FINAL

HYDROMODIFICATION MANAGEMENT PLAN

Prepared for
County of San Diego, California
March 2011

Brown AND Caldwell
9665 Chesapeake Drive, Suite 201
San Diego, California 92123

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LIST OF ACRONYMS

ACCCMP	Alameda Countywide Clean Water Program
BAHM	Bay Area Hydrology Model
BEHI	Bank Erosion Hazard Index
BMP	Best Management Practice
CASQA	California Stormwater Quality Association
CCCWP	Contra Costa Clean Water Program
CEM	Channel Evolution Model
CEQA	California Environmental Quality Act
D50	Median grain size diameter
Ep	Erosion potential index
ET	Evapotranspiration
FSURMP	Fairfield-Suisun Urban Runoff Management Program
GIS	Geographical Information System
HEC-HMS	Hydrologic Modeling System; distributed by the US Army Corps of Engineers Hydrologic Engineering Center
HMP	Hydromodification Management Plan
HR	Hydraulic Radius
HSPF	Hydrologic Simulation Program FORTRAN, distributed by USEPA
IMP	Integrated Management Practices
LID	Low Impact Development
LSPC	Loading Simulation Program in C++
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resource Conservation Service
PLS	Pervious Land Surface
PWA	Philip Williams & Associates
Q	Flow
Qcrit	Critical flow
RWQCB	Regional Water Quality Control Board
SCCWRP	Southern California Coastal Water Research Project
SCVURPPP	Santa Clara Valley Urban Runoff Pollution Prevention Program
STOPPP	San Mateo County Stormwater Pollution Prevention Program
SUSMP	Standard Urban Stormwater Mitigation Plan
SWM	Stanford Watershed Model
SWMM	Storm Water Management Model; distributed by USEPA
SWMP	Storm Water Management Plan
SWWM	Storm Water Management Model
TAC	Technical Advisory Committee
TMDL	Total Maximum Daily Load
USACE	United States Army Corps of Engineers
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey

EXECUTIVE SUMMARY

Background

The need to address hydromodification and its influence on water quality is included in the San Diego Regional Water Board Order R9-2007-001, Provision D.1.g of California Regional Water Quality Control Board San Diego Region Order R9-2007-0001, which requires the San Diego Stormwater Copermittees to implement a Hydromodification Management Plan (HMP) "…to manage increases in runoff discharge rates and durations from all Priority Development Projects, where such increased rates and durations are likely to cause increased erosion of channel beds and banks, sediment pollutant generation, or other impacts to beneficial uses and stream habitat due to increased erosive force."

To address this permit condition, the Copermittees, represented by the County of San Diego, hired a consultant team and proceeded with developing an HMP that meets the intent of the Permit Order. The permit requires the Copermittees to develop an HMP for all Priority Development Projects (PDP), with certain exemptions. The HMP must develop standards to control flows within the geomorphically-significant flow range. Supporting analyses must be based on continuous hydrologic simulation modeling.

As required by Permit Order No. R9-2007-0001, each Copermittee shall incorporate the approved HMP into its local Standard Urban Storm Water Mitigation Plan (SUSMP) and implement the HMP for all applicable PDPs by January 14, 2011.

HMP Development Process

All 21 Copermittees participated in the development of the HMP, both financially and through their participation in the Copermittees Hydromodification/SUSMP Workgroup. The Workgroup was convened 14 times over the course of the project at times that corresponded with key decision points in developing the HMP and the update to the SUSMP. The Workgroup reviewed and commented on all drafts of the HMP and SUSMP, as well as reviewed all of the public comments received on these documents and responses to comments.

A key element of the San Diego HMP was the creation and involvement of a Technical Advisory Committee (TAC). The TAC members consisted of respected individuals from academia, technical resource agencies, the development community, consulting engineers, and environmental organizations. The TAC was tasked with providing technical input to the scientific approach and interpretation of results integral to the establishment of numerical flow control standards for the HMP, and met 11 times since October 2007.

Literature Review

Pursuant to Permit Section D.1.g(1)(e), the consultant team conducted a literature review as a basis for the initial development of the HMP. The review focused on several key technical areas, including an analysis of the flow control approaches used in past hydromodification management efforts. Concepts of effective work, critical flow, and erosion potential were reviewed along with noted stream classification strategies. Finally, hydromodification management strategies were reviewed, including LID, flow duration control basins, and in-stream mitigation. The literature review also focused on continuous simulation modeling approaches, rainfall data management, determination of rainfall losses due to infiltration, and determination of rainfall losses due to evaporation.

To assess the effectiveness of storm water devices to meet hydromodification criteria, peak flow frequency, and duration statistics were required to be developed. A literature review examining these statistical methods indicated that the use of a partial-duration series is preferred for climates similar to San Diego County. The need for partial-duration statistics is more pronounced for control standards based on more frequent return intervals (such as the 2-year design storm), since the peak annual series statistics do not perform as well in the estimation of such events. This phenomenon is especially pronounced in the San Diego region's semi-arid climate. Partial-duration series frequency calculations consider multiple storm events in a given year while the peak annual series considers just the peak storm event. The Hydrologic Research Center (HRC), which is located in San Diego, recommended use of the partial duration series method to most accurately estimate flow frequency response in the San Diego climate.

Methodology and Technical Approach

Per the Permit Order, a range of runoff flow rates was required to be determined to identify the range for which Priority Development Project post-project runoff flows and durations shall not exceed pre-project runoff flows and durations. The Order further required a continuous hydrologic simulation of the entire rainfall record be generated. In January 2008, Interim HMP standards were developed in order to meet the Regional Board Order. These requirements pertained only to projects disturbing 50 acres or more.

Per final hydromodification management criteria developed for San Diego County, which will be applicable to all Priority Development Projects, results of a hydromodification management analysis must adhere to the following criteria:

- For flow rates between the pre-project lower flow threshold (see below) and the pre-project 10-year runoff event, the post-project discharge rates, and durations may not deviate above the pre-project discharge rates and durations by more than 10 percent over more than 10 percent of the length of the flow duration curve.
- Lower flow thresholds may be determined using the HMP Decision Matrix (located in Chapter 6) along with a critical flow calculator and channel screening tools developed by the Southern California Coastal Water Research Project (SCCWRP), detailed in Chapter 5. These methods identify lower flow thresholds for a range of channel conditions. The critical flow calculator recommends a lower flow value of $0.1Q_2$, $0.3Q_2$, or $0.5Q_2$ dependent on the receiving channel material and dimensions. This value will be compared to the channel susceptibility rating (High, Medium, or Low) as determined from the SCCWRP screening tools located in Appendix B to determine the final lower flow threshold.
- The lower flow threshold may alternately be determined as 10 percent of the pre-project 2-year runoff event, or $0.1Q_2$. This approach, which is outlined in the HMP Decision Matrix, is available if the project applicant chooses not to complete the channel screening analysis.

Information regarding the analysis and categorization of streams from a geomorphic context has been prepared in a concurrent grant-funded hydromodification study by the Southern California Coastal Water Research Project (SCCWRP) and the County of San Diego. Screening tools developed by SCCWRP identify channel susceptibility to hydromodification impacts. These include tools to classify receiving streams as having either a High, Medium, or Low susceptibility to channel erosion impacts. Where receiving stream channels are already unstable, the standard is to avoid acceleration of the existing erosion problems. Where receiving channels are in a state of dynamic equilibrium, hydromodification management may prevent the onset of erosion or other problems.

Requirements/Standards for Projects

Priority Development Projects are required to implement hydromodification mitigation measures so that post-project runoff flow rates and durations do not exceed pre-project flow rates and durations where such increases would result in an increased potential for erosion or significant impacts to beneficial uses. Hydromodification mitigation can provide:

- Demonstration of no post-project increase in impervious area and resultant peak flow rates as compared to pre-project conditions;
- Installation of LID BMPs, such as bioretention facilities, to control runoff flows and durations from new impervious areas;
- Mitigation of flow and durations through implementation of extended detention flow duration control basins;
- Preparation of continuous simulation hydrologic models and comparison of the pre-project and mitigated post-project runoff peaks and durations (with hydromodification flow controls) until compliance is achieved; and
- Implementation of in-stream rehabilitation controls to demonstrate that projected increases in runoff peaks and/or durations would not accelerate erosion to the rehabilitated receiving stream reach.

The HMP Decision Matrix, which leads project applicants through the HMP compliance options, is located in Chapter 6.

Exemptions

The HMP Decision Matrix outlines potential exemptions from hydromodification management criteria. These potential exemptions include discharges to exempt receiving waters such as the Pacific Ocean, to hardened conveyance systems that extend to exempt systems, as well as discharges to highly urbanized watersheds (greater than 70 percent imperviousness).

Selection and Implementation of BMPs

The project proponent may use Low-Impact Development (LID) integrated management practices to mitigate hydromodification impacts, using design procedures, criteria and sizing factors developed by the consultant team with input from the TAC and Copermittees. The sizing factor development protocol, which includes the use of a continuous simulation of runoff from the long-term rainfall record, is detailed in Chapter 7.

LID facilities must be designed to be practically built and maintained within the urban environment. Since the HMP will be implemented through the municipal development review process, design criteria have been specified and will be incorporated into conditions of approval. This HMP advocates the use of LID design approaches to provide both treatment of the 85th percentile water quality event as well as flow control to meet hydromodification criteria. To assure compliance with hydromodification flow control requirements, design criteria, specifications, and long-term operations and maintenance requirements have been provided in the Model SUSMP for a variety of LID-based flow control methods including bioretention basins, flow-through planter boxes, and bioretention systems in combination with cisterns and vaults. Provisions will also be provided for the design of larger extended detention flow duration control scenarios subsequent to approval of the HMP by the Regional Board and subsequent approval of local SUSMPs.

Details regarding rainwater harvesting, the collection of storm water for future reuse and a potentially effective storm water quality mitigation approach, are discussed in the San Diego Model SUSMP document. Because the release of the collected water is not standardized and since a full collection facility at the onset of rainfall would provide no flow control benefit, rainwater harvesting methods are not discussed in this HMP.

Proof of a long-term, ongoing maintenance responsibility and mechanism will be required for all post-construction BMP and flow control facilities. If not properly designed or maintained, hydromodification flow control devices may create a habitat for vectors such as mosquitoes or rodents. Maintenance activities for flow control and LID devices will be specified in the proposed Project Submittal.

Monitoring and BMP Evaluation

Chapter 8 of this HMP includes an outline for a monitoring program to assess the effectiveness of hydromodification management facilities. Monitoring activities will include inflow and outflow monitoring from BMPs, baseline cross section monitoring, and flow-based sediment monitoring. These monitoring efforts will coordinate with ongoing hydromodification monitoring work conducted by SCCWRP.

1. INTRODUCTION

Hydromodification refers to changes in the magnitude and frequency of stream flows as a result of urbanization and the resulting impacts on receiving channels in terms of erosion, sedimentation, and degradation of in-stream habitat. The degree to which a channel will erode is a function of the increase in driving force (shear stress), the resistance of the channel (critical shear stress), the change in sediment delivery, and the geomorphic condition of the channel. Critical shear stress is the stress threshold above which erosion occurs. Not all flows cause erosion -- only those that generate shear stress in excess of the critical shear stress of the bank and bed materials. Urbanization increases the shear stress exerted on the channel by stream flows and can trigger erosion in the form of incision (channel downcutting), widening (bank erosion), or both. Increases in flow below critical shear stress levels have little or no effect on the channel.

Provision D.1.g of the San Diego Regional Water Quality Control Board (RWQCB) Permit Order R9-2007-0001 requires the Copermittees to implement a HMP "…to manage increases in runoff discharge rates and durations from all Priority Development Projects, where such increased rates and durations are likely to cause increased erosion of channel beds and banks, sediment pollutant generation, or other impacts to beneficial uses and stream habitat due to increased erosive force." Where receiving stream channels are already unstable, hydromodification management can be thought of as a method to avoid accelerating or exacerbating existing problems. Where receiving stream channels are in a state of dynamic equilibrium, hydromodification management may prevent the onset of erosion or other problems.

To address the permit condition, the San Diego Storm Water Copermittees, represented by the County of San Diego, hired a consultant team and proceeded with developing an HMP that meets the intent of the Order. Permit Order R9-2007-0001 contains certain requirements that strongly influence the methodology chosen in development of the HMP. The Permit requires the Copermittees to develop an HMP for all Priority Development Projects (with certain exemptions) and develop standards to control flows within the geomorphically-significant flow range. Supporting analyses must be based on continuous hydrologic simulation modeling.

The Copermittees will incorporate HMP requirements into the local approval processes via incorporation of HMP criteria into local SUSMPs. The San Diego region's updated Model SUSMP will incorporate the Final HMP criteria. HMP criteria will be incorporated into the local SUSMP and municipal ordinances no later than 180 days following RWQCB adoption of the HMP.

It should be noted that the San Diego RWQCB jurisdiction area covers the majority of San Diego County. A portion of eastern San Diego County, all of which is part of the unincorporated County of San Diego, is under the jurisdiction of the Colorado River RWQCB and is not subject to the provisions of this HMP.

2. COPERMITTEE HMP DEVELOPMENT PROCESS

Although the County of San Diego serves as the lead agency for development of the HMP, all 20 of the other Copermittees have participated in its development, both financially and through participation in the Copermittees' Hydromodification/SUSMP Workgroup, which is a subcommittee of the Copermittees' Land Development Workgroup. The Hydromodification/SUSMP Workgroup was convened periodically over the course of the project at times corresponding with key decision points in developing the HMP and the update to the Model SUSMP.

This workgroup was tasked with providing regional standards and consistency in the development, implementation, assessment, and reporting of urban runoff activities and programs related to hydromodification management. As required by Permit Section D.1.g, the Workgroup assisted in the development of the regional HMP.

It should be noted that Copermittees' Regional Land Development Workgroup will continue to meet to discuss and resolve any issues that may arise during the HMP implementation phase. The Workgroup will also assist in the refinement and reinforcement of methodologies, criteria, and standards established in the HMP. This Workgroup has provided training regionally to municipal staffs as well as the local engineering community on LID and hydromodification management concepts, as well as requirements in the updated Model SUSMP and HMP.

The Copermittee HMP Workgroup met 14 times since July 2007. The table below summarizes meeting dates, locations, and agenda items. In addition to the formal meetings, the Copermittee HMP Workgroup coordinated via email on countless occasions to review and discuss technical documents, deliberate regarding specific HMP-related topics and reach consensus to provide direction for the consultant team.

Table 2-1. Copermittee Workgroup Meetings Summary

Date	Location	Agenda
July 26, 2007	County of San Diego 9325 Hazard Way San Diego, CA	• Formation of a Technical Advisory Committee • Discussion of HMP requirements in other permits • Consultant contract for HMP
August 23, 2007	City of San Diego 2392 Kincaid Road San Diego, CA	• Formation of a Technical Advisory Committee • Consultant contract for HMP
October 18, 2007	County of San Diego 9325 Hazard Way San Diego, CA	• Development of interim hydromodification criteria • Technical Advisory Committee
November 5, 2007	County of San Diego 5201 Ruffin Road San Diego, CA	• Development of interim hydromodification criteria
December 13, 2007	County of San Diego 5201 Ruffin Road San Diego, CA	• Development of interim hydromodification criteria

Table 2-1. Copermittee Workgroup Meetings Summary

Date	Location	Agenda
May 12, 2008	County of San Diego 5201 Ruffin Road San Diego, CA	• Development of interim hydromodification criteria
June 19, 2008	County of San Diego 5201 Ruffin Road San Diego, CA	• HMP progress report
October 21, 2008	County of San Diego 5201 Ruffin Road San Diego, CA	• HMP submittal to the Regional Board
December 16, 2008	County of San Diego 5201 Ruffin Road San Diego, CA	• HMP submittals to the Regional Board • Rain gauge data for HMP continuous simulation modeling
January 15, 2009	City of Chula Vista 1800 Maxwell Road Chula Vista, CA	• Approval of Draft HMP for submittal to RWQCB • Approval of Model SUSMP for submittal to RWQCB
July 20, 2009	County of San Diego 5201 Ruffin Road San Diego, CA	• HMP Decision Matrix • Discussion of potential exemptions
October 28, 2009	City of Chula Vista 1800 Maxwell Road Chula Vista, CA	• Discussion of Draft Final HMP document • Discussion of HMP implementation • HMP Design Standards
June 22, 2010	County of San Diego 5201 Ruffin Road San Diego, CA	• HMP Monitoring Plan
September 22, 2010	City of Santee 10601 Magnolia Avenue Santee, CA	• HMP Monitoring Plan • QAPP Development Process

The Copermittees will incorporate HMP requirements into the local approval processes via incorporation of HMP criteria into their local SUSMPs and municipal ordinances no later than 180 days following RWQCB adoption of the HMP. The San Diego region's updated Model SUSMP will also incorporate the Final HMP criteria.

3. TECHNICAL ADVISORY COMMITTEE

A key element of the San Diego HMP was the creation and involvement of a TAC. The TAC members consist of respected individuals from academia, technical resource agencies, the development community, consulting engineers, and environmental organizations. Dennis Bowling of the San Diego American Public Works Association (APWA), Chair of the Water Resources Committee, chairs the TAC. A list of all TAC members and attendees to the meetings is included at the end of this section. The TAC, which has been convened on t11 occasions that correlated with key decision-making points in the development of the HMP, was tasked with providing technical input to the HMP's scientific approach and interpretation of results integral to the establishment of numerical flow control standards as well as to the Copermittees for their policy determinations. At each TAC meeting, the consultant team presented a PowerPoint presentation describing the technical approach, and solicited feedback and buy-in from TAC members. While the TAC did not always achieve consensus on recommendations to the Copermittee workgroup, its discussions and alternate views were presented to the Copermittees for their consideration. An example involves comments provided by the Natural Resources Defense Council (NRDC) and Coastkeeper. While some of their comments, such as their opinion that storm events up to the 100-year event should be considered for hydromodification mitigation, differed from the majority consensus of the TAC, their comments were considered and specifically addressed. A comment response document to Coastkeeper comments is included in Appendix C.

Some of the key input received from the TAC included agreement with the Consultant Team's approach to using a synthetic watershed modeling approach to develop flow control standards (due to time constraints and a lack of published information on local geomorphology); agreement with the selection of 20 representative rain gauges and methodology to address data gaps (to provide the historical rainfall record for the required continuous simulation hydrologic modeling); agreement on the use of scaled Lindbergh Field data to conduct the initial modeling efforts (since available local rain gauge data sets were not in a format suitable for use with continuous simulation software at the time they were required); input on development of the HMP decision matrix; lower flow threshold calculator; and SCCWRP channel screening tools/domain of analysis.

The table below summarizes meeting dates, locations, and agenda items for all TAC meetings.

Table 3-1. Technical Advisory Group Meeting Summary

Date	Location	Agenda
February 20, 2008	City of San Diego Metro Biosolids Conference Rm. San Diego, CA	• Formation of a Technical Advisory Committee • Introduction of Consultant Team • Proposed approach to developing HMP and Model SUSMP Update (presentations by Dan Cloak, Dan Cloak Environmental Consulting and Andy Collison, PWA) • Input on how much channel erosion is tolerable • Input on how aggrading channels should be addressed

Table 3-1. Technical Advisory Group Meeting Summary		
Date	**Location**	**Agenda**
May 29, 2008	City of San Diego Metro Biosolids Conference Rm. San Diego, CA	• Recap of Interim HMP Standard • Input on/agreement with approach on synthetic watershed modeling approach (presentation by Andy Collison, PWA) • Input on/agreement with approach to conducting geomorphic assessment • Discussion of approach to conducting continuous hydrologic simulation modeling
August 5, 2008	City of San Diego Metro Biosolids Conference Rm. San Diego, CA	• Input on/agreement with approach to selection of representative gauges and management of rainfall data (Presentation by Eric Mosolgo, Brown and Caldwell) • Overview of approach to conducting continuous hydrologic simulation modeling (Presentation by Eric Mosolgo, Brown and Caldwell) • Overview of BMP Sizing Tool Development (Presentation by Eric Mosolgo) • Initial results of synthetic watershed modeling based on 2 watersheds in San Diego County (Presentation by Andy Collison, PWA)
October 14, 2008	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Recap of meeting with Regional Board to discuss HMP and Model SUSMP Update submittals • Input on/agreement with approach to supplementing rain gauge data sets and selection of proper rain gauge(s) for a project (Presentation by Eric Mosolgo, Brown and Caldwell) • Additional discussion of continuous hydrologic simulation modeling, including use of partial duration series data (Presentation by Eric Mosolgo and Tony Dubin, Brown and Caldwell) • Discussion of findings of synthetic watershed modeling (Presentation by Andy Collison and Christie Beeman, PWA)
February 12, 2009	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Review of Draft HMP submittal to RWQCB, review of concurrent SCCWRP modeling, summary of flow threshold modeling efforts (Presentation by Eric Mosolgo, Brown and Caldwell) • Presentation of flow threshold analysis and lower threshold alternatives including watershed position and channel characteristics (Presentation by Andy Collison and Christie Beeman, PWA)
April 21, 2009	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Review of comments prepared by Dr. Richard Horner, prepared on behalf of Coastkeeper, pertaining to the Draft HMP submitted to the RWQCB; review of SCCWRP work for San Diego HMP; requirements for partial duration rainfall series analysis; watershed position affects on lower flow threshold; and development of the HMP implementation decision matrix (Presentation by Eric Mosolgo, Brown and Caldwell) • Development of lower flow threshold nomograph and determination of alternate minimum flow rate (Presentation by Christie Beeman, PWA)
June 17, 2009	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Summary and review of SCCWRP progress on developing the Channel Susceptibility Analysis and Domain of Analysis (Presentation by Eric Stein, SCCWRP, via telephone) • Review and discussion of lower flow threshold nomograph (Presentation by Andy Collison, PWA, via telephone) • Review of minimum flow rate and cumulative impacts (Eric Mosolgo, Brown and Caldwell) • Response to Coastkeeper comments on Draft HMP (Eric Mosolgo, Brown and Caldwell) • Discussion of BMP Sizing Calculator development (Presentation by Tony Dubin, Brown and Caldwell, via telephone, and Eric Mosolgo) • Discussion of Draft HMP Decision Matrix (Eric Mosolgo, Brown and Caldwell)

Table 3-1. Technical Advisory Group Meeting Summary

Date	Location	Agenda
July 29, 2009	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Review of SCCWRP progress on developing the channel screening tools (Eric Mosolgo, Brown and Caldwell) • Discussion of Revised Draft HMP Decision Matrix (Eric Mosolgo, Brown and Caldwell) • Responses to RWQCB comments on Draft HMP submittal (Eric Mosolgo, Brown and Caldwell)
September 30, 2009	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Summary and review of SCCWRP progress on developing the Channel Susceptibility Analysis and Domain of Analysis (Presentation by Eric Stein, SCCWRP) • Discussion of Track 1 and Track 2 flow threshold analysis development (Presentation by Andy Collison, PWA, via telephone and Webcast) • Discussion of Draft HMP Decision Matrix, HMP exemptions, design standards technical memo, and proposed monitoring plan (Eric Mosolgo, Brown and Caldwell)
October 16, 2009	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• Discussion of minimum orifice size (Eric Mosolgo, Brown and Caldwell) • Review of proposed monitoring plan (Eric Mosolgo, Brown and Caldwell) • Review of lower flow threshold analysis and modification to the PWA calculator (Eric Mosolgo, Brown and Caldwell) • Review and discussion of revised HMP Decision Matrix incorporating the SCCWRP Channel Susceptibility tools (Eric Mosolgo, Brown and Caldwell)
June 21, 2010	City of San Diego Stormwater Dept. Conference Rm. 9370 Chesapeake Drive San Diego, CA	• HMP Monitoring Plan

The tables below list TAC members, non-TAC member meeting attendees, and the HMP Consultant Team.

Table 3-2. Technical Advisory Committee (TAC)

Name and Entity	Sector Represented
Sara Agahi, County of San Diego	San Diego Stormwater Copermittees
Edward Beighley, San Diego State University	BMP and Erosion Control Expert
Livia Borak, San Diego Coastkeeper, Natural Resources Defense Council (NRDC)	Environmental Community
Dennis Bowling, Rick Engineering	Chair of TAC
Dr. Howard Chang, San Diego State University	Geomorphology Expert
Rob Hawk, City of San Diego	Geotechnical Expert
Mikhail Ogawa, Mikhail Ogawa Engineering	TAC Coordinator
Eric Reichard, U.S. Geological Survey	Geology Expert
Eric Sattler, Spear & Associates	North County Engineers Council
Gabriel Solmer, San Diego Coastkeeper, Natural Resources Defense Council (NRDC)	Environmental Community
Eric Stein, Southern California Coastal Water Research Project (SCCRWP)	Technical Resource Agency
Garret Tam Sing, CA Department of Water Resources	Technical Resource Agency
Martin Teal, West Consultants	Consulting Engineers
Tory Walker, Tory Walker Engineering	Building Industry Association

Table 3-3. TAC Meeting Attendees (Non-TAC Members)	
Name	**Entity/Affiliation**
David Hauser	City of Carlsbad
Glen Van Peski	City of Carlsbad
Khosro Aminpour	City of Chula Vista
Silvester Evetovich	City of Chula Vista
Tom Adler	City of Chula Vista
Jaime Campos	City of El Cajon
Masih Maher	City of Encinitas
Erik Steenblock	City of Encinitas
Cheryl Filar	City of Escondido
Homi Namdari	City of Escondido
Mo Lahsaie	City of Oceanside
Alison Witheridge	City of Oceanside
Billy Walker	City of Oceanside
Danis Bechter	City of Poway
Roger Morrison	City of Poway
Sumer Hasenin	City of San Diego
James Nabong	City of San Diego
Sassan Haghgoo	City of San Marcos
Julie Procopio	City of Santee
Greg Mayer	City of Vista
Karen Franz	Coastkeeper
Vaikko Allen	Contech Stormwater Solutions
Chris Crompton	County of Orange
George Edwards	County of Orange
Anthony Barry	County of San Diego
John Quenzer	D-MAX Engineering
Arsalan Dadkhah	D-MAX Engineering/City of National City
Dick Rol	Foothill Engineering
Jeff O'Connor	Home Fed
Dave Hammar	Hunsaker & Associates
Luis Parra	Hunsaker & Associates, Adams Engineering, URS
Eylon Shamir	Hydrologic Research Center
Rosanna Lacarra	PBS&J
Debby Reece	Project Design Consultants
Allison Gutierrez	Port of San Diego
Karen Holman	Port of San Diego
Rich Lucera	RBF
Braeden Macguire	RBF

Table 3-3. TAC Meeting Attendees (Non-TAC Members)	
Name	**Entity/Affiliation**
Laura Henry	Rick Engineering
Bob Cullen	Riverside County Flood Control & Water Conservation District
Jason Uhley	Riverside County Flood Control & Water Conservation District
Tyler Schemper	Tory Walker Engineering
Matt Moore	URS Corporation/Port of San Diego

Table 3-4. HMP Consultant Team	
Name	**Company**
Christie Beeman	Philip Williams & Associates
Dr. Andrew Collison	Philip Williams & Associates
Dan Cloak	Dan Cloak Environmental Consulting
Tony Dubin	Brown and Caldwell
Nancy Gardiner	Brown and Caldwell
Eric Mosolgo	Brown and Caldwell

4. LITERATURE REVIEW

Pursuant to Permit Section D.1.g(1)(e), this section provides the results of a literature review conducted as a basis for the initial development of the HMP.

4.1 Flow Control Approach

HMPs that have been developed in the San Francisco Bay Area of California (Contra Costa, Santa Clara, and Alameda Counties) vary with regard to the emphasis placed on lower flow control thresholds as compared to other approaches, such as distributed LID methods. However, there is consensus in that both the frequency and duration of flows must be controlled, requiring the use of continuous simulation hydrologic modeling (as opposed to the more standard design storm approach used for flood control design) for evaluation of potential development impacts. It is also generally accepted that events smaller than the 10-year design flow are the most critical for hydromodification management.

The Santa Clara HMP focused on the use of detention basins for hydromodification management and strongly emphasized the lower flow control limit for site runoff. Extended detention flow control basins can utilize multi-stage outlets to mitigate both the duration and magnitude of flows within a prescribed range. To avoid the erosive effects of extended low flows, the maximum rate at which runoff is discharged is set below the erosive threshold. Per the Santa Clara HMP, the lower flow control limit was defined as the flow rate that generates critical shear stress on the channel bed and banks. Both Santa Clara and Alameda Counties correlated the lower flow control limit to a value equal to 10 percent of the 2-year runoff event.

The Contra Costa HMP strongly emphasized the use of LID methods to meet hydromodification management criteria. LID approaches to hydromodification management rely on site design and distributed LID Best Management Practices (BMPs) to control the frequency and duration of flows and to mitigate hydrograph modification impacts. By minimizing directly connected impervious areas and promoting infiltration, LID approaches mimic natural hydrologic conditions to counteract the hydrologic impacts of development. Because more runoff is retained onsite and in distributed facilities the lower discharge limit is less critical for LID facilities since different facilities discharge to the stream system at different times.

The County of San Diego and Copermittees interviewed three consultant teams as part of the selection process to develop the HMP. The selection panel; which included representatives from the County of San Diego, City of San Diego, City of Chula Vista, and the City of Encinitas, selected the team led by Brown and Caldwell and included Phillip Williams Associates and Dan Cloak Engineering. This team had previously developed the HMP for Contra Costa County and thus, the Contra Costa approach was selected as the base approach for the San Diego HMP.

For the San Diego region's Interim Hydromodification Management Criteria, the range of flows to be managed under the hydrograph curve-matching approach (matching of peak flows and durations within the geomorphically significant range) was expressed as a percentage of the 5-year runoff event, based on the understanding that the 5-year runoff event is considered the dominant channel-forming discharge for Southern California streams. This assumption was based upon the paper titled, "Effect of Increases in Peak Flows and Imperviousness on the Morphology of Southern California Streams," by Coleman, MacRae, and Stein. The following list details the range of flows recommended in the San Diego region's Interim Hydromodification Criteria.

- For flow rates between 20 percent of the pre-project 5-year runoff event and the pre-project 10-year runoff event, the post-project discharge rates and durations may not deviate above the pre-project discharge rates and durations by more than by more than 10 percent over more than 10 percent of the length of the flow duration curve.
- For flow rates between 20 percent of the pre-project 5-year runoff event and the pre-project 5-year runoff event, the post-project flows shall not exceed pre-project flows. For flow rates between the 5-year and 10-year runoff events, post-project flows may exceed pre-project flows by up to 10 percent for a 1-year frequency interval.
- The project proponent may also use LID integrated management practices to manage hydromodification impacts, using design procedures, criteria, and sizing factors (ratio of the required LID area to the tributary impervious area) specified by the Copermittees.

The Interim Hydromodification Management Criteria listed above were put in place beginning in January 2008 for development projects that disturb 50 acres or more.

Hydromodification in the context of this project refers to changes in the magnitude and frequency of stream flows as a result of urbanization and the resulting impacts on the receiving channels in terms of erosion, sedimentation, and degradation of instream habitat. The processes involved in this degradation are complex, but involve an alteration of the hydrologic regime of a watershed due to increases in impervious surfaces, more efficient and dense storm drain networks, and a change in historic sediment sources. The study of hydromodification is an evolving field, and regulations to manage the impacts of hydromodification must take into account the latest science available.

HMPs seek ways to mitigate erosion impacts by establishing requirements for controlling runoff from new development. In order to establish appropriate regulations, it is important to understand 1) how land use changes alter storm water runoff; and 2) how these changes can impact stream channels. This literature review focuses on how these issues have been addressed in HMPs adopted within the state of California as well as relevant journal articles, books, and other reports. This report builds upon previous literature reviews developed for other HMPs, and attempts to not repeat information that can be found in those reports. Instead this report is a synthesis of information that can be found in those studies and is augmented with either more recent studies or information relevant to Southern California.

4.1.1 Previous Studies

Previous hydromodification literature reviews were conducted by Geosyntec Consultants (Mangarella and Palhegyi, 2002) for the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP) and by the Contra Costa Clean Water Program (CCCWP 2004). Mangarella and Palhegyi provide a detailed overview of the geomorphic and hydrologic processes involved in hydromodification and the reader is directed there for more detailed information on the mechanics of stream erosion. Channel Assessment methods described in Section 2 of this report rely heavily on those reviewed by Bledsoe et al. (2008) for SCCWRP.

As of the date of this report, five approved HMPs have been published. These include HMPs for SCVURPPP (2005), the CCCWP (2005), the Fairfield-Suisun Urban Runoff Management Program FSURMP (2005), the Alameda Countywide Clean Water Program (ACCCMP 2005), and the San Mateo County Stormwater Pollution Prevention Program (STOPPP 2005). In addition, a number of HMPs were implemented while agencies developed their final plans. Interim HMPs are not detailed in this report due to the fact that these plans have adopted findings from the above listed HMPs.

4.1.2 Hydrograph Modification Processes

The effects of urbanization on channel response have been the focus of many studies (see Paul and Meyer, 2001 for a review), and the widely accepted consensus is that increases in impervious surfaces associated with urbanizing land uses can cause irreversible channel degradation. Urbanization generally leads to a change in the amount and timing of runoff in a watershed, which leads to increases in erosive forces on bank and bed material. This can cause large-scale channel enlargement, stream bank failure, loss of aquatic habitat and degradation of water quality.

Channel erosion, like most physical processes, is a complex system based on a variety of influences. Channel erosion is non-linear (Philips 2003) meaning the response of streams is not directly proportional to changes in land use and flow regimes. Small changes or temporary disturbances in a watershed may lead to unrecoverable channel instability (Kirkby 1995). These disturbances may give rise to feedback systems whereby small instabilities can be propagated into larger and larger instabilities (Thomas 2001).

A variety of factors have been documented to contribute to instability in streams. These include historic land use practices such as grazing (Trimble and Mendel 1995), logging (Jana et al. 1975), wildfire patterns, (Benda et al., 2003), geologic uplift (Colin and Burbank 2007), climatic changes (Leeder 1998), or removal of flora or fauna from the watershed (Ripple et al. 2001).

Although these parameters are varied, urban runoff control programs focus on managing the effect that new impervious surfaces have on stream channels. Stream channels show some form of temporal stability, whereby they resist change until a threshold of system parameters are exceeded (Thomas 2001). A number of studies have sought to correlate the amount of urbanization in a watershed and stream instability (Bledsoe 2001; Booth 1990, 1991; Both and Jackson 1997; MacRae 1992; 1993; 1996; Coleman et al. 2005). Evidence from these studies suggests that streams resist instability until a watershed urbanization threshold is crossed. This threshold appears to be around seven to ten percent watershed urbanization for perennial streams (Schueler 1998 and Booth 1997), but may be much lower for intermittent streams such as those found in Southern California. Studies done in Santa Fe, New Mexico (Leopold and Dunne 1978) suggest that dramatic changes occur at four percent impervious area of the watershed. Initial studies by Coleman et al. (2005) suggest that this urbanization threshold may be as low as two to three percent for intermittent streams in Southern California. It is important to understand that use of impermeable cover alone is a poor predictor of channel erosion due to regional differences and differences in storm water detention and infiltration within regions.

Though it is well established that watershed urbanization causes channel degradation, a detailed understanding of how development alters runoff and how this altered runoff in turn causes erosion is still being developed. This section briefly describes these processes and summarizes methods used to quantify hydromodification impacts.

4.1.2.1 Effective Work

The ability of a stream to transport sediment is proportional to the amount of flow in the stream: as flow increases, the amount of sediment moved within a channel also increases. The ability of a stream channel to transport sediment is termed stream power, which integrated over time is work. Leopold (1964) introduced the concept of effective work, whereby the flow-frequency relationship of a channel is multiplied by sediment transport rate. This gives a mass-frequency relationship for erosion rates in a channel. Flows on the lower end of the relationship (e.g., two-year flows) may transport less material, but occur more frequently than higher flows, thereby having a greater overall effect on the work within the channel. Conversely, higher magnitude events, while transporting more material, occur infrequently so as to have less effective work. Leopold found that the maximum point on the effective work curve occurred around the 1- to 2-year frequency range. This maximum point is commonly referred to as the dominant discharge and corresponds roughly to a bankfull event (a flow that fills the actively scoured portion of the channel up to a well defined break in the bank slope).

Urbanization tends to have the greatest relative impact on flows that are frequent and small, and which tend to generate less-than-bankfull flows. Change is greatest in these events because prior to urbanization infiltration would have absorbed much or all of the potential runoff, but following urbanization a high percent of the rainfall runs off. Thus, events that might have generated little or no flow in a non-urbanized watershed can contribute flow in urban settings. These smaller less-than-bankfull events have been found to do a significant proportion of the work in urban streams (MaCrae 1993) due to their high frequency, and can lead to channel instability. Less frequent, larger magnitude flows (e.g., flows greater than Q10) are less strongly affected by urbanization because during such large storm events the ground rapidly becomes saturated and acts in a similar manner as impervious surfaces.

4.1.2.2 Erosion Potential

As part of the SCVURPPP's HMP process, GeoSyntec Consultants (2004) studied the Lower-Silver-Thompson Creek subwatershed in Santa Clara County to characterize the pre-development effective work and compare it to modeled post-development effective work. Stability was assessed by comparing these effective work curves via an erosion potential index (Ep). This value is the ratio of the effective work of a pre-development stream to that of a post-development stream. A developed stream with an Ep of 1.0 has the same ability to transport sediment as an undeveloped stable stream. Managing the Ep of a stream can focus on managing the hydrologic regime of a watershed or on managing the stream itself. Both of these methods are discussed in Section 4.1.4.

Ep was adopted as a hydromodification metric for the SCVURPPP's hydromodification management program, and was later incorporated into four of the five approved HMPs. In addition, its use is being promoted by several research and regulatory bodies.

4.1.2.3 Estimating Critical Q_c

Due to the increase in impervious surfaces and fewer opportunities for infiltration of storm water, urbanization creates more runoff volume than an un-urbanized watershed. Opportunities for infiltration of excess storm water exist in some areas, but many times are infeasible due to cost or land use constraints. Therefore, some of the excess storm water must be discharged to a receiving stream. In order to achieve a comparable Ep to a pre-developed condition, this excess runoff volume must be discharged at a rate at which no additional stream work is done.

Bed load sediment moves through transmission of shear stress from the flow of water to the bed load material. An increase in velocity of water corresponds to an increase in shear stress. In order to initiate movement of bed material, however, a shear stress threshold must be exceeded. This is commonly referred to as critical shear stress, and is dependent on sediment and channel characteristics. For a given point on a channel where the cross-section is known, the critical shear can be related to a stream flow. The flow that corresponds to the critical shear is known as the critical flow, or Qc. For a given cross-section, flows that are below the value for Qc do not initiate bed movement, while flows above this value do.

The SCVURPPP expressed Qc as a percentage of the two-year flow in order to develop a common metric across watersheds of different size, and allow for easy application of HMP requirements For the two watersheds studied, a similar relationship was found where Qc corresponded to 10 percent of the two-year flow. This became the basis for the lower range of geomorphically significant flows under the SCVURPPP HMP and is referred to as Qcp to indicate that it is a percentage of flow. That program also adopted the 10-year flow as the upper end of the range of flows to control with the justification that increases in stream work above the 10-year flow were small for urbanized areas.

A similar study was conducted for the FSURMP on two watersheds in Fairfield, California following a geomorphic assessment. That study found Qcp to be 20 percent of the pre-development two-year flow. The differences in the two values may be attributable to differences in watershed characteristics in Santa Clara County and Fairfield. Channels in Fairfield were found to have a more densely vegetated riparian corridor and may have a higher resistance to increases in shear stresses (FSURMP). Values for Qcp appear to be similar among neighboring watersheds, but there appears to be no evidence for a 'universal' Qcp, and the characteristics of individual biomes (climatically and geographically defined areas of ecologically similar climatic conditions such as communities of plants, animals, and soil organisms, and are often referred to as ecosystems) should be taken into account when developing a Qcp. For example, Western Washington State, which has more densely vegetated riparian zones than either Fairfield or Santa Clara County, has adopted a Qcp of 50 percent of the 2-year flow.

A summary of flow control standards adopted in each of these HMPs is given in Table 4-1.

Table 4-1. Flow Control Standards Adopted by Selected Agencies for Hydromodification Management.

Permitting Agency	Qcp	Largest Managed Flow
Santa Clara County	10 percent of the 2-year flow (0.1Q2)	10-year flow (Q10)
Alameda County	10 percent of the 2-year flow (0.1Q2)	10-year flow (Q10)
San Mateo County	10 percent of the 2-year flow (0.1Q2)	10-year flow (Q10)
Contra Costa County	10 percent of the 2-year flow (0.1Q2)	10-year flow (Q10)
Fairfield-Suisun Urban Runoff Management Program	20 percent of the 2-year flow (0.2Q2)	10-year flow (Q10)
Western Washington State	50 percent of the 2-year flow (0.5Q2)	50-year flow (Q50)

4.1.3 Stream Channel Stability

Numerous stream channel stability assessment methods have been proposed to help identify which channels are most at risk from hydrograph modification impacts and/or define where HMP requirements should apply. Assessment strategies range from purely empirical approaches to channel evolution models to energy-based models (see Simon et al., 2007 for a critical evaluation).

4.1.3.1 Stream Classification Systems

A recent study by Bledsoe et al. (2008) for SCCWRP describes nine types of classification and mapping systems with an emphasis on assessing stream channel susceptibility in Southern California. The summary below is taken from that study. Bledsoe also provides a summary of the implications of these classification and mapping systems to the development of hydromodification tools for Southern California. The article provides a detailed breakdown of guidelines for developing hydromodification tools given the advantages and disadvantages of each system previously assessed.

Planform Classifications and Predictors

Alluvial channels form a continuum of channel types whose lateral variability is primarily governed by three factors: flow strength, bank erodibility, and relative sediment supply. Though many natural channels conform to a gradual continuum between straight and intermediate, meandering, and braided patterns, abrupt transitions in lateral variability imply the existence of geomorphic thresholds where sudden change can occur. The conceptual framework for geomorphic thresholds has proven integral to the study of the effects of disturbance on river and stream patterns. Many empirical and theoretical thresholds have been proposed relating stream power, sediment supply and channel gradient to the transition between braiding and meandering channels. Accounting for the effects of bed material size has been shown to provide a vital modification to the traditional approach of defining a discharge-slope combination as the threshold between meandering and braided channel patterns. The many braided planforms in Southern California indicate the need to refine and calibrate established thresholds to river networks of interest. However, at this time there is not a well accepted model to predict how hydromodification affects channel planform.

Energy-Based Classifications

The link between channel degradation and urbanization has been exhaustively studied; however, impervious area is not the solitary factor influencing channel response. Studies have shown that the ratio between specific stream power and median bed material size D50b, where b is approximately 0.4 to 0.5 for both sand- and gravel-bed channels, can be used as a valuable predictor of channel form. Stream power, which is related to the square root of total discharge, is the most comprehensive descriptor of hydraulic conditions and sedimentation processes in stream channels. Several studies have been performed relating channel stability to a combination of parameters such as discharge, median bed-material size, and bed slope, as an analog for stream power.

General Stability Assessment Procedures

By assessing an array of qualitative and quantitative parameters of stream channels and floodplains, several investigators have developed qualitative assessment systems for stream and river networks. These assessment methods have been incorporated into models used to analyze channel evolution and stability. Many parameters used to establish methodologies such as the Rosgen approach are extendable to a qualitative assessment of channel response in Californian river networks. Field investigations in Southern California have shown that grade control can be the most important factor in assessing the severity of channel response to hydromodification. Qualitative methodologies have proven extendable to many regions and utilize many parameters that may provide valuable information for similar assessments in California.

Sand vs. Gravel Behavior / Threshold vs. Live-Bed Contrasts

It is well recognized that the fluvial-geomorphic behavior varies greatly between sand and gravel/cobble systems. Live bed channels (of which sand channels are good examples) are systems where sediment moves at low flows, and where sediment is frequently in motion. Threshold channels such as gravel streams, by contrast, require considerable flow to initiate bedload movement. Live bed channels are more sensitive both to increases in flow and decreases in sediment supply than threshold channels. Scientific consensus shows that sand bed streams lacking vertical control show greater sensitivity to changes in flow and sediment transport regimes than do their gravel/cobble counterparts. Factors such as slope which affect discharge and sedimentation regimes are known to have greater impact on sand-bed streams. This can be an important issue for storm water systems that receive runoff from watersheds composed primarily of streams with sandy substrate. The transition between sand and gravel bed behavior can be rapid which may make it possible to utilize geographic mapping methods to identify channel segments according to their susceptibility to the effects of hydromodification.

Channel Evolution Models of Incising Channels

The Channel Evolution Model (CEM) developed by Schumm et al. (1984) posits five stages of incised channel instability organized by increasing degrees of instability severity, followed by a final stage of quasi-equilibrium. Work has been done to quantify channel parameters such as sediment load and specific stream power through each phase of the CEM. A dimensionless stability diagram was developed by Watson et al. (2002) to represent thresholds in hydraulic and bank stability. This conceptual diagram can be useful for engineering planning and design purposes in stream restoration projects requiring an understanding of the potential for shifts in bank stability.

Channel Evolution models Combining Vertical and Lateral Adjustment Trajectories

Originally, CEMs focused primarily on incised channels with geotechnically, rather than fluvially, driven bank failure. Several CEMs have been proposed that incorporate channel responses to erosion and sediment transport into the original framework for channel instability. In these new systems, an emphasis is placed on geomorphic adjustments and stability phases that consider both fluvial and geomorphic factors. The state of Vermont has developed a system of stability classification that suggests channel susceptibility is primarily a function of the existing Rosgen stream type and the current stream condition referenced to a range of variability. This system places more weight on entrenchment (vertical erosion of a channel that occurs faster than the channel can widen, so that the resulting channel is more confined than the original channel) and slope than differentiation between bed types.

Equilibrium Models of Supply vs. Transport-capacity / Qualitative Response

The qualitative response model builds on an understanding of the dynamic relationship between the erosive forces of flow and slope relative to the resistive forces of grain size and sediment supply to describe channel responses to adjustments in these parameters. In this system qualitative schematics provide predictions for channel response to positive or negative fluctuations in physical channel characteristics and bed material. Refinements to such frameworks have been made to account for channel susceptibility relative to existing capacity and riparian vegetation among other influential characteristics.

Bank Instability Classifications

Early investigations provided the groundwork for bank instability classifications by analyzing shear, beam, and tensile failure mechanisms. The dimensionless stability approach developed by Watson characterized bank stability as a function of hydraulic and geotechnical stability. Rosgen (1996) proposed the widely applied Bank Erosion Hazard Index (BEHI) as a qualitative approach based on the general stability assessment procedures outlined above. Other classification systems, like the CEM, identify bank instability according to channel characteristics that control hydrogeomorphic behavior.

Hierarchical Approaches to Mapping Using Aerial Photographs / GIS

It has become increasingly common practice to characterize stream networks as hierarchical systems. This practice has presented the value in collecting channel and floodplain attributes on a regional scale. Multiple studies have exploited geographical information systems (GIS) to assess hydrogeomorphic behavior at a basin scale. Important valley scale indices such as valley slope, confinement, entrenchment, riparian vegetation influences, and overbank deposits can provide indispensable information for river networks in California. Many agencies are developing protocol for geomorphic assessment using GIS and other database associated mapping methodologies.

4.1.4 Managing Hydromodification

Most HMPs provide guidance on how Copermittees can meet the goals of their program. There are many different approaches and most HMPs provide multiple options for achieving and documenting compliance. In general, hydrograph management approaches focus on managing runoff from a developed area so as to not increase instability in a channel, and in-stream solutions focus on managing the receiving channel to accept an altered flow regime without becoming unstable. This chapter briefly summarizes various approaches for HMP compliance.

4.1.4.1 Hydrograph Management Solutions

Facilities that detain or infiltrate runoff to mitigate development impacts are the focus of most HMP implementation guidance. They work either by reducing the volume of runoff (infiltration facilities) or by holding water and releasing it below Qc (retention facilities). These facilities, sometimes referred to as BMPs, can range from regional detention basins designed solely for flow control, to bioretention facilities that serve a number of functions. A number of BMPs including swales, bioretention, flow-through planters, and extended detention basins have been developed to manage storm water quality, and there are several resources that describe the design of storm water quality BMPs (CASQA 2003; Richman et al. 2004). In many cases these facilities can be designed to also meet hydromodification management requirements.

Many HMPs also provide guidance for applying LID approaches to site design and land use planning to preserve the hydrologic cycle of a watershed and mitigate hydromodification impacts. These plans typically include decentralized storm water management systems and protection of natural drainage features, such as wetlands and stream corridors. Runoff is typically directed toward infiltration-based storm water BMPs that slow and treat runoff.

The following sections summarize implementation guidance for designing hydromodification management BMPs that have been developed for existing HMPs.

Sizing Hydromodification BMPS

Hydromodification BMPs differ slightly from those BMPs used to meet water quality objectives in that they focus more on matching undeveloped flow-regimes than on filtering storm runoff, although these two functions can be combined into one facility. Various methods exist for sizing Hydromodification BMPs.

Hydrograph Matching. This is an approach whereby the outflow hydrograph for a particular site matches closely with the pre-project hydrograph for a design storm. This method is most traditionally used to design flood-detention facilities to mitigate for a particular storm recurrence interval (e.g., the 100-year storm). Although hydrograph matching can be employed for multiple storms, this method generally does not take into account the smaller, more frequent storms where a majority of the erosive work in stream channel is done and is therefore not widely accepted for HMP compliance.

Volume Control. This is a method for matching the pre-project and post-construction runoff volume for a project site. Any increase in runoff volume is either infiltrated on site, or discharged to another location where streams will not be impacted. The magnitude of peak flows is not controlled, and therefore this method, while ensuring that there is no increase in total volume of runoff, can result in higher erosive forces during storms.

Flow Duration Control. Refers to matching both the duration and magnitude of a specified range of storms. The entire hydrologic record is taken into account and pre-project and post-construction runoff magnitudes and volumes are matched as closely as possible. Excess runoff is either infiltrated on site, or is discharged below Qcp.

The SCVUPPP HMP reviewed each of these design approaches and concluded that a Flow Duration Control design approach was the most effective in controlling erosive flows. Two examples were evaluated using this approach, one on the Thompson Creek subwatershed in Santa Clara Valley and one on the Gobernadora Creek watershed in Orange County. The evaluation approach used continuous simulation modeling to generate flow-duration curves, and then designed a test hydromodification management facility to match pre-project durations and flows.

In addition to the SCVURPP, the flow duration control design approach has been applied by ACCWP, STOPPP, FSURMP, and CCCWP. Among these agencies, different approaches have emerged as to how to demonstrate that proposed BMPs meet flow-duration control guidelines. Both methods employ continuous simulation to match flow-durations, but differences exist in how continuous simulation is used (site-specific simulation vs. unit area simulation). Differences also exist in the focus of the two approaches (regional detention facilities vs. on-site LID facilities). Both approaches were evaluated by the RWQCB, and deemed to be valid approaches (Butcher 2007).

BAHM Approach

The Bay Area Hydrology Model (BAHM) is a continuous simulation rainfall-runoff hydrology model developed for ACCWP, STOPPP, and SCVURPP . It was developed from the Western Washington Hydrology Model, which focuses primarily on meeting hydromodification management requirements using storm water detention ponds alone or combined with LID facilities (Butcher 2007). The Western Washington Hydrology model is based on the Hydrologic Simulation Program FORTRAN (HSPF) modeling platform, developed by the USEPA, and uses HSPF parameters in modeling watersheds.

BAHM is a standalone modeling package that is available free of charge to the public. Project proponents who want to size a hydromodification BMP select the location of their project site from a map of the county and BAHM correlates the project location to the nearest rainfall gauge and applies an adjustment factor. The adjustment factor is applied to the hourly rainfall for the nearest gauge, to produce a weighted hourly rainfall at the project site. The user then enters parameters for the proposed project site that describe soil types, slope, and land uses. BAHM then runs the continuous rainfall-runoff simulation for both the pre-project and the post-construction conditions of the project site. Output is provided in the form of flow-duration curves that compare the magnitude and timing of storms between the pre-project and the post-construction modeling runs.

If an increase in flow durations is predicted, the user can select and size mitigation BMPs from a list of modeling elements. An automatic sizing subroutine is available for sizing detention basins and outlet orifices that matches the flow duration curves between the pre-project scenario and a post-construction mitigation scenario. Manual sizing is necessary for other BMPs included in the program, such as storage vaults, bioretention areas, and gravel trenches. The program is designed so that once a BMP is selected and sized, the modeling run can be transferred to the local agency for approval. The model reviewer at the local agency can open the program and verify modeling parameters and sizing techniques.

CCCWP Approach

The CCCWP developed their own protocol for selecting and sizing hydromodification BMPs, which are referred to as Integrated Management Practices (IMPs) in their guidebook. Instead of a project proponent running a site-specific continuous simulation to size hydromodification control facilities, the CCCWP provides sizing factors for designing IMPs. Sizing factors are based on the soil type of the project site and are adjusted for Mean Annual Precipitation. Sizing factors are provided for Bioretention Facilities, Flow-Through Planters, Dry Wells and a combination Cistern and bioretention facility.

Sizing factors were developed through continuous-simulation HSPF modeling runs for a variety of development scenarios. Flow-durations were developed for a range of soil types, vegetation and land use types, and rainfall patterns for development areas in Contra Costa County. Then, based on a unit area (one acre) of impervious surface, flow-durations were modeled using several IMP designs. These IMPs were then sized to achieve flow control for the range of storms required, (from 10 percent of the 2-year storm up to the 10-year storm). These sizing factors were then transferred to a spreadsheet form for use by project proponents.

The primary difference between the CCCWP approach and the BAHM approach is the focus on type of BMP used. Whereas the CCCWP approach focuses on meeting hydromodification management goals using lot-scale LID facilities, the BAHM approach is geared toward employing detention basins. Although the CCCWP approach is based on utilizing sizing factors for specific BMPs, the program does allow for application of site-specific continuous simulation modeling, such as HSPF, if the relevant sizing factor has not been developed, such as storm water detention basins or constructed wetlands. This approach can be used for larger developments where regional hydromodification facilities will be used.

4.1.4.2 In-Stream Stabilization Solutions

In-stream solutions focus on managing the stream corridor to protect stability and, if necessary, modify stream channels to accept an altered flow regime. In cases where development is proposed in an already degraded watershed it may be beneficial to focus on rehabilitating the stream channel with an altered flow regime in mind rather than retrofitting the watershed or only controlling a percentage of the runoff. In addition, in some cases where a master-planned watershed development plan is being implemented it may be more feasible to design a new channel to be stable under the proposed watershed land use rather than to construct distributed on-site facilities.

Newhall Ranch Natural River Management Plan

An example where in-stream solutions are being designed at the Master Plan level can be found in the Newhall Ranch Natural River Management Plan. The proposed Newhall Ranch development near Valencia, California is employing a combination of distributed storm water quality facilities to manage storm water pollutants and in-stream management actions to manage an altered flow regime. The management plan began with an analysis of post-development flow conditions, then found slopes and channel cross-sections that would be stable under these altered conditions. Biotechnical bank stabilization and stable step-pools were included to allow the new channel to resist higher shear forces. The plan has been approved by Los Angeles County Department of Public Works.

The key objectives for the in-stream channel design employed for the Newhall Ranch development were:

- Accommodate runoff flows from existing and future development;
- Stabilize the channel bed and banks so that they do not degrade;
- Preserve the waterway and canyon characteristics and environment, where applicable;
- Minimize riparian and bank disturbance during construction, where applicable;
- Implement improvements that are the most compatible with the environment and character of the region, yet sustainable on a long-term basis and
- Minimize channel maintenance requirements.

Other Methods

A number of methods exist for managing channels to accept altered flow regimes and higher shear forces. These have been covered in detail in a number of sources available to watershed groups and public agencies. (A few helpful sources include Riley 1998, Watson and Annable 2003, and FISRWG 1998.)

4.2 Continuous Simulation Modeling

As part of the HMP development, Brown and Caldwell is preparing flow control sizing tools to assess the effectiveness of hydromodification controls. A beta version of the HMP Sizing Calculator will be available by early 2010 and will be reviewed by the HMP TAC. Since those sizing tools are not yet available, Brown and Caldwell has identified specific evaluation criteria for the design and analysis of hydromodification controls using continuous simulation hydrologic modeling. Evaluation criteria discussed herein focuses on the following items:

- Continuous Simulation Hydrologic Modeling
- Continuous Simulation Modeling Software
- Long-Term Hourly Precipitation Gauge Data
- Parameter Validation for Rainfall Losses
- Hydromodification Control Processes
- Peak Flow and Flow Duration Statistics

Pursuant to criteria set forth by the San Diego RWQCB and by the San Diego County Copermittees in the Hydromodification Criteria, the use of continuous simulation hydrologic modeling is required to size storm water facilities to mitigate hydromodification effects. Continuous simulation modeling uses an extended time series of recorded precipitation data as input and generates hydrologic output, such as surface runoff, groundwater recharge, and evapotranspiration, for each model time step.

Continuous hydrologic models are typically run using either 1-hour or 15-minute time steps. Based on a review of available rainfall records in San Diego County, we are recommending the use of a 1-hour time step (15-minute time series rainfall data are very limited). Continuous models generate model output for each time step. In this case, hydrologic output would be generated for each hour of the continuous model. A continuous simulation model with 35 years of hourly precipitation data will generate 35 years of hourly runoff estimates, which corresponds to runoff estimates for 306,600 time steps over the 35-year simulation period.

Use of the continuous modeling approach allows for the estimation of the frequency and duration by which flows will exceed a particular threshold. The limitations to increases of the frequency and duration of flows within that geomorphically significant flow range is the key component to San Diego County's approach to hydromodification management.

For a more detailed review of continuous simulation modeling, refer to a memo prepared by Brown and Caldwell titled *Using Continuous Simulation to Size Storm Water Control Facilities* (May 2008). This memo is attached as Appendix E.

4.2.1 Continuous Simulation Modeling Software

The following public domain software models may be used to assess hydromodification controls for storm water facilities to meet the Hydromodification Criteria:

- HSPF - Hydrologic Simulation Program-FORTRAN, distributed by United States Environmental Protection Agency (USEPA)
- HEC-HMS – Hydrologic Modeling System; distributed by the US Army Corps of Engineers Hydrologic Engineering Center
- SWMM – Storm Water Management Model; distributed by USEPA

Third-party and proprietary software can be used to meet the Hydromodification Criteria provided that the software incorporates minimum design parameters summarized below:

- Input and output data from the software can interface with public domain software such as HSPF HEC-HMS, or SWMM. In other words, input files from the third-party software should have sufficient functionality to allow export to public domain software for independent validation.
- Rainfall data are selected according to an existing rainfall gauge location that is geographically and meteorologically similar to the project site location.

- Rainfall loss parameters used in the software can be substantiated and fully referenced.
- The software's hydromodification control processes, detailed later in this memo, are substantiated and fully referenced.

All third-party and proprietary software will be subject to more rigorous review upon the adoption of the Final HMP. This review would include further testing of various development and treatment scenarios as well as an in-depth analysis of software functionality and processes.

As stated previously, Brown and Caldwell is currently preparing flow control sizing tools to assess the effectiveness of hydromodification controls. These tools will be available in association with implementation of the final HMP.

4.2.2 Parameter Validation for Rainfall Losses

In preparing computer models to assess storm water controls and meet Hydromodification Criteria, rainfall loss parameters describing soil characteristics, land cover descriptions, and evapotranspiration data should be validated to prove consistency with the local environment and climatic conditions. The validation process should include documentation of the source of evapotranspiration data and commentary of the effects of varying evapotranspiration patterns between the subject site and parameter data source. A full review of local pan evaporation and potential evapotranspiration data will be included as part of development of the final hydromodification flow control sizing tool.

To meet Hydromodification Criteria, soil and land cover parameter validation can be based on the following:

- Calibration to local stream flow data, where applicable. Examples of local calibration studies include, but are not limited to, total maximum daily load (TMDL) modeling efforts prepared for the San Diego RWQCB (*TMDL for Indicator Bacteria Project I – Beaches and Creeks in the San Diego Region*, Tetra Tech, December 2007).
- Published parameter values consistent with previous studies for San Diego County and Southern California, such as HSPF-related regional calibration studies, research projects, regional soil surveys, etc.
- Specific data prepared as part of a site-specific geotechnical investigation
- If parameters are transposed or modified from calibration efforts outside of Southern California, the source should be identified and justification should be provided stating why such data are applicable for San Diego County. Details should be provided justifying how parameters from such studies were adjusted to be applicable to San Diego conditions.
- Recommended parameter value ranges from *BASINS Technical Notice 6, Estimating Hydrology, and Hydraulic Parameters for HSPF*, USEPA, July 2000.

Storm water flow control devices designed to meet Hydromodification Criteria should be analyzed pursuant to the following criteria:

- Infiltration processes should be modeled with sufficient complexity to properly quantify the flow control benefit to the receiving streams. These infiltration processes should be transparent and fully documented.
- Infiltration quantification should include provisions for water head and pore suction effects for multiple layers of varying materials (i.e., ponding areas, amended soil layer, gravel layer, etc.), or provide justification why such complex processes are not included.
- Storage processes associated with each layer of the storm water device should be fully explained and quantified.
- Device outflow curves should consider controls associated with device underdrains. The methodology by which such stage-discharge relationships are developed should be fully documented.

4.2.3 Peak Flow and Flow Duration Statistics

To assess the effectiveness of storm water flow control devices in mitigating hydromodification effects to meet Hydromodification Criteria, peak flow frequency statistics should be developed. Peak flow frequency statistics estimate how often flow rates will exceed a given threshold. In this case, the key peak flow frequency values would be the lower and upper bounds of the geomorphically significant flow range. Peak flow frequency statistics should be developed using either a partial-duration or peak annual series. Partial-duration series frequency calculations consider multiple storm events in a given year while the peak annual series considers just the peak annual storm event.

Flow duration statistics must also be summarized to determine how often a particular flow rate is exceeded. To determine if a storm water facility meets hydromodification criteria, peak flow frequency and flow duration curves must be generated for pre-project and post-project conditions. Both pre-project and post-project simulation runs should extend for the entire length of the rainfall record.

For a more detailed review of peak flow frequency and flow duration curves, refer to the aforementioned Brown and Caldwell memo titled *Using Continuous Simulation to Size Storm Water Control Facilities* (May 2008).

The need for partial-duration statistics is more pronounced for control standards based on more frequent return intervals (such as the 2-year runoff event), since the peak annual series does not perform as well in the estimation of such events. This phenomenon is especially pronounced in the San Diego County region's semi-arid climate. Per the advice of the Hydrologic Research Center, with whom the project team has consulted throughout the project, and a review of supporting literature, the use of a partial-duration series is recommended for semi-arid climates similar to San Diego County, where prolonged dry periods can skew peak flow frequency results determined by a peak annual series for more frequent runoff events.

For the statistical analysis of the rainfall record, partial duration series events have been separated into discrete rainfall events assuming the following criteria.

- To determine a discrete rainfall event, a lower flow limit was set to a very small value, equal to 0.002 cfs per acres of contributing drainage area.
- A new discrete event is designated when the flow falls below 0.002 cfs per acre for a time period of 24 hours.

4.3 Rainfall Data

Standards developed as part of this HMP to control runoff peak flows and durations are based on a continuous simulation of runoff using local rainfall data. To provide for clear climatic designation between coastal, foothill and mountain areas of the County, and to distinguish between the major watershed units, historical records for a series of 20 rainfall data stations located throughout San Diego County were compiled, formatted and quality controlled for analysis.

Long-term hourly rainfall records have been prepared for the 20 rainfall stations. These rainfall record files are located on the *Project Clean Water* web site for public use (www.projectcleanwater.org). Sources of the rainfall data include ALERT data from the County of San Diego (which extend back to 1982), the California Climatic Data Archive, National Oceanic and Atmospheric Administration (NOAA), the National Climatic Data Center, and the Western Regional Climate Center. In all cases, the length of the overall rainfall station record is 35 years or the overall length of the rainfall record, whichever is longer.

Gauge selection was further governed by minimum continuous simulation modeling requirements including the following:

- The selected precipitation gauge data set should be located near the project site to ensure that long-term rainfall records are similar to the anticipated rainfall patterns for the site. Thus, gauges were selected in proximity to areas planned for future development and redevelopment.
- Recording frequency for the gauge data set should be hourly (or more frequent).
- The gauge rainfall record should extend for the entire length of the record. Where the gauge record length is less than 35 years, then adjacent gauge records were used to extend the rainfall record to at least 35 years.
- Use of the most applicable long-term rainfall gauge data, as opposed to the scaling of rainfall patterns from Lindbergh Field, is required to account for the diverse rainfall patterns across San Diego County.

Precipitation gauges identified by Brown and Caldwell, summarized in Table 4-2 below, all have recording frequencies of one hour and recording data ranges of at least 35 years.

Table 4-2. Rainfall Station Summary

Station	Elevation	Watershed
Bonita	120	Sweetwater River
Encinitas	242	San Elijo Lagoon and Batiquitos Lagoon and ocean outlets
Escondido	645	Escondido Creek
Fallbrook	675	San Luis Rey River (near ridge with Santa Margarita River watershed)
Fashion Valley	20	Lower San Diego River
Flinn Springs	880	San Diego River
Kearny Mesa	425	San Diego River (near ridge with San Clemente Canyon watershed)
Lake Cuyamaca	4,590	Upper San Diego River
Lake Heneshaw	2,990	Upper San Luis Rey River
Lake Wohlford	1,490	Upper Escondido Creek
Lindbergh Field	Near Sea Level	Coastal – San Diego Bay
Lower Otay Reservoir	491	Otay River
Morena Dam	3,075	Upper Tijuana River
Oceanside	30	San Luis Rey River
Poway	440	Los Penasquitos Canyon
Ramona	1,450	Upper San Dieguito River
San Onofre	162	North County Coastal – Pacific Ocean
San Vicente Reservoir	663	San Diego River
Santee	300	San Diego River

For a given project location, the following factors should be considered in the selection of the appropriate rainfall data set.

- In most cases, the rainfall data set in closest proximity to the project site will be the appropriate choice. A rainfall station map has been posted to the *Project Clean Water* web site for public use.
- In some cases, the rainfall data set in closest proximity to the project site may not be the most applicable data set. Such a scenario could involve a data set with an elevation significantly different from the project site. In addition to a simple elevation comparison, the project proponent may also consult with the San Diego County's average annual precipitation isopluvial map, which is provided in the San Diego County Hydrology Manual (2003). Review of this map could provide an initial estimate as to whether the project site is in a similar rainfall zone as compared to the rainfall stations. Generally, precipitation totals in San Diego County increase with increasing elevation.
- Where possible, rainfall data sets should be chosen so that the data set and the project location are both located in the same topographic zone (coastal, foothill, mountain) and major watershed unit (Upper San Luis Rey, Lower San Luis Rey, Upper San Diego River, Lower San Diego River, etc.).

Upon implementation of final hydromodification criteria, the hydromodification flow control sizing calculator being developed by Brown and Caldwell will automate the rainfall gauge selection process.

4.4 Rainfall Losses - Infiltration Parameters

Standards developed as part of this HMP to control runoff peak flows and durations are based on a continuous simulation of runoff using locally derived parameters for initial infiltration. A review was conducted of available continuous hydrologic simulation modeling reports in Southern California. These included TMDL models developed for the San Diego RWQCB (RWQCB), regional continuous models developed by the Southern California Coastal Water Research Project (SCCWRP), and watershed-level continuous models developed for river and large creek systems in Ventura County. In conducting this review, particular interest was focused on determining how local and regional continuous hydrologic models simulated the pervious land surface[1] for various combinations of soils and land use types, because this component of hydrologic modeling is typically the most variable and difficult to describe.

The HSPF software package is the industry standard for continuous simulation hydrologic modeling, though HEC-HMS and SWMM also provide public domain continuous modeling alternatives. The Final HMP provides the option to use HEC-HMS for a project submittal but only provides infiltration data review for HSPF modeling approaches. Therefore, if a project applicant chooses to use HEC-HMS, prior authorization should be provided by the governing municipality.

[1] Characterized by PERLND/PWATER parameters in the EPA's public domain Hydrologic Simulation Program – FORTRAN, HSPF.

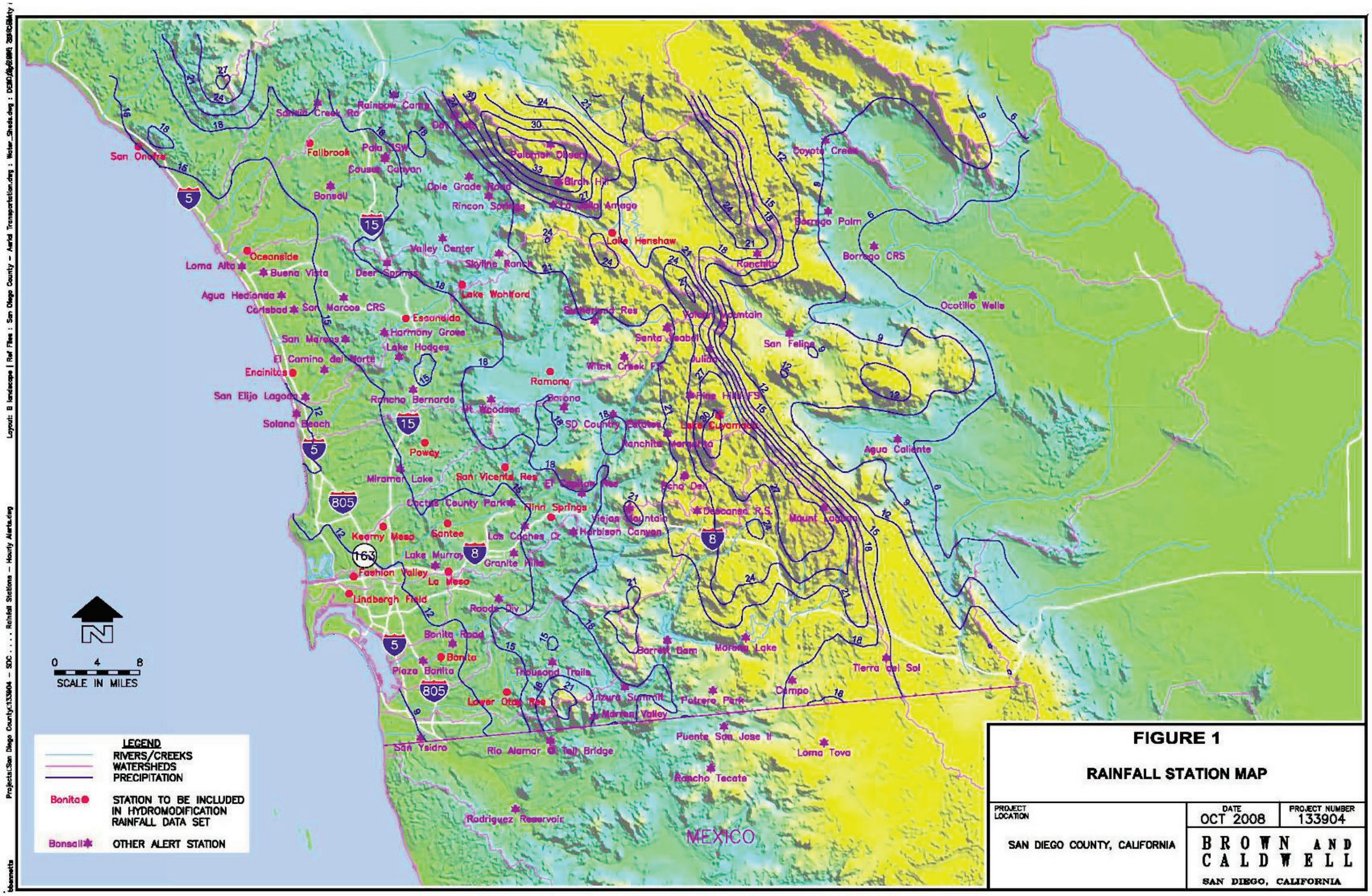

Figure 4-1. Rainfall Station Map

In preparing computer models to assess storm water controls and meet Hydromodification Criteria, rainfall loss parameters describing soil characteristics, land cover descriptions, and slope should be validated to prove consistency with the local environment and climatic conditions. The goal, with regard to the San Diego HMP, is to develop a set of appropriate parameter ranges to account for variations of these key parameters. The final selection of rainfall loss parameters and evaporation data is part of the Sizing Calculator development process as part of HMP implementation in winter 2010.

In addition to the reports listed below, other TMDL reports from San Diego County and elsewhere in Southern California were reviewed in Table 4-3. However, only those reports with a substantial description of modeling activities were summarized in the table.

Table 4-3. Summary of HSPF Modeling Reports for Southern California

No.	Title	Authors	Date	Summary/Comments
1	TMDL to Reduce Bacterial Indicator Densities at Santa Monica Bay Beaches During Wet Weather (Preliminary Draft)	Los Angeles RWQCB / Tetra Tech	June 21, 2002	Combination of hydrologic and water quality modeling to estimate bacterial loadings to Santa Monica Bay HSPF/Loading Simulation Program in C++ (LSPC) model was calibrated and validated using stream flow data collected on Malibu Creek and Ballona Creek. (LSPC is a recoded C++ version of HSPF) No HSPF model parameters included in report
2	Technical Report – TMDLs for Indicator Bacteria in Baby Beach and Shelter Island Shoreline Park	San Diego RWQCB / Tetra Tech	June 11, 2008	Combination of hydrologic and water quality modeling HSPF/LSPC model was calibrated to flow data collected in Aliso Creek and Rose Creek. Calibrated infiltration rates were reported for Natural Resources Conservation Survey (NRCS) Group A, B, C, and D soils (in Appendix F). However, it is unclear if these rates correspond to specific HSPF model parameters. This issue of how to apply the calibrated infiltration rates should be addressed through correspondence with study authors.
3	Evaluating HSPF in an Arid, Urbanized Watershed (in Journal of the American Water Resources Association, 2005, p477-486	Drew Ackerman, Kenneth Schiff, Stephen Weisburg (SCCWRP)	February 2005	HSPF was used to simulate hydrologic processes in arid region, e.g., precipitation on dry soils, effect of irrigation. Model was calibrated to gauge data collected in lower reaches of Malibu Creek. The calibration set aggregated the soil and land cover variations in the watershed (i.e., spatially "lumped" parameters). Pervious land surface (PWATER) parameters were included in the paper.
4	TMDL for Indicator Bacteria Project I – Beaches and Creeks in the San Diego Region	San Diego RWQCB / Tetra Tech	December 12, 2007	HSPF/LSPC model parameters were selected from regional calibration. Calibration efforts used daily average stream flows as the baseline calibration condition. Appendices describe the regional calibration process. The modeling files have been provided by the San Diego RWQCB
5	Lake Elsinore and Canyon Lake Nutrient Source Assessment (Final Report) for Santa Ana Watershed Project Authority	Tetra Tech, Inc.	January 2003	HSPF/LSPC model was calibrated and validated using United States Geological Survey (USGS) gauging site data in the San Jacinto watershed Model simulated pollutant loading to Lake Elsinore and Canyon Lake Pervious land surface (PWATER) parameters were not published in the report.

The technical reports listed in Table 4-3 demonstrate that a variety of detailed HSPF modeling studies have been conducted in the past 10 years in Southern California. However, adapting these modeling efforts for use on the San Diego HMP project would require additional work. This is because the reports listed above did not publish their HSPF parameter sets, with the exception of the Ackerman study (see No. 3 above), which published a set of generalized parameters that aggregate or "spatially lump" the contributions of different soil/land use combinations in the upper watershed.

The HSPF model described in the Ackerman paper simulates all soil and land use combinations using a single composite parameter set. In a follow-on conversation in May 2008, Drew Ackerman explained that the "Arid, Urbanized" HSPF model was calibrated only to gauge data in the lower Santa Monica Bay watershed, because the model's purpose was to estimate pollutant loadings to area beaches and water bodies. His study was understandably less focused on characterizing the variation in runoff rates and volumes among the different land uses in the upper portions of the watershed. Additionally, the effect of upstream surface water impoundments would have made the development of an accurate, detailed calibration at the sub-catchment scale very difficult to achieve. Unfortunately, this "spatially lumped" parameter set is of limited usefulness for the purpose of the HMP project, given the need to develop parameter sets that describe a variety of common soil and land use combinations.

Continuous simulation modeling files associated with the report titled *Bacteria Project I – Beaches and Creeks in the San Diego Region* (February 2009) include infiltration parameter calibrations based upon 15-20 years of average daily flows. Per discussions with Tetra Tech in November 2008 and January 2009, ongoing work related to TMDL development for San Diego County lagoons may also prove to be beneficial to the future San Diego HMP model parameter estimation effort.

The consultant team will continue to review additional HSPF studies in preparation for development of a hydromodification flow control sizing tool for San Diego County (to be completed in Winter 2010). The consultant team has had discussions with Tony Donigian of Aquaterra, who has prepared numerous HSPF models and serves as an EPA-sponsored trainer for HSPF modeling.

Aquaterra's HSPF modeling efforts in Southern California have focused on Ventura County. Aquaterra has requested permission from Ventura County to allow the San Diego HMP consultant team to review modeling results and input data sets for the Ventura County HSPF modeling efforts.

To better utilize the existing HPSF models for use in the San Diego HMP project and development of the San Diego HMP Sizing Calculator, the consultant team will be conducting the following activities:

- Contact the authors of the studies listed in Table 4-3 (and others provided by Tetra Tech, Aquaterra and SCCWRP) to obtain copies of the HSPF pervious land surface (PERLND/PWATER) parameter sets. .
- Relate the HSPF parameters to NRCS soil groups and common land use types. Develop a range of recommended HSPF input parameters that could be used to characterize the range of soil and land use types common to San Diego County.

The following model parameters were published in the Drew Ackerman et al. paper described in Figure 4-2. The specific values were selected by calibrating an HSPF model to flow monitoring data in the Santa Monica Bay watershed, specifically on Malibu Creek. The values represent a composite of the various upstream soils and land uses.

Table 2. Model parameters utilized for modeling of Santa Monica Bay.

Pervious Parameters		Value	Units
Fraction of Remaining E-T from Active Groundwater Storage	AGEWTP	0.05	None
Basic Groundwater Recession Rate	AGWRC	0.92	1/d
Fraction of Remaining E-T from Baseflow	BASETP	0.05	None
Interception Storage Capacity	CEPSC	0.25	cm
Fraction of Groundwater to Deep Aquifer	DEEPFR	0.40	None
Forest Fraction	FOREST	0.0	%
Infiltration Equation Exponent	INFEXP	2.0	None
Ratio between the Maximum and Mean Infiltration Capacities	INFILD	2.0	None
Infiltration Capacity	INFILT	0.10	cm/hr
Interflow Inflow Parameter	INTFW	1.50	None
Interflow Recession Parameter	IRC	0.70	1/d
Groundwater Recession Flow Coefficient	KVARY	7.6	1/cm
Overland Flow Length	LSUR	61	m
Lower Zone E-T Parameter	LZETP	0.70	None
Lower Zone Nominal Storage	LZSN	25	cm
Manning's n for Overland Flow	NSUR	0.20	Complex
Temperature Maximum for Evapotranspiration (E-T)	PETMAX	1.7	°C
Temperature that E-T is Zero	PETMIN	-1.1	°C
Overland Flow Slope	SLSUR	0.03	None
Upper Zone Nominal Storage	UZSN	3.0	cm

Figure 4-2. Model Parameters Utilized by SCCWRP for Modeling of Santa Monica Bay

Additional reference material can be located in the document titled, *BASINS Technical Notice 6, Estimating Hydrology and Hydraulic Parameters for HSPF,* prepared by the U.S. EPA (July 2000). This document provides details regarding pervious and impervious land hydrology parameters along with flow routing parameters. Parameter and value range summary tables are included in the document.

4.4.1 Pervious Land Hydrology (PWATER) Parameters

The HSPF hydrology parameters of PWATER are divided into four sections, titled PARM1-4. PARM1 is a series of checks to outline any monthly variability versus constant parameter values within the simulated algorithm; whereas, PARM2 and 3 are a series of climate, geology, topography, and vegetation parameters that require numerical values to be inputted.

PARM2 involves the basic geometry of the overland flow, the impact of groundwater recession, potential snow impact due to forest cover and the expected infiltration and soil moisture storage. The main parameters of groundwater recession are KVARY and AGWRC. The infiltration and soil moisture storage parameters are INFILT and LZSN.

PARM3 involves the impact of climate temperature during active snow conditions, a wide range of evaporation parameters due to the variability of the onsite soil and existing vegetation and subsurface losses due to groundwater recharge or the existing geology. The main evaporation parameters are INFEXP, INFILD, BASETP, and AGWETP. The parameter for subsurface loss is DEEPFR which accounts for one of only three major losses from the PWATER water balance (i.e., in addition to evaporation, and lateral and stream outflows).

PARM4 involves the flow and hydrograph characteristics, the expectation of rain interception due to the inherent moisture storage capacity from existing vegetation, land use and/or near surface soil conditions and evaporation due to the root zone of the soil profile. The main interception parameters are CEPSC and UZSN. The parameter for evaporation as a primary function of vegetation is LZETP.

PARM2

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. Value ranges are shown in Figure 4-3. 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 can be used to estimate the recession rate from observed daily flow data (such as plotting on a logarithmic scale). Value ranges are shown in Figure 4-3.

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 runoff. 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:

Table 4-4. INFILT Parameters

SCS Hydrologic Soil Group	INFILT Estimate		Runoff Potential
	(in/hr)	(mm/hr)	
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.

LZSN. Lower zone nominal soil moisture storage *(inches), (estimate, then calibrate)*.

LZSN is related to both precipitation patterns and soil characteristics in the region. 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.

PARM3

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).*

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 gauge used for calibration, such as flow around or under the gauge site. 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 gauge. DEEPFR should be set to 0.0 initially or estimated based on groundwater studies, and then calibrated, in conjunction with adjustments to evapotranspiration (ET) parameters.

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 Figure 4-3. If significant riparian vegetation is present in the watershed then non-zero values of BASETP should be used. 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 pervious land segment (PLS), 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 affect 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.

PARM4

CEPSC. Amount of rainfall, in inches, which is retained by vegetation, that 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 4-5. Interception Parameters

Land Cover	Maximum Interception (in)
Grassland	0.10
Cropland	0.10 – 0.25
Forest Cover, light	0.15
Forest Cover, heavy	0.20

LZETP. 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 Figure 4-3, and the following ranges for different vegetation are expected for the ‘maximum’ value during the year:

Table 4-6. LZETP Coefficients

Land Cover	Input Coefficient
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

HSPF HYDROLOGY PARAMETERS AND VALUE RANGES

			RANGE OF VALUES					
NAME	DEFINITION	UNITS	TYPICAL		POSSIBLE		FUNCTION OF ...	COMMENT
			MIN	MAX	MIN	MAX		
PWAT - PARM2								
FOREST	Fraction forest cover	none	0.0	0.50	0.0	0.95	Forest cover	Only impact when SNOW is active
LZSN	Lower Zone Nominal Soil Moisture Storage	inches	3.0	8.0	2.0	15.0	Soils, climate	**Calibration**
INFILT	Index to Infiltration Capacity	in/hr	0.01	0.25	0.001	0.50	Soils, land use	**Calibration**, divides surface and subsurface flow
LSUR	Length of overland flow	feet	200	500	100	700	Topography	Estimate from high resolution topo maps or GIS
SLSUR	Slope of overland flow plane	ft/ft	0.01	0.15	0.001	0.30	Topography	Estimate from high resolution topo maps or GIS
KVARY	Variable groundwater recession	1/inches	0.0	3.0	0.0	5.0	Baseflow recession variation	Used when recession rate varies with GW levels
AGWRC	Base groundwater recession	none	0.92	0.99	0.85	0.999	Baseflow recession	**Calibration**
PWAT - PARM3								
PETMAX	Temp below which ET is reduced	deg. F	35.0	45.0	32.0	48.0	Climate, vegetation	Reduces ET near freezing, when SNOW is active
PETMIN	Temp below which ET is set to zero	deg. F	30.0	35.0	30.0	40.0	Climate, vegetation	Reduces ET near freezing, when SNOW is active
INFEXP	Exponent in infiltration equation	none	2.0	2.0	1.0	3.0	Soils variability	Usually default to 2.0
INFILD	Ratio of max/mean infiltration capacities	none	2.0	2.0	1.0	3.0	Soils variability	Usually default to 2.0
DEEPFR	Fraction of GW inflow to deep recharge	none	0.0	0.20	0.0	0.50	Geology, GW recharge	Accounts for subsurface losses
BASETP	Fraction of remaining ET from baseflow	none	0.0	0.05	0.0	0.20	Riparian vegetation	Direct ET from riparian vegetation
AGWETP	Fraction of remaining ET from active GW	none	0.0	0.05	0.0	0.20	Marsh/wetlands extent	Direct ET from shallow GW
PWAT - PARM4								
CEPSC	Interception storage capacity	inches	0.03	0.20	0.01	0.40	Vegetation type/density, land use	Monthly values usually used
UZSN	Upper zone nominal soil moisture storage	inches	0.10	1.0	0.05	2.0	Surface soil conditions, land use	Accounts for near surface retention
NSUR	Manning's n (roughness) for overland flow	none	0.15	0.35	0.05	0.50	Surface conditions, residue, etc.	Monthly values often used for croplands
INTFW	Interflow inflow parameter	none	1.0	3.0	1.0	10.0	Soils, topography, land use	**Calibration,** based on hydrograph separation
IRC	Interflow recession parameter	none	0.5	0.7	0.3	0.85	Soils, topography, land use	Often start with a value of 0.7, and then adjust
LZETP	Lower zone ET parameter	none	0.2	0.7	0.1	0.9	Vegetation type/density, root depth	**Calibration**

Figure 4-3. HSPF Hydrology Parameters and Value Ranges

Source: USEPA BASINS Technical Note 6

Model assumptions for stream reach infiltration rates were derived through calibration based on data collected within the reaches of Aliso Creek (11 stations) and Rose Creek (6 stations). In the model, infiltration rates vary by soil type. Stream infiltration was calibrated by adjusting a single infiltration value, which was varied for each soil type by factors established from literature ranges (USEPA 2000) of infiltration rates specific to each soil type. The final resulting infiltration rates were 1.368 in/hr (Soil Group A), 0.698 in/hr (Soil Group B), 0.209 in/hr (Soil Group C) and 0.084 in/hr (Soil Group D). The infiltration rates for Soil Groups B, C, and D are within the infiltration range given in literature (Wanielisata et al. 1997). The result for Soil Group A is below the range given in Wanielisata et al. (1997); however, this result only represented one watershed in this TMDL study.

The technical reports reviewed demonstrate that a variety of detailed HSPF modeling studies have been conducted in the past 10 years in Southern California. However, adapting these modeling efforts for use on the San Diego HMP project will require additional work, which will be completed in association with development of the implementation Sizing Calculator. That effort includes meetings with report authors, including representatives from the SCCWRP, as well as meetings with HSPF modeling experts from Aquaterra to ascertain appropriate values for initial infiltration parameters.

4.5 Rainfall Losses - Evapotranspiration Parameters

Standards developed as part of this HMP to control runoff peak flows and durations are based on a continuous simulation of rainfall runoff using locally derived parameters for evaporation and evapotranspiration.

Known data sources for evaporation and evapotranspiration data in San Diego County are listed below.

- California Irrigation Management and Information System web site – evapotranspiration stations include San Diego, Oceanside, Escondido, Ramona, Otay Lakes, Miramar, Torrey Pines, and Borrego Springs.
- Historical Reservoir Level and Evaporation Data for Lake Heneshaw.
- Historical Evaporation Data from City of San Diego Reservoirs.
- Historical Evaporation Data from Helix Water District for Lake Cuyamaca.

The evaporation / evapotranspiration parameter validation process includes documentation of the source of data and analysis of the effects of varying patterns between the subject site and parameter data source. A full review of local pan evaporation and potential evapotranspiration data is being conducted as part of the development of the final hydromodification flow control sizing tool.

Table 4-6 below summarizes available evaporation and evapotranspiration data sources in San Diego County. Most of the available evaporation data are located close to reservoirs in the inland valley and mountain areas of the County. Monthly evaporation records are available for multiple reservoirs within the County. Evapotranspiration sensing data are generally collected in agricultural zones.

The California Irrigation Management Information Systems web site (wwwcimis.water.ca.gov/cimis/data.jsp) provides access to real-time and summarized evapotranspiration data (ETo) throughout California. For the San Diego region, average evapotranspiration values are summarized for the coastal and foothill zones of San Diego County.

Table 4-7. Summary of Evaporation and Evapotranspiration Data for San Diego County

Station Name ID	Data Type	Data Source	Recording Frequency	Start Date	End Date
Barratt Lake	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Borrego Springs	Evapotranspiration	CIMIS	Monthly	2008	2008
Chula Vista	Pan Evaporation	Western Regional Climate Center	Monthly Averages	1948	2005
El Capitain Reservoir	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Escondido / 74	Evapotranspiration	CIMIS	Monthly	1988	1998
Escondido / 153	Evapotranspiration	CIMIS	Monthly	1999	2008
Lake Cuyamaca	Pan Evaporation	Helix Water District	Monthly	1985	2006
Lake Heneshaw	Pan Evaporation	County of San Diego	Daily	1999	2005
Lake Heneshaw	Pan Evaporation	County of San Diego	Monthly	1957	2008
Lake Hodges	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Lake Jennings	Pan Evaporation	Helix Water District	Monthly	1985	2006
Lake Murray	Pan Evaporation	City of San Diego	Monthly	1950	2008

Table 4-7. Summary of Evaporation and Evapotranspiration Data for San Diego County					
Station Name ID	**Data Type**	**Data Source**	**Recording Frequency**	**Start Date**	**End Date**
		Water Department			
Lake Sutherland	Pan Evaporation	City of San Diego Water Department	Monthly	1954	2008
Lower Otay Reservoir	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Lower Otay / 147	Evapotranspiration	CIMIS	Monthly	1999	2008
Miramar Lake	Pan Evaporation	City of San Diego Water Department	Monthly	1960	2008
Miramar Lake / 150	Evapotranspiration	CIMIS	Monthly	1999	2008
Morena Lake	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Oceanside / 49	Evapotranspiration	CIMIS	Monthly	1986	2003
Ramona / 98	Evapotranspiration	CIMIS	Monthly	1991	1998
San Diego / 45	Evapotranspiration	CIMIS	Monthly	1985	1989
San Diego / 66	Evapotranspiration	CIMIS	Monthly	1989	2001
San Diego II / 184	Evapotranspiration	CIMIS	Monthly	2002	2008
San Vicente Reservoir	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Torrey Pines / 173	Evapotranspiration	CIMIS	Monthly	2000	2008

Long-term evaporation / evapotranspiration data sets are being generated to correspond with long-term rainfall records. The final selection of rainfall loss parameters and evaporation data is part of the Sizing Calculator development process.

In summary, the published literature reviewed as part of this study support the methods and approach taken in developing the San Diego Hydromodification Management Plan.

5. METHODOLOGY AND TECHNICAL APPROACH TO REGIONAL HYDROMODIFICATION DEVELOPMENT

As outlined in Permit Section D.1, the San Diego Copermittees shall implement a program to manage increases in runoff discharge rates and durations from Priority Development Projects that are likely to cause increases to erosion of channel beds or banks, silt pollutant generation, or other impacts to beneficial uses and stream habitat due to increased erosive force. This section provides a detailed description of the methodology and approach used in the development of the HMP (Permit Section D.1.g(1)). Section 5.1 specifically focuses on the approach taken to identify the geomorphically significant flow range, Section 5.2 focuses on channel screening tools developed in association with this HMP, and Section 5.3 discusses cumulative watershed impacts.

5.1 Flow Control Limit Determination

5.1.1 Background

The purpose of the HMP is to identify guidelines for managing 'geomorphically-significant' flows that, if not controlled, would cause increased erosion in receiving water channels. Specifically, the HMP must identify low and high flow thresholds between which flows should be controlled so that the post-project flow rates and durations do not exceed pre-project levels between these two flow magnitudes. Specifically, the Board Order requires that the HMP shall:

> Utilize continuous simulation of the entire rainfall record to identify a range of runoff flow[1] for which Priority Development Project post-project runoff flow rates and durations shall not exceed pre-project runoff flow rates and durations, where the increased flow rates and durations will result in increased potential for erosion or other significant adverse impacts to beneficial uses, attributable to changes in the flow rates and durations. The lower boundary of the range of runoff flows identify shall correspond with the critical channel flow that produces the critical shear stress that initiates channel bed movement or that erodes the toe of channel banks. The identified range of runoff flows may be different for specific watersheds, channels or channel reaches.

For the purposes of this project, 'hydrograph modification' or 'hydromodification' is understood to mean changes to the frequency, duration and magnitude of surface runoff that, when unmitigated, cause an increase in erosion of the receiving water body. Hydromodification occurs when urbanization replaces areas of vegetated, uncompacted soil with impermeable surfaces such as buildings, roads, and compacted fill. The reduction in permeability results in increased volumes of runoff, and faster and more concentrated delivery of this water to receiving waters. These changes have the potential to cause creeks to erode faster than before development. Although the focus of hydromodification management plans has been on increased erosion, it should be noted that in rivers that are depositional, hydromodification can cause creeks to regain some

[1] The identified range of runoff flows to be controlled should be expressed in terms of peak flow rates of rainfall events, such as "10% of the pre-project 2-year peak flow up to the pre-project 10-year peak flow."

transport equilibrium. This phenomenon is the basis for providing exemptions for river reaches which are aggrading (depositional).

Stream flows are often expressed in terms of the frequency with which a particular flow occurs. For example, Q_2 refers to the flow rate that occurs once every two years, on average over the long term runoff record. Flow frequencies are a function of rainfall and watershed characteristics, and are unique to each stream channel (and location along the channel). The effects of urbanization tend to increase the magnitude of the flow associated with a given frequency (e.g., post-development Q_2. is higher than pre-development Q_2.). Similarly, urbanization tends to increase the frequency with which any given flow rate occurs. The purpose of the HMP is to control runoff from new developments so that flow magnitudes and frequencies match pre-development conditions within a critical range of flows.

