

DESIGN HYDROLOGY MANUAL

**VENTURA COUNTY WATERSHED
PROTECTION DISTRICT**

VENTURA COUNTY, CALIFORNIA

UPDATED DECEMBER, 2010

UPDATED DECEMBER, 2006

REPRINTED 1991

REPRINTED 1985

REVISED 1978



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List of Acronyms and Abbreviations

ac.	Acres
ASCE	American Society of Civil Engineers
cfs	Cubic feet per second
cy	cubic yards
District	Ventura County Watershed Protection District
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
fps	Feet per second
Ft	Feet
HEC	USACE Hydrologic Engineering Center
HEC-HMS	Hydrologic Modeling System
HEC-RAS	River Analysis System
in.	Inches
mg/l	milligrams/liter
mi.	Miles
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
SCS	Soil Conservation Service
sq. mi.	Square miles
Tc	Time of Concentration
USACE	U.S. Army Corps of Engineers
USBR	U. S. Bureau of Reclamation
USGS	U. S. Geological Survey
VCRat	Ventura County Modified Rational Method Model
VCWPD	Ventura County Watershed Protection District
Vw	Wave velocity

SECTION 1 INTRODUCTION

1.1 PURPOSE AND SCOPE

The intent of this manual is to present guidelines and sufficient input data for computing design hydrology in Ventura County (see Figure 1). The methods should be used for projects subject to Ventura County Watershed Protection District (District) requirements. Input data and design procedures are intended to be sufficiently flexible that the methods can be adopted for use by all agencies and engineering consultants engaged in the design of flood control works throughout the county. The methods can be used on all small, intermediate, and large size watersheds under various conditions of development. The methods are used in computer programs such as the District's VCRat program, the U.S. Army Corps of Engineers (USACE) HEC-HMS program, the Environmental Protection Agency's (EPA) Hydrologic Simulation Program – FORTRAN (HSPF) model, and the USACE HEC-SSP implementation of the Bulletin 17B method for flood flow frequency analyses.

The majority of watersheds considered in the flood protection program of cities, subdivision engineering, and the District's jurisdictional facilities generally range in size from a few acres up to about five to ten square miles. Projected development in those watersheds ranges from complete urbanization to urbanization interspersed with parks and open space land. Runoff conveyances consist of streets, pipes, concrete channels, and channels in their natural condition. For the analysis of design hydrology for small to intermediate size watersheds up to 5,000 acres (ac.) with urbanized areas, the flexibility and simplicity of the modified rational method (MRM) is useful for obtaining design peak flows and hydrographs. For intermediate to large size undeveloped watersheds, the District has developed unit-hydrographs and corresponding S-Graphs for use with the HEC-HMS program. For developed watersheds of any size, the EPA's HSPF continuous simulation model can be used in conjunction with design storm rainfall and provides the ability to model erosion, sediment scour, and water quality parameters. If stream gage data exist, those can be used to develop flow frequency analysis results for design following Bulletin 17B (USGS, 1982). The results can also be used to calibrate the above hydrology runoff models.

When appropriate, the District will use other methods in the formulation of project hydrology, especially if those methods are used in analyses of the large river systems of the county in conjunction with Federal agencies. The recommended approach in each case will be determined by the availability and expected reliability of necessary input data. Considerable engineering judgment will be necessary in formulating project hydrology for watersheds of approximately 5,000 acres or more to ensure that peak flows computed by the various appropriate methods are consistent and compatible.

Project design in the vicinity of watersheds with a high wildfire and erosion potential may need to consider the effect of debris bulking on peak flows. Techniques for bulking peak flows in Ventura County have been developed and are applied on a case-by-case basis. The District will determine the need for debris bulking and specify appropriate procedures in all cases.

Although it is anticipated that the overall approach described in this manual will not significantly change, any hydrologic process should be considered a dynamic process and subject to periodic refinement as additional input data become available. Techniques described in this manual will be continuously reviewed and updated, as necessary, to provide a thorough and complete package for hydrologic design.

1.2 ACKNOWLEDGMENT

The modified rational method presented in this manual for computing peaks and runoff hydrographs was developed by the Los Angeles Department of Public Works (formerly known as the Los Angeles County Flood Control District) over the past 50 years and is recommended by them for use in Los Angeles County. It has been adapted for use in Ventura County by the Hydrology Section of the Ventura County Watershed Protection District.

The District gratefully acknowledges the very generous cooperation of the then-named Los Angeles County Flood Control District in assisting with the development of necessary input data for adapting their method for use in Ventura County. Velocity-discharge relationships for natural channels, slope correction factors, and wave velocity relationships used in this manual were supplied by their staff. The computer program presented in this manual was supplied by them and originally adapted for use on the IBM 370 and Control Data Corporation 6600 computers, and currently for use on PCs.

Many of the procedures and data contained in this document were developed and refined under the direction of the longtime Hydrology Section Manager, Dolores Taylor, PE. She has been indispensable in training many of the engineers and hydrologists that have worked for the District in these techniques.

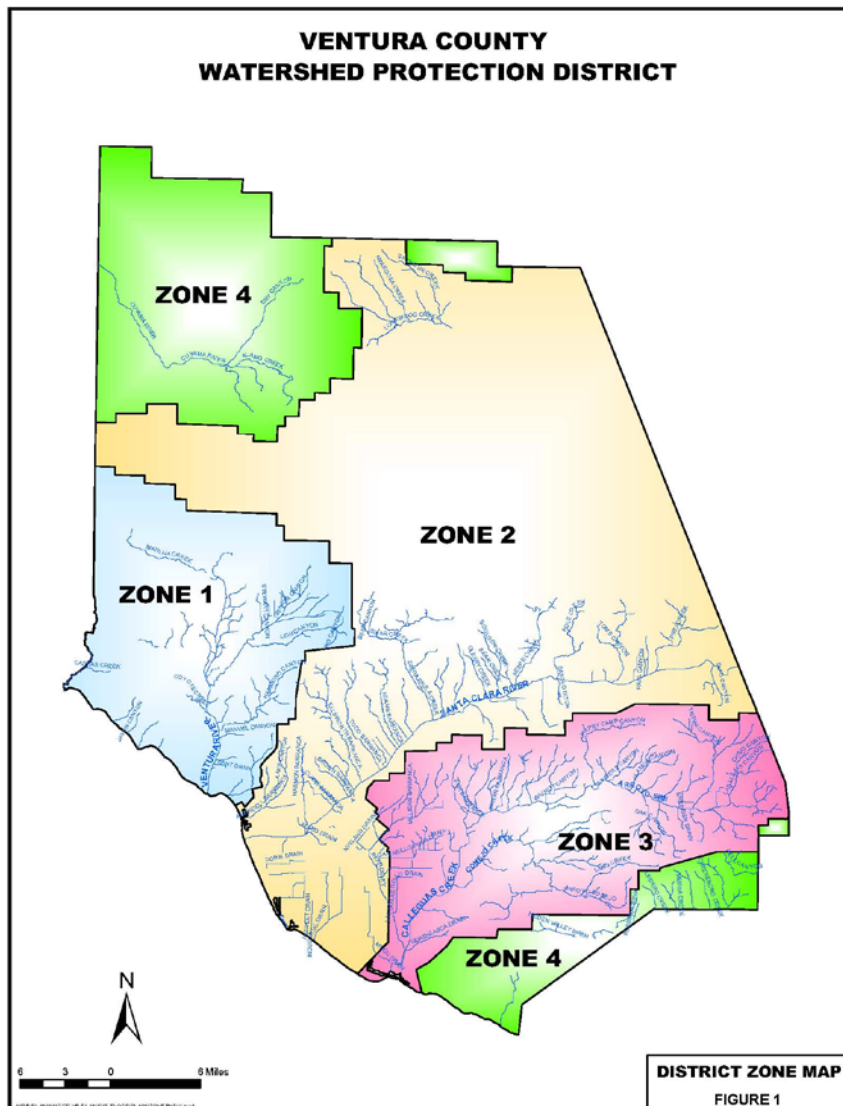
1.3 REPRINTS, REVISIONS, AND UPDATES

In August 1978, the first reprinting of the Hydrology Manual was done. Because of recent interest in 100-year peak flow rates, and the need for more guidelines in estimating overland flow velocities for computing times of concentration, certain additions were incorporated in the manual at that time. A plate was added to include intensity-duration curves for the 100-year storm event. A new plate, Exhibit 8, was added which shows suggested minimum overland flow velocities for use in consistently estimating time of concentration throughout a study area and from watershed to watershed. Appropriate revisions were made to each section in the text to support the added information.

In a previous Manual update (2006), the following procedures were updated:

1. The District's VCRat 2.2 computer program was modified to create a more user-friendly Windows version (VCRat 2.5, updated into 2.6). The program still includes all of the specialized operations available in VCRat 2.2 so that previous VCRat input files can be imported and yield the same results.
2. A Tc Calculator program has been developed to facilitate Tc calculations for the modified rational method program. Because the program requires less time to calculate Tc's the number of required subareas with calculated Tc's has been increased to 75 percent of developed areas and 25 percent of undeveloped areas as described in Section Five.
3. Information and procedures for adjusting the hydrograph volumes to match the yield predicted by the SCS Curve Number method for the watershed have been added. This information is intended for use in designing detention facilities for developments receiving runoff from tributary areas less than one sq. mi. in size. The data required for use in the yield adjustment are provided in Appendix A.
4. The program requirements for performing areal reduction simulations are discussed.

5. The recommended effective imperviousness percents for use in hydrology analyses have been expanded to cover more land uses and reflect current construction techniques that lead to less pervious area in the watershed.



The current 2010 Manual update provides the following additional information:

1. Design procedures using HSPF.
2. Design procedures using HEC-HMS.
3. Flow frequency analysis procedures and references.
4. Detention basin design procedures.

5. Simplified detention basin design procedures for small project areas.
6. Revised Curve Numbers (Exhibits 14a and b) for use in yield assessments
7. Areal reduction run procedures using VCRat2.2
8. Bulking factor information

SECTION 2 DESIGN HYDROLOGY SUMMARY

The hydrology methods presented in this Manual yield peak flows and hydrographs that can be used for facility design or to show the impacts of proposed improvement projects. This section presents general information about the District models and their calibration and use in design studies. The historical data and studies referenced in this section can be downloaded from the District's website at <http://www.vcwatershed.org/>. The information is provided in one of more of three locations: 1) The downloads section of the website; 2) The webpage listing models and studies for an individual watershed; and 3) The webpage for a major project in a watershed.

