Order R2-2003-0022, Provision C.3.f.i.

Guidebook increase time of concentration, particularly for smaller storms, and are considered to substantially reduce drainage efficiency.

2. **Implementation of hydrograph modification IMPs.** The applicant may select and size IMPs to manage hydrograph modification impacts, using the design procedure, criteria, and sizing factors specified by the Contra Costa Clean Water Program and incorporated in the Program's *Stormwater C.3 Guidebook*.

3. **Estimated post-project runoff durations and peak flows do not exceed pre-project durations and peak flows.** The applicant may use a continuous simulation hydrologic computer model such as USEPA's Hydrograph Simulation Program—Fortran (HSPF) to simulate pre-project and post-project runoff, including the effect of proposed IMPs, detention basins, or other stormwater management facilities. To use this method, the applicant shall compare the pre-project and post-project model output for a rainfall record of at least 30 years, using limitations and instructions provided by the Contra Costa Clean Water Program, and shall show the following criteria are met:

    a. For flow rates from one-half the pre-project 2-year runoff event (0.5Q2) to Q2, post-project runoff durations shall not exceed pre-project runoff durations. For flow rates above Q2, post-project durations may exceed pre-project durations no more than 10% of the time.

    b. For flow rates from 0.5Q2 to Q2, the post-project peak flows shall not exceed pre-project peak flows. For flow rates from Q2 to Q10, post-project peak flows may exceed pre-project flows by up to 10% for a 1-year frequency interval. For example, post-project flows could exceed pre-project flows by up to 10% for the interval from Q9 to Q10 or from Q5.5 to Q6.5, but not from Q8 to Q10.

4. **Projected increases in runoff peaks and durations will not accelerate erosion of receiving stream reaches.** The applicant may show that, because of the specific characteristics of the stream receiving runoff from the project site, or because of proposed stream restoration projects, or both, there is little likelihood that the incremental increase in flow due to the project could increase the net rate of stream erosion to the extent that beneficial uses would be significantly impacted. To use this option, the applicant shall evaluate the receiving stream to determine the relative risk of erosion impacts and take the appropriate actions as described below:

    a. **"Low Risk."** In a report or letter report, signed by an engineer or qualified environmental professional, the applicant shall show that all downstream channels between the project site and the Bay/Delta fall into one of the following "low-risk" categories.

i. Enclosed pipes.

ii. Channels with continuous hardened beds and banks engineered to withstand erosive forces and composed of concrete, engineered riprap, sackcrete, gabions, mats, etc. This category excludes channels where hardened beds and banks are not engineered continuous installations (i.e., have been installed in response to localized bank failure or erosion).

iii. Channels subject to tidal action.

iv. Channels shown to be aggrading, i.e., subject to the accumulation of sediments.

b. **"Medium Risk."** Medium risk channels are those where the boundary shear stress could exceed critical shear stress as a result of hydrograph modification, but where either the sensitivity of the boundary shear stress to flow is low (e.g., an oversized channel with high width to depth ratios) or where the resistance of the channel materials is relatively high (e.g. cobble or boulder beds and vegetated banks). In "medium-risk" channels, accelerated erosion due to increased watershed imperviousness is not likely but is possible, and the uncertainties can be more easily and effectively addressed by mitigation than by additional study.

In a preliminary report, the applicant's engineer or qualified environmental professional will apply the Program's methods and criteria to show all downstream reaches between the project site and the Bay/Delta are either at "low-risk" or "medium-risk" of accelerated erosion due to watershed development. In a following, detailed report, a qualified stream geomorphologist[3] will use the Program's criteria, available information, and current field data to evaluate each "medium-risk" reach. For each "medium-risk" reach, the report shall show one of the following:

i. A detailed analysis, using the Program's criteria, showing the particular reach may be reclassified as "low-risk."

ii. A detailed analysis, using the Program's criteria, confirming the "medium-risk" classification, and:

1. A preliminary plan for a mitigation project to stabilize stream beds or banks, improve stream functions, and/or improve habitat values, and

2. A commitment to implement the mitigation project timely in connection with the proposed development project

[3] Typically, detailed studies will be conducted by a stream geomorphologist retained by the lead agency (or, on the lead agency's request, another public agency such as the Contra Costa County Flood Control and Water Conservation District) and paid for by the applicant.

(including milestones, schedule, cost estimates, and funding), and

3. An opinion and supporting analysis by one or more qualified environmental professionals that the expected environmental benefits of the mitigation project substantially outweigh the potential impacts of an increase in runoff from the development project, and

4. Communication, in the form of letters or meeting notes, indicating consensus among staff representatives of regulatory agencies having jurisdiction that the mitigation project is feasible and desirable. (This is a preliminary indication of feasibility required as part of the development project Stormwater Control Plan. All applicable permits must be obtained before the mitigation project can be implemented.)

c. **"High Risk."** High-risk channels are those where the sensitivity of boundary shear stress to flow is high (e.g., incised or entrenched channels, channels with low width-to-depth ratios, and narrow channels with levees) or where channel resistance is low (e.g., channels with fine-grained, erodible beds and banks, or with little bed or bank vegetation). In a "high-risk" channel, it is presumed that increases in runoff flows will accelerate bed and bank erosion.

To implement this option (i.e., to allow increased runoff peaks and durations to a high-risk channel), the applicant must perform a comprehensive analysis to determine the design objectives for channel restoration and must propose a comprehensive program of in-stream measures to improve channel functions while accommodating increased flows. Specific requirements are developed case-by-case in consultation with regulatory agencies having jurisdiction. The analysis will typically involve watershed-scale continuous hydrologic modeling (including calibration with stream gauge data where possible) of pre-project and post-project runoff flows, sediment transport modeling, collection and/or analysis of field data to characterize channel morphology including analysis of bed and bank materials and bank vegetation, selection and design of in-stream structures, and project environmental permitting.

The Program plans to develop an assessment of one watershed, and further recommendations for future comprehensive watershed assessments, later in 2005.

BROWN AND CALDWELL

Attachment 2

Memorandum

Date: May 12, 2005

To: Tom Dalziel, Contra Costa Clean Water Program
Dan Cloak, Dan Cloak Environmental Consultants

Cc: Christie Beeman, Philip Williams Associates
Jeff Haltiner, Philip Williams Associates

From: Tony Dubin, BC-Seattle
Steve Anderson, BC-Seattle

Subject: Contra Costa Clean Water Program Hydrograph Modification Management Plan Integrated Management Practices Modeling: Methods and Results

1. Introduction

To comply with California Regional Water Quality Control Board for the San Francisco Bay Region (Water Board) Order R2-2003-0022, Contra Costa Clean Water Program (Program) Co-permittees are requiring additional treatment controls to limit stormwater pollutant discharges associated with certain new development and significant redevelopment projects.

The Program's *Stormwater C.3 Guidebook* provides comprehensive, step-by-step guidance, including standard designs and details, to assist project applicants to select and design integrated management practices (IMPs) to retain, or detain and treat, stormwater runoff from new development. Municipal staff use the *Guidebook's* checklists and design criteria to review applicant submittals for compliance with each of the C.3 provisions. Implementation of these requirements began February 15, 2005.

Provision C.3.f of Order R2-2003-0022 requires the Co-permittees to submit, by May 15, 2005, a Hydrograph Modification Management Plan (HMP). The HMP, once approved by the Water Board, will be implemented so that "… post-project runoff shall not exceed estimated pre-project rates and/or durations, where the increased stormwater discharge rates and/or durations will result in increased potential for erosion…."

The Program aims to develop methods that fully implement all requirements of Provision C.3.f, while also being consistent with the user-friendly, step-by-step approach exemplified by the current *Stormwater C.3 Guidebook.*

The Program retained the consultant team of Philip Williams and Associates, Brown and Caldwell, and Pace Engineers to assist with this objective.

The consultant team developed a method to size IMPs so that the peaks and durations of runoff flows from a developed area will be equivalent to, or less than, peaks and durations of flows that existed before the project was built, for the specified range of flows.

To develop the method, 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.
- Incorporated the sizing factor calculations into a user-friendly spreadsheet-based interface.

This memorandum documents the modeling methods and assumptions used to determine IMP sizing factors. A separate memorandum describes use of the spreadsheet-based interface.

The remainder of this memorandum is arranged as follows:

- **Section 2 – Background**. This section describes the hydrologic model Hydrologic Simulation Program—Fortran (HSPF), why it was selected over other models for use in this analysis, the general steps used to develop the sizing factors, and how the sizing factors are built into the spreadsheet.
- **Section 3 – HSPF Model Development**. This section describes parameters used by the HSPF model, and how local values for those parameters were selected. This included review of local rainfall and soils, other modeling parameters such as plant cover, and sensitivity analyses conducted to isolate which parameters were most significant in runoff calculations.
- **Section 4 – HSPF Approach for Modeling IMPs**. This section describes the modeling approach used to size IMPs. One IMP, the In-Ground Planter, is used as an example.
- **Section 5 – HSPF Modeling Results**. This section presents the modeling results and the corresponding sizing factors developed for each of the IMPs simulated.
- **Section 6 – IMP Sizing Factors Adjustment**. This section describes how IMP sizing factors are adjusted for variability of rainfall at different locations in the

County, and for tributary areas that include pervious in addition to impervious areas.

- **Section 7 – Simplifying Assumptions and Potential Model Refinements**. This section identifies simplifying assumptions made in the current study and suggests potential model refinements, including field testing the accuracy of the IMP sizing results.
- **Appendix A – HSPF Modeling Parameters**. Appendix A provides background on each of the parameters used in HSPF.
- **Appendix B – Soil Physics.** Appendix B describes technical approaches used to simulate accumulation and movement of water in the IMPs (e.g., soil physics theory used to specify hydraulic conductivity, etc.).
- **Appendix C – Sensitivity Analyses**. Appendix C contains a memorandum written to describe the sensitivity of HSPF model results to varying certain key parameters.

2. Background

Low Impact Design (LID) stormwater control facilities (also called IMPs) have been developed and implemented by municipalities across the country. Commonly, these facilities are sized to retain or treat a specified proportion (e.g., 80%) of average annual rainfall. Control of runoff peaks and durations is a considerably more complex criterion. To ensure the IMPs are sized appropriately for hydrograph modification control, the consultant team modeled key physical parameters in HSPF and created a continuous simulation of hourly IMP inflows and outflows over a multi-year period.

2.1 CONTINUOUS VERSUS EVENT MODELS

There are two commonly used approaches for designing stormwater management facilities. The older approach to runoff modeling is called the "event-based" approach. Event models calculate runoff from a single hypothetical rainfall event. This rainfall event is represented by a "synthetic hyetograph", an idealized distribution of rainfall intensity during a storm. The total volume of rainfall during the storm event is determined by the storm recurrence interval (e.g., a 2-year, 24-hour storm may have a total volume of X inches). Distribution of a portion of the total rainfall depth to each time increment (typically one hour) is based on characteristic rainfall patterns for the region.

The event-based approach can be used to quickly and conservatively estimate peak flow rates required to design stormwater conveyance and detention facilities, and was particularly useful in the pre-computer era. With increased computing power, "continuous" runoff simulation models can now be readily used. These continuous models overcome many of the limitations of the event models. In particular, the continuous models can be used to evaluate the hydrology of smaller storms.

Continuous hydrologic modeling involves establishing a mathematical representation of watershed physical characteristics (e.g., soils, vegetation, etc.) and applying a long-term time series of rainfall (e.g., 30 years of hourly rainfall) to that watershed. The continuous model tracks infiltration, evaporation, and other losses, and generates a runoff time series for the specified watershed conditions and rainfall time series.

The continuous modeling approach offers several advantages over event-based methods for sizing IMPs:

- The runoff time series generated by continuous models can be analyzed to determine return frequencies for different runoff peak rates (e.g., the 2- year event, the 100-year event, etc.), including those generated by small, frequent events. Event methods simply assume that the runoff event frequency corresponds to the rainfall event frequency. For example, in the past, it was common to assume that a two-year storm generated a two-year runoff event.

- The runoff time series generated by continuous models can be analyzed to determine duration of flows above various levels. This is critical to evaluating the effects of land development because stream erosion correlates to duration of flows above a specific threshold.

- Rainfall records used in continuous models include the variations in storm length, volume and intensity that occur at a particular location. Event-based methods are based on a synthetic hyetograph that attempts to represent rainfall event behavior for a larger region with a single, idealized rainfall distribution.

- Continuous models better account for antecedent moisture conditions and the impacts of back-to-back storms. Event-based methods typically assume a facility (e.g., an infiltration basin) is empty at the start of a design storm event, but during the wet season this assumption may not hold true.

In summary, because they encompass a wide range of actual rainfall-runoff conditions, continuous models can best assess whether stormwater flow control IMPs serve their key purpose, which is protecting local creeks against the increase in the frequency of geomorphically significant, channel forming flows.

2.2 DESCRIPTION OF HSPF MODEL

A variety of continuous models were available to the Program to perform the analyses required to implement the C.3.f requirements (e.g., SWMM, HEC-HMS, etc.). The HSPF model was selected as the preferred tool. The remainder of this section briefly describes HSPF, and summarizes reasons for this selection. The descriptive text was obtained and abbreviated from the Hydrocomp website (www.hydrocomp.com). Hydrocomp, Inc. is the firm that was commissioned by USEPA in 1976 to create HSPF.

2.2.1 HSPF History

HSPF is a comprehensive, conceptual, continuous watershed simulation model designed to simulate all the water quantity and water quality processes that occur in a watershed. The model simulates watershed spatial variability by dividing each basin into hydrologically homogeneous land segments and simulating runoff for each land segment independently, using different meteorologic input data and watershed parameters. The model includes fitted parameters as well as parameters that can be measured in the watershed.

HSPF has its origin in the Stanford Watershed Model developed by Crawford and Linsley (1966). Crawford and Linsley refined the original model and created HSP, the Hydrocomp Simulation Program, which included sediment transport and water quality simulation. During the early 1970's Hydrocomp developed the ARM (Agricultural Runoff Management Model) and the NPS (Nonpoint Source Pollutant Loading Model) for the EPA (U.S. Environmental Protection Agency). In 1976, the US EPA commissioned Hydrocomp, Inc. to combine the various modules into one program, resulting in what is now known as HSPF.

2.2.2 HSPF General Description

HSPF simulates hydrologic processes on pervious and impervious land surfaces and in streams and impoundments. The model can be applied to most watersheds using existing meteorologic and hydrologic data. According to Hydrocomp, EPA recommends its use as the most accurate and appropriate management tool available for the continuous simulation of hydrology and water quality in watersheds.

In HSPF, the various hydrologic processes are represented as flows and storages. In general, each flow is an outflow from storage, usually expressed as a function of the current storage amount and the physical characteristics of the subsystem. For simulation with HSPF, the basin is represented in terms of land segments and reaches/reservoirs. A land segment is a subdivision of the simulated watershed. The boundaries are established according to the user's needs, but generally, a segment is defined as an area with similar hydrologic characteristics. For modeling purposes, water, sediment and water quality constituents leaving the watershed move laterally to a downslope segment or to a reach/reservoir. A segment of land that has the capacity to allow enough infiltration to influence the water budget is considered pervious. Otherwise it is considered impervious.

In pervious land segments HSPF models the movement of water along three paths: overland flow, interflow and groundwater flow. Each of these three paths experiences differences in time delay. A variety of storage zones are used to represent the storage processes that occur on the land surface and in the soil horizons. Processes that occur in an impervious land segment are also simulated. Even though there is no infiltration, precipitation, overland flow and evaporation occur.

The hydraulic and water quality processes that occur in the river channel network are simulated by reaches. The outflow from a reach may be distributed across several targets to represent normal outflow or diversions. Evaporation, precipitation and other fluxes that take place in the surface are also represented. Routing is done using a modified version of the kinematic wave equation.

2.2.3 Advantages of Using HSPF

HSPF allows for effective simulation of watershed processes, identification of continuous runoff time series, and integration and evaluation of the performance of control measures such as IMPs. Although data intensive, the model allows for simulation of all the losses associated with IMPs (e.g., evaporation, transpiration, surface runoff, interflow, deep infiltration).

HSPF has been widely embraced in a variety of regions throughout the Country for evaluation and design of hydrograph modification management for new developments. For example, HSPF is the standard for watershed modeling in the Pacific Northwest. The Western Washington Hydrologic Model (WWHM) consists of an HSPF-based simulation with an interface to facilitate user input and formatting of results. As a consequence of its widespread use, there is a growing community of practitioners and a significant body of literature available to support the use of the model in hydrograph modification management applications.

2.3 HSPF MODELING APPROACH SUMMARY

The purpose of the runoff simulation for existing and post-development site conditions is to evaluate the effectiveness of IMPs, which mitigate the increase in stormwater runoff peak flow and duration resulting from the conversion of pervious land surfaces to impervious surfaces. The pre-project runoff regime must be characterized for a variety of baseline soil group, cover and rainfall scenarios. Increases in runoff peaks and durations from each of these baseline scenarios establish the impacts to be fully mitigated by an IMP to be incorporated into a particular development. This section summarizes the overall steps used in this study to size IMPs, and how this information will be made accessible to applicants for project approvals.

2.3.1 Develop Pre-Project and Post-Project Runoff Time Series

The Program's approach to compliance with Provision C.3.f is to ensure that post-project runoff at any given development does not exceed pre-project runoff peaks or durations for the range of flows which could potentially have significant impacts on receiving streams. This approach aims to address the potential impacts of an individual development and the cumulative effects of many developments in the same watershed.

The consultant team developed sets of HSPF model parameters to represent a range of pre-project site conditions that may be encountered in Contra Costa County (these scenarios are presented in Section 3). The various possible combinations of these parameters determined the number of "scenarios" that might be required to adequately characterize the pre-project condition for any given development project in the County.

Runoff from each scenario was simulated using a rainfall time series from a gauge located in Martinez.

Once a continuous runoff time series was generated for the rainfall period of record for each scenario, frequency and duration analyses were performed on each time series to identify recurrence frequencies and durations for different size runoff events. (This step is needed to characterize the peak flows for various recurrence intervals).

Consistent with the general design guidance in the *Stormwater C.3 Guidebook*, designers are expected to minimize the amount of pervious surface that drains to IMPs. Post-project site runoff was therefore evaluated by simulating runoff from a unit area converted to 100% impervious surface. Comparing the pervious surface model output with the impervious surface model output shows the effects of development prior to adding an IMP.

2.3.2 Model IMPs

The project team constructed representations of each IMP in HSPF. For example, a "dry" swale is represented in HSPF by length, cross-section geometry, layers of soil and underdrain material, and transmissivity of underlying soils. These parameters can be varied to determine the configuration that provides the best performance in the least amount of space. The HSPF tool for representing storage facilities is called an F-TABLE, and is described further in Section 4.

2.3.3 Establish Sizing Factors

For each IMP and pre-project scenario of site conditions, impervious surface runoff was routed through the IMP to develop a post-project "mitigated" runoff time series. Each IMP mitigates post-project runoff by providing infiltration and/or reduction of discharge rates to the drainage system. The post-project time series was then compared to the pre-project runoff time series to assess IMP performance. The IMP size (typically surface area) was varied over the course of multiple model iterations until a size was identified that adequately matched post-project to pre-project runoff. The runoff comparison was performed both for peak rates and durations. The standard applied in this comparison was as follows:

- Peak flow control. Runoff from the site under post-project conditions should not exceed pre-project runoff for events ranging from half the pre-project two-year peak flow to the pre-project 10-year peak flow (0.5Q2 to Q10). This is evaluated by comparing flow-frequency curves for pre- and post-project conditions for all events from 0.5Q2 to Q10.

- Flow duration control. Runoff durations from the site under post-project conditions should not exceed pre-project runoff durations for events ranging from 0.5Q2 to Q10. This is evaluated by comparing duration-frequency curves for pre- and post-project conditions for all flows from (pre-project) 0.5Q2 to Q10

Section 4 below provides a detailed explanation of how the above standards were actually applied.

2.3.4 Incorporate Sizing Factors Into Spreadsheet Tool

The sizing factors computed using the above process were then incorporated into an IMP Sizing Worksheet that developers will use to describe site hydrology, compute pre- and post-project runoff rates, and size IMPs.

During the site design process, the applicant's engineer will divide a project site into separate drainage management areas that will drain to individual IMPs. Based on the type of IMP selected, the amount of impervious and pervious tributary land, and local soil group, the IMP Sizing Worksheet will look up the appropriate value derived from the HSPF modeling analysis. An adjustment will be applied to the IMP sizing factor based on the location of the project in the County to account for the different rainfall characteristics (see Section 3.2.1 and Section 6).

The IMP Sizing Worksheet also provides prescriptive guidance on using self-retaining landscaping, soil amendments, and other techniques to limit site runoff, and contains a conservative approach to scale IMPs based on tributary pervious areas (i.e., in addition to the tributary impervious areas). The approaches used for scaling of sizing factors according to local rainfall and tributary pervious areas are summarized in Section 6 below and will be described in greater detail in a separate technical memorandum focused on the IMP Sizing Worksheet.

3. HSPF Model Development

This section describes how HSPF models were developed to simulate pre-project and post-project runoff for Contra Costa County.

3.1 HSPF MODELING OVERVIEW

An HSPF modeling study of a single watershed typically begins with gathering hydrologic information about the area, such as precipitation data, soil groups, soil layer depths, vegetation types, vegetation canopy thickness, etc. This information is used to develop appropriate input parameters to the HSPF model. HSPF parameters fall into three general categories:

1. Prescriptive parameters that set flags and specify algorithms to use.
2. Measured or estimated parameters, such as basin area, that are set by GIS analysis or physical measurement.
3. Calibration parameters that may be estimated by measurement, but must be adjusted during the model calibration process. Examples of calibration parameters are infiltration rates, upper soil depth, and groundwater conductivity.

Together these parameters describe the vertical movement (e.g. interception, depression storage, infiltration, evapotranspiration) and lateral movement (e.g. surface runoff, interflow, groundwater flow) of water in HSPF. For studies of individual watersheds, the values of calibration parameters are adjusted, or tuned, until the model simulations reproduce an observed stream flow record.

The purpose of this study was to produce a County-wide hydrologic assessment tool for sizing IMPs. This required a modified approach relative to the typical steps taken in evaluating a single watershed. Sets of regional, representative parameters were applied to a theoretical unit area, instead of developing and calibrating a specific watershed model. The representative model parameters were initially selected based on other, calibrated HSPF models including the US Geological Survey's regional calibration for Calabazas Creek in Santa Clara County and WWHM. In addition, local model curve numbers for the USDA's TR55 event-based model were examined to determine relative runoff production by soil group and cover type.

3.2 CONTRA COSTA HSPF MODEL DATA AND PARAMETERS

Adapting the compiled HSPF parameters for use in Contra Costa County required an assessment of the local characteristics that affect surface runoff, such as precipitation data, basic soil groups and vegetation cover. This section summarizes the data and parameters selected for use in the model, and their integration into the model development and IMP sizing process.

3.2.1 Rainfall Data Evaluation

Evaluating the distribution of rainfall across the County helped determine (1) which precipitation gauge to use as input to HSPF for modeling simulations and (2) whether rainfall quantities vary enough to require an adjustment to IMP sizing factors based on location.

The Contra Costa County Flood Control District (FCD) operates a series of precipitation gauges across the County, and the National Oceanographic and Atmospheric Administration (NOAA) operates a COOP (Cooperative Network) station in Martinez. Six rainfall data series with at least 30 years of hourly data were available for analysis. Partial duration series statistics were computed for each precipitation station to determine the rainfall intensity and volume at different recurrence intervals (e.g. 2-year, 24-hour storm; 10-year, 72-hour storm). In addition to comparing the rainfall frequency by station, this process included an assessment of data quality.

Table 1 lists reference information about the gauges and Figure 1 shows the variation in rainfall depth, based on a partial duration series analysis of the rainfall records at each site. Storm volumes clearly vary at different locations, with the highest mean annual precipitation (MAP) sites also producing the largest storm volumes. Higher rainfall volumes occur at higher elevation gauges, with the notable exception of the Dublin-San Ramon station which is located to the south and east of the other stations. In addition to the storm frequency-volume curves shown in Figure 1, rainfall accumulations over specific periods of time (e.g. 24 hours, 48 hours, etc.) were examined and showed results similar to the frequency-volume curves: sites with larger MAPs also had larger 24-hour and 48-hour accumulations.

Table 1. List of Long-Term Rain Gauging Stations in Contra Costa County.

Station Name	Location	Period of Record	Latitude; Longitude	Elev. (ft)	Mean Annual Rain
Martinez[A]	City of Martinez	7/48 thru 2/04	37° 58' N; -122° 08' W	70.1	20.2 in
Flood Control	CCC Flood Control HQ	9/71 thru 5/04	37° 59' N; 122° 05' W	160'	16.4 in
St. Mary's	St. Mary's College	9/72 thru 5/04	37° 51' N; 122° 06' W	620'	24.8 in
Orinda Fire	Orinda Fire Station 3	9/73 thru 5/04	37° 54' N; 122° 10' W	700'	25.1 in
Los Medanos	Chevron Pipeline Pump Plant	7/74 thru 5/04	38° 00' N; 121° 51' W	130'	8.4 in
Dublin Fire	Dublin-San Ramon Fire House	9/73 thru 5/04	37° 44' N; 121° 56' W	355'	12.5 in

A. Our examination of the Martinez Gauge record showed several questionable records where an entire storm's depth was recorded in a single hour. For these questionable storms, we distributed the rainfall depth recorded at Martinez according to the storm timing recorded at the nearest gauge (Flood Control District Gauge 11). We were only able to correct the Martinez gauge back to 1969. The HSPF simulations therefore contain 35 years of rainfall data.

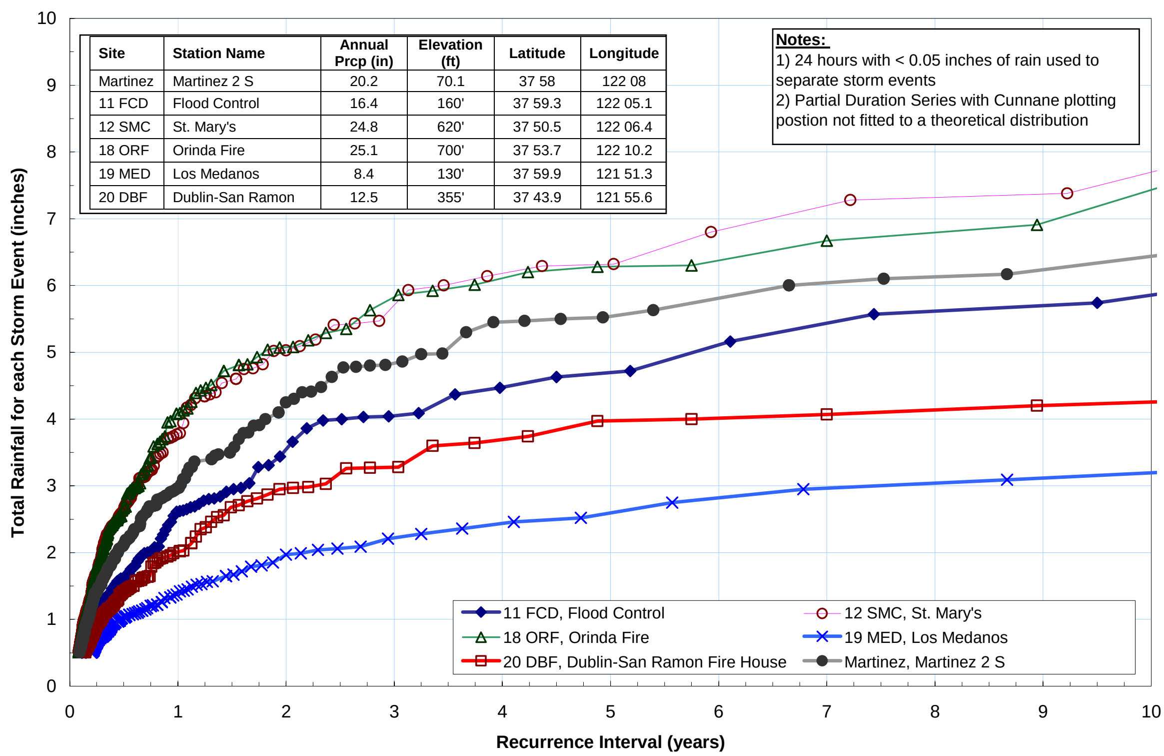

Figure 1. Rainfall Variation in Contra Costa County

Attachment 2

The required size of each IMP should be closely tied to local storm volume, because these devices function by capturing stormwater runoff and slowly releasing it. The sizing factors must assure that the IMPs are sufficiently large to manage all flows within the 0.5Q2 to Q10 range. This assessment of rainfall data suggested that there is sufficient variability in countywide rainfall patterns to require adjustment of IMP sizing factors for location. The method of adjustment was determined through a modeling analysis and is discussed in the *IMP Sizing Adjustment Factors* section.

The Martinez station was selected as the basis for the long-term HSPF simulations and IMP 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.

3.2.2 Contra Costa Soils Map Evaluation

The HSPF model development focused on scenarios that represent the most commonly occurring soil groups in Contra Costa County, with added consideration for areas of likely future development. Three sources of information were considered: (1) Natural Resources Conservation Service (NRCS) mapping, (2) the Contra Costa County Watershed Atlas and (3) the practical local experience of the project team.

According to project team experience (and backed by NRCS data), approximately two-thirds of Contra Costa County is classified as NRCS Group D soils (e.g. clays). Approximately one-quarter of the County consists of Group C soils, but most of these occur in the steepest and least developable sections of the County. There are smaller sections of Group A and Group B soils, although the project team's local experience suggested the Group A soils located in the north and east parts of the County should be included in the assessment because this represents much of the remaining developable land that is not characterized as Group D soil.

Thus, two bookend soil conditions were modeled in HSPF: fast-draining Group A (sand) soils and slow-infiltrating Group D (clay) soils. Developing HSPF parameters for two of the four NRCS hydrologic group classifications is consistent with the approach used Western Washington, where the WWHM model combines Group A with Group B soils, and Group C with Group D soils for hydrologic and detention pond sizing calculations. In the IMP Sizing Worksheet, the sizing factors for Group B soils are set equal to those for Group A soils, and sizing factors for Group C soils are set equal to those for Group D soils. The most important HSPF parameter related to the soil group is the INFILT parameter, which is a measure of the infiltration rate when the underlying soil is approximately half saturated (HSPF uses the INFILT parameter value and decay equations to compute the infiltration capacity of soils through the full range of conditions from dry to saturated). For Group D soils, the INFILT parameter was set to 0.03 in/hr, which is the value used in the USGS study of Calabazas Creek in Santa Clara County and is consistent with other values from the literature. The Group A soil INFILT value was selected by assessing the sensitivity of HSPF model outputs, for

INFILT values ranging from 0.2 to 0.7 in/hr based on typical values from the literature. The model results with INFILT set to 0.3 in/hr most closely matched the project team's local experience in the County (see Appendix C for sensitivity analysis).

3.2.3 Other Model Parameters

HSPF differs from event-based or purely conceptual hydrologic models in the large number of parameters and equations it contains to describe in great detail the movement of water through a watershed. Due to the greater physical basis for HSPF's calculations, some of the familiar conceptual parameters used in event-based modeling are not emphasized in HSPF. For example, HSPF does not contain an initial abstraction like the NRCS's TR55 model, but similar above-ground storage is modeled by specifying the interception storage in the vegetation canopy (CEPSC parameter) and near-surface depression storage (UZSN parameter). Time of concentration is not an HSPF parameter, but HSPF does require the hydraulic terms used to compute time of concentration (i.e. basin area, overland flow path length and average basin slope).

The parameter values selected for interception storage and depression storage are consistent with the major vegetation/cover types in the County. HSPF scenarios included scrub, range, irrigated pasture and live oak as cover types. All simulations assumed gentle to moderate slopes (SLSUR parameter = 10 percent) to represent typical ground slopes in areas of likely future development. The single slope approach is similar to the regional calibration method incorporated into WWHM, which applies one slope for till soils and one slope for outwash soils in each County (e.g. in Lewis County, WA, SLSUR = 10 percent in till soils and SLSUR = 5 percent in outwash soils).

The EPA publication, *EPA Basins Technical Note 6 Estimating Hydrologic and Hydraulic Parameters for HSPF* (July 2000) is a very useful guide that describes key HSPF parameters and suggests initial values. Appendix C reproduces text from *Technical Note 6* to describe the function of many HSPF parameters and lists the values selected for for characterizing pre-project runoff. Table 2 lists values for the key HSPF parameters described above.

Table 2. Summary of HSPF Inputs and Parameters

HSPF Input	Source or Value
Precipitation Data	Martinez Gauge (COOP ID = 45371)
Mean Infiltration Group D Soils (INFILT)	0.03 in/hr
Mean Infiltration Group A Soils (INFILT)	0.30 in/hr
Interception Storage (CEPSC)	0.02 to 0.10 in[A]
Upper Zone Nominal Storage (UZSN)	0.50 in
Overland Flow Slope (SLSUR)	0.10

A. The interception storage depth ranged from 0.02 inches for range cover to 0.10 inches for live oak cover.

3.3 SCENARIOS MODELED

Using the data and parameters described above, HSPF was used to simulate runoff for eight combinations of pervious soils and cover types (Group A and D soils, and four different cover types) and impervious areas (see

Table 3 below). For each HSPF scenario simulated, peak flow frequency graphs were produced from a partial duration series analysis of the model output. Flow duration statistics were also computed. Together, these model results were used to evaluate the hydromodification effects of development and formed the basis for the IMP sizing process.

Table 3. HSPF Land Cover and Soil Group scenarios

Scenario No.	Land Cover	Soil Group [A]
1	Scrub	A
2	Scrub	D
3	Range	A
4	Range	D
5	Live Oak	A
6	Live Oak	D
7	Irrigated Pasture	A
8	Irrigated Pasture	D
9	Impervious	N/A

A. The soil classes refer to the NRCS soil classification system, where Group A soils correspond to sands and Group D soils correspond to clays.

4. Hydrologic Modeling Approach to Sizing IMPs

This section describes the technical approach used to represent IMPs in the HSPF model and describes the model inputs for the In-Ground Planter as an example. To represent the behavior of storage reservoirs (such as IMPs), hydrologic models typically use stage-storage-discharge tables. The stage represents depth of water in the facility, the storage represents the volume of water stored in the facility for that stage, and the discharge is the calculated outflow for that stage. Outflow may be via an orifice, infiltration, evaporation, or any other mechanism for which a relationship to stage or storage can be defined. Stage-storage-discharge relationships are represented in HSPF with FTABLES.

4.1 GENERAL IMP CHARACTERISTICS

Each IMP design selected by the Program includes a surface reservoir, a layer of gravel or drain rock, and an overflow outlet. Some IMPs have an additional layer of sandy loam soil; some also have an underdrain. In general, runoff flows into the surface

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storage reservoir and either infiltrates into the soil or flows through the overflow outlet structure.

For IMPs that include a soil layer, water that does not overflow the surface-storage reservoir infiltrates into the upper soil medium and is stored as soil water. Once in the soil, water percolates downward at a rate that is dependent on the soil moisture content, the hydraulic properties of the soil and the boundary conditions of the soil layer.

All IMPs include a gravel or aggregate layer. Discharge from this layer is by percolation to native soil and, in some cases, through an underdrain. Maximum discharge from the underdrain is limited by an orifice outlet. Figure 2 below is an illustration of an In Ground Planter, which incorporates all of the elements described above.

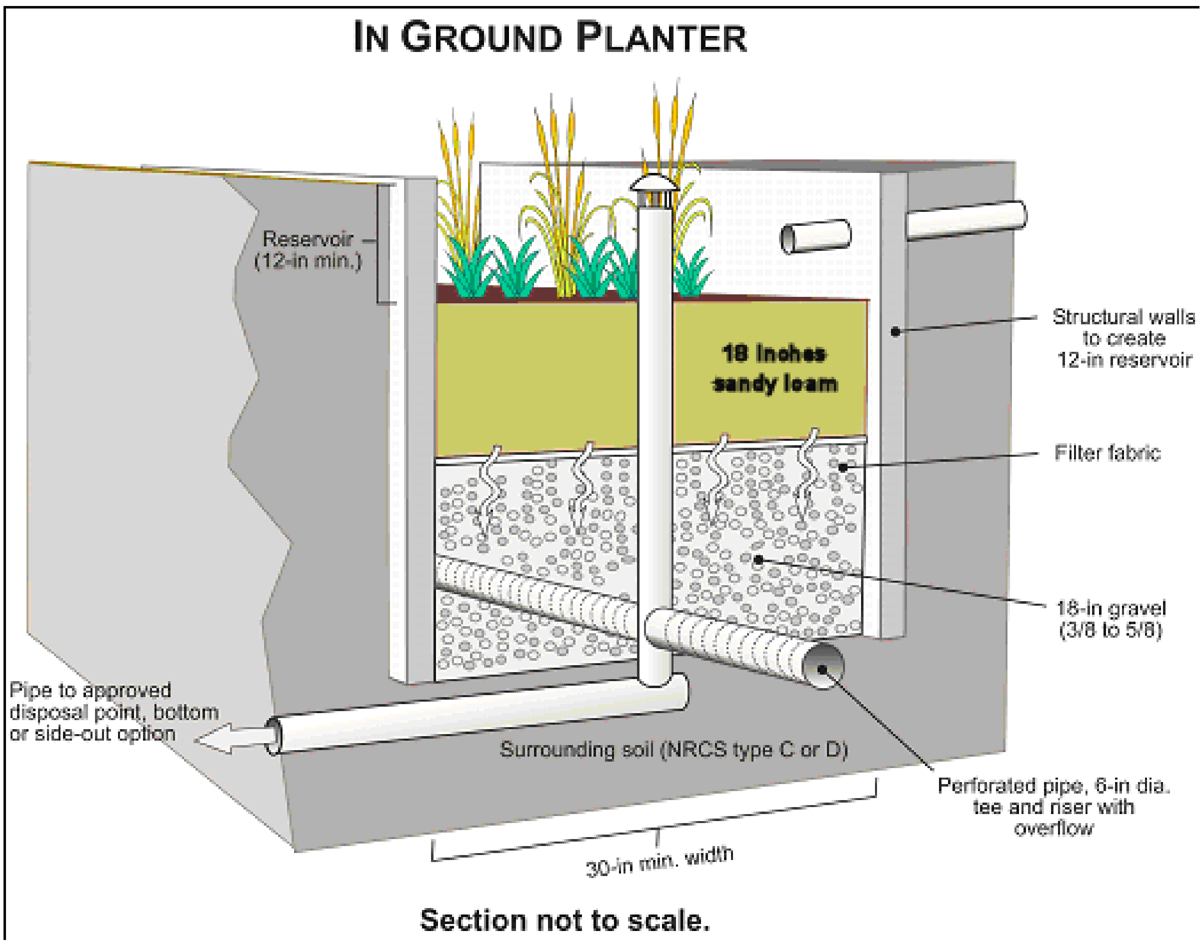

Figure 2. Cross-Section View of In-Ground Planter, Group C/D Soil Configuration

4.2 ALTERNATIVE IMP MODELING METHODS CONSIDERED

Prior to simulating all of the IMPs, the consultant team performed a pilot analysis of one IMP. The analysis considered three different methods for representing the movement of water through the In Ground Planter in HSPF. The analysis was necessary because IMP hydraulics are more complex than typical detention basins, and therefore cannot accurately be represented with just one stage-storage-discharge table. Because the component parts are consistent among the selected IMPs , it was reasoned that the same approach could be re-used for each of the other IMPs.

4.2.1 The Bathtub Approach

The first approach considered was the "bathtub" approach. In this approach, the planter is assumed to function like a stormwater detention vault, with only soil porosity dictating the stage-storage relationship. While this approach is easy to conceptualize, it is overly simple, since it does not capture the movement of the wetting front downward in the soil profile or the way water remains bound in the upper soil layer (i.e., until reaching field capacity) before moving into the gravel layer in the planter bottom. As a result, the "bathtub" approach does not accurately reflect the way the IMP detains and releases runoff.

4.2.2 Two-Layer Approach

This approach was a significant improvement over the bathtub approach. For this approach, the loamy top layer and pea gravel lower layer in the IMP are represented separately in the model with their own distinct stage-storage-discharge tables. Stormwater runoff drains to the loamy layer, progressively filling it (including, potentially, the reservoir storage area above the soil). When the upper layer is relatively dry, the percolation rate is low. As the loamy layer approaches saturation, percolation from the upper layer to the lower layer increases to the saturated conductivity rate for sandy loam soils. For the pea gravel layer, water moves freely to the bottom, with discharge via 1) an orifice outflow with discharge computed as a function of head (for planters with underdrains) or 2) infiltration from the bottom of the planter.

4.2.3 Pervious Land Surface Over a Pea Gravel Vault

The third approach considered involved treating the loamy soil layer as a separate pervious land surface instead of representing it with a stage-storage-discharge table. In this approach, HSPF would discharge impervious runoff to the IMP pervious land surface (essentially distributing the runoff over the surface as if it were rainfall). Water that was then discharged from the pervious surface (i.e., the top soil layer of the IMP) would then be routed into the pea gravel portion of the stormwater planter. In this approach, the depth of the soil layer would match the available storage in the loamy top portion of the planter. This method was expected to provide good representation of both evapotranspiration and vertical water movement as the soils become increasingly wetted during a rain event. However, upon closer examination, HSPF did not support the manipulation of the pervious areas to provide the detailed soil hydraulic characteristics that the FTABLES would allow.

4.3 SELECTED METHOD

After a feasibility assessment of each of the above methods, the consultant team selected the two-layer approach for representing IMPs. The bathtub method clearly

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oversimplified the processes occurring within the different components of an IMP, and the third approach, while useful in concept, was not well supported by HSPF.

4.3.1 Hydraulic Assumptions for All IMPs

Once the preferred approach was selected for modeling the IMPs, certain assumptions were required to model storage and discharge. The following general hydraulic assumptions were applied to all of the IMPs modeled:

- Inflow is uniformly distributed over the area of the IMP (i.e. level-pool ponding).
- Infiltration and soil water movement is a 1-dimensional flux in the vertical direction (neglecting lateral flows is a conservative assumption).
- Soil moisture within a homogeneous soil layer is assumed to be evenly distributed throughout the soil layer both vertically and horizontally. This assumes an engineered IMP would be free of macropores.
- The interface between an upper soil layer and a lower gravel layer is at atmospheric pressure when the lower layer is not saturated (i.e. water flows freely out of the upper layer).
- Water flows out the bottom of the IMP into the surrounding soil at the rate of saturated hydraulic conductivity.
- The sandy loam soil used for the growing medium has an effective porosity of 0.412, based on Table 5.3.2 in the *Handbook of Hydrology* (Maidment, 1994). A sensitivity analyses conducted to determine the effect of porosity on IMP performance determined that porosity has little influence on the required sizing factor (see Appendix C for sensitivity analysis results).

4.3.2 IMP Modeling Steps

Once a representation of each IMP was created for use in the model, sizing for each involved the following procedure:

- HSPF simulations were performed to compute continuous, hourly runoff-hydrographs for pre-project and impervious land segments with equal unit areas (1 acre).
- Stage-storage-discharge relationships summarized in the FTABLEs were developed uniquely for each IMP based on detailed soil physics equations (see Appendix A). The stage-storage-discharge relationships reflected the unique configurations of each IMP, including ponding depth, side slope, soil and gravel layer depths, and outflow configuration.
- The surface runoff from the impervious land segment was routed to an IMP, represented by one or more FTABLEs in HSPF.
- HSPF was used to track infiltration and outflow from the IMP's overflow pipe and underdrain (when applicable).

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- The IMP outflow time series (HSPF output) was analyzed and compared with pre-project flows. IMP size was adjusted and the new outflow time series compared to pre-project flows until pre-project and post-project flows and durations matched, according to established criteria for "goodness of fit."

4.4 IMP MODELING EXAMPLE: IN GROUND PLANTER

This section describes in detail the sizing analysis performed for this IMP.

4.4.1 In-Ground Planter HSPF Representation

The In-Ground Planter consists of a 12-inch ponding reservoir over an 18-inch upper soil layer (growing medium), and below that an 18-inch gravel layer. A vertical riser (pipe) was used as an overflow outlet from the ponding reservoir. The overflow pipe is 6 inches in diameter and the inlet is 10 inches above the soil. (See Appendix C for an analysis of the sensitivity of the In-Ground Planter sizing factor to overflow height.) The upper soil layer is a sandy loam with a specified infiltration rate of 5 inches per hour. The ponding depth, overflow height, soil and gravel layer depths and hydraulic properties are specified in the *Stormwater C.3 Guidebooks.* The area of the planter was varied with each iteration.

There are two configurations for the In-Ground Planter: 1) With a lateral underdrain for use in areas where native soils have low hydraulic conductivity (Group C/D soils and/or the water table is close to the ground surface, see Figure 2 above). 2) With no underdrain in areas where surrounding soils have high hydraulic conductivity (Group A/B soils) and groundwater infiltration is acceptable. The In-Ground planter was modeled using two FTABLEs. The first FTABLE represents the upper soil layer, the ponding reservoir and the overflow outlet. The second FTABLE represents the lower gravel layer and the underdrain. Percolation outflow from the first FTABLE is routed as inflow to the second FTABLE.

FTABLE 1: Upper Soil Layer, Ponding Storage and Overflow Outlet

Stormwater routed from impervious surfaces first enters the upper layer of the In-Ground Planter, represented by FTABLE 1 (Figure 3). The HSPF model assumes that all inflow will infiltrate if the layer is not saturated. This is a reasonable assumption based on the anticipated range of inflows (see Appendix A for a complete discussion of soils physics). The soil layer is represented by depths from 0 to 1.5 feet. The volume of storage at 1.5 ft is equal to the storage within the soil layer at saturation. Above this depth water is stored in the ponding reservoir.

Water contained in the upper soil layer is stored as soil moisture. Although there are depths indicated in the first column of the FTABLE, the soil water is considered to be evenly distributed throughout the soil layer (e.g. a soil depth of 0.5 feet in FTABLE 1 corresponds to one-third saturated, not water filling the bottom 0.5 feet of the upper soil layer). Above 1.5 ft, water ponds on the planter surface, and the FTABLE 1 depth column corresponds to the actual water surface.

The fourth column in FTABLE 1 lists the rate of soil water percolation out the bottom of the upper soil layer and into the lower gravel layer. This column is calculated using Darcy's Law and the van Genuchten relations (see Appendix A). Percolation does not occur unless the soil water content exceeds the holding capacity of the soil (i.e. the gravitational head is greater than the suction or *matric head* within the soil pores). The percolation rate calculations assume a free surface at the interface with the lower layer. However, the percolation rate is limited if the lower layer reaches capacity and becomes saturated. In this case the percolation rate through the upper layer is limited to the percolation rate though the lower layer, which in itself is limited by the total outflow from the lower layer through the underdrain orifice and percolation to the surrounding soil. Thus, the percolation rate through the upper layer is limited to underdrain outflow rate plus a small amount of percolation to the surrounding soil when the planter reaches capacity.

The fifth column in the FTABLE is the outflow through the overflow pipe, which is calculated using a weir equation (see Appendix A). Outflow through the overflow pipe does not occur until the depth of storage in the ponding reservoir is above the pipe inlet.

```
  FTABLE      1
 rows cols                                                ***
   31    5
     Depth       Area     Volume     Q Perc     Q Over   ***
      (ft)    (acres) (acre-ft)     (cfs)      (cfs)    ***
      0.00       0.03     0.0000     0.0000      0.000
      0.10       0.03     0.0012     0.0000      0.000
      0.20       0.03     0.0024     0.0000      0.000
        :          :         :          :          :
      1.40       0.03     0.0168     0.0132      0.000
      1.50       0.03     0.0180     0.0707      0.000
   -----------------------------------------------------------
      1.60       0.03     0.0210     0.0760      0.000
        :          :         :          :          :
      2.40       0.03     0.0495     0.1957      0.100
      2.50       0.03     0.0525     0.1957      0.312
  END FTABLE1
```

Soil

Ponding

Figure 3. Example FTABLE Describing Upper Layer of In-Ground Planter

FTABLE 2: Lower Gravel Layer, Percolation to Surrounding Soils, Underdrain Outlet

The second FTABLE represents the lower gravel layer and the underdrain (Figure 4). Percolation outflow from the first FTABLE is routed as inflow to the second FTABLE. This FTABLE represents the lower gravel layer, which has a depth of 1.5 ft. Water is stored as volumetric water content with a maximum storage limited to saturation of the gravel medium. The percolation rate out the bottom of the lower layer is limited by the hydraulic conductivity of the surrounding soil, which is a conservative assumption (percolation will actually be faster when native soils are unsaturated).