Not all runoff causes erosion. Runoff in receiving channels below a critical discharge (Qcrit) does not exert sufficient force to overcome the erosion resistance of the channel banks and bed materials. Flows greater than Qcrit cause erosion, with larger flows causing proportionally greater erosion. It has been determined through calculations and field measurements that most erosion in natural creeks is caused by flows between some fraction of Q_2 and Q_{10}. Flows in this range are referred to as 'geomorphically-significant' because they cause the majority of erosion and sediment transport in a channel system.

Flows greater than Q_{10}, though highly erosive *per event*, occur too infrequently to do as much work as smaller but more frequent flows. Hydromodification also has less impact on flows greater than Q_{10} since at such high rainfall intensities, the soil becomes saturated and the infiltration capacity of undeveloped landscapes is rapidly exceeded. When the soil is saturated, runoff rates become more similar to those from impervious surfaces. For these reasons, HMPs have focused on identifying a low flow threshold that is close to Qcrit for most receiving channels, and controlling flows between that value and Q_{10} (see Literature Review in Chapter 4 for review of HMPs completed in Santa Clara, Contra Costa, Alameda, and San Mateo Counties). By requiring mitigation (storage and either infiltration or detention) of excess runoff within the control range, and by limiting the release of excess water to Qcrit or less, HMPs seek to prevent additional erosion in receiving water channels.

5.1.2 Identifying a Low Flow Threshold

Erosion occurs when the shear stress exerted on the channel by flowing water (*boundary shear stress*) exceeds the resistance of the channel (*critical shear stress*). Critical shear stress varies by several orders of magnitude for different channel materials (Figure 5-1). *Critical flow* (Qcrit) is the channel flow which produces boundary shear stress equal to the critical shear stress for a given channel. In other words, critical flow is the flow rate that can initiate erosion in a channel. Qcrit is a function not only of the critical shear stress of the channel materials, but also channel size and channel geometry. A particular flow rate (expressed as a number of cubic feet per second) in a small, steep, confined channel will create more shear stress than the identical flow rate in a large, flat, wide open channel. Thus, Qcrit can be extremely variable depending on channel and watershed characteristics and will be different in each channel, and in each watershed.

Boundary Category	Boundary Type	Permissible Shear Stress (lb/sq ft)
Soils	Fine colloidal sand	0.02 - 0.03
	Sandy loam (noncolloidal)	0.03 - 0.04
	Alluvial silt (noncolloidal)	0.045 - 0.05
	Silty loam (noncolloidal)	0.045 - 0.05
	Firm loam	0.075
	Fine gravels	0.075
	Stiff clay	0.26
	Alluvial silt (colloidal)	0.26
	Graded loam to cobbles	0.38
	Graded silts to cobbles	0.43
	Shales and hardpan	0.67
Gravel/Cobble	1-in.	0.33
	2-in.	0.67
	6-in.	2.0
	12-in.	4.0
Vegetation	Class A turf	3.7
	Class B turf	2.1
	Class C turf	1.0
	Long native grasses	1.2 – 1.7
	Short native and bunch grass	0.7 - 0.95
	Reed plantings	0.1-0.6
	Hardwood tree plantings	0.41-2.5
Temporary Degradable RECPs	Jute net	0.45
	Straw with net	1.5 – 1.65
	Coconut fiber with net	2.25
	Fiberglass roving	2.00
Non-Degradable RECPs	Unvegetated	3.00
	Partially established	4.0-6.0
	Fully vegetated	8.00
Riprap	6 – in. d_{50}	2.5
	9 – in. d_{50}	3.8
	12 – in. d_{50}	5.1
	18 – in. d_{50}	7.6
	24 – in. d_{50}	10.1
Soil Bioengineering	Wattles	0.2 – 1.0
	Reed fascine	0.6-1.25
	Coir roll	3 - 5
	Vegetated coir mat	4 - 8
	Live brush mattress (initial)	0.4 – 4.1
	Live brush mattress (grown)	3.90-8.2
	Brush layering (initial/grown)	0.4 – 6.25
	Live fascine	1.25-3.10
	Live willow stakes	2.10-3.10
Hard Surfacing	Gabions	10
	Concrete	12.5

[1] Ranges of values generally reflect multiple sources of data or different t

A. Chang, H.H. (1988).
B. Florineth. (1982)
C. Gerstgraser, C. (1998).
D. Goff, K. (1999).
E. Gray, D.H., and Sotir, R.B. (1996).
F. Julien, P.Y. (1995).
G. Kouwen, N.; Li, R. M.; and Simons, D.B., (1980).
H. Norman, J. N. (1975).
I. Schiechtl, H. M. and R. Stern. (1996).
J. Schoklitsch, A. (1937).

Figure 5-1. Range of Critical Shear Stresses (τ_{cr}) for Different Materials (from Fischenich)

It was the original intent of the HMP consulting team to identify a single low flow threshold for the entire County (per previous HMPs). However, an extensive assessment of channel and runoff conditions led the team to conclude that there was a very wide range in critical flows, based largely on channel material but also on channel dimensions, rainfall, and watershed area. Adopting a single standard that is conservative for the most vulnerable channels would result in controls that were excessively conservative for more resilient channels, while adopting an 'average' value would leave some channels unprotected.

Because of this natural variability, the team pursued an analytical approach for estimating Qcrit as a function of parameters such as channel materials, channel dimensions, and watershed area. Because the low flow standard is required to correspond to Qcrit (Order No. R9-2007-0001), this approach allows the low flow standard to be customized for local conditions. The following sections describe an analysis of Qcrit as a fraction of Q_2 for the range of channel conditions in San Diego County. This is followed by a description of a calculator tool that may be used to calculate Qcrit for a specific channel based on parameters that may be readily measured in the field. The analyses described in this report provide background for the selection of low flow thresholds identified in the HMP.

5.1.3 Critical Flow Analysis

The low flow thresholds were calculated by conducting a sensitivity analysis in which a wide range of channel sizes and geometries, rainfalls, watershed areas and channel materials were modeled in a flow-erosion model to identify Qcrit as a function of Q_2. In all, 170 combinations of channel, rainfall, and watershed conditions were assessed. Based on the results of this sensitivity analysis, a series of low flow thresholds was identified.

The steps used to conduct the sensitivity analysis and determine the recommended flow thresholds were as follows:

1. Identify the typical range of rainfall conditions for the HMP area (western San Diego County).
2. Identify the range of typical watershed areas likely to be developed.
3. Identify a range of typical receiving channel dimensions for each watershed area.
4. Identify a range of typical channel materials for receiving channels.
5. Simulate a range of flows and develop rating curves (relationships between discharge and boundary shear stress).
6. Identify the flow rate at which boundary shear stress exceeds critical shear stress for the channel and material.
7. Express this flow rate as a function of Q_2.
8. Group critical flow rates by channel materials and identify appropriate low flow thresholds for each channel material type.

Steps 1 through 4 were used to define the range of parameters to use in the sensitivity testing. The intent was to identify a typical range of conditions likely to occur in the HMP area (western San Diego county), rather than provide an exhaustive description of possible watershed and channel conditions. Sensitivity testing on many combinations of parameters within this typical range allows identification of the range of channel responses and appropriate flow thresholds.

Each step in the critical flow analysis is explained in detail in the following sections.

Identify the Typical Range Of Rainfall Conditions for the HMP Area (West San Diego County)

Mean annual rainfall was used to estimate receiving channel size, Q_2, Q_5 and Q_{10} (methods described in subsequent sections). Figure 5-2 shows mean annual rainfall for San Diego County. Based on the map, three mean annual rainfalls were selected to represent the range of rainfall conditions for the simulations: 10-inch, 20-inch, and 30-inch.

Identify the Range of Typical Watershed Areas Likely to be Developed

Based on discussions with the TAC, a range of representative watershed areas for development projects was identified. These were: 0.1 sq mi, 0.5 sq mi, 1 sq mi, and 2 sq mi. The consultant team assumed that in project watersheds larger than 2 sq mi the development would either require site specific continuous simulation modeling, or would be broken into multiple smaller sub watersheds with individual points of compliance.

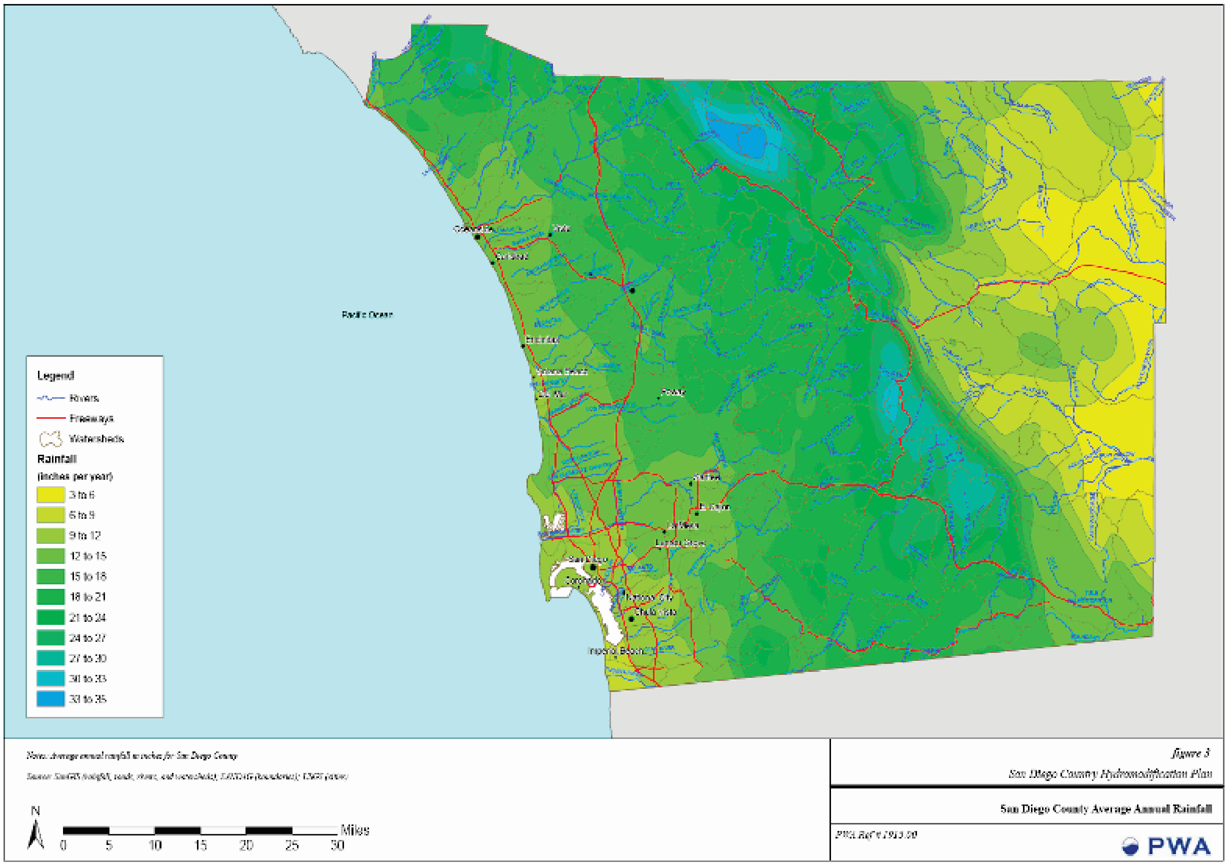

Figure 5-2. Rainfall Distribution in San Diego County

Identify a Range of Typical Receiving Channel Dimensions for Each Watershed Area

Empirical relationships have been developed to express channel dimensions (width, depth and, to a lesser extent, gradient) as a function of dominant discharge. Dominant discharge for a creek channel is the flow rate that transports the majority of sediment and creates/maintains the characteristic size and shape of the channel over time. Dominant discharge may also be referred to as bankfull flow. For undeveloped channels in semi arid parts of the US, dominant discharge is approximately equivalent to Q_5. For example, Coleman et al. (2005) found dominant discharge for streams in Southern California to average $Q_{3.5}$ (range = $Q_{2.1}$ – $Q_{6.7}$). Goodwin (1998) found dominant discharge to vary from Q_2 to Q_{10} for semi arid regions.

To capture natural variability in channel geometry, three different empirical channel geometry relationships were used to estimate receiving channel dimensions for the range of watershed areas and rainfall characteristics used in this study. The relationships were:

Coleman et al. 2005 (modified by Stein – personal communication) – derived from undeveloped channels in Southern California, tends to predict narrow, deep, steep dimensions.

$$\text{Width (ft)} = 0.6012 * Q_{bf}^{0.6875}$$
$$\text{Depth (ft)} = 0.3854 * Q_{bf}^{0.3652}$$

Where Q_{bf} is in cfs.

Parker et al. 2007 – suitable for gravel channels, tends to predict wide, shallow, flat braided dimensions.

$$\text{Width (m)} = 4.63 * (Q_{bf}^{2/5}) / (9.81^{1/5}) * (Q_{bf} / \text{Sqrt}(9.81 * d_{50}) * d_{50}^{2}))^{0.0667}$$
$$\text{Depth (m)} = 0.382*((Q_{bf}^{2/5})/(9.81^{1/5})))$$

Where Q_{bf} is bankfull discharge in m^3/sec and d_{50} (diameter of median channel material) is in m.

The Parker equation was only used to assess gravel and cobble channel conditions.

Hey and Thorne 1986 tends to predict medium width, depth, and gradient channels.

$$\text{Width (m)} = 2.73*Q_{bf}^{0.5}$$

$$\text{Depth (m)} = 0.22 * \text{Width}^{0.37} * d_{50}^{-0.11}$$

Where Q_{bf} is in m^3/sec and d_{50} is in m.

(Note that original combinations of English and metric units described in the source papers were used rather than standardized these equations in one set of measurements.)

The three equations cover a wide range of likely field conditions, from deeply incised channels (Coleman et al. 2005) to wide, braided conditions (Parker 2007). Note that for the sensitivity analysis we set d50 in the Parker et al equation to the d_{50} of the channel material being tested, and did not use the equation for channels where the material was sand or silt.

The equations produce estimations of width and depth. To estimate a slope for each combination of channel dimensions, the velocity associated with each cross section was calculated (by dividing discharge by width multiplied by depth) and then the slope was calculated that corresponded with that velocity using Manning's equation.

$$\text{Velocity (ft/sec)} = \frac{1.486\ HR^{0.67} * s^{0.5}}{n}$$

Where HR is channel hydraulic radius, s is slope, and n is Manning's roughness coefficient (see definitions).

For the purposes of the sensitivity analysis, a value of n of 0.035 was assumed, corresponding to a non vegetated, straight channel with no riffles and pools. This is a reflection of the small, ephemeral receiving channels which are most prevalent in Southern California developments. A relatively low value was used at the request of the San Diego RWQCB so that the values erred on the conservative side. Some members of the TAC considered the value of n to be too conservative.

These equations all require a value for bankfull discharge. Bankfull discharge (assumed to be approximately Q_5) was estimated using the USGS regional regression for undeveloped watersheds in the South Coast region (Waananen and Crippen 1977). This equation calculates Q_5 as a function of watershed area and mean annual precipitation, based on empirical observations of USGS gauges. The relationship is:

$$Q5\ \text{(cfs)} = 0.4 * \text{Watershed Area}^{0.77} * \text{Mean Annual Precipitation}^{1.69}$$

Where watershed area is in square miles and precipitation is in inches.

For each combination of typical watershed area and mean annual rainfall, Q_5 was calculated using the USGS regression equation, then three sets of channel dimensions were calculated based on the three channel equations. This provided the range of channel conditions to simulate for the critical flow analysis. The total number of channel conditions was as follows:

3 rainfalls (10, 20, 30 inches per year)
4 watershed areas (0.1, 0.5, 1, 2 square miles)
3 channel width, depth, and slope combinations (narrow/deep, medium, wide/shallow)
= 36 combinations of receiving channel geometry

Identify a Range of Typical Channel Materials for Receiving Channels

The consultant team identified a range of typical channel materials based on feedback from the TAC and experience gained working in San Diego County. The identified materials are not intended as a comprehensive list of possible channel materials, but to cover the range of critical shear stresses likely to be encountered in typical western San Diego County channels. The identified range is as follows:

Table 5-1. Critical Shear Stress Range in San Diego County Channels

Material	Critical Shear Stress (lb/sq ft)
Coarse Unconsolidated Sand	0.01
Alluvial Silt (Non Colloidal)	0.045
Medium gravel	0.12
Alluvial silt/clay	0.26
2.5 inch cobble	1.1

Combining the five channel material types (coarse unconsolidated sand, alluvial silt, medium gravel, alluvial silt/clay, and 2.5 inch cobble) with the 36 combinations of channel geometry produces 180 potential combinations of receiving channel characteristics. Ten sets of combinations were omitted from the analysis because they produced physically unrealistic conditions, such as slopes that were too steep to be developed. Exclusion of these results did not significantly affect the overall results.

Develop Shear Stress Rating Curves

Rating curves for the 36 different combinations of receiving channel characteristics were developed using the same Excel worksheet that forms the basis for the Qcrit calculator developed for Track 2 (described in later sections). Using channel cross section, roughness, and gradient input by the user, the tool calculates the average boundary shear stress associated with a range of different flow depths to construct a rating curve (discharge on the x axis versus shear stress on the y axis). It then identifies the flow rate where average boundary shear stress equals critical shear stress for the channel materials. This is the critical flow (Qcrit). By dividing this number by Q_2, we identify the low flow threshold for each simulation as a function of Q_2. (e.g., $0.1Q_2$ where the critical flow is one tenth of the Q_2 flow).

The tool calculates a shear stress rating curve for a range of flows between 1 and 100 percent of the bankfull flow depth. Bankfull flow depth is defined as the flow depth that corresponds to the dominant discharge for a given channel. The range 1 to 100 percent of bankfull is used because critical flow rarely falls outside these values. The tool then calculates a power function between the points to allow for interpolation. For each of the depths, the tool calculates discharge and average boundary shear stress exerted on the bed, as described below.

Calculating Average Boundary Shear Stress

Average boundary shear stress is the force that flowing water exerts on channel materials. For a given channel cross-section, it is calculated as follows:

$$\tau_b = \gamma * HR * s$$

where τ_b = average boundary shear stress (lb/ft^2)

γ = unit weight water (62.4 lb/ft^3)

HR = Hydraulic radius (cross section area / wetted perimeter)

s = channel slope (ft/ft)

For each depth increment between 1 and 100 percent of bankfull, cross section area, wetted perimeter, HR and τ_b are calculated. Slope is a constant for the cross section. These calculations produce a rating curve for boundary shear as a function of flow depth.

Calculating Discharge

This step converts flow depth to flow rate (Q) so that the rating curve may be expressed as a function of Q. For each depth increment between 1 and 100 percent of bankfull, the flow rate is calculated using Manning's equation:

$$\text{Velocity (ft/sec)} = 1.486 \frac{HR^{0.67} * s^{0.5}}{n}$$

where V = velocity (ft/sec)

n = Manning's roughness coefficient

HR = hydraulic radius (ft)

For the sensitivity analysis, Manning's n was assumed to be 0.035, which is typical for a non-vegetated ephemeral channel. This assumption was made because most developments covered by the HMP would discharge to receiving channels relatively high in the watershed and with little summer flow. Interim sensitivity analysis found that relative to other factors such as critical shear stress, the range of roughness factors found in receiving channels had little effect on the estimated critical shear flow rate.

Discharge is calculated as velocity multiplied by cross section area (calculated for each cross section). The result of these calculations is a rating curve showing boundary shear stress for the receiving channel as a function of discharge, with the highest point representing bankfull depth (see Figure 5-3 below). Rating curves were created for each of the 36 combinations of channel characteristics.

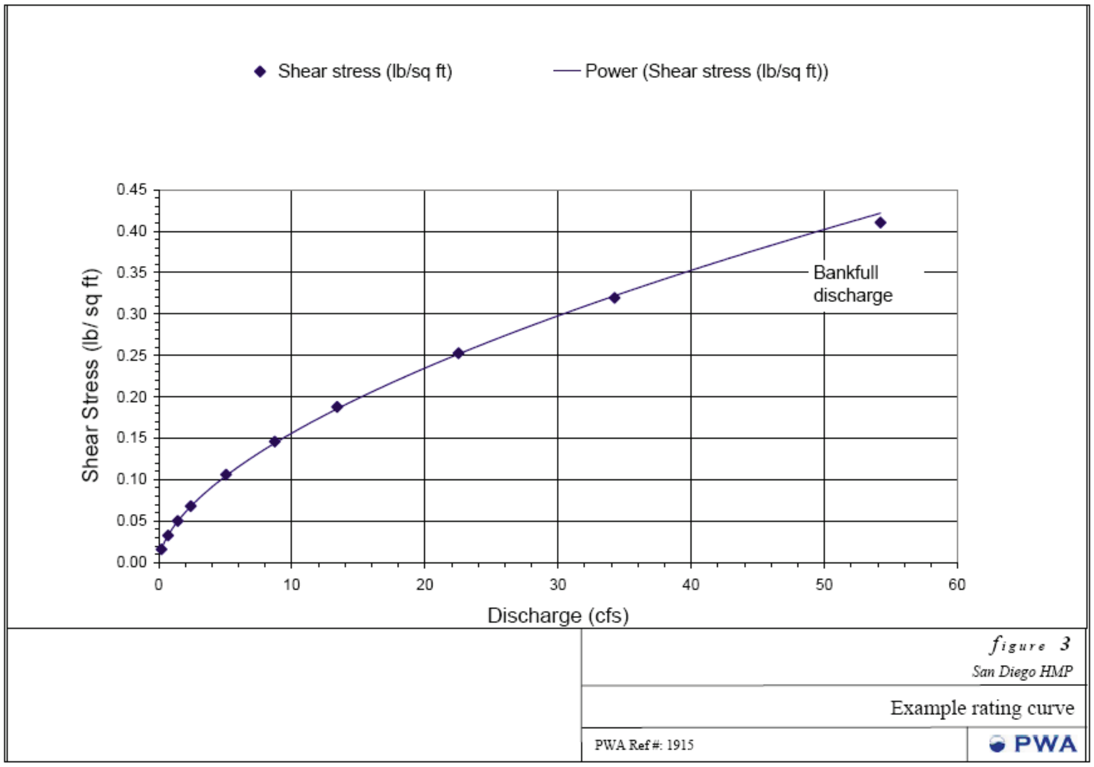

Figure 5-3. Shear Stress Rating Curve for an Example Channel (0.6%, 14 Feet Wide, 1.3 Feet Deep).

These curves were created for 36 different combinations of channel characteristics.

Identify Critical Flow for the Channel and Material

Qcrit is the flow rate at which boundary shear stress equals critical shear stress. A power function interpolates the discharge versus boundary shear stress rating curve, to allow calculation of an intercept between the rating curve and critical shear stress. The critical shear stress for each channel material was plotted horizontally from the Y axis until it intercepted the rating curve. The intercept point was extended vertically to the X axis, showing the Qcrit (Figure 5-4). In this way, Qcrit was calculated for each of the five channel materials using each of the 36 rating curves representing different channel dimensions. As mentioned above, 10 combinations unlikely to occur in nature were eliminated, resulting in a total of 170 Qcrit calculations.

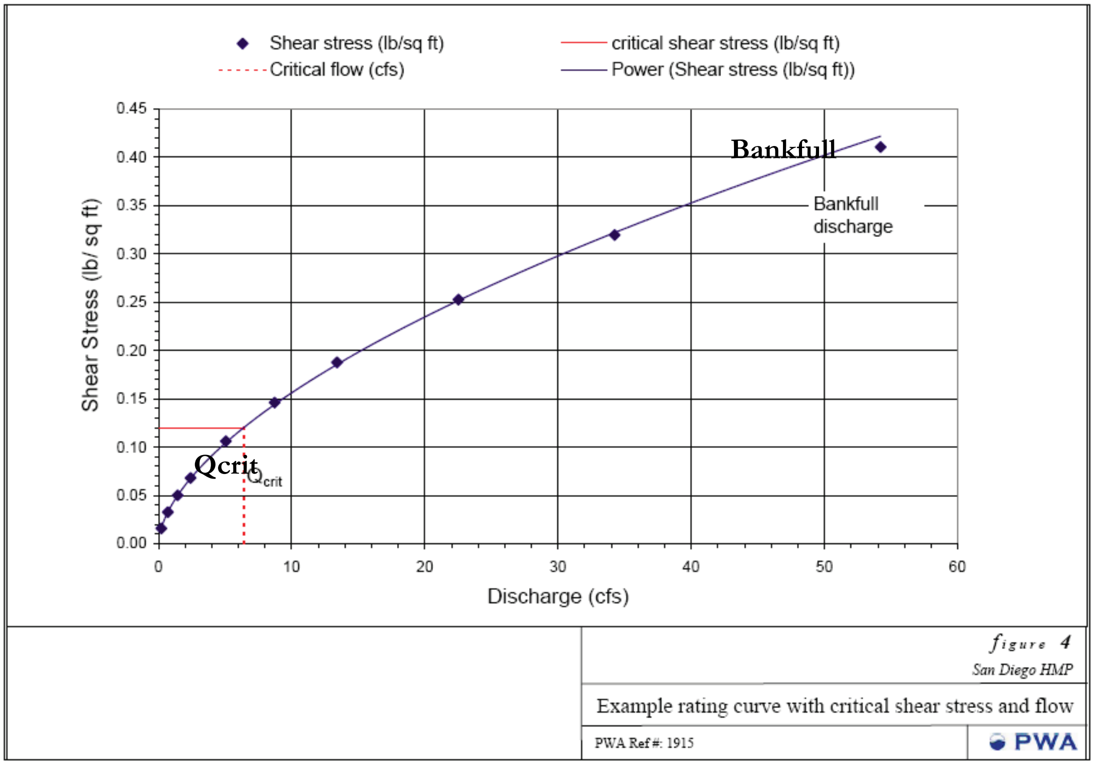

Figure 5-4. Example of a Rating Curve with Critical Shear Stress for Medium Sized Gravel.
In this example critical shear stress = 0.12 lb/sq ft and critical flow Qcrit = 6.4 cfs.

Express Critical Flow as a Function of Q_2

As described above, each rating curve represents a particular combination of watershed area and channel dimensions. Q_2 was calculated for each combination using the USGS regional regression for Q_2 as described above. By dividing the calculated Qcrit by the appropriate Q_2, Qcrit as a proportion of Q_2 was calculated for the 170 scenarios. These Qcrits were then plotted by material type, showing mean and one standard deviation either side of the mean. Note that although Q_5 is assumed as bankfull discharge, critical flow is expressed as a function of Q_2 as has become standard for HMPs.

The results show the high degree of variability in Qcrit based on different channel materials. It is important to note that in field conditions many of the most extreme cases (examples with very high or very low thresholds) would tend to evolve to conditions that yielded critical flows closer to the bankfull discharge because channels have a tendency to self equilibrate. For example, channels with materials that have very low critical flows such as unconsolidated sand tend to erode and either flatten (lowering shear stress, and so increasing critical flow rate) or armor (increasing flow resistance, and increasing critical flow rate). Likewise, channels with materials that have very high thresholds tend to either become steeper due to deposition (increasing shear stress and lowering critical flow rate) or fill in with finer material (reducing resistance and lowering critical flow rate).

As the results of this analysis demonstrate, critical flow is extremely variable among channel materials and, for a given channel material, can vary significantly with channel configuration (slope, width/depth ratio etc.). Unconsolidated fine sediments can be mobilized by extremely low flows in the absence of clays or other consolidating elements with the structure of the channel. This result is based on literature values for critical shear stress for unconsolidated materials and may not be realistic for natural channels. Therefore in setting flow thresholds this result should be balanced with the recognition that natural channels are likely to include some consolidating fraction within their structure, as well as practical considerations associated with controlling trickle flows that represent the smaller fractions of Q_2 analyzed in this study.

5.1.4 Tool for Calculating Site-Specific Critical Flow

Background

The consultant team developed a tool for calculating a site-specific low flow threshold based on local conditions. The low flow threshold is based on Qcrit for the receiving channel, which is calculated based on channel geometry (width, depth, and gradient), channel materials, and watershed area.

The approach taken was to develop an Excel spreadsheet model to calculate the boundary shear stress associated with a range of flows up to Q_5 for a given channel width, depth and slope, then plot the critical shear stress for the channel material on this rating curve over to identify the flow where boundary shear stress equals critical shear stress.

The development steps were as follows:

1. Develop simplified channel cross section and gradient inputs
2. Calculate a shear stress rating curve
3. Characterize channel materials in terms of critical shear stress
4. Plot critical shear stress of the receiving channel on the rating curve to determine Qcrit
5. Divide the critical low flow by the project area as a proportion of the receiving water watershed area to determine the allowable flow at the point of compliance

Simplified Channel Cross Section and Gradient Inputs

The tool generates a flow rating curve based on user inputs describing the receiving channel dimensions (cross section) and gradient. The first step in developing the tool was to create a template for inputting the required channel parameters. The template assumes a simple trapezoidal cross section, with the following elements:

1. Channel width at a well defined break point corresponding to top of bank (a)
2. Channel width at the toe of the bank (b)
3. Channel depth (elevation difference between bank top and channel bed) (c)

Assumptions:

1. Receiving channels can be reasonably represented by a simple trapezoidal cross section
2. The top of bank corresponds reasonably to the level inundated by the dominant discharge (approximately equal to Q_5)

If top of bank is much higher than the dominant discharge flow depth (e.g. in an incised channel) the applicant should adjust the cross section to represent the lower part of the channel so that depth (c) corresponds approximately to the Q_5 depth.

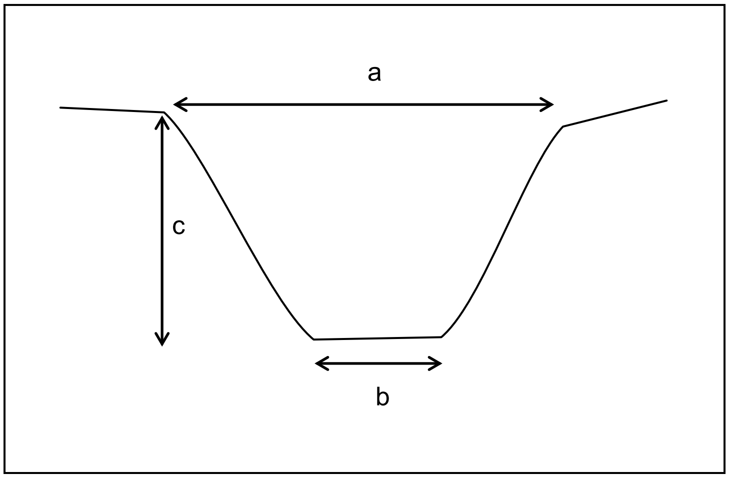

Figure 5-5. Bankfull Cross Section

Develop a Shear Stress Rating Curve

The tool creates a shear stress rating curve for a range of flows between 1 percent of the bankfull flow depth and bankfull depth [flow at depth (c)]. The range 1 to 100 percent of bankfull is used because critical flow rarely lies outside these values. The tool then calculates average boundary shear stress and discharge as previously described in section 5.1.3.

Characterize Receiving Channel Materials in Terms of Critical Shear Stress

The critical shear stress of the channel materials is estimated using a look-up table based on values published by the U.S. Army Corps of Engineers (Fischenich). The tool provides values of critical shear stress for a wide range of channel materials in a drop down box so the user can select from the list. The calculator also allows the user to input a vegetated channel material when this is appropriate (when the channel is completely lined in vegetation).

Calculating Critical Flow for the Receiving Water

Critical flow is the discharge at which boundary shear stress equals critical shear stress. The tool uses a power function to interpolate the discharge versus boundary shear stress rating curve. The critical shear stress for the weaker of the bed or banks is plotted horizontally from the Y axis until it intercepts the rating curve. The intercept point is extended vertically to the X axis, showing the critical flow (see Figure 5-4). This represents the low flow threshold for the receiving water. This value is compared to the 2-year flow in the channel to determine the allowable release rate from BMP facilities, expressed as a fraction of the 2-year flow (0.1, 0.3, or $0.5Q_2$).

Calculating Critical Flow for the Point of Compliance

The tool calculates critical flow based on the characteristics of the receiving water. Where the project watershed does not make up the entire watershed area for the receiving water, the low flow threshold can be prorated based on the percentage of the watershed that is occupied by the project site[2]. For example, if a project occupies one tenth of the receiving water's watershed at the point of compliance and the critical flow

[2]. It is not necessary to adjust the "off-the-shelf" thresholds developed for Track 1 for point of compliance, since they are expressed as a fraction of Q2 for the relevant project area.

level is 50 cfs, the project's 'share' of the non-erosive flow is 5 cfs (50 x 1/10). This prevents the cumulative impact of future developments from exceeding critical flow in the receiving water, since the critical flow is apportioned according to watershed area.

$$\text{Critical flow at Point of Compliance} = \text{Critical Flow at Receiving Water} \times \frac{\text{Project Area}}{\text{Watershed Area}}$$

The critical flow at the point of compliance is the low flow threshold for the project draining to this point.

Conversion of Critical Flow to Flow Class

To avoid having an infinite range of flow control standards, the calculator assigns the discharge into one of three classes based on its value as a function of the estimated Q_2. These classes are: $0.1Q_2$, $0.3Q_2$, $0.5Q_2$. For example, a channel where the critical flow is $0.15Q_2$ would be assigned a flow threshold of $0.1Q_2$. Channels with critical flows less than $0.1Q_2$ are assigned to the $0.1Q_2$ class. The class flow rate is calculated (i.e. the critical flow corresponding to the assigned fraction of Q_2) and expressed as the final output of the tool.

5.1.5 Third Party Review

West Consultants conducted an in-depth, independent third-party review of the preliminary flow threshold analysis in December 2008. The following list presents a summary of the third-party review.

- Concern was noted regarding the lower flow control limit suggested by the modeling results (10 percent of the 2-year runoff event), especially with regard to implementation practicality and its derivation based solely on sediment movement.
- The review noted that literature suggests standard hydrologic design practices may be inadequate for characterizing cumulative effects of urbanization for flow events more frequent than the 2-year runoff event – specifically with regard to sediment transport and channel disturbance potential.
- The review questioned the use of a specific frequency discharge as an indicator of shear stress to move particles given the variability of other site-specific parameters such as grain size, slope, roughness, and channel shape.
- The review suggested that hydraulic and sediment transport results should be supplemented with actual field data (slope, sediment properties, roughness, and channel shape) to set thresholds (flows, shear stresses, or velocities).
- Concern was noted regarding the use of a single and conservative uniform size for sediment grain sizes. The use of a distribution of sediment grain sizes was recommended.

PWA agreed with the recommendation that additional field data (channel dimensions and slope, and sediment size distribution) is desirable both to verify receiving channel conditions and to make direct measurements of critical shear stress. Efforts were made to pursue the former data, but it was not possible to obtain field permission in time to meet the project deadlines. As the third-party review notes, any revised lower flow threshold calculated using field data is as likely to decrease as increase.

Subsequent to the preliminary flow threshold analysis, PWA ran sediment transport models using a distribution of grain sizes (rather than a single uniform size) for two channel configurations. The results of this limited sensitivity test (see discussion below titled "Summary of Sensitivity Analysis") did not show a consistent trend toward more or less erosion.

For more detailed information regarding West Consultants' independent third-party review, refer to the memo titled *Review of Hydromodification Work by Phillip Williams and Associates (PWA),* prepared by West Consultants and dated December 19, 2008 (Appendix D).

5.1.6 Summary of Sensitivity Analysis

Subsequent to the preliminary flow threshold analysis, the Copermittees requested that a sensitivity analysis be conducted based on historical rainfall records in the vicinity of the test watershed sites. The purpose of the sensitivity analysis based on the revised rainfall input data is described below. There were two potential concerns associated with the use of the hydrologic analysis developed in the preliminary flow threshold analysis.

- First, the analysis used a single rainfall time series (Lindbergh Field) for all simulations. Rainfall records for other areas were synthesized by taking the difference in mean annual rainfall between a nearby rain gauge and developing a linear adjustment for the Lindbergh series (e.g., if the test site's mean annual rainfall is 15 percent greater than the mean annual rainfall at Lindbergh, then 15 percent is added to all hourly rainfalls). The preliminary flow threshold analysis used this scaled data approach since other data were not available at the time of the initial analysis. Long-term rainfall data for 20 gauges throughout San Diego County have subsequently been prepared and are thus more relevant to the test simulations performed for this study. A test hydrologic analysis showed significant hydrologic response differences between the historical rainfall record for Lower Otay Reservoir and the scaled data from Lindbergh Field.
- Second, the preliminary flow threshold analysis used an "annual peak" method to calculate the rainfall recurrence interval, rather than a partial duration series method. The two methods result in significantly different predictions of the two year runoff event (Q_2.). From discussions with rainfall statistical experts at the Hydrologic Research Center, it has been determined that the partial duration series is a more applicable rainfall series for the semi-arid climate in San Diego County. Partial duration flow statistics have been prepared and the test hydrologic analysis showed significant hydrologic response differences between the partial duration series and annual peak series methods.

There is significant variability in the HEC-RAS modeling results for the different channel and sediment scenarios, as reflected in the results of the preliminary flow threshold analyses. Therefore, it is important to focus on the general trends reflected in the sensitivity analysis results rather than the specific numerical results. As such, the sensitivity analysis modeling results confirm that the selection of rainfall data, flow frequency methodology, and sediment size distribution do affect the results of the flow control analysis. However, the cumulative effect of these changes did not affect the consultant's preliminary conclusion that a singular countywide lower flow threshold limit would converge on 10 percent of the 2-year runoff event.

For more detailed information regarding the PWA sensitivity analysis based on revised rainfall data, refer to the memo titled *Sensitivity of Changing Rainfall Series and Analysis on Erosion Threshold*, prepared by PWA and dated December 29, 2009 (Appendix A).

5.2 Categorization of Streams

Information for this section was prepared in association with a concurrent hydromodification study by the SCCWRP. As discussed with the San Diego RWQCB staff, results of the SCCWRP study have been included in the San Diego HMP to comply with the following Permit Order requirement.

- Identification of geomorphic standards for channel segments receiving storm water discharges from Priority Projects (Permit Section D.1.g.(1)(a) and (m)). The purpose of these standards is to maintain or improve channel stability.

The SCCWRP study, which is being conducted for the entire Southern California region between Ventura and San Diego Counties, was originally funded by a Prop 50 grant. Because of funding issues that required a work stoppage in late 2008, the County of San Diego has provided funding to SCCWRP to continue its work and meet deadlines required for the San Diego HMP submittal timeline. The overall SCCWRP study approach is summarized in the document titled, "*Stream Channel Mapping and Classification Systems: Implications for Assessing Susceptibility to Hydromodification Effects in Southern California," SCCWRP Technical Report 562, April 2008.*

Screening tools, prepared by SCCWRP to identify channel susceptibility to hydromodification impacts, are now available in 2009 on a testing basis. Such tools include the following:

- A tiered, hierarchical approach for channel erosion susceptibility evaluation of multiple channel types. This approach includes determination of a vertical channel stability analysis (including transportability of channel bed material) and a lateral channel stability analysis (including potential erodibility of channel banks and subsequent channel migration). These rapid assessment tools provide a preliminary rating of stream susceptibility to erosion (Very High, High, Medium, or Low) and are provided for a variety of geomorphic scenarios including alluvial fans, broad valley bottoms, incised headwater channels, etc.

Eventually, SCCWRP tools will be expanded to help quantify the effect of a proposed project on the receiving stream's susceptibility to erosion, based upon factors such as size of the project, impervious footprint, location of the project within the watershed, and stability of the receiving water body.

Development of HMPs in most Southern California counties is correlated to the ultimate findings of the SCCWRP study, which was originally scheduled for release in March 2010. Though individual regions and municipalities would not be tied to acceptance of the SCCWRP results, it is generally acknowledged that SCCWRP's formulation of regional standards for hydromodification management will serve as a solid baseline for development of HMPs for specific regions in Southern California.

For implementation with the San Diego HMP, the SCCWRP screening tools will be used in association with the decision matrix to determine the appropriate level of mitigation required for a particular project. Where receiving streams have a high susceptibility to erosion, then more restrictive mitigation solutions will be required as compared to receiving streams with a low susceptibility to erosion.

The full lateral and vertical susceptibility decision matrices are included on Pages 4 and 5 of the overall HMP Decision Matrix, located in Chapter 6 of this HMP. Page 3 of the HMP Decision Matrix includes recommendations regarding the appropriate lower flow threshold, based upon the SCCWRP susceptibility analysis as well as the critical flow calculator result.

Channel screening tools will assess the domain of analysis from a proposed project. The domain of analysis is defined as the reach lengths upstream and downstream from a project for which hydromodification assessment is required. The domain of analysis determination includes an assessment of the incremental flow accumulations downstream of the site, identification of hard points in the downstream conveyance system, and quantification of downstream tributary influences.

The effects of hydromodification may propagate for significant distances downstream (and sometimes upstream) from a point of impact such as a stormwater outfall. Accordingly, it may be necessary to conduct geomorphic screening reconnaissance across a domain spanning multiple channel types/settings and property owners.

For purposes of this HMP, the extents of the domain of analysis are defined as follows:

- Proceed downstream until reaching one of the following:
  - At least one reach downstream of the first grade-control point (preferably second downstream grade control location)
  - Tidal backwater/lentic (still water) waterbody
  - Equal order tributary (Strahler 1952)

- Accumulation of 50 percent drainage area for stream systems (note that SCCWRP is still determining specific flow accumulation percentage)
- Accumulation of 100 percent drainage area for urban conveyance systems (storm drains, hardened channels, etc.)

OR demonstrate sufficient flow attenuation through existing hydrologic modeling.

- Proceed upstream for 20 channel top widths OR to the first grade control point, whichever comes first. Identify hard points that can check headward migration and evidence of active headcutting.

If the screening analysis is conducted on a project-specific basis, there may be instances in which a high susceptibility rating is obtained at the first point of field observation. In these cases, it may be sufficient to limit the analysis to the point/property of impact.

The SCCWRP screening tools, as well as details to determine the domain of analysis, are provided in Appendix B.

5.3 Cumulative Watershed Impacts

California Environmental Quality Act (CEQA) Guidelines §15065 mandate a finding that a project has a significant effect on the environment when it has:

> "…possible environmental effects that are individually limited but cumulatively considerable. 'Cumulatively considerable' means that the incremental effects of an individual project are significant when viewed in connection with the effects of past projects, the effects of other current projects, and the effects of probably future projects."

Such assessments are inherently difficult and rarely quantifiable. However, it is often possible to incorporate within a project measures that limit or offset potential impacts to such a degree that reasonable minds can agree the net incremental impact of the project is insignificant regardless of the connections to, or multiplying effects of, other projects. To this end, a river reach sensitivity analysis was performed for the San Diego River. The intent of this analysis was to determine the level of cumulative watershed impacts that would result in a significant alteration to the San Diego River's flow duration curve. Data from this analysis are being used to determine exemption criteria for similar-sized river systems in San Diego County, since detailed long-term hourly streamflow data is not available for most of those rivers. The results of the sensitivity analysis are discussed in detail in Section 6.1.

5.3.1 Hydromodification Management

The purpose of the HMP is to address the cumulative effect of many individual development projects on stream erosion. In the HMP, the watershed-scale effect is addressed through conditions placed on individual development projects.

Also, the HMP implements a regulatory standard. A project's compliance with regulatory standards may be used to help determine whether the project may have a significant impact on the environment, either individually or cumulatively.

Two questions have been raised with regard to how the HMP addresses cumulative impacts:

1. Are the low-flow thresholds (the maximum rate at which on-site detention facilities can be drained) low enough to prevent stream erosion, when viewed in connection with the effects of other projects?

2. With regard to the specific exemption proposed for discharges where downstream sub-watershed imperviousness is at least 70 percent and the potential for cumulative impacts is "minimal," how will that potential be assessed?

Low Flow Thresholds

How does the low flow threshold for a receiving stream relate to the flow that must be controlled at a project site?

A low flow threshold for a receiving stream can be articulated as a particular runoff event (e.g., 0.1 Q_2., or one-tenth the 2-year peak runoff flow). Runoff to the stream is modeled based on the watershed area tributary to the stream, and the model is then calibrated to stream gauge data. In some cases, the low-flow runoff threshold developed from watershed-scale stream analysis has then been applied to each project area within the watershed. Implicitly, this standard is set so that if the entire watershed was made impervious, and runoff from the entire watershed was controlled through the facilities built to this standard, no increase in stream erosion would result.

This is an exceedingly conservative assumption, because:

- Not all areas of the watershed will be developed.
- Very low flows trickling from individual detention facilities will have losses before reaching streams.
- If bioretention facilities are used, the losses to infiltration and evapotranspiration are likely to be underestimated.

In addition to this general conservative bias, additional conservatism is built in when a project is located downstream from headwaters. To use an extreme example, a discharge from a development project near the mouth of a stream draining a large watershed would have an insignificant impact throughout the range of runoff flow rates encountered in the stream.

The degree to which these factors contribute to a conservative bias in the hydromodification standard can only be estimated; however, reasonable judgments can be made. These judgments should be subject to revision based on further insights gained in the first years of implementation.

Proposed Exemptions

How would the potential for cumulative impacts from many exempted projects be assessed? There are too many possible scenarios of development proposal and watershed condition to establish firm standards or guidelines. In this context, the concept of "minimal potential" for cumulative impacts means a judgment that—based on knowledge of the specific land use patterns and policies in the watershed—it is unlikely the total of all future newly developed projects discharging at the selected low-flow threshold would be significant when compared to the current (pre-project) total flow from the watershed. The requirements that Priority Development Projects on previously developed sites implement LID and use LID facilities such as bioretention for storm water treatment further ensure runoff rates and durations in highly developed watersheds will decrease rather than increase over time.

A similar approach applies to other proposed exemptions and in-lieu mitigation projects within the HMP. For example, the HMP states "the project proponent may consider implementation of planning measures such as buffers and restoration activities.… in lieu of implementation of storm water flow controls." In this case, cumulative impacts are addressed by the proviso that this option is available "in situations where the benefits of a proposed stream restoration project would *substantially outweigh the potential impacts* of additional runoff from a proposed project…" (emphasis added). The requirement that benefits "substantially outweigh" potential impacts for each individual project addresses the potential for cumulative impacts by ensuring that, even if many such scenarios were implemented in a watershed, the cumulative benefits would outweigh the cumulative impacts.

5.3.2 Summary

As with other cumulative impacts, the cumulative impacts on stream erosion of individual land development projects within a watershed can only be estimated. Judgments on the significance of potential cumulative impacts are based on a weight of evidence approach. Despite the lack of quantification and certainty, it is possible for stakeholders to agree that, in a given set of conditions, the potential for cumulative impacts is highly unlikely.

The HMP supports assessment of cumulative impacts through hydrologic modeling of entire watersheds. Translating the results of watershed modeling to standards applicable to individual development sites is a matter of estimation and judgment.

In the HMP, the potential for cumulative impacts is addressed through a built-in conservative bias in quantitative estimates of impacts and the effectiveness of flow-control measures needed to address those impacts, and through a conservative approach and individual review of proposed exemptions and in-lieu projects.

6. REQUIREMENTS AND STANDARDS FOR PROJECTS

Priority Development Projects are required to implement hydrologic control measures so that post-project runoff flow rates and durations do not exceed pre-project flow rates and durations where they would result in an increased potential for erosion or significant impacts to beneficial uses or violate the channel standard (Permit Section D.1.g(1)(c)). The purpose of this chapter is to detail HMP applicability requirements, present hydromodification mitigation criteria and implementation options, and provide a framework for in-stream rehabilitation options.

6.1 HMP Applicability Requirements

To determine if a proposed project must implement hydromodification controls, refer to the HMP Decision Matrix in Figure 6-1 on the following page.

The HMP Decision Matrix can be used for all projects. For redevelopment projects, flow controls would only be required if the redevelopment project increases impervious area or peak flow rates as compared to pre-project conditions.

It should be noted that all Priority Development Projects will be subject to the Permit's LID and water quality treatment requirements even if hydromodification flow controls are not required.

As noted in Figure 6-1, projects may be exempt from HMP criteria under the following conditions.

- If the project is not a Priority Development Project
- If the proposed project does not increase the impervious area or peak flows to any discharge location.
- If the proposed project discharges runoff directly to an exempt receiving water such as the Pacific Ocean, San Diego Bay, an exempt river reach, an exempt reservoir, or a tidally-influenced area.
- If the proposed project discharges to a stabilized conveyance system that extends to the Pacific Ocean, San Diego Bay, a tidally-influenced area, an exempt river reach or reservoir.
- If the contributing watershed area to which the project discharges has an impervious area percentage greater than 70 percent
- If an urban infill project discharges to an existing hardened or rehabilitated conveyance system that extends beyond the "domain of analysis," the potential for cumulative impacts in the watershed are low, and the ultimate receiving channel has a Low susceptibility to erosion as defined in the SCCWRP channel assessment tool.

If the proposed project decreases the pre-project impervious area and peak flows to each discharge location, then a flow-duration analysis is implicitly not required. If continuous simulation flow-frequency and flow duration curves were developed for such a scenario, the unmitigated post-project flows and durations would be less as compared to pre-project curves.

Proposed exemptions for projects discharging runoff directly to the Pacific Ocean, San Diego Bay or to hardened conveyance systems which transport runoff directly to the Pacific Ocean or San Diego Bay are referred to the 2007 Municipal Permit. Per the Permit, hardened conveyance systems can include existing concrete channels, storm drain systems, etc.

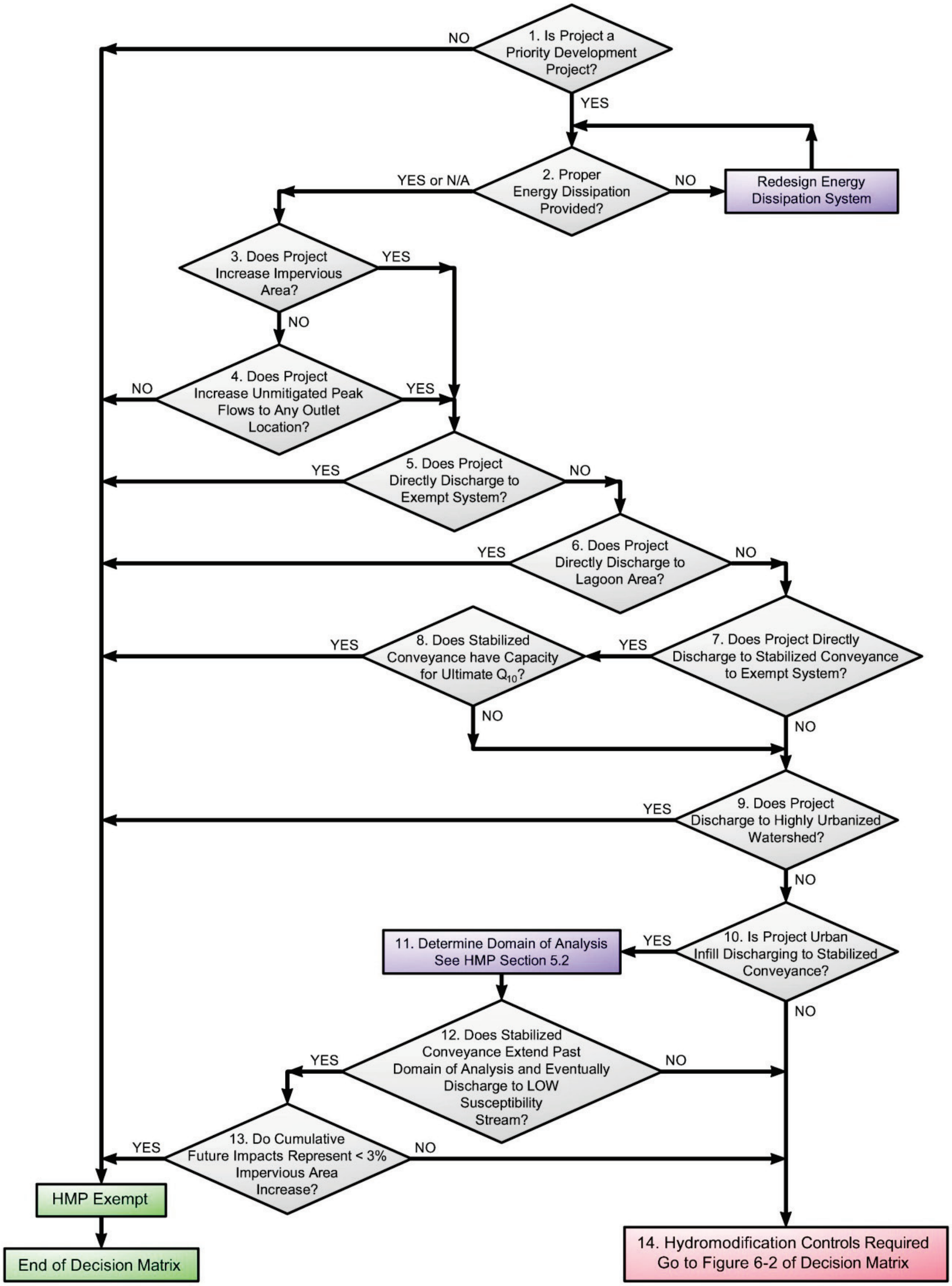

Figure 6-1. HMP Applicability Determination

The Municipal Permit also contains language to support exemptions for projects located in highly urbanized areas where the impervious percentage exceeds 70 percent (as calculated for the sub-watershed between the project outfall downstream to the exempt receiving water).

- Figure 6-1, Node 1 – Hydromodification mitigation measures are only required if the proposed project is a Priority Development Project.
- Figure 6-1, Node 2 – Properly designed energy dissipation systems are required for all project outfalls to unlined channels. Such systems should be designed in accordance with the County of San Diego's Drainage Design Manual to ensure downstream channel protection from concentrated outfalls.
- Figure 6-1, Nodes 3 and 4 – Projects may be exempt from hydromodification criteria if the proposed project reduces the pre-project impervious area and if unmitigated post-project outflows (outflows without detention routing) to each outlet location are less as compared to the pre-project condition. The pre and post-project hydrologic analysis should be conducted for the 2 and 10-year design storms and follow single-event methodology set forth in the San Diego Hydrology Manual. This scenario may apply to redevelopment projects in particular.
- Figure 6-1, Node 5 – Potential exemptions may be granted for projects discharging runoff directly to an exempt receiving water, such as the Pacific Ocean, San Diego Bay, an exempt river system (detailed in Table 6-1), or an exempt reservoir system (detailed in Table 6-2).
- Figure 6-1, Node 6 – For projects discharging runoff directly to a tidally-influenced lagoon, potential exemptions may also be granted. Exemptions related to runoff discharging directly to tidally-influenced areas were drafted based upon precedent set in the Santa Clara HMP. Regarding the potential exemption, additional analysis would be required to assess the effects of the freshwater / saltwater balance and the resultant effects on lagoon-system biology. This assessment, which would be required by other permitting processes such as the Army Corps of Engineers, California Department of Fish and Game, etc., must be provided by a certified biologist or other specialist as approved by the governing municipality. Such discharges would include an energy dissipation system (riprap, etc.) designed to mitigate 100-year outlet velocities based upon a free outfall condition. Such a design would be protective of the channel bed and bank from an erosion standpoint.
- Figure 6-1, Nodes 7 and 8 – For projects discharging runoff directly to a hardened conveyance or rehabilitated stream system that extends to exempt receiving waters detailed in Node 5, potential exemptions from hydromodification criteria may be granted. Such hardened or rehabilitated systems could include existing storm drain systems, existing concrete channels, or stable engineered unlined channels. To qualify for this exemption, the existing hardened or rehabilitated conveyance system must continue uninterrupted to the exempt system. In other words, the hardened or rehabilitated conveyance system cannot discharge to an unlined, non-engineered channel segment prior to discharge to the exempt system. Additionally, the project proponent must demonstrate that the hardened or rehabilitated conveyance system has capacity to convey the 10-year ultimate condition flow through the conveyance system. The 10-year flow should be calculated based upon single-event hydrologic criteria as detailed in the San Diego County Hydrology Manual.
- Figure 6-1, Node 9 – As allowed per the Municipal Permit, projects discharging runoff to a highly urbanized watershed (defined as an existing, pre-project impervious percentage greater than 70 percent) may be eligible for an exemption from hydromodification criteria.

Watershed impervious area calculations for this potential exemption will be measured between the project site discharge location and the connection to a downstream exempt receiving conveyance system, such as the Pacific Ocean, San Diego Bay, or an exempt river system. If a tributary area connects with the main line drainage path between the project site and the exempt system, then the entire watershed area contributing to the tributary shall be included in the calculation. Initial review of County land use indicates that this exemption will likely only apply in a limited number of urbanized coastal areas.

Percent imperviousness will be calculated based on an area-weighted average of impervious areas associated with commercial, industrial, single-family residential, multi-family residential, open space, and other miscellaneous areas (schools, churches, etc.) representative for the watershed. Representative percent imperviousness values for each land use type may correspond to values recommended in Table 3-1 of the County of San Diego's Hydrology Manual and detailed below or by more specific representative percent impervious calculations (using GIS, etc.), which are often required to represent impervious area percentages for park, school and church sites.

- Figure 6-1, Nodes 10 through 13 – For urban infill projects discharging runoff to an existing hardened or rehabilitated conveyance system, potential limited exemptions from hydromodification criteria may apply where the existing impervious area percentage in the watershed exceeds 40 percent. For the potential exemption application, the domain of analysis must be determined and the existing hardened or rehabilitated conveyance system must extend beyond the downstream terminus of the domain of analysis. The hardened or rehabilitated conveyance system must discharge to a receiving channel with a Low potential for channel susceptibility for this exemption to be granted (channel susceptibility determined using SCCWRP tool). Finally, continuous simulation sensitivity analysis shows that an exemption could only be granted if the potential future development impacts in the watershed would increase the watershed's impervious area percentage by less than 3 percent (as compared to the existing condition in the year 2010). If the potential future cumulative impacts in the watershed could increase the impervious area percentage by more than 3 percent (as compared to existing condition), then no exemption could be granted based on this item. Watershed impervious area calculations for this potential exemption, in which a project discharges to a watershed with an existing impervious areas greater than 40 percent, will be measured upstream from the outfall of the urban conveyance system (to a non-concrete, non-riprap-lined or non-engineered channel) to the contributing watershed boundary (the entire watershed contributing to the discharge outfall).

  Percent imperviousness will be calculated based on an area-weighted average of impervious areas associated with commercial, industrial, single-family residential, multi-family residential, open space, and other miscellaneous areas (schools, churches, etc.) representative for the watershed. Representative percent imperviousness values for each land use type may correspond to values recommended in Table 3-1 of the County of San Diego's Hydrology Manual and detailed below or by more specific representative percent impervious calculations (using GIS, etc.), which are often required to represent impervious area percentages for park, school and church sites.

Exemptions related to runoff discharging directly to certain river reaches were initially based upon the majority TAC opinion that such river reaches were depositional (aggrading) and that the effects of cumulative watershed impacts to these reaches is minimal. Subsequent justifications for the river reach exemptions were the result of a flow duration curve analysis for the San Diego River.

Potential river reaches that would be exempt from hydromodification criteria include only those reaches for which the contributing drainage area exceeds 100 square miles and which have a 100-year design flow in excess of 20,000 cfs. For reference, proposed Caltrans HMP criteria allows for river/creek exemptions for drainage areas of only 10 square miles.

Per recommendations from members of the TAC, San Diego river systems meeting the drainage area and peak flow criteria are typically aggrading (depositional) and have very wide floodplain areas when in the natural condition. In all cases, river reaches meeting the drainage area and peak flow criteria are located downstream of large reservoir systems which effectively block outflows for most storm events. In addition, the river systems meeting these criteria typically have very low gradients. The combination of low gradients, significant peak flow attenuation, and wide floodplain areas translate to a low potential for channel erosion at the upper limit of the proposed geomorphic flow range (10-year flow event).

The intent of the San Diego River flow duration analysis was to determine the level of cumulative watershed impacts that would result in a significant alteration to the San Diego River's flow duration curve. Both the Fashion Valley and Mast Boulevard USGS stream gauge stations were used to develop long-term flow duration curves for the San Diego River. Data from this analysis will be used to determine exemption criteria for similar-sized river systems in San Diego County, since detailed long-term hourly streamflow data is not available for most of those rivers. Since the findings of the sensitivity analysis are planned to be extrapolated to other large river systems, implementation of additional gauging stations along other major river systems is recommended to analyze the differences in watershed response between the major watershed systems.

Assumptions related to the San Diego River sensitivity analysis are provided below:

- The flow duration charts show the San Diego River flow durations, plus simulated river flows durations for additional development scenarios.
- HSPF models were built to simulate converting existing undeveloped areas in the watershed into development with no stormwater flow controls.
- Increasing drainage area increments were modeled.
- To produce the 'simulated development' flow duration curves, the difference between developed and undeveloped flow duration curves was calculated for proposed hypothetical development .sites of varying sizes Then, the "difference hydrograph" was added to the San Diego River flow duration curve. This approach was used to avoid the potential problem of double-counting areas.

Tasks Related to Development of Flow Duration Analysis of San Diego River:

- Acquired 15-minute stream flow data from USGS (available from 1988 to present)
- Aggregated to 1-hour historical record
- Computed flow duration statistics for both records and determined if there is any substantial difference between the records (this is a QA step that allowed for removal any high flow 'outliers' in the record that could affect the results).
- Prepared a simple, characteristic HSPF model for the lower watershed for "existing conditions in an undeveloped area" – assumed Group D soils with sparse vegetation.
- Prepared a simple, parallel HSPF model for "developed conditions"

- Ran both models and examined the difference between the resulting hydrographs (the hydromodification). A couple of different pre- and post-development models were generated to analyze the differences on a per unit area basis.
- Using the "difference hydrograph" created from the model simulations, progressively added development and recomputed the flow duration statistics.
- Examined the modified flow duration statistics and determined at what level of increased development the statistics became noticeably altered.

Results showed that increasing levels of development, in excess of 1,000 acres assumed to occur at the same location as the stream gauge station, would produce a very minor influence on the river's flow duration curve. These results demonstrated that certain portions of the San Diego River could be exempt from hydromodification requirements. Such HMP exemptions would only be granted for projects discharging runoff directly to the exempt river reach. Each municipality must define "direct discharge" based on the project site conditions. To qualify for the potential exemption, the outlet elevation must be between the river bottom elevation and the 100-year floodplain elevation and properly designed energy dissipation must be provided The supporting HSPF continuous modeling analysis results are summarized in a Technical Memo in Appendix F.

All exempt river reaches, which are presented in Table 6-1, have drainage areas in excess of 100 square miles and 100-year flow rates in excess of 20,000 cfs. In addition, all proposed river reaches are subject to significant upstream reservoir flow regulation, have wide floodplain or stabilized channel areas, and low gradients. This combination of factors, in association with field observations and years of historical perspective from the TAC members, justifies exemptions for direct discharges to the exempt river reaches provided that properly sized energy dissipation is provided at the outfall location.

Table 6-1. Summary of Exempt River Reaches in San Diego County

River	Downstream Limit	Upstream Limit
Otay River	Outfall to San Diego Bay	Lower Otay Reservoir Dam
San Diego River	Outfall to Pacific Ocean	Confluence with San Vicente Creek
San Dieguito River	Outfall to Pacific Ocean	Lake Hodges Dam
San Luis Rey River	Outfall to Pacific Ocean	Upstream river limit of Basin Plan subwatershed 903.1 upstream of Bonsall and near Interstate 15
Sweetwater River	Outfall to San Diego Bay	Sweetwater Reservoir Dam

Table 6-2 provides a summary of exempt reservoirs in San Diego County. Large reservoirs can be exempt systems from a hydromodification standpoint since reservoir storm water inflow velocities are naturally mitigated by the significant tailwater condition in the reservoir. HMP exemptions would only be granted for projects discharging runoff directly to the exempt reservoirs. Each municipality must define "direct discharge" based on the project site conditions. To qualify for the potential exemption, the outlet elevation of the conveyance system must be within (or below) the normal operating water surface elevations of the reservoir and properly designed energy dissipation must be provided.

Table 6-2. Summary of Exempt Reservoirs in San Diego County	
Reservoir	**Watershed**
Barrett Lake	Tijuana River
El Capitain Reservoir	San Diego River
Lake Dixon	Escondido Creek
Lake Heneshaw	San Luis Rey River
Lake Hodges	San Dieguito River
Lake Jennings	San Diego River
Lake Murray	San Diego River
Lake Poway	San Dieguito River
Lake San Marcos	San Marcos Creek
Lake Wohlford	Escondido Creek
Loveland Reservoir	Sweetwater River
Lower Otay Reservoir	Otay River
Miramar Lake	Los Penasquitos Creek
San Vicente Reservoir	San Diego River
Sweetwater Reservoir	Sweetwater River
Upper Otay Reservoir	Otay River

The final exemption category focuses on small urban infill projects where the potential for future cumulative watershed impacts is minimal. Continuous simulation models have been prepared for subwatershed areas containing between 40 and 70 percent existing imperviousness (as measured from the project site downstream to existing storm drain outfall) with the following assumptions.

Sensitivity Analysis for Urban Watersheds:

- Prepared HSPF models for 10-, 100-, and 500-acre watersheds with 40, 50, 60 percent imperviousness. Ran simulations and computed flow duration statistics for each of the urban watershed scenarios.
- Progressively increased the level of imperviousness to simulate infill development for the 10-, 100-, 500-acre watersheds.
- Ran infill scenario simulations and computed flow duration statistics
- Examined the infill flow duration statistics and determined at what level of increased development the statistics became noticeably altered.

Per results of the continuous simulation modeling and analysis of the resultant flow duration curves, urban infill projects have a relatively minor effect on the overall watershed's flow duration curve if the future cumulative additional impacts have the potential to increase the existing watershed impervious area by less than 3 percent. Potential urban infill project exemptions are only considered if the existing impervious area percentage of the sub-watershed is at least 40 percent. For sub-watersheds containing less than 40 percent existing impervious area, continuous simulation models indicated a more pronounced response to the flow duration curve with small urban infill developments.

Urban infill projects may be exempt from HMP criteria if:

1. The potential future development impacts within the sub-watershed, as measured from the entire sub-watershed area draining to the existing conveyance system outfall, would not increase the composite impervious area percentage of the sub-watershed by more than 3 percent
2. The project discharges runoff to an existing hardened or rehabilitated conveyance system (storm drain, concrete channel, or engineered vegetated channel) that extends beyond the Domain of Analysis determined for the project site, and
3. The stabilized conveyance system eventually discharges to a channel with a Low susceptibility to erosion, as designed by the SCCWRP channel assessment tool.

The supporting HSPF continuous modeling analysis results, which analyzed existing sub-watershed scenarios of 40, 50, and 60 percent impervious area, are summarized in a Technical Memo in Appendix F.

As mentioned in Section 5.2, the Domain of Analysis is defined to extend downstream of a proposed project site to a location in a natural stream section to where a 50 percent flow accumulation is added to the stream system. For existing storm drain systems or hardened conveyance systems, the Domain of Analysis shall extend downstream to a location where a 100 percent flow accumulation is added to the storm drain or hardened conveyance system. These definitions may be revised in the future subsequent to ongoing work being conducted by the SCCWRP.

6.2 Flow Control Performance Criteria

Figures 6-2 and 6-3, which are part of the HMP Decision Matrix and are presented on the following pages, detail how lower flow thresholds would be determined for a project site. Figures 6-4 and 6-5, which detail the SCCWRP lateral and vertical channel susceptibility requirements, complete the HMP Decision Matrix.

The project applicant must first determine whether field investigations will be conducted pursuant to the SCCWRP channel screening tools. If the screening tools are not completed for a proposed project, then the site must mitigate peak flows and durations based on a pre-project condition lower flow threshold of $0.1Q_2$. While a project applicant would be held to the $0.1Q_2$ standard if channel screening tools and assessments are not conducted, less restrictive standards are possible for more erosion-resistant receiving channel sections if the screening tools are completed and the SCCWRP method indicates either a Medium or Low susceptibility to channel erosion.

In such a scenario, the project applicant would also use the critical shear stress calculator to assist in determination of the predicted lower flow threshold. The SCCWRP screening tools and critical shear stress calculator work in concert to determine the lower flow threshold for a given site. Lower flow limits determined by the calculator have been grouped into one of three thresholds – $0.1Q_2$, $0.3Q_2$ or $0.5Q_2$. “Low” susceptibilities from the SCCWRP tool generally correspond to the $0.5Q_2$ threshold, “Medium” susceptibilities generally correspond to the $0.3Q_2$ threshold, and “High” susceptibilities generally correspond to the $0.1Q_2$ threshold. The SCCWRP channel screening tools are required to identify channel conditions not considered by the critical shear stress calculator, which focuses on channel material and cross section. Conversely, the SCCWRP channel screening tools considers other channel conditions including channel braiding, mass wasting, and proximity to the erosion threshold. In cases where the critical shear stress calculator and the SCCWRP screening tools return divergent values, then the most conservative value shall be used as the lower flow threshold for the analysis.

Low-Impact Development (LID) and extended detention facilities are required to meet peak flow and duration controls as follows:

1. For flow rates ranging from 10 percent, 30 percent or 50 percent of the pre-project 2-year runoff event ($0.1Q_2$, $0.3Q_2$, or $0.5Q_2$) to the pre-project 10-year runoff event (Q_{10}), the post-project discharge rates and durations shall not deviate above the pre-project rates and durations by more than 10 percent over and more than 10 percent of the length of the flow duration curve. The specific lower flow threshold will depend on results from the SCCWRP channel screening study and the critical flow calculator.
2. For flow rates ranging from the lower flow threshold to Q_5, the post-project peak flows shall not exceed pre-project peak flows. For flow rates from Q_5 to Q_{10}, post-project peak flows may exceed pre-project flows by up to 10 percent for a 1-year frequency interval. For example, post-project flows could exceed pre-project flows by up to 10 percent for the interval from Q9 to Q10 or from Q5.5 to Q6.5, but not from Q8 to Q10.

This HMP recommends the use of LID facilities to satisfy both 85th percentile water quality treatment as well as HMP flow control criteria. The Copermittees and the consultant team have developed detailed standards for LID implementation. These standards are provided in the San Diego County Model SUSMP.

The following methods may be used to meet mitigation requirements.

- Install BMPs that meet design requirements to control runoff from new impervious areas. BMPs including bioretention basins, vegetated swales, planter boxes, extended detention basins, etc. shall be designed pursuant to standard sizing and specification criteria detailed in the Model SUSMP and the HMP/LID Sizing Calculator to ensure compliance with hydromodification criteria.
- Use of the automated sizing calculator (San Diego Sizing Calculator) that will allow project applicants to select and size LID treatment devices or flow control basins. The tool, akin to the sizing calculator developed for compliance with the Contra Costa HMP, uses pre-calculated sizing factors to determine required footprint sizes for flow control BMPs. Continuous simulation hydrologic analyses are currently being developed to determine the sizing factors for various flow control options and development scenarios. The Sizing Calculator also includes an automated pond sizing tool to assist in the design of extended detention facilities for mitigation of hydromodification effects. Because of the Sizing Calculator's ease of implementation, and since hydromodification BMPs can also serve as treatment BMPs, it is anticipated that most project applicants will choose this option instead of seeking compliance through site-specific continuous simulation model preparation. The HMP/LID Sizing Calculator is an implementation tool, which is currently under development by the consultant team and will be completed by the time final HMP criteria go into effect.
- Prepare continuous simulation hydrologic models and compare the pre-project and mitigated post-project runoff peaks and durations (with hydromodification flow controls) until compliance to flow control standards can be demonstrated. The project applicant will be required to quantify the long-term pre- and post-project runoff response from the site and establish runoff routing and stage-storage-discharge relationships for the planned flow control devices. Public domain software such as HSPF, HEC-HMS and SWMM can be used for preparation of a continuous simulation hydrologic analysis.
- Points of compliance must be selected to conduct the comparisons of pre-project and post-project flows and durations. Generally, points of compliance are selected at locations along the project boundary where concentrated flows discharge from the project site. If a point of compliance is selected downstream of the project boundary, then the governing municipality should be consulted in advance of the hydromodification analysis. For projects which convey offsite runoff through the site, it is assumed that the offsite runoff would be separated from site runoff. If this is not the case, then the governing municipality should be consulted to further refine the points of compliance for the site (an interior project site point of compliance could be required in such a scenario).

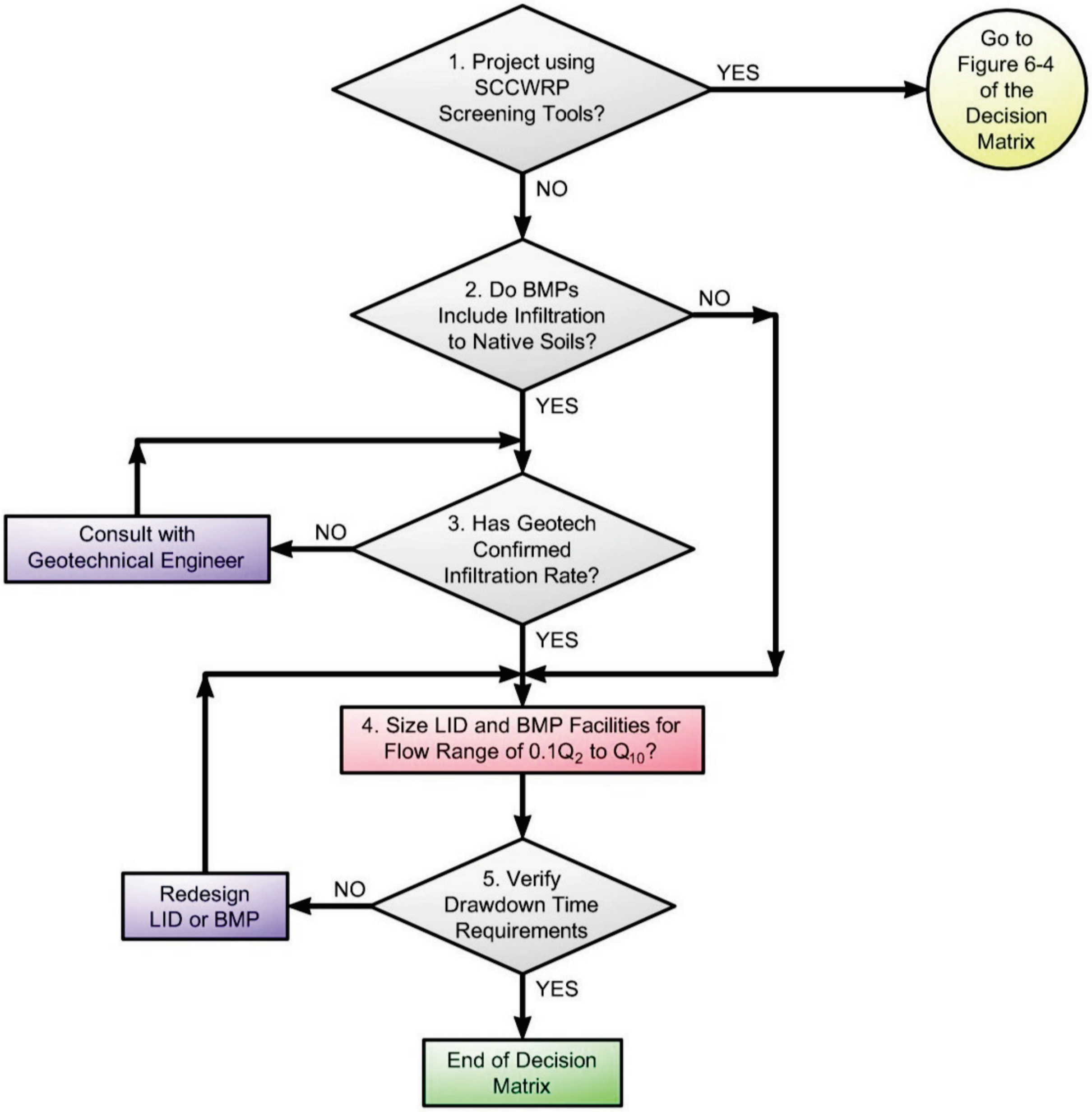

Figure 6-2. Mitigation Criteria and Implementation

- Figure 6-2, Node 1 – If the project applicant chooses to complete SCCWRP channel screening tools, then the applicant moves to Figures 6-4 and 6-5 to assess the vertical and lateral susceptibility of the receiving channel systems. Depending on the results of the SCCWRP screening tools and critical flow calculator, it is possible that lower flow thresholds in excess of $0.1Q_2$ may be used. If the project applicant chooses not to complete the SCCWRP channel assessment, then the applicant proceeds with Figure 6-2 of the Decision Matrix.
- Figure 6-2, Node 2 – If the project's LID or BMP approach accounts for the infiltration of runoff to native surrounding soils (below amended soil layers), then consultation with a geotechnical engineer is required (Box 3). If the project mitigation approach does not account for infiltration of runoff, then the applicant would proceed to Box 4.
- Figure 6-2, Node 3 – A geotechnical engineer should determine the allowable infiltration rates to be used for the design of each LID or BMP facility. The geotechnical assessment should also identify potential portions of the project which are feasible for infiltration of runoff.
- Figure 6-2, Node 4 – In this scenario, the SCCWRP channel assessment was not conducted. Therefore, the project applicant would be held to the $0.1Q_2$ lower flow threshold. LID and extended detention facilities must be sized so that the mitigated post project flows and durations do not exceed pre-project flows and durations for the geomorphically-significant flow range of $0.1Q_2$ to Q_{10}.
- Figure 6-2, Node 5 - The Decision Matrix includes language regarding a drawdown time requirements so that standards set forth by the County's Department of Environmental Health are met. As a side note, the County's Department of Environmental Health has stated that the drawdown requirement would be applied to underground vaults in addition to extended detention basins and the surface ponding areas of LID facilities. Proper maintenance of hydromodification mitigation facilities is essential to guard against potential vector issues as well potential safety issues resulting from long-term standing water. If mitigation facility outlets clog, then runoff will bypass the system and potentially result in additional erosion problems downstream of a site.

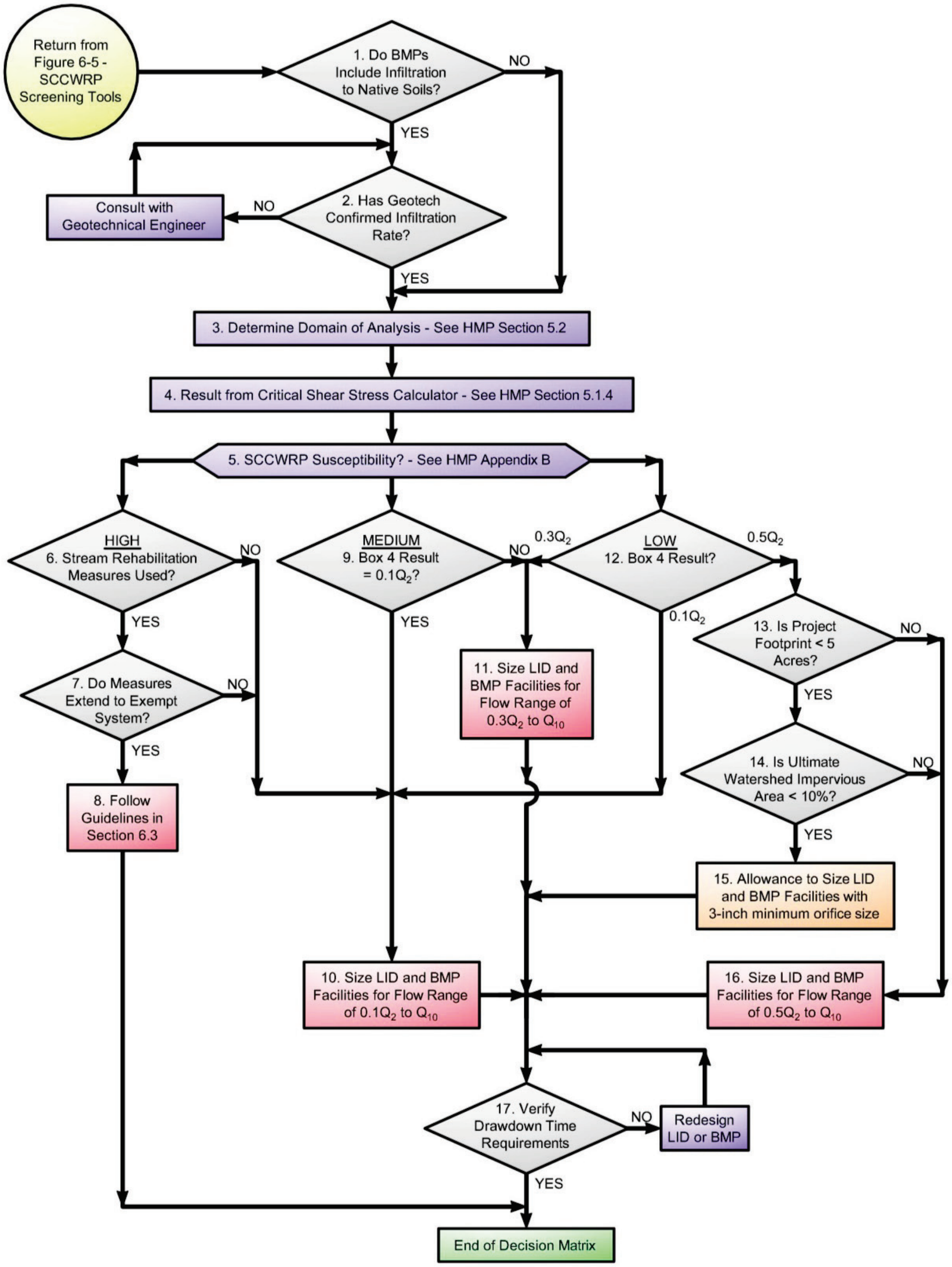

Figure 6-3. Mitigation Criteria and Implementation

- Figure 6-3, Node 1 – Use of Figure 6-3 assumes that the project applicant conducted the SCCWRP channel assessment. Box 1 would begin following completion of both the lateral and vertical susceptibility flow charts depicted in Figures 6-4 and 6-5. Box 1 is a decision box asking if the project's LID or BMP approach accounts for the infiltration of runoff to native surrounding soils (below amended soil layers). If the answer is Yes, then consultation with a geotechnical engineer is required (Box 2). If the project mitigation approach does not account for infiltration of runoff, then the applicant would proceed to Box 3.
- Figure 6-3, Node 2 – A geotechnical engineer should determine the allowable infiltration rates to be used for the design of each LID or BMP facility. The geotechnical assessment should also identify potential portions of the project which are feasible for infiltration of runoff.
- Figure 6-3, Node 3 – Pursuant to criteria detailed in HMP Section 5.2, the Domain of Analysis is determined downstream and upstream of the project site. This determination is used to ascertain the required reach length for data collection (channel bed and bank material, channel cross section data, etc.) required for the critical flow calculator (see Box 4),
- Figure 6-3, Node 4 – Pursuant to criteria detailed in HMP Section 5.1.4, the project applicant would run the critical shear stress calculator to determine if the recommended critical flow threshold should be $0.1Q_2$, $0.3Q_2$, or $0.5Q_2$. This result will be compared to the result from the SCCWRP screening analysis (Box 5) to determine the final lower flow threshold for the project.
- Figure 6-3, Node 5 – Pursuant to criteria detailed in HMP Appendix B, the project applicant would determine both the lateral and vertical channel susceptibility rating per guidelines set forth by SCCWRP. If the lateral and vertical tools returned divergent results, then the more conservative result would be used. SCCWRP susceptibility ratings include "High", "Medium" and "Low."
- Figure 6-3, Node 6 – A project applicant would arrive at Box 6 if the SCCWRP channel susceptibility rating was determined to be "High." This decision box inquires as to whether stream rehabilitation measures such as grade control and channel widening will be used as a mitigation measure instead of flow control. It should be noted that stream rehabilitation options are only allowed if the existing receiving channel susceptibility is considered to be "High."
- Figure 6-3, Node 7 – Stream rehabilitation measures are only allowed if the proposed mitigation project extends to a downstream exempt system (such as an exempt river system). If the mitigation measure did not extend to an exempt system, then the potential for cumulative watershed impacts would be more pronounced.
- Figure 6-3, Node 8 – If stream rehabilitation measures are allowed, then guidelines outlined in Section 6.3 of the HMP should be followed to design the in-stream mitigation approach.
- Figure 6-3, Node 9 - A project applicant would arrive at Box 9 if the SCCWRP channel susceptibility rating was determined to be "Medium." If the result from the critical shear stress calculator is also "Medium" (or $0.3Q_2$), then the lower flow threshold would be $0.3Q_2$ (Box 11). If the result from the critical shear stress calculator is "High" (or $0.1Q_2$), then the more conservative value would be used and the lower flow threshold would be $0.1Q_2$ (Box 10).
- Figure 6-3, Node 10 – For stream reaches determined by either the critical flow calculator or the SCCWRP screening tools to have a "High" susceptibility to erosion, LID and extended detention flow control facilities should be sized so that the mitigated post project flows and durations do not exceed pre-project flows and durations for the geomorphically-significant flow range of $0.1Q_2$ to Q_{10}.

- Figure 6-3, Node 11 - For stream reaches determined by either the critical flow calculator or the SCCWRP screening tools to have a "Medium" susceptibility to erosion, LID and extended detention flow control facilities should be sized so that the mitigated post project flows and durations do not exceed pre-project flows and durations for the geomorphically-significant flow range of $0.3Q_2$ to Q_{10}.
- Figure 6-3, Node 12 - A project applicant would arrive at Box 12 if the SCCWRP channel susceptibility rating was determined to be "Low." If the result from the critical shear stress calculator is also "Low" (or $0.5Q_2$), then the lower flow threshold would be $0.5Q_2$ (Box 16 – note potential waiver in Box 13). If the result from the critical shear stress calculator is "High" (or $0.1Q_2$), then the more conservative value would be used and the lower flow threshold would be $0.1Q_2$ (Box 10). If the result from the critical flow calculator is "Medium" (or $0.3Q_2$), then the more conservative value would be used and the lower flow threshold would be $0.3Q_2$ (Box 11).
- Figure 6-3, Node 13 – In some limited situations, namely small developments in rural or lightly developed areas, an allowance for a minimum outlet orifice size may be granted when the receiving channel susceptibility is "Low." This criteria may potentially be used for project footprints less than 5 acres. If the project footprint is greater than 5 acres, then the allowance may not be granted and the applicant would proceed to Box 16.
- Figure 6-3, Node 14 – The potential allowance discussed in Box 13 could only be granted if the ultimate potential impervious area in the sub-watershed is less than 10 percent. If there is potential for the sub-watershed impervious area to exceed 10 percent, then the minimum orifice size criteria may not be granted.
- Figure 6-3, Node 15 – If Boxes 12, 13, and 14 are satisfied, then mitigation facilities may be designed using a 3-inch minimum outlet orifice size.
- Figure 6-3, Node 16 - For stream reaches determined by either the critical flow calculator or the SCCWRP screening tools to have a "Low" susceptibility to erosion – and for projects where the minimum outlet orifice criteria does not apply - LID and extended detention flow control facilities should be sized so that the mitigated post project flows and durations do not exceed pre-project flows and durations for the geomorphically-significant flow range of $0.5Q_2$ to Q_{10}.
- Figure 6-3, Node 17 – For all hydromodification mitigation designs, the Decision Matrix includes language regarding drawdown time requirements so that standards set forth by the County's Department of Environmental Health are met. As a side note, the County's Department of Environmental Health has stated that the drawdown requirement would be applied to underground vaults in addition to extended detention basins and the surface ponding areas of LID facilities. Proper maintenance of hydromodification mitigation facilities is essential to guard against potential vector issues as well potential safety issues resulting from long-term standing water. If mitigation facility outlets clog, then runoff will bypass the system and potentially result in additional erosion problems downstream of a site.

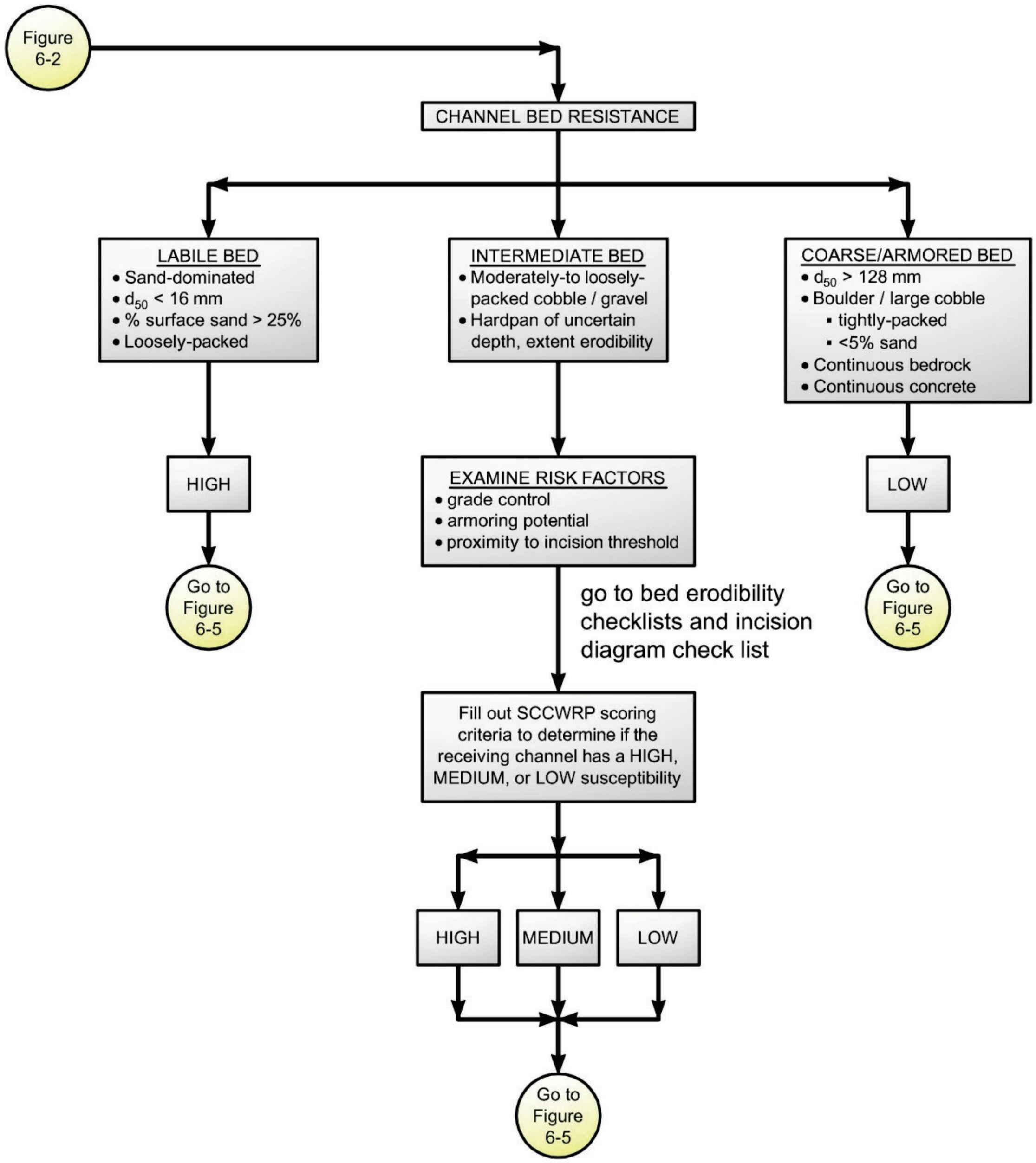

Figure 6-4. SCCWRP Vertical Susceptibility

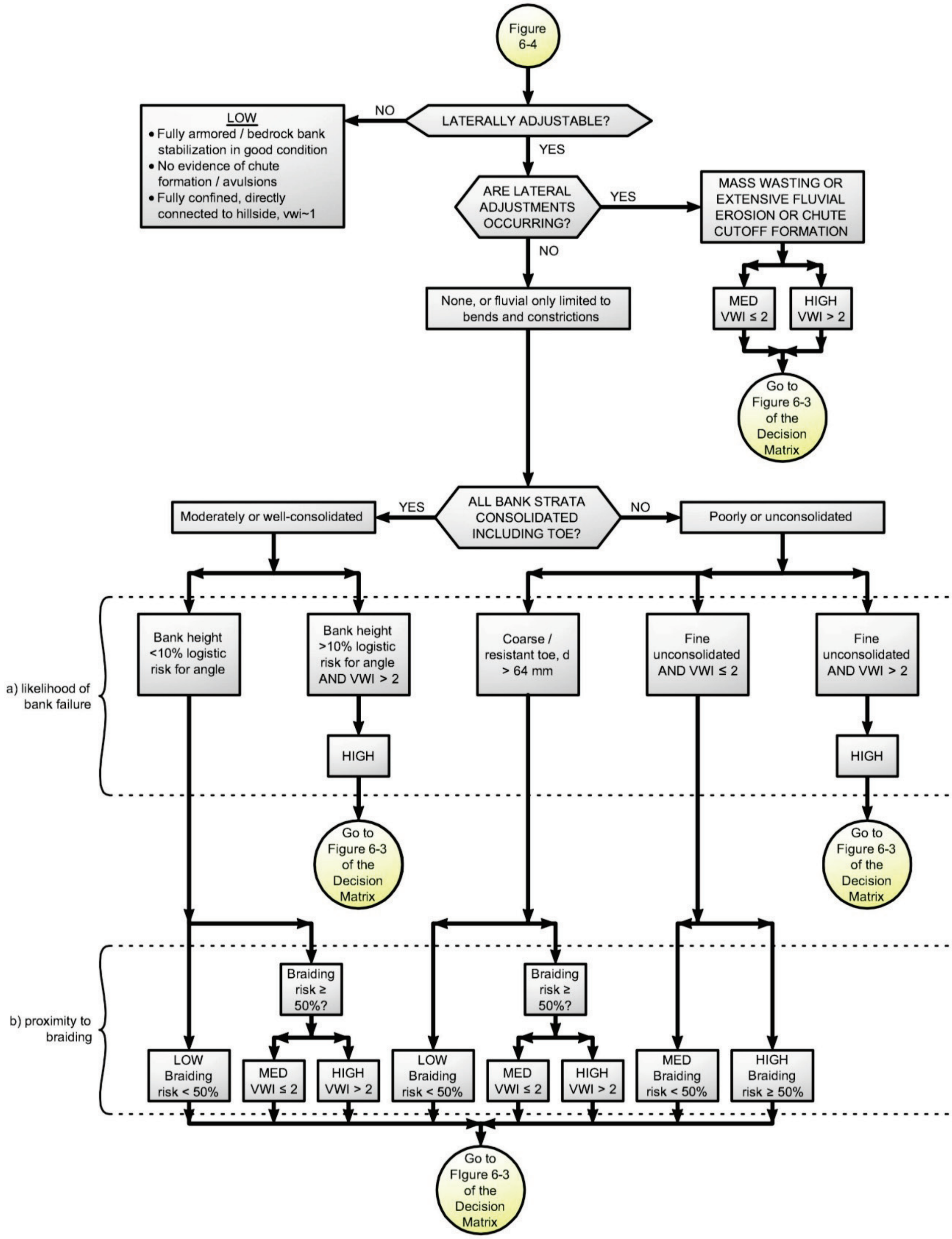

Figure 6-5. Lateral Channel Susceptibility

A continuous simulation analysis was conducted to identify situations where a 3-inch minimum orifice size standard could be applied. For small sites where orifices less than 3-inches would be required to achieve HMP mitigation, LID implementation is recommended in lieu of extended detention facilities.

