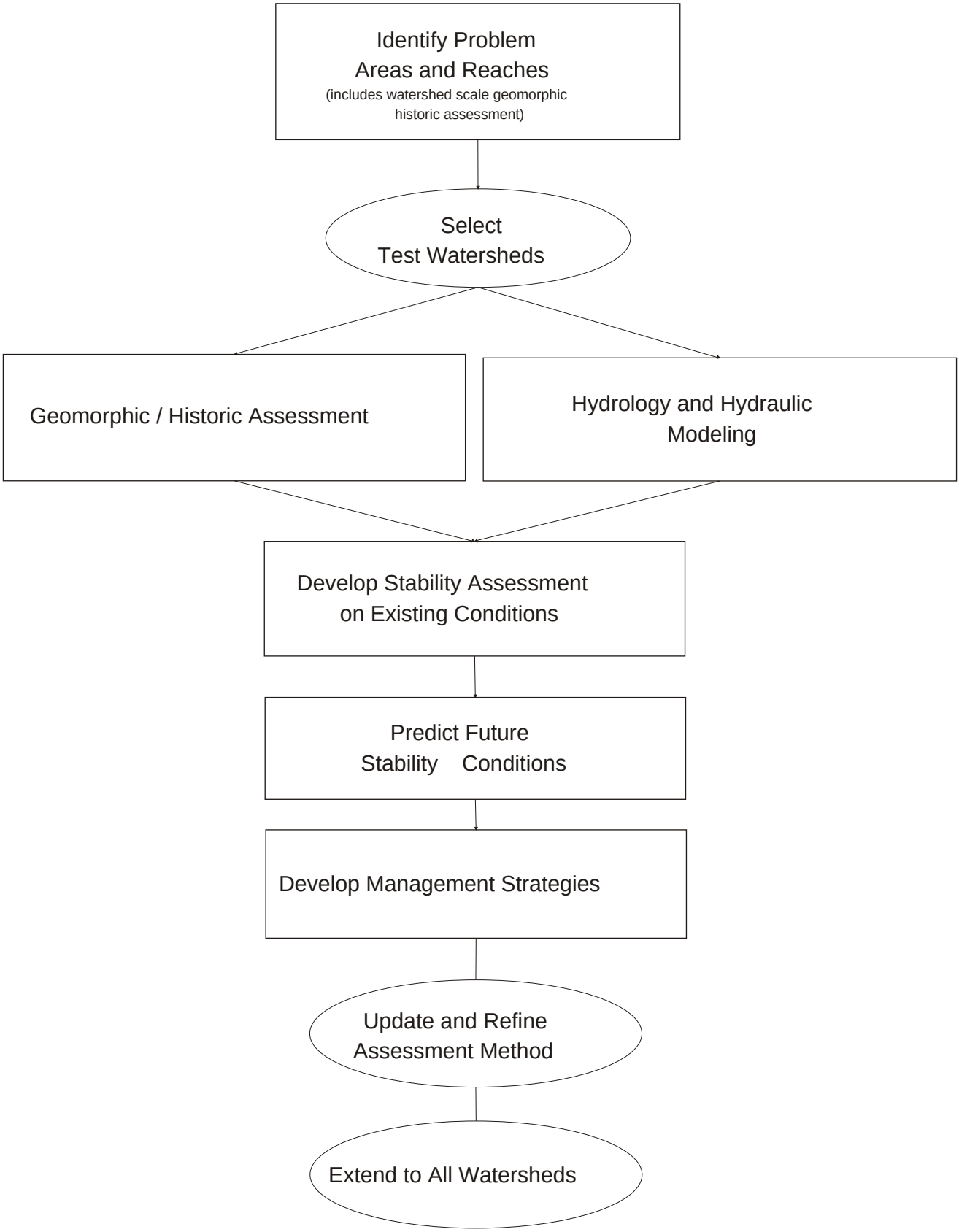
Appendix C

Technical Memoranda

FIGURE 1. OVERVIEW OF RECOMMENDED ASSESSMENT METHOD



T E C H N I C A L
M E M O R A N D U M # 1

TO: Santa Clara Valley Urban Runoff Pollution Prevention Program

FROM: Geosyntec Consultants HMP Project Team

DATE: October 24, 2002 (Draft)

SUBJECT: **RECOMMENDED ASSESSMENT METHOD FOR DEVELOPING THE HYDROMODIFICATION MANAGEMENT PLAN, INCLUDING DATA REQUIREMENTS**

Background

The Geosyntec Project Team[1] (Team) is assisting the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP) in developing a Hydromodification Management Plan (HMP) as required in Provision C.3.f. of the updated stormwater permit. One of the tasks to be completed, as described in the Work Plan submitted to the Regional Water Quality Control Board (RWQCB) on March 1, 2002, is to develop and recommend an *Assessment Method* that can be used to evaluate hydromodification from urban development. Technical Memorandum #1 summarizes the Team's recommended assessment method. This memorandum is intended to be a summary of the overall approach including tables of expected data requirements.

A Literature Review and Conceptual Model for the HMP were previously developed and submitted to the RWQCB on September 15, 2002. The goals and objectives in the Literature Review and the Conceptual Model were used to guide the development of the assessment method. The overall assessment method is subdivided into four major phases: 1) Identify Problem Areas and Reaches/Prioritization, 2) Geomorphic/Historic Assessment, 3) Hydrology/Hydraulic Modeling, and 4) Stability Assessment. Figure 1 illustrates the assessment method graphically and shows the sequence and links between phases. The following sections summarize criteria for selection of the assessment method and describe the purpose, use and data requirements for each phase of the assessment method.

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[1] Project Team consists of Jill Bicknell, SCVURPPP; Peter Mangarella and Gary Palhegyi, GeoSyntec Consultants; Jeff Haultiner and Christie Beeman, Phillip Williams & Associates; Barry Hecht, Balance Hydrologics; and Laurel Collins, Watershed Sciences.

Criteria for Method Selection

The goal of the Team was to develop a method that can evaluate changes in hydrology and associated stream channel conditions and predict the potential for erosion and deposition or other impacts attributed to hydromodification from future urban development. The method was developed based on methods described in the Literature Review and on the expertise of the Team, including review comments by the Expert Panel[2] on the Literature Review and comments from Santa Clara Valley Water District. The assessment method should have the following characteristics:

1. Be able to predict existing erosion and distinguish between stable and unstable conditions under existing land use conditions with an acceptable level of accuracy.
2. Be able to predict future erosion (instability) under future land use conditions and under future land use conditions with BMP's.
3. Be applicable to natural stream segments, modified segments (i.e., existing conditions are not pristine), and restored stream segments (in-stream BMP's).
4. Be able to distinguish between direct and indirect anthropogenic effects.
5. Be testable and verifiable.
6. Be defendable if challenged legally.
7. Be based on recognized standards of practice.
8. Be cost effective and able to be completed by the regulatory due dates.
9. Be adaptable to new scenarios and updateable as new knowledge is gained during implementation.

Conceptual Model

The Conceptual Model of the hydrologic and geomorphic processes to be considered in the assessment, showing specific attributes considered important in addressing hydromodification, is presented in Figure 2. The process drivers are the regional factors of climate, geology, and physiography, which in turn affect the amount of runoff and sediment sources discharged to stream channels. Land use, soil and vegetation characteristics affect the proportion of rainfall that infiltrates the ground or runs off the surface as overland flow. The nature of local climate, geology, and physiography affect the frequency and type of sediment supplied to the stream system. The imposed changes in stream flow and sediment supply characteristics caused by urbanization and hydromodification ultimately change the physical and ecological characteristics of stream channels.

[2] Expert Panel review consisted of comments from Professor Tom Dunne, SBSU; and Professor Brian Bledsoe, CSU.

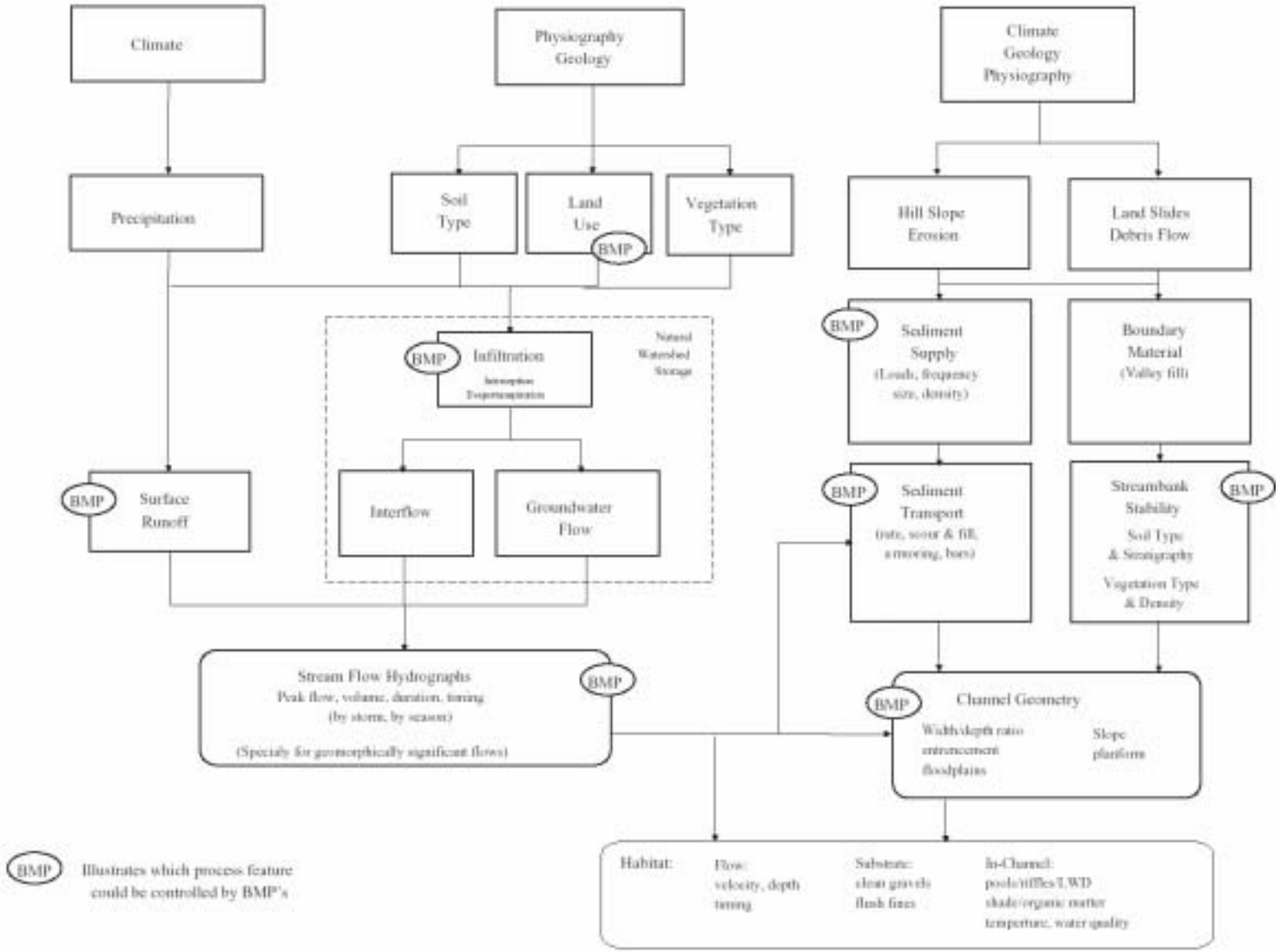

Figure 2. CONCEPTUAL MODEL ILLUSTRATING THE LINKAGES BETWEEN THE HYDROLOGIC AND GEOMORPHIC PROCESSES TO BE ADDRESSED IN HYDROMODIFICATION

The assessment method must incorporate features that address elements described in the Conceptual Model. The hydrologic process can be handled by traditional rainfall-runoff modeling as long as it includes the ability to account for the amount, frequency, duration, and timing of geomorphically significant flows (Literature Review). The hydrologic model also must be able to account for variations in land use characteristics as well as urban runoff best management practices (BMP's).

Geomorphic processes will not be modeled in the same manner as the hydrologic processes. The geomorphic processes and their attributes are described through mapping and field measurement of select physical characteristics as part of a geomorphic / historic assessment. An important aspect of the geomorphic / historic assessment is developing an understanding of the existing watershed and stream conditions in sufficient detail to correctly interpret which observed channel instabilities are due to urban hydromodification.

Identify Problem Areas/Reaches & Prioritize (Completed Basin Wide)

The objective of this phase is to identify stream segments basin wide that are currently subject to erosion and/or deposition, and those segments that could potentially be affected by future development. Recommended data for problem identification are listed in Table 1.

The primary activity is to review background data, interview stream management personnel, and visit suspected problem sites to identify stream segments that are currently unstable from erosion or deposition. The location of the eroding stream segments will be mapped. Future development plans from agency master plans will be mapped and overlain on maps showing stream channels to identify streams at risk of being impacted by hydromodification. Some stream channels have been hardened by concrete, riprap, gabions or significantly modified such that they may be considered exempt from the HMP requirements. Channel characteristics will be identified and mapped to show which stream segments can be considered exempt and which segments are subject to regulation.

Following a basin wide review of the suspected problem areas and reaches, a screening level analysis of the eroding stream segments will be conducted to determine which segments are the most critical, or that could be considered the highest priority. The screening will consist of site visits to verify the condition of suspected problem areas and reaches, and to identify patterns or other features that indicate larger scale system wide problems.

Geomorphic/Historic Assessment. Although there is a separate phase describing the geomorphic/historic assessment, there are a few features of this assessment that will be valuable in assessing problem areas and reaches and selecting test watersheds. Generally, the larger watershed scale elements of a geomorphic/historic assessment will be completed under this phase. The geomorphic/historic assessment is described in more detail in following sections and recommended data are listed in Tables 2 and 3.

Within the Santa Clara Valley Basin, the climatic, geologic and physiographic nature of Basin would be mapped and used to identify and group watersheds with similar features. However, based on the Team's experience, the Basin has been conceptually divided into three major watersheds that represent the range of climatic, geologic and geomorphic conditions in the Basin: 1) Diablo Mt. Range, Coyote Creek Watershed (east side of the basin), 2) Guadalupe River Watershed (central basin), and 3) Santa Cruz Mt. Range (west side of the basin). One of the important questions to address early in the assessment is "how have past land use practices and water related civil works (e.g., dams and other structures) affected the observed stream channel conditions?" The HMP C.3.f. regulations require the assessment to address changes in watershed hydrology from urbanization and to develop and implement management strategies for new development and significant redevelopment. The assessment must include elements to distinguish between urbanizing impacts and impacts caused by past land use practices and changes drainage patterns. Review of aerial photography of pre-urban versus urban periods provides a cost effective way to evaluate changes in stream channels.

Products:

a) Map(s) illustrating the stream network, exempt and non-exempt stream segments, jurisdictional boundaries, and watershed and sub-watershed boundaries.

b) Map(s) illustrating existing conditions of the stream channel network, exempt and non-exempt stream segments, location of the problem areas and reaches, highlighting the higher priority segments, and the areas projected for future development.

c) Map(s) illustrating the location and timing of past land use activities and changes in water related infrastructure or re-routing of stream channels.

d) Map(s) illustrating the climatic, geologic and physiographic nature of Basin with suggested divisions between distinct zones.

Test Watersheds

Using the Basin wide information compiled above, the Team will select sub-watersheds to develop, test and verify the assessment method. Considering the Santa Clara Basin, one sub-watershed should be selected from each of the three major watersheds to develop and test the assessment method that later could potentially be extrapolated to other sub-watersheds within each major watershed.

The test watersheds will also be used to test the importance of the recommended assessment method tools and parameters. It may be possible to define a minimum level of analysis that still provides the same results as using a more complex approach.

The first test sub-watershed will be used to develop, test and verify the assessment method itself and ask broader questions of adequacy. For example, "can we correctly predict existing erosion and deposition?" "How successful can we define erosion thresholds?" The second test sub-watershed will be used to verify that the method works in a watershed with

different characteristics. At this point, it may be possible to define a reduced level of analysis that provides the same results as that proposed in this memorandum. For example, can we reach the same conclusions using hydrologic modeling with stream power as we do using hydrologic/hydraulic modeling and excess shear stress? Ultimately, the final approach should satisfy the criteria described on page 2 of this technical memorandum.

Product:

a) A tested and verified assessment method that meets the identified criteria, and that can be used on the remaining watersheds and sub-watersheds in the Basin.

Geomorphic/Historic Assessment (For Test Watersheds)

The goal of the geomorphic/historic assessment is to characterize features of the watersheds and stream channels necessary to understand the nature and extent of the problem areas and stream reaches, explain the existing conditions, and to correctly interpret results and formulate solutions. The objectives of the geomorphic/historic assessment are to:

- Describe the current watershed/stream conditions and dominant physical processes that control stream attributes.
- Describe the extent and modes of failure for observed eroding banks.
- Define the sensitivity of stream segments to hydromodification.
- Define stable channel attributes for in-stream solutions (reference reach).
- Extrapolate the results for the test watershed to other watersheds with similar characteristics.

The geomorphic/historic assessment involves collecting data at several different spatial scales ranging from broad characterization of the watersheds to site-specific descriptions of eroding segments, and over different time scales ranging from pre-urban periods to present time. Recommended data are listed in Tables 2 and 3.

Two important geomorphic data sets are recommended that describe the location of the study sites relative to the stream's longitudinal profile: 1) stream type and/or stream order and 2) sediment transport zones; i.e., source, transport, and deposition zones. Stream channel sensitivity to hydromodification, its response, and ultimately its stability are influenced by its location within the watershed. Impacts from urbanization may not occur immediately downstream from the discharge location, but may become apparent in more sensitive reaches some distance downstream. The assessment will consider potential impacts both upstream (from headcuts) and downstream of stormwater discharge locations (outfalls).

An evaluation of historical conditions can be conducted using a wide range in levels-of-effort and detail. At a minimum, the assessment should include a comparison of larger scale features between pre-urban and urban periods, as well as a comparison between stream attributes over the past 15 to 20 years, to understand the stream system and sufficiently predict erosion and deposition problems resulting from urbanization. The analysis need not be elaborate, but should consider the effects from changing land use, constructed water

works, El Nino, the Loma Prieta earthquake, major fires, and subsidence on channel stability, incision, deposition and sediment supply. The episodic nature of sediment loads from landslides, earth flows, etc. and transport of this material through the stream network will not be captured quantitatively, but will be incorporated through characterization where data exist. Rainfall episodes will be captured using long-term continuous simulation (see description of hydrologic/hydraulic modeling).

Stream planform, longitudinal profile, cross sectional geometry, bed and bank characteristics describe the existing morphology of the stream channel system. Data on channel sinuosity, cross-sections, channel slopes, headcuts, nick points, bed material size (D50, D84) and field measured bankfull discharge are needed. The characterization of stream segments geomorphic parameterization provides the basis to assess stream bank resilience. Locations of significant bed and bank erosion, bank height and side slope, estimate of bank material, vegetation type and density are necessary to evaluate bank stability. The importance of individual parameters in Santa Clara Basin streams will be tested while evaluating the test watersheds.

Products.

a) Map(s) illustrating important landforms and stream types, such as narrow canyons, alluvial fans, incised channels, deltas; and sediment transport zones, stream type and/or stream order.

b) Plots of the longitudinal profile over time. The profile should identify channel incision and deposition, headcuts, nick points, grade control structures and slopes.

c) Map(s) of recent landslides, debris flows and earth flows (from existing information) with reference to the cause of the event, such as El Nino or the Loma Prieta earthquake and how such conditions might have affected stream conditions.

Hydrologic/Hydraulic Modeling

The Team recommends using a long-term time series of precipitation data and continuous simulation modeling to analyze the effects of hydromodification. This approach is required to correctly analyze the frequency, duration and timing of geomorphically significant flows. The data required for the hydrologic and hydraulic models are listed in Table 4.

The method incorporates the full probability distribution of historical rainfall events and antecedent conditions, rather than using artificial design storms. The product is a time series of daily urban runoff and stream flow, channel depth and velocity to be analyzed for changes in peak flow, volume, frequency, duration and timing between development scenarios. Stream channel velocity and depth will be predicted using uniform flow calculations for individual cross sections of interest (HEC-RAS will be used where sufficient data exists). Roughness coefficients will be estimated based on channel characteristics during bankfull, or dominant discharges. These can be different from estimates made for flood flow conditions.

The analysis needs to evaluate thresholds between stable and unstable segments as well as predict potential future problems in currently stable reaches. The assessment will be completed at locations along the stream network and longitudinal profile for both eroding segments and healthy stable segments.

Products.
The following will be included in technical memoranda:

1) For existing and future land use development scenarios, a time series of watershed runoff, stream flows, depth, velocity, and shear stress will be provided at selected locations along the stream network.
2) Results are to be generated at locations of known bed and bank erosion as well as locations of healthy-stable stream reaches.
3) Results are to be calibrated and verified using measured stream flow data and existing erosion problem areas and reaches.

Sediment Transport Modeling

Sediment transport modeling is not recommended at this time. Because of the costs and complexity of sediment transport modeling, flow energy and erosion indices have been developed to simplify the procedure and predict stream bank erosion and instability. The effects of sediment supply and transport will be incorporated through the geomorphic / historic assessment by identifying and describing sources of sediment, zones of transport and deposition.

Channel Stability Assessment

The channel stability assessment will use the time series of stream flow data generated from the hydrologic / hydraulic modeling at selected cross sections along the stream network. The data required for the stability assessment are listed in Table 5. Stream flow records need to be generated for both eroding and non-eroding stream segments to compare results between stable and unstable conditions, and identify stability thresholds. Potential future instabilities need to be assessed along the stream network including healthy stable reaches.

The literature reported a range of potential stability indices that should be considered and carried through the analysis to see if one or more could accurately predict stream bank instability (Table 5). The literature review suggested that each index has varying degrees of success at predicting potential erosion. Using the existing condition scenario, the measures of stream flow energy and erosion potential will be compared to the observed conditions in the field to evaluate the accuracy of the method. Thresholds will then be defined and used as criteria to limit or control hydromodification.

The Team recommends that the stability assessment incorporate both the applied hydraulic forces (stream flows) and the streams boundary materials ability to resist erosion (resilience), i.e. a physical measure of the streams resistance to erosion and hydromodification. Several authors in the literature have used bed material size (D50 or

D84) as an indicator of the stream's resilience. These authors suggest that flows in excess of the critical flow for bed mobility is required to cause channel erosion for both the bed and banks. In general, the analysis involves evaluating the frequency and duration of flows greater than the critical flow (velocity or shear stress). Most authors recognize the importance of bank material and its characteristics (and specifically the least resistant layer) in accurately predicting instability.

The approach to evaluating stability using one, or more of the energy/erosion potential indices would likely consist of the following:

a) Compute velocity, shear stress, stream power, work done, etc. for each location along the stream network where computations of erosion potential is desired.
b) Evaluate the critical velocity, critical shear stress, and critical stream slope for bed mobility and compare eroding stream segments to healthy-stable stream segments.
c) Evaluate potential thresholds values by comparing computed results and critical values for eroding and non-eroding stream segments. Compare the results between watershed and stream attributes.
d) Compute and compare statistical results between data sets (e.g., absolute values, frequency distributions, percent change, box plots, tables, etc.).
e) Develop probability distributions for "Potential Erosion".
f) Predict future stability problems areas and reaches.

Products.

a) Time series and integrated measures of stream flow energy and erosion potential.
b) Statistical analysis of results showing differences between eroding and non-eroding stream segments, difference between stream attributes or sediment transport zones, and other relationships that may prove useful.
c) Thresholds of instability and/or erosion potential.
d) Probability distributions of results and potential erosion as a function of stream attributes (if possible).
e) Predictions of potential instability of currently healthy-stable stream segments under future land use conditions, and future land use conditions with BMP's.

Table 1. PROBLEM IDENTIFICATION

GOAL: To focus the assessment on watersheds and stream segments with existing erosion problems and those most likely to be affected by near-term future development.

1) Collect and review background data on existing problem areas and stream reaches
   a) Interview District maintenance staff and watershed managers to identify and map known and suspected eroding stream segments.
   b) Collect sediment removal data from stream maintenance crews (location, volume, grain size distribution).
   c) Collect information on past stream maintenance projects: location, date, and type of repair (e.g., Concrete, sack-concrete, gabions, riprap, biotechnical, restoration).
   d) Obtain and review stream channel capital improvement plans.