2.1 DESIGN HYDROLOGY MODELS

2.1.1 Modified Rational Method

The modified rational method (MRM) is appropriate for modeling partially- to fully-developed urbanized catchments of up to about 5,000 ac. It has been used on larger catchments in the past but those efforts required calibration of the model to stream gage analysis results. The MRM requires intensive effort to digitize boundaries and calculate Tc's for the relatively numerous subareas.

The MRM provides a hydrograph characterized by a relatively sharp and narrow peak, and generally the yield under the hydrograph is less than would be expected from a NRCS yield evaluation using Curve Numbers. Therefore, the MRM program hydrographs cannot be used for detention basin design without being adjusted for yield- a process the District refers to as "fattening". However, watersheds over 70 sq mi are too large to use the VCRat program yield adjustment or "fattening" due to the constraint of the 24-hr simulation length of the model. For hydrographs from large watersheds, the District has developed special methods using HEC-HMS to provide design hydrographs discussed later in this document.

The District has prepared MRM models for many of the developed areas of the County and generally has model results available by request. Some of the models were developed years ago and require updating of their outdated land use assumptions. Availability of the models can be checked by sending a data request to the District's hydrologic data request email at: Hydrodata@ventura.org. The District is also preparing a webmap with links showing MRM hydrology study locations and allowing the results to be downloaded. The webmap is expected to be posted on the District's website sometime in 2011.

The most ambitious study using the MRM was the preparation of 2000 and 2020 condition hydrology models of the 250 sq. mi. Calleguas Creek Watershed. Each model contains about 5,000 subareas. The hydrology supported the FEMA Flood Insurance Study (FIS) update for that watershed and the report and the model results are available on the District's website and also on the Calleguas Creek Watershed Management Study website. The 2000 Condition Model was finalized and is considered official while the Future Condition Model has not been finalized and is still considered draft.

The MRM has a scale effect so the Tc used in the model is limited to range from 5 to 30 minutes, with subarea sizes of 20 to 80 acres recommended. If numerous very small subareas are used with 5-min Tc's to represent a development, a relatively high peak can result. Therefore, if a model is prepared with numerous small subareas to design local drainage for a development the results should not be submitted to the District for evaluation of the impacts to the District's jurisdictional facilities ("Redline" Channels).

2.2 HSPF MODELING

For developed and undeveloped watersheds of any size, the EPA's HSPF continuous simulation model can be used in conjunction with design storm rainfall to produce design storm peaks that also have reasonable yields. HSPF is a comprehensive watershed model of hydrology and water quality that includes modeling of both land surface and subsurface hydrologic and water quality processes, linked and closely integrated with corresponding stream and reservoir processes. HSPF has enjoyed widespread usage and acceptance, since its initial release in 1980, as demonstrated through hundreds of applications across the U.S. and abroad. HSPF is the primary watershed model included in the EPA BASINS modeling system and it has recently been incorporated into the U.S. Army Corps of Engineers Watershed Modeling System.

Subsequent sections in this Manual discuss topics relevant to using HSPF for design storm model such as preparation of design storm rainfall hyetographs, selection of modeling period to reflect antecedent moisture conditions for design storms, and undeveloped channel routing effects inherent in the program. Because HSPF is a more complex hydrologic model than the MRM, care must be taken in preparing and running the design storm model. However, good results and agreement with stream gage frequency results have been obtained for the Santa Clara Watershed ([AQUA TERRA, 2009](#)), and the Ventura River Watershed ([Tetra Tech 2009](#) and [VCWPD, 2010](#)). The project to obtain design storm results for the Calleguas Creek Watershed Model is expected to be completed in 2011.

Many of the catchments in the Santa Clara and Ventura River watersheds been evaluated in those regional models and the results are contained in the reports posted on the District's website in the pages for each watershed. These results can be used for design purposes. If detailed modeling results are required, the HSPF model can be refined appropriately. For urban and developing areas, a MRM model can also be developed.

2.3 HEC-HMS MODELING

For intermediate to large undeveloped watersheds, the District has developed unit-hydrographs and their associated S-Graphs for use with the HEC-HMS model. A District study (VCWPD, 2007, Draft) calibrated the unit hydrograph with a number of undeveloped watersheds using runoff data from the 2005 storms. Those storms had recurrence intervals ranging from 10- to 50-yrs depending on the location. The resulting unit hydrographs were then averaged into final unit hydrographs that can be used for undeveloped watersheds across the county. The method includes the use of the Snyder Unit Hydrograph, the USACE lag equation, and a uniform and constant loss approach so that the hydrograph yield matches that obtained through a NRCS Curve Number yield approach. Through this methodology, the District was able to create design storm models with peaks matching stream gage frequency analysis results within 10% or better for most of the gages included in the study.

[USACE \(2003\)](#) prepared a regional HMS model of the Calleguas watershed in support of the FEMA FIS update for that basin and to confirm the MRM results used in the FIS. This model presented results for the natural, existing, and future conditions and has been used for various studies in the watershed. The model used an S-Graph developed for Sespe Creek and design storm hyetographs using the alternating block technique. The resultant hydrographs are very peaky, similar to MRM hydrographs, and generally have yields that are much less than would be expected based on NRCS Curve Number yield analyses for the watersheds. Therefore, the hydrographs should not be used for hydrology and floodplain mapping

studies requiring accurate volumetric hydrographs. The model did not explicitly include urban hydraulic constraints and required the use of the standard areal reduction factors and also a runoff calibration factor to match the peaks provided by the stream gage frequency analyses published in the USACE report.

2.4 STREAM GAGE FLOW FREQUENCY ANALYSES

If stream gage data exist near a project site, those can be used to develop flow frequency analysis results for use in design following Bulletin 17B (USGS, 1982). The stream data can also be used to calibrate the hydrograph volumes. The resultant hydrographs and peak can then be used to calibrate hydrology models or for design of regional facilities. The District (VCWPD, 2007) has prepared a draft report that summarizes the frequency results and describes the methodologies used to provide the results for the stream gages operated by the USGS and VCWPD in Ventura County. This report can be obtained from the District upon request. The methodologies include the following:

1. Bulletin 17B (USGS, 1982) analyses of stream gages performed by the VCWPD on various stream gages.
2. U. S. Bureau of Reclamation (USACE, 2005) top-fitting of the seven highest historic peaks on the Ventura River to obtain design peak flows for the two gages on the Ventura River mainstem. The topfitting model was used because the log Pearson III statistical model does not fit the data from the regulated watershed adequately. The USBR used historic hydrology modeling results to provide estimated flows at other intermediate locations between the two gages.
3. USACE (2003) graphical frequency analyses of the Calleguas Creek watershed stream gages that used engineering judgment and review of all of the Calleguas Creek stream gage data to plot of family of curves to obtain the design storm peaks.

2.5 WATERSHED MODEL SUMMARIES

2.5.1 Ventura River Models

The 2005 Condition HSPF Continuous model of the Ventura River watershed developed by Tetra Tech (2009) under contract to the District is the best source of design peak data for this watershed for subareas studied by FEMA. Model results have been used to provide design storm peaks after calibration with stream gage data and comparison to other hydrology model results. The peaks were provided to FEMA for their FIS study for many of the tributaries along the river. The model was also calibrated to match the mainstem 100-yr peaks provided to FEMA based on a stream gage frequency analyses done by the USBR.

A recent VCRat study of Dent Drain using the criteria outlined in the District's 2006 Hydrology Manual is considered to be a good source of intermediate flows for that watershed. Design peak data for a number of subareas in the San Antonio River watershed are available from the VCRat model developed in the 1990s for intermediate flows. The VCRat model results were compared to the HSPF model results at coincident points during the HSPF model calibration and are considered to be secondary to the HSPF model results.

Various historical VCRat models of individual subareas in this watershed generally do not reflect current land uses or have sufficient backup information to consider them to be official model results and should

not be used for design peak info without being updated using current criteria. A future conditions model of the Ventura River watershed is not available and there are currently no plans to develop one.

The District uses a HEC-1 model to forecast flow peaks due to predicted rain conditions and has recently converted the HSPF model of the watershed for the same purpose. It was tested during the 2009-2010 water year and will be used for forecasting in subsequent years.

2.5.2 Santa Clara River Models

The 2003 Condition HSPF Continuous model of the Santa Clara River watershed (Aqua Terra, 2009) is the best source of design peak data for this watershed. Model results have been used to provide design storm peaks after calibration with stream gage data and comparison to other hydrology model results. The peaks were provided to FEMA for their FIS study for 5 of the streams. The peaks from most of the other major tributaries along the river were provided to the COE for additional floodplain mapping as part of the feasibility study. The model was also calibrated to match the mainstem 100-yr peaks provided to FEMA based on a stream gage frequency analysis done by the District in 2006. For the streams included in the HSPF report, this model is the source of the best design peak information for watersheds along the river.

Other available models for the Santa Clara River watershed include various historical VCRat, HEC-1, HEC-HMS, and COE models of individual watersheds that generally do not reflect current land uses or have sufficient backup information for them to be considered official model results. The results from these models should not be used unless they are updated according to the District's current criteria.

A future conditions model is planned for development as part of the Santa Clara River Watershed Feasibility Study but the work has not been started or contracted out as of summer 2010. The technical team on the study has not resolved how the future conditions will be represented in the model, including curb inlets limiting flow in future development and whether new development shown on General Plans will be assumed to have detention basins to mitigate peak flows.

The District uses a HEC-1 model to forecast flow peaks due to predicted rain conditions and is converting the HSPF model of the watershed for the same purpose during the fall of 2010. It will be used for forecasting in subsequent years.

2.5.3 Calleguas Creek Models

A HEC-HMS model of the watershed developed by the COE with regional-scale subareas providing natural, 2000, and 2050 condition design peaks for the watershed is considered to be the best source of regional-scale peak information for design studies. The model was calibrated for peaks only and does not have sufficient volume in the hydrographs to be used for studies requiring accurate volumetric data.