The continuous simulation analysis was based on the following assumptions:

Sensitivity Analysis for Minimum Orifice Diameter:

- Prepared HSPF models for 100 and 500 acre undeveloped watersheds – assumed Group D soils with sparse vegetation.
- Prepared simple, parallel HSPF models for 1 and 5 acre developments.
- Added detention ponds to the development models, matching the flow duration curve as much as possible using a 3-inch minimum diameter. For the pond configuration, assumed the ponds are 4 feet deep and contained two outflow control structures (low orifice = 3 inches; high weir to prevent overtopping).
- Ran the undeveloped scenario model simulations and computed the flow duration statistics
- Ran the development scenario model simulations (various levels of development) and computed flow duration statistics.
- Determined the increased level of development that would produce a noticeable difference in the flow duration statistics

The sensitivity analysis showed that cumulative basin outflows from multiple 3-inch outlet orifices from 1- and 5-acre developments would have less than significant cumulative impacts to the watershed's flow duration curve provided the impervious areas in the watershed is less than 10 percent. The supporting HSPF continuous modeling analysis results are summarized in a Technical Memo in Appendix F.

For project sites 1 acre or less in size:

1. HMP mitigation must be attained through the use of LID facilities (because a 3-inch outlet orifice would provide no significant mitigation). If LID facilities cannot fully mitigate flows to meet hydromodification criteria, then small detention facilities can be used in combination with the LID facilities.

For project sites greater than 1 acre and less than 5 acres in size:

1. HMP mitigation should be attained through the use of LID facilities to the maximum extent practicable.
2. A 3-inch minimum outlet orifice size may be used provided that the potential cumulative impacts in the subwatershed area, as measured for the entire sub-watershed (containing the project site) downstream to a natural creek confluence, would not increase the ultimate-condition composite impervious area in the subwatershed to more than 10 percent.

If the potential cumulative impacts in the subwatershed areas would result in an impervious area percentage greater than 10 percent, then the 3-inch minimum orifice size waiver would not be granted.

6.3 Stream Rehabilitation Performance Criteria

If the SCCWRP channel screening tools indicate the existing downstream channel condition has a High susceptibility to erosion, then stream rehabilitation options may be considered. Such mitigation measures must extend downstream to an exempt receiving conveyance system. If such options are chosen as hydromodification mitigation for the project site, then the following criteria must be analyzed.

- Show that projected increases in runoff peaks and/or durations, along with sediment reductions associated with development, would not accelerate degradation or erosion of rehabilitated receiving stream reaches.
- A proposed stream rehabilitation mitigation measure can accommodate additional runoff from a proposed project, the project proponent may consider implementation of planning measures such as buffers and restoration activities, revegetation, and use of less-impacting facilities at the point of discharge in lieu of implementation of storm water flow controls.
- Such scenarios include the modification of the channel gradient, cross section, or boundary materials to achieve stable conditions in the altered flow regime. Implementation of such measures would require a geomorphic analysis to show that the proposed changes to the stream channel cross sections, vegetation, discharge rates, velocities, and durations would not have adverse impact to the receiving channel's beneficial uses.
- Such measures could not include concrete.
- Such measures must be designed considering the ultimate condition 100-year flows (as well as lower return frequency events) to the rehabilitated channel segment.

The San Diego HMP has a provision for in-channel mitigation as an alternative, or supplement, to flow volume and duration control. In-stream mitigation involves the modification of the receiving channel (primarily by altering its width, depth, slope and channel materials) to accommodate the increased flow following development. The purpose of this section is to outline for applicants and permitees what components should go into designing and implementing an in-channel mitigation program. It is not intended as an exhaustive 'cookery book' approach to designing an instream approach, but to present the principles that should be used to develop a plan. Most projects will require detailed site-specific analyses and approaches and due to differences in scale, channel type and historic condition there is not necessarily a single approach that will be applicable in all sites.

6.3.1 Goal of In-Channel Hydromod Mitigation

The goal of in-channel hydromod mitigation is to modify a receiving channel such that it supports the beneficial uses and physical and ecological functions of the channel to the same extent or greater than it did prior to the proposed development. More specifically it should:

- Be in geomorphic dynamic equilibrium (it is desirable that is should have small amounts of local scour and deposition to support biological processes, but it should not experience significant net erosion or deposition of sediment over the entire reach over a sustained period of several years).
- Provide the appropriate physical processes and forms to sustainably support the flora and fauna that existed prior to development.

A key step in any project will be to define these goals more clearly. In particular, applicants and permitees will need to agree upon whether the goal is to maintain the creek at pre-project conditions or to restore it to a previous, higher level of function. For example, if the existing condition is an incised channel with little ecological value due to historic impacts, there is little value in stabilizing the creek in this condition to accommodate higher future flows, and an alternate goal will be required such as restoring to a previous condition that is more stable.

6.3.2 Design Principals

Understand Pre-Project Conditions and Potential Project Impacts

All proposed projects must display a clear understanding of the existing physical and ecological condition of the receiving water prior to project implementation. In particular, applicants must identify the ecological functions and values of the existing channel corridor, the physical processes that control or influence them, and the impact of the proposed project on those factors. Table 6-3 provides a hypothetical example but is not intended to be exhaustive.

Identifying the ecological conditions will require the services of a trained riparian and aquatic biologist, while identifying the physical conditions will require a trained geomorphologist or hydrologist. Methods may include field surveys and use of historical documents (maps, aerial photos).

It is important to draw a distinction between 'stability' and 'stasis', and to understand that many ecological functions require a degree of channel disturbance. For example, willow and mulefat assemblages (a common ecotype for many San Diego creeks) require somewhat depositional conditions to form, with alternating periods of sand deposition to create low terraces and subsequent scour and reformation. Many constructed and armored channels are static and do not support the geomorphic functions that underpin these ecological functions.

Table 6-3. Creek Assessment and Mitigation Approaches

Creek Function or Attribute	Current Controlling / Influencing Factors	Project Impacts on Controlling Factors	Potential Mitigation Approach
Vertical channel stability (bed erosion or deposition)	e.g. balance between coarse sediment and water supply, nature of bed materials.	e.g. runoff likely to increase, coarse sediment supply likely to decrease.	Reduce bed gradient using step-pool structures.
Lateral channel stability (e.g. widening, lateral migration)	e.g. vertical stability, riparian vegetation.	e.g. runoff likely to increase, coarse sediment supply likely to decrease.	Widen channel to appropriate geometry and stabilize with biotechnical approaches.
Mulefat assemblage	e.g. requires braided channel with low terraces subject to periodic scour and deposition.	e.g. excess sediment transport capacity over supply will erase terraces and prevent deposition.	Widen channel to lower sediment transport capacity, allow braiding and support terrace formation. Lower gradient to achieve same.
Willow assemblage	e.g. proximity of floodplain to water table.	e.g. incision will lower water table and prevent regeneration.	Prevent incision by grade control, gradient flattening, or channel widening.
Ephemeral vegetation assemblage	e.g. absence of summer nuisance flows.	e.g. presence of summer nuisance flows will allow perennial vegetation to colonize.	Elimination of nuisance flows.
Fish spawning	e.g. presence of gravel, relative absence of fine sediment, relatively low shear stresses during winter/spring flows.	e.g. fine sediment will bury spawning gravel.	Promote sediment sorting and reduce bank erosion or other fine sediment sources.
Fish rearing	e.g. channel complexity, riparian shade cover, relative rarity of high velocity flows.	e.g. excess shear stress will erode and simplify channel features, wash out fish.	Widen and flatten channel to reduce shear stresses.

Design Criteria

In-stream mitigation projects must meet the following design criteria:

1. The proposed channel and riparian corridor must provide the same acreage of habitat as the pre-project channel and riparian corridor, and should support geomorphic processes that can reasonably be considered to sustain those acreages.
2. The cumulative sediment transport capacity of the proposed channel under the post project flow regime must not exceed that of the pre-project channel under the pre-project flow regime. Sediment transport capacity should be assessed at cross sections along the channel at least every 500 feet (minimum of three cross-sections for channels shorter than 500 feet), with the net proposed sediment transport capacity being equal or less than pre-project net sediment transport capacity, and no individual cross section having a sediment transport capacity more than 10 percent greater than under pre-project conditions.

Proposed plans for in-stream HMP mitigation must demonstrate that these criteria will be met by proving a biological report and maps showing the acreage of habitat in pre-and post project conditions, and by providing hydraulic and sediment transport analyses that show the following:

1. For projects larger than 50 acres the analysis should be based on continuous rainfall-runoff modeling, and continuous sediment transport capacity modeling. The analysis should demonstrate that the cumulative sediment transport capacity in the proposed channel based on the channel dimensions and watershed runoff under post-project conditions is the same or less than the cumulative sediment transport capacity for the existing channel based on the channel dimensions and watershed runoff under pre-project conditions. The period of analysis should be the approved rainfall record for the closest appropriate rain gauge as found on the www.projectcleanwater.org web site.
2. For projects smaller than 50 acres the analysis may be based on sediment transport capacity for a series of designated runoff events. The analysis should demonstrate that the sediment transport capacity in the proposed channel based on the channel dimensions and watershed runoff under post-project conditions is the same or less than the sediment transport capacity for the existing channel based on the channel dimensions and watershed runoff under pre-project conditions for the following events: 0.1Q2, Q2 and Q10.

Methods for performing this analysis are described below.

Matching Pre- and Post-Project Cumulative Sediment Transport Capacity

A key component of any in-channel project will be to quantify and balance the pre- and post-project sediment transport regime in channel that are stable under pre-development conditions, and to lower sediment transport capacity for channels that are unstable under existing conditions. This method is sometimes referred to as the Erosion Potential method. There are several potential tools to assess this and design the channel to meet these goals, but certain principals must be incorporated in whatever approach is used.

For developments larger than 50 acres the analysis must be based on continuous rainfall-runoff modeling, rather than event-based modeling. This is because research has shown that in most urbanized watersheds significant amounts of sediment transport occur during low magnitude, high frequency events (smaller than the two-year flow). Quantification of sediment transport capacity will not capture these processes unless continuous rainfall-runoff simulation is used. Potential models to achieve this include HEC-HMS in continuous mode, SWMM, HSPF, and the San Diego Hydrology Model. Modeling should include at least 40 years of rainfall data from a nearby rain gauge. Modeling should include pre- and post-project conditions. Output (a time series of flow) should be used to quantify pre- and post-project cumulative sediment transport capacity. This can be achieved in several ways, varying from a simple spreadsheet-based sediment transport model to a full one-dimensional hydraulic and sediment transport model such as HEC-RAS (sediment transport module), HEC-6, Fluvial-12, or MIKE-11. The model should simulate the existing and proposed channel morphology in sufficient detail to allow analysis of potential modifications to cross sections and gradient. A hypothetical example is described below.

A hypothetical analysis might include modeling the existing watershed land use in HEC-HMS and generating a 40-year time series of flow at hourly intervals. This time series would be the input for a HEC-RAS hydraulic and sediment transport model of the existing receiving channel. The time series would be run using a sediment transport equation appropriate to the channel materials, and the cumulative sediment transport capacity over 40 years calculated. The proposed development would then be simulated in HEC-HMS and the 40-year flow output run through a HEC-RAS model of the proposed in-channel mitigation (for example with a lower gradient, in-channel step-pool structures and wider cross section). Cumulative sediment transport capacity would again be calculated. If the proposed channel-with-project cumulative sediment transport capacity was equal to or less than the existing pre-project channel cumulative sediment transport capacity then the channel would have met the sediment transport goals. If the cumulative sediment transport capacity was higher the channel design would have to be refined to lower transport rates or some flow control would be required in the watershed, until the transport capacities either matched or were lower than pre-project condition.

For developments smaller than 50 acres event based analysis may be used. The applicant must calculate the flows for $0.1Q_2$, Q_2 and Q_{10} using continuous rainfall-runoff modeling, and determine the sediment transport capacity using either a sediment transport model or spreadsheet model. If the proposed channel has an equal or lower sediment transport capacity at all three flows it would meet the sediment transport criteria. If it did not the applicant would need to iteratively vary the channel dimensions or manage runoff until the criteria were met.

Methods of Reducing Sediment Transport Capacity

It is highly likely that in a watershed experiencing hydromod without significant flow control the sediment transport capacity will be greatly increased (commonly by a factor of 5 or more for highly developed watersheds) while sediment supply will be reduced. This will likely require a significant modification in channel geometry to bring sediment transport capacity back to pre-project levels. This can be achieved in several ways:

Slope Reduction by Construction of Step-Pools or Roughened Channels

Step-pools are vertical or near vertical sections in a channel profile (step) with a flat section that dissipates the energy of the step (pool). A natural feature of upland creeks, step-pools are sometimes built into creek rehabilitation projects to concentrate bed elevation loss in a small number of hardened areas where erosion is unlikely to occur and allow the remainder of the bed to be designed at a lower gradient that reduces sediment transport capacity. Step-pools can be constructed from uncemented boulders of appropriate size (designed to be stable during design flood events such as the 100-year flow), or from soil cement or other hard materials. The gradient between steps can be designed to match the EP for the pre-project condition without the need for armor, with the difference between the channel's existing and post-project gradient being taken up in vertical steps. Steps should be designed to meet any relevant fish passage and animal migration requirements (e.g., for fish bearing streams steps should be no higher than 3 feet).

Roughened channels are a similar approach where the elevation loss occurs at armored rock reaches typically with a gradient of 10 percent over a few tens of feet (e.g., 3 feet of drop over 30 feet of roughened channel). As with step-pools these are employed between longer reaches of un-armored stable channel at a lower gradient.

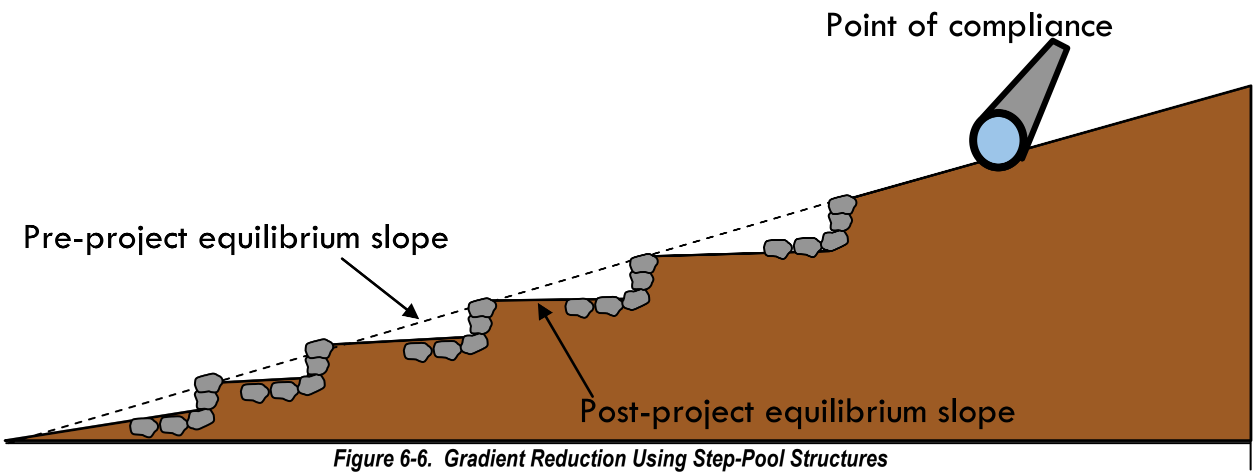

Figure 6-6. Gradient Reduction Using Step-Pool Structures

Slope Reduction by Sinuosity Increase

In some cases small reductions in slope can be achieved by increasing sinuosity (ratio of channel distance between two points to straight line distance). For example, a 30 percent reduction in slope can be achieved by converting a straight receiving channel into a channel with a sinuosity of 1.3 (typical for a meandering channel). However, it is important to understand that channel sinuosity is a dependent variable that is influenced by the valley gradient and the sediment and water regime of the watershed. As a general rule Forcing a channel to a sinuosity that is inappropriately high is likely to lead to subsequent channel avulsion to a straighter course. Channel sinuosity needs to be supported by a geomorphic basis of design that shows the proposed form and gradient to be appropriate for the valley slope and sediment and water regime. This may take the form of reference reaches in similar watersheds that have support the proposed morphology over a significant period of time, or comparison between the proposed form and typical literature values.

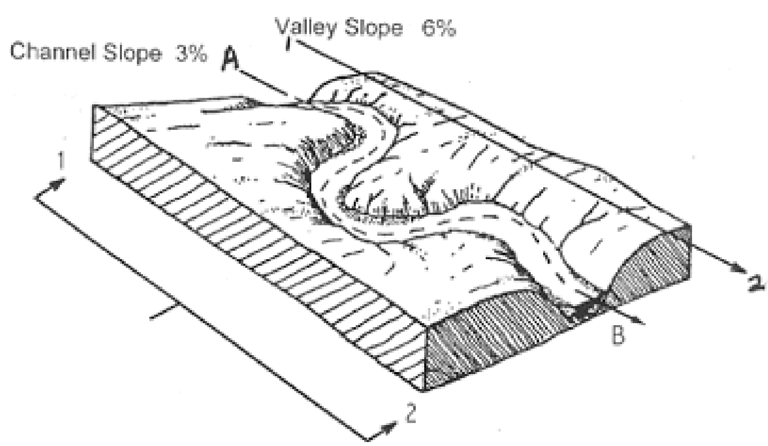

Figure 6-7. Gradient Reduction by Increasing Sinuosity

Increased Width:Depth Ratio

Sediment transport capacity can be reduced by increasing width:depth ratio for the channel (both low flow channel and floodplain). By spreading flows out over a wider cross section with lower depths, shear stress is reduced for any given flow rate. This approach can be a useful mitigation strategy in incised creeks to bring them back to equilibrium conditions once vertical incision has ceased. However, as with sinuosity, it is important to develop a robust geomorphic basis of design that shows the increase in width:depth ratio to be sustainable. For example, for sand bed channels in watersheds where the coarse sediment supply is greatly reduced by urbanization, low flows may cut into the bed of an over-widened channel, leading to a positive feedback loop of incision and flow concentration. Proposed designs will need to show (using stable analogous reference reaches or analytical methods such as sediment transport analysis) that width:depth ratios are sustainable.

6.3.3 Size Channel for Changed Dominant Discharge

A mitigated channel is likely to consist of a low flow channel that provides the aquatic functions of the pre-project channel, and a floodplain corridor that supports the pre-project riparian functions. The low flow channel should be sized to meet the new dominant discharge of the post-project watershed. In most cases this will be a more frequent event than under pre-development conditions. For example, a low flow channel may accommodate the five year flow under pre-development conditions but be sized for the one-two year flow under developed conditions. For large developments, the EP analysis used to determine cumulative sediment transport capacity will provide the dominant discharge. If EP is plotted as a histogram of sediment transport capacity binned into flow ranges the flow range that produces the highest sediment transport capacity will be the dominant discharge. The floodplain area of the riparian corridor should be designed so as to match the inundation frequencies, areas, and elevations of the pre-project channel.

6.3.4 Upstream and Downstream Limits of In-Channel Mitigation Projects

It is likely that in-channel mitigation projects will have to be negotiated with permitting agencies on a case by case basis due to different site conditions. However, for guideline purposes we recommend the following approach to identifying the limits of in-channel mitigation projects.

The upstream limit of an in-channel mitigation project will typically be the point of compliance (PoC ; point at which stormwater is discharged into the receiving water). However, as a precaution against potential unplanned erosion following a project it is recommended that either the project extend upstream to the next grade control, or that grade control be added immediately upstream of the point of compliance.

The downstream limit of an in-channel project would be the connection to an exempt system (such as the confluence with an exempt river system).

6.3.5 Relationship Between In-Channel HMP Mitigation and Existing Permit Requirements

The HMP does not replace existing permit requirements for in-channel projects. In addition to meeting the HMP requirements, applicants proposing an in-channel mitigation project will likely require the following permits:

- A CEQA/NEPA review and document
- California Department of Fish and Game – 1602 Streambed Alteration Agreement
- US Fish and Wildlife Service – Authorization Under the Endangered Species Act

- US Army Corps of Engineers – Nationwide 404 Permit
- Regional Water Quality Control Board – 401 Water Quality Certification
- County of San Diego – Grading Permit

These permits have their own requirements that may involve additional studies beyond those described above.

6.4 HMP Design Standards

6.4.1 Introduction

This Technical Memorandum details criteria for the analysis and methodology used to assess mitigation of hydromodification effects. As mandated by Regional Water Quality Control Board (RWQCB) Order R9-2007-0001, San Diego Copermittees must develop criteria for the mitigation of development-related increases to peak flows and flow durations within the geomorphically significant flow range. The purpose of the hydromodification management criteria is to prevent development-related changes in storm water runoff from causing, or further accelerating, stream channel erosion or other adverse impacts to beneficial stream uses.

Three specific areas are discussed in this memorandum.

- Partial Duration Series Calculations
- Drawdown Calculations
- Offsite Area Restrictions

Information contained in this memorandum will be incorporated into a Final Technical Reference Document in support of the final HMP document.

6.4.2 Partial Duration Series Calculations

Preliminary review of continuous simulation hydrologic analyses prepared for multiple project sites throughout the County of San Diego indicates the need for partial duration series calculations to determine estimated return flow frequencies. Because of San Diego's semi-arid climate, in which long periods of time can elapse between significant rainfall events, use of the peak annual series tends to unrealistically underestimate flow return event values (since only the peak event in any given year is considered in the analysis). This effect is particularly pronounced for more frequent return events such as the 2-year flow and the 5-year flow (note: the 2-year flow is the runoff rate which statistically has a 50 percent chance of occurrence in any given year). The partial duration series calculations consider all significant rainfall events in the long-term rainfall record (which for the San Diego area corresponds to a minimum historical record of hourly rainfall totals for 35 years).

This partial duration series data provided below were prepared based on a sample project in south San Diego County. Using the Lower Otay Reservoir rainfall gauge as the historical rainfall record, the subsequent commentary shows how a partial duration series analysis should be conducted to estimate peak runoff rates for frequencies of 2-, 5-, and 10-year recurrence given hydrologic modeling results for hypothetical Basins A, B, C and D.

6.4.3 Data

Four modeling files, corresponding to Basins A, B, C and D from a proposed development project, were prepared using the HSPF hydrologic modeling software. Relevant time series were output to WDM files, which were named for the modeled basin (e.g., *Basin A.wdm*). Two land use conditions were generated:

- Pre-developed flow in cubic feet per second (cfs)
- Post-developed (unmitigated) flow in cfs

Given two flow scenarios (above) and four basins, a total of 8 sets of time series data were identified for flow frequency analysis. Plots of these flow data are included later in this document.

6.4.4 Analysis

Each of the 8 time series data files described in the previous section were exported from the WDM file using *WDM Util*. The exported files were then imported into MatLAB and a previously developed script was used to convert the complete duration-time series to a partial duration time series using the criteria shown in Table 6-4 below.

The previously developed partial duration script was developed in association with development of the Contra Costa HMP / LID Sizing Tool, which was approved by the San Francisco Regional Water Quality Control Board for review of project-specific hydromodification plans (*Contra Costa Hydromodification Management Plan* - May 15, 2005). Similar methodology is included in the San Diego HMP / LID Sizing Calculator.

Table 6-4. Partial Duration Series Criteria

Basin and Scenario		Separation Event (hours)	Flow Floor (cfs)	Number of Events
Basin A	Pre-developed	24	0.1	357
	Post-developed (unmitigated)	24	0.1	620
Basin B	Pre-developed	24	0.01	63
	Post-developed (unmitigated)	24	0.1	540
Basin C	Pre-developed	24	0.01	73
	Post-developed (unmitigated)	24	0.1	535
Basin D	Pre-developed	24	0.1	104
	Post-developed (unmitigated)	24	0.1	558

The columns listed in Table 6-4 describe criteria detailed below.

- A separation event, defined as time period in which runoff does not exceed a prescribed threshold, is required to parse the long-term flow records into discrete runoff events. The separation event corresponds to the required number of consecutive time intervals (hours in this case because the long-term rainfall records were prepared in hourly time steps) with a flow value less than Flow Floor 1 (which is calculated as an artificially low flow value based on a fraction of the contributing watershed areas – for instance, the flow floor could correspond to ratios in the range of 0.002 cfs/acre to 0.005 cfs/acre).
- Flow Floor 1 is the maximum value for the inter-event time period (allows for separation of events). In other words, if no flow value exceeds the Flow Floor 1 value for a time equal to or greater than the Separation Event, then the preceding runoff event is viewed as a discrete runoff event. Flow Floor 1 is typically set as an artificially low flow value based on a fraction of the contributing watershed area.

- Flow Floor 1 is also the minimum value for the rainfall event. In other words, if no flow value in the event exceeds Flow Floor 1, then the minor runoff is not considered a discrete runoff event.
- Number of events corresponds to the total number of discrete runoff events generated for the long-term rainfall record. As noted in Table 6-4 and graphically depicted in the figures at the end of this section, impervious area addition associated with development dramatically increases the number of discrete runoff events for the sample basins.

The partial duration series data were ranked and the plotted using the Cunnane equation for plotting return frequency. The Cunnane equation documentation can be referenced in the "Handbook of Hydrology" by David R. Maidment, published in 1994 (Table 18.3.1).

6.4.5 Results

Flow frequency plots are included later in this document. Flow frequency estimates were obtained from these plots for the 2-, 5-, and 10-year recurrence intervals. The results are summarized in Table 6-5.

Table 6-5. Flow Frequency by Partial Duration Series Analysis

Basin and Scenario		Peak Runoff (cfs) by Recurrence Interval		
		2-year	5-year	10-year
Basin A	Pre-developed	1.2	3.1	6.3
	Post-developed (Unmitigated)	4.8	6.9	8.8
Basin B	Pre-developed	0.2	0.6	1.2
	Post-developed (Unmitigated)	1.2	2.0	2.4
Basin C	Pre-developed	0.2	0.9	1.8
	Post-developed (Unmitigated)	1.5	2.5	3.0
Basin D	Pre-developed	0.5	1.2	2.5
	Post-developed (Unmitigated)	1.4	2.0	2.5

As shown in Figures 6-8 and 6-9, impervious area increases associated with proposed development dramatically increases the frequency and intensity of flows throughout the rainfall record. While the scenario modeled above depicts a worst-case scenario where undeveloped land is converted to highly impervious industrial land, similar but less pronounced increases to flow frequency and peak flows would be expected for other development types. The degree of change is dependent on the degree of impervious areas and landform modification.

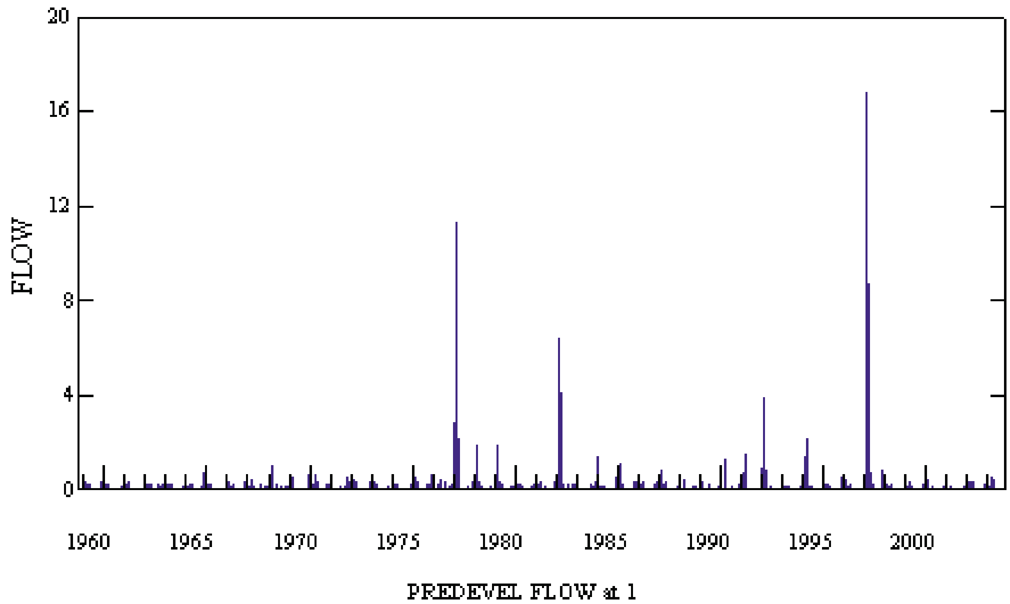

Figure 6-8. Pre-Developed Flow Time Series for Basin A

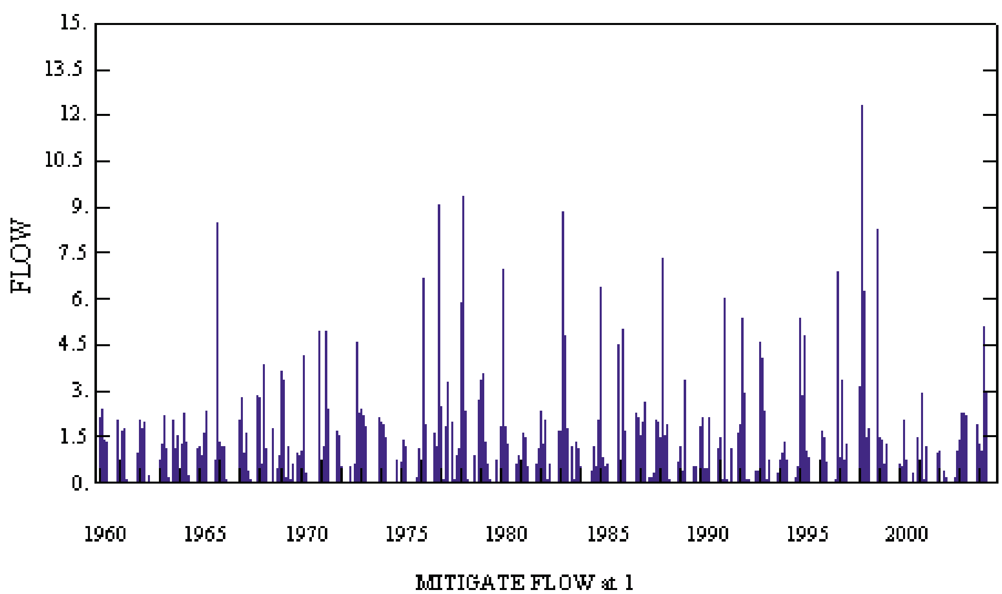

Figure 6-9. Post-Developed (unmitigated) Flow Time Series for Basin A

As shown in Figures 6-10 and 6-11, increases in impervious areas associated with development create a significant flow regime change for the full range of flows. These changes are most pronounced for frequent flow events. As detailed on the figures, development would increase the 1-year pre-project flow of 0.5 cfs to a 1-year post-project flow of 3.0 cfs. At the 5-year event, the pre-project flow is 3 cfs while the post-project flow increases to 7 cfs. At the 10-year event, the pre-project is 6.5 cfs while the post-project flow is 9 cfs.

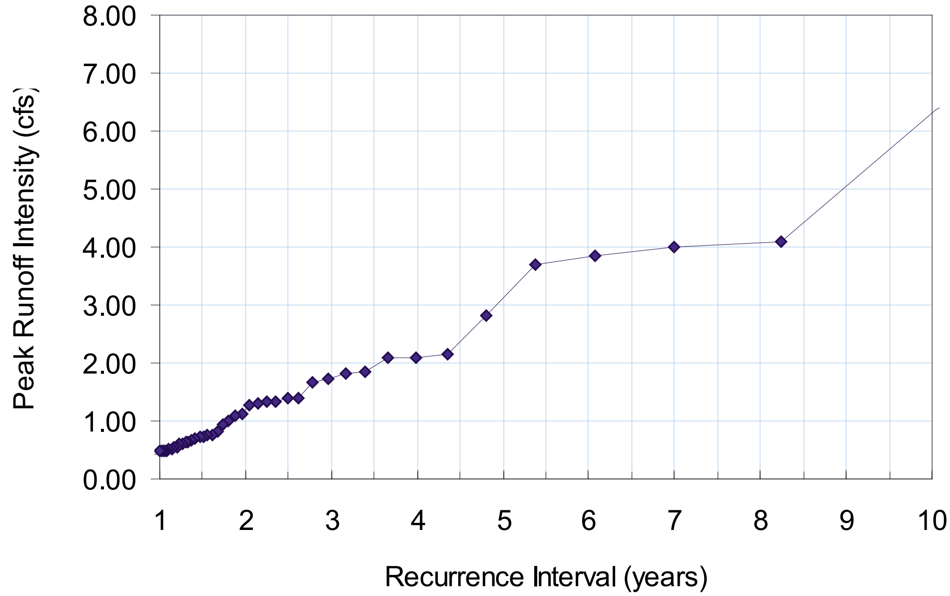

Figure 6-10. Pre-Developed Flow Frequency Basin A

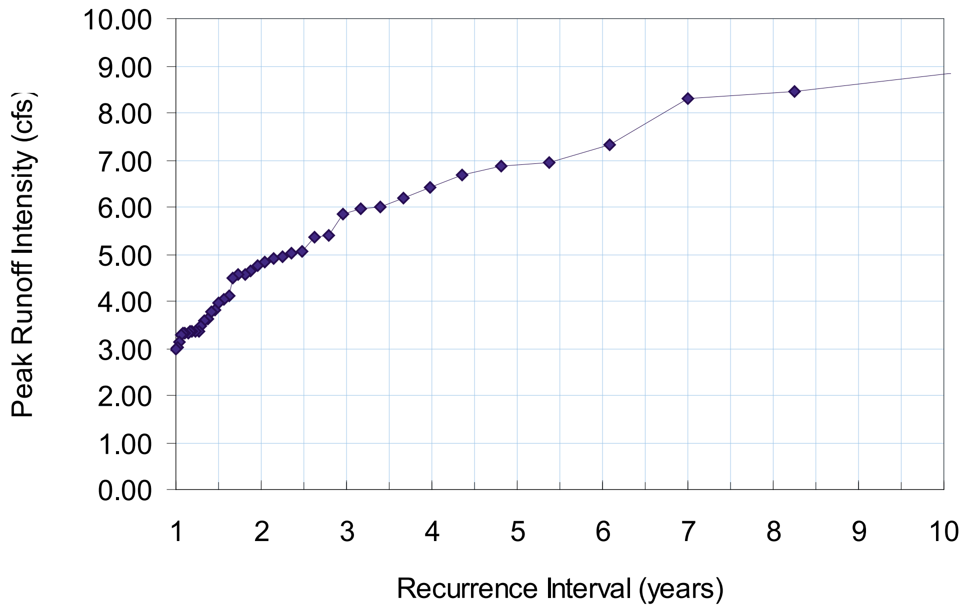

Figure 6-11. Post-Developed (Unmitigated) Flow Frequency Basin A

6.4.6 Drawdown Calculations

Per instruction from the County of San Diego's Department of Environmental Health (DEH), the drawdown time in hydromodification flow control facilities, as well as other flow control facilities such as peak flow attenuation detention basins and water quality extended detention basins, shall be limited to 96 hours. This restriction was implemented as mitigation to potential vector breeding issues and the subsequent risk to human health. The standard applies to, but is not limited to, the following flow control facilities:

- Detention basins (extended detention and peak flow attenuation)
- Underground storage vaults
- Above-ground storage area in LID facilities

As is the case for peak flow attenuation detention basins and water quality extended detention basins, the drawdown time for hydromodification flow control facilities can be calculated by assuming a starting water surface elevation coincident with the peak operating level in the facility (such as the elevation at the riser overflow or emergency spillway overflow).

Using a hydrologic computer program such HEC-HMS or other public domain software, the basin's dewatering time can be determined given the basin's stage-storage and stage-discharge information. Provided that the basin has completely dewatered after 96 hours, the basin is considered to meet the drawdown criteria.

If an applicant cannot achieve the 96-hour drawdown requirement, a vector management plan may be an acceptable alternative if approved by the governing municipality.

For extended detention flow control facilities, protective fencing may be required to address safety concerns associated with the extended duration of ponded water. Specifications regarding protective fencing requirements will be determined by each individual Copermittee. If a riser is installed in the basin, it is assumed that flows would exit the basin via a small orifice or a series of orifices cut into the side of the riser. To prevent clogging, debris capture devices should be designed to protect the principal outflow orifice. Failure to prevent clogging could actually make downstream erosion problems worse, since basin inflows would simply overtop the riser and flow unattenuated downstream.

6.4.7 Offsite Area Restrictions

Runoff from offsite areas should be routed around hydromodification flow control facilities. This is required because of the following:

- Offsite areas containing sediment should be allowed to pass to the receiving channel to maintain the natural sediment balance in the receiving conveyance system. This is especially true when the offsite area contains significant loads of coarse sediment. Capture and removal of natural sediment from the downstream watercourse can create "hungry water" conditions and the increased potential for downstream erosion. The "hungry water" phenomenon occurs when the natural sediment load decreases and the erosive force of the runoff increases as a natural counterbalance, as described by Lane's Equation.
- The addition of runoff from offsite areas to a hydromodification flow control facility increases the total runoff volume to the basin, which increases the required water quality treatment volume as well as the hydromodification and peak flow attenuation design peak inflows to the basin.

If geometric constraints prohibit the rerouting of flows around a hydromodification flow control facility, then a detailed description of the constraints should be submitted to the governing municipality. Methods to route flows around flow control facilities include the addition of parallel storm drain systems and by simply designing the site to avoid natural drainage courses. It is assumed that off-site runoff would be separated from site runoff. If this is not the case, then the governing municipality should be consulted to further refine the points of compliance for the site (an interior project site point of compliance could be required in such a scenario).

7. SELECTION AND IMPLEMENTATION OF BMPS

7.1 BMP Selection Criteria

As detailed in Permit Section D.1.d(4), LID BMPs should be implemented where feasible. Selection of the appropriate flow control treatment device will depend on the susceptibility of the receiving channel, geologic conditions in the area surrounding the proposed mitigation facility, impacts of the proposed development, and water quality sensitivity of the receiving streams.

Use of LID BMPs minimizes the impacts of urban runoff discharges to receiving waters by collectively minimizing directly connected impervious areas. By directing urban runoff to landscaped areas, LID BMPs help restore the pre-development condition hydrologic cycle of the site, allowing for filtration and infiltration of urban runoff which can significantly reduce post-development peak runoff rates, velocities, volumes, and pollutant loadings in urban runoff.

The San Diego HMP encourages the use of LID facilities for the dual treatment of the 85th percentile water quality event as well as hydromodification mitigation flow control. When LID facilities are used for both functions, they are known as Integrated Management Practices.

Unless specifically deemed infeasible, LID practices are encouraged to be implemented on the vast majority of proposed development sites to meet hydromodification criteria. Defining the infiltration potential of a site is recommended to provide for sound engineering design. In some cases, infiltration to native soils may not be feasible. These situations include the following:

- Underlying native soils with very low infiltration rates (clay soils, etc.)
- Lenses beneath soil layers that cause lateral migration of flows
- Potential for structural foundation or roadway damage from infiltrated runoff
- High groundwater table

Even if infiltration is shown to be infeasible, LID facilities can be designed as filtration-type or evaporation-type facilities instead of infiltration-based facilities. Filtration type facilities, such as bioretention basins, can be implemented through the use of amended soils. In some cases, LID approaches may need to be implemented in series or in combination with an extended detention type approach to satisfy vector control and hydromodification criteria.

To assure compliance with hydromodification flow control requirements, design criteria and specifications have been provided in the San Diego Model SUSMP for a variety of LID-based flow control methods including the following:

- Bioretention basins
- Flow-through planter boxes
- Infiltration facilities
- Bioretention in series with a cistern
- Bioretention in series with an underground vault
- Self-retaining areas.

Sizing factors have been developed by the consultant team through the use of continuous simulation hydrologic modeling and these factors will be built into the San Diego LID/HMP Sizing Calculator to assist with HMP implementation. Sizing factors are ratios of the required mitigation size (in area or volume) as compared to the contributing developed area. The same concepts used to develop sizing factors in Contra Costa County are being used to develop sizing factors based on conditions in the San Diego area. Tables 7-1 through 7-5 detail sizing factors which have been determined to ensure compliance with peak flow and flow duration criteria as outlined in this HMP.

Table 7-1. Sizing Factors for Bioretention Facilities

Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.5Q_2$	A	Flat	Lindbergh	0.060	0.0500	N/A
$0.5Q_2$	A	Moderate	Lindbergh	0.055	0.0458	N/A
$0.5Q_2$	A	Steep	Lindbergh	0.045	0.0375	N/A
$0.5Q_2$	B	Flat	Lindbergh	0.093	0.0771	N/A
$0.5Q_2$	B	Moderate	Lindbergh	0.085	0.0708	N/A
$0.5Q_2$	B	Steep	Lindbergh	0.065	0.0542	N/A
$0.5Q_2$	C	Flat	Lindbergh	0.100	0.0833	0.0600
$0.5Q_2$	C	Moderate	Lindbergh	0.100	0.0833	0.0600
$0.5Q_2$	C	Steep	Lindbergh	0.075	0.0625	0.0450
$0.5Q_2$	D	Flat	Lindbergh	0.080	0.0667	0.0480
$0.5Q_2$	D	Moderate	Lindbergh	0.080	0.0667	0.0480
$0.5Q_2$	D	Steep	Lindbergh	0.060	0.0500	0.0360
$0.5Q_2$	A	Flat	Oceanside	0.070	0.0583	N/A
$0.5Q_2$	A	Moderate	Oceanside	0.065	0.0542	N/A
$0.5Q_2$	A	Steep	Oceanside	0.060	0.0500	N/A
$0.5Q_2$	B	Flat	Oceanside	0.098	0.0813	N/A
$0.5Q_2$	B	Moderate	Oceanside	0.090	0.0750	N/A
$0.5Q_2$	B	Steep	Oceanside	0.075	0.0625	N/A
$0.5Q_2$	C	Flat	Oceanside	0.075	0.0625	0.0450
$0.5Q_2$	C	Moderate	Oceanside	0.075	0.0625	0.0450
$0.5Q_2$	C	Steep	Oceanside	0.060	0.0500	0.0360
$0.5Q_2$	D	Flat	Oceanside	0.065	0.0542	0.0390
$0.5Q_2$	D	Moderate	Oceanside	0.065	0.0542	0.0390
$0.5Q_2$	D	Steep	Oceanside	0.050	0.0417	0.0300
$0.5Q_2$	A	Flat	L Wohlford	0.050	0.0417	N/A
$0.5Q_2$	A	Moderate	L Wohlford	0.045	0.0375	N/A
$0.5Q_2$	A	Steep	L Wohlford	0.040	0.0333	N/A
$0.5Q_2$	B	Flat	L Wohlford	0.048	0.0396	N/A
$0.5Q_2$	B	Moderate	L Wohlford	0.045	0.0375	N/A
$0.5Q_2$	B	Steep	L Wohlford	0.040	0.0333	N/A

Table 7-1. Sizing Factors for Bioretention Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.5Q_2$	C	Flat	L Wohlford	0.065	0.0542	0.0390
$0.5Q_2$	C	Moderate	L Wohlford	0.065	0.0542	0.0390
$0.5Q_2$	C	Steep	L Wohlford	0.050	0.0417	0.0300
$0.5Q_2$	D	Flat	L Wohlford	0.055	0.0458	0.0330
$0.5Q_2$	D	Moderate	L Wohlford	0.055	0.0458	0.0330
$0.5Q_2$	D	Steep	L Wohlford	0.045	0.0375	0.0270
$0.3Q_2$	A	Flat	Lindbergh	0.060	0.0500	N/A
$0.3Q_2$	A	Moderate	Lindbergh	0.055	0.0458	N/A
$0.3Q_2$	A	Steep	Lindbergh	0.045	0.0375	N/A
$0.3Q_2$	B	Flat	Lindbergh	0.098	0.0813	N/A
$0.3Q_2$	B	Moderate	Lindbergh	0.090	0.0750	N/A
$0.3Q_2$	B	Steep	Lindbergh	0.070	0.0583	N/A
$0.3Q_2$	C	Flat	Lindbergh	0.110	0.0917	0.0660
$0.3Q_2$	C	Moderate	Lindbergh	0.110	0.0917	0.0660
$0.3Q_2$	C	Steep	Lindbergh	0.085	0.0708	0.0510
$0.3Q_2$	D	Flat	Lindbergh	0.100	0.0833	0.0600
$0.3Q_2$	D	Moderate	Lindbergh	0.100	0.0833	0.0600
$0.3Q_2$	D	Steep	Lindbergh	0.070	0.0583	0.0420
$0.3Q_2$	A	Flat	Oceanside	0.070	0.0583	N/A
$0.3Q_2$	A	Moderate	Oceanside	0.065	0.0542	N/A
$0.3Q_2$	A	Steep	Oceanside	0.060	0.0500	N/A
$0.3Q_2$	B	Flat	Oceanside	0.098	0.0813	N/A
$0.3Q_2$	B	Moderate	Oceanside	0.090	0.0750	N/A
$0.3Q_2$	B	Steep	Oceanside	0.075	0.0625	N/A
$0.3Q_2$	C	Flat	Oceanside	0.100	0.0833	0.0600
$0.3Q_2$	C	Moderate	Oceanside	0.100	0.0833	0.0600
$0.3Q_2$	C	Steep	Oceanside	0.080	0.0667	0.0480
$0.3Q_2$	D	Flat	Oceanside	0.085	0.0708	0.0510
$0.3Q_2$	D	Moderate	Oceanside	0.085	0.0708	0.0510
$0.3Q_2$	D	Steep	Oceanside	0.065	0.0542	0.0390
$0.3Q_2$	A	Flat	L Wohlford	0.050	0.0417	N/A
$0.3Q_2$	A	Moderate	L Wohlford	0.045	0.0375	N/A
$0.3Q_2$	A	Steep	L Wohlford	0.040	0.0333	N/A
$0.3Q_2$	B	Flat	L Wohlford	0.060	0.0500	N/A
$0.3Q_2$	B	Moderate	L Wohlford	0.055	0.0458	N/A
$0.3Q_2$	B	Steep	L Wohlford	0.045	0.0375	N/A
$0.3Q_2$	C	Flat	L Wohlford	0.075	0.0625	0.0450

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Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.3Q_2$	C	Moderate	L Wohlford	0.075	0.0625	0.0450
$0.3Q_2$	C	Steep	L Wohlford	0.060	0.0500	0.0360
$0.3Q_2$	D	Flat	L Wohlford	0.065	0.0542	0.0390
$0.3Q_2$	D	Moderate	L Wohlford	0.065	0.0542	0.0390
$0.3Q_2$	D	Steep	L Wohlford	0.050	0.0417	0.0300
$0.1Q_2$	A	Flat	Lindbergh	0.060	0.0500	N/A
$0.1Q_2$	A	Moderate	Lindbergh	0.055	0.0458	N/A
$0.1Q_2$	A	Steep	Lindbergh	0.045	0.0375	N/A
$0.1Q_2$	B	Flat	Lindbergh	0.100	0.0833	N/A
$0.1Q_2$	B	Moderate	Lindbergh	0.095	0.0792	N/A
$0.1Q_2$	B	Steep	Lindbergh	0.080	0.0667	N/A
$0.1Q_2$	C	Flat	Lindbergh	0.145	0.1208	0.0870
$0.1Q_2$	C	Moderate	Lindbergh	0.145	0.1208	0.0870
$0.1Q_2$	C	Steep	Lindbergh	0.120	0.1000	0.0720
$0.1Q_2$	D	Flat	Lindbergh	0.160	0.1333	0.0960
$0.1Q_2$	D	Moderate	Lindbergh	0.160	0.1333	0.0960
$0.1Q_2$	D	Steep	Lindbergh	0.115	0.0958	0.0690
$0.1Q_2$	A	Flat	Oceanside	0.070	0.0583	N/A
$0.1Q_2$	A	Moderate	Oceanside	0.065	0.0542	N/A
$0.1Q_2$	A	Steep	Oceanside	0.060	0.0500	N/A
$0.1Q_2$	B	Flat	Oceanside	0.103	0.0854	N/A
$0.1Q_2$	B	Moderate	Oceanside	0.090	0.0750	N/A
$0.1Q_2$	B	Steep	Oceanside	0.075	0.0625	N/A
$0.1Q_2$	C	Flat	Oceanside	0.130	0.1083	0.0780
$0.1Q_2$	C	Moderate	Oceanside	0.130	0.1083	0.0780
$0.1Q_2$	C	Steep	Oceanside	0.110	0.0917	0.0660
$0.1Q_2$	D	Flat	Oceanside	0.130	0.1083	0.0780
$0.1Q_2$	D	Moderate	Oceanside	0.130	0.1083	0.0780
$0.1Q_2$	D	Steep	Oceanside	0.065	0.0542	0.0390
$0.1Q_2$	A	Flat	L Wohlford	0.050	0.0417	N/A
$0.1Q_2$	A	Moderate	L Wohlford	0.045	0.0375	N/A
$0.1Q_2$	A	Steep	L Wohlford	0.040	0.0333	N/A
$0.1Q_2$	B	Flat	L Wohlford	0.090	0.0750	N/A
$0.1Q_2$	B	Moderate	L Wohlford	0.085	0.0708	N/A
$0.1Q_2$	B	Steep	L Wohlford	0.065	0.0542	N/A
$0.1Q_2$	C	Flat	L Wohlford	0.110	0.0917	0.0660
$0.1Q_2$	C	Moderate	L Wohlford	0.110	0.0917	0.0660

Table 7-1. Sizing Factors for Bioretention Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.1Q_2$	C	Steep	L Wohlford	0.090	0.0750	0.0540
$0.1Q_2$	D	Flat	L Wohlford	0.100	0.0833	0.0600
$0.1Q_2$	D	Moderate	L Wohlford	0.100	0.0833	0.0600
$0.1Q_2$	D	Steep	L Wohlford	0.075	0.0625	0.0450

Q_2 = 2-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records

Q_{10} = 10-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records

A = Surface area sizing factor

V_1 = Surface volume sizing factor

V_2 = Subsurface volume sizing factor

Table 7-2. Sizing Factors for Bioretention Plus Cistern Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.5Q_2$	A	Flat	Lindbergh	0.020	0.1200	N/A
$0.5Q_2$	A	Moderate	Lindbergh	0.020	0.1000	N/A
$0.5Q_2$	A	Steep	Lindbergh	0.020	0.1000	N/A
$0.5Q_2$	B	Flat	Lindbergh	0.020	0.3900	N/A
$0.5Q_2$	B	Moderate	Lindbergh	0.020	0.2000	N/A
$0.5Q_2$	B	Steep	Lindbergh	0.020	0.1200	N/A
$0.5Q_2$	C	Flat	Lindbergh	0.020	0.1200	N/A
$0.5Q_2$	C	Moderate	Lindbergh	0.020	0.1200	N/A
$0.5Q_2$	C	Steep	Lindbergh	0.020	0.1000	N/A
$0.5Q_2$	D	Flat	Lindbergh	0.020	0.1000	N/A
$0.5Q_2$	D	Moderate	Lindbergh	0.020	0.1000	N/A
$0.5Q_2$	D	Steep	Lindbergh	0.030	0.0800	N/A
$0.5Q_2$	A	Flat	Oceanside	0.020	0.1600	N/A
$0.5Q_2$	A	Moderate	Oceanside	0.020	0.1400	N/A
$0.5Q_2$	A	Steep	Oceanside	0.030	0.1200	N/A
$0.5Q_2$	B	Flat	Oceanside	0.020	0.1900	N/A
$0.5Q_2$	B	Moderate	Oceanside	0.025	0.1600	N/A
$0.5Q_2$	B	Steep	Oceanside	0.035	0.1400	N/A
$0.5Q_2$	C	Flat	Oceanside	0.030	0.1400	N/A
$0.5Q_2$	C	Moderate	Oceanside	0.035	0.1400	N/A
$0.5Q_2$	C	Steep	Oceanside	0.040	0.1200	N/A
$0.5Q_2$	D	Flat	Oceanside	0.035	0.1200	N/A
$0.5Q_2$	D	Moderate	Oceanside	0.040	0.1200	N/A
$0.5Q_2$	D	Steep	Oceanside	0.040	0.1000	N/A
$0.5Q_2$	A	Flat	L Wohlford	0.025	0.1800	N/A
$0.5Q_2$	A	Moderate	L Wohlford	0.040	0.1400	N/A

Table 7-2. Sizing Factors for Bioretention Plus Cistern Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.5Q_2$	A	Steep	L Wohlford	0.040	0.0800	N/A
$0.5Q_2$	B	Flat	L Wohlford	0.040	0.2100	N/A
$0.5Q_2$	B	Moderate	L Wohlford	0.040	0.2000	N/A
$0.5Q_2$	B	Steep	L Wohlford	0.040	0.1400	N/A
$0.5Q_2$	C	Flat	L Wohlford	0.040	0.1400	N/A
$0.5Q_2$	C	Moderate	L Wohlford	0.040	0.1400	N/A
$0.5Q_2$	C	Steep	L Wohlford	0.040	0.1000	N/A
$0.5Q_2$	D	Flat	L Wohlford	0.040	0.1000	N/A
$0.5Q_2$	D	Moderate	L Wohlford	0.040	0.1000	N/A
$0.5Q_2$	D	Steep	L Wohlford	0.040	0.0800	N/A
$0.3Q_2$	A	Flat	Lindbergh	0.020	0.1200	N/A
$0.3Q_2$	A	Moderate	Lindbergh	0.020	0.1000	N/A
$0.3Q_2$	A	Steep	Lindbergh	0.020	0.1000	N/A
$0.3Q_2$	B	Flat	Lindbergh	0.020	0.5900	N/A
$0.3Q_2$	B	Moderate	Lindbergh	0.020	0.3600	N/A
$0.3Q_2$	B	Steep	Lindbergh	0.020	0.1800	N/A
$0.3Q_2$	C	Flat	Lindbergh	0.020	0.1800	N/A
$0.3Q_2$	C	Moderate	Lindbergh	0.020	0.1800	N/A
$0.3Q_2$	C	Steep	Lindbergh	0.020	0.1400	N/A
$0.3Q_2$	D	Flat	Lindbergh	0.020	0.1400	N/A
$0.3Q_2$	D	Moderate	Lindbergh	0.020	0.1400	N/A
$0.3Q_2$	D	Steep	Lindbergh	0.020	0.0800	N/A
$0.3Q_2$	A	Flat	Oceanside	0.020	0.1600	N/A
$0.3Q_2$	A	Moderate	Oceanside	0.020	0.1400	N/A
$0.3Q_2$	A	Steep	Oceanside	0.020	0.1200	N/A
$0.3Q_2$	B	Flat	Oceanside	0.020	0.2200	N/A
$0.3Q_2$	B	Moderate	Oceanside	0.020	0.1800	N/A
$0.3Q_2$	B	Steep	Oceanside	0.020	0.1600	N/A
$0.3Q_2$	C	Flat	Oceanside	0.020	0.1600	N/A
$0.3Q_2$	C	Moderate	Oceanside	0.020	0.1600	N/A
$0.3Q_2$	C	Steep	Oceanside	0.025	0.1400	N/A
$0.3Q_2$	D	Flat	Oceanside	0.020	0.1400	N/A
$0.3Q_2$	D	Moderate	Oceanside	0.025	0.1400	N/A
$0.3Q_2$	D	Steep	Oceanside	0.030	0.1200	N/A
$0.3Q_2$	A	Flat	L Wohlford	0.020	0.1800	N/A
$0.3Q_2$	A	Moderate	L Wohlford	0.025	0.1400	N/A
$0.3Q_2$	A	Steep	L Wohlford	0.030	0.0800	N/A

Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
Table 7-2. Sizing Factors for Bioretention Plus Cistern Facilities						
$0.3Q_2$	B	Flat	L Wohlford	0.025	0.2600	N/A
$0.3Q_2$	B	Moderate	L Wohlford	0.025	0.2400	N/A
$0.3Q_2$	B	Steep	L Wohlford	0.030	0.1800	N/A
$0.3Q_2$	C	Flat	L Wohlford	0.030	0.1800	N/A
$0.3Q_2$	C	Moderate	L Wohlford	0.030	0.1800	N/A
$0.3Q_2$	C	Steep	L Wohlford	0.035	0.1400	N/A
$0.3Q_2$	D	Flat	L Wohlford	0.030	0.1400	N/A
$0.3Q_2$	D	Moderate	L Wohlford	0.035	0.1400	N/A
$0.3Q_2$	D	Steep	L Wohlford	0.040	0.1000	N/A
$0.1Q_2$	A	Flat	Lindbergh	0.020	0.1200	N/A
$0.1Q_2$	A	Moderate	Lindbergh	0.020	0.1000	N/A
$0.1Q_2$	A	Steep	Lindbergh	0.020	0.1000	N/A
$0.1Q_2$	B	Flat	Lindbergh	0.020	0.5400	N/A
$0.1Q_2$	B	Moderate	Lindbergh	0.020	0.7800	N/A
$0.1Q_2$	B	Steep	Lindbergh	0.020	0.3400	N/A
$0.1Q_2$	C	Flat	Lindbergh	0.020	0.3600	N/A
$0.1Q_2$	C	Moderate	Lindbergh	0.020	0.3600	N/A
$0.1Q_2$	C	Steep	Lindbergh	0.020	0.2400	N/A
$0.1Q_2$	D	Flat	Lindbergh	0.020	0.2600	N/A
$0.1Q_2$	D	Moderate	Lindbergh	0.020	0.2600	N/A
$0.1Q_2$	D	Steep	Lindbergh	0.020	0.1600	N/A
$0.1Q_2$	A	Flat	Oceanside	0.020	0.1600	N/A
$0.1Q_2$	A	Moderate	Oceanside	0.020	0.1400	N/A
$0.1Q_2$	A	Steep	Oceanside	0.020	0.1200	N/A
$0.1Q_2$	B	Flat	Oceanside	0.020	0.5100	N/A
$0.1Q_2$	B	Moderate	Oceanside	0.020	0.3400	N/A
$0.1Q_2$	B	Steep	Oceanside	0.020	0.2400	N/A
$0.1Q_2$	C	Flat	Oceanside	0.020	0.2600	N/A
$0.1Q_2$	C	Moderate	Oceanside	0.020	0.2600	N/A
$0.1Q_2$	C	Steep	Oceanside	0.020	0.2000	N/A
$0.1Q_2$	D	Flat	Oceanside	0.020	0.2000	N/A
$0.1Q_2$	D	Moderate	Oceanside	0.020	0.2000	N/A
$0.1Q_2$	D	Steep	Oceanside	0.020	0.1800	N/A
$0.1Q_2$	A	Flat	L Wohlford	0.020	0.1800	N/A
$0.1Q_2$	A	Moderate	L Wohlford	0.020	0.1400	N/A
$0.1Q_2$	A	Steep	L Wohlford	0.020	0.0800	N/A
$0.1Q_2$	B	Flat	L Wohlford	0.020	0.4400	N/A

Table 7-2. Sizing Factors for Bioretention Plus Cistern Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.1Q_2$	B	Moderate	L Wohlford	0.020	0.4000	N/A
$0.1Q_2$	B	Steep	L Wohlford	0.020	0.3200	N/A
$0.1Q_2$	C	Flat	L Wohlford	0.020	0.3200	N/A
$0.1Q_2$	C	Moderate	L Wohlford	0.020	0.3200	N/A
$0.1Q_2$	C	Steep	L Wohlford	0.020	0.2200	N/A
$0.1Q_2$	D	Flat	L Wohlford	0.020	0.2400	N/A
$0.1Q_2$	D	Moderate	L Wohlford	0.020	0.2400	N/A
$0.1Q_2$	D	Steep	L Wohlford	0.020	0.1800	N/A

Q_2 = 2-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
Q_{10} = 10-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
A = Bioretention surface area sizing factor
V_1 = Cistern volume sizing factor

Table 7-3. Sizing Factors for Bioretention Plus Vault Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.5Q_2$	A	Flat	Lindbergh	N/A	N/A	N/A
$0.5Q_2$	A	Moderate	Lindbergh	N/A	N/A	N/A
$0.5Q_2$	A	Steep	Lindbergh	N/A	N/A	N/A
$0.5Q_2$	B	Flat	Lindbergh	0.040	0.3600	N/A
$0.5Q_2$	B	Moderate	Lindbergh	0.040	0.2400	N/A
$0.5Q_2$	B	Steep	Lindbergh	0.040	0.1400	N/A
$0.5Q_2$	C	Flat	Lindbergh	0.040	0.1600	N/A
$0.5Q_2$	C	Moderate	Lindbergh	0.040	0.1600	N/A
$0.5Q_2$	C	Steep	Lindbergh	0.040	0.1200	N/A
$0.5Q_2$	D	Flat	Lindbergh	0.040	0.1400	N/A
$0.5Q_2$	D	Moderate	Lindbergh	0.040	0.1400	N/A
$0.5Q_2$	D	Steep	Lindbergh	0.040	0.1000	N/A
$0.5Q_2$	A	Flat	Oceanside	N/A	N/A	N/A
$0.5Q_2$	A	Moderate	Oceanside	N/A	N/A	N/A
$0.5Q_2$	A	Steep	Oceanside	N/A	N/A	N/A
$0.5Q_2$	B	Flat	Oceanside	0.040	0.2100	N/A
$0.5Q_2$	B	Moderate	Oceanside	0.040	0.1800	N/A
$0.5Q_2$	B	Steep	Oceanside	0.040	0.1400	N/A
$0.5Q_2$	C	Flat	Oceanside	0.040	0.1400	N/A
$0.5Q_2$	C	Moderate	Oceanside	0.040	0.1400	N/A
$0.5Q_2$	C	Steep	Oceanside	0.040	0.1200	N/A
$0.5Q_2$	D	Flat	Oceanside	0.040	0.1400	N/A

Table 7-3. Sizing Factors for Bioretention Plus Vault Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.5Q_2$	D	Moderate	Oceanside	0.040	0.1400	N/A
$0.5Q_2$	D	Steep	Oceanside	0.040	0.1200	N/A
$0.5Q_2$	A	Flat	L Wohlford	N/A	N/A	N/A
$0.5Q_2$	A	Moderate	L Wohlford	N/A	N/A	N/A
$0.5Q_2$	A	Steep	L Wohlford	N/A	N/A	N/A
$0.5Q_2$	B	Flat	L Wohlford	0.040	0.2600	N/A
$0.5Q_2$	B	Moderate	L Wohlford	0.040	0.2200	N/A
$0.5Q_2$	B	Steep	L Wohlford	0.040	0.1200	N/A
$0.5Q_2$	C	Flat	L Wohlford	0.040	0.1400	N/A
$0.5Q_2$	C	Moderate	L Wohlford	0.040	0.1400	N/A
$0.5Q_2$	C	Steep	L Wohlford	0.040	0.1000	N/A
$0.5Q_2$	D	Flat	L Wohlford	0.040	0.1200	N/A
$0.5Q_2$	D	Moderate	L Wohlford	0.040	0.1200	N/A
$0.5Q_2$	D	Steep	L Wohlford	0.040	0.0800	N/A
$0.3Q_2$	A	Flat	Lindbergh	N/A	N/A	N/A
$0.3Q_2$	A	Moderate	Lindbergh	N/A	N/A	N/A
$0.3Q_2$	A	Steep	Lindbergh	N/A	N/A	N/A
$0.3Q_2$	B	Flat	Lindbergh	0.040	0.4500	N/A
$0.3Q_2$	B	Moderate	Lindbergh	0.040	0.3200	N/A
$0.3Q_2$	B	Steep	Lindbergh	0.040	0.1800	N/A
$0.3Q_2$	C	Flat	Lindbergh	0.040	0.1800	N/A
$0.3Q_2$	C	Moderate	Lindbergh	0.040	0.1800	N/A
$0.3Q_2$	C	Steep	Lindbergh	0.040	0.1400	N/A
$0.3Q_2$	D	Flat	Lindbergh	0.040	0.1600	N/A
$0.3Q_2$	D	Moderate	Lindbergh	0.040	0.1600	N/A
$0.3Q_2$	D	Steep	Lindbergh	0.040	0.1200	N/A
$0.3Q_2$	A	Flat	Oceanside	N/A	N/A	N/A
$0.3Q_2$	A	Moderate	Oceanside	N/A	N/A	N/A
$0.3Q_2$	A	Steep	Oceanside	N/A	N/A	N/A
$0.3Q_2$	B	Flat	Oceanside	0.040	0.2500	N/A
$0.3Q_2$	B	Moderate	Oceanside	0.040	0.2000	N/A
$0.3Q_2$	B	Steep	Oceanside	0.040	0.1600	N/A
$0.3Q_2$	C	Flat	Oceanside	0.040	0.1600	N/A
$0.3Q_2$	C	Moderate	Oceanside	0.040	0.1600	N/A
$0.3Q_2$	C	Steep	Oceanside	0.040	0.1400	N/A
$0.3Q_2$	D	Flat	Oceanside	0.040	0.1400	N/A
$0.3Q_2$	D	Moderate	Oceanside	0.040	0.1400	N/A

Table 7-3. Sizing Factors for Bioretention Plus Vault Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.3Q_2$	D	Steep	Oceanside	0.040	0.1200	N/A
$0.3Q_2$	A	Flat	L Wohlford	N/A	N/A	N/A
$0.3Q_2$	A	Moderate	L Wohlford	N/A	N/A	N/A
$0.3Q_2$	A	Steep	L Wohlford	N/A	N/A	N/A
$0.3Q_2$	B	Flat	L Wohlford	0.040	0.2900	N/A
$0.3Q_2$	B	Moderate	L Wohlford	0.040	0.2600	N/A
$0.3Q_2$	B	Steep	L Wohlford	0.040	0.1600	N/A
$0.3Q_2$	C	Flat	L Wohlford	0.040	0.1600	N/A
$0.3Q_2$	C	Moderate	L Wohlford	0.040	0.1600	N/A
$0.3Q_2$	C	Steep	L Wohlford	0.040	0.1200	N/A
$0.3Q_2$	D	Flat	L Wohlford	0.040	0.1200	N/A
$0.3Q_2$	D	Moderate	L Wohlford	0.040	0.1200	N/A
$0.3Q_2$	D	Steep	L Wohlford	0.040	0.0800	N/A
$0.1Q_2$	A	Flat	Lindbergh	N/A	N/A	N/A
$0.1Q_2$	A	Moderate	Lindbergh	N/A	N/A	N/A
$0.1Q_2$	A	Steep	Lindbergh	N/A	N/A	N/A
$0.1Q_2$	B	Flat	Lindbergh	0.040	0.5900	N/A
$0.1Q_2$	B	Moderate	Lindbergh	0.040	0.5000	N/A
$0.1Q_2$	B	Steep	Lindbergh	0.040	0.3200	N/A
$0.1Q_2$	C	Flat	Lindbergh	0.040	0.3400	N/A
$0.1Q_2$	C	Moderate	Lindbergh	0.040	0.3400	N/A
$0.1Q_2$	C	Steep	Lindbergh	0.040	0.2400	N/A
$0.1Q_2$	D	Flat	Lindbergh	0.040	0.2600	N/A
$0.1Q_2$	D	Moderate	Lindbergh	0.040	0.2600	N/A
$0.1Q_2$	D	Steep	Lindbergh	0.040	0.1800	N/A
$0.1Q_2$	A	Flat	Oceanside	N/A	N/A	N/A
$0.1Q_2$	A	Moderate	Oceanside	N/A	N/A	N/A
$0.1Q_2$	A	Steep	Oceanside	N/A	N/A	N/A
$0.1Q_2$	B	Flat	Oceanside	0.040	0.4300	N/A
$0.1Q_2$	B	Moderate	Oceanside	0.040	0.3400	N/A
$0.1Q_2$	B	Steep	Oceanside	0.040	0.2400	N/A
$0.1Q_2$	C	Flat	Oceanside	0.040	0.2600	N/A
$0.1Q_2$	C	Moderate	Oceanside	0.040	0.2600	N/A
$0.1Q_2$	C	Steep	Oceanside	0.040	0.2000	N/A
$0.1Q_2$	D	Flat	Oceanside	0.040	0.2200	N/A
$0.1Q_2$	D	Moderate	Oceanside	0.040	0.2200	N/A
$0.1Q_2$	D	Steep	Oceanside	0.040	0.1600	N/A

Table 7-3. Sizing Factors for Bioretention Plus Vault Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
$0.1Q_2$	A	Flat	L Wohlford	N/A	N/A	N/A
$0.1Q_2$	A	Moderate	L Wohlford	N/A	N/A	N/A
$0.1Q_2$	A	Steep	L Wohlford	N/A	N/A	N/A
$0.1Q_2$	B	Flat	L Wohlford	0.040	0.4300	N/A
$0.1Q_2$	B	Moderate	L Wohlford	0.040	0.3800	N/A
$0.1Q_2$	B	Steep	L Wohlford	0.040	0.2800	N/A
$0.1Q_2$	C	Flat	L Wohlford	0.040	0.2800	N/A
$0.1Q_2$	C	Moderate	L Wohlford	0.040	0.2800	N/A
$0.1Q_2$	C	Steep	L Wohlford	0.040	0.2000	N/A
$0.1Q_2$	D	Flat	L Wohlford	0.040	0.2200	N/A
$0.1Q_2$	D	Moderate	L Wohlford	0.040	0.2200	N/A
$0.1Q_2$	D	Steep	L Wohlford	0.040	0.1400	N/A

Q_2 = 2-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
Q_{10} = 10-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
A = Bioretention surface area sizing factor
V_1 = Cistern volume sizing factor

Table 7-4. Sizing Factors for Flow-Through Planters						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.5Q2	A	Flat	Lindbergh	N/A	N/A	N/A
0.5Q2	A	Moderate	Lindbergh	N/A	N/A	N/A
0.5Q2	A	Steep	Lindbergh	N/A	N/A	N/A
0.5Q2	B	Flat	Lindbergh	N/A	N/A	N/A
0.5Q2	B	Moderate	Lindbergh	N/A	N/A	N/A
0.5Q2	B	Steep	Lindbergh	N/A	N/A	N/A
0.5Q2	C	Flat	Lindbergh	0.115	0.0958	0.0690
0.5Q2	C	Moderate	Lindbergh	0.115	0.0958	0.0690
0.5Q2	C	Steep	Lindbergh	0.080	0.0667	0.0480
0.5Q2	D	Flat	Lindbergh	0.085	0.0708	0.0510
0.5Q2	D	Moderate	Lindbergh	0.085	0.0708	0.0510
0.5Q2	D	Steep	Lindbergh	0.065	0.0542	0.0390
0.5Q2	A	Flat	Oceanside	N/A	N/A	N/A
0.5Q2	A	Moderate	Oceanside	N/A	N/A	N/A
0.5Q2	A	Steep	Oceanside	N/A	N/A	N/A
0.5Q2	B	Flat	Oceanside	N/A	N/A	N/A
0.5Q2	B	Moderate	Oceanside	N/A	N/A	N/A
0.5Q2	B	Steep	Oceanside	N/A	N/A	N/A
0.5Q2	C	Flat	Oceanside	0.075	0.0625	0.0450

Table 7-4. Sizing Factors for Flow-Through Planters						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.5Q2	C	Moderate	Oceanside	0.075	0.0625	0.0450
0.5Q2	C	Steep	Oceanside	0.065	0.0542	0.0390
0.5Q2	D	Flat	Oceanside	0.070	0.0583	0.0420
0.5Q2	D	Moderate	Oceanside	0.070	0.0583	0.0420
0.5Q2	D	Steep	Oceanside	0.050	0.0417	0.0300
0.5Q2	A	Flat	L Wohlford	N/A	N/A	N/A
0.5Q2	A	Moderate	L Wohlford	N/A	N/A	N/A
0.5Q2	A	Steep	L Wohlford	N/A	N/A	N/A
0.5Q2	B	Flat	L Wohlford	N/A	N/A	N/A
0.5Q2	B	Moderate	L Wohlford	N/A	N/A	N/A
0.5Q2	B	Steep	L Wohlford	N/A	N/A	N/A
0.5Q2	C	Flat	L Wohlford	0.070	0.0583	0.0420
0.5Q2	C	Moderate	L Wohlford	0.070	0.0583	0.0420
0.5Q2	C	Steep	L Wohlford	0.050	0.0417	0.0300
0.5Q2	D	Flat	L Wohlford	0.055	0.0458	0.0330
0.5Q2	D	Moderate	L Wohlford	0.055	0.0458	0.0330
0.5Q2	D	Steep	L Wohlford	0.045	0.0375	0.0270
0.3Q2	A	Flat	Lindbergh	N/A	N/A	N/A
0.3Q2	A	Moderate	Lindbergh	N/A	N/A	N/A
0.3Q2	A	Steep	Lindbergh	N/A	N/A	N/A
0.3Q2	B	Flat	Lindbergh	N/A	N/A	N/A
0.3Q2	B	Moderate	Lindbergh	N/A	N/A	N/A
0.3Q2	B	Steep	Lindbergh	N/A	N/A	N/A
0.3Q2	C	Flat	Lindbergh	0.130	0.1083	0.0780
0.3Q2	C	Moderate	Lindbergh	0.130	0.1083	0.0780
0.3Q2	C	Steep	Lindbergh	0.100	0.0833	0.0600
0.3Q2	D	Flat	Lindbergh	0.105	0.0875	0.0630
0.3Q2	D	Moderate	Lindbergh	0.105	0.0875	0.0630
0.3Q2	D	Steep	Lindbergh	0.075	0.0625	0.0450
0.3Q2	A	Flat	Oceanside	N/A	N/A	N/A
0.3Q2	A	Moderate	Oceanside	N/A	N/A	N/A
0.3Q2	A	Steep	Oceanside	N/A	N/A	N/A
0.3Q2	B	Flat	Oceanside	N/A	N/A	N/A
0.3Q2	B	Moderate	Oceanside	N/A	N/A	N/A
0.3Q2	B	Steep	Oceanside	N/A	N/A	N/A
0.3Q2	C	Flat	Oceanside	0.105	0.0875	0.0630
0.3Q2	C	Moderate	Oceanside	0.105	0.0875	0.0630
0.3Q2	C	Steep	Oceanside	0.085	0.0708	0.0510
0.3Q2	D	Flat	Oceanside	0.090	0.0750	0.0540

Table 7-4. Sizing Factors for Flow-Through Planters						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.3Q2	D	Moderate	Oceanside	0.090	0.0750	0.0540
0.3Q2	D	Steep	Oceanside	0.070	0.0583	0.0420
0.3Q2	A	Flat	L Wohlford	N/A	N/A	N/A
0.3Q2	A	Moderate	L Wohlford	N/A	N/A	N/A
0.3Q2	A	Steep	L Wohlford	N/A	N/A	N/A
0.3Q2	B	Flat	L Wohlford	N/A	N/A	N/A
0.3Q2	B	Moderate	L Wohlford	N/A	N/A	N/A
0.3Q2	B	Steep	L Wohlford	N/A	N/A	N/A
0.3Q2	C	Flat	L Wohlford	0.085	0.0708	0.0510
0.3Q2	C	Moderate	L Wohlford	0.085	0.0708	0.0510
0.3Q2	C	Steep	L Wohlford	0.060	0.0500	0.0360
0.3Q2	D	Flat	L Wohlford	0.065	0.0542	0.0390
0.3Q2	D	Moderate	L Wohlford	0.065	0.0542	0.0390
0.3Q2	D	Steep	L Wohlford	0.050	0.0417	0.0300
0.1Q2	A	Flat	Lindbergh	N/A	N/A	N/A
0.1Q2	A	Moderate	Lindbergh	N/A	N/A	N/A
0.1Q2	A	Steep	Lindbergh	N/A	N/A	N/A
0.1Q2	B	Flat	Lindbergh	N/A	N/A	N/A
0.1Q2	B	Moderate	Lindbergh	N/A	N/A	N/A
0.1Q2	B	Steep	Lindbergh	N/A	N/A	N/A
0.1Q2	C	Flat	Lindbergh	0.250	0.2083	0.1500
0.1Q2	C	Moderate	Lindbergh	0.250	0.2083	0.1500
0.1Q2	C	Steep	Lindbergh	0.185	0.1542	0.1110
0.1Q2	D	Flat	Lindbergh	0.200	0.1667	0.1200
0.1Q2	D	Moderate	Lindbergh	0.200	0.1667	0.1200
0.1Q2	D	Steep	Lindbergh	0.130	0.1083	0.0780
0.1Q2	A	Flat	Oceanside	N/A	N/A	N/A
0.1Q2	A	Moderate	Oceanside	N/A	N/A	N/A
0.1Q2	A	Steep	Oceanside	N/A	N/A	N/A
0.1Q2	B	Flat	Oceanside	N/A	N/A	N/A
0.1Q2	B	Moderate	Oceanside	N/A	N/A	N/A
0.1Q2	B	Steep	Oceanside	N/A	N/A	N/A
0.1Q2	C	Flat	Oceanside	0.190	0.1583	0.1140
0.1Q2	C	Moderate	Oceanside	0.190	0.1583	0.1140
0.1Q2	C	Steep	Oceanside	0.140	0.1167	0.0840
0.1Q2	D	Flat	Oceanside	0.160	0.1333	0.0960
0.1Q2	D	Moderate	Oceanside	0.160	0.1333	0.0960
0.1Q2	D	Steep	Oceanside	0.105	0.0875	0.0630
0.1Q2	A	Flat	L Wohlford	N/A	N/A	N/A

Table 7-4. Sizing Factors for Flow-Through Planters						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.1Q2	A	Moderate	L Wohlford	N/A	N/A	N/A
0.1Q2	A	Steep	L Wohlford	N/A	N/A	N/A
0.1Q2	B	Flat	L Wohlford	N/A	N/A	N/A
0.1Q2	B	Moderate	L Wohlford	N/A	N/A	N/A
0.1Q2	B	Steep	L Wohlford	N/A	N/A	N/A
0.1Q2	C	Flat	L Wohlford	0.135	0.1125	0.0810
0.1Q2	C	Moderate	L Wohlford	0.135	0.1125	0.0810
0.1Q2	C	Steep	L Wohlford	0.105	0.0875	0.0630
0.1Q2	D	Flat	L Wohlford	0.110	0.0917	0.0660
0.1Q2	D	Moderate	L Wohlford	0.110	0.0917	0.0660
0.1Q2	D	Steep	L Wohlford	0.080	0.0667	0.0480

Q_2 = 2-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
Q_{10} = 10-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
A = Surface area sizing factor
V_1 = Surface volume sizing factor
V_2 = Subsurface volume sizing factor

Table 7-5. Sizing Factors for Infiltration Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.5Q2	A	Flat	Lindbergh	0.040	0.1040	N/A
0.5Q2	A	Moderate	Lindbergh	0.040	0.1040	N/A
0.5Q2	A	Steep	Lindbergh	0.035	0.0910	N/A
0.5Q2	B	Flat	Lindbergh	0.058	0.1495	N/A
0.5Q2	B	Moderate	Lindbergh	0.055	0.1430	N/A
0.5Q2	B	Steep	Lindbergh	0.050	0.1300	N/A
0.5Q2	C	Flat	Lindbergh	N/A	N/A	N/A
0.5Q2	C	Moderate	Lindbergh	N/A	N/A	N/A
0.5Q2	C	Steep	Lindbergh	N/A	N/A	N/A
0.5Q2	D	Flat	Lindbergh	N/A	N/A	N/A
0.5Q2	D	Moderate	Lindbergh	N/A	N/A	N/A
0.5Q2	D	Steep	Lindbergh	N/A	N/A	N/A
0.5Q2	A	Flat	Oceanside	0.045	0.1170	N/A
0.5Q2	A	Moderate	Oceanside	0.045	0.1170	N/A
0.5Q2	A	Steep	Oceanside	0.040	0.1040	N/A
0.5Q2	B	Flat	Oceanside	0.065	0.1690	N/A
0.5Q2	B	Moderate	Oceanside	0.065	0.1690	N/A
0.5Q2	B	Steep	Oceanside	0.060	0.1560	N/A
0.5Q2	C	Flat	Oceanside	N/A	N/A	N/A
0.5Q2	C	Moderate	Oceanside	N/A	N/A	N/A

Table 7-5. Sizing Factors for Infiltration Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.5Q2	C	Steep	Oceanside	N/A	N/A	N/A
0.5Q2	D	Flat	Oceanside	N/A	N/A	N/A
0.5Q2	D	Moderate	Oceanside	N/A	N/A	N/A
0.5Q2	D	Steep	Oceanside	N/A	N/A	N/A
0.5Q2	A	Flat	L Wohlford	0.050	0.1300	N/A
0.5Q2	A	Moderate	L Wohlford	0.050	0.1300	N/A
0.5Q2	A	Steep	L Wohlford	0.040	0.1040	N/A
0.5Q2	B	Flat	L Wohlford	0.078	0.2015	N/A
0.5Q2	B	Moderate	L Wohlford	0.075	0.1950	N/A
0.5Q2	B	Steep	L Wohlford	0.065	0.1690	N/A
0.5Q2	C	Flat	L Wohlford	N/A	N/A	N/A
0.5Q2	C	Moderate	L Wohlford	N/A	N/A	N/A
0.5Q2	C	Steep	L Wohlford	N/A	N/A	N/A
0.5Q2	D	Flat	L Wohlford	N/A	N/A	N/A
0.5Q2	D	Moderate	L Wohlford	N/A	N/A	N/A
0.5Q2	D	Steep	L Wohlford	N/A	N/A	N/A
0.3Q2	A	Flat	Lindbergh	0.040	0.1040	N/A
0.3Q2	A	Moderate	Lindbergh	0.040	0.1040	N/A
0.3Q2	A	Steep	Lindbergh	0.035	0.0910	N/A
0.3Q2	B	Flat	Lindbergh	0.058	0.1495	N/A
0.3Q2	B	Moderate	Lindbergh	0.055	0.1430	N/A
0.3Q2	B	Steep	Lindbergh	0.050	0.1300	N/A
0.3Q2	C	Flat	Lindbergh	N/A	N/A	N/A
0.3Q2	C	Moderate	Lindbergh	N/A	N/A	N/A
0.3Q2	C	Steep	Lindbergh	N/A	N/A	N/A
0.3Q2	D	Flat	Lindbergh	N/A	N/A	N/A
0.3Q2	D	Moderate	Lindbergh	N/A	N/A	N/A
0.3Q2	D	Steep	Lindbergh	N/A	N/A	N/A
0.3Q2	A	Flat	Oceanside	0.045	0.1170	N/A
0.3Q2	A	Moderate	Oceanside	0.045	0.1170	N/A
0.3Q2	A	Steep	Oceanside	0.040	0.1040	N/A
0.3Q2	B	Flat	Oceanside	0.065	0.1690	N/A
0.3Q2	B	Moderate	Oceanside	0.065	0.1690	N/A
0.3Q2	B	Steep	Oceanside	0.060	0.1560	N/A
0.3Q2	C	Flat	Oceanside	N/A	N/A	N/A
0.3Q2	C	Moderate	Oceanside	N/A	N/A	N/A
0.3Q2	C	Steep	Oceanside	N/A	N/A	N/A
0.3Q2	D	Flat	Oceanside	N/A	N/A	N/A
0.3Q2	D	Moderate	Oceanside	N/A	N/A	N/A

Table 7-5. Sizing Factors for Infiltration Facilities						
Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.3Q2	D	Steep	Oceanside	N/A	N/A	N/A
0.3Q2	A	Flat	L Wohlford	0.050	0.1300	N/A
0.3Q2	A	Moderate	L Wohlford	0.050	0.1300	N/A
0.3Q2	A	Steep	L Wohlford	0.040	0.1040	N/A
0.3Q2	B	Flat	L Wohlford	0.078	0.2015	N/A
0.3Q2	B	Moderate	L Wohlford	0.075	0.1950	N/A
0.3Q2	B	Steep	L Wohlford	0.065	0.1690	N/A
0.3Q2	C	Flat	L Wohlford	N/A	N/A	N/A
0.3Q2	C	Moderate	L Wohlford	N/A	N/A	N/A
0.3Q2	C	Steep	L Wohlford	N/A	N/A	N/A
0.3Q2	D	Flat	L Wohlford	N/A	N/A	N/A
0.3Q2	D	Moderate	L Wohlford	N/A	N/A	N/A
0.3Q2	D	Steep	L Wohlford	N/A	N/A	N/A
0.1Q2	A	Flat	Lindbergh	0.040	0.1040	N/A
0.1Q2	A	Moderate	Lindbergh	0.040	0.1040	N/A
0.1Q2	A	Steep	Lindbergh	0.035	0.0910	N/A
0.1Q2	B	Flat	Lindbergh	0.058	0.1495	N/A
0.1Q2	B	Moderate	Lindbergh	0.055	0.1430	N/A
0.1Q2	B	Steep	Lindbergh	0.050	0.1300	N/A
0.1Q2	C	Flat	Lindbergh	N/A	N/A	N/A
0.1Q2	C	Moderate	Lindbergh	N/A	N/A	N/A
0.1Q2	C	Steep	Lindbergh	N/A	N/A	N/A
0.1Q2	D	Flat	Lindbergh	N/A	N/A	N/A
0.1Q2	D	Moderate	Lindbergh	N/A	N/A	N/A
0.1Q2	D	Steep	Lindbergh	N/A	N/A	N/A
0.1Q2	A	Flat	Oceanside	0.045	0.1170	N/A
0.1Q2	A	Moderate	Oceanside	0.045	0.1170	N/A
0.1Q2	A	Steep	Oceanside	0.040	0.1040	N/A
0.1Q2	B	Flat	Oceanside	0.065	0.1690	N/A
0.1Q2	B	Moderate	Oceanside	0.065	0.1690	N/A
0.1Q2	B	Steep	Oceanside	0.060	0.1560	N/A
0.1Q2	C	Flat	Oceanside	N/A	N/A	N/A
0.1Q2	C	Moderate	Oceanside	N/A	N/A	N/A
0.1Q2	C	Steep	Oceanside	N/A	N/A	N/A
0.1Q2	D	Flat	Oceanside	N/A	N/A	N/A
0.1Q2	D	Moderate	Oceanside	N/A	N/A	N/A
0.1Q2	D	Steep	Oceanside	N/A	N/A	N/A
0.1Q2	A	Flat	L Wohlford	0.050	0.1300	N/A
0.1Q2	A	Moderate	L Wohlford	0.050	0.1300	N/A

Lower Flow Threshold	Soil Group	Slope	Rain Gauge	A	V_1	V_2
0.1Q2	A	Steep	L Wohlford	0.040	0.1040	N/A
0.1Q2	B	Flat	L Wohlford	0.078	0.2015	N/A
0.1Q2	B	Moderate	L Wohlford	0.075	0.1950	N/A
0.1Q2	B	Steep	L Wohlford	0.065	0.1690	N/A
0.1Q2	C	Flat	L Wohlford	N/A	N/A	N/A
0.1Q2	C	Moderate	L Wohlford	N/A	N/A	N/A
0.1Q2	C	Steep	L Wohlford	N/A	N/A	N/A
0.1Q2	D	Flat	L Wohlford	N/A	N/A	N/A
0.1Q2	D	Moderate	L Wohlford	N/A	N/A	N/A
0.1Q2	D	Steep	L Wohlford	N/A	N/A	N/A

Table 7-5. Sizing Factors for Infiltration Facilities

Q_2 = 2-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
Q_{10} = 10-year pre-project flow rate based upon partial duration analysis of long-term hourly rainfall records
A = Surface area sizing factor
V_1 = Infiltration volume sizing factor

Rainfall basin boundaries were determined based upon mean annual precipitation values as determined by the County of San Diego and specific precipitation totals at the three base rainfall stations (Lindbergh Field, Oceanside and Lake Wohlford). The final rainfall basin map is provided in the San Diego BMP Sizing Calculator.

Per the County's chief hydrologist Rand Allan, the 3 base rainfall stations have the following mean annual precipitation values for the time period of 1971-2001 (period of time depicted on the mean annual precipitation map created by the County of San Diego).

Lindbergh Field = 10.2 inches
Oceanside = 13.3 inches
Lake Wohlford = 20.0 inches

To determine the east-west boundary between Oceanside and Lake Wohlford, the average of the mean annual precipitation values between Oceanside and Lake Wohlford was determined:

(13.3 inches + 20.0 inches) / 2 = 16.7 inches

The 17 inch isopluvial line was used as the boundary – anything east of the 17 inch isopluvial line would be part of the Lake Wohlford basin.

To determine the east-west boundary between Oceanside and Lindbergh, the average of the mean annual precipitation values between Oceanside and Lindbergh was determined:

(13.3 inches + 10.2 inches) = 11.8 inches

The 12 inch isopluvial line was used as the boundary – anything west of the 12 inch isopluvial line would be part of the Lindbergh basin.

To determine the east-west boundary between Lindbergh and Lake Wohlford (used only for extreme south county areas), the average of the mean annual precipitation values between Lindbergh and Lake Wohlford was determined:

(10.2 inches + 20.0 inches) / 2 = 15.1 inches

The 15 inch isopluvial line was used as the boundary – anything east of the 15-inch isopluvial line would be part of the Lake Wohlford basin.

Areas located between the 12 inch and 17 inch isopluvial lines and also located north of Sweetwater Reservoir/San Miguel Mountain were designated as part of the Oceanside basin.

Additional notes:

1. The southern extent of the Oceanside basin was limited to the area near Sweetwater Reservoir and San Miguel Mountain. At that location, the previously discussed 15 inch and 17 inch isopluvial lines are in close proximity.
2. There is a short reach of 12 inch isopluvial line near the coastline between Encinitas and Los Penasquitos lagoon. Even though this area has a mean annual precipitation less than 12 inches, it is included in the Oceanside basin pursuant to Rand Allan's determination that a north-south divide between the Lindbergh and Oceanside basins occurs north of La Jolla.

For situations where LID implementation cannot fully achieve the required hydromodification flow mitigation, the project applicant will have the option to implement extended detention facilities in combination with LID facilities. The San Diego HMP / LID sizing calculator will have a basin sizing component to assist with the design of extended detention flow duration control facilities as well as LID facilities.

Facilities must be designed, built, and maintained to practically function within the urban environment. Soil compaction associated with grading activities affects infiltration rates and should be considered. Underdrains are typically required for urban projects where the anticipated infiltration rate is low or where infiltrated runoff could pose an adjacent stability risk.

Since the HMP will be implemented through the municipal development review process, design criteria must be specified and be incorporated into conditions of approval.

Development of sizing factors and the San Diego HMP / LID sizing calculator is currently being conducted and includes the following tasks.

- Develop and document the major assumptions and model parameters that will be used in subsequent HSPF simulations to size Low-Impact Design (LID) facilities, detention ponds, and non-structural stormwater controls. This task includes three steps.
    1. Document LID facility configurations included in the Sizing Calculator
    2. Select the range of input parameters to use in the HSPF model simulations
    3. Develop and document the approach to computing BMP sizing factors
- Develop configurations of each of the LID BMPs included in the Model SUSMP, including the dimensions of the ponding layers, growing medium, storage layer, and outlet piping. The following LID BMPs will be modeled for flow control and water quality treatment:
    1. Bioretention
    2. Cistern with bioretention
    3. Bioretention with flow control vault
    4. Flow-through planter

5. Dry well
6. Vegetated bioswale (for water quality treatment only)

- Select a recommended set of HSPF input parameters for simulating hydrologic processes on pervious surfaces, known in HSPF as PERLNDs. Parameters will be selected that represent specific combinations of the following:
  - Soils
  - Land Cover
  - Slope
- Develop LID sizing factors for NRCS Group A, B, C, and D soils. LID facilities built in Group C and D soils will include an underdrain and a flow control orifice. LID facilities built in Group A and B soils will have no underdrain, requiring infiltration to surrounding soils.
- Develop sizing for traditional stormwater BMPs using an automated approach to size stormwater detention ponds. The automated pond sizing algorithm is incorporated into the BMP Sizing Calculator.
  - Select specific allowable range of pond configuration parameters, such as side slopes and the number of outlets
  - Develop an algorithm that will read in long-term model simulation results and iteratively vary pond volume and outlet dimensions until the flow control requirements are met
- Develop BMP Sizing Calculator. The BMP Sizing Calculator will help streamline the process of sizing the BMPs listed in the County's Model SUSMP. The software will have the following features:
  - Sizing of BMPs for "flow control + water quality treatment" and "water quality treatment-only" permit requirements
  - Include all LID BMPs listed in the County's SUSMP, including stormwater detention ponds
  - Include sizing criteria for self-retaining areas and self-treating areas, as described in the Model SUSMP

7.2 Inspection and Maintenance Schedule

If not properly designed or maintained, hydromodification flow control devices may create a habitat for vectors such as mosquitoes or rodents as well as potential safety hazards due to standing water. Vector habitat creation can be avoided through collaboration with municipalities and both local vector control agencies and the State Department of Health Services during the development and implementation of Project Submittals (Storm Water Management Plans or Water Quality Technical Reports).