2) Collect and review available historic land use information and water related civil works (refer to Table 2 Historical Data) and determine how these activities might contribute to the observed channel conditions.
   a) Evaluate date, location and significance of channel disturbing events.
   b) Consider this activity on a watershed wide scale.

3) Collect and review current land use information and future development projections.
   a) City and County Master Plans and General Plans
   b) ABAG projections and other sources.

Table 2. Historical Assessment Data Requirements

GOAL: To estimate the extent to which observed erosion in streambeds and banks is caused by urban development.

1) Assess past land use activities (watershed scale review)
   a) Date, location and description of land use types in the basin.
      i) Time lines of major growth and development (for example, from agricultural and grazing practices to present time urbanization).
   b) Date, location and description of various stream or water related civil works.
      i) Dams, reservoirs, diversions, canals, levees.
      ii) In-stream gravel mining and re-routing of stream channels.
      iii) Bed control structures; weirs.
   c) Review of aerial photos to see how channels have changed from pre-urban periods to today.
      i) Aerials of pre- urban development (railroad vintage maps, circa 1880's; USDA "ten chains" maps, circa 1930's)
      ii) Aerials of present time and past 50 years as available (50's, 70's, 90's, 2000)
   d) Groundwater overdraft maps and characteristic data (date, region, depth of water table)

2) Assess channel changes over time (stream/reach scale review)
   a) Data, date and location of past stream cross-sections where available.
      i) Typically at bridges, pipe crossings, improvement projects, levee construction
   b) Compare cross sections, aerial photos, longitudinal profiles, and data from bridge and pipeline crossings; channel armoring projects and date constructed (riprap, etc.)
   c) Determine changes in channel characteristics (width, depth, slope, roughness, etc.) in the last 15 to 20 years.

Table 3. Geomorphic Assessment Data Requirements

GOAL: To define the geomorphologic processes and attributes in sufficient detail to understand the nature of existing problems, to predict the potential for erosion, and to formulate adequate solutions.

1) Describe watershed/valley scale attributes
   a) Climate, topography, aspect, geology, soils,
   b) Major land features; i.e., narrow V-shaped valleys, alluvial valleys, alluvial fans, etc.
   c) Describe sediment transport regimes
      i) Zones of source material, transport, and deposition,
   d) Stream type, stream order, and drainage area.
   e) Location of land subsidence.

2) Measure stream channel attributes (at locations along stream network for both eroding stream segments and healthy stream segments; e.g., Reference Reaches)
   a) Cross sections; widths, depths, W/D ratio's, entrenchment ratio's,
   b) Longitudinal profile; slope, locate headcuts & nick points
   c) Plan form dimensions and sinuosity.
   d) Bed material (D50, D84).
   e) Bankfull dimensions, estimated roughness coefficients at bankfull.
   f) Locations of grade control structures and alignment controls.

3) Existing Stream Bed and Bank Erosion
   a) Estimate of bank material & stratigraphy, vegetation type and density.
   b) Measure eroding bank height, side slope, and length.
   c) Existing bank material data where available (composition, strength; plasticity, density, etc.).
   d) Predict mode of failure (incision, widening, loss of vegetation, landslide, etc.).
   e) Note erosion upstream and downstream of bridges, culverts, weirs, etc.

Table 4. Hydrologic Modeling Data Requirements

GOAL: To predict the nature of storm runoff and stream flow along a stream network under existing and future land use conditions, and under future conditions with BMP's.

1) Incorporate continuous simulation of long-term rainfall records.
   a) Account for antecedent conditions and track soil moisture.
   b) Analyze frequency and duration of stream flows.
   c) Incorporate the historic probability distribution of rainfall events.
2) Predict peak flow, volume, duration and timing of stream flow.
3) Predict stream flow at defined locations along the stream network
   a) At locations of known erosion and healthy stream segments.
4) Calibrate model to available measured stream flow data (gage data) and verify results.
5) Predict time-series of stream velocity, depth, and bed and bank shear stress

Hydrologic Model Input Data

1) Watershed and Drainage Network
   a) Area, length, width, slope
   b) Land cover (impervious, pervious)
   c) Roughness
2) Precipitation
   a) Design storm(s)
   b) Recording gages, locations
   c) Continuous long-term records
3) Rainfall Loss
   a) Interception/depression storage
   b) Infiltration & percolation
   c) Soils data
4) Evaporation
   a) Monthly
5) Flow Routing
   a) Node elevations of channel, pipe, etc.
   b) Channel/pipe dimensions (length, width, depth, slope, etc.)
   c) Roughness coefficients
6) Stream Flow Records (calibration & verification)
   a) Recording gages, location
   b) Baseflows
7) Storage Routing
   a) Basins, reservoirs, etc.
   b) Stage – discharge relationships

Hydraulic Model Input Data

1) Time series of flows from hydrologic model
2) Cross section data
   a) Surveyed or roughly measured using tape measure
3) Estimate of roughness coefficients
   a) Field determined
4) Boundary conditions
   a) Channel slope
5) Structures data
   a) Bridges, culverts, weirs, etc.

Table 5. Channel Stability Assessment Data Requirements

GOAL: To correctly predict the potential for streambed and bank erosion and deposition (instability) under future land development conditions, including future conditions with BMP's.

INPUT DATA REQUIREMENTS

1. Stream Flow Record (uniform flow, continuous time series)
    a. Flow rate, depth, and velocity
    b. Applied shear stress & stream power
2. Channel Geometry / Hydraulic Roughness
    a. Cross sectional dimensions and longitudinal slope
    b. Observed hydraulic roughness
3. Bed Material
    a. Particle size distribution or D50 and D84
4. Bank Stability Data (geomorphic / historical assessment)
    a. Visually observed bank erosion location, length, etc.
    b. Determination of failure mechanisms
    c. Mode of channel changes over time
    d. Vegetation type and density
    e. Root depth and density

POTENTIAL STABILITY INDICES

1) Thresholds of Mobility
    a) Critical shear stress
    b) Critical velocity
2) Time series (statistical analysis: frequency distributions, etc.)
    a) Peak flow, velocity
    b) Stream power, Bledsoe's erosion index
    c) Shear stress, shear stress ratio
3) Time Integrated
    a) Work done
    b) MacRae's potential erosion index

DRAFT

T E C H N I C A L

M E M O R A N D U M # 4

TO: Onsite Management Measures Subgroup
Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP)

FROM: GeoSyntec Consultants and SCVURPPP Staff

DATE: April 1, 2004 (REVISED FINAL DRAFT)

SUBJECT: **Evaluation of the Range of Storms for HMP Performance Criteria**

Under Provision C.3.f.iv.3 of the SCVURPPP NPDES permit, "the HMP proposal shall identify the maximum rainfall event below which the standard applies, or range of rainfall events for which the standard applies"[1]. This memorandum provides a discussion of technical results for evaluating which flows, or range of flows, are the most important when considering stream channel erosion and hydromodification impacts. This analysis is based on the Erosion Potential (Ep) methodology developed on Thompson Creek. Results from the Ross Creek and San Tomas Creek studies will be incorporated into this work as they become available.

The results for two land development scenarios are being compared in this memo: pre-development represented by 1970 conditions and future development in 2020. The results and discussion should simply be considered as an example of a "before" and "after" watershed condition. The same analysis would be done if we were evaluating a proposed future development in a currently undeveloped watershed. We believe that it is important that the Work Group develops an understanding of this simple (and perhaps ideal) comparison before moving forward and applying the HMP to more complex mixed development scenarios (e.g., partial development, infill, etc.).

By making this comparison, there is no intent to suggest that cities should require developers to reduce existing runoff to pre-urban conditions. The proposed Standard (TM #3), for example, requires discharge from future development to be maintained relative to existing conditions at the time of development. Whether the proposed development is located in an undeveloped watershed (or portion thereof) discharging to a healthy creek, or in a partially developed watershed discharging to an impacted creek, the intent is to not increase the current potential for erosion. Where the District has plans to repair and/or restore certain reaches of a stream, they

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[1] RWQCB staff has indicated that, although paragraph C.3.f.iv.3 refers to rainfall events, management of hydromodification should be focused on runoff and stream flows (personal communication with Jan O'Hara, SFRWQCB). See discussion in Appendix A regarding the distinction between rainfall event and flow event frequencies or return periods.

would likely allow for known future development (refer to TM #3 for more specific details). In cases where a developer chooses not to participate with District's plans, or the development is unknown at the time the plans are made, the developer would be required to maintain *existing runoff conditions* and not increase the potential for erosion. In all cases, the HMP methodology can be used to measure changes in the erosion potential, account for altered watershed and stream conditions, and evaluate proposed control measures.

This memo considers the case where the watershed is mostly undeveloped and is being evaluated for impacts caused by future build-out conditions. In this case, the baseline stream condition is assumed to be a healthy stable stream. Once the Work Group understands this scenario, the next step is to analyze the scenario where a development is proposed in an already developing watershed and the objective is to not increase the potential for erosion, i.e., maintain the existing conditions.

Section 1 presents the predicted range of geomorphically significant flows before development to show which flows appear most important in influencing sediment transport and the erosion processes. Section 2 presents similar results under developed conditions to show how this range of important flow changes. Section 3 presents a discussion on managing the impacts of hydromodification and which flows must be controlled to minimize impacts.

1 Geomorphically Significant Flows of Thompson Creek before Urban Development

Figure 1 presents the cumulative work curves predicted for a subset of stream cross sections in the Thompson Creek subwatershed when the subwatershed was relatively undeveloped. Representative cross sections were selected to show the range of flow conditions. The curves are in order of upstream cross section (higher identifying number) to downstream (lower number) moving from left to right. The upstream cross sections, having smaller drainage areas, are represented by the lower flow values.

The intent of Figure 1 is to show that the flows which appear most important in controlling sediment transport and the erosion processes range from near zero up to the 10-year peak flow. These curves illustrate that a significant amount of the total work done (approx. 90%) on the channel bed and banks is done by flows up to the 10-year peak flow. Flows up to the 2-year peak flow are predicted to perform from 30% to 60% of the total work done on the channel depending on the local flows, channel slope and geometry.

One important concept here is that, before urban development, no single storm event size is controlling the erosion processes and channel form, but that a wide range of storms up to the 10-year event are all influencing the physical characteristics of stream channels. The definition of geomorphically significant flows has been provided in previous HMP submittals.

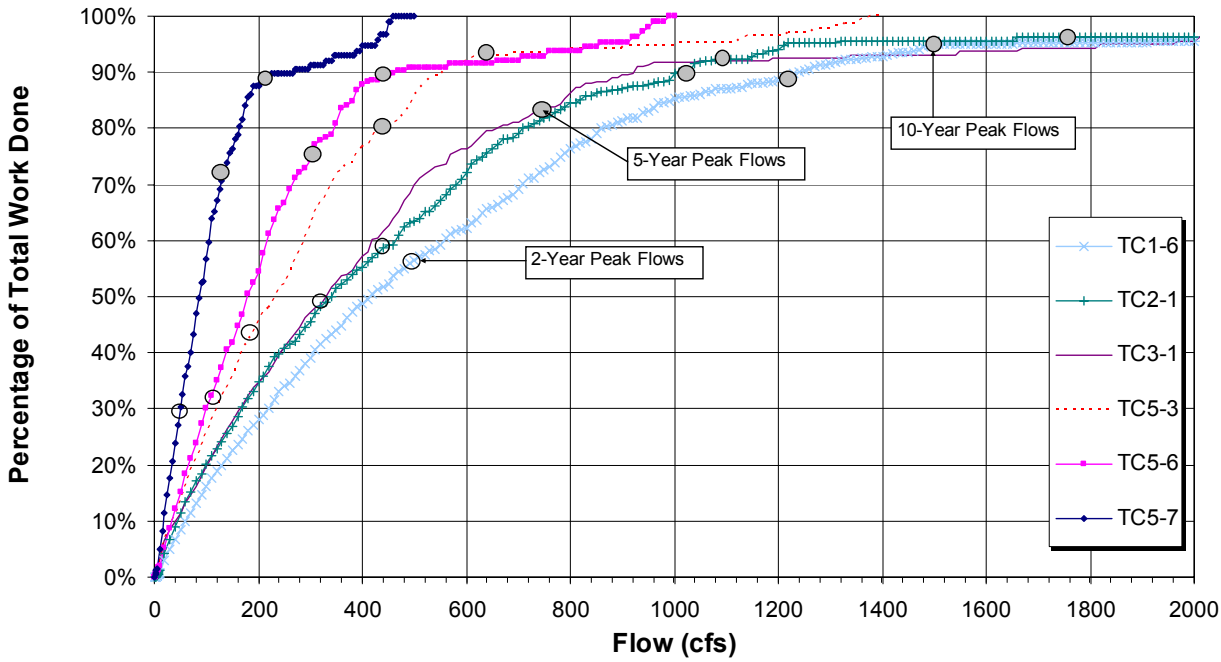

Figure 1 - Cumulative Work Curves Showing Range of Significant Flows before Development (using Thompson Creek stream segments as an example)

2 Geomorphically Significant Flows for the Future Built-Out Scenario

Figure 2 presents the cumulative work curves predicted for the same stream cross-sections after the Thompson Creek subwatershed is developed to the build-out condition, based on General Plan information. Comparison of this figure with Figure 1 shows the changes predicted in the cumulative work curves resulting from development of the subwatershed. Peak flows for the 2-, 5-, and 10-year storms for the pre-development condition have been indicated on Figure 2 to illustrate how much additional work is done on the stream for the same flow rate under post-development conditions.

Generally, a larger percentage of work is done by lower flows than what was done before development. This is shown by the faster rise in the cumulative work curve between zero and approximately 300 cfs. The percentage of work done by flows up to the 2-year peak flow changes widely. The upper Segment 5 cross section (TC5-7) changes very little, while the others show work increases of 10% to 20% (e.g., for TC1-6, work done by flows up to the 2-year peak flow increases from 55% to 75%). The percentage of work done by flows up to the 10-year peak flow changes slightly.

As discussed above, these results help indicate which flows are contributing most to the total work done and suggest which flows should be managed for hydromodification. Flows less than and up to the 2-year event become more significant than they were before development,

however, flows between the 2-year and the 10-year peak flow still contribute a significant percentage of the total. Management of hydromodification should definitely include flows up to the 2-year peak flow where most of the change is observed. However, flows greater than the 2-year peak flow are also causing increases in the total work done on the channel. This is discussed further in the following section.

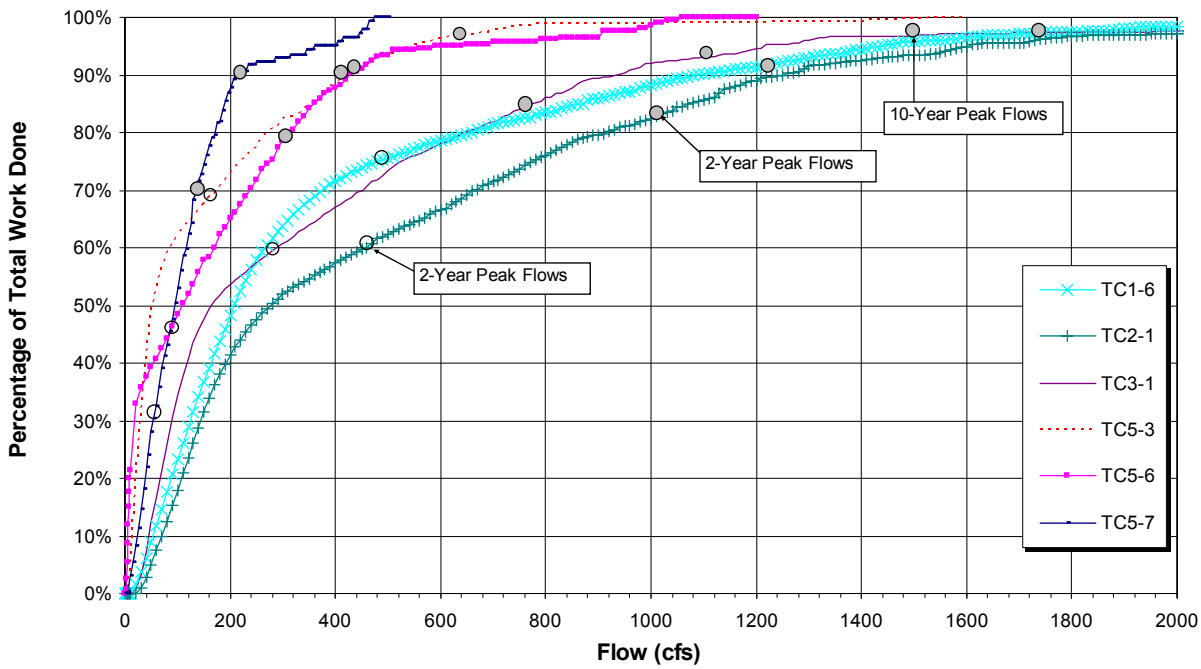

Figure 2 - Cumulative Work Curves Showing Range of Significant Flows after Development

For comparison, Table 1 provides the pre- and post-development peak discharges for the 2-, 5- and 10-year events.

Table 1. Return Period Peak Discharges

Cross Section	Pre-Development Condition			Post-Development Condition		
	2-year	5-year	10-year	2-year	5-year	10-year
TC 1-6	530	1,229	1,752	1,188	2,002	2,519
TC 2-1	438	1,032	1,480	963	1,707	2,179
TC 3-1	322	764	1,099	697	1,298	1,676
TC 5-3	189	451	653	356	700	922
TC 5-6	127	311	455	182	405	562
TC 5-7	56	139	203	72	163	232

3 Selecting the Range of Storms to Manage

Consider the case in which an existing undeveloped area is being significantly developed. Cumulative work curves such as those presented in Figure 3 can be developed at a cross section just downstream from the project's proposed discharge, for the existing undeveloped land use condition (pre-development flows) and the proposed development without any flow controls in-place (post-development flows). As described in TM #3, a proposed HMP performance criterion is to maintain the Ep ratio (the ratio of post-development to pre-development erosion potential) at 1.2 or less (i.e., within 20% of the predeveloped condition). In the example illustrated in Figure 3, the total work done for the existing undeveloped condition is 940 units. Allowing an increase of 20% (i.e., Ep ratio = 1.2) results in an allowable increase in work up to 1128 units. If flows are uncontrolled, the post-development Ep ratio would be 11 (10,340 work units) in this stream segment.

Equating the range of flows to manage to discrete storm sizes helps specify which flows should be managed to achieve the HMP objectives. Both the 2-year and 10-year peak flow magnitudes are shown on Figure 3.

There are two important flow ranges illustrated in Figure 3: 1) the range of flows to be managed, and 2) the flows that can be discharged uncontrolled (from an erosion perspective). The results indicate that flows must be controlled in the range where the post-development work curve increases most dramatically. Work must be matched up to a point where the remaining uncontrolled flows do not increase the total work beyond the allowable 1128 units. Although the increase in work done by high flows is small relative to the increase caused by lower flows, the increase is still measurable. These higher uncontrolled flows add the allowable increase in work (20%).

Figures 4 and 5 show example post-development work curves for two other stream cross sections in Thompson Creek, each with smaller predicted Ep ratios for the post-development condition. Cross sections TC3-7 and TC1-6, with Ep ratios of 4 and 7 respectively, are presented to show the range of possible conditions and the variability in the extent of hydromodification impacts. When flow conditions are the same between cross sections, channel slope, critical shear stress and geometry determine the magnitude of the erosion potential.

All stream systems are likely to have reaches with varying degrees of predicted impacts depending on location specific channel characteristics, even within relatively short distances. Selecting management criteria on the basis of less sensitive reaches is only partially protective if there are more sensitive ones downstream. The location of a development may influence the influence the frequency of storm that the development will be required to mitigate. However, this analysis suggests that in general, for Thompson Creek and similar watersheds on the eastern side of the Santa Clara Basin, all flows up to the 10-year peak flow should be controlled to fully protect streams from hydromodification caused by future development.

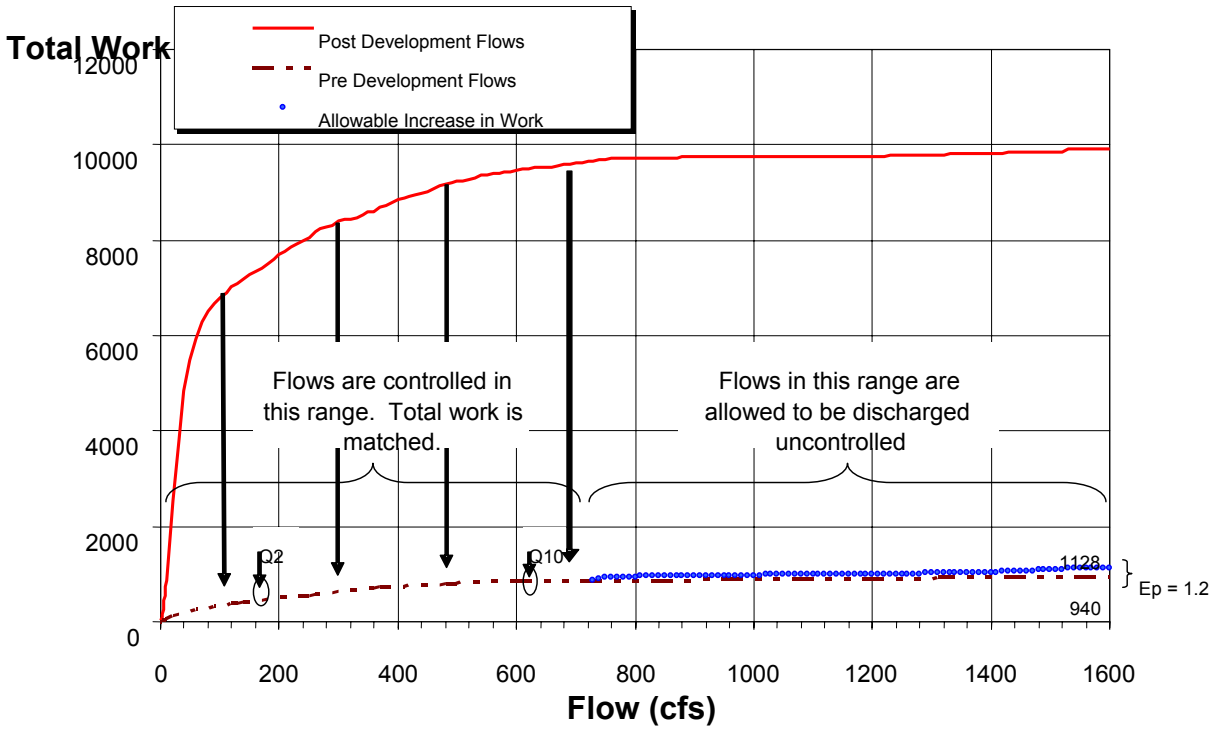

Figure 3 - Cumulative Work Curves Illustrating Range of Flows to Manage using Cross Section TC 5-4 (Ep ratio for uncontrolled post-development condition = 11)