A VCRat model of the watershed representing 2000 conditions land use is considered to be the second best source of design peak data and also can provide intermediate peaks for smaller subareas. The hydrograph volumes are too low to be used for volumetric studies without yield adjustment following the District's methods.

Draft future condition peaks can be obtained from two additional VCRat models representing 2020 conditions developed in 2005, with one model using the channel routing assumptions from the 2000 conditions model (without project) and the other assuming that known hydraulic constrictions in the

watershed were fixed (with project). These models were never made official and outside review by a consultant showed that a number of issues related to the ways that future conditions were represented in the model limited their usefulness for providing peak flows for design studies. Use of the peak information for project evaluation should be discussed with the District on a case-by-case basis. The model hydrographs cannot be used for volumetric studies unless adjusted as discussed for the 2000 condition model.

An HSPF model of the watershed providing design storm peaks is under development as of summer 2010. The plan is to also provide a future conditions version that can provide hydrographs and peaks for design studies. The model will also be used to evaluate urban storage effects on peak flows as this is an important issue for the communities that are part of the Calleguas Creek watershed.

The District uses a HEC-1 model to forecast flow peaks due to predicted rain conditions and will be converting the HSPF model of the watershed for the same purpose during the fall of 2010. It will be used for forecasting in subsequent years.

2.6 MODEL CALIBRATION

For watersheds with stream gages, each hydrology model should be calibrated to the stream gage data, including flow frequency analysis results for design storm models and continuous data if it is a continuous model. The daily stream gage data for Ventura County are available from the District's website. For high resolution stream gage data at the 5-min level, files can be requested from the District. Some gages are operated by the USGS and the data are officially published by them. The District generally has copies of those data sets for use in our studies and will provide those to requestors. If the user wishes to obtain the USGS data sets they can be obtained from their website or by contacting them directly for high-resolution data.

For ungaged watersheds, design peak results from different models should be considered similar and adequate if they are within 5%. Because HSPF and HMS models of the District's watersheds developed so far tend to represent large watersheds with relatively few subareas, they are considered to be regional models. Calibration of ungaged watersheds in models is done by comparing the results to the following sources in order of preference subject to availability:

1. Stream gage results from gaged watersheds with similar characteristics.
2. Other model results for the same watershed,
3. Other model results from watersheds of similar size and hydrologic characteristics, and
4. Available USGS regression equations peaks.

For calibration of hydrograph volumes the NRCS Curve Number yields should be calculated and the hydrographs adjusted to match those values.

Due to the use of VCRat in many studies and channel design projects that have been found to perform well over time, VCRat model results are considered calibrated without any adjustment to the input parameters. Due to the scale effect, however, if VCRat is used to study large watersheds the resultant peak can be much larger than the Q100 obtained from stream gage frequency studies. In this case, the areal reduction curves used to convert the model output to design peak numbers are adjusted by adding a point that provides the correct peak flow at the total area corresponding to the stream gage watershed and

fitting a regression line through the points. This approach was used to calibrate the 2000 and 2020 condition Calleguas VCRat models.

2.6.1 Urban Storage Effects

In developed watersheds where hydraulic constraints exist, the model results must be evaluated carefully before being used for design. This is especially important for models representing developed areas where culvert inlets are often designed for the 10-yr storm peak and any flow above that level is expected to be contained in the streets and be delayed in discharging to one of the District's jurisdictional facilities. Typical urban area models developed by the District represent theoretical peak flow results in that system storage is not modeled explicitly. For these results, it is the responsibility of the hydraulic engineer to evaluate system hydraulic constraints and adjust peaks and hydrographs provided by the hydrologist accordingly. In the limited cases where the hydrology model simulates explicit hydraulic constraints, the hydrology cannot be used to show the resultant flow once the system has been improved. If this is required, then the hydrologist should prepare two models, one representing pre-project conditions, and one showing post-project conditions.

At this time urban channel deficiencies are rarely represented in the District's hydrology models. At a very limited number of locations flow splits caused by hydraulic constrictions in the District's channels have been included in models such as the Tapo Canyon channel in the VCRat model of the Calleguas watershed and one location in the Oxnard Industrial Drain VCRat model. The urbanized watersheds in the Ventura River HSPF model were calibrated so that the peak matched the stream gage results, which required the application of a factor in the model equivalent to reducing the peak rain intensity by 0.7 but preserving the total hydrograph volume. The official model results discussed in the following sections are available for download at the District's website. Unofficial model results are available through requests sent to HydroData@ventura.org.

SECTION 3 MODIFIED RATIONAL METHOD AND VCRAT

The MRM is based on the rational equation that has been widely accepted for use in the design of flood protection measures. It is relatively simple, is in wide use, and has been found to provide reasonable results for project design. The basic equation is:

$$Q = CIA$$

where:

Q = Peak discharge in cubic feet per second (cfs).

C = Coefficient of runoff (dimensionless).

I = Average rainfall intensity for a duration equal to the time of concentration of the watershed (inches/hour).

A = Drainage area of the watershed (acres).

Dimensionally the equation is approximately homogeneous. Units of discharge are acre-inches per hour which converts to cubic feet per second with a conversion factor of 1.008. In view of the uncertainties associated with measurement of necessary input parameters, the conversion factor is usually ignored and flow rates are considered to be in units of cfs.

The rational equation is dependent upon the following basic assumptions: 1) The peak rate of runoff at any point in a subarea is a direct function of the average rainfall intensity during the time of concentration to that point; and 2), The time of concentration is the time required for runoff to become established and flow from the hydraulically-most-remote part of the drainage area to the subarea outlet.

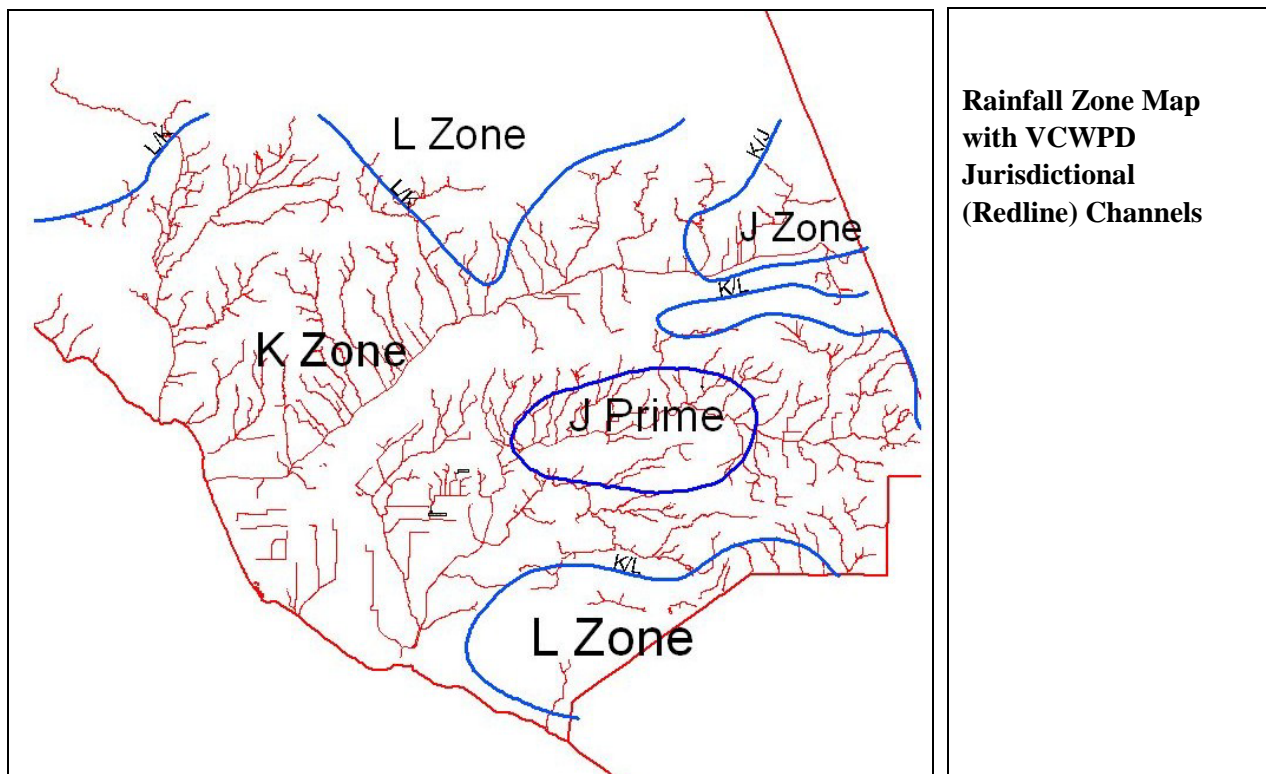
The method presented in this manual is based on input parameters developed specifically for soil types and rainfall characteristics in Ventura County. In addition to the watershed drainage area, input parameters for the rational equation include rainfall intensity-duration curves, runoff coefficient curves, and time of concentration. The rational equation has been enhanced through the use of rainfall mass curves so that a hydrograph is developed that can be routed downstream and combined with hydrographs from other subareas.

3.1 Rainfall Intensity-Duration

Most precipitation in the Ventura County area results from general winter storms associated with extra-tropical cyclones that develop in the North Pacific during the months of November through April. Precipitation during these storms may occur over large areas and a major storm causing high flows and flooding can last four days or more. The first several days of the storm normally contain light, persistent rainfall of moderate intensities. Rainfall during this period satisfies the soil moisture deficiency. The latter portion of the storm is characterized by short periods of high rain intensities falling on saturated soils.

Using theoretical techniques proposed by the California Department of Water Resources, intensity-duration-frequency relationships were computed for all long-term recording rain gage data in Ventura County. Frequency analyses used storm intensities for durations of 5, 10, 15, and 30 minutes, and for 1, 2, 3, 4, 5, 12, and 24 hours. From this analysis, intensity-duration curves were plotted for each gage for

selected frequencies and similar curves from gages in similar rainfall regimes were averaged, resulting in a family of intensity-duration curves that were labeled J', J, K, and L. Using the 50-year, 24-hour isohyetal map of the county, and considering orographic influences, each curve was assigned to a specific area in the county. The curves are presented in this manual for the 100-, 50-, 25-, and 10-year events. The appropriate areas in the county for each curve are shown in the hydrology maps, Exhibit 1a through d and Appendix E maps. The maximum intensities for the various design storm and concentration times are listed in [Exhibit 2](#).