Proof of long-term ongoing maintenance responsibility and mechanism are required for all post-construction BMPs, including hydromodification mitigation facilities. Maintenance activities for flow control and LID devices will be specified in the proposed Project Submittal (Storm Water Management Plan or Water Quality Technical Report).

A blockage in the storm drain system can cause water to back up into the treatment facilities and cause damage. For this reason, inspection and maintenance of the storm drain system is considered part of the inspection and maintenance of the treatment facilities. Normal functioning of the facilities may involve retention of water for up to 72 hours following significant storm events.

As required by Permit Provision D.1.c.(5), local municipalities require submittal of proof of a mechanism under which ongoing long-term maintenance of stormwater treatment and flow-control facilities will be conducted. Municipalities may require one of more of the following items be included in the Project Submittal:

1. A means to finance and implement facility maintenance in perpetuity.
2. Acceptance of responsibility for maintenance from the time the facilities are constructed until responsibility for operation and maintenance is legally transferred. A warranty covering a period following construction may also be required.
3. An outline of general maintenance requirements for the selected treatment and flow-control facilities.

Local municipalities may also require preparation and submittal of a detailed plan that sets forth a maintenance schedule for each of the treatment and flow-control facilities built on the project site and names the responsible parties for this action.

Before completing the Project Submittal, the applicant should ensure the stormwater control design is fully coordinated with the site plan, grading plan, and landscaping plan being proposed for the site.

Information submitted and presentations to design review committees, planning commissions, and other decision-making bodies must incorporate relevant aspects of the stormwater design. In particular, ensure:

- Curb elevations, elevations, grade breaks, and other features of the drainage design are consistent with the delineation of Drainage Management Areas (DMAs).
- The top edge (overflow) of each bioretention facility is level all around its perimeter—this is particularly important in parking lot medians.
- The resulting grading and drainage design is consistent with the design for parking and circulation.
- Bioretention facilities and other IMPs do not create conflicts with pedestrian access between parking and building entrances.
- Vaults and utility boxes can be accommodated outside bioretention facilities and will not be placed within bioretention facilities.
- The visual impact of stormwater facilities, including planter boxes at building foundations and any terracing or retaining walls required for the stormwater control design, is shown in renderings and other architectural drawings.
- Landscaping plans, including planting plans, show locations of bioretention facilities, and the plant requirements are consistent with the engineered soils and conditions in the bioretention facilities.
- Renderings and representation of street views incorporate any stormwater facilities located in street-side buffers and setbacks

Other design considerations to assist with long-term maintenance include:

- For effective, low-maintenance operation, locate facilities so drainage into and out of the device is by gravity flow. Pumped systems are feasible, but are expensive, require more maintenance, are prone to untimely failure, and can cause mosquito control problems. Most IMPs require 3 feet or more of head.
- If the property is being subdivided now or in the future, the facility should be in a common, accessible area. In particular, avoid locating facilities on private residential lots. Even if the facility will serve only one site owner or operator, make sure the facility is located for ready access by inspectors from the local municipality and local mosquito control agency.
- The facility must be accessible to equipment needed for its maintenance. Access requirements for maintenance will vary with the type of facility selected. Planter boxes and bioretention areas will typically need access for the same types of equipment used for landscape maintenance.

8. MONITORING AND BMP EVALUATION

8.1 Introduction

This section presents a summary of the San Diego HMP's revised Monitoring Plan. The summary explains technical concepts and proposes approaches to monitor the effectiveness of the HMP as required by provision D.1.g of Regional Board Order No. R9-2007-0001.

Part 1(k) of provision D.1.g requires that the HMP shall "include a description of pre- and post-project monitoring and other program evaluations to be conducted to assess the effectiveness of implementation of the HMP." For the purposes of developing an HMP monitoring approach, an effective HMP is defined as a program that ensures compliance with HMP design criteria and results in no significant stream degradation due to increased erosive force caused by new development.

The proposed monitoring approach provides for the optimum 5-year effectiveness assessment within currently available funding resources. Monitoring Plan activities were selected to achieve statistical data collection requirements while balancing regional financial constraints and highly variable scientific, regulatory, and physical elements. Monitoring plan activities presented herein have been developed to answer the following questions regarding HMP program effectiveness assessment:

- **Do field observations confirm that the HMP appropriately defines the flow rate (expressed as a function of the 2-year runoff event) that initiates movement of channel bed or bank materials?** Since most of the sediment transport modeling prepared as part of the HMP development relied on laboratory flume data, it is important to supplement the sediment transport data set with field observations. This data may be used in the next permit cycle to determine whether critical shear stress is the appropriate parameter for selecting the lower flow threshold of the geomorphically significant flow range.
- **Are mitigation facilities adequately meeting flow duration design criteria outlined in the HMP?** Observed HMP mitigation facility outflow data can be analyzed to determine if mitigation facilities are reducing the mitigated post-project peak flow frequency and flow duration curves to the pre-project curves (within tolerances set forth in the HMP). This data can also be used to analyze the precision of LID sizing factors, extended detention facility design criteria, and to potentially recommend changes to more closely match the mitigated post-project curves to pre-project condition peak flow frequency and flow duration curves.
- **What is the effect of development on downstream cross section incision and widening?** Since the mitigation of accelerated channel degradation as a result of development is the central purpose of the HMP, analysis of channel cross sections downstream of development projects is a component of the monitoring plan. However, uncertainties involved with this comparison tool (namely the determination of pre-project condition trends regarding channel incision and channel widening rates) make policy determinations less likely within the time frame of the 5-year monitoring plan (as compared to sediment transport modeling and flow duration modeling detailed in the previous two questions).

Such a question-driven plan is consistent with the draft hydromodification monitoring framework prepared by the Southern California Coastal Water Research Project (SCCWRP – report dated December 9, 2009).

In an effort to effectively address the wide variability of potential monitoring scenarios and competing needs outlined above, the Copermittees and Brown and Caldwell have consulted with technical experts in a variety of critical disciplines including Dr. Eric Stein of SCCWRP (geomorphology expert), San Diego Regional Water Quality Control Board staff, Dr. Andy Collison of Phillip Williams Associates (geomorphology expert), Dr. Khalil Abusaba (formerly of the San Francisco Regional Water Quality Control Board and currently with Brown and Caldwell – expert in statistical analysis of water quality data), and members of the San Diego HMP Technical Advisory Committee.

8.2 Technical Concepts

8.2.1 Hydromodification Concepts

As required in the Permit, the evaluation of increased erosive force is limited to the geomorphically significant flow range, which is defined between the flow associated with critical shear stress and the ten-year return flow (Q10). The value of the lower flow threshold indicates the flow at which sediment erosion from the stream bed or banks begins to occur. The HMP uses two calculation tools (the low flow calculator and the SCCWRP channel assessment tool) to determine the low flow threshold based upon substrate type, channel slope, roughness, channel cross section, and other stream assessment conditions. The resulting lower flow threshold will be expressed as a multiple of the two-year return flow (Q2):

- $0.1Q_2$ for streams with HIGH susceptibility to channel erosion
- $0.3Q_2$ for streams with MEDIUM susceptibility to channel erosion
- $0.5Q_2$ for streams with LOW susceptibility to channel erosion

8.2.2 HMP Effectiveness Validation Measures

Sediment Transport Studies. This approach monitors sediment concentration (SSC) throughout a storm event and can be used to directly evaluate the validity of a lower flow threshold for a particular stream segment. Measuring the continuous SSC to flow relationship over a range of flows allows HMP effectiveness to be evaluated based on whether or not significant post-project increases in SSC (as compared to pre-project conditions) are observed at a given flow rate. This approach is the most costly, because it involves measuring flow and SSC. The SSC measurements will involve continuous turbidity monitoring, which would include calibration of turbidity meters using stream cross-sectional sediment sampling to correlate SSC to turbidity, or an approved equivalent metric. SSC values can also be determined through a laboratory analysis using United States Geological Survey (USGS) procedures. The final analysis method, along with data collection specifications, will be determined following future discussions with the Copermittees and members of the Technical Advisory Committee. These approaches are most likely to produce information on HMP effectiveness on a relatively short time frame, provided that a sufficient range of storm event sizes can be sampled in a given year.

Flow Duration Curves. Another measure of HMP effectiveness is determining if, within the geomorphically significant flow range, the post-project flow-duration curve is comparable to or below the pre-project flow duration curve. Flow-duration curves are monitored by installing continuous flow monitoring devices downstream of a planned project prior to development to establish pre-project conditions. If the flow monitoring facilities used for the sediment transport studies (detailed above) are located just downstream of a proposed development, then data from the sediment transport studies can be used for the pre-project flow duration data. This approach is consistent with the draft SCCWRP monitoring framework, which recommends stream flow monitoring to be provided just downstream of a hydromodification mitigation management device. Post-development mitigated flow duration monitoring data is analyzed to evaluate whether significant changes in the flow-duration curve have occurred. This monitoring approach can also be used to validate sizing factors for LID and extended detention BMPs. Depending on the range of rainfall events encountered in a particular year, monitoring of flow-duration curves can help develop pre-project conditions and evaluate post-project effectiveness on a relatively short time scale (i.e., 2 to 3 years each).

Channel Incision and Widening. The most obvious measure of stream degradation is to physically measure the pre-project and post-project cross sections, and determine if the channel is incising and / or widening. This is accomplished by conducting geomorphic assessments and channel surveys downstream of a planned development before and after construction. In addition to physical measurements, comparison of current and historical photos, aerial photography, and site inspection for signs of channel degradation can provide important supporting evidence. The labor for conducting such an assessment at a single location is lower compared to the effort needed to conduct sediment transport studies. Costs are driven by the number of sites assessed, as well as the need for establishing pre-project trends (e.g., rate of pre-project channel incision per year). Although this monitoring approach is the most direct measure of whether stream degradation is occurring, it is difficult to use the method to differentiate between existing geomorphic effects and post-project geomorphic effects. To do so would require a long-term baseline of pre-project channel incision and widening rates along with post-project monitoring. To capture the range of annual rainfall conditions encountered in Southern California, decades of information are generally recommended to quantify pre-project baseline trends. Therefore, while baseline data will be collected and be useful for future comparison analyses, this monitoring plan focuses on validation measures likely to provide meaningful data within 2 to 5 years. It is possible that tentative conclusions may be reached regarding channel incision and widening at the conclusion of the 5-year monitoring plan. Finally, it should be noted that the Copermittees will centralize stream assessment information collected as part of project development processes. This information may be used for future channel condition assessments and will be utilized by the Copermittees to the extent practicable. While such stream assessment information will not be required for all Priority Development Projects, it would be required for all projects proposing the use of stream rehabilitation mitigation measures (e.g., constructed channel widening, drop structures) and for projects using lower flow thresholds in excess of 0.1Q2. The Copermittees are currently considering other requirements for pre-project stream assessments, including project size, contributing impervious area cover, and receiving channel material.

8.2.3 Temporal and Spatial Variability of Monitoring Locations

Temporal Variability. As noted above, the single most important factor affecting the temporal variability inherent to measuring stream degradation is variable inter-annual rainfall frequency and intensity. Droughts in California can last years, with little to no rainfall occurring in Southern California. During El Nino years, anomalously high storm frequencies and intensities can result in sudden geomorphic changes. Rainfall intensity also varies intra-annually. However, if a sufficient range of storm intensities is encountered in a particular year, then short duration monitoring approaches, such as flow-duration curves and sediment transport studies can provide some information on HMP effectiveness on shorter timescales.

Spatial Variability. Sampling an adequate variety of channel susceptibility types, along with a reasonable number of replicates within for each susceptibility type, is important to capture the range of watershed conditions present in the permit coverage area. Other important factors that affect stream responses to hydromodification include channel grade, watershed area, vegetated cover, and stream sinuosity. In addition to channel and watershed features, location within the watershed is an important consideration. Monitoring stations should be located in the watershed headwaters just downstream of a development project of sufficient size, so that hydromodification effects from the proposed development can be isolated for comparison purposes to the maximum extent practicable. Upper watershed sites provide more definitive measures of HMP effectiveness because they can more directly correlate effects to specific development projects. Middle watershed and lower watershed sites would be influenced by confounding variables such as mass wasting and impacts from natural tributary confluences and other existing development projects, including phased developments over many years, in the watershed. Therefore, middle and lower watershed monitoring sites would require much more time to assess overall program effectiveness. However, the Copermittees will attempt to utilize data from concurrent water quality monitoring programs to develop a database of middle/lower watershed flow data. Specifically, monitoring station located in middle to lower watershed locations will be identified for the two proposed channel susceptibility types. While the San Diego HMP has been written to require onsite hydromodification flow controls at each applicable new development and redevelopment site, thus minimizing the potential for cumulative watershed impacts as a result of new development and redevelopment, monitoring station locations will be selected, where possible, to include the effects of multiple upstream developments. The concept of providing hydromodification effectiveness measurements in the watershed headwaters is supported by SCCWRP. Research by SCCWRP has shown that hydromodification effects of a development project become muted with increasing distance from the development site (defined by SCCWRP as the Domain of Effect). To the extent practicable, monitoring locations detailed in the Monitoring Plan will be distributed throughout the Permit coverage area Hydrologic Units to provide for geographic and climatic variability across San Diego County.

8.3 Recommended Approaches to Assess Effectiveness

Selection of HMP effectiveness assessment monitoring techniques is subject to two primary constraints. The schedule constraint involves the RWQCB's desire to have information on HMP effectiveness prior to re-issuance of the Municipal Separate Storm Sewer System (MS4) Permit for San Diego County, currently scheduled for 2012. This schedule constraint creates an added "practicality" issue, since it is unlikely that meaningful data can be acquired in such an abbreviated timeline. While the monitoring plan detailed in this memorandum extends for five years, interim data may be provided to the Regional Board to assist with development of the next Permit.

The budget constraint involves the San Diego County Copermittees' limited resources for monitoring. Given the fact that the Copermittees are currently committed to a $2,500,000 annual regional water quality monitoring plan effort, and given the current economic climate in which multiple local municipalities have been forced to reduce both budget and staff, expansion of existing monitoring mandates requires significant financial consideration and analysis. Thus, the Copermittees are compelled to evaluate how to develop the best possible monitoring approach to evaluate HMP effectiveness within the available budget.

Details of the monitoring plan are above and beyond details of the existing regional water quality monitoring effort. Wherever possible, the Copermittees will seek opportunities to utilize relevant data from the existing water quality monitoring efforts to achieve an economy of scale. The Copermittees will also ensure there is no duplication of effort between the two monitoring programs.

This monitoring plan focuses on using continuous monitoring data to obtain the maximum amount of data regarding sediment transport and flow duration monitoring. It is the opinion of the Copermittees that acquisition of continuous data at a statistically justified number of monitoring locations is more valuable (from a data analysis standpoint) as compared to obtaining a finite number of isolated runoff events from more monitoring locations.

Considering the constraints and technical approach detailed above, the following approaches are recommended for the revised HMP Monitoring Plan.

- **Monitor effectiveness using Sediment Transport and Flow Duration Studies.** As noted above, continuous sediment transport and flow duration studies can provide direct measures of HMP effectiveness on a relatively short timescale. These studies are important to verify HMP assumptions about the lower flow thresholds and to verify flow duration design criteria is being achieved. Development of the sediment transport studies would also provide stream cross section data, as well as photographic evidence, that could serve as a baseline for future stream morphology comparisons.

- **Monitor the Upper Watershed.** Upper watershed monitoring is recommended to eliminate confounding lower watershed variables that would skew the analysis and minimize the potential for reaching meaningful conclusions.
- **Monitor Replicates of Two Channel Susceptibility Types.** In the development of the San Diego County HMP, receiving streams will be classified into one of three channel types, pursuant to a State Board-funded study conducted by SCCWRP. The stream classification system is consistent with the analysis, findings, and tools developed in the SCCWRP study and classifies streams into the following stream susceptibility categories:
    - HIGH susceptibility
    - MEDIUM susceptibility
    - LOW susceptibility

  Monitoring locations should be selected from HIGH and MEDIUM susceptibility channel segments.
- **Monitor three replicates and one reference station for each susceptibility type.** Providing three replicates of each channel susceptibility type would begin the characterization of the range of conditions present in San Diego County. The reference monitoring station associated with each channel susceptibility type would be located in a watershed for which no upstream development (existing or future) is anticipated. Data from the reference stations can be used to supplement pre-project condition data obtained at the replicate sites, since the amount of pre-project condition data that can be obtained at such sites is dependent on the land development process. Providing three replicate stations balances the need to characterize spatial variability against the cost of monitoring and provides the data needed to estimate the median and range of the lower flow threshold for a given susceptibility type, or to estimate the standard deviation of an average value.
- **Monitor the Middle Watershed.** Middle watershed monitoring will be provided at two monitoring locations, both of which will be located downstream of existing urbanized areas with watershed impervious areas greater than 40 percent.

8.4 Summary and Conclusions

The revised Monitoring Plan, scheduled for implementation over a 5-year period, will recommend the following specific activities:

Baseline Monitoring Plan Requirements:

- Development of QAPP
- Rainfall gauge analysis and installation
- Rainfall gauge, stream gauge, and HMP facility outflow station inspection and maintenance (Fiscal Year 2012 through 2016)
- Annual data analysis (2012 – 2016)
- Reevaluation of the Monitoring Plan after review of findings from Statewide HMP Monitoring Technical Advisory Group and review of final SCCWRP Hydromodification Monitoring Report (2013)
- Report preparation (final report to be prepared in 2016)

Channel Assessments:

- Initial geomorphic assessment at each monitoring location (to determine stream susceptibility type – 2011-2012)
- Baseline cross section surveys at each monitoring location (2011-2012)

- Annual geomorphic assessments at each monitoring location (to assess channel condition and response - 2012 – 2016)
- Cross section surveys (after 5 years) at each monitoring location (2016)

Sediment Transport Analysis:

- Flow and sediment monitoring station installation
- Continuous pre-project, post-project and reference station flow, sediment and rainfall data collection (2012 – 2016)

Flow Duration Analysis:

- HMP facility outflow monitoring station installation
- Continuous post-project HMP facility outflow data collection (2013 – 2016)

9. CONCLUSIONS

Implementation of this HMP satisfies Provision D.1.g of Board Order R9-2007-0001. Adherence to guidelines outlined in the HMP is required "to manage increases in runoff discharge rates and durations for all Priority Development Projects, where such increased rates and durations are likely to cause increased erosion of channel beds or banks, sediment pollutant generation, or other impacts to beneficial uses and stream habitat due to increased erosive force."

Order R9-2007-0001 contains requirements that strongly influence the methodology contained in this HMP. As recommended in the HMP, post-project flows must match pre-project flows within the prescribed geomorphically significant flow range.

Flow control options to meet the criteria include LID facilities, which promote infiltration and filtration to attain the required flow mitigation, and extended flow duration control detention basins. Continuous hydrologic modeling is required to prove conformance with the standards presented in this HMP.

Specific permit requirements, detailed below, have been addressed by this HMP.

- Provide performance criteria for Priority Development Projects (Chapter 6)
- Include a protocol to evaluate potential hydrograph change impacts to downstream watercourses from PDPs (Chapter 6).
- Provide a description of how the Copermittees will incorporate the HMP requirements into their local approval processes (Chapter 1).
- Include a description of pre- and post-project monitoring and other program evaluations to be conducted to assess the effectiveness of implementation of the HMP (Chapter 8).
- Include mechanisms for addressing cumulative impacts within a watershed on channel morphology (Chapter 5).
- Utilize a continuous rainfall record to identify the geomorphically significant flow range (Chapter 5).
- Include a pertinent literature review (Chapter 4)
- Include criteria on management practices designed to mitigate increases to peak flows and durations (Chapter 7)
- Include information on the evaluation of channel form and condition (Chapter 5)

The Copermittees will incorporate HMP requirements into the local approval processes via incorporation of HMP criteria into their local SUSMPs. The San Diego region's updated Model SUSMP will incorporate the Final HMP criteria. HMP criteria will be incorporated into the local SUSMP and municipal ordinances no later than 180 days following RWQCB adoption of the HMP.

Information presented in the HMP has been prepared in association with the County of San Diego, San Diego Storm Water Copermittees, the Technical Advisory Committee, and the consultant team.

10. LIMITATIONS

Report Limitations

This document was prepared solely for the County of San Diego in accordance with professional standards at the time the services were performed and in accordance with the contract between the County of San Diego and Brown and Caldwell dated September 6, 2007. This document is governed by the specific scope of work authorized by the County of San Diego; it is not intended to be relied upon by any other party except for regulatory authorities contemplated by the scope of work. We have relied on rainfall data provided by the County of San Diego and other parties and have made no independent investigation as to the validity or accuracy of such data.

REFERENCES

Alameda Countywide Clean Water Program (ACCWP). 2005. *Hydrograph Modification Management Plan*. May 2005.

Amos, C.B., Burbank, D.W. 2007. Channel width response to differential uplift. *Journal of Geophysical Research*. Vol. 112.

Beighley, R.E., J.M. Melack, and T. Dunne. 2003. *Impacts of California's Climatic Regimes and Coastal Land Use Change on Streamflow Characteristics;* Journal of the American Water Resources Association.

Bendaa, L., Miller, D., Bigelowa, P., Andrasa, K.. 2003. Effects of post-wildfire erosion on channel environments, Boise River. *Forest Ecology and Management* 178 (2003)105–119

Beschta R.L and W.J. Ripple. 2008. Wolves, trophic cascades, and rivers in the Olympic National Park, USA. *Ecohydrol*. 1, 118–130 (2008)

Bicknell, B.R., Imhoff, J.C., et al. 1997. *Hydrological Simulation Program--Fortran, User's Manual for Version 11:* U.S. Environmental Protection Agency, National Exposure Research Laboratory, Athens, Ga., EPA/600/R-97/080, 755 p.

Bledsoe, B.P. 2001. Relationships of Stream Responses to Hydrologic Changes, Linking Stormwater BMP Designs and Performance to Receiving Water Impact Mitigation. Proceedings of an Engineering Foundation Conference, August 19-24, 2001, Snowmass Village, CO.

______. 2002. Stream Erosion Potential and Stormwater Management Strategies. *Journal of Water Resources Planning and Management 128*: 451.

______. 2007. Assessment of Stream response to Urbanization. *Cities of the Future: Towards Integrated Sustainable Water and Landscape Management.*

Bledsoe, B.P., Hawley, R., and Stein E. 2008. Channel Classification and Mapping Systems with Implications for Assessing Susceptibility to Hydromodifcation in Southern California. Southern California Coastal Water Research Project.

Bledsoe, Brian and Hawley, Robert. 2009. *Hydromodification Screening Tool for Southern California Draft for Internal Field Testing/Review* July.

Booth, D.B. 1990. Stream Channel Incision Following Drainage Basin Urbanization. *Journal of the American Water Resources Association* 26(3): 407-417.

______. 1991. Urbanization and the Natural Drainage System—Impacts, Solutions, and Prognoses. *The Institute for Environmental Studies* 7(1).

Booth, D.B. and C.R. Jackson 1997. Urbanization of Aquatic Systems: Degradation Thresholds, Stormwater Detection, and the Limits of Mitigation. *Journal of the American Water Resources Association* 33(5): 1077-1090.

Brown and Caldwell. 2007. *Development of Interim Hydromodification Criteria.* October 30.

______. 2008. *HMP Submittal Requirements.* September 25.

______. 2008. *Using Continuous Simulation to Size Storm Water Control Facilities.* May.

______. 2009. *Flow Threshold Analysis for the San Diego HMP.* April 30.

______. 2009. *Lower Flow Threshold Alternatives.* April 30.

______. 2009. *Minimum Criteria for Evaluation of Storm Water Controls to Meet Interim Hydromodification Criteria.* April 30.

______. 2009. *Rainfall Station Selection Criteria.* April 30.

______. 2009. *Summary of Evaporation and Evapotranspiration Data for the San Diego Region.* April 30.

______. 2009. *Summary of HSPF Modeling Efforts in Southern California.* April 30.

Butcher, J. 2007. Comparison of BAHM and Contra Costa Approaches to Hydromodification Management Plan Requirements. Tetra Tech, Inc. Memorandum to Janet O'Hara (CA RWQCB Region 2). December 7, 2007.

California Code of Regulations, Title 14, Division 6, Chapter 3, Sections 15000-15387 (California Environmental Quality Act - CEQA)

California Irrigation Management Information Systems web site (wwwcimis.water.ca.gov/cimis/data.jsp)

California Regional Water Quality Control Board - San Diego Region. 2007. *Order No. R9-2007-0001.* January 24.

California Stormwater Quality Association (CASQA). 2003. Stormwater Best Management Practice Handbook. New Development and Redevelopment. January 2003.

Clear Creek Solutions, Inc., 2007. *Bay Area Hydrology Model User Manua*l, Prepared for Alameda Countywide Clean Water Program, San Mateo Countywide Water Pollution Prevention Program, Santa Clara Valley Urban Runoff Pollution Prevention Program. July 2007.

Coleman, D., C. MacRae, and E.D. Stein. 2005. *Effect of increases in peak flows and imperviousness on the morphology of southern California streams. Research Project Technical Report #450.* Report from the Stormwater Monitoring Coalition, Southern California Coastal Water, Westminster, CA.

Contra Costa Clean Water Program (CCCWP). 2004. Work Plan and Literature Review for the Hydrograph Modification Management Plan.

______. 2005. Hydrograph Modification Management Plan. Prepared by Philip Williams & Associates. May 15, 2005.

______. 2008. Stormwater C.3 Guidebook. Stormwater Quality Requirements for Development Applications. Fourth edition, September 10, 2008. California Irrigation Management Information System – Department of Water Resources – Office of Water Use Efficiency. Real-Time and Summarized Evapotranspiration Data for the State of California.

County of San Diego Department of Public Works – Flood Control Division. 1982- present. *ALERT Rainfall Records*.

County of San Diego. 2007. *Low-Impact Development Handbook.* December 31.

Donigan, A.S., and H.H. Davis. 1978. *User's Manual for Agricultural Runoff Management (ARM) Model* . EPA-600/3-78-080. U.S. Environmental Protection Agency.

Dunne, T. and L.B. Leopold. 1978. *Water in Environmental Planning.* New York, W.H. Freeman and Company. 818 pp.

Fairfield-Suisun Urban Runoff Management Program (FSURMP). 2006. *Hydromodification Management Plan for the Fairfield-Suisun Urban Runoff Management Program.* Prepared by Balance Hydrologics, Inc. April, 2006.

Federal Interagency Stream Restoration Work Group (FISRWG). 1998. S*tream Corridor Restoration: Principles, Processes, and Practices.* United States Department of Agriculture.

Fischenich, C. 2001. Stability of Stream Restoration Materials. ERDC TN-EMRRP-SR-29

Geosyntec Consultants. 2004. *Evaluation of the Range of Storms for HMP Performance Criteria*. Memorandum to Onsite Management Measures Subgroup, Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP). April 1, 2004. Revised Final Draft.

Henshaw, P.C. and D.B. Booth 2000. Natural Restabilization of Stream Channels in Urban Watersheds. *Journal of the American Water Resources Association* 36(6): 1219-1236.

Hey, R. D., and Thorne, C. R. 1986. "Stable Channels with Mobile Gravel Beds," Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol. 112, No. 8, pp 671-689.

Hollis, G.E. 1975. *The Effect of Urbanization on Floods of Different Recurrence Intervals.* Water Resources Research.

Leeder, M.R., T. Harris, et al. 1998. Sediment supply and climate change: implications for basin stratigraphy. *Basin Research* 10(1): 7-18.

Leopold, L.B., M.G. Wolman, J.P. Miller. 1964. *Fluvial Processes in Geomorphology.* San Francisco, W.H. Freeman and Company. 522 pp.

Los Angeles Regional Water Quality Control Board. 2002. *TMDL to Reduce Bacterial Indicator Densities at Santa Monica Bay Beaches During Wet Weather.* June 21.

MacRae C.R., 1997. Experience from Morphological Research on Canadian Streams: Is Control of the Two-Year Frequency Runoff Event the Best Basins for Stream Channel Protection? Effects of *Watershed Development And Management Of Aquatic Ecosystems,* L.A. Roesner, ed., ASCE Reston, VA., 144-162.

MacRae, C.R. 1993. An Alternate Design Approach for the Control of Instream Erosion Potential in Urbanizing Watersheds. Sixth International Conference on Urban Storm Drainage, Niagara Falls, Ontario.

Mangarella, P. and Palhegyi, G. Geosyntec Consultants. 2002. Santa Clara Valley Urban Runoff Pollution Prevention Program Hydromodification Management Plan Literature Review.

Palhegyi, G.,P. Mangarella, et al. 2004. Developing Management Plans to Address Impacts from Urbanization on Stream Channel Integrity, ASCE.

Parker, G.P., P.R. Wilcock, C. Paola, et al. 2007. Physical basis for quasi-universal relations describing bankfull hydraulic geometry of single-thread gravel bed rivers. *J. Geophysical Research – Earth Surface*. vol.112

Paul, M.J. and J.L Meyer. 2001. Streams in the Urban Landscape. *Annual Review of Ecologic Systems*. 32:333–65

Phillip Williams and Associates. 2003. Sources of nonlinearity and complexity in geomorphic systems. *Progress in Physical Geography* 27(1): 1.

______. 2008. *Determining Flow Control Thresholds to Avoid and Minimize Channel Erosion due to Hydrograph Modification.* October 12.

______. 2009a. *Sensitivity of Changing Rainfall Series and Analysis on Erosion Threshold.* January 5.

______. 2009b. *Literature Review for the San Diego Hydromodification Management Plan.* October 19.

______. 2009c. *Flow Control Threshold Analysis.* December 29.

Richman, T., P. E. Jill Bicknell, et al. 2004. Start at the Source: Site Planning and Design Guidance Manual for Storm Water Quality Protection, ASCE.

Ripple, W.J., E.J. Larsen, et al. 2001. Trophic cascades among wolves, elk and aspen on Yellowstone National Park's northern range. *Biological Conservation* 102(3): 227-234.

San Diego Co-Permittees. 2009. *Countywide Model SUSMP.* January.

San Diego County Department of Public Works, Flood Control. 2003. *Hydrology Manual*. June.

San Diego Regional Water Quality Control Board. 2008. *TMDLs for Indicator Bacteria in Baby Beach and Shelter Island Shoreline Park.* June 11.

______. 2007. *TMDLs for Indicator Bacteria Project 1 – Beaches and Creeks in the San Diego Region.* December 12.

San Mateo Countywide Stormwater Pollution Prevention Program (STOPPP). 2005.

Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP). 2005. *Hydromodification Management Plan Final Report*. April 21, 2005.

Santa Clara Valley Urban Runoff Pollution Prevention Program. 2006. *Santa Clara Hydromodification Management Plan (HMP).* February.

Schueler, T., 1998. Basic Concepts in Watershed Planning. Chapter 1 from *The Rapid Watershed Planning Handbook*, Center for Watershed Protection, Ellicott City, MD.

Simon, A.,M. Doyle, M. Kondolf, F.D. Shields Jr., B. Rhoads, and M. McPhillips. 2007. Critical evaluation of how the Rosgen Classification and Associated Natural Channel Design Methods fail to Integrate and Quantify Fluvial Processes and Channel Response. J*ournal of the American Water Resources Association.* 43(5):1117-1131.

Southern California Coastal Water Research Project (SCCWRP). 2005. *Evaluating HSPF in an Arid, Urbanized Watershed;* Journal of the American Water Resources Association.

______. 2008. *Stream Channel Classification and Mapping Systems: Implications for Assessing Susceptibility To Hydromodification Effects in Southern California – Technical Report 562.* April.

______. 2010. *Hydromodification Screeening Tools: Field Manual for Channel Susceptibility.* 2010. March.

Stein, E.D., S. Zaleski, et al. 2005. Managing Runoff to Protect Natural Streams: The Latest Developments on Investigation and Management of Hydromodification in California: Proceedings of a Special Technical Workshop, University of Southern California.

Tetra Tech, Inc. 2003. *Lake Elsinore and Canyon Lake Nutrient Source Assessment for Santa Ana Watershed Authority.* January.

Thomas, M.F. 2001. Landscape sensitivity in time and space—an introduction. *Catena* 42(2-4): 83-98.

Trimble, S.W. and A.C. Mendel 1995. The cow as a geomorphic agent—A critical review. *Geomorphology* 13(1-4): 233-253.

United States Environmental Protection Agency. 2000. *BASINS Technical Note 6 – Estimating Hydrology and Hydraulic Parameters for HSPF.* July.

USGS Web Site. Water Resources Applications Software. *Summary of HSPF – Hydrologic Simulation Program – FORTRAN.*

Viessman, Warren, Lewis, Gary L, and Knapp, John W. 1989. *Introduction to Hydrology.* Harper & Row, New York

Waananen, AO and Crippen, JR. 1977. Magnitude and frequency of floods in California. *US Geological Survey Water-Resources Investigation* 77-21. 96 p.

Washington State Department of Ecology. 2001. *Storm Water Management Manual for Western Washington, Volume III*.

West Consultants. 2008. *Review of Hydromodification Work Prepared by Phillip Williams and Associates.* December 19.

Western Regional Climate Center.

Wolman, M.G. and J.P. Miller. 1960. *Magnitude and Frequency of Forces in Geomorphic Processes,* Journal of Geology.

APPENDIX A

Flow Threshold Report

Flow Control Threshold Analysis
for the
San Diego Hydrograph Modification Management Plan

Prepared for

San Diego County and Copermittees

Prepared by

Philip Williams & Associates, Ltd.

December 23rd 2009

PWA REF. # 1915

Services provided pursuant to this Agreement are intended solely for the use and benefit of San Diego County and the Hydromodification Management Plan Copermitees. No other person or entity shall be entitled to rely on the services, opinions, recommendations, plans or specifications provided pursuant to this agreement without the express written consent of Philip Williams & Associates, Ltd., 550 Kearny Street, Suite 900, San Francisco, CA 94108.

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1. INTRODUCTION AND BACKGROUND

1.1 PURPOSE OF THE REPORT

San Diego County and its copermittees are required to develop a Hydromodification Management Plan (HMP) under their National Pollutant Discharge Elimination System (NPDES) MS4 permit. The purpose and requirements of the HMP are described in a 2007 Regional Water Quality Control Board (RWQCB) order renewing the NPDES permit (Order No. R9-2007-0001). The purpose of the HMP is to identify guidelines for managing 'geomorphically-significant' flows that, if not controlled, would cause increased erosion of receiving waters. Specifically, the HMP must identify a low and high flow threshold between which flows should be controlled so that the post-project flow rates and durations do not exceed pre-project levels between these two flow magnitudes. The lower flow threshold is required to correspond to critical flow producing critical shear stress in the channel. The flow control language in the Board Order is as follows:

> Utilize continuous simulation of the entire rainfall record to identify a range of runoff flows[8] for which Priority Development Project post-project runoff flow rates and durations shall not exceed pre-project runoff flow rates and durations, where the increased flow rates and durations will result in increased potential for erosion or other significant adverse impacts to beneficial uses, attributable to changes in the flow rates and durations. The lower boundary of the range of runoff flows identify shall correspond with the critical channel flow that produces the critical shear stress that initiates channel bed movement or that erodes the toe of channel banks. The identified range of runoff flows may be different for specific watersheds, channels or channel reaches.
>
> [8] The identified range of runoff flows to be controlled should be expressed in terms of peak flow rates of rainfall events, such as "10% of the pre-project 2-year peak flow up to the pre-project 10-year peak flow."

1.2 CONCEPTS BEHIND 'GEOMORPHICALLY-SIGNIFICANT FLOWS', CRITICAL FLOWS AND FLOW CONTROL

For the purposes of this project 'hydrograph modification' or 'hydromodification' is understood to mean changes to the frequency, duration and magnitude of surface runoff that, when untreated, cause an increase in erosion of the receiving water body. Hydromodification occurs when urbanization replaces areas of vegetated, uncompacted soil with impermeable surfaces such as buildings, roads and compacted fill. The reduction in permeability results in increased volumes of runoff, and faster and more concentrated delivery of this water to receiving waters. These changes have the potential to cause creeks to erode faster than before development.[1]

---

[1] Although the focus of hydromodification management plans has been on increased erosion it should be noted that in rivers that are depositional hydromodification can cause creeks to regain some transport equilibrium.

Stream flows are often expressed in terms of the frequency with which a particular flow occurs. For example, Q_2 refers to the flow rate that occurs once every two years, on average over the long term. Flow frequencies are a function of rainfall and watershed characteristics, and are unique to each stream channel (and location along the channel). The effects of urbanization tend to increase the magnitude of the flow associated with a given frequency (e.g. post-development Q_2 higher than pre-development Q_2). Similarly, urbanization tends to increase the frequency with which any given flow rate occurs. The purpose of the HMP is to control runoff from new developments so that flow magnitudes and frequencies match pre-development conditions within a critical range of flows.

Not all runoff causes erosion: runoff in receiving channels below a critical discharge (Q_{crit}) does not exert sufficient force to overcome the erosion resistance of the channel banks and bed materials. Flows greater than Q_{crit} cause erosion, with larger flows causing proportionally greater erosion. It has been determined by calculations and field measurements that most erosion in most natural creeks is caused by flows between some fraction of Q_2 and Q_{10} (see for example Leopold, 1964). Flows in this range are referred to as 'geomorphically-significant' because they cause the majority of erosion and sediment transport in a channel system.

Flows greater than Q_{10}, though highly erosive *per event*, occur too infrequently to do as much work as smaller but more frequent flows (see Figure 1). Hydromodification also has less impact on flows greater than Q_{10} since at such high rainfall intensities the soil becomes saturated and the infiltration capacity of undeveloped landscapes is rapidly exceeded. When the soil is saturated, runoff rates become more similar to those from impervious surfaces. For these reasons, HMPs have focused on identifying a low flow threshold that is close to Q_{crit} for most receiving channels, and controlling flows between that value and Q_{10} (see for example the HMPs completed in Santa Clara, Contra Costa, Alameda and San Mateo Counties). By requiring treatment (storage and either infiltration or detention) of excess runoff within the control range, and by limiting the release of excess water to Q_{crit} or less, HMPs seek to prevent additional erosion in receiving channels.

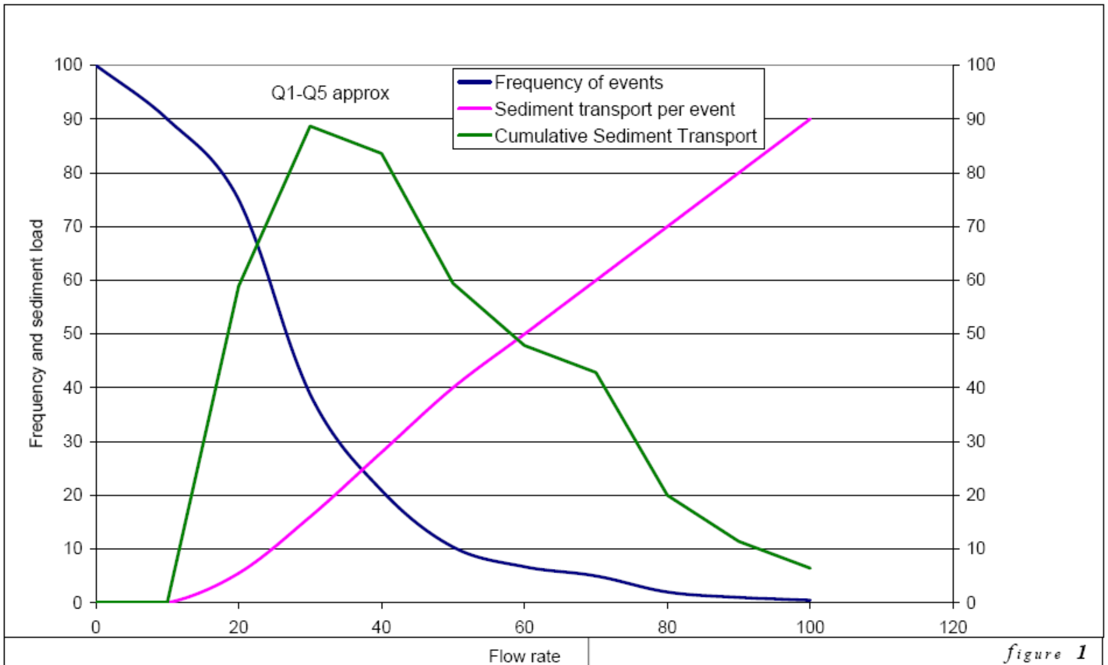


figure **1**
San Diego HMP

Hypothetical cumulative work curve

PWA Ref #: 1915

PWA

2. IDENTIFYING A HIGH FLOW THRESHOLD

Previous HMPs have focused considerable attention on the low flow threshold, but little on the high flow threshold. The use of an upper flow threshold is based on two assumptions:

1. Flows above this level cause relatively little cumulative erosion in receiving waters due to their low recurrence
2. Flows above this level are relatively unaffected by hydromodification because at such high rainfall intensities and durations the pre-development ground cover become saturated and most rain runs off, similar to in a post development condition.

The five HMPs developed to date in California have all adopted a value of Q_{10} as the upper threshold. We propose adopting the same value for the San Diego HMP.

3. IDENTIFYING A LOW FLOW THRESHOLD

Erosion occurs when the shear stress exerted on the channel by flowing water (*boundary shear stress*) exceeds the resistance of the channel (*critical shear stress*). Critical shear stress varies by several orders of magnitude for different channel materials (Table 1). *Critical flow* (Q_{crit}) is the channel flow which produces boundary shear stress equal to the critical shear stress for a given channel. That is, the flow rate that can initiate erosion in a channel. Q_{crit} is a function not only of the critical shear stress of the channel materials, but also channel size, and channel geometry. A particular flow rate (expressed as a number of cubic feet per second) in a small, steep, confined channel will create more shear stress than the identical flow rate in a large, flat, wide open channel. Thus Q_{crit} can be extremely variable depending on channel and watershed characteristics and will be different in each channel, and in each watershed.

Boundary Category	Boundary Type	Permissible Shear Stress (lb/sq ft)
Soils	Fine colloidal sand	0.02 - 0.03
	Sandy loam (noncolloidal)	0.03 - 0.04
	Alluvial silt (noncolloidal)	0.045 - 0.05
	Silty loam (noncolloidal)	0.045 - 0.05
	Firm loam	0.075
	Fine gravels	0.075
	Stiff clay	0.26
	Alluvial silt (colloidal)	0.26
	Graded loam to cobbles	0.38
	Graded silts to cobbles	0.43
	Shales and hardpan	0.67
Gravel/Cobble	1-in.	0.33
	2-in.	0.67
	6-in.	2.0
	12-in.	4.0
Vegetation	Class A turf	3.7
	Class B turf	2.1
	Class C turf	1.0
	Long native grasses	1.2 – 1.7
	Short native and bunch grass	0.7 - 0.95
	Reed plantings	0.1-0.6
	Hardwood tree plantings	0.41-2.5
Temporary Degradable RECPs	Jute net	0.45
	Straw with net	1.5 – 1.65
	Coconut fiber with net	2.25
	Fiberglass roving	2.00
Non-Degradable RECPs	Unvegetated	3.00
	Partially established	4.0-6.0
	Fully vegetated	8.00
Riprap	6 – in. d_{50}	2.5
	9 – in. d_{50}	3.8
	12 – in. d_{50}	5.1
	18 – in. d_{50}	7.6
	24 – in. d_{50}	10.1
Soil Bioengineering	Wattles	0.2 – 1.0
	Reed fascine	0.6-1.25
	Coir roll	3 - 5
	Vegetated coir mat	4 - 8
	Live brush mattress (initial)	0.4 – 4.1
	Live brush mattress (grown)	3.90-8.2
	Brush layering (initial/grown)	0.4 – 6.25
	Live fascine	1.25-3.10
	Live willow stakes	2.10-3.10
Hard Surfacing	Gabions	10
	Concrete	12.5

[1] Ranges of values generally reflect multiple sources of data or different t

A. Chang, H.H. (1988). F. Julien, P.Y. (1995).
B. Florineth. (1982) G. Kouwen, N.; Li, R. M.; and Simons, D.B., (1980).
C. Gerstgraser, C. (1998). H. Norman, J. N. (1975).
D. Goff, K. (1999). I. Schiechtl, H. M. and R. Stern. (1996).
E. Gray, D.H., and Sotir, R.B. (1996). J. Schoklitsch, A. (1937).

Table 1. Range of critical shear stresses (τ_{cr}) for different materials. From Fischenich, 2001.

It was the original intent of the HMP project team to identify a single low flow threshold for the entire county (per previous HMPs). However, an extensive assessment of channel and runoff conditions led the team to conclude that there was a very wide range in critical flows, based largely on channel material but also on channel dimensions, rainfall, and watershed area[2]. Adopting a single standard that is conservative for the most vulnerable channels would result in controls that were excessively conservative for more resilient channels, while adopting an 'average' value would leave some channels unprotected. As the ongoing SCCWRP Hydromod project is showing, individual creeks have different risk categories and respond in different ways to the same level of hydromodification. Because of this natural variability, we pursued an analytical approach for estimating Q_{crit} as a function of parameters such as channel materials, channel dimensions and watershed area. The following sections of this report describe an analysis of Q_{crit} as a fraction of Q_2 for the range of channel conditions in San Diego County. This is followed by a description of a calculator tool developed by PWA that may be used to calculate Q_{crit} for a specific channel based on parameters that may be readily measured in the field. The analyses described in this report provide background for the selection of low flow thresholds identified in the HMP.

[2] These early analyses are summarized in Appendix D of the Final Hydromodification Management Plan.

4. CRITICAL FLOW ANALYSIS

4.1 BACKGROUND

PWA conducted a sensitivity analysis in which a wide range of channel sizes and geometries, rainfalls, watershed areas and channel materials were modeled in a flow-erosion model to identify Q_{crit} as a function of Q_2. In all, 170 combinations of channel, rainfall and watershed conditions were assessed (described below). Based on the results of this sensitivity analysis, a range of Q_{crits} were identified for several categories of channel materials.

The steps used to conduct the sensitivity analysis:

1. Identify the typical range of rainfall conditions for the HMP area (west San Diego County)
2. Identify the range of typical watershed areas likely to be developed
3. Identify a range of typical receiving channel dimensions for each watershed area
4. Identify a range of typical channel materials for receiving channels
5. Simulate a range of flows and develop rating curves (relationships between discharge and boundary shear stress)
6. Identify the flow rate at which boundary shear stress exceeds critical shear stress for the channel and material
7. Express this flow rate as a function of Q_2
8. Group critical flow rates by channel materials.

Steps 1 through 4 were used to define the range of parameters to use in the sensitivity testing. The intent was to identify a typical range of conditions likely to occur in the HMP area (west San Diego County), rather than provide an exhaustive description of possible watershed and channel conditions. Sensitivity testing on many combinations of parameters within this typical range allows identification of the range of channel responses and critical flows.

Each step in the critical flow analysis is explained in detail in the following sections.

4.2 IDENTIFY THE TYPICAL RANGE OF RAINFALL CONDITIONS FOR THE HMP AREA (WEST SAN DIEGO COUNTY)

Mean annual rainfall was used to estimate receiving channel size, Q_2, Q_5 and Q_{10} (methods described in subsequent sections). Figure 2 shows mean annual rainfall for San Diego County. Based on the map, three mean annual rainfalls were selected to represent the range of rainfall conditions for the simulations: 10”, 20” and 30”.

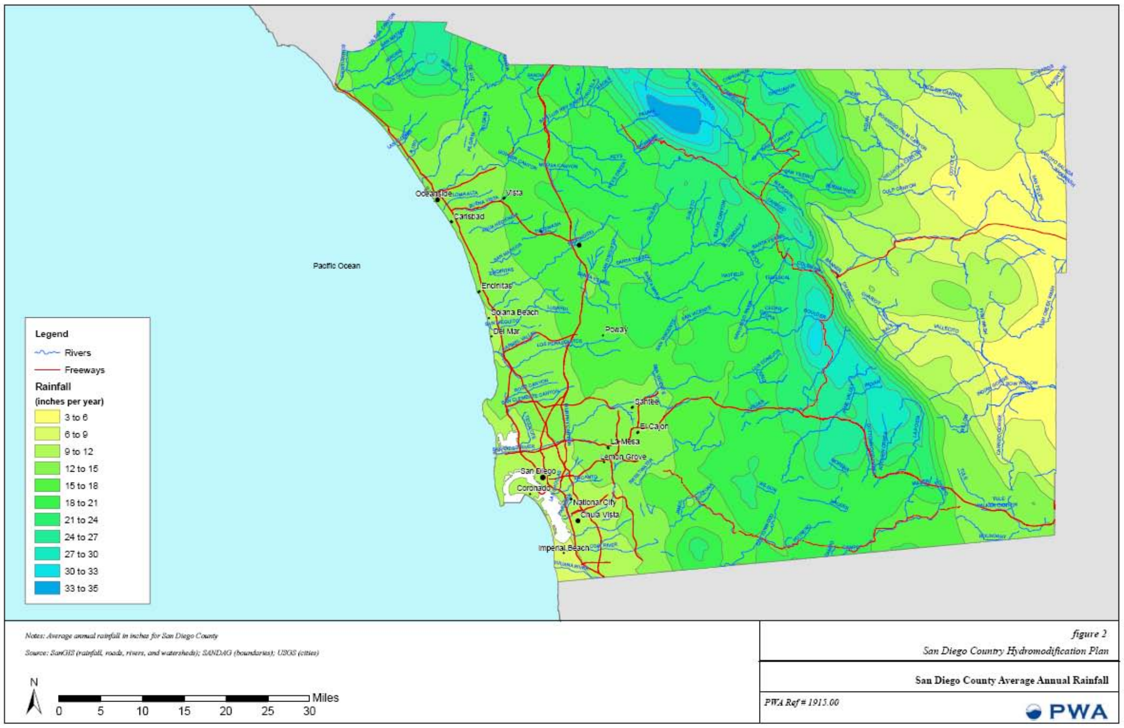


Figure 2. Rainfall distribution in San Diego County

4.3 IDENTIFY THE RANGE OF TYPICAL WATERSHED AREAS LIKELY TO BE DEVELOPED

Based on discussions with the Technical Advisory Committee, a range of representative watershed areas for development projects was identified. These were: 0.1 sq mi, 0.5 sq mi, 1 sq mi, 2 sq mi. We assumed that in project watersheds larger than 2 sq mi the development would either require site specific continuous simulation modeling, or be broken into multiple smaller sub watersheds with individual points of compliance.

4.4 IDENTIFY A RANGE OF TYPICAL RECEIVING CHANNEL DIMENSIONS FOR EACH WATERSHED AREA

Empirical relationships have been developed to express channel dimensions (width, depth and, to a lesser extent, gradient) as a function of dominant discharge. Dominant discharge for a creek channel is the flow rate that transports the majority of sediment and creates/maintains the characteristic size and shape of the channel over time. Dominant discharge may also be referred to as bankfull flow. For undeveloped channels in semi arid parts of the US, dominant discharge is approximately equivalent to Q_5. For example, Coleman et. al. (2005) found dominant discharge for streams in Southern California to average $Q_{3.5}$ (range = $Q_{2.1}$ – $Q_{6.7}$.) Goodwin (1998) found dominant discharge to vary from Q_2 to Q_{10} for semi arid regions.

To capture natural variability in channel geometry, we used three different empirical channel geometry relationships to estimate receiving channel dimensions for the range of watershed areas and rainfall characteristics used in this study. The relationships were:

Coleman et. al. 2005 (modified by Stein – personal communication) – derived from undeveloped channels in Southern California, tends to predict narrow, deep, steep dimensions.

$$\text{Width (ft)} = 0.6012 * Q_{bf}^{0.6875}$$

$$\text{Depth (ft)} = 0.3854 * Q_{bf}^{0.3652}$$

Where Q_{bf} is in cfs.

Parker et al. 2007 – suitable for gravel channels, tends to predict wide, shallow, flat braided dimensions.

$$\text{Width (m)} = 4.63 * (Q_{bf}^{2/5}) / (9.81^{1/5}) * (Q_{bf} / \text{Sqrt } (9.81 * d50) * d50^{2}))^{0.0667}$$

$$\text{Depth (m)} = 0.382*((Qbf^{2/5})/(9.81^{1/5})))$$

Where Q_{bf} is bankfull discharge in m^3/sec and d50 (diameter of median channel material) is in m.

The Parker equation was only used to assess gravel and cobble channel conditions.

Hey and Thorne 1986 tends to predict medium width, depth, and gradient channels.

$$\text{Width (m)} = 2.73*Q_{bf}^{0.5}$$

$$\text{Depth (m)} = 0.22 * \text{Width}^{0.37} * d50^{-0.11}$$

Where Q_{bf} is in m^3/sec and d50 is in m.

(Note that we have used the original combinations of English and metric units described in the source papers rather than standardized these equations in one set of measurements.)

The three equations cover a wide range of likely field conditions, from deeply incised channels (Coleman et al, 2005) to wide, braided conditions (Parker, 2007). Note that for the sensitivity analysis we set d50 in the Parker et al. equation to the d50 of the channel material being tested, and did not use the equation for channels where the material was sand or silt.

The equations produce estimations of width and depth. To estimate a slope for each combination of channel dimensions we calculated the velocity associated with each cross section (by dividing discharge by width multiplied by depth) and calculated the slope that corresponded with that velocity using Manning's equation.

$$\text{Velocity (ft/sec)} = 1.486\ \frac{HR^{0.66} * s^{0.5}}{n}$$

Where HR is channel hydraulic radius, s is slope, and n is Manning's roughness coefficient (see definitions). For the purposes of the sensitivity analysis a value of n 0.035 was assumed, corresponding to a non vegetated, straight channel with no riffles and pools. This is a reflection of the small, ephemeral receiving channels which are most prevalent in Southern California developments. A relatively low value was used at the request of the San Diego RWQCB so that the values erred on the conservative side.

These equations all require a value for bankfull discharge. Bankfull discharge (assumed to be approximately Q_5) was estimated using the USGS regional regression for undeveloped watersheds in the South Coast region (Waananen and Crippen, 1977). This equation calculates Q_5 as a function of watershed area and mean annual precipitation, based on empirical observations of USGS gages. The relationship is:

$$Q_5\ \text{(cfs)} = 0.4 * \text{Watershed Area}^{0.77} * \text{Mean Annual Precipitation}^{1.69}$$

Where watershed area is in square miles and precipitation is in inches.

For each combination of typical watershed area (Section 2.2) and mean annual rainfall (Section 2.3) we calculated Q_5 using the USGS regression, then calculated three sets of channel dimensions based on the three channel equations. This provided the range of channel conditions to simulate for the critical flow analysis. The total number of channel conditions was as follows:

3 rainfalls (10, 20, 30 inches per year)
4 watershed areas (0.1, 0.5, 1, 2 square miles)
3 channel width, depth and slope combinations (narrow/deep, medium, wide/shallow)
= 36 combinations of receiving channel geometry

4.5 IDENTIFY A RANGE OF TYPICAL CHANNEL MATERIALS FOR RECEIVING CHANNELS

We identified a range of typical channel materials based on feedback from the TAC and experience gained working in San Diego County. The identified materials are not intended as a comprehensive list of possible channel materials, but to cover the range of critical shear stresses likely to be encountered in typical western San Diego County channels. The identified range is as follows:

Material	Critical shear stress (lb/sq ft)
Coarse unconsolidated sand	0.025
alluvial silt (non coloidal)	0.045
medium gravel	0.12
alluvial silt/clay	0.26
2.5 inch cobble	1.1

Combining the 5 channel material types with the 36 combinations of channel geometry produces 180 potential combinations of receiving channel characteristics. Ten sets of combinations were omitted from the analysis because they produced physically unrealistic conditions, such as slopes that were too steep to be developed. Exclusion of these results did not significantly affect the overall results.

4.6 DEVELOP SHEAR STRESS RATING CURVES

Rating curves for the 36 different combinations of receiving channel characteristics were developed using the same Excel worksheet that forms the basis for the Qcrit calculator developed for Track 2 (described in later sections). Using channel cross section, roughness and gradient input by the user, the tool calculates the average boundary shear stress associated with a range of different flow depths to construct a rating curve (discharge on the x axis versus shear stress on the y axis). It then identifies the flow rate where average boundary shear stress equals critical shear stress for the channel materials. This is the critical flow (Q_{crit}). By dividing this number by Q_2 we identify the critical flow for each simulation as a function of Q_2 (e.g. $0.1Q_2$ where the critical flow is one tenth of the Q_2 flow).

The tool calculates a shear stress rating curve for a range of flows between 1% and 100% of the bankfull flow depth. Bankfull flow depth is defined as the flow depth that corresponds to the dominant discharge for a given channel. The range 1% to 100% of bankfull is used because critical flow rarely falls outside these values. The tool then calculates an equation that allows for interpolation between the points. For each of the depths, the tool calculates discharge and average boundary shear stress exerted on the bed, as described below.

4.6.1 <u>Calculating Average Boundary Shear Stress</u>

Average boundary shear stress is the force that flowing water exerts on channel materials. For a given channel cross-section, it is calculated as follows:

$$\tau_b = \gamma * HR * s$$

where τ_b = average boundary shear stress (lb/ft^2)
γ = unit weight water (62.4 lb/ft^3)
HR = Hydraulic radius (cross section area / wetted perimeter)
S = channel slope (ft/ft)

For each depth increment between 1% and 100% of bankfull, cross section area, wetted perimeter, HR and τ_b are calculated. Slope is a constant for the cross section. These calculations produce a rating curve for boundary shear as a function of flow depth.

4.6.2 Calculating Flow Rate

This step converts flow depth to flow rate (Q) so that the rating curve may be expressed as a function of Q. For each depth increment between 1% and 100% of bankfull, the flow rate is calculated using Manning's equation:

$$\text{Velocity (ft/sec)} = 1.486 \frac{HR^{0.66} * s^{0.5}}{n}$$

where V = velocity (ft/sec)

n = Manning's roughness coefficient

For the sensitivity analysis Manning's n was assumed to be 0.035, which is typical for a non-vegetated ephemeral channel. We assumed that for most developments covered by the HMP the receiving channels would be relatively high in the watershed and would have received little summer flow. In interim sensitivity analysis found that relative to other factors such as critical shear stress, the range of roughness factors found in receiving channels had little effect on the estimated critical shear flow rate.

Discharge is calculated as velocity multiplied by cross section area (calculated for each cross section, above). The result of these calculations is a rating curve showing boundary shear stress for the receiving channel as a function of discharge, with the highest point representing bankfull depth (see Figure 3 below). Rating curves were created for each of the 36 combinations of channel characteristics.

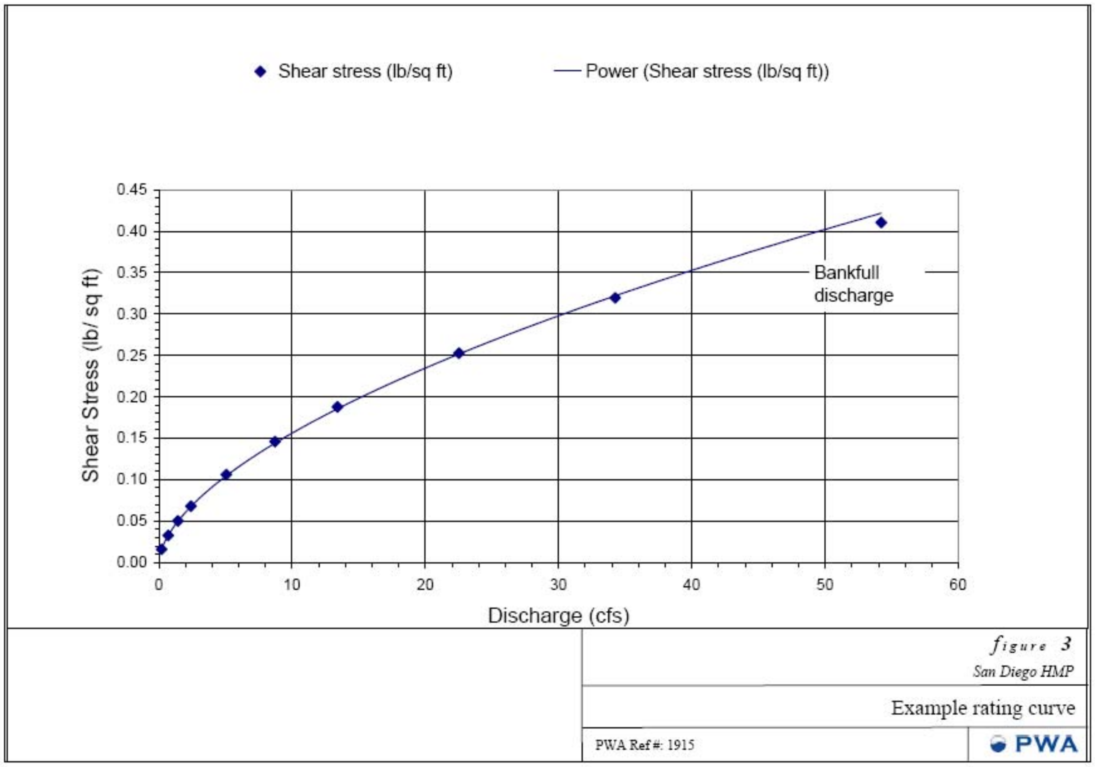


Figure 3. Shear stress rating curve for an example channel (0.5%, 10 feet wide, 2 feet deep). These curves were created for 36 different combinations of channel characteristics.

4.7 IDENTIFY CRITICAL FLOW FOR THE CHANNEL AND MATERIAL

Q_{crit} is the flow rate at which boundary shear stress equals critical shear stress. The tool uses a power function to interpolate the discharge versus boundary shear stress rating curve, to allow calculation of an intercept between the rating curve and critical shear stress. The critical shear stress for each channel material was plotted horizontally from the Y axis until it intercepted the rating curve. The intercept point was extended vertically to the X axis, showing the Q_{crit} (see Figure 4 below). In this way, Q_{crit} was calculated for each of the five channel materials using each of the 36 rating curves representing different channel dimensions. As mentioned above, 10 combinations unlikely to occur in nature were eliminated, resulting in a total of 170 Q_{crit} calculations.

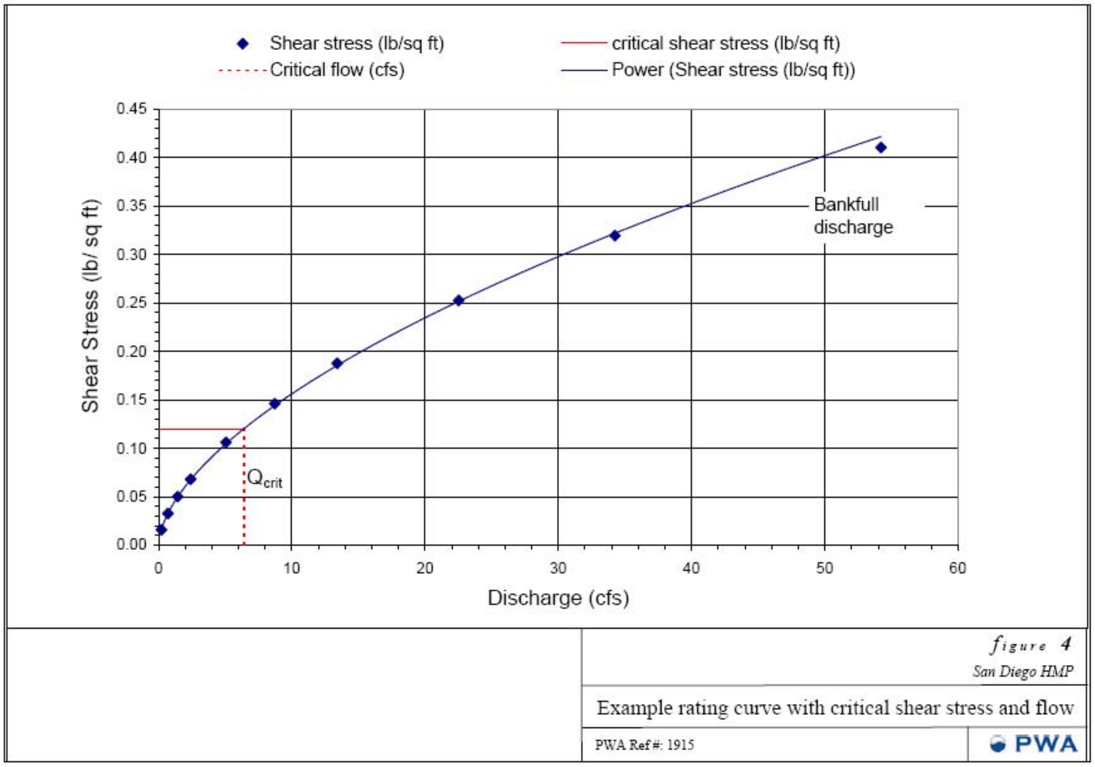


Figure 4. Example of a rating curve with critical shear stress for medium sized gravel. In this example critical shear stress = 0.12 lb/sq ft and critical flow Q_{crit} = 6.4 cfs.

4.8 EXPRESS CRITICAL FLOW AS A FUNCTION OF Q_2

As described above, each rating curve represents a particular combination of watershed area and channel dimensions. Q_2 was calculated for each combination using the USGS regional regression for Q_2 as described in section 4.4. By dividing the calculated Q_{crit} by the appropriate Q_2, Q_{crit} as a proportion of Q_2 was calculated for the 170 scenarios. These Q_{crits} were then plotted by material type, showing mean and one standard deviation either side of the mean. Note that although we assume that Q_5 is bankfull discharge, we express the critical flow as a function of Q_2 as has become standard for HMPs.

4.9 CRITICAL FLOW ANALYSIS RESULTS

The results show the high degree of variability in Q_{crit} based on different channel materials. It is important to note that in field conditions many of the most extreme cases shown in the figure (examples with very high or very low thresholds) would tend to evolve to conditions that yielded critical flows closer to the bankfull discharge because channels have a tendency to self equilibrate. For example, channels with materials that have very low critical flows such as unconsolidated sand tend to erode and either flatten (lowering shear stress, and so increasing critical flow rate) or armor (increasing flow resistance, and increasing critical flow rate). Likewise, channels with materials that have very high thresholds tend to either become steeper due to deposition (increasing shear stress and lowering critical flow rate) or fill in with finer material (reducing resistance and lowering critical flow rate).

4.10 DISCUSSION

As the results of this analysis demonstrate, critical flow is extremely variable among channel materials and, for a given channel material, can vary significantly with channel configuration (slope, width/depth ratio etc.). Unconsolidated fine sediments can be mobilized by extremely low flows in the absence of clays or other consolidating elements with the structure of the channel. This result is based on literature values for critical shear stress for unconsolidated materials and may not be realistic for natural channels. Therefore in setting flow thresholds this result should be balanced with the recognition that natural channels are likely to include some consolidating fraction within their structure, as well as practical considerations associated with controlling trickle flows that represent the smaller fractions of Q_2 analyzed in this study.

5. TOOL FOR CALCULATING SITE-SPECIFIC CRITICAL FLOW

5.1 BACKGROUND

PWA developed a tool for calculating a site-specific critical flow (Q_{crit}) based on local conditions. Q_{crit} for the receiving channel is calculated based on channel geometry (width, depth and gradient), channel materials, and watershed area.

The approach taken was to develop an Excel spreadsheet model to calculate the boundary shear stress associated with a range of flows up to Q_5 for a given channel width, depth and slope, then plot the critical shear stress for the channel material on this rating curve over to identify the flow where boundary shear stress equals critical shear stress (see example graph below).

The development steps were as follows:

1. Develop simplified channel cross section and gradient inputs
2. Calculate a shear stress rating curve
3. Characterize channel materials in terms of critical shear stress
4. Plot critical shear stress of the receiving channel on the rating curve to determine Q_{crit}
5. Divide the critical low flow by the project areas as a proportion of the receiving water watershed area to determine the allowable flow at the point of compliance

5.2 SIMPLIFIED CHANNEL CROSS SECTION AND GRADIENT INPUTS

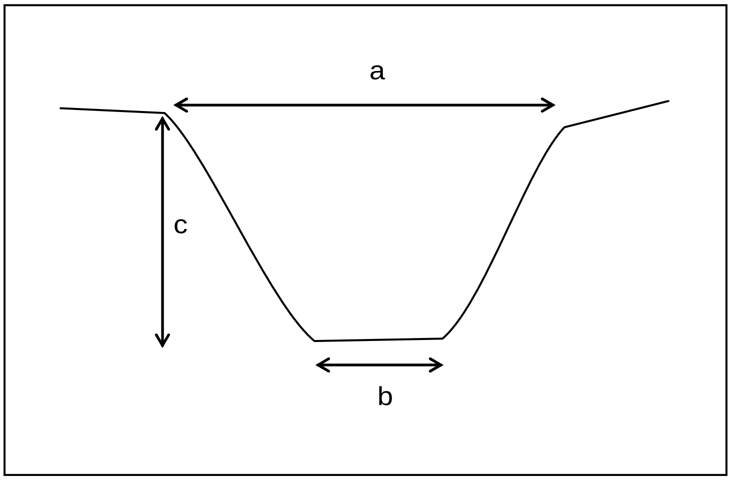
The tool generates a flow rating curve based on user inputs describing the receiving channel dimensions (cross section) and gradient. The first step in developing the tool was to create a template for inputting the required channel parameters. The template assumes a simple trapezoidal cross section, with the following elements:

1. Channel width at a well defined break point corresponding to top of bank (a)
2. Channel width at the toe of the bank (b)
3. Channel depth (elevation difference between bank top and channel bed) (c)

Assumptions:

1. Receiving channels can be reasonably represented by a simple trapezoidal cross section
2. The top of bank corresponds reasonably to the level inundated by the dominant discharge (approximately equal to Q_5)

If top of bank is much higher than the dominant discharge flow depth (e.g. in an incised channel) the applicant should adjust the cross section to represent the lower part of the channel so that depth (c) corresponds approximately to the Q_5 depth.



5.3 DEVELOP A SHEAR STRESS RATING CURVE

The tool creates a shear stress rating curve for a range of flows between 1% of the bankfull flow depth and bankfull depth (flow at depth (c).) The range 1% to 100% of bankfull is used because critical flow rarely lies outside these values. The tool then calculates a power function between the points to allow for interpolation. For each of the flows the tool calculates average boundary shear stress exerted on the bed, and discharge, as described below.

5.3.1 Calculating Average Boundary Shear Stress

Average boundary shear stress is the force that erodes channel materials. It is calculated as follows:

$$\tau_{crit} = \gamma * HR * s$$

where τ_{crit} = average boundary shear stress (lb/ft^2)
γ = unit weight water (62.4 lb/ft^3)
HR = Hydraulic radius (cross section area / wetted perimeter)
S = channel slope (ft/ft)

For each depth increment between 1% of bankfull and bankfull, cross section area, wetted perimeter, HR and τ_{crit} are calculated. Slope is assumed to be constant for the cross section; therefore multiple calculations may be required for variable slope conditions. These calculations produce a rating curve for boundary shear stress as a function of flow depth.

5.3.2 Calculating Discharge

For each depth increment between 1% of bankfull and bankfull discharge is calculated using Manning's equation:

$$V = 1.486 \frac{HR^{0.66} * S^{0.5}}{n}$$

where V = velocity (ft/sec)
n = Manning's roughness coefficient

Manning's n is entered by the user from a drop down dialogue box ranging from 0.03 (smooth, straight earth channel with no vegetation) to 0.12 (windy, rough bed channel with dense vegetation).

Discharge is calculated as velocity multiplied by cross section area. The product of these calculations is a rating curve showing boundary shear stress for the receiving channel as a function of discharge, with the highest point representing bankfull flow (see Figure 7 below).

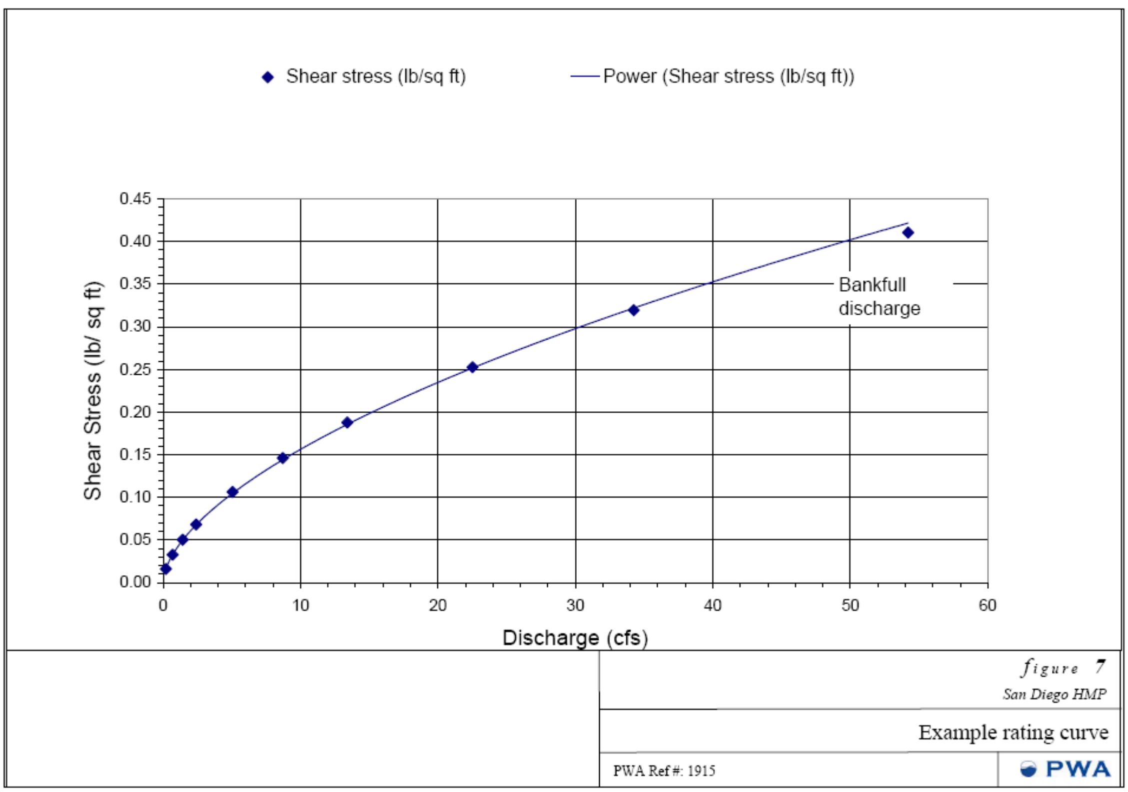


Figure 7. Shear stress rating curve for an example channel (0.5%, 10 feet wide, 2 feet deep)

5.4 CHARACTERIZE RECEIVING CHANNEL MATERIALS IN TERMS OF CRITICAL SHEAR STRESS

The critical shear stress of the channel materials is estimated using a look-up table based on values published by the USACE (Fischenich, 2001). The tool provides values of critical shear stress for a wide range of channel materials in a drop down box so the user can select from the list, or select a median particle size (d50). The values are shown in Table 1. The calculator also allows the user to input a vegetated channel material when this is appropriate (when the channel is completely lined in vegetation). The process for identifying representative materials is covered in the implementation chapter.

5.5 CALCULATING CRITICAL FLOW FOR THE RECEIVING WATER

Critical flow is the discharge at which boundary shear stress equals critical shear stress. The tool uses a power function to interpolate the discharge versus boundary shear stress rating curve. The critical shear stress for the weaker of the bed or banks is plotted horizontally from the Y axis until it intercepts the rating curve. The intercept point is extended vertically to the X axis, showing the critical flow (see Figure 8 below). This represents Q_{crit} for the receiving water. Note that the creation of a site-specific rating curve allows Q_{crit} to be expressed as a specific flow rate (Q) rather than a fraction of Q_2.

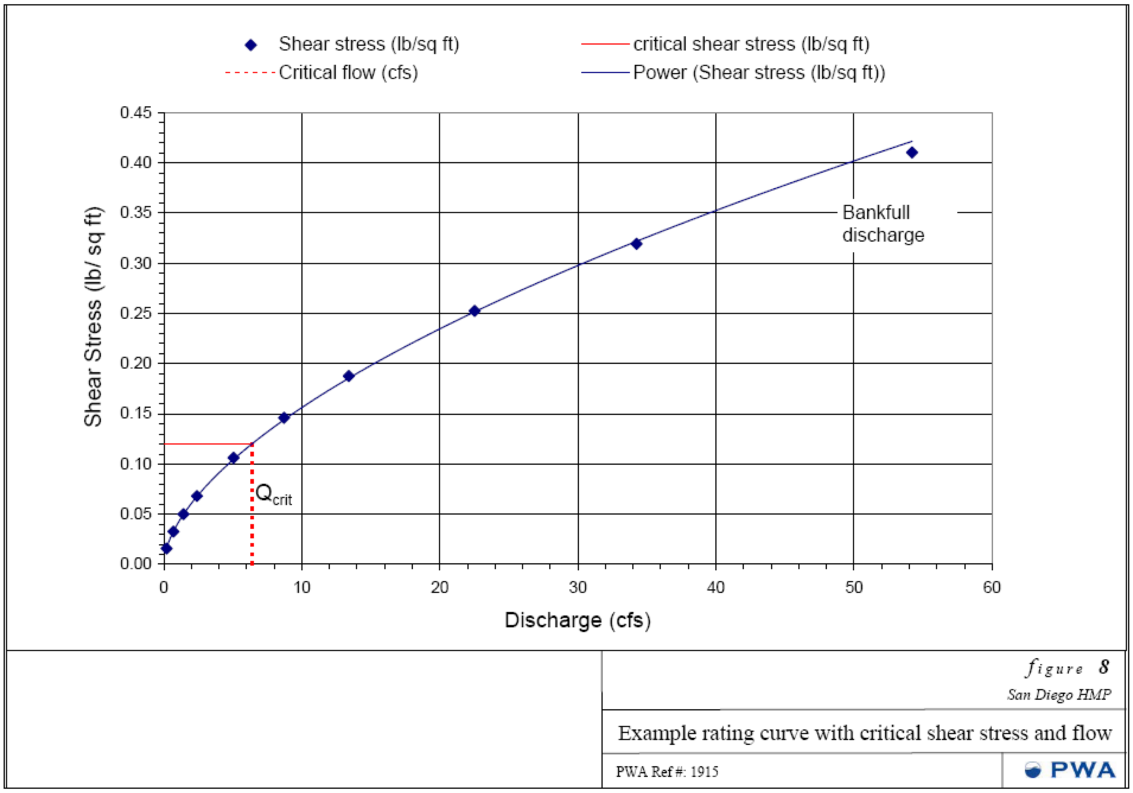


Figure 8. Example of a rating curve with critical shear stress for medium sized gravel. In this example critical shear stress = 0.12 lb/sq ft and critical flow Q_{crit} = 6.4 cfs.

5.6 CALCULATING CRITICAL FLOW AT THE POINT OF COMPLIANCE

The tool calculates critical flow based on the characteristics of the receiving water. Where the project watershed does not make up the entire watershed area for the receiving water, it is necessary to divide the estimated Qcrit based on the percentage of the watershed that is occupied by the project site[3]. For example, if a project occupies one tenth of the receiving water's watershed at the point of compliance and the critical flow level is 50 cfs, the project's 'share' of the non-erosive flow is 5 cfs (50 x 1/10). This prevents the cumulative impact of future developments from exceeding critical flow in the receiving water, since the critical flow is apportioned according to watershed area.

$$\text{Critical flow at Point of Compliance} = \text{Critical flow at receiving water} \times \frac{\text{project area}}{\text{watershed area}}$$

[3]. It is not necessary to adjust the "off-the-shelf" thresholds developed for Track 1 for point of compliance, since they are expressed as a fraction of Q2 for the relevant project area.

5.7 CONVERSION OF CRITICAL FLOW TO FLOW CLASS

To avoid having an infinite range of flow control standards the calculator assigns the discharge into one of three classes based on its value as a function of the estimated Q_2. These classes are: $0.1Q_2$, $0.3Q_2$, $0.5Q_2$. For example, a channel where the critical flow is $0.15Q_2$ would be assigned a flow threshold of $0.1Q_2$. Channels with critical flows less than $0.1Q_2$ are assigned to the $0.1Q_2$ class. The class flow rate is calculated (i.e. the critical flow corresponding to the assigned fraction of Q_2 and expressed as the final output of the tool.

6. GLOSSARY

Bankfull depth
The water depth between the deepest part of the channel and the water surface, during bankfull discharge. Also the vertical distance between the uppermost 'bankfull indicators' and the deepest part of the channel.

Bankfull discharge
The flow rate at which the actively scoured portion of the creek channel is filled with water. In southern California bankfull discharge has typically been found to be between Q2 and Q7, with an average of approximately Q5.

Bankfull indicators
Morphological evidence for the portion of a creek channel that is subject to active scour and sediment transport processes. Typical indicators include scour lines along a bank, the highest vertical level on point bars, base of undercut tree roots.

Bankfull width
The width of the channel at the water surface during bankfull discharge. Also the horizontal distance between 'bankfull indicators' across a channel.

Critical flow
The discharge corresponding to Critical Shear Stress. Varies with channel geometry and materials.

Critical shear stress
The shear stress at which a given channel material is eroded. In non cohesive sediments larger particles have higher critical shear stresses. In cohesive sediments (those smaller than 0.063 mm) sediment has higher critical shear stresses than fine, non cohesive materials

d50
The median sediment particle size in a sample of material taken from a creek bed (diameter of the 50^{th} percentile)

Geomorphically-significant flows
The range of flows that, over a period of several decades, erode and transport the majority of the sediment in a creek system. The mid range of this flow range tends to be similar to "bankfull" discharge, leading people to infer that these flows shape the channel as well as moving most sediment. Calculated by integrating the flow frequency curve with the sediment rating curve.

Point of Compliance

The point at which collected stormwater from a development is delivered from a constructed or modified drainage system into the natural creek receiving water. Note that the HMP applies only to discharge into a natural creek of receiving water, and does not apply to sheet flow or overland flow from a developed site.

Q2

The discharge that recurs on average every 2 years, and that has a 50% probability of occurring in any single year.

Q10

The discharge that recurs on average every 10 years, and that has a 10% probability of occurring in any single year.

Shear stress (also known as boundary shear stress or average boundary shear stress)

The average force exerted by flowing water on the channel boundary. Shear stress is the force responsible for eroding sediment from the channel boundary. It is a function of water surface gradient (related to channel gradient), water depth, and water density.

7. REFERENCES

Coleman, D., MacRae, C., and Stein, E.D. 2005. Effect of Increases in Peak Flows and Impervious on the Morphology of Southern California Streams, Technical Report 450. Southern California Coastal Water Research Project.

Hey, R. D., and Thorne, C. R. 1986. "Stable Channels with Mobile Gravel Beds," Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol 112, No. 8, pp 671-689.

Fischenich, C. 2001. Stability of Stream Restoration Materials. ERDC TN-EMRRP-SR-29

Leopold, L.B., M.G. Wolman, J.P. Miller. 1964. Fluvial Processes in Geomorphology. San Francisco, W.H. Freeman and Company. 522 pp.

Parker, G.P., P.R. Wilcock, C. Paola, W.E. Dietrich, J.C. Pitlick. 2007. "Physical basis for quasi-universal relations describing bankfull hydraulic geometry of single-thread gravel bed rivers". J. Geophysical Research – Earth Surface. vol.112

Waananen, A.O., and Crippen, J.R., 1977, Magnitude and frequency of floods in California: U.S. Geological Survey Water-Resources Investigations Report 77-21, 96 p.

8. LIST OF PREPARERS

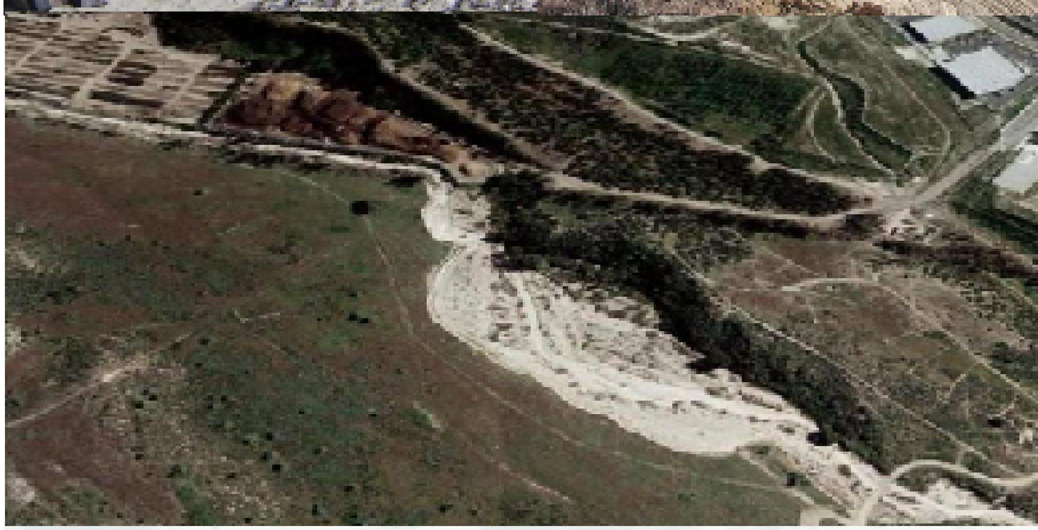
This report was prepared by the following PWA staff:

Andrew Collison, PhD
Setenay Bozkurt, MS
Christie Beeman, PE

APPENDIX B

SCCWRP Channel Screening Report

HYDROMODIFICATION SCREENING TOOLS: FIELD MANUAL FOR ASSESSING CHANNEL SUSCEPTIBILITY



SCCWRP
Established 1969

Colorado State University

Stillwater Sciences

Brian P. Bledsoe
Robert J. Hawley
Eric D. Stein
Derek B. Booth

Southern California Coastal Water Research Project

Technical Report 606 - March 2010

Hydromodification Screening Tools: Field Manual for Assessing Channel Susceptibility

Brian P. Bledsoe[1], Robert J. Hawley[1]
Eric D. Stein[2] and Derek B. Booth[3]

[1]Colorado State University, Fort Collins, CO
[2]Southern California Coastal Water Research Project, Costa Mesa, CA
[3]Stillwater Sciences, Inc., Santa Barbara, CA

March 2010

Technical Report 606

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EXECUTIVE SUMMARY

Managing the effects of hydromodification (physical response of streams to changes in catchment runoff and sediment yield) has become a key element of most stormwater programs in California. Although straightforward in intent, hydromodification management is difficult in practice. Shifts in the flow of water and sediment, and the resulting imbalance in sediment supply and capacity can lead to changes in channel planform and cross-section via wide variety of mechanisms. Channel response can vary based on factors such as boundary materials, valley shape and slope, presence of in-stream or streamside vegetation, or catchment properties (e.g., slope, land cover, geology).

Management prescriptions should be flexible and variable to account for the heterogeneity of streams; a given strategy will not be universally well-suited to all circumstances. Management decisions regarding a particular stream reach(s) should be informed by an understanding of susceptibility (based on both channel and catchment properties), resources potentially at risk (e.g., habitat, infrastructure, property), and the desired management endpoint (e.g., type of channel desired, priority functions; see Figure ES1).

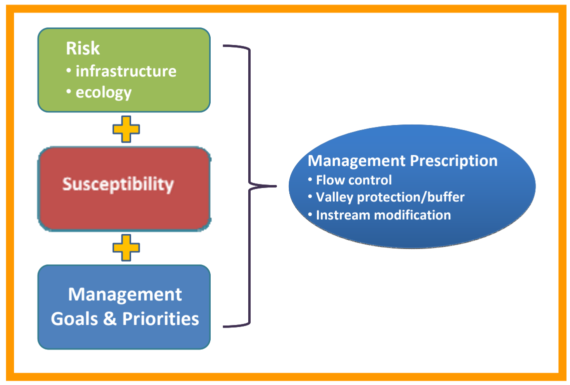

Figure ES1: Decision nodes that influence the management prescription for a particular stream reach.

We have produced a series of documents that outline a process and provide tools aimed at addressing the decision node associated with assessing channel susceptibility. The three corresponding hydromodification screening tool documents are:

1. *GIS-based catchment analyses of potential changes in runoff and sediment discharge* which outlines a process for evaluating potential change to stream channels resulting from watershed-scale changes in runoff and sediment yield.
2. *Field manual for assessing channel susceptibility* which describes an in-the-field assessment procedure that can be used to evaluate the relative susceptibility of channel reaches to deepening and widening.
3. *Technical basis for development of a regionally calibrated probabilistic channel susceptibility assessment* which provides technical details, analysis, and a summary of field data to support the field-based assessment described in the field manual.

The catchment analyses and the field manual are designed to support each other by assessing channel susceptibility at different scales and in different ways. The GIS-based catchment analyses document is a planning tool that describes a process to predict likely effects of hydromodification based on potential change in water and sediment discharge as a consequence of planned or potential landscape alteration (e.g., urbanization). Data on geology, hillslope, and land cover are compiled for each watershed of interest, overlaid onto background maps, grouped into several discrete categories, and classified independently across the watershed in question.

The classifications are used to generate a series of Geomorphic Landscape Units (GLUs) at a resolution defined by the coarsest of the three data sets (usually 10 to 30 m). Three factors: geology, hillslope, and land cover are used because the data are readily available; these factors are important to controlling sediment yield. The factors are combined into categories of High, Medium, or Low relative sediment production. The current science of sediment yield estimation is not sophisticated enough to allow fully remote (desktop) assignment of these categories. Therefore initial ratings must be verified in the field.

Once the levels of relative sediment production (i.e., Low, Medium, and High) are defined across a watershed under its current configuration of land use, those areas subject to future development are identified, and corresponding sediment-production levels are determined by substituting Developed land cover for the original categories and modifying the relative sediment production as necessary (Figure ES2). Conversely, relative sediment production for currently developed watershed areas can be altered to estimate relict sediment production for an undeveloped land use and used to assess the impact of watershed development on pre-development sediment production. The resultant maps can be used to aid in planning decisions by indicating areas where changes in land use will likely have the largest (or smallest) effect on sediment yield to receiving channels.

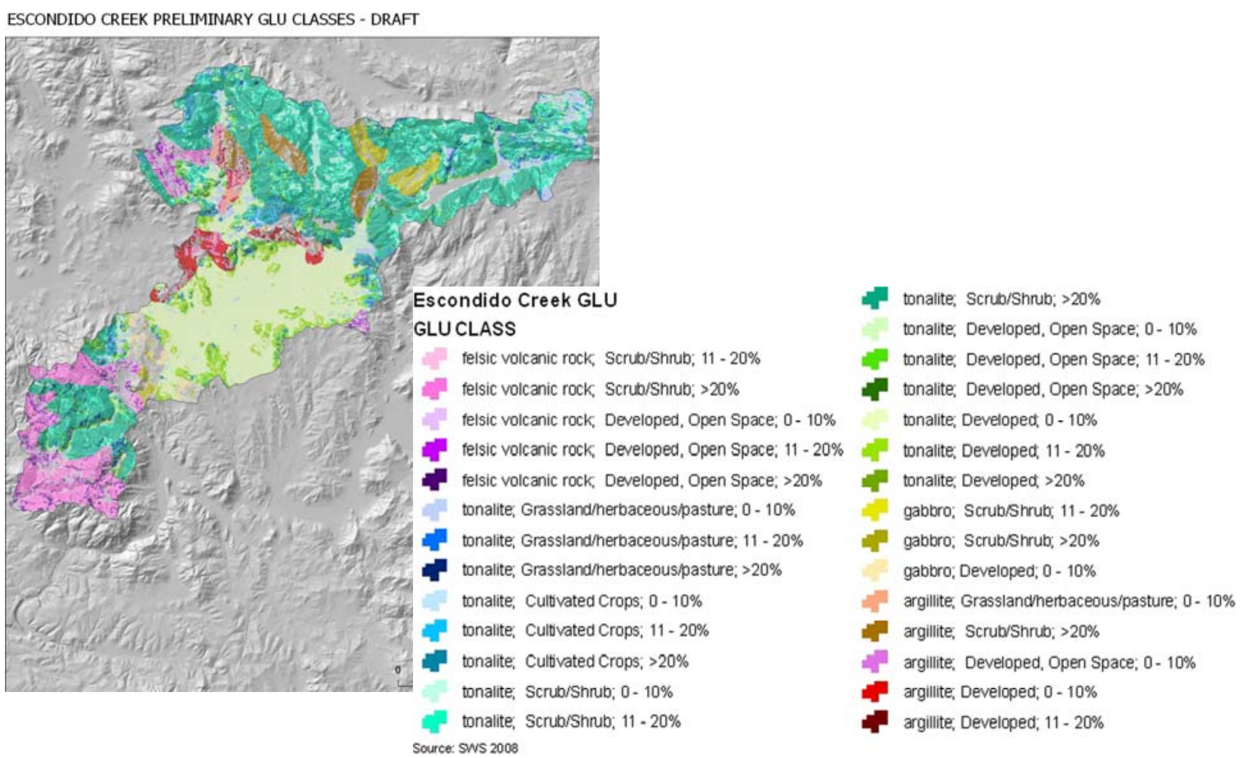

Figure ES2: Example of Geomorphic Landscape Units for the Escondido Creek Watershed.