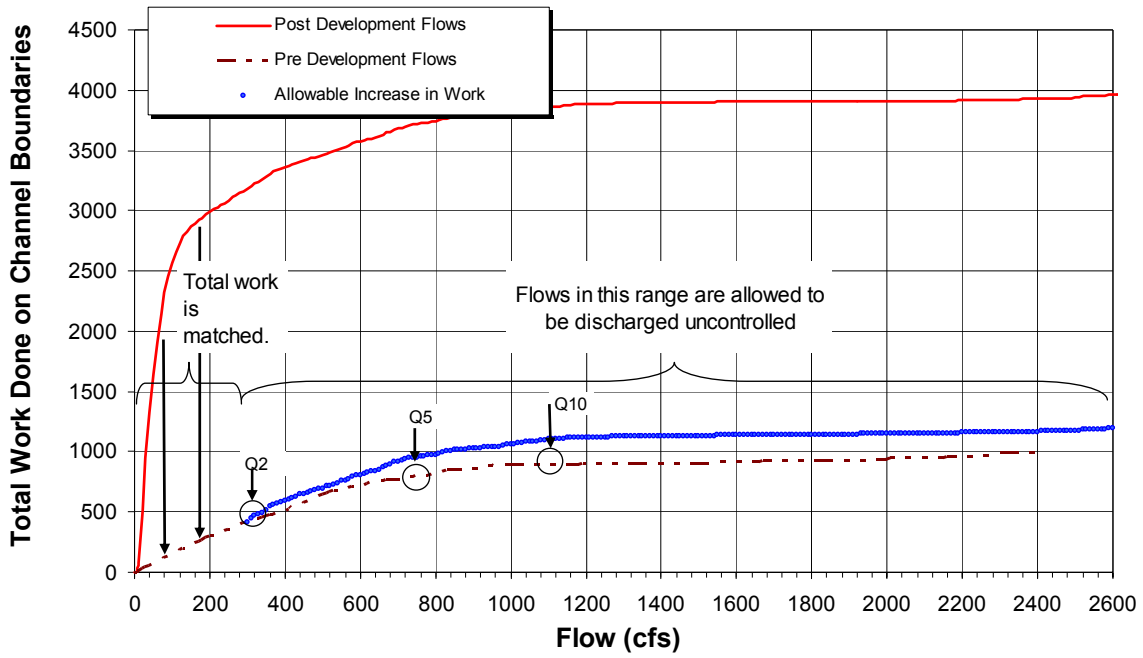

Figure 4 - Cumulative Work Curves for Cross Section TC3-7 (Ep ratio for uncontrolled post-development condition = 4)

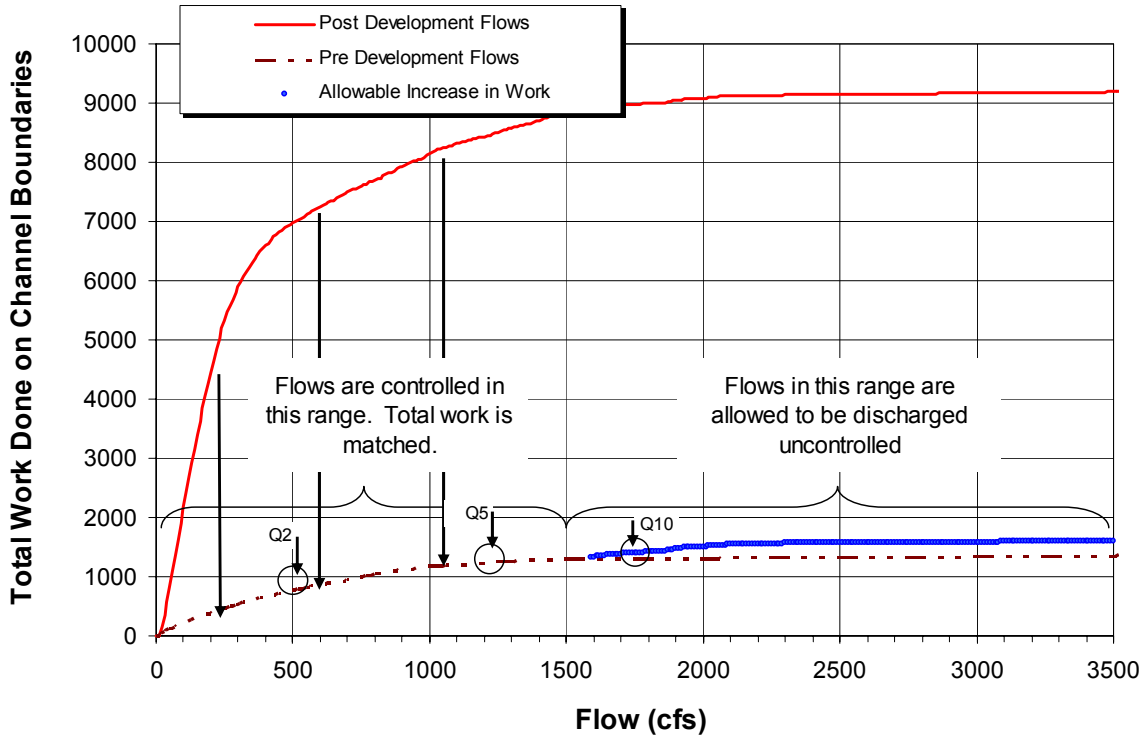

Figure 5 - Cumulative Work Curves for Cross Section TC1-6 (Ep ratio for uncontrolled post-development condition = 7)

3.1 Discussion on the Range of Storms to Manage

There have been some questions raised regarding whether selecting the 10-year peak flow as the upper limit for the range of storms to manage is conservatively high, and whether this selection is appropriate Basin wide. This section provides a response to these questions.

Within the Thompson Creek study area, measured Ep ratios vary due to stream channel characteristics, with medium and high ratios intermixed with low ratios. There are several factors for this, but slope is an easy one to consider. Figure 6 provides a simplified illustration of a longitudinal profile with both shallow slopes and steep slopes downstream from an outfall location.

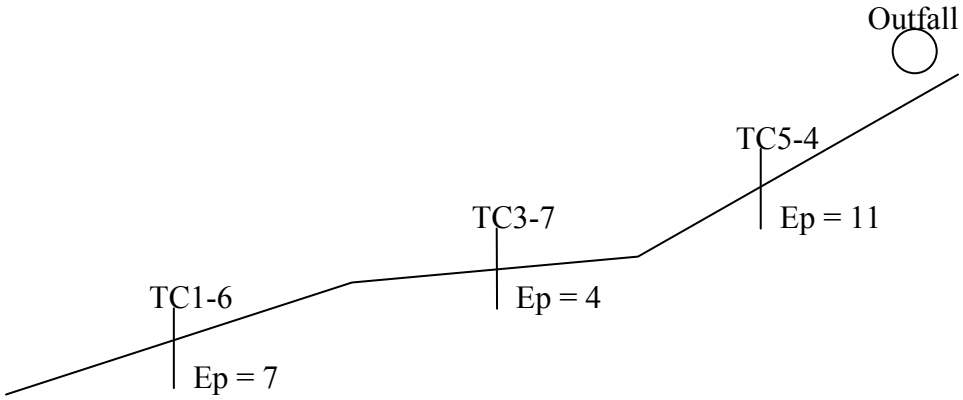

Figure 6 – Illustration of Longitudinal Profile Showing Cross Sections and Outfall

1. Considering this illustration and the results discussed above, a range of flows should be selected to be fully protective of all stream segments downstream of the discharge point. For this reason, the 10-year peak flow is believed to be an appropriate upper limit for the range of flows to be managed within the Thompson Creek Subwatershed. There may be locations in the downstream portion of the subwatershed that could control for less than the 10-year peak flow and still be protective of downstream segments; however, an Ep analysis would have to be performed to verify this for a particular project.
2. Both flow duration and volume control basins are sized to control the *difference* in runoff volume between pre- and post- condition storm events, not the entire event volume. Results show that the difference in runoff volume between pre- and post- 10-year storm events is roughly equivalent to the total 1.1-year, or slightly larger, storm event volume.
3. Western Washington selected the 50-year peak flow as its upper limit. Our tests show flows greater than the 10-year peak flows contribute less than 5% to 10% of the total work done on the stream channel.

The assumption that Thompson Creek is representative of other east side subwatersheds was one of the basic assumptions made at the beginning of the project when the Lower Silver/Thompson Creek Subwatershed was selected as the first test watershed. The results from the Thompson Creek assessment, indicating that the climate, soils, geology, and vegetation characteristics of this subwatershed are the same as those for the east side watersheds, confirms this assumption. For this reason, the 10-year peak flow is believed to be an appropriate upper limit for other east side subwatersheds of the Coyote Watershed, as well as Thompson Creek.

Extrapolating these results to the west side watersheds is much less certain, which is one of the reasons why the Ross/San Tomas subwatersheds were selected as additional test subwatersheds. Results from these subwatersheds will help us determine if aggregating results is appropriate, or whether different criteria must be developed.

Appendix A

References To Rainfall Event Versus Flow Event Characteristics in C.3.f of the NPDES Permit

Under Provision C.3.f.iv.3 of the SCVURPPP NPDES permit, the HMP "shall identify the maximum rainfall event below which the standard applies, or range of rainfall events for which the standard applies". RWQCB staff has indicated that, although paragraph C.3.f.iv.3 refers to rainfall events, management of hydromodification should be focused on runoff and stream flows (personal communication with Jan O'Hara, SFRWQCB). Reference to rainfall events helps communicate management requirements and limitations to developers and others who may be involved with implementation. Management strategies that rely on on-site controls where traditional engineering practices are used for design need references to storm size for analysis, such as the storm with a 10% chance of occurring in any given year (i.e., 10-year rainfall event).

It is well known that the 10-year recurrence interval storm (or any other storm) is not necessarily the same storm that produces the 10-year peak flow. Any given event that occurs in the fall when soils are relatively dry does not produce as much runoff as it does when the same storms occurs in the spring when the soils are saturated. This effect is more prevalent for smaller storms than larger ones (e.g., 2-year events vs. 100-year events) and for low percentage impervious areas than for higher ones. One of the primary purposes of using the continuous simulation modeling approach is to account for the effects of antecedent conditions and generate a flow record with a specific probability distribution.

However, for simplicity, water resource professionals frequently assume that the 10-year rainfall event produces a 10-year runoff event and peak flow. This is probably acceptable for computations of drainage infrastructure in urban areas and less appropriate for watershed wide and stream network modeling, where large areas are undeveloped.

Assuming, for example, that the 10-year rainfall event is equivalent to the 10-year runoff event from a management perspective is reasonable for on-site management strategies, like volume control. Implementing flow duration control or an in-stream erosion control strategy requires continuous simulation modeling such that the 10-year event and peak flow can be estimated from the predicted long-term flow record.

DRAFT

T E C H N I C A L

M E M O R A N D U M # 5

TO: HMP Onsite Management Measures Subgroup
Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP)

FROM: GeoSyntec Consultants and SCVURPPP Staff

DATE: April 1, 2004 (REVISED DRAFT)

SUBJECT: **Evaluation of Volume Control Effectiveness**

Background

This Technical Memorandum (TM) #5 provides a discussion on the effectiveness of using volume control to manage hydromodification, for review and discussion by the OMM Subgroup and the HMP Work Group.

TM #3 discusses a proposed hydromodification standard, performance criteria, and implementation options. The three performance strategies that can be used include: project runoff volume control, project runoff flow duration control, and in-stream erosion potential (Ep) control. These strategies differ by the level of detail in the analysis and costs for implementation. Although the erosion potential strategy is considered the most comprehensive approach to evaluate HMP impacts and control measures, the project team recognized that a simpler approach such as volume control may be needed for smaller projects.

Volume control means that only the amount of runoff generated from the existing (pre-project) site may be discharged from the site after development, and the difference between the post-project and pre-project volumes must be retained on-site, discharged to a non-eroding stream segment, or discharged at a flow rate that does not increase erosion. The performance criteria for the volume control strategy does not stipulate the shape of the post-project discharge hydrograph, only that its volume (i.e., the area under the hydrograph) must be the same as the pre-project conditions. However, this approach may not be as accurate as flow duration control in maintaining the existing Ep of stream segment receiving the discharge. This memorandum evaluates and discusses the effectiveness of volume control by comparing the work done on the stream by different shaped hydrographs using the erosion potential concept.

Methodology

The evaluation was performed using the discrete events as defined in the District's Design Flood Flows Manual ("Green Book", 1979) and modeled for the Thompson Creek subwatershed. The method used was to compare the work done by various post-development hydrographs with

volume control to the pre-project hydrograph (in this case, the "pre-urban" hydrograph for segments of Thompson Creek), which is assumed to represent the target work value for this example.

The questions to be addressed are:

1) What is the affect of hydrograph shape on in-stream work when storm runoff volume is maintained at pre-project levels?
2) Is volume control effective?

Hypothetical Hydrograph Shape Test

To evaluate the effect of hydrograph shape while maintaining pre-project volume, the work done by several hypothetical hydrographs of different shapes is compared to the work done by the pre-project hydrograph. It was recognized in TM #3 that volume control is not as accurate as flow duration control because it does not account for the differences in erosive power for the same excess volume at higher flows as compared to lower flows. Referring back to the work index in the draft HMP assessment for Lower Silver/Thompson Creek Subwatershed (equation 1, Section 5.3, GeoSyntec, July 2003), the reader will notice that work is a non-linear function of excess shear stress. This non-linearity results in these differences in erosive power between hydrographs of the same volume which have different flow rates.

The hypothetical hydrographs, called test hydrographs, were created to compare the amount of work done on a stream channel between hydrographs of different shapes, but having the exact same total volume (area under hydrograph). The test hydrographs (Figure 1.b.) were designed to represent a range of possible conditions between the post-project hydrograph and the pre-project hydrograph (Figure 1.a.). In this example, the pre-development runoff volume 99 ac-ft, and the post-development volume is 166 ac-ft. It is assumed that a developer would begin with a post-project hydrograph and then consider BMPs to mitigate the effects. Uniform (rectangular shape) hydrographs were selected to have peaks ranging from the post-project peak to the pre-project peak, with duration changing as needed to maintain pre-project runoff volume (99 ac-ft.). Test Hydrographs 1 through 3 were selected in this manner (Figure 1.b.). Test Hydrograph 4 was designed to represent the case in which a volume control BMP captured the beginning portion of the storm until it is filled, at which time the remaining runoff is discharged uncontrolled. Test Hydrograph 4 consists of the remaining portion of the existing conditions hydrograph that is not captured in the BMP, and has a volume equal to the pre-project volume as required.

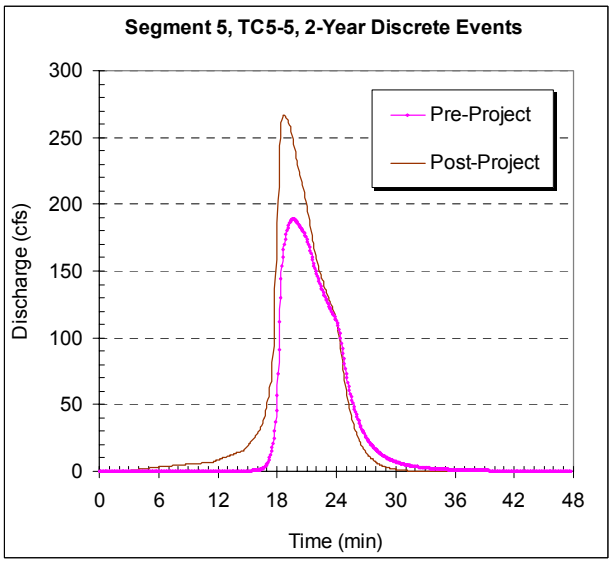

Figure 1.a. Pre- and Post-Development Hydrographs for TC5-5, 2-Year Event

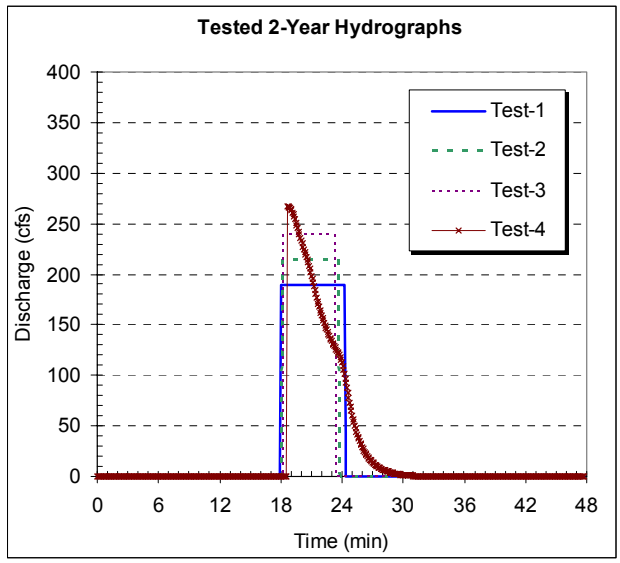

Figure 1.b. Test Hydrographs Used in Volume Control Analysis

The work done by each of these hydrographs was computed and compared. The results are summarized in Table 1 below.

Table 1. Estimated Work Done at TC5-5 by 2-Year Test Hydrographs of Varying Shapes

Hydrographs	Peak Flow (cfs)	Volume (ac-ft)	WORK (ft-lbs/sq.ft.)	Difference in Work from Pre-Project (ft-lbs/sq.ft.)	Percent Higher than Pre-Project Work
Pre-Project	189	99	555,937		
Post-Project	267	166	763,129	+207,192	37.3
Test Hydrograph 1	189	99	656,481	+100,544	18.1
Test Hydrograph 2	215	99	681,109	+125,171	22.5
Test Hydrograph 3	240	99	703,608	+147,671	26.6
Test Hydrograph 4	266	99	615,000	+59,063	10.6

Discussion

Table 1 shows that each of the test hydrographs does more work on the channel than the pre-project 2-year storm event hydrograph. With channel geometry and discharge volume held constant, the differences in work are solely a result of changing the hydrograph shapes.

Other hydrographs, such as uniform hydrographs with smaller peak flows than pre-project and triangular shaped hydrographs, could be selected to bring these differences closer to the pre-project value. However, there are some observations that we can use to infer the results of most other shapes.

With volume remaining constant, the following conditions are observed:

- Higher peaked hydrographs with shorter duration do more work than lower peaked hydrographs with longer duration.
- Test Hydrograph 1 has the same peak as the pre-project hydrograph, but also does more work than the pre-project hydrograph because of concentrating volume in a shorter duration.
- Test Hydrograph 4 produces the least difference in work when compared to pre-project, because flows are distributed over a longer time with a portion of the hydrograph volume discharged at low peak rates creating the long tail of the hydrograph.

Discrete Storm Event Test

In a second test, hydrographs were developed for the 1.1-year, 2-year, 5-year, and 10-year discrete events, for Segment 5 and for a small sub-catchment discharging to Misery Creek, a tributary to Thompson Creek. Two cross sections per segment were used in the evaluation. For example, in Segment 5, both TC5-4 and TC5-6 were evaluated and each of these cross sections had 4 tests conducted, so all total there were 8 tests conducted per segment. The results for Segment 5 and Misery Creek are plotted in Figures 2 and 3.

Discussion

Figure 2 plots the results for Segment 5 for all the design storms tested. The results express the range of potential work done in excess of the pre-project amount. For example, the results for the 2-year storm show a range of work from 1 to 32 percent greater than the pre-project amount, depending on the hydrograph shape. The range for 1.1-year events is significantly higher at 300 percent (the largest increase in work would be created by a uniform hydrograph with a high peak flow). The minimum value of 1 is assumed because it is possible to find shapes that create work very close to the pre-project amount.

It is interesting that the magnitude of work done above the pre-project amount gets smaller as the storm size increases. A possible reason for this is that the difference between pre and post conditions is larger, percentage wise, for smaller volume hydrographs than larger ones.

This trend is also observed when comparing downstream cross sections to upstream cross sections. This suggests that smaller catchments with volume controls discharging to smaller creeks could potentially have higher post-development increases in work and stream channel erosion, than what is shown in Figure 2.

To test this theory, a volume analysis was conducted on a 250 acre sub-catchment discharging to Misery Creek, a tributary to Thompson Creek. (The previous cross sections in Segment 5 have contributing catchment areas of 2,400 to 3,700 acres.)

As expected, the tests of this smaller catchment area show higher increases in work above the targeted pre-project amount. For example, work done by a post 2-year hydrograph could be up to 80% more than the pre-project amount, depending on the shape of the hydrograph. For both the 5 and 10-year storms, the work done by the post-project runoff hydrographs could be up to 50% more than the pre-project hydrograph.

This analysis suggests that specifying only volume control could lead to unexpected increases in work and stream erosion, unless other criteria are specified that control the shape of potential discharge hydrographs. One approach would be to require "hydrograph matching" instead of volume matching. Hydrograph matching maintains the volume and distribution of flows for a single discrete storm event in a similar manner as that used for flow duration control. [Note – the effectiveness of the hydrograph matching approach is evaluated in TM #7.]

Conclusions

Conclusions that can be drawn from the volume control evaluation include the following:

- Hydrograph shape does matter and unexpected stream erosion could occur when managing volume control alone.
- Volume control does not appear effective unless other controls are imposed to maintain the pre-project hydrograph shape.
- Adding peak flow matching to the performance criteria helps improve effectiveness, but is still not enough. For example, Test Hydrograph 1 in Figure 1.b. has the same peak as the pre-project hydrograph, yet it does more work because the tails of the pre-project hydrograph are concentrated in a shorted duration uniform hydrograph.
- The volume control performance criteria in TM #3 may need to be changed to specify "hydrograph" matching, as opposed to "volume" matching.

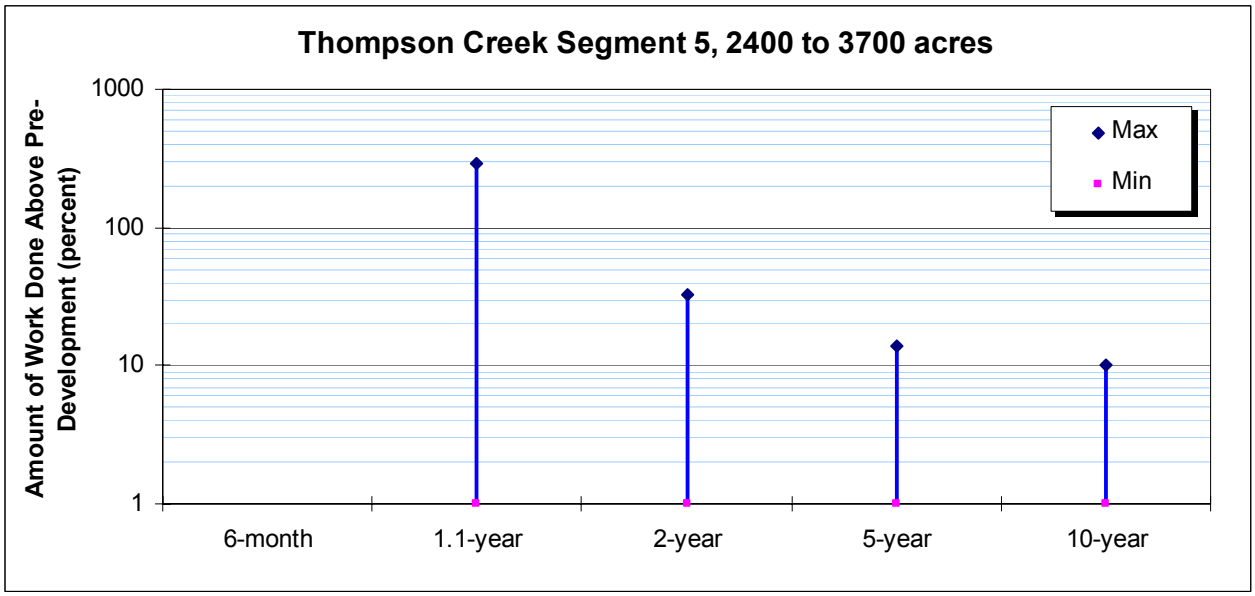

Figure 2. Difference in Work Done for Varying Hydrograph Shapes Having the Same Volume.