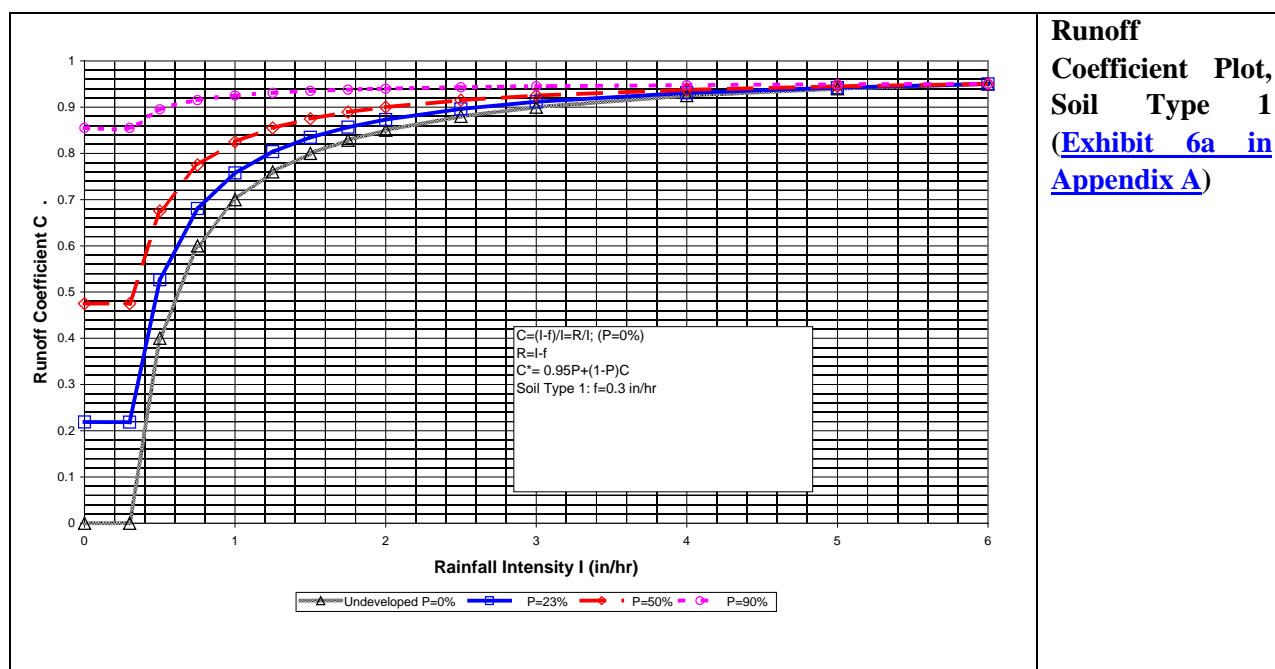
The intensity duration curves were originally used to developed 30-point mass curves for use in developing hydrographs in the modified rational method model. As more accuracy was needed in resolving the 100-yr peaks for design, a 200-pt mass curve was developed for each zone. The 100-yr mass curves are designed to produce 80% of the volume under the hydrograph at 80% of the hydrograph time for the one-day simulations. This provided for more conservative design of detention basins.

3.2 RUNOFF COEFFICIENT CURVES

For each acre of drainage area, a runoff coefficient used in the rational equation represents the ratio of runoff to rainfall. It is the percentage of rainfall on a watershed that occurs as runoff and ranges from zero on very porous soils to 0.95 on impervious surfaces (where runoff is limited due to depression storage and other abstractive losses). It includes the composite effect of such watershed variables as infiltration, ground slope, ground cover, surface and depression storage, antecedent precipitation and soil moisture, and shape of the drainage basin. Runoff is not a constant percentage of rainfall, but is the residual of rainfall remaining after losses. Losses increase as precipitation increases in minor storms, but the increase in losses is not proportional to the increase in rain. Therefore, the ratio of runoff to rainfall

increases as storm intensity increases and in the method presented in this manual, the runoff coefficient is a function of intensity.

Using the Soil Survey completed by the SCS (now NRCS) of the U. S. Department of Agriculture in April 1970, soils in Ventura County were grouped into seven hydrologically homogeneous families. Two families were assigned to each SCS Hydrologic Soil Group A, B, and C; while only one family was assigned to SCS Hydrologic Soil Group D. The upper and lower limits of the group of runoff coefficient curves for Ventura County were defined by considering a composite plot of all those developed by Los Angeles County Flood Control District using ring infiltrometer tests. Individual curves for undeveloped soils were determined theoretically by considering representative infiltration rates for various soil textures recommended by the SCS and the ASCE in their Hydrology Handbook. Some adjustment was necessary at high rainfall intensities to ensure a family of approximately parallel runoff coefficient curves.



At some point, the hardcopy soil maps with their assigned VCWPD soil numbers were digitized so that they could be used in GIS analyses of the watersheds. Recent evaluation of those historic digital maps showed that the digitizing process resulted in some offset and skewing compared to the recent digital maps provided by the NRCS. Also, the historic maps did not cover the entire county and did not extend across County lines to provide coverage of watersheds located in both Counties. Therefore, as of 2010, the soils data was downloaded from the NRCS and a new digital soils map was developed for use in hydrology studies. The new soils GIS files are provided in Appendix E.

3.3 TIME OF CONCENTRATION

Time of concentration (T_c) is defined as the time required for runoff to travel from the hydraulically most distant point of a watershed to its outlet. It is a summation of the travel times associated with overland flow and concentrated flow in streets, pipes, and stream channels. Because of the complex flow path

associated with most watersheds, generalized charts and nomographs such as Kirpich are not considered adequate for computing travel times.

To determine time of concentration, the watershed is divided into contributing subareas using topographic barriers as well as streets, known improvement plans, and projected ultimate development. Ideally, the subarea size should range between 20 and 80 acres; however, the method can be used on smaller subareas providing the time of concentration is greater than five minutes. If the subarea of interest is too small to provide a T_c calculation greater than 5-minutes, a larger subarea should be evaluated and the discharge pro-rated based on the peak flow per unit area as described in Section Six. The upper limit on T_c is 30 minutes.

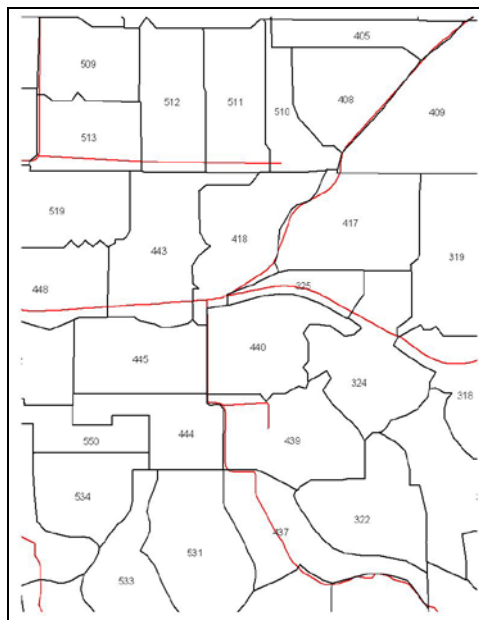
For any given subarea, the time of concentration must be computed by the method of successive approximations. For a given flow path and drainage area, the travel time is a function of the peak flow, which is itself a function of the rainfall intensity. Intensity is a function of the time of concentration. The proper approach for computing an appropriate time of concentration is discussed in more detail in below and demonstrated in [Appendix B](#). Examples are also provided in the T_c Calculator Program Manual.

The rational equation computes an instantaneous flow rate based upon average rainfall intensity over a period equal to the time of concentration of a watershed. When the intensity is the maximum for a given storm, the associated flow rate represents the peak for that storm. When storm intensities that are less than the peak period intensity are used in the rational equation, the resulting flow rates represent some point on a theoretical storm hydrograph.

Every design storm is made up of a series of intensities that change more or less uniformly throughout. A mass curve of rainfall represents an accumulation of storm totals at any point in a storm and its slope at any point represents the rainfall intensity at that point. By determining intensities at representative times throughout a design storm, a runoff hydrograph can be generated using the rational equation.

Each of the intensity-duration curves used by VCWPD has an associated mass curve. The mass curves are included in the District's VCRat computer program and they are used to define intensities for all hydrographs developed by VCRat. Intensity tables may be used for hand computations of runoff hydrographs. A typical calculation is shown in [Appendix B](#).

Once the runoff hydrograph is generated, additional subareas can be evaluated and added together using the timing information contained in the hydrographs. In this way the rational method can be extended for use in watersheds with numerous distinct subareas such is shown below. The use of the method to confluence the hydrographs from different subareas is known as the modified rational method.



Subareas with Node I.D.s and Channels in Calleguas VCRat Model

3.4 VCRAT GENERAL DESCRIPTION

The VCRat computer program performs a number of functions in implementing the modified rational method, such as subarea hydrograph generation, channel routing, basin routing, and subarea confluencing. First, the consistency of input data that may include runoff coefficient curves, rainfall mass curves, and subarea and hydrograph data input are verified. When errors are encountered, processing is terminated and error messages are printed. After the input data have been verified to have enough information to do a simulation, the program proceeds with specified hydrologic calculations.

3.4.1 Conceptual Model of Runoff

The conceptual model of flow from a multiple subarea model is as follows: first an outflow hydrograph is generated for the most upstream subarea. Next, that hydrograph is routed through the next downstream subarea using the channel information provided as input. Then the program calculates a hydrograph for the downstream subarea and combines it with the routed hydrograph from the upstream subarea. The combined hydrograph is then routed through the next downstream subarea and so on. Based on this conceptual model, the runoff from a subarea does not add to the water level or affect velocities in a channel going through that subarea.

VCRat does not have the capability to easily model hydraulic constraints in the drainage system such as 10-yr storm curb inlet limitations, or undersized drainage pipes detaining flow and attenuating peaks. Therefore, with limited exceptions model results provide peaks assuming that all of the flow in the watershed can be discharged to the outlet in adequately sized pipes. This allows the hydraulic engineer to adjust the hydrographs as necessary to reflect constraints and also show the impacts of improving the drainage system on the downstream peak.