The field assessment procedure is intended to provide a rapid assessment of the relative susceptibility of a specific stream reach to effects of hydromodification. The intrinsic sensitivity of a channel system to hydromodification as determined by the ratio of disturbing to resisting forces, proximity to thresholds of concern, probable rates of response and recovery, and potential for spatial propagation of impacts. A combination of relatively simple, but quantitative, field indicators are used as input parameters for a set of decision trees. The decision trees follow a logical progression and allow users to assign a classification of Low, Medium, High, or Very High susceptibility rating to the reach being assessed. Ratings based on likely response in the vertical and lateral directions (i.e., channel deepening and widening) are assigned separately. The screening rating foreshadows the level of data collection, modeling, and ultimate mitigation efforts that can be expected for a particular stream-segment type and geomorphic setting. The field assessment is novel in that it incorporates the following combination of features:

- Integrated field and office/desktop components
- Separate ratings for channel susceptibility in vertical and lateral dimensions
- Transparent flow of logic via decision trees
- Critical nodes in the decision trees are represented by a mix of probabilistic diagrams and checklists
- Process-based metrics selected after exhaustive literature review and analysis of large field dataset
- Metrics balance process fidelity, measurement simplicity, and intuitive interpretability
- Explicitly assesses proximity to geomorphic thresholds delineated using field data from small watersheds in southern California
- Avoids bankfull determination, channel cross-section survey, and sieve analysis, but requires pebble count in some instances
- Verified predictive accuracy of simplified logistic diagrams relative to more complex methods, such as dimensionless shear-stress analyses and Osman and Thorne (1988) geotechnical stability procedure
- Assesses bank susceptibility to mass wasting; field-calibrated logistic diagram of geotechnical stability vetted by Colin Thorne (personal communication)
- Regionally-calibrated braiding/incision threshold based on surrogates for stream power and boundary resistance
- Incorporates updated alternatives to the US Geological Survey (USGS; Waananen and Crippen 1977) regional equations for peak flow (Hawley and Bledsoe In Review)
- Does not rely on bank vegetation given uncertainty of assessing the future influence of root reinforcement (e.g., rooting depth/bank height)
- Channel evolution model underpinning the field procedure is based on observed responses in southern California using a modification of Schumm *et al*. (1984) five-stage model to represent alternative trajectories

The probabilistic models of braiding, incision, and bank instability risk embedded in the screening tools were calibrated with local data collected in an extensive field campaign. The models help users directly assess proximity to geomorphic thresholds and offer a framework for gauging susceptibility that goes beyond expert judgment. The screening analysis represents the first step toward determining appropriate management measures and should help inform decisions about subsequent more detailed analysis.

The GIS-based catchment-scale analysis and the field screening procedure are intended to be used as a set of tools to inform management decisions (Figure ES3). The catchment-scale analysis provides an overall assessment of likely changes in runoff and sediment discharge that can be used to support larger-scale land use planning decisions and can be applied prospectively or retrospectively. The field screening procedure provides more precise estimates of likely response of individual stream reaches based on direct observation of indicators. The field assessment procedure also provides a method to evaluate the extent of potential upstream and downstream propagation of effects (i.e., the analysis domain). In concept, the catchment-scale analysis would be completed for a watershed of interest before conducting the field analysis. However, this is not required and the two tools can be used independent of each other. It is not presently possible to describe a mechanistic linkage between the magnitude of the *drivers* of hydromodification (i.e., changes in the delivery of water and sediment to downstream channels), the *resistance* of channels to change, and the net expression on channel form. For this reason, the results of the catchment and field analyses must be conducted independently and the results cannot be combined to produce an overall evaluation of channel susceptibility to morphologic change (Figure ES3).

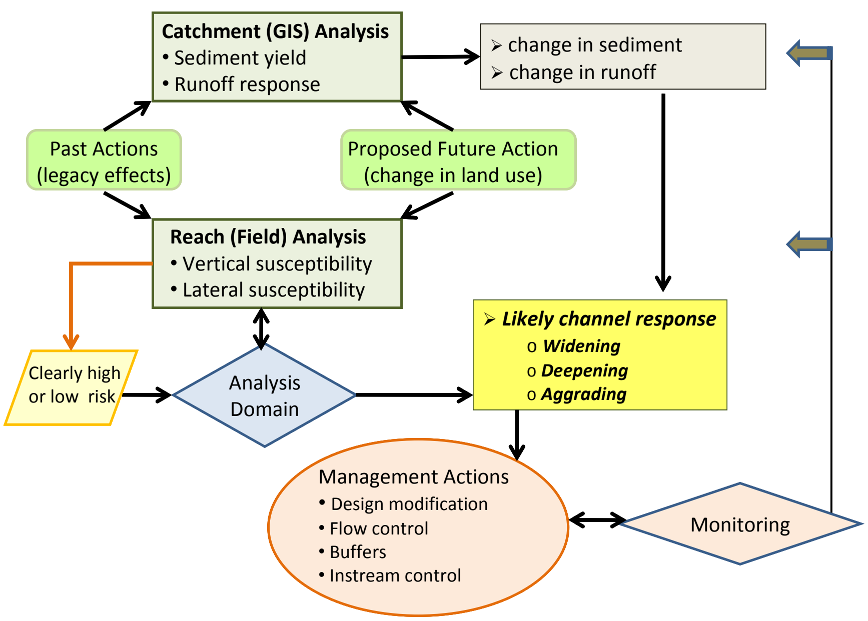

Figure ES3: Relationship of catchment and field screening tools to support decisions regarding susceptibility to effects of hydromodification.

Finally, it is important to note that these tools should be used as part of larger set of considerations in the decision making process (see Figure ES1). For example, the tools do not provide assessments of the ecological or economic affects of hydromodification. Similarly, they do not allow attribution of current conditions to past land use actions. Although the screening tool is designed to have management implications via a decision framework, policy/management decisions must be made by local stakeholders in light of a broader set of considerations.

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INTRODUCTION

Hydromodification, the response of streams to changes in flow and sediment input, is an area of active investigation and emerging regulation. Previous research that led to screening tool development has concluded that 1) urbanization markedly affects the flow regimes of streams in southern California, 2) the corresponding imbalances in sediment-transport capacity result in substantial geomorphic instabilities across most stream settings, 3) channels in southern California may be more sensitive than streams in other regions of the United States (US) for equivalent flows, bed-material sizes, valley slopes, and bank heights/angles, and 4) widely varying degrees of susceptibility to hydromodification are clearly reflected across the field study sites as an interaction between flow energy and the resistance of channel boundaries to lateral and vertical adjustments (Hawley 2009).

Many management schemes currently use a one-size-fits-all approach to managing hydromodification effects, whereby a single criterion is applied to all streams within a given area. However, factors such as dominant bed material, channel planform, grade control, vegetation, and existing infrastructure can influence the rate and manner in which streams respond to changes in flow and sediment. Consideration of these differences in management programs requires a tool to rate stream reaches in terms of their relative susceptibility to hydromodification effects.

This document provides the steps and process to apply a process-based hydromodification susceptibility screening tool. The tool builds on studies conducted in other regions, as summarized by Bledsoe *et al*. (2008), to provide a means to rank stream reaches in terms of their relative likelihood of response to hydromodification. The screening tool consists of two elements: 1) Geographic Information System (GIS) based landscape-scale analyses of relative runoff and sediment yield to stream channels, and 2) field-based assessment of channel condition. Together these two elements can be used to assess susceptibility of a specific stream reach based on both landscape and local influences (**Figure 1**). The GIS based analysis is intended mainly as a planning tool to allow potential changes in runoff and sediment yield to be considered during the siting and design of new developments. This tool is presented in the companion to this document. The field-based tool is intended to provide a rapid assessment of the relative susceptibility of a specific stream reach to effects of hydromodification. This tool is presented in this document.

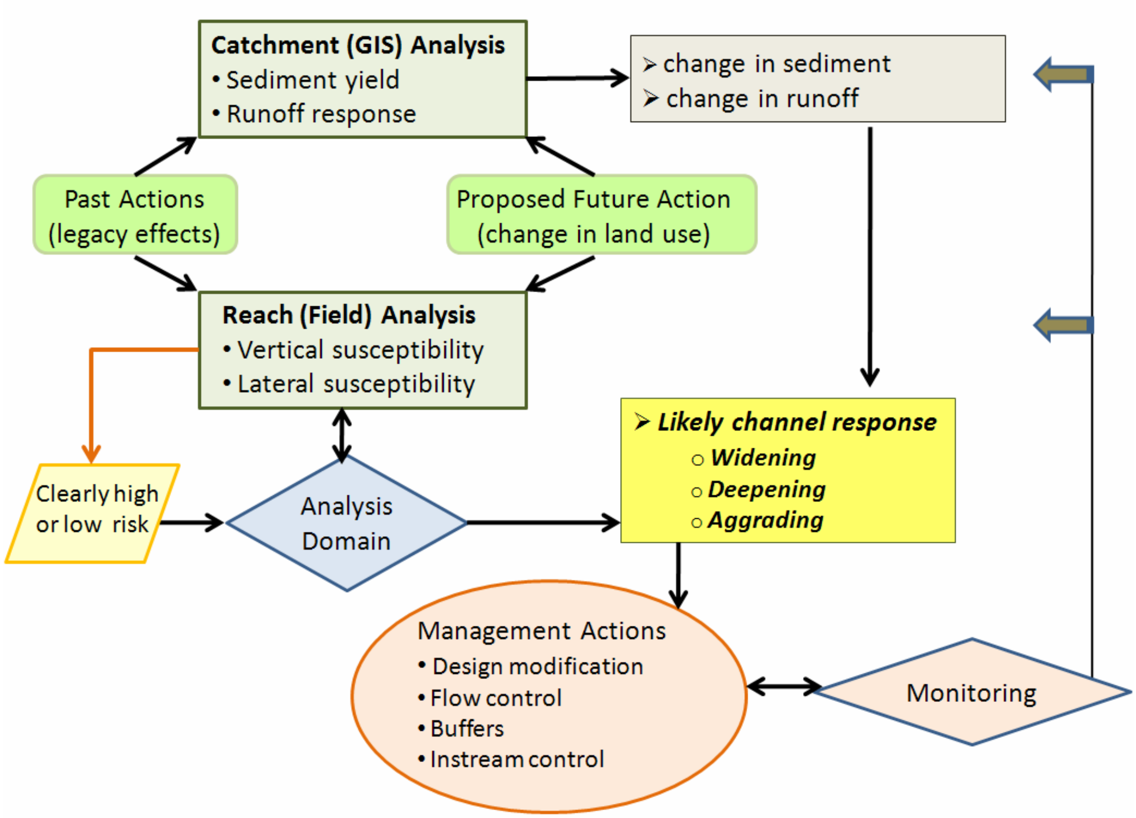

Figure 1. Conceptual application of GIS- and field-based screening tools.

General features of the field screening tool:

- Integrated field and office/desktop components
- Separate ratings for channel susceptibility in vertical and lateral dimensions
- Transparent flow of logic via decision trees
- Critical nodes in the decision trees are represented by a mix of probabilistic diagrams and checklists
- Process-based metrics selected after exhaustive literature review and analysis of large field dataset
- Metrics balance process fidelity, measurement simplicity, and intuitive interpretability
- Explicitly assesses proximity to geomorphic thresholds delineated using field data from small watersheds in southern California
- Avoids bankfull determination, channel cross-section survey, and sieve analysis, but requires pebble count in some instances

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- Does not rely on bank vegetation given uncertainty of assessing the future influence of root reinforcement (e.g., rooting depth/bank height)
- Channel evolution model (CEM) underpinning the field procedure is based on observed responses in southern California using a modification of Schumm *et al*. (1984) five-stage model to represent alternative trajectories

The Field Screening Tool <u>DOES NOT</u>:

⊗ **Make policy/management decisions**: although the screening tool is designed to have management implications via a decision framework, policy/management decisions must be made by local stakeholders

⊗ **Incorporate ecological/economic considerations**: the screening tool is exclusively focused on geomorphic stability and does not include ecological/economic aspects that stakeholders may consider

⊗ **Assess historical attribution**: the screening tool is designed to assess the current susceptibility of a channel, independent of attributing degraded conditions to historical land users, and policies.

OFFICE AND FIELD COMPONENTS FOR FIELD SCREENING TOOL

Office Components

The screening tool presented in this report is predominantly designed for field-based assessment. The field tool requires some preparatory office work to provide context and familiarity with the site prior to conducting the field evaluation. The following addresses:

- Examination of Overall Setting (using Google Earth or equivalent aerials)
- Quantification of important remotely-sensed parameters (using GIS software)
- Identification of Analysis Domain (tentatively defining upstream and downstream extents of field reconnaissance, locations of likely grade control, and valley transitions)

Overall Setting

Using satellite imagery/aerial photography, gather a baseline understanding of the watershed. Consider aspects such as development extent, fires and vegetation coverage, sediment sources and bottlenecks, ecologically-sensitive areas, etc. Examine the valley setting near the project in greater detail, identifying tributary confluences, potential grade control (e.g., road crossings), and infrastructure (e.g., stormwater outfalls, drainage 'improvements', etc.) *sensu* Chin and Gregory (2005). Specifically consider:

- Geologic setting, basin type, valley context, and tributaries
- Recent watershed history – urbanization and fire
- Obvious grade-control locations, human influences, and existing infrastructure

Printed screen shots of aerials, specifically near the project site, may be helpful in the field. In addition, the results from the GIS-based assessment (if completed) should be reviewed prior to beginning the field assessment.

GIS Metrics

Using publicly available GIS data, measure four readily quantifiable watershed- and valley-scale variables that will be used to compute the simple, but statistically-significant, screening indices (i.e., flow, screening index, and valley width index). Measurement details in Form 1 (**Figure 2**).

- Spatial: contributing drainage area
- Topographic: valley slope at site(s)
- Precipitation: mean annual area-weighted precipitation
- Geomorphic Confinement: valley bottom width at site(s)

These variables are explained in more detail in Form 1 Table 1 (**Figure 2**). A digital data entry form is available as well (Data Entry Form.xls).

Analysis Domain

The effects of hydromodification may propagate for significant distances downstream (and sometimes upstream) from a point of impact such as a stormwater outfall. Accordingly, it may be necessary to conduct geomorphic screening reconnaissance across a domain spanning multiple channel types/settings and property owners.

The maximum spatial unit for assigning a susceptibility rating is defined as a *ca.* 20 channel width reach not to exceed 200 m. Before conducting the field screening, the analyst should identify the following attributes as part of the office analysis to estimate the maximum extent of the analysis domain for field refinement.

Begin by defining the points or zones along the channel reach(es) where changes in discharge or channel type are likely to occur (e.g., potential locations of outfalls or tributary inputs). Document any observed outfalls for final desktop synthesis and define the upstream and downstream extents of analysis as follows:

- **Downstream** – until reaching the closest of the following:
    - at least one reach downstream of the first grade-control point (but preferably the second downstream grade-control location)
    - tidal backwater/lentic waterbody
    - equal order tributary (Strahler 1952)[1]
    - a 2-fold increase in drainage area[2]

OR demonstrate sufficient flow attenuation through existing hydrologic modeling

- **Upstream** – extend the domain upstream for a distance equal to 20 channel widths OR to grade control in good condition – whichever comes first. Within that reach, identify hard points that could check headward migration, evidence that head cutting is active or could propagate unchecked upstream

Within the analysis domain there may be several reaches that should be assessed independently based on either length or change in physical characteristics. In more urban settings, segments may be logically divided by road crossings (Chin and Gregory 2005), which may offer grade control, cause discontinuities in the conveyance of water or sediment, etc. In more rural settings, changes in valley/channel type, natural hard points, and tributary confluences may be more appropriate for delineating assessment reaches. In general, the following criteria should trigger delineation of a new reach and hence a separate susceptibility assessment:

- 200 m or ca. 20 bankfull widths – it is difficult to integrate over longer distances
- Distinct or abrupt change in grade or slope due to either natural or artificial features
- Distinct or abrupt change in dominant bed material or sediment conveyance
- Distinct or abrupt change in valley setting or confinement
- Distinct or abrupt change in channel type, bed form, or planform

[1] In the absence of proximate downstream grade control or backwater, the confluence of an 'equal order tributary' should correspond to substantial increases in flow and channel capacity that should, in theory, correspond to significant flow attenuation; however, there is no scientific basis to assume that downstream channels of higher stream order are less susceptible than their upstream counterparts. This (practically-driven) guidance should not supersede the consideration of local conditions and sound judgment. Stakeholders may elect to use a more regionally-preferred guidance.

[2] An increase in drainage area greater than or equal to 100% would roughly correspond to the addition of an equal-order tributary

FORM 1: INITIAL DESKTOP ANALYSIS

Complete all shaded sections.

IF required at multiple locations, circle one of the following site types:

Applicant Site / Upstream Extent / Downstream Extent

Location: Latitude: ____________ Longitude: ____________

Description (river name, crossing streets, etc.): ____________

GIS Parameters: The International System of Units (SI) is used throughout the assessment as the field standard and for consistency with the broader scientific community. However, as the singular exception, US Customary units are used for contributing drainage area (A) and mean annual precipitation (P) to apply regional flow equations after the USGS. See SCCWRP Technical Report 607 for example measurements and "Screening Tool Data Entry.xls" for automated calculations.

Form 1 Table 1. Initial desktop analysis in GIS.

	Symbol	Variable	Description and Source	Value
Watershed properties (English units)	A	Area (mi^2)	Contributing drainage area to screening location via published Hydrologic Unit Codes (HUCs) and/or ≤ 30 m National Elevation Data (NED), USGS seamless server	
	P	Mean annual precipitation (in)	Area-weighted annual precipitation via USGS delineated polygons using records from 1900 to 1960 (which was more significant in hydrologic models than polygons delineated from shorter record lengths)	
Site properties (SI units)	S_v	Valley slope (m/m)	Valley slope at site via NED, measured over a relatively homogenous valley segment as dictated by hillslope configuration, tributary confluences, etc., over a distance of up to ~500 m or 10% of the main-channel length from site to drainage divide	
	W_v	Valley width (m)	Valley bottom width at site between natural valley walls as dictated by clear breaks in hillslope on NED raster, irrespective of potential armoring from floodplain encroachment, levees, etc. (imprecise measurements have negligible effect on rating in wide valleys where VWI is >> 2, as defined in lateral decision tree)	

Form 1 Table 2. Simplified peak flow, screening index, and valley width index. Values for this table should be calculated in the sequence shown in this table, using values from Form 1 Table 1.

Symbol	Dependent Variable	Equation	Required Units	Value
Q_{10cfs}	10-yr peak flow (ft^3/s)	$Q_{10cfs} = 18.2 * A^{0.87} * P^{0.77}$	A (mi^2) P (in)	
Q_{10}	10-yr peak flow (m^3/s)	$Q_{10} = 0.0283 * Q_{10cfs}$	Q_{10cfs} (ft^3/s)	
INDEX	10-yr screening index ($m^{1.5}/s^{0.5}$)	$INDEX = S_v * Q_{10}^{0.5}$	Sv (m/m) Q_{10} (m^3/s)	
W_{ref}	Reference width (m)	$W_{ref} = 6.99 * Q_{10}^{0.438}$	Q_{10} (m^3/s)	
VWI	Valley width index (m/m)	$VWI = W_v/W_{ref}$	W_v (m) W_{ref} (m)	

(Sheet 1 of 1)

Figure 2. Form 1: Initial Desktop Analysis. Complete set of assessment forms in Appendix B.

Conceptual Basis for 10-yr Flow Analysis

The geomorphic thresholds presented in the field-screening sections below correspond to the 10-yr peak flow calculated using the regional hydrologic model presented in Form 1 Table 2 (**Figure 2**; Hawley and Bledsoe, In review). This peak flow model is substantially more accurate for small watersheds in southern California than previously published regional regression equations. The 10-yr flow was selected for several reasons. First, it better represents a channel-filling flow than alternative return intervals such as Q_2. Second, it typically requires a 10-yr instantaneous peak flow to create a geomorphically significant *duration* at the 2-yr flow magnitude (i.e., the 10-yr instantaneous peak flow typically corresponds to a daily-mean flow equal to a 2- to 3-yr peak magnitude). Finally, the 10-yr hydrologic models had the best prediction accuracy of all return intervals. Out of 5 peak-flow model forms (Hawley and Bledsoe, In review), the model based on drainage area and precipitation had the best cross-validation performance. With respect to modeling Q_{10}, the standard error as percentage of mean for validation samples was 41% (arithmetic space), with an R^2 during final calibration of 0.81 (geometric space). Because of the relatively-robust model performance and overall simplicity, we selected the model form of $Q = f\,(A, P)$ for use in this screening tool.

Field Components

After completing the Initial Desktop Analysis (**Figure 2**), the user should have a first-order estimate of an appropriate analysis domain, a baseline understanding of the watershed, and critical indices to use during the field assessment(s). At this juncture it is essential to examine the stream (and its valley setting) in greater detail. Minimally, the following items should be taken to the field, although Form 2: Pebble Count (**Figure 4**) is not needed in every case:

- Assessment forms and/or field book for sketches/notes
- Digital camera for photographic documentation
- Pocket rod and/or tape for some basic measurements and reference/scale in photographs
- Protractor (e.g., gravity-driven) for measuring bank angle (**Figure 3a**)
- Gravelometer (i.e., US SAH-97 half-phi template) for standardized pebble count (**Figure 3b**)

a) b)

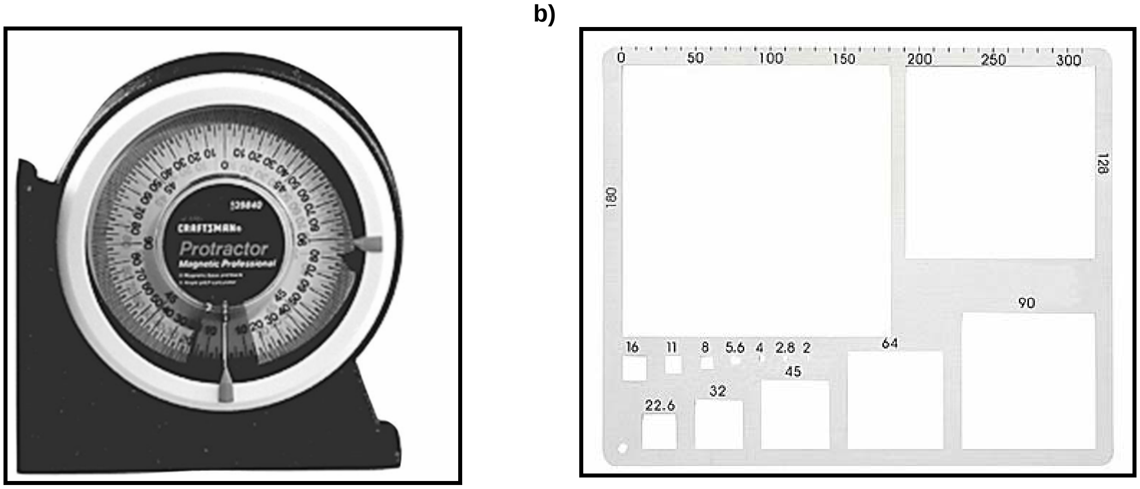


Figure 3. Craftsman magnetic protractor (a) and US SAH-97 half-phi template gravelometer (b).

FORM 2: PEBBLE COUNT

If it is necessary to estimate d_{50}, perform a pebble count, after Bunte and Abt (2001a,b), using a minimum of 100 particles and a standard half-phi template, or by measuring along the intermediate axis of each pebble. Use a grid and tape for equally spaced samples over systematic/complete transects across riffle sections (i.e., if the 100th particle is in the middle of a transect, complete the full transect before stopping the count; if more than 125 particles, record data near the bottom of Sheet 2 of 2). If the source of fines (sand/silt d <2 mm; see Form 2 Table 2 below) is less than ½ inch thick (approximately one finger width) at the sampling point, sample the coarser buried substrate; otherwise record observation of fines. Take photographs to support observations (Detailed instructions in Appendix A.3).

Form 2 Table 1. 100-pebble count tabulation for Vertical Susceptibility. Record station (Sta) and diameter (d) in millimeters.

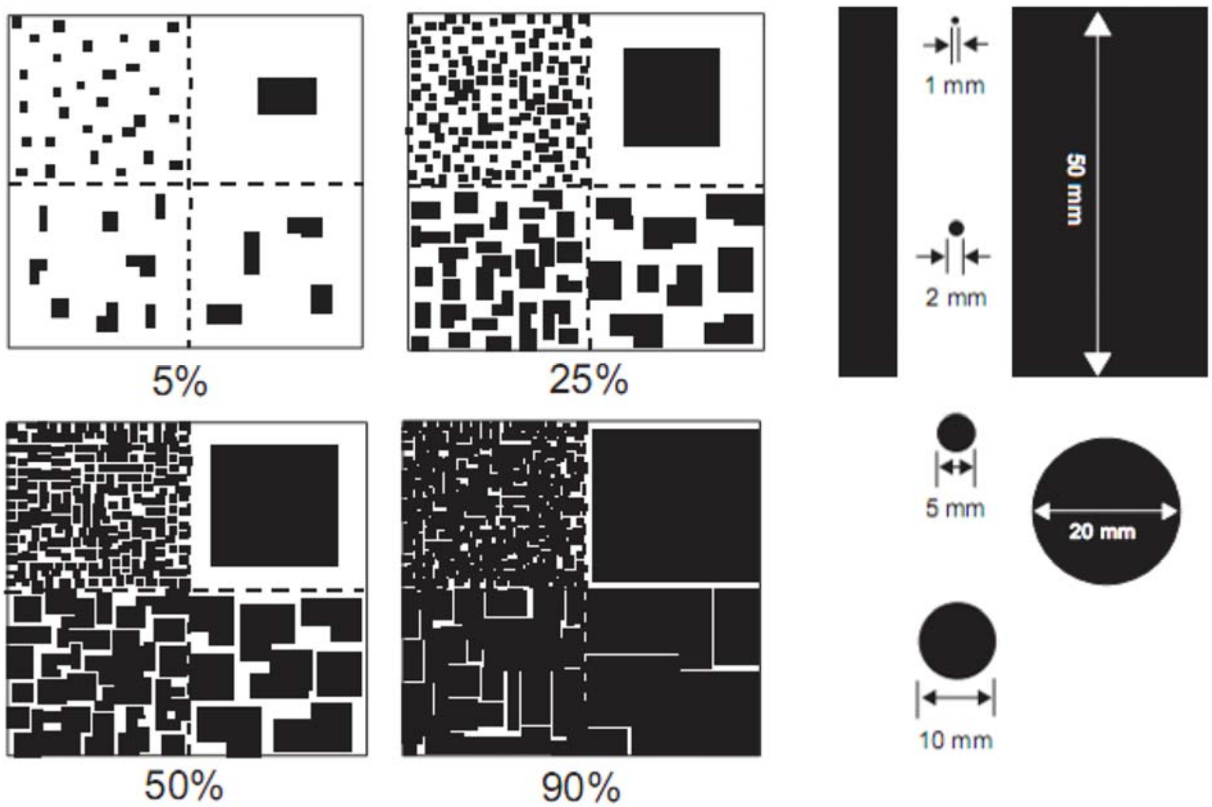
#	Sta	d (mm)	#	Sta	d (mm)	#	Sta	d (mm)	#	Sta	d (mm)	#	Sta	d (mm)
1			26			51			76			101		
2			27			52			77			102		
3			28			53			78			103		
4			29			54			79			104		
5			30			55			80			105		
6			31			56			81			106		
7			32			57			82			107		
8			33			58			83			108		
9			34			59			84			109		
10			35			60			85			110		
11			36			61			86			111		
12			37			62			87			112		
13			38			63			88			113		
14			39			64			89			114		
15			40			65			90			115		
16			41			66			91			116		
17			42			67			92			117		
18			44			68			93			118		
19			44			69			94			119		
20			45			70			95			120		
21			46			71			96			121		
22			47			72			97			122		
23			48			73			98			123		
24			49			74			99			124		
25			50			75			100			125		

Form 2 Table 2. d_{50} for Screening Index Threshold.

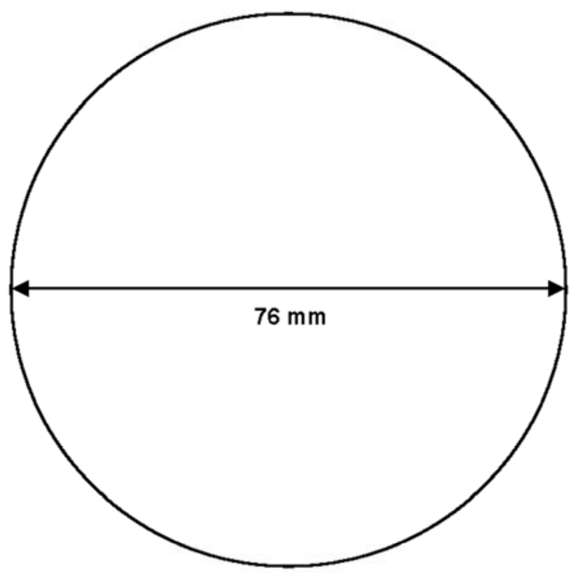
Class Name	Diameter (mm)	Helpful Descriptions for Field Identification
Boulder	> 256	Difficult to lift by hand
Cobble	> 64	Typically able to lift
Gravel	> 2	Fits in one hand
Sand	> 0.0625	Can feel between fingers
Silt	> 0.004	Can feel with tongue
Clay	≤ 0.004	Can not feel individual particle

(Sheet 1 of 2)

Figure 4. Form 2: Pebble Count. Complete set of assessment forms in Appendix B.



Note: Each quadrant within each box contains the same total area covered using different sized objects.



Form 2 Figure 1. Examples of % coverage by volume and substrate sizing adapted from *NRCS Field Book for Describing and Sampling Soils* (Schoeneberger *et al.* 2002) and Julien (1998).

(Sheet 2 of 2)

Figure 4. Continued

Susceptibility Rating Definitions

The field screening tool uses a combination of relatively simple, but quantitative, field indicators as input parameters to a set of decision trees. The decision trees follow a logical progression and allow users to assign a classification of Low, Medium, High, or Very High susceptibility rating (Table 1) to the reach being assessed.

Table 1. Vertical and Lateral Susceptibility rating definitions.

Susceptibility Rating	Definitions of Susceptibility
LOW	• Low ratio of disturbing forces to resisting forces
	• Far from geomorphic thresholds of concern (based on explicit quantification of probability if feasible – < 1% probability of exceedance)
	• Relatively rapid relaxation time
	• Low potential for positive feedbacks, nonlinear response, sensitivity to initial conditions
	• Very limited or no spatial propagation (*ca.* 10 m)
MEDIUM	• Moderate ratio of disturbing forces to resisting forces
	• Not proximate to geomorphic thresholds of concern (based on explicit quantification of probability if feasible – e.g., < 10% probability of exceedance)
	• Moderately rapid relaxation time
	• Low to moderate potential for positive feedbacks, nonlinear response, sensitivity to initial conditions
	• Local spatial propagation, contained within *ca.* 100 m
HIGH	• High ratio of disturbing forces to resisting forces
	• Proximate to geomorphic thresholds of concern (based on explicit quantification of probability if feasible – e.g., > 10 to 50% probability of exceedance)
	• Relaxation time may be relatively long given magnitude and spatial extent of change
	• Moderate to high potential for positive feedbacks, nonlinear response, sensitivity to initial conditions
	• Potential spatial propagation – headcutting/base-level change upstream and downstream but contained within *ca.* 100 to 1,000 m domain of control
VERY HIGH	• High ratio of disturbing forces to resisting forces
	• At geomorphic thresholds of concern (based on explicit quantification of probability if feasible – e.g., ≥ 50% probability of exceedance)
	• Relaxation time may be relatively long given magnitude and spatial extent of change
	• High potential for positive feedbacks, nonlinear response, sensitivity to initial conditions
	• Potential widespread spatial propagation – headcutting/base-level change upstream and downstream uncontained within *ca.* 1,000 m domain of control
	• Specifically, the VERY HIGH rating is reserved for the following geomorphic thresholds/states (clear and present danger): o **Vertical** ▪ Currently unstable (Channel Evolution Model (CEM) Type III or IV) with incision past critical bank height for mass wasting and active bank failure ▪ Currently stable (CEM Type I or II) with banks less than critical height, but p ≥ 50% for incision or braiding in labile bed (d_{50} <16 mm) with ineffective/absent grade control o **Lateral** ▪ Currently unstable with active braiding/extensive mass wasting/fluvial erosion (> 50% of banks) in a wide valley ▪ Currently stable consolidated bank in wide valley with High Vertical rating combined with p > 10% for mass wasting ▪ Currently stable unconsolidated banks with fine toe material in wide valley with High Vertical rating

Recall that it may be necessary to perform the field assessment at several locations based on an analysis domain that could span multiple stream reaches up and downstream (see Analysis Domain above). At each distinct reach type, the user will follow the guidelines below to separately assess susceptibility in vertical and lateral dimensions. Although vertical and lateral responses are often interdependent, vertical and lateral susceptibility are assessed separately for several reasons. First, vertical and lateral responses are primarily controlled by different types of resistance, which, when assessed separately, may improve ease of use and lead to increased repeatability among users compared to an integrated, cross-dimensional assessment. Second, the mechanistic differences between vertical and lateral responses point to different modeling tools and potentially different management strategies. Having separate screening ratings may better direct users and managers to the most appropriate tools for subsequent analyses.

The field screening tool uses combinations of decision trees, checklists, tables and calculations. We attempt to employ decision trees when a question can be answered fairly definitively and/or quantitatively (e.g., median grain diameter (d_{50}) < 16 mm; see Form 2 (**Figure 4**)). Alternatively, checklists are used in places where answers are relatively qualitative (e.g., grade control).

The tool is designed to first classify the current state of the assessment area. Next, the user identifies the type and number of risk factors that are present; risk factors are then combined with current state to determine a final rating. Users should take photographs to support their assessment. If uncertain about a given decision node, the user should use the more precautionary pathway that results in a higher rating of susceptibility. The field-assessment process is described in detail below:

- Decision Trees
    - Vertical Susceptibility
    - Lateral Susceptibility

- Design/Setup
    - Assess the Analysis Domain (defined above), which may include multiple stream types and settings; conduct separate analyses for reaches distinguished by distance, change in valley type, dominant bed material, and other significant geomorphic considerations
    - Assign susceptibility ratings of Low, Medium, High, and Very High (as defined in Table 1 above) independently to the vertical and lateral conditions of each channel reach

- Consult susceptibility decision trees and photographic supplements for rating guidance; to clearly highlight rating endpoints within the decision trees, non-terminal and terminal nodes in the decision trees have been color coded (**Figure 5**) to prompt users to proceed to another step

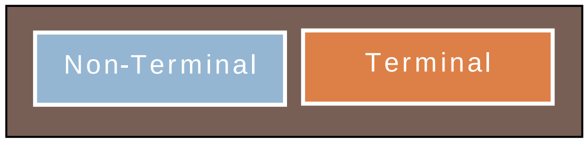


Figure 5. Color scheme for non-terminal and terminal nodes in susceptibility decision trees.

- Overall logic of susceptibility decision trees (**Figure 6**)

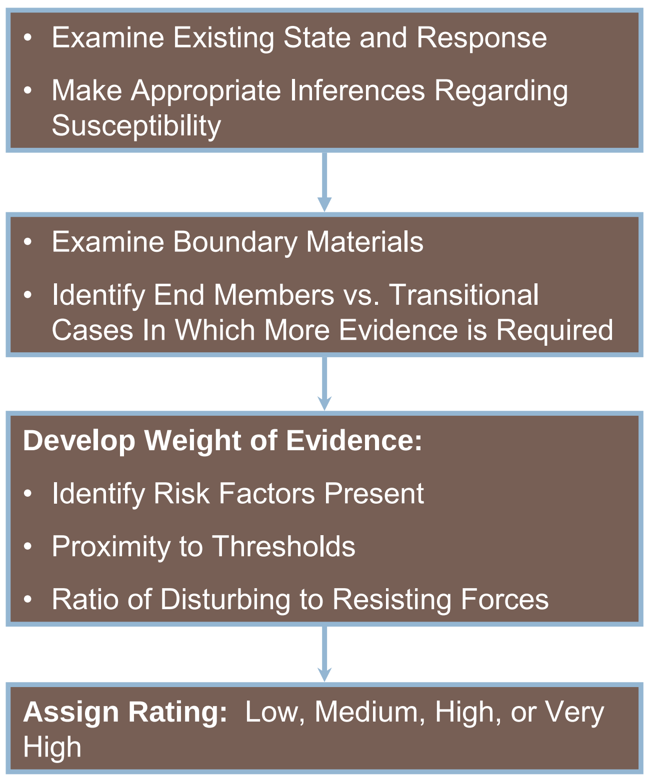


Figure 6. Logical flow of susceptibility decision trees.

CHANNEL SUSCEPTIBILITY DECISION TREES AND FORMS

Vertical Susceptibility

In the Vertical Susceptibility decision tree, there are three potential states of bed material based on broad classes of armoring potential. These states are listed below from most susceptible to least with definitions and photographic examples provided in Form 3 (**Figure 7**):

- Labile Bed – sand-dominated bed, little resistant substrate
- Transitional/Intermediate Bed – bed typically characterized by gravel/small cobble, intermediate level of resistance of the substrate and uncertain potential for armoring
- Threshold Bed (Coarse/Armored Bed) – armored with large cobbles or larger bed material or highly-resistant bed substrate (i.e., bedrock)

Threshold beds composed of boulders and large cobbles and/or highly-resistant bedrock are the region's most resistant channel beds with geologic grade control and a natural capacity to armor (see Form 3 (**Figure 7**)). Consequently, threshold beds correspond to a vertical rating of low. Conversely, labile beds have little to no capacity to self-armor and have a high probability of vertical adjustments in response to hydromodification. Depending on two additional decision tree questions that consider the current state of incision and grade control, labile beds receive a rating of High or Very High. Finally, transitional/intermediate beds are involved in a wide range of potential susceptibility responses and must be assessed in greater detail in order to develop weight of evidence for appropriate screening ratings. Three primary risk factors used to assess vertical susceptibility for channels with transitional/intermediate bed materials:

- Armoring Potential – Form 3 Checklist 1 (**Figure 7**)
- Grade Control – Form 2 Checklist 1 (**Figure 7**)
- Probability of Incision/Braiding based on a Regionally-Calibrated Screening Index – Form 3 Figure 1/Table 1 (**Figure 7**)

These risk factors are assessed using checklists and a diagram, then calculated using the instructions and equation at the bottom of Form 3 Sheet 4 of 4 (**Figure 7**) to provide an overall vertical susceptibility rating for the intermediate/transitional bed-material group.

Vertical Susceptibility Decision Tree

The purpose of the vertical susceptibility decision tree is to assess the state of the channel bed with a particular focus on the risk of incision (i.e., down cutting). Vertical stability is a prerequisite for lateral stability because a stream that incises can increase bank heights to the point of collapse and channel widening. Accordingly, vertical susceptibility is assessed first because it affects the lateral rating in most instances.

Conceptual Basis

Channel bed material is one of the main factors controlling vertical stability. Bed material is assessed using the photographic supplement Form 3 Figure 1 (**Figure 7**), with Form 2 Figure 1 (**Figure 4**) provided as a reference for some particle sizes and to assist with estimating the percentage of surface sand. Some reaches may require a pebble count, Form 2 (**Figure 4**), for a more definitive assessment of bed material size.

For threshold (coarse/armored) beds, document the channel substrate with photographs, and a supporting pebble count[3] if d_{50} is near 128 mm. For labile beds, use supplemental photographs in Form 3 Figure 1 (**Figure 7**) and the diagram of the five-stage CEM presented in Appendix A, Figure A.3, to assess the current state of channel incision. For intermediate/transitional beds, assess: armoring potential using Form 3 Checklist 1 (**Figure 7**), grade-control condition using Form 3 Checklist 1 (**Figure 7**), and risk of incision/braiding using Form 3 Table 1 (**Figure 7**).

Form 3 Checklist 1 (**Figure 7**): Armoring potential is assessed because it is a primary mechanism in which a channel can self-check channel incision/headcutting. Coarser particles naturally provide greater resistance and, therefore, yield a lower susceptibility rating. Additionally, the tighter the particles are packed, the more resistant the armor layer, which can also influence the rating. Finally, the amount of sand-sized particles can adversely affect the resistance of an armor layer (Wilcock and Kenworthy, 2002; Wilcock and Crowe, 2003).

Form 3 Checklist 2 (**Figure 7**): Grade control is another way in which incision/headcutting can be arrested. When channels adjust their slope, the incision typically hinges around a hard point such as a natural or artificial grade control. Grade control has been clearly demonstrated to be a statistically-significant predictor of channel enlargement in southern California (Hawley 2009). Adjustments may also revolve around channel base-level, which could be set by an estuary, large waterbody (such as a lake or reservoir), or confluence with a larger river.

Form 3 Figure 4 (**Figure 7**): Risk of incising or braiding is based on the potential specific stream power of the valley relative to d_{50}. Beyond armoring potential and grade control, channels with intermediate/transitional beds may also have a relatively-energetic valley setting that creates an inherently higher risk for incision than lower energy settings. The threshold is based on regional data from unconfined, unconstructed valley settings and modeled after similar analyses from various regions (e.g., Chang (1988), van den Berg (1995), and Bledsoe and Watson (2001)).

Hawley (2009) performed separate logistic regression analyses on incising and braiding systems relative to their stable, unconfined counterparts that returned similar thresholds. In developing this revised screening tool, we combined unstable states of braided or incising into one model for parsimony. Well over 100 total model variations were developed that segregated unstable (braided or incising) channels from stable, single-thread, unconfined, unconstructed channels, using different measures of erosive energy (i.e., dimensionless shear stress, specific stream power, and screening index) and different hydrologic models to estimate the 2- and 10-yr instantaneous peak flow events.

In addition, a large body of previous fluvial geomorphic research suggests that the behavior and response potential of coarse versus fine-grained systems is markedly different (e.g., Chang (1988), Montgomery and MacDonald (2002), and Simons and Simons (1987)). We assessed both combined and separated models, based on different grain-size discriminators between sand-

[3] If d_{50} is clearly greater than 128 mm, there is no need to conduct a pebble count, only visual documentation with photographs and general description of substrate type is recommended.

dominated gravels and gravel/cobble armored systems. Out of 108 total models, all but 6 were significant ($p < 0.05$) with the simplified specific stream power and grain-size surrogate(screening index) regularly performing similarly or superior to the more rigorous indices. Indeed, 5 of the 12 models of the screening index for coarse-size fractions offered complete segregation of unstable/stable sites (i.e., 100% correctly classified). Although that clearly delineates a threshold (Form 3 Table 1 (**Figure 7**)) it precludes using the logistic model to represent risk levels in terms of a range of probabilities. This explains why the 90% and 10% lines converge to the 50% risk level for $d_{50} > 16$ mm in Form 3 Figure 4 (**Figure 7**).

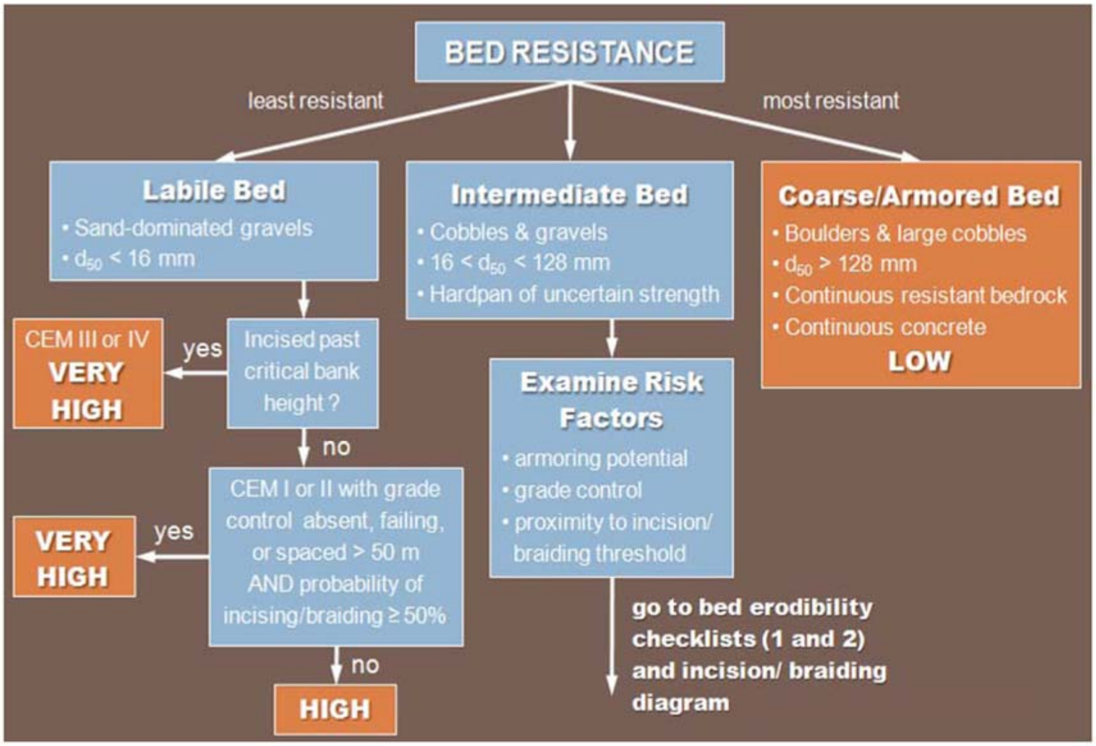
Vertical Flow and Forms

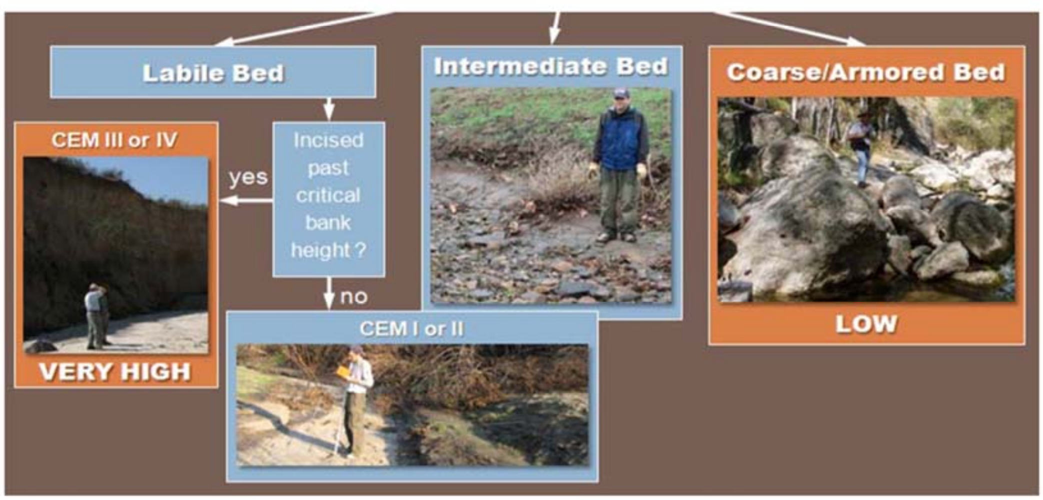
Forms 3: Vertical Susceptibility Field Sheet (**Figure 7**) is used to assess vertical susceptibility. The logical flow of this form is summarized through a series of decisions outlined below:

1) Assess the initial 'state': which of the following (a, b, or c) best describes the bed condition/material
    a. If the bed is Coarse/Armored with d_{50} >128 mm or continuous bedrock/concrete, then Vertical Rating = Low; see Form 3 Figure 1 (**Figure 7**)
    b. If the bed is labile with sand dominated gravels and d_{50} <16 mm, then assess level of incision:
        i. If channel is incised past critical bank height for mass wasting (CEM III or IV), Vertical Rating = Very High; see Form 3 Figure 1 (**Figure 7**) and Form 2 (**Figure 4**)
        ii. If channel is not incised past critical bank height (CEM I or II), assess Grade Control using Form 3 Checklist 2 (**Figure 7**) and Probability of Incision/Braiding using Form 3 Table 1 (**Figure 7**)
            1. If CEM I or II with grade control absent, failing, or spaced at intervals larger than 50 m, AND probability of incising/braiding ≥50%, Vertical Rating = Very High; see Form 3 Figure 1 (**Figure 7**)
            2. If CEM I or II with grade control in good condition and spaced at intervals less than 50 m, OR probability of incising/braiding < 50%, Vertical Rating = High; see Form 3 Figure 1 (**Figure 7**)
    c. If the bed is Intermediate with cobbles and gravels and $16 < d_{50} < 128$ mm or hardpan of uncertain strength, proceed to Form 3 Checklist 1 and 2 (**Figure 7**) to assess Armoring Potential and Grade Control, respectively, and Form 3 Figure 4 (**Figure 7**) to estimate Probability of Incising/Braiding.

FORM 3: VERTICAL SUSCEPTIBILITY FIELD SHEET

Circle appropriate nodes/pathway for proposed site.





Form 3 Figure 1. Vertical Susceptibility photographic supplement to be used in conjunction with Form 3 Bed Resistance above.

(Sheet 1 of 4)

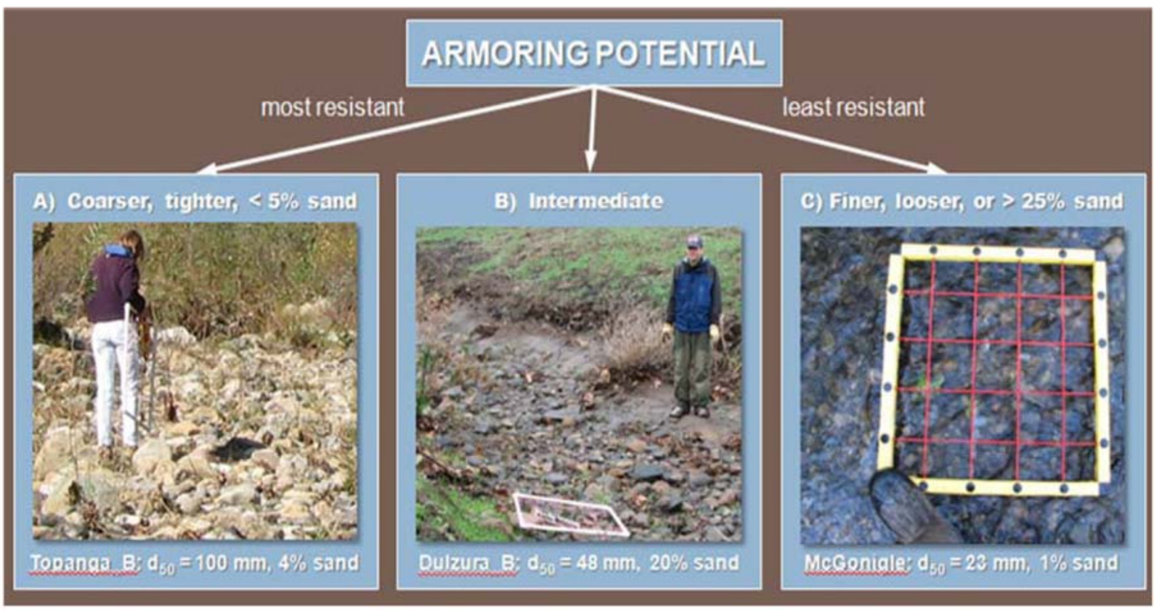
Figure 7. Form 3: Vertical Susceptibility Field Sheet. Complete set of assessment forms in Appendix B.

Form 3 Support Materials

Form 3 Checklists 1 and 2, along with information recording in Form 3 Table 1, are intended to support the decisions pathways illustrated in Form 3 Overall Vertical Rating for Intermediate/Transitional Bed.

Form 3 Checklist 1: Armoring Potential

☐	A	A mix of coarse gravels and cobbles that are tightly packed with <5% surface material of diameter <2 mm
☐	B	Intermediate to A and C or hardpan of unknown resistance, spatial extent (longitudinal and depth), or unknown armoring potential due to surface veneer covering gravel or coarser layer encountered with probe
☐	C	Gravels/cobbles that are loosely packed or >25% surface material of diameter <2 mm



Form 3 Figure 2. Armoring potential photographic supplement for assessing intermediate beds (16 < d_{50} < 128 mm) to be used i**n conjunction with Form 3 Checklist 1.**

(Sheet 2 of 4)

Figure 7. Continued

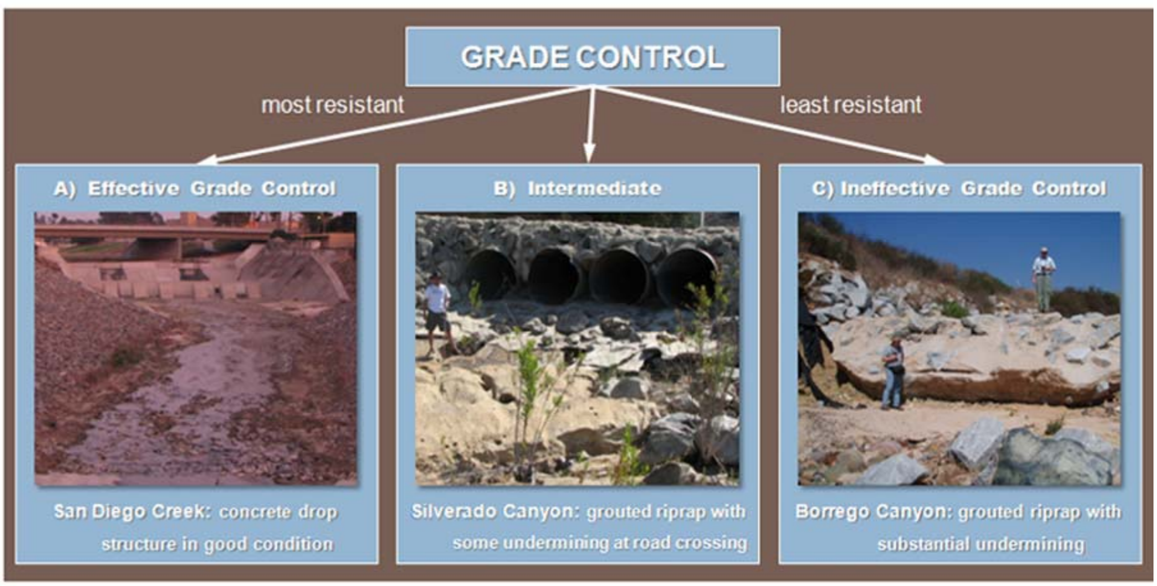
Form 3 Checklist 2: Grade Control

☐ A Grade control is present with spacing <50 m or $2/S_v$ m

- No evidence of failure/ineffectiveness, e.g., no headcutting (>30 cm), no active mass wasting (analyst cannot say grade control sufficient if mass-wasting checklist indicates presence of bank failure), no exposed bridge pilings, no culverts/structures undermined
- Hard points in serviceable condition at decadal time scale, e.g., no apparent undermining, flanking, failing grout
- If geologic grade control, rock should be resistant igneous and/or metamorphic; For sedimentary/hardpan to be classified as 'grade control', it should be of demonstrable strength as indicated by field testing such as hammer test/borings and/or inspected by appropriate stakeholder

☐ B Intermediate to A and C – artificial or geologic grade control present but spaced 2/Sv m to 4/Sv m or potential evidence of failure or hardpan of uncertain resistance

☐ C Grade control absent, spaced >100 m or $>4/S_v$ m, or clear evidence of ineffectiveness



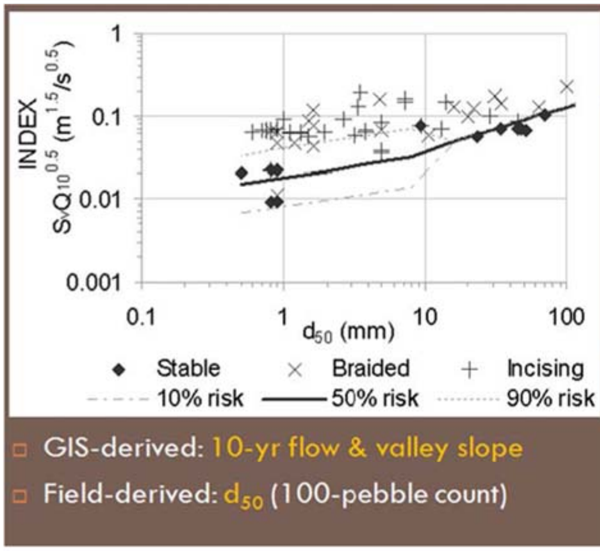
Form 3 Figure 3. Grade-control (condition) photographic supplement for assessing intermediate beds (16 < d_{50} < 128 mm) to be used in conjunction with Form 3 Checklist 2.

(Sheet 3 of 4)

Figure 7. Continued

Regionally-calibrated Screening Index Threshold for Incising/Braiding

For transitional bed channels (d_{50} between 16 and 128 mm) or labile beds (channel not incised past critical bank height), use Form 3 Figure 3 to determine Screening Index Score and complete Form 3 Table 1.



Model Type	d_{50} (mm)	50% Risk $S_vQ_{10}^{0.5}$ ($m^{1.5}/s^{0.5}$)
Logistic Regression $d_{50} \geq 16$ mm	128	0.145
	96	0.125
	80	0.114
	64	0.101
	48	0.087
	32	0.070
	16	0.049
Logistic Reg. $d_{50} < 16$ mm	8	0.031
	4	0.026
	2	0.022
	1	0.018
	0.5	0.015

Form 3 Figure 4. Probability of incising/braiding based on logistic regression of Screening Index and d_{50} to be used in conjunction with Form 3 Table 1.

Form 3 Table 1. Values for Screening Index Threshold (probability of incising/braiding) to be used in conjunction with Form 3 Figure 4 (above) to complete Form 3 Overall Vertical Rating for Intermediate/Transitional Bed (below).. Screening Index Score: A = <50% probability of incision for current Q_{10}, valley slope, and d_{50}; B = Hardpan/d_{50} indeterminate; and C = $\geq$50% probability of incising/braiding for current Q_{10}, valley slope, and d_{50}.

d_{50} (mm) *From Form 2*	**$S_v{*}Q_{10}^{0.5}$ ($m^{1.5}/s^{0.5}$)** *From Form 1*	**$S_v{*}Q_{10}^{0.5}$ ($m^{1.5}/s^{0.5}$)** *50% risk of incising/braiding from table in Form 3 Figure 3 above*	**Screening Index Score (A, B, C)**

Overall Vertical Rating for Intermediate/Transitional Bed

Calculate the overall Vertical Rating for Transitional Bed channels using the formula below. Numeric values for responses to Form 3 Checklists and Table 1 as follows: A = 3, B = 6, C = 9.

$$Vertical\ Rating = \sqrt{\{(\sqrt{armoring * grade\ control}) * screening\ index\ score\}}$$

Vertical Susceptibility based on Vertical Rating: <4.5 = LOW; 4.5 to 7 = MEDIUM; and >7 = HIGH.

(Sheet 4 of 4)

Figure 7. Continued

Lateral Susceptibility

In terms of lateral stability, there are five primary states of bank characteristics. These states are listed below, roughly in order of most susceptible to least:

- Mass wasting or fluvial erosion/braiding existing and extensive
- Poorly consolidated or unconsolidated with fine/nonresistant toe material
- Poorly consolidated or unconsolidated with coarse/resistant toe material
- Consolidated
- Fully-armored bedrock/engineered reinforcement or fully confined by hillslope

In addition to the present channel state/response and bank materials, there are three primary risk factors used to develop a weight of evidence for lateral susceptibility:

- Valley width index (VWI) from Form 1 (**Figure 2**): a measure of valley bottom width versus reference channel width (calculated in the office) used to assess the potential for lateral movement of the channel; see Forms 4 and 5 (**Figures 12 and 13**, respectively)
- Proximity to a regionally-calibrated bank stability threshold: geotechnical probability diagram based on bank height and angle; see Form 6 (**Figure 14**)
- The Vertical Susceptibility Rating: from Form 3 Sheet 4 of 4 (**Figure 7**)

Lateral Susceptibility Decision Tree

The purpose of the lateral decision tree is to assess the state of the channel banks with a particular focus on the risk of widening. Channels can widen from either bank failure or through fluvial processes such as chute cutoffs, avulsions, and braiding (see Figure A.2 in Appendix A). Widening through fluvial avulsions/active braiding is a relatively straightforward observation. If braiding is not already occurring, the next logical question is to assess the condition of the banks. Banks fail through a variety of mechanisms (see Figures A.4a and A.4b in Appendix A); however, one of the most important distinctions is whether they fail in mass (as many particles) or by fluvial detachment of individual particles. Although much research is dedicated to the combined effects of weakening, fluvial erosion, and mass failure (Beatty 1984, Hooke 1979, Lawler 1992, Thorne 1982), we found it valuable to segregate bank types based on the inference of the dominant failure mechanism (as the management approach may vary based on the dominant failure mechanism).

Conceptual Basis

Cohesive banks have been documented in both flume and field experiments as being much more resistant to fluvial entrainment than non-cohesive banks (Thorne 1982). Despite the fact that most of the banks that observed in southern California had relatively low amounts of cohesion when compared to other US regions, it is generally acknowledged that truly non-cohesive banks are rare in nature given the effective cohesion introduced by pore-water suction even in banks formed in coarse materials (Lawler *et al*. 1997). Furthermore, there was clear evidence of mass wasting at a large number of sites, including the presence of tension cracks and discrete failure surfaces deep within the banks exhibiting corresponding planar, slab, and rotational failures.

Because cohesivity is difficult to assess in the field, Hawley (2009) segregated banks by relative degree of consolidation. Failure in banks composed of recently deposited alluvium with little time to consolidate (i.e., <~10 yrs, unconsolidated) was generally dominated by the resistance of individual particles. Banks composed of much older fluvial deposits with more time to both acquire more cohesive particles and become more consolidated (i.e., well-consolidated) were controlled by mass failure. Intermediate poorly- and moderately-consolidated bank types were generally found to be controlled by mass wasting with the latter and fluvial entrainment with the former; however, the segregation is both subjective and somewhat difficult to determine, especially in stable banks. For the present study, in addition to the current bank condition, we considered key risk factors including 1) the potential for lateral instability triggered by vertical instability, and 2) potential severity of the lateral response based on the available valley width (i.e., how large of a valley bottom is there for the channel to access?).

<u>Lateral Susceptibility Definitions and Forms</u>

- Channel Banks – vertically inclined surfaces that are generally perpendicular to flow and contain approximately the 10-year flow (i.e., the 'walls' of the active channel)

- *Extensive mass wasting* – >50% of banks exhibiting planar, slab, or rotational failures, and/or scalloping, undermining, and/or tension cracks (**Figure 8**)

a)

b)

Figure 8. Planar/Slab failure with tension cracks, exhibiting cohesive consolidated banks, at San Timetao, San Bernardino County, CA (a) and Acton, Los Angeles County, CA (b).

- *Extensive fluvial erosion* – significant and frequent bank cuts (> 50% of banks) and not limited to bends and constrictions (**Figure 9**)

Figure 9. Bank failure at Hicks Canyon, Orange County, CA, exhibiting combinations of fluvial erosion, shallow slips, and mass failure in weakly cohesive, poorly consolidated banks.

- *Moderately to highly consolidated* – hard when dry with little evidence of crumbling. Bank appears as a composite of tightly-packed particles that are difficult to delineate even with close inspection of the bank; moderately dry block/ped sample (1 in^2) is not crushable between fingers and bank material stratification not prevalent or contributing to failure (**Figure 10**)

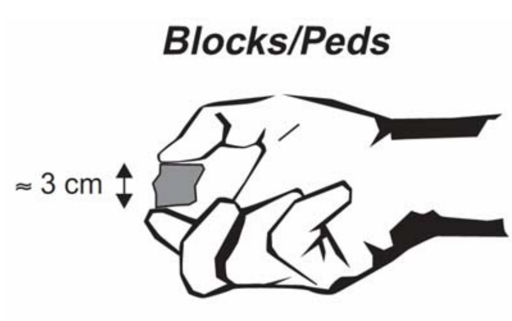


Figure 10. Moderately dry block/ped sample. Figure adapted from Schoeneberger *et al.* (2002); Not to scale.

- *Poorly consolidated to unconsolidated* – relatively weak with evidence of crumbling (**Figure 11**). Bank appears as a loose pile of recently deposited alluvium and block/ped samples (if attainable) can be crushed between fingers

a) b)

Figure 11. Failure of poorly consolidated banks with some cohesivity, but bank stability largely controlled by resistance of the individual particles of the bank toe in Stewart Canyon, Ventura County, CA, (a) and Hasley Canyon, Los Angeles County, CA (b).

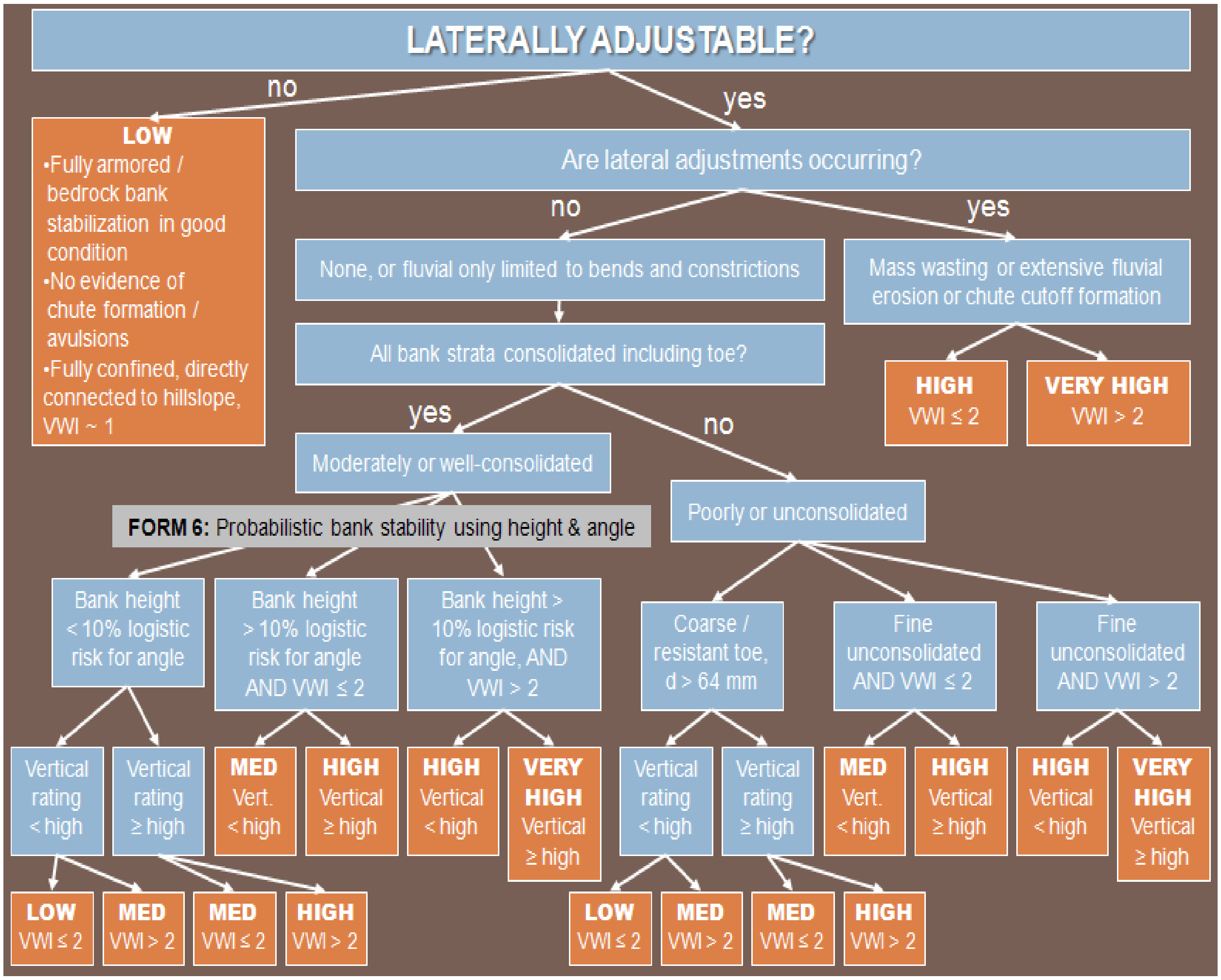
In assessing the potential for incision-induced bank failure we selected a vertical rating of high as a key discriminator. This decision was made primarily because such an approach inherently captures braiding risk as channels with high amounts of erosive energy relative to their bed material and >50% risk of incision/braiding using Form 3 Table 1 (**Figure 7**) would most likely result in a vertical rating of high unless exceptionally resistant and well-protected by armoring. We also defined a VWI of 2 as a key discriminator because doing so successfully distinguished between channels with valley bottoms 'confined by bedrock or hillslope' versus unconfined channels in the field data set. Unconfined valley settings were typically well above a VWI of 2.

The Lateral Susceptibility decision tree in Form 4 (**Figure 12**) and the series of questions in Form 5 (**Figure 13**) are provided for use in conducting the lateral susceptibility assessment. Either may be used depending on the user's preference. Definitions and photographic examples above are intended to support the lateral susceptibility assessment.

Additionally, Hawley (2009) performed logistic regression analysis of stable versus mass wasting in moderately- to well-consolidated banks using bank height and angle, consistent with geotechnical stability theory presented by Osman and Thorne (1988). The model was highly significant ($p < 0.0001$) and correctly classified unstable and stable states with ~95% accuracy, as shown in Form 6 (**Figure 14**), using a shape that was analogous to the Culmann relationship. As an alternative, by including the poorly consolidated sites, the model accuracy was ~90% with a lower 50% threshold and a much broader 10 to 90% risk range.

FORM 4: LATERAL SUSCEPTIBILTY FIELD SHEET

Circle appropriate nodes/pathway for proposed site
OR use sequence of questions provided in Form 5.



(Sheet 1 of 1)

Figure 12. Form 4: Lateral Susceptibility Field Sheet. Complete set of assessment forms in Appendix B.

FORM 5: SEQUENCE OF LATERAL SUSCEPTIBILITY QUESTIONS OPTION

Enter Lateral Susceptibility (Very High, High, Medium, Low) in shaded column.
Mass wasting and bank instability from Form 6, VWI from Form 4, and Vertical Rating from Form 3.

			Lateral Susceptibility
Channel fully confined with VWI ~1 – connected hillslopes OR fully-armored/engineered bed and banks in good condition?	If YES, then LOW		
If NO, Is there active **mass wasting** or extensive fluvial erosion (> 50% of bank length)?	If YES, VWI ≤ 2 = HIGH, VWI > 2 = VERY HIGH		
If NO, Are both banks consolidated?	If YES, How many risk factors present? **Risk Factors:** o Bank instability p > 10% o VWI > 2 o Vertical rating ≥ High	• All three = VERY HIGH • Two of three = HIGH • One of three = MEDIUM • None = LOW	
If NO, Are banks either consolidated or unconsolidated with coarse toe of d > 64 mm?	If YES, How many risk factors present? **Risk Factors:** o VWI > 2 o Vertical rating ≥ High	• Two = HIGH • One = MEDIUM • None = LOW	
If NO, At least one bank is unconsolidated with toe of d < 64 mm	How many risk factors present? **Risk Factors:** o VWI > 2 o Vertical rating ≥ High	• Two = VERY HIGH • One = HIGH • None = MEDIUM	

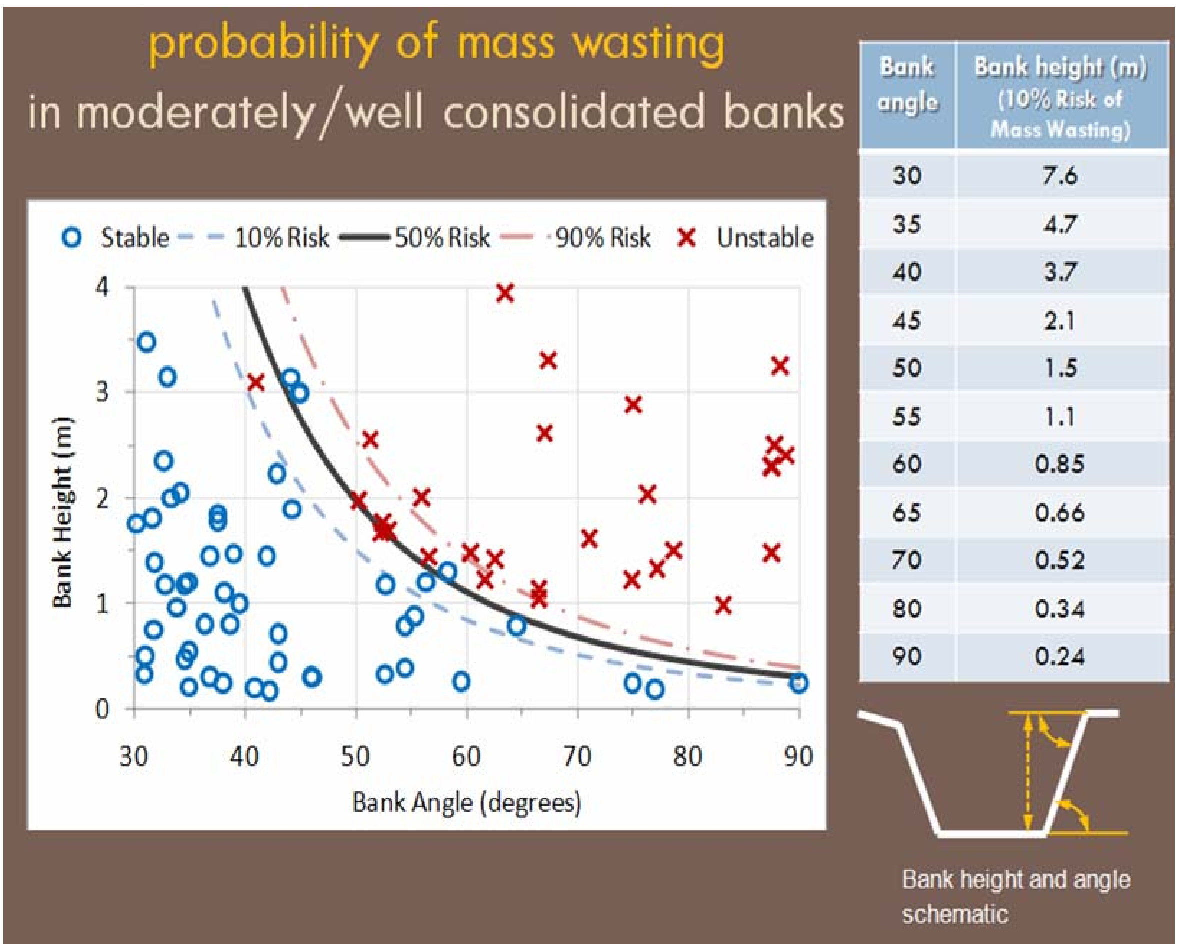
(Sheet 1 of 1)

Figure 13. Form 5: Sequence of Lateral Questions Option for lateral susceptibility assessment. Complete set of assessment forms in Appendix B.

FORM 6: PROBABILITY OF MASS WASTING BANK FAILURE

If mass wasting is not currently extensive and the banks are moderately- to well-consolidated, measure bank height and angle at several locations (i.e., at least three locations that capture the range of conditions present in the study reach) to estimate representative values for the reach. Use Form 6 Figure 1 below to determine if risk of bank failure is >10% and complete Form 6 Table 1. Support your results with photographs that include a protractor/rod/tape/person for scale.

	Bank Angle (degrees) *(from Field)*	**Bank Height (m)** *(from Field)*	**Corresponding Bank Height for 10% Risk of Mass Wasting (m)** *(from Form 6 Figure 1 below)*	**Bank Failure Risk** *(<10% Risk)* *(>10% Risk)*
Left Bank				
Right Bank				



Bank angle	Bank height (m) (10% Risk of Mass Wasting)
30	7.6
35	4.7
40	3.7
45	2.1
50	1.5
55	1.1
60	0.85
65	0.66
70	0.52
80	0.34
90	0.24

Form 6 Figure 1. Probability Mass Wasting diagram, Bank Angle:Height/% Risk table, and Band Height:Angle schematic.

(Sheet 1 of 1)

Figure 14. Form 6: Probability of Mass Wasting Bank Failure for lateral susceptibility assessment. Complete set of assessment forms in Appendix B.

SUMMARY AND CONCLUSIONS

After completing the initial desktop and field components, the user should return to the office to summarize the reconnaissance information. Some values that were measured in the field may require (or be simplified by) computer assistance (e.g., sorting and ranking the pebble count data to determine the median particle size). A data entry spreadsheet (Data Entry Form.xls) has been provided to automate the necessary calculations from your field data.

At a minimum, we suggest outlining the following aspects from the field reconnaissance:

- Aerial photo of analysis domain with demarcation of reaches assessed and locations of critical features such as hard points, outfalls, changes in valley type, etc.
- A minimum of four photos from each assessed reach
    - Overview/cross-section
    - Representative bed material
    - Representative bank from right and left side of channel
- Applicable Vertical Susceptibility forms and final rating from each assessed reach
- Applicable Lateral Susceptibility forms and final rating from each assessed reach

In depth information describing the development and scientific basis of the field screening tool is provided in SCCWRP Technical Report 607, available at www.sccwrp.org We expect that the field screening tool presented herein will be systematically improved over time through a variety of monitoring and adaptive management activities, as well as through user feedback. Accordingly, comments, questions, and suggestions are welcome and may be submitted to Eric Stein at SCCWRP (**erics@sccwrp.org**) and Brian Bledsoe at CSU (brian.bledsoe@colostate.edu).

LITERATURE CITED

Beatty, D.A. 1984. Discussion of "Channel migration and incision on the Beatton River" by Gerald C. Nanson and Edward J. Hickin (March, 1983). *Journal of Hydraulic Engineering* 110:1681-1682.

Bledsoe, B.P., 2002. Stream erosion potential associated with stormwater management strategies. Journal of Water Resources Planning and Management, 128: 451-455.

Bledsoe, B.P. and C.C. Watson. 2001. Effects of urbanization on channel instability. *Journal of the American Water Resources Association* 37:255-270.

Bledsoe, B.P., R.J. Hawley and E.D. Stein. 2008. Systems: Implications for Assessing Susceptibility to Hydromodification Effects in Southern California. Technical Report 562. Southern California Coastal Water Research Project. Costa Mesa, CA.

Brice, J.C. 1960. Index for description of channel braiding. *Geological Society of America Bulletin* 71:1833.

Brice, J.C. 1964. Channel patterns and terraces of the Loup Rivers in Nebraska. *US Geological Survey Professional Paper* 422-D.

Bunte, K. and S.R. Abt. 2001a. Sampling frame for improving pebble count accuracy in coarse gravel-bed streams. *Journal of the American Water Resources Association* 37:1001-1014.

Bunte, K. and S.R. Abt. 2001b. Sampling surface and subsurface particle-size distributions in wadable gravel-and cobble-bed streams for analyses in sediment transport, hydraulics, and streambed monitoring. pp. 448 *in*: US Department of Agriculture (ed.), General Technical Report RMRS-GTR-74. USDA, Rocky Mountain Research Station. Fort Collins, CO.

Chang, H.H. 1988. Fluvial Processes in River Engineering. John Wiley & Sons. New York, NY.

Chin, A. and K.J. Gregory. 2005. Managing urban river channel adjustments. *Geomorphology* 69:28-45.

Downs, P.W. and K.J. Gregory. 1995. Approaches to river channel sensitivity. *Professional Geographer* 47:168-175.

Hawley, R.J. 2009. Effects of urbanization on the hydrologic regimes and geomorphic stability of small streams in southern California. Ph.D. Dissertation, Colorado State University. Fort Collins, CO.

Hawley, R.J. and B.P. Bledsoe. In review. Long-term effects of urbanization on the flow regimes of southern California streams. *Journal of Hydrology*.

Hey, R.D., G.L. Heritage, N.K. Tovey, R.R. Boar, N. Grant and R.K. Turner. 1991. Streambank Protection in England and Wales. National Rivers Authority. London, UK.

Hooke, J.M. 1979. An analysis of the processes of river bank erosion. *Journal of Hydrology* 42:39-62.

Julien, P.Y. 1998. Erosion and Sedimentation. Cambridge University Press. Cambridge, UK.

Knighton, A.D. 1998. Fluvial Forms and Processes: A New Perspective. John Wiley & Sons. New York, NY.

Lawler, D.M. 1992. Process dominance in bank erosion systems. pp. 117-143 *in*: P. Carling and G.E. Petts (eds.), Lowland Floodplain Rivers: Geomorphological Perspectives. Wiley. Chichester, UK.

Lawler, D.M., C.R. Thorne J.M. and Hooke. 1997. Bank Erosion and Instability. pp. 137-172 *in*: C.R. Thorne, R.D. Hey and M.D. Newson (eds.), Applied Fluvial Geomorphology for River Engineering and Management. Wiley. Chichester, UK.

Leopold, L.B. and M.G. Wolman. 1957. River channel patterns -- braided, meandering, and straight. *US Geological Survey Professional Paper* 282-B.

Montgomery, D.R. and L.H. MacDonald. 2002. Diagnostic approach to stream channel assessment and monitoring. *Journal of the American Water Resources Association* 38:1-16.

Nanson, G.C. and J.C. Croke. 1992. A genetic classification of floodplains. *Geomorphology* 4:459-486.

Osman, A.M. and C.R. Thorne. 1988. Riverbank stability analysis I: Theory. *Journal of Hydraulic Engineering* 114:134-150.

Schoeneberger, P.J., D.A. Wysocki, E.C Benham and W.D. Broderson (eds.). 2002. Field Book for Describing and Sampling Soils, Version 2.0. Natural Resources Conservation Service, National Soil Survey Center. Lincoln, NE.

Schumm, S.A. 1985. Patterns of alluvial rivers. *Annual Review of Earth and Planetary Sciences 13:5-27.*

Schumm, S.A., 1991. To Interpret the Earth: Ten Ways to be Wrong. Cambridge University Press. Cambridge, UK.

Schumm, S.A., M.D. Harvey and C.C. Watson. 1984. Incised Channels: Morphology, Dynamics, and Control. Water Resources Publications. Littleton, CO.

Simons, D.B. and R.K. Simons. 1987. Differences between gravel and sand-bed rivers. *in*: C.R. Thorne, J.C. Bathurst and R.D. Hey (eds.), Sediment Transport in Gravel-bed Rivers, John Wiley & Sons. New York, NY.

Strahler, A.N. 1952. Hypsomic analysis of erosional topography. *Geological Society of America Bulletin* 63:1117-1142.

Thorne, C.R. 1982. Processes and mechanisms of river bank erosion. pp. 227-271 *in*: R.D. Hey, J.C. Bathurst and C.R. Thorne (eds.), Gravel-bed Rivers. Wiley. Chichester, UK.

van den Berg, J.H. 1995. Prediction of alluvial channel pattern of perennial rivers. *Geomorphology* 12:259-279.

Waananen, A.O. and J.R. Crippen. 1977. Magnitude and Frequency of Floods in California. US Geological Survey. Menlo Park, CA.

Watson, C.C., D.S. Biedenharn and B.P. Bledsoe. 2002. Use of incised channel evolution models in understanding rehabilitation alternatives. *Journal of the American Water Resources Association* 38:151-160.

Wilcock, P.R. and J.C. Crowe. 2003. Surface-based transport model for mixed-size sediment. *Journal of Hydraulic Engineering* 129:120-128.

Wilcock, P.R. and S.T. Kenworthy. 2002. A two-fraction model for the transport of sand/gravel mixtures. Water Resources Research 38: 1194-2003.

APPENDIX A: GENERAL DEFINITIONS

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APPENDIX A: GENERAL DEFINITIONS

A.1 SUSCEPTIBILITY/SENSITIVITY DEFINITIONS

What is susceptibility?

The *intrinsic* sensitivity of a channel system to hydromodification as determined by the ratio of disturbing to resisting forces, proximity to thresholds of concern, probable rates of response and recovery, and potential for spatial propagation of impacts.

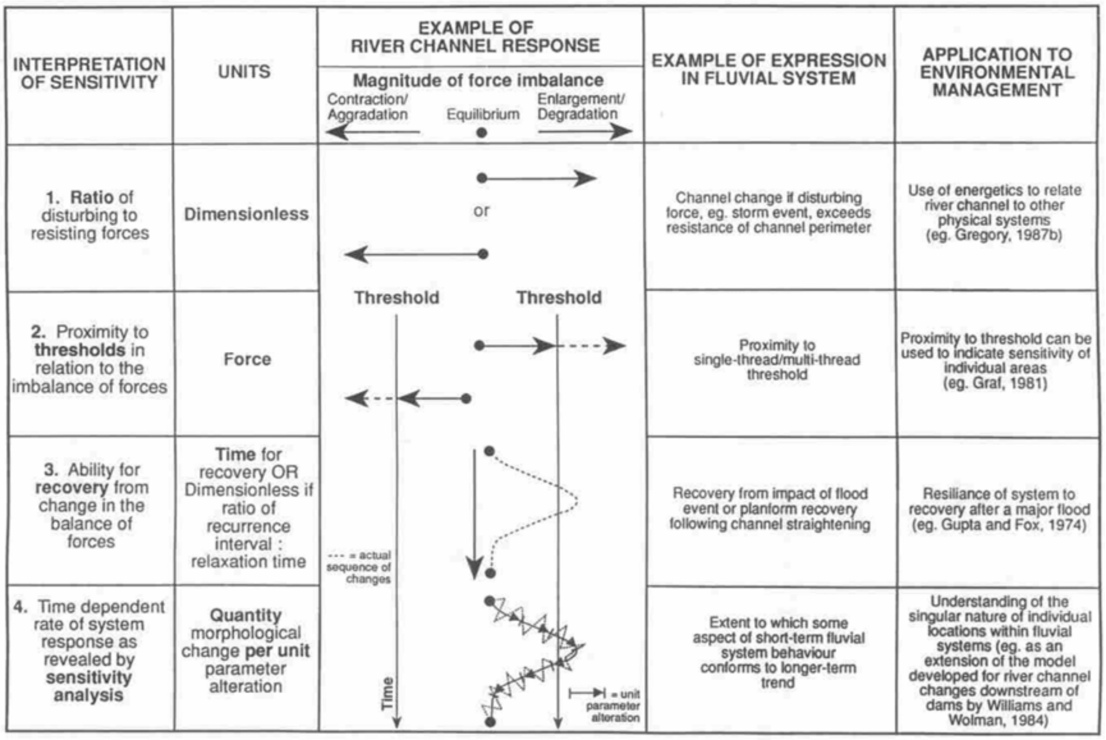
What is sensitivity?

Schumm defined *sensitivity* as:

"One aspect of (landform) singularity that must be treated separately is the sensitivity of landscape components... The reason for such variable response… is the existence of threshold conditions, which when exceeded produce a large change. In contrast, apparently similar landforms may show little or no response to a similar change. Thus, within a landscape composed of singular landforms there will be sensitive and insensitive landforms." Schumm (1985, page 13)

"Sensitivity refers to the propensity of a system to respond to a minor external change. The changes occur at a threshold, which when exceeded produces a significant adjustment. If the system is sensitive and near a threshold it will respond to an external influence; but if it is not sensitive it may not respond." Schumm (1991, page 78)

Downs and Gregory (1995) illustrated *sensitivity* as:



INTERPRETATION OF SENSITIVITY	UNITS	EXAMPLE OF RIVER CHANNEL RESPONSE Magnitude of force imbalance Contraction/ Aggradation — Equilibrium — Enlargement/ Degradation	EXAMPLE OF EXPRESSION IN FLUVIAL SYSTEM	APPLICATION TO ENVIRONMENTAL MANAGEMENT
1. Ratio of disturbing to resisting forces	**Dimensionless**	or	Channel change if disturbing force, eg. storm event, exceeds resistance of channel perimeter	Use of energetics to relate river channel to other physical systems (eg. Gregory, 1987b)
2. Proximity to **thresholds** in relation to the imbalance of forces	**Force**	**Threshold** **Threshold**	Proximity to single-thread/multi-thread threshold	Proximity to threshold can be used to indicate sensitivity of individual areas (eg. Graf, 1981)
3. Ability for **recovery** from change in the balance of forces	**Time** for recovery OR Dimensionless if ratio of recurrence interval : relaxation time	--- = actual sequence of changes	Recovery from impact of flood event or planform recovery following channel straightening	Resiliance of system to recovery after a major flood (eg. Gupta and Fox, 1974)
4. Time dependent rate of system response as revealed by **sensitivity analysis**	**Quantity** morphological change **per unit** parameter alteration	Time = unit parameter alteration	Extent to which some aspect of short-term fluvial system behaviour conforms to longer-term trend	Understanding of the singular nature of individual locations within fluvial systems (eg. as an extension of the model developed for river channel changes downstream of dams by Williams and Wolman, 1984)

Figure A.1: Interpretation of sensitivity from Downs and Gregory (1995)

We add to this, the potential spatial extent of impacts over a common engineering time scale of *ca*. 50 yrs. That is, some effects may propagate throughout drainage networks relatively quickly and result in headcutting, base-level lowering of tributaries, complex response, etc.