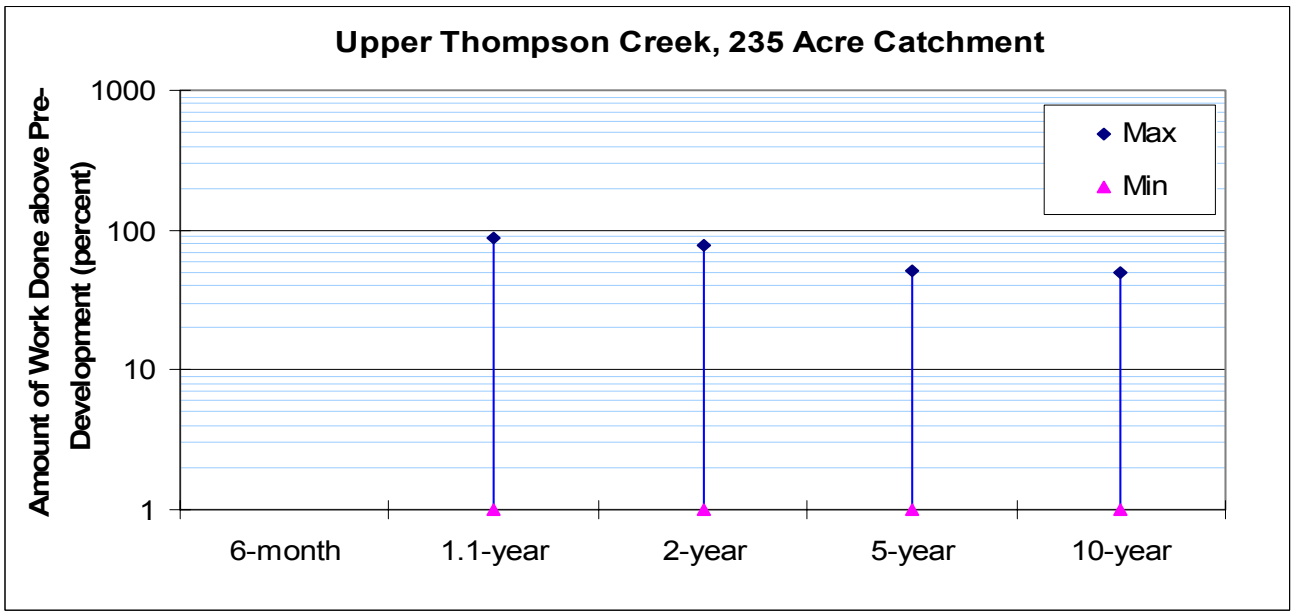

Figure 3. Difference in Work Done for Varying Hydrograph Shapes for a 250 acre Catchment.

DRAFT

T E C H N I C A L

M E M O R A N D U M # 6

TO: HMP Onsite Management Measures Subgroup
Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP)

FROM: GeoSyntec Consultants and SCVURPPP Staff

DATE: April 1, 2004 (REVISED DRAFT)

SUBJECT: **Volume Control Sizing Example and Cost Analysis**

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1 Introduction

Technical Memorandum #3 – Draft Hydromodification Control Standard, Performance Criteria, and Implementation Options (revised draft dated January 16, 2004) includes a simplified hydromodification management strategy centered on maintaining the pre-project runoff volume. GeoSyntec concurrently evaluated the effectiveness of this option in Technical Memorandum (TM) #5.

Volume control requires that the increase in surface runoff created by the installation of impervious surfaces be retained on-site and not discharged. As a result, the BMPs considered must be able to capture and store a design volume and then dispose this volume through infiltration and/or evapotranspiration, or discharge at a very slow rate.

The draft performance criterion for volume control is that certain projects may meet HMP requirements by providing stormwater controls that match pre-project runoff volumes. The Work Group is currently trying to determine the appropriate range of storms to be managed under this criterion. The objective of TM #6 is to assist the Work Group with this decision by providing an example with costs for the range of possible storms.

This memo discusses the application of the volume control approach to a 17-acre proposed development in a city in Santa Clara Valley (Figure 1). It is assumed that this development discharges to a nearby creek and is non-exempt and the HMP requirements apply. The developer chooses to implement the volume control strategy.

The total proposed development area consists of 9 acres of residential housing, a 1-acre park, and a 6-acre senior housing complex. This example considers the residential area only. The residential area consists of 87 lots at 3,000 square feet each, with 65% of impervious surfaces, or 1,950 sq. ft per lot, plus 131,040 sq.ft. of street right-of-way (ROW).

Two types of facilities are evaluated: 1) surface storage and infiltration (basin), and 2) sub-surface storage and infiltration (bio-swale & infiltration trench combination). By-passes and low-impact design strategies that minimize the increase in surface runoff should be considered; however, this example is intentionally focused on sizing and costs of the two facilities listed above.

There are two fundamental questions to be addressed in this memo related to technical feasibility given local soils and rainfall, and to economic feasibility of construction and possibly reserving land for HMP controls. Questions to be addressed include:

1) What does it take to fully retain and infiltrate the increase in surface runoff for the range of possible storm events being considered for HMP management?
2) What is technically and economically feasible to accomplish with on-site measures?

The first question is addressed by developing sizing requirements for a range of soil types and design conditions as a function of storm size. A family of curves presenting the results is discussed below. The second question is addressed by applying the volume control methodology to a real proposed development, including costs, to help determine what is actually feasible given local conditions.

This example problem also will help us answer the question:

3) What storm size should be used (2-year, 5-year or 10-year) as a requirement for sizing HMP volume controls?[1]

1.1 Building-Blocks

Two fairly simple building-block computational tools were developed and used in this example: 1) basin sizing charts and 2) cost curves. The sizing charts are used to help determine what is needed in order to meet the HMP requirements for various storm sizes. These charts express the basin size required as a function of the design storm, for several soil types found in the Santa Clara Valley. The cost curves help evaluate the feasibility of volume controls. These curves present construction costs as a function of storm size and soil type.

1.1.1 Sizing Chart Computations

Table 1a presents the calculations for sizing a surface basin intended for infiltration. Table 1b presents calculations for sizing sub-surface facilities filled with gravel. The intent is to develop a family of curves representing a range of conditions.

This section summarizes table computations and assumptions:

- The computations are based on the area of impervious surfaces contributing to the facility (column 1). The intent is to capture only the increase in surface runoff caused by the impervious surfaces. Any runoff from pervious surfaces, for example lawns and

[1] The effectiveness of the volume control approach is evaluated in TM #5.

landscaped areas, is considered normal and not used in sizing the facility. Results are plotted as a percentage of the impervious area created.

- Column 2 lists the range of storm sizes considered in terms of total precipitation depth, for a 24 hour design storm event, including the 1.1-year, 2-year, 5-year, and 10-year return period storms[2]. Precipitation of 0.5-inches and 0.75-inches are approximately the 60th and 85th percentile storm volume for local gages in Santa Clara Valley.
- Column 3 shows the estimated increase in surface runoff from the area that is planned to be impervious. This increase in runoff is the amount that must be retained and disposed of, and is a function of the soil type and infiltration rates (Column 4). For a given storm size, impervious surfaces constructed on soils with high infiltration rates have greater increases in surface runoff when compared to soils with low infiltration rates. The method used to compute runoff volume is the SCS Curve Number method.
- The infiltration rates listed in Column 4 are taken from the Soil Conservation Service's *Soils of Santa Clara County* (1968), and represent the range of published values for C and D soils[3] typical of those found in Santa Clara County.
- Columns 5 and 6 represent assumed design limitations on basin drain time and depth, primarily from a public nuisance, construction and safety perspective. These values are fixed to generate the resulting family of curves.
- Column 7 is the computed volume that must be captured and retained on-site, and is simply the product of columns 1 and 3 (converted to acre-feet).
- Column 8 represents the depth of water that can be infiltrated given the infiltration rate and maximum drain time. In other words, given a certain soil type, how much water can be disposed of in 3 days? In conditions where this value is larger than the maximum allowable depth (e.g., row 4: 6 feet vs. 4 feet), the maximum allowable is used in the calculation of infiltration area (4 feet).
- Column 9 computes the surface area required for the range of conditions presented in the table. The required surface area is computed as volume / depth (column 7/column 8).
- Column 10 computes the surface area as a percentage of the contributing impervious surface.

1.1.2 Sizing Chart Results

Figure 2 presents the resulting family of curves generated from Table 1. For each of the four soil types (infiltration rates), the results indicate how much infiltrating area is required to retain and infiltrate the increase in runoff from the selected storm size. For example, given a 2-year design storm of 2.25 inches and a clay soil type with 0.06 in/hr of infiltration capacity, the required basin area is nearly 20% of the impervious surface area contributing to the facility. This is also

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[2] Storm depths for the 2-, 5-, and 10-year storms were obtained from the City of San Jose and are consistent with the City's hydrologic design standards.

[3] "C" and "D" soils are NRCS Hydrologic Soil Group designations for moderately to poorly infiltrating soils.

true for the 5-year and 10-year storm sizes. For better infiltrating soils, or engineered basin soils, the required basin area ranges from 3 to 10% of the contributing impervious surface area. (If the total catchment area draining to the basin is 50 % impervious, then the basin area would range from 1.5% to 5% of the catchment area.) The more a developer can disconnect and reduce directly connected impervious areas, the smaller HMP volume control facilities would need to be.

The results suggest that there is a decreasing dependency of basin size on storm size as storms get larger. This is most apparent for soils with infiltration rates of 0.06 in/hr. The reason this occurs can be traced back to the SCS CN Method, where the difference between pervious and impervious runoff volume becomes minimal for large storms. Another way to think about this is, no matter how big the storm is, only a certain amount of precipitation can infiltrate the ground surface under saturated conditions, and thus for larger storms the difference between pre- and post- surface runoff volumes become very small and nearly constant as storm size increases.

In Figure 2, the two lower curves overlap. The reason is that the 1.0 inch/hour curve is controlled by the maximum allowable depth rather than the infiltration rates. Referring back to Table 1, for a 1 in/hr soil, 6 feet of water storage could be infiltrated in 3 days. However, basin depth is limited to 4 feet for ease of construction and safety.

Figure 3 shows the same results for a sub-surface facility that would be filled with gravel, like an infiltration trench. A porosity of 40% has been assumed, and the drain time has been increased to 7 days. The same general trends seen in Figure 2 are also repeated in Figure 3. Note the overlap in the bottom three curves. Again, the depth limitation is preventing the two infiltration curves, 0.63 and 1.0 in/hr, from dropping lower on the graph.

For all soil types, except 0.06 in/hr, from 8% to 11% of the impervious surface area is needed to control storms in the 2 to 10-year range. Using our 50% impervious catchment area as an example, this translates to 4% to 5.5% of the overall drainage area. There is only a small difference in area requirements among the facilities sized for the 2-, 5-, and 10-year storms.

1.1.3 Cost Curves

Figures 4 and 5 show the construction cost curves for a surface basin (such as a flow duration basin) and a sub-surface facility (such as a bioswale/infiltration trench), respectively. These curves were derived from well detailed cost spreadsheets, including such elements as site preparation, earth work, structures, piping, re-vegetation and so forth. Costs do not include soil disposal fees, hauling, or contaminated soil testing, mitigation, or disposal. In addition, land costs have been excluded from this analysis at the request of the Subgroup.

Detailed spreadsheets are available for review. These are applied to the example problem in the following section.

2 Example Problem – Residential Area

Recapping from Section 1, the volume control sizing methodology was applied to a 9-acre proposed residential housing development (Figure 1). The residential area consists of 87 lots at 3,000 square feet each, with 65% of impervious surfaces, or 1,950 sq. ft. per lot, plus 131,040 sq.ft. of street right-of-way (ROW).

Table 2 shows the computational table for a surface infiltration basin to retain and infiltrate just the street runoff (131,040 sq.ft. of impervious surface). These surfaces are considered separately from the impervious surfaces on the home lots in case pre-treatment of street runoff is required before infiltration. Drainage swales along the roadside may be possible to treat street runoff.

The differences from Table 1 presented earlier are the total area of impervious surfaces, storage volume, and the resulting BMP areas. In Column 10, the percentages are the same as shown in Table 1, because all other parameters are the same.

Table 3 shows the computational table for a sub-surface basin on each individual lot to retain and infiltrate roof runoff, patio and walkway runoff. Runoff from these surfaces can be infiltrated without pre-treatment. Like Table 2, only the areas and volumes change and the percentage of total impervious area stay the same as shown in Table 1b.

2.1 Cost Estimates

Figures 6 and 7 provide the resulting cost curves for volume control for the 9-acre residential area. Figure 6 provides costs for the surface basin capturing runoff from streets and ROW. Figure 7 provides the costs for each lot with a house.

Because land costs are not included in the analysis, the figures show a slight but insignificant increase in facility cost as storm size increases. In addition, the cost of the facility increases with infiltration rate; i.e., the curves for the higher infiltrating soils plot highest on the figures. This is because the higher the infiltration rate of the pervious areas on site, the greater the difference in runoff volume between the pre-project and post-project condition. When land costs are taken into account, there are two effects on the figures: 1) the order of the curves switches and the facilities on sites with lower infiltrating soils are more expensive because surface area of the basin becomes the dominant cost factor; and 2) the land costs overwhelm the differences between basin costs for different storm sizes.

Figure 6 indicates that costs for a surface infiltration basin range from approximately $115,000 to $119,000 for all soil types and storm sizes. The results for the sub-surface lot-level system (Figure 7) suggest that costs will range from $3,600 to $4,000 per lot, and are again only slightly dependent on soil type and storm size.

2.2 Discussion of Feasibility

This section lists a summary of findings:

- The results suggest that volume control is technically feasible for soil types with an infiltration rate of 0.2 inches/hour and larger, and may be economically feasible if basin area requirements are below 10% of the contributing impervious area.[4]
- Given a dense clayey soil type with 0.06 in/hr infiltration capacity, can volume control can be achieved? Assuming the 0.20 in/hr curve represents a feasible solution, there are two approaches: 1) increase allowable storage time to 7 days (not acceptable to vector

[4] Another criterion for feasibility is whether costs to implement C.3., including HMP requirements, exceed 2% of project costs. This criterion would have to be evaluated on a site-specific basis.

control agencies for surface basins), or 2) decrease the area requirements by half. Assuming 3 days detention time, then half of the storage requirements could be obtained on-site and the remainder would have to be treated regionally or discharged at a slow rate. Sites with dense clayey soils can achieve about half of the storage requirements on-site.

- There appears to be only small differences in the sizes and costs of facilities designed to capture the pre- vs. post- volume difference for the 2-year, 5-year or 10-year storms. Soils with the lowest infiltration rates have less dependency on storm size than soils with higher infiltration rates.

2.3 Example Application to the Residential Area

The volume control BMPs were applied without making any changes in the layout or number of houses. Other development configurations might be possible that make better use of the BMPs proposed in this example.

Proposed Surface Concept Collecting Street Runoff:

- Considering the range of results in Table 2, a surface facility size of approximately 15,000 cubic feet (0.35 ac-ft) would be capable of capturing the 10-year storm volume difference for soils with an infiltration rate of 0.2 inches/hour or better. This works out to be a facility that is 0.30 acres in area, or about 10%, of the street ROW area. A basin sized for the 2-year storm volume difference would be 0.21 acres in area, or about 7% of the street ROW area.
- Figure 8 illustrates the placement of this facility using a surface infiltration basin. This facility could be designed as a multi-purpose facility within the designated park area.

Proposed Sub-Surface Concept Collecting Roof and Patio Runoff:

- Considering the range of results in Table 3, a sub-surface facility size of 230 cu.ft. and surface area of 82 sq.ft. would be capable of capturing the 10-year storm volume difference for soils with an infiltration rate of 0.2 inches/hour or better. This could be designed as a facility that is 40-feet long by 2-feet wide, per house. For a 2-year storm volume difference, a facility size of 160 cu.ft. (57 sq.ft. surface area) would be needed.
- Figure 9 illustrates the placement of facilities using a sub-surface bioswale/infiltration trench combination concept. Essentially, a 4-foot wide strip running along the fence line could accept runoff from each adjacent lot. This strip could be a vegetated garden and/or bioswale underlain by an infiltration trench, or bioretention system.
- A public facility including a trail system could be created using these strips.

Table 1a. Surface Basin Computations for a 1-Acre Impervious Catchment

	Open Surface Basin									
	Impervious Surface Area	Design Storm Rainfall Depth	Increased Runoff	Soil Permeability	Maximum Allowable Infiltration Basin Drain Time	Maximum Basin Depth	Detention Volume	Maximum Depth for Max Drain Time (plus direct rainfall)	Infiltration Basin Surface Area	Percent Total SA
	(ft^2)	(in)	(in)	(in/hr)	(days)	(ft)	(ac-ft)	(ft)	(acre)	
60%	43,560	0.5	0.31	0.06	3	4	0.026	0.36	0.072	7.2%
	43,560	0.5	0.32	0.2	3	4	0.027	1.20	0.022	2.2%
	43,560	0.5	0.32	0.63	3	4	0.027	3.78	0.007	0.7%
	43,560	0.5	0.32	1	3	4	0.027	6.00	0.007	0.7%
85%	43,560	0.75	0.48	0.06	3	4	0.040	0.36	0.111	11.1%
	43,560	0.75	0.53	0.2	3	4	0.044	1.20	0.037	3.7%
	43,560	0.75	0.55	0.63	3	4	0.046	3.78	0.012	1.2%
	43,560	0.75	0.55	1	3	4	0.046	6.00	0.011	1.1%
1.1-yr	43,560	1	0.62	0.06	3	4	0.052	0.36	0.144	14.4%
	43,560	1	0.71	0.2	3	4	0.059	1.20	0.049	4.9%
	43,560	1	0.76	0.63	3	4	0.063	3.78	0.017	1.7%
	43,560	1	0.79	1	3	4	0.066	6.00	0.016	1.6%
2-yr	43,560	1.5	0.83	0.06	3	4	0.069	0.36	0.192	19.2%
	43,560	1.5	0.99	0.2	3	4	0.083	1.20	0.069	6.9%
	43,560	1.5	1.11	0.63	3	4	0.093	3.78	0.024	2.4%
	43,560	1.5	1.2	1	3	4	0.100	6.00	0.025	2.5%
5-yr	43,560	2.2	1.03	0.06	3	4	0.086	0.36	0.238	23.8%
	43,560	2.2	1.28	0.2	3	4	0.107	1.20	0.089	8.9%
	43,560	2.2	1.49	0.63	3	4	0.124	3.78	0.033	3.3%
	43,560	2.2	1.65	1	3	4	0.138	6.00	0.034	3.4%
10-yr	43,560	2.6	1.11	0.06	3	4	0.093	0.36	0.257	25.7%
	43,560	2.6	1.41	0.2	3	4	0.118	1.20	0.098	9.8%
	43,560	2.6	1.66	0.63	3	4	0.138	3.78	0.037	3.7%
	43,560	2.6	1.87	1	3	4	0.156	6.00	0.039	3.9%

Table 1b. Sub-Surface Basin Computations for a 1-Acre Impervious Catchment

	Below Ground Storages w/Rock Fill							Porosity of gravel ->	0.4	
	Impervious Surface Area	Design Storm Rainfall Depth	Increased Runoff	Soil Permeability	Maximum Allowable Infiltration Basin Drain Time	Maximum Storage Depth	Req'rd Detention Volume	Maximum Depth for Max Drain Time (plus direct rainfall)	Infiltration Basin Surface Area	Percent Total SA
	(ft^2)	(in)	(in)	(in/hr)	(days)	(ft)	(ac-ft)	(ft)	(acre)	
60%	43,560	0.5	0.31	0.06	7	4	0.026	0.84	0.031	3.1%
	43,560	0.5	0.32	0.2	7	4	0.027	2.80	0.010	1.0%
	43,560	0.5	0.32	0.63	7	4	0.027	8.82	0.007	0.7%
	43,560	0.5	0.32	1	7	4	0.027	14.00	0.007	0.7%
85%	43,560	0.75	0.48	0.06	7	4	0.040	0.84	0.048	4.8%
	43,560	0.75	0.53	0.2	7	4	0.044	2.80	0.016	1.6%
	43,560	0.75	0.55	0.63	7	4	0.046	8.82	0.011	1.1%
	43,560	0.75	0.55	1	7	4	0.046	14.00	0.011	1.1%
1.1-yr	43,560	1	0.62	0.06	7	4	0.052	0.84	0.062	6.2%
	43,560	1	0.71	0.2	7	4	0.059	2.80	0.021	2.1%
	43,560	1	0.76	0.63	7	4	0.063	8.82	0.016	1.6%
	43,560	1	0.79	1	7	4	0.066	14.00	0.016	1.6%
2-yr	43,560	1.5	0.83	0.06	7	4	0.069	0.84	0.082	8.2%
	43,560	1.5	0.99	0.2	7	4	0.083	2.80	0.029	2.9%
	43,560	1.5	1.11	0.63	7	4	0.093	8.82	0.023	2.3%
	43,560	1.5	1.2	1	7	4	0.100	14.00	0.025	2.5%
5-yr	43,560	2.2	1.03	0.06	7	4	0.086	0.84	0.102	10.2%
	43,560	2.2	1.28	0.2	7	4	0.107	2.80	0.038	3.8%
	43,560	2.2	1.49	0.63	7	4	0.124	8.82	0.031	3.1%
	43,560	2.2	1.65	1	7	4	0.138	14.00	0.034	3.4%
10-yr	43,560	2.6	1.11	0.06	7	4	0.093	0.84	0.110	11.0%
	43,560	2.6	1.41	0.2	7	4	0.118	2.80	0.042	4.2%
	43,560	2.6	1.66	0.63	7	4	0.138	8.82	0.035	3.5%
	43,560	2.6	1.87	1	7	4	0.156	14.00	0.039	3.9%