When an underdrain is included in the configuration, the 'Q Outlet' column is included in the FTABLE for the outflow rate. This rate is calculated using the orifice equation (see Appendix A) so that the underdrain flow will match 0.5Q2 when the lower gravel layer is fully saturated. The *Stormwater C.3 Guidebook* will specify criteria for sizing pipe perforations and/or flow control orifices to ensure that the underdrain flow is limited to 0.5Q2.

```
  FTABLE      2
 rows cols                                                        ***
   16    5
     Depth      Area    Volume    Q Perc  Q Outlet    ***
      (ft)   (acres) (acre-ft)     (cfs)     (cfs)    ***
      0.00      0.03    0.0000    0.0000     0.000
      0.10      0.03    0.0012    0.0001     0.000
      0.20      0.03    0.0025    0.0007     0.001
      0.30      0.03    0.0037    0.0007     0.005
      0.40      0.03    0.0050    0.0007     0.018
      0.50      0.03    0.0062    0.0007     0.047
      0.60      0.03    0.0075    0.0007     0.104
      0.70      0.03    0.0087    0.0007     0.133
      0.80      0.03    0.0100    0.0007     0.142
      0.90      0.03    0.0112    0.0007     0.151
      1.00      0.03    0.0125    0.0007     0.159
      1.10      0.03    0.0137    0.0007     0.167
      1.20      0.03    0.0149    0.0007     0.174
      1.30      0.03    0.0162    0.0007     0.181
      1.40      0.03    0.0174    0.0007     0.190
      1.50      0.03    0.0187    0.0007     0.195
  END FTABLE2
```

Figure 4. Example FTABLE Describing Lower Gravel Layer of In-Ground Planter

4.4.2 Iterative IMP Sizing Steps

Once the geometric characteristics of the In-Ground Planter were represented in FTABLEs, the sizing factors were computed using an iterative process involving multiple HSPF simulations and statistical analyses. The process involved varying the surface area until peak flow and flow duration control were achieved.

The ability of the IMP to achieve peak flow and flow duration control was evaluated by generating and comparing partial duration series statistics and flow duration statistics for (a) the pre-project runoff from a pervious land surface and (b) the post-project outflow from the planter serving an equivalent area that has been converted to an impervious surface. A 24-hour inter-event period (as defined by 24 hours with IMP outflow less than 0.05 cfs/ac) was used to separate storm events in the partial duration series (see Section 5.1 for further discussion of the statistical analyses). The footprint of the IMP was included in the calculations to preserve equivalence between the pre-project and post-project analysis (i.e. Pre-project Area = Impervious Area + IMP Area). The HSPF model allowed rainfall directly on the IMP.

IMP surface area was increased incrementally with each iteration until flow and duration control were achieved. Flow and duration control were considered to be achieved when the post-project peak flows and flow durations were less than or equal to the pre-project flows for flow rates ranging from half the 2-year flow (0.5Q2) to the 10-year flow (Q10), within a "goodness of fit" standard (see Section 5.1.2 for a more detailed discussion of the curve matching procedure).

5. HSPF Modeling Results

This section describes:

- The statistical analyses used to analyze and summarize HSPF runoff time series (i.e., peak flow curves and duration plots), and how those analyses were used to evaluate IMP performance.
- Runoff time series generated by the HSPF model using the pre- and post-project parameters described in Section 3. Included in this section is a discussion of sensitivity analyses conducted to evaluate the importance of various parameters.
- The results of IMP modeling and sizing factors selected for each IMP.

5.1 FLOW AND DURATION CONTROL CURVES

As discussed in earlier sections, the consultant team used peak flow and duration curves to summarize pre- and post-project conditions for the different modeling scenarios, and to size hydrograph modification IMPs to mitigate post-project runoff.

5.1.1 Background

Peak flow analysis has been used for years as a way to analyze flow data sets and develop probability-based predictions for likely flood events. Most people are familiar with the concept of the 100-year flood, which is the flooding resulting from a flow event calculated to have a statistical probability of occurring of 0.01 in any given year. Figures 5 and 6 (in Section 5.2 below) are examples of peak flow curves. The Y axis of the curve represents simulated flow magnitude, and the X axis represents the flow recurrence frequency (e.g. the 5 year peak flow represents the peak flow rate that is equaled or exceeded an average of once every five years).

The peak flow curves were generated for each pervious land surface and impervious land surface scenario as follows. First, peak flows were calculated by parsing the 35-year HSPF runoff time series into individual runoff events to create a partial duration series[1]. Each runoff event has an associated peak flow and recurrence interval (as well as volume, starting time, duration, etc.). The peak flow, recurrence interval pairs were plotted for each scenario (as in Figure 6) to clearly demonstrate the influence of soil group and the impact of the conversion from pervious to impervious. For example, converting an acre of Group A soil pervious land to impervious increases the peak 10-year flow from 0.17 cfs to 0.68 cfs.

Flow durations are a newer concept, calculation of which has been made possible by use of continuous runoff models such as HSPF. Flow durations can be calculated directly from runoff time series, with less statistical manipulation than event peak flow analysis. For duration analysis the total amount of time a given flow is equaled or exceeded over the whole runoff time series (for the rainfall period of record) is plotted as a percent. Figure 7 illustrates a runoff duration curve. Low percentages of time exceeded are simply an illustration that over much of the simulation time series the pre-project unit area modeled did not generate runoff.

5.1.2 <u>Use of Peak Flow and Duration Curves in IMP Sizing</u>

As discussed in earlier sections, IMPs were sized by varying surface area until mitigated post-project discharges were less than or equal to the pre-project flows for flow rates ranging from half the 2-year flow (0.5Q2) to the 10-year flow (Q10). Because of the difficulty in achieving a precise match across the range of flows, jurisdictions commonly apply a "goodness of fit" standard to measure achievement of the matching objective.

The "goodness of fit" approach applied by the consultant team was based on the standard published in Washington State Department of Ecology (WADOE) 2001 Stormwater Management Manual for Western Washington, with modifications for Contra Costa County. The WADOE approach specifies a flow duration control standard and assumes that sites meeting flow duration control will also achieve peak flow control. The Western Washington flow duration control is summarized as follows:

- From 0.5Q2 to Q2, the post-project flow durations should not exceed the pre-developed condition. This recognizes the impact of these relatively frequent events on the stream channel stability.
- For flow rates above Q2, post-project flow durations should not exceed pre-development flow durations more than 10 percent of the time.

When adapting this standard to Contra Costa County, *pre-project* conditions are considered instead of *pre-development* conditions. In addition, the IMP modeling results

[1] Partial duration series statistics were used in this study instead of the more commonly used peak annual series statistics because the partial duration series provides better flow estimates for small events (particularly events 5-years and less). This is because the partial duration series includes all runoff events, whereas the peak annual series only includes the single largest event for each year and neglects all others, even if they are geomorphically significant.

indicate there are instances when the flow duration control standard is met, but the post-project peak flows exceed pre-project flows near the peak 10-year flow. Therefore, the WADOE "goodness of fit" standard was applied to flow duration control as-is, but it was modified for peak flows as follows:

- From 0.5Q2 to Q2, the post-project peak flows should not exceed the pre-project condition.
- For flow rates from Q2 to Q10, the post-project peak flows may exceed pre-project flows by up to 10 percent for a 1-year band within the 2 to 10 year recurrence interval range. For example, the post-project flows could exceed the pre-project flows by up to 10 percent between Q9 and Q10 or from Q5.5 to Q6.5, but not from Q8 to Q10.

5.2 PRE- AND POST-PROJECT RUNOFF SIMULATION RESULTS

The HSPF model was first run for the pre-project and post project scenarios to establish the differences in runoff to be mitigated by IMPs. As discussed in Section 3, the pre-project HSPF scenarios included Group A and Group D soils in combination with four cover types (scrub, range, irrigated pasture and live oak). However, after running the pre-project simulations, the consultant team discovered that the HSPF model was very insensitive to variation in cover type. This is because the cover types most commonly encountered in the region include little rainfall interception storage. Figure 5 shows peak flow results for Group D soils and demonstrates the similarity among the results for the four cover types. Group A soil simulations and flow duration statistics also showed very little sensitivity to the selected cover type.

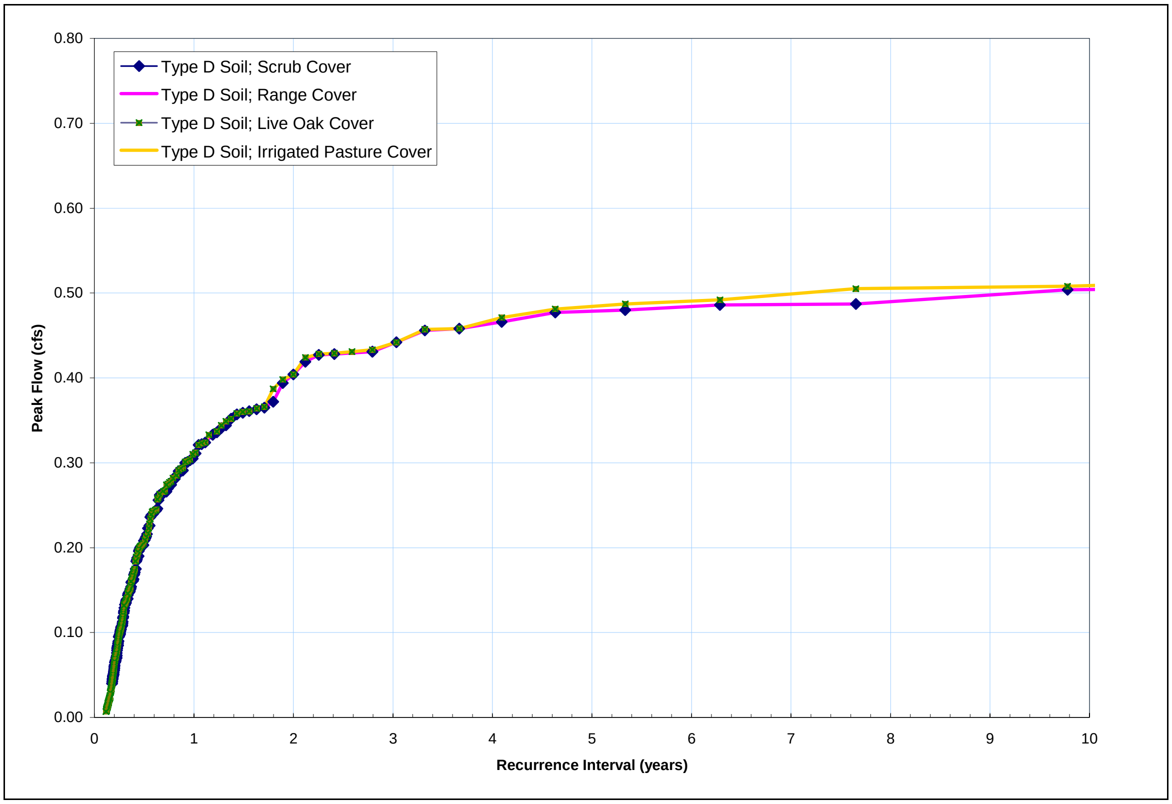

Figure 5. HSPF Peak Flow Frequency Statistics for Scrub, Range, Live Oak and Irrigated Pasture Covers

Because of the lack of sensitivity to cover type, the IMP simulations discussed in the remainder of this section all assume *scrub* cover. Cover type will however be included in the IMP Sizing Worksheet as a placeholder for possible future refinements to IMP sizing factors. Figure 6 shows the peak runoff frequency for Group A and Group D soil pervious areas with scrub cover, and for impervious surfaces, based on rainfall recorded at the Martinez gauge. Figure 7 shows flow duration statistics for the same pervious and impervious surfaces.

The peak flow graph in Figure 6 shows that Group D soils produce peak runoff rates that are approximately 70 to 90 percent of impervious surface peak runoff rates. Peak runoff rates from Group A soils are less than 10 percent of those from impervious surfaces. The flow duration graph in Figure 7 demonstrates a key difference between impervious surface and Group D soil runoff. While the peak flows are generally similar, high runoff rates occur for much longer periods for impervious surfaces than Group D soil pervious surfaces. Statistical analyses of the modeling results suggest that Group D soils produce approximately half the runoff volume of impervious surfaces over time.

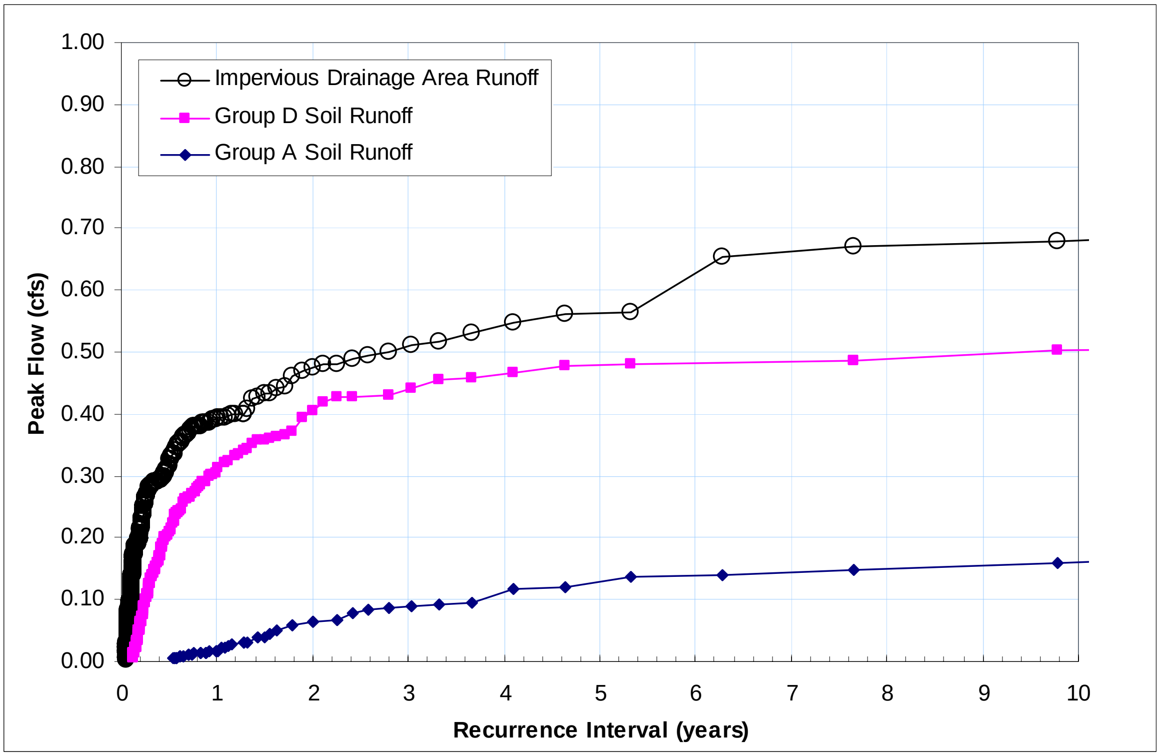

Figure 6. HSPF Runoff Peak Flow Frequency Statistics for 1-Acre Pervious and Impervious Surfaces

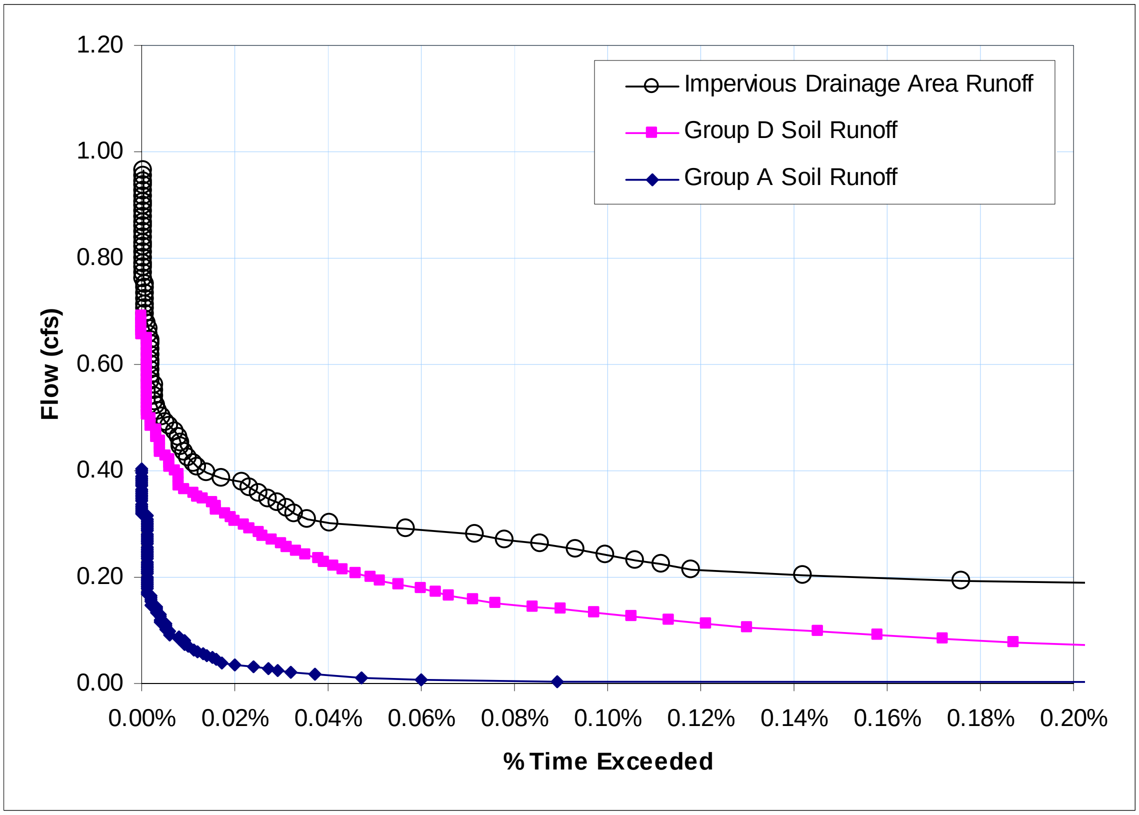

Figure 7. HSPF Runoff Flow Duration Statistics for 1-Acre Pervious and Impervious Surfaces

5.3 IMP MODELING RESULTS AND SELECTED SIZING FACTORS

This section contains a description of each IMP, the sizing factors selected, and the associated peak flow frequency and flow duration charts. Table 4 below summarizes the IMPs evaluated.

Table 4. List of IMPs for Contra Costa Clean Water Program

IMP	Description	Sizing Factors
In-Ground Planter	• Concrete-walled container for capturing and slowly infiltrating or transpiring stormwater. Top loamy soil layer captures and slowly percolates water to the lower layer. Plants may take up water, particularly between storms. • Above-ground ponding area provides peak flow storage. • May be adapted to Group A/B and Group C/D soils.	Group A: 0.08 Group D: 0.04
Flow-Through Planter	• Similar to In-Ground Planter except that concrete bottom prevents infiltration to surrounding soils. Appropriate for installation next to buildings.	Group D: 0.05
Vegetated/Grassy Swale	• Modified trapezoidal section with an upper layer of loamy soil and a lower layer of aggregate or drain rock. • Check dams encourage ponding and infiltration of low and moderate flows in addition to conveyance of very high flows. • Varying depths and bottom widths were considered. • May be adapted to Group A/B and Group C/D soils.	Group A: 0.10 to 0.14 Group D: 0.07 to 0.115
Bioretention Basin	• Depressed landscaping feature that encourages capture and infiltration of stormwater. Includes an upper layer of loamy soil and a lower layer of aggregate or drain rock. • May be adapted to Group A/B and Group C/D soils.	Group A: 0.13 Group D: 0.06
Dry Well	• Buried well filled with drain rock. An upper sand layer filters runoff and helps delay clogging at the interface to native soils.Provides a hidden approach to controlling runoff. Often used to receive roof downspout flow. Utility hatch hides entrance to dry well. • Stormwater infiltrates to native soils. Installed in Group A/B soils only.	Group A: 0.05 to 0.06
Infiltration Trench	• Similar to Dry Well without the utility hatch and below-ground feed from a downspout. A surrounding berm provides temporary detention while stormwater infiltrates into the trench. • Stormwater infiltrates to surrounding soils. Installed in Group A/B soils only.	Group A: 0.05 to 0.06
Infiltration Basin	• Excavated basin detains runoff for infiltration to exposed permeable layers of native soil. • Installed in Group A/B soils only.	Group A: 0.05 to 0.10

Attachment 2

5.3.1 In-Ground Planter

Table 5 below lists the recommended sizing factors that resulted from applying the process described in this memorandum to the In-Ground Planter described in Section 4.4. These values represent the size of the planter relative to the impervious area draining to it. For example, one acre of new impervious area in Group D soils could be controlled by installing an In-Ground Planter measuring 0.04 acres. The sizing factor for Group A soils is larger than for Group D soils for two reasons. First, Group A soils produce little runoff, requiring a greater reduction of the runoff from new impervious surfaces. Second, the infiltration rate to surrounding soils is less than the peak allowable underdrain flow (0.5Q2) in Group D applications.

Table 5. In-Ground Planter Performance: Sizing Factors

IMP Group / Soil	Sizing Factor
In-Ground Planter / Group A soils	0.08
In-Ground Planter / Group D soils	0.04

Figure 8 through Figure 11 below are the peak and duration matching curves that illustrate the performance of In-Ground Planters sized according to the selected sizing factors.

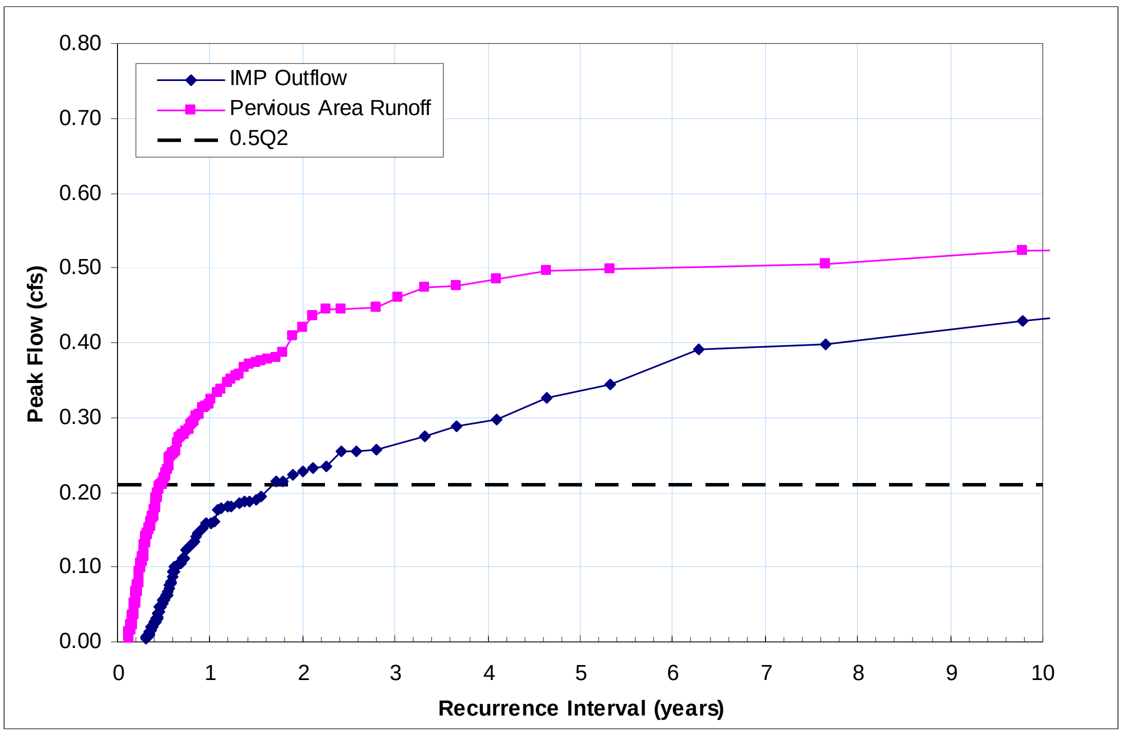

Figure 8. Peak Flow Statistics for In-Ground Planter Controlling 1-ac Impervious in Group D Soils Sizing Factor = 0.04

Attachment 2

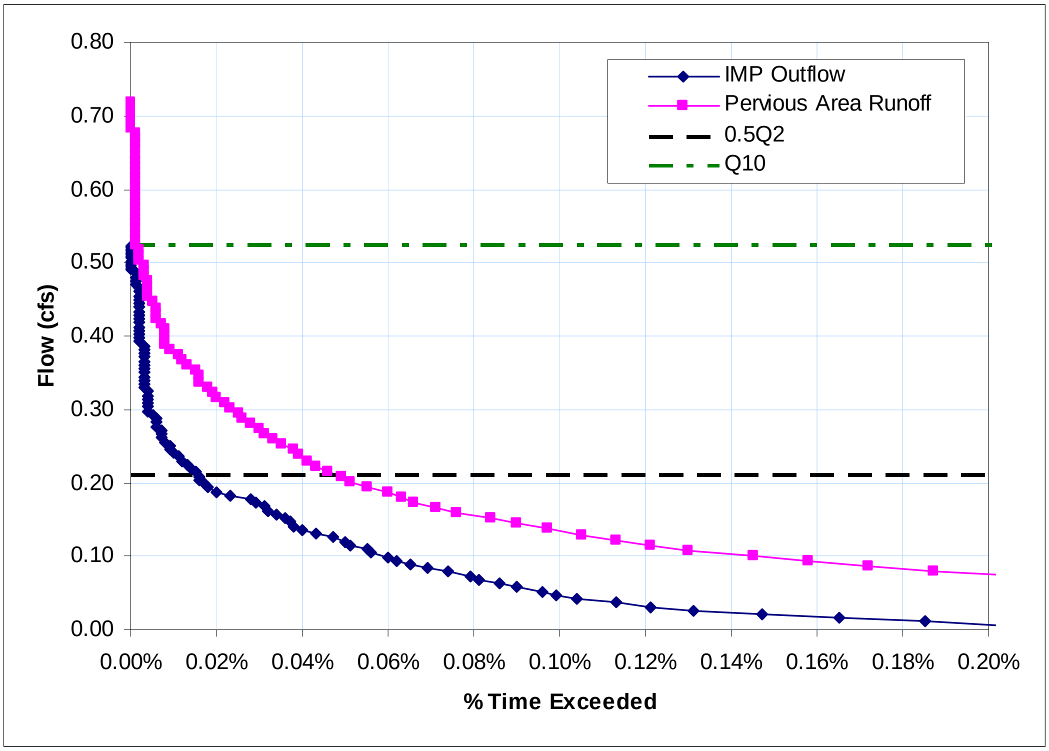

Figure 9. Flow Duration Statistics for In-Ground Planter Controlling 1-ac Impervious in Group D Soils Sizing Factor = 0.04

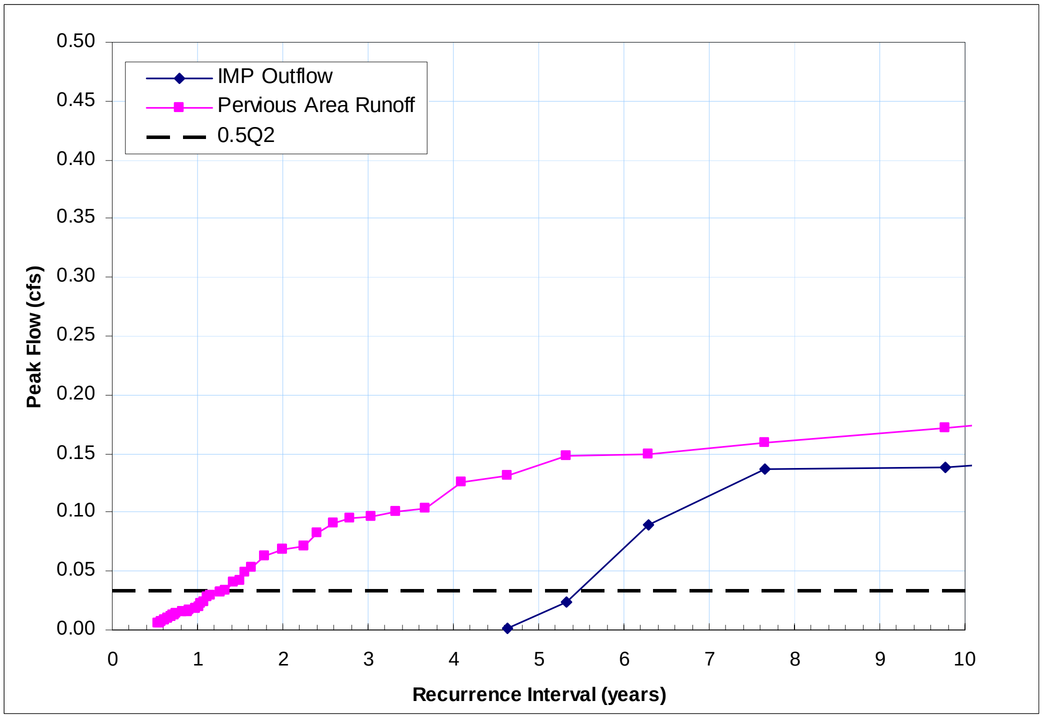

Figure 10. Peak Flow Statistics for In-Ground Planter Controlling 1-ac Impervious in Group A Soils Sizing Factor = 0.08

Attachment 2

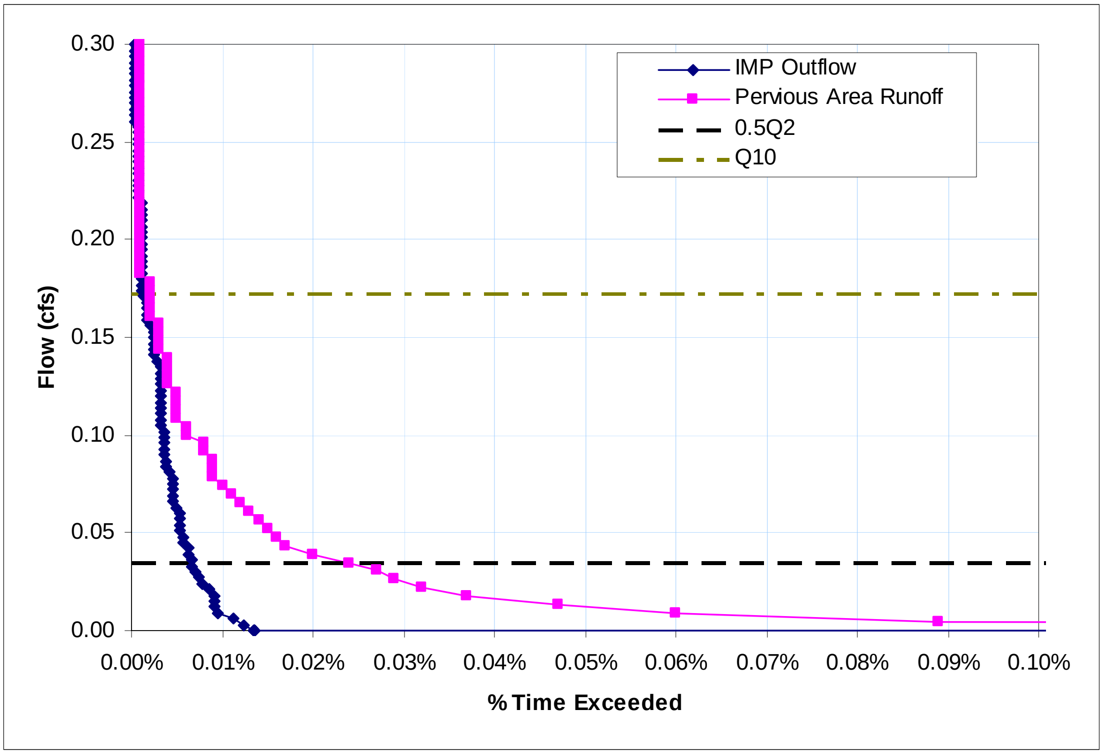

Figure 11. Flow Duration Statistics for In-Ground Planter Controlling 1-ac Impervious in Group A Soils Sizing Factor = 0.08

5.3.2 Flow-Through Planter

This IMP is similar to the In-Ground Planter, except that it includes a concrete bottom to eliminate infiltration to the surrounding soils (see Figure 12 below). The concrete bottom will prevent infiltrated water from pooling near foundations, making the Flow-Through Planter appropriate for use adjacent to buildings. Because infiltration to surrounding soils is prevented, the Flow-Through Planter must contain an underdrain and as such is only appropriate to use in NRCS Group C and Group D soils.

Attachment 2

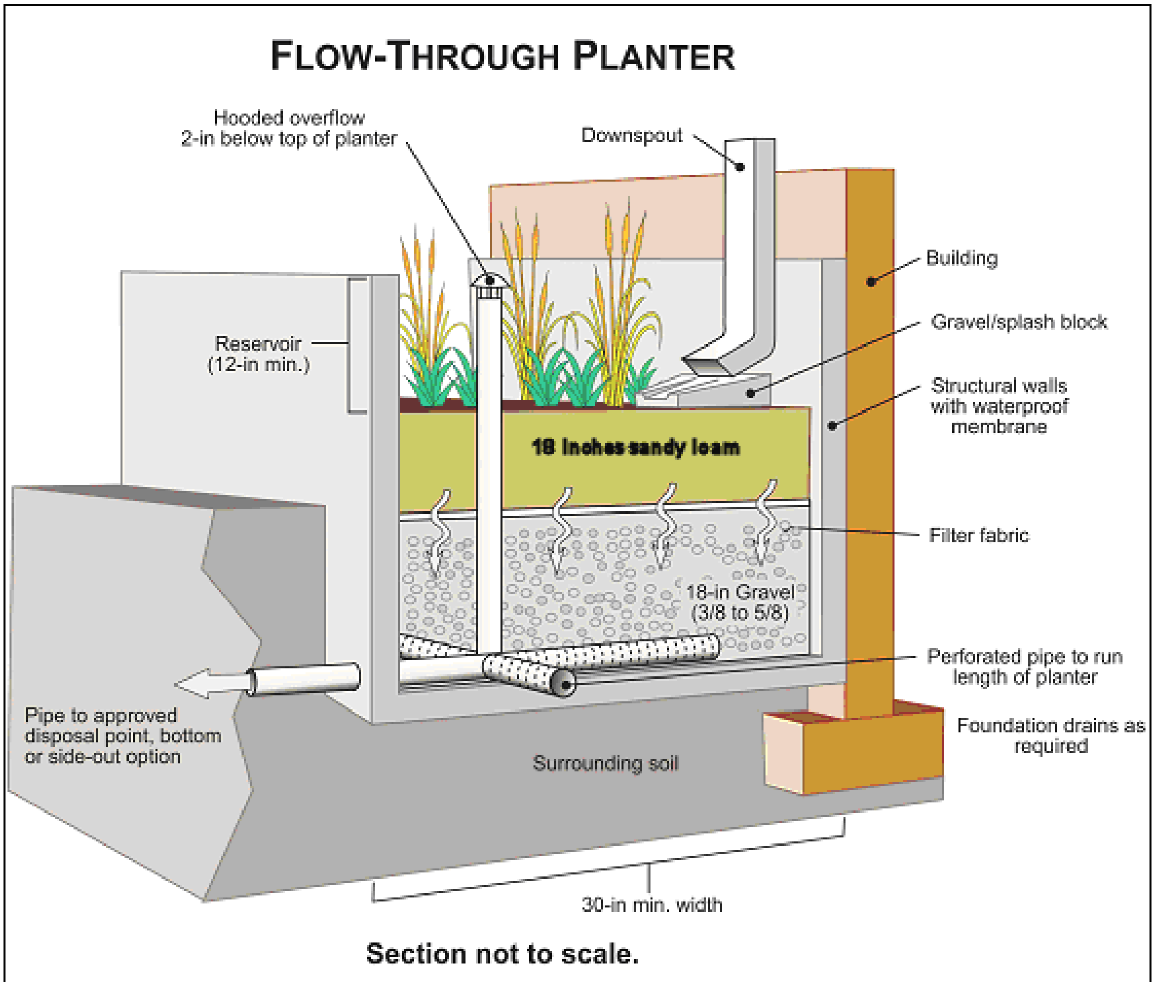

Figure 12. Cross-Section View of Flow-Through Planter, Group C/D Soil Configuration

The sizing factor for Group D soils is equivalent to the In-Ground Planter (Table 6).

Table 6. Flow-Through Planter Performance: Partial Duration Statistics

IMP Group / Soil	Sizing Factor
Flow-Through Planter / Group D soils	0.05

Figure 13 and Figure 14 below are the peak and duration matching curves that illustrate the performance of Flow-Through Planters sized according to the selected sizing factors.

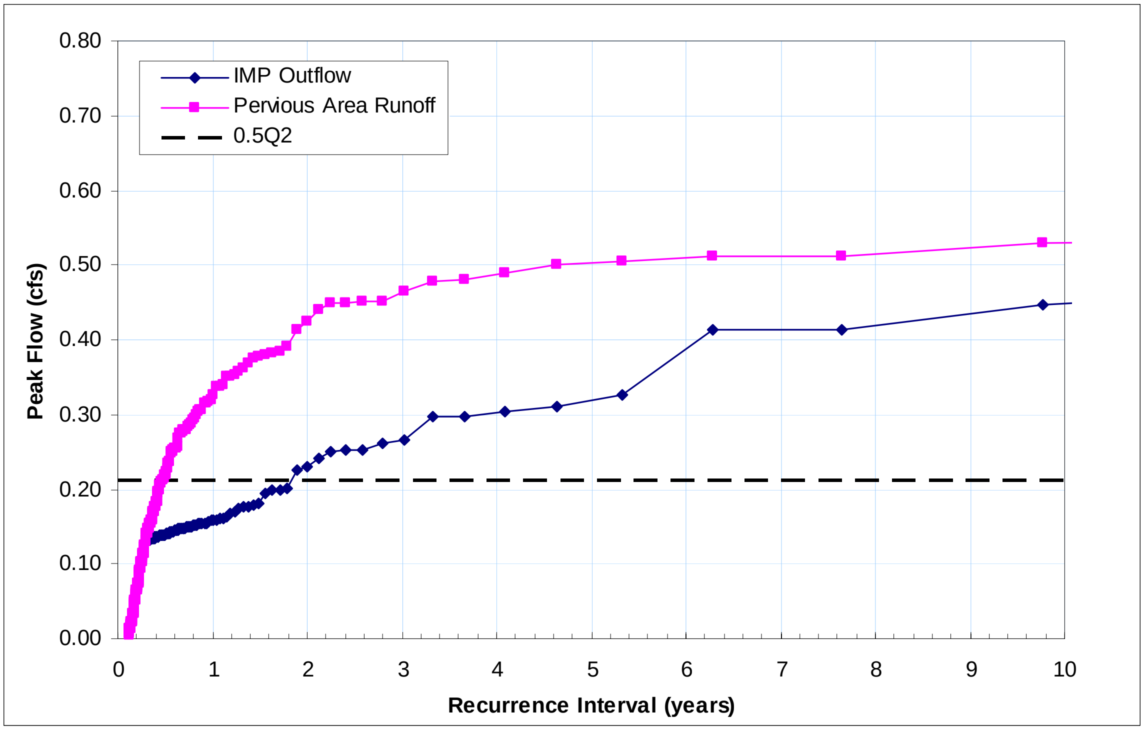

Figure 13. Peak Flow Statistics for Flow-through Planter Controlling 1-ac Impervious in Group D Soils Sizing Factor = 0.05

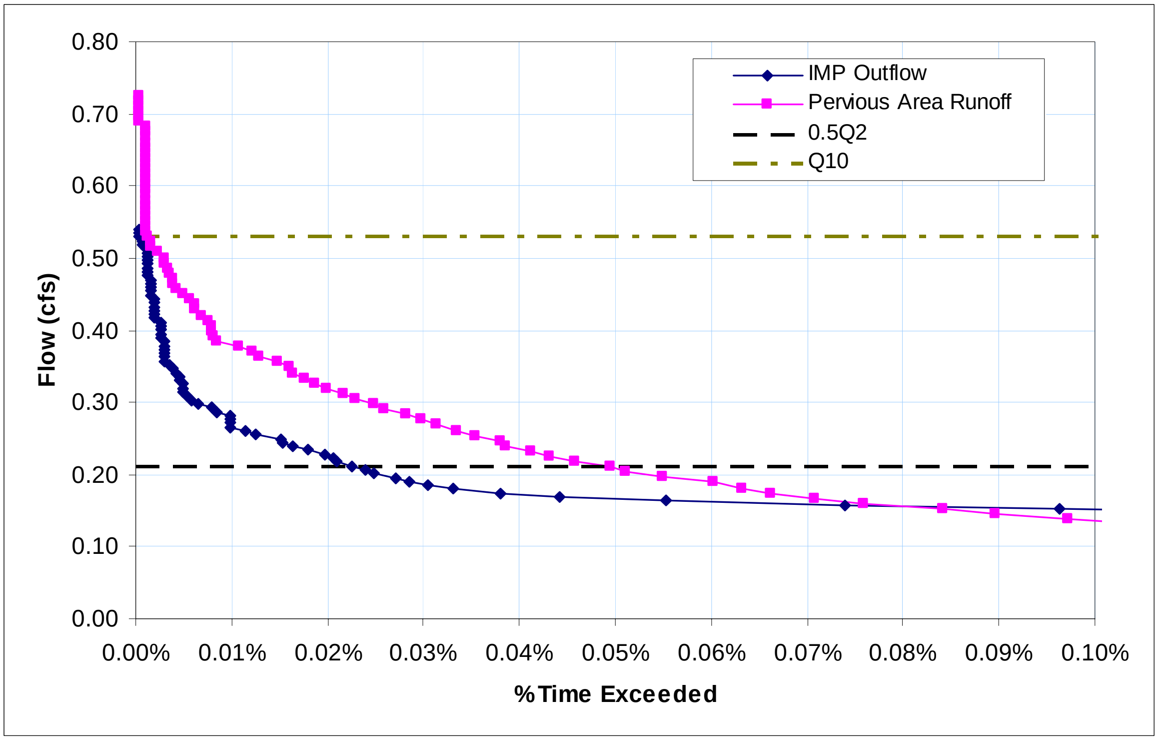

Figure 14. Flow Duration Statistics for Flow-through Planter Controlling 1-ac Impervious in Group D Soils Sizing Factor = 0.05

Attachment 2

5.3.3 Vegetated/Grassy Swale

A vegetated/grassy swale consists of a series of linear, trapezoidal channel segments with check dams to encourage ponding and infiltration rather than surface flow between segments. In hilly areas, a vegetated/grassy swale could be constructed as a series of near-flat channel segments with vertical drop structures between the segments, enabling the swale to capture and infiltrate water while following the slope of the land. The channel consists of 18-inches of growing medium along the bottom and side slopes. This growing medium is specified to be a sandy loam soil. No underdrain is included when the swale is placed in well-draining Group A or B soils (Figure 15). In Group C or D soils an additional 2-ft gravel layer is placed beneath the growing medium and an underdrain is placed along the bottom (Figure 16).

The FTABLEs used to model the swale are set up in a similar form as those for stormwater planters. The overflow calculations are calculated differently because a straight weir across the channel is used for overflow instead of a circular pipe. The weir is assumed to be a sharp-crested weir with the crest set approximately 2 inches below the top of the channel. The weir also included a 90-degree v-notch with a notch height of approximately half the total height of the weir. For Group A and B soil applications, the lower gravel layer was not included in the configuration and a second FTABLE was not used in the HSPF model.

Vegetated/Grassy Swales may be built in various dimensions, with bottom widths and ponding depths chosen to match available rights-of-way, easements or required IMP volume. Rather than computing a single sizing factor for each soil Group like in the earlier examples, required lengths for the Vegetated/Grassy Swale were computed for bottom widths of 2 ft, 4 ft and 6 ft; and depths of 0.5 ft, 1 ft and 1.5 ft (Table 7,Table 8). The side slopes area assumed to always be 4H:1V.

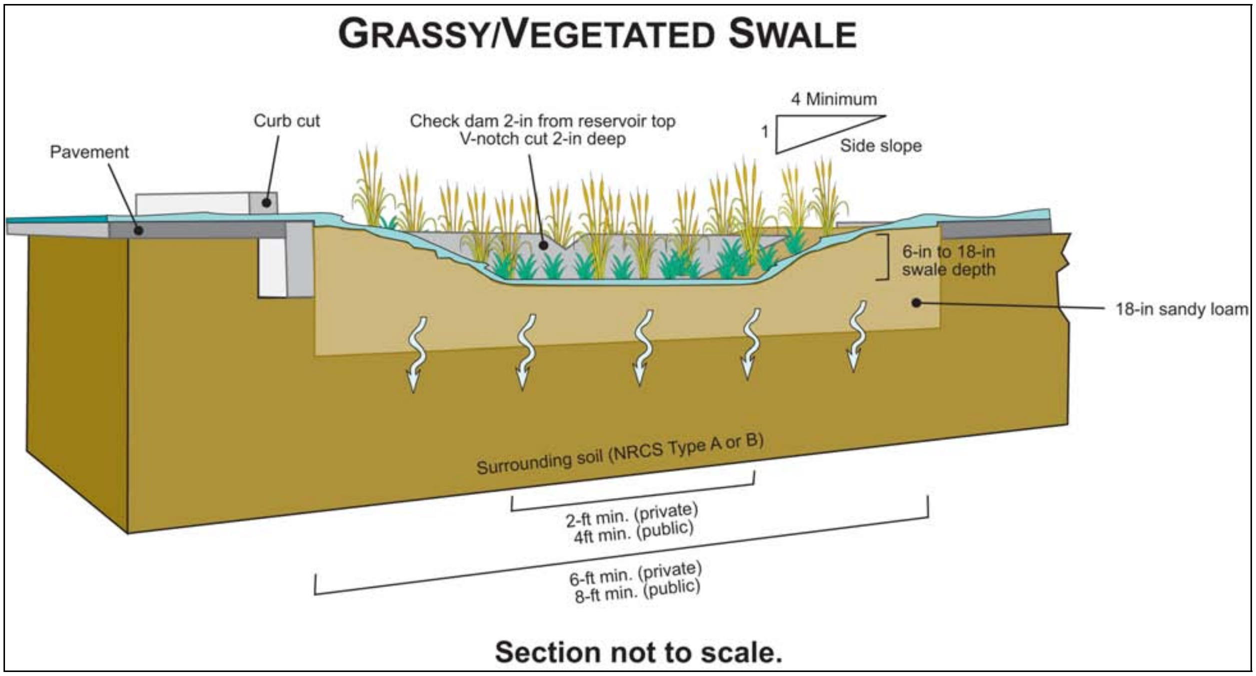

Figure 15. Cross-Section View of Vegetated/Grassy Swale, Group A/B Soil Configuration

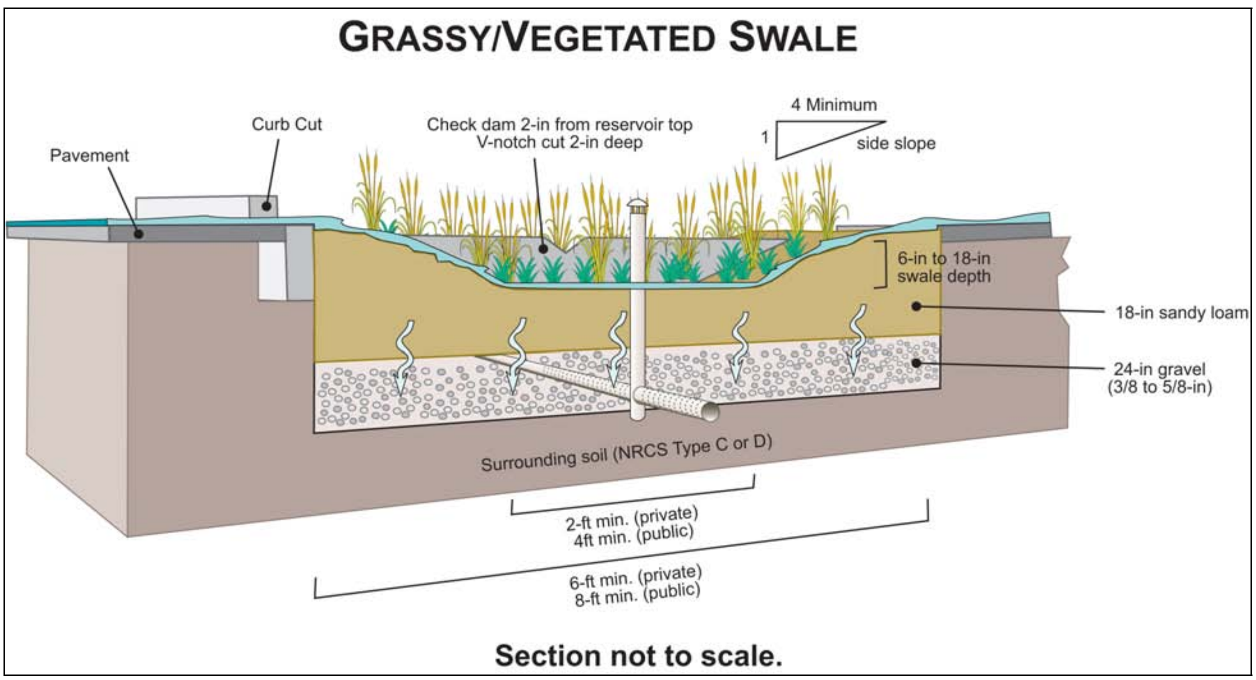

Figure 16. Cross-Section View of Vegetated/Grassy Swale, Group C/D Soil Configuration

Table 7. Vegetated/Grassy Swale Performance: Sizing Factors for Group D Soils

Bottom Width (ft)	Swale Depth (ft)	Total Swale Width (ft)	Required Length (ft)	Swale Footprint (ac)[A]
2	0.5	6	700	0.096
2	1.0	10	350	0.080
2	1.5	14	225	0.072
4	0.5	8	550	0.101
4	1.0	12	275	0.076
4	1.5	16	200	0.073
6	0.5	10	500	0.115
6	1.0	14	225	0.072
6	1.5	18	175	0.072

A. The Swale Footprint is analogous to the Sizing Factor computed for the other IMPs, because it corresponds to the land space occupied by the Vegetated/Grassy Swale.