A.2 Braiding Definitions

- Broadest definition: multi-channel patterns (Leopold and Wolman 1957)
- Definition illustrations of sinuosity, braiding, and anabranching (Figure A.2), incision-driven CEM (Figure A.3), bank failure (Figures A.4a and A.4b)
- Flow separated by bars within a defined channel, where bars (Knighton 1998):
    - o may be inundated at higher flows, appearing as a single channel at/near 'bankfull'
    - o tend to be unvegetated, temporary, with little cohesion
- Characterized by repeated division and joining of channels (i.e., divergence and convergence of flow) resulting in high rates of fluvial activity relative to other rivers (Knighton 1998)

- Non-cohesive floodplains with braid-channel accretion as the main sediment accretion mechanism (Nanson and Croke 1992)

- Informed by the aforementioned definitions, we classify 'braided' channels for the purposes of this screening tool as:

  ➔ ***Multiple flow paths through over 50% of the reach length at low to moderate flows (see 35 – 65% 'degree of braiding', Figure A.2)***

  ➔ ***OR, if stakeholders are not concerned about 'anastomosing'/ 'anabranching' systems, augment above with: where paths are temporary and the result of dynamic, mostly unvegetated/non-cohesive bars***

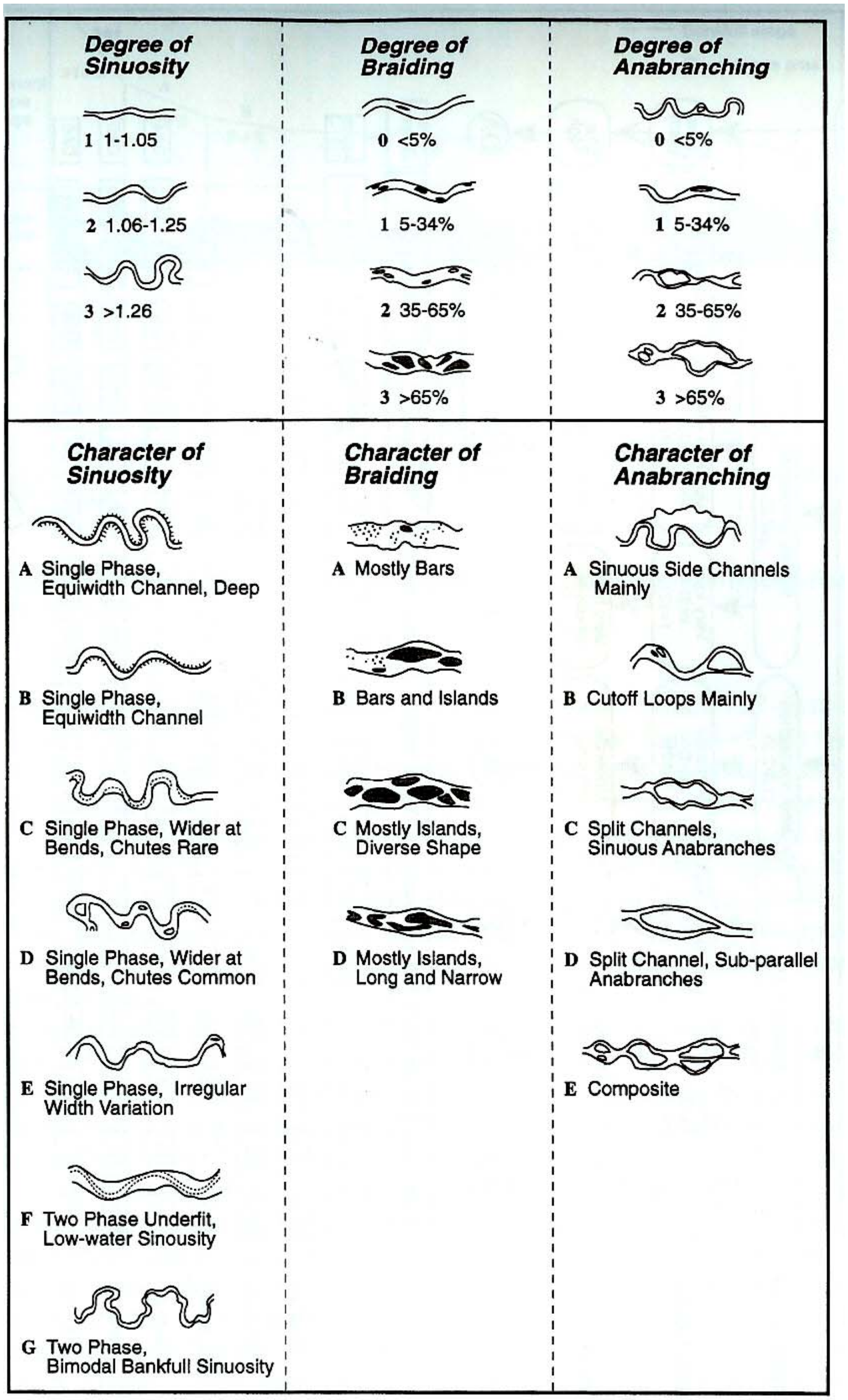
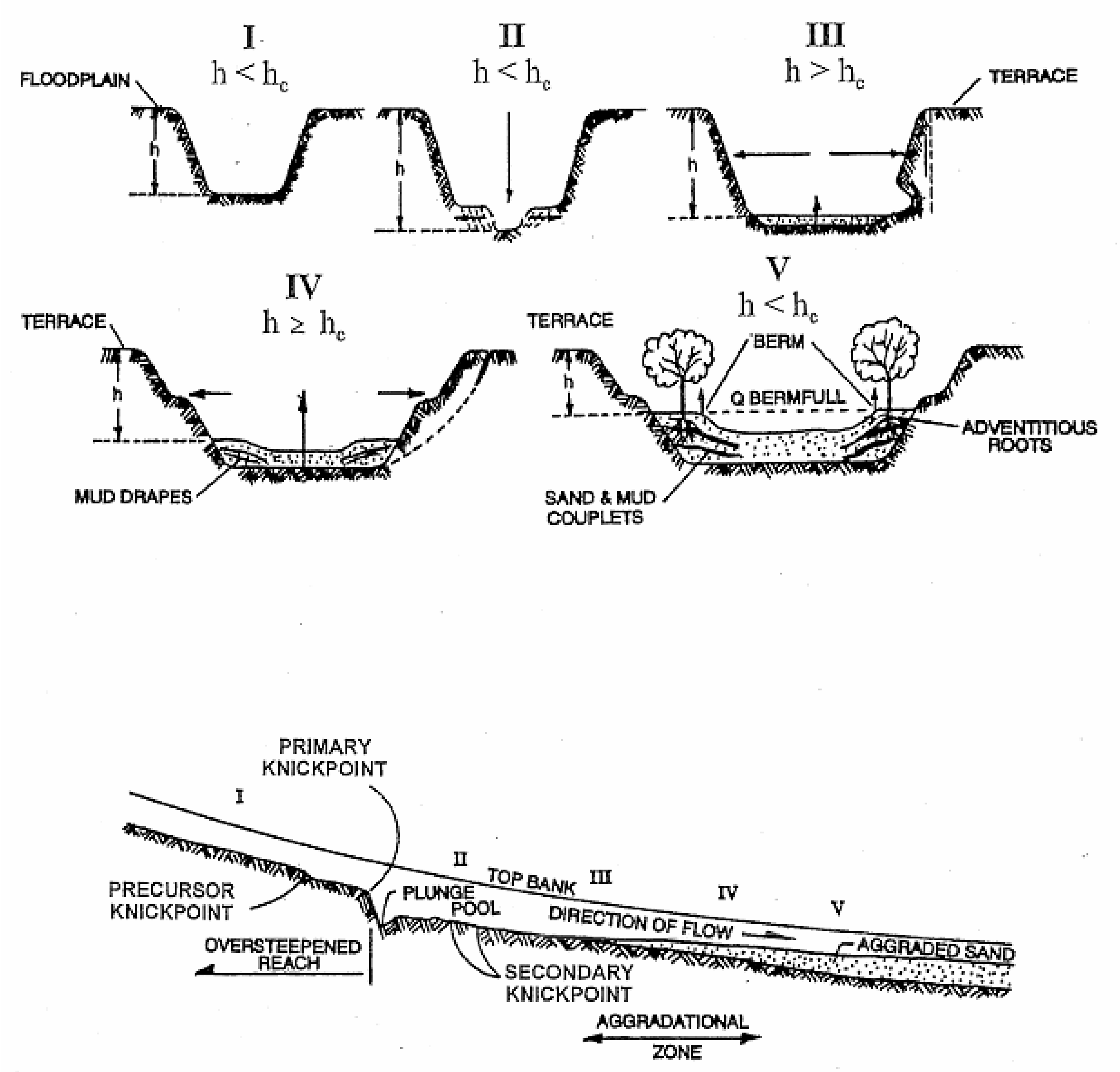


Figure A.2: Illustration of sinuosity, braiding, and anabranching (from Brice (1960, 1964))



h_c = critical bank height for mass failure

Figure A.3: Incision-driven CEM after Schumm *et al.* (1984); figure adapted from Watson *et al*. (2002)

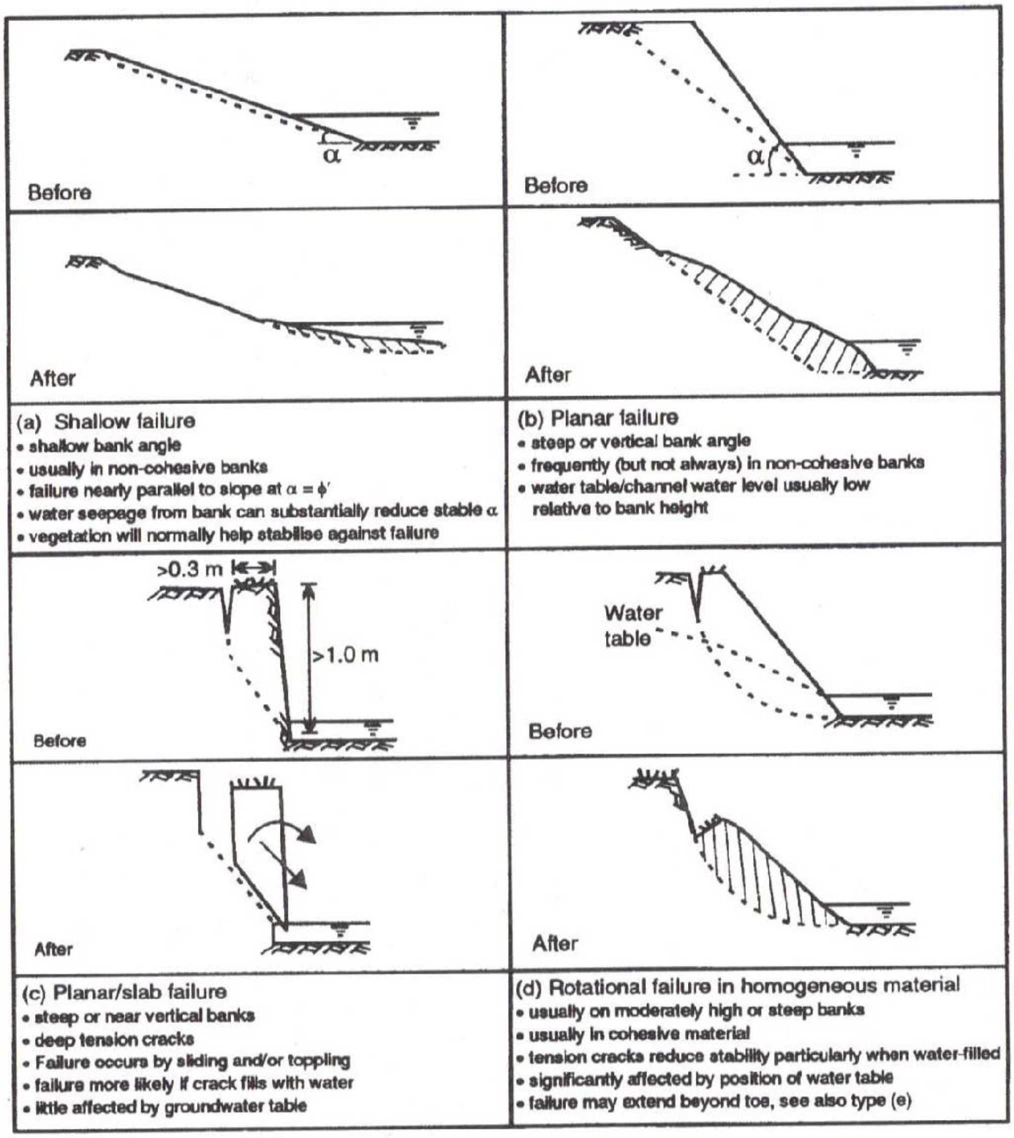


Figure A.4a: Bank-failure illustrations (a through d) after Hey *et al.* (1991); figure adapted from Lawler *et al.* (1997)

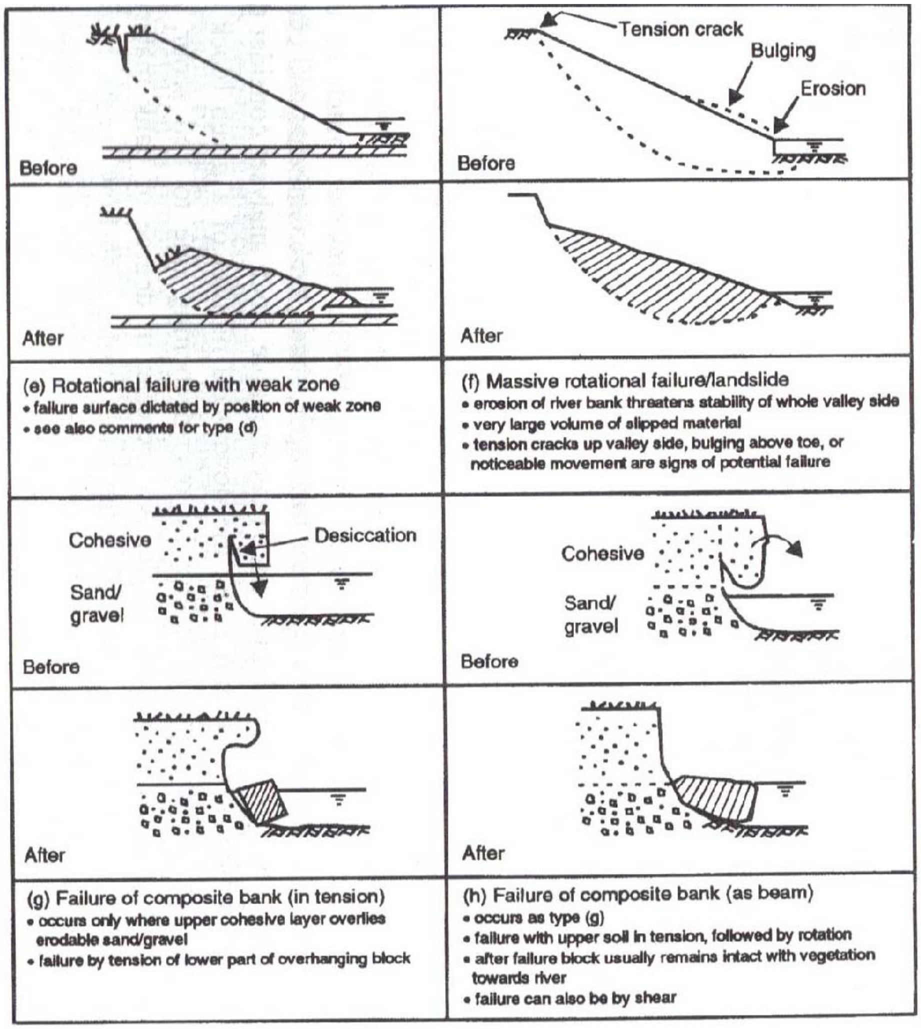


Figure A.4b: Bank-failure illustrations (e through h) after Hey *et al.* (1991); figure adapted from Lawler *et al.* (1997)

A.3 PEBBLE COUNT INSTRUCTIONS

Sampling with a frame – Excerpt from Bunte and Abt, 2001b:

"A tape measure is stretched from bank to bank. The sampling frame is placed onto the stream bottom so that one of the corners aligns with even-spaced marks on the tape, e.g., every three feet or one meter. Grid points derived by the elastic bands are used to visually define the particle to be selected. If the flow is deep and fast, and vision is blurred, looking at the grid intersection can help identify the particle to be included in the sample. If, for example, the grid intersection is between two cobbles, the operator knows that a small interstitial particle should be selected, but neither of the cobbles.

If flow is too deep or too fast to see the particle under the grid intersection, the particle to be included in the sample has to be identified by touch. A pointed index finger is placed in a corner of the grid intersection, and vertically lowered onto the sediment surface. The grid intersection serves as a guide for the position of the finger as it is lowered to the bed surface. Using the grid intersection as a reference point as opposed to the tip of the boot helps the operator select a particle more representatively because the operator works in a more comfortable posture when bending down to the sampling frame as opposed to bending down to the tip of the boot. The elastic bands in the sampling frame do not hinder the removal of a particle from the streambed. Particles are collected from under all four grid points, measured with a template, and placed back approximately into the same position from which they were taken. The frame is then moved to the next position along the tape. For many coarse gravel-bed rivers, a 30-cm grid within a 60 by 60 cm frame placed at 1 m, or 3 feet increments along the tape will be adequate. The sampling frame can be used on both sides of a transect. Individual transects should be 3 - 4 m apart to avoid overlap between sampled areas."

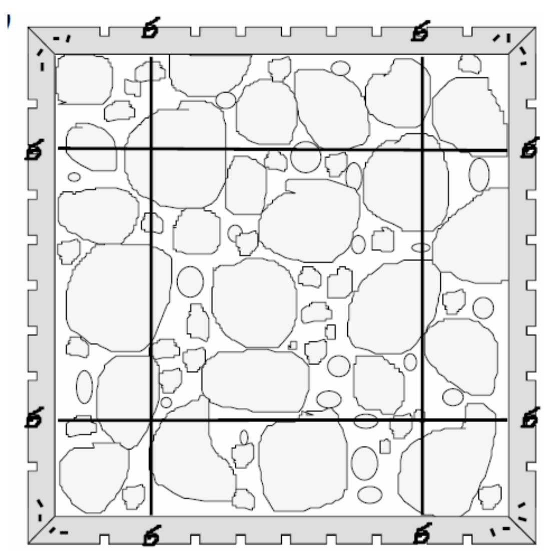


Figure A.5: Pebble Count Sampling Frame and Instructions from Bunte and Abt (2001b)

APPENDIX B: ASSESSMENT FORMS

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APPENDIX B: ASSESSMENT FORMS

Overview

This appendix compiles the field forms necessary to conduct the field susceptibility analysis. The field assessment uses a combination of relatively simple, but quantitative field, indicators as input parameters to a set of decision trees. The decision trees follow a logical progression and allow users to assign a classification of Low, Medium, High, or Very High susceptibility rating to the reach being assessed. Each stream reach is assessed independently in terms of its vertical and lateral susceptibility.

The susceptibility assessment consists of the following steps:

1. Determine the **Analysis Domain**
2. Conduct the initial **Office Assessment**
3. Rate the **Vertical Susceptibility** of the stream reach
4. Rate the **Horizontal Susceptibility** of the stream reach

The following forms and instructions provided to conduct these assessments include:

- Instruction on determining Analysis Domain
- Office Assessment Forms
    - Form 1: Initial Desktop Analysis
    - Form 2: Pebble Count
        - Example coverage and substrate sizing guidance
    - Form 3: Vertical Susceptibility Field Sheet
        - Checklist 1: Armoring Potential
        - Checklist 2: Grade Control
        - Probability of Incising/Braiding Diagram and Screening Index Threshold Calculations
        - Overall Vertical Susceptibility Decision Tree
    - Form 4: Lateral Susceptibility Field Sheet
    - Form 5: Sequence of Lateral Susceptibility Questions Option
    - Form 6: Probability of Mass Wasting Bank Failure

In order to complete the field assessment, the following items should be taken to the field:

- Additional forms and/or field book for sketches/notes
- Digital camera for photographic documentation
- Pocket rod and/or tape for some basic measurements and reference/scale in photographs
- Protractor (e.g., gravity-driven) for measuring bank angle
- Gravelometer (i.e., US SAH-97 half-phi template) for standardized pebble count

Analysis Domain

Prior to initiating the assessment, it is necessary to define the domain of analysis that will be covered. The maximum spatial unit for assigning a susceptibility rating is defined as a *ca.* 20 channel width 'reach' not to exceed 200 m. Before conducting the field screening, the analyst should identify the following attributes as part of the office analysis to estimate the maximum extent of the analysis domain for field refinement.

Begin by defining the points or zones along the channel reach(es) where changes in discharge or channel type are likely to occur (e.g., potential locations of outfalls or tributary inputs). Document any observed outfalls for final desktop synthesis and define the upstream and downstream extents of analysis as follows:

- **Downstream** – until reaching the closest of the following:
    - at least one reach downstream of the first grade-control point (but preferably the second downstream grade-control location)
    - tidal backwater/lentic waterbody
    - equal order tributary (Strahler 1952)[1]
    - a 2-fold increase in drainage area[2]

OR demonstrate sufficient flow attenuation through existing hydrologic modeling

- **Upstream** – extend the domain upstream for a distance equal to 20 channel widths OR to grade control in good condition – whichever comes first. Within that reach, identify hard points that could check headward migration, evidence that head cutting is active or could propagate unchecked upstream

Within the analysis domain there may be several reaches that should be assessed independently based on either length or change in physical characteristics. In more urban settings, segments may be logically divided by road crossings (Chin and Gregory, 2005), which may offer grade control, cause discontinuities in the conveyance of water or sediment, etc. In more rural settings, changes in valley/channel type, natural hard points, and tributary confluences may be more appropriate for delineating assessment reaches. In general, the following criteria should trigger delineation of a new reach and hence a separate susceptibility assessment:

- 200 m or ca. 20 bankfull widths – it is difficult to integrate over longer distances
- Distinct or abrupt change in grade or slope due to either natural or artificial features
- Distinct or abrupt change in dominant bed material or sediment conveyance
- Distinct or abrupt change in valley setting or confinement
- Distinct or abrupt change in channel type, bed form, or planform

Assessment Forms

Assessment Forms 1 - 6, beginning on the next page, have been collected for printing as a group.

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[1] In the absence of proximate downstream grade control or backwater, the confluence of an 'equal order tributary' should correspond to substantial increases in flow and channel capacity that should, in theory, correspond to significant flow attenuation; however, there is no scientific basis to assume that downstream channels of higher stream order are less susceptible than their upstream counterparts. This (practically-driven) guidance should not supersede the consideration of local conditions and sound judgment. Stakeholders may elect to use a more regionally-preferred guidance.

[2] An increase in drainage area greater than or equal to 100% would roughly correspond to the addition of an equal-order tributary (see above).

FORM 1: INITIAL DESKTOP ANALYSIS

Complete all shaded sections.

IF required at multiple locations, circle one of the following site types:

Applicant Site / **Upstream Extent** / **Downstream Extent**

Location: Latitude: ____________ Longitude: ____________

Description (river name, crossing streets, etc.): ____________

GIS Parameters: The International System of Units (SI) is used throughout the assessment as the field standard and for consistency with the broader scientific community. However, as the singular exception, US Customary units are used for contributing drainage area (A) and mean annual precipitation (P) to apply regional flow equations after the USGS. See SCCWRP Technical Report 607 for example measurements and "Screening Tool Data Entry.xls" for automated calculations.

Form 1 Table 1. Initial desktop analysis in GIS.

	Symbol	Variable	Description and Source	Value
Watershed properties (English units)	A	Area (mi^2)	Contributing drainage area to screening location via published Hydrologic Unit Codes (HUCs) and/or ≤ 30 m National Elevation Data (NED), USGS seamless server	
	P	Mean annual precipitation (in)	Area-weighted annual precipitation via USGS delineated polygons using records from 1900 to 1960 (which was more significant in hydrologic models than polygons delineated from shorter record lengths)	
Site properties (SI units)	S_v	Valley slope (m/m)	Valley slope at site via NED, measured over a relatively homogenous valley segment as dictated by hillslope configuration, tributary confluences, etc., over a distance of up to ~500 m or 10% of the main-channel length from site to drainage divide	
	W_v	Valley width (m)	Valley bottom width at site between natural valley walls as dictated by clear breaks in hillslope on NED raster, irrespective of potential armoring from floodplain encroachment, levees, etc. (imprecise measurements have negligible effect on rating in wide valleys where VWI is >> 2, as defined in lateral decision tree)	

Form 1 Tabl e 2. Simplif ied peak flo w, screening index, and valley width index. Values for this table should be calculated in the sequence shown in this table, using values from Form 1 Table 1.

Symbol	Dependent Variable	Equation	Required Units	Value
Q_{10cfs}	10-yr peak flow (ft^3/s)	$Q_{10cfs} = 18.2 * A^{0.87} * P^{0.77}$	A (mi^2) P (in)	
Q_{10}	10-yr peak flow (m^3/s)	$Q_{10} = 0.0283 * Q_{10cfs}$	Q_{10cfs} (ft^3/s)	
INDEX	10-yr screening index ($m^{1.5}/s^{0.5}$)	$INDEX = S_v * Q_{10}^{0.5}$	Sv (m/m) Q_{10} (m^3/s)	
W_{ref}	Reference width (m)	$W_{ref} = 6.99 * Q_{10}^{0.438}$	Q_{10} (m^3/s)	
VWI	Valley width index (m/m)	$VWI = W_v/W_{ref}$	W_v (m) W_{ref} (m)	

(Sheet 1 of 1)

FORM 2: PEBBLE COUNT

If it is necessary to estimate d_{50}, perform a pebble count, after Bunte and Abt (2001a,b), using a minimum of 100 particles and a standard half-phi template, or by measuring along the intermediate axis of each pebble. Use a grid and tape for equally spaced samples over systematic/complete transects across riffle sections (i.e., if the 100^{th} particle is in the middle of a transect, complete the full transect before stopping the count; if more than 125 particles, record data near the bottom of Sheet 2 of 2). If the source of fines (sand/silt d <2 mm; see Form 2 Table 2 below) is less than ½ inch thick (approximately one finger width) at the sampling point, sample the coarser buried substrate; otherwise record observation of fines. Take photographs to support observations (Detailed instructions in Appendix A.3).

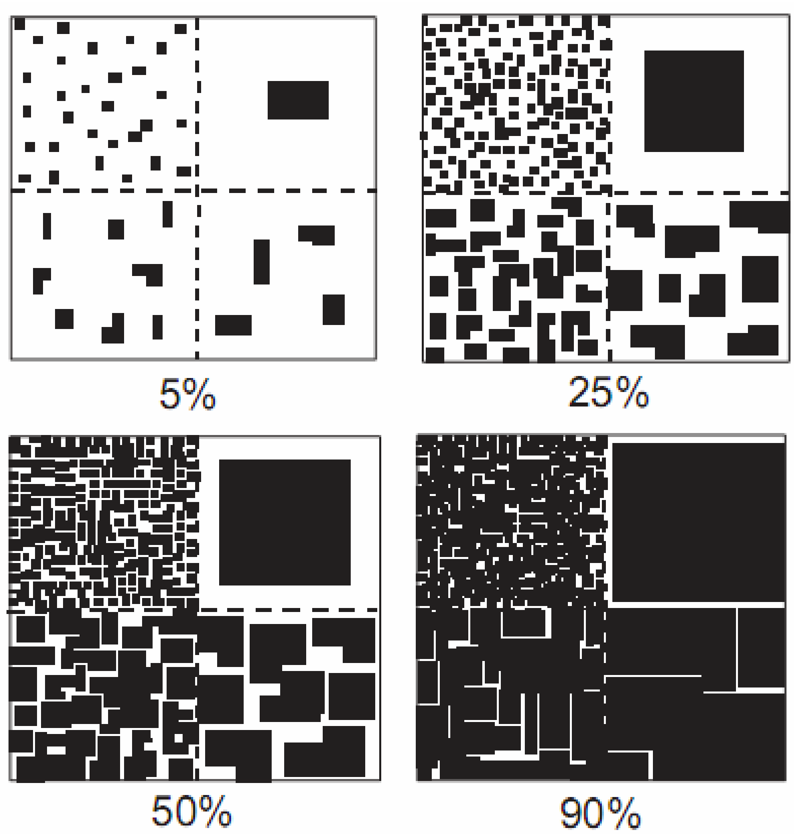
Form 2 Table 1. 100-pebble count tabulation for Vertical Susceptibility. Record station (Sta) and diameter (d) in millimeters.

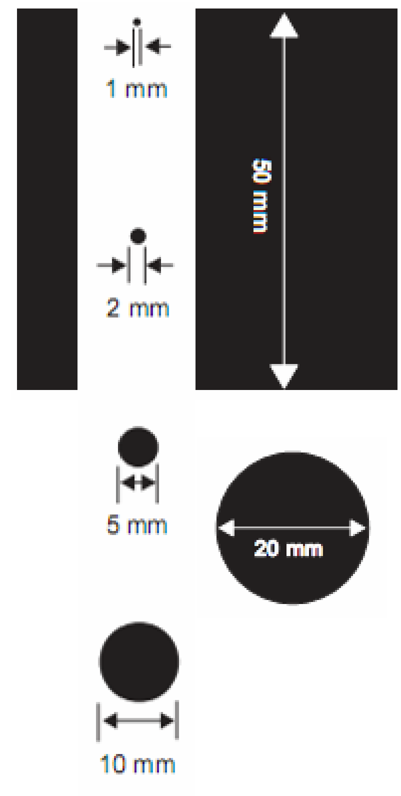
#	Sta	d (mm)	#	Sta	d (mm)	#	Sta	d (mm)	#	Sta	d (mm)	#	Sta	d (mm)
1			26			51			76			101		
2			27			52			77			102		
3			28			53			78			103		
4			29			54			79			104		
5			30			55			80			105		
6			31			56			81			106		
7			32			57			82			107		
8			33			58			83			108		
9			34			59			84			109		
10			35			60			85			110		
11			36			61			86			111		
12			37			62			87			112		
13			38			63			88			113		
14			39			64			89			114		
15			40			65			90			115		
16			41			66			91			116		
17			42			67			92			117		
18			44			68			93			118		
19			44			69			94			119		
20			45			70			95			120		
21			46			71			96			121		
22			47			72			97			122		
23			48			73			98			123		
24			49			74			99			124		
25			50			75			100			125		

Form 2 Table 2. d_{50} for Screening Index Threshold.

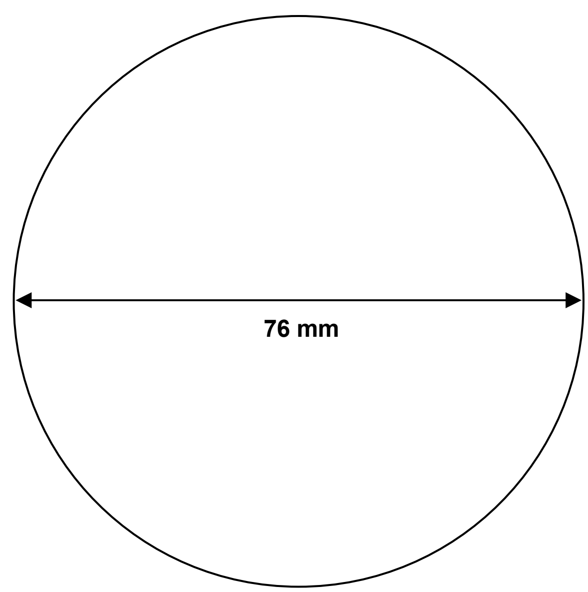
Class Name	Diameter (mm)	Helpful Descriptions for Field Identification
Boulder	> 256	Difficult to lift by hand
Cobble	> 64	Typically able to lift
Gravel	> 2	Fits in one hand
Sand	> 0.0625	Can feel between fingers
Silt	> 0.004	Can feel with tongue
Clay	≤ 0.004	Can not feel individual particle

(Sheet 1 of 2)





Note: Each quadrant within each box contains the same total area covered using different sized objects.

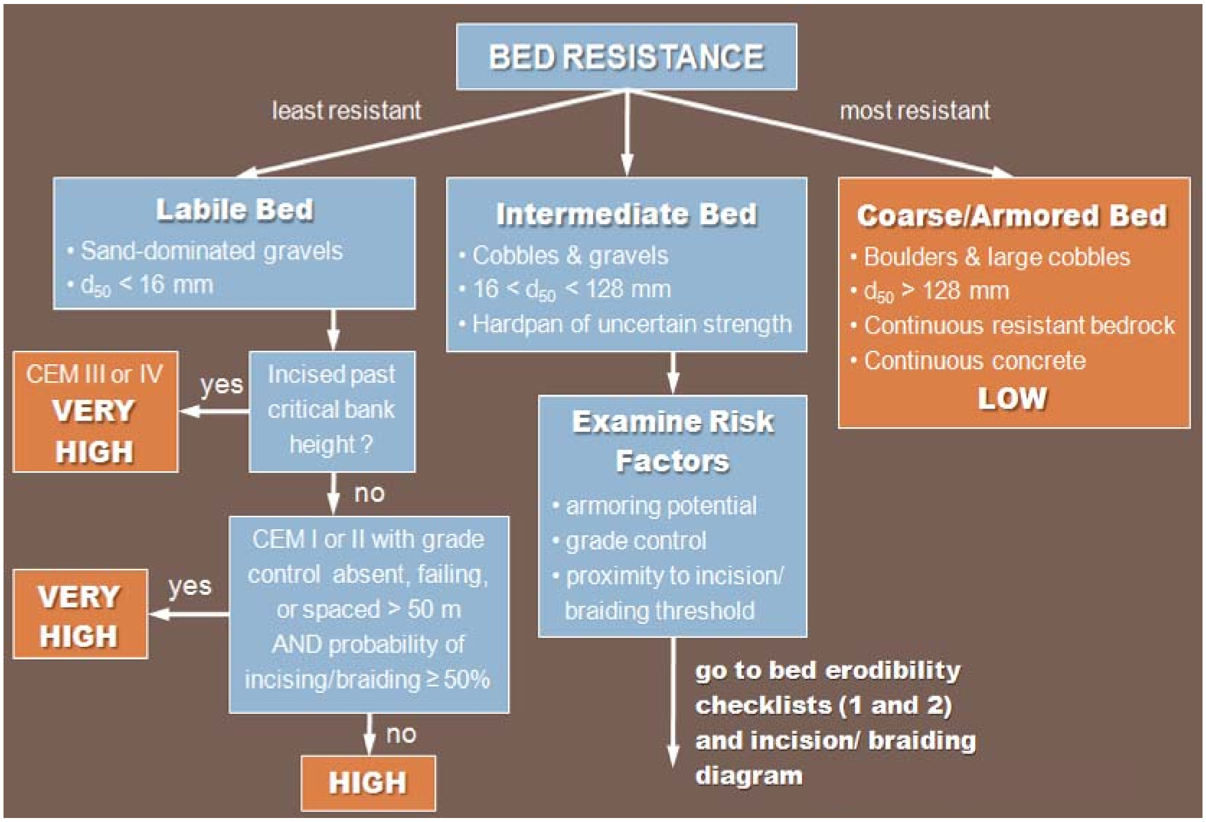


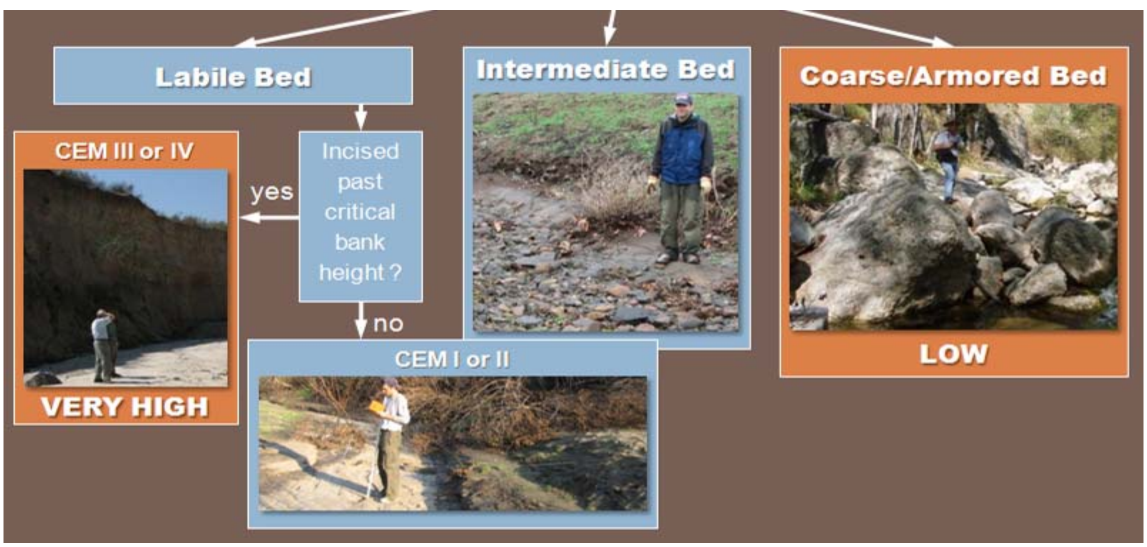
Form 2 Figure 1. Examples of % coverage by volume and substrate sizing adapted from *NRCS Field Book for Describing and Sampling Soils* (Schoeneberger *et al.* 2002) and Julien (1998).

(Sheet 2 of 2)

FORM 3: VERTICAL SUSCEPTIBILITY FIELD SHEET

Circle appropriate nodes/pathway for proposed site.





Form 3 Figure 1. Vertical Susceptibility photographic supplement to be used in conjunction with Form 3 Bed Resistance above.

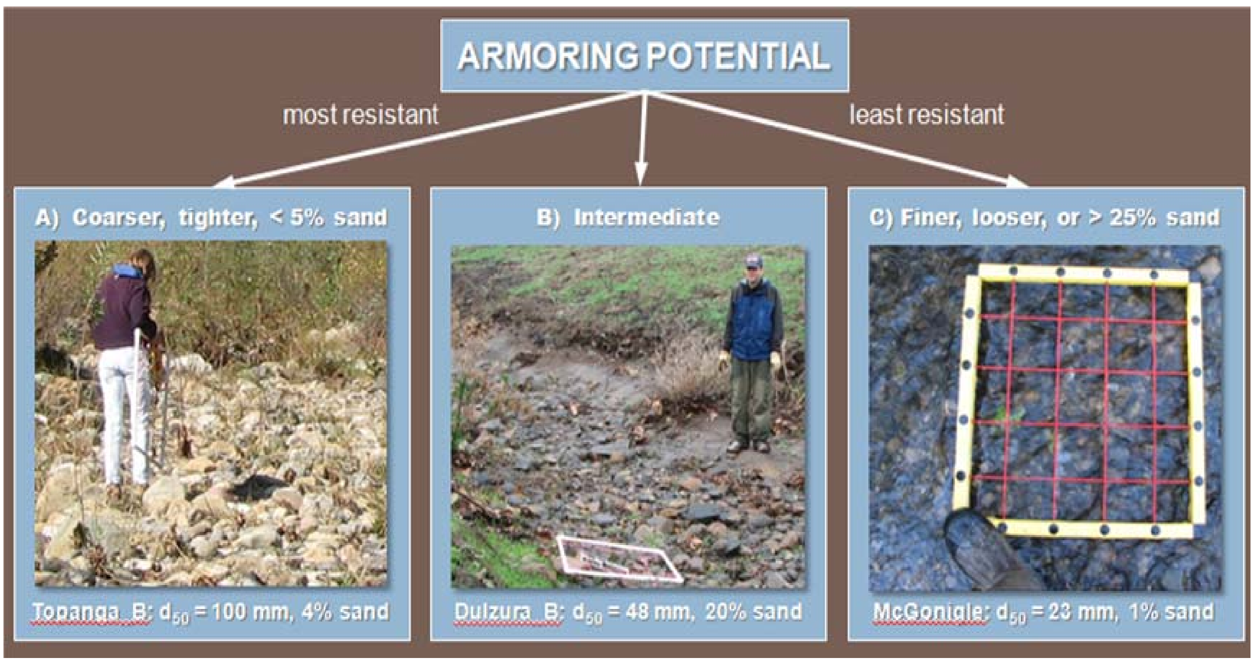
(Sheet 1 of 4)

Form 3 Support Materials

Form 3 Checklists 1 and 2, along with information recording in Form 3 Table 1, are intended to support the decisions pathways illustrated in Form 3 Overall Vertical Rating for Intermediate/Transitional Bed.

Form 3 Checklist 1: Armoring Potential

□	A	A mix of coarse gravels and cobbles that are tightly packed with <5% surface material of diameter <2 mm
□	B	Intermediate to A and C or hardpan of unknown resistance, spatial extent (longitudinal and depth), or unknown armoring potential due to surface veneer covering gravel or coarser layer encountered with probe
□	C	Gravels/cobbles that are loosely packed or >25% surface material of diameter <2 mm



Form 3 Figure 2. Armoring potential photographic supplement for assessing intermediate beds (16 < d_{50} < 128 mm) to be used i**n conjunction with Form 3 Checklist 1.**

(Sheet 2 of 4)

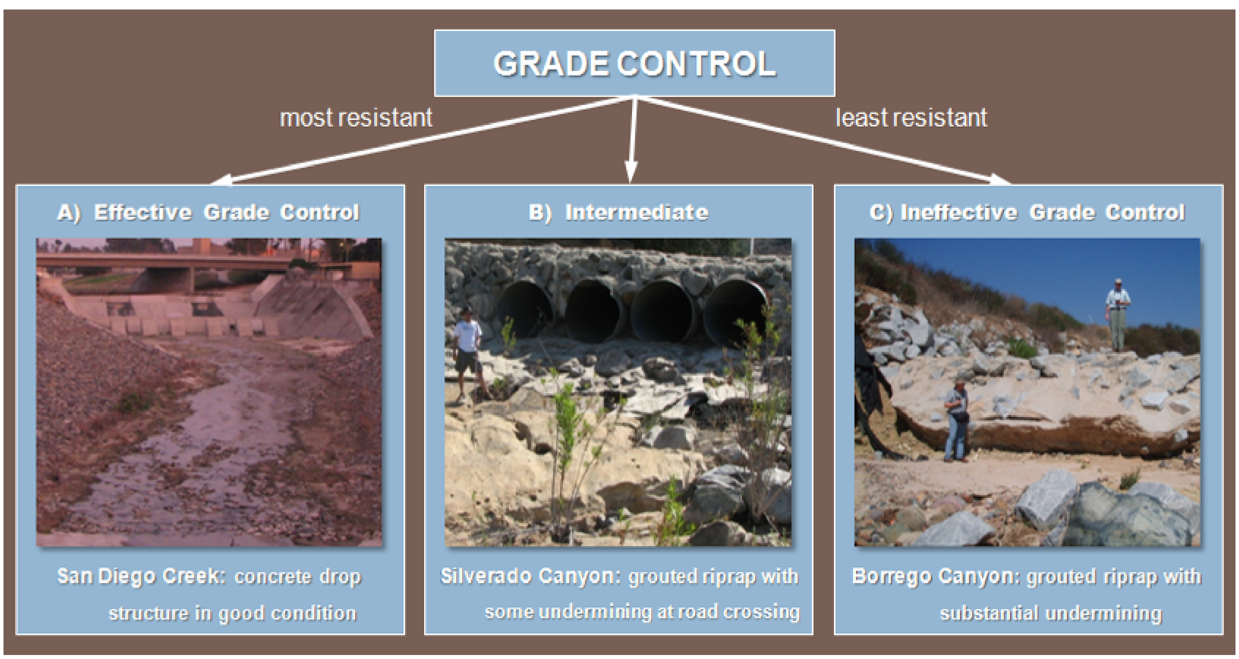
Form 3 Checklist 2: Grade Control

□ A Grade control is present with spacing <50 m or $2/S_v$ m

- No evidence of failure/ineffectiveness, e.g., no headcutting (>30 cm), no active mass wasting (analyst cannot say grade control sufficient if mass-wasting checklist indicates presence of bank failure), no exposed bridge pilings, no culverts/structures undermined
- Hard points in serviceable condition at decadal time scale, e.g., no apparent undermining, flanking, failing grout
- If geologic grade control, rock should be resistant igneous and/or metamorphic; For sedimentary/hardpan to be classified as 'grade control', it should be of demonstrable strength as indicated by field testing such as hammer test/borings and/or inspected by appropriate stakeholder

□ B Intermediate to A and C – artificial or geologic grade control present but spaced 2/Sv m to 4/Sv m or potential evidence of failure or hardpan of uncertain resistance

□ C Grade control absent, spaced >100 m or $>4/S_v$ m, or clear evidence of ineffectiveness

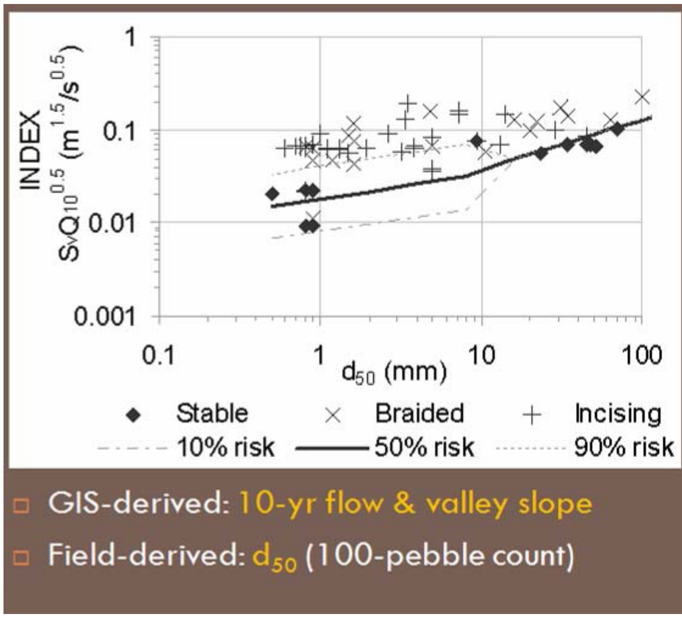


Form 3 Figure 3. Grade-control (condition) photographic supplement for assessing intermediate beds ($16 < d_{50} < 128$ mm) to be used in conjunction with Form 3 Checklist 2.

(Sheet 3 of 4)

Regionally-Calibrated Screening Index Threshold for Incising/Braiding

For transitional bed channels (d_{50} between 16 and 128 mm) or labile beds (channel not incised past critical bank height), use Form 3 Figure 3 to determine Screening Index Score and complete Form 3 Table 1.



Model Type	d_{50} (mm)	50% Risk $S_vQ_{10}^{0.5}$ ($m^{1.5}/s^{0.5}$)
Logistic Regression $d_{50} \geq 16$ mm	128	0.145
	96	0.125
	80	0.114
	64	0.101
	48	0.087
	32	0.070
	16	0.049
Logistic Reg. $d_{50} < 16$ mm	8	0.031
	4	0.026
	2	0.022
	1	0.018
	0.5	0.015

Form 3 Figure 4. Probability of incising/braiding based on logistic regression of Screening Index and d_{50} to be used in conjunction with Form 3 Table 1.

Form 3 Table 1. Values for Screening Index Threshold (probability of incising/braiding) to be used in conjunction with Form 3 Figure 4 (above) to complete Form 3 Overall Vertical Rating for Intermediate/Transitional Bed (below).. Screening Index Score: A = <50% probability of incision for current Q_{10}, valley slope, and d_{50}; B = Hardpan/d_{50} indeterminate; and C = ≥50% probability of incising/braiding for current Q_{10}, valley slope, and d_{50}.

d_{50} (mm) *From Form 2*	**S_v*$Q_{10}^{0.5}$ ($m^{1.5}/s^{0.5}$)** *From Form 1*	**S_v*$Q_{10}^{0.5}$ ($m^{1.5}/s^{0.5}$)** *50% risk of incising/braiding from table in Form 3 Figure 3 above*	**Screening Index Score (A, B, C)**

Overall Vertical Rating for Intermediate/Transitional Bed

Calculate the overall Vertical Rating for Transitional Bed channels using the formula below. Numeric values for responses to Form 3 Checklists and Table 1 as follows: A = 3, B = 6, C = 9.

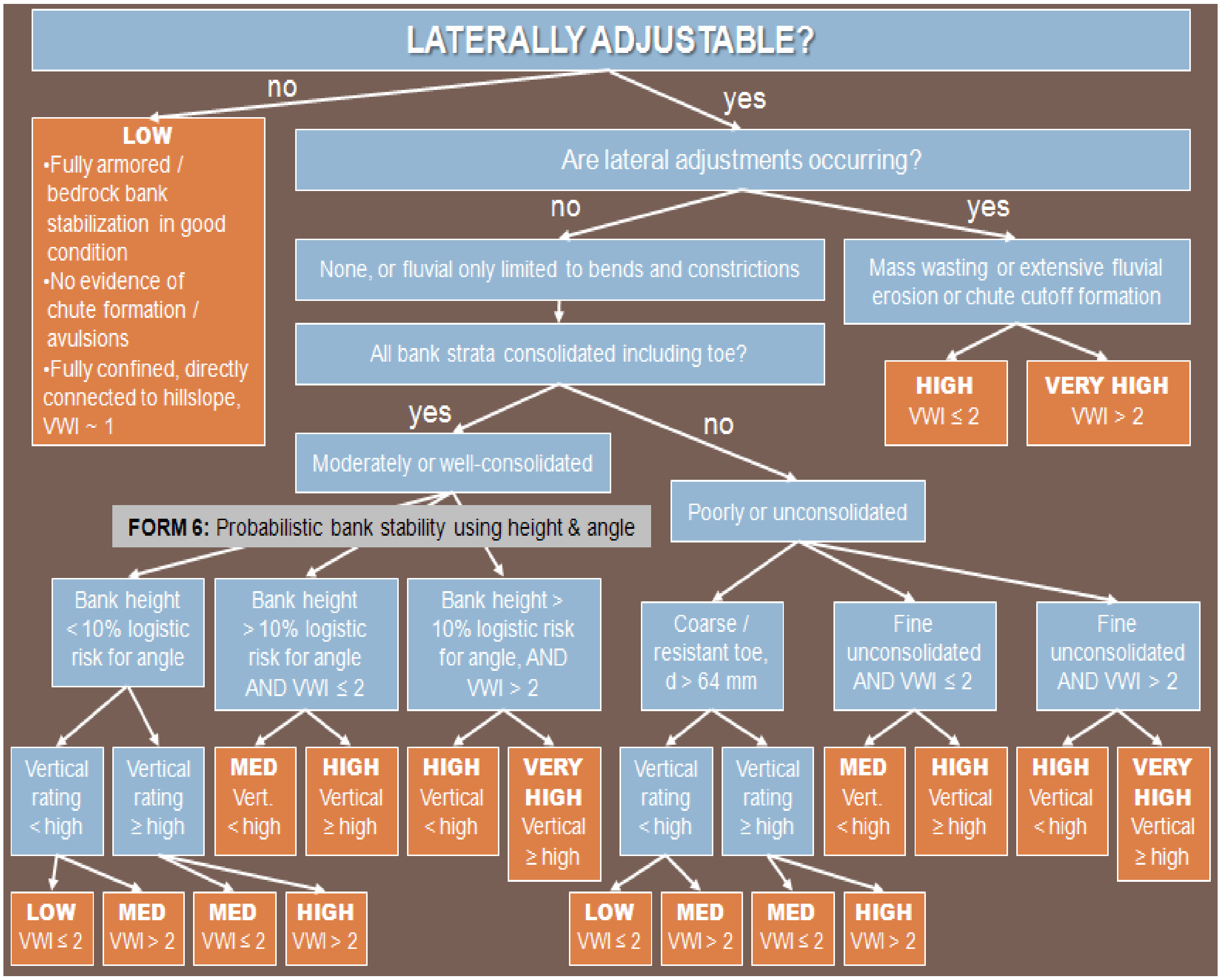
$$Vertical\ Rating = \sqrt{\{(\sqrt{armoring * grade\ control}) * screening\ index\ score\}}$$

Vertical Susceptibility based on Vertical Rating: <4.5 = LOW; 4.5 to 7 = MEDIUM; and >7 = HIGH.

(Sheet 4 of 4)

FORM 4: LATERAL SUSCEPTIBILTY FIELD SHEET

Circle appropriate nodes/pathway for proposed site
OR use sequence of questions provided in Form 5.



(Sheet 1 of 1)

FORM 5: SEQUENCE OF LATERAL SUSCEPTIBILITY QUESTIONS OPTION

Enter Lateral Susceptibility (Very High, High, Medium, Low) in shaded column.
Mass wasting and bank instability from Form 6, VWI from Form 4, and Vertical Rating from Form 3.

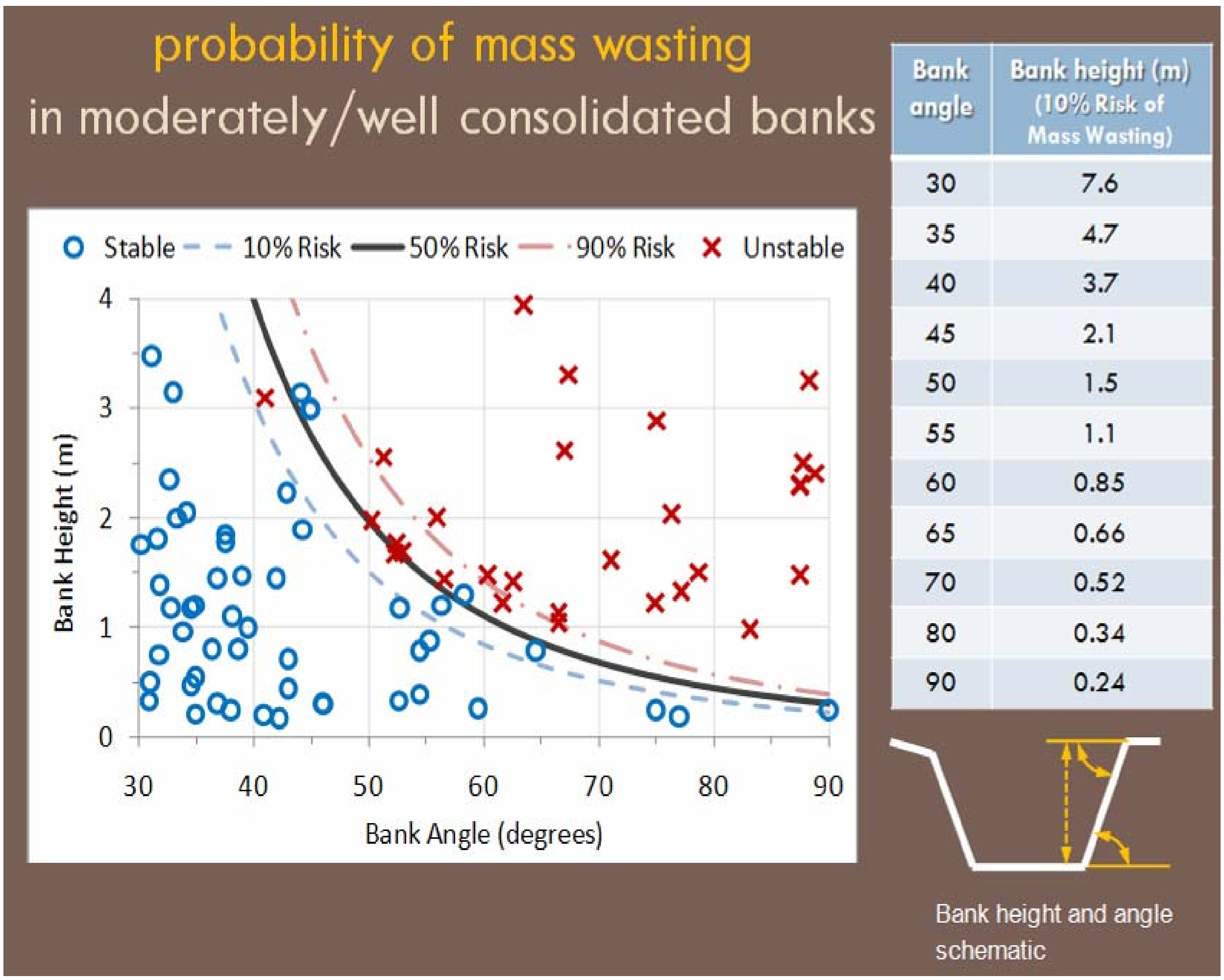
			Lateral Susceptibility
Channel fully confined with VWI ~1 – connected hillslopes OR fully-armored/engineered bed and banks in good condition?	If YES, then LOW		
If NO, Is there active **mass wasting** or extensive fluvial erosion (> 50% of bank length)?	If YES, VWI ≤ 2 = HIGH, VWI > 2 = VERY HIGH		
If NO, Are both banks consolidated?	If YES, How many risk factors present? **Risk Factors:** o Bank instability p > 10% o VWI > 2 o Vertical rating ≥ High	• All three = VERY HIGH • Two of three = HIGH • One of three = MEDIUM • None = LOW	
If NO, Are banks either consolidated or unconsolidated with coarse toe of d > 64 mm?	If YES, How many risk factors present? **Risk Factors:** o VWI > 2 o Vertical rating ≥ High	• Two = HIGH • One = MEDIUM • None = LOW	
If NO, At least one bank is unconsolidated with toe of d < 64 mm	How many risk factors present? **Risk Factors:** o VWI > 2 o Vertical rating ≥ High	• Two = VERY HIGH • One = HIGH • None = MEDIUM	

(Sheet 1 of 1)

FORM 6: PROBABILITY OF MASS WASTING BANK FAILURE

If mass wasting is not currently extensive and the banks are moderately- to well-consolidated, measure bank height and angle at several locations (i.e., at least three locations that capture the range of conditions present in the study reach) to estimate representative values for the reach. Use Form 6 Figure 1 below to determine if risk of bank failure is >10% and complete Form 6 Table 1. Support your results with photographs that include a protractor/rod/tape/person for scale.

	Bank Angle (degrees) *(from Field)*	**Bank Height (m)** *(from Field)*	**Corresponding Bank Height for 10% Risk of Mass Wasting (m)** *(from Form 6 Figure 1 below)*	**Bank Failure Risk** *(<10% Risk)* *(>10% Risk)*
Left Bank				
Right Bank				



Form 6 Figure 1. Probability Mass Wasting diagram, Bank Angle:Height/% Risk table, and Band Height:Angle schematic.

(Sheet 1 of 1)

APPENDIX C

Response to Coastkeeper Comments

BROWN AND CALDWELL

TECHNICAL MEMO

9665 Chesapeake Drive, Suite 201
San Diego, CA. 92123
Tel: 858-514-8822
Fax: 858-514-8833

Project No: 133904

San Diego Hydromodification Management Plan

Subject: Responses to Comments Provided by San Diego Coastkeeper

Date: February 16, 2010

To: Sara Agahi, P.E. – County of San Diego

From: Eric Mosolgo, P.E. – Brown and Caldwell

This draft technical memorandum has been prepared per the request of the County of San Diego to summarize responses to comments made in reference to the San Diego Hydromodification Management Plan (HMP) by San Diego Coastkeeper. These comments were submitted to the County of San Diego in letters dated April 14th, September 29th, and November 30th, 2009.

As mandated by Regional Water Quality Control Board (RWQCB) Order R9-2007-0001 Provision D.1.g, the purpose of hydromodification criteria is to prevent development-related changes in storm water runoff from causing, or further accelerating, stream channel erosion or other adverse impacts to beneficial stream uses.

The responses detailed in this memo have been incorporated into the Final HMP submitted to the San Diego Regional Water Quality Control Board (RWQCB) on December 29, 2009.

Responses to Coastkeeper Comments Dated November 30, 2009

Coastkeeper Comment – The inadequacies in applying LID are the HMP's most serious faults. They start with regarding LID as almost entirely a matter of infiltrating runoff, diminishing or ignoring the mechanisms of evapotranspiration and water harvesting and the practices associated with those mechanisms. Furthermore, the plan recommends basing infiltration assessments on coarse U.S. Department of Agriculture concepts and data instead of site-specific analysis and almost totally ignores the great potential of organic soil amendments to improve infiltration and evapotranspiration and reduce surface runoff quantities. The HMP reveals a poor appreciation of the status, performance, and practice of LID techniques today.

Response

- LID options modeled in determination of sizing factors account for both infiltration and evapotranspiration. Continuous simulation models are currently in development to determine the sizing factors for a wide range of development types, rainfall gauges, soil types, and BMP mitigation options. The evapotranspiration (ET) data is a key component of the continuous simulation models, along with the infiltration capacity of the soil, and is more certainly not an ignored mechanism. That said, BMPs studied in this analysis have to meet the County of San Diego's vector control guidelines along with the 85th percentile water quality and hydromodification standards. Thus, storage of runoff in excess of 72 hours will not be allowed.

- While water harvesting and reuse have obvious benefits, these criteria are not addressed or mandated in the Permit. From a hydromodification standpoint, water reuse facilities have some benefit for isolated rainfall events. When back-to-back storms occur, however, the hydromodification benefit is often not sufficient since the storage facilities are filled and provide no attenuation for the multiple concurrent storms. The use of rain water storage as a hydromodification control measure has not been ruled out. Rather, Copermittees can consider developer proposed storage facilities on a case by case basis. Such design strategies must prove compliance with hydromodification design criteria considering the long-term historical rainfall record.

- The Decision Matrix, located in Chapter 6 of the Final HMP, specifically states that site-specific geotechnical investigations be conducted to determine site-specific infiltration rates. Copermittees already require major development projects and many smaller projects to submit geotechnical soils reports which typically include identification of soil types. The referenced USDA information is part of the required Literature Review, which is located in Chapter 4 of the Final HMP. Infiltration parameters for the San Diego Region will be reviewed in details as part of the Sizing Calculator development process and further refined as part of the HMP implementation process.

- The use of amended soils has always been part of the HMP mitigation approach and the text of the Final HMP explicitly encourages the use of amended soils in the design of bioretention facilities. This concept is chronicled in both the HMP and the Model SUSMP. Similar to the approach used in Contra Costa County, several of the proposed BMP facilities will use an amended soil layer with an approximate infiltration rate of 5 inches per hour. Criteria provided in the Model SUSMP and HMP will work in concert. It should be noted that the use of amended soils will not promote deep infiltration for Types C and D soils, which are the dominant soil types in San Diego County. Thus, the use of underdrains may be required in urban environments.

- The Copermittees and the consultant team have developed detailed standards for LID implementation. These standards are provided in the Model SUSMP and are referenced in the Final HMP. The Final HMP recommends the use of LID facilities to satisfy HMP and 85th percentile water quality criteria.

- The intent of the HMP, as well as the Model SUSMP, is to encourage the use of LID facilities to meet hydromodification criteria. The text of Chapters 6 and 7 of the Final HMP were reviewed in detail and revised accordingly to encourage implementation of LID facilities.

- Defining the infiltration potential of a site is recommended to provide for sound engineering design. Even if infiltration is shown to be infeasible, LID facilities can be designed as filtration-type or evaporation-type facilities instead of infiltration-based facilities.
- Chapter 7 of the Final HMP has been revised to allow for evaporation-type facilities. It should be noted that such facilities may require implementation in series with more traditional LID approaches, such as biofiltration basins, in order to satisfy vector control and hydromodification criteria.

Coastkeeper Comment- Concerning the critical flow rate, the HMP presents an alternative to using a single value, a practice adopted elsewhere. The concept of multiple values is theoretically sound, but the plan falls short in specifying how the method it develops should be applied to assure proper use. Unless and until that gap can be filled, the appropriate single value, 10 percent of the 2-year flow event, should be used for the critical flow rate.

Response

- The San Diego HMP's varying lower flow threshold is a major advancement in the field of hydromodification management. This concept has been endorsed by the State Water Resources Control Board and other experts in the field. It is intuitive that erosion-prone streams should be held to a more stringent lower flow threshold as compared to erosion-resistant streams.
- Decision Matrices located in Chapter 6 clearly specify the method for determining the appropriate lower flow threshold. The method uses data from both the Southern California Coastal Water Research Project's (SCCWRP) channel screening tools (discussed in Chapter 5.2 and Appendix B) and the consultant team's critical flow calculator (discussed in Chapter 5.1) to determine the appropriate lower flow threshold.

Coastkeeper Comment – Exemptions put forward by the HMP fall into two categories: those that have been poorly thought through and, as presented in the plan, will continue to allow substantial hydromodification; and those that will forever consign degraded streams to that status. Both must be seriously reconsidered.

Response

- Exemptions proposed in the San Diego HMP have been thoroughly reviewed, discussed and analyzed.
- The exemption regarding projects that decrease both the pre-project impervious area and outlet discharge rates is logical. If the unmitigated post-project condition results in no increase to either impervious surface or resultant outflows as compared to pre-project conditions, then the project has no negative impact on downstream erosion .
- Exemptions regarding direct discharges to existing concrete channels have been thoroughly discussed with both the TAC and the Copermittee Work Group. This potential exemption was referenced in the Permit. A direct discharge to a concrete channel which connects to a

downstream exempt system poses an insignificant hydromodification related issue provided that the concrete channel has capacity to convey the ultimate condition 10-year flow. Note that if the downstream conveyance system passes through a stream segment susceptible to erosion, if the concrete channel does not have capacity to convey the ultimate condition 10-year flow, or if the project does not discharge directly to the existing concrete channel, then the existing concrete channel exemption may not be granted.

- Exemptions regarding direct discharges to large river systems have been analyzed using continuous simulation modeling and review of the resultant flow duration curves. This item has also been discussed in detail with the Copermittee Work Group, the TAC, and the Regional Board. This potential exemption applies only to river reaches with 100-year flows in excess of 20,000 cfs and drainage areas in excess of 100 square miles. The upstream limits of the specific potential exempt reaches, which are detailed in Table 6-1, were set based upon reach-specific review of the floodplain width, degree of upstream reservoir attenuation, etc. A detailed flow duration analysis was conducted to test the variability in flow duration curves based upon hypothetical additions of master development areas. Historical flow duration curves were based upon streamflow data in the San Diego River, as provided by USGS.

- Exemptions regarding urban infill projects in highly urbanized watersheds have been analyzed using continuous simulation modeling and review of the resultant flow duration curves. This item has also been discussed in detail with the Copermittee Work Group, the TAC, and the Regional Board. This potential exemption applies only to projects that discharge runoff directly to a stabilized conveyance system that extends beyond the Domain of Analysis. The exemption is only valid for watersheds with an existing impervious area of 40 percent or greater and with the potential for no more than a 3 percent impervious area increase in ultimate developed conditions (as compared to existing impervious area for the watershed). A detailed flow duration analysis was conducted to test the variability in flow duration curves based upon hypothetical additions of watershed impervious areas. It should be noted that the Permit allows for an exemption when the project discharges to a watershed with an existing impervious area percentage greater than 70 percent. Thus, this particular exemption is focused on highly urbanized watersheds containing an existing impervious area percentage between 40 and 70 percent.

Coastkeeper Comment – The subject of monitoring is only partially developed. At this stage it appears to be missing an in-stream component to determine if indeed the program is meeting its charge to manage channel erosion and impacts to beneficial uses and stream habitat.

Response

- As detailed in Chapter 8 of the Final HMP, in-stream monitoring is required at locations downstream of the monitored project site. Baseline cross section monitoring would be required prior to construction of the project. Subsequent cross section monitoring would then be required at defined intervals following construction of the site to assess effects of hydromodification mitigation controls.
- Chapter 8 of the Final HMP includes requirements for flow-based sediment monitoring. Results of such sediment monitoring can be used to determine the low flows which initiate sediment movement. This data can be used to further refine the low flow thresholds.
- Chapter 8 of the Final HMP also includes requirements for the monitoring of BMP inflows and outflows to assess BMP effectiveness. These protocols are similar to the monitoring requirements for the Contra Costa HMP.

Responses to Coastkeeper Comments Dated September 29, 2009

Coastkeeper Comment – The HMP is disconnected from the purpose and requirements of the MS4 permit. Following the first few meetings, we submitted an email that asked the TAC to take the opportunity to think more holistically and to stem the growing disconnect between the direction of the development of the HMP and the intent of the NPDES permit. We received assurances that the TAC and the consulting team were looking to take this opportunity to create "the most holistic HMP carried out to date in California." Unfortunately, this promise has not been kept. We understand the HMP must address erosion, but it must also address water quality issues. The Copermittee Working Group and TAC's silo approach may have devastating consequences down the line. When one regulatory effort moves forward without consideration of other ongoing efforts, implementation becomes impossible. This is especially true in light of significant movements by various Regional Boards (including San Diego Regional Board) to move toward a more holistic approach to MS4 Permit implementation.

Response

- Throughout the HMP development process, the Copermittees and the consultant team have held regular meetings with the Regional Board to discuss the approach. Through this process, the HMP direction has focused on the purpose and requirements of the MS4 permit.
- The San Diego HMP, Model SUSMP and subsequent implementation sizing tools explicitly recommend integrated facilities that provide for both water quality treatment and hydromodification flow control. The recommended implementation of Integrated Management Practices, such as LID bioretention basins, will provide for both 85th percentile water quality treatment and hydromodification flow control. Water quality issues have been addressed extensively in the Model SUSMP.

Coastkeeper Comment - The HMP inappropriately includes policy and compliance provisions. It appears the Copermittees misunderstand the role of the TAC itself. Throughout the HMP development process, *decisions have been made based not on science, but on "policy" grounds. For example, at the June HMP TAC meeting, a discussion centered on minimum orifice size for BMPs to meet HMP and flow rate requirements. The TAC recognized the conflict between the model and minimum orifice requirements predicted for fine-grained systems. In the end, the decision was labeled a "policy" choice to be made by the Copermittees. However, such decisions must be based on sound science to meet the goals of the Permit. The HMP contains other policy choices made by the TAC and Copermittee working group that are inappropriate for the technical document, and circumvent the Permit. For example, with regard to implementation of the HMP, restoration activities are listed as an alternative to compliance with flow control criteria. The Permit allows for implementation of such activities without adverse impacts to channel beneficial uses. However, the HMP proposes a cost-benefit analysis for implementation of the HMP design requirement. The Permit does not contain such "in-lieu of" language, nor can it be inferred from the Permit. Moreover, injecting such cost-benefit analysis into the Permit creates a loophole in implementation of the HMP. Such subjective analysis should not be part of the HMP in light of the mandate to "manage increases in runoff discharge rates and durations." Additionally, implementation of buffers, revegetation, etc. does not meet the twin roles of the HMP: addressing the "changes in a watershed's runoff characteristics resulting from development, together with associated morphological changes to channels receiving runoff." The in-lieu of planning measures does not address the change in watershed runoff characteristics. The HMP exemption for the lower third of the watershed is also an unsubstantiated policy decision. Impacts to all areas of a watershed need to be addressed. No support has been given for such an exemption, nor is it considered in the Permit. Runoff from impervious surfaces not only causes erosion, but also carries pollutants to receiving waters. As the Permit requires HMP implementation to prevent "significant adverse impacts to beneficial uses, attributable to changes in the discharge rates and durations," wholesale exemptions for portions of a watershed are inappropriate.*

Response

- As was the case in both the Santa Clara and Contra Costa HMPs, the San Diego HMP included some policy decisions. These policy decisions, which were ultimately made by the Copermittee Workgroup considering advice provided by the TAC, were based upon scientific investigations and analysis as well as practical considerations. The Hydromodification/SUSMP Workgroup was convened periodically over the course of the project at times corresponding with key decision points in developing the HMP and the update to the Model SUSMP. This workgroup was tasked with providing regional standards and consistency in the development, implementation, assessment, and reporting of urban runoff activities and programs related to hydromodification management. As required by Permit Section D.1.g, the Workgroup assisted in the development of the regional HMP. A key element of the San Diego HMP was the creation and involvement of a Technical Advisory Committee (TAC). The TAC members consist of respected individuals from academia, technical resource agencies, the development community, consulting engineers, and environmental organizations. The TAC, which has been convened on ten occasions that correlated with key decision-making points in the development of the HMP, was tasked with providing technical input to the HMP's scientific approach and interpretation of results integral to the establishment of numerical flow control standards as well as to the Copermittees for their policy determinations.

- Regarding the minimum orifice size issue, detailed analyses were prepared using continuous simulation hydrology to assess the effects of the minimum orifice size criteria. As a result, the minimum orifice size criteria may only be used in very limited scenarios to avoid problems resulting from clogged orifices and uncontrolled overflows. These scenarios are detailed in Chapter 6.2 of the Final HMP. The policy decisions regarding the minimum orifice size criteria were based on a detailed continuous simulation hydrologic analysis. This detailed analysis was combined with practical considerations regarding facility maintenance (specifically, the clogging of small orifices which would cause riser overflows and the potential for increased erosion downstream) to maximize HMP facility effectiveness.

- Regarding the stream restoration / rehabilitation options, this issue was fully discussed with the Regional Board, the TAC, and the Copermittee Work Group. As worded in the Final HMP, such channel rehabilitation options may be constructed in limited situations. Specifically, such options may only be constructed if the existing channel susceptibility is determined to be "High" (as determined by SCCWRP assessment), if the stream rehabilitation project extends downstream to an HMP exempt system, and if the stream rehabilitation project is constructed assuming ultimate development conditions upstream of the project. Details of the stream rehabilitation protocols are detailed in Chapter 6.3 of the Final HMP. Additionally, permits from resource agencies are necessary in most cases, and improvement to habitat and the environment are expected.
- The Final HMP contains no mention of a cost-benefit analysis regarding stream rehabilitation measures. However, developers will ultimately use cost-benefit analyses when selecting alternative methods for meeting Permit requirements.
- The final HMP contains no mention of the "lower third of the watershed" exemption.

Coastkeeper Comment – TAC consensus has been misrepresented to the Regional Board. Recently, we have become aware of the Copermittees misrepresentation of TAC consensus regarding decisions made in developing the HMP. Our continuing disagreements with the current conclusions of the draft HMP are evident from: our emailed comments submitted by Karen Franz on February 2, 2008; our comment letter from our expert Dr. Horner; submitted on April 14, 2009; and our requests for underlying technical data to support the HMP. Following the receipt of the responses to comments from Dr. Horner, we requested the supporting references and technical papers that were the basis for the development of the design storm formulation for the Santa Clara and Contra Costa HMPs. The request was made at the June 17th meeting, and no communication of the references or technical papers followed the request. Further, the draft HMP was not give to TAC members until after it was first presented to the Regional Board. A TAC meeting was held in October 2008, and another meeting was not held until February 2009. In the interim, the consultants met with the Copermittee working group, obtained approval of the draft HMP, and submitted it to the Regional Board. It was not until February 4, 2009 that TAC members were sent an electronic copy of the HMP. We obtained a physical copy of the draft HMP at the Copermittee meeting in January shortly after it was submitted to the Regional Board and before it was sent to the TAC. TAC consensus and approval are also misrepresented on key issues, such as HMP compliance through "no increase to pre-project impervious area and no increase to pre-project flow." Contrary to the document assertion, this has not been "discussed and approved by the TAC." Coastkeeper has and will continue to insist upon natural, pre-project flows and reduction in overall impervious area.

Response

- In the Final HMP, the phrase "majority TAC approval" was used to indicate the majority opinion of the cumulative TAC members.
- Technical memos detailing the preparation of the Santa Clara and Contra Costa HMPs are public information and located at the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPP) web site and the Contra Costa Clean Water Program (CCCWP) web site.
- The majority of the members of the TAC agreed that HMP requirements should not be imposed on developments that decrease the pre-project impervious cover and decrease the design flows to each outlet location.

Coastkeeper Comment – Coastkeeper's effectiveness has been stymied by a lack of transparency and unavailability of key documents. Coastkeeper concurs in the Regional Board's comments made on June 29, 2009. The lack of detail and transparency highlighted in the letter has been a particular concern for Coastkeeper as well. For instance, the BMP sizing tools and their reporting should be a transparent process. Although the tools go beyond the scope of the HMP development, they are a necessary piece of the process, and as such, the HMP should provide more oversight on their use. Additionally, Coastkeeper's specific comments from our technical expert Dr. Horner remain largely ignored or dismissed out of hand. Even to get an electronic copy of the draft HMP for our expert to review proved challenging. Several attempts were made to request the document by email, without success. We were ultimately forced to scan a paper copy we obtained from a Stormwater Copermittee meeting where the draft HMP was distributed. At a TAC meeting following submission of the comment letter, several TAC meeting attendees and members opined about the radical nature of our comments and marginalized Coastkeeper. This type of discussion is indicative of the limited role Coastkeeper was able to play in participating on the TAC. This process of excluding the TAC from critical decision-making, and information exchange has also hindered the usefulness of the TAC.

Response

- All documents prepared in association with the Final HMP are available for public review. These documents were presented on multiple occasions for review by the TAC, Copermittee Workgroup and the Regional Board. These documents are posted on the Project Clean Water web site.
- The BMP sizing tool development is a transparent and ongoing process. These are implementation tools and were not required as part of the HMP document. Key technical memos and data reviews will be circulated to the TAC, Copermittee Working Group and Regional Board throughout the Sizing Calculator development process.
- Dr. Horner's comments have been addressed in previous comments response document and in this comment response document.

Coastkeeper Comment – A lack of data inhibits progress. In addition to the lack of transparency in information exchange by consultants and Copermittees to TAC members, the delay in production of key *aspects of the HMP prohibits meaningful input from the TAC. For example, the San Diego region has three distinct geomorphic and hence geologic regions. The geologic conditions of a watershed/catchment area are the factors affecting the low flow threshold values. Other critical components that may never be reviewed by the TAC include development of maintenance and long-term monitoring protocols and the required approval process for Priority Development Projects. The incorporation of these tools into the decision matrix and preparation of consultant technical memos are critical steps in the HMP which have yet to be conducted, and may largely take place outside of the TAC.*

Response

- The HMP was submitted on time to the Regional Board on December 29, 2009.
- Prior to the final submittal, multiple iterations of the HMP document and supporting memos were distributed to the TAC, Copermittee Work Group and the Regional Board.

Coastkeeper Comment – Exemptions remain ill-conceived and overused. The Draft HMP makes exemptions for hardened channels as arguably allowed by the current Permit, but these exemptions are neither required nor prudent. First, the Permit language gives some discretion to the Copermittees, not requiring exemptions and qualifying such decisions with the requirement not to impact beneficial uses. Moreover, the proposed South Orange County stormwater permit specifically requires hydromodification considerations for restoration of such hardened channels. Also, the Copermittees attempt to create an exemption for projects with "no net increase" in impervious area is also not in line with the Regional Board's interpretation of "pre-project" as highlighted in the proposed South Orange County Permit. Therefore, pre-project conditions in the current Permit should not make exceptions for "no net increase" unless such projects mimic naturally occurring conditions. Further, the "adoption and implementation of this NPDES permit relieves the Copermittee from developing a non-point source plan, for the urban category, under CZARA." CZARA requires implementation of management measures to prevent non-point source pollution from impacting or threatening coastal water quality. Therefore, exemptions for the lower portions of watersheds or large receiving waters are not allowed.

Response

- The exemption regarding projects that decrease both the pre-project impervious area and outlet discharge rates is logical. If there no increase to either impervious surface or resultant outflows as compared to pre-project conditions, then the project has no negative impact on downstream erosion.
- Exemptions regarding direct discharges to existing concrete channels have been thoroughly discussed with both the TAC and the Copermittee Work Group. This potential exemption was referenced in the Permit. A direct discharge to a concrete channel which connects to a downstream exempt system poses an insignificant hydromodification related issue provided that the concrete channel has capacity to convey the ultimate condition 10-year flow. Note that if the downstream conveyance system passes through a stream segment susceptible to erosion, if the concrete channel does not have capacity to convey the ultimate condition 10-year flow, or if the project does not discharge directly to the existing concrete channel, then the existing concrete channel exemption may not be granted.

- Exemptions regarding direct discharges to large river systems have been analyzed using continuous simulation modeling and review of the resultant flow duration curves. This item has also been discussed in detail with the Copermittee Work Group, the TAC, and the Regional Board. This potential exemption applies only to river reaches with 100-year flows in excess of 20,000 cfs and drainage areas in excess of 100 square miles. The upstream limits of the specific potential exempt reaches, which are detailed in Table 6-1, were set based upon reach-specific review of the floodplain width, degree of upstream reservoir attenuation, etc. A detailed flow duration analysis was conducted to test the variability in flow duration curves based upon hypothetical additions of master development areas. Historical flow duration curves were based upon streamflow data in the San Diego River, as provided by USGS.

- Exemptions regarding urban infill projects in highly urbanized watersheds have been analyzed using continuous simulation modeling and review of the resultant flow duration curves. This item has also been discussed in detail with the Copermittee Work Group, the TAC, and the Regional Board. This potential exemption applies only to projects that discharge runoff directly to a stabilized conveyance system that extends beyond the Domain of Analysis. The exemption is only valid for watersheds with an existing impervious area of 40 percent or greater and with the potential for no more than a 3 percent impervious area increase in ultimate developed conditions (as compared to existing impervious area for the watershed). A detailed flow duration analysis was conducted to test the variability in flow duration curves based upon hypothetical additions of watershed impervious areas. It should be noted that the Permit allows for an exemption when the project discharges to a watershed with an existing impervious area percentage greater than 70 percent. Thus, this particular exemption is focused on highly urbanized watersheds containing an existing impervious area percentage between 40 and 70 percent.

- The San Diego HMP complied with permit provision for the San Diego region, not the South Orange County permit.

Coastkeeper Comment – Selection and implementation of BMPs are vague or missing. The Draft HMP does not provide a list possible preferred BMPs, and the explanation of BMPs thus far at TAC meetings have been equally vague. At the outset we find that the BMP specific design criteria will be much more useful and transparent. It is unclear why the TAC has not chosen this route. Additionally, although the age of a BMP system has a great influence on the efficacy of that BMP, no provisions or requirements exist to address this issue. We have also asked to include infiltration and rainwater harvesting in the list of BMPs, but apparently only dry wells have been added so far. San Diego's reliance on imported water and its precipitation patterns create a tremendous regional opportunity for the development of rainwater harvesting systems to not only capture and reuse this resource, but also to reduce flow (and sediment) from Priority Development Projects. The Ventura County permit requires all features constructed to render impervious surfaces "ineffective:" to "infiltrate, store for reuse, or evapotranspire, without any runoff at least the volume of water that results from" the 85th percentile, 24-hour runoff event, annual runoff based on unit basin storage to achieve 80 percent or more volume treatment, or a 0.75 inch storm event. The San Diego HMP should contain greater emphasis on infiltration, reuse and evapotranspiration as well.

Response

- Chapter 7 of the Final HMP and the Model SUSMP include a suite of BMPs that can be used for water quality treatment and hydromodification flow control. The suite of BMPs listed, including bioretention basins, biorentention in series with cisterns, bioretention in series with vaults, extended detention basins, and flow-through planter boxes, corresponds to the BMP selection list that will be provided in the Sizing Calculator.
- While water harvesting and reuse have obvious benefits, these criteria are not addressed or mandated in the Permit. From a hydromodification standpoint, water reuse facilities have some benefit for isolated rainfall events. When back-to-back storms occur, however, the hydromodification benefit is often not sufficient since the storage facilities are filled and provide no attenuation for the multiple storms. The San Diego permit does not require rainwater harvesting for hydromodification mitigation. The use of rain water storage as a hydromodification control measure has not been ruled out. Rather, Copermittees can consider developer proposed storage facilities on a case by case basis. Such design strategies must prove compliance with hydromodification design criteria considering the long-term historical rainfall record.
- The 5 percent EIA requirement from the Ventura permit is not included in the San Diego MS4 permit.

Coastkeeper Comment – The HMP does not consider climate and land use change. Effects of climate and land use changes on low flows and other hydrologic responses have been well documented as to the hydrological effects that will result in our region. When employed singly and in combination, climate and land use changes have significant and varying effects on flow conditions. The draft HMP contemplates only one rate of land-use change. The HMP needs to consider the potential impacts of climate change and the effects that it will have on regional hydrologic conditions through its modeling. Hydrologic data is being generated by the Hydrologic Research Center, a San Diego-based international research center.

Response

- While climate change effects were not considered in this version of the HMP, it is possible that the rainfall data sets prepared in association with the HMP could be updated once predictive rainfall models have been developed. These data sets could be used to refine recommendations of future HMP updates.

Coastkeeper Comment – Implementation of a standard of 3 percent maximum allowable Effective Impervious Area (EIA) in all regulated projects, with a narrowly crafted alternative compliance provision for developments where severe site constraints, such as non-infiltrative soils, render compliance with the 3 percent EIA limitation impossible.

Response

- The Effective Impervious Area (EIA) requirement was not part of the San Diego MS4 permit.

Coastkeeper Comment – As a hydromodification standard, post-development peak flow rates and volumes shall not exceed the modeled peak flow rates and volumes of pre-European-settlement native land cover for all storms from the channel-forming event to the 100-year frequency stream flow. This requirement shall be satisfied to the maximum possible extent by retention of runoff on the development site through infiltration, evapotranspiration, and/or rainwater harvesting. If the requirement cannot be fully met by onsite retention, there shall be a demonstration and convincing justification, according to specific criteria, of why it is not achievable at that site. If such a convincing demonstration and justification can be made, the differential between the required retention and the amount that can be provided onsite shall be offset by performing or contributing to an offsite project, within the same watershed, to retain an equal or greater volume of runoff from such other site.

Response

- The hydromodification standard, as interpreted from the San Diego MS4 permit, requires the control of peak flows and durations within the geomorphically significant flow range to pre-project conditions. No mention of pre-European settlement is included in the San Diego MS4 permit.

Coastkeeper Comment – Monitoring of HMP compliance must be conducted at more than 5 sites in the entire County. At least one site per watershed must be monitored. Additionally, monitoring should begin before development, not after completion. Monitoring site selection should also be made with Regional Board staff input, not solely by Copermittees.

Response

- No HMP monitoring plan in the State of California proposes more than 5 countywide monitoring sites. The recommendations detailed in Chapter 8 exceed the requirements for Contra Costa County as approved by the San Francisco Regional Board.
- As detailed in Chapter 8 of the Final HMP, monitoring will begin before development and extend into the future following development.

Coastkeeper Comment – Individual Priority Development Projects must be required to monitor effectiveness and maintain HMP BMPs and compliance measures. A real, tangible monitoring mechanism and compliance determination must be implemented into the HMP. Without such requirements in the HMP, no assurance of long-term effectiveness will be provided. Such tools would also help Copermittees monitor specific BMP effectiveness in different watersheds.

Response

Monitoring of the 5 sites will be a regional Copermittee effort. Individual Priority Development Projects are required to inspect and maintain their treatment control and HMP facilities through maintenance agreements. Additionally, Copermittees conduct annual inspections of treatment BMPs and HMP facilities as required by the Municipal Permit.

Coastkeeper Comment – Urge the Regional staff to ensure strict compliance with the current Permit and look toward future consistency with other MS4 Permits in southern California, as setting the MEP standard.

Response

- We will defer to the Regional Board for a response to this comment.

Coastkeeper Comment – Future development, implementation, and monitoring of the HMP should be more transparent, including more availability for public input.

Response

- We will continue to provide technical memos and materials available for public review through the TAC, Copermittee Work Group and the Regional Board. These documents can be accessed at the Project Clean Water web site.

Coastkeeper Comment – High, Medium and Low susceptibility ratings should be removed. All watersheds should be treated as susceptible to erosion. Moreover, the classification of streams does not correlate to an appropriate HMP objective. For instance, for already unstable channels the standard is to "avoid acceleration of the existing erosion problems." This is unacceptable, and does not meet the spirit of intent of the Permit.

Response

- Stream classification, as provided for in this HMP by the SCCWRP channel susceptibility analysis, is a requirement of the MS4 permit (Permit Section D.1.g.(1)(a) and (m)). Therefore, this information will not be removed from the HMP. It is a critical component of the HMP for San Diego County and all counties in southern California.

Responses to Coastkeeper Comments Dated April 14, 2009

Coastkeeper Comment - Comparing the stated San Diego County criteria to hydromodification standards elsewhere, the County's criteria are relatively highly protective of runoff receiving waters in the cases of flows of 5- and 10-year frequencies. On the other hand, these criteria do not extend to the larger storms of less frequency. Some hydromodification criteria cover a range of storms up to the 50- and even 100-

year events. In the central city area of San Diego, rainfalls of 24-hour duration for different frequencies are approximately (http://ponce.sdsu.edu/noaa_24hr_sd_2x.html): 5-year—2.4, 10-year—2.8, 50-year—3.5, and 100-year—4.1 inches. Thus, it may be seen that extending the assessment from the 10- to the 100-year frequency enlarges the time period over which resource protection is evaluated by an order of magnitude (1000 percent) with an increase of just 46 percent in the rainfall quantity. The criteria should be extended to these larger storms, or the County should show why doing so is not necessary to protect and recover stream ecosystems.

Response

- Similar to the two previously approved hydromodification management plans in the State of California (Santa Clara County and Contra Costa County), the San Diego Final HMP recommends flow and duration control for a range of flows between a fraction of the 2-year flow event to the 10-year flow event. Neither the approved Santa Clara HMP nor the approved Contra Costa HMP required controls for flow recurrence events in excess of the 10-year design flow.
- The referenced 24-hour rainfall totals in the comment above refer to a single-event design storm approach, which is not applicable with the continuous simulation hydrologic modeling approach mandated in Permit R9-2007-0001. The Permit goes on to say that determination of peak flow frequency values shall be developed from analysis of the full rainfall record. In other words, hourly data from the entire rainfall record (35 to 50+ years) is used in the analysis as opposed to use of a singular rainfall depth as noted in the comment above.
- Finally, it should be noted that various geomorphologists across California and the nation have concurred that controls above the 10-year flow event have a minimal impact on cumulative sediment movement across the historical record. Sediment transport studies based on a continuous flow record, such as the long-term analysis prepared in association with the Santa Clara Hydromodification Management Plan, have shown that roughly 90 percent of the cumulative work exerted on a channel occurs within the relative flow ranges detailed in the Santa Clara, Contra Costa, and San Diego HMPs. Thus, it can be demonstrated that the significant cost associated with controls above the 10-year event would not result in significant additional protection to the stream processes from a hydromodification standpoint.

Coastkeeper Comment - Criteria setting is, "... based on the understanding that the 5-year design flow is considered the dominant channel-forming discharge for Southern California streams." If the basis is merely an "understanding", it is not strong enough. The basis must be rooted in detailed analyses. Such analyses elsewhere in the nation have identified flows having frequencies around 1.5 to 2-year to be the channel-forming discharges.

Response

- Per the Final HMP, lower flow threshold criteria were based upon a fraction of the 2-year design flow. This determination was made using a synthetic modeling approach which used the continuous rainfall record to determine hydrologic response. Sediment transport models were then simulated for the entire historical record for a wide variety of channel conditions.

- The commentary regarding the 5-year design flow in the comment above was provided in reference to determination of interim flow control standards. As a reasonable first step for the setting of the interim standards, initial determinations were made based upon previous research conducted by the Southern California Coastal Water Research Project (SCCWRP) and others. The final flow control standards are based upon detailed hydrologic and sediment transport analyses.

Coastkeeper Comment - The plan contains exemptions from requirements that will foreclose future stream restoration options, or at least substantially increase their difficulty. One such instance is allowance of planning measures as alternatives in lieu of stormwater flow controls. Another is the allowance of a demonstration that projected increases in runoff peaks and/or durations will not accelerate stream channel erosion. The plan further provides a dispensation for controls if a project applicant conducts a sediment transport analysis and shows no adverse impact. Such demonstrations could be convincingly made when a channel is hardened or already cut to bedrock, but each permitted increment of flow further reduces the opportunity to recover a natural stream, and its ecological values. The plan goes on to state specifically that hydromodification management flow controls will not be required for discharges into hardened channels or the downstream sub-watershed imperviousness is at least 70 percent and the potential for cumulative impacts is "minimal". This policy essentially consigns these channels perpetually to their artificial, highly degraded status with almost no ecological function. These exemptions should be removed, at least until a broad assessment of restoration potential can be completed and the most opportune cases prioritized for implementation.

Response

- The exemptions listed in the HMP closely follow recommendations provided in Permit R9-2007-0001, especially with regard to discharges to existing hardened channels, storm drain systems, and into existing highly urbanized watersheds (with a percent imperviousness > 70%).
- Planning measures such as implementation of Low-Impact Development (LID) facilities would still be required to demonstrate that the mitigated condition would meet mandated flow and duration control criteria.
- Planning measures such as the implementation of riparian buffers or non-hardened stream restoration/rehabilitation projects would require mitigation proof in the form of an accompanying hydraulic and/or sediment transport analysis of sufficient technical rigor. The HMP does not allow for the implementation of concrete channel solutions as a method for stream restoration/rehabilitation.

Coastkeeper Comment - The plan is silent on how the potential for cumulative impacts can or should be assessed and what "minimal" is. It should be explicit on these subjects.

Response

- Chapter 5.3 provides a discussion of cumulative watershed impacts.

- Definition of cumulative watershed impacts was quantified in the detailed continuous simulation models prepared in association with the river system exemption, highly urbanized watershed scenario, and minimum orifice size. This discussion is detailed in Appendix F.

Questions related to this comment response document should be directed to Sara Agahi at (858) 694-2665.

APPENDIX D

Flow Threshold Analysis Third Party Review

December 19, 2008

Mrs. Sara Agahi, P.E.
County of San Diego.
5201 Ruffin Road, Suite D
San Diego, CA 92123

Subject: Review of Hydromodification Work by Phillip Williams and Associates (PWA)

Dear Mrs. Agahi:

This letter summarizes our findings from review of the subject PWA work as subconsultant to Rick Engineering Company and as authorized under County of San Diego Agreement Number 525773, Task Order Number 5.

According to the County of San Diego (the County) Municipal Separate Storm Sewer System (MS4) permit, the Hydromodification Program (HMP) must use standards to manage increases in runoff discharge rates and durations where these are likely to cause increased erosion of channel bed and banks, sediment pollutant generation, or other impacts to beneficial uses and stream habitat due to increased erosive force. Under the permit's definition of "flow duration" it is noted that flow duration within the range of geomorphically significant flows is important for managing erosion. The permit also requires that the HMP be based on continuous rainfall-runoff modeling. The purpose of the work by PWA is to help establish the flow thresholds for use with the County HMP.

The review consisted of examining the underlying assumptions of the analyses, the methodology followed in the analyses themselves (including the modeling techniques employed), development of results from the analyses, and conclusions reached based on those results. The data, analyses and models submitted for review were contained on a portable hard drive provided by Brown & Caldwell on 11/20/08. A memorandum from PWA to the County of San Diego dated 11/12/08 describing the watershed and channel modeling was also provided by Brown & Caldwell via e-mail on 11/25/08. Other background data was gathered from the periodic reports submitted by Brown & Caldwell and/or PWA to the Technical Advisory Group.

Method

The general methodology employed by PWA was to conduct a large simulation-based sensitivity analysis to cover the range of potential channel and watershed conditions found in western San Diego County. Three sample watersheds within the size to be regulated by the HMP were chosen in areas where development is expected to occur. Specifics of the analyses are commented upon below.

Hydrology

The hydrology for each site was developed using the San Diego Hydrology Model (SDHM) for pre-development, post-development, and post-development with flow mitigation (one, one, and six simulations, respectively). WEST verified the input data contained in the SDHM models for the Otay (Rolling Hills) and Peñasquitos basins. Tables 1 and 2 present a summary of pond sizes, outlet dimensions, and LID parameters (infiltration rate and reduction factors) for each of the scenarios for the two watersheds.

In the Otay input files, the same outlet dimensions (notch width, height, and orifice diameter) were maintained for each flow duration criteria simulation (10% Q_2, 10% Q_5, and 20% Q_5) for both the "non-LID" and "with-LID" cases (see Table 1). The pond size changes slightly, decreasing in the "with-LID" case because flow is lost through infiltration and the pond size can decrease while still meeting the duration criteria. However, WEST found that in the 10% Q_2 scenario, the riser diameter is set to 400 inches, while it is fixed at 48 inches for all other scenarios. The corresponding pond size changes from a square 750 feet on each side for the 10% Q_2 scenario to one 318 feet on each side for the 10% Q_2 with LID scenario. WEST suggests changing the diameter to 48 inches and re-running the simulation.

For the Peñasquitos watershed analyses we observed that while the riser dimensions were the same for all simulations, no consistent choice of notch height, width, and orifice diameter was maintained.

In addition, the SDHM uses only rainfall data from the Lindbergh Field gage in the simulations. Potential pitfalls with this assumption have already been pointed out by Brown & Caldwell elsewhere. All simulations used a 40-year period of record from this gage as input and runoff hydrographs were generated for the eight cases discussed above for each of the sample basins. Eight cases multiplied by three basins resulted in a total of 24 hydrologic simulations.

PWA assumptions for land use (land cover, vegetation, percent impervious) for the test watersheds were not confirmed by measurement in a geographic information system (GIS), but seemed reasonable by inspection.

The assumption that all runoff would be routed into a single runoff control facility is probably not realistic (especially given the resulting single basin sizes compared to the overall watershed area), but is justified for this type of comparative analysis.

Table 1. Otay SDHM Parameters

OTAY	Pond Length (ft)	Pond Width (ft)	Depth (ft)	Riser Height (ft)	Riser Diameter (in)	Notch Height (ft)	Notch Width (ft)	Orifice diameter (in)	Pond Volume at Riser Head (ac-ft)	Infiltration Rate (in/hr)	Reduction Factor	Percent Infiltrated
10% Q_2	750	750	5	4	400	0.0954	4	0.86	54.09	n/a	n/a	n/a
10% Q_5	232	232	5	4	48	0.0878	3.9584	5.543	5.55	n/a	n/a	n/a
20% Q_5	211	211	5	4	48	0.0954	3.94	7.9314	4.624	n/a	n/a	n/a
10% Q_2 with LID	318	318	5	4	48	0.0954	4	0.86	10.154	0.7	0.25	88.74
10% Q_5 with LID	225	225	5	4	48	0.0878	3.9584	5.543	5.241	0.7	0.25	41.65
20% Q_5 with LID	210	210	5	4	48	0.0954	3.94	7.9314	3.5	0.7	0.25	34.47

Table 2. Peñasquitos SDHM Parameters

PEÑASQUITOS	Pond Length (ft)	Pond Width (ft)	Depth (ft)	Riser Height (ft)	Riser Diameter (in)	Notch Height (ft)	Notch Width (ft)	Orifice diameter (in)	Pond Volume at Riser Head (ac-ft)	Infiltration Rate (in/hr)	Reduction Factor	Percent Infiltrated
10% Q_2	307	307	7	6	72	0.14	6.00	2.82	14.73	n/a	n/a	n/a
10% Q_5	172	172	7	6	72	0.2835	5.94	7.7877	5.041	n/a	n/a	n/a
20% Q_5	162	162	7	6	72	0.178	5.94	10.673	4.54	n/a	n/a	n/a
10% Q_2 with LID	251	251	7	6	72	0.178	5.94	2.9834	10.121	0.7	0.25	53.82
10% Q_5 with LID	179	179	7	6	72	0.3185	5.94	8.2677	5.445	0.7	0.25	24.13
20% Q_5 with LID	183	183	7	6	72	0.2626	6	7.5051	5.632	0.7	0.25	25.2

Hydraulics and Sediment Transport

As opposed to the site-specific characteristics employed in the hydrologic analysis, the hydraulic and sediment transport analyses appear to be completely hypothetical in nature. The eight hydrographs produced from the SDHM simulations previously discussed were used as input to the HEC-RAS hydraulic and sediment transport model. Other key input parameters such as cross section geometry, channel slope, roughness, and sediment characteristics were selected to cover a "representative" range corresponding to potential field conditions. Forty-two combinations of basin, grain size, slope and width-to-depth ratio were simulated in HEC-RAS for each of the eight hydrologic scenarios resulting in at least 336 models created and executed (additional models were created for sensitivity analyses). The volume of sediment leaving a "project reach" over the 40-year simulation for each of the post-development analyses were compared with pre-development yield and the results interpreted to select the minimum flow rated that should be regulated in the HMP.