Proposed Subdivision

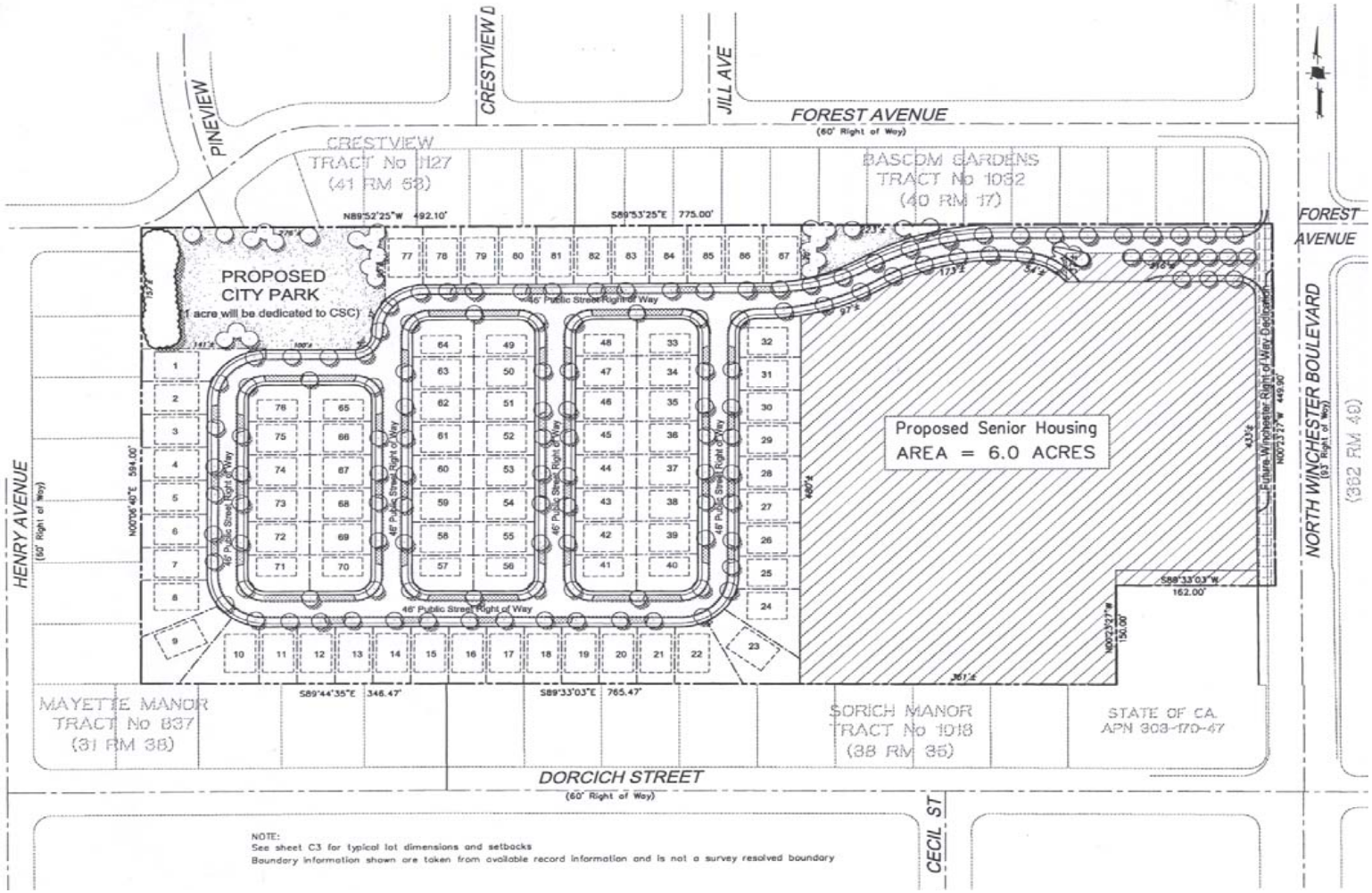


Figure 1

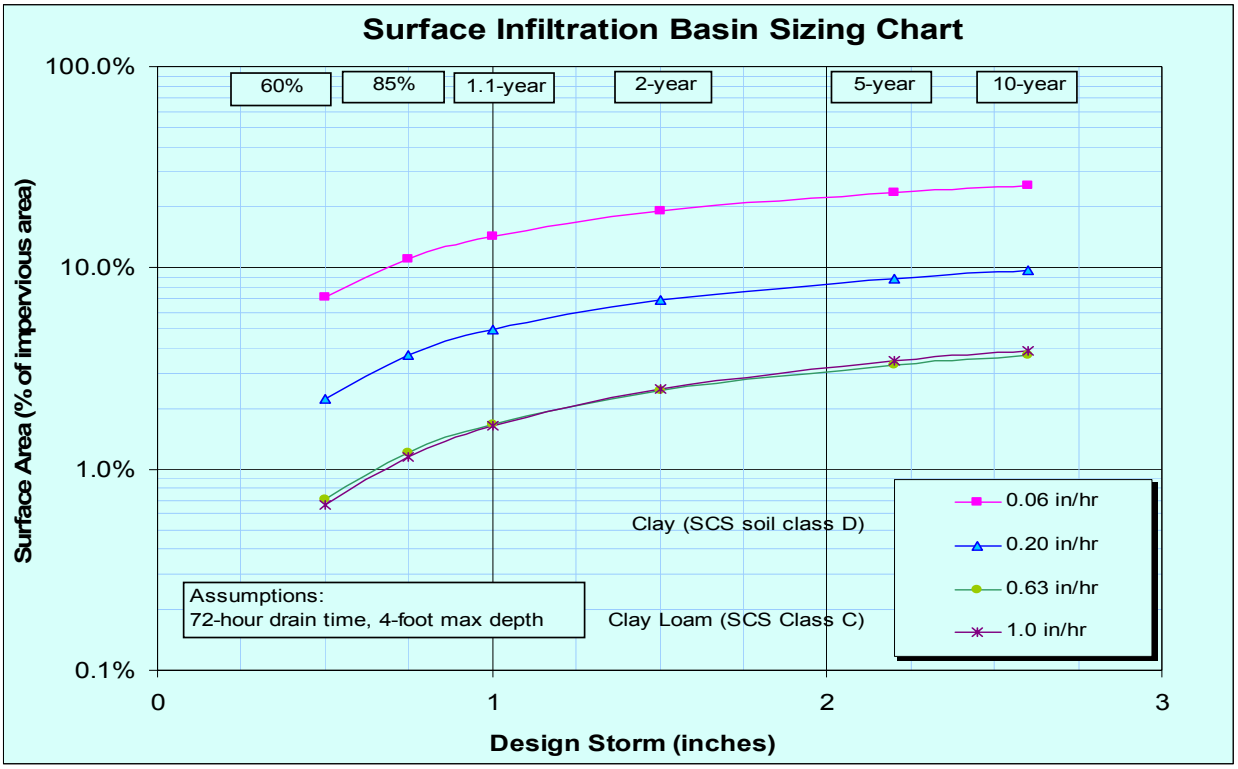


Figure 2 Results from the Surface Basin Sizing Table

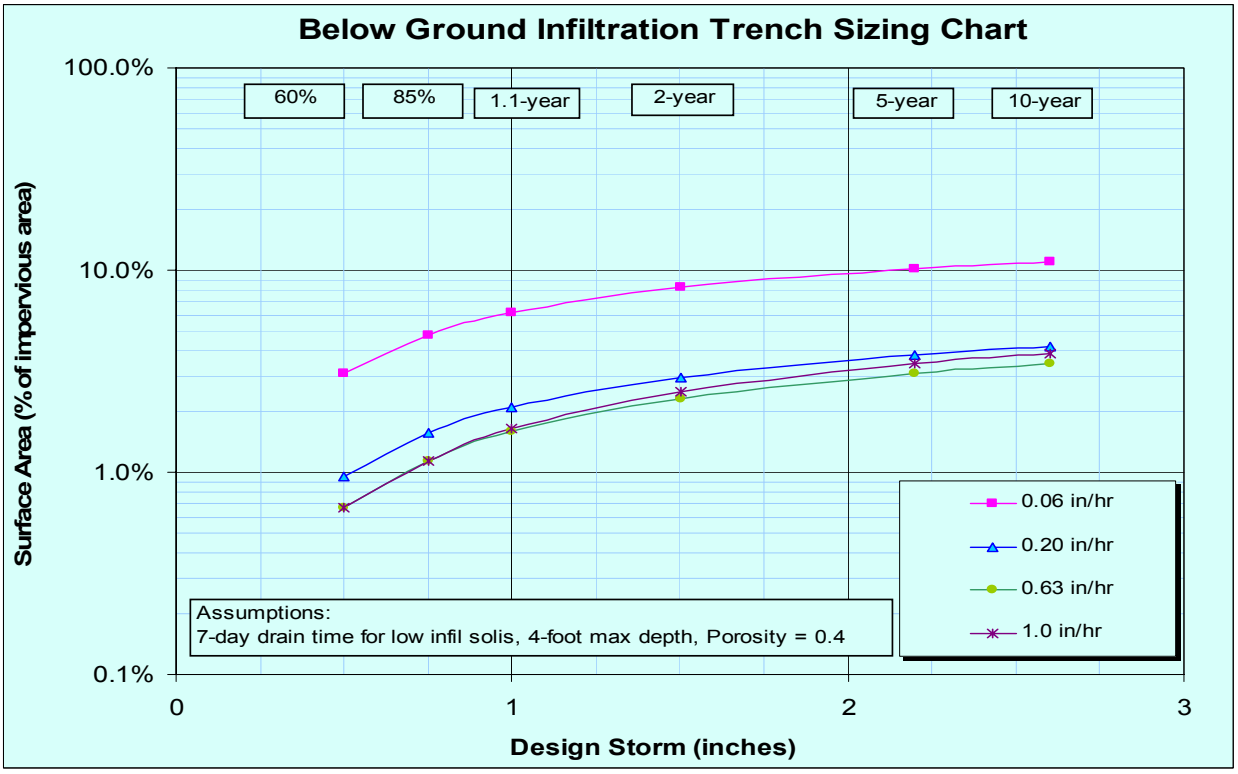


Figure 3 Results from the Sub-Surface Basin Sizing Table

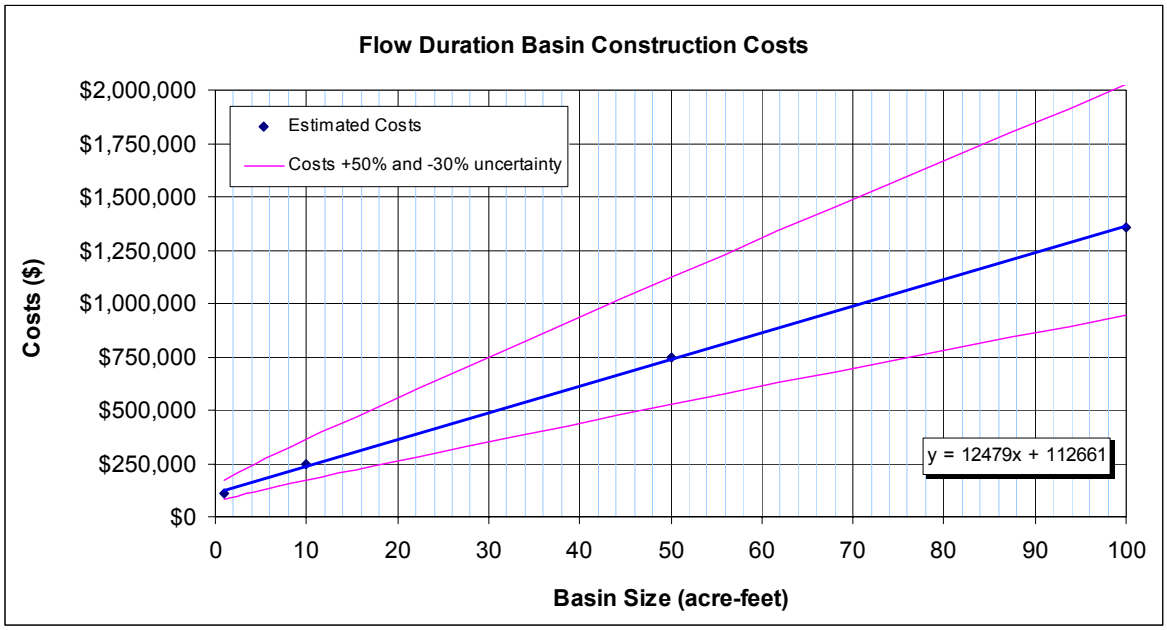


Figure 4 Estimated Cost for Surface Basin

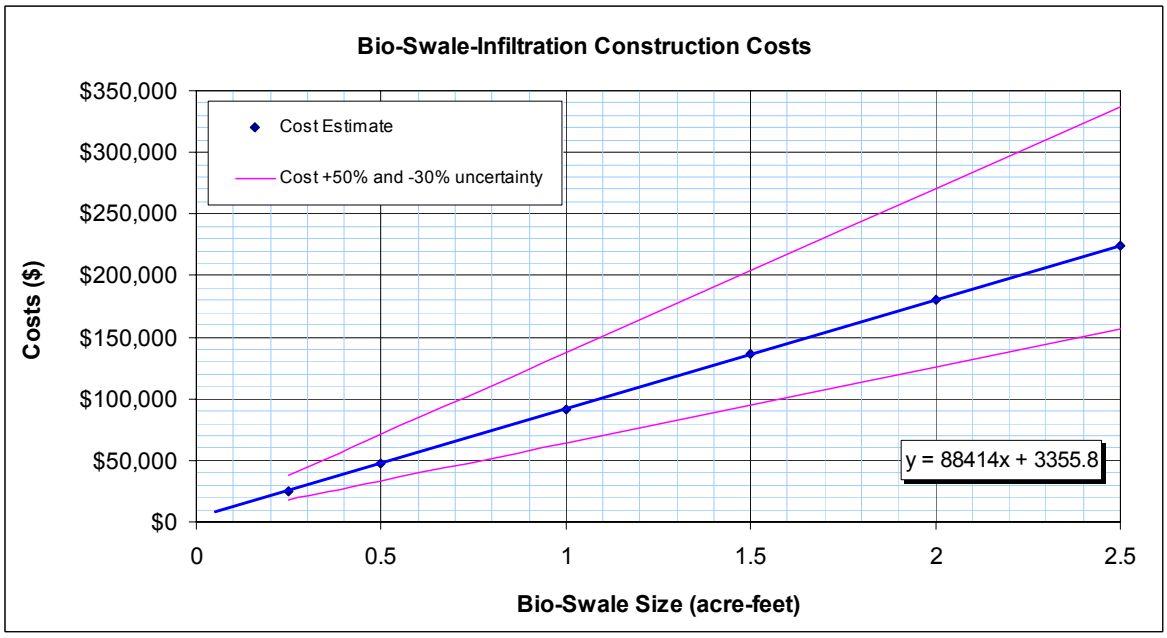


Figure 5 Estimated Costs for Sub-Surface Facility

Table 2. Pavement Areas, Street Drainage

	Example 1: Pavement Areas									
	Impervious Surface Area	Design Storm Rainfall Depth	Increased Runoff	Soil Permeability	Maximum Allowable Infiltration Basin Drain Time	Maximum Basin Depth	Detention Volume	Maximum Depth for Max Drain Time (plus direct rainfall)	Infiltration Basin Surface Area	Percent Total SA
	(ft^2)	(in)	(in)	(in/hr)	(days)	(ft)	(ac-ft)	(ft)	(acre)	
60%	131,040	0.5	0.31	0.06	3	4	0.078	0.36	0.216	7.2%
	131,040	0.5	0.32	0.2	3	4	0.080	1.20	0.067	2.2%
	131,040	0.5	0.32	0.63	3	4	0.080	3.78	0.021	0.7%
	131,040	0.5	0.32	1	3	4	0.080	6.00	0.020	0.7%
85%	131,040	0.75	0.48	0.06	3	4	0.120	0.36	0.334	11.1%
	131,040	0.75	0.53	0.2	3	4	0.133	1.20	0.111	3.7%
	131,040	0.75	0.55	0.63	3	4	0.138	3.78	0.036	1.2%
	131,040	0.75	0.55	1	3	4	0.138	6.00	0.034	1.1%
1.1-yr	131,040	1	0.62	0.06	3	4	0.155	0.36	0.432	14.4%
	131,040	1	0.71	0.2	3	4	0.178	1.20	0.148	4.9%
	131,040	1	0.76	0.63	3	4	0.191	3.78	0.050	1.7%
	131,040	1	0.79	1	3	4	0.198	6.00	0.050	1.6%
2-yr	131,040	1.5	0.83	0.06	3	4	0.208	0.36	0.578	19.2%
	131,040	1.5	0.99	0.2	3	4	0.248	1.20	0.207	6.9%
	131,040	1.5	1.11	0.63	3	4	0.278	3.78	0.074	2.4%
	131,040	1.5	1.2	1	3	4	0.301	6.00	0.075	2.5%
5-yr	131,040	2.2	1.03	0.06	3	4	0.258	0.36	0.717	23.8%
	131,040	2.2	1.28	0.2	3	4	0.321	1.20	0.267	8.9%
	131,040	2.2	1.49	0.63	3	4	0.374	3.78	0.099	3.3%
	131,040	2.2	1.65	1	3	4	0.414	6.00	0.103	3.4%
10-yr	131,040	2.6	1.11	0.06	3	4	0.278	0.36	0.773	25.7%
	131,040	2.6	1.41	0.2	3	4	0.353	1.20	0.295	9.8%
	131,040	2.6	1.66	0.63	3	4	0.416	3.78	0.110	3.7%
	131,040	2.6	1.87	1	3	4	0.469	6.00	0.117	3.9%

Table 3. Roof Drainage, patios, and walkways.

	Below Ground Storages w/Rock Fill							Porosity of gravel ->	0.4	
	Impervious Surface Area	Design Storm Rainfall Depth	Increased Runoff	Soil Permeability	Maximum Allowable Infiltration Basin Drain Time	Maximum Storage Depth	Req'rd Detention Volume	Maximum Depth for Max Drain Time (plus direct rainfall)	Infiltration Basin Surface Area	Percent Total SA
	(ft^2)	(in)	(in)	(in/hr)	(days)	(ft)	(ac-ft)	(ft)	(acre)	
60%	1,950	0.5	0.31	0.06	7	4	0.001	0.84	0.001	3.1%
	1,950	0.5	0.32	0.2	7	4	0.001	2.80	0.000	1.0%
	1,950	0.5	0.32	0.63	7	4	0.001	8.82	0.000	0.7%
	1,950	0.5	0.32	1	7	4	0.001	14.00	0.000	0.7%
85%	1,950	0.75	0.48	0.06	7	4	0.002	0.84	0.002	4.8%
	1,950	0.75	0.53	0.2	7	4	0.002	2.80	0.001	1.6%
	1,950	0.75	0.55	0.63	7	4	0.002	8.82	0.001	1.1%
	1,950	0.75	0.55	1	7	4	0.002	14.00	0.001	1.1%
1.1-yr	1,950	1	0.62	0.06	7	4	0.002	0.84	0.003	6.2%
	1,950	1	0.71	0.2	7	4	0.003	2.80	0.001	2.1%
	1,950	1	0.76	0.63	7	4	0.003	8.82	0.001	1.6%
	1,950	1	0.79	1	7	4	0.003	14.00	0.001	1.6%
2-yr	1,950	1.5	0.83	0.06	7	4	0.003	0.84	0.004	8.2%
	1,950	1.5	0.99	0.2	7	4	0.004	2.80	0.001	2.9%
	1,950	1.5	1.11	0.63	7	4	0.004	8.82	0.001	2.3%
	1,950	1.5	1.2	1	7	4	0.004	14.00	0.001	2.5%
5-yr	1,950	2.2	1.03	0.06	7	4	0.004	0.84	0.005	10.2%
	1,950	2.2	1.28	0.2	7	4	0.005	2.80	0.002	3.8%
	1,950	2.2	1.49	0.63	7	4	0.006	8.82	0.001	3.1%
	1,950	2.2	1.65	1	7	4	0.006	14.00	0.002	3.4%
10-yr	1,950	2.6	1.11	0.06	7	4	0.004	0.84	0.005	11.0%
	1,950	2.6	1.41	0.2	7	4	0.005	2.80	0.002	4.2%
	1,950	2.6	1.66	0.63	7	4	0.006	8.82	0.002	3.5%
	1,950	2.6	1.87	1	7	4	0.007	14.00	0.002	3.9%

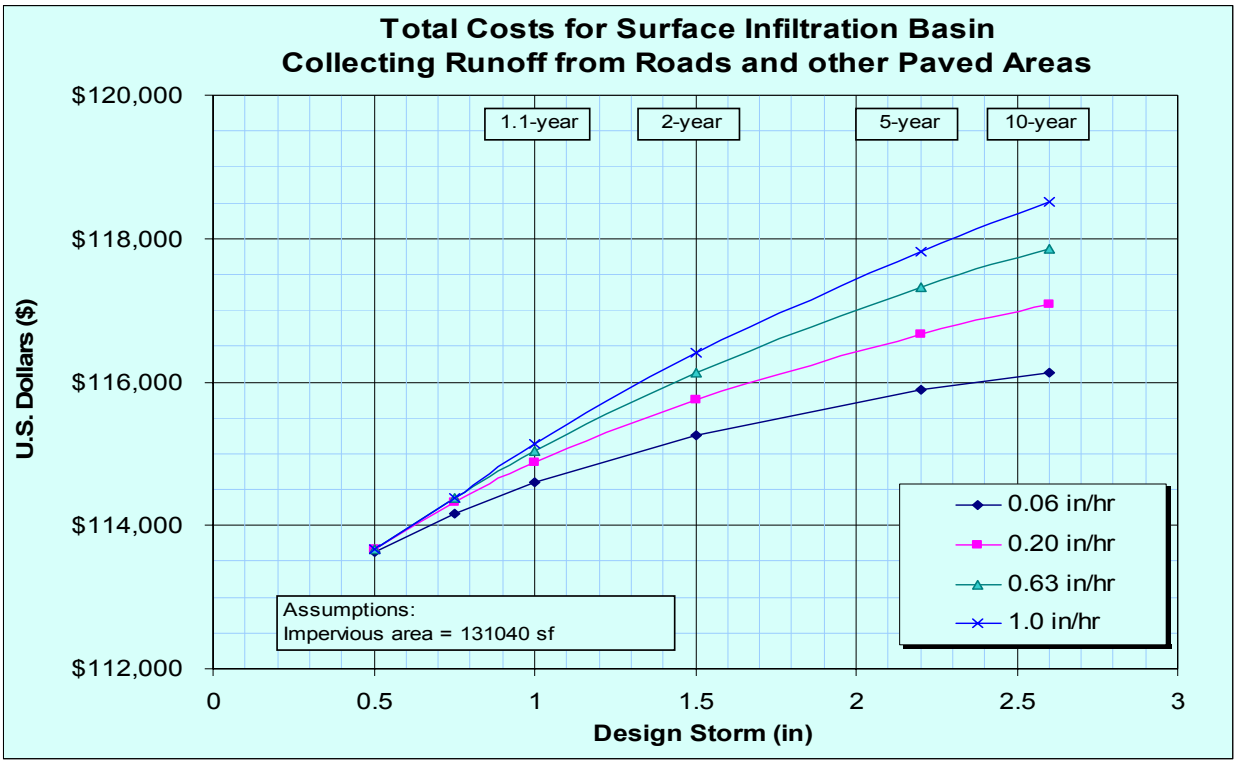


Figure 6

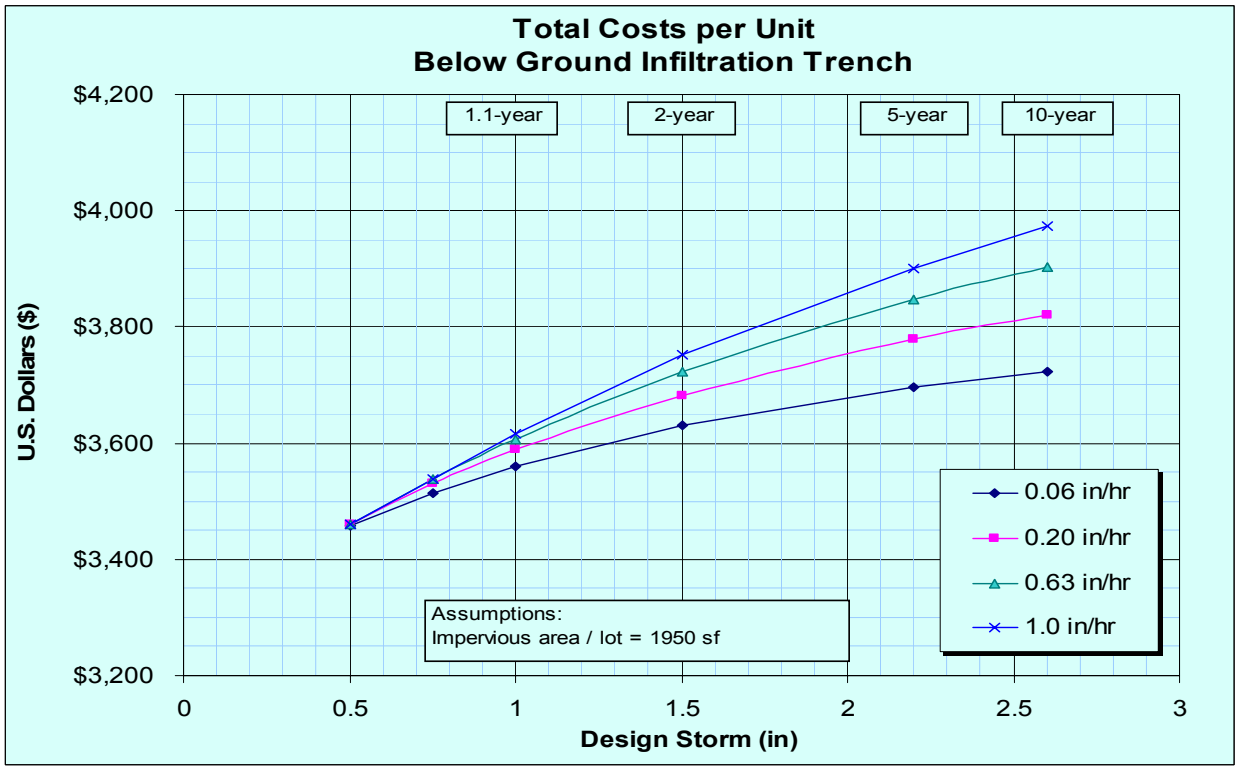
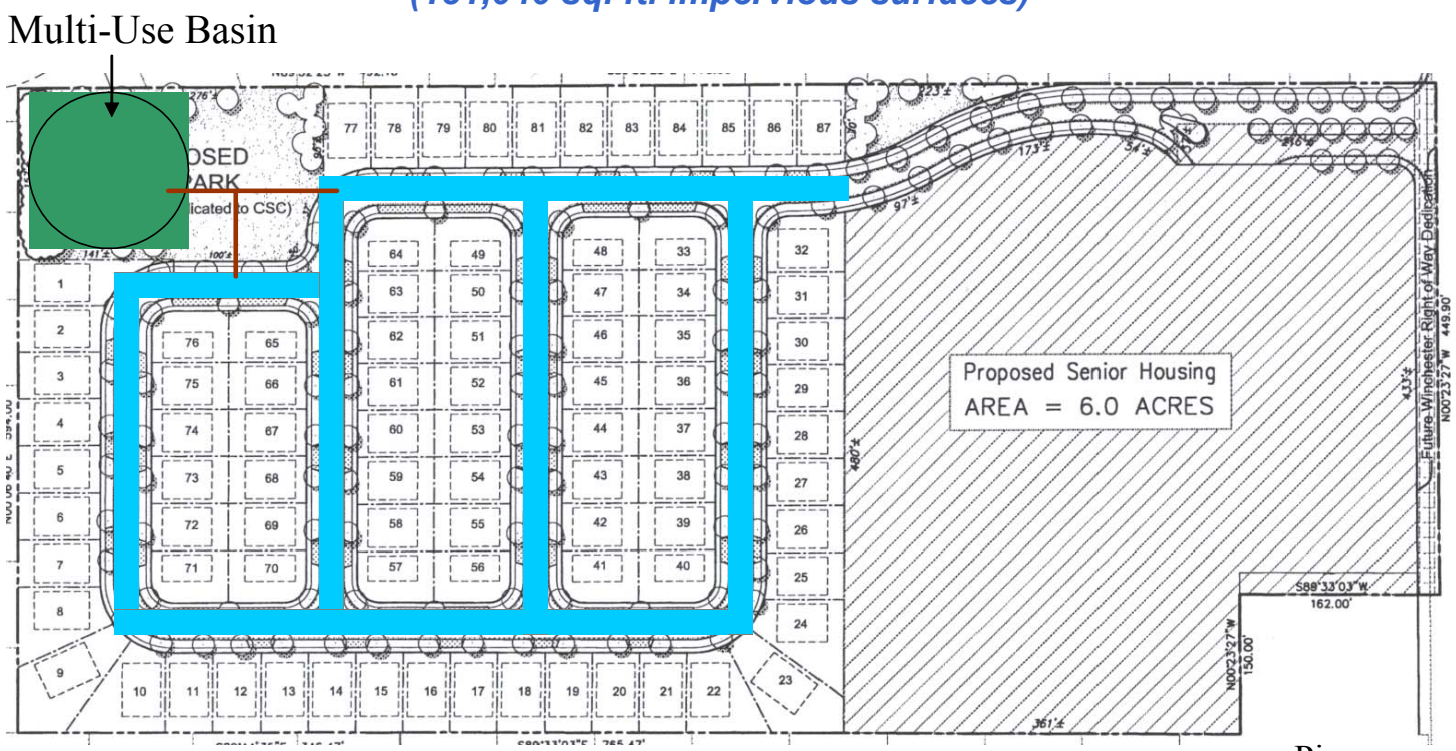