Table 8. Vegetated/Grassy Swale Performance: Sizing Factors for Group A Soils

Bottom Width (ft)	Swale Depth (ft)	Total Swale Width (ft)	Required Length (ft)	Swale Footprint (ac)[A]
2	0.5	6	1,000	0.138
2	1.0	10	500	0.115
2	1.5	14	325	0.104
4	0.5	8	750	0.138
4	1.0	12	375	0.103
4	1.5	16	275	0.101
6	0.5	10	625	0.143
6	1.0	14	325	0.104
6	1.5	18	250	0.103

A. The Swale Footprint is analogous to the Sizing Factor computed for the other IMPs, because it corresponds to the land space occupied by the Vegetated/Grassy Swale.

Figure 17 through Figure 20 below are the peak and duration matching curves that illustrate the performance of Vegetated/Grassy Swales sized according to the selected sizing factors.

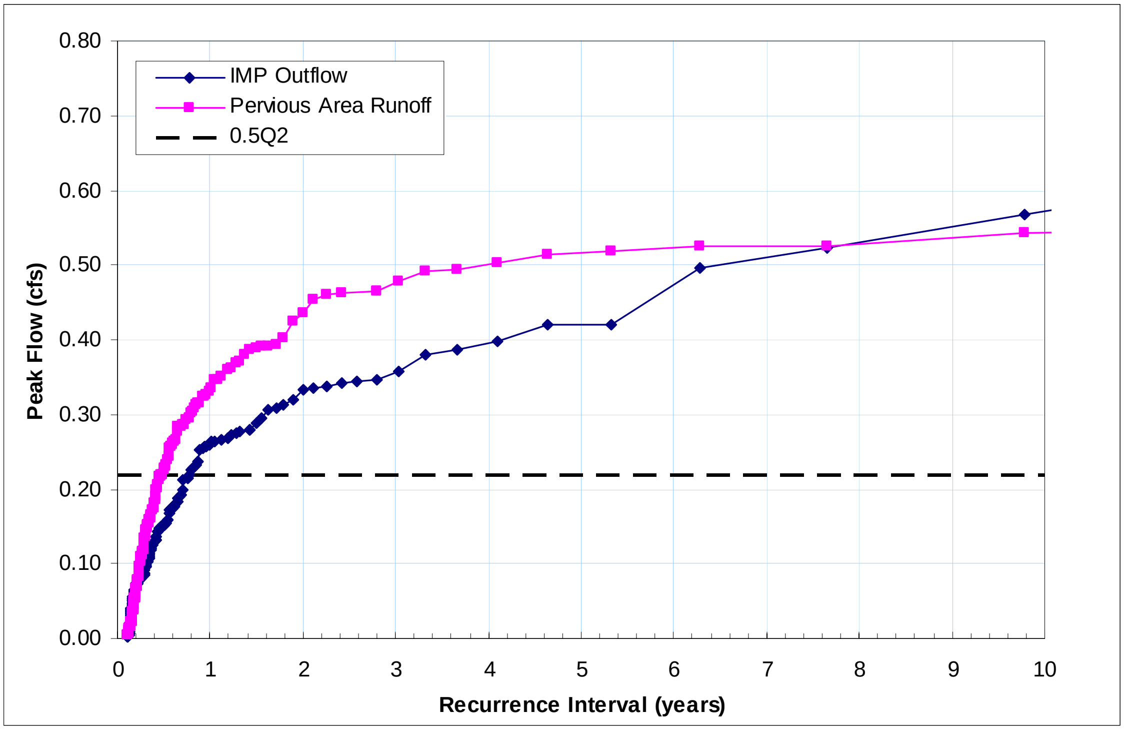

Figure 17. Peak Flow Statistics for Vegetated/Grassy Swale Controlling 1-ac Impervious in Group D Soils; Bottom Width = 2 ft; Depth = 1 ft; Length = 350 ft

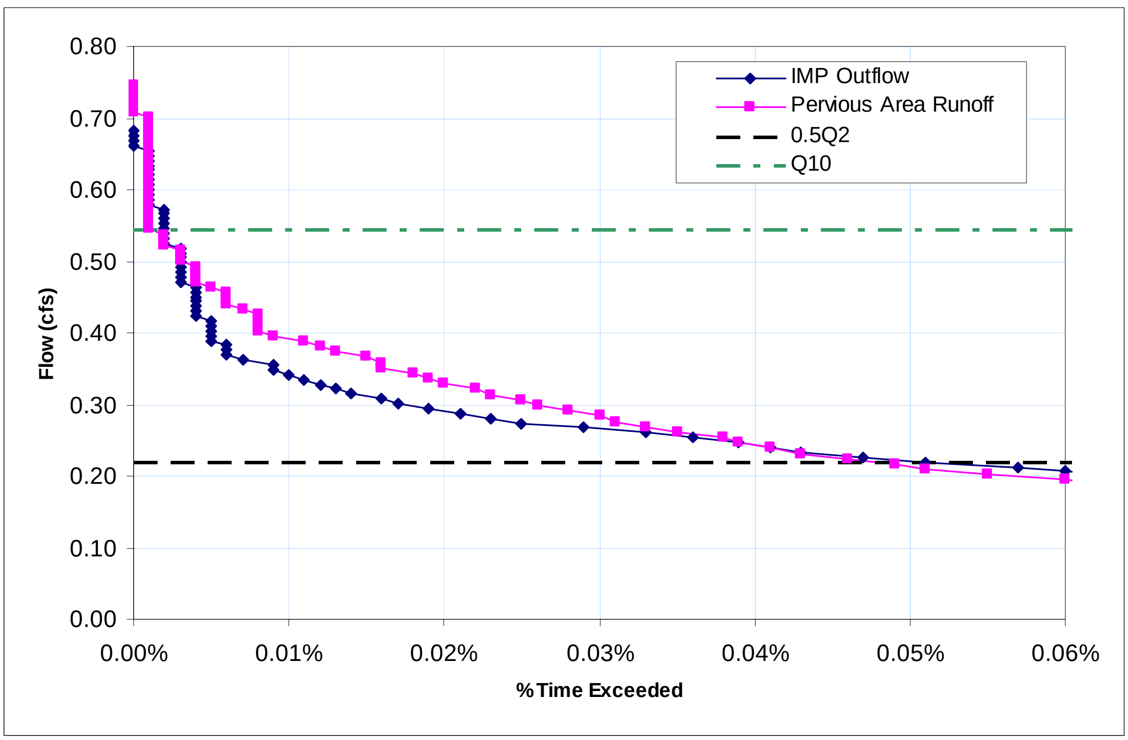

Figure 18. Flow Duration Statistics for Vegetated/Grassy Swale Controlling 1-ac Impervious in Group D Soils; Bottom Width = 2 ft; Depth = 1 ft; Length = 350 ft

Attachment 2

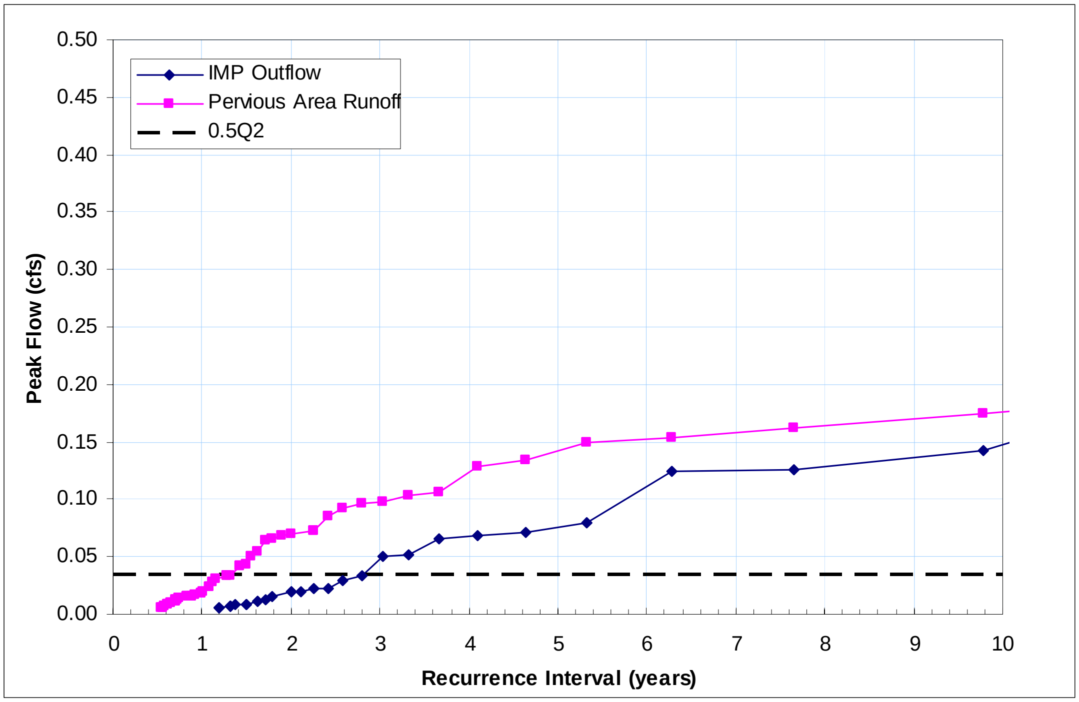

Figure 19. Peak Flow Statistics for Vegetated/Grassy Swale Controlling 1-ac Impervious in Group A Soils Bottom Width = 2 ft; Depth = 1 ft; Length = 500 ft

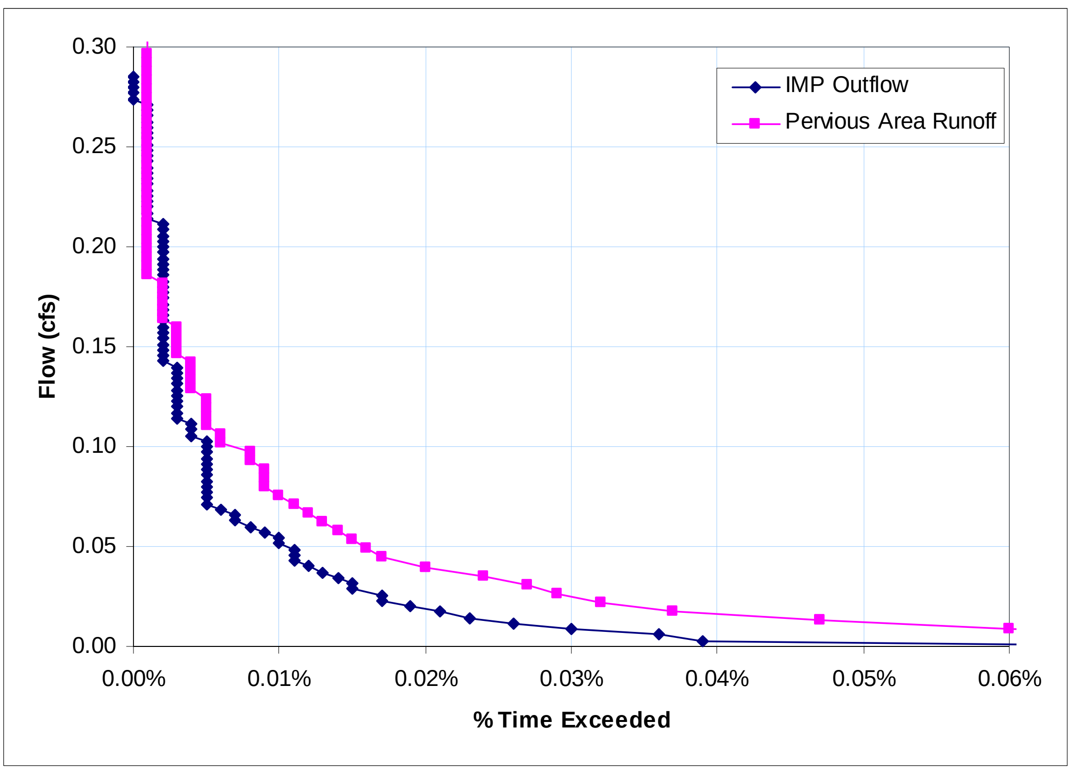

Figure 20. Flow Duration Statistics for Vegetated/Grassy Swale Controlling 1-ac Impervious - Group A Soils Bottom Width = 2 ft; Depth = 1 ft; Length = 500 ft

5.3.4 Bioretention Basin

The Bioretention Basin is similar to the vegetated/grassy swale, except that it is configured as a rectangular or irregular shape instead of a channel. Also, a vertical riser pipe outlet is used for overflow instead of a weir. The Bioretention Basin contains 12-inches of surface ponding and an 18-inch thick growing medium layer, assumed to be a sandy loam soil. If the basin is placed in Group A or B soils there is no underdrain. If the basin is installed in Group C or D soils, a 48-in thick gravel layer is included below the growing medium. The gravel layer contains an underdrain, because Group C and D soils are poor draining (Figure 21).

The volume-to-area relation for the Bioretention Basin is dependent on the shape of the basin (in plan view). For the purposes of sizing the basin the minimum width of the basin is assumed to be 12 ft. The side slopes are assumed to be 4H:1V on all four sides. The FTABLEs are set up similar to the stormwater planter and vegetated/grassy swale. If a gravel layer and underdrain are not included then only the first FTABLE is used.

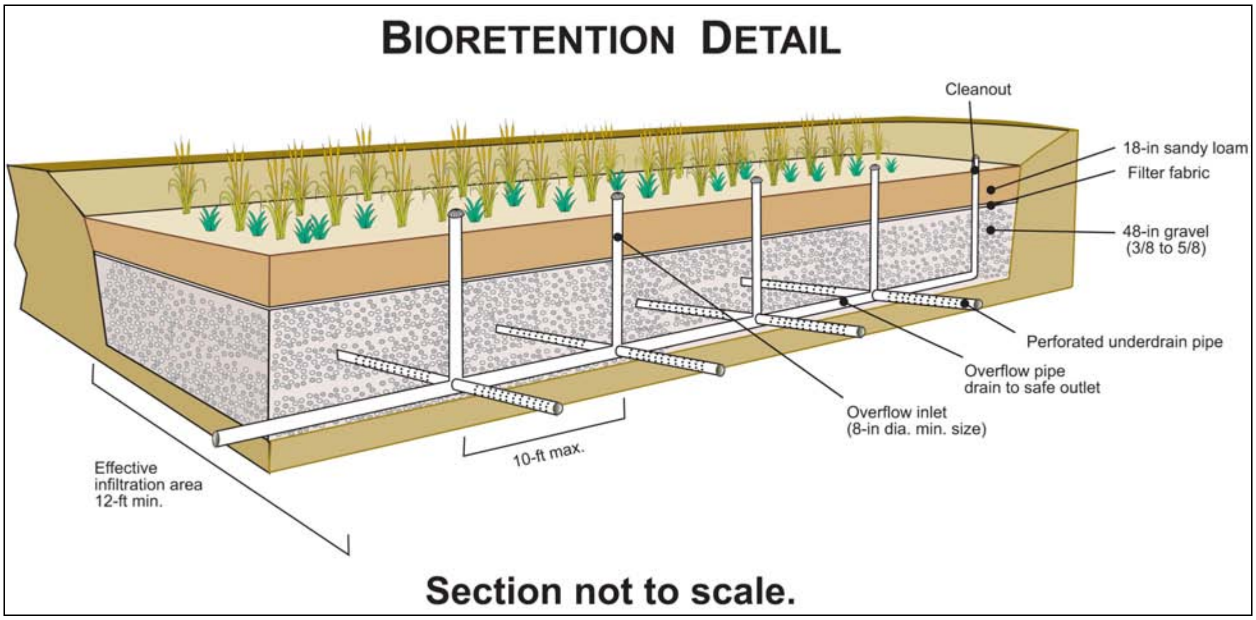

Figure 21. Cross-Section View of Bioretention Basin

Attachment 2

Table 9. Bioretention Basin Performance: Sizing Factors

Soil Group	Bioretention Basin Sizing Factor
A	0.13
D	0.06

Figure 22 through Figure 25 illustrate the performances of an appropriately sized Bioretention Basin for Group A and Group D soil areas. The sizing factors are larger than most other IMPs because the gentle side slopes provide less above-ground storage for a given footprint.

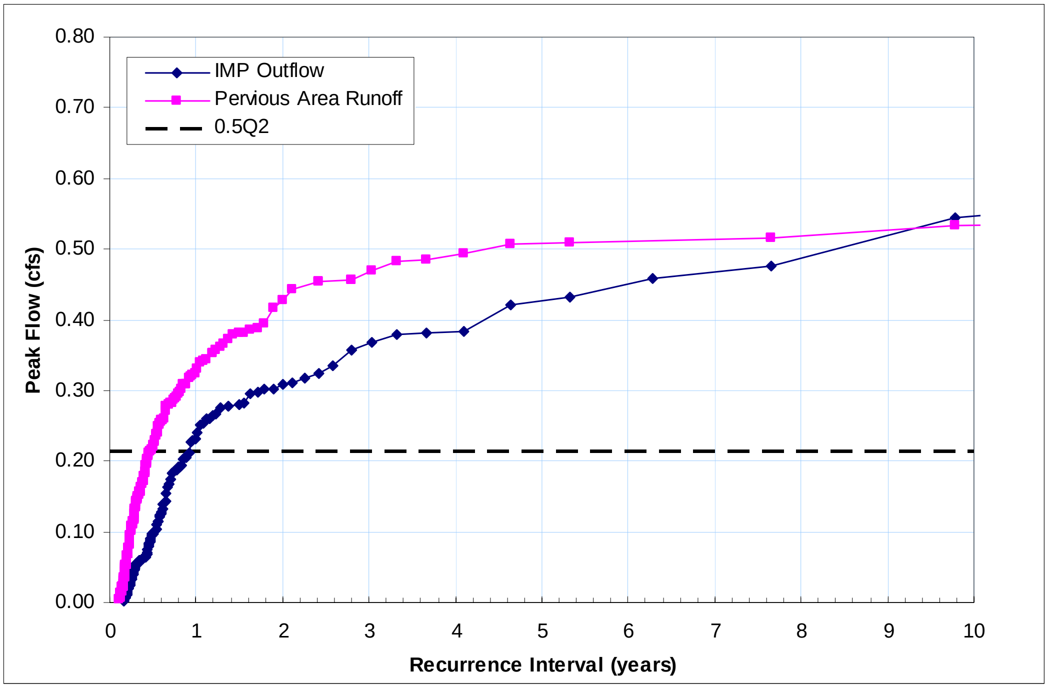

Figure 22. Peak Flow Statistics for Bioretention Basin IMP Controlling 1-ac Impervious in Group D Soils Max. Ponding Depth = 1 ft; Sizing Factor = 0.06

Attachment 2

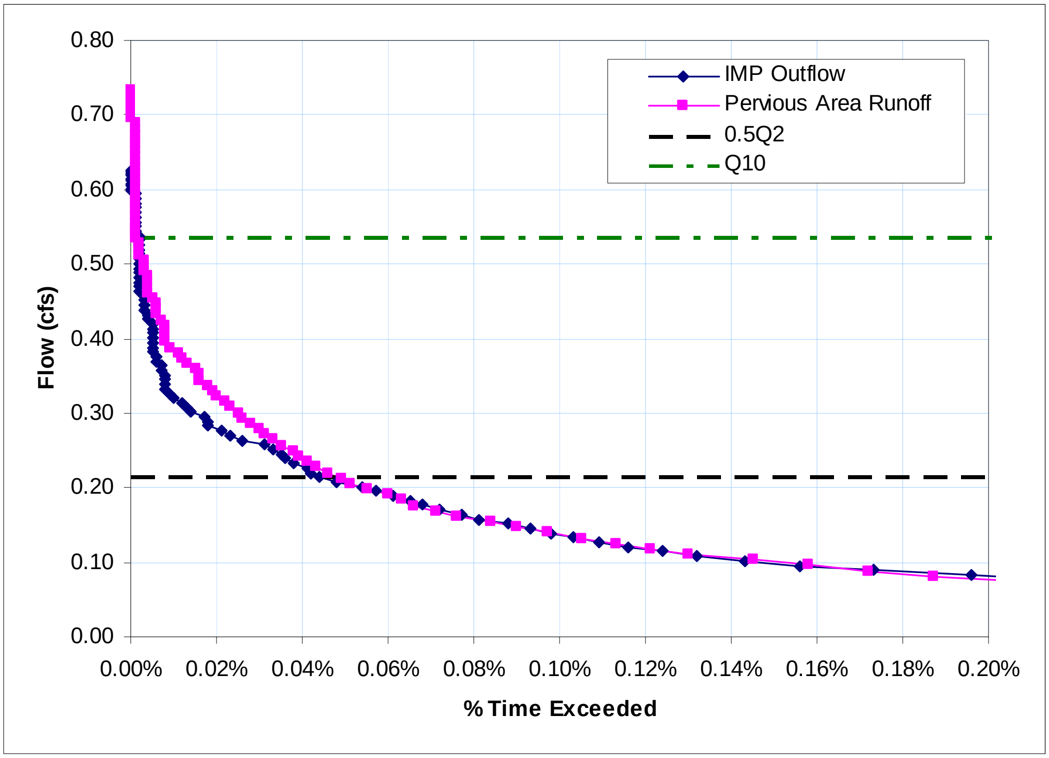

Figure 23. Flow Duration Statistics for Bioretention Basin IMP Controlling 1-ac Impervious - Group D Soils Max. Ponding Depth = 1 ft; Sizing Factor = 0.06

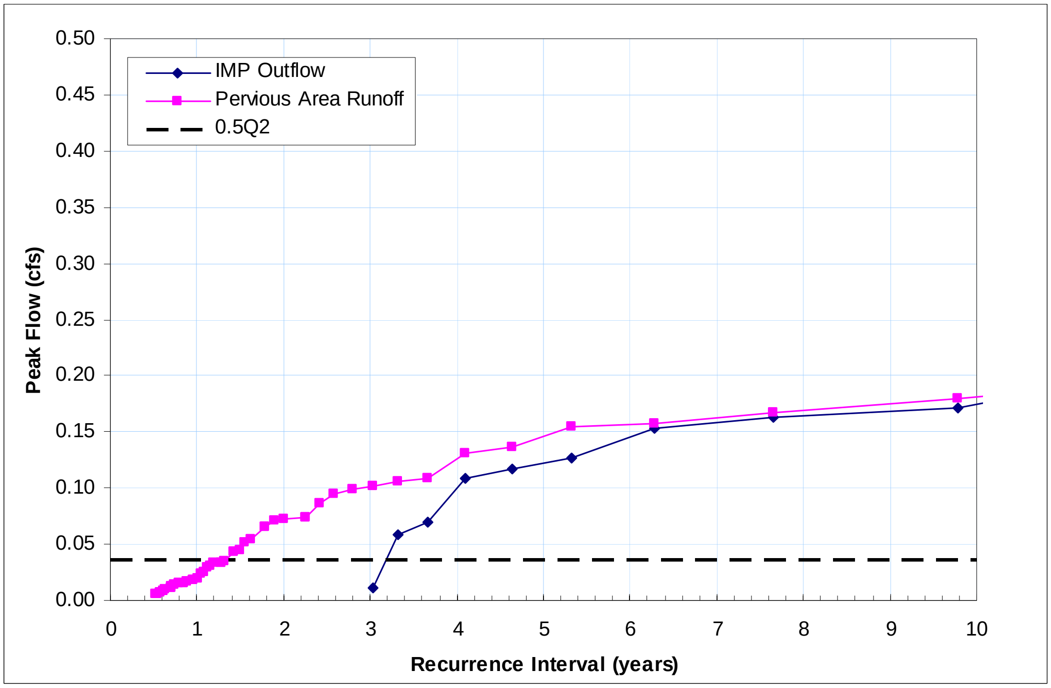

Figure 24. Peak Flow Statistics for Bioretention Basin IMP Controlling 1-ac Impervious in Group A Soils Max. Ponding Depth = 1 ft; Sizing Factor = 0.13

Attachment 2

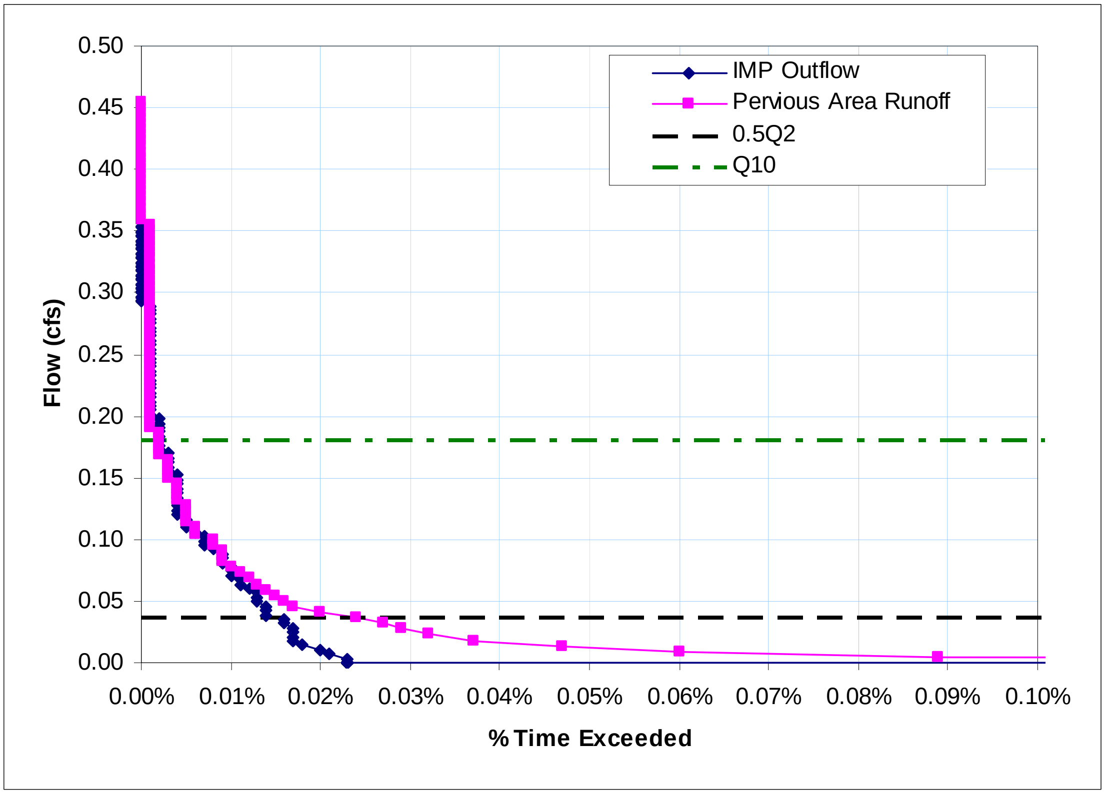

Figure 25. Flow Duration Statistics for Bioretention Basin IMP Controlling 1-ac Impervious - Group A Soils Max. Ponding Depth = 1 ft; Sizing Factor = 0.13

5.3.5 Dry Well

The dry well consists primarily of an aggregate backfill with a thin layer of sand on top. Runoff is collected and piped into a reservoir above the soil layer. This reservoir has a depth of one foot below the ground surface. The top of the reservoir is covered by a utility hatch or similar safety device (Figure 26).

Stormwater infiltrates into the top soil layer, percolates through the aggregate and sandy soil, and exfiltrates out the bottom into the surrounding soil. The rate of exfiltration into the surrounding soil is the controlling rate for flow through the drywell. The exfiltration rate is assumed to be limited to the saturated hydraulic conductivity of the surrounding soil. This IMP can only be used in well-draining (Group A or B) soils. For simplification, the aggregate and sandy layers were combined into one layer in HSPF and modeled as a free-draining gravel layer. Sizing factors were computed for four different aggregate depths: 3 ft, 4 ft, 5 ft and 6 ft (Table 10).

Only one FTABLE is necessary to model the dry well. For depths below the ponding reservoir the volume of water stored is assumed to be the volume of the drywell multiplied by the porosity of the aggregate, which was assumed to be 0.4. The outflow

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from the drywell (into the surrounding soil) is equal to the saturated hydraulic conductivity of the surrounding soils.

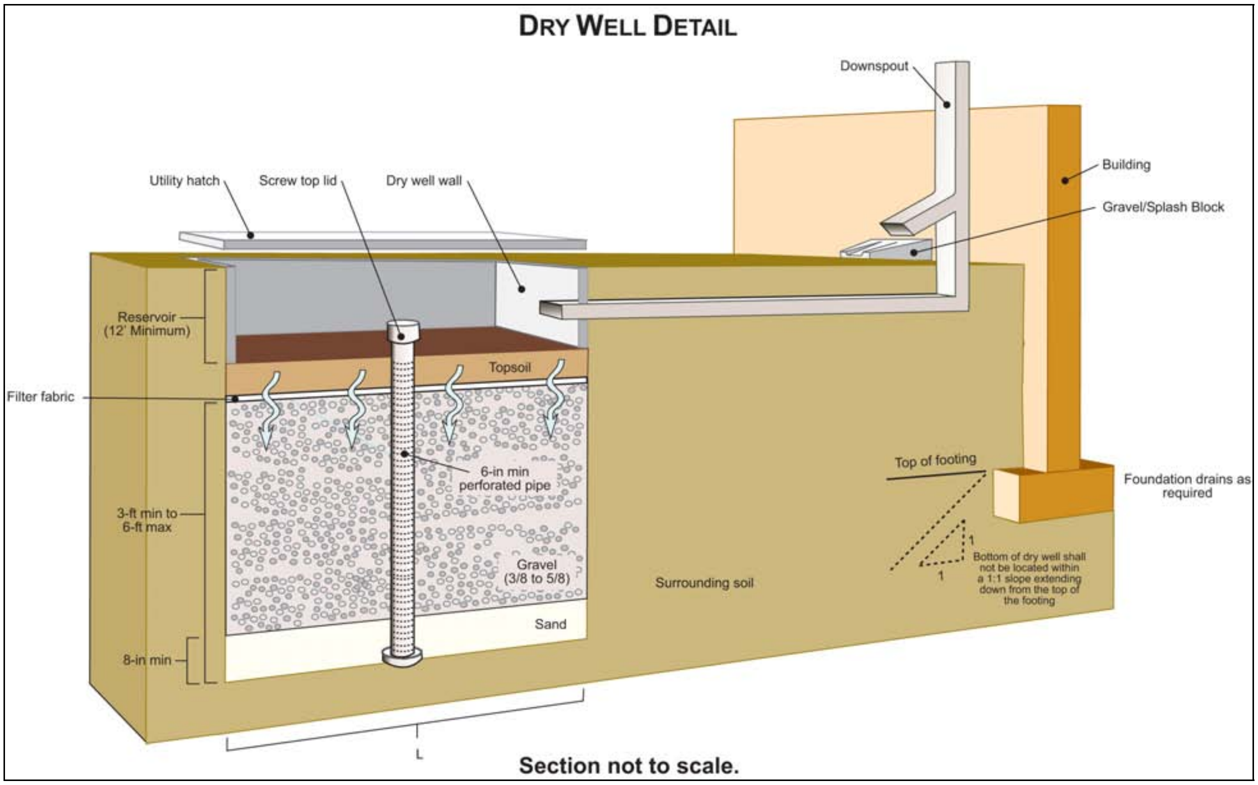

Figure 26. Cross-Section View of Dry Well IMP, Group A/B Soil Configuration

Table 10. Dry Well IMP Performance: Sizing Factors

Dry Well Depth	Sizing Factor Group A Soil
3 ft	0.06
4 ft	0.05
5 ft	0.05
6 ft	0.045

Figure 27 and Figure 28 illustrate the performances of an appropriately sized drywell for Group A soils.

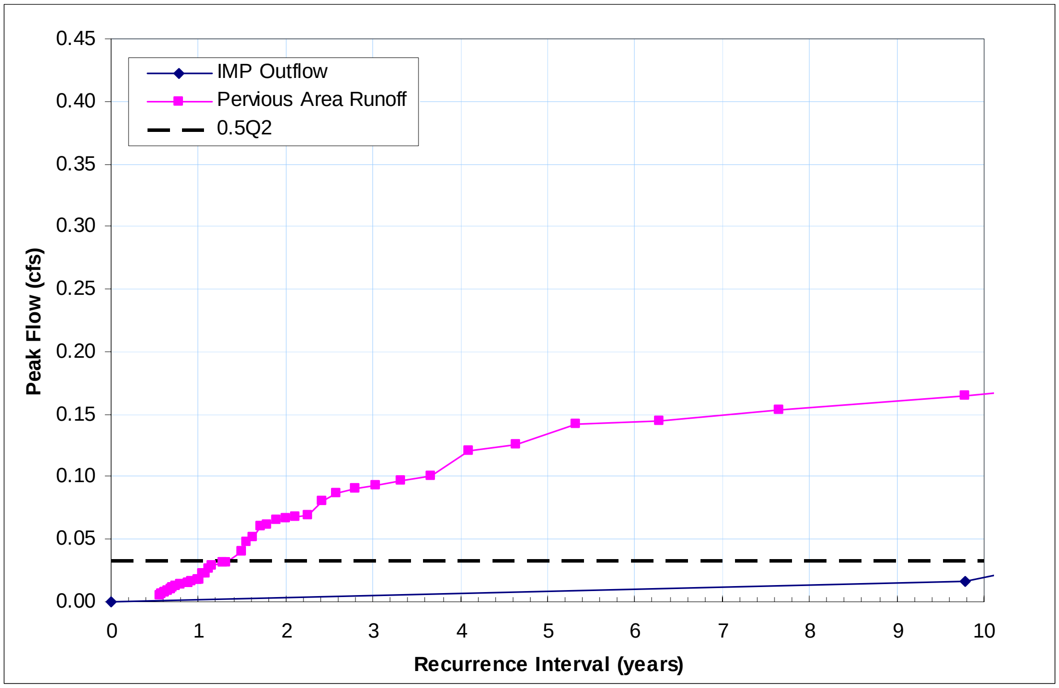

Figure 27. Peak Flow Statistics for Dry Well IMP for 1-ac Impervious in Group A Soils Depth = 3 ft; Sizing Factor = 0.06

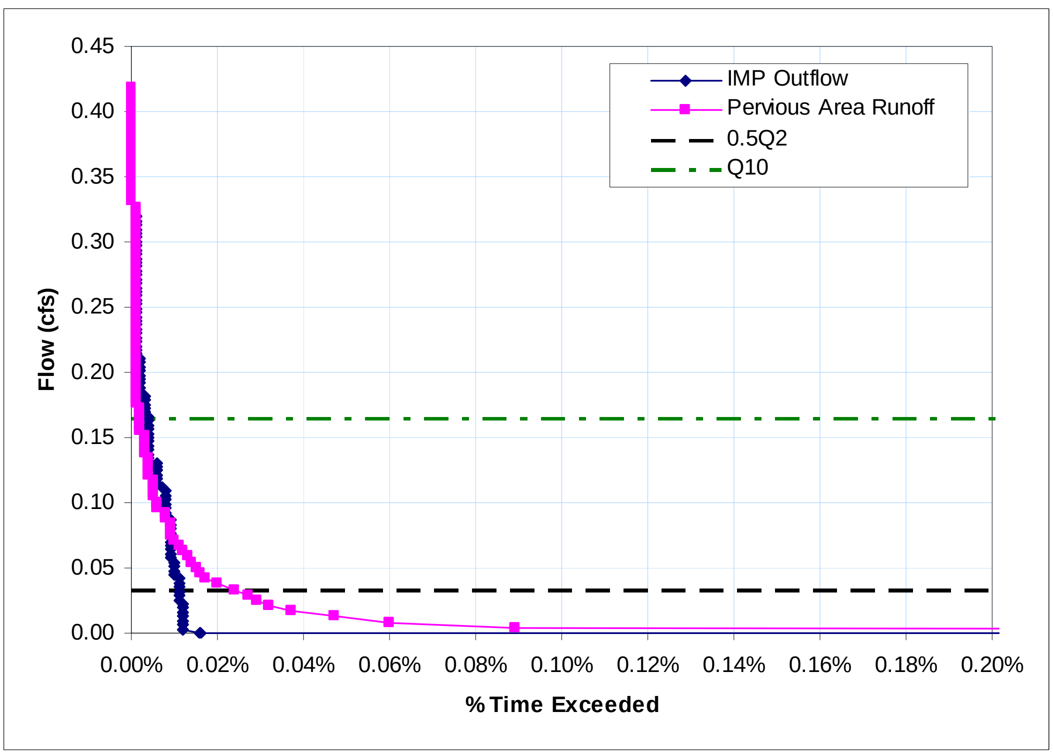

Figure 28. Flow Duration Statistics for Dry Well IMP for 1-ac Impervious in Group A Soils Depth = 3 ft; Sizing Factor = 0.06

5.3.6 Infiltration Trench

The infiltration trench is similar to the drywell except it is not covered and the top is not below the surface. Instead of a covered ponding reservoir the infiltration trench has a small berm surrounding it to contain overflow and allow shallow ponding (Figure 28).

As with the drywell, the rate at which water flows through the trench is limited by the rate at which water exfiltrates into the surrounding soil. This rate is assumed to be equal to the saturated hydraulic conductivity of the soil and is constant for all depths of storage within the trench. The infiltration trench can only be used in well-draining (A or B-Group) soils.

The trench is sized for surface area. Sizing factors were developed for various combinations of trench depth and overflow berm height. Two berm heights were selected: 0.5 ft and 1.0 ft. Three trench depths were selected: 3, 4 and 5 ft. Only one FTABLE is used to model the infiltration trench. Table 11 lists the appropriate sizing factors.

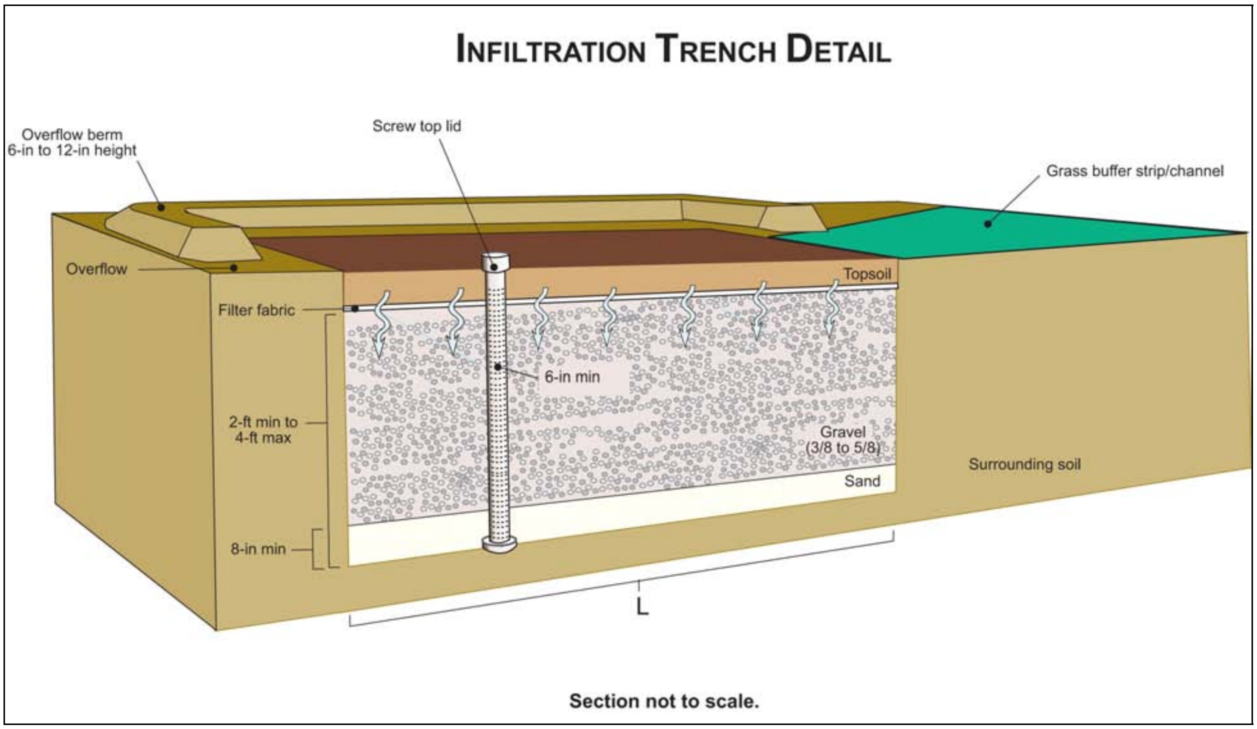

Figure 29. Cross-Section View of Infiltration Trench IMP, Group A/B Soil Configuration

Attachment 2

Table 11. Infiltration Trench IMP Performance: Sizing Factors

Overflow Berm Height	Infiltration Trench Depth	Sizing Factor Group A Soil
0.5 ft	3 ft	0.065
0.5 ft	4 ft	0.060
0.5 ft	5 ft	0.055
1 ft	3 ft	0.060
1 ft	4 ft	0.055
1 ft	5 ft	0.050

Figure 30 and Figure 31 illustrate the performances of an appropriately sized Infiltration Trench IMP with a depth of 3 feet and a surrounding berm height of 0.5 feet.

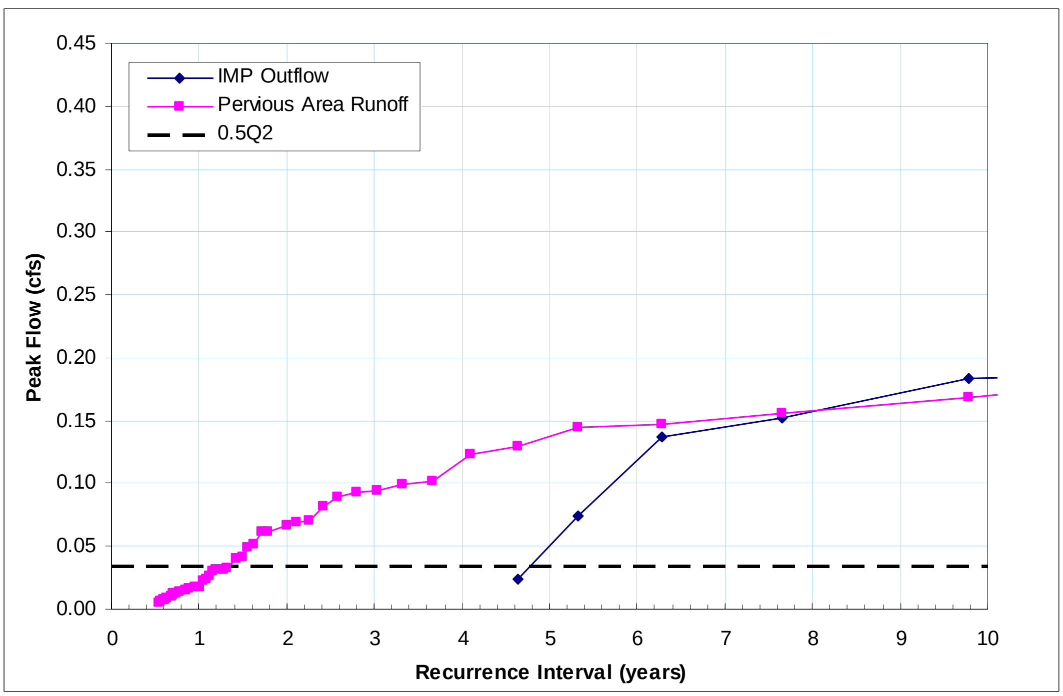

Figure 30. Peak Flow Statistics for Infiltration Trench IMP Controlling 1-ac Impervious in Group A Soils Infiltration Trench Depth = 3 ft; Surround Berm Height = 0.5 ft; Sizing Factor = 0.06

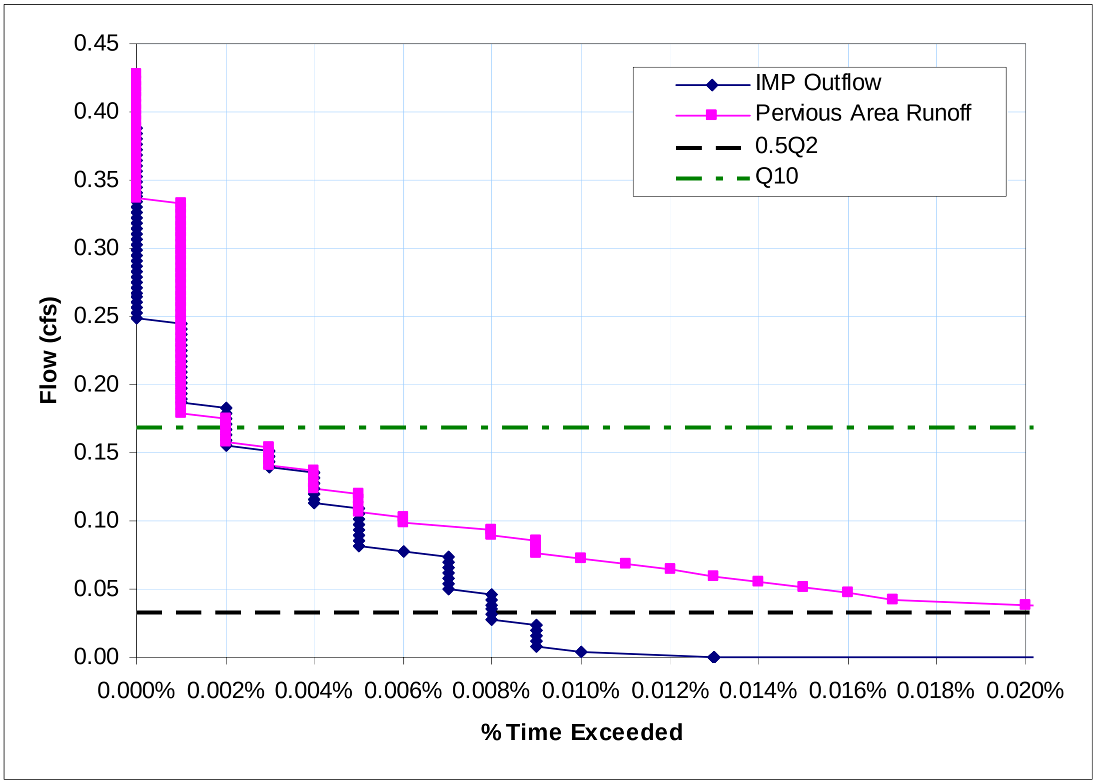

Figure 31. Flow Duration Statistics for Infiltration Trench IMP Controlling 1-ac Impervious in Group A Soils Infiltration Trench Depth = 3 ft; Surround Berm Height = 0.5 ft; Sizing Factor = 0.06

5.3.7 Infiltration Basin

The infiltration basin functions as a small detention pond that stores runoff and allows it to infiltrate into the underlying soil. The basin can be any shape (in plan view) and the side slopes can vary. For sizing purposes, the Program recommended use of a constant area for all depths of storage (vertical walls). A vertical riser pipe is used to control outflow. The diameter of the pipe is assumed to be 6 inches and the flow through the pipe is calculated using the same method as for the stormwater planter (Figure 31).