Significant time and effort was obviously spent in preparing, executing and debugging the numerous models. Model instabilities led to using a sediment rating curve approach for computing sediment yield. There are significant issues regarding the modeling and computations which throw the validity of the results and the conclusions drawn from them into question. Specific comments are provided in the following sections.

Cross Section Geometry

The synthetic cross section geometry (width and depth) used for the analyses was generated using empirical relationships developed from various sources. These include equations for gravel-bed rivers in the UK and US, relations for sand-bed streams, and regression equations developed from measurements of Southern California streams (references in PWA memorandum of 11/12/08). Application of some of these equations to San Diego Country streams is problematic, while other similar equations developed from US data (e.g., Lee and Julien[1]) were not employed. In any case, cross sections were developed by imposing a small "bankfull" channel at the bottom of a v-shaped section with 10% side slopes (10 horizontal feet for each 1 vertical foot). Width to depth (W:D) values were computed using the various methods, and a set of width to depth values were chosen, apparently only loosely linked to the specific method results. A trapezoidal channel containing three bottom points was created at the bottom of each cross section. Based on spreadsheets and models provided W:D ratios of 3, 6, and 10 were run for both the Peñasquitos and Otay sites. A W:D ratio of 20 was also used for the Otay site for certain combinations of grain size and slope. Channel depths in HEC-RAS, based on the equation results, were set between 0.25 to 0.5 feet for Peñasquitos and 0.7 feet or less for Otay. Therefore, even though numerous combinations of W:D ratio were used, the absolute dimensions were still very small (for a depth of 0.5 feet the top width would vary from 1.5 feet to 5 feet for W:D ratios of 3 and 10, respectively).

Several of these cross section geometry relationships rely on bankfull or channel-forming discharge as an input parameter. This discharge was estimated by PWA using USGS regression

[1] "Downstream Hydraulic Geometry of Alluvial Channels," Lee and Julien, *Journal of Hydraulic Engineering* Vol. 132, No. 12, December 2006.

equations[2]. Although it is recognized that few methods exist outside of site-specific investigations to estimate this discharge, the USGS equations are known to be very approximate. The equations are based on gauging stations from Santa Barbara to San Diego, from the coast to elevations above 5,000 feet and using data available in 1975. The mean annual precipitation (MAP), also an input to the regression equation, was assumed to be 15 inches for this exercise although it does vary from about 9 inches at the coast to over 25 inches at higher elevations. The MAP at both the Peñasquitos and Otay sites is close to 12 inches. It could be argued that a 5-year return interval would be a more appropriate indicator of bank-full discharge than a 2-year flow, but this is a topic that is still being researched and is far from resolved for semi-arid regions such as San Diego.

Roughness was held constant apparently for all simulations with Manning's coefficients of 0.03 for the channel and 0.05 for the overbanks. This could have an impact on the overall results and conclusions as roughness will usually increase with both increasing grain size and increasing slope (two of the variables in the PWA analysis).

Boundary Conditions

The combination of a short (500 foot) channel length and uncertain boundary conditions casts doubt on the results. Modelers recognize that results near boundaries often reflect inaccuracies in assumptions at those boundaries and will therefore extend their models beyond the immediate area of concern to minimize these boundary effects. The current models incorporate boundary effects at both the upstream and downstream ends. An "equilibrium" inflowing sediment load was developed with the HEC-RAS model such that the upstream most cross section would neither aggrade nor degrade with time. This load was based on uniform sediment size, slope, cross section shape, etc. and is a necessary but fictitious assumption to perform the simulations. At the downstream end, the assumption of normal depth at a fixed energy slope can have similar results. In addition, using a depth rather than an elevation at the downstream end with a movable bed model can prevent the model from ever reaching an equilibrium state. For example, at an aggrading downstream boundary, instead of increased velocity (increased sediment transport potential at a shallower flow depth) the water surface elevation will simply increase to match the bed increase in order to maintain the computed normal depth.

Sediment Grain Sizes

The uniform grain sizes used in the simulations are not representative of field conditions and the model results cannot reflect preferential transport of various size classes nor armoring of the bed ("hiding" of smaller size particles by larger ones on the surface).

Hydrologic Record

Model run times and output were larger than necessary because **all** flows were simulated, even zero flows. Typically in arid regions modeling, zero flows and very low flows estimated not to be able to move particles are excluded from simulations. In the arid Southwest, it is not unusual to have a 50 year period of record with only 10-20 years of actual flow data modeled.

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[2] "Magnitude and Frequency of Floods in California," Waananen and Crippen, USGS *Water Resources Investigations 77-21*, 1977.

Maximum Erosion Depth
The maximum depth of erosion (or "hard bottom") was set to 5 feet for all models. By itself, this is a reasonable value given the very small channel dimensions. However, the fact that the cross sections hit this hard bottom many times, prompting the switch to the analytical (rating curve) approach, *even for existing conditions* (no increased flows) should have been an indicator that other modeling problems were present. An example is given in Figure 1.

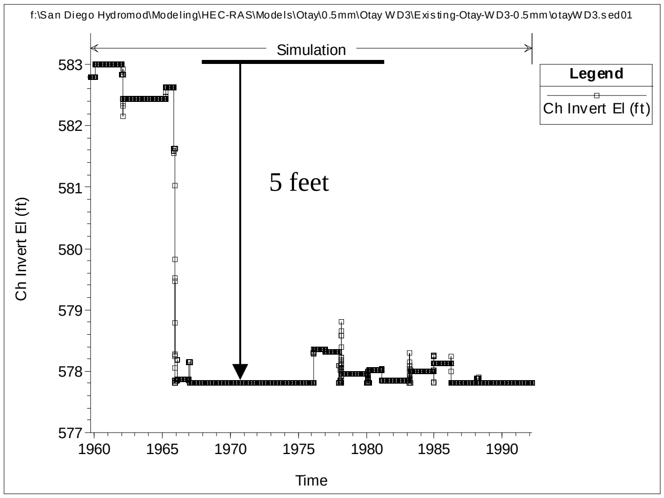


Figure 1. Otay site, existing conditions, bed hits hard bottom December, 1965

Overall, it appears that given all of the assumptions, uncertainty with inputs, and modeling problems, that a stable slope type analysis would have given similar results with much less effort involved.

Conclusions

Based on examination of the materials provided, this reviewer has serious concerns about the results obtained and their application to flow thresholds for hydromodification requirements. All results are related to baseline conditions – good practice in sedimentation modeling – but it is not clear that the baseline results are reasonable. Additionally, as noted by PWA in their memorandum, implementation of a threshold of 0.1Q2 will be a challenge in practical terms as this will encompass a very large range of flows. However, is 0.1Q2 a reasonable threshold based solely on sediment movement? Based on the EPA Nationwide Urban Runoff Program (NURP) results and continuous simulation modeling, Bledsoe and Watson[3] argue that standard hydrologic design practices are inadequate for characterizing the cumulative effects of urbanization on flow events that are *more frequent than Q2* (emphasis added) in terms of sediment transport and channel disturbance potential. That is to say, additional work leading from questions about the methodology and/or results of the PWA study may not result in an increase in a lower flow threshold for the HMP. Because of site-specific values of grain size, slope, roughness, and

[3] "Effects of Urbanization on Channel Instability," Bledsoe and Watson, *Journal of the American Water Resources Association*, Volume 37, No. 2, April 2001.

channel shape, it is not clear that using any specific frequency discharge as an indicator of shear stress that will move particles is a tenable approach.

Recommendations

Clearly the goal of the County must be to meet intent of MS4 permit with a reasonable effort to quantify flow thresholds. PWA's hydraulic and sediment transport results should be supplemented with real data from sites in order to set thresholds (flows, shear stresses, or velocities). With the help of the technical advisory group and others, existing information could be gathered to provide additional base data. Slope, sediment properties, roughness, and channel shape data from other studies could be used to compute shear stresses that would move significant amounts of sediment. If frequency discharges are available for a site, the critical shear could be related to a return period. If enough sites are available, the data could be analyzed to see if there is a consistent value of return period. If such a value is found, this could used for a regulatory threshold. If not, a site specific analysis may be required for each project.

Thank you for the opportunity to perform this review and contribute to stormwater management practice in San Diego County. Please call me at (858) 487-9378 if you have any questions.

Sincerely,

Martin J. Teal, P.E., P.H., D.WRE
Vice President

APPENDIX E

SDHMP Continuous Simulation Modeling Primer

BROWN AND CALDWELL

Memorandum

9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
Tel: 858-514-8822
Fax: 858-514-8833

Project Title: San Diego County Hydrograph Modification Plan

Project No: 133904

San Diego County Hydrograph Modification Plan

Subject: Using Continuous Simulation to Size Storm water Control Facilities

Date: April 30, 2008

To: Sara Agahi, San Diego County

From: Tony Dubin, Brown and Caldwell
Nancy Gardner, Brown and Caldwell

Brown and Caldwell prepared this memo to help civil engineers through the process of sizing storm water control facilities to meet San Diego County's Interim Hydromodification Criteria (IHC). Since the publication of the IHC this past January, the County has been engaged in outreach activities to explain the new storm water modeling methods required by the IHC and storm water facilities that could meet the IHC performance standard. In response to the outreach efforts, the County has received several questions and comments along a common theme:

1. How do we perform continuous hydrologic modeling analyses to size storm water control facilities?
2. What is the precise meaning of the peak flow and flow duration curve matching standard described in the IHC memo?

This document is not a complete "how-to manual" for conducting continuous hydrologic modeling to meet the County's IHC, but we hope it addresses the major technical concerns of the local engineering community.

Using Continuous Simulation Models to Size Storm Water Facilities

The IHC requires continuous simulation hydrologic modeling to adequately size storm water control facilities. This is a significant break with the common local practice of using event-based modeling to determine whether a storm water pond, swale or other device was properly sized. Event-based modeling computes storm water runoff rates and volumes generated by a synthetic rainfall event with a total depth that matches local records (e.g., rainfall depths shown in County isopluvial maps). By contrast, continuous modeling uses a long time series of actual recorded precipitation data as input a hydrologic model. The model in turn simulates hydrologic fluxes (e.g., surface runoff, groundwater recharge, evapotranspiration) for each model time step.

Continuous hydrologic models are usually run using one-hour or 15-minute time steps, depending on the type of precipitation data available and computational complexity of the model. Continuous models generate outputs for each model time step and most software packages allow the user to output a variety of different hydrologic flux terms. For example, a continuous simulation model setup with 25 years of hourly precipitation data will generate 25 years of hourly runoff estimates, which corresponds to runoff estimates for each of the 219,000 time steps (each date and hour) of the 25 year simulation period. While creating and running continuous simulation models involves more effort than running event-based models, the clear benefit of the continuous approach is that these models allow an engineer to estimate how often and for how long flows will exceed a particular threshold. Limiting how often and for how long geomorphically significant flows occur is at the heart of San Diego County's approach to hydrograph modification management.

Two common models were presented at a recent APWA workshop on HMP issues: HSPF and HEC-HMS. HSPF refers to the Hydrologic Simulation Program-FORTRAN and is distributed by the USEPA. HEC-HMS refers to the Hydrologic Modeling System (HMS) produced by the US Army Corps of Engineers Hydraulic Engineering Center (HEC). Engineers unfamiliar with these software packages should seek out training opportunities and online guidance. The USEPA conducts training workshops around the US to help teach engineers how to use HSPF. HEC-HMS training is provided through ASCE and third-party vendors.

The following list describes the major elements of developing a hydrologic model and using that model to size storm water facilities that meet the IHC.

1. Select an appropriate historical precipitation dataset for the analysis.
   a. The precipitation station should be located near the project site or at least receive similar rainfall intensities and volumes as the project site.
   b. The station should also have a minimum of 25-years of data recorded at hourly intervals or more frequently.
2. Develop a model to represent the pre-project conditions, including
   a. Land cover types
   b. Soil characteristics
   c. General drainage direction
3. Develop a model to represent the post-project conditions, including
   a. New land cover types – more impervious surfaces
   b. Soil characteristics
   c. Any modifications to the drainage layout
4. Examine the model results to determine how the proposed development affects storm water flows
   a. Compute peak flow recurrence statistics (described below)
   b. Compute flow duration series statistics (described below)
5. Iteratively size storm water control facilities until the post-project peak flows and durations meet the performance standard described below.

Understanding the Peak Flow and Flow Duration Performance Criteria

The IHC is based on a peak flow and flow duration performance standard. To compute the peak flow and flow duration statistics described in the standard, model users must have a method for evaluating long time series outputs (usually longer than the 65,000 rows available in MS Excel 2003 and earlier versions) and computing both peak flow frequency statistics and flow duration statistics.

We recommend computing **peak flow frequency statistics** by constructing a partial-duration series (rather than an "annual maximum" series). This involves examining the entire runoff time series generated by the model, dividing the runoff time series into a set of discrete unrelated events, determining the peak flow for each event, ranking the peak flows for all events and then computing the recurrence interval or plotting position for each storm event. To limit the number of discrete events to a manageable number, we usually only select events that are larger than a 3-month recurrence when generating the partial duration series. We consider flow events to be "separate" when flow rates drop below a threshold value for a period of at least 24 hours.

The exercise described above will generate a table of peak flows and corresponding recurrence intervals (i.e., frequency of occurrence for a particular flow). For continuous modeling and peak flow frequency statistics, it is important to remember that events refer to *flow events* and not precipitation events. Peak flow frequency statistics estimate how often flow rates will exceed a given threshold. For example, the 5-year flow event represents the flow rate that is equaled or exceeded an average of once per 5 years (and the storm generating this flow does not necessarily correspond to the 5-year precipitation event). Ranking the storm events generated by a continuous simulation and computing the recurrence interval of each storm will generate a table similar to Table 1 below.

Readers who are unfamiliar with how to compute the partial-duration series should consult reference books or online resources for additional information. For example, *Hydrology for Engineers*, by Linsley et all, 1982, discusses partial-duration series on pages 373-374 and computing recurrence intervals or plotting positions on page 359. *Handbook of Applied Hydrology*, by Chow, 1964, contains a detailed discussion of flow frequency analysis, including Annual Exceedance, Partial-Duration and Extreme Value series methods, in Chapter 8. The US Geological Survey (USGS) has several hydrologic study reports available online that use partial-duration series statistics (see http://water.usgs.gov/ and http://water.usgs.gov/osw/bulletin17b/AGU_Langbein_1949.pdf).

Table 1. Example Peak Flow Frequency Statistics

Recurrence Interval (years)	Peak Flow (cfs per acre)
58.5	0.73
21.9	0.69
13.5	0.53
9.8	0.53
7.6	0.51
6.3	0.51
5.3	0.50
4.6	0.50
4.1	0.49
3.7	0.48
3.3	0.48
3.0	0.46
2.8	0.45
2.6	0.45
2.4	0.45
2.3	0.45
2.1	0.44
2.0	0.42

Flow duration statistics are more straightforward to compute than peak flow frequency statistics. Flow duration statistics provide a simply summary of how often a particular flow rate is exceeded. To compute the flow duration series, rank the entire runoff time series output and divide the results into discrete bins. Then, compute how often the flow threshold dividing each bin is exceeded. For example, let's assume the results of a 35-year continuous simulation hydrologic model with hourly time steps show that flows leaving a project site exceeded 5 cfs an average of about once per year for 30 hours at a time. This corresponds to a total of

1050 hours of flows exceeding 5 cfs over 35 years. Another way to express this information is to say a flow rate of 5 cfs is exceeded 0.34 percent of the time. Computing the "exceedance percentage" for other flow rates will fill out the flow duration series. Table 2 lists an example flow duration series.

Table 2. Example Flow Duration Statistics

Flow (cfs per acre)	Percent of Time Flow Rate is Exceeded
0.02	0.67%
0.03	0.43%
0.04	0.34%
0.06	0.27%
0.07	0.21%
0.09	0.17%
0.10	0.15%
0.12	0.12%
0.13	0.11%
0.15	0.09%
0.16	0.08%
0.17	0.07%
0.19	0.06%
0.20	0.05%
0.22	0.05%
0.23	0.04%
0.25	0.04%
0.26	0.03%

The intention of the IHC performance standard is to limit the potential for new development to generate accelerated erosion of stream banks and stream bed material in the local watershed by matching the post-project hydrograph to the pre-project hydrograph for the range of flows that are likely to generate significant amounts of erosion within the creek. The IHC memo identified the geomorphically significant flow range as extending from two-tenths of the 5-year flow to the 10-year flow (0.2Q5 to Q10). The performance standard requires the following:

A. For flow rates from 20% of the pre-project 5-year runoff event (0.2Q5) to the pre-project 10-year runoff event (Q10), the post-project discharge rates and durations shall not deviate above the pre-project rates and durations by more than 10% over more than 10% of the length of the flow duration curve.

B. For flow rates from 0.2Q5 to Q5, the post-project peak flows shall not exceed pre-project peak flows. For flow rates from Q5 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.

Determining When a Storm Water Control Facility Meets the IHC Performance Standard

The previous section discussed how to calculate peak flow frequency and flow duration statistics. By comparing the peak flow frequency and flow duration series for pre-project and post-project conditions, an engineer can determine whether a stormwater control facility would perform adequately or if its size should be increased or decreased. The easiest way to determine if a particular storm water facility meets the IHC performance standard is to plot peak flow frequency curves and flow duration curves for the pre-project and post-project conditions.

Figure 1 shows a **flow duration curve** for a hypothetical development. The three curves show what percentage of the time a range of flow rates are exceeded for three different conditions: pre-project, post-project and post-project with storm water mitigation. Under pre-project conditions the minimum geomorphically significant flow rate (assumed to be 0.2Q5) is 0.10 cfs and flows would equal or exceed this value about 0.14% of the time (about 12 hours per year). For post-project conditions, this flow rate would occur more often – about 0.38% of the time (about 33 hours per year). This increase in the duration of the geomorphically significant flow after development illustrates why duration control is closely linked to protecting creeks from accelerated erosion. Higher flows that last for longer durations provide the energy necessary to increase the amount of erosion in local creeks. The post-project mitigated condition would include stormwater controls designed to limit the duration of geomorphically significant flows. Figure 1 shows that flows exceed 0.10 cfs only 0.08% of the time, which is less than pre-project conditions. This means the stormwater control mitigations would counteract the effects of the increased pavement associated with development projects.

An engineer can easily interpret the flow duration plots to determine whether a stormwater control facility would meet the IHC. Looking at the flow range between 0.2Q5 and Q10, the post-project mitigated curve should plot on or to the left of the pre-project curve. If the post-project curve plots to the left of the pre-project curve, this means a particular flow would occur for shorter durations due to storm water controls. Minor deviations where the post-project durations exceed the pre-project durations are allowed over a short portion of the flow range as described in IHC item A above.

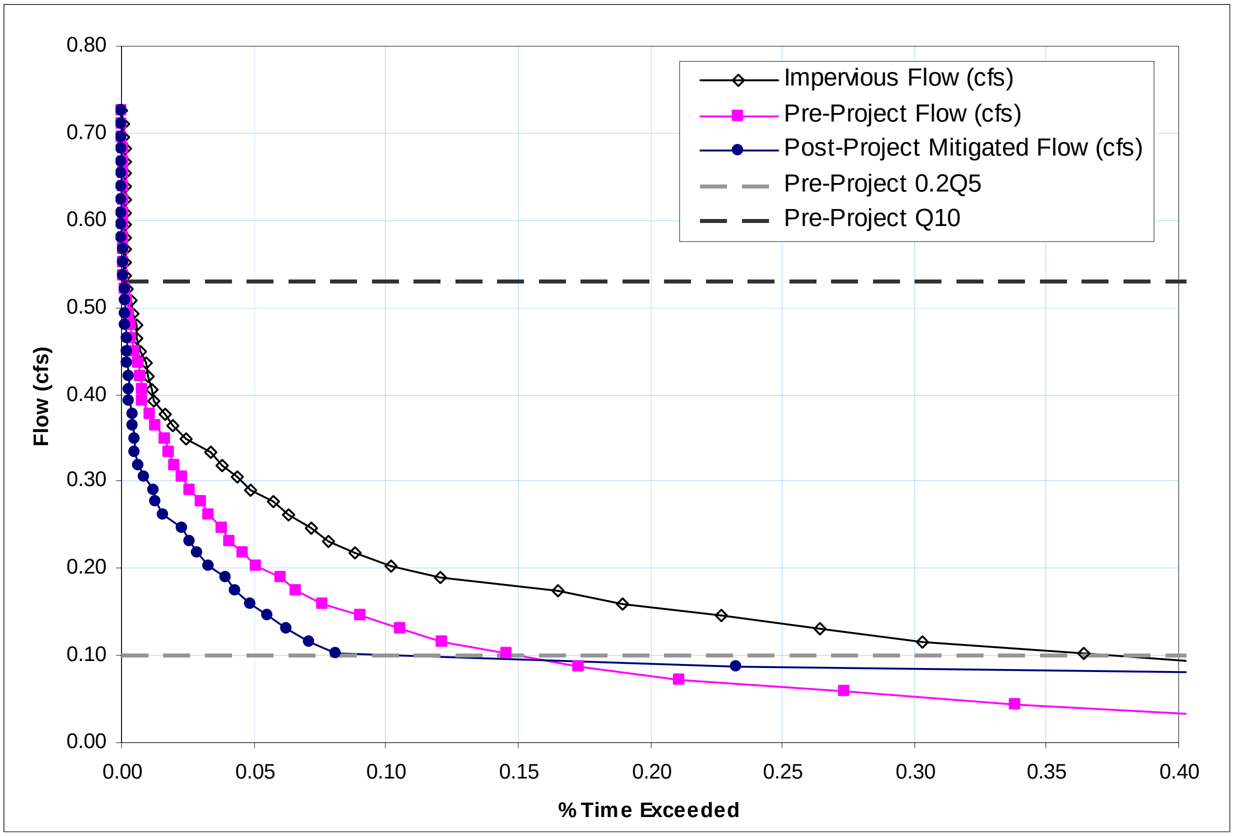


Figure 1. Flow Duration Series Statistics for a Hypothetical Development Scenario

Figure 2 shows a **peak flow frequency curve** for pre-project, post-project and post-project with storm water mitigation scenarios. The curves indicate how often a particular flow rate would be equaled or exceeded. For example, the pre-project 5 year flow rate would be 0.5 cfs per acre. This means under pre-project conditions, a flow rate of 0.5 cfs per acre would be equaled or exceeded an average of once per 5 years. For developed conditions, this 0.5 cfs per acre peak flow rate occur more often – about once per 1.5 years or, expressed another way, more than 3 times as often. The developed 5 year flow rate would increase by 30 percent over the pre-project condition, from 0.5 cfs per acre to about 0.65 cfs per acre.

Storm water control facilities should reduce peak flows from the site to levels less than or equivalent to the pre-project conditions. To determine whether a storm water facility provides sufficient protection, examine the peak flow frequency curves to see if the post-project mitigated peak flows are lower than pre-project peak flows of the same recurrence interval. The post-project mitigated scenario curve should plot below the pre-project curve for recurrence intervals between 0.2Q5 and Q10 to meet the IHC performance standard, with the possible exception of the small, allowable deviations described above in IHC item B.

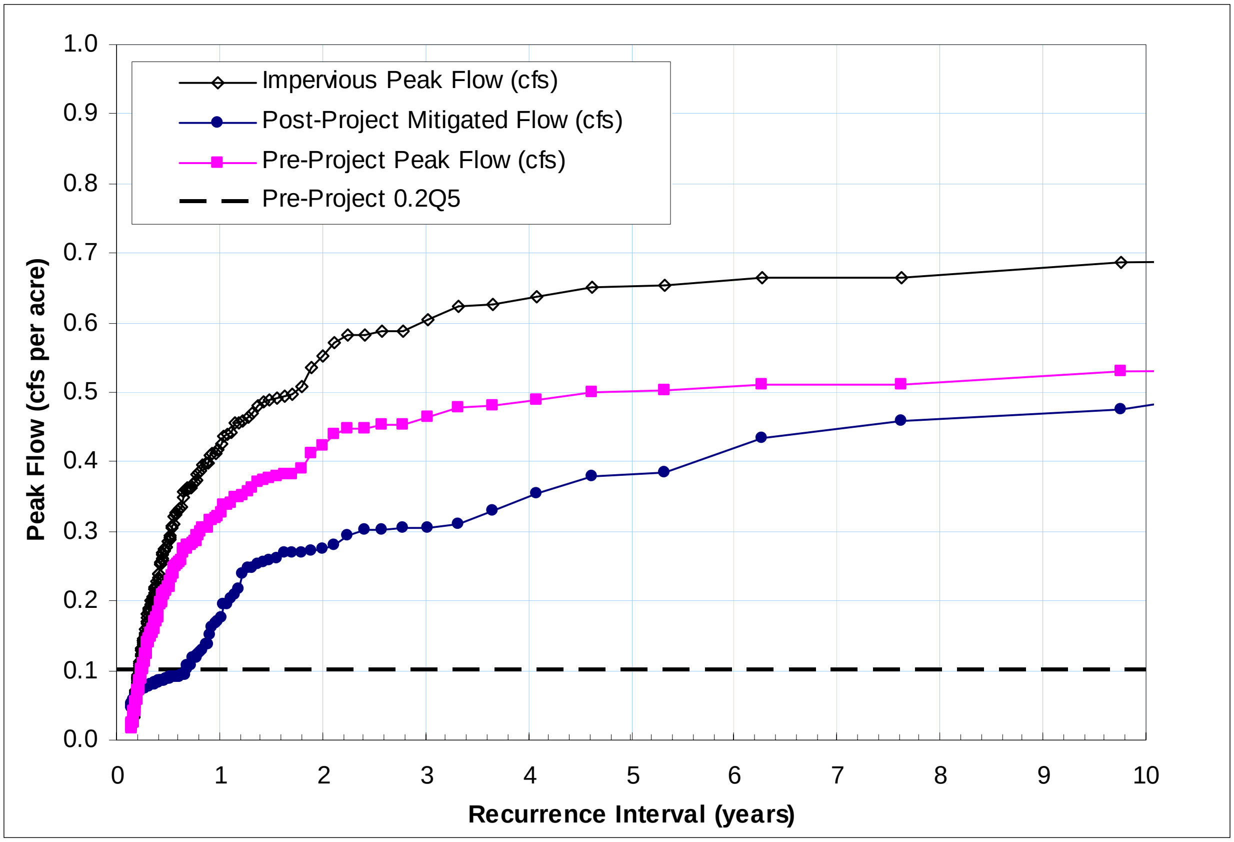


Figure 2. Peak Flow Frequency Statistics for a Hypothetical Development Scenario

References

Linsley, RK Jr.; Koher, MA; Paulhas, JLH; *Hydrology for Engineers,* 1982; McGraw-Hill Inc.

Chow, VT; *Handbook of Applied Hydrology,* 1964; McGraw-Hill Inc.

APPENDIX F

HSPF Modeling Analysis - Technical Memos

BROWN AND CALDWELL

9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
Tel: 858-514-8822
Fax: 858-514-8833

Memorandum

Project Title: San Diego County Hydrograph Modification Plan

Project No: 133904

San Diego County Hydrograph Modification Plan

Subject: HMP Sensitivity Analysis

Date: December 16, 2009

To: Sara Agahi, San Diego County

From: Tony Dubin, Brown and Caldwell
Eric Mosolgo, Brown and Caldwell

This memorandum evaluates three conditions where the HMP requirements could be modified without appreciable impacts on the receiving water body:

1. Development that is directly tributary to a large river
2. Development in highly urbanized watershed
3. Limited small developments within a watershed

The following sections describe the technical analysis that was performed to test the sensitivity of river flow durations to specific modifications in the HMP requirements. The results of the technical analysis can be used to justify and/or discard any planned special conditions that allow the HMP requirements to be modified.

Issue #1: Could Developments Near Large Rivers Be Exempted from Flow Duration Requirements?

To test whether development that is directly tributary to large rivers could potentially be exempted from flow duration control requirements, we examined the historical flow record for the San Diego River and evaluated how much additional development could occur without an appreciable change in the range of flows within the San Diego River channel.

We acquired the historical, hourly stream flow records for the San Diego River at Fashion Valley (USGS 11023000) and San Diego River at Mast Road (USGS 11022480) directly from the US Geological Survey. The data was available from October 1988 through November 2009. Next, we computed flow duration statistics for the river and computed relevant statistics, such as the peak 2-year flow rate.

After summarizing the river flows, we built HSPF models to simulate the conversion of undeveloped land to suburban development, assuming a 10-acre hypothetical development. We then ran the HSPF models and computed flow duration curves for the pre- and post-development conditions. We ran one scenario that used the Fashion Valley rain gauge and another scenario that used the Santee rain gauge. Table 1 lists the NRCS soil groups, land uses, and rain gauges that were used to simulate the different development scenarios.

Table 1. HSPF Model Assumptions for Large River Exemption Simulations							
No.	Scenario Description	Rain Gauge	Basin Acres	Soil and Land Use Combinations (area in acres)			
				Impervious	C/D, Dirt	C/D, Grass	C/D, Shrub
1	Undeveloped conditions in lower watershed	Fashion Valley	10	0	5	0	5
2	Developed (unmitigated) conditions in lower watershed	Fashion Valley	10	4	0	4	2
3	Undeveloped conditions in lower watershed	Santee	10	0	5	0	5
4	Developed (unmitigated) conditions in lower watershed	Santee	10	4	0	4	2

To simulate the incremental effects of development on flow durations, the pre-development flow duration curve was subtracted from the post-development flow duration curve. To represent multiple developments, the flow portion of this *difference flow duration curve* was scaled linearly with area to represent 100, 500, 1000, and 2000-acres of additional development within the San Diego River watershed. The simple scaling of the flow duration curves ignores the curve smoothing that could result from the staggered timing of flows reaching the San Diego River, and as such, this simple scaling of the flow duration curves should provide a conservative approximation of the impacts of multiple developments.

Finally, to gauge the impact of multiple developments on the range of San Diego River flows, the *difference flow duration curves* for 10, 100, 500, 1000, and 2000-acres of additional development were superimposed on the observed San Diego River flow duration curve. Figure 1 and Figure 2 (on the following pages) show the combined effect of multiple developments in the vicinity of the Fashion Valley stream flow gauge. Figure 2 shows the same information, but with the scale that focuses in on the part of the curve where the differences are most noticeable. Figure 3 and Figure 4 show the same results for the Mast Road (near Santee) stream flow gauge.

Recommendations

The post-development flow duration curves show very little difference from existing condition flow duration curves until about 2,000 acres or more of additional development occurs. Even when there are differences in the flow duration curves, the flow rates are sufficiently high that the incremental difference would not appreciably increase the level of sediment movement and river bank erosion. As such, we recommend exemptions for these reaches of the San Diego River and other similar rivers from flow duration control requirements.

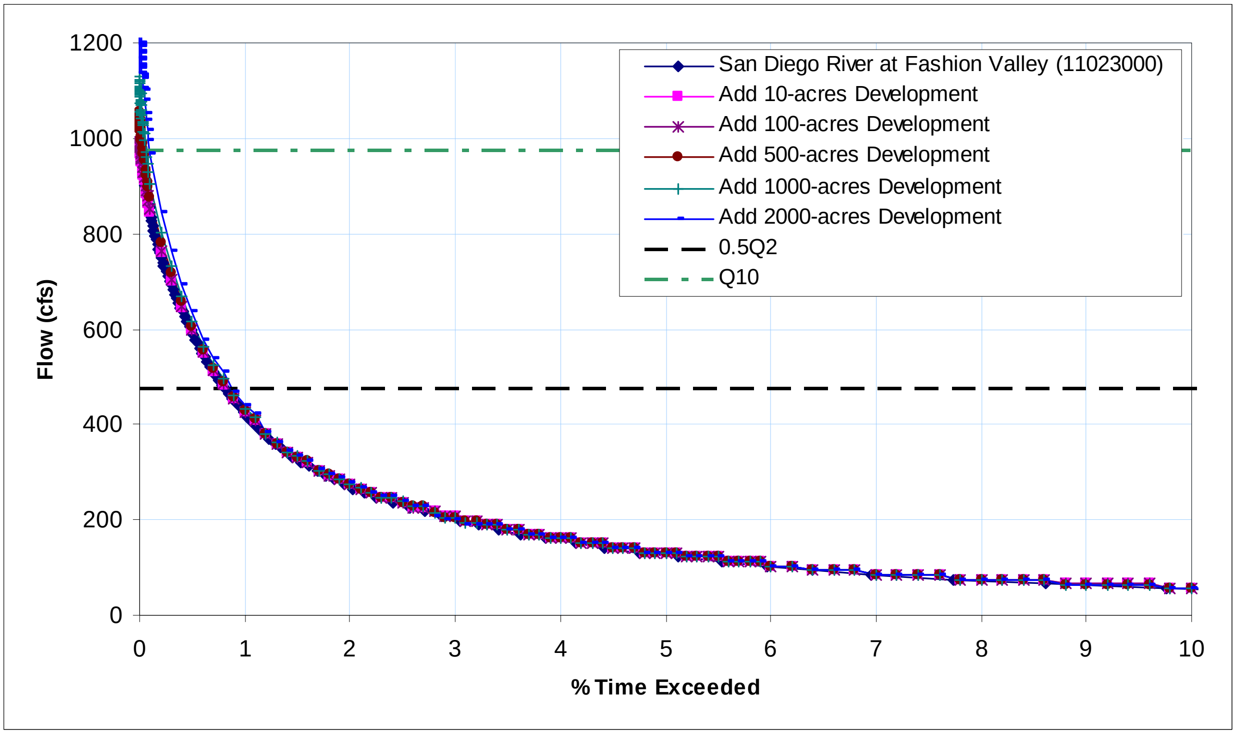


Figure 1. Effects of Additional Development near the San Diego River at Fashion Valley

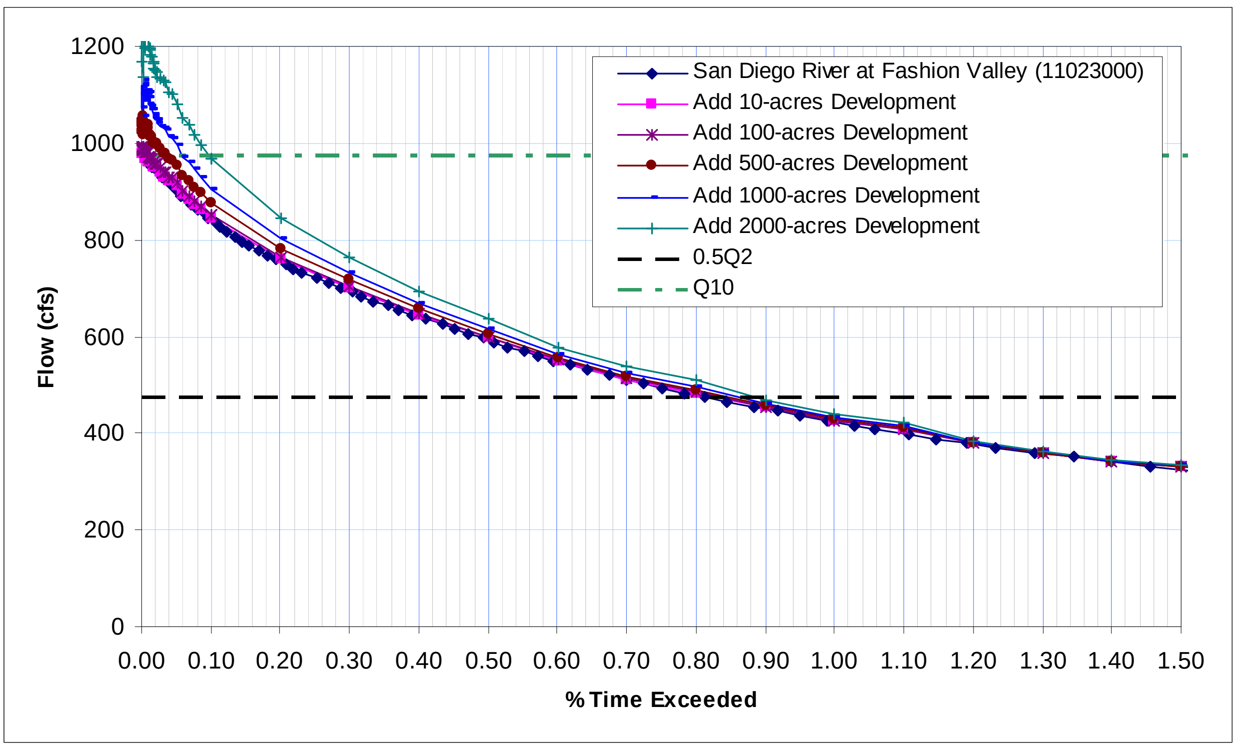


Figure 2. Effects of Additional Development near the San Diego River at Fashion Valley, Zoomed View

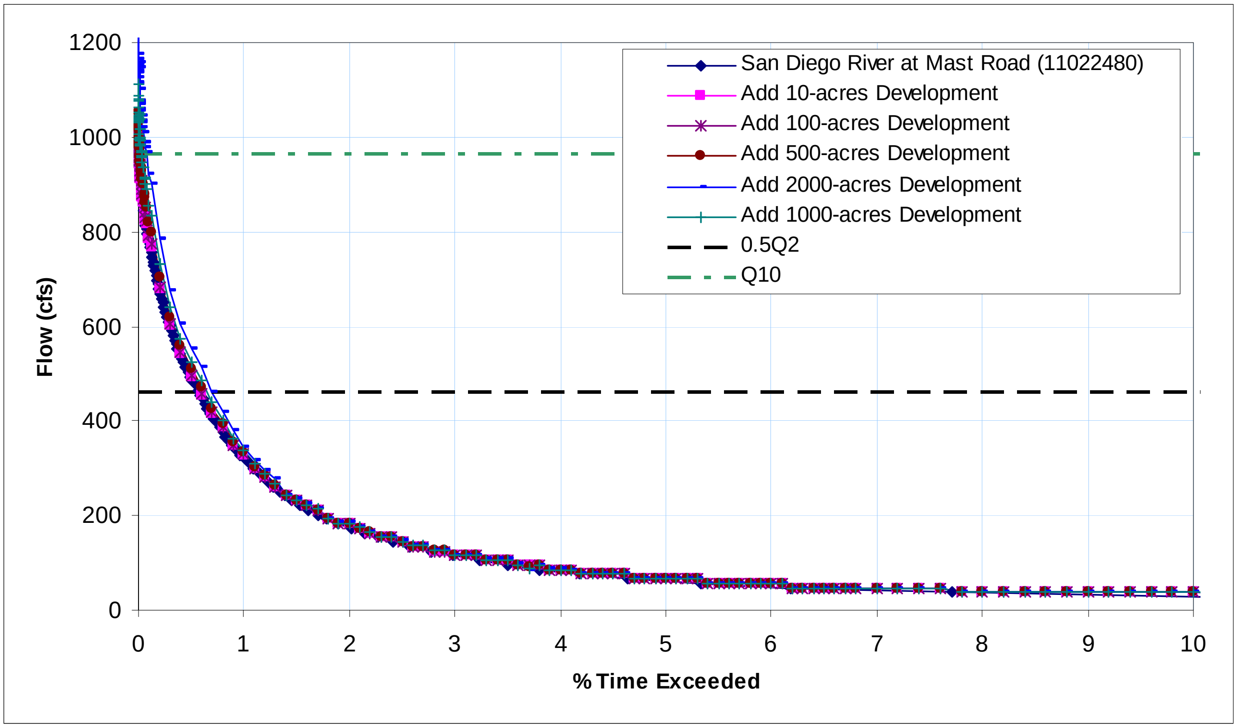


Figure 3. Effects of Additional Development near the San Diego River at Mast Road (near Santee)

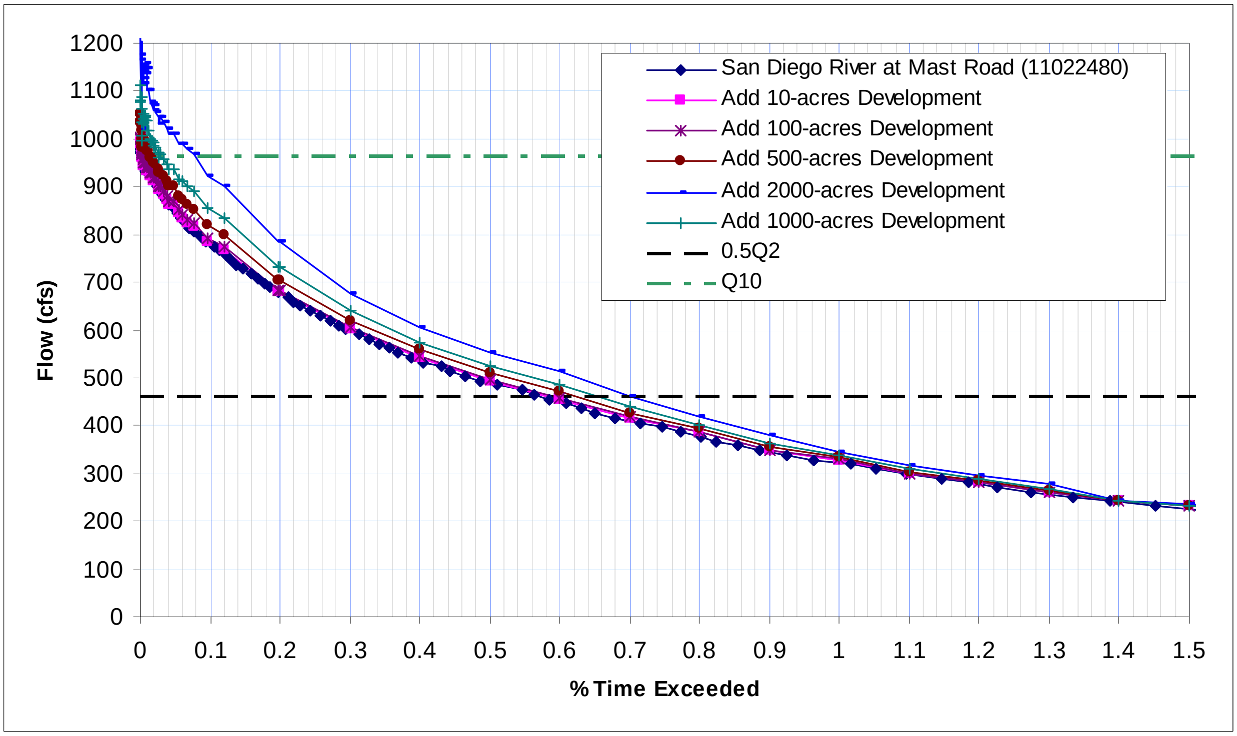


Figure 4. Effects of Additional Development near the San Diego River at Mast Road (near Santee), Zoomed View

Issue #2: Could Limited Infill Development within Highly Urban Watersheds Be Exempted from Flow Duration Control Requirements?

To test whether limited infill development within urbanized watersheds would appreciably impact flow durations in receiving water bodies, we built HSPF models to simulate the stormwater runoff that would occur in 10-acre, 100-acre, and 500-acre urbanized watersheds with 40, 50, and 60-percent total impervious areas. Table 2 lists the soil, land use, and rain gauges that were used to develop the models.

Table 2. HSPF Model Assumptions for Urban Infill Exemption Simulations

No.	Scenario Description	Rain Gauge	Basin Acres	Soil and Land Use Combinations (area in acres)			
				Impervious	C/D, Dirt	C/D, Grass	C/D, Shrub
1	10-acre urban watershed (40% impervious)	Fashion Valley	10	4	0	4	2
2	100-acre urban watershed (40% impervious)	Fashion Valley	100	40	0	40	20
3	500-acre urban watershed (40% impervious)	Fashion Valley	500	200	0	200	100
4	10-acre urban watershed (50% impervious)	Fashion Valley	10	5	0	4	1
5	100-acre urban watershed (50% impervious)	Fashion Valley	100	50	0	40	10
6	500-acre urban watershed (50% impervious)	Fashion Valley	500	250	0	200	50
7	10-acre urban watershed (60% impervious)	Fashion Valley	10	6	0	3	1
8	100-acre urban watershed (60% impervious)	Fashion Valley	100	60	0	30	10
9	500-acre urban watershed (60% impervious)	Fashion Valley	500	300	0	150	50

Figure 5 (on the following page) shows a peak flow frequency curve for the 100-acre, 40, 50, and 60-percent impervious scenarios. Figure 6 shows the flow duration curves for these 100-acre watershed scenarios. Figure 7 focuses on the portion of the flow duration curves where the differences in the simulations are most noticeable. The 10-acre and 500-acre scenarios produced similar results (on a unit area basis).

Recommendations

The extent of the spread among the 40, 50, and 60-percent model scenarios demonstrates that unchecked development within urbanized watershed would have a noticeable effect on the peak flows and flow durations observed within the receiving waters. However, some modest level of urbanized development would produce minor or negligible effects on the peak flows and flow durations. Based on our examination of the peak flow frequency and flow duration curves, we recommend the following allowances in highly urbanized watersheds:

> For subwatershed areas containing between 40 percent and 70 percent existing imperviousness (as measured from the project site downstream to a natural creek confluence), projects may be exempt from HMP criteria if:

1. The potential cumulative impacts within the subwatershed would not increase the composite impervious area percentage by more than 3 percent, and;
2. The project discharges runoff to an existing hardened system (storm drain or concrete channel) that extends beyond the Domain of Analysis determined for the project site.

For subwatershed areas containing existing impervious percentages greater than 70 percent (as measured downstream to the Pacific Ocean, San Diego Bay, a tidally influenced lagoon, or an exempt river system), projects are exempt from HMP criteria. Additionally, for subwatershed areas containing less than 40 percent existing imperviousness, projects are subject to HMP criteria unless they qualify for another exemption (per HMP Decision Matrix).

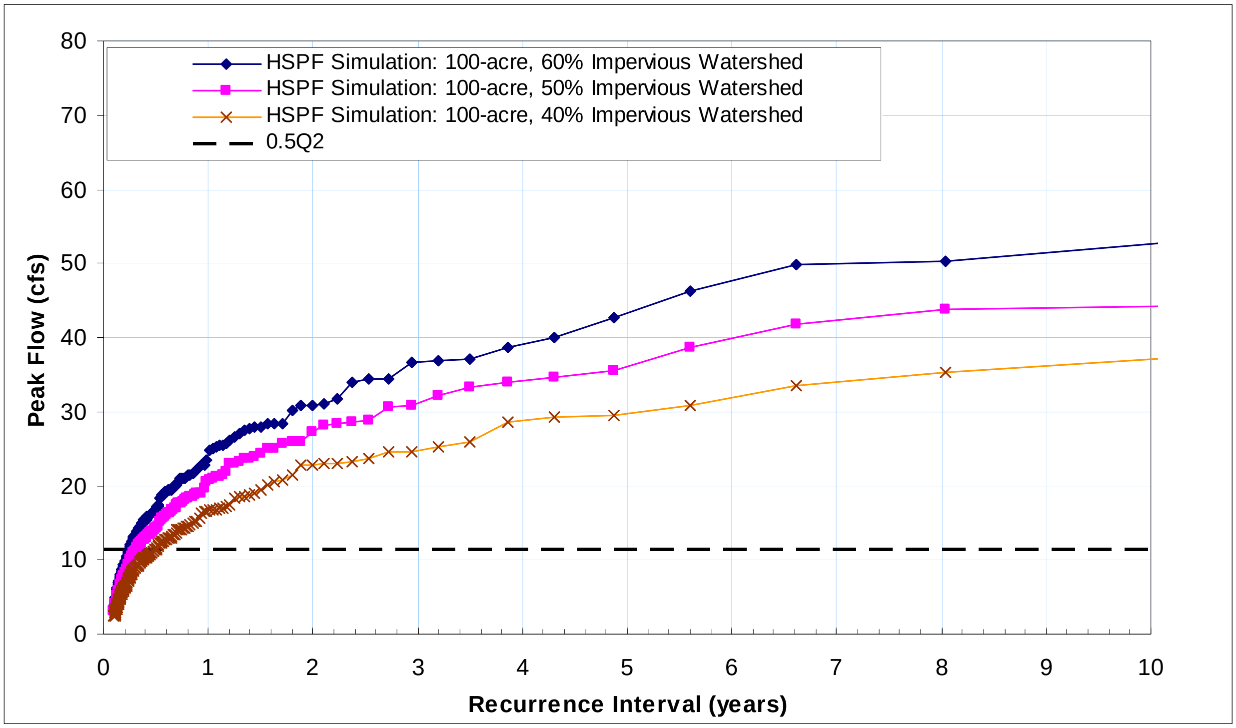


Figure 5. Simulated Peak Flow Frequencies for 100-acre Urbanized Watershed (Fashion Valley rainfall)

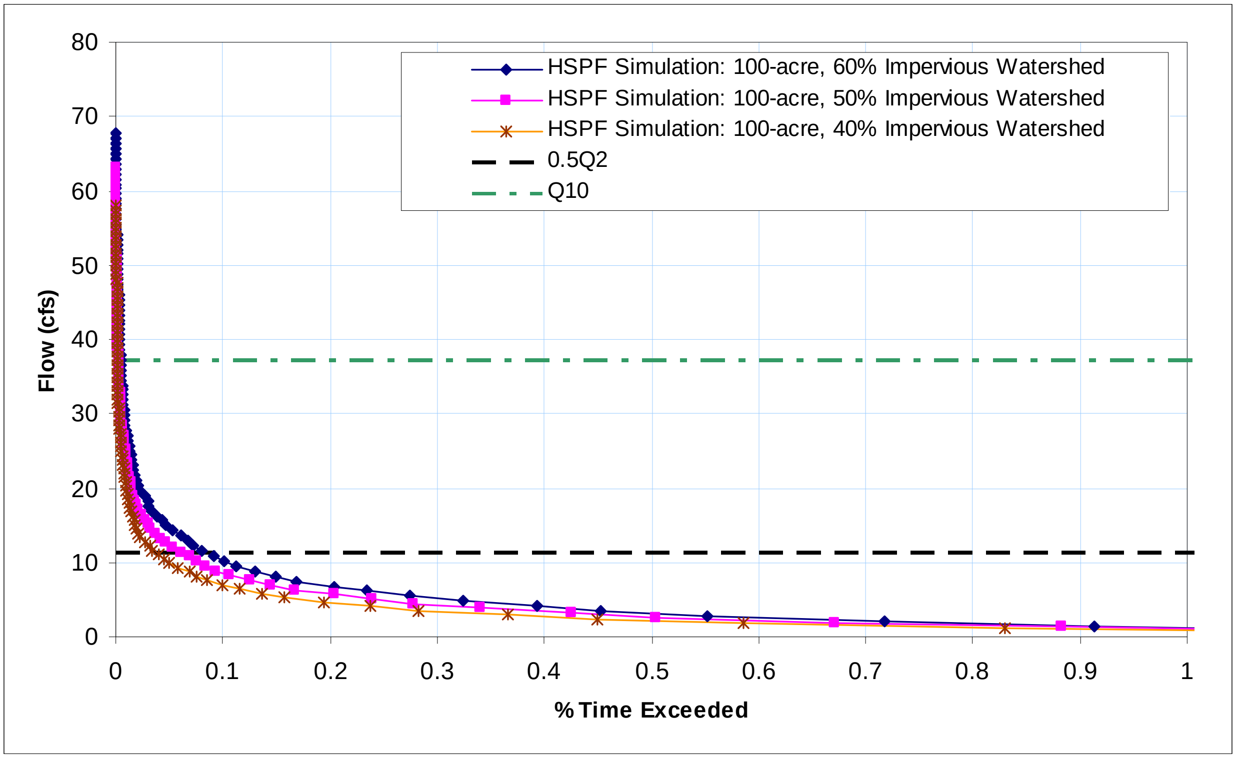


Figure 6. Simulated Flow Durations for 100-acre Urbanized Watershed (Fashion Valley rainfall)

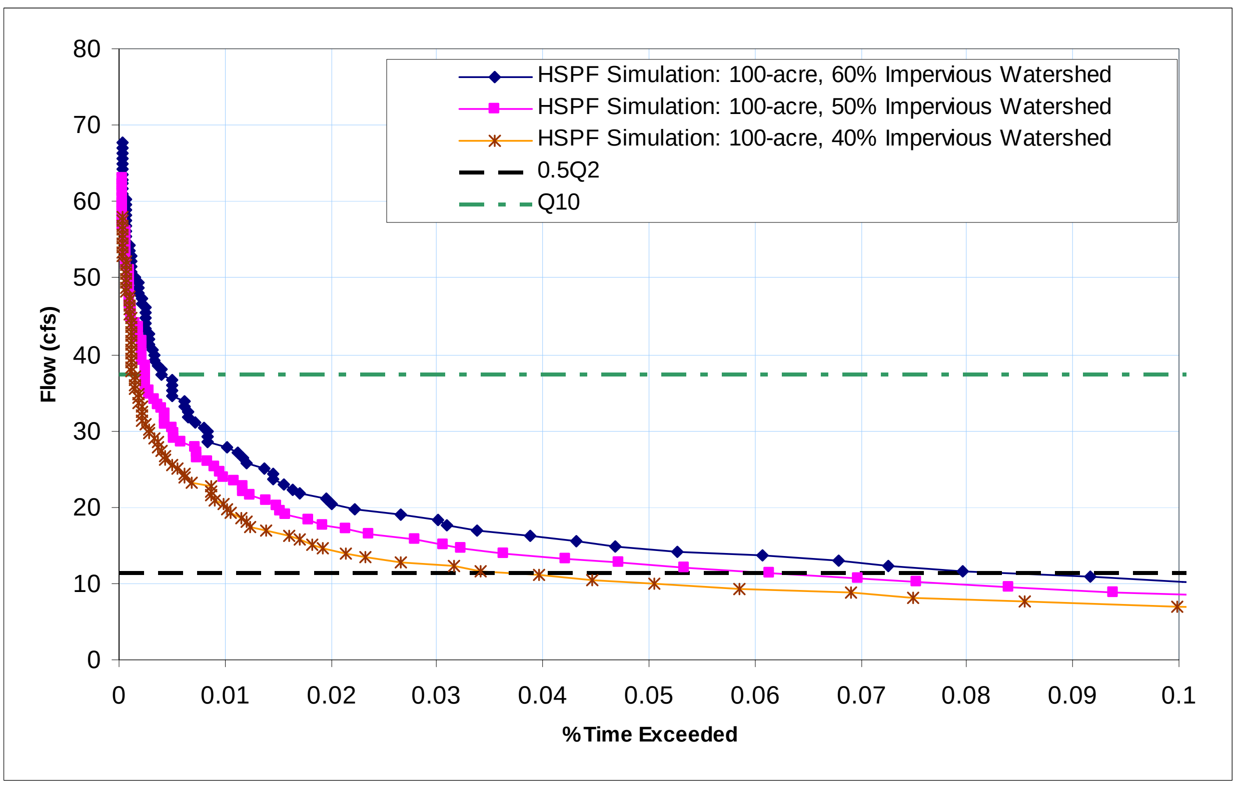


Figure 7. Simulated Flow Durations for 100-acre Urbanized Watershed (Fashion Valley rainfall), Zoomed View

Issue #3: Could Limited Small Developments Specify a Minimum 3-inch Diameter Orifice for Detention Pond Design without Affecting the Receiving Water's Flow Durations?

Due to concerns about clogging, a 3-inch diameter minimum diameter orifice has been proposed for stormwater detention pond design. This size orifice would not provide the required level of flow restriction for small developments, because the 3-inch diameter orifice capacity is greater than the lower flow control range (0.1Q2, 0.3Q2, 0.5Q2) in the Final HMP. As such, we tested whether a limited number of small developments could use a 3-inch minimum orifice diameter without generating appreciable cumulative effects on the receiving water's flow durations.

We built HSPF models to represent undeveloped 100-acre and 500-acre watersheds in the vicinity of the Lower Otay rain gauge. We then built HSPF models to represent undeveloped and developed-mitigated conditions for 5-acre and 10-acre development sites. The developed-mitigated scenarios included detention ponds with 3-inch diameter lower orifice and an upper high flow release.

Similar to the large watershed development scenarios evaluated for Issue #1 above, we computed flow duration curves for the undeveloped and developed-mitigated scenarios, and then subtracted the undeveloped flow duration curve from the developed-mitigated flow duration curve to estimate the difference in conditions. Then, we scaled the difference flow duration curve in increments of 5, 10, 25, and 50-acres and superimposed these curves on the 100-acre and 500-acre undeveloped scenarios to determine when the cumulative impacts would be noticeable. Table 3 lists the soil, land use, and rain gauges that were used for this analysis.

Table 3. HSPF Model Assumptions for 3-inch Minimum Orifice Diameter Simulations

No.	Scenario Description	Rain Gauge	Basin Acres	Soil and Land Use Combinations (area in acres)			
				Impervious	C/D, Dirt	C/D, Grass	C/D, Shrub
1	100 ac undeveloped conditions	Lower Otay	100	0	50	0	50
2	500 ac undeveloped conditions	Lower Otay	500	0	250	0	250
3	1 ac undeveloped conditions	Lower Otay	1	0	0.5	0	0.5
4	5 ac undeveloped conditions	Lower Otay	5	0	2.5	0	2.5
5	10 ac undeveloped conditions	Lower Otay	10	0	5	0	5
6	1 ac mitigated conditions with 3-in diameter outlet	Lower Otay	1	0.4	0	0.4	0.2
7	5 ac mitigated conditions with 3-in diameter outlet	Lower Otay	5	2	0	2	1
8	10 ac mitigated conditions with 3-in diameter outlet	Lower Otay	10	4	0	4	2

Figure 8 shows the flow durations curves for the 100-acre undeveloped scenario, plus developed-mitigated scenarios with increments of 5, 10, 25, and 50-acres of development. For the developments, we are assuming the ponds serve 10 acre increments of development (except for the 5-acre increment scenario) and include a 3-inch diameter lower control orifice.

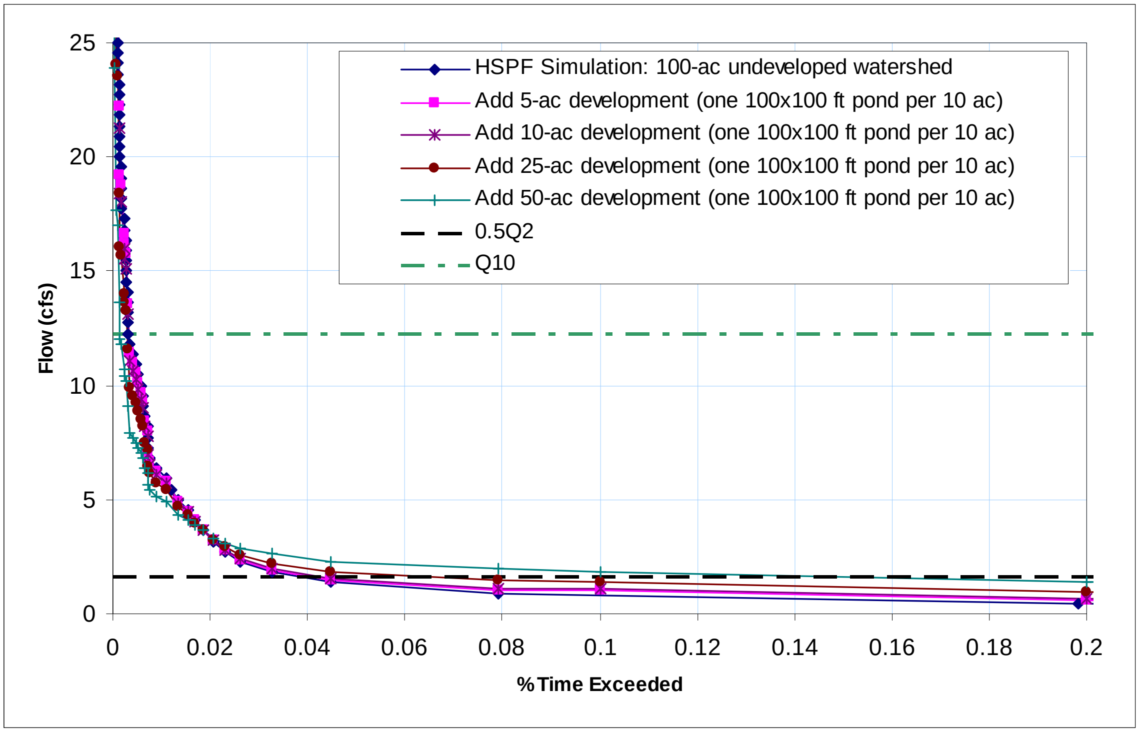


Figure 8. Simulated Flow Durations for Adding Development and Extended Duration Ponds with 3-inch Minimum Diameters to a 100-acre Undeveloped Watershed (Lower Otay rainfall)

The results show that the cumulative development flow duration curve approximately matches the undeveloped flow duration curves when development occurs in 10 percent or less of the watershed. For development levels in excess of 10 percent, the cumulative flow duration curve deviates noticeably from the undeveloped condition. The 500-acre undeveloped watershed simulations indicated a similar threshold sensitivity to development.

Recommendations

The HSPF analysis indicated limited situations where a 3-inch minimum orifice size standard could be applied. However, it should be noted that for small sites where orifices less than 3-inches would be required for HMP mitigation, we recommend an LID requirement in lieu of extended detention facilities.

For project sites 1 acre of less in size:

- HMP mitigation must be attained through the use of LID facilities (because a 3-inch outlet orifice would provide no tangible mitigation)

For project sites greater than 1 acre and less than 5 acres in size:

- HMP mitigation should be attained through the use of LID facilities

If LID implementation is not possible and extended detention basins are used:

- A 3-inch minimum outlet orifice size may be used provided that the potential cumulative impacts in the subwatershed area, as measured from the project site downstream to a natural creek confluence, would not increase the composite impervious area in the subwatershed to more than 10 percent.

If the potential cumulative impacts in the subwatershed areas would result in an impervious area percentage greater than 10 percent, then the 3-inch minimum orifice size waiver would not be granted.

Appendix A: Assumed Water Movement Hydraulics for Modeling BMPs

At minimum, each BMP 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 BMPs 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 BMPs. 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 bioretention and the flow-through planter BMPs. Hydraulically, this is sufficiently similar to the overflow gate and weir designs shown in the Countywide SUSMP.

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-controlled. As the head over the opening increases the flow condition transitions to become orifice-controlled and eventually pipe-controlled (the pipe flows full).

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

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

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 bioswale with check dams. 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:

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

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.

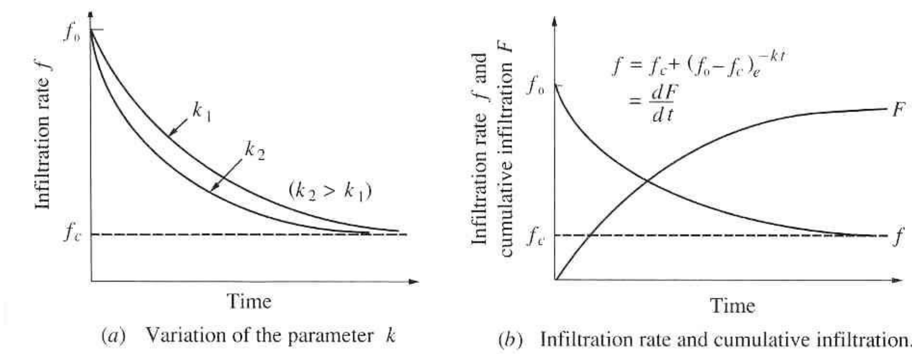


Figure B1– Horton's Equation for Infiltration (graphs from Chow et al. 1988)

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}.

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

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.

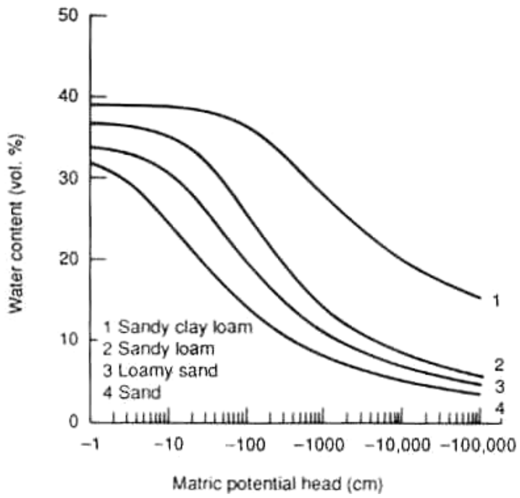


Figure B2 – Typical water retention curves (graph from Maidment, 1993)

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

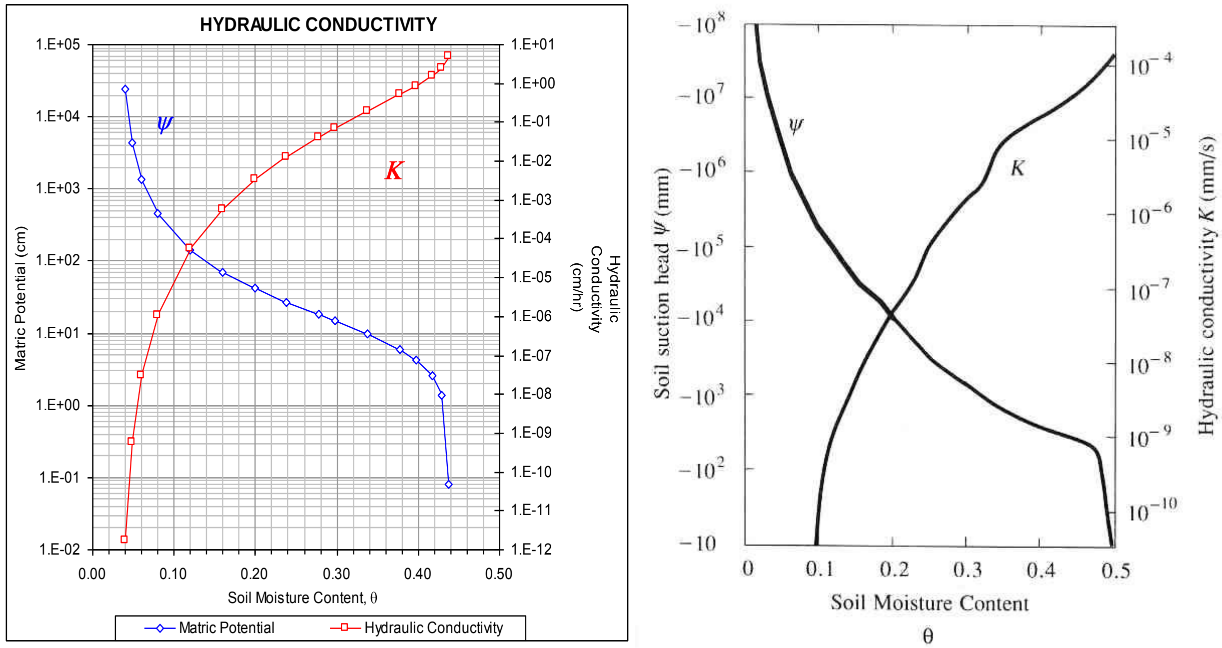
1 – Values for gravel were obtained from Fayer (1992) as presented in INEEL (2002).

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.

References

1. Chow, V.T.; D.R. Maidment; L.W. Mays. 1988. Applied Hydrology. McGraw-Hill Series in Water Resources and Environmental Engineering. McGraw-Hill, Inc. San Francisco. pp 103.
2. Eagleson, P.S. 1970. Dynamic Hydrology. McGraw-Hill, Inc. pp 291-305.
3. Fayer, M.J.; M.L. Rockhold; and M.D. Campbell. 1992. "Hydrologic modeling of protective barriers: Comparison of field data and simulation results," Soil Science Society of America Journal, Vol 56 pp 690-700.
4. Idaho National Engineering and Environmental Laboratory (INEEL). March 2002. "Engineering Design File: Central Facilities Area Sewage Treatment Drainfield (CFA-08) Protective Cover Infiltration Study." Project File number 021048, Prepared for the U.S. Department of Energy, Idaho Operations Office, Idaho Falls, Idaho. Document Number EDF-2696.

5. Maidment, D.R. 1993. Handbook of Hydrology. McGraw-Hill, Inc. San Francisco. pp 5.6, 5.14, 5.34.

6. National Soil Conservation Service (NRCS). 2001. *National Engineering Handbook, Part 630 Hydrology, Appendix A: Hydrologic Soil Groups*. United States Depertment of Agriculture.

7. United States Bureau of Reclamation (USBR). 1987. Design of Small Dams, Third Edition. United States Department of the Interior, Water Resources Technical Publication. U.S. Government Printing Office, Washington D.C.

8. van Genuchten, M. Th. 1980. "A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils." Soil Science Society of America Journal. Vol. 44, pp 892-898.

BROWN AND CALDWELL

Draft Technical Memorandum

9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
Tel: 858-514-8822
Fax: 858-514-8833

Project Title: San Diego County Hydromodification Management Plan

Project No: 133904

San Diego County Hydromodification Management Plan

Subject: HMP Modeling Approach and BMP Configurations

Date: March 2, 2010

To: Sara Agahi, San Diego County

From: Tony Dubin, Brown and Caldwell
Eric Mosolgo, Brown and Caldwell

This memorandum describes the modeling approach that is being used to size low-impact development dbest management practices (LID BMPs) for the San Diego Hydromodification Management Plan (HMP), including the range of scenarios performed and key assumptions for describing pre-project and post-project conditions, and BMP hydraulics. The memo also describes the type, configurations and dimensions of the LID BMPs that will be modeled in support of the BMP Sizing Tool software. The memo is organized into the following sections:

- **Section 1** provides a brief overview of the HSPF model setup and BMP sizing process.
- **Section 2** describes in greater detail how the HSPF models are setup for the San Diego HMP, including key input data.
- **Section 3** summarizes the general process for computing LID BMP sizing factors.
- **Section 4** describes the physical configurations of the BMPs.

1. Overview of HSPF Modeling and BMP Sizing Approach

The purpose of the runoff simulation for existing and post-development site conditions is to evaluate the effectiveness of BMPs which mitigate the increase in stormwater runoff 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 groups, land cover, slope and rainfall scenarios. Increases in runoff peaks and durations from each of these baseline scenarios establish the impacts to be fully mitigated by a BMP in a particular site development project. This section summarizes the overall steps used in this study to size BMPs.

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

San Diego County and its Copermittees' approach to compliance with the stormwater runoff control provision of its NDPES permit 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 that 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.

Brown and Caldwell has developed sets of HSPF model parameters to represent a range of pre-project site conditions that may be encountered in San Diego County. The parameter selection process and parameter values are described in a separate technical memorandum entitled, *San Diego HMP HSPF Model Parameter Selections*, dated March 2010. 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 locally collected rainfall time series data.

Once a continuous runoff time series was generated for the rainfall period of record for each scenario, partial duration 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 *Countywide Model SUSMP*, designers are expected to minimize the amount of pervious surface that drains to BMPs. 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 a BMP.

1.2 Model the Hydraulic Response of BMPs

The project team has constructed representations of each BMP in HSPF. For example, a bioretention basin is represented with separate surface ponding, growing medium, storage layers, an overflow relief outlet, a restricted underdrain outlet (as appropriate), and transmissivity of underlying soils. The configuration of these BMP elements and associated hydraulic characteristics can be varied to determine the configuration that provides the best performance in the least amount of space. The HSPF method for representing storage facilities is called an F-TABLE, and is described further in Section 3.1.1.

1.3 Establish BMP Sizing Factors

To compute sizing factors for each BMP, the impervious runoff time series was routed through the BMP to develop a post-project "mitigated" runoff time series. Each BMP mitigates post-project runoff by providing infiltration and/or reduction of discharge rates to the drainage system. The post-project mitigated time series is then compared to the pre-project runoff time series to assess BMP performance. The BMP 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 following standard applied to assess BMP performance:

- Flow duration control -_For flow rates ranging from 10%, 30% or 50% of the pre-project 2-year runoff event ($0.1Q_2$, $0.3Q_2$, or $0.5Q_2$) to the pre-project 10-year runoff event (Q_{10}), the post-project discharge rates and durations shall not deviate above the pre-project rates and durations by more than 10% over and more than 10% of the length of the flow duration curve. The specific lower flow threshold will depend on results from the SCCWRP channel screening study and the critical flow calculator.
- Peak flow control - For flow rates ranging from the lower flow threshold to Q_5, the post-project peak flows shall not exceed pre-project peak flows. For flow rates from Q_5 to Q_{10}, 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 Q_9 to Q_{10} or from $Q_{5.5}$ to $Q_{6.5}$, but not from Q_8 to Q_{10}.

1.4 Incorporate Sizing Factors into BMP Sizing Calculator

The sizing factors computed using the above process will be incorporated into a BMP Sizing Calculator that development engineers and municipal plan review staff will use to describe site hydrology, compute pre- and post-project runoff rates, and size BMPs.

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

The BMP Sizing Calculator will also provide prescriptive guidance on using self-retaining landscaping, soil amendments, and other techniques to limit site runoff, and contains a conservative approach to scale BMPs based on tributary pervious areas (i.e., in addition to the tributary impervious areas). The approaches to be used for scaling the sizing factors according to local rainfall variations will be described in a separate technical memorandum.

2. HSPF Model Development

This section describes in detail how HSPF models are developed to simulate pre-project and post-project runoff for San Diego County.

2.1 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, growing medium 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 hydrologic modeling within the HMP is to produce a County-wide assessment tool for sizing BMPs. This requires several modifications to the approach used 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 HSPF studies in the area, such as the Santa Monica Bay HSPF watershed-scale model developed by staff at the Southern California Coastal Water Research Project (SCCWRP). In addition, the range of parameter variations across different soil types and slope values were estimated using other references, including EPA Technical Note 6 and various Brown and Caldwell studies. The HSPF model parameters that are used to characterize the hydrologic response of pervious land surfaces to rainfall area (e.g., PERLND parameters) are described in detail in a separate memo entitled, *San Diego HMP HSPF Model Parameter Selections*, dated March, 2010.

Adapting the compiled HSPF parameters for use in San Diego County required an assessment of the local characteristics that affect surface runoff, such as precipitation data, basic soil groups and vegetation cover. The following subsections briefly summarize the range and variability in rainfall volumes and soil types within the County.

2.1.1 Rainfall Data Evaluation

Evaluating the distribution of rainfall across the County helped determine (1) which precipitation gauges to use as input to HSPF for modeling simulations and (2) the extent of rainfall variation throughout the County. The San Diego Alert Network operates a series of precipitation stations across the County, and the National Oceanographic and Atmospheric Administration (NOAA) operates a station at Lindberg Field in San Diego. Eighteen stations with datasets containing at least 30 years of hourly data were evaluated in detail by Brown and Caldwell. Brown and Caldwell prepared and submitted summary technical memoranda that assessed the data records, identified data gaps, and provided recommendations for filling the data gaps.

Table 1 lists reference information about the gauges and Figure 1 shows the variation in mean annual rainfall depth. Mean annual precipitation values vary from 8.7 inches at Bonita to 30.4 inches at Lake Cuyamaca, with the majority of stations recording annual rainfall amounts between 10 and 15 inches.

Table 1. San Diego County Rain Gauge Station Reference Information

Station Name	Watershed	Start Date	End Date	Length of Record	Latitude	Longitude	Elevation (ft)	Max Hour Rain (in)	Mean Annual Rain (in)
Bonita	Sweetwater River	11/25/1970	5/25/2008	37 years	32.3922	-117.0203	120	1.10	8.7
Encinitas	North County Coastal	9/4/1963	6/30/2008	45 years	33.0237	-117.1639	242	0.88	10.2
Escondido	Escondido Creek	9/24/1964	5/23/2008	44 years	33.0711	-117.0542	645	0.88	13.7
Fallbrook	San Luis Rey River	7/25/1951	6/30/2008	57 years	33.213	-117.1513	675	1.40	15.1
Fashion Valley	San Diego River	1/2/1968	6/30/2008	40 years	32.4555	-117.1033	20	0.96	10.3
Flinn Springs	San Diego River	8/8/1963	6/30/2008	45 years	325055	-1165129	880	1.05	13.1
Kearny Mesa	San Diego River	9/8/1964	6/30/2008	44 years	32.5003	-117.0744	425	1.40	11.0
Lake Cuyamaca	Upper San Diego River	9/1/1967	6/30/2008	41 years	32.5921	-116.3513	4590	2.30	30.4
Lake Henshaw	Upper San Luis Rey River	1/2/1950	6/30/2008	58 years	331419	-1164542	2990	1.79	22.3
Lake Wohlford	Upper Escondido Creek	10/8/1949	7/7/2008	59 years	33.0959	-117.0016	1490	1.60	16.8
Lindbergh Field	Coastal – San Diego Bay	10/17/1948	6/30/2008	60 years	32.7333	-117.1833	15	1.36	9.8
Lower Otay	Otay River	8/28/1951	6/30/2008	57 years	32.3632	-116.554	491	0.84	10.3
Oceanside	San Luis Rey River	7/1/1951	6/30/2008	57 years	33.1238	-117.2112	30	1.20	11.7
Poway	Los Penasquitos River	10/4/1962	6/30/2008	46 years	32.5658	-117.0346	440	0.80	12.0
Ramona	Upper San Dieguito River	8/8/1963	6/30/2008	45 years	33.0253	-116.5139	1450	1.16	14.2
San Onofre	North County Coastal	11/25/1970	6/30/2008	38 years	33.2105	-117.3155	162	1.60	11.3
San Vicente	San Diego River	1/1/1973	6/10/2008	35 years	32.55	-116.5558	663	1.00	12.7
Santee	San Diego River	1/1/1973	9/26/2008	36 years	32.502	-117.013	300	1.00	13.2

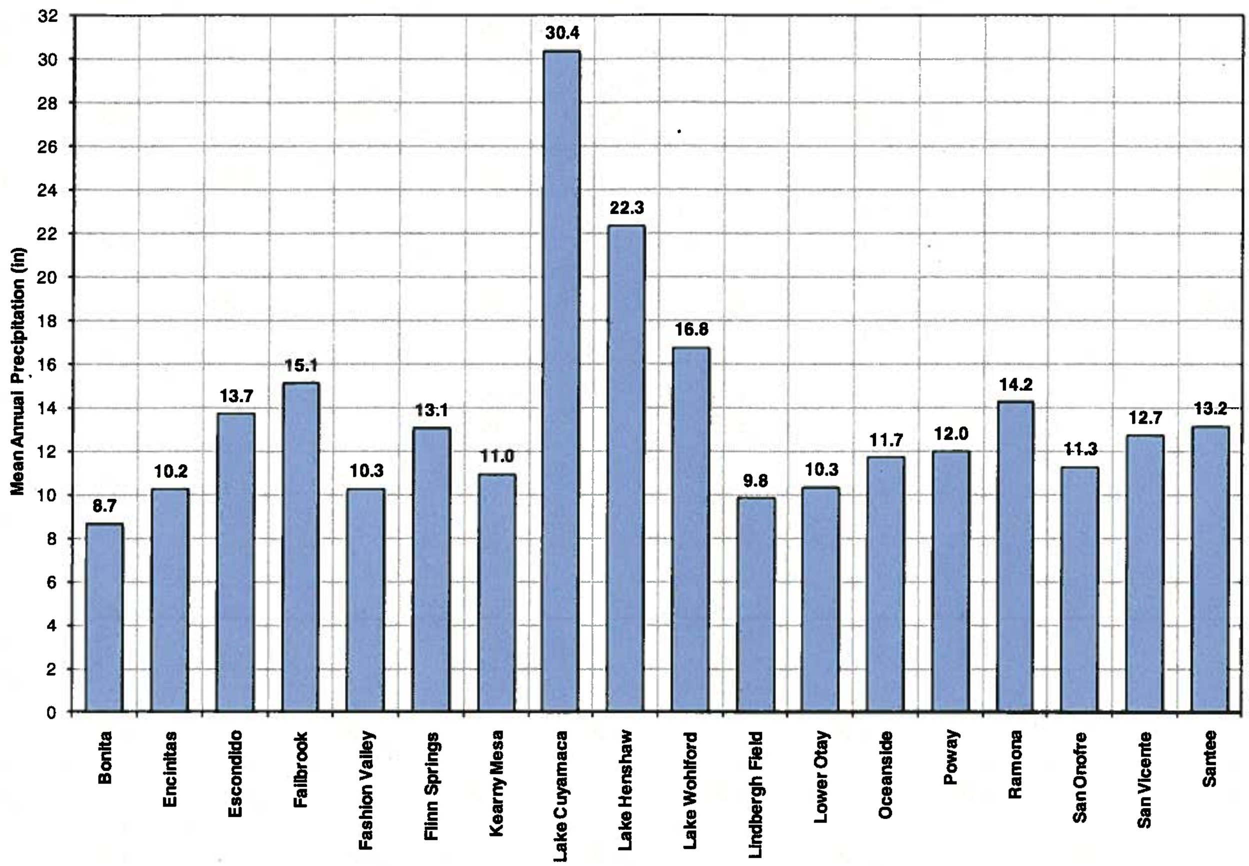


Figure 1. Rainfall Variation in San Diego County

2.1.2 San Diego Soils Map Evaluation

The HSPF model development was based on the commonly occurring and easy-to-identify soil hydrologic groupings used by the National Soils Conservation Service (NRCS). The NRCS uses four groupings called (in decreasing order of hydraulic conductivity) Group A, B, C and D. Group A soils are sandy and exceedingly well drained, while Group D soils are typically poorly drained clays. Group B and Group C soils exhibit hydraulic characteristics between those of Group A and Group D soils.

Figure 2 shows NRCS soil mapping for San Diego County. According NRCS data, about 43 percent of San Diego County is classified as NRCS Group D soils. Approximately one-quarter of the County consists of Group C soils and one-quarter Group B soils. The remaining 7 percent is classified as Group A soils. The well drained Group A and Group B soils occur more commonly in the eastern portions of the County that are not covered under this HMP. The central and western portions of the county consist mainly of the less hydraulically conductive Group C and Group D soils.

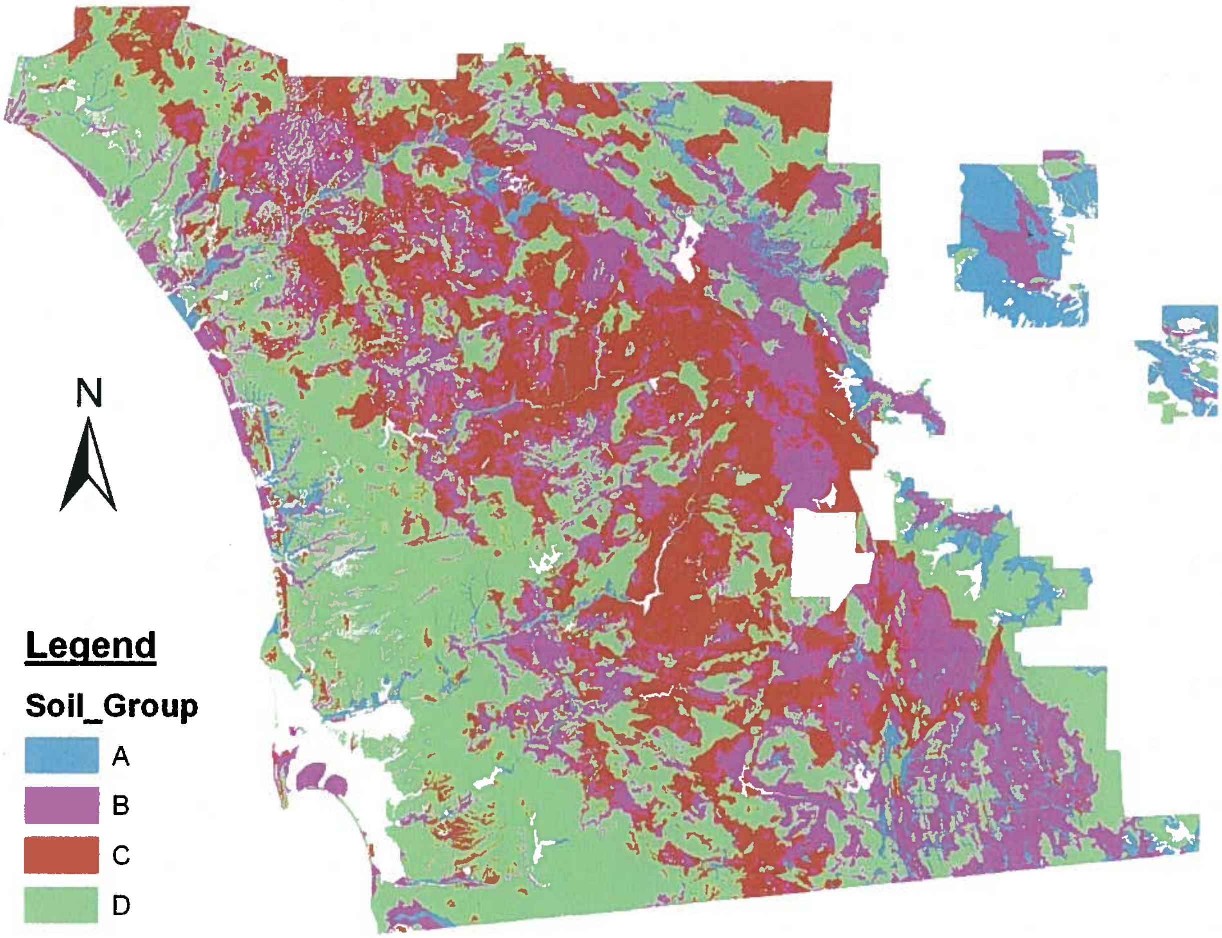


Figure 2. NRCS Soils Mapping of San Diego County

2.2 Scenarios Modeled

HSPF was used to characterize 12 different pre-project runoff scenarios corresponding to 4 soil types and 3 ranges of slopes. The range of land cover and vegetation types is not sufficiently variable among developable lands to require separate scenarios for different pre-project pervious land cover types. Table 2 below summarizes the scenario components. The specific HSPF pervious land surface parameters for these scenarios are described separately in the *San Diego HMP HSPF Model Parameter Selections* technical memo.

Table 2. HSPF Scenarios for Characterizing Pre-Project Conditions

Scenario No.	NRCS Soil Group	Land Cover	Slope
1	A	Scrub, Shrub	Low (<5%)
2	A	Scrub, Shrub	Moderate (10%)
3	A	Scrub, Shrub	Steep (>15%)
4	B	Scrub, Shrub	Low (<5%)
5	B	Scrub, Shrub	Moderate (10%)
6	B	Scrub, Shrub	Steep (>15%)

Table 2. HSPF Scenarios for Characterizing Pre-Project Conditions			
Scenario No.	**NRCS Soil Group**	**Land Cover**	**Slope**
7	C	Scrub, Shrub	Low (<5%)
8	C	Scrub, Shrub	Moderate (10%)
9	C	Scrub, Shrub	Steep (>15%)
10	D	Scrub, Shrub	Low (<5%)
11	D	Scrub, Shrub	Moderate (10%)
12	D	Scrub, Shrub	Steep (>15%)

3. Hydrologic Modeling Approach to Sizing BMPs

This section describes the technical approach used to represent BMPs in the HSPF model. The discussion focuses on the key physical aspects of BMP performance (i.e., how a BMP routes water through its different layers) and how these physical processes are represented in HSPF. This section also describes key hydraulic and modeling assumptions and how these assumptions impact both the modeling process and the accuracy of the results across the full range of flow conditions.

3.1 General BMP Characteristics

The flow control BMP designs selected by San Diego County and its Copermittees all include some combination of detention storage and water quality treatment media. For example, the bioretention BMP includes (in order of vertical routing) a surface ponding layer, a growing medium layer, and a storage layer. Each layer has its configuration, porosity, volume, and hydraulic conditions that influence the rate of flow to the next layer (see Figure 3).

HSPF uses stage-storage-discharge tables to represent the hydraulic behavior of devices that detain and discharge water (e.g., all of the LID BMPs included in the HMP). 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.

The following general hydraulic assumptions were applied to all of the BMPs modeled:

- Inflow is uniformly distributed over the area of the BMP (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 growing medium layer is assumed to be evenly distributed throughout the growing medium layer both vertically and horizontally. This assumes an engineered BMP would be free of macropores.
- Percolation from the growing medium layer to the storage layer is computed based on unsaturated or saturated hydraulic equations, based on the amount of moisture contained in the growing medium during each model time step.
- Water flows out the bottom of the BMP 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 BMP performance determined that porosity has little influence on the required sizing factor.

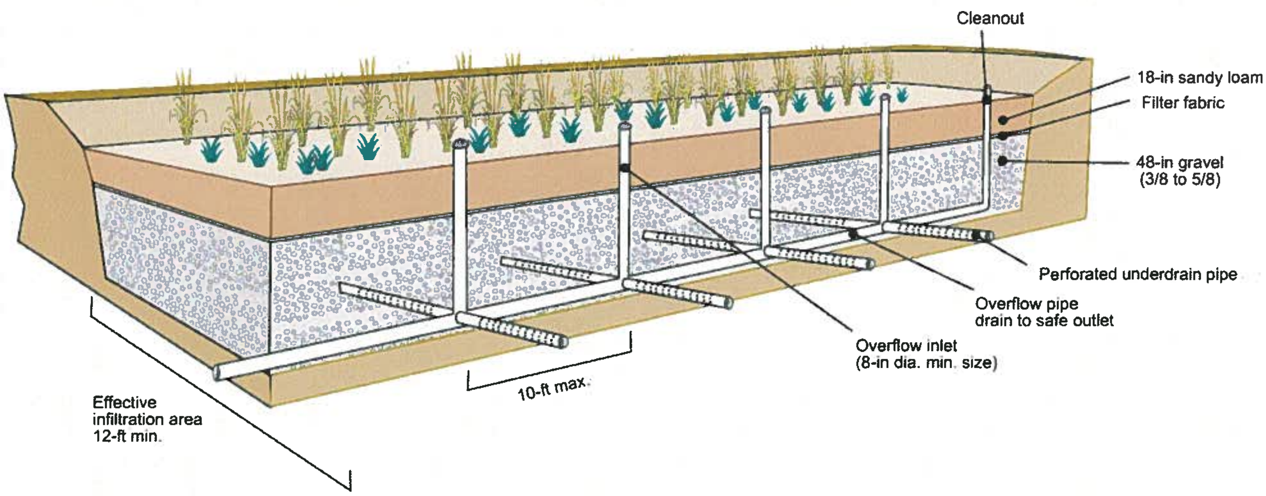


Figure 3. Cross-Section View of Bioretention BMP, Group C/D Soil Configuration

3.1.1 Bioretention BMP HSPF Representation

The bioretention BMP is modeled using two FTABLEs. The first FTABLE represents the surface ponding layer, growing medium layer, and overflow outlet. The second FTABLE represents the storage layer, exfiltration to surrounding soils, and underdrain outflow, if applicable. Percolation from the growing medium to the storage layer is modeled as an outflow from the first FTABLE and inflow to the second FTABLE.

FTABLE 1: Upper Growing medium layer, Ponding Storage and Overflow Outlet

Stormwater routed from impervious surfaces first enters the upper layer of an In-Ground Planter, represented by FTABLE 1 (Figure 4). 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 growing medium 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 growing medium layer at saturation. Above this depth water is stored in the ponding reservoir.

Water contained in the upper growing medium 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 growing medium 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 growing medium 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 growing medium 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
```

Figure 4. 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. Percolation outflow from the first FTABLE is routed as inflow to the second FTABLE (Figure 5). 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 lower flow control rate when the lower gravel layer is fully saturated.

```
 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 5. Example FTABLE Describing Lower Gravel Layer of In-Ground Planter

3.1.2 Iterative BMP Sizing Steps

Once the geometric characteristics of the BMP were represented in FTABLEs, as described above, 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 BMP 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 BMP serving an equivalent area that has been converted to an impervious surface. A 24-hour inter-event period (as defined by 24 hours with BMP outflow less than 0.05 cfs/ac) was used to separate storm events in the partial duration series. The footprint of the BMP was included in the calculations to preserve equivalence between the pre-project and post-project analysis (i.e. Pre-project Area = Impervious Area + BMP Area). The HSPF model allowed rainfall directly on the BMP.

BMP 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 mitigated post-project peak flows and flow durations were less than or equal to the pre-project flows, as defined by the performance criteria in the Final HMP.

4. Low-Impact Development (LID) BMP Descriptions

This section describes the LID BMPs that are included in the Countywide Model SUSMP, focusing on the elements that are explicitly represented within HSPF. The following LID BMPs will be evaluated for flow control and/or water quality treatment:

1. Bioretention
2. Cistern with bioretention
3. Bioretention with flow control vault
4. Flow-through planter
5. Dry well
6. Vegetated bioswale (for water quality treatment only)

Non-structural strategies for stormwater management, such as pervious pavement, self-retaining areas, and self-treating areas will be described in separate memoranda.