3.4.2 Computation Summary

The required input data for a subarea consists of the area, T_c , soil type, and design storm type. Channel routing information can be added if appropriate. For a specified design storm, VCRat calculates a runoff hydrograph at all subarea collection points within the watershed, combines hydrographs from each subarea, and routes the combined hydrograph through the channel system. The hydrographs are calculated assuming the T_c associated with the peak does not vary with intensity changes in the rainfall mass curve. The program accounts for channel storage and reports peak flow rates in each reach of each drain, coincident flow rates of tributaries at confluence structures, and hydrographs at subarea collection points as specified in the program input data.

Options are available for the computation of hydrographs for a multiple day design storm to obtain the runoff volume data necessary for retention basin and pump plant design. Separation of flow at the junction of two drains may be made by either specifying a percentage split of the inflow hydrograph, or a split with all flow up to a specified discharge placed in one drain and the remaining flow placed in the other.

If the type and size of a drain are not specified, the system will automatically begin routing flow in a street section in the upper limits of a watershed; change from street to pipe section when flow depth reaches the property line (located at the development boundary of an assumed 10-ft wide sidewalk adjacent to the street); from pipe to rectangular channel when a pipe diameter of 8 feet is exceeded; and from the hydraulically most efficient rectangular section to a rectangular section with a maximum depth of 13 feet deep when the depth reaches 13 feet. Consistency of drain type and size are automatically maintained such that a drain does not decrease in size in a downstream direction and drain type will progress continuously from street to pipe to rectangular channel as necessary.

3.4.3 Program Requirements

The VCRat program is written in FORTRAN and compiled to run on PCs with minimal memory and storage requirements. Program VCRat 2.2 was generated by revising and linking together a number of historic FORTRAN modules with a Visual Basic interface to run a composite input data file combining subarea information with rainfall and infiltration data. Historic operations such as hydrograph “fattening” and routing that were performed with separate codes were combined in VCRat 2.2 so that all of the operations could be done in one simulation. These options continue to be incorporated in the most current version of the program, VCRat 2.6, which replaced VCRat2.5.

A more user-friendly version of the program, VCRat 2.6, is a Windows-compliant program developed using Visual Basic.Net. Minimum requirements are Windows 2000 or Windows XP operating system with 256 MB of RAM and minimum 10MB hard drive capacity for storing the resultant output files. VCRat2.6 was revised to run on PCs with the Vista Operating System which prevents programs from writing to the root directory. Microsoft .Net Framework is required for VCRat 2.6 to run. Microsoft .Net Framework can be obtained through Windows Updates, from <http://www.microsoft.com> or by following the instructions provided in the help file provided with VCRat 2.6. The current program does not run on computers with Windows 64-bit operating systems.

3.4.4 Computer Output

Typical computer input and output data shown in [Appendix B](#) includes appropriate subarea input data, subarea Q's, and confluence Q's. At specified confluence structures, storm times and coincident Q's at the peak in each tributary are listed, and the time of peak and combined Q leaving the confluence are shown. Typical printed outputs of hydrograph data are also shown in [Appendix B](#). Hydrograph data in an output file may be exported to be used as input for future runs.

Because of the conceptual model of flow where the program shows the results for a subarea before routing is done, if the model has successive subarea input data lines the possible attenuation of the peak due to channel routing is often obscured by the addition of another subarea hydrograph to the channel flow. If the user would like to see the effects of channel routing on the peak, a placeholder node can be added after the subarea node of interest and the flow peak printed out in the cumulative output for the placeholder node will show the routing effects.

3.4.5 Limitations of Program

The following computations should be specified separately and cannot be combined with any other computation:

1. Compute a subarea hydrograph and add it to other flow in drain.
2. Add two hydrographs at a confluence.
3. Read hydrograph data input.
4. Modify a hydrograph at a relief drain junction and transfer a portion of the flow to the other drain.

A maximum of 200 points during the standard 1,500-minute-long storm period may be used to define rainfall mass curves and these same points must be used to define a storm runoff hydrograph. Selected times in the following table for storm patterns J through L apply to the 100-yr hydrograph data points. Mass curves for smaller design storms only have 40 points.

100-Yr Hydrograph Discretization

Time Period Minutes	Point Spacing Minutes	Number of Points
0 - 1000	100	11
1000 - 1100	50	2
1100 - 1130	10	3
1130 - 1300	1	170
1300 - 1400	10	10
1400 - 1460	20	3
1460 - 1500	40	1
		Total 200

3.4.6 Areal Reduction Simulations

Areal reduction reduces the peak flows to reflect the fact that the storm cells yielding the maximum historic intensities in the design storm rainfall mass curves are limited in extent. In the VCRat method, any watershed greater than about 1 sq. mi. is considered to have spatially variable rainfall that will result in a reduced peak flow/area ratio compared to a smaller watershed with the same runoff characteristics. The VCRat 2.2 program applies a reduction factor to the 100-yr mass curve intensities and runs the model a second time to show the AR peak at one location of interest per run. If repeated for a number of locations, an AR curve can be developed that can be used to estimate the AR values for intermediate locations and nearby watersheds.

The VCRat 1.0 program had the capability of doing simulations that would automatically perform areal reduction on the rainfall data to provide areally-reduced flows at a specific model location. However, VCRat 1.0 did not incorporate hydrograph fattening and detention basin routing capabilities that are part of VCRat 2.2. In order to use VCRat 2.2 for areal reduction simulations, the modeler must remove any fattening and basin routing operations from the VCRat 2.2 input file. If a basin exists in a model where an areal reduction run is desired, the procedure is as follows:

1. Apply appropriate areal reduction factors and fattening to the flow entering the basin.
2. Export the resulting basin outflow hydrograph
3. At the node corresponding to the basin, delete the basin routing info, clear the hydrograph bank, and import the basin outflow hydrograph to that node.
4. The AR run will then proceed correctly.

VCRat 2.6 does not currently have the capability of doing areal reduction simulations. The rainfall mass curves used in the AR run must be the 200-pt A97, B98, and C99 100-yr storms or the T01 50-yr storm (similar to K zone 50-yr rainfall) for the areal reduction simulation. AR should only be done for watersheds with a cumulative area of 640 ac or greater. The AR factor for a watershed of 600 ac is assumed to be 1.0. Between 600 and 640 ac, the AR factor varies linearly with area up to the calculated value for an area of 640 ac. The way to turn on areal reduction using VCRat 2.2 is do the following to the first 006 data input line:

```

          10          20          30          40          50          60          70
1234567890123456789012345678901234567890123456789012345678901234567890
006  8822   1A 040  0  7612B981 1550005940                               G1 7

```

The “1” in column 65 prints the heading at the top of the input file in the output file. In column 67 at node 1A, any number from 1 to 7 will perform Areal Reduction. In column 64, “G” clears the memory of all the hydrographs stored in the system (this is not required when only running a single model at a time). Areal reduction will occur with or without the column 64 command.

At the point where AR results are desired, areal reduction is specified for a VCRat2.2 model location on an A line by entering a “1” for AR in column 67. If AR is desired for a B line, a “2” is used in column 67. When this is done, for any proceeding node on the specified drainage line, including dummies and confluences, the program recognizes that areal reduction is needed and will use the embedded rainfall reduction factors in the VCRATP.for program to obtain an Areal Reduction Factor for Rain (ARr). The

factor will be applied to the model rainfall mass curve to reduce the intensities for the subareas in the line of interest. For this example provided in [Appendix B](#), areal reduction was set for node 30AB. The program ran normally until it reached node 30AB and recognized the command for areal reduction. VCRat 2.2 then ran again with reduced rainfall rates to provide an areally reduced peak. The output then shows the non- AR and AR results for the model.

```
1234567890123456789012345678901234567890123456789012345678901234567890
```

```
006 8822 30AB030 B985 1050002630200 800 11 1035035
```

```
006 8822 31A 030 23 1608B98 2
```

The AR factor for flow (AR_q) is calculated as the areally-reduced flow/unareally-reduced flow or in this case 1,467/1,583=0.9267 for an 851 ac watershed (from Appendix B). Another characteristic of an AR run is that the flow values up to the point where AR has been specified (1 in col 67) cannot be used to calculate AR_q values because the AR_r value is only good at the specified node point. Also, the flow values below where AR was specified are not valid model results and cannot be used for design or in AR_q calculations.

If a hydrograph is printed for the node where AR is requested, the reduction factor printed at the top of the hydrograph is the AR_r value for the node location, not the AR_q value. The AR_q value has to be hand calculated from the model output. The depth-area-reduction curve used in the VCRat program is obtained from the 1967 Hydrology Manual (Source: USDA SCS Engineering Handbook, Supp. A 1964). The VCRat2.2 program can be obtained from the District's Hydrology Section.

The output file for a VCRat2.6 run includes the VCRat2.2 format input file at the end. The VCRat2.2 input data file can be extracted from the VCRat2.6 output file and used for AR runs. If there is a need to do an AR run using VCRAT 2.2 for J zone rainfall (copying VCRAT 2.6's VCRAT 2.2 input file), VCRAINJZ.DAT file provided in [Appendix E](#) should be used in the model run. AR curves developed for 100-yr models are applied to the peak flow results for other recurrence intervals such as the 5-, 10-, 25-, and 50-yr storms. It is assumed that the AR factors do not vary with design storm recurrence interval. Additional info on areal reduction simulations is provided in [Section 3.12.14](#).