The basin is sized for area and sizing factors were developed for the following basin depths: 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 ft (Table 12). Only one FTABLE is required to model the infiltration basin.

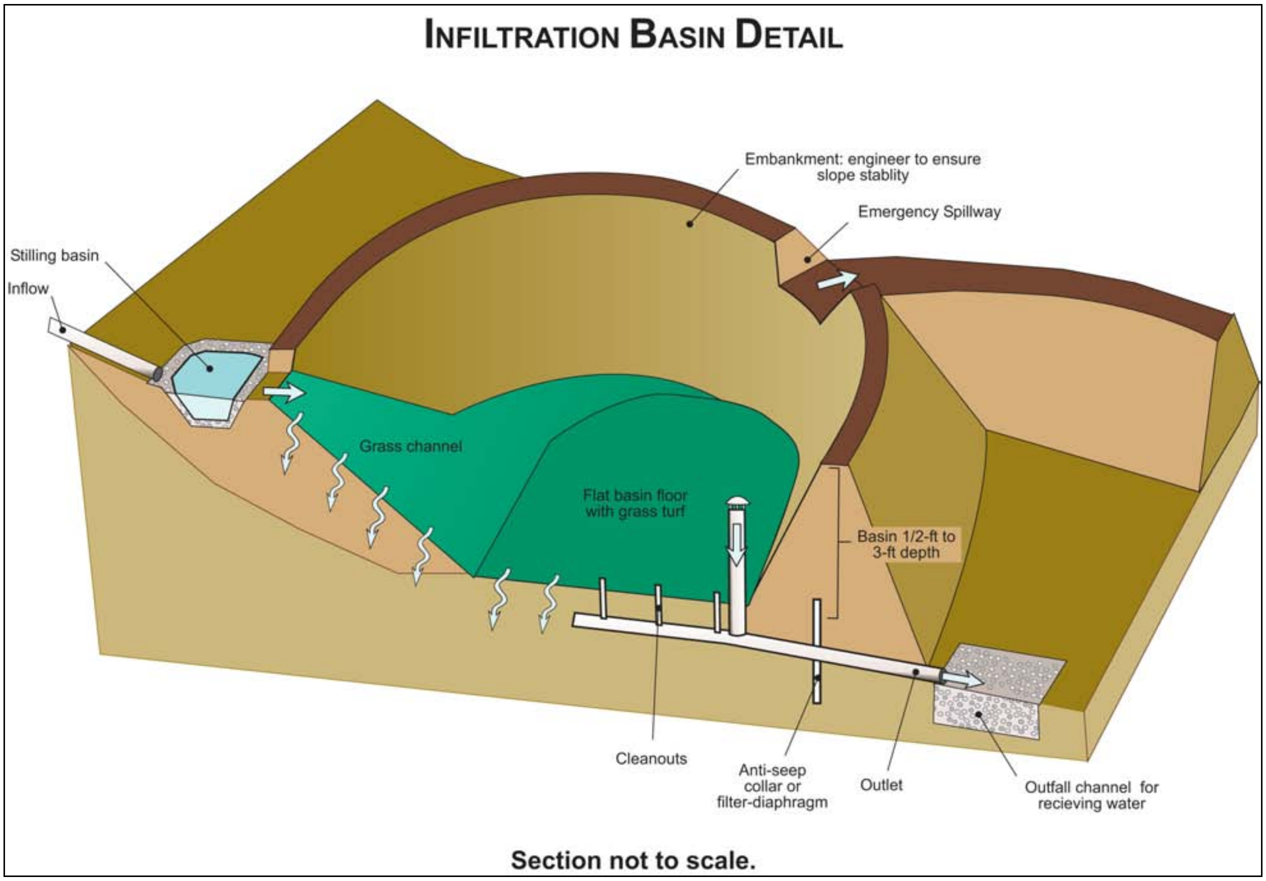

Figure 32. Cross-Section View of Infiltration Basin, Group A/B Soil Configuration

Table 12. Infiltration Basin Performance: Sizing Factors

Infiltration Basin Depth	Sizing Factor Group A Soil
0.5 ft	0.10
1.0 ft	0.07
1.5 ft	0.06
2.0 ft	0.06
2.5 ft	0.05
3.0 ft	0.05

Figure 33 and Figure 34 illustrate the performance of an Infiltration Basin IMP that provides 1 foot of above ground storage.

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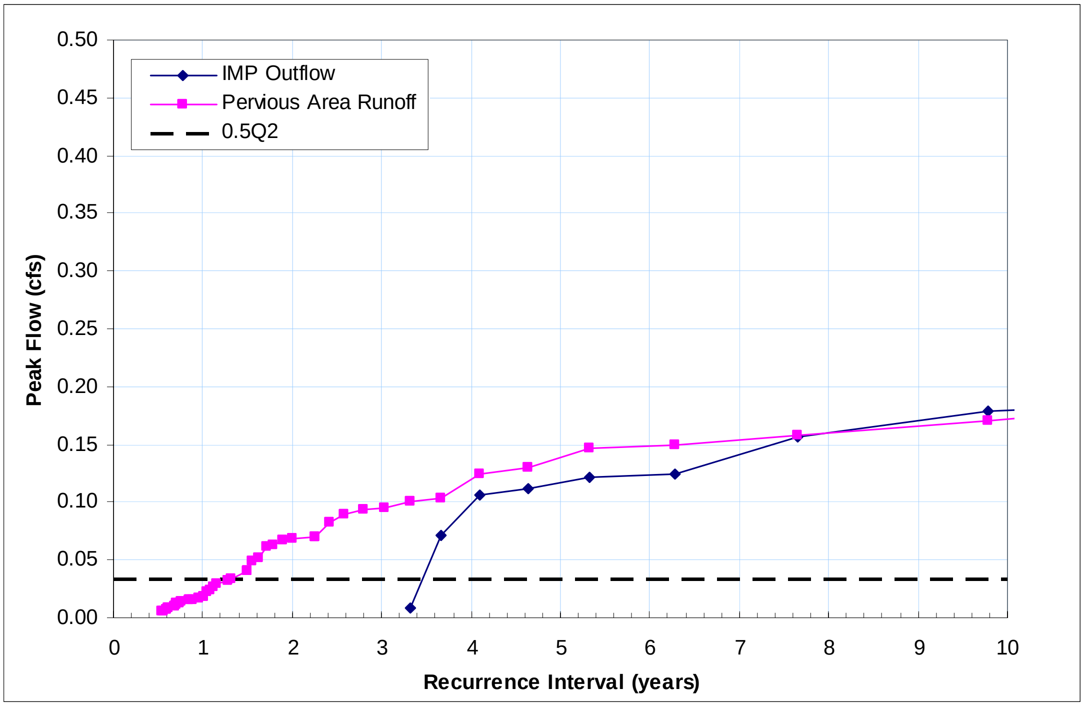

Figure 33. Peak Flow Statistics for Infiltration Basin IMP Controlling 1-ac Impervious in Group A Soils Max. Ponding Depth = 1 ft; Sizing Factor = 0.07

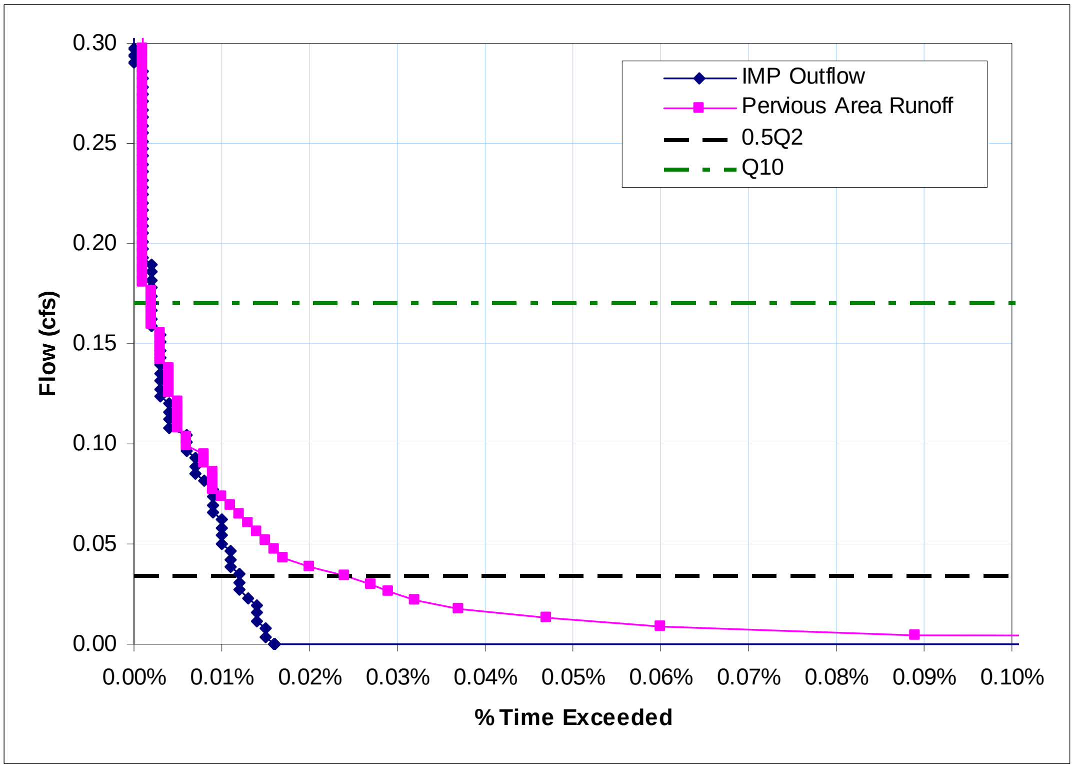

Figure 34. Peak Flow Statistics for Infiltration Basin IMP Controlling 1-ac Impervious in Group A Soils Max. Ponding Depth = 1 ft; Sizing Factor = 0.07

Attachment 2

6. IMP Sizing Factors Adjustments

The consultant team has developed adjustments to the calculated sizing factors to address two simplifying assumptions made during the HSPF modeling process:

1) The model used rainfall data from the Martinez rain gage only, though rainfall rates and patterns vary significantly across the county.
2) The model assumed that the area draining to the IMP is 100% impervious, though it will not always be possible to design site drainage so that IMPs only receive runoff from impervious areas (i.e., bypassing all pervious area runoff).

Therefore the spreadsheet tool used to implement the sizing factors will adjust the factors for each site based on the nearest rainfall gauge, and for each IMP based on the breakdown of pervious vs. impervious area tributary to the IMP.

6.1 ADJUSTMENT FOR RAINFALL SPATIAL VARIABILITY

Regional stormwater models usually include a method to account for spatial differences in rainfall. For example, WWHM uses a single rainfall gauge for each County, and then adjusts the local rainfall based on mean annual precipitation (MAP) determined from built-in isopluvial curves. The King County (WA) Regional Time Series model, an HSPF-based stormwater model, allows the user to select one of two available precipitation gauges. The Contra Costa County Flood Control District provides isopluvial maps to adjust the local runoff for new projects.

The consultant team considered a variety of options for adjusting IMP sizes in Contra Costa County for rainfall variability. Initially, it was assumed that IMP sizing would vary according to a linear relationship between the MAP at a project site and the MAP at the reference Martinez gauge. However, sizing factors computed for the In-Ground Planter using the FCD11 precipitation gauge suggested that the relationship is more complex. The FCD11 simulations generated larger sizing factors for Group D soils but smaller sizing factors for Group A soils, relative to the sizing factors computed with the Martinez gauge rainfall data. Since the mean annual precipitation at FCD11 is less than the Martinez gauge, sizing factors would have been uniformly smaller had the assumption of linear variability been true.

This limited assessment shows that Group A and Group D soil IMP sizing factors do not scale similarly. Different IMP sizing factor adjustments should be applied to Group A and Group D IMPs. IMPs on Group A soils contain no underdrain, suggesting their sizing factor adjustments should be related to the volume of local rainfall. MAP is a simple and easily obtainable measure to incorporate into the IMP Sizing Worksheet that appears to represent the rainfall-based adjustment to Group A soil IMP sizing factors.

The key difference for Group D soil IMPs is that they have underdrains that can release flow at up to 0.5Q2. Differing rainfall rates throughout the County affect both the

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amount of inflow to the IMP and the rate at which the IMP underdrain may discharge. Accordingly, the sizing factor adjustment should be related to both the rate of inflow and the rate of outflow from the IMP. For example, In-Ground Planter sizing factors computed from Martinez and FCD11 gauge simulations suggest the IMP sizing factor in Group D soils scales by the relative ratios of Q10/Q2 at the two sites. The ratio of Q10/Q2 for the FCD11 simulations are 50 percent higher than the Martinez simulations, and the recommended sizing factor for the FCD11 simulations is also 50 percent higher (sizing factor = 0.06 for FCD11; sizing factor = 0.04 for Martinez).

Based on the discussion above, the IMP Sizing Worksheet will include scaling factors for project site location, based on the MAP for Group A soils and the ratio of Q10 / Q2 for Group D soils (Table 13). The IMP Sizing Worksheet will require users to enter the MAP for the project site. This entry will scale sizing factors Group A soils. For Group D soils, the IMP Sizing Worksheet will select the Q10 / Q2 ratio for the precipitation gauge with the most similar MAP and adjust the sizing factors accordingly.

Table 13. IMP Sizing Factor Rainfall Variation Adjustment

Soil Group	IMP Sizing Adjustment
Group A	$MAP_{project\_site} / MAP_{Martinez}$
Group D	$\dfrac{Q10_{project\_site}}{Q2_{project\_site}} \bigg/ \dfrac{Q10_{Martinez}}{Q2_{Martinez}}$

6.2 ADJUSTMENT FOR PERVIOUS AREA TRIBUTARY TO IMP

The IMP sizing factors assume that the area draining to the IMP is 100% impervious. However, in some cases, site grading and landscaping will cause runoff from pervious land surfaces to drain to an IMP, such as sloped landscaping near a roadside Vegetated/Grassy Swale. Whether the pervious landscaping represents an increase in runoff (over pre-project conditions) or not, this flow should be factored into the IMP size in order to ensure the design performance standard is met.

The IMP Sizing Worksheet will adjust IMP sizing factors by converting tributary pervious areas to an equivalent impervious area. Table 14 lists the equivalent impervious area for Group A and Group D soils applied in the spreadsheet.

These adjustment factors were created by comparing the peak runoff and volumes for Group A and Group D soils with peak runoff and volume from impervious areas. For the peak runoff comparison, peaks were compared across the range from 0.5Q2 to Q10. This comparison indicated that peaks from Group D soils varied from approximately 70 to 90 percent of those for impervious areas. Total volume was compared by summing runoff volume over the entire time series. For example, for the Group D soil total runoff volume was approximately 50 percent of that from the impervious area. The higher value of 90 percent from the peak comparison was averaged with the 50 percent value from the volume comparison to arrive at the selected equivalency factor of 0.7. This same approach was used in developing the equivalency factor for Group A soils.

These values will form a background lookup table in the IMP Sizing Worksheet. When a user selects a pervious area and soil group draining to a particular type of IMP, the IMP Sizing Worksheet will multiply the drainage area, sizing factor and equivalent impervious conversion factor to size the IMP.

Table 14. Sizing Factor Adjustment for Pervious Areas Draining to IMPs

Pervious Area to Equivalent Impervious Area Conversion
1 acre of Group A soils → 0.1 acre Impervious
1 acre of Group D soils → 0.7 acre Impervious

7. Simplifying Assumptions and Potential Modeling Refinements

During the course of this project, a number of simplifying assumptions were made, similar to those made in programs in other parts of the Country. The list below discusses these assumptions, and potential associated modeling refinements that the Program may address in the future. These refinements are not likely to result in large changes to the overall range of sizing factors, but may result in more specific results for individual sites. The order of the refinements listed below reflects their relative priority.

7.1 MODEL TESTING

The model results presented in this memorandum are based on regional parameters and are not calibrated to local stream flow data. As with all hydrologic models, calibration would produce a more accurate result set. The regional HSPF model parameters for NRCS Group A and Group D soil, and scrub cover could be tested by comparing model output with stream gauging records in watersheds with limited development. Once calibration is achieved, the adjusted HSPF parameters could be used to refine the IMP sizing factors.

An additional approach to model testing would be to assess cumulative effects of multiple IMPs at a watershed scale. One possible method to test the aggregated effects is to create an HSPF model that links several spatially separated IMPs to compare with runoff from undeveloped watersheds.

7.2 RAINFALL GAUGES

As described in Section 6.1 above, the current sizing factors were all computed using the rainfall time series from the Martinez Gage. To address the variability of rainfall within the County, these sizing factors are corrected in the spreadsheet. The correction is based on comparison of runoff results from the gage with rainfall characteristics most similar to the project site to results from the Martinez Gage. Therefore, a future model refinement would be to explicitly develop IMP sizing factors for each of the six available rainfall gauges.

7.3 NRCS SOIL GROUPS

The model is currently based on modeling runs using NRCS hydrologic soils groups A and D. This was due to the conclusion that much of remaining developable areas in the County consist of these soils. As a consequence, B and C are lumped with A and D respectively in the spreadsheet, similar to the approach used in the Western Washington Hydrologic Model. A future refinement could involve developing HSPF model parameters for B and C soils groups, and then generating sizing factors specific to these soil groups.

7.4 TRIBUTARY PERVIOUS AREAS

Section 6.2 above describes how the spreadsheet currently represents pervious areas tributary to an IMP as "equivalent" impervious area. A potential model refinement would be to extend the HSPF modeling to directly consider a combination of impervious and pervious surfaces draining to a single IMP.

7.5 HSPF COVER TYPES

As discussed earlier, analysis of pre-project HSPF scenarios including Group A and Group D soils in combination with four cover types (scrub, range, irrigated pasture and live oak) indicated that the HSPF model was very insensitive to variation in cover type. This is because the cover types most commonly encountered in the region include little rainfall interception storage. Group A soil simulations and flow duration statistics also showed very little sensitivity to the selected cover type.

Because of the lack of sensitivity to cover type, the IMP simulations for sizing factor development all assumed *scrub* cover. A potential future model refinement could therefore involve development of sizing factors for each cover type, as well as modeling of additional cover types with more potential for rainfall interception, such as forest.

8. References

1. City of Portland. 2004. Stormwater Management Manual, Revision #3. Bureau of Environmental Services. September.
2. Maidment, D.R. 1993. Handbook of Hydrology. McGraw-Hill, Inc. San Francisco. pp 5.6, 5.14, 5.34.
3. United States Environmental Protection Agency (USEPA). 2000. Estimating Hydrologic and Hydraulic Paramters for HSPF. Office of Water, BASINS Technical Note 6. Washington D.C.
4. Washington State Department of Ecology. 2001. Stormwater Management Manual for Western Washington. August.

Attachment 2

Appendix A: HSPF Parameters for Pervious Land Surfaces (PERLNDs): Parameter Values and Descriptions

This section provides a list of values and descriptions for the pervious land surface (PERLND) parameters used in the HSPF model for Contra Costa. As described in Section 3, the model parameters were derived from numerous sources: the USGS regional calibration on Calabazas Creek in Santa Clara County, the WWHM, and the EPA publication, *EPA Basins Technical Note 6 Estimating Hydrologic and Hydraulic Parameters for HSPF* (July 2000), from which the parameter descriptions below are reproduced.

Table A1 below links the scenario descriptions with the PERLND Code (PLS) numbers used in HSPF input file. Table A2, which was copied directly from an HSPF input file, lists the actual PERLND parameters used to describe the various modeling scenarios in HSPF. The leftmost column in Table A2 lists the PERLND PLS numbers and the remaining columns list the values for each attribute of the pervious land surfaces.

Table A1. HSPF Modeling Scenarios and PERLND Code Numbers

PERLND Code (PLS) → Cover/Soil Combination
101→ Scrub; Type A Soil
102 → Scrub; Type D Soil
111 → Range; Type A Soil
112 → Range; Type D Soil
121 → Live Oak; Type A Soil
122 → Live Oak; Type D Soil
131 → Irrigated Pasture; Type A Soil
132 → Irrigated Pasture; Type D Soil

Table A2. Complete Set of HSPF Parameters for Pervious Land Segments

```
PWAT-PARM1
  <PLS > ***************** Flags *********************************************
  # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE IFFC                  ***
101  199    0    1    1    1    0    0    0    0    1    1
END PWAT-PARM1
```

```
PWAT-PARM2
  <PLS > ***************** Flags *********************************************
  # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
101           0.00    7.0000    0.7000    660.00    0.1000    0.0000    0.9500
102           0.00    7.0000    0.0300    660.00    0.1000    0.0000    0.9500
111           0.00    7.0000    0.7000    660.00    0.1000    0.0000    0.9500
112           0.00    7.0000    0.0300    660.00    0.1000    0.0000    0.9500
121           0.00    7.0000    0.7000    660.00    0.1000    0.0000    0.9500
122           0.00    7.0000    0.0300    660.00    0.1000    0.0000    0.9500
131           0.00    7.0000    0.7000    660.00    0.1000    0.0000    0.9500
132           0.00    7.0000    0.0300    660.00    0.1000    0.0000    0.9500
END PWAT-PARM2
```

Table A2. Complete Set of HSPF Parameters for Pervious Land Segments (Continued)

```
PWAT-PARM3
    <PLS > ***************** Flags *********************************************
    # -  #*** PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
  101           40.0      35.0    2.0000    2.0000      0.45       0.0       0.0
  102           40.0      35.0    2.0000    2.0000      0.10       0.0       0.0
  111           40.0      35.0    2.0000    2.0000      0.45       0.0       0.0
  112           40.0      35.0    2.0000    2.0000      0.10       0.0       0.0
  121           40.0      35.0    2.0000    2.0000      0.45       0.0       0.0
  122           40.0      35.0    2.0000    2.0000      0.10       0.0       0.0
  131           40.0      35.0    2.0000    2.0000      0.45       0.0       0.0
  132           40.0      35.0    2.0000    2.0000      0.10       0.0       0.0
  END PWAT-PARM3
```

```
PWAT-PARM4
    <PLS > ***************** Flags *********************************************
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP       ***
  101         0.1000    0.5000    0.3000     0.400    0.3000    0.0000
  102         0.1000    0.5000    0.3000     0.400    0.3000    0.0000
  111         0.0200    0.5000    0.3000     0.400    0.3000    0.0000
  112         0.0200    0.5000    0.3000     0.400    0.3000    0.0000
  121         0.1000    0.5000    0.3000     0.400    0.3000    0.0000
  122         0.1000    0.5000    0.3000     0.400    0.3000    0.0000
  131         0.0300    0.5000    0.3000     0.400    0.3000    0.0000
  132         0.0300    0.5000    0.3000     0.400    0.3000    0.0000
  END PWAT-PARM4
```

```
  MON-INTERCEP
    <PLS >  Interception storage capacity at start of each month          ***
    # -  #  JAN  FEB  MAR  APR  MAY  JUN  JUL  AUG  SEP  OCT  NOV  DEC       ***
  101      0.10 0.10 0.10 0.10 0.06 0.06 0.06 0.06 0.06 0.10 0.10 0.10
  102      0.15 0.15 0.15 0.15 0.08 0.08 0.08 0.08 0.08 0.15 0.15 0.15
  111      0.10 0.10 0.10 0.10 0.06 0.06 0.06 0.06 0.06 0.10 0.10 0.10
  112      0.15 0.15 0.15 0.15 0.08 0.08 0.08 0.08 0.08 0.15 0.15 0.15
  121      0.10 0.10 0.10 0.10 0.06 0.06 0.06 0.06 0.06 0.10 0.10 0.10
  122      0.10 0.10 0.10 0.10 0.06 0.06 0.06 0.06 0.06 0.10 0.10 0.10
  131      0.15 0.15 0.15 0.15 0.08 0.08 0.08 0.08 0.08 0.15 0.15 0.15
  132      0.10 0.10 0.10 0.10 0.06 0.06 0.06 0.06 0.06 0.10 0.10 0.10
  END MON-INTERCEP
  MON-LZETPARM
    <PLS >  Lower zone evapotranspiration parm at start of each month      ***
    # -  #  JAN  FEB  MAR  APR  MAY  JUN  JUL  AUG  SEP  OCT  NOV  DEC       ***
  101       0.4  0.4  0.4  0.4  0.6  0.6  0.6  0.6  0.6  0.4  0.4  0.4
  102       0.5  0.5  0.5  0.5  0.7  0.7  0.7  0.7  0.7  0.5  0.5  0.5
  111       0.4  0.4  0.4  0.4  0.6  0.6  0.6  0.6  0.6  0.4  0.4  0.4
  112       0.5  0.5  0.5  0.5  0.7  0.7  0.7  0.7  0.7  0.5  0.5  0.5
  121       0.4  0.4  0.4  0.4  0.6  0.6  0.6  0.6  0.6  0.4  0.4  0.4
  122       0.4  0.4  0.4  0.4  0.6  0.6  0.6  0.6  0.6  0.4  0.4  0.4
  131       0.5  0.5  0.5  0.5  0.7  0.7  0.7  0.7  0.7  0.5  0.5  0.5
  132       0.4  0.4  0.4  0.4  0.6  0.6  0.6  0.6  0.6  0.4  0.4  0.4
  END MON-LZETPARM
```

```
  PWAT-STATE1
    <PLS >  PWATER state variables***
    # -  #***   CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
  101            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  102            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  111            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  112            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  121            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  122            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  131            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  132            0.0    0.0000    0.1500     0.000    4.0000      0.05     0.000
  END PWAT-STATE1
```

PWAT-PARM1 Table: Sets PERLND Flags

The PWAT-PARM1 table includes flags to indicate the selected simulation algorithm option, other selection of monthly variability versus constant values for selected parameters. Where flags indicate monthly variability, the corresponding monthly values must be provided in Monthly Input Parameters (see below following the PWAT_PARM4 Table section). That section also provides guidance on which parameters are normally specified as monthly values.

CSNOFG Flag to use snow simulation data; must be checked (CSNOFG=1) if SNOW is simulated.

RTOPFG Flag to select overland flow routing method; choose either the method used in predecessor models (HSPX, ARM, and NPS) or the alternative method as described in the HSPF User Manual. Recommendation: Set RTOPFG=1; This method, used in the predecessor models is more commonly used, and has been subjected to more widespread application.

UZFG Flag to select upper zone inflow computation method; choose either the method used in predecessor models (HSPX, ARM, and NPS) or the more exact numerical solution to the integral of inflow to upper zone, i.e the alternative method. Recommendation: Set UZFG=1; This method, used in the predecessor models, is more commonly used, and has been subjected to more widespread application.

VCSFG Flag to select constant or monthly-variable interception storage capacity, CEPSC. Monthly value can be varied to represent seasonal changes in foliage cover; monthly values are commonly used for agricultural, and sometimes deciduous forest land areas.

VUZFG Flag to select constant or monthly-variable upper zone nominal soil moisture storage, UZSN. Monthly values are commonly used for agricultural areas to reflect the timing of cropping and tillage practices.

VMNFG Flag to select constant or monthly-variable Manning=s n for overland flow plane, NSUR. Monthly values are commonly used for agricultural, and sometimes deciduous forest land areas.

VIFWFG Flag to select constant or monthly-variable interflow inflow parameter, INTFW. Monthly values are not often used.

VIRCFG Flag to select constant or monthly varied interflow recession parameter, IRC. Monthly values are not often used.

VLEFG Flag to select constant or monthly varied lower zone ET parameter, LZETP. Monthly values are commonly used for agricultural, and sometimes deciduous forest land areas.

PWAT-PARM2 Table:

FOREST Fraction of land covered by forest (unitless) (measure/estimate). FOREST is the fraction of the land segment which is covered by forest which will continue to transpire in winter (i.e. coniferous). This is only relevant if snow is being considered (i.e., CSNOFG=1 in PWATER-PARM1).

LZSN Lower zone nominal soil moisture storage (inches), (estimate, then calibrate). LZSN is related to both precipitation patterns and soil characteristics in the region. The ARM Model User Manual (Donigian and Davis, 1978, p. 56, LZSN variable) includes a mapping of calibrated LZSN values across the country based on almost 60 applications of earlier models derived from the Stanford-based hydrology algorithms. LaRoche et al (1996) shows values of 5 inches to 14 inches, which is consistent with the 'possible' range of 2 inches to 15 inches shown in the Summary Table. Viessman, et al, 1989, provide initial estimates for LZSN in the Stanford Watershed Model (SWM-IV, predecessor model to HSPF) as one-quarter of the mean annual rainfall plus four inches for arid and semiarid regions, or one-eighth annual mean rainfall plus 4 inches for coastal, humid, or subhumid climates. These formulae tend to give values somewhat higher than are typically seen as final calibrated values; since LZSN will be adjusted through calibration, initial estimates obtained through these formulae may be reasonable starting values.

INFILT Index to mean soil infiltration rate (in/hr); (estimate, then calibrate). In HSPF, INFILT is the parameter that effectively controls the overall division of the available moisture from precipitation (after interception) into surface and subsurface flow and storage components. Thus, high values of INFILT will produce more water in the lower zone and groundwater, and result in higher baseflow to the stream; low values of INFILT will produce more upper zone and interflow storage water, and thus result in greater direct overland flow and interflow. LaRoche et al (1996) shows a range of INFILT values used from 0.004 in/hr to 0.23 in/hr, consistent with the 'typical' range of 0.01 to 0.25 in/hr in the Summary Table. Fontaine and Jacomino (1997) show sediment and sediment associated transport to be sensitive to the INFILT parameter since it controls the amount of direct overland flow transporting the sediment. Since INFILT is not a maximum rate nor an infiltration capacity term, it's values are normally much less than published infiltration rates, percolation rates (from soil percolation tests), or permeability rates from the literature. In any case, initial values are adjusted in the calibration process. INFILT is primarily a function of soil characteristics, and value ranges have been related to SCS hydrologic soil groups (Donigian and Davis, 1978, p.61, variable INFIL) as follows: NRCS Hydrologic INFILT Estimate Soil Group (in/hr) (mm/hr) Runoff Potential

A 0.4 - 1.0 10.0 - 25.0 Low

B 0.1 - 0.4 2.5 - 10.0 Moderate

C 0.05 - 0.1 1.25 - 2.5 Moderate to High

D 0.01 - 0.05 0.25 - 1.25 High

An alternate estimation method that has not been validated, is derived from the premise that the combination of infiltration and interflow in HSPF represents the infiltration commonly modeled in the literature (e.g. Viessman et al, 1989, Chapter 4). With this assumption, the value of 2.0*INFILT*INTFW should approximate the average measured soil infiltration rate at saturation, or mean permeability.

LSUR Length of assumed overland flow plane (ft) (estimate/measure). LSUR approximates the average length of travel for water to reach the stream reach, or any drainage path such as small streams, swales, ditches, etc. that quickly deliver the water to the stream or waterbody. LSUR is often assumed to vary with slope such that flat slopes have larger LSUR values and vice versa; typical values range from 200 feet to 500 feet for slopes ranging from 15% to 1 %. It is also often estimated from topographic data by dividing the watershed area by twice the length of all streams, gullies, ditches, etc that move the water to the stream. That is, a representative straight-line reach with length, L, bisecting a representative square areal segment of the watershed, will produce two overland flow planes of width ½ L. However, LSUR values derived from topographic data are often too large (i.e. overestimated) when the data is of insufficient resolution to display the many small streams and drainage ways. Users should make sure that values calculated from GIS or topographic data are consistent with the ranges shown in the Summary Table.

SLSUR Average slope of assumed overland flow path (unitless) (estimate/measure). Average SLSUR values for each land use being simulated can often be estimated directly with GIS capabilities. Graphical techniques include imposing a grid pattern on the watershed and calculating slope values for each grid point for each land use.

KVARY Groundwater recession flow parameter used to describe non-linear groundwater recession rate (/inches) (initialize with reported values, then calibrate as needed) KVARY is usually one of the last PWATER parameters to be adjusted; it is used when the observed groundwater recession demonstrates a seasonal variability with a faster recession (i.e. higher slope and lower AGWRC values) during wet periods, and the opposite during dry periods. LaRoche, et al, 1996 reported an extremely high 'optimized' value of 0.66 mm^{-1} or (17 in^{-1}) (much higher than any other applications) while Chen, et al, 1995 reported a calibrated value of 0.14 mm^{-1} (or 3.6 in^{-1}). Value ranges are shown in the Summary Table. Users should start with a value of 0.0 for KVARY, and then adjust (i.e. increase) if seasonal variations are evident. Plotting daily flows with a logarithmic scale helps to elucidate the slope of the flow recession.

AGWRC Groundwater recession rate, or ratio of current groundwater discharge to that from 24 hours earlier (when KVARY is zero) (/day) (estimate, then calibrate). The overall watershed recession rate is a complex function of watershed conditions, including climate, topography, soils, and land use. Hydrograph separation techniques (see any hydrology or water resources textbook) can be used to estimate the recession rate from observed daily flow data (such as plotting on a logarithmic scale, as noted above); estimated values will likely need to be adjusted through calibration. Value ranges are shown in the Summary Table. LaRoche, et al, 1996 reported an optimized value of 0.99; Chen, et al, 1995 reported values that varied with land use type, ranging from 0.971 for grassland and clearings to 0.996 for high density forest; Fontaine and Jacomino, 1997 reported a calibrated value of 0.99. This experience reflects normal practice of using higher values for forests than open, grassland, cropland and urban areas.

PWAT-PARM3 Table:

PETMAX Temperature below which ET will be reduced to 50% of that in the input time series (deg F), unless it=s been reduced to a lesser value from adjustments made in the SNOW routine (where ET is reduced based on the percent areal snow coverage and fraction of coniferous forest). PETMAX represents a temperature threshold where plant transpiration, which is part of ET, is reduced due to low temperatures (initialize with reported values, then calibrate as needed). It is only used if SNOW is being simulated because it requires air temperature as input (also a requirement of the SNOW module), and the required low temperatures will usually only occur in areas of frequent snowfall. Use the default of 40°F as an initial value, which can be adjusted a few degrees if required. PETMIN Temperature at and below which ET will be zero (deg F).

PETMIN represents the temperature threshold where plant transpiration is effectively suspended, i.e. set to zero, due to temperatures approaching freezing (initialize with reported values, then calibrate as needed). Like PETMAX, this parameter is used only if SNOW is being simulated because it requires air temperature as input (also a requirement of the SNOW module), and the required low temperatures will usually only occur in areas of frequent snowfall. Use the default of 35°F as an initial value, which can be adjusted a few degrees if required.

INFEXP Exponent that determines how much a deviation from nominal lower zone storage affects the infiltration rate (HSPF Manual, p. 60) (initialize with reported values, then calibrate as needed). Variations of the Stanford approach have used a POWER variable for this parameter; various values of POWER are included in Donigian and Davis (1978, p. 58). However, the vast majority of HSPF applications have used the default value of 2.0 for this exponent. Use the default value of 2.0, and adjust only if supported by local data and conditions.

INFILD Ratio of maximum and mean soil infiltration capacities (initialize with reported value). In the Stanford approach, this parameter has always been set to 2.0, so that the maximum infiltration rate is twice the mean (i.e. input) value; when HSPF was developed, the INFILD parameter was included to allow investigation of this assumption. However, there has been very little research to support using a value other than 2.0. Use the default value of 2.0, and adjust only if supported by local data and conditions.

DEEPFR The fraction of infiltrating water which is lost to deep aquifers (i.e. inactive groundwater), with the remaining fraction (i.e. 1-DEEPFR) assigned to active groundwater storage that contributes baseflow to the stream (estimate, then calibrate). It is also used to represent any other losses that may not be measured at the flow gage used for calibration, such as flow around or under the gage site. This accounts for one of only three major losses from the PWATER water balance (i.e. in addition to ET, and lateral and stream outflows). Watershed areas at high elevations, or in the upland portion of the watershed, are likely to lose more water to deep groundwater (i.e. groundwater that does not discharge within the area of the watershed), than areas at lower elevations or closer to the gage (see discussion and figures in Freeze and Cherry, 1979, section 6.1). DEEPFR should be set to 0.0 initially or estimated based on groundwater studies, and then calibrated, in conjunction with adjustments to ET parameters, to achieve a satisfactory annual water balance.

BASETP ET by riparian vegetation as active groundwater enters streambed; specified as a fraction of potential ET, which is fulfilled only as outflow exists (estimate, then calibrate). Typical and possible value ranges are shown in the Summary Table. If significant riparian vegetation is present in the watershed then non-zero values of BASETP should be used. Adjustments to BASETP will be visible in changes in the low-flow simulation, and will effect the annual water balance. If riparian vegetation is significant, start with a BASETP value of 0.03 and adjust to obtain a reasonable low-flow simulation in conjunction with a satisfactory annual water balance.

AGWETP Fraction of model segment (i.e. pervious land segment) that is subject to direct evaporation from groundwater storage, e.g. wetlands or marsh areas, where the groundwater surface is at or near the land surface, or in areas with phreatophytic vegetation drawing directly from groundwater. This is represented in the model as the fraction of remaining potential ET (i.e. after base ET, interception ET, and upper zone ET are satisfied), that can be met from active groundwater storage (estimate, then calibrate). If wetlands are represented as a separate PLS (pervious land segment), then AGWETP should be 0.0 for all other land uses, and a high value (0.3 to 0.7) should be used for the wetlands PLS. If wetlands are not separated out as a PLS, identify the fraction of the model segment that meets the conditions of wetlands/marshes or phreatophytic vegetation and use that fraction for an initial value of AGWETP. Like BASETP, adjustments to AGWETP will be visible in changes in the low-flow simulation, and will effect the annual water balance. Follow above guidance for an initial value of AGWETP, and then adjust to obtain a reasonable low-flow simulation in conjunction with a satisfactory annual water balance.

PWAT_PARM4 Table:

CEPSC Amount of rainfall, in inches, which is retained by vegetation, never reaches the land surface, and is eventually evaporated (estimate, then calibrate). Typical guidance for CEPSC for selected land surfaces is provided in Donigian and Davis (1978, p. 54, variable EPXM) as follows:

Land Cover Maximum Interception (in)

Grassland 0.10

Cropland 0.10 - 0.25

Forest Cover, light 0.15

Forest Cover, heavy 0.20

Donigian et al (1983) provide more detail guidance for agricultural conditions, including residue cover for agricultural BMPs. As part of an annual water balance, Viessman, et al. 1989 note that 10-20% of precipitation during growing season is intercepted and as much as 25% of total annual precipitation is intercepted under dense closed forest stands; crops and grasses exhibit a wide range of interception rates - between 7% and 60% of total rainfall. Users should compare the annual interception evaporation (CEPE) with the total rainfall available (PREC in the WDM file), and then adjust the CEPSC values accordingly. (See Monthly Input Values below).

UZSN Nominal upper zone soil moisture storage (inches) (estimate, then calibrate). UZSN is related to land surface characteristics, topography, and LZSN. For agricultural conditions, tillage and other practices, UZSN may change over the course of the growing season. Increasing UZSN value increases the amount of water retained in the upper zone and available for ET, and thereby decreases the dynamic behavior of the surface and reduces direct overland flow; decreasing UZSN has the opposite effect. Donigian and Davis (1978, p. 54) provide initial estimates for UZSN as 0.06 of LZSN, for steep slopes, limited vegetation, low depression storage; 0.08 LZSN for moderate slopes, moderate vegetation, and moderate depression storage; 0.14 LZSN for heavy vegetal or forest cover, soils subject to cracking, high depression storage, very mild slopes. Donigian et al., (1983) include detailed guidance for UZSN for agricultural conditions. LaRoche shows values ranging from 0.016 in to 0.75 in. Fontaine and Jacomino showed average daily stream flow was relatively insensitive to this value but sediment and sediment associated contaminant outflow was sensitive; this is consistent with experience with UZSN having an impact on direct overland flow, but little impact on the annual water balance (except for extremely small watersheds with no baseflow). Typical and possible value ranges are shown in the Summary Table.

NSUR Manning's n for overland flow plane (estimate). Manning's n values for overland flow are considerably higher than the more common published values for flow through a channel, where values range from a low of about 0.011 for smooth concrete, to as high as 0.050-0.1 for flow through unmaintained channels (Hwang and Hita, 1987). Donigian and Davis (1978, p. 61, variable NN) and Donigian et al (1983) have tabulated the following values for different land surface conditions:

Smooth packed surface 0.05

Normal roads and parking lots 0.10

Disturbed land surfaces 0.15 - 0.25

Moderate turf/pasture 0.20 - 0.30

Heavy turf, forest litter 0.30 - 0.45

Agricultural Conditions

Conventional Tillage 0.15 - 0.25

Smooth fallow 0.15 - 0.20

Rough fallow, cultivated 0.20 - 0.30

Crop residues 0.25 - 0.35

Meadow, heavy turf 0.30 - 0.40

For agricultural conditions, monthly values are often used to reflect the seasonal changes in land surfaces conditions depending on cropping and tillage practices. Additional tabulations of Manning's n values for different types of surface cover can be found in: Weltz, et al, 1992; Engman, 1986; and Mays, 1999. Manning's n values are not often calibrated since they have a relatively small impact on both peak flows and volumes as long as they are within the normal ranges shown above. Also, calibration requires data on just overland flow from very small watersheds, which is not normally available except at research plots and possibly urban sites.

<u>INTFW</u> Coefficient that determines the amount of water which enters the ground from surface detention storage and becomes interflow, as opposed to direct overland flow and upper zone storage (estimate, then calibrate). Interflow can have an important influence on storm hydrographs, particularly when vertical percolation is retarded by a shallow, less permeable soil layer. INTFW affects the timing of runoff by effecting the division of water between interflow and surface processes. Increasing INTFW increases the amount of interflow and decreases direct overland flow, thereby reducing peak flows while maintaining the same volume. Thus it affects the shape of the hydrograph, by shifting and delaying the flow to later in time. Likewise, decreasing INTFW has the opposite effect. Base flow is not affected by INTFW. Rather, once total storm volumes are calibrated, INTFW can be used to raise or lower the peaks to better match the observed hydrograph. Typical and possible value ranges are shown in the Summary Table.

<u>IRC</u> Interflow recession coefficient (estimate, then calibrate). IRC is analogous to the groundwater recession parameter, AGWRC, i.e. it is the ratio of the current daily interflow discharge to the interflow discharge on the previous day. Whereas INTFW affects the volume of interflow, IRC affects the rate at which interflow is discharged from storage. Thus it also affects the hydrograph shape in the 'falling' or recession region of the curve between the peak storm flow and baseflow. The maximum value range is 0.3 – 0.85, with lower values on steeper slopes; values near the high end of the

range will make interflow behave more like baseflow, while low values will make interflow behave more like overland flow. IRC should be adjusted based on whether simulated storm peaks recede faster/slower than measured, once AGWRC has been calibrated. Typical and possible value ranges are shown in the Summary Table.

<u>LZETP</u> Index to lower zone evapotranspiration (unitless) (estimate, then calibrate). LZETP is a coefficient to define the ET opportunity; it affects evapotranspiration from the lower zone which represents the primary soil moisture storage and root zone of the soil profile. LZETP behaves much like a 'crop coefficient' with values mostly in the range of 0.2 to 0.7; as such it is primarily a function of vegetation; Typical and possible value ranges are shown in the Summary Table, and the following ranges for different vegetation are expected for the 'maximum' value during the year:

Forest 0.6 - 0.8

Grassland 0.4 - 0.6

Row crops 0.5 - 0.7

Barren 0.1 - 0.4

Wetlands 0.6 - 0.9

<u>Monthly Input Parameter Tables:</u>

In general, monthly variation in selected parameters, such as CEPSC and LZETP should be included with the initial parameter estimates. However, adjustments to the monthly values should be addressed only after annual flow volumes are matched well with monitored data. All monthly values can be adjusted to calibrate for seasonal variations.

<u>MON-INTERCEP Table:</u>

Monthly values for interception storage. Monthly values can be developed based on the data presented in the discussion in PWAT-PARM4/CEPSC and the Summary Tables.

<u>MON-UZSN Table:</u>

Monthly values for upper zone storage. For agricultural areas under conventional tillage, lower values are used to reflect seedbed preparation in the spring with values increasing during the growing season until harvest and fall tillage. See PWAT-PARM4/UZSN discussion and Summary Tables for guidance.

<u>MON-MANNING Table:</u>

Monthly values for Manning=s n for the overland flow plane. Monthly values can be used to represent seasonal variability in ground cover including crop and litter residue. See discussion in PWAT-PARM4/NSUR for Manning's n as a function of agricultural conditions.

<u>MON-INTERFLW Table:</u>

Monthly values for interflow parameter (INTFW) are not often used.

MON-IRC Table:

Monthly values for interflow recession parameter are not often used.

MON-LZETPARM Table:

Monthly values for LZETP for evapotranspiration from the lower zone can be developed using an expected maximum value from the PWAT-PARM4/LZETP discussion and the range of values presented in the Summary Tables. Monthly variable values should be used to reflect the seasonality of evapotranspiration, in response to changes in density of vegetation, depth of root zone, and stage of plant growth.

PWAT-STATE1 Table:

CEPS, SURS, IFWS, UZS, LZS, AGWS, are initial values for storage of water in interception, surface ponding, interflow, the upper zone, lower zone, and active groundwater, respectively, and GWVS is the initial index to groundwater slope. All these storages pertain to the first interval of the simulation period. The surface related storages (i.e. CEPS, SURS, IFWS) are highly dynamic, and will reach a dynamic equilibrium within a few days, at most. These state variables can be left blank, or set to 0.0 unless an individual storm is being simulated. The soil storages (i.e. UZS, LZS, and AGWS, and the GWVS) are much less dynamic, so their beginning values can impact the simulation for a period of months to a few years.

If possible, users should allow as long a startup time period as possible (i.e. set the simulation period to begin prior to the period you=ll use for comparison against monitoring data or other use); as noted each of these storages should reach a dynamic equilibrium within a few years of simulation. UZS and LZS should be set equal to UZSN and LZSN respectively, unless it is known that the starting date is during a particularly wet or dry period; starting values can be increased or decreased if wet or dry conditions were evident prior to the simulation period. AGWS is a bit more problematic. If far too high or too low, baseflow will be excessive or skewed low for several months or years, depending on AGWRC and KVARY. Improper values of GWVS can also cause simulation accuracy problems again for lengths of time depending on values of AGWRC and KVARY. However, since when KVARY is set to 0.0 seasonal recession is not represented and GWVS is not calculated. To avoid problems, then, AGWS should be set to 1.0 inch and GWVS to 0.0 for initial simulation runs. If the simulation period is limited in duration, you can check and reset these state variables to values observed for the same period in subsequent years with similar climatic conditions. However, if major calibration changes are made to the parameters controlling these storages (e.g. UZSN, LZSN, INFILT), then the initial conditions should be checked and adjusted during the calibration process. The values for AGWS and GWVS should be checked and adjusted as noted above, which assuming a yearly cycle of groundwater storage, can be compared to values during similar seasons in the simulation period. If the initial simulated baseflow (before the first significant rainfall) is much different from the initial observed streamflow, then further adjustments can be made to raise or lower the flow rates.

Appendix B: Assumed Water Movement Hydraulics for Modeling IMPs

At minimum, each IMP consists of a reservoir for surface water storage, an overflow outlet and a soil medium. In general, runoff flows into the surface storage reservoir and either infiltrates into the soil or flows through the overflow outlet structure.

Water that does not overflow the surface-storage reservoir infiltrates into the top soil medium and is stored as soil water. Once in the soil, water percolates downward at a rate that is dependent on the soil moisture content, the hydraulic properties of the soil and the boundary conditions of the soil layer.