Figure 7

Residential Subdivision

Treatment BMP for Streets and Paved Surfaces

(131,040 sq. ft. impervious surfaces)



$I_f = 0.2$ in/hr

Surface Infiltration Basin

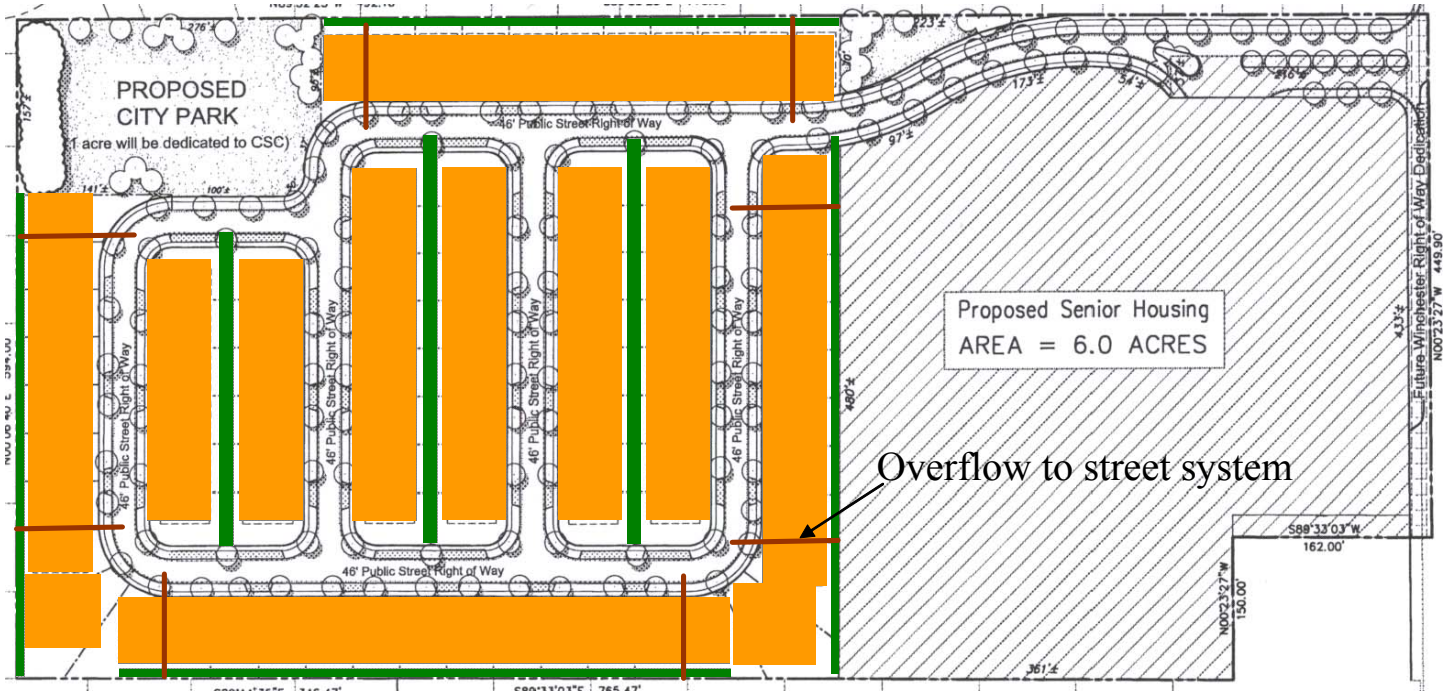
(12,850 sq.ft., 0.295 ac, 10% of impervious area)

Figure 8

Residential Subdivision

Treatment BMP for Roof Tops & Patio's

(1950 sq. ft. impervious surface per lot)



$I_f = 0.2$ in/hr

Pipe

Bio-Swale

Backvard Bio-Swales

(2'x40' = 80 sq.ft. per lot, 4% of impervious area)

Figure9

DRAFT

T E C H N I C A L

M E M O R A N D U M # 7

TO: HMP Onsite Management Measures Subgroup
Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP)

FROM: GeoSyntec Consultants and SCVURPPP Staff

DATE: April 1, 2004 (REVISED DRAFT)

SUBJECT: **Flow Duration Control Example**

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1 Introduction

Technical Memorandum #3 – Draft Hydromodification Control Standard, Performance Criteria, and Implementation Options, includes a hydromodification management strategy centered on maintaining the pre-project flow duration curves for runoff from the project site. Matching flow duration of runoff from project areas is an effective way to manage hydromodification and maintain the in-stream erosion potential.

The draft standard requires that the pre-project flow duration curves be maintained for all flows between a lower limit and an upper limit. Flow duration control is similar to volume control but is more effective for managing hydromodification than volume control alone. Hydrograph matching improves on volume control, but is for a single discrete event as opposed to the full record of flows. Matching flow duration curves also maintains runoff volume for the full distribution of flows from the lower and upper limit. Unlike volume control or hydrograph matching, flow duration control considers the full multi-year discharge record, including antecedent conditions and back-to-back storms. The objective of this analysis is to apply the flow duration control strategy as described in TM#3 (Section 3.2.3). This memo discusses the application of this strategy on a 716-acre residential development in Thompson Creek subwatershed (Figure 1) and for a 274-acre development in southern California (Figure 2). The flow duration results of these two projects are compared.

Flow duration control requires that the increase in surface runoff volume created by the installation of impervious surfaces be retained on-site and discharged at less than the critical flow of the stream, Qc (the flow rate that begins to cause erosion). For this example, Qc is assumed to be zero because it is assumed that the developer does not have information about the Qc of the stream. As a result, the BMPs considered must be able to capture and store a design volume and then dispose of this volume through infiltration and/or evapotranspiration. This may not be

feasible for sites in Santa Clara Valley, and options that allow a slow discharge of water from the BMP (a proposed limit is 10% of the 2-year storm runoff for the site) may need to be considered.

The type of facility considered and evaluated is a surface basin including detention and outlet control, storage and infiltration. Infiltration is assumed to take place within the flow duration basin, as opposed to a separate infiltration facility. By-passes and low-impact development (LID) site design strategies that minimize the increase in surface runoff should be considered; however, this example is focused on sizing and costs of the flow duration basin without LID strategies.

2 Flow Duration Basin Configuration

The flow duration approach involves: 1) simulating the runoff from the project site, pre- and post-development, using a continuous rainfall record; 2) generating flow-duration curves from the results; and 3) designing a flow control facility such that when the post-development time series of runoff is routed through the facility, the discharge pattern matches the pre-development flow-duration curve. In this example, 50 years of rainfall data from the San Jose Airport gage is used to generate the runoff time series. The continuous runoff time series from the future development is routed through a detention basin that diverts and retains a certain portion of the runoff. This portion to be retained is essentially the increase in surface runoff volume created between a pre-project and post-project condition. This captured runoff is assumed to be infiltrated in the basin for this example.

The flow duration basin is designed to have two pools (Figure 3), a low flow pool (Zone A) and a high flow pool (Zone B). The low flow pool is designed to capture the difference in volume of runoff between the pre- and post-development conditions. It will also capture small to moderate size storms, the initial portions of larger storms, and dry weather flows. The high flow pool is designed to store and release higher flows to maintain, to the extent possible, the pre-project runoff conditions. The flow duration basin also serves as a water quality treatment facility and can be designed to treat dry and wet weather flows using a combination of extended detention and natural treatment processes. Most dry weather "nuisance flows" will also infiltrate in the basin.

The flow duration basin is sized using an iterative process of adjusting basin storage as well as selecting and adjusting orifice sizes in the outlet structure. The low flow pool within the basin is initially sized to capture the increase in runoff volume that is generated from the impervious surfaces. This capture volume is dependent on the development characteristics, the soil types, and the magnitude of increase in runoff volume created by the proposed development. Previous analyses have shown that area requirements have less to do with the range of storms selected for management and more to do with site and development characteristics.

Once the lower pool is sized to capture the correct volume of runoff, the upper pool of the basin is sized to detain and discharge larger flows through a specific set of orifices in such a way as to reproduce the flow duration curve. The number, diameter, and elevation of these orifices are determined by a trial and error approach first developed in Western Washington (King County, 1998). The combination of sizing the lower portion of the basin and the upper portion to detain and discharge high flows has the affect of capturing the correct volume of runoff and matching the pre-development distribution of hourly flows.

In this example, the basin functions as an infiltration basin such that the excess runoff volume is infiltrated. Alternatively, a by-pass pipeline could carry the excess runoff to a safe discharge location or other infiltration site, if feasible. Combining flow duration and infiltration into a single facility reduces the overall land requirements for stormwater management.

The outlet structure is designed to reproduce the pre-developed flow duration (runoff histogram such as that in Figure 5) using orifice diameter and elevation above the bottom of the basin. Figure 4 illustrates the outlet structure. The number, size and placement will vary from basin to basin depending on project conditions. A weir and orifice combination could also be designed to accomplish the same level of control.

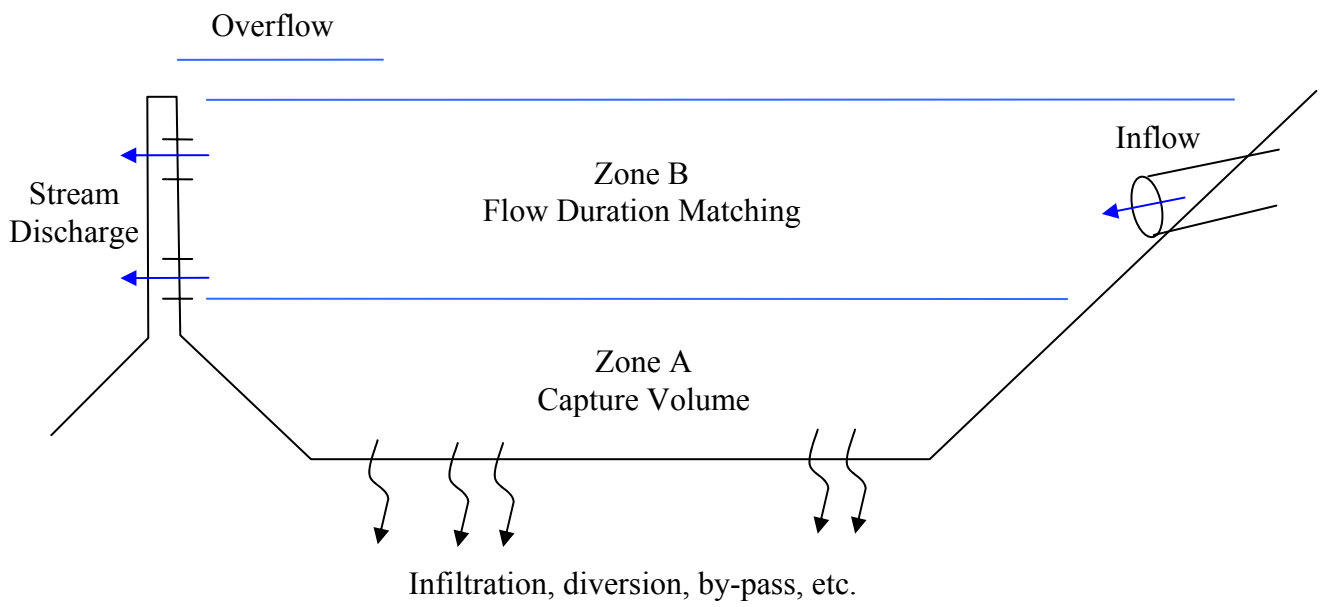


Figure 3. Generalized Configuration of Flow Duration Basin

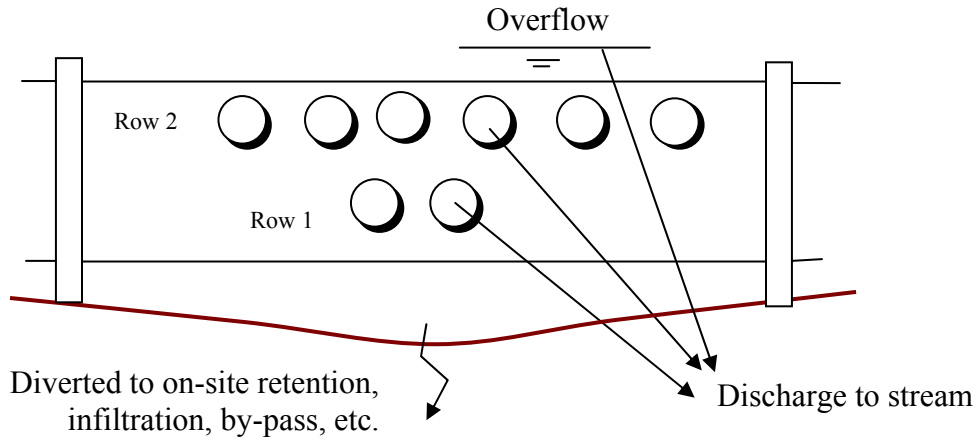


Figure 4. Generalized Configuration of Outlet Structure

3 Thompson Creek Example

The Lower Silver – Thompson Creek subwatershed (approximately 42 square miles) is located within the Coyote Creek watershed, the largest watershed in the Santa Clara Basin, with some of its area partially in the City of San Jose and partially in unincorporated Santa Clara County.

Thompson Creek originates in the Diablo Range foothills at an elevation of about 2,300 feet and presently flows northerly to its confluence with Lower Silver Creek near the Eastridge Shopping Center at an elevation of approximately 125 feet. Its tributary streams include, from north to south: Quimby Creek, Fowler Creek, Evergreen Creek, Yerba Buena Creek, and Cribari Creek. Thompson Creek discharges to Lower Silver Creek. The Thompson Creek subwatershed encompasses about 17.5 square miles.

This example sizes HMP controls for an existing 716-acre residential development between Yerba Buena Creek and upper Thompson Creek, as if HMP controls were required when the project was constructed (i.e., the pre-project condition at the site is assumed to be the "pre-urban" condition simulated in the Thompson Creek modeling effort). This area now has 71% impervious cover. Existing land use is predominantly single family residential, streets, and park and golf course properties. The infiltration rate for soils at the basin site is assumed to be 0.2 inches/hour.

3.1 Results

This section summarizes the flow duration sizing results for the Thompson Creek example.

Figure 5 presents the results of this method illustrating the pre-project flow duration curve, post-project (future) and post-project curves with controls at the site outlet point, using a 50-year record simulation results from the Thompson Creek hydrologic model. The post-development curve illustrates that the effect of development is to increase the duration of flows; that is, the flow duration curve moves to the right indicating that both volume and duration of flows increase. Also note that this is a logarithmic scale on the horizontal axis, so small changes along the axis may indicate large changes in volume and duration. The effect of the BMP is to reduce the durations to more closely replicate the existing condition. This means that the magnitude of hourly runoff and the number of hours (duration) that flows occur at those magnitudes are nearly the same between pre- and post-project conditions.

Note that for the simplicity of this example, the analysis assumes that no individual on-site control measures or low impact site design techniques were used. These options would be strongly encouraged if an actual project like this was implementing HMP requirements.

For the 716 acres of development, the resulting basin characteristics are as follows:

- Basin volume is 120 acre-feet.
- Basin surface area is 15 acres (2% of the sub-catchment area), assuming a maximum basin depth of 10-feet and 3:1 side slopes. (Note: if the basin depth were limited to 4 feet, the required surface area would be about 35 acres, or 5% of the drainage area).
- Fifteen orifices having diameters of 9-inches each discharge to the creek.

When comparing the pre-project curve to the post-project with BMP curve, the results shown in Figure 5 would be considered a very good match. Design of flow duration basins is somewhat of a trial and error process, and some design configurations match pre-project curves better than others. Western Washington allows no more than a 10% deviation between the curves over no more than 10% of the curves' length, or range of flows.

The shape of the post-project with BMP curve is dependent on the type of outlet structure. In our case, we are using orifice holes in a headwall, following Western Washington's guidelines. Orifice holes create the convex shaped sections of the curve, where each section represents the change in flows from adding the next row of orifice holes. In the example in Figure 5, there are three convex curve sections between zero and 51 cfs. Above 51 cfs, flows are spilling over the overflow structure.

3.1.1 Cost Curves

Figure 6 shows the construction cost curves for a surface basin infiltration facility. This curve was derived from well detailed cost spreadsheets, including such elements as site preparation, earth work, structures, piping, re-vegetation and so forth. Costs do not include soil disposal fee, hauling, or contaminated soil testing, mitigation, or disposal. Detailed spreadsheets are available for review. Land costs are <u>not</u> included per direction of the Subgroup.

The cost estimates presented in this section represent a planning level cost estimate, and represent an accuracy of + 50% to -30%.

The Thompson Creek example construction costs would be approximately $1.6 Million for the basin itself. This does not include pipelines, design, environmental documents, or land costs.

The $1.6 Million construction cost is equivalent to $2,250 per acre of development area. Assuming 4 houses per acre in this location, this flow duration basin construction costs $563 per lot, excluding land costs.

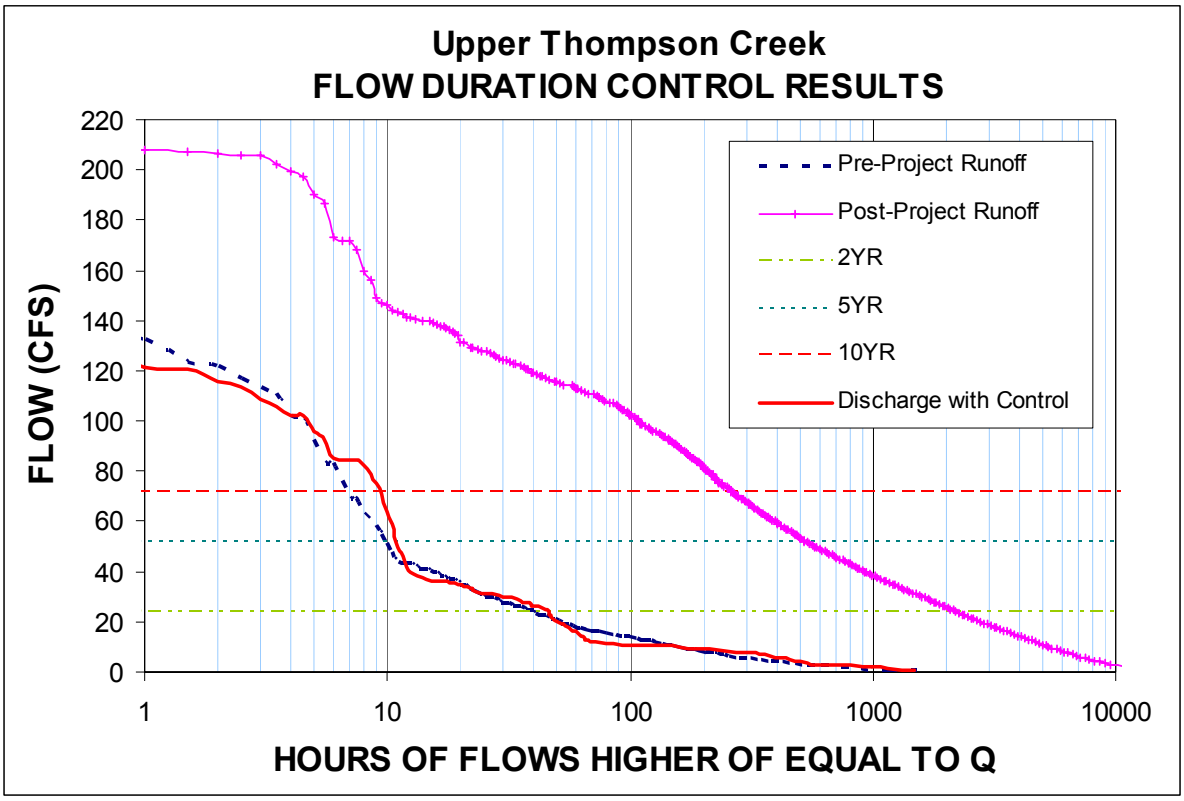


Figure 5. Flow Duration Curves for Thompson Creek Example

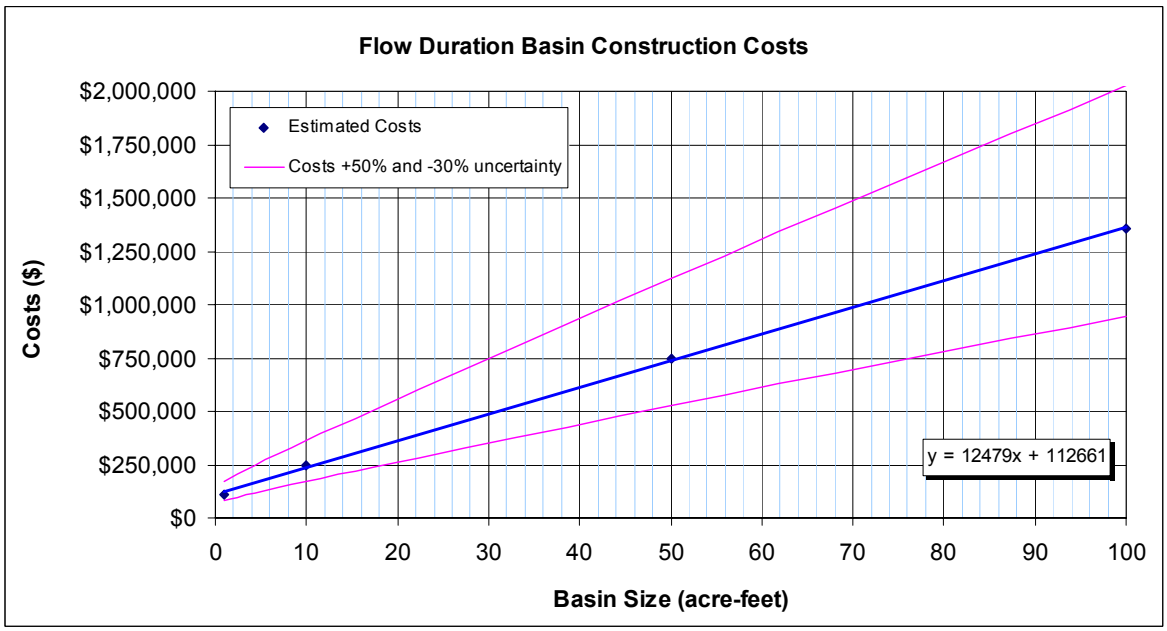


Figure 6. Estimated Cost for Surface Basin

4 Discussion of Gobernadora Creek (Orange County) Example

GeoSyntec is responsible for preparing the Water Quality Management Plan (WQMP) for a project in Southern California to support planning efforts of the San Juan and Western San Mateo Watersheds. The purpose of the WQMP is to assess potential effects of the proposed project on water quality, water balance, and hydromodification and recommend control measures to address potential impacts. The Southern California project consists of nine sub-watersheds evaluated separately, one of which is Gobernadora Creek subbasin.