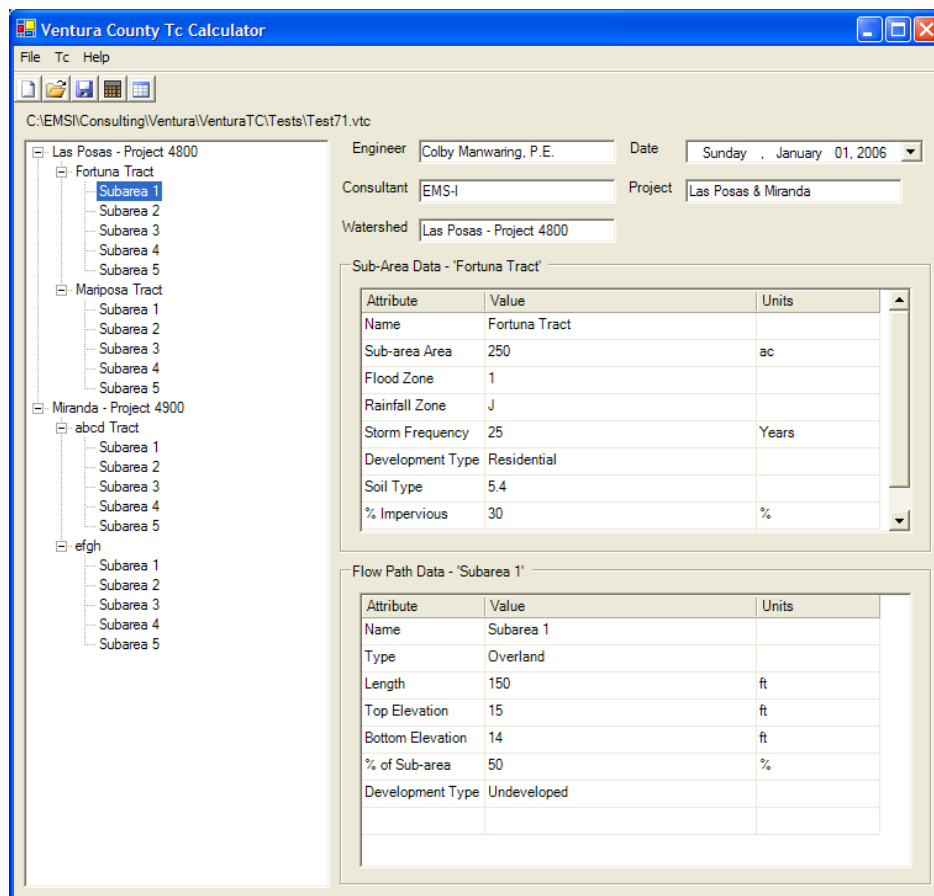
The task to include areal reduction simulations in VCRat 2.6 has been incorporated into the Hydrology Section's workplan as of 2010. The effort will require extensive modifications to the code, however, and so will not be done by the time this Manual receives Board approval. It is expected that the modified program will be available sometime in calendar year 2011.

3.5 TIME OF CONCENTRATION (T_c) CALCULATIONS

The iterative hand calculations that were required to provide T_c data for the VCRat model were replaced in 2006 through the development of a T_c calculator program. The T_c calculator program was developed using Microsoft Visual Studio.NET 2003 and was written in Visual Basic programming language. The program was designed to run on Microsoft Windows 2000/XP with .NET Framework, and should run on any newer Microsoft Windows OS platforms. The current program does not run on Windows Vista 64-bit machines.

The interface uses a data tree style of organizing the watershed with its sub-areas and flow-paths and is a user-friendly tool to facilitate hydrologic studies in Ventura County. A complete report describing the T_c Calculator methodology (EMSI, 2006) is provided with the VCRat2.6 program download.

The Tc Calculator program is a self-contained executable intended to be run from a folder on the local PC's harddrive after a simple installation process. It is not guaranteed to work if launched across a network. The following files are required for program execution: VenturaTc.exe – the program executable; and VCRAIN.DAT – data file containing soil and rainfall data compatible with VCRat 2.6- available from the [District's website](#). These two files may be placed in any folder on the local computer. The program is run by executing VenturaTc.exe (double-click this file or create a desktop shortcut that can be double-clicked).



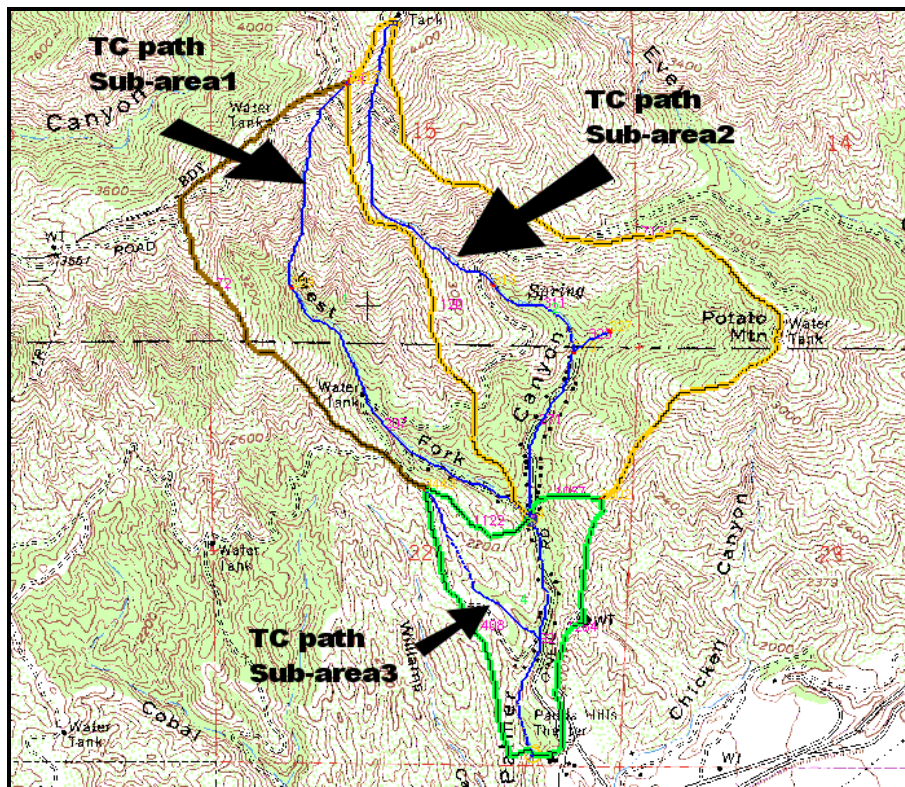
Sample Tc Calculation Window from Tc Calculator

3.5.1 Tc Calculation Methodology

Information about the watershed and its subareas is collected according to the guidance in this manual. It is then required that the subareas be subdivided into flow-paths segments. These flow-paths are created according to the type of flow that exists or the proposed type of flow. After suitable data are collected and the subarea is divided into an appropriate number of flow-paths according to the various types of flow, the calculation of a time of concentration is done. This process will be described briefly below

along with a description of the preparatory work that is done to collect data. The process is described in more detail in the Tc Calculator Manual provided with the program.

Collection of data usually involves obtaining a representation of the terrain and creating a map of the watershed and subarea boundaries, sub-dividing the subareas into segments that contribute to each time of concentration flow-path.



Sub-areas with Tc flow-paths

As shown in the figure above, the longest flow-path in each subarea must be determined for Tc calculations. The flow-path in each subarea should be subdivided based on flow type (overland, natural channel, street, pipe, etc.) and the direct runoff area to each segment determined. This flow-path information and runoff data are the basics needed for the calculation of Tc with the Tc Calculator program.

Travel times are based on flow velocities associated with overland flow and flow in various watershed conveyance facilities modified by wave velocity considerations. Overland flow on a canyon side slope or undeveloped hillside is dependent upon slope, topography, cover conditions, and travel distance. Previous versions of this Manual did not provide a length limit on the overland flow-path used in the Tc calculation. Some calculations used flow-path lengths of 1,500-ft or more. The District is currently limiting the maximum overland flow-path for undeveloped areas to 1,000 ft for the following reasons:

1. The selection of an overland flow-path length is critical to the results because the overland flow-path portion of the calculation is generally responsible for most of the time in a resultant Tc, and because the resultant peak flow is very sensitive to the Tc.

2. It is likely that overland flow in undeveloped areas will be channelized after a flow distance of 1,000-ft.

Minimum overland flow velocity for undeveloped overland flow areas is shown in Exhibit 8 for various frequency storm events. These are minimum values that apply to slopes with dense vegetal growth. Using engineering judgment, velocities assumed for bare slopes and slopes with sparse vegetal growth should be increased consistent with the reduced surface resistance. The length of the undeveloped overland flow-path is further limited for mountain channels (slopes greater than five percent) by evaluating the point where there is sufficient tributary area to produce flow with a scouring velocity of six to eight feet per second in a natural mountain channel. If there is enough flow to exceed the scour velocity, it is assumed that channelization will develop and the overland flow assumption is invalid.

It is recognized that because the flow used to determine if the scour point has been reached is based on a pro-rated value from the entire subarea under evaluation, the scour velocity point will change depending on downstream conditions. The scour velocity point will also change with design storm due to the changes in peak intensities and their effects on peak flows. The current 1,000 ft limit will minimize these effects to some extent. The maximum overland flow-path length in developed areas is limited to 200-ft. These limits are embedded in the Tc Calculator Program and are discussed in more detail in the Manual provided with the Program.

Parking lots or large paved areas such as are found in commercial or industrial areas have a sheet flow velocity of a minimum of 1.0 foot per second for 50- and 100-yr storms. For these storms, sheet flow in areas of extreme low relief where overland flow at 1.0 foot per second may exceed street flow velocity, an overland flow velocity of 0.5 foot per second for these storms may be acceptable if previously approved by the Hydrology Section. The minimum velocity for these areas for 25- and 10-yr storms is 0.5 fps.

In determining average travel velocities for streets, pipes, and open channels, uniform flow is assumed. Average velocity data for natural mountain channels are shown in Exhibit 9; natural valley channel velocity data are shown in Exhibit 10. To account for drops and falls that develop in steep mountain channels, slopes greater than ten percent determined from topographic information must be adjusted. Effective slopes were determined empirically from a survey of typical steep mountain channels in the San Gabriel Mountains and a correction curve is shown in Exhibit 7.

Using Manning's Equation and typical street cross sections from the Ventura County Road Standards Manual, normal velocities in common street sections were determined. The velocities were related to discharge and slope, and graphs of the data are shown in Exhibits 11a-d for streets of 32- and 40-foot widths, and six- and eight-inch curbs. Since graphs, tables, and programs are readily available from a variety of sources for determining flow velocities in pipes and lined channels, these data were not included in this manual.

Because a hydrograph represents a translatory flood wave, travel time between subarea collection points is based on wave velocity. Wave velocities for rectangular channels, trapezoidal channels, and pipes were determined theoretically by the Los Angeles County Flood Control District and checked using controlled experiments. The relationships are shown in Exhibits 12a-c. Wave velocities for natural mountain channels, natural valley channels, and for street sections are considered to be 1.5 times the average velocity.