Many IMPs also include a gravel or aggregate layer below the upper soil layer. Similarly, the rate at which water percolates downward through the gravel/aggregate layer is dependent on the soil moisture content, the hydraulic properties of the soil and the boundary conditions. The lower boundary is often controlled using an underdrain with an orifice outlet.

The following sections describe the theoretical relationships used to develop the FTABLEs for HSPF modeling of the IMPs. The first four sections of this appendix describe the discharge equations used for each of three overflow outlet types and the underdrain orifice:

- Circular Overflow Outlet,
- Straight, Sharp-crested Weir,
- V-notch Weir,
- Underdrain Orifice.

The last three sections describe infiltration, soil water storage and soil water movement.

Circular Overflow Outlet

A circular overflow outlet is basically a vertical pipe with a horizontal opening set to a specific height. This type of outlet is used for the in-ground planter, the flow-through planter, the bioretention basin and the infiltration basin.

Outflow control conditions vary as head over the pipe opening increases. As the water level begins to rise above the opening the pipe acts as a circular weir and flow is crest-controled. As the head over the opening increases the flow condition transitions to become orifice-controled and eventually pipe-controled (the pipe flows full).

Under crest-controled conditions outflow is calculated using a modified weir equation:

$$Q = C_d \left(2\pi R\right) H^{3/2} \qquad \textbf{Equation 1}$$

Where Q = outflow in cfs, C_d = discharge coefficient, R = pipe radius in ft, and H = the head over the crest in ft.

The discharge coefficient for crest-controlled flow is highly variable depending on the head over the crest, the radius of the circular weir, and the ratio of the inlet height to radius. USBR (1987) published a series of curves that are used to determine the appropriate discharge coefficient for each water surface level.

Straight Sharp-crested Weir

A second type of overflow outlet is a straight sharp-crested weir. A sharp-crested weir is used to control overflow in a vegetated/grassy swale. The following weir equation is used to calculate overflow discharge:

$$Q = C_d L H^{3/2} \qquad \textbf{Equation 2}$$

Where Q = outflow in cfs, C_d = discharge coefficient, L = weir length in ft and H = head over the weir crest in ft. The weir coefficient is assumed to be 3.10 for straight sharp-crested weirs.

V-notch Weir

In some cases a v-notch is added to the overflow weir. A v-notch weir is incorporated into the overflow weir of the vegetated/grassy swale. The flow through the v-notch is calculated using the following equation.

$$Q = C_d \tan\left(\frac{\phi}{2}\right) H^{5/2} \qquad \textbf{Equation 3}$$

Where Q = outflow in cfs, C_d = discharge coefficient, ϕ = angle of the v-notch, and H = head over the weir crest in ft. The v-notch is assumed to be 90 degrees and the weir coefficient was assumed to be 2.55.

Underdrain Outlet

The perforated pipe of lateral underdrains is assumed to be sufficiently large as to not limit the flow into the drain. Drain outflow is limited by single orifice at the end of the drain pipe. Outflow through this orifice was calculated using the orifice equation:

$$Q = C_d A \sqrt{2gH} \qquad \textbf{Equation 4}$$

Where Q = outflow, C_d = discharge coefficient, A = area of the orifice, g = gravitational constant, H = head over the centerline of the orifice. The discharge coefficient is assumed to be 0.6 in all cases.

Infiltration

Infiltration is the process of water penetrating from the ground surface into the soil (Chow et al. 1988). Many factors influence the rate of infiltration including ground cover, soil hydraulic properties and soil moisture. As water infiltrates into the soil the soil moisture and hydraulic gradient change. As a result the infiltration rate itself changes over time. This non-linear relation is given by Richard’s equation, which is the governing equation for unsteady unsaturated flow in a porous medium. Eagleson (1970) presents Richard’s equation in its one-dimensional form:

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z}\left[D\frac{\partial \theta}{\partial z} + K\right]. \qquad \textbf{Equation 5}$$

Where D = diffusivity, K = hydraulic conductivity, q = soil moisture content, z = elevation and t = time.

Numerous equations have been developed as approximate solutions to Richard’s equation. Eagleson (1970) shows that Horton’s equation is derived from Richard’s equation by assuming D and K are constants independent of soil moisture:

$$f(t) = f_c + \left(f_0 - f_c\right)\cdot e^{-kt}. \qquad \textbf{Equation 6}$$

Where, f_0 = initial infiltration rate, k = decay constant and f_c = final constant infiltration rate. Using Horton’s approximate solution we can see how infiltration rate changes over time.

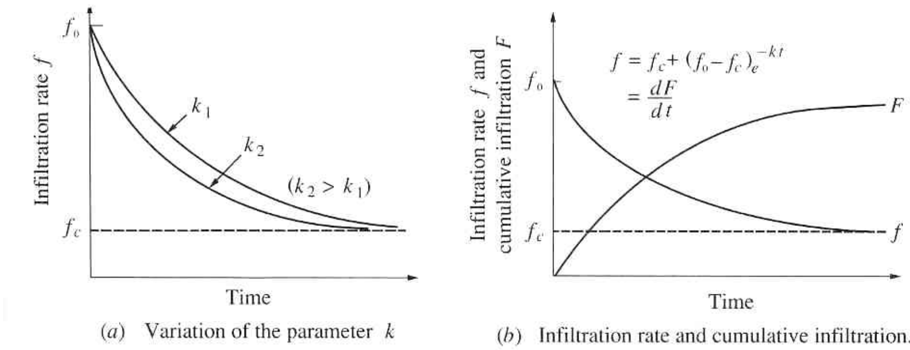


Figure B1– Horton’s Equation for Infiltration (graphs from Chow et al. 1988)

We can see from Figure B1 that infiltration begins at a very high rate due to the high matric potential in a dry soil and decreases exponentially as the soil becomes saturated, matric potential becomes insignificant and gravity governs the hydraulic gradient. Thus the infiltration rate approaches a steady-state final rate that approximately corresponds to the saturated hydraulic conductivity of the soil.

After water has been infiltrated into the soil the movement of water through the soil is termed percolation. The rate of percolation can be calculated using Darcy’s Law (see Soil Water Movement Section).

Horton’s equation showed that the potential infiltration rate of water into the soil always exceeds the saturated hydraulic conductivity of the soil. Conversely, the percolation rate of soil water is limited by the saturated hydraulic conductivity of the soil. Therefore, it is reasonable to assume that the potential infiltration rate is always greater than the percolation rate, and that the percolation rate will limit the flow rate through the soil layer.

Water Storage

The amount of water stored in soils (soil moisture) is expressed as a dimensionless ratio called the volumetric water content, θ. For any given water content the total volume of water stored in the soil, V_{water}, is equal to the volumetric water content (θ) times the total volume of soil, V_{total}.

$$\theta = \frac{V_{water}}{V_{total}} \quad \textbf{Equation 7}$$

The total void space within a soil is the porosity, η. Soil is saturated when the volumetric water content is equal to the porosity.

Some voids do not actively store and convey water. The void space within the soil that is hydrodynamically effective is called the effective porosity, θ_e. The difference between the total porosity and the effective porosity is known as the residual water content, θ_r. Maidment (1993) provides typical porosity, effective porosity and residual water content values by soil texture (see Table B1).

Table B1– Soil Porosity, Effective Porosity and Residual Water Content by Soil Texture (Maidment, 1993)

Soil Type	Porosity η	Effective Porosity θ_e	Residual Water Content θ_r
GRAVEL[1]	0.420	0.415	0.005
SAND	0.437	0.417	0.020
LOAMY SAND	0.437	0.401	0.035
SANDY LOAM	0.453	0.412	0.041
LOAM	0.463	0.434	0.027
SILT LOAM	0.501	0.486	0.015
SANDY CLAY LOAM	0.398	0.330	0.068
CLAY LOAM	0.464	0.390	0.075
SILTY CLAY LOAM	0.471	0.432	0.040
SANDY CLAY	0.430	0.321	0.109
SILTY CLAY	0.479	0.423	0.056
CLAY	0.475	0.385	0.090

1 – Values for gravel were obtained from Fayer (1992) as presented in INEEL (2002).

Porosity, effective porosity and residual water content values by hydrologic soil group were obtained for this project by assuming each group corresponds with a specific soil texture.

- Group A → Sand
- Group B → Loam
- Group C → Sandy Clay Loam

- Group D → Clay

These assumptions were based on the hydrologic soil group descriptions provided by NRCS (2001). Table B2 provides the assumed porosity, effective porosity and residual water content values by hydrologic soil group.

Table B2 – Soil Porosity, Effective Porosity and Residual Water Content by Hydrologic Soil Group

Soil Type	Porosity η	Effective Porosity θ_e	Residual Water Content θ_r
HYDROLOGIC SOIL GROUP: A	0.437	0.417	0.020
HYDROLOGIC SOIL GROUP: B	0.463	0.434	0.027
HYDROLOGIC SOIL GROUP: C	0.398	0.330	0.068
HYDROLOGIC SOIL GROUP: D	0.475	0.385	0.090

Soil Water Movement

Darcy's Law is used to calculate the rate of water movement through a porous medium:

$$q = -K \frac{\partial h}{\partial z} \qquad \textbf{Equation 8}$$

Where q = Darcy flux, K = hydraulic conductivity of the porous medium, h = total hydraulic head, and z = elevation. The total head, h, is the sum of the matric head, ψ, and the gravity head, z (velocity head is negligible):

$$h = \psi + z \ . \qquad \textbf{Equation 9}$$

Assuming flow only in the vertical direction and substituting for h, Equation 1 becomes:

$$q = -K \frac{d(\psi + z)}{dz} \ . \qquad \textbf{Equation 10}$$

The matric potential within a soil varies greatly with soil moisture. The relation between matric potential and soil moisture for a specific soil is known as the water-retention characteristic of that soil. Figure B2 shows some examples of typical water-retention curves for soils of various textures.

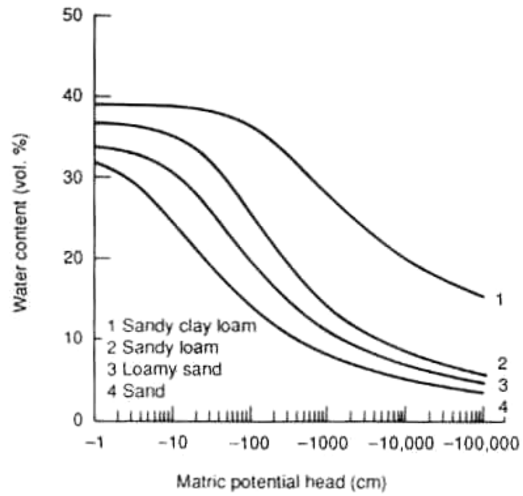


Figure B2 – Typical water retention curves (graph from Maidment, 1993)

Several equations have been developed to approximate water-retention relationships based on the physical characteristics of the soil. One such equation was developed by van Genuchten (1980):

$$\frac{\theta - \theta_r}{\eta - \theta_r} = \left[\frac{1}{1 + (\alpha\psi)^n}\right]^m$$ **Equation 11**

Where the constants α, *n* and *m* are given by:

$$\alpha = (h_b)^{-1}$$ **Equation 12**

$$n = \lambda + 1$$ **Equation 13**

$$m = \frac{\lambda}{\lambda + 1}\ .$$ **Equation 14**

The bubbling pressure head, h_b, and pore-size index, λ, are soil-specific parameters. Maidment (1993) provides typical bubbling pressures and pore-size index values by soil texture (see Table B3).

Table B3 – Bubbling Pressure and Pore-size Index by Soil Texture (Maidment, 1993)

Soil Type	Bubbling Pressure (cm) h_b	Pore-size Distribution λ
GRAVEL[1]	0.20	1.190
SAND	7.26	0.694
LOAMY SAND	8.69	0.553
SANDY LOAM	14.66	0.378
LOAM	11.15	0.252
SILT LOAM	20.76	0.234
SANDY CLAY LOAM	28.08	0.319
CLAY LOAM	25.89	0.242
SILTY CLAY LOAM	32.56	0.177
SANDY CLAY	29.17	0.223
SILTY CLAY	34.19	0.150
CLAY	37.30	0.165

1 – Values for gravel were obtained from Fayer (1992) as presented in INEEL (2002).

As discussed previously, soil properties were assigned to hydrologic soil groups based on soil textures. Table B4 provides the bubbling pressure and pore-size index values by hydrologic soil group.

Table B4 – Bubbling Pressure and Pore-size Index by Hydrologic Soil Group

Soil Type	Bubbling Pressure (cm) h_b	Pore-size Distribution λ
HYDROLOGIC SOIL GROUP: A	7.26	0.694
HYDROLOGIC SOIL GROUP: B	11.15	0.252
HYDROLOGIC SOIL GROUP: C	25.89	0.242
HYDROLOGIC SOIL GROUP: D	37.30	0.165

Hydraulic Conductivity, K, is also dependent on soil moisture. Van Genuchten (1980) also developed a relationship to approximate the hydraulic conductivity of soils based on soil properties:

$$\frac{K(\theta)}{K_s} = \left(\frac{\theta-\theta_r}{\eta-\theta_r}\right)^{1/2} \left\{1-\left[1-\left(\frac{\theta-\theta_r}{\eta-\theta_r}\right)^{1/m}\right]^m\right\}^2 . \qquad \textbf{Equation 15}$$

Saturated hydraulic conductivity, K_s, is a measure of a saturated soil's ability to transmit water along a hydraulic gradient. This value is highly variable in field conditions; however, Maidment (1993) does provide estimates of saturated hydraulic conductivity by soil texture (see Table B5).

Table B5 – Saturated Hydraulic Conductivity by Soil Texture (Maidment, 1993)

Soil Type	Saturated Hydraulic Conductivity (cm/hr) K_s
GRAVEL[1]	1260
SAND	23.56
LOAMY SAND	5.98
SANDY LOAM	2.18
LOAM	1.32
SILT LOAM	0.68
SANDY CLAY LOAM	0.3
CLAY LOAM	0.2
SILTY CLAY LOAM	0.2
SANDY CLAY	0.12
SILTY CLAY	0.1
CLAY	0.06

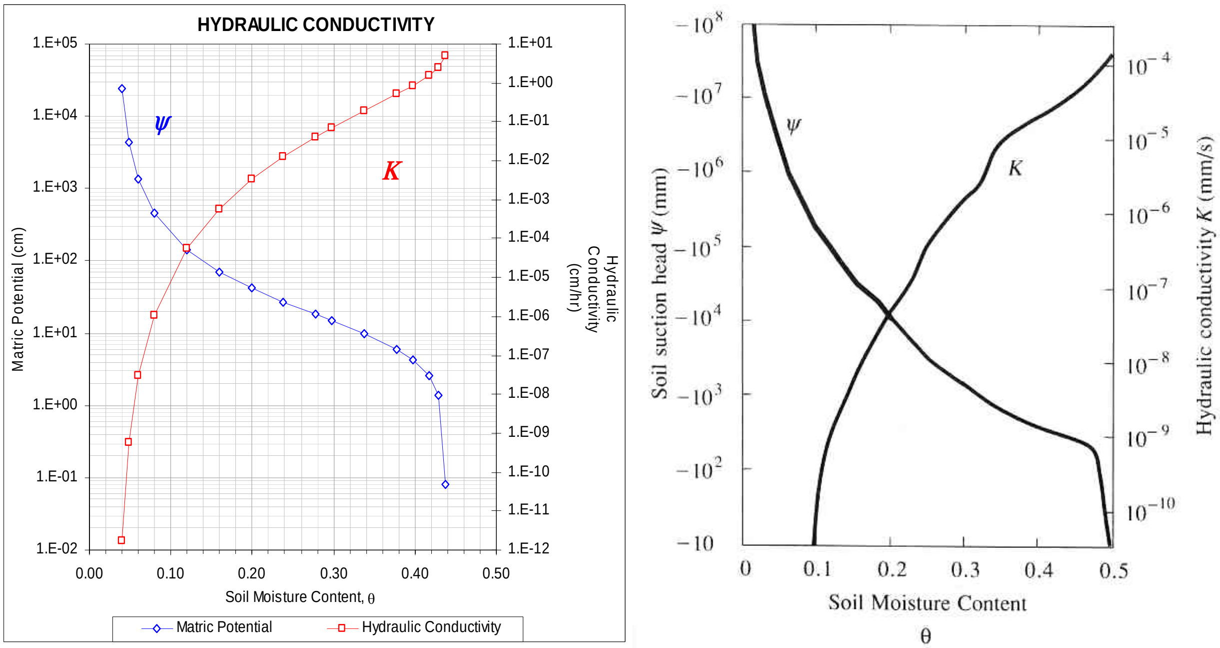
1 – Values for gravel were obtained from Fayer (1992) as presented in INEEL (2002).

As discussed previously, soil properties were assigned to hydrologic soil groups based on soil textures. Table B6 provides the saturated hydraulic conductivity by hydrologic soil group.

Table B6 – Saturated Hydraulic Conductivity by Hydrologic Soil Group

Soil Type	Saturated Hydraulic Conductivity (cm/hr) K_s
HYDROLOGIC SOIL GROUP: A	23.56
HYDROLOGIC SOIL GROUP: B	1.32
HYDROLOGIC SOIL GROUP: C	0.20
HYDROLOGIC SOIL GROUP: D	0.06

Figure B3(a) shows a plot of the van Genuchten relationships using the soil properties assumed for a loamy sand soil. Figure B3(b) is a graph from Chow et al. (1988) that shows the typical variation of matric head and hydraulic conductivity based on experimental data for an example soil.



(a) (b)

Figure B3 - (a) variation of matric head and hydraulic conductivity for a loamy sand using van Genuchten relations, (b) example provided in Chow et al. (1988)

The van Genuchten relations were used to calculate the matric head and hydraulic conductivity for a given soil moisture content. These results were then used in the Darcy equation to compute the flow through the soil. Calculated over a range of soil moisture contents, a table can be created relating soil water storage and flow through the soil layer.

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Appendix C: Summary of Sensitivity Analysis for the HSPF Modeling and IMP Sizing

BROWN AND CALDWELL

Memorandum

Date: March 7, 2005

To: Christie Beeman, PWA

From: Tony Dubin, BC-Seattle
Steve Anderson, BC-Seattle

Subject: Summary of Sensitivity Analysis for the HSPF Modeling of a Stormwater Planter

At the December 14, 2004, project team meeting, the Brown and Caldwell (BC) team presented preliminary sizing factors (i.e. the ratio of IMP size to impervious area controlled) for the Stormwater Planter. Afterward, the group discussed several assumptions that were built into the HSPF model of the planter box system, such as the porosity of the planter box soil and the height of the overflow pipe. Dan Cloak asked the BC team to investigate how the values of several of the assumed model parameters affect the modeling results. Specifically, Dan Cloak asked BC to perform sensitivity analyses to determine how varying the following parameters would affect IMP sizing:

1. The assumed infiltration rate for sandy soils (NRCS Type A),
2. The height of the overflow pipe from the planter box,
3. The assumed porosity of the upper soil layer in the planter box, and
4. The discharge limit through the planter box underdrain.

These issues were thoroughly investigated with dozens of long-term HSPF simulations and statistical analyses that varied model input parameters, evaluated changes in modeling results, and most importantly interpreted the reasons for these changes. The following sections provide a brief summary of the sensitivity analysis results.

Sensitivity Analysis 1: How does the assumed infiltration rate for Type A sandy soils (as determined by the INFILT parameter in HSPF) affect the rate of surface runoff?

During the December project team meeting, team members suggested that in their local experience, Type A soils in the East Bay area generate more surface runoff than the HSPF modeling showed. The quantity of runoff produced from Type A soils is a key factor for determining the Stormwater Planter size, because it sets the 'existing site condition' that the surface outflow from the Stormwater Planter must match.

The initial assumed value for the INFILT parameter was 0.7, which is in the middle of the range recommended by EPA for Type A soils[1]. We incrementally reduced the INFILT parameter value from 0.7 to 0.2 and ran HSPF simulations for each value. Figure 1 shows the resulting surface runoff rates up to the 10-year recurrence interval, with the runoff from impervious surfaces also shown to provide perspective. Table 1 lists the ratio of pervious to impervious runoff for Type A soils for various assumed INFILT parameter values. (Note: Q9 is the closest point to the 10-year flow in the partial duration series analysis of the 32 year rainfall series used in the HSPF model).

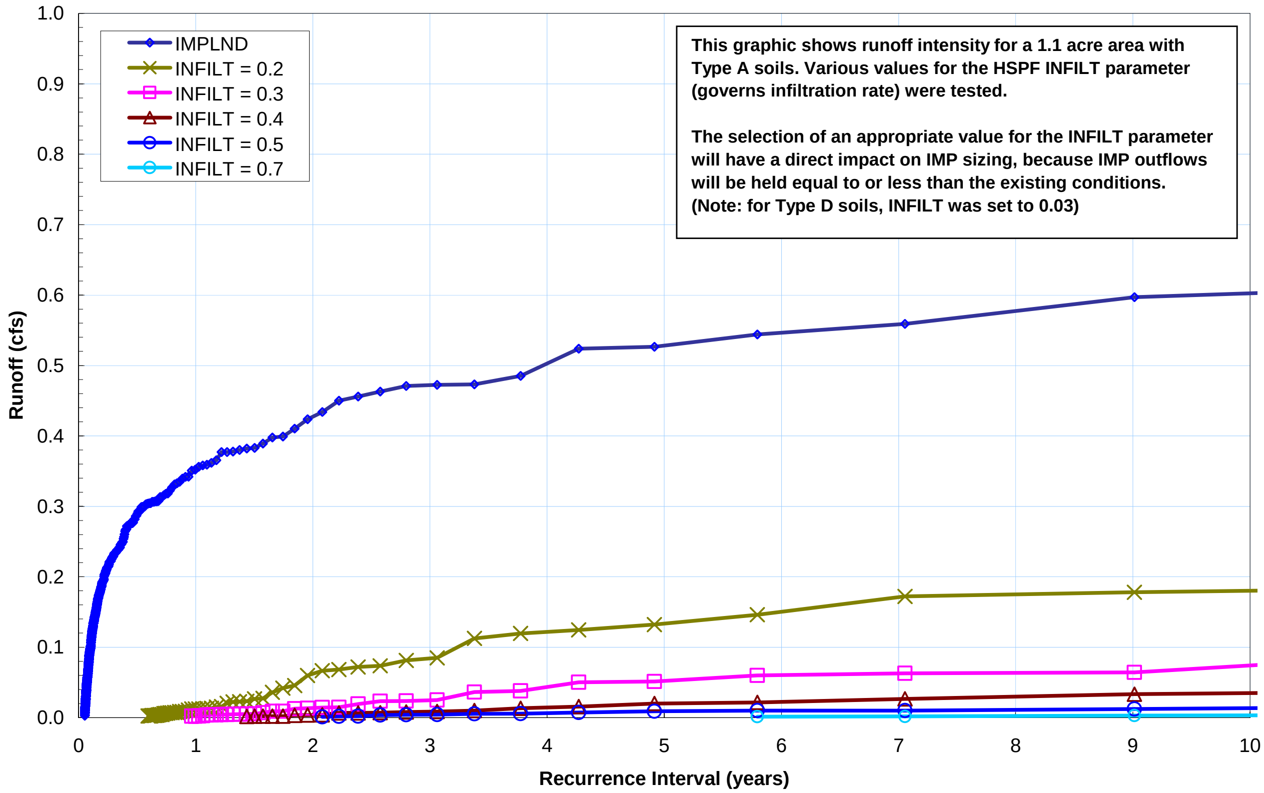

Figure 1. Surface Runoff Rates for Type A Soils for Various INFILT Parameter Values

Table 1. Ratio of Pervious to Impervious Runoff for Various INFILT Parameter Values

INFILT Value	Q9 (cfs)	Q9 Runoff Ratio: Type A / Impervious
0.2	0.18	30%
0.3	0.06	11%
0.4	0.03	6%
0.5	0.01	2%
0.6	0.01	1%
0.7	0.00	0%

[1] While the INFILT parameter does not relate directly to dry field infiltration tests, its value is generally 10 to 50 times lower than field values (in inches per hour).

The INFILT parameter value selected to size the remaining IMPs should be the value that best reflects typical rainfall runoff conditions based on the project team's experience working in East Bay watersheds. Surface runoff is minimal for INFILT values greater than 0.4. INFILT values of 0.3 and 0.4 were considered reasonable values for further consideration; sizing factors were computed for these INFILT values (Table 2). The impacts of the decision are clearly shown in Table 2: selecting INFILT = 0.4 would result in sizing factors 0.01 larger than selecting INFILT = 0.3.

Table 2. Stormwater Planter Sizing Factors for Various INFILT Parameter Values

INFILT Value	Sizing Factor (Overflow at 8 inches)	Sizing Factor (Overflow at 10 inches)
0.3	0.09	0.08
0.4	0.10	0.09[A]

A. The combination of INFILT = 0.4, overflow height = 10 inches and sizing factor = 0.09 was not one of the combinations modeled in the sensitivity analysis, but has been inferred here from the results of other modeling simulations.

Sensitivity Analysis 2: How does the opening height for the overflow pipe affect the frequency of discharges from the Stormwater Planter and its sizing factor?

The height of the overflow relief pipe determines the volume of active storage above the soil layer and the freeboard available to prevent overtopping of the planter box. To test the impact of the overflow height on the frequency of discharges through the overflow pipe, long-term HSPF simulations were conducted for overflow pipe heights of 6, 8, and 10 inches above the soil layer. All simulations assumed Type A soils and a sizing factor of 0.08. The overflow pipe was modeled as a 6-inch diameter standpipe.

Figure 2 and Table 3 show a statistical summary of the HSPF modeling results for the three overflow pipe heights. The height of the overflow pipe affects discharges in two ways:

1. Lowering the pipe generates more discharge events, because there is less storage available above the soil layer
2. Lowering the pipe produces higher peak flows during discharge events, because more head builds over the standpipe.

The impact of selecting a specific overflow height on the required sizing factor was shown earlier in Table 2. *If setting the overflow at 10 inches is a feasible construction standard, then this would provide the most efficient use of storage.* Drawings in the City of Portland's stormwater manual show overflow pipes oriented 10 inches above the soil layer with 2 inches of freeboard.

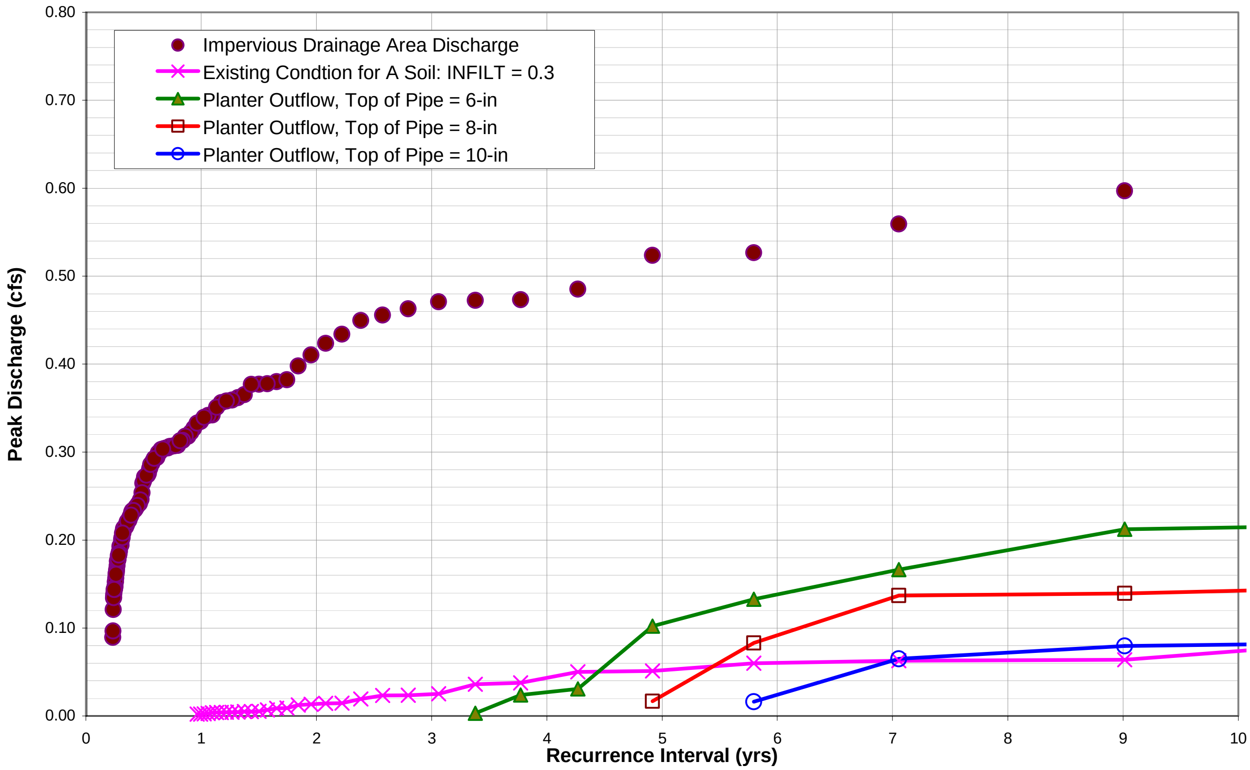

Figure 2. Overflow Pipe Discharge for Different Overflow Heights (Type A Soils; SF = 0.08)

Table 3. Summary of Overflow Pipe Discharges for 33 year HSPF Simulation

Overflow Height	# Events	Q9 (cfs/ac)
6-in	10	0.21
8-in	7	0.14
10-in	6	0.08

Sensitivity Analysis 3: How does the assumed porosity of the loamy soil layer in planter box (i.e. the growing medium) affect the Stormwater Planter sizing factor?

The porosity of the upper layer of the planter box determines the maximum available soil water storage. To test the effects of varying soil porosity on the Stormwater Planter sizing factor, the effective porosity was varied from 0.35 to 0.50[2]. All simulations assumed Type A soils and a sizing factor of 0.08. The overflow pipe was modeled as a 6-inch diameter standpipe.

Varying the porosity value had very little impact on the frequency and magnitude of discharges from the planter box and would have *no significant impact on the planter box sizing factor*. Figure 3 shows the

[2] For a natural sandy-loam soil, the average effective porosity is approximately 0.4 according to the Maidment's 1994 Handbook of Hydrology.

Stormwater Planter outflow rates across the range of porosity tested. While these results may seem counterintuitive, a closer evaluation of the modeling outputs revealed the reasons why porosity has little impact on the results:

1. In the 18-in growing medium, increasing the porosity from 0.35 to 0.50 only adds 2.7 inches of pore space (Table 4).
2. Large storms most often occur during the winter season when the soils are near-saturated and the evapotranspiration rate is low. For example, if frequent storms leave the soils 80 percent saturated prior to a large storm event, the difference in pore space across our range of tested porosities would be about half an inch (e.g. 0.2 x 2.7 inches).

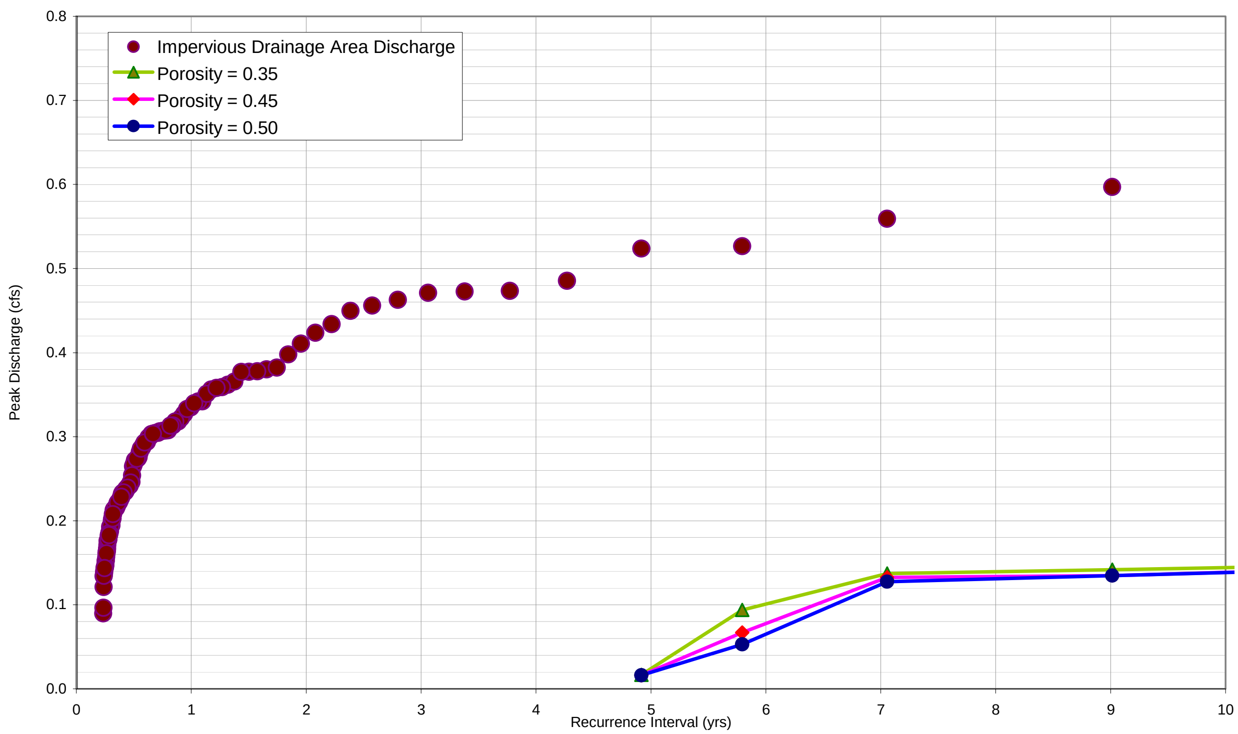

Figure 3. Stormwater Planter Outflows for Various Assumed Porosity Values (Type A Soils)

Table 4. Total Pore Space Available in 18-in Loamy Top Layer

Porosity	Pore Space (in)
0.35	6.3
0.40	7.2
0.50	9.0

Sensitivity Analysis 4: How does the underdrain discharge limit from the Stormwater Planter (for Type D soils) affect the required sizing factor?

Long-term HSPF simulations were conducted with underdrain discharge limits of 0.5Q2, 0.3Q2 and 0.1Q2 for a planter box with Type D soils. The simulations used the Martinez[3] rain gauge and an overflow pipe height of 10 inches. The computed sizing factors range from 0.04 to 0.07.

Table 5. Planter Box Sizing Factors for Various Discharge Limits for Type D Soils

Discharge Limit	Sizing Factor
0.5Q2	0.04
0.3Q2	0.05
0.1Q2	0.07

After computing these sizing factors, we examined the reasons why they are lower than the sizing factors computed in December. Two reasons were apparent:

1. The December simulations used the Flood Control District rain gauge 11, for which HSPF produces 0.5Q2 flow rates that are approximately 25 percent less than the Martinez rain gauge.
2. The December simulations allowed ponded water to discharge from the Stormwater Planter before the water reached a height of 10 inches. This produced a large number of flow hours slightly higher than 0.5Q2. The stricter overflow modeling used in the current simulations produces fewer discharges through the overflow pipe.

[3] The Martinez 2S rain gauge has a period of record spanning from 1948 to present. The data contained several erroneous readings. These readings were assumed to be actual rainfall depths; however, rainfall that had occurred over many hours was lumped into one single reading (instead of hourly increments). These readings were corrected by proportionally distributing the total rainfall to match the pattern that was recorded on Flood Control District rain gauge 11 over the same date and time. Only the years of record that could be corrected were used in the HSPF simulations.

Attachment 2, Appendix C

BROWN AND CALDWELL

Attachment 3

Memorandum

Date: May 4, 2005

To: Tom Dalziel, Contra Costa Clean Water Program

CC: Christie Beeman, Philip Williams Associates
Jeff Haltiner, Philip Williams Associates

From: Tony Dubin, BC-Seattle
Steve Anderson, BC-Seattle

Subject: Contra Costa County Clean Water Program Hydrograph Modification Program HSPF Modeling Guidance

Introduction

This memorandum provides technical guidance on how to build an HSPF (Hydrologic Simulation Program-Fortran) model to evaluate the performance of hydrograph modification facilities within Contra Costa County. As an alternative to the simplified IMP sizing approach,[1] an HSPF model may be used to ensure site-specific stormwater facilities are designed to achieve the Contra Costa County Clean Water Program's standard for runoff peak flows and durations.

Building an HSPF model for a project may be a better alternative than using simplified IMP sizing:

- When it is proposed to control runoff peaks and durations by routing runoff through detention basins, constructed wetlands, or other facilities for which a simplified sizing procedure has not been developed.
- For large drainage areas with complex drainage where the simplified approach cannot adequately represent project and pre-project conditions.
- To design facilities that serve more than one project site.
- For sites with steep slopes, dense vegetation, thin top soil, or other atypical hydrologic conditions.

The following sections of this memorandum discuss how to obtain HSPF software, the major data entry components of an HSPF runoff model, and the model parameters used to develop the IMP sizing factors. The memo is intended as a guide to building an HSPF model in Contra Costa County, but it is not a general HSPF user manual. The technical level of the discussion assumes the user is an experienced hydrologic modeler and has some familiarity with HSPF.

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[1] The simplified IMP sizing approach uses a spreadsheet tool to select the necessary sizes for hydromodification facilities based on the user's description of a project site's drainage characteristics. The IMP sizes were computed through an extensive HSPF modeling process. The simplified IMP sizing approach is summarized in a technical memorandum, *Contra Costa County Clean Water Program Hydrograph Modification Program: Integrated Management Practices Modeling Methods and Results*, dated April 29, 2005.

The remainder of this memorandum is arranged as follows:

- The **Obtaining HSPF Software** section describes where to download the software and identifies some valuable tools for model creation and data management for new HSPF users.
- **The Building and Running an HSPF Model** section describes the major data requirements of HSPF and describes the components of the model, with particular emphasis on the model elements that are used for hydromodification simulations.
- The **HSPF Modeling Analysis** section describes the iterative procedure for sizing hydromodification facilities with HSPF simulations.
- **Appendix A** provides detailed descriptions of the pervious and impervious land segment model parameters included in HSPF.

Obtaining HSPF Software

HSPF is publicly available software maintained and distributed by the US Environmental Protection Agency (EPA). HSPF is distributed as part of the EPA BASINs software suite, which includes HSPF, Soil Water Assessment Tool (SWAT), and PLOAD, which is a GIS-based model for estimating non-point source pollutant loads, and other GIS-based watershed analysis tools. BASINs also includes three utilities for building and running HSPF models and for managing time series data (Table 1) that may be very helpful to the novice HSPF user who is getting started building a model. The BASINs software suite may be downloaded from the EPA's web site, http://www.epa.gov/OST/BASINS/.

Table 1. EPA BASINs Utilities for HSPF

Utility	Description
WinHSPF	WinHSPF provides a Windows-based graphical user interface with menus and input forms for building HPSF models. This tool may be particularly valuable to new HSPF users who would prefer to develop models interactively rather than using a text editor to create a user control input (UCI) file from scratch.
WinHSPF Lite	WinHSPF Lite is a convenient tool for running already-built HSPF models. This utility loads separately-prepared HSPF input files (i.e., UCI files) and launches the HSPF executable.
WDM Utility	WDM Utility is a useful tool for managing WDM (watershed data management) files, which are the binary-formatted files used by HSPF to store time series data. WDM Utility can create time series datasets, and perform basic statistical, graphical and data aggregation functions.

Building and Running an HSPF Model for the Project Site

Building an HSPF model to simulate stormwater runoff and evaluate the performance of stormwater control facilities involves examining the drainage patterns of the site, computing the pervious and impervious areas, examining the local soil types, collecting time series input data, and expressing the site hydrology using a collection of model parameters. Building the HSPF model and analyzing the stormwater runoff are parts of the overall site development process. The procedure may be summarized as follows:

1. The developer's team develops a site plan that includes existing and proposed grading, new impervious areas, changes in land cover and soil depth, and other site characteristics that affect stormwater runoff.

2. The developer's team divides the project areas into separate drainage areas (referred to as Drainage Management Areas in the simplified IMP sizing approach) and determines where stormwater control facilities, such as integrated management practices (IMPs) and detention ponds, will be located. These first two steps should be completed before attempting to model the site runoff.
3. Once a proposed site plan is in place, an HSPF model should be built to reflect the site conditions, linking stormwater runoff from different parts of the project site with proposed IMPs and other stormwater capture devices. Building the model involves time series data collection, estimating appropriate model parameter values, and adding any necessary flow routing and stormwater control facilities to the model.

HPSF Input File Components and Data Requirements

HSPF requires extensive input information to define the hydrology of the project site. Time series data are compiled in a WDM file; hydrologic parameters, stormwater control facilities, flow routing and data output controls are all defined in the UCI input file. The following section lists recommended sources for time series data, model parameter values, and instructions on building the stage-storage-discharge relationships that define how hydromodification facilities perform.

Time Series Data Sources

HSPF requires, at a minimum, two time series datasets: precipitation and pan evaporation. Including a temperature time series improves HSPF's representation of evapotranspiration. The time series should have uniform time steps no greater than one hour. All time series should cover the entire simulation period. (In fact, the length of the time series data usually determines the length of the model simulation period.)

For Contra Costa County, precipitation and evapotranspiration data are available from the National Oceanic and Atmospheric Administration (NOAA) and the Contra Costa Flood Control District (Table 2). These datasets were used to develop the sizing factors used in the simplified IMP sizing approach.

Table 2. Time Series Input Data Sources

Station Name	Location	Period of Record	Latitude; Longitude	Elev. (ft)	Mean Annual Rain
Hourly Precipitation Data Sources					
Martinez[A]	City of Martinez	7/48 thru 2/04	37° 58' N; -122° 08' W	70.1	20.2 in
Flood Control	CCC Flood Control HQ	9/71 thru 5/04	37° 59' N; 122° 05' W	160'	16.4 in
St. Mary's	St. Mary's College	9/72 thru 5/04	37° 51' N; 122° 06' W	620'	24.8 in
Orinda Fire	Orinda Fire Station 3	9/73 thru 5/04	37° 54' N; 122° 10' W	700'	25.1 in
Los Medanos	Chevron Pipeline Pump Plant	7/74 thru 5/04	38° 00' N; 121° 51' W	130'	8.4 in
Dublin Fire	Dublin-San Ramon Fire House	9/73 thru 5/04	37° 44' N; 121° 56' W	355'	12.5 in
Hourly Evaporation Data Sources[B]					
Source	**Location**		**Data Type**		**Period of Record**
Los Alamitos	Los Alamitos Recharge Basin, San Jose		Pan Evaporation		1960 to 1996
SFO	San Francisco Airport		Pan Evaporation		1948 to 2004

A. Our examination of the Martinez Gauge record showed several questionable records where an entire storm's depth was recorded in a single hour. For these questionable storms, the recorded rainfall depth at Martinez was distributed according to the storm timing recorded at the nearest gauge (Flood Control District Gauge 11). A similar procedure should be used for simulations that use the Martinez gauge data.

B. The two data sources were combined because the higher quality dataset from Los Alamitos did not cover the entire modeling period.

HSPF Land Segment Parameters

The project site should be divided into separate drainage management areas (DMAs) based on project drainage design (e.g., location of grade breaks, direction of roof drainage, and routing of surface and piped drainage) and preliminary location of the hydrograph modification management facilities. DMAs should be configured to minimize the amount of undeveloped or landscaped area draining to the hydrograph modification management facilities. Each drainage management area should be represented by a combination of PERLND and IMPLND land segments in HSPF. The hydrograph modification management facilities should be located to capture runoff from all impervious areas while minimizing capture of runoff from pervious areas. PERLNDs represent pervious land surfaces and IMPLNDs represent impervious surfaces. Table 3 and Table 4 below contain a set of recommended PERLND and IMPLND parameters, respectively, for Contra Costa County. Appendix A contains a more detailed description of the PERLND and IMPLND parameters below. These parameters values were used in the IMP sizing analysis.

The recommended parameters may be modified if appropriate technical justification is provided. Consult the EPA publication, *EPA BASINs Technical Note 6 Estimating Hydrologic and Hydraulic Parameters for HSPF* (July 2000) for recommended ranges of HSPF parameter values. Examples of appropriate technical justification for modifying the parameters listed below include:

1. Local field measurements that differ from the recommended parameters.
2. Local land cover may differ from the cover types provided. For example, heavy forest cover could be represented by increasing the interception storage (CEPSC) and evapotranspiration fractions.

Table 3. HSPF PERLND Parameters for use in Contra Costa County

PERLND Parameter	Value	Units	Description
CSNO	0	None	Flag to determine whether snow data are used in simulation
RTOP	1	None	Flag to select overland flow routing method (see Appendix A)
UZFG	1	None	Flag to select upper zone inflow computation method
VCS	1	None	Flag to select constant or monthly-variable interception storage capacity
VUZ	0	None	Flag to select constant or monthly-variable upper zone nominal soil moisture storage
VNN	0	None	Flag to select constant or monthly-variable Manning's n parameter
VIFW	0	None	Flag to select constant or monthly-variable interflow parameter
VIRC	0	None	Flag to select constant or monthly varied interflow recession parameter
VLE	1	None	Flag to select constant or monthly varied lower zone ET parameter
FOREST	0	None	Fraction of forest covered area that will continue to transpire in winter
LZSN	7	Inch	Nominal lower zone soil moisture storage
INFILT	0.7 0.03	inch/hour	Mean soil infiltration rate. Ranges of values for NRCS Hydrologic Group B and C soils are in Appendix A. The upper value of INFILT = 0.7 was used for Group A soils; INFILT = 0.03 was used for Group D soils.
LSUR	660	Feet	Length of assumed overland flow plane. Value provided for generic 1-acre basin. For specific projects, the value should be calculated from the site plan.
SLSUR	0.1	None	Average slope of assumed overland flow path. For specific project sites, the value may be computed drafting or GIS software.
KVARY	0	per inch	Groundwater recession flow parameter used to describe non-linear groundwater recession rate. This parameter affects groundwater flow rates and is relevant to larger watershed studies that track groundwater influence on local streams.
AGWRC	0.95	per day	Groundwater recession rate, or ratio of current groundwater discharge to that from 24 hours earlier (when KVARY = 0)
PETMAX	40	deg F	Temperature below which ET will be reduced to 50% of that in the input time series
PETMIN	35	deg F	Temperature threshold where plant transpiration is effectively suspended, i.e. set to zero, due to temperatures approaching freezing

Table 3. HSPF PERLND Parameters for use in Contra Costa County (Cont.)

PERLND Parameter	Value	Units	Description
INFEXP	2	None	Exponent that determines how much a deviation from nominal lower zone storage affects the infiltration rate
INFILD	2	None	Ratio of maximum and mean soil infiltration capacities
DEEPFR	0.45 0.10	None	The fraction of infiltrating water which is lost to deep aquifers (i.e. inactive groundwater). DEEPFR = 0.45 was used for Group A soils; DEEPFR = 0.1 was used for Group D soils.
INFEXP	2	None	Exponent that determines how much a deviation from nominal lower zone storage affects the infiltration rate
AGWETP	0	None	Fraction of PERLND that is subject to direct evaporation from groundwater storage, e.g. wetlands or marsh areas
CEPSC	0.02 to 0.10	Inch	Amount of rainfall that is retained by vegetation, never reaches the land surface, and is eventually evaporated. CEPSC = 0.10 for Live Oak cover; CEPSC = 0.02 for Range cover.
UZSN	0.5	Inch	Nominal upper zone soil moisture storage
NSUR	0.3	None	Manning's friction coefficient, n, for overland flow plane
INTFW	0.4	None	The fraction of water in surface detention that becomes interflow, as opposed to direct overland flow or upper zone storage
IRC	0.3	None	The interflow recession coefficient is the ratio of the current daily interflow discharge to the interflow discharge on the previous day
LZETP	0	None	Lower zone evapotranspiration coefficient defines portion of the the ET opportunity that occurs in the lower soil zone (i.e. rooting zone)
CEPS	0	Inch	Interception storage initial value
SURS	0	Inch	Surface ponding storage initial value
UZS	0.15	Inch	Upper zone storage initial value
IFWS	0	Inch	Interflow storage initial value
LZS	4	Inch	Lower zone storage initial value
AGWS	0.05	Inch	Active groundwater storage initial value
GWVS	0	None	Initial groundwater storage slope

Table 4. HSPF IMPLND Parameters for use in Contra Costa County

IMPLND Parameter	Value	Unit	Description
CSNO	0	None	Flag to determine whether snow data are used in simulation
RTOP	0	None	Flag to select overland flow routing method (see Appendix A)
VRS	0	None	Flag to select constant or monthly-variable retention storage capacity
VNN	0	None	Flag to select constant or monthly-variable Manning's n parameter
RTLI	1	None	Flag to determine if lateral surface inflow to the impervious land segment will be subject to retention storage
LSUR	100	None	Length of assumed overland flow plane. Value provided for generic 1-acre basin. For specific projects, the value should be calculated from the site plan.
SLSUR	0.035	None	Average slope of assumed overland flow path. For specific project sites, the value may be computed drafting or GIS software.
NSUR	0.05	None	Manning's friction coefficient, n, for overland flow plane
RETSC	0.1	Inch	Retention (interception) storage of the impervious surface
PETMAX	40	deg F	Temperature below which ET will be reduced to 50% of that in the input time series
PETMIN	35	deg F	Temperature threshold below which evaporation is set to zero
RETS	1.00E-03	Inch	Retention storage initial value
SURS	1.00E-03	Inch	Surface ponding storage initial value

Linking Land Segments

HSPF includes two general schemes for routing water from land segments (PERLNDs and IMPLNDs) through a watershed. Either all outflows are moved from one land segment to the next land segment or facility at each time step, or a specific routing algorithm is used to weight the distribution of outflows over multiple time steps based on the travel time between model elements in the watershed. Linking separate land segments with routing algorithms becomes more important in larger analysis areas.