One of the objectives of the this project is to strategically layout the development to protect natural resources and reserve certain areas, like high infiltration soils, for stormwater management facilities. The results discussed in this section show the effect of strategic site planning when compared to the Thompson Creek example. The reader will see that the difference between the pre- and post-project flow duration curves is small when compared to Thompson Creek example.

The Gobernadora subbasin is an elongated valley that is characterized by deep alluvial deposits in the canyon floor. Hill slopes and ridges are overlain by hardpan or exposed rock. The proposed development addresses approximately 2,156 acres. Approximately 881 acres would remain as open space, with the remainder being developed into estate, single, and multi-family residential housing, golf resort, and transportation.

The Gobernadora subbasin has ten sub-catchments. The flow duration basin sizing results for Sub-catchment 3 is discussed here. Sub-catchment 3 consists of 274 acres of single family residential area and transportation corridors, at 44 percent imperviousness. The pervious area is mostly grassland and riparian area.

4.1 Results

Figure 7 presents the results for a Gobernadora Creek example. The post-project flow duration curve with BMPs closely matches the pre-project curve, with better accuracy than the curves shown in Figure 5. This means that the magnitude of hourly runoff and the number of hours (duration) that flows occur at those magnitudes are nearly the same between pre- and post-project conditions.

For the 274 acres of development, the resulting basin characteristics are as follows:

- Basin volume is 15 acre-feet.
- Basin surface area is 3.7 acres (or 1.4% of the sub-catchment area) assuming maximum basin depth of 5 feet and 3:1 side slopes.
- 5 orifices having diameters of 7 to 20-inches each discharge to the creek.

Of all the flow duration basins sized for Gobernadora, the range of land area required for the facilities ranged from 1.2 to 1.4 percent of their total sub-catchment areas. It should be noted that the soil infiltration rate where the flow duration basins are located were assumed to have 1

inch/hour infiltration rates. The range of percent imperviousness is 40 to 44%. One sub-catchment has 50% imperviousness with development on sandier soils. The resulting basin is 2.8% of its catchment area.

When comparing Figures 5 and 7, notice that the difference between the pre- and post-project conditions for development in Gobernadora Creek watershed is much smaller than the differences observed in Thompson Creek subwatershed. In Gobernadora Creek watershed, residential development was located on low infiltration clay soil and bedrock, reserving the high infiltration soils for stormwater management BMP's.

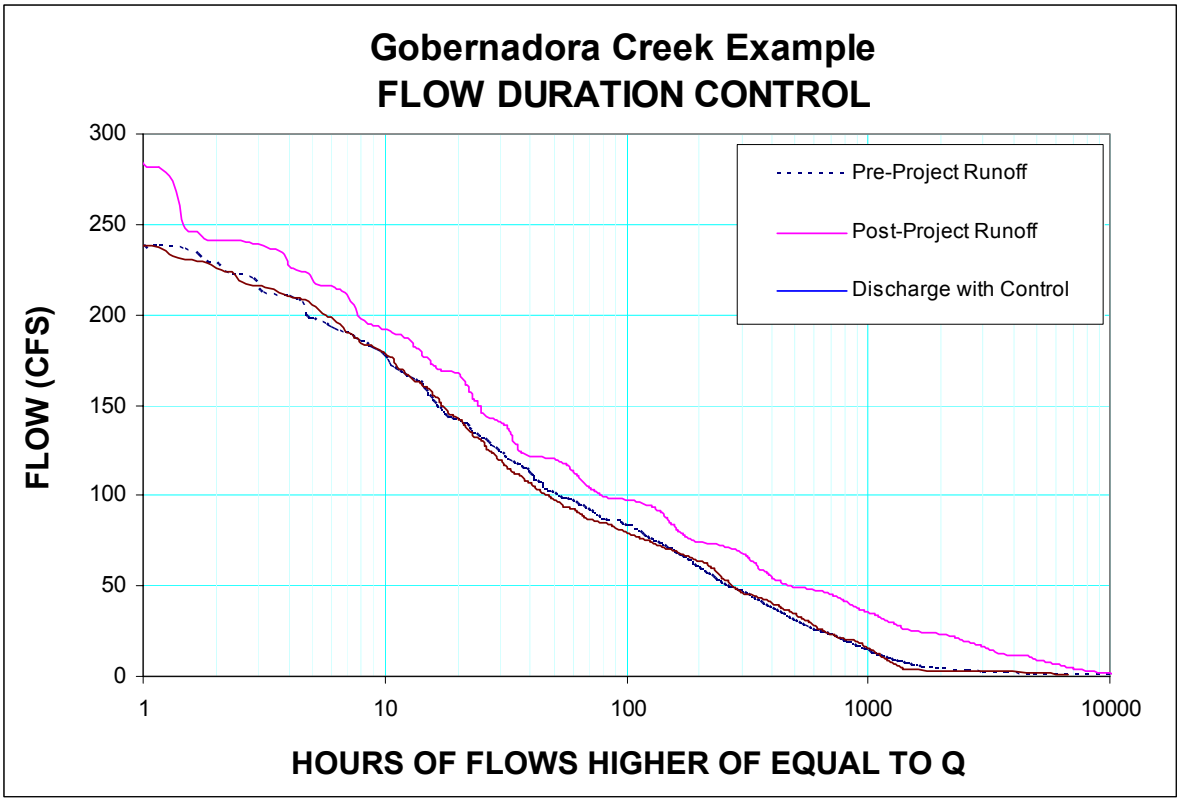


Figure 7. Flow Duration Curves for Gobernadora Creek Example

5 Hydrograph Matching (Volume Control) Effectiveness

In TM #5 – Volume Control, volume control as a strategy was found not to be very effective at controlling hydromodification impacts, because of the differences in erosive power for the same volume at higher flows when compared to lower flows. The memo concluded that hydrograph matching may significantly improve the results by not only controlling runoff volume, but also controlling the shape of the hydrograph and preventing higher flows from occurring.

This section presents and discusses an effectiveness evaluation of hydrograph matching when compared to the full 50-year flow duration matching option using the 716-acre sub-catchment in Thompson Creek. Matching the full flow duration curve is the most effective option at maintaining the pre-development erosion potential.

This analysis compares the effectiveness of different sizing strategies at reproducing the pre-development flow duration curve. Four control basins were sized and their outlets designed using three independent discrete storm sizes (2-year, 10-year, and 50-year), and one sized using the full 50-year hour record of runoff generated by the HEC-HMS hydrologic model. The basins for the 2-, 5- , and 10-year storms were sized and designed to match the discrete pre-project hydrographs for those storms, such that both pre-project volume and shape are maintained.

Figure 7a, 7b and 7c present the results of matching the discrete event hydrographs. Both the storm hydrographs and flow duration curves are plotted for each storm size. In all cases, the storm hydrographs were matched closely in volume and shape. The basin discharge hydrograph lags the pre-project hydrograph by a couple of hours, but this is not a problem. The result for matching the full flow duration curve was presented in Figure 5 above.

The discrete hydrograph basins are designed to capture and retain the difference in volume between the pre- and post-project hydrographs. It is assumed that this capture volume is slowly infiltrated, diverted to another disposal facility, or discharged at a very slow rate.

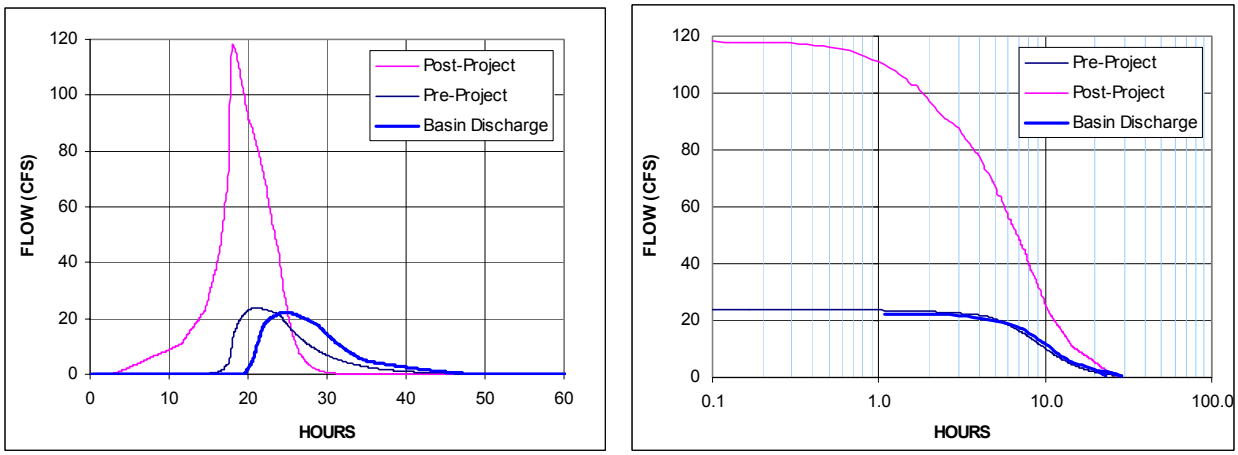


Figure 7a. 2-Year Discrete Hydrograph Matching Results

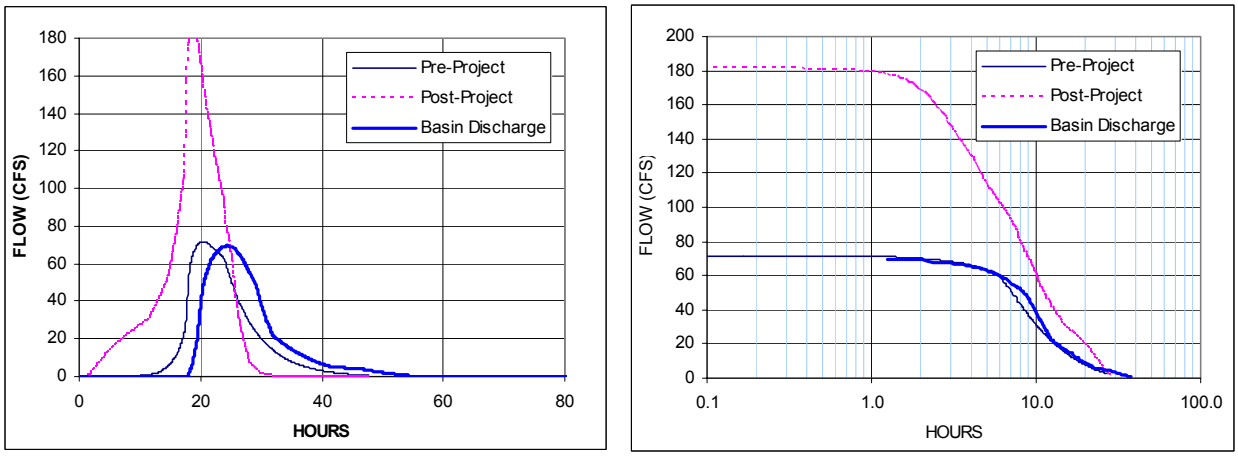


Figure 7b. 10-Year Discrete Hydrograph Matching Results

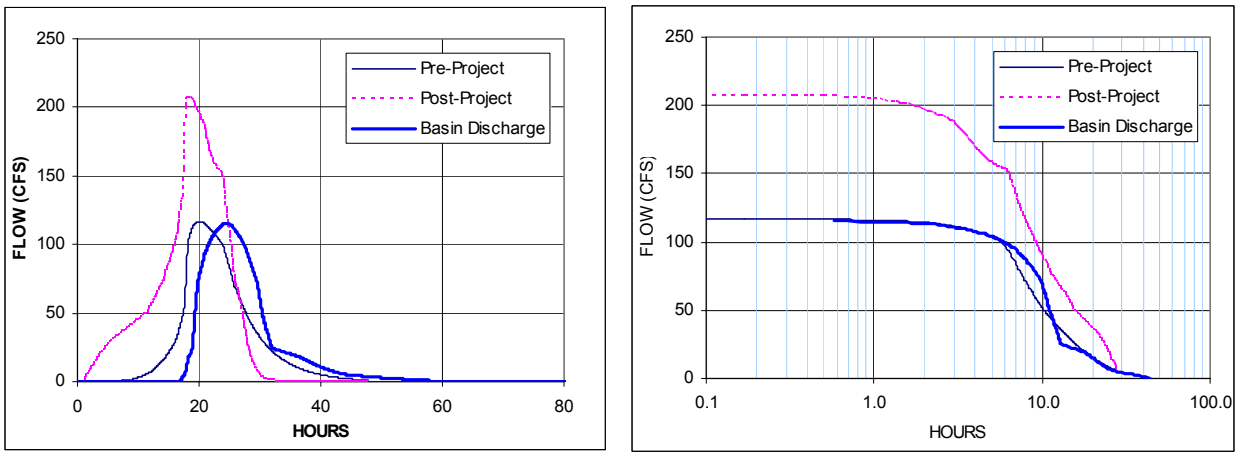


Figure 7c. 50-Year Discrete Hydrograph Matching Results

In TM #3 – Draft Standard, one of the control options is intended to provide a simple discrete event based approach that could replicate the control effectiveness of continuous simulation analysis and flow duration matching. The next test that was conducted was to evaluate the effectiveness of the discrete basins by routing the full 50-year flow record through them and plotting the resulting flow duration curves.

Figure 8 below plots the resulting flow duration curves for each of the four sizing criteria, 2-year, 10-year, 50-year, and the full 50-year time series. As shown, the 50-year time series approach closely reproduces the pre-project flow duration curve. The three basins sized using discrete events do not match the pre-project curve and would result in stream channel erosion if a large percentage of the watershed development was designed using discrete events.

Also shown in Figure 8 are the resulting basin sizes. Basin sizes range from 71 acre-feet to 155 acre-feet for the 2-year and 50-year storms, respectively. Note that for this example, the size of the 50-year time series basin (116 acre-feet) is close to the basin size using the 10-year storm (113 acre-feet). Also recall that these basins are being sized to capture the difference in runoff volume between pre- and post-project conditions, not the entire storm event.

Generally, the end points of the flow duration curves in Figure 8 reflect the different basin sizes. For example at the low flow end, the 2-year basin releases more small flows to the stream than the 10-year or 50-year basins, all of which release about 5 to 10 times more than the pre-project condition. The high flow end shows the affect of large storm peak flows. The 2-year basin is small such that large storms spill over the overflow uncontrolled. The 10-year and 50-year basins can provide peak control of the larger events and closely approximate pre-urban peaks. The middle portion of the flow duration curves is primarily influenced by the outlet structure. The shape of the discrete basin curves reflect the design of the outlet structure, where each break in the curve shows the affect of stormwater discharge through orifice holes at higher elevations.

One interesting observation is that the 50-year discrete basin does not match the full time series curve even though its basin size is larger. The reason for this is in the design of the outlet structure, which was designed to reproduce the 50-year hydrograph alone.

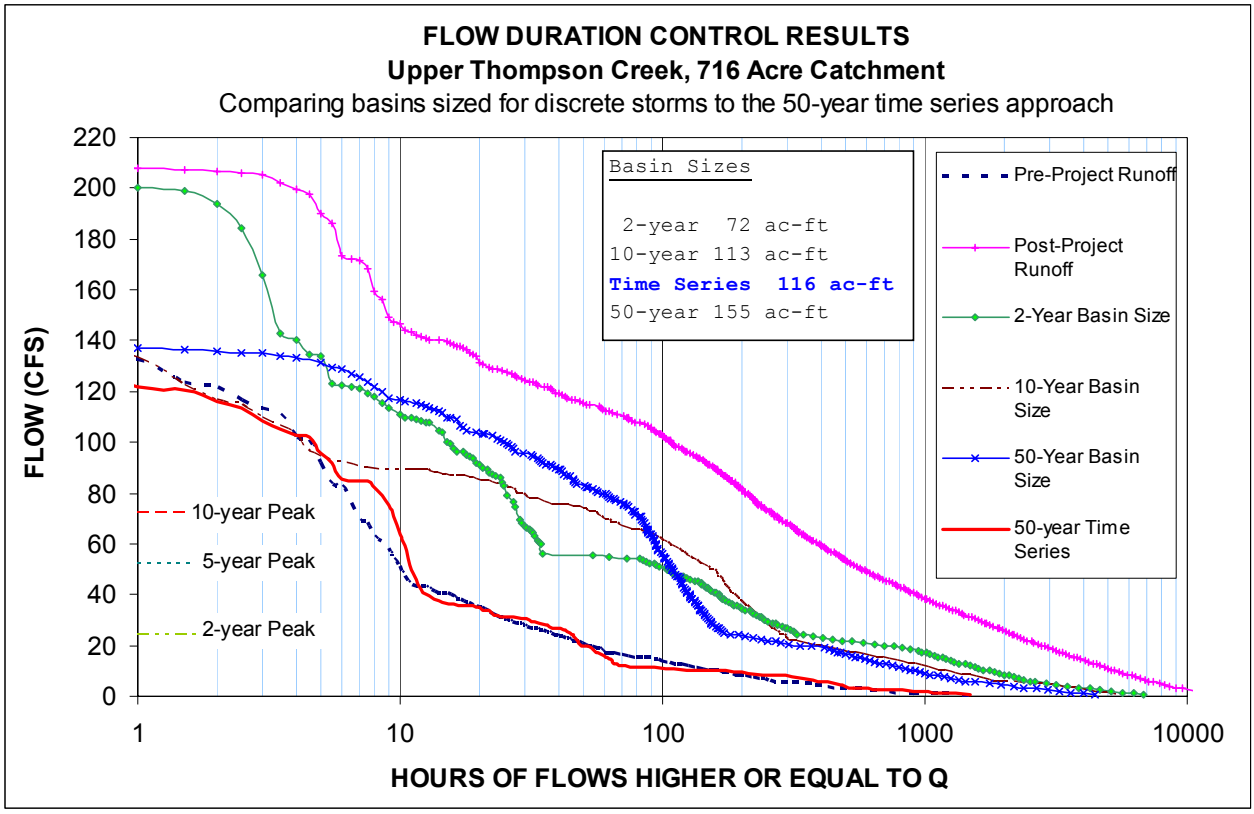


Figure 8. Comparison of Different Basin Sizing Criteria

6 Conclusions

Sizing hydromodification control basins using discrete events does not closely reproduce the pre-project flow-duration conditions and will lead to stream channel erosion if widely used throughout a catchment.

The required area for flow duration basins seems to be between 2 to 5 percent of the contributing catchment area. Where projects have good soil infiltration rates and utilize low-impact development strategies, less land area is required (1 to 2 percent).

If infiltration cannot be achieved in the flow duration basin and pre-treatment is required before infiltration, then a separate infiltration facility may be required. In this case, additional land area would be required. Based on the examples completed by GeoSyntec, about twice as much land area would be required if flow duration and infiltration had to be done in separate facilities.

DRAFT

T E C H N I C A L

M E M O R A N D U M # 8

TO: HMP Onsite Management Measures Subgroup
Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP)

FROM: GeoSyntec Consultants and SCVURPPP Staff

DATE: April 29, 2004

SUBJECT: **Sizing Flow-Duration Controls for a Small Development Project in San Jose**

This memorandum summarizes the application of the flow duration basin sizing technique to a small 12-lot subdivision in San Jose. The site is located near Alum Rock, and discharges to South Babb Creek through the storm drain system.

The objective was to apply the flow duration methodology to a small size project in San Jose that is typical of developments expected in the future. The first example illustrating the design of a flow duration basin was completed for a 716-acre development in the Thompson Creek subwatershed. The site is located between Yerba Buena Creek and Thompson Creek in the Village Parkway area and is predominantly single family residential, streets, park and a golf course. The Subgroup requested the application of the flow duration method on a smaller development.

A second objective was to test the effectiveness of the Rational formula and SCS curve number method applied in continuous mode to size the flow-duration basin, compared to the HEC-HMS continuous model results, to see if a simpler analytical method could be used. The justification for this second objective is described later in this memorandum.

1 Project Description

This section briefly summarizes relevant project information required for modeling purposes.

- The catchment area is 3.6 acres, with twelve residential lots of 3,000 square feet each. Run-on from the adjacent hill slopes is assumed to be zero. According to the City of San Jose, a ditch will be constructed along the upper boundary of the property to collect hill slope runoff and route it around the development to the storm drain system.
- The estimated time of concentration (Tc) for the pre- and post- project condition is 5.5 and 5 minutes, respectively. However, HMS limits the time of concentration to one-half of time increment used in running the simulation, thus the modeled Tc = 7.5 minutes. The time increment used for this continuous hydrologic example is 15 minutes. More explanation is provided later in this memorandum.

- The pre-project and post-project percent imperviousness are estimated to be 9 and 45 percent, respectively. The post-development impervious surface includes paved roads and driveways, patios and roofs of the 12 houses.
- The soil type in this area is classified as "D" type soils using the SCS classification system. The infiltration rates for this soil type range from 0.06 to 0.20 in/hr. For hydrologic modeling purposes, an average infiltration rate of 0.13 in/hr was used.

2 Discussion

Time of Concentration

Because the project size is small, the time of concentration, Tc (i.e., the time for runoff to travel from the farthest point in the catchment to the flow duration basin), is also small. The time step of the hydrologic model should be one-half to one-third of Tc. However, file sizes for a time step of 2 or 3 minutes of continuous records for 50 years becomes unmanageable. This example was run with a time step of 15 minutes, which is the smallest manageable time step that can be used with 50-years of continuous record given the desk-top computing power available. The smallest permissible Tc within HEC-HMS, regardless of the time step, is 6 minutes.

Running the hydrologic model with a 15 minute time increment when the Tc is 7.5 minute is acceptable when the input rainfall record is hourly. Using hourly rainfall data means rainfall is constant for that hour. Even if the hourly value was subdivided into 3 minute data, the 3 minute values will be constant for that hour. Given constant rainfall data, with the exception of the first 7.5 minutes, the hydrograph will be constant. During the first 7.5 minutes, the hydrograph rises from zero to the peak flow at that rainfall magnitude. Runoff continues at a constant rate for the remainder of the hour, when the rainfall magnitude potentially changes. Running HMS with a time increment of 15 minutes with constant rainfall computes the average flow for that period, the same value derived with 3 minute constant data, after the first 7.5 minutes.

In studies done with the Western Washington Hydrology Model, dividing an hourly rainfall record into shorter intervals for a shorter time step does not make much difference in the sizing of flow control basins, especially considering the detention storage provided in the basin[1]. In addition, when a rainfall record with 15-minute data was applied to a project at SEATAC Airport, the resulting basin size was <u>smaller</u> that when hourly data were used, because the pre-development peak flows were higher and there was less difference between pre- and post-development flow duration curves. This is because short term peak intensities in the pre-development condition did not infiltrate into pervious areas. Based on this information, GeoSyntec and Program staff believe that the analyses for small sites are not affected by short times of concentration

.Infiltration Rate Assumptions

As described above, the infiltration rate for class "D" soils ranges from 0.06 in/hr to 0.20 in/hr. For modeling, the average infiltration rate was used. For basin sizing however, it is assumed that the basin will be constructed to maximize its infiltration performance. For example, the basin

[1] Personal communication with J. Brascher, Aquaterra Consultants, 4-21-04.

could be excavated below final grade of the basin and the soils amended to improve infiltration and minimize compactness. Infiltration at the basin location is assumed to be 0.20 in/hr, the upper limit of saturated hydraulic conductivity for class "D" soils.