3.5.2 Calculation Process

Once a watershed has been delineated and flow-paths determined, the Tc calculation is performed based on flow-path properties (length, slope, type, etc.), rainfall, and soil data. Several parameters are needed for input as follows:

1. Zone number, rainfall zone, and soil type are obtained by locating the area on the maps included in this Manual. If a weighted soil number is calculated for the subarea, it must be rounded to the nearest whole number.
2. A desired rainfall frequency storm is selected for analysis or all frequencies can be done.
3. The fraction of effective impervious area is determined from the actual impervious area from calculations from development plans and [Exhibits 14a and b \(Appendix A\)](#).
4. A generalized development type of the area is needed (Residential, Commercial, Undeveloped, Industrial).
5. Total area of the basin. (20-80-acres recommended)
6. Each subarea is divided into flow-path segments and each flow-path is specified as one of the following types and required data must be entered: Overland Flow; Natural Channel; Street; Pipe; and Channel.
7. Areas, lengths, and beginning and ending elevations for each flow-path must be calculated and entered.

A Tc between 5 and 30 minutes based on the required range in the Hydrology Manual is assumed. The rainfall intensity for the assumed Tc is obtained from the file VCRAIN.DAT compatible with VCRat 2.6 corresponding to the rainfall zone and design storm recurrence interval. A Runoff Coefficient for the basin for pervious area is determined automatically from the file VCRAIN.DAT through linear interpolation of the values in the file. The runoff coefficient is dependent on the given intensity and soil type. The runoff coefficient that is used for the subarea is altered by the given % effective impervious area as follows:

$$C_{\text{total}} = C * (1 - \% \text{ Impervious}/100) + 0.95 * (\% \text{ Impervious}/100)$$

Total flow for the assumed Tc is calculated using the rational equation.

$$Q_{\text{total}} = C_{\text{total}} * \text{Intensity} * \text{Area}$$

Amount of flow for each flow-path is calculated.

$$Q_{\text{segment}} = Q_{\text{total}} * \text{Fraction of total area}$$

Wave velocity is calculated for each flow-path except for the initial overland flow-path segment. Travel time for each flow-path is computed by:

$$\text{Travel Time} = \text{Length} / \text{Wave Velocity}$$

Summation of travel times is equal to the calculated T_c . If the calculated T_c is within 0.5 minutes of the assumed T_c , it can be used in the hydrology calculation. If calculated T_c is not within 0.5 minutes of assumed T_c , the assumed T_c is adjusted, then the above steps are repeated:

The resultant T_c from the calculations must be greater than 4.5 minutes and less than 30.5 minutes. Subarea extents may have to be adjusted to produce a T_c in this range. Specific steps in launching and using the T_c Calculator program are described in the Time of Concentration Calculation Report (EMSI, 2006).

3.6 VCRAT PROGRAM INPUT DATA

The structure of the VCRat 2.2 program is complex and input data must be in the correct fields in order to get the program to execute successfully. Input data must be included in the order shown in [Appendix B](#). VCRat 2.6 has been designed to be a user-friendly program that does the required formatting and builds the input data file for the user with the following features:

1. System Data.
2. Page Heading Data.
3. Hydrograph Header Data. (Optional)
4. Hydrograph Import Data. (Optional).
5. Location Data.
6. Subarea Data
7. Confluence Data
8. Channel Routing Data
9. Runoff Coefficient Curve Data.
10. Rainfall Mass Curve Header Data.
11. Rainfall Mass Curve Data.

Additional features provided in the VCRat 2.6 program include the following:

1. Performs the yield calculation for a watershed given a rainfall depth and SCS Curve Number
2. Header information can be assigned to each model node as desired and are no longer limited to nodes where hydrograph data output is desired. Header information is printed in the output so that it is not necessary to refer to the input to find the hydrograph or node description information.
3. Can launch the T_c calculator in the VCRat 2.6 data input window
4. Provides additional channel routing and time-of-peak data in the output
5. Indicates in the output where a routing channel type has been changed by the program during machine routing
6. Provides additional reservoir routing info in the output
7. Can run an existing VCRat 2.2 file without altering it

8. Can import an existing VCRat 2.2 file, evaluate it to see if any complex routing assumptions or data outside the recommended ranges were used, and provide a report to the modeler
9. Displays the input and output hydrographs in a data window
10. Has additional error checking capabilities over VCRat 2.2
11. Has an extensive help file that explains the various parameters and options
12. Can import and export hydrographs in Excel-compatible formats for additional analysis

Detailed information about the program is provided in the VCRat 2.6 Help Manual available after launching the program. The technical details of how the program operates are still the same as provided in the description of VCRat 2.2 included in [Appendix B](#).

3.7 COMPUTATION PROCEDURES

3.7.1 Storm Rainfall Relationships

The VCRat 2.2 and VCRat 2.6 interfaces construct an input data file from the subarea and mass curve data input files and launch the VCRATP.EXE FORTRAN executable. The VCRATP.EXE executable has numerous subroutines (names provided in [Appendix B](#)) that are called to perform the hydrologic calculations. The VCRat programs interpolate rainfall mass curve data entered as data input in the VCRAIN.DAT file and constructs a system storage table of accumulated total rainfall by one-minute increments from zero rainfall to the time of the last point indicated by input data. Design storm rainfall mass curve data included in VCRAIN.DAT have cumulative rainfall totals as shown in the following table. These rainfall totals are obtained after building the VCRat rainfall hyetograph using rain frequency analysis results in the alternating block method and are not intended to match N-yr 24-hr rain depths across the County.

Design Storm Fourth Day Rainfall Total, Inches

Rainfall Zone	100-Year Storm	50-Year Storm	25-Year Storm	10-Year Storm
J	7.00	5.0	3.91	3.17
J' (J prime)	6.66	6.0	5.28	4.38
K	10.60	8.0	6.41	5.53
L	15.00	11.0	8.81	7.21

The average rainfall intensity for any specified duration (time of concentration) at a selected storm time is calculated as follows:

1. Determine the cumulative storm rainfall from the mass curve at the selected time
2. Determine the cumulative storm rainfall at an earlier storm time by the length of the specified duration
3. Rainfall intensity in inches per hour is calculated as the incremental cumulative storm total, times 60 (minutes in an hour) divided by the specified duration.

For the design storm, rainfall intensities are reduced if computations for other than the fourth day (maximum day) rainfall are specified. The reduction factor is 0.10, 0.40, 0.35, and 1.00 for first through fourth day, respectively.

3.7.2 Rainfall-Runoff Relationships

The system linearly interpolates runoff coefficient curve data as a function of intensity entered as data input in VCRAIN.DAT and constructs a system storage table with up to 14 points, with one curve for each soil type. Infiltration rate (loss rate) data may be specified as data input as an alternative to runoff coefficient curve data. Through use of this table, runoff rate may be computed for a specified rainfall intensity by either (a) runoff coefficient times rainfall intensity or (b) rainfall rate minus infiltration rate (loss rate).

The watershed acreage, effective impervious area of the watershed, and time of concentration data plus systems tables provide the information necessary for the system to compute watershed discharge using the rational formula. The following equations are used to compute runoff rates:

$$Q = CIA$$

$$R = I-f$$

$$C = R/I = (I-f)/I \text{ (No Impervious Area)}$$

$$C_{imp} = P(0.95) + C(1-P)$$

$$Q = [P(0.95)I + (1-P)R]A$$

where:

A = Watershed area in acres.

C = Runoff coefficient.

f = Infiltration rate in inches per hour; f varies with intensity for each soil.

I = Rainfall intensity in inches per hour, assumed to be constant

P = Percent imperviousness/100.

Q = Flow rate in cubic feet per second.

R = Runoff rate in inches per hour.

3.7.3 Effective Impervious Values

Effective imperviousness is less than total impervious area because some of the impervious area runoff passes through pervious areas where some infiltration can occur. For example, roof drains that discharge onto a lawn may see a reduction in runoff due to infiltration. Roof drain discharge onto a driveway provides no opportunity for infiltration and thus the effective impervious area would be the same as the total impervious area. Compaction is also generally done uniformly across the development site, decreasing the amount of infiltration that can occur, even in a pervious area. These and other applicable factors should be evaluated by the engineer prior to performing the runoff calculations.

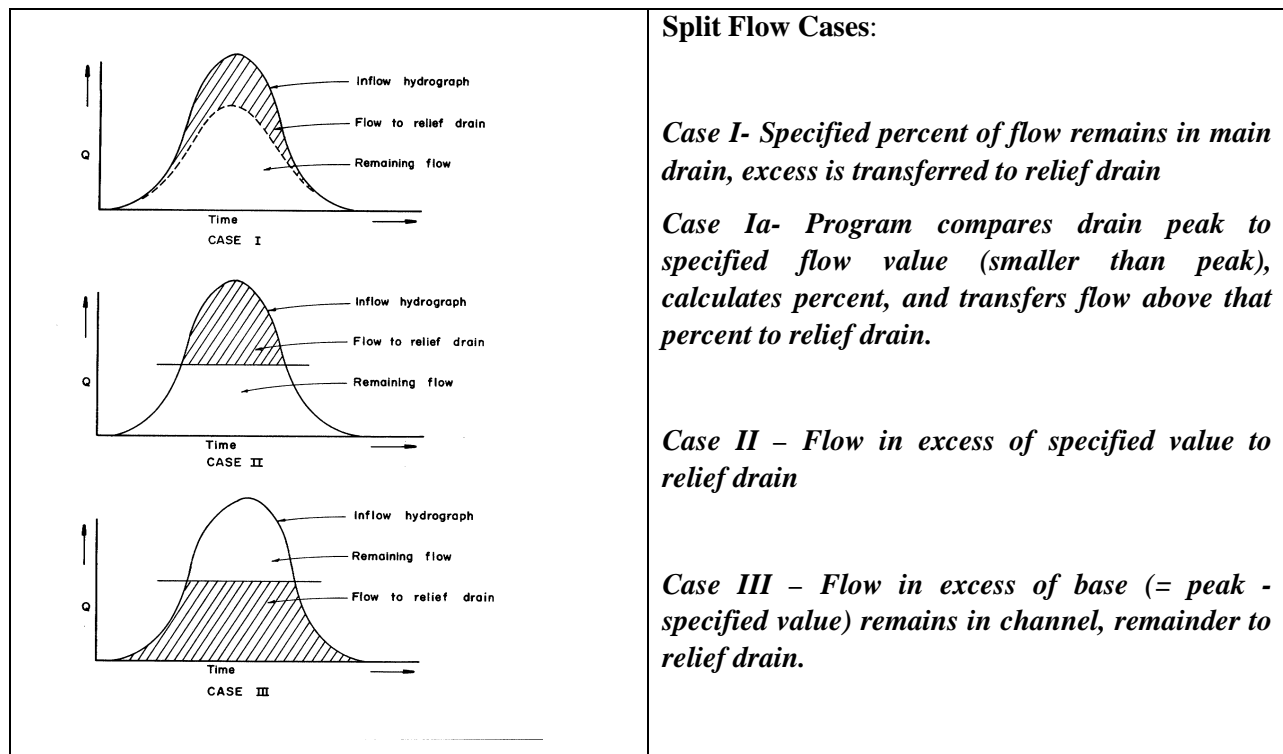
Computations evaluate runoff from pervious and impervious portions of the watershed separately. It is assumed that 5 percent of rain falling on impervious surfaces will not appear as runoff due to evaporation and surface storage. Previous versions of the Hydrology Manual only provided for three recommended values for effective imperviousness for residential, commercial, and industrial development. In this Manual, additional values are provided for various development densities and types. Recommended effective impervious percentages generally used for projected ultimate development are provided in [Appendix A in Exhibits 14a and b](#) for use in VCRat modeling. However, it is the responsibility of the hydrologist to evaluate the effective impervious area of their project if it is different than the recommended values.