As a general rule, if the overland flow timing is similar to or longer than the model time step, then explicit routing algorithms should be considered. Flow routing is managed using RCHRES elements within HSPF. Otherwise flow from adjacent land segments may be routed directly, without weighting algorithms, using either the NETWORK or MASSLINK element.

Representing DMAs That Have IMPs

A special case exists for sites that include a mixture of IMPs and traditional downstream stormwater control facilities that collect both treated and untreated flows. This circumstance was listed in the introduction as an example that requires an HSPF model, particularly if the IMPs have underdrains. In areas with Group D soils, the IMP underdrains will discharge to the local stormwater conveyance system, so downstream hydromodification facilities may need to be sized to manage all flows (if flows from upstream IMPs cannot be segregated). Two methods are proposed for modeling these combination sites:

- One method is to include the IMPs in an HSPF model of the entire project site. The IMP outflows could be routed to the stormwater conveyance system and to any downstream control facilities. In the IMP sizing analysis, the IMPs were modeled with two-layer FTABLEs in HSPF that characterized the geometry and soil moisture holding characteristics of each IMP type. The *Low Impact Design Technical Guidance Manual for Puget Sound*, released in January 2005, provides a survey of various analysis methods used to size IMPs in Western Washington.
- As an alternative, the DMAs that contain IMPs could be modeled as the pre-project soil/cover type. This method is conservative for the range of flows controlled by the IMPs.

Modeling Downstream Hydromodification Facilities

HSPF models storage-based facilities with the FTABLE element, which defines the stage-storage-discharge relationship for a facility. Figure 1 shows an example FTABLE that could be used to model a gravel-filled detention device that allows percolation through the bottom and a flow-control release to the local stormwater conveyance. The first three columns define the stage-area-volume relationship. The final two columns define stage-discharge relationships for this facility.

```
  FTABLE      2
 rows cols                                         ***
   11    5
      Depth      Area    Volume    Q Perc  Q Outlet    ***
       (ft)   (acres) (acre-ft)     (cfs)     (cfs)    ***
       0.00      0.03    0.0000    0.0000     0.000
       0.10      0.03    0.0012    0.0001     0.000
       0.20      0.03    0.0025    0.0007     0.001
       0.30      0.03    0.0037    0.0007     0.005
       0.40      0.03    0.0050    0.0007     0.018
       0.50      0.03    0.0062    0.0007     0.047
       0.60      0.03    0.0075    0.0007     0.104
       0.70      0.03    0.0087    0.0007     0.133
       0.80      0.03    0.0100    0.0007     0.142
       0.90      0.03    0.0112    0.0007     0.151
       1.00      0.03    0.0125    0.0007     0.159
  END FTABLE2
```

Figure 1. Sample FTABLE for Stormwater Detention Facility

While the layout of the FTABLE is straightforward, the values in each column and the number of outflow columns depend on the design of the facility. First, the model developer must select the type of facility to model, including its geometry, its detention and infiltration characteristics, and the height and size of any flow control orifices or weirs.

For detention basins, the careful selection of initial orifice sizes and heights can help streamline the process of sizing the facility. The height and diameter of any flow control orifices should be sized to allow the basin outflow to match the requirements of limiting post-project peak flows and durations to pre-project levels from one half the pre-project flow with an average recurrence interval of two years (0.5Q2) to the pre-project flow with an average recurrence interval of 10 years (Q10). For example, a detention basin with two flow control orifices could have its lower orifice sized to pass 0.5Q2 when the water in the basin is just below the height of the *upper orifice*. The upper orifice could pass flows up to Q10 when the water surface reaches the height of an overflow relief weir. If the basin volume is sized to trigger the overflow relief an average of once per 10 years, this setup should come close to approximating the flow and duration control standard, and reduce the number of modeling simulations needed in the iterative facility sizing process.

HSPF Modeling Analysis of the Project Site

After compiling the required input dataset, defining model parameters, and specifying the stormwater control scheme for the project area, the next step involves running the HSPF model to determine if the post-project flows are controlled to the pre-project levels. The program requires that projects subject to hydrograph modification control must meet a specific peak flow and duration standard. Partial duration series statistics should be used to (1) parse the HSPF output time series into discrete flow events and (2) compute the recurrence interval and peak flow for each flow event. The peak flow and duration control standard is summarized as follows:

<u>Peak Flow Control</u>

- From 0.5Q2 to Q2 (inclusive), the post-project peak flows should not exceed pre-project peak flows.
- For recurrence intervals from Q2 to Q10, the post-project peak flows may exceed pre-project peak flows by up to 10 percent for a 1-year band within the 2 to 10 year recurrence interval range. For example, the post-project flows could exceed the pre-project flows by up to 10 percent between Q9 and Q10 or from Q5.5 to Q6.5, but not from Q8 to Q10.

<u>Flow Duration Control</u>

- From 0.5Q2 to Q2 (inclusive), the post-project flow durations (i.e., the aggregate time for which the site discharge exceeds a specific flow rate) should not exceed the pre-project flow durations. This recognizes the impact of these relatively frequent events on the stream channel stability.
- For flow rates above Q2, post-project flow durations should not exceed pre-project flow durations by more than 10 percent at any flow rate.
- The post-project durations should not exceed pre-project durations for more than 50 percent of the flow levels from 0.5Q2 to Q10.

Sizing facilities to meet the peak flow and duration control standard is often an iterative process that involves several HSPF simulations and statistical analyses. The following steps outline a general procedure for applying the HSPF model to compute pre-project and post-project flows and assess the performance of hydromodification facilities.

1. Conduct long-term HSPF simulations to compute hourly runoff-hydrographs for the following conditions:
    a. Pre-project site conditions
    b. Proposed post-project site conditions
    c. Mitigated post-project site conditions with hydromodification facilities included
2. Calculate peak flow frequencies using partial duration series statistics, which may be produced using available data analysis software packages.
3. Calculate flow duration statistics using database queries or data analysis software.

4. Produce summary peak flow and flow duration graphics to assess the performance of the hydromodification approach (see Figure 2 and Figure 3). The example shown in the figures meets the peak flow and flow duration standards because the mitigated post-project peak flow and flow duration curves are below the corresponding pre-project curves in the range from 0.5Q2 to Q10. If the post-project flows do not meet the peak flow and flow duration standards, the hydrograph modification management facilities or site design components should be revised and the HSPF modeling process repeated.

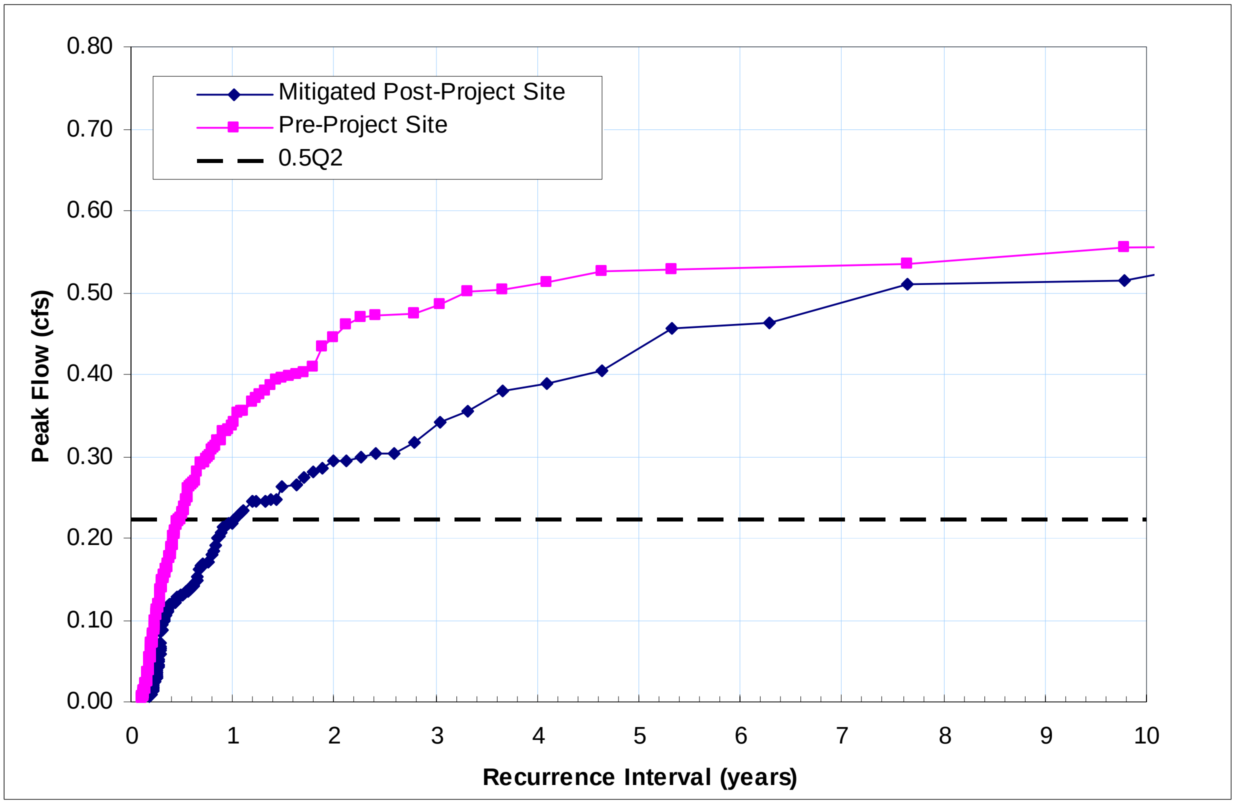

Figure 2. Example Peak Flow Frequency Plot for Post-Project Flows that Meet Control Standard

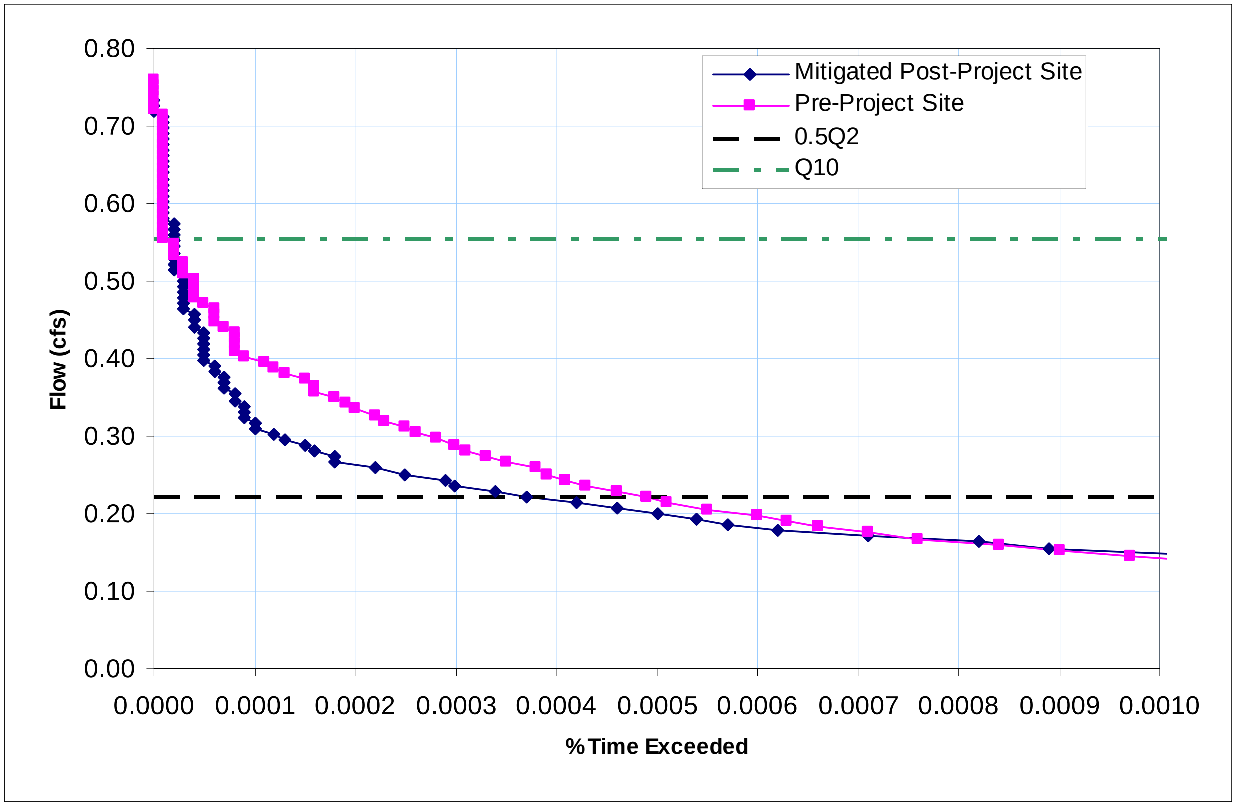

Figure 3. Example Flow Duration Plot for Post-Project Flows that Meet Control Standard

APPENDIX A: HSPF PARAMETER DESCRIPTIONS

This section provides a list of descriptions for the pervious and impervious land surface parameters (PERLND and IMPLND, respectively) used in the HSPF model for Contra Costa. The values for these parameters were derived from numerous sources: the USGS regional calibration on Calabazas Creek in Santa Clara County, the WWHM, and the EPA publication, *EPA Basins Technical Note 6 Estimating Hydrologic and Hydraulic Parameters for HSPF* (July 2000), from which the parameter descriptions below are reproduced.

PERLND Parameters

PWAT-PARM1 Table: Sets PERLND Flags

The PWAT-PARM1 table includes flags to indicate the selected simulation algorithm option, other selection of monthly variability versus constant values for selected parameters. Where flags indicate monthly variability, the corresponding monthly values must be provided in Monthly Input Parameters (see below following the PWAT_PARM4 Table section). That section also provides guidance on which parameters are normally specified as monthly values.

CSNOFG Flag to use snow simulation data; must be checked (CSNOFG=1) if SNOW is simulated.

RTOPFG Flag to select overland flow routing method; choose either the method used in predecessor models (HSPX, ARM, and NPS) or the alternative method as described in the HSPF User Manual. Recommendation: Set RTOPFG=1; This method, used in the predecessor models is more commonly used, and has been subjected to more widespread application.

UZFG Flag to select upper zone inflow computation method; choose either the method used in predecessor models (HSPX, ARM, and NPS) or the more exact numerical solution to the integral of inflow to upper zone, i.e the alternative method. Recommendation: Set UZFG=1; This method, used in the predecessor models, is more commonly used, and has been subjected to more widespread application.

VCSFG Flag to select constant or monthly-variable interception storage capacity, CEPSC. Monthly value can be varied to represent seasonal changes in foliage cover; monthly values are commonly used for agricultural, and sometimes deciduous forest land areas.

VUZFG Flag to select constant or monthly-variable upper zone nominal soil moisture storage, UZSN. Monthly values are commonly used for agricultural areas to reflect the timing of cropping and tillage practices.

VMNFG Flag to select constant or monthly-variable Manning=s n for overland flow plane, NSUR. Monthly values are commonly used for agricultural, and sometimes deciduous forest land areas.

VIFWFG Flag to select constant or monthly-variable interflow inflow parameter, INTFW. Monthly values are not often used.

VIRCFG Flag to select constant or monthly varied interflow recession parameter, IRC. Monthly values are not often used.

VLEFG Flag to select constant or monthly varied lower zone ET parameter, LZETP. Monthly values are commonly used for agricultural, and sometimes deciduous forest land areas.

PWAT-PARM2 Table:

FOREST Fraction of land covered by forest (unitless) (measure/estimate). FOREST is the fraction of the land segment which is covered by forest which will continue to transpire in winter (i.e. coniferous). This is only relevant if snow is being considered (i.e., CSNOFG=1 in PWATER-PARM1).

LZSN Lower zone nominal soil moisture storage (inches), (estimate, then calibrate). LZSN is related to both precipitation patterns and soil characteristics in the region. The ARM Model User Manual (Donigian and Davis, 1978, p. 56, LZSN variable) includes a mapping of calibrated LZSN values across the country based on almost 60 applications of earlier models derived from the Stanford-based hydrology algorithms. LaRoche et al (1996) shows values of 5 inches to 14 inches, which is consistent with the 'possible' range of 2 inches to 15 inches shown in the Summary Table. Viessman, et al, 1989, provide initial estimates for LZSN in the Stanford Watershed Model (SWM-IV, predecessor model to HSPF) as one-quarter of the mean annual rainfall plus four inches for arid and semiarid regions, or one-eighth annual mean rainfall plus 4 inches for coastal, humid, or subhumid climates. These formulae tend to give values somewhat higher than are typically seen as final calibrated values; since LZSN will be adjusted through calibration, initial estimates obtained through these formulae may be reasonable starting values.

INFILT Index to mean soil infiltration rate (in/hr); (estimate, then calibrate). In HSPF, INFILT is the parameter that effectively controls the overall division of the available moisture from precipitation (after interception) into surface and subsurface flow and storage components. Thus, high values of INFILT will produce more water in the lower zone and groundwater, and result in higher baseflow to the stream; low values of INFILT will produce more upper zone and interflow storage water, and thus result in greater direct overland flow and interflow. LaRoche et al (1996) shows a range of INFILT values used from 0.004 in/hr to 0.23 in/hr, consistent with the 'typical' range of 0.01 to 0.25 in/hr in the Summary Table. Fontaine and Jacomino (1997) show sediment and sediment associated transport to be sensitive to the INFILT parameter since it controls the amount of direct overland flow transporting the sediment. Since INFILT is not a maximum rate nor an infiltration capacity term, it's values are normally much less than published infiltration rates, percolation rates (from soil percolation tests), or permeability rates from the literature. In any case, initial values are adjusted in the calibration process. INFILT is primarily a function of soil characteristics, and value ranges have been related to SCS hydrologic soil groups (Donigian and Davis, 1978, p.61, variable INFIL) as follows: NRCS Hydrologic INFILT Estimate Soil Group (in/hr) (mm/hr) Runoff Potential

Table A1. Recommended INFILT Parameter Range for Initial Model Setup

NRCS Hydrologic Soil Group	Initial Model Setup: INFILT range (in/hr)	Runoff Potential
A	0.4 to 1.0	Low
B	0.1 to 0.4	Moderate
C	0.05 to 0.1	Moderate to High
D	0.01 to 0.05	High

An alternate estimation method that has not been validated, is derived from the premise that the combination of infiltration and interflow in HSPF represents the infiltration commonly modeled in the literature (e.g. Viessman et al, 1989, Chapter 4). With this assumption, the value of 2.0*INFILT*INTFW should approximate the average measured soil infiltration rate at saturation, or mean permeability.

LSUR Length of assumed overland flow plane (ft) (estimate/measure). LSUR approximates the average length of travel for water to reach the stream reach, or any drainage path such as small streams, swales, ditches, etc. that quickly deliver the water to the stream or waterbody. LSUR is often assumed to vary with slope such that flat slopes have larger LSUR values and vice versa; typical values range from 200 feet to 500 feet for slopes ranging from 15% to 1 %. It is also often estimated from topographic data by dividing the watershed area by twice the length of all streams, gullies, ditches, etc that move the water to the stream. That is, a representative straight-line reach with length, L, bisecting a representative square areal segment of the watershed, will produce two overland flow planes of width ½ L. However, LSUR values derived from topographic data are often too large (i.e. overestimated) when the data is of insufficient resolution to display the many small streams and drainage ways. Users should make sure that values calculated from GIS or topographic data are consistent with the ranges shown in the Summary Table.

<u>SLSUR</u> Average slope of assumed overland flow path (unitless) (estimate/measure). Average SLSUR values for each land use being simulated can often be estimated directly with GIS capabilities. Graphical techniques include imposing a grid pattern on the watershed and calculating slope values for each grid point for each land use.

<u>KVARY</u> Groundwater recession flow parameter used to describe non-linear groundwater recession rate (/inches) (initialize with reported values, then calibrate as needed) KVARY is usually one of the last PWATER parameters to be adjusted; it is used when the observed groundwater recession demonstrates a seasonal variability with a faster recession (i.e. higher slope and lower AGWRC values) during wet periods, and the opposite during dry periods. LaRoche, et al, 1996 reported an extremely high 'optimized' value of 0.66 mm^{-1} or (17 in^{-1}) (much higher than any other applications) while Chen, et al, 1995 reported a calibrated value of 0.14 mm^{-1} (or 3.6 in^{-1}). Value ranges are shown in the Summary Table. Users should start with a value of 0.0 for KVARY, and then adjust (i.e. increase) if seasonal variations are evident. Plotting daily flows with a logarithmic scale helps to elucidate the slope of the flow recession.

<u>AGWRC</u> Groundwater recession rate, or ratio of current groundwater discharge to that from 24 hours earlier (when KVARY is zero) (/day) (estimate, then calibrate). The overall watershed recession rate is a complex function of watershed conditions, including climate, topography, soils, and land use. Hydrograph separation techniques (see any hydrology or water resources textbook) can be used to estimate the recession rate from observed daily flow data (such as plotting on a logarithmic scale, as noted above); estimated values will likely need to be adjusted through calibration. Value ranges are shown in the Summary Table. LaRoche, et al, 1996 reported an optimized value of 0.99; Chen, et al, 1995 reported values that varied with land use type, ranging from 0.971 for grassland and clearings to 0.996 for high density forest; Fontaine and Jacomino, 1997 reported a calibrated value of 0.99. This experience reflects normal practice of using higher values for forests than open, grassland, cropland and urban areas.

<u>PWAT-PARM3 Table:</u>

<u>PETMAX</u> Temperature below which ET will be reduced to 50% of that in the input time series (deg F), unless it=s been reduced to a lesser value from adjustments made in the SNOW routine (where ET is reduced based on the percent areal snow coverage and fraction of coniferous forest). PETMAX represents a temperature threshold where plant transpiration, which is part of ET, is reduced due to low temperatures (initialize with reported values, then calibrate as needed). It is only used if SNOW is being simulated because it requires air temperature as input (also a requirement of the SNOW module), and the required low temperatures will usually only occur in areas of frequent snowfall. Use the default of 40$_{o}$F as an initial value, which can be adjusted a few degrees if required. PETMIN Temperature at and below which ET will be zero (deg F).

<u>PETMIN</u> represents the temperature threshold where plant transpiration is effectively suspended, i.e. set to zero, due to temperatures approaching freezing (initialize with reported values, then calibrate as needed). Like PETMAX, this parameter is used only if SNOW is being simulated because it requires air temperature as input (also a requirement of the SNOW module), and the required low temperatures will usually only occur in areas of frequent snowfall. Use the default of 35$_{o}$F as an initial value, which can be adjusted a few degrees if required.

<u>INFEXP</u> Exponent that determines how much a deviation from nominal lower zone storage affects the infiltration rate (HSPF Manual, p. 60) (initialize with reported values, then calibrate as needed). Variations of the Stanford approach have used a POWER variable for this parameter; various values of POWER are included in Donigian and Davis (1978, p. 58). However, the vast majority of HSPF applications have used the default value of 2.0 for this exponent. Use the default value of 2.0, and adjust only if supported by local data and conditions.

INFILD Ratio of maximum and mean soil infiltration capacities (initialize with reported value). In the Stanford approach, this parameter has always been set to 2.0, so that the maximum infiltration rate is twice the mean (i.e. input) value; when HSPF was developed, the INFILD parameter was included to allow investigation of this assumption. However, there has been very little research to support using a value other than 2.0. Use the default value of 2.0, and adjust only if supported by local data and conditions.

DEEPFR The fraction of infiltrating water which is lost to deep aquifers (i.e. inactive groundwater), with the remaining fraction (i.e. 1-DEEPFR) assigned to active groundwater storage that contributes baseflow to the stream (estimate, then calibrate). It is also used to represent any other losses that may not be measured at the flow gage used for calibration, such as flow around or under the gage site. This accounts for one of only three major losses from the PWATER water balance (i.e. in addition to ET, and lateral and stream outflows). Watershed areas at high elevations, or in the upland portion of the watershed, are likely to lose more water to deep groundwater (i.e. groundwater that does not discharge within the area of the watershed), than areas at lower elevations or closer to the gage (see discussion and figures in Freeze and Cherry, 1979, section 6.1). DEEPFR should be set to 0.0 initially or estimated based on groundwater studies, and then calibrated, in conjunction with adjustments to ET parameters, to achieve a satisfactory annual water balance.

BASETP ET by riparian vegetation as active groundwater enters streambed; specified as a fraction of potential ET, which is fulfilled only as outflow exists (estimate, then calibrate). Typical and possible value ranges are shown in the Summary Table. If significant riparian vegetation is present in the watershed then non-zero values of BASETP should be used. Adjustments to BASETP will be visible in changes in the low-flow simulation, and will effect the annual water balance. If riparian vegetation is significant, start with a BASETP value of 0.03 and adjust to obtain a reasonable low-flow simulation in conjunction with a satisfactory annual water balance.

AGWETP Fraction of model segment (i.e. pervious land segment) that is subject to direct evaporation from groundwater storage, e.g. wetlands or marsh areas, where the groundwater surface is at or near the land surface, or in areas with phreatophytic vegetation drawing directly from groundwater. This is represented in the model as the fraction of remaining potential ET (i.e. after base ET, interception ET, and upper zone ET are satisfied), that can be met from active groundwater storage (estimate, then calibrate). If wetlands are represented as a separate PLS (pervious land segment), then AGWETP should be 0.0 for all other land uses, and a high value (0.3 to 0.7) should be used for the wetlands PLS. If wetlands are not separated out as a PLS, identify the fraction of the model segment that meets the conditions of wetlands/marshes or phreatophytic vegetation and use that fraction for an initial value of AGWETP. Like BASETP, adjustments to AGWETP will be visible in changes in the low-flow simulation, and will effect the annual water balance. Follow above guidance for an initial value of AGWETP, and then adjust to obtain a reasonable low-flow simulation in conjunction with a satisfactory annual water balance.

PWAT PARM4 Table:

CEPSC Amount of rainfall, in inches, which is retained by vegetation, never reaches the land surface, and is eventually evaporated (estimate, then calibrate). Typical guidance for CEPSC for selected land surfaces is provided in Donigian and Davis (1978, p. 54, variable EPXM) as follows:

Table A2. Recommended CEPSC Parameter Range for Initial Model Setup

Land Cover	Maximum Interception (in)
Grassland	0.1
Cropland	0.1 to 0.25
Forest Cover, light	0.15
Forest Cover, heavy	0.20

Donigian et al (1983) provide more detail guidance for agricultural conditions, including residue cover for agricultural BMPs. As part of an annual water balance, Viessman, et al. 1989 note that 10-20% of precipitation during growing season is intercepted and as much as 25% of total annual precipitation is intercepted under dense closed forest stands; crops and grasses exhibit a wide range of interception rates - between 7% and 60% of total rainfall. Users should compare the annual interception evaporation (CEPE) with the total rainfall available (PREC in the WDM file), and then adjust the CEPSC values accordingly. (See Monthly Input Values below).

UZSN Nominal upper zone soil moisture storage (inches) (estimate, then calibrate). UZSN is related to land surface characteristics, topography, and LZSN. For agricultural conditions, tillage and other practices, UZSN may change over the course of the growing season. Increasing UZSN value increases the amount of water retained in the upper zone and available for ET, and thereby decreases the dynamic behavior of the surface and reduces direct overland flow; decreasing UZSN has the opposite effect. Donigian and Davis (1978, p. 54) provide initial estimates for UZSN as 0.06 of LZSN, for steep slopes, limited vegetation, low depression storage; 0.08 LZSN for moderate slopes, moderate vegetation, and moderate depression storage; 0.14 LZSN for heavy vegetal or forest cover, soils subject to cracking, high depression storage, very mild slopes. Donigian et al., (1983) include detailed guidance for UZSN for agricultural conditions. LaRoche shows values ranging from 0.016 in to 0.75 in. Fontaine and Jacomino showed average daily stream flow was relatively insensitive to this value but sediment and sediment associated contaminant outflow was sensitive; this is consistent with experience with UZSN having an impact on direct overland flow, but little impact on the annual water balance (except for extremely small watersheds with no baseflow). Typical and possible value ranges are shown in the Summary Table.

NSUR Manning's n for overland flow plane (estimate). Manning's n values for overland flow are considerably higher than the more common published values for flow through a channel, where values range from a low of about 0.011 for smooth concrete, to as high as 0.050-0.1 for flow through unmaintained channels (Hwang and Hita, 1987). Donigian and Davis (1978, p. 61, variable NN) and Donigian et al (1983) have tabulated the following values for different land surface conditions:

Table A3. Recommended NSUR Parameter Range for Initial Model Setup

Overland Flow Surface	Manning's n Value (NSUR)
Smooth packed surface	0.05
Normal roads and parking lots	0.10
Disturbed land surfaces	0.15 to 0.25
Moderate turf/pasture	0.20 to 0.30
Heavy turf, forest litter	0.30 to 0.45
Conventional Tillage	0.15 to 0.25
Smooth fallow	0.15 to 0.20
Rough fallow, cultivated	0.20 to 0.30
Crop residues	0.25 to 0.35
Meadow, heavy turf	0.30 to 0.40

For agricultural conditions, monthly values are often used to reflect the seasonal changes in land surfaces conditions depending on cropping and tillage practices. Additional tabulations of Manning's n values for different types of surface cover can be found in: Weltz, et al, 1992; Engman, 1986; and Mays, 1999. Manning's n values are not often calibrated since they have a relatively small impact on both peak flows and volumes as long as they are within the normal ranges shown above. Also, calibration requires data on just overland flow from very small watersheds, which is not normally available except at research plots and possibly urban sites.

<u>INTFW</u> Coefficient that determines the amount of water which enters the ground from surface detention storage and becomes interflow, as opposed to direct overland flow and upper zone storage (estimate, then calibrate). Interflow can have an important influence on storm hydrographs, particularly when vertical percolation is retarded by a shallow, less permeable soil layer. INTFW affects the timing of runoff by effecting the division of water between interflow and surface processes. Increasing INTFW increases the amount of interflow and decreases direct overland flow, thereby reducing peak flows while maintaining the same volume. Thus it affects the shape of the hydrograph, by shifting and delaying the flow to later in time. Likewise, decreasing INTFW has the opposite effect. Base flow is not affected by INTFW. Rather, once total storm volumes are calibrated, INTFW can be used to raise or lower the peaks to better match the observed hydrograph. Typical and possible value ranges are shown in the Summary Table.

<u>IRC</u> Interflow recession coefficient (estimate, then calibrate). IRC is analogous to the groundwater recession parameter, AGWRC, i.e. it is the ratio of the current daily interflow discharge to the interflow discharge on the previous day. Whereas INTFW affects the volume of interflow, IRC affects the rate at which interflow is discharged from storage. Thus it also affects the hydrograph shape in the 'falling' or recession region of the curve between the peak storm flow and baseflow. The maximum value range is 0.3 – 0.85, with lower values on steeper slopes; values near the high end of the range will make interflow behave more like baseflow, while low values will make interflow behave more like overland flow. IRC should be adjusted based on whether simulated storm peaks recede faster/slower than measured, once AGWRC has been calibrated. Typical and possible value ranges are shown in the Summary Table.

<u>LZETP</u> Index to lower zone evapotranspiration (unitless) (estimate, then calibrate). LZETP is a coefficient to define the ET opportunity; it affects evapotranspiration from the lower zone which represents the primary soil moisture storage and root zone of the soil profile. LZETP behaves much like a 'crop coefficient' with values mostly in the range of 0.2 to 0.7; as such it is primarily a function of vegetation; Typical and possible value ranges are shown in the Summary Table, and the following ranges for different vegetation are expected for the 'maximum' value during the year:

Table A4. Recommended LZETP Parameter Range for Initial Model Setup

Vegetation / Crop Type	Lower Zone ET Potential (LZETP)
Forest	0.6
Grassland	0.4
Row crops	0.5
Barren	0.1
Wetlands	0.6

<u>Monthly Input Parameter Tables:</u>

In general, monthly variation in selected parameters, such as CEPSC and LZETP should be included with the initial parameter estimates. However, adjustments to the monthly values should be addressed only after annual flow volumes are matched well with monitored data. All monthly values can be adjusted to calibrate for seasonal variations.

MON-INTERCEP Table:

Monthly values for interception storage. Monthly values can be developed based on the data presented in the discussion in PWAT-PARM4/CEPSC and the Summary Tables.

MON-UZSN Table:

Monthly values for upper zone storage. For agricultural areas under conventional tillage, lower values are used to reflect seedbed preparation in the spring with values increasing during the growing season until harvest and fall tillage. See PWAT-PARM4/UZSN discussion and Summary Tables for guidance.

MON-MANNING Table:

Monthly values for Manning=s n for the overland flow plane. Monthly values can be used to represent seasonal variability in ground cover including crop and litter residue. See discussion in PWAT-PARM4/NSUR for Manning's n as a function of agricultural conditions.

MON-INTERFLW Table:

Monthly values for interflow parameter (INTFW) are not often used.

MON-IRC Table:

Monthly values for interflow recession parameter are not often used.

MON-LZETPARM Table:

Monthly values for LZETP for evapotranspiration from the lower zone can be developed using an expected maximum value from the PWAT-PARM4/LZETP discussion and the range of values presented in the Summary Tables. Monthly variable values should be used to reflect the seasonality of evapotranspiration, in response to changes in density of vegetation, depth of root zone, and stage of plant growth.

PWAT-STATE1 Table:

CEPS, SURS, IFWS, UZS, LZS, AGWS, are initial values for storage of water in interception, surface ponding, interflow, the upper zone, lower zone, and active groundwater, respectively, and GWVS is the initial index to groundwater slope. All these storages pertain to the first interval of the simulation period. The surface related storages (i.e. CEPS, SURS, IFWS) are highly dynamic, and will reach a dynamic equilibrium within a few days, at most. These state variables can be left blank, or set to 0.0 unless an individual storm is being simulated. The soil storages (i.e. UZS, LZS, and AGWS, and the GWVS) are much less dynamic, so their beginning values can impact the simulation for a period of months to a few years.

If possible, users should allow as long a startup time period as possible (i.e. set the simulation period to begin prior to the period you=ll use for comparison against monitoring data or other use); as noted each of these storages should reach a dynamic equilibrium within a few years of simulation. UZS and LZS should be set equal to UZSN and LZSN respectively, unless it is known that the starting date is during a particularly wet or dry period; starting values can be increased or decreased if wet or dry conditions were evident prior to the simulation period. AGWS is a bit more problematic. If far too high or too low, baseflow will be excessive or skewed low for several months or years, depending on AGWRC and KVARY. Improper values of GWVS can also cause simulation accuracy problems again for lengths of time depending on values of AGWRC and KVARY. However, since when KVARY is set to 0.0 seasonal recession is not represented and GWVS is not calculated. To avoid problems, then, AGWS should be set to 1.0 inch and GWVS to 0.0 for initial simulation runs. If the simulation period is limited in duration, you can check and reset these state variables to values observed for the same period in subsequent years with similar climatic conditions. However, if major calibration changes are made to the parameters controlling these storages (e.g. UZSN, LZSN, INFILT), then the initial conditions should be checked and adjusted during the calibration process. The values for AGWS and GWVS should be checked and adjusted as noted above, which assuming a yearly cycle of groundwater storage, can be compared to values during similar seasons in the simulation period. If the initial simulated baseflow (before the first significant rainfall) is much different from the initial observed streamflow, then further adjustments can be made to raise or lower the flow rates.

IMPLND Parameters

IWAT-PARM1 Table:

The IWAT-PARM1 table includes a number of flag variables to indicate either the selection of a simulation algorithm option, or whether the parameter will be treated as a constant or be varied monthly. As with PWAT-PARM1, where flags indicate monthly variability, corresponding monthly values must be provided in Monthly Input Parameter tables (see below following IWATPARM3 section).

CSNOFG Flag to use snow simulation data; must be checked (CSNOFG=1) if SNOW module is run.

RTOPFG Flag to select overland flow routing method. If RTOPFG=0, a new routing algorithm is used. RTOPFG=1 results in the use of the method used by predecessor models (HSPX, ARM, and NPS). Recommendation: set RTOPFG=1; this method is more commonly used and has been subjected to more widespread application.

VRSFG Flag to select constant or monthly-variable retention storage capacity, RETSC. Monthly values are not often used.

VNNFG Flag to select constant or monthly-variable Manning's n for overland flow plane, NSUR. Monthly values are not often used.

RTLIFG Flag to determine if lateral surface inflow to the impervious land segment will be subject to retention storage (RTLIFG=1). This flag only has an impact if the another land segment drains to the impervious land segment; otherwise lateral surface inflow is nonexistent. This feature is not commonly used in most HSPF applications.

IWAT-PARM2 Table:

LSUR Length of assumed overland flow plane (feet), (measure/estimate). See PWATPARM2/ LSUR discussion. For impervious areas, LSUR reflects the overland flow length on directly connected, or effective impervious area (EIA), and is usually in the range of 50 to 150 feet, although longer lengths may apply in commercial or industrial regions of large metropolitan areas. Impervious surfaces that drain to pervious land, rather than to a reach, are considered part of the pervious land segment and not part of the EIA.

SLSUR Average slope of the assumed overland flow path (unitless), (measure/estimate). See PWAT-PARM2 / SLSUR discussion.

NSUR Manning's n for overland flow plane (estimate). See PWAT-PARM4 / NSUR discussion. Recommendation: set NSUR within the range of 0.05 to 0.10 for paved roads and parking lots.

RETSC Retention (interception) storage of the impervious surface (inches) (estimate). RETSC is the impervious equivalent to the interception storage variable (CEPSC) used for pervious land segments. RETSC is the depth of water that collects on the impervious surface before any runoff occurs. A study of five urban watersheds in the Puget Sound region conducted by the U.S. Geological Survey (Dinicola, 1990) found that a value of 0.10 for RETSC was appropriate. If parking lots and rooftops are designed for detention storage, larger values up to 0.5 inches may be reasonable.

IWAT-PARM3 Table:

The following two parameters are used only if SNOW is being simulated.

PETMAX Temperature below which ET will be reduced by 50% of that in the input time series (degree F), (estimate, then calibrate). See PWAT-PARM3 /PETMAX discussion.

PETMIN Temperature at and below which ET will be set to zero (degree F), (estimate, then calibrate). See PWAT-PARM3 /PETMIN discussion.

Monthly Input Parameter Tables:

MON-RETN Table:

Monthly values for retention storage. Monthly values can be varied to represent seasonal changes in surface retention storage due to litter accumulation or sediment deposition on the impervious surface. Monthly values are not often used.

MON-MANNING Table:

Monthly values for Manning's n for the overland flow plane. As described above for MONRETN, monthly values can be changed to represent seasonal changes on the surface of the impervious area. Monthly values are not often used.

IWAT-STATE1 Table:

RETS and SURS are initial values for storage of water in retention and surface ponding, respectively. Both of these storages pertain to the first day of the simulation period. RETS and SURS are highly dynamic and are only non-zero if the simulation starts during or just following a storm event. They can be left blank or set to zero unless an individual storm is being simulated.

Attachment 4

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M E M O R A N D U M

DATE: May 4, 2005

TO: Tom Dalziel

COMPANY: Contra Costa Clean Water Program

FROM: Andy Collison
Mike Liquori

RE: Stream Classification Methodology

PWA Ref. #: 1742

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1. INTRODUCTION

The purpose of the stream classification methodology is to evaluate the sensitivity of a stream to erosion due to development-related increases in flow peak and duration, and to assign a "risk" classification for stream erosion (high, medium or low). The method outlined in this memo will facilitate consistent classification of stream risk by a qualified stream geomorphologist based on a basic field assessment. Guidelines for more detailed analyses—required when the basic field assessment does not yield a clear and unambiguous result—are included in Section 6.

2. SUMMARY OF TECHNICAL BASIS

Erosion occurs when *boundary shear stress* (the force of water flowing on a surface such as the bed or banks of a creek) exceeds *critical shear stress* (the amount of force required to cause erosion of the bank or bed material). A stream's potential for erosion due to increased flow is determined by two factors: the rate at which boundary shear stress increases relative to increases in flow (shear stress sensitivity), and the margin between the boundary shear stress of the flow and the critical shear stress of the channel materials (channel resistance).

Shear stress sensitivity is proportional to flow depth, and is therefore generally related to the shape of a stream channel. In some channel reaches, large increases in peak flow bring about relatively small increases in boundary shear stress, and so erosion vulnerability is likely to be relatively low (low shear stress sensitivity). Such channels are likely to be wide and shallow, or have unconfined floodplains where additional peak flows can escape out of the channel. In entrenched channels, small increases in peak flow bring about relatively large increases in flow depth, and boundary shear stress is increased proportionally (high shear stress sensitivity). These channels are more vulnerable to flow increases.

Channel resistance is generally related to the characteristics of the stream bed and banks. Channels composed of resistant materials, such as gravel and cobbles, are less sensitive to additional peak flows because the channel resistance is large. Channels composed of erodible materials, such as clay and silt, are vulnerable to smaller increases in boundary shear stress (low channel resistance). Channel resistance can be partially assessed by comparing the boundary shear stresses associated with stream flows, with the critical shear stresses of the bed materials.

	Low Shear Stress Sensitivity	High Shear Stress Sensitivity
High Channel Resistance	Wide shallow channel with coarse materials or armor	Narrow, deep channel cut in resistant or armored materials
Low Channel Resistance	Wide, shallow channel with fine materials	Narrow, deep channel cut in fine materials

Increasing stream vulnerability

Increasing stream vulnerability

Figure 1. Conceptual Approach to Stream Classification

We have used the conceptual model of stream vulnerability shown in Figure 1 to develop the following definitions for low, medium and high sensitivity channels.

2.1 LOW RISK CHANNEL

In low risk channels, the boundary shear stress is likely to be greatly below the critical shear stress, either because the channel is hardened or because the gradient is so low that the channel is dominated by deposition rather than transport or erosion. The criteria for identifying low risk channels closely follow the exemptions described in C.3.f.ii. Note that channels that have hardened banks but without erosion resistant bottoms should not be exempted, as bed erosion may still occur.

2.2 MEDIUM RISK CHANNEL

Medium risk channels are those where boundary shear stress could exceed critical shear stress as a result of hydrograph modification, but where either the sensitivity of boundary shear stress to increased flow is low or the resistance of the channel materials is relatively high. The first condition is met in oversize channels with high width to depth ratios, where increases in peak flow have little effect on flow depth and therefore shear stress (e.g. large earth flood control channels or some naturally wide, shallow channels with easy access to floodplains during high flow events). The second condition is met in channels that have coarse or armored bed material (e.g. cobble or boulder beds), and vegetated banks.

2.3 HIGH RISK CHANNEL

High-risk channels are those where the sensitivity of boundary shear stress to flow is high and/or the channel resistance is low. The first condition is met in confined channels where flow increases result in large increases in flow depth (e.g. incised or entrenched channels, and channels with low width to depth ratios). The second condition is met in channels that are composed of fine-grained, erodible bed or bank materials, or that have little or no bank vegetation. Channels that show evidence of active erosion will also be defined as high risk, since active erosion indicates that boundary shear stress already exceeds critical shear stress during high flows, and there is no margin of channel resistance.

3. OVERVIEW OF THE CLASSIFICATION METHOD

The classification method is outlined in flow chart form in Figure 2. The first step is to check whether the stream meets the low risk criteria. If not, a basic geomorphic assessment will, in most cases, distinguish whether a channel is medium or high risk. The basic assessment involves collection of field data at the project site and a variety of analyses typical of geomorphic watershed assessment methods.

Two parameters, entrenchment ratio and entrainment ratio, form the primary basis for evaluating shear stress sensitivity and channel resistance. A scoring system using these two parameters is sufficient for a conclusive classification in most cases.[1] In some cases, the results of the scoring may be inconclusive with respect to distinguishing between the medium and high risk classes. Guidelines are provided for making the distinction in these cases based on the preponderance and weight of evidence from the other analyses performed as part of the basic geomorphic assessment. This method is designed to provide a consistent framework for stream classification.

The classification method includes up to 4 steps, depending on the size of the project and the outcome from the initial steps.

- **Identification of projects greater than 20 acres**: If the proposed project includes more than 20 acres total area, the basic geomorphic assessment is insufficiently detailed, and a more detailed geomorphic assessment is required.
- **Identification of low risk channels**: If the receiving stream meets the criteria for a 'low' risk channel (stated in the Program's HMP Standard), no further stream assessment is required.
- **Basic Geomorphic Assessment**: If the project is smaller than 20 acres total area and the receiving channel does not meet the criteria for 'low' risk, the basic geomorphic assessment can be used to distinguish between medium and high risk channels. Two primary factors are used to make the classification: entrenchment ratio and entrainment ratio. When the two parameters conflict (one suggests a 'medium' risk while one suggests 'high' risk), consideration of secondary factors may still allow classification using basic field data. In rare cases where the classification cannot be resolved using the basic assessment method, the municipal reviewer may require a more detailed assessment.
- **Detailed Assessment**: For projects larger than 20 acres total area, or where the basic analysis does not provide a clear classification, additional analysis is required. As described in Section 6, the analysis may involve methods such as a detailed geomorphic assessment of channel stability and sediment transport and/or numerical modeling of sediment transport processes.

[1] Based on our own evaluation of 20 typical medium and high risk streams in Contra Costa County we have found that the two parameter test leads to a conclusive classification in approximately two thirds of cases.