Comparison of Analysis Methods

The Subgroup has concerns that continuous simulation modeling is a complicated method for developers to use (and agency staff to review), so it has requested GeoSyntec to look at using other analytical methods for estimating runoff, i.e. the Rational Method and Soil Conservation Service (SCS) Method. In order to create flow-duration curves and size basins with these methods, these methods were applied in a "continuous mode", i.e., runoff flows were estimated for each interval of rainfall in the 50-year rainfall record. It is assumed that the Rational, SCS and HEC-HMS methods have increasing accuracy (and level of detail) in predicting runoff, with the HEC-HMS model being the most accurate. For this reason, the HEC-HMS model was used to test the effectiveness of the Rational and SCS methods. Neither the Rational nor the SCS method accounts for routing, lag, or changing antecedent conditions. Routing and lag are insignificant for a small project such as this one; however, antecedent conditions (e.g., soil moisture before a storm event) can be important.

3 Results

Figure 1 compares the SCS and Rational method results to the HMS model results for pre-project and post-project conditions. The Rational formula predicts runoff using a simple linear relationship to rainfall. Every hour of rainfall produces an hour of runoff. Because every rainfall produces runoff, the pre- and post- total hours of runoff are always the same. The rational formula does not adequately reproduce the HMS flow duration curves.

The SCS curve number method more closely resembles the HMS flow duration curves (although the closeness shown in Figure 1 could be somewhat coincidental). The SCS method is capable of predicting the change in the number of hours of runoff and closely matches the HMS results. The post-project SCS results reasonably reproduce the post-project HMS flow duration curve.

The HMS model predicts the smallest hydrologic change and difference in pre- and post- flow duration curves and thus results in the smallest basin size. One likely reason is that the soil moisture stays fairly saturated throughout periods of frequent rainfall, predicting runoff closer to impervious conditions. On the other hand, the SCS method initial abstraction is re-set every storm event and does not track soil moisture.

Table 1 presents the resulting flow duration (FD) basin size information and other details for the Rational, SCS and HMS results. The HMS model predicts the smallest FD basin size, primarily because of the small difference in pre- and post- flow duration curves. The SCS basin is 20% larger than the HMS basin. The Rational basin is twice the volume estimated using the HMS method. Figure 2 presents the matched flow duration curves using the HMS results as an example.

The resulting area requirements range from 2.1% to 2.6% of the total project area (3.6 acres), or 4.6% and 4.8% of the DCIA (45% imperviousness). A water quality basin would be about 2% of the total project area. Basin depths range from 1.75 feet to 2.75 feet. During the sizing procedure, basin depth was limited to a maximum of 3 feet.

Table 1 also lists the basin volume as a function of the inches over the project area. Values for HMS and SCS range from 0.35 to 0.42 inches. A water quality basin is often between 0.5 inches to 1 inch, thus the flow duration basin is not any larger than and could be combined with a water quality basin.

Drain time ranges from 2.7 days to 3.7 days. Drain times of 3 days and less are recommended to prevent mosquito production.

Because the scale of this project is small, the resulting flows, basin sizes, and orifice sizes are all small. Orifice sizes are 3 inches in diameter.

Figure 3 presents the estimated flow duration curves for the Rational and SCS basins when routing the HMS time series through each basin. In other words, the effectiveness of the SCS and Rational methods are tested by routing the HMS post-project runoff through these basins. The SCS basin provides better results when compared to the HMS basin than the Rational method basin. The SCS method provides better control over the number of hours of flow than the Rational method. Both the Rational and SCS methods under-control flows of 0.2 cfs and smaller, while over-controlling flows greater than 0.2 cfs.

Figure 4 presents the HMS flow duration analysis with each house assumed to have runoff controls. Each lot is assumed to have its roof and patios disconnected from the storm drain system and controlled. Since these areas only need to control the increase in runoff caused by the impervious surfaces, runoff is assumed to function like pre-project pervious area. In other words, each house has a bio-infiltration trench (as described for volume control, TM #6) or similar BMP designed to maintain pre-project runoff conditions.

Figure 4 shows that HMS predicts a much smaller difference between pre- and post- flow duration curves. The flow duration basin area and volume are 62% and 77% smaller, respectively, than the HMS basin sized to collect roof runoff.

3.1 Discussion of the Application of Qc

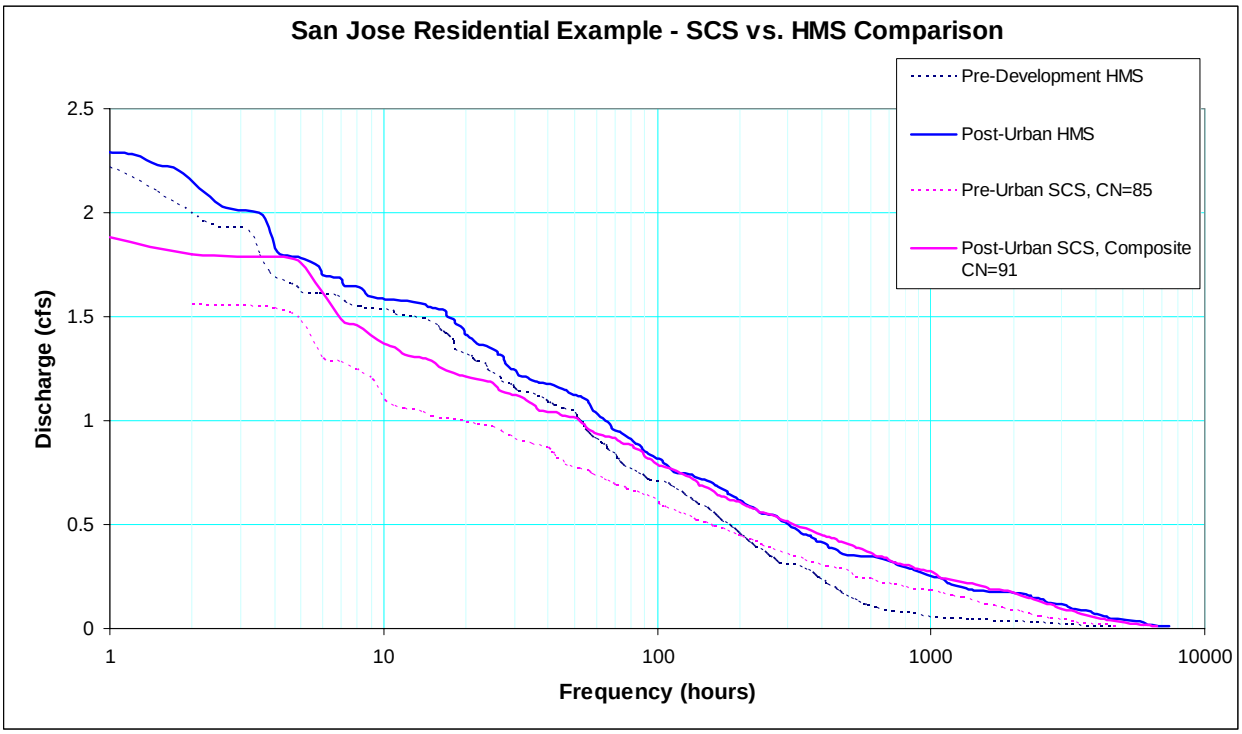
The HMS analysis and flow duration basin sizing was re-computed after adding an allowable discharge equal to a Qc value of 0.10 cfs. Qc is computed as 10% of the 2-year pre-project peak flow. The 2-year peak flow was determined by performing a flood frequency analysis of the HMS model results and was estimated to be 1.0 cfs. Figure 5 presents the resulting matched flow duration curves under this scenario.

In the first flow duration basin example on Thompson Creek, Qc was insignificant. However, Qc turns out to be important when sizing small flow duration basins like the one sized for the San Jose small development example. Qc can be thought of as the discharge rate for the extra volume of runoff caused by the impervious surfaces. Infiltration through the bottom of the basin is the other primary means of discharge of this extra volume of runoff.

For large basins like the one in Thompson Creek, the area is large (29 acres) and thus the infiltration loss rate (I_f * area = 5.5 cfs) is also large. When compared to the Qc rate of 2.4 cfs (10% of the 2-year peak flow of 24 cfs), the infiltration loss rate is twice as important and Qc has little affect on basin size. For small basins, the area is small (0.06 acres) and the infiltration loss

rate (I_f * area = 0.01 cfs) is also small. The estimated Qc for this small basin is 0.10 cfs and much more important for basin sizing.

The importance of Qc appears to vary depending on the area of the basin and the soil infiltration rates. Qc could be incorporated in every basin design and the analysis would reveal its importance. The Qc rates for small basins require small orifice diameters down to about 2 inches.



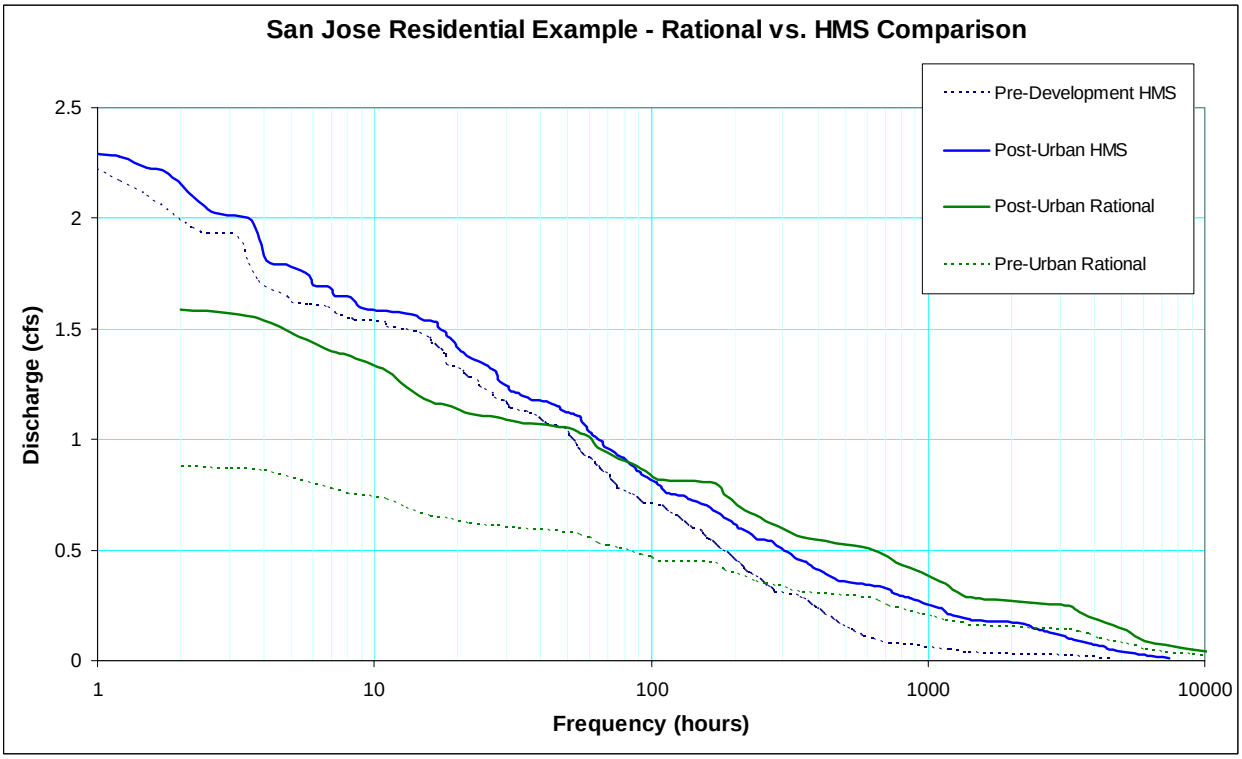


Figure 1. Comparison Between the Rational Formula and the SCS Method to the HEC-HMS Model

Table 1. COMPARISON OF MODELS AND RESULTS FOR FLOW DURATION BASIN SIZING

Model	Size			Percent of Catchment Area (%)	Inches Over Catchment (inches)	Orifice Sizes (inches)	Drain Time (days)
	Area (acres)	Volume (Ac-ft)	Depth (ft)				
HMS	0.074	0.106	1.75	2.1%	0.35	1 3-inch	3.7
SCS	0.078	0.125	2	2.2%	0.42	2 3-inch	3.1
Rational	0.095	0.211	3	2.6%	0.70	3 3-inch	2.7
Individual House Controls	0.046	0.082	1.75	1.3%	0.27	1 3-inch	3.6
HMS, with Qc = 0.10 cfs	0.060	0.102	2.25	1.2%	0.34	1 1.6 -inch 2 6-inch	<1 day

1) Infiltration rate: 0.2 inches/hour

2) Basin has 3:1 side slopes

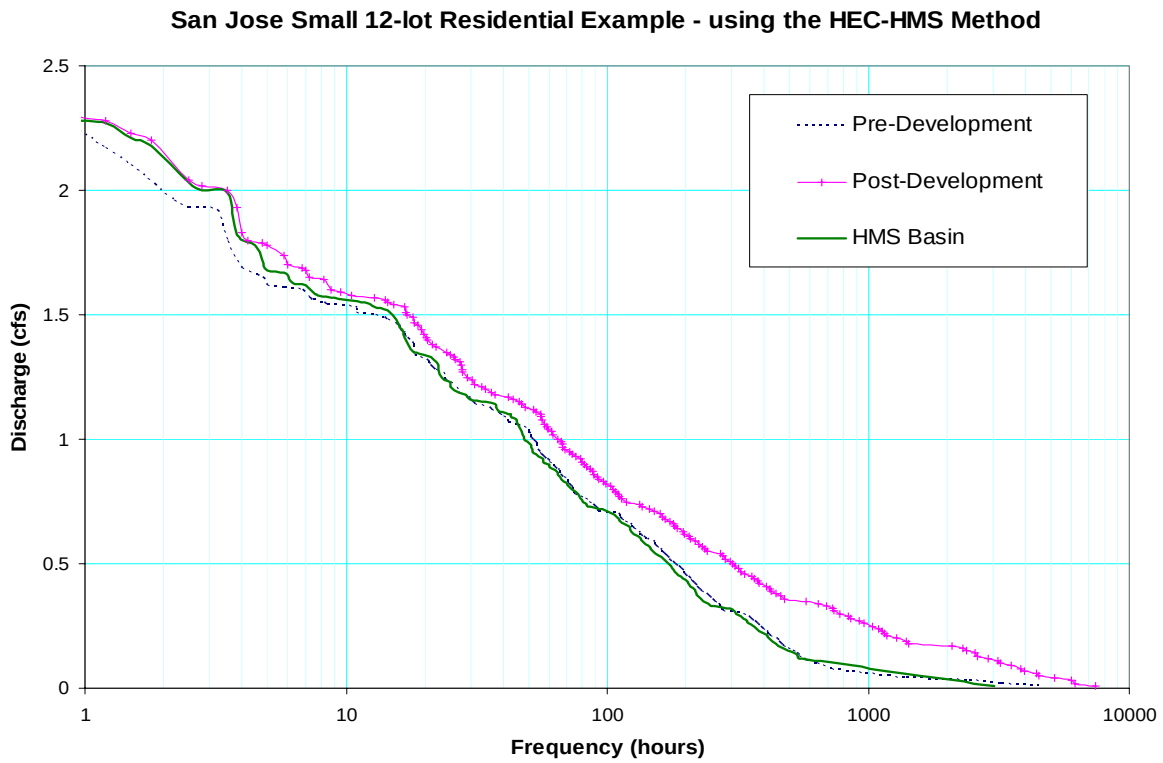


Figure 2. Matched Flow Duration Curves using the HEC-HMS Model Results

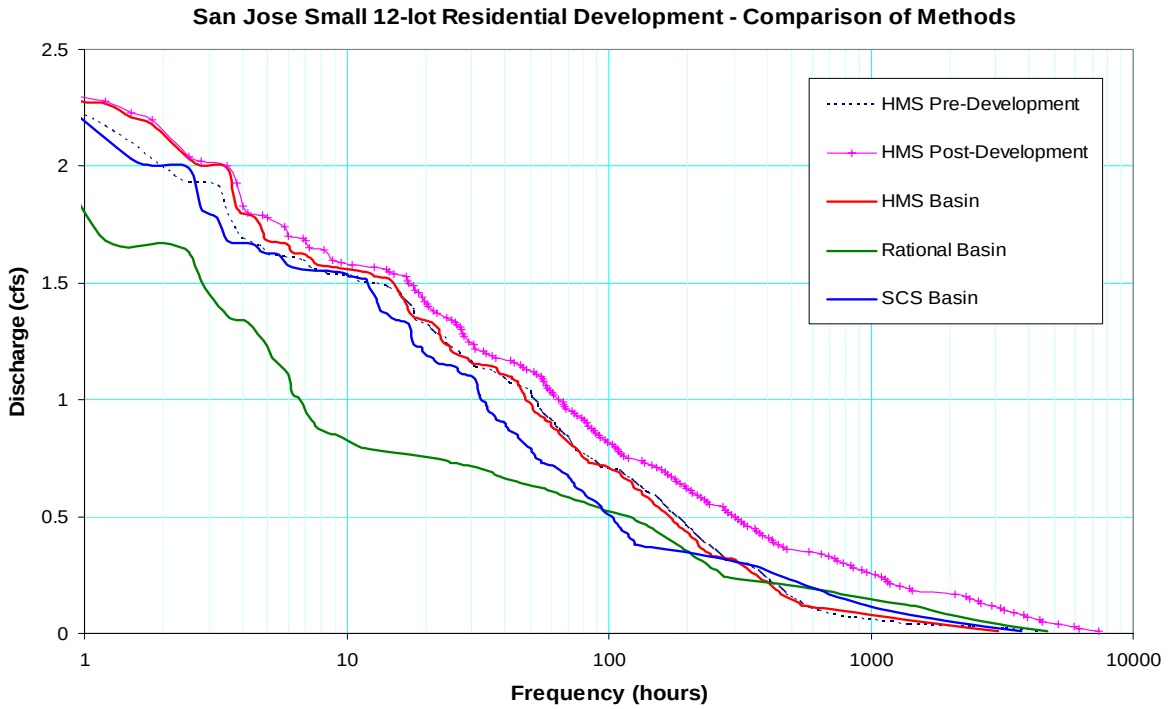


Figure 3. Comparison of Method Effectiveness at Reproducing HMS Results

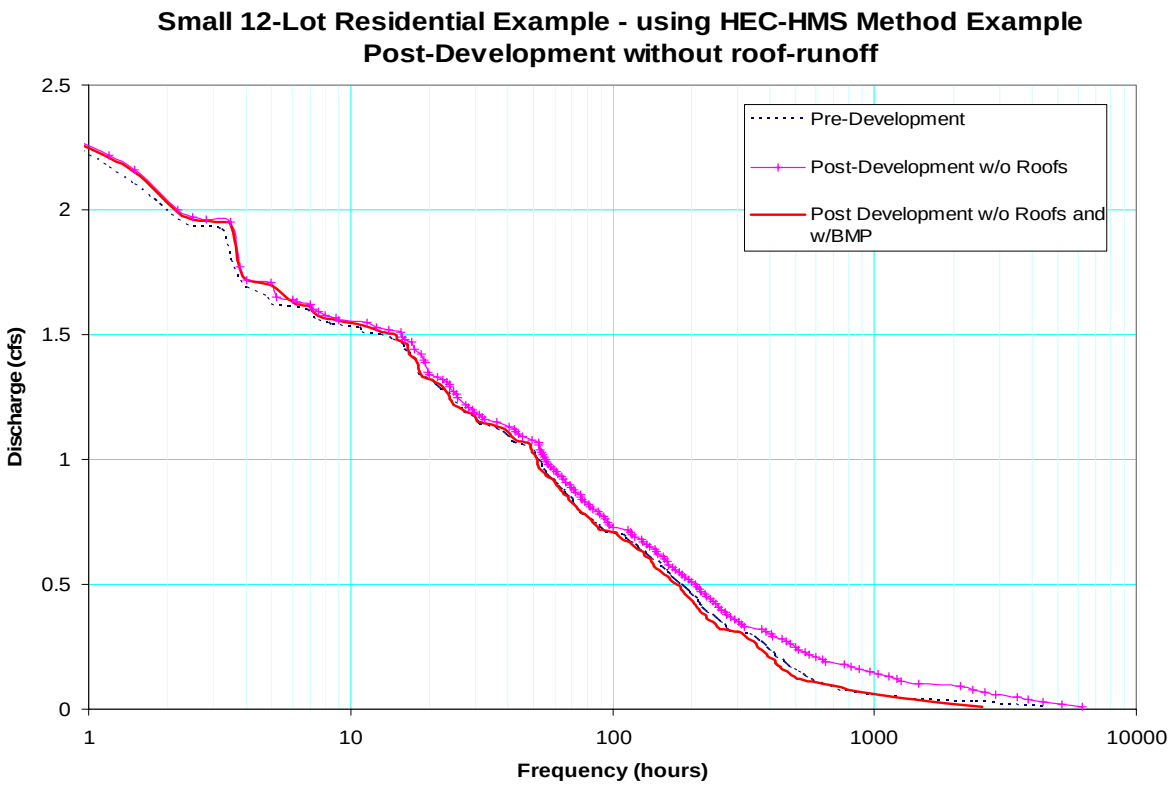


Figure 4. Matched Flow Duration Curves When Disconnecting House Roof Tops and Patios from the DCIA

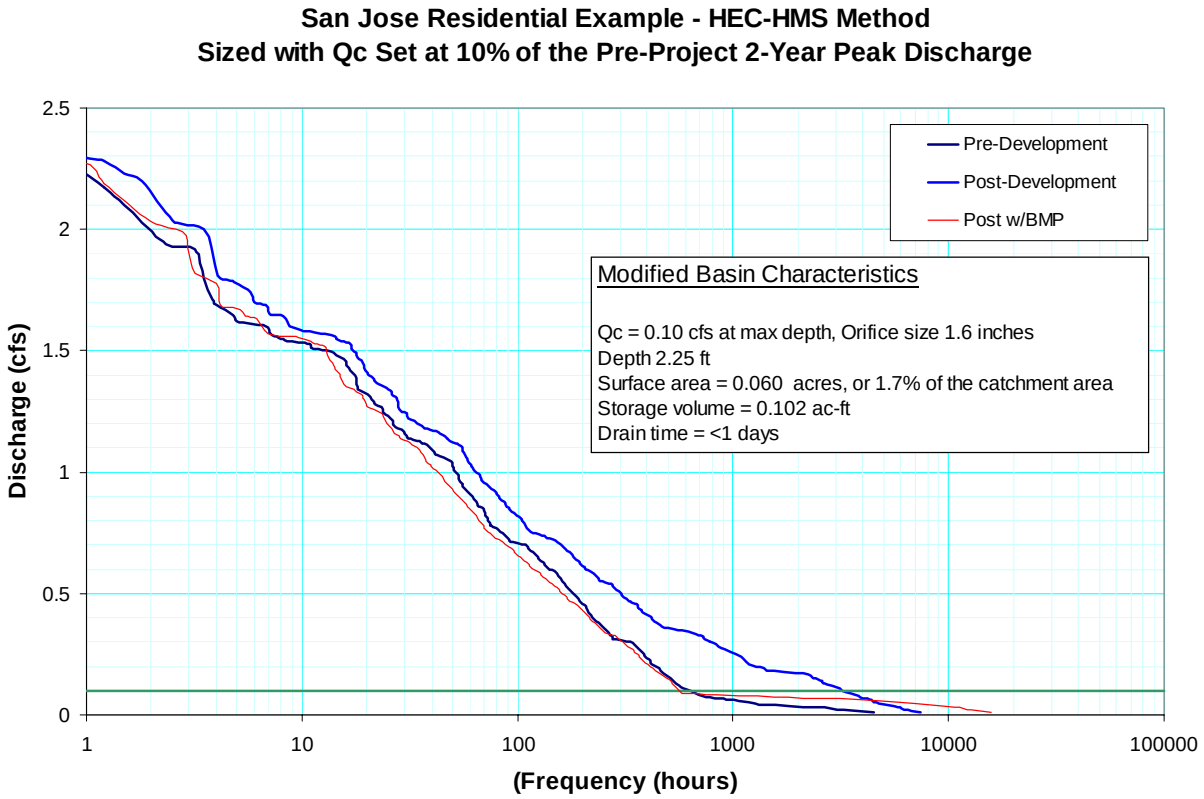


Figure 5. Final Basin Sizing Incorporating a Low Flow Discharge at 10% of the Pre-Project 2-year Peak Flow