4.1 Bioretention

The bioretention facility consists of a surface ponding layer, a growing medium layer, and a below ground storage layer (Figure 6). The bioretention BMP captures water in the ponding layer, filters it through a growing medium that consists of soil and plant roots, percolates water from the growing medium into a storage layer, and then slowly discharges treated stormwater via exfiltration to surrounding native soils and regulated discharge through an underdrain pipe to the local stormwater drainage system. For applications with well-draining native soils (e.g., NRCS hydrologic group A or B soils), an underdrain pipe would not be included.

For the HMP, we will simulate the bioretention BMP using separate a) ponding layer, b) growing medium, and c) storage layer components. We will assume the following depths for each layer:

- **Ponding layer:** 10-inches active storage, 2-inches of freeboard above overflow relief
- **Growing medium:** 18-inches of soil at 40 percent porosity
- **Storage layer:** 30-inches of gravel at 40 percent porosity

As described above in Section 3.1.2,the plan area of the BMP will be iteratively sized until the BMP controls limit outflows to levels that are less than or equal to pre-project conditions across flow rates ranging from the lower flow control limit ($0.1Q_2$, $0.3Q_2$ or $0.5Q_2$) to the upper flow control limit (Q_{10}). The sizes of the ponding layer and storage layer will be converted into volumes, so that the project designer can flexibly configure the ponding layer and storage layer to meet site constraints. For example, the design engineer could configure the ponding layer with half the depth but twice the plan area called for by the sizing factor if this fits the project site. Additionally, the designer could use commercially-available storage vessels to meet the volume requirements instead of using gravel.

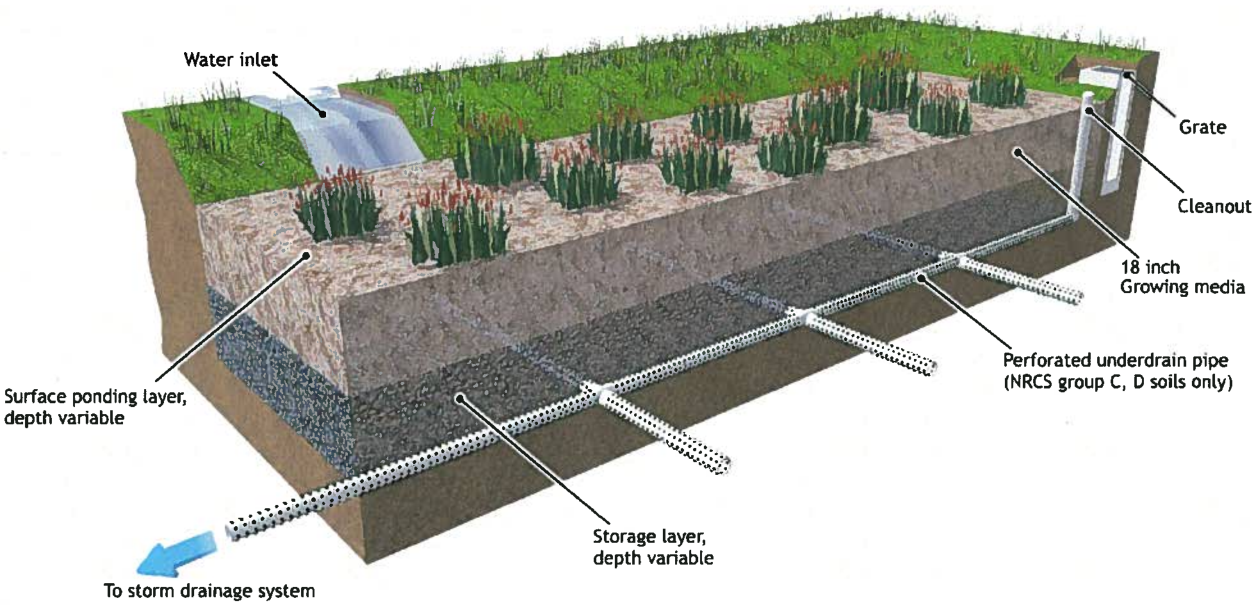


Figure 6. Bioretention BMP Example Illustration

4.2 Cistern with Bioretention

The cistern with bioretention BMP is a flow-control and treatment train BMP. There is no water quality treatment-only option. The cistern component captures and detains site runoff, and then slowly releases the water to a nearby bioretention device that provides water quality treatment by filtering the stormwater through its soil matrix.

The cistern will contain two outlets. A lower orifice will be located at the bottom of the cistern and will be designed to release water at the lower flow control limit ($0.1Q_2$, $0.3Q_2$ or $0.5Q_2$) where it will be routed through the bioretention device. Because the cistern accomplishes the flow control requirement, the bioretention only provides water quality treatment and an underdrain is permissible for all soil groups. However, due to the high infiltration capacity of NRCS hydrologic group A soils, the underdrain should only be used in Group B, C, and D soils. For Group A soils, the bioretention element is not necessary and cistern discharges should be routed into native soils for infiltration and treatment. A small depression should be included in the landscaping to provide sufficient time for infiltration to occur.

For the HMP, we will simulate the performance of the cistern with bioretention BMP using the following key assumptions:

- **Cistern configuration:** The cistern is modeled as a 4-foot tall vessel. However, designers could use other configurations, so long as the lower outlet orifice is sized to properly restrict outflows.
- **Cistern upper outlet:** The upper outlet from the cistern would consist of a weir or other flow control structure set at an elevation of 7/8th of the way to the top of the cistern (see Figure 7). The overflow weir would be sized to pass approximately 1 cfs per acre of tributary impervious area.
- **Bioretention configuration:** The bioretention needs only a small depression/ponding area to settle inflows prior to infiltration. For water quality treatment, the bioretention should be 1.5 feet deep and

contain the soil mixture specified in the *Countywide Model SUSMP* that allows a continuous infiltration rate of 5 inches per hour. The bioretention basin should be sized to pass the cistern outlet flows.

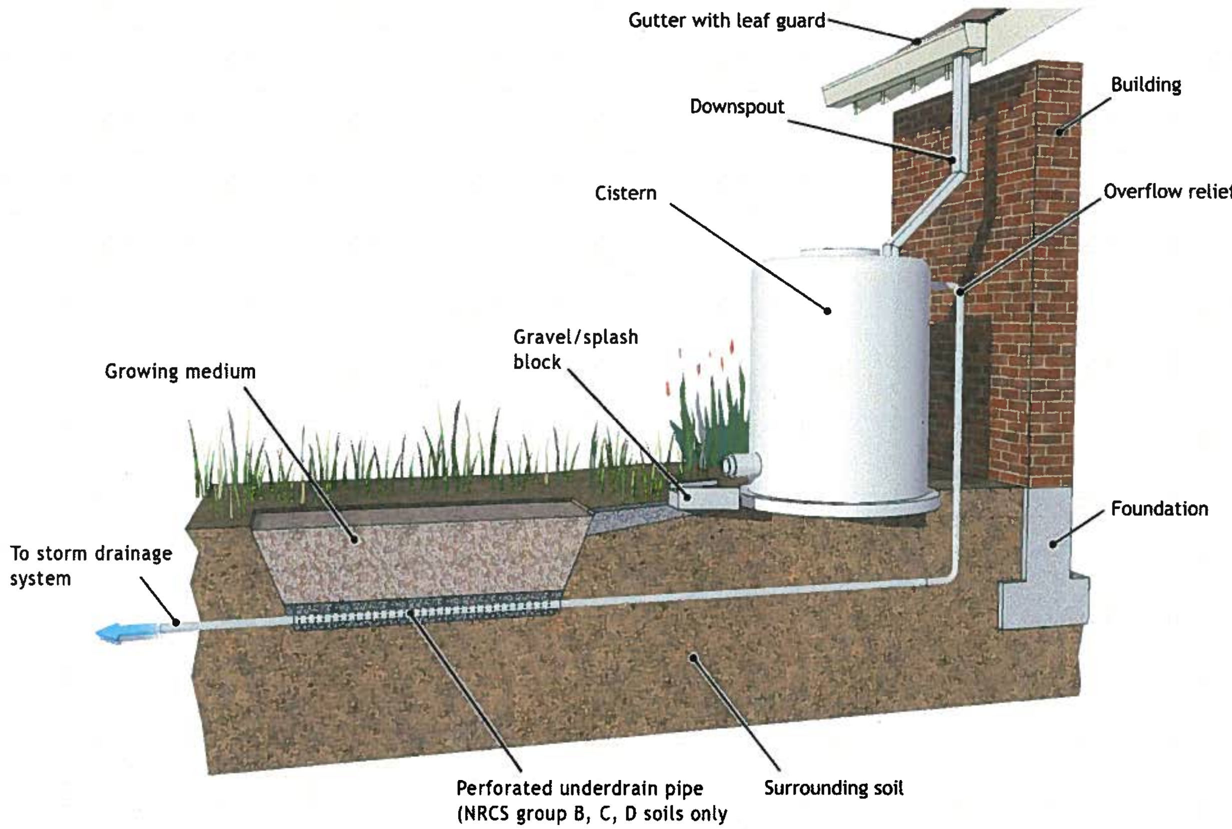


Figure 7. Cistern with Bioretention BMP Example Illustration

4.3 Bioretention with Vault

This BMP configuration would route stormwater through a bioretention basin for water quality treatment, and then discharge water to a nearby vault for detention and release (Figure 8). The vault would contain a lower orifice to restrict outflows to meet the HMP's flow control requirements. The vault portion of the BMP could be located below, adjacent or farther away from the bioretention portion of the BMP. This BMP is particularly effective in commercial applications where distributed water quality treatment outflows could be collected into a single vault for flow control underneath a parking lot. There is not water quality treatment-only option.

For the HMP, we will simulate the performance of the bioretention with vault BMP using the following key assumptions:

- **Bioretention configuration:** The bioretention portion of this BMP would be designed similarly to the bioretention BMP, except that the storage layer would be only deep enough to contain a perforated underdrain pipe that would convey treated runoff to the vault portion of the BMP.
- **Vault configuration:** The vault would contain concrete side wall and top, as well as an access hatch for inspection and maintenance. The bottom of the vault would be open to allow infiltration to the

surrounding soils. The vault was simulated as a 4-foot deep chamber, but the designer could select other configurations that were similar or lesser depths.

- **Vault outlets:** The vault would contain two outlets. The lower outlet would be a flow control orifice that would release water at a maximum rate equal to the lower flow control limit ($0.1Q_2$, $0.3Q_2$, $0.5Q_2$). The upper outlet from the vault would be located at 80 percent of the vault's height and would have a capacity of approximately 1 cfs per acre of tributary impervious area. The overflow relief should be located no lower than the elevation of the vault's inlet pipe.

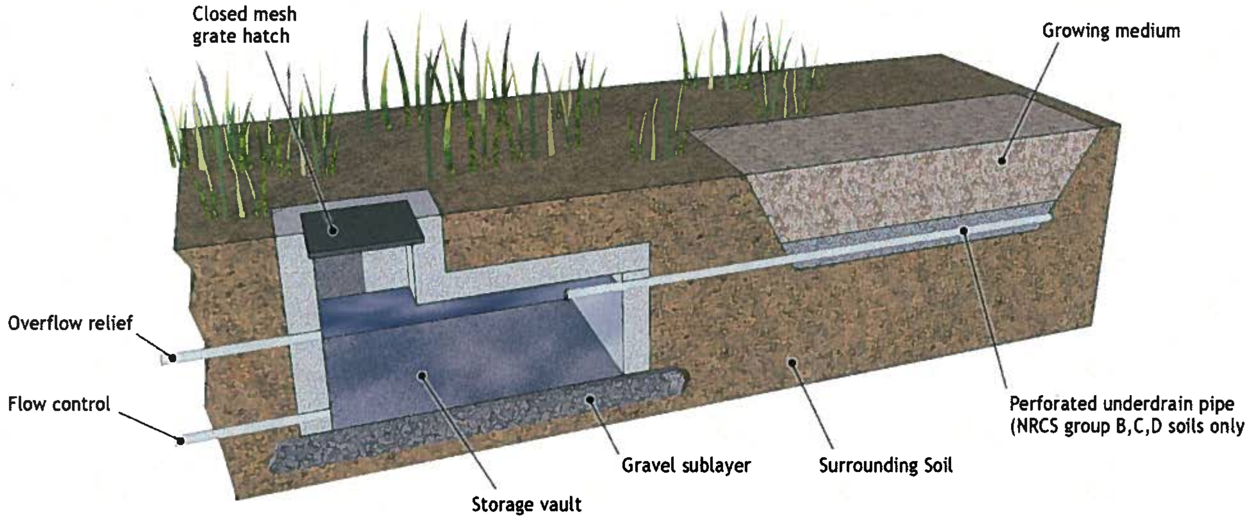


Figure 8. Bioretention with Vault BMP Example Illustration

4.4 Flow Through Planter

Flow-through planters treat and detain runoff without allowing seepage into the underlying soil. Typical applications would be next to buildings or on steep slopes, where the infiltration associated with bioretention facilities could cause problems. Flow-through planters typically receive runoff via downspouts leading from the roofs of adjacent buildings. However, they can also be set in-ground and receive sheet flow from adjacent paved areas.

Pollutants are removed as runoff passes through the growing medium layer and is collected in an underlying storage layer (Figure 9). A perforated-pipe underdrain is typically connected to a storm drain or other discharge point. An overflow inlet conveys flows which exceed the capacity of the planter. The flow through planter BMP should only be used in Group C or D soil applications.

For the HMP, we will simulate the flow through planter BMP using separate a) ponding layers, b) growing medium, and c) storage layer components. We will assume the following depths for each layer:

- **Ponding layer:** 10-inches active storage, 2-inches of freeboard above overflow relief
- **Growing medium:** 18-inches of soil at 40 percent porosity
- **Storage layer:** 30-inches of gravel at 40 percent porosity

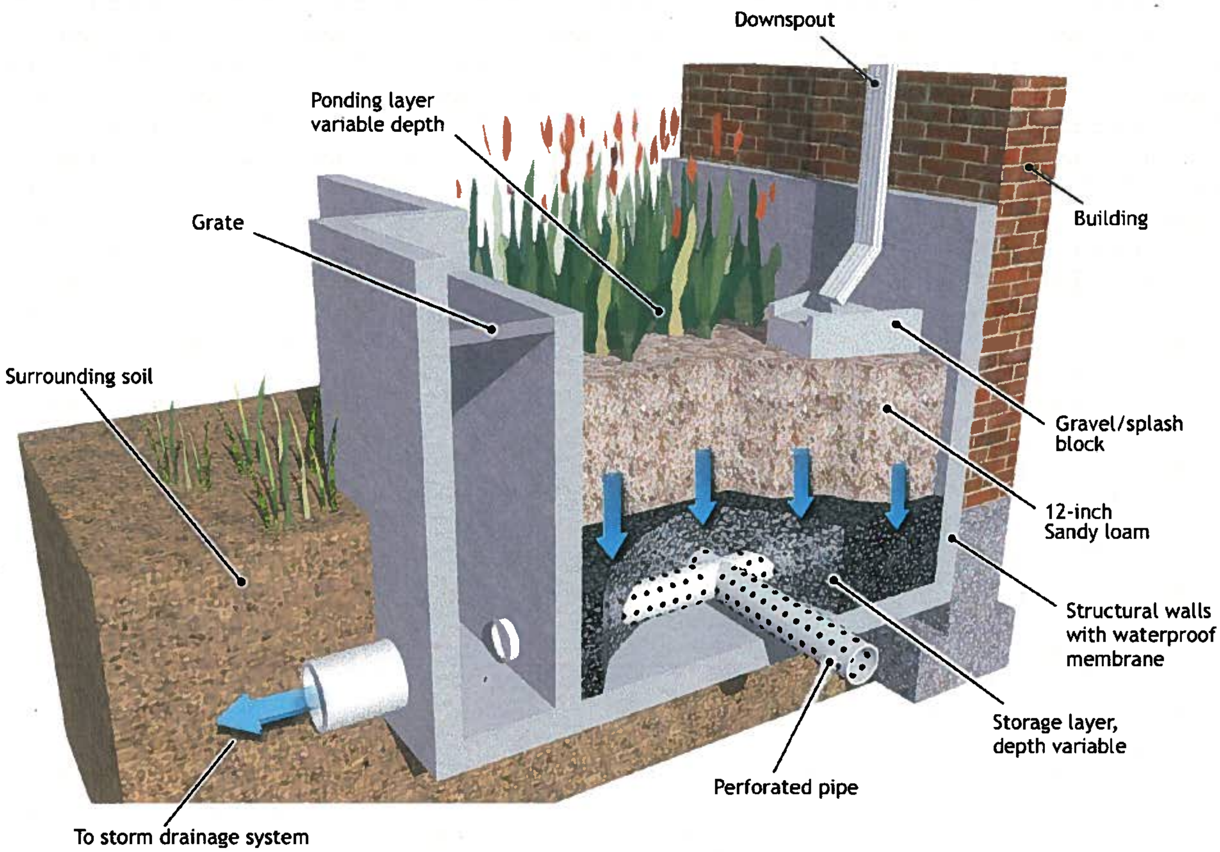


Figure 9. Flow Through BMP Example Illustration

4.5 Drywell

The drywell BMP is a below ground structure that can be used in areas with well-drained soils, such as NRCS Group A or B soils. The drywell consists of an initial soil layer to trap pollutants underlain with gravel, drain rock or some other free draining material (Figure 10). The dry well should have an access hatch to limit access.

For the HMP, we will simulate the drywell BMP using the following key assumptions:

- **Ponding layer:** a nominal 6-inch ponding layer should be included below the access hatch to allow for water spreading and infiltration during intense storms.
- **Soil layer:** 12 inches of soil should be included to remove pollutants
- **Free draining layer:** The drywell is sized assuming a 6-foot deep free draining layer. However, designers could use shallower drywells.

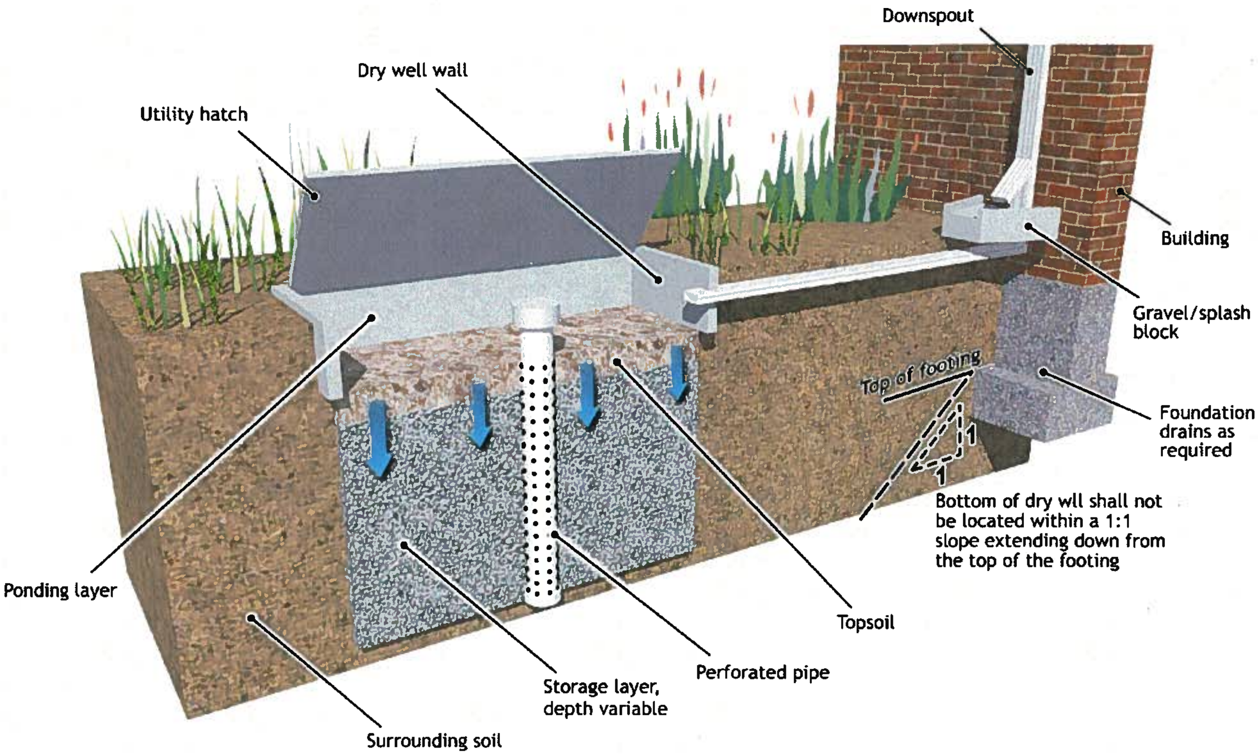


Figure 10. Dry Well BMP Example Illustration

4.6 Vegetated Bioswale

The vegetated bioswale BMP could be used to provide water quality treatment but not to provide stormwater flow control (Figure 11). The conventional swale design uses available on-site soils and does not include an underdrain system. Where soils are clayey, there is little infiltration. Treatment occurs as runoff flows through grass or other vegetation before exiting at the downstream end.

For the simulations, the vegetated bioswale will be sized to meet the minimum detention time specified in the Model SUSMP. If the bioswale is designed to pass stormwater through at a steady rate, its length will be computed based on flow velocities (i.e., slope and dimensions). If the vegetated bioswale is designed with a check dam structure to hold water, its size will be computed based on the volume provided by the swale.

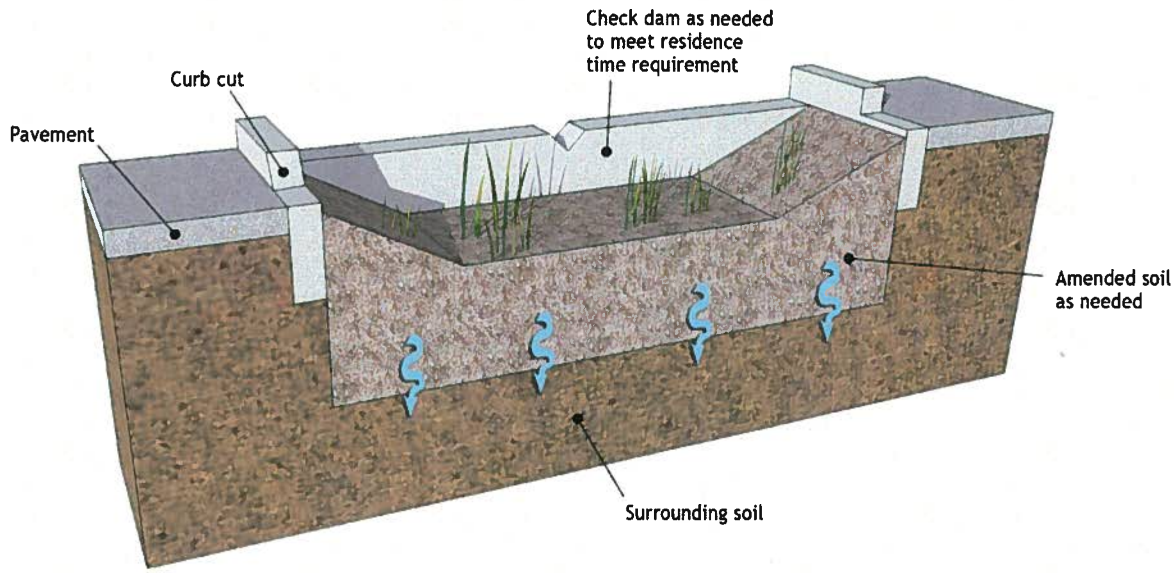


Figure 11. Vegetated Bioswale BMP Example Illustration

BROWN AND CALDWELL

Draft Technical Memorandum

9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
Tel: 858-514-8822
Fax: 858-514-8833

Project Title: San Diego County Hydromodification Management Plan

Project No: 133904

San Diego County Hydromodification Management Plan (HMP)

Subject: Selection of PERLND Parameters for HSPF Modeling

Date: April 23, 2010

To: Sara Agahi, San Diego County

From: Tony Dubin, Brown and Caldwell
Eric Mosolgo, Brown and Caldwell

This memorandum presents the HSPF PERLND parameters recommended for the San Diego HMP's Best Management Practice (BMP) sizing analysis. These parameter values will be used in HSPF to simulate runoff rates and other hydrologic processes across a range of pervious surface conditions. The resulting long-term runoff time series (and key statistical series computed from these time series) will form the pre-project condition baseline that new and redevelopment projects must match by mitigating site runoff rates and durations through the use of BMPs.

This memo is organized as follows:

Section 1 defines a PERLND and describes how HSPF simulates water movement on and through pervious surfaces.

Section 2 describes the published studies using HSPF that were reviewed for this project.

Section 3 summarizes the available PERLND parameter sets that were reviewed.

Section 4 describes how Brown and Caldwell (BC) tested various parameter values to identify sensitive parameters and examined how the selection of specific parameter values would affect the runoff time series.

Section 5 presents conclusions and recommendations.

1. PERLND Description and Schematic

The PERLND block within the HSPF input file contains parameters that affect the vertical and lateral movement of water moisture through a pervious land segment. Figure 1 is a schematic view of the PERLND water budget terms and key HSPF parameters. The schematic illustrates the movement of water among interception storage, upper zone storage, lower zone storage, groundwater storage, and deep/inactive groundwater storage. The schematic also illustrates flux terms, such as overland flow and interflow.

The algorithms that control the movement among these storage layers are described thoroughly in the HSPF User's Manual, which is available from the US EPA as part of the BASINS documentation (http://www.epa.gov/waterscience/basins/bsnsdocs.html). The parameters listed in Figure 1 are described in greater detail in Section 1.1.

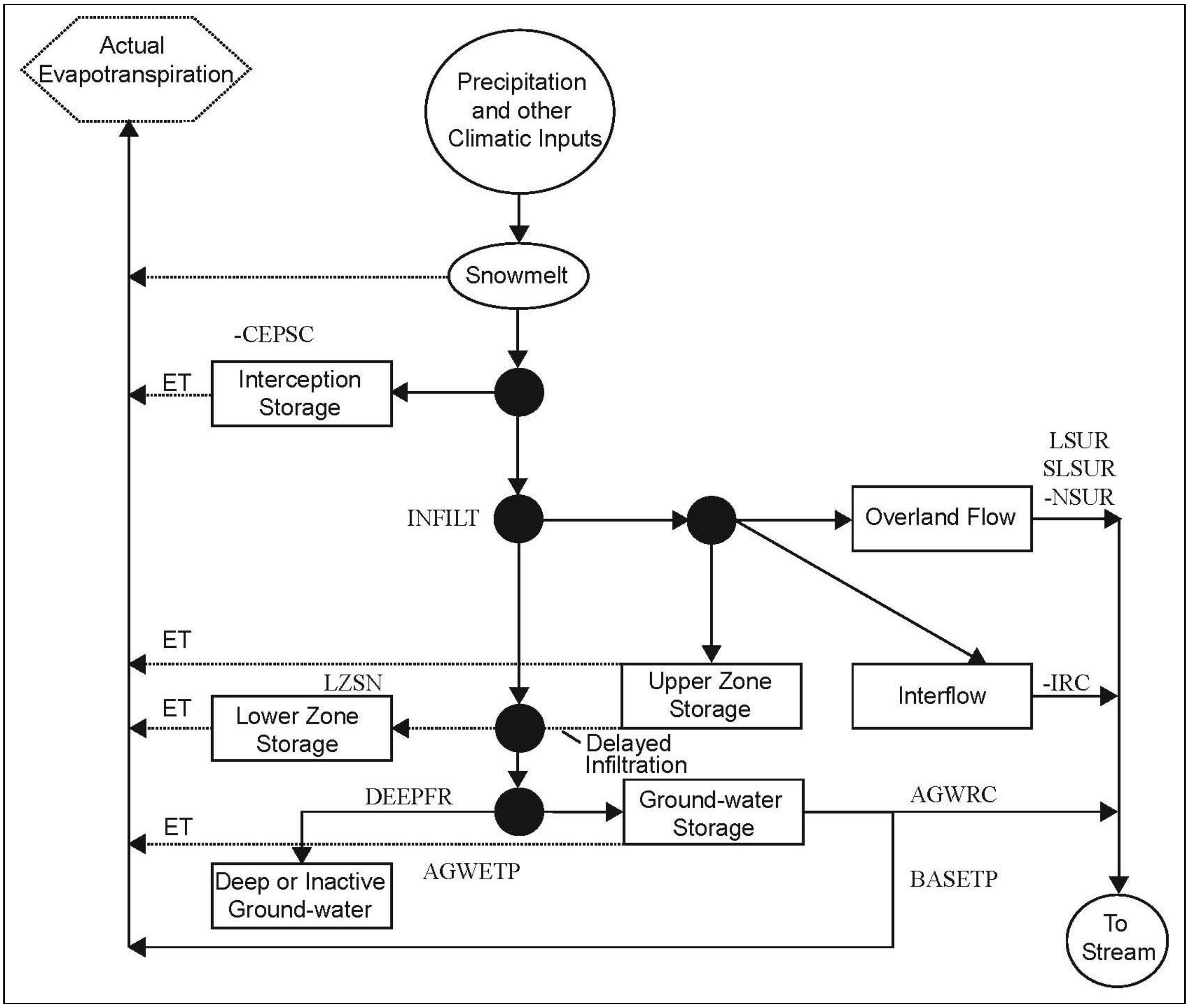


FIGURE 1

HSPF PERLND Water Moisture Schematic (Adapted from HSPF User's Manual)

1.1 PERLND Characteristics

The PERLND parameters shown in Figure 1 are located in the PWATER section of the PERLND block. PWATER, in turn, is divided into four sections, titled PWAT-PARM1, PWAT-PARM2, PWAT-PARM3, and PWAT-PARM4.

- PWAT-PARM1 is a series of flags that specify how various algorithms are to be used to compute hydrologic functions.
- PWAT-PARM2, PWAT-PARM3 and PWAT-PARM4 contain a series of climate, geology,

topography, and vegetation parameters and initial conditions.

Table 1 contains brief descriptions of the HSPF parameters used to characterize pervious land surfaces, along with commonly used ranges of values for these parameters. The parameters that often affect stormwater runoff most (INFILT, LZSN, LZETP) are highlighted in the table below. These highlighted parameters were the focus of our investigation of the range and variation among local HSPF studies and our testing of prospective parameters. The descriptions and parameter ranges in the table were adapted from *EPA BASINS Technical Note 6 – Estimating Hydrologic and Hydraulic Parameters for HSPF (Technical Note 6)*, which is available from the EPA web site, http://www.epa.gov/waterscience/basins/bsnsdocs.html.

TABLE 1
List of PERLND PWATER Parameters, Definitions and Common Range of Values[A]

PWAT-PARM1 – Flags			
Parameter	*Units*	*Description*	*Range of Values*
CSNOFG	None	Flag to use snow simulation data; must be set to if the SNOW simulation algorithms are to be used.	0 or 1
RTOPFG	None	Flag to select overland flow routing method. Set TOPFG=1; This method has been subjected to more widespread application.	1
UZFG	None	Flag to select upper zone inflow computation method Set UZFG=1; This method has been subjected to more widespread application.	1
VCSFG	None	Flag to select constant or monthly-variable interception storage capacity, CEPSC. Monthly value can be varied to represent seasonal changes in foliage cover	0 or 1
VUZFG	None	Flag to select constant or monthly-variable upper zone nominal soil moisture storage, UZSN.	0 or 1
VMNFG	None	Flag to select constant or monthly-variable Manning=s n for overland flow plane, NSUR. .	0 or 1
VIFWFG	None	Flag to select constant or monthly-variable interflow inflow parameter, INTFW. Monthly values are not often used.	0 or 1
VIRCFG	None	Flag to select constant or monthly varied interflow recession parameter, IRC. Monthly values are not often used.	0 or 1
VLEFG	None	Flag to select constant or monthly varied lower zone evapotranspiration (ET) parameter, LZETP.	0 or 1
PWAT-PARM2			
Parameter	*Units*	*Description*	*Range of Values*
FOREST	None	Fraction of land covered by forest that 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).	0 to 0.95
LZSN	**Inches**	**Lower zone nominal soil moisture storage. This parameter affects the proportion of water going to surface runoff, interflow and active groundwater**	**2 to 15**
INFILT	**in/hr**	**INFILT is the parameter that controls the overall division of the available moisture from precipitation (after interception) into surface runoff. This is NOT equivalent to a field-measured infiltration rate.**	**0.001 to 0.50**
LSUR	Feet	Length of assumed overland flow plane. LSUR approximates the average length of travel for water to reach any drainage path such as streams, swales, ditches, etc.	Estimate from mapping or GIS
SLSUR	ft/ft	Average slope of assumed overland flow path. Average SLSUR values for each land use being simulated can often be estimated directly with GIS capabilities.	Estimate from mapping or GIS
KVARY	1/inches	Groundwater recession flow parameter used to describe non-linear groundwater recession rate	0.0 to 5.0
AGWRC	None	Groundwater recession rate, or ratio of current groundwater discharge to that from 24 hours earlier	0.85 to 0.999
PWAT-PARM3			
Parameter	*Units*	*Description*	*Range of Values*

TABLE 1 List of PERLND PWATER Parameters, Definitions and Common Range of Values[A]			
PETMAX	Deg F	Temperature below which ET will be reduced to 50% of that in the input time series	32 to 48
PETMIN	Deg F	Temperature at and below which ET will be zero. PETMIN represents the temperature threshold where plant transpiration is effectively suspended	30 to 40
INFEXP	None	Exponent that determines how much a deviation from nominal lower zone storage affects the infiltration rate. This parameter is commonly set to a value of 2.	1 to 3
INFILD	None	Ratio of maximum and mean soil infiltration capacities. This parameter is commonly set to a value of 2.	1 to 3
DEEPFR	None	The fraction of infiltrating water that is lost to deep/inactive aquifers with the remaining fraction assigned to active groundwater storage that contributes base flow to the stream.	0.0 to 0.5
BASETP	None	ET by riparian vegetation as active groundwater enters streambed; specified as a fraction of potential ET, which is fulfilled only as outflow exists.	0.0 to 0.2
AGEWTP	None	Fraction of PERLND that is subject to direct evaporation from groundwater storage, e.g. wetlands or marsh areas.	0.0 to 0.2
PWAT-PARM4			
Parameter	*Units*	*Description*	*Range of Values*
CEPSC	inches	Amount of rainfall, in inches, which is retained by vegetation, never reaches the land surface, and is eventually evaporated.	0.01 to 0.40
UZSN	inches	Nominal upper zone soil moisture storage. UZSN is related to land surface characteristics, topography, and LZSN.	0.05 to 2.0
NSUR	None	Manning's friction coefficient, n, for overland flow plane.	0.02 to 0.50
INTFW	None	Coefficient that determines the amount of water that enters the ground from surface detention storage and becomes interflow	1.0 to 10.0
IRC	None	Interflow recession coefficient IRC is the ratio of the current daily interflow discharge to the interflow discharge on the previous day.	0.3 to 0.85
LZETP	**None**	**Index to lower zone evapotranspiration LZETP affects ET from the lower zone, which represents the primary soil moisture storage and root zone of the soil profile.**	**0.1 to 0.9**

A. The parameter descriptions and ranges were obtained from the EPA BASINS Technical Note 6.

2. Available Studies and HSPF Parameter Sources

Brown and Caldwell collected and examined published Southern California studies that used HSPF to perform hydrologic modeling. We previously summarized this effort in the technical memorandum entitled *Summary of HSPF Modeling Reports in Southern California*, dated May 2009. Whenever possible, we also collected the HSPF input files that were used in these studies. We examined studies of the following models and study areas:

- **Santa Monica Bay Watershed** - The Southern California Coastal Water Research Project (SCCWRP) and Tetra Tech created HSPF models to simulate hydrologic processes and pollutant loadings to Santa Monica Bay. The specific parameter values were selected by calibrating an HSPF model to flow monitoring data in the Santa Monica Bay watershed, specifically on Malibu Creek. The values represent a composite of the various upstream soils and land uses.

- **Calleguas Creek** - This project was a pilot study to evaluate the use of HSPF as a management tool for comprehensive watershed assessment within the climatic, physiographic, and topographic conditions of Ventura County. The Calleguas Creek model, developed by Aqua Terra Consultants, simulates watershed hydrology using a combination of six different land use categories, topographic data and soils data.

- **San Diego Hydrology Model (SDHM)** - The San Diego Hydrology Model (SDHM) uses a graphical user interface and pre-selected HSPF parameters to simulate stormwater runoff from development sites and size stormwater control facilities to mitigate the impacts of land use changes. SDHM includes HSPF parameters for common soil and land use combinations. The SDHM user's manual is available in the download section of Clear Creek Solutions' web site, http://www.clearcreeksolutions.com/SearchResults.asp?Cat=17.

We also examined other HSPF input sources for relevant information:

- **EPA BASINS Technical Note 6** - The EPA publication (July 2000) is a very useful guide that describes key HSPF parameters and suggests initial values. This technical note provides BASINS users with guidance in how to estimate the input parameters in the ATEMP, SNOW, PWATER, IWATER, HYDR, and ADCALC portions of the HSPF model.

- **Western Washington Hydrology Model (WWHM)** was developed by Clear Creek Solutions for the Washington Department of Ecology to size stormwater control facilities in western Washington. The model runs HSPF to generate over 40 years of hourly runoff data. The interface and range of input types are generally similar to the SDHM.

- **Calabazas Creek** - In 1997, Aqua Terra Consultants used HSPF to study multipurpose design of detention facilities in Calabazas Creek watershed for the Santa Clara Valley Water District.

3. Range of Available Southern California HSPF Parameters

Brown and Caldwell has compiled and assessed the similarities and variations among the PERLND parameters used for the Santa Monica Bay, Calleguas Creek and SDHM work efforts. For reference, BC also compiled the parameters contained in EPA BASINS Technical Note 6, WWHM version 3, and the Contra Costa HMP. Table 2 lists the minimum, maximum and average values of the PERLND PWATER parameters for each study.

It is difficult to make a direct comparison among the parameters used in previous studies, because these modeling efforts examined entire watersheds with varying levels of development, reservoirs and regulation, and water demands and usages. However, focusing on the general range of specific parameter can be informative. For example, the Santa Monica Bay and Calleguas Creek model files use generally similar values for the key parameters, such as INFILT and LZSN (lower zone storage nominal), while the Santa Monica study used a substantially higher value of LZETP (lower zone evapotranspiration potential). The SDHM, which specifies parameters for ranges of soils, land uses and slopes, has INFILT, LZSN and LZETP parameters that are in the same range as the Santa Monica Bay and Calleguas Creek models.

TABLE 2

Compilation of PERLND Parameters

		Southern California HSPF Research							General HSPF Research										Contra Costa HSPF Research						
		Santa Monica Bay	Calleguas			SDHM			Tech Note 6				WWHM v.3 (moderate slopes)						Calabazas Creek				Contra Costa HMP		
									Typical		Full Range		NRCS Group C			NRCS Group A/B			Developed		Open Space				
		Value	Min	Max	Avg	Min	Max	Avg	Min	Max	Min	Max	Forest	Grass	Pasture	Forest	Grass	Pasture	Min	Max	Min	Max	Min	Max	Avg
PWAT_PARM2	**Units**																								
FOREST	none	0	0	0	0	N/A	N/A	N/A	0	0.5	0	0.95	N/A	N/A	N/A	N/A	N/A	N/A	0	0	0	0	0	0	0
LZSN	inches	9.8	3	12.5	8.7	3.5	5.2	4.5	3	8	2	15	5	5	5	4.5	4.5	4.5	7	7	7	7	7	7	7
INFILT	in/hr	0.04	0.02	0.2	0.11	0.02	0.10	0.05	0.01	0.25	0.001	0.5	2	1.5	0.8	0.08	0.06	0.03	0.03	0.03	0.03	0.03	0.3	0.03	0.1595
LSUR	feet	201	150	400	319	200.0	400.0	312.5	200	500	100	700	400	400	400	400	400	400	200	250	150	200	660	660	660
SLSUR	ft/ft	0.03	0.00	0.30	0.11	0.1	0.3	0.1	0.01	0.15	0.001	0.3	0.1	0.1	0.1	0.1	0.1	0.1	0.0065	0.0533	0.068	0.28	0.1	0.1	0.1
KVARY	1/inches	3.0	0.5	1	0.61	0.8	3.0	1.5	0	3	0	5	0.3	0.3	0.3	0.5	0.5	0.5	0	0	0	0	0	0	0
AGWRC	none	0.92	0.80	1.00	0.91	1.0	1.0	1.0	0.92	0.99	0.85	0.999	0.996	0.996	0.996	0.996	0.996	0.996	0.8	0.95	0.8	0.95	0.95	0.95	0.95
PWAT_PARM3																									
PETMAX (F)	F	35	40	40	40	NA	NA	NA	35	45	32	48	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	40	40	40
PETMIN (F)	F	30	35	35	35.0	NA	NA	NA	30	35	30	40	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	35	35	35
INFEXP	none	2	2	2	2	2.0	3.0	2.3	2	2	1	3	2	2	2	2	2	2	2	2	2	2	2	2	2
INFILD	none	2	2	2	2	2.0	2.0	2.0	2	2	1	3	2	2	2	2	2	2	2	2	2	2	2	2	2
DEEPFR	none	0.4	0	0.8	0.67	0.0	0.0	0.0	0	0.2	0	0.5	0	0	0	0	0	0	0.1	0.45	0.1	0.45	0.45	0.1	0.275
BASETP	none	0.05	0	0.26	0.05	0.0	0.0	0.0	0	0.05	0	0.2	0	0	0	0	0	0	0	0	0	0	0	0	0
AGWETP	none	0.05	0	0	0	0.0	0.0	0.0	0	0.05	0	0.2	0	0	0	0	0	0	0	0	0	0	0	0	0
PWAT_PARM4																									
CEPSC	inches	0.10	0.06	0.12	0.08	0.10	0.19	0.13	0.03	0.20	0.01	0.40	0.2	0.15	0.1	0.2	0.15	0.1	0.0	0.0	0.0	0.0	0.1	0.02	0.07
UZSN	inches	1.18	0.50	0.80	0.59	0.20	0.50	0.31	0.1	1	0.05	2	0.5	0.5	0.5	0.43	0.35	0.22	0.4	0.4	0.6	0.6	0.5	0.5	0.5
NSUR	none	0.20	0.15	0.25	0.18	0.20	0.35	0.27	0.15	0.35	0.02	0.5	0.35	0.3	0.25	0.35	0.3	0.25	0.1	0.2	0.4	0.4	0.3	0.3	0.3
INTFW	none	1.50	1.00	1.80	1.35	0.35	1.00	0.81	1	3	1	10	0	0	0	6	6	6	0.4	0.4	0.5	0.5	0.4	0.4	0.4
IRC	none	0.70	0.20	0.60	0.35	0.30	0.80	0.46	0.50	0.70	0.30	0.85	0.70	0.70	0.70	0.43	0.43	0.43	0.30	0.30	0.40	0.40	0.30	0.30	0.30
LZETP	none	0.70	0.40	0.50	0.43	0.20	0.69	0.51	0.20	0.70	0.10	0.90	0.70	0.40	0.25	0.70	0.40	0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00

4. Evaluating HSPF Parameter Values

To determine the mix of pre-project conditions to include in the BMP Sizing Calculator, Brown and Caldwell examined the extent of variation among the PERLND parameters among the Santa Monica Bay, Calleguas Creek, and SDHM models.

Figures 2, 3 and 4 show the variation in the INFILT parameter used in the SDHM as function of slope and land cover. The INFILT parameter values clearly vary with slope. However, the INFILT parameter value is the same for the most common pre-project land cover types for new developments in San Diego County – shrub, grass, and dirt. The INFILT parameter value is higher for forest and lower for urban (i.e., compacted soils and irrigated landscapes), but these do not represent pre-project conditions that will be commonly managed by the BMP Sizing Calculator.

- Since the INFILT parameters are identical across the three most common pre-project land cover types, the modeling effort will focus on a single composite land cover type.
- The INFILT values vary significantly for different slopes. As such, parameter sets will be prepared for low, moderate, and steep slope classifications (5, 10 and 15 percent, respectively). In many cases, LID BMPs will not be feasible in areas with slopes that are steeper than this range. Further, because the pre-sizing analysis would potentially under-estimate pre-project runoff rates from very steep sites, any LID facilities designed in such areas using the BMP Sizing Calculator would be conservatively sized.
- An urban parameter set is not needed for the BMP Sizing Calculator. The *Countywide Model SUSMP* encourages developers to manage runoff from landscaped surfaces using grading and soil amendments that emphasize infiltration to reduce site runoff from landscaped areas without implementing LID BMPs. An urban parameter set can be developed for the automated pond sizing tool, because ponds are expected to capture flows from a combination of impervious and urban landscaped surfaces.

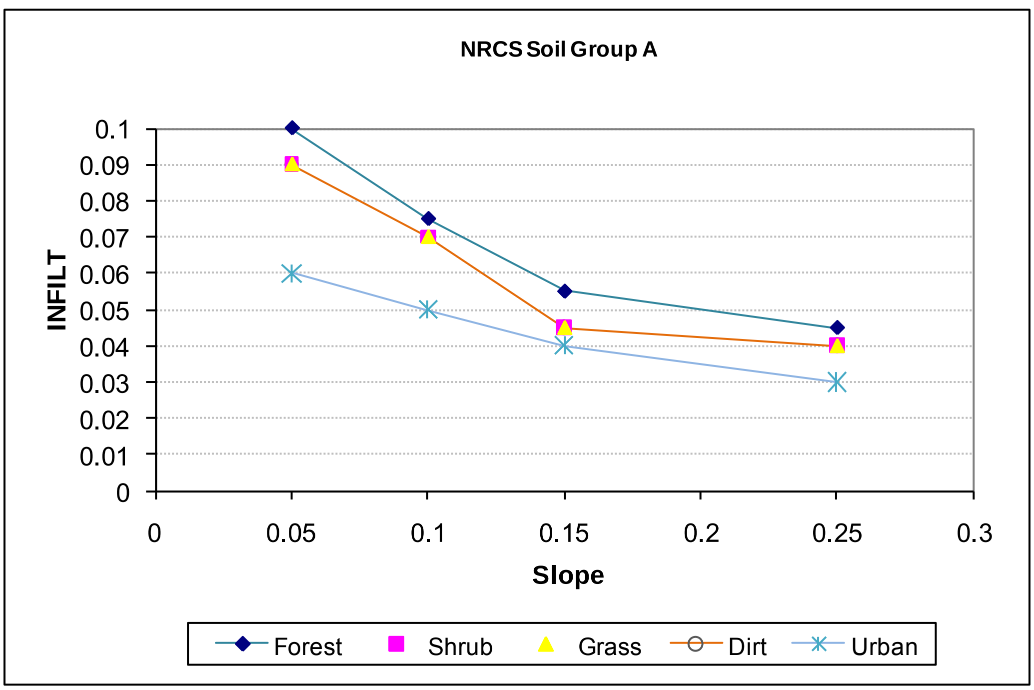


FIGURE 2
SDHM Variation in INFILT Parameter, NRCS Group A Soils

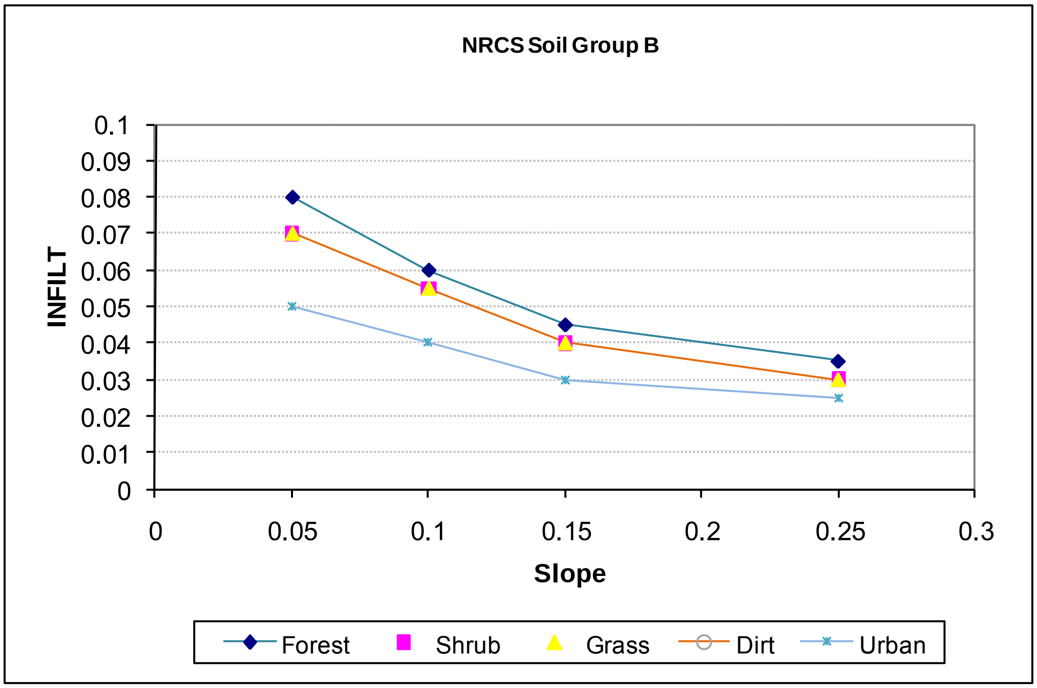


FIGURE 3

SDHM Variation in INFILT Parameter, NRCS Group B Soils

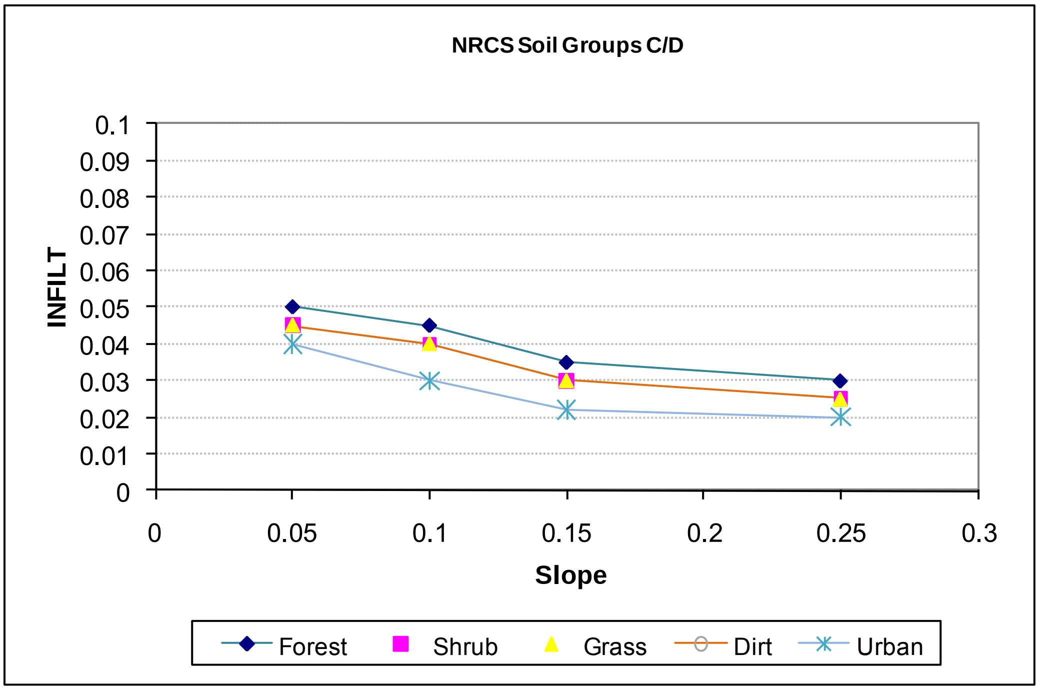


FIGURE 4

SDHM Variation in INFILT Parameter, NRCS Group C/D Soils

Figure 5, 6 and 7 show the SDHM model's assumed variations in the LZSN parameter as a function of slope and land cover type. Similar to the INFILT evaluation above, LZSN values are identical for the most common land cover types that will be incorporated in the BMP Sizing Calculator. These figures further reinforce the intention to focus on a single composite land cover type, while focusing on the differences in runoff generation potential associated with different soils and slopes.

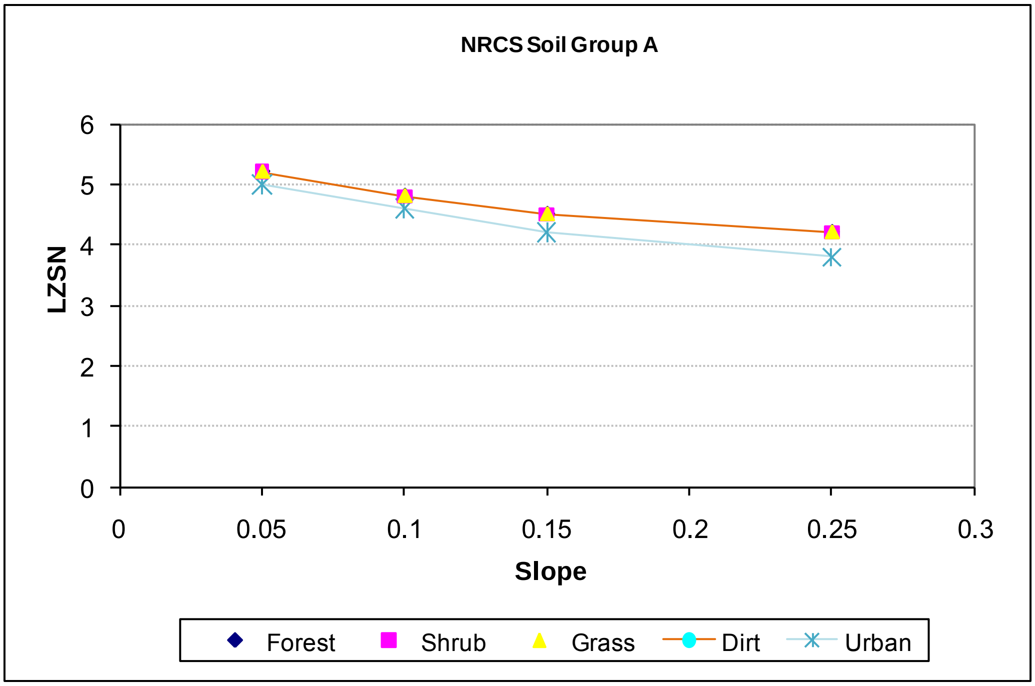


FIGURE 5

SDHM Variation in LZSN Parameter, NRCS Group A Soils

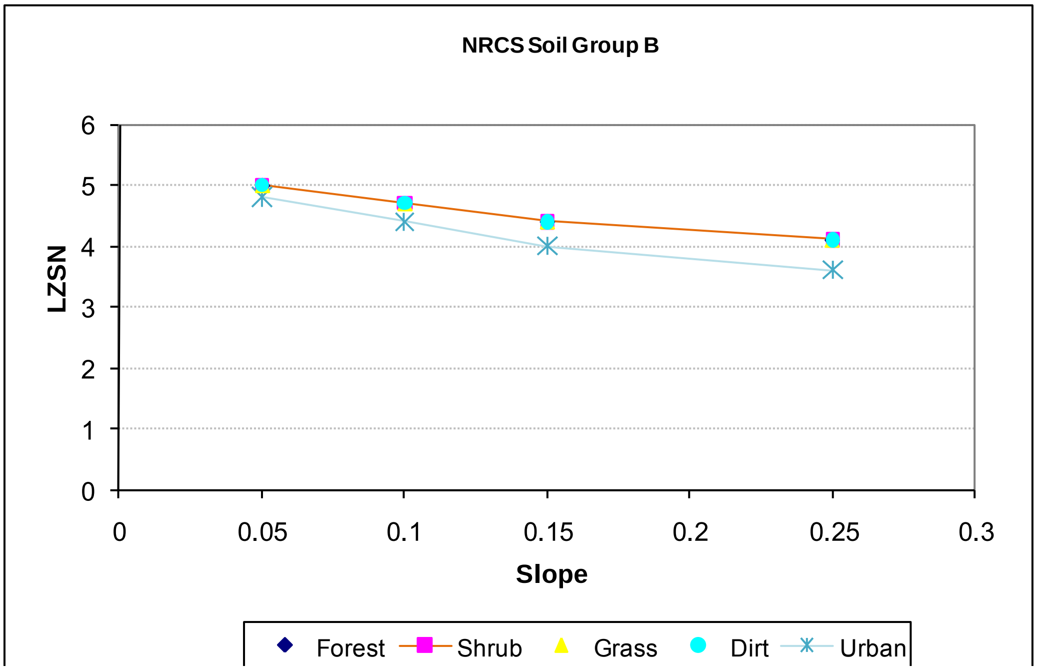


FIGURE 6

SDHM Variation in LZSN Parameter, NRCS Group B Soils

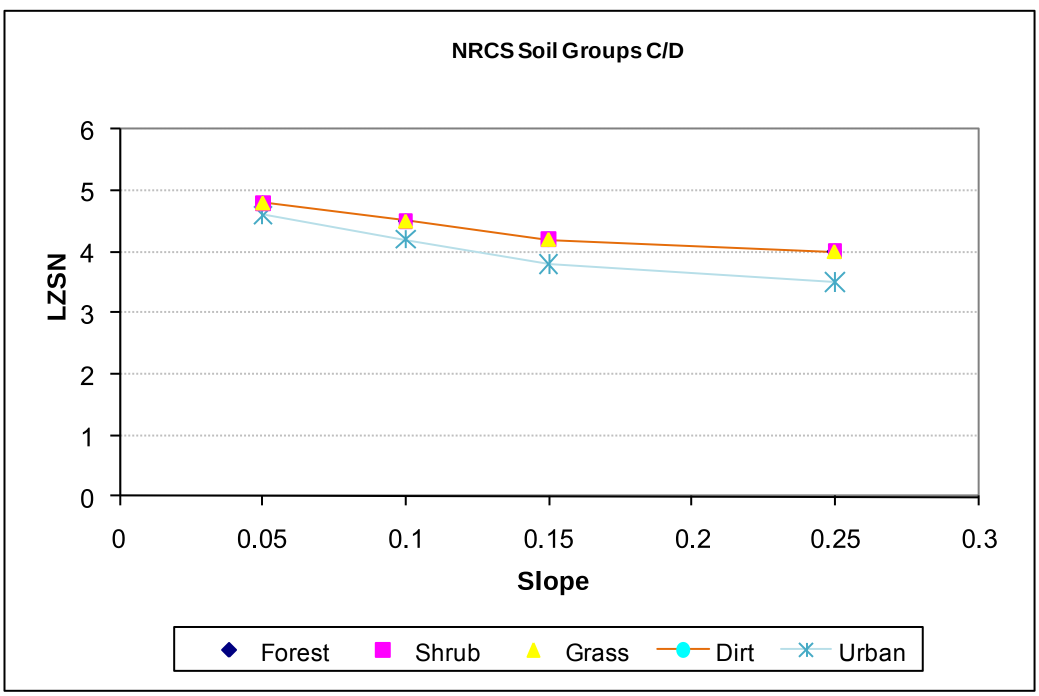


FIGURE 7

SDHM Variation in LZSN Parameter, NRCS Group C/D Soils

5. Recommended HSPF PERLND Parameters

The following recommended HSPF PERLND parameter values have been developed to use for LID pre-sizing factor analysis that will be included in the BMP Sizing Calculator. The 12 parameter sets cover the four NRCS soil groups and three separate slopes. The precise values were obtained by combining the Santa Monica Bay, Calleguas Creek, and SDHM parameter sets.

TABLE 3

Recommended HSPF PERLND Parameters for BMP Modeling

		Group A			Group B			Group C			Group D		
		5%	10%	15%	5%	10%	15%	5%	10%	15%	5%	10%	15%
PWAT_PARM2	**Units**												
FOREST	None	0	0	0	0	0	0	0	0	0	0	0	0
LZSN	inches	5.2	4.8	4.5	5.0	4.7	4.4	4.8	4.5	4.2	4.8	4.5	4.2
INFILT	in/hr	0.090	0.070	0.045	0.070	0.055	0.040	0.050	0.040	0.032	0.040	0.030	0.040
LSUR	Feet	200	200	200	200	200	200	200	200	200	200	200	200
SLSUR	ft/ft	0.05	0.1	0.15	0.05	0.1	0.15	0.05	0.1	0.15	0.05	0.1	0.15
KVARY	1/inches	3	3	3	3	3	3	3	3	3	3	3	3
AGWRC	None	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
PWAT_PARM3													
PETMAX (F)	F	35	35	35	35	35	35	35	35	35	35	35	35
PETMIN (F)	F	30	30	30	30	30	30	30	30	30	30	30	30
INFEXP	None	2	2	2	2	2	2	2	2	2	2	2	2
INFILD	None	2	2	2	2	2	2	2	2	2	2	2	2
DEEPFR	None	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
BASETP	None	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
AGEWTP	None	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
PWAT_PARM4													
CEPSC	inches	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08	0.08
UZSN	inches	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
NSUR	None	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
INTFW	None	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
IRC	None	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
LZETP	None	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5

BROWN AND CALDWELL

DRAFT TECHNICAL MEMO

9665 Chesapeake Drive, Suite 201
San Diego, CA. 92123
Tel: 858-514-8822
Fax: 858-514-8833

Project No: 133904

San Diego Hydromodification Management Plan

Subject: Responses to Comments on HMP Modeling Approach and BMP Configurations

Date: April 23, 2010

To: Sara Agahi, P.E. – County of San Diego

From: Eric Mosolgo, P.E. – Brown and Caldwell
Tony Dubin, P.E. – Brown and Caldwell

This draft technical memorandum summarizes the review comments received regarding the HMP Modeling Approach and BMP Configurations Draft Technical Memorandum (dated March 2, 2010) and Brown and Caldwell's (BC's) responses to these comments.

Review comments were received from the following groups:

- San Diego County Flood Control - Anthony Barry
- Clear Creek Solutions - Doug Beyerlein and Joe Brascher
- West Consultants - Marty Teal
- Hunsaker & Associates - Luis Parra

Table 1 below provides a summary of each comment, the corresponding page location from the originial Draft Technical Memorandum, and Brown and Caldwell's response.

No.	Reviewer	Page	Comment	Response
1	Anthony Barry, San Diego County	7	It is mentioned on page 7 that the vegetation types are not "sufficiently variable among developable lands to require separate scenarios". This statement should be further justified in a manner similar to the effects of porosity in the last bullet on page 8.	We made the decision to focus on one land cover type for the following reason: 1) We assume the vast majority of projects regulated by the HMP flow control standard will be in previously undeveloped areas with scrub vegetation. Conversely, many of the areas with "landscape/grass" as a pre-project land cover may not be covered by the flow control requirements, because these projects would be small or located in urban areas that qualify for some type of exemption. 2) We examined the range HSPF PERLND parameter values used in previous Southern CA studies and values included in the SDHM software – in particular the INFILT parameter. The parameter values were less variable for different land cover types than we expected, and this makes it likely that the variation in land cover type would have little impact on the computed LID BMP sizes. 3) The modeling "pre-sizing" analysis used to compute the sizing factors for the Sizing Calculator requires us to constrain or limit the variability of input parameters as much as possible. Because scrub/shrub vegetation will be encountered in most of the development projects, we think this is a good place to start with HSPF simulations and the Sizing Calculator. The County and its Copermittees could add more land cover types, BMPs or other features in V2.0 of the Sizing Calculator.

No.	Reviewer	Page	Comment	Response
2	Anthony Barry, San Diego County	11	Is the BMP area discussed in the second paragraph of Section 3.1.2 considered to be pervious area in the modeling? If so, and alterations to the standard (modeled) design are allowed (as suggested in the last paragraph on page 12), the depth of a BMP could be increased and the plan area decreased. This could allow an increase in the actual impervious area above what was considered in the modeling, without requiring the appropriate increase in required BMP volume. <u>FOLLOW-UP COMMENT:</u> After reviewing the response to my second comment (#2 on Page 3 of the response document) and looking back at the Modeling Approach, I discovered that the BMP area is modeled as being pervious as outlined in the 5th bullet on page 8 (of the document dated March 2, 2010), which states that there is flow out of the bottom of the basin (percolation) equal to the rate of hydraulic conductivity. It seems logical that significant reductions in the BMP area would also significantly reduce the area available for this outflow, and thereby increase the necessary volume. If this were not the case you would have to wonder if the percolation has any effect at all, and if it doesn't, why is it included in the modeling?	The BMPs are modeled using "FTABLEs" in HSPF, which detail stage-area-volume-discharge relationships. The model allows rainfall to occur directly on the BMP (as would happen in real life). The Sizing Calculator contains a "check" to ensure that the total contributing watershed area to the mitigation facility plus the mitigation facility area equates to the total project area. If a development engineer incorporates a narrow/deep ponding layer, the computed drainage management area (DMA) tributary to the BMP must accurately reflect the entire paved area draining to the BMP. We suggest allowing development engineers to vary the configuration of the surface ponding layer to better fit site constraints (e.g., wider/shallower to limit trip hazards; narrower/deeper to limit at-grade footprint). However, the sizing factor is based on the plan area of the growing medium underneath. For the Contra Costa HMP, we tested the sensitivity of BMP sizes to different ponding layer configurations and found that as long as the recommended volume is provided, wide/shallow, deep/narrow surface ponding layer configurations performed similarly enough not to impact BMP sizing factors. <u>RESPONSE TO FOLLOW-UP COMMENT:</u> As discussed by phone, all of the infiltration that occurs beneath the BMP is managed in the FTABLE (see Q Perc column in Figure 5), and not through the hydrologic elements of the HSPF model.
3	Anthony Barry, San Diego County	17	If vegetated bioswales are for water quality only, and do not provide flow control (as mentioned on page 17), why are they being included? Details on how to design a vegetated swale are covered in the CASQA BMP handbook.	This "treatment only" option has been proposed in response to a request from the Copermittees. The Sizing Calculator will allow users to size BMPs to meet either the "water quality treatment only" OR "flow control + treatment" requirement. The vegetated bioswale option will only be available as a selection in the Sizing Calculator if the user chooses "water quality treatment only" as the project design goal.

No.	Reviewer	Page	Comment	Response
4	Doug Beyerlein, Joe Braschcher; Clear Creek Solutions	7	Only one land cover vegetation type is offered: scrub, shrub. Different vegetation types change the pre-development runoff. The user should be given more vegetation type options.	See response to comment #1.
5	Doug Beyerlein, Joe Braschcher; Clear Creek Solutions	7	The sizing calculator assumes no increase in pervious runoff with development. Runoff from pervious surfaces can and does increase with development due to soil compaction from construction activities, the replacement of native vegetation with urban vegetation, and the addition of irrigation. These effects should be included in the sizing of BMP facilities.	The Countywide Model SUSMP adequately addresses this issue with its requirements for managing runoff from developed/pervious areas. For example, the SUSMP directs project proponents to control pervious runoff as much as possible using grading patterns, soil amendments, etc., so that these areas do not contribute runoff to paved areas and do not increase overall site runoff (relative to pre-project conditions). If a pervious area does drain to a paved area, and then into a BMP, this area must be accounted for within the Sizing Calculator to ensure the BMP is appropriately sized.
6	Doug Beyerlein, Joe Braschcher; Clear Creek Solutions	N/A	The HSPF parameter values selected for the BMP sizing calculator are critical in the computation of the existing and development runoff and the sizing of BMP facilities. However, we have had no opportunity to review and comment on these parameter values.	We will issue a separate technical memorandum detailing the selection of HSPF model parameters.
7	Doug Beyerlein, Joe Braschcher; Clear Creek Solutions	N/A	We have had no opportunity to review and comment on the HSPF FTABLEs used to represent different BMP facilities nor their associated HSPF UCI files. Nor have we had the opportunity to review and comment on the assumptions used in the construction of the HSPF FTABLEs and UCI files that produce the facility sizing results reported by the BMP sizing calculator.	The soil physics and key assumptions used to route water through the BMPs are described in Appendix A. This will be distributed to the TAC.
8	Marty Teal, West Consultants	2	Will it be obvious which of the various lower control threshold values someone is supposed to use/analyze?	Yes. The critical flow calculator allows a project proponent to determine which lower control threshold will apply to a specific project site. The critical flow calculator will be included in the overall BMP Sizing Calculator.
9	Marty Teal, West Consultants	2	Will the Sizing Calculator automatically determine/report whether a proposed BMP will meet the peak flow and flow duration performance requirement?	Yes. The Sizing Calculator will compute and report a BMP's minimize required size to meet the HMP stormwater control performance requirements.

No.	Reviewer	Page	Comment	Response
10	Marty Teal, West Consultants	8	Page 8 states, “Infiltration and soil water movement is a 1-dimensional flux in the vertical direction (neglecting lateral flows is a conservative assumption).” Why is neglecting lateral flows a conservative assumption?	If we assumed water would move laterally out of the BMP, the BMP would have a higher capacity to capture and mitigate stormwater flows.
11	Luis Parra, Hunsaker	7	The model does not include enough variability of vegetation cover to characterize the expected variation on infiltration. Among the most important vegetation type excluded, grass in fair to good condition comes to mind, as many of the developed areas will occur in this type of existing vegetation.	See response to comment #1.
12	Luis Parra, Hunsaker	9	Figure 3 is wrong and does not correspond with the one presented in page 78 of the Countywide Model SUSMP. The French drain should be placed on top, with only few inches on gravel above the top of the pipe, and the gravel below. This way, water retained below the French drain will be incorporated into the underground media.	Figure 3 was included simply to describe the function of LID BMPs. It could be replaced with Figure 6, which is consistent with the Countywide Model SUSMP.
13	Luis Parra, Hunsaker	9-11	As the hydraulic conductivity of the amended soil is the flow constraining factor (and less than the conductivity of the gravel) the only way that gravel on top makes sense is if an orifice constraining the flow in the French drain is used. This aspect, however, does not exclude the possibility of having some retention below the French drain. For instance, in a soil Type D with a hydraulic saturated conductivity of 0.1 in/hr, the equivalent of 7.2 in of ponding can be placed below the invert elevation of the French drain. With an assumed porosity of 0.4, this corresponds to an additional retention depth of 18” below the French drain. The model should allow retention as a function of the hydraulic conductivity of the bottom soil.	The gravel layer is proposed below the amended soil layer. We will follow up with reviewer to clarify this comment.

No.	Reviewer	Page	Comment	Response
14	Luis Parra, Hunsaker	N/A	Water table constraints should be included in the model. For example, it should be recommended that the water table must be at a given depth below the bottom of the gravel to be sure that the vertical assumption is valid.	The Countywide Model SUSMP describes specific site conditions that affect the feasibility of LID BMPs (e.g., steep slopes, high groundwater). Furthermore, the HMP Decision Matrix requires applicants to complete a geotechnical investigation which would identify such design constraints. A project applicant would first need to determine whether these constraints apply. If not, the project proponent could use the Sizing Calculator to plan BMPs. The Sizing Calculator will not apply in high groundwater conditions.
15	Luis Parra, Hunsaker	9	It is not clear to me if growing medium as a maximum limit of 1.5 ft. The user should be able to increase this depth. As a matter of fact, the Maryland Manual (the one that initiated the bio-retention revolution) suggests using at least 2 ft of amended soil. The user should have the option to increase this depth up to 3-4 ft.	A project proponent could specify a deeper growing medium. However, to pre-size BMPs for the Sizing Calculator we need to limit the number of potential BMP configurations. Other design scenarios can be modeled through the preparation of continuous simulation hydrologic models such as HSPF.
16	Luis Parra, Hunsaker	9	Van Genuchen relations are mentioned in page 9 but never shown.	The Van Genuchten relations are included in Appendix A. This document will be distributed to the reviewers and the TAC.
17	Luis Parra, Hunsaker	9-10	Neglecting lateral percolation and limiting infiltration to the saturated hydraulic conductivity of the existing soil is a reflection of simplicity and building upon conservative assumptions rather than a reality. Also neglecting the influence of the water pressure of the gravel in the infiltration occurring at the bottom soil is another conservative assumption not discussed. I would suggest to allow for adding an increase infiltration dimensionless factor if measurements demonstrate that the discharge is actually much less than what the model predicts.	The conservative assumptions detailed in the HMP Modeling Approach memo serve as a hedge against real-world installation problems, occasional BMP failures, etc., so that the integrated effectiveness of distributed BMP performance is consistent with the requirements of the NPDES permit. The accuracy of the sizing factors will be measured by the Copermittees' monitoring program, which will be conducted over the ensuing 5 years and beyond. If the monitoring results indicate deviations from the sizing factor predictions, then adjustments to the sizing factors will be proposed.

No.	Reviewer	Page	Comment	Response
18	Luis Parra, Hunsaker	12	The user should have flexibility to determine design parameters of the bio-retention: ponding depth (it can be more than 10 inches, no reason why this has to be the limit); growing medium (it can be more than 18" and as a matter of fact many references recommend at least 2 to 3 ft); storage layer: it can be more than 30" and it is associated with the possibility of the Bio-retention to be able to drain in 72 hrs. Also remember the possibility to add gravel below the invert of the French drain for retention purposes and groundwater recharge purposes.	Regarding ponding depth comment: The selection of a maximum ponding depth is a policy decision. Because bioretention is often installed in pedestrian-friendly areas, these systems often have limited ponding depth to eliminate trip hazards. Regarding growing medium and gravel comment: see response to comment #15.
19	Luis Parra, Hunsaker	12	Drawdown considerations for the bio-retention should be included to determine the maximum depth of the combination of ponding, amended soil, and gravel than can drain in 72 hrs.	Vector control is a major benefit of stormwater LID. Conventionally, drawdown considerations only apply to the surface ponding layer of bioretention devices and not the below ground layers. The surface ponding layer will fully drain within a few hours of the end of a storm event. The sizing calculator will include drawdown calculations.
20	Luis Parra, Hunsaker	12	It is not clear in page 12 if the other option of conversion is valid: the example describes a situation where the design engineer can convert the ponding layer with half the depth but twice the area, and actually design engineers are more interested in doing exactly the opposite: half the area and twice the depth. I am assuming that this is also a valid option.	The Countywide Model SUSMP specifies the allowable configurations for bioretention BMPs.
21	Luis Parra, Hunsaker	13	Comments 2 and 3 (#12 and #13 in this table) are also applicable to figure 6 of page 13.	The figures on pages 77 and 78 of the Countywide Model SUSMP should be modified to show the underdrain pipe at or near the bottom of the storage layer (i.e., the gravel layer).
22	Luis Parra, Hunsaker	N/A	There is no opportunity to make comments in hidden parameters or assumptions made by the program, but to trust blindly on the results. Unfortunately the engineer will become more of a technician running a black-box program than an engineer using criteria an experience to come up with a good design.	The Sizing Calculator is a simple-to-use tool that allows engineers to quickly size stormwater BMPs based on detailed "pre-sizing" modeling exercise (performed by Brown and Caldwell). Project proponents could perform their own hydrologic and hydraulic modeling analyses to size modified BMP designs, if desired.

BROWN AND CALDWELL
9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
Tel: 858-514-8822
Fax: 858-514-8833

Draft Technical Memorandum

Project Title: San Diego County Hydromodification Management Plan

Project No: 133904

San Diego County Hydromodification Management Plan (HMP)

Subject: Description of Automated Pond Sizing Procedure

Date: May 6, 2010

To: Sara Agahi, San Diego County

From: Tony Dubin, Brown and Caldwell
Eric Mosolgo, Brown and Caldwell

This memorandum describes the automated pond sizing procedure that Brown and Caldwell (BC) is currently developing to support the implementation of the San Diego HMP. The purpose of the automated pond sizing procedure is to provide both project proponents and municipal review staff with a technically sound yet streamlined method for sizing stormwater ponds that meet the performance requirements of the HMP.

Pond Sizing Procedure

The automated pond sizing procedure will be built into the BMP Sizing Calculator software to allow project proponents to select detention ponds as the method of stormwater runoff control. The general process will work as follows:

1. The project proponent will enter information about the area tributary to the proposed detention pond for the pre-project and post-project conditions. The information will include drainage area, soil types, slopes, and land cover information (e.g., scrub land, landscaping, impervious).
2. The BMP Sizing Calculator software will construct pre-project and post-project (unmitigated) long-term runoff time series data that correspond to the site conditions (i.e., the information described in the item #1 above). The time series will be created through a pre-modeling exercise that involves running HSPF with real, historical rainfall data and developing long-term, unit area hydrographs for each combination of soils, slopes, and land covers.
3. The project proponent will next enter an initial configuration for the detention pond, including area, depth and side slopes. Alternatively, the user could supply a stage-storage-discharge table. The user will also enter preferences for how the automated pond sizing procedure should iteratively adjust the pond configuration if the initial configuration does not meet the HMP's performance requirement for flow duration and peak flow control. The user will not need to supply any information about the outlet control structure, because the automated pond sizing algorithm will use a pre-defined configuration that includes 2 flow control orifices and an overflow weir (sizes of the outflow facilities will be determined by the pond sizing algorithm).

4. The software will route the post-project unmitigated, long-term runoff time series through the detention pond. The reservoir routing routine will compute the following quantities for each hourly time step:

    - stormwater inflow
    - water depth
    - pond exfiltration
    - pond outflow through the outlet control structure

    The pond outflows will form the "post-project mitigated" time series that will be compared to the pre-project conditions.

5. The software will compare the pond outflow **flow durations** and **peak flows** with the pre-project flow durations and peak flows to determine if the pond configuration meets the performance requirements of the HMP.

6. If the current configuration does not meet the HMP performance requirements, the automated pond sizing procedure will apply the user's stated preference for modifying the pond configuration (see item #3 above) and perform the reservoir routing and statistical post-processing calculations again (and again) until the pond is properly sized and meets HMP requirements.

Time Series Data

As described above, the automated pond sizing procedure will prepare pre-project and post-project time series for the area tributary to the proposed pond, and then determine how large the pond must be to mitigate the impacts of development or redevelopment activities. The site-specific time series will be developed by adding together the component time series data that describe the different parts of the project area (e.g., 10-acres impervious *time series* + 5 acres Group D soils and scrub vegetation with moderate slopes *time series* + 12 acres Group D soils with urban/landscaped cover *time series*).

The component runoff time series will be developed by running HSPF simulations for each of the 12 scenario conditions described in the *HMP Modeling Approach and BMP Configurations* technical memo dated March 2, 2010 (see Section 2.2 of the technical memo). Runoff time series will also be developed corresponding to four (4) "urban" landscaped conditions describing landscaped areas with compacted soils that would be typical of urban and suburban-style development.

These landscaped cover types are necessary for the automated pond sizing procedure, because it is not feasible for site developments that include large ponds for stormwater control to segregate runoff from pervious and impervious surfaces (in the way that LID-focused developments typically separate contributions from pervious and impervious sources).

Table 1 below lists the pervious site conditions that will be available for the automated pond sizing procedure.

TABLE 1
HSPF Scenarios for Characterizing Pervious Site Conditions

Scenario No.	NRCS Soil Group	Land Cover	Slope
1	A	Scrub, Shrub	Low (<5%)
2	A	Scrub, Shrub	Moderate (10%)
3	A	Scrub, Shrub	Steep (>15%)
4	B	Scrub, Shrub	Low (<5%)
5	B	Scrub, Shrub	Moderate (10%)
6	B	Scrub, Shrub	Steep (>15%)
7	C	Scrub, Shrub	Low (<5%)
8	C	Scrub, Shrub	Moderate (10%)
9	C	Scrub, Shrub	Steep (>15%)
10	D	Scrub, Shrub	Low (<5%)
11	D	Scrub, Shrub	Moderate (10%)
12	D	Scrub, Shrub	Steep (>15%)
13	A	Urban/Landscaped	Moderate (10%)
14	B	Urban/Landscaped	Moderate (10%)
15	C	Urban/Landscaped	Moderate (10%)
16	D	Urban/Landscaped	Moderate (10%)

Pond Configuration Preferences

After describing the pre-project and post-project conditions based on local soils, slopes and land covers, the project proponent will describe an initial pond configuration and preferences for modifying the configuration during the automated sizing process.

Since each project site has its unique constraints on pond configurations, the user should be allowed to express preferences with regard to configuration modification. To minimize the pond footprint area, engineers commonly provide the required storage volume by constructing a deeper pond. However, site specific constraints, community concerns, and municipal regulations could require an engineer to set a maximum depth for a pond. Potential concerns associated with pond depths include public safety, drawdown times, vector control, or aesthetics, among others.

Figure 1 below shows how preferences will be incorporated into the iterative pond sizing process. Specifically, the figure illustrates how the automated pond sizing procedure could test an initial configuration, iteratively test increasing pond depths, and finally test increasing pond areas until a solution is found that meets the flow duration and peak flow performance requirements of the HMP.

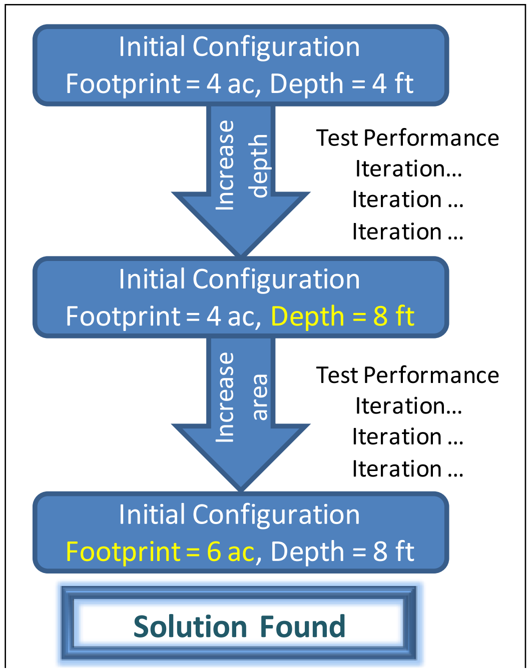


FIGURE 1

Illustration of Pond Configuration Preferences for Automated Iterative Sizing

BROWN AND CALDWELL

Draft Technical Memorandum

9665 Chesapeake Drive, Suite 201
San Diego, CA 92123
Tel: 858-514-8822
Fax: 858-514-8833

Project Title: San Diego County Hydromodification Management Plan

Project No: 133904

San Diego County Hydromodification Management Plan (HMP)

Subject: Rainfall and Evapotranspiration Data for BMP Sizing Calculator

Date: May 20, 2010

To: Sara Agahi, San Diego County

From: Eric Mosolgo, Brown and Caldwell
Tony Dubin, Brown and Caldwell

This memorandum describes the rainfall and evapotranspiration data that Brown and Caldwell (BC) is using to develop BMP sizing factors and pond sizing time series data for incorporation in the San Diego BMP Sizing Calculator. The purpose of the BMP Sizing Calculator is to provide both project proponents and municipal review staff with a technically sound yet streamlined method for sizing stormwater facilities that meet the performance requirements of the HMP.

Rainfall Data

Standards developed as part of this HMP to control runoff peak flows and durations are based on a continuous simulation of runoff using local rainfall data. To provide for clear climatic designation between coastal, foothill and mountain areas of the County, and to distinguish between the major watershed units, historical records for a series of 18 rainfall data stations located throughout San Diego County were compiled and quality controlled for analysis.

Long-term hourly rainfall records have been prepared for the 18 rainfall stations. These rainfall record files are located on the *Project Clean Water* web site for public use (www.projectcleanwater.org). Sources of the rainfall data include ALERT data from the County of San Diego (which extend back to 1982), the California Climatic Data Archive, National Oceanic and Atmospheric Administration (NOAA), the National Climatic Data Center, and the Western Regional Climate Center.

Gauges were selected based on minimum continuous simulation modeling requirements including the following:

1. Since the selected precipitation gauge data set should be located near the project site to ensure that long-term rainfall records are similar to the anticipated rainfall patterns for the site, gauges were selected in proximty to areas planned for future development and redevelopment.
2. Recording frequency for the gauge data set should be hourly (or more frequent).
3. The gauge rainfall record should extend for the entire length of the record. Where the gauge record length is less than 35 years, then adjacent gauge records were used to extend the rainfall record to at least 35 years.

4. Use of the most applicable long-term rainfall gauge data, as opposed to the scaling of rainfall patterns solely from Lindbergh Field, is required to account for the diverse rainfall patterns across San Diego County.

Precipitation gauges summarized in Table 1 below all have recording frequencies of one hour and recording data ranges of at least 35 years.

TABLE 1
Rainfall Station Summary

Station	Elevation	Watershed
Bonita	120	Sweetwater River
Encinitas	242	San Elijo Lagoon and Batiquitos Lagoon and ocean outlets
Escondido	645	Escondido Creek
Fallbrook	675	San Luis Rey River (near ridge with Santa Margarita River watershed)
Fashion Valley	20	Lower San Diego River
Flinn Springs	880	San Diego River
Kearny Mesa	425	San Diego River (near ridge with San Clemente Canyon watershed)
Lake Cuyamaca	4,590	Upper San Diego River
Lake Heneshaw	2,990	Upper San Luis Rey River
Lake Wohlford	1,490	Upper Escondido Creek
Lindbergh Field	Near Sea Level	Coastal – San Diego Bay
Lower Otay Reservoir	491	Otay River
Oceanside	30	San Luis Rey River
Poway	440	Los Penasquitos Canyon
Ramona	1,450	Upper San Dieguito River
San Onofre	162	North County Coastal – Pacific Ocean
San Vicente Reservoir	663	San Diego River
Santee	300	San Diego River

For a given project location, the following factors should be considered in the selection of the appropriate rainfall data set when developing continuous simulation hydrologic models.

1. In most cases, the rainfall data set in closest proximity to the project site will be the appropriate choice. A rainfall station map is included in Figure 1 of this technical memo and has been posted to the *Project Clean Water* web site for public use.
2. In some cases, the rainfall data set in closest proximity to the project site may not be the most applicable data set. Such a scenario could involve a data set with an elevation significantly different from the project site. In addition to a simple elevation comparison, the project proponent may also consult with the San Diego County's average annual precipitation isopluvial map, which is provided in the San Diego County Hydrology Manual (2003). Review of this map could provide an initial estimate as to whether the project site is in a similar rainfall zone as compared to the rainfall station. Generally, average annual precipitation totals in San Diego County increase with increasing elevation.
3. Where possible, rainfall data sets should be chosen so that the data set and the project location are both located in the same topographic zone (coastal, foothill, mountain) and/or major watershed unit (Upper San Luis Rey, Lower San Luis Rey, Upper San Diego River, Lower San Diego River, etc.).

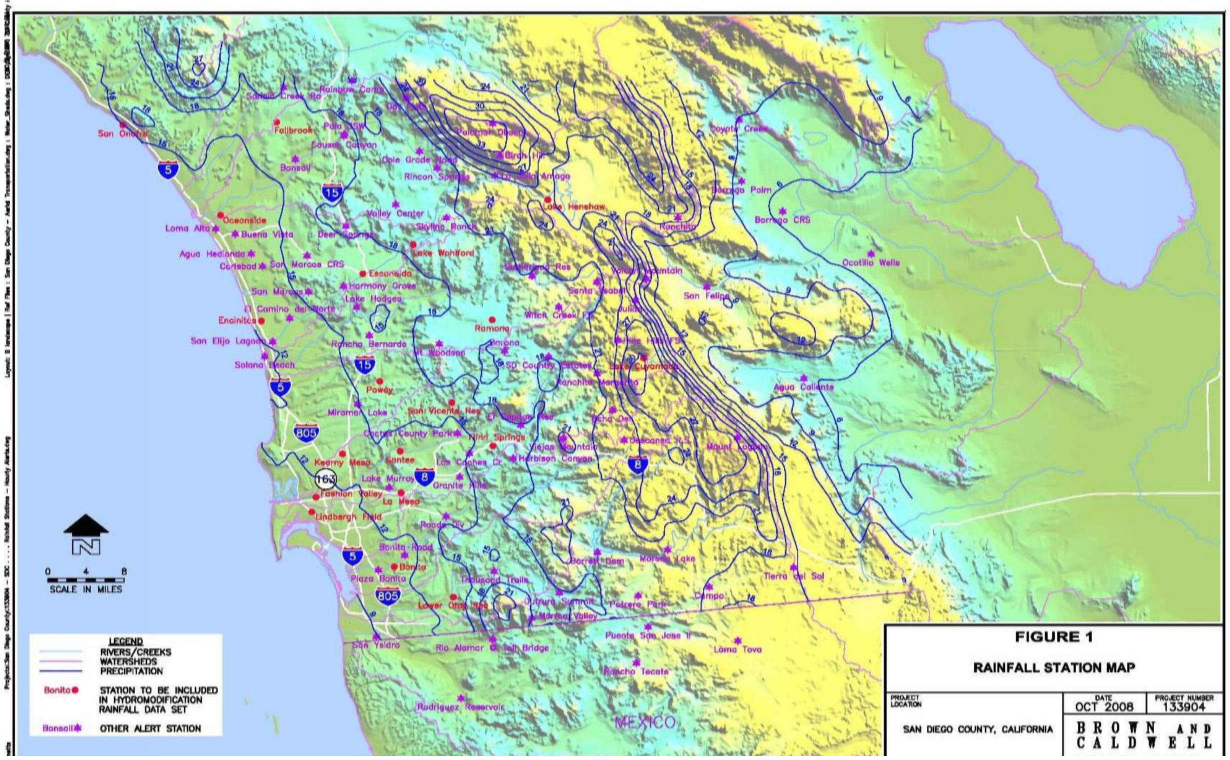


FIGURE 1

Rainfall Station Map

The BMP Sizing Calculator will automate the rainfall gauge selection process considering the factors detailed in this technical memo. For the purposes of the sizing factor modeling development effort, four (4) of the rainfall data sets were considered for analysis. The selected rainfall data sets include:

1. Lindbergh Field (coastal area, San Diego Bay watershed, central San Diego County, elevation near sea level)
2. Oceanside (coastal area, San Luis Rey River watershed, northern San Diego County, elevation = 30 feet)
3. Lower Otay Reservoir (inland valley area, Otay River watershed, southern San Diego County, elevation = 491 feet)
4. Ramona (mountain area, San Dieguito River watershed, eastern San Diego County, elevation = 1,450 feet)

To account for topographic, geographic and climatic variability across San Diego County, scaling factors will be developed for each of the (4) rainfall stations listed above. Projects will be assigned to a "rainfall basin" corresponding to one of the (4) rainfall stations. Then, rainfall data will be scaled based upon either mean annual precipitation of single-event isopluvial data (such as the 2-year, 24-hour or 85th percentile, 24-hour rainfall totals) differences as compared to the selected rainfall station.

Evapotranspiration Data

Known data sources for evaporation and evapotranspiration data in San Diego County are listed below.

1. California Irrigation Management and Information System web site – evapotranspiration stations include San Diego, Oceanside, Escondido, Ramona, Otay Lakes, Miramar, Torrey Pines, and Borrego Springs.
2. Historical Reservoir Level and Evaporation Data for Lake Heneshaw.
3. Historical Evaporation Data from City of San Diego Reservoirs.
4. Historical Evaporation Data from Helix Water District for Lake Cuyamaca.

Table 2 below summarizes available evaporation and evapotranspiration data sources in San Diego County. Most of the available data are located close to reservoirs in the inland valley and mountain areas of the County. Monthly evaporation records are available for multiple reservoirs within the County. Evapotranspiration sensing data are generally collected in agricultural zones.

The California Irrigation Management Information Systems web site (www.cimis.water.ca.gov/cimis/data.jsp) provides access to real-time and summarized evapotranspiration data (ETo) throughout California. For the San Diego region, average evapotranspiration values are summarized for the coastal and foothill zones of San Diego County.

TABLE 2
Summary of Evaporation and Evapotranspiration Data for San Diego County

Station Name ID	Data Type	Data Source	Recording Frequency	Start Date	End Date
Barratt Lake	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Chula Vista	Pan Evaporation	Western Regional Climate Center	Monthly Averages	1948	2005
El Capitain Reservoir	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Escondido / 74	Evapotranspiration	CIMIS	Monthly	1988	1998
Escondido / 153	Evapotranspiration	CIMIS	Monthly	1999	2008
Lake Cuyamaca	Pan Evaporation	Helix Water District	Monthly	1985	2006
Lake Heneshaw	Pan Evaporation	County of San Diego	Daily	1999	2005
Lake Heneshaw	Pan Evaporation	County of San Diego	Monthly	1957	2008
Lake Hodges	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Lake Jennings	Pan Evaporation	Helix Water District	Monthly	1985	2006
Lake Murray	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Lake Sutherland	Pan Evaporation	City of San Diego Water Department	Monthly	1954	2008
Lower Otay Reservoir	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Lower Otay / 147	Evapotranspiration	CIMIS	Monthly	1999	2008
Miramar Lake	Pan Evaporation	City of San Diego Water Department	Monthly	1960	2008
Miramar Lake / 150	Evapotranspiration	CIMIS	Monthly	1999	2008

TABLE 2
Summary of Evaporation and Evapotranspiration Data for San Diego County

Station Name ID	Data Type	Data Source	Recording Frequency	Start Date	End Date
Morena Lake	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Oceanside / 49	Evapotranspiration	CIMIS	Monthly	1986	2003
Ramona / 98	Evapotranspiration	CIMIS	Monthly	1991	1998
San Diego / 45	Evapotranspiration	CIMIS	Monthly	1985	1989
San Diego / 66	Evapotranspiration	CIMIS	Monthly	1989	2001
San Diego II / 184	Evapotranspiration	CIMIS	Monthly	2002	2008
San Vicente Reservoir	Pan Evaporation	City of San Diego Water Department	Monthly	1950	2008
Torrey Pines / 173	Evapotranspiration	CIMIS	Monthly	2000	2008

For the purposes of the sizing factor modeling development effort, the four (4) rainfall data sets were associated with evapotranspiration/evaporation data as detailed below.

1. Lindbergh Field (San Diego/45, San Diego/66, San Diego II/184, Chula Vista, Lake Murray and Torrey Pines/173)
2. Oceanside (Oceanside/49, Lake Hodges, Lake Heneshaw)
3. Lower Otay Reservoir (Lower Otay Reservoir, Lower Otay/147)
4. Ramona (Ramona / 98, Lake Hodges, San Vicente Reservoir)