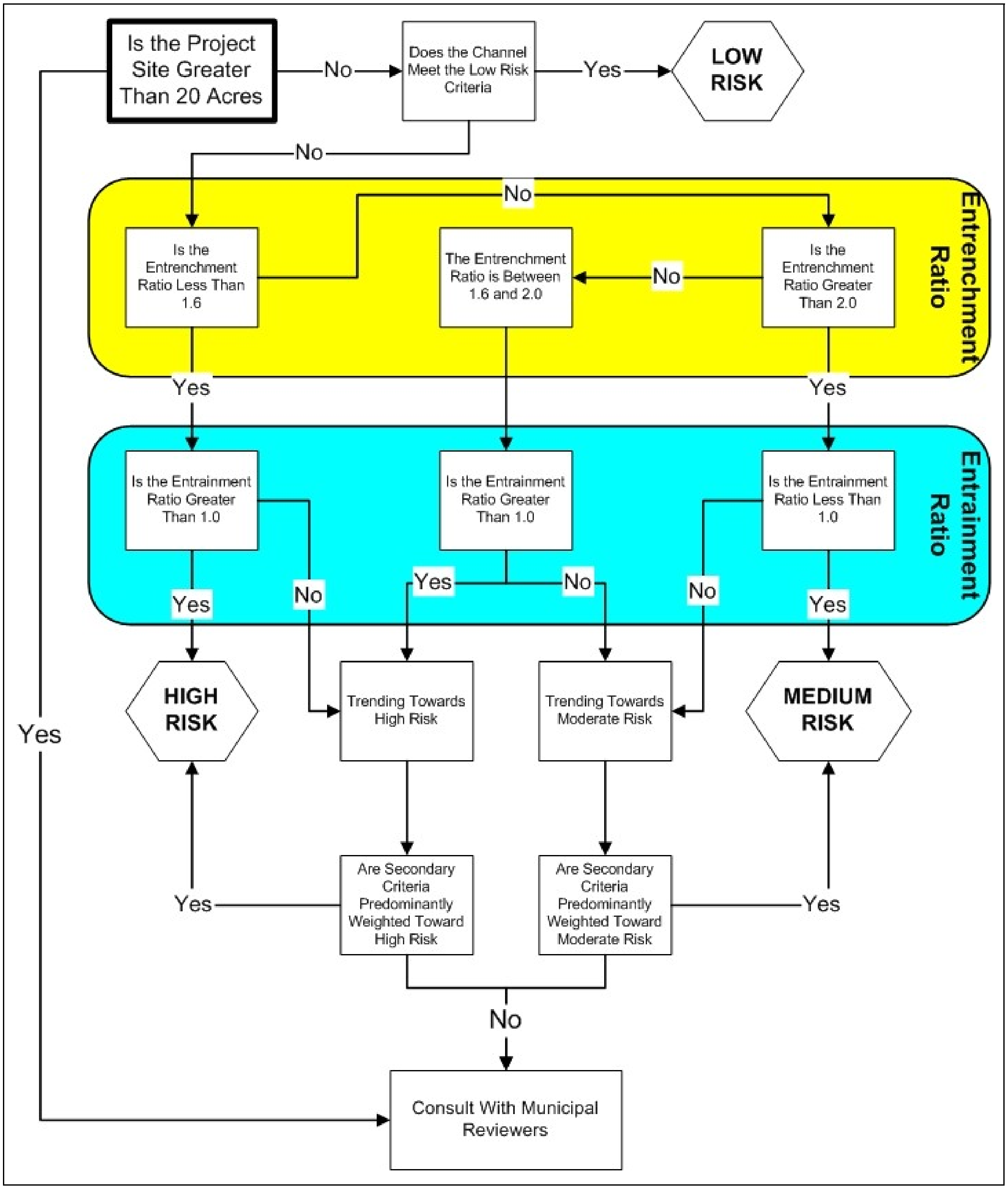


Figure 2. Flowchart of stream classification process

4. LOW RISK CHANNEL CRITERIA

To demonstrate that the low risk criteria are met, the applicant must provide a report or letter report, signed by an engineer or qualified environmental professional, demonstrating that all downstream channels between the project site and the Bay/Delta fall into one of the following "low-risk" categories:

1) Enclosed pipes.

2) Channels with continuous hardened beds and banks engineered to withstand erosive forces and composed of concrete, engineered riprap, sackcrete, gabions, mats, etc. Channels where hardened beds and banks are not engineered continuous installations (i.e., have been installed in response to localized bank failure or erosion) are excluded.

3) Channels subject to tidal action.

4) Aggrading channels (i.e., those consistently subject to the accumulation of sediments).

5. BASIC GEOMORPHIC ASSESSMENT

The basic geomorphic assessment requires a field site review and data collection to characterize the stream in its existing condition. Field data will be recorded onto a form (Appendix A), and used to make calculations during the subsequent analyses. Data fall into two types: primary data used in the classification scoring system, and secondary data that may be used to make a final determination if the primary data are inconclusive.

5.1 LIMITS OF ASSESSMENT

The basic geomorphic assessment involves data collection at selected sites on the receiving channel adjacent to the discharge point(s) from the proposed projects. The assessment reach should extend upstream and downstream far enough to encompass the likely area of influence of potential hydrograph modification. Therefore the channel reach must be a minimum of 500 feet long, and must also meet the upstream and downstream criteria listed below.

Upstream: The assessment should extend upstream to the next permanent grade control (either natural bedrock or a channel bottom-spanning structure such as a box culvert or weir).

Downstream: The assessment should extend downstream to a point where at least one of the following is true:

a) All further downstream reaches are classified as 'low' vulnerability (e.g. the start of hardened flood channel).

b) The channel enters a reservoir managed and maintained by a public entity.

c) The watershed area contributing to the discharge point(s) of the project (may or may not be the same as the project boundary) is less than 5% of the total watershed area contributing to the stream channel.

5.2 DATA COLLECTION SITE SELECTION

Data collection sites should be selected to be representative of typical geomorphic conditions found along the reach. Sites should be located along portions of the reach with relatively uniform width and gradient. Sections of the stream immediately upstream or downstream of steps, culverts, grade controls, tributary junctions, or other features and structures that significantly affect the shape and behavior of the channel should be avoided.

The number of data collection sites required in the assessment reach depends on the size of the channel and the length of the reach, though a minimum of two sites must be sampled. Data should be collected from at least one site every 30 bankfull channel widths. For example, a 30 foot wide channel should be sampled approximately every 1,000 feet. Where the channel character significantly changes more frequently, additional data sites should be included. Surveys on streams with bankfull widths less than 10 feet may be sampled less frequently than 30 bankfull widths.

5.3 PRIMARY CHANNEL DATA

The following section describes the data collection required to complete the Field Data form (Appendix A).

5.3.1 Site Identifier

Each data collection site should have a unique identifier that describes the site. This could include a stream (or project) name followed by a unique site number (For example, Deadhorse Creek - G1).

5.3.2 Map Identifier

A detailed description of the location where the data were collected should be noted on the field form. This information should be sufficiently detailed that reviewers can find the precise site again using a map or Global Positioning System (GPS) receiver.

5.3.3 Site Photographs

Photos provide documentation to generally verify existing conditions and make comparisons with the existing database of classified sites.

Digital photographs should be taken of each data collection site and include, at a minimum, an upstream channel view, a downstream channel view, and at least one landscape overlook. Multiple photos generated from various perspectives are preferred. Photos should include close-ups and annotations as needed to show the bankfull indicators used in the analysis.

5.3.4 General Notes

The Notes section should be used to provide a general written description of the data collection site. Observations regarding unusual conditions, unique situations, or other notable information about the site should also be included.

5.3.5 Bankfull Depth

Purpose: Bankfull depth is used to calculate floodprone width (below), which is used in the entrenchment ratio calculation. It is also used to calculate entrainment ratio.

Method: Bankfull depth should be measured vertically from the channel bottom to the bankfull indicators on the bank (Figure 3). Bankfull indicators normally show the transition between erosion and deposition processes in a channel, and typically include: exposed tree roots on the bank, the top of a point bar formed within or adjacent to the channel, and the top of a terrace formed within an entrenched channel. As a rule of thumb, bankfull is likely to be found in the lower 3 feet of small channels in Contra Costa (narrower than 30 feet). If the channel bottom has a lot of depth variation (e.g. pools or riffles), then this measure should be taken along the midpoint in the downstream riffle transition. Alternatively, measurements can be taken systematically along the riffle crest and averaged for a given site.

5.3.6 Bankfull Width

Purpose: The bankfull width is used to calculate floodprone width.

Method: Bankfull width measurements should be taken at the same location as the bankfull depths. It should be measured horizontally between the top of the bankfull indicators on each side of the channel (Figure 3). When in doubt, use the indicator that would result in a smaller channel width.

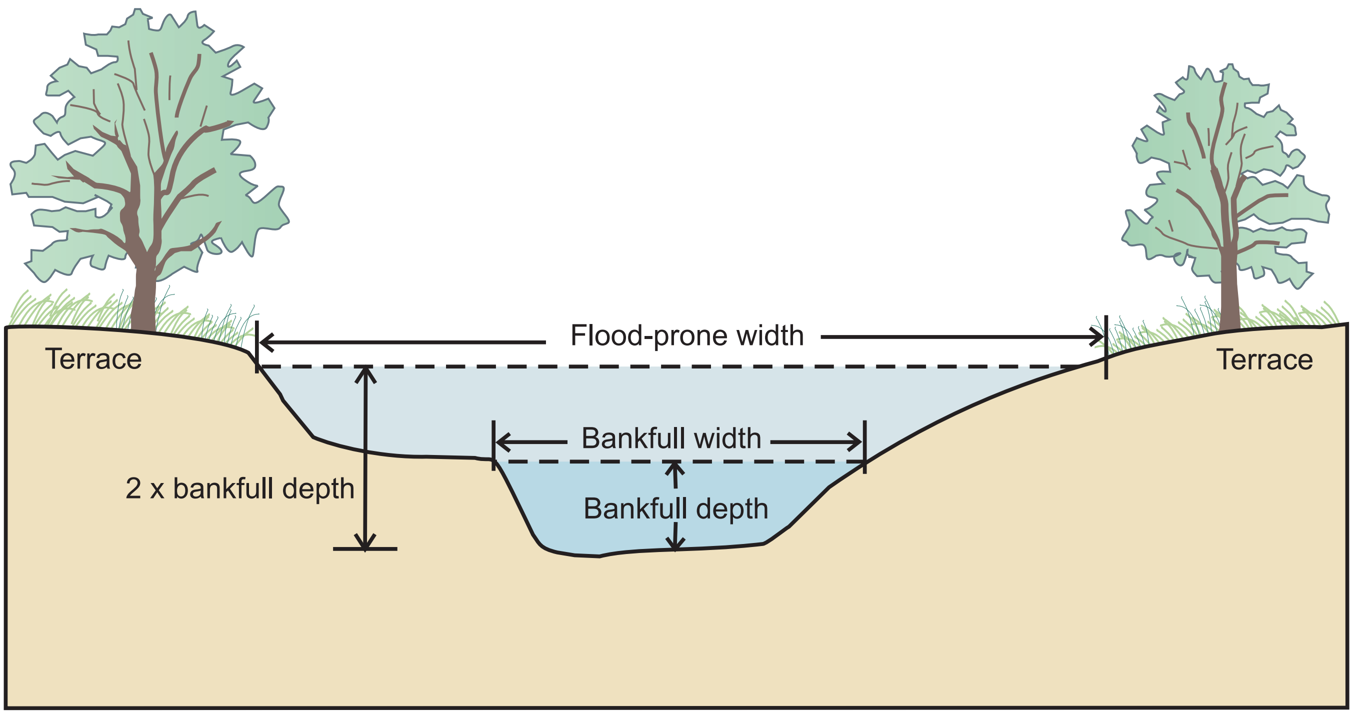


figure 3

Bank Full Measurements

5.3.7 Floodprone Width

Purpose: Floodprone width is used to calculate the entrenchment ratio.

Method: Floodprone width is the width of the valley bottom measured at an elevation that is twice the bankfull depth (Figure 3).

5.3.8 Entrenchment Ratio (after Rosgen)

Purpose: Entrenchment is used an indicator of a channel's shear stress sensitivity.

Method: Entrenchment ratio is calculated using the approach developed by Rosgen. The calculated floodprone width is divided by the measured bankfull width to obtain the entrenchment ratio. Note: the lower the ratio, the more entrenched the system.

LOW entrenchment ratio for values of 1.4 or less

MEDIUM entrenchment ratio for values of 1.41 to 1.6

HIGH entrenchment ratio for values greater than 1.6

5.3.9 Bed Sediment Materials

Purpose: The dominant bed particle size provides an indication of the relative resistance to erosion.

Method: The dominant bed sediment size classes are estimated in the field. The bed material should be described according to the most appropriate of the following classes: bedrock, boulder (> 256 mm), cobble (64-256 mm), coarse gravel (16-64 mm), pebbles (2-16 mm), sand (0.125 – 2 mm), silt (gritty between fingers) or clay (smooth between fingers). In many cases, two dominant grain sizes of bed sediment exist (e.g. silty-gravel), in which case, record both the dominant (most prevalent) and sub-dominant (2nd-most prevalent) sizes. Materials should be estimated based on the overall character of the channel. Sampling should avoid the bottom of pools (which are too fine) or the tops of gravel bars (too coarse).

5.3.10 Channel Gradient

Purpose: The channel gradient is used in the entrainment ratio calculation to evaluate channel resistance.

Method: Channel gradient is calculated from measured differences in the elevation of the water surface (not the channel bed) over a measured length of channel. Five to seven measurements of elevation/channel length should be taken over a cumulative length of at least 200 feet. In dry streams, the channel elevation can be measured provided measurements are taken at appropriate locations in the channel bed.

5.3.11 Entrainment Ratio

Purpose: Entrainment ratio is the ratio between shear stress and critical shear stress, and represents the channel erosion potential.

Method: First, estimate the critical grain size diameter (Dc), in millimeters, that will be entrained by bankfull flows using the equation:

$$D_c = 4.18 * \text{gradient} * \text{bankfull depth (in feet)}$$

Then compare D_c with the bed sediment materials classified above and calculate the ratio of critical grain diameter to actual bed grain diameter using the equation below. Grain Sizes used in the denominator are: silt and finer (0.001 mm); sand (0.125 mm); pebbles (2.00 mm); gravel (16.0 mm); boulders (256 mm); bedrock (999 mm).

$$\text{Entrainment Ratio} = (D_c)/\text{Grain Size}$$

The threshold for Medium is values less than 0.5

The threshold for High is values greater than 0.5

This method is conservative since is compares the size class rather than the average particle size, and includes a factor of safety of 2 relative to the bankfull entrainment threshold (entrainment ratio of 1.0).

5.4 SECONDARY CHANNEL DATA

5.4.1 Confinement Class

Purpose: Confinement is a measure of the amount of room that exists for the channel to actively move laterally. It can be a useful indicator of a channel's vulnerability to erosion.

Method: Confinement classes are assigned based on the width of the valley bottom as a function of channel width, as follows:

WELL CONFINED (WC) channels are in valleys that are less than 2 channel widths wide. They usually have very limited meandering potential.

MODERATELY CONFINED (MC) channels are in valleys that are 2-4 channel widths wide. They tend to have minor amounts of meandering, which commonly results in bank erosion that oscillates from side to side.

UNCONFINED (UC) channels are in valleys with large floodplains, and exceed 4 channel widths wide. Unconfined channels are generally less influenced by hillslope processes and local sediment supply.

5.4.2 Bank Materials

Purpose: The bank materials reflect local sediment sources and also provide an indication of the banks' resistance to erosion by the channel.

Method: The dominant bank sediment size classes should be assigned in the field based on the same classes as are described under Bed Materials in Section 4.4.9. Materials should be estimated based on the overall character of the lower portion of the adjacent banks, at about the elevation of bankfull flows.

5.4.3 Bank Vegetation Classification

Purpose: Bank vegetation classes provide an indication of the resistance to lateral erosion. More vegetated streams are less likely to erode.

Method: A bank vegetation classification assignment should be made in the field based on several classes:

BARE: Unvegetated.

GRASSLAND: Consists of grasses that extend from upslope areas to the channel. Occasional trees may exist in isolated locations, but do not form a closed canopy.

SHRUBS: A riparian community dominated by shrubs with a low, dense canopy, often with limited human access.

OPEN FOREST: A stand of open trees with sparse understory vegetation.

DENSE RIPARIAN FOREST: A dense stand of riparian trees with closed canopy and extensive understory vegetation.

5.4.4 Active Bank Erosion Classification

Purpose: Current bank erosion is an indicator of high vulnerability to future erosion, and large amounts of erosion indicate channel instability.

Method: Indications of active bank erosion include exposed, bare, or unvegetated soils, recently scoured gullies, areas of adjacent slumps or slides along the banks, or freshly scoured banks. Assessment of active erosion should include observations of all banks adjacent to the channel. Classes for active erosion include:

LOW: No evidence of active erosion; well vegetated banks; no oversteepened slopes along channel.

MODERATE: Patches of exposed soils or small, localized bank slumps or slides; limited areas of recent bank erosion

HIGH: Extensive areas of exposed and unvegetated banks; evidence of recent, chronic sediment input from bank failures; evidence of active channel incision.

5.4.5 Active Sedimentation Classification

Purpose: An active sedimentation classification provides an approximate estimate of the amount of sediment supplied to the channel. Active sedimentation can be an indication of instability in the channel.

Method: An evaluation should be made of active sedimentation features such as channel deposits, multiple channels, mid channel bars, as well as sediment sources adjacent to the channel, such as bare or unvegetated soils and actively widening banks. Assessment of active sedimentation should include observations of the channel bed, and all banks adjacent to the channel, as well as other proximate sediment sources. Classes for active sedimentation include:

LOW: no evidence of active sedimentation. Adjacent hillslopes are well vegetated. No deposits of sediment in or near the channel. No obvious sources of sediment from either hillslope or upstream sources.

MODERATE: patches of exposed soils or small, localized slumps, gullies or slides that delivered sediment to the channel.

HIGH: extensive evidence of sediment delivered to the channel. Indications may include multiple channel threads, poorly defined channel margins, or sediment deposits that have not been re-worked by the stream into sand or gravel bars. Evidence of recent, chronic sediment from bank failures. Extensive or large hillslope gullies that deliver sediment to the stream.

5.4.6 Width:Depth Ratio

Purpose: Width:depth ratio is a measure of how concentrated flows are, and gives an indication of shear stress sensitivity.

Method: The measured bankfull width is divided by the measured bankfull depth to obtain the width:depth ratio.

LOW width to depth ratios < 12

HIGH width to depth ratios >12

6. CALCULATING CHANNEL RISK CLASS

Channel risk class is initially calculated using two metrics: the channel entrenchment ratio and the entrainment ratio. If the channel entrenchment ratio is less than 1.6, the metric supports a classification as ‘high’ risk. If the ratio is greater than 1.6, the metric supports a classification of ‘medium’ risk. Where the entrainment ratio is less than 0.5 the metric supports a classification as ‘medium’ risk. Where the entrainment ratio is more than 0.5 the metric supports a classification as ‘high’ risk.

Where both metrics indicate a classification of ‘medium’ or ‘high’ risk, no further analysis is needed. Where one metric supports a ‘medium’ risk and the other a ‘high’ risk class, the secondary data are used to see if there is a preponderance of evidence for either risk class. The table below shows how the primary and secondary data should be used to arrive at a conclusion. If there is still no agreement, additional more detailed analyses may be required.

	Medium Risk	**High Risk**
	Primary Criteria	
Entrenchment Ratio	> 1.6	< 1.6
Entrainment Ratio	< 1.0	> 1.0
	Secondary Criteria	
Confinement Class	UC	WC or MC
Active Bank Erosion Class	Low	Moderate or High
Active Sedimentation Class	varies	varies
Width to Depth Ratio	> 12	< 12
Schumm State Class	1, 5 & 6	2, 3 & 4

6.1 COMPARISON TO STREAM STABILITY HANDBOOK

The stream stability handbook will be incorporated into the Contra Costa Clean Water Program’s Stormwater C.3 Guidebook. It provides detailed examples of ‘low’, ‘medium’ and ‘high’ risk streams throughout Contra Costa County including photos, maps, and site data as required for the geomorphic assessment (examples in Appendix B).

7. DETAILED GEOMORPHIC AND HYDROLOGIC ASSESSMENT

The detailed geomorphic assessment will be performed by a qualified fluvial geomorphologist for projects in one of three situations:

1. *For projects larger than 20 acres that do not meet the low risk criteria*.
   Developments larger than 20 acres potentially have a higher and more widespread impact than smaller developments, and as such both require and can support more detailed analyses than smaller developments.

2. *Where applicants dispute a classification derived from the Basic Geomorphic Assessment and seek a more detailed method of refining or validating the classification*.
   To allow the Basic Geomorphic Assessment to be easily employed it has a degree of conservatism built into it. More detailed and rigorous analyses may reduce the need for conservatism, potentially allowing channels to be classified at a lower risk level, albeit at the cost of more expensive analysis to the applicant. For example, a detailed sediment study may show that a channel initially classified as 'medium' risk is aggrading, and so should be classified as 'low' risk.

3. *To identify the mitigation approach to be taken for channels that are ranked as 'high' risk*.
   The Detailed Geomorphic Assessment will be used to develop specific instream mitigation strategies and designs for channels that are ranked as 'high' risk.

It is undesirable to be over-mechanistic in setting up the methods to be used in the Detailed Geomorphic Assessment, since different project types, receiving channels and geographic locations will demand different investigative approaches. However, the following framework can be tailored to most situations.

- Site historic conditions
- Existing site geomorphic conditions
- Project impacts on site hydrology and sediment supply
- Predicted impact of receiving channels
- Mitigation or avoidance of predicted impact

7.1 PROJECT ELEMENTS IN THE DETAILED GEOMORPHIC ASSESSMENT

The following is a set of methods that may be utilized in the detailed assessment, depending on project size, site and characteristics. It is not intended to be exhaustive, but to offer suggestions for methods the authors have found effective in the past.

7.1.1 Site Historic Conditions

Purpose: The historic assessment gives an indication of how the channel is likely to evolve and respond to changes in the watershed, and provides context for rates of change (e.g. how prone was the site to lateral and vertical channel erosion prior to development, how has it responded to watershed disturbance in the past?) The emphasis should be on identifying why the channel is in its current condition, whether the processes that shaped the channel in the past persist or are likely to change in future, and to gain an understanding of the dynamism and sensitivity of the channel to changes in the watershed.

Methods: Historic assessments can utilize historic maps, aerial and oblique photos, channel cross sections and thalweg profiles, eyewitness accounts and other sources to develop an understanding of the site. Historic maps and aerial photos can be overlain in GIS to show past channel alignments and to calculate rates of movement. Past and present cross sections and log profiles can be overlain to identify rates of channel incision or aggradation, and to see if such trends are ongoing. Where old cross sections exist, new sections can be commissioned. The historic assessment should develop an understanding of the site and channel *trajectory* (e.g. is the channel becoming more or less in equilibrium with its watershed over time, is it widening or deepening, is it likely to avulse?)

7.1.2 Existing site geomorphic conditions

Purpose: The existing site conditions assessment provides a view of the processes that currently shape the site and receiving channel. This is related to the historic assessment (e.g. what processes currently shape the channel?) and also to identifying requirements and opportunities for mitigation (e.g. which stream banks are most likely to erode if peak flows increase, which sections of channel are most likely to incise?)

Methods: The existing conditions assessment should include a site assessment carried out by a geomorphologist or engineer with relevant training and experience. It may involve some or all of the following elements:

- a thalweg long profile to identify channel gradient and knickpoints
- a walk through of the channel, mapping and prioritizing features such as eroding or unstable banks, failing or stable bank protection, knickpoints, bedrock outcrops, grade control structures, areas of high value habitat, sedimentary features, accessible floodplain
- measured and monumented cross sections to assess bank geometry, evaluate stability and identify mitigation sites, and to allow post project monitoring of impacts
- sampling of bed and bank sediment for particle size distribution
- an assessment of the watershed and channel to identify sediment sources, relative sediment abundance, possible disruptions in sediment supply

- photo documentation of site conditions

7.1.3 Project impacts on site hydrology and sediment yield

Purpose: The predicted change in flow peak and duration will be an input into the sediment transport assessment. The impact of the project on sediment supply may affect the receiving channel response. For example, overly effective sediment control on the site could cause a ‘hungry water’ effect, leading to channel erosion.

Methods: The hydrologic impacts will be assessed using continuous hydrologic modeling as outlined in the HSPF Modeling Guidance Memo. Sediment yield may be calculated using a physically-based computer model such as WEPP, or simpler empirical methods where appropriate.

7.1.4 Predicted impact on receiving channels

Purpose: This is one of the key elements of the assessment, and provides an evaluation of the effect that changes in project site hydrology and sediment yield will have on erosion and deposition in the receiving streams.

Methods: Some form of hydraulic and sediment transport assessment is required for this phase. In most developments over 20 acres, we assume that a one dimensional hydraulic model (e.g. HEC-RAS) will be developed, and will be available to assess channel response. There may be cases where a simpler spreadsheet analysis (e.g. Manning’s equation) is appropriate. In larger or more complex cases, full sediment transport modeling should be employed, using one or two dimensional models such as HEC-6, MIKE-11 or MIKE-21. The methods should be used to evaluate the changes in flow frequency, depth, velocity and boundary shear stress that the receiving channel will experience following project implementation. Based on the observed sediment characteristics of the channel bed and banks, applicants should assess the likely change in erosion or deposition rates. In simple cases, this may involve comparing pre- and post-project sediment transport capacity and available sediment characteristics, for example to assess the percentage change in total sediment transport capacity. In larger or more complex settings, the actual quantity and location of eroded sediment may be calculated.

7.1.5 Mitigation or avoidance of predicted impact

Purpose: This element of the assessment will identify the most appropriate and effective management approach to avoid or mitigate for impacts predicted in the earlier phases.

Method: Where the predicted impact of the development on the channel is high, applicants will be encouraged to revisit the site design to lower impacts. When an applicant opts for mitigation, the investigator will work with the public agency (municipality and/or Flood Control District) to develop a design using an approach similar to the science-based stream stabilization approach advocated by the San Francisco Bay Regional Water Quality Control Board. This approach emphasizes designing stable channels and reducing the sources of geomorphic instability as far as possible, rather than hardening channels to be able to resist impacts. The approach involves determining the dimensions of channel (width, depth, gradient, roughness) that would be stable given the inputs of water and sediment from the watershed. We recommend that several methods be used to assess stable channel dimensions, due to the large amount of uncertainty and variance in most methods. Investigators should look for several methods to produce a convergence of evidence for a stable channel design. Typical methods to design a stable channel may include:

- Regional hydraulic geometry relationships (e.g. relationships between discharge and channel width, depth and gradient)
- Sediment transport models to model a stable channel (e.g. SAM)
- Reference reaches (based on channels that have been exposed to similar disturbances and that have regained equilibrium, rather than reaches that are undisturbed)

Where the site constraints do not allow a stable channel to be designed (e.g. the impact of re-engineering the channel alignment may exceed the impact of the development) mitigation may be employed. However, mitigation should seek to bring the system into equilibrium where possible. For example, in incised systems erosion mitigation may involve cutting lowered floodplain benches into the channel side to lower excess shear stress at high flows. Mitigation may also involve reducing erosion utilizing biotechnical bank stabilization methods, which also increase roughness and reduce shear stress.

7.1.6 Monitoring and adaptive management

Purpose: Monitoring provides a measure of the success or failure of the risk assessment scheme and the subsequent mitigation approach. Adaptive management allows practices to be improved over time.

Methods: Data for the existing conditions assessment should be collected in such a way as to allow subsequent resurveys (e.g. establishing permanent cross section monuments, identifying sediment sample locations and photo record points using GPS). The investigator should produce a conceptual model for how the site is evolving, and how the project and mitigation will affect that evolution, leading to statements and metrics of project success that can be verified during project monitoring. For example: ‘the channel banks will be vegetated and stabilized such that where there is less than 100 cubic yards of net erosion through the reach, to be confirmed by resurveying five cross sections on an annual basis’. Where post project monitoring reveals that a channel is not functioning as predicted, the mitigation approach may be reviewed.

Appendices: Attachments

Appendix A
Field Data Sheet

Appendix B
Sample Stream Stability Handbook Sheets

Attachment 4

Site ID:
Map Identifier:

Photo Numbers:

Site Coordinates: **Coordinate Datum:**

Primary Attributes		Confinement Class	
Entrenchment Ratio *Floodprone Width/Bankfull Width*		Active bank erosion	
Entrainment Ratio *(4.18 * Bankfull depth (in feet) * Average Gradient)/Grain Size*		Active sediment supply	
Secondary Attributes		Bed Materials	
Bankfull Width (ft)		Bank Materials	
Bankfull Depth (ft)		Average Gradient	
Width/Depth Ratio		**Field Risk Classification**	
Floodprone Width (ft)			

Bed Materials	Bedrock	Boulder	Cobble	Gravel	Pebble	Sand	Silt	Clay
Size Class *(for Entrainment Ratio Calcs)*	9999	256	64	16	4.0	0.125	0.001	

Site Notes:

Field Justification Notes:

Gradient Data:

Location	***X'*** *Relative Elevation*	***Z'*** *Relative Elevation*	***(ΔZ/ΔX)*** *Local Gradient Calculation*	*Note*

Site 32 – Marsh Creek

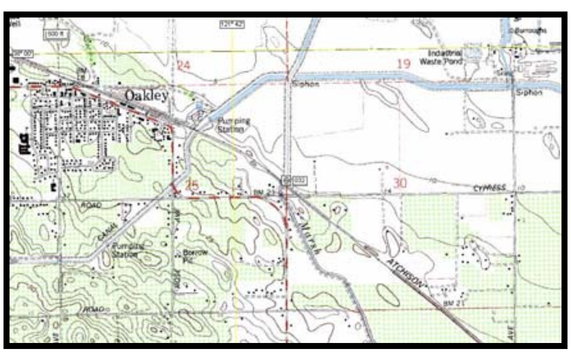
Site Coordinates **614520, 4205694** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**WC (h)**
Entrenchment Ratio	**2.67 (m)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**2.54 (h)**	Active sediment supply	**Low (m)**
Secondary Attributes		Bed Materials	**Clay silt (m)**
Bankfull Width (ft)	**12**	Bank Materials	**silt loam (h)**
Bankfull Depth (ft)	**3.5**	Average Gradient	**0.17%**
Width/Depth Ratio	**3.4 (h)**	**CLASSIFICATION**	**MEDIUM**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes are mixed. Predominantly cohesive clay substrate with patches of silty deposits. Low grade implies incision unlikely. Levees well developed and apparently stable, although not hardened.

SITE NOTES: Trapezoidal channel confined within constructed levees. Well vegetated. Minor internal meandering within levee caused by localized point bars. Terrace formed within levees implies excess capacity within the levee. Simplified channel morphology. Cohesive clay substrate with limited non-cohesive sediment in transit.




AirPhoto USA
Dutch Slough
Oakley
Site 32
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 38°00'22.46" N 121°43'16.02" W elev 7 ft
Streaming |||||||||| 100%
Eye alt 13288 ft


AirPhoto USA
Site 32
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com
Pointer 37°51'58.59" N 121°38'46.39" W elev 39 ft
Streaming |||||||||| 100%
Eye alt 3759 ft

Site 37 – Marsh Creek

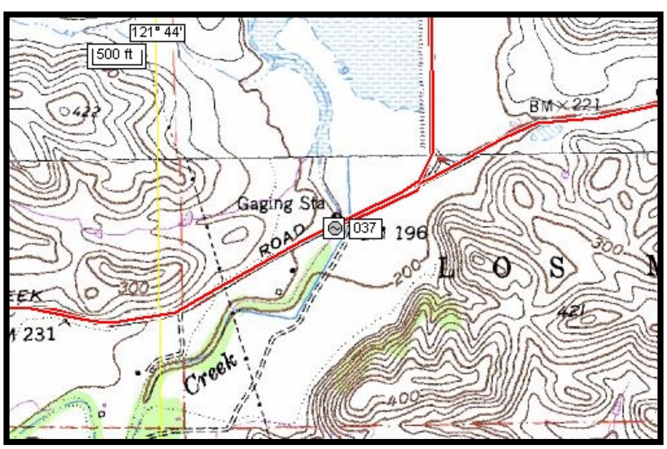
Site Coordinates **611946, 4192484** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**MC (m)**
Entrenchment Ratio	**3.29 (m)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**0.00293 (m)**	Active sediment supply	**Moderate (m)**
Secondary Attributes		Bed Materials	**Gravel (m)**
Bankfull Width (ft)	**24**	Bank Materials	**gravel & silts (m)**
Bankfull Depth (ft)	**1.6**	Average Gradient	**0.70%**
Width/Depth Ratio	**15 (m)**	**CLASSIFICATION**	**MEDIUM**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes are moderate. Gravel dominated substrate implies limited entrainment potential. Relatively unconfined channel with will vegetated point bars. No evidence of recent incision. Wide channel indicates small increase in shear stress during large flows. All secondary criteria indicate a Medium risk.

SITE NOTES: Pool-riffle channel with well vegetated bars and banks. Well graded gravels in riffles, presumably from upstream supply. Silty/mucky in pools. Localized bank erosion at outer bends consistent with normal channel migration processes and rates.




Briones Valley
Site 37
Image © 2004 AirPhotoUSA
©2004 keyhole.com


Site 37
Image © 2004 AirPhotoUSA
©2004 keyhole.com

Site 38 – Marsh Creek

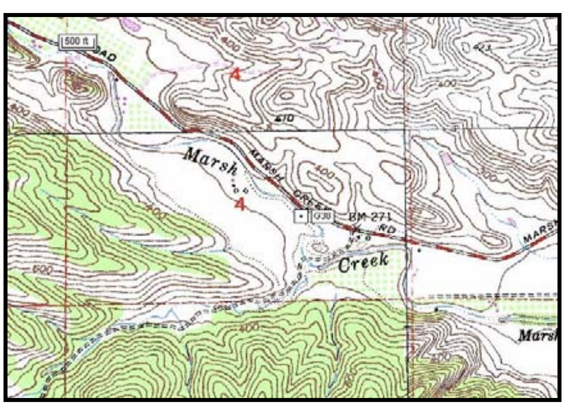
Site Coordinates **609263, 4192377** Site Datum: **UTM WGS 1984**

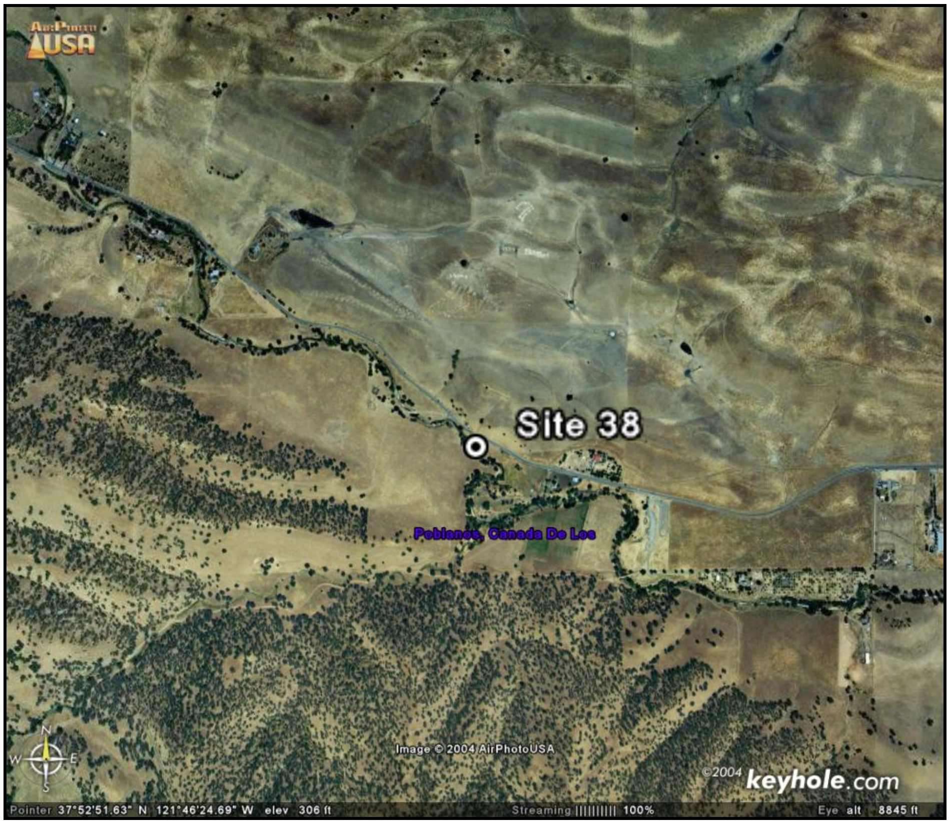
Primary Attributes		Confinement Class	**MC (h)**
Entrenchment Ratio	**1.9 (M)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**0.00041 (M)**	Active sediment supply	**Mod (m)**
Secondary Attributes		Bed Materials	**Cobble**
Bankfull Width (ft)	**22.5**	Bank Materials	**Silt**
Bankfull Depth (ft)	**1**	Average Gradient	**0.62%**
Width/Depth Ratio	**22.5 (m)**	**CLASSIFICATION**	**MEDIUM**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes both support ‘medium’ classification. Coarse substrate combined with limited evidence of erosion under existing conditions (a notable exception occurring at an upstream paved ford); moderate supply of sediment that will help maintain an alluvial mantle and prevent incision; high width/depth ratio, indicating the ability to distribute shear stress during large flows; large roughness elements that will retard stream power.

SITE NOTES: Locally steep eroded bluffs providing a moderate amount of sediment supply. Banks mostly stable and well vegetated. Occasional boulders and bedrock blocks in channel. Channel incised about 9 feet into valley floor.




Site 38
Poblanos, Canada De Los
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°52'51.63" N 121°46'24.69" W elev 306 ft
Streaming 100%
Eye alt 8845 ft


Site 38
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com
Pointer 37°51'33.36" N 121°45'53.39" W elev 984 ft
Streaming 100%
Eye alt 4440 ft

Site 43 – Marsh Creek

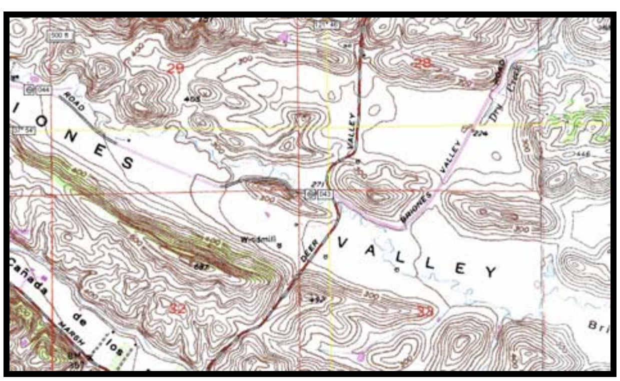
Site Coordinates **608316, 4194998** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**WC (h)**
Entrenchment Ratio	**1.06 (h)**	Active bank erosion	**High (h)**
Entrainment Ratio	**5.24 (h)**	Active sediment supply	**Moderate (m)**
Secondary Attributes		Bed Materials	**clay silt (h)**
Bankfull Width (ft)	**9**	Bank Materials	**silty loam (h)**
Bankfull Depth (ft)	**0.5**	Average Gradient	**2.51%**
Width/Depth Ratio	**18.0 (m)**	**CLASSIFICATION**	**HIGH**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes are high. Channel clearly incised under current land-use. Secondary criteria also lean strongly to High risk. Incision promotes continued erosion, largely due to fine sediment and lack of functional floodplain. High increases in shear stress during peak flows.

SITE NOTES: Plane-bed channel incised about 5 feet into historic floodplain. Small incipient floodplain forming within channel banks, primarily formed from bank colluvium and downstream transport. Minimal meandering but variable active channel width, primarily due to bank slumping.




Site 43
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°54'08.32" N 121°46'26.85" W elev 319 ft
Streaming |||||||||| 100%
Eye alt 5223 ft


Site 43
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com
Pointer 37°53'17.25" N 121°46'25.45" W elev 394 ft
Streaming |||||||||| 100%
Eye alt 2333 ft

Site 56 – Pinole Creek Tributary

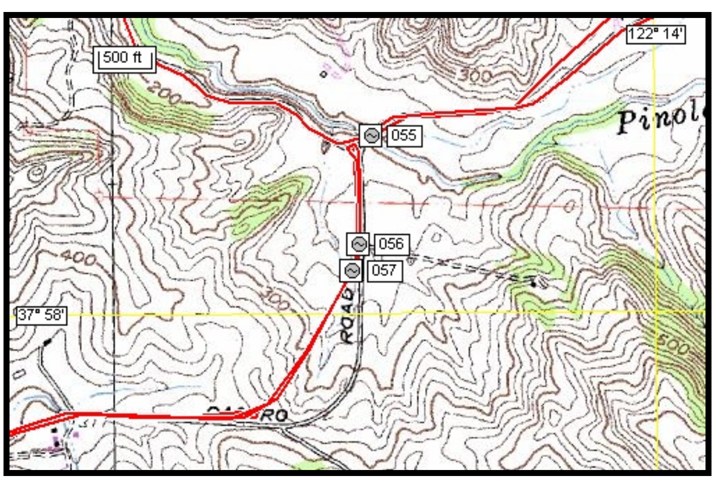
Site Coordinates **566485, 4202587** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**WC (h)**
Entrenchment Ratio	**Varies (h)**	Active bank erosion	**High (h)**
Entrainment Ratio	**2.7 (h)**	Active sediment supply	**High (h)**
Secondary Attributes		Bed Materials	**Silt/Sand**
Bankfull Width (ft)	**2**	Bank Materials	**silty sand**
Bankfull Depth (ft)	**2.0**	Average Gradient	**4.04%**
Width/Depth Ratio	**1.0 (h)**	**CLASSIFICATION**	**High**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes are high. Channel actively degrading. Incision leading to active bank slumping and instability. Secondary criteria also trending to high risk.

SITE NOTES: Massive degradation of channel due to incision and destabilized bank failures under existing land-uses. Degradation partially due to poorly designed and/or maintained infrastructure.




Site 56
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°58'13.08" N 122°15'20.80" W elev 601 ft
Streaming |||||||||| 100%
Eye alt 7807 ft


Site 54
Site 56
Image © 2004 AirPhotoUSA
Image © 2004 I3
©2004 keyhole.com
Pointer 37°58'25.08" N 122°13'37.45" W elev 327 ft
Streaming |||||||||| 100%
Eye alt 2752 ft

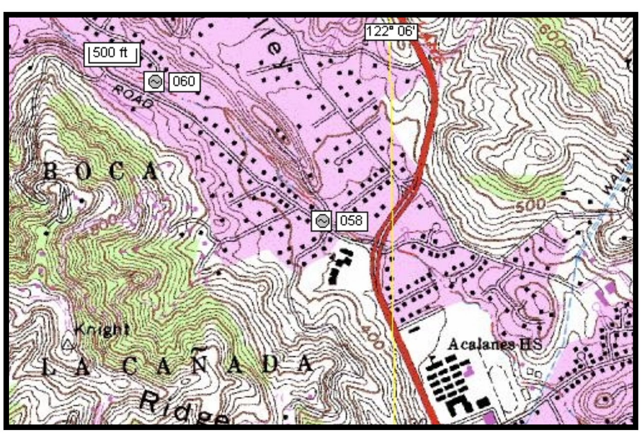
Site 58 – Releiz Creek

Site Coordinates	**578889, 4196097**	Site Datum:	**UTM WGS 1984**
Primary Attributes		Confinement Class	**WC (h)**
Entrenchment Ratio	**1.43 (h)**	Active bank erosion	**High (h)**
Entrainment Ratio	**0.00723 (m)**	Active sediment supply	**Moderate (m)**
Secondary Attributes		Bed Materials	**Gravel (m)**
Bankfull Width (ft)	**9**	Bank Materials	**silt**
Bankfull Depth (ft)	**1.4**	Average Gradient	**1.98%**
Width/Depth Ratio	**6.4 (h)**	**CLASSIFICATION**	**HIGH**

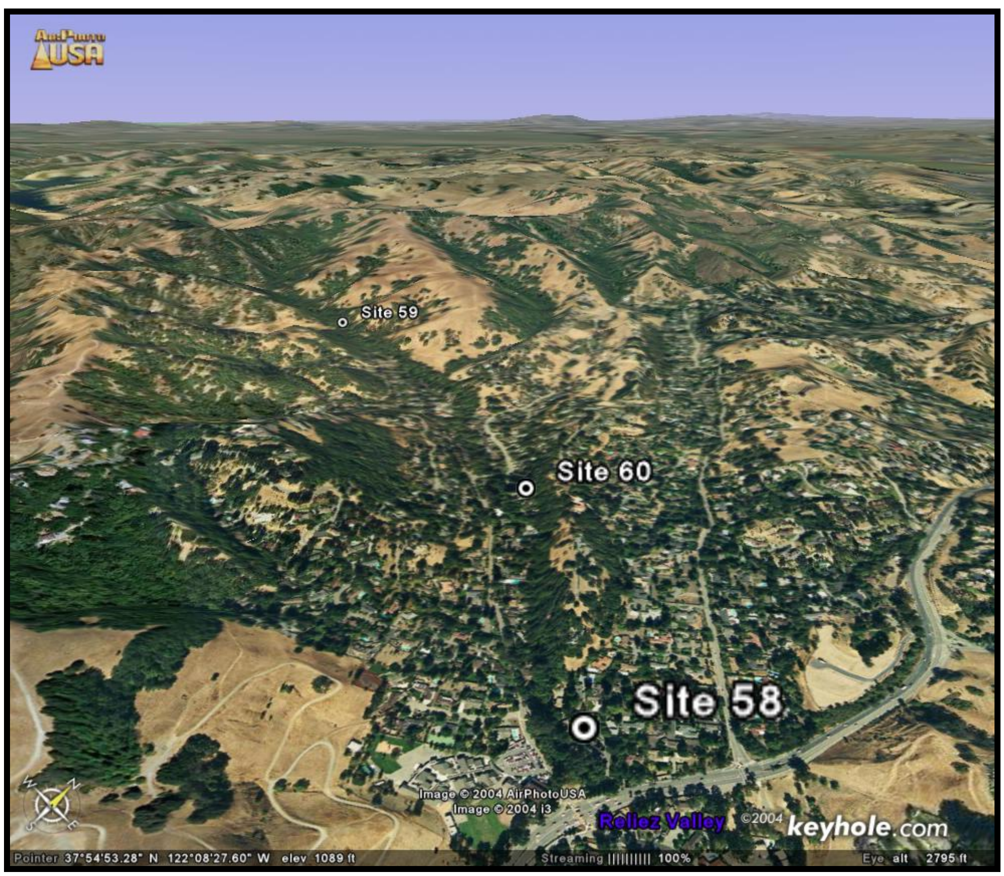
(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary and secondary attributes are mixed. High risk class assignment due to confinement, evidence of historic incision. Coarse bed materials suggest a low risk for incision, although confinement indicates that erosion potential during large storms may be significant.

SITE NOTES: Channel incised about 15-18 feet into a small, confined valley along older sub-division. Road is adjacent to channel. Clear signs of bank instability evident through informal hardening efforts (e.g. brick, old asphalt, log revetments etc.).




Site 60
Site 58
Reliez Valley
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°54'55.47" N 122°06'46.03" W elev 455 ft
Streaming |||||||||| 100%
Eye alt 6039 ft


Site 59
Site 60
Site 58
Image © 2004 AirPhotoUSA
Image © 2004 i3
Reliez Valley
©2004 keyhole.com
Pointer 37°54'53.28" N 122°08'27.60" W elev 1089 ft
Streaming |||||||||| 100%
Eye alt 2795 ft

Site 59 – Upper Releiz Creek

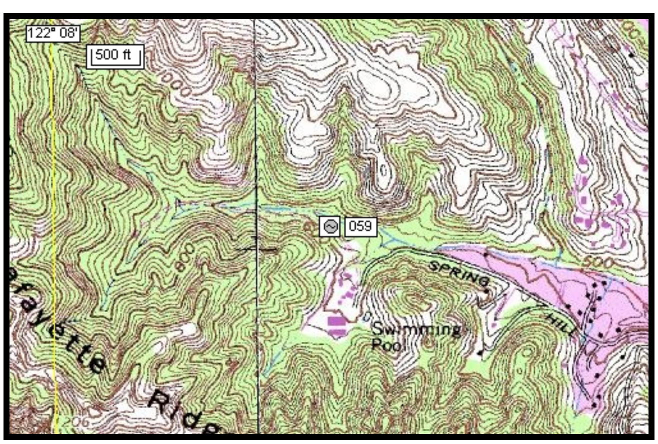
Site Coordinates **577042, 4196995** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**MC (m)**
Entrenchment Ratio	**2.0 (m)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**7.6 (h)**	Active sediment supply	**High (h)**
Secondary Attributes		Bed Materials	**Silt (h)**
Bankfull Width (ft)	**7**	Bank Materials	**Silt (h)**
Bankfull Depth (ft)	**0.6**	Average Gradient	**3.29%**
Width/Depth Ratio	**12.7 (m)**	**CLASSIFICATION**	**HIGH**

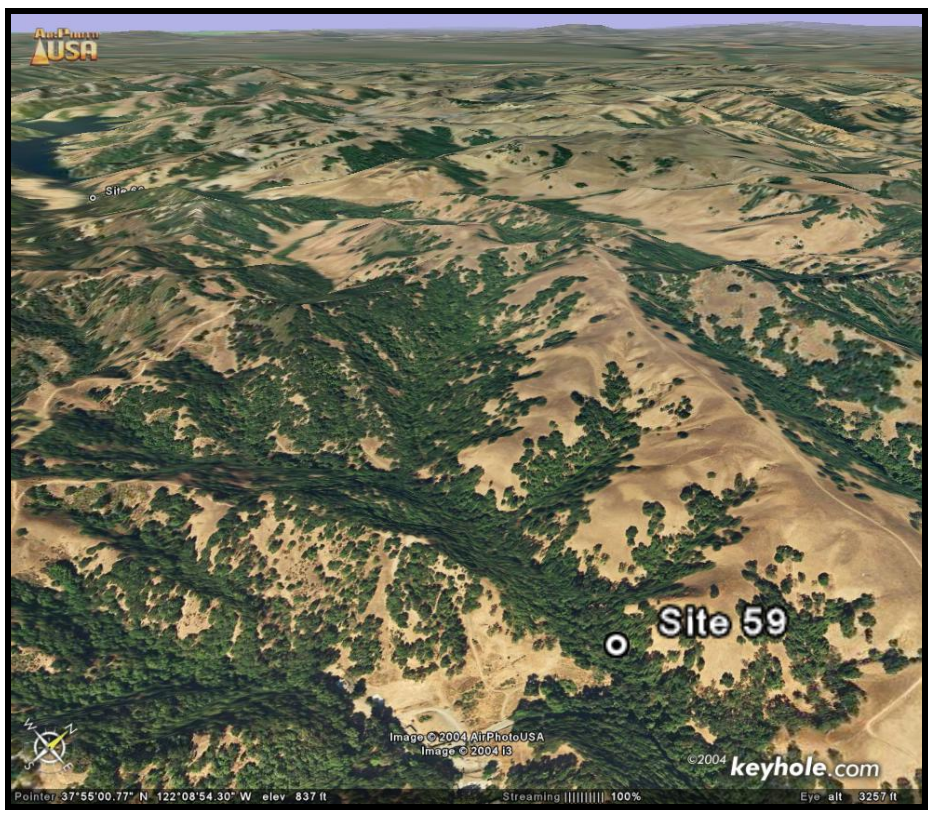
(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary and secondary attributes are mixed. High risk class assignment due to steep grade, fine substrate and potential to destabilize adjacent slopes, leading to large increases in downstream sediment supply.

SITE NOTES: Steep channel in relatively undisturbed park environment with well developed forest canopy providing good root strength to support banks. Person in photo (left) standing on terrace likely formed from incising into landslide deposits. Small inset floodplain and localized bars formed. Substrate dominated by fines. Hillslope failures provide sediment and control local valley morphology.




Site 59
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°55'24.78" N 122°08'34.63" W elev 1027 ft
Streaming |||||||||| 100%
Eye alt 8494 ft


Site 59
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com
Pointer 37°55'00.77" N 122°08'54.30" W elev 837 ft
Streaming |||||||||| 100%
Eye alt 3257 ft

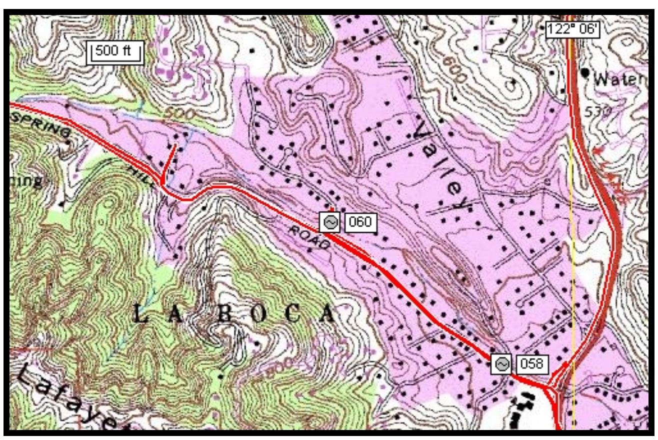
Site 60 –Releiz Creek

Site Coordinates	**578351, 4196542**	Site Datum:	**UTM WGS 1984**
Primary Attributes		Confinement Class	**WC (h)**
Entrenchment Ratio	**1.9 (m)**	Active bank erosion	**Moderate (m)**
Entrainment Ratio	**0.00013 (m)**	Active sediment supply	**Moderate (m)**
Secondary Attributes		Bed Materials	**gravels & silt (m)**
Bankfull Width (ft)	**9**	Bank Materials	**gravel & silts (m)**
Bankfull Depth (ft)	**1.3**	Average Gradient	**0.04%**
Width/Depth Ratio	**7.4 (h)**	**CLASSIFICATION**	**MEDIUM**

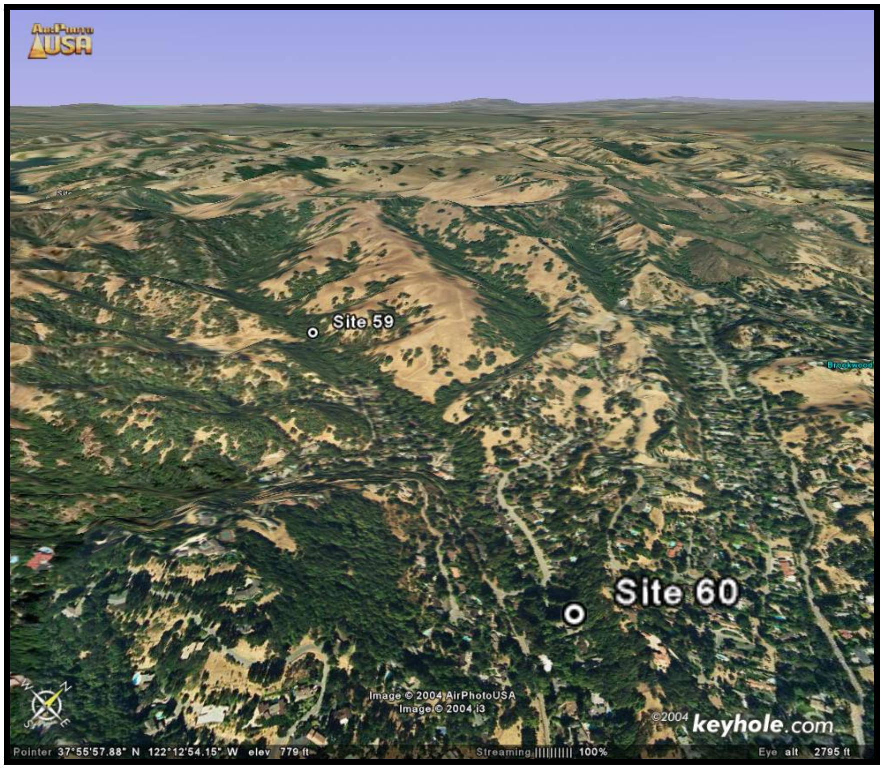
(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary criteria are moderate. Reaching apparent stability with surrounding sub-urban land use. Coarse substrate limits further incision. Deposition of alluvium within valley indicates dynamic erosion and deposition processes are active, an indication of dynamic equilibrium. Existing land-use largely developed.

SITE NOTES: Confined channel within sub-urban land use area. Valley walls appear stable and well vegetated. Gravels and fines are mixed in substrate. Simple channel with small pools and debris-formed steps. Minor localized incision in confined sections.




Site 60
Site 58
Image © 2004 AirPhotoUSA
keyhole.com


Site 59
Site 60
Image © 2004 AirPhotoUSA
keyhole.com

Site 62 – Pinole Ck: Amber Swartz Park

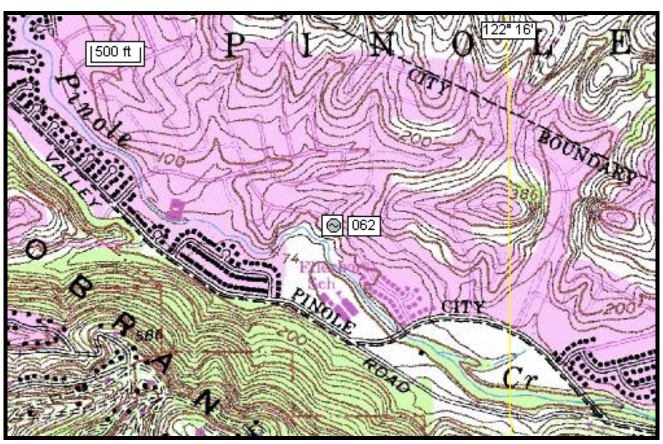
Site Coordinates **563851, 4204298** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**MC (h)**
Entrenchment Ratio	**2.89 (m)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**0.22 (m)**	Active sediment supply	**Moderate (m)**
Secondary Attributes		Bed Materials	**Silty Gravel (m)**
Bankfull Width (ft)	**10**	Bank Materials	**clay silt (m)**
Bankfull Depth (ft)	**2.0**	Average Gradient	**3.34%**
Width/Depth Ratio	**4.9 (h)**	**CLASSIFICATION**	**MEDIUM**

(m) = Medium Criterion; (h) = High Criterion

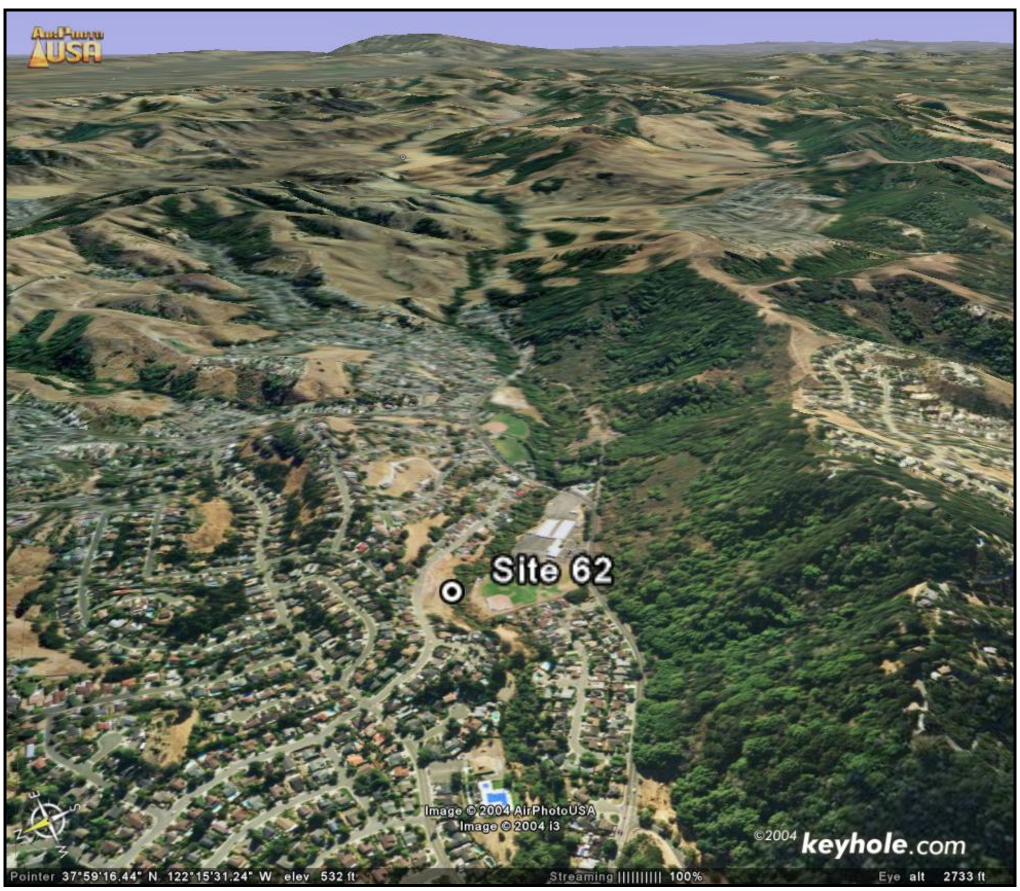
RISK JUSTIFICATION: Primary and most secondary attributes are Medium. Well-developed riparian areas limit coarse sediment supply. Bedrock exposures indicate limited down-cutting potential. However, landslide evidence indicates that continued incision may destabilize local valley walls, which may affect local channel stability.

SITE NOTES: Channel incised about 6-8 feet from former surface as indicated by local terraces adjacent to channel. Large, deep-seated landslide adjacent to site, which may have been triggered from channel incision that destabilized landslide toe. Pool-riffle channel with clay bedrock locally exposed. Well vegetated riparian community along banks and valley walls. Surrounding land use heavily developed into suburban housing.



Attachment 4, Appendix B


Site 62
Sobrante Reservoir
El Sobrante
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°59'22.50" N 122°17'44.29" W elev 148 ft
Streaming |||||||||| 100%
Eye alt 11393 ft


Site 62
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com
Pointer 37°59'16.44" N 122°15'31.24" W elev 532 ft
Streaming |||||||||| 100%
Eye alt 2733 ft

Site 64 – Pinole Ck

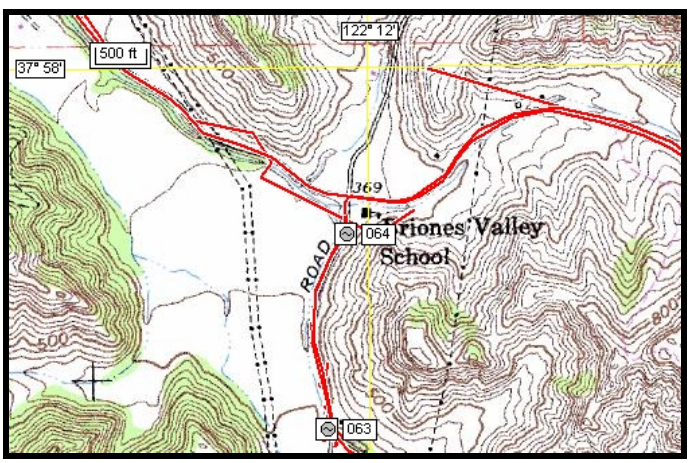
Site Coordinates **570204, 4201919** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**WC (h)**
Entrenchment Ratio	**2.0 (m)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**0.00289 (m)**	Active sediment supply	**Low (m)**
Secondary Attributes		Bed Materials	**Gravel (m)**
Bankfull Width (ft)	**9**	Bank Materials	**silty clay (h)**
Bankfull Depth (ft)	**0.9**	Average Gradient	**1.23%**
Width/Depth Ratio	**10.0 (h)**	**CLASSIFICATION**	**MEDIUM**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary and most secondary attributes are Medium. Well-developed riparian areas prevent local erosion. No evidence of instability in either banks or channel. While channel is confined, it also has a coarse bed and well vegetated and stabilized banks.

SITE NOTES: Stable pool-riffle channel with gravel riffles and well developed riparian corridor. High root strength in banks, moderate sinuosity. Incised slightly into valley floor. Limited inset floodplain development.




Site 64
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°58'11.29" N 122°13'07.80" W elev 303 ft
Streaming |||||||||| 100%
Eye alt 10042 ft


Site 50
Site 66
Site 64
Image © 2004 AirPhotoUSA
Image © 2004 I3
©2004 keyhole.com
Pointer 37°58'32.40" N 122°11'17.58" W elev 630 ft
Streaming |||||||||| 100%
Eye alt 4712 ft

Site 65 – Pinole Ck

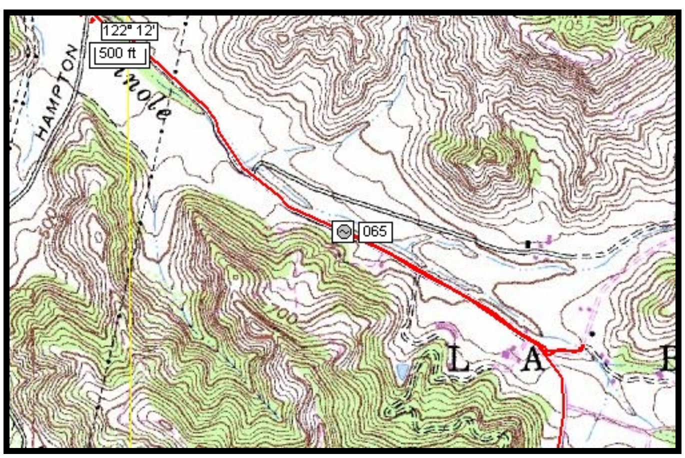
Site Coordinates **570927, 4200653** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**MC (m)**
Entrenchment Ratio	**1.4 (h)**	Active bank erosion	**High (h)**
Entrainment Ratio	**0.00951 (m)**	Active sediment supply	**High (h)**
Secondary Attributes		Bed Materials	**Pea Gravel (h)**
Bankfull Width (ft)	**13**	Bank Materials	**silty clay (h)**
Bankfull Depth (ft)	**1.7**	Average Gradient	**1.07%**
Width/Depth Ratio	**7.6 (h)**	**CLASSIFICATION**	**HIGH**

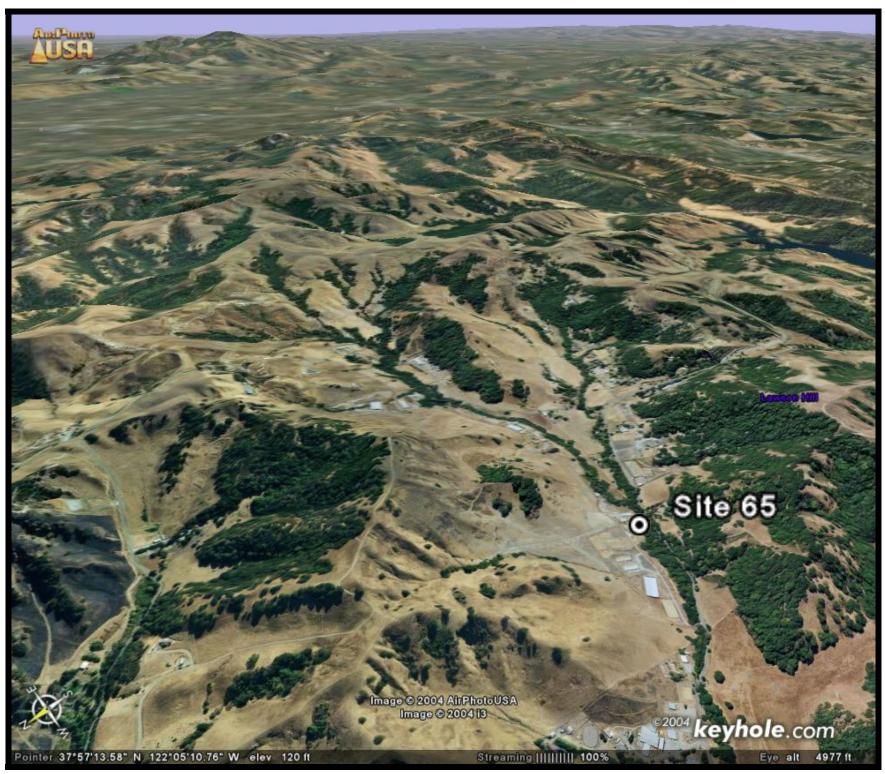
(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary and most secondary attributes are mixed. Sediment supply is high due to local erosion sources, artificially aggrading the bed. Steep valley side slopes and local erosion sources indicate excessive sediment load that, if reduced, could result in long-term degradation of the channel bed.

SITE NOTES: Localized channel aggradation due to large inputs of sediment from failing culverts and road fill failures adjacent to channel. Pool-riffle channel with localized trash and debris buried in channel, offering artificial stabilization features.




Site 65
Lawson Hill
Boy Scout Camp Cove
Image © 2004 AirPhotoUSA
Deep Cove
©2004 keyhole.com
Pointer 37°57'46.74" N 122°13'33.11" W elev 650 ft
Streaming 100%
Eye alt 18224 ft


Site 65
Lawson Hill
Image © 2004 AirPhotoUSA
Image © 2004 I3
©2004 keyhole.com
Pointer 37°57'13.58" N 122°05'10.76" W elev 120 ft
Streaming 100%
Eye alt 4977 ft

Site 74 – Briones Valley Headwaters

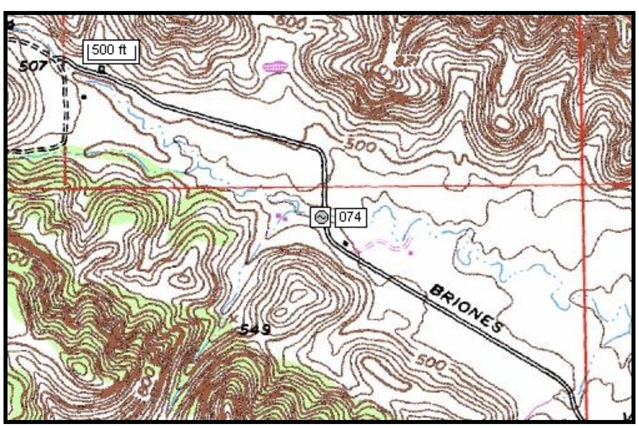
Site Coordinates **604189, 4196462** Site Datum: **UTM WGS 1984**

Primary Attributes		Confinement Class	**WC (H)**
Entrenchment Ratio	**1.06 (H)**	Active bank erosion	**Moderate**
Entrainment Ratio	**1.22 (H)**	Active sediment supply	**Moderate**
Secondary Attributes		Bed Materials	**silty clay (H)**
Bankfull Width (ft)	**9**	Bank Materials	**silty clay (H)**
Bankfull Depth (ft)	**0.5**	Average Gradient	**0.58%**
Width/Depth Ratio	**18.0 (M)**	**CLASSIFICATION**	**HIGH**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes are High. Fine substrate combined with evidence of localized bank erosion and significant channel incision under existing conditions; moderate supply of sediment will help maintain an alluvial mantle and prevent incision, although reduction in supply could destabilize the channel; high width/depth ratio may be a result of channel widening associated with cattle-driven sedimentation.

SITE NOTES: Locally steep eroded bluffs providing a moderate amount of sediment supply. Banks mostly stable and well vegetated. Occasional boulders and bedrock blocks in channel. Channel incised about 9 feet into valley floor.




Long Valley
Site 74
Image © 2004 AirPhotoUSA
©2004 keyhole.com
Pointer 37°54'49.28" N 121°49'31.45" W elev 548 ft
Streaming 100%
Eye alt 5661 ft


Long Valley
Site 74
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com
Pointer 37°53'25.21" N 121°51'05.89" W elev 619 ft
Streaming 100%
Eye alt 4829 ft

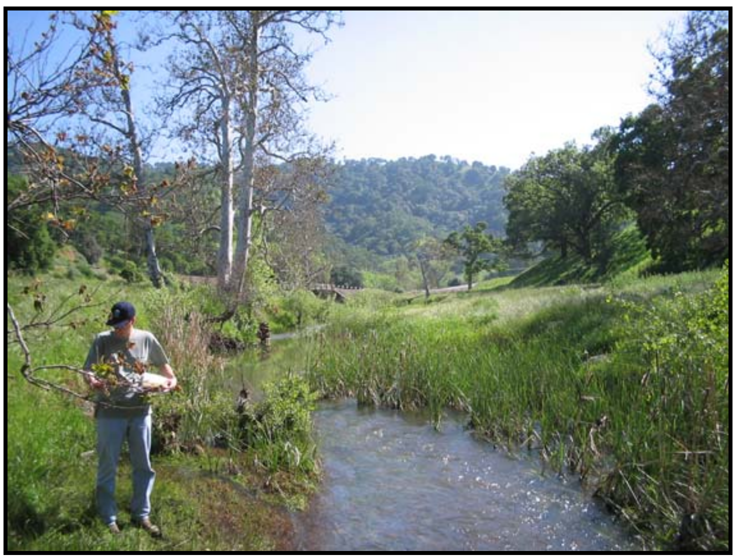
Site 75 – Marsh Creek Headwaters

Site Coordinates **605619, 4194606** Site Datum: **UTM WGS 1984**

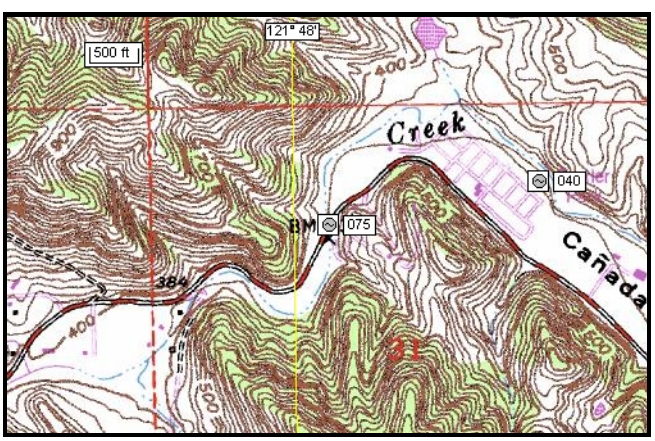
Primary Attributes		Confinement Class	**MC (h)**
Entrenchment Ratio	**1.64 (h)**	Active bank erosion	**Low (m)**
Entrainment Ratio	**0.00708 (m)**	Active sediment supply	**Low (m)**
Secondary Attributes		Bed Materials	**cobble/gravel (m)**
Bankfull Width (ft)	**28**	Bank Materials	**gravel (m)**
Bankfull Depth (ft)	**2.2**	Average Gradient	**1.23%**
Width/Depth Ratio	**12.5 (m)**	**CLASSIFICATION**	**MEDIUM**

(m) = Medium Criterion; (h) = High Criterion

RISK JUSTIFICATION: Primary attributes are mixed. However, most secondary criteria are medium. Coarse substrate limits down-cutting potential. Stable, well-vegetated banks and moderate confinement indicate limited channel widening risk. Low sediment supply and relatively steep gradient indicates limited risk of active sedimentation, suggesting a low risk of channel widening.



SITE NOTES: Pool-riffle channel. Old sycamores rooted in alluvium with approximately 3 feet of very coarse roots exposed, indicating some historic incision. Old inset floodplain now acting as terrace (e.g. no longer exposed to flooding).




Site 75
Image © 2004 AirPhotoUSA
Image © 2004 i3
©2004 keyhole.com


Site 75
Dark Canyon
Long Canyon
Hog Canyon
Image © 2004 AirPhotoUSA
©2004 keyhole.com

Attachment 5

Timeline of HMP Preparation
and Coordination with Water Board Staff

February 19, 2003	Water Board adopts permit, including Provision C.3.f
June 2003	Program adopts budget for C.3 implementation, including preparation on the HMP.
October 2003	Program convenes a Technical Work Group composed of municipal and flood control district engineers to oversee C.3 technical issues, including preparation of the HMP.
October 2003	Program engages consultant to prepare the HMP Work Plan.
October-November, 2003	Program invites two consultants who have been involved in preparing the Santa Clara Valley Urban Runoff Pollution Prevention Program HMP to make presentations to the Technical Work Group.
November 19, 2003	Program Management Committee discusses and endorses a conceptual approach to implement the HMP.
November 24, 2003	At Program staff's invitation, Christine Boschen and Keith Lichten of Water Board staff attend a Technical Work Group meeting, which includes a presentation of the Contra Costa Clean Water Program's conceptual approach to implementing the HMP. The details of the conceptual approach are discussed extensively.
December 8, 2003	Technical Work Group approves an outline for the HMP Work Plan.
December 11, 2003	Christine Boschen email states that Water Board staff will be providing comments on the November 24 presentation.
December 15, 2003	Program staff provides outline of HMP Work Plan to Water Board staff and notes that work is proceeding according to the outline.
January 26, 2004	Technical Work Group begins review of draft HMP Work Plan.
January 28, 2004	Water Board staff letter provides comments on the November 24 presentation.

February 15, 2004	Permit deadline for HMP Work Plan. Program staff obtains Water Board permission to extend the deadline.
February 18, 2004	Program Management Committee approves HMP Work Plan.
February 27, 2004	HMP Work Plan transmitted to Water Board staff with request for approval by March 15, 2004
April 12, 2004	Program receives Water Board staff comments on HMP Work Plan.
April 28, 2004	Tom Dalziel (Program staff) and Dan Cloak (Program consultant) meet with Christine Boschen and Keith Lichten. The Water Board's comment letter is reviewed point-by-point and a resolution of each item is agreed upon.
May 4, 2004	Program's letter to Christine Boschen documents resolution of comments in the Water Board's letter. A revised Work Plan incorporates the changes. Program requests an extension of time to submit the draft HMP; the deadline for the final HMP would remain the same pending timely response from Water Board staff.
May 28, 2004	Program issues RFP for preparation of the HMP. The RFP includes the Work Plan as an attachment.
June 15, 2004	Program holds a clarification meeting for consultants preparing HMP proposals.
June 30, 2004	Program receives four proposals from Consultant teams.
July 1, 2004	At a meeting to discuss C.3 implementation, Water Board staff proposes new, additional requirements for the HMP.
July 8, 2004	Program letter to Water Board Executive Officer Bruce Wolfe notes inconsistent and ill-timed input from Water Board staff may affect the Program's ability to produce an implementable HMP within the Permit deadlines.
July 14, 2004	Contra Costa Council, Contra Costa Clean Water Program, and the Homebuilders Association of Northern California co-sponsor a half-day workshop targeted to developers and decision-makers. The Water Board Executive Officer and Water Board staff make presentations and discuss C.3 with the attendees.

July 20, 2004	Program C.3 Technical Work Group selects the consultant team of Philip Williams and Associates, Brown and Caldwell, and PACE Engineers to prepare the HMP.
August 24, 2004	Consultant team begins work on the HMP.
September 27, 2004	Eight Water Board staff members attend a meeting of the Program's C.3 Technical Work Group. The consultant team presents initial work on the HMP.
November 15, 2004	The Program submits a draft HMP to Water Board staff as required by the Permit.
November 15, 2004	The Program sponsors the first of four half-day workshops on C.3, each of which includes a brief presentation of plans for the forthcoming HMP. Water Board staff (Christine Boschen) provides opening remarks at each of the workshops.
March 14, 2005	At the invitation of Program staff, Water Board staff members (Christine Boschen and Jan O'Hara) attend a meeting of the C.3 Technical Work Group. Santa Clara Valley Urban Runoff Pollution Prevention Program Assistant Program Manager Jill Bicknell also attends. The consultant team provides a detailed presentation of the approach to be implemented in the final HMP and answers questions.
March 17, 2005	Water Board staff forward a 7-page comment letter on the November 15, 2004 draft HMP. In an email, Christine Boschen explains the comments were prepared in December 2004 but not sent until now.
April 21, 2005	At Water Board staff request, Program staff Don Freitas and Tom Dalziel, and Program consultants Dan Cloak and Christie Beeman (Philip Williams and Associates) meet with Water Board staff members Keith Lichten and Jan O'Hara to discuss Water Board staff's March 17 comment letter.
May 15, 2005	Program submits final HMP on the date required in the Permit.

Attachment 6

Water Board Comments on the Program's November 15, 2004 Draft HMP Submittal and Program Responses

<u>Cover Letter</u>

1. This document is an excellent example of the organization you have brought to this effort. The cover letter gives us a clear indication of what the Program has been doing and where you are going with development of your HMP.

   No response required.

2. The concept of adapting the continuous simulation approach for easier use in designing "integrated management practices" looks innovative and worthwhile. We commend the Program for its approach, from the outset, of determining how to make hydromodification control workable at small development sites. We look forward to learning the specifics as you develop them.

   No response required.

3. On page 3, in the first bullet: you note that a determination was made that the Martinez gauge had a long enough record and was able to represent other areas of the County by completing some sort of adjustment. Please submit the technical report that includes the quantitative analysis completed and explains how the record can be adjusted to accurately reflect (for the purposes of hydromodification) the remainder of the County.

   To develop the equations for adjusting IMP sizing factors, the Program's consultant modeled the performance for one IMP using two different sets of hourly rainfall data from two different locations within the County. As described in Attachment 2, the equations adjust for differences in the 2-year storm as well as the 10-year storm, because these differences were found to affect the required IMP size. The Program will examine whether it is necessary or cost-effective to conduct additional modeling to further refine the adjustment equations (e.g. to create adjustment equations for each IMP).

4. On page 3, in the fourth bullet: We concur that the event-based procedure should be abandoned.

   No response required.

5. On page 5, first three paragraphs: This approach—simulation of 50 years of rainfall and runoff flows from a set of pre-designed devices—is interesting and seems reasonable. However, please submit the supporting analysis. (Please see our comments on the Appendices).

The requested analysis is in Final HMP Attachment 2.

6. On page 6, second paragraph: Proposes, "…The applicant may opt to assess the risk of downstream erosion and to develop additional watershed-specific measures." This responsibility cannot only be placed upon the applicant, without the Program first completing a detailed, step-by-step methodology, and ensuring its effectiveness through application to at least one and probably more stream systems/catchments. In essence, for the Program to hand over this potentially complex work to a developer without providing a product that is substantially ready to be easily applied does not ensure that the HMP will be carried through to compliance.

   The detailed, step-by-step methodology in Final HMP Attachment 4 fulfills the permit requirement to provide "a protocol to evaluate potential hydrograph change impacts to downstream watercourses from proposed projects," and the intent (expressed in the permit findings) that the Program should provide an "analysis template" to be used by developers. The methodology is ready to be applied and will be reviewed and refined through the Program's continuous improvement process.

7. On Figure 1, square at lower left "Evaluate risk of contributing to downstream erosion…": Again, this step needs to be clearly developed by the Program, and should be comprised of a step-by-step approach to address both the cumulatively and individually significant impacts that a project may have. The HMP must insure, but does not yet, that it does not allow small increases in erosion from one project here, another project there, because that has the potential to result in cumulatively significant erosion that this section is intended to prevent.

   The flowchart is not included in the final HMP. In the final HMP, the potential for cumulative impacts is addressed by using conservative assumptions throughout the analysis and by selecting conservative standards to be applied to individual projects.

<u>Attachment 1: Hydrograph Modification Assessment Methodology Memo</u>

1. We support the concept that "if all new developed areas drain to appropriately sized IMPs, then the site runoff is considered controlled for peak flow and durations." Please describe how the discharge rate of the IMPs will be estimated. In addition, the long-term inspection and maintenance of the IMPs will be doubly important, and this should be addressed in the final HMP.

   Discharge from IMP underdrains is limited to one-half the pre-project runoff event with a recurrence interval of two years (0.5Q2). Actual discharge from underdrains and from overflows has been modeled using HSPF. Results are shown in Final HMP Attachment 2.

Hydrograph modification IMPs will be subject to the same operation and maintenance requirements as the Program currently requires for treatment IMPs. See the Stormwater C.3 Guidebook, Chapter Six and Appendix F.

2. In the inputs for the Site Design IMP Sizing Worksheet, please explain how the average annual rainfall relates to flow duration control. Also, it is not clear where the maximum allowable release rate for underdrains is accounted for in the Worksheet.

   For IMPs in Type A and B soils, the sizing factor relates most closely to the volume and intensity of larger storms. The volume and intensity of larger storms are co-variant with average annual rainfall. These relationships allow us to use average annual rainfall (which is easily computed for a particular location based on existing isopluvial maps) to be used as a scaling factor for different locations in the County. Sizing factors for IMPs in Type C and D soils are more closely related to the volume and intensity of smaller storms (those that produce pre-project Q2), which are not co-variant with average annual rainfall. To adjust these sizing factors, the spreadsheet uses average annual rainfall to select the most similar rain gauge, then uses the ratio to adjust for differences in rainfall statistics between that gauge and the Martinez gauge.

3. The text clearly states that if IMPs do not account for all runoff, then the user *either* does a Stream Vulnerability Risk Assessment *or* installs flow duration control BMPs. This is not so clear on the flow chart in Figure 1 of the cover letter. Please also clarify that IMP/BMPs must be implemented at the site unless the site discharges to an exempt water body.

   The requirements are clarified in the Program's HMP Policy (Final HMP Attachment 1).

4. On page 1, first paragraph: "…areas up to 20 acres…. Projects larger than 20 acres may be subdivided for this purpose." Why has this 20-acre threshold been chosen? Is this because of stated detention basin limits? Where are the discussion and citations of literature to support this threshold? For example, if basins are the issue, where is the discussion and analysis showing why basins cannot be used/are never effective below 20 acres, and then connecting that idea to the use of this threshold? Why shouldn't this threshold be 1 acre? 5 acres? 50 acres? Why is it that IMPs are effective for areas of up to 20 acres? Please submit a technical analysis supporting this approach and the threshold.

   Separately, what does it mean to "subdivide" a project that is larger than 20 acres down to 20 acres? Is this an area that would go to a single catch basin? A single outfall? A single creek system? While the concept may be sound, the absence of supporting data and analysis for the 20-acre threshold renders it presently unacceptable.

In the final HMP, this threshold no longer applies to the use of IMPs. To use IMPs, applicants are required to divide the project site (regardless of size) into drainage management areas. Typically, the maximum size of a drainage management area will be determined by practical considerations in designing drainage to the IMP.

5. On page 2, first full paragraph (discusses the idea of using off-the-shelf BMP designs to meet hydromod requirements): Parameters must be clearly spelled out and incorporated into conditions of approval, to be satisfied prior to the granting of occupancy permits (or a similarly effective restriction.)

   Contra Costa municipalities require submittal of a Stormwater Control Plan with applications for development approval. The Stormwater C.3 Guidebook will require Stormwater Control Plans that propose IMPs to incorporate the design parameters in Final HMP Attachment 2.

6. Site Design IMP Sizing Worksheet: The worksheet appears to have only two input factors—predevelopment cover, and predevelopment soil type. It does not appear to address other issues that may be significant, and which we have previously identified should be addressed. These may include: slope, travel distance, and changes in drainage pattern. To take the approach of only using its two specified factors of cover and soil type, the Program should complete a technical analysis that appropriately demonstrates that other factors are individually and cumulatively insignificant, or that they are somehow accounted for in the spreadsheet analysis. As a note, here, cumulatively refers to both "other factors, taken together," and "if this method is applied to many small projects, that the changes resulting from not incorporating the factors are cumulatively insignificant."

   Separately, the worksheet must incorporate, and we need to see, the minimum required design parameters for the specified controls. The design parameters should be basic items such as length, width, drainage time, soil porosity, presence/length/design of subdrains, etc. These design parameters must clearly address known significant problems, such as infiltration into tight soils, construction compaction, need to temporarily pond water in a control (e.g., ponding of a few inches in a swale by raising the storm drain inlet), and whether any deviation in designs is allowed, and under what circumstances.

   IMPs are designed and sized so that the outflow matches the pattern of pre-project flows; accordingly, the relevant factors are the factors that change when an undeveloped site is developed. The dominant factor that changes is perviousness, which is characterized by the change from pre-project soil type to impervious surface. (Sensitivity analyses showed estimated pre-project runoff is insensitive to the different vegetative cover types found in Contra Costa County.)

   The sizing factors are conservative (i.e., post-project flows will be lower, most of *the time, than pre-project flows for each drainage management area and each project). This minimizes the potential for cumulative impacts. As part of continuous improvement of the HMP, the Program will model a watershed-scale scenario where IMPs are used on a number of projects in the same watershed.*

Design parameters for IMPs are in Final HMP Attachment 2.

7. On page 5, last paragraph "…the final HMP will provide guidance to developers on the use of…HSPF….": What does "guidance" mean? Our expectation is that developers will be provided an enforceable approach that has easy-to-measure outputs (although the process itself may be complex) enabling municipalities to determine whether a developer's approach/design is acceptable. Developers will be able to view the Program's completed and documented case studies, since those case studies will be completed as a part of the final HMP and included in it.

   The guidance (Final HMP Attachment 3) includes both general requirements for the modeling approach and specification of some input parameters. Co-permittees will require that Stormwater Control Plans for projects using site-specific modeling include documentation of the model construction and of the parameters used.

8. On page 7: "Continuous Simulation Modeling Guidance." – provides list of items that will be included in the final HMP. Again, what does "guidance" mean (see previous comment)? Also, please submit drafts of these well before the due date of the final HMP, since these are "rubber-meets-the-road" sorts of details that can be controversial.

   We regret we were unable to submit drafts "well before" the due date of the final HMP, as we received these Water Board comments only eight weeks before that date.

<u>Attachment 2: Development of IPM Sizing Factors</u>

1. In the second paragraph, we recommend using a "goodness of fit" standard[1], rather than simply stating the post-development curve shall not exceed the pre-development curve.

   The HMP Policy (Final HMP Attachment 1) includes a standard based on a Washington Department of Ecology standard.

2. Would it make sense to include porous pavement and permeable pavers with underground water detention capacity in the worksheet along with the other IMPs?

[1] As described in Appendix F (pp. 5,6) of the Santa Clara Valley Urban Runoff Pollution Prevention Program's, "Hydromodification Management Plan—Public Review Draft, June 2004"

These treatments may be incorporated into "self-retaining areas," which are disconnected from the storm drain system for the purposes of the HMP.

3. A good deal of technical analyses and assumptions have gone into development of this methodology and are not presented in this brief memorandum. For example, the use of Q2 and upper and lower discharge bounds (Q10 and 0.5Q2) must be explained. In general, we would like to have more information about the development of sizing factors.
   The hydrologic models you mention should be explained and fully named—for example, "HSPF" should be spelled out and referenced in the bibliography.

   Technical analyses and assumptions are explained in Final HMP Attachments 1 and 2.

4. Page 1 "[Using the Martinez gauge record]…we will apply adjustment factors to the basic rainfall record to reflect the range of average annual rainfall experienced throughout the county." Please submit the supporting technical analysis demonstrating how this is done and that it appropriately reflects local variations. Such a technical analysis may include comparisons to other existing County rainfall records that incorporate statistical and/or other analyses demonstrating that the adjustments are appropriate and, as applied, will result in meeting the standards listed in the Permit.

   This is addressed in Final HMP Attachment 1. See also the response to a similar comment above.

5. On page 2, Table 1 (cover/soil class factors): What variables are not included (see earlier comment that the Program needs to demonstrate that a two-parameter model is sufficient)? Also, please submit the detailed, County-specific information that was used to develop these numbers (e.g., the impervious surface percentage for each type of density. For example, "low" is a relative descriptor that has different meanings in different local planning documents, so the definitions of these terms must include clear definitions of what land use intensities are related to each stated cover class, in commonly available terms (e.g., DU/ac for a particular development style/type). We share the Program's goal here of trying to have an analytical descriptor that all can be confident appropriately describes ultimate built conditions on the ground.

   In the final HMP, the IMP sizing procedure is not based on land use; rather, the applicant must directly calculate square footage of pervious and impervious surfaces.

   See response to earlier comment regarding the use of soil type as the key parameter needed to compare pre-project to post-project runoff.

6. On page 3, second paragraph: What is the "recommended depth" for each site design IMP? That is, what does "recommended depth" mean? What are the other factors that will be specified (e.g., soil type, relative compaction, side slope min/max, width, etc.)? It is acceptable not to specify a factor, as long as the IMP's effectiveness is not a function of that factor.

   Final HMP Attachment 1 includes the key design parameters for each IMP.

7. Page 3, Sizing Factor Example (1): Please include a reference to the necessary mulch layer at the top of the bioretention example.

   Mulch is recommended but optional, as it is not critical to performance of the bioretention area.

8. Page 3, Sizing Factor Example (2): States "…Assume the vertical transport rate in the planter is high relative to the other terms, so the planter fills from the bottom up during a rainstorm, and that planter soil becomes saturated before any discharge from underdrains occurs." It was not obvious to us that this is a conservative assumption with respect to hydromod and the planter design. Could you explain this assumption further?

   In the final HMP, Appendix A to Attachment 2 contains a detailed discussion of the physics of water movement through soil.

<u>Attachment 3: Stream Classification Methodology</u>

1. From the limited information presented, this appears to not be a precise enough method for exempting water bodies from HMP requirements. It is not evident that cumulative impacts of development on streams, among other things, are considered.

   Where project runoff flows to storm drains, hardened channels, tidally influenced streams, or depositional streams, (i.e., "low risk") we propose to require only "maximum extent practicable" limitations on imperviousness. This is consistent with Permit Provision C.3.f.ii.

   Final HMP Attachment 4 contains a more detailed methodology for distinguishing "medium risk" from "high risk" situations.

2. While three risk classifications are given (high, medium, and low), this memorandum does not state how the classifications are to be applied. We infer that projects discharging to "low risk" streams will be exempt from hydromodification controls. Also, project proponents and municipal reviewers can "use their professional judgment in borderline cases": does this mean medium risk streams are decided on a case-by-case basis? We do not necessarily see the benefit in applying professional judgment on a case-by-case basis, because the

room left open for interpretation does not insure compliance with the Permit. A system that classifies water bodies as either exempt (low risk) or non-exempt would seem to have more merit. Please look back to the permit language, which gave examples of the types of creeks and storm drains where the potential for erosion or other impacts to beneficial uses is minimal, for guidance on exempting creeks.

To implement Provision C.3.f.vi.6, which calls for: "stream buffers and stream restoration activities, including restoration-in-advance of floodplains, revegetation, and use of less-impacting facilities at the point of discharge, etc.," Co-permittees will have to apply professional judgment on a case-by-case basis of the potential for stream erosion as well as professional judgment on a case-by-case basis of the effectiveness of the alternative measures.

These staff suggestions to classify water bodies as either exempt or non-exempt, while also disallowing Co-permittees to use professional judgment in determining equivalent limitation of impacts, would make it effectively impossible to implement Permit Provisions C.3.f.vi.6 and C.3.f.vii.

3. It seems somewhat contradictory that the Program emphasizes simplifying the design of hydromodification controls for individual projects, while leaving the burden of classifying streams to each developer. Will the Program provide an inventory of all hardened channels in the final HMP (or before)? What field work or other data collection will the Program do to identify unstable or eroding streams?

   The Program's policy will encourage applicants to use IMPs to control runoff to pre-project peaks and durations, rather than attempting to establish exemptions. The Program does not plan to conduct field inventories, except as needed to establish guidance for stream classification.

4. On page 3, 1st full paragraph: Mentions "concrete brick" and "gabions." These need to be better defined. For example, do gabions include thinner rock "mattresses," and is there a specified required thickness for them? Does "concrete brick" include sackrete and reinforced concrete erosion control products? Please clearly define what is included in each category. Water Board staff will consider whether it is appropriate to accept increases in erosive flows for creeks lined with such products, but not with straight concrete. To accept these types of hardening as effectively exempt is not something the Water Board has done previously.

   Provision C.3.f.ii. states in part: "Such situations may include discharges into creeks that are concrete-lined or significantly hardened (e.g., with rip-rap, sackcrete, etc.)…"

   .

5. Page 3, 2nd full paragraph: "…we propose developing empirical relationships so that channels that are depositional or very low gradient can also be designated as low-risk after an initial assessment." This approach could be acceptable, as long as this is confirmed in the field and an acceptable analysis of potential changes in boundaries between depositional and transport/erosive reaches is completed. It seems unlikely, at first glance, that we would want to accept this proposal for "low gradient" creeks, since it is not immediately clear that the set of low-gradient creeks and the set of non-erodable creeks intersect, or fully intersect. Depositional creeks and non-erodable creeks seem a much closer fit, at first glance. The analysis should also address potential bank erosion—frequently an issue in trapezoidal flood control channels, even where the creek bottom may not be downcutting (because of bank saturation and/or flow velocities in the creek and associated limited vegetation). Such creeks should be excluded from the "low-risk" category.

   We agree that depositional and non-erodable creeks are a closer fit, as reflected in the final HMP Policy (Attachment 1).

6. Page 3, Section 4.2.2, Medium Risk Channel Definition: To have this category does not appear to be a conservative approach. The Program can propose it, but the proposal should be accompanied by the level of analysis and scientific substantiation needed to demonstrate the protectiveness of the approach.

   Also, the proposal (further down in the same section) to allow municipal reviewers to use their best professional judgment in borderline cases is not acceptable. There needs to be a specifically defined deciding factor created by initial analyses and/or the current more highly-trained and experienced consulting crew. We recognize that there are huge pressures to approve developments, and that there may be concomitantly huge pressures to avoid fully mitigating HMP impacts, where such mitigation may be viewed as slowing a project. To leave this up to the best professional judgment of engineers and planners whose training is largely in other subjects is unacceptable; it does not insure that the HMP will be carried through to compliance with the Permit. In general, the Medium-Risk category should be eliminated, in favor of just two categories (Low/High). If the Medium-Risk category is included, then these analyses should be completed now, for review by Board staff (and subsequent Board review), or a very detailed approach should be prepared and practiced on one or more stream systems, and the results provided as a part of the HMP and for future projects.

   "Medium risk" is defined in final HMP Attachment 1, which also details the options available to an applicant upon a "medium risk" finding. The approach and guidance for evaluating "medium risk" vs. "high risk" is in final HMP Attachment 4.