3.8 CONFLUENCE AND DIVERSION STRUCTURE FLOWS

Flow rates calculated for a subarea at specified storm times provide the storm runoff hydrograph from the subarea. At each collection point, subarea hydrographs are added to the flow present in the drain system from upstream portions of the watershed. Where detailed confluence output is specified, coincident peak flow rates are printed at the peak flow rate time of each hydrograph for use in hydraulic design of the drain system. Confluence inflow and outflow hydrographs are printed and output files generated as needed. Confluence information provided by the program includes the following three sets of data:

1. At the time of peak flow in primary drain, flow information includes primary peak flow, combined flow, and flow in lateral.
2. At the time of peak flow in lateral drain, flow information includes lateral drain peak flow, combined flow, and flow in primary drain.
3. At the time of peak combined flow in primary drain, flow information includes combined peak flow, primary drain flow, and lateral drain flow.

At the junction of two drains where one of the drains functions as a relief drain, the hydrograph may be separated into two parts as shown below. The input data required to specify the various options are shown in [Appendix B](#).



3.9 FLOOD ROUTING

Specific channel sections, including natural mountain channel, natural valley channel, standard street section, circular pipe and rectangular or trapezoidal channels, may be specified as water conveyances. The pipe diameter or channel side slopes, bottom width, and depth may be specified. In the case of a trapezoidal section with an unlined bottom, the maximum velocity to control scour through use of drop structures, and various composite lining roughness values may be specified. The channel length and slope plus channel section data provide the information necessary for the system to route the hydrographs in a drain from one subarea to the next. The program uses the Modified Puls or storage-indication method for channel routing.

3.9.1 Machine Routing

If machine routing is specified, the system will begin flood routing in a drain at the upstream limits of the watershed using a 40 ft wide street section and will automatically change from street to pipe section when flow depth reaches the property line (at the sidewalk limit), from pipe to rectangular channel when pipe diameter of 8 feet is exceeded, and from the hydraulically most efficient rectangular section to a rectangular section with maximum depth of 13 feet when that depth is reached. To maintain consistency of drain type and size, the system checks the type of drain used in the preceding upstream channel reach, and retains that section if it is large enough to contain the flow. In a rectangular or trapezoidal section, the upstream reach bottom width is retained until flow depth increases by more than one foot from the previous reach depth. Pipe sizes used by the system increase in 3-inch increments from 2.0 to 8.0-feet. Rectangular and trapezoidal bottom widths used by the system increase in 2-foot increments from 2 to 30

feet, 10-foot increments from 30 to 80 feet, 50-foot increments from 100 to 300 feet, and 100-foot increments from 300 to 1000 feet.

Machine routing is selected by leaving the channel type blank in the 006 data line or subarea editing window in VCRat but entering a channel length and slope to be used in flow routing. Additionally, a channel type can be selected in the 006 data input line or editing window but the width or diameter left blank. In this case, the program will size the specified channel. If the flow is too large for the type of channel selected, the program will automatically select a larger channel type and size it appropriately. VCRat 2.6 will show the channel change by adding the “#” symbol to the output. However, if street routing is specified with an assigned street width the program will not select a different type of channel—the flow, no matter how large, will be analyzed with the street routing equations.

3.9.2 Wave Velocity and Flood Routing

Because the VCRat hydrograph constitutes a translatory flood wave, routing is performed at wave velocity. At the lower end of each reach the hydrograph is checked for a time shift and redefined by interpolation to obtain flow rates at storm times as specified by hydrograph data points. The hydrograph, as affected by channel storage, is next computed using a reservoir routing method (Modified Puls), with time periods for routing determined by spacing of rainfall data input.

3.9.3 Flood Routing Equations

The following equations are used for flood routing purposes:

Equation	Application
$T=L/(60V_w)$	Travel time in minutes
$V=Q/A$	Velocity, ft/sec
$V=5.6Q^{0.333}S^{0.5}$	Natural Mountain Channels
$V=(7+8Q^{0.352})S^{0.5}$	Natural Valley Channels
$V=11.9Q^{0.191}S^{0.411}$	40-ft Street Sections
$V=1.486R^{0.667}S^{0.5}/n$	Manning's Equation for pipes, rectangular, and trapezoidal channels
$D=B/\{2[(Z^2+1)0.5-Z]\}$	Most Efficient Open Channel Section
$n=[n_1B+n_2(2\ell)]/(B+2\ell)$	Composite Lining of Trapezoidal Channel
$V_w=1.5V$	Wave Velocity- Natural Mountain and Valley Channels and Streets
See Appendix A	Wave Velocity, Partially Full Pipe
See Appendix A	Wave Velocity, Rectangular and Trapezoidal Channels
$I_1+I_2+2S_1/t-O_1=2S_2/t+O_2$	Storage Routing Equation
$\Theta=4\sin^{-1}(D/d)^{0.5}$	Angle in Radians for Partially Full Pipe

Where:

A	=	cross section area in square feet
B	=	channel bottom width in feet
D	=	flow depth in feet
D	=	pipe diameter in feet
I	=	inflow to channel reach in cfs
I ₁	=	instantaneous inflow at beginning of time period t.
I ₂	=	instantaneous inflow at end of time period t.
L	=	length of channel reach in ft
<i>L</i>	=	length of wetted channel wall in ft
n	=	channel roughness coefficient
n ₁	=	roughness coefficient for channel bottom
n ₂	=	roughness coefficient for channel walls
O	=	outflow from channel reach in cfs
O ₁	=	instantaneous outflow at beginning of time period t
O ₂	=	instantaneous outflow at end of time period t
Q	=	flow rate in cubic feet per second
R	=	hydraulic radius
S	=	slope of channel reach in feet per foot
S ₁	=	channel storage at beginning of time period t
S ₂	=	channel storage at end of time period t
T	=	travel time in minutes
t	=	time period for routing purposes in minutes
V	=	mean velocity in fps
V _w	=	wave velocity in fps
Z	=	channel side slope computed as horizontal projection of wall divided by depth in feet per foot

3.10 FLOOD ROUTING METHODS AND LIMITATIONS IN VCRAT

The use of the Modified Puls method of routing for channels in VCRat treats the channel like a long linear reservoir. It is not possible to enter in irregular channel cross-sections with overbank areas in the routing input so the routing scheme applied to prismatic channels results in relatively little peak flow attenuation. The largest peak attenuation observed in the VCRat model is for relatively wide channels

with low slopes such as the Camarillo Hills Drain adjacent to the Camarillo Airport. A peak flow of about 4,500 cfs at the upstream end of the 1-mi reach was reduced to about 4,250 cfs at the downstream end, a reduction of less than six percent. When using the hydrology data in a hydraulic model, the attenuation can be simulated in the model by applying the upstream and downstream discharge and using linear interpolation for any intermediate cross-sections. Additional issues and methods for modeling hydraulic constraints in the drainage system are discussed in Sections 6.1 and 6.2.

If significant peak attenuation is suspected of occurring due to extensive overbank flow in a study reach, the hydrograph from the hydrology model can be exported and a better estimate of attenuation obtained from an unsteady-state hydraulic model run.

3.11 WATERSHED YIELD ADJUSTMENT

The MRM hydrograph produces a conservative peak flow for use in drainage system design. The hydrograph has a relatively short duration peak due to the shape of the design storm hyetograph. Because of this, the volume under the hydrograph is generally smaller than the watershed yield obtained through application of other hydrological methods such as the NRCS (formerly SCS) curve number (CN) approach. Because the hydrograph volume is critical in detention and storage facility design, the District has developed the VCRat 2.2 and 2.6 programs to adjust the yield of the resultant hydrograph (often called hydrograph “fattening”) for use in basin design. This methodology should not be used in the design of regional detention and storage facilities. For those facilities, other standard hydrologic methods pre-approved by the District should be used to generate the design inflow hydrograph so that the outlet works design is done correctly. Design of these facilities should be undertaken after consultation with District staff.

For small developments and basins, however, the VCRat method can be used to generate a yield-adjusted design hydrograph as shown in [Appendix B](#) with the following steps:

1. Use VCRatX.X to produce an inflow hydrograph for the design storm and tributary watershed.
2. Find the total design storm rainfall depth (N-yr 24-hour) at the centroid of the tributary watershed.
3. Use shapefile or maps supplied by the District to find the average soil type in the watershed.
4. Generate a list of land uses, associated areas, and NRCS AMC II CNs from Exhibits 14a and b that represents the weighted soil type from Step 3 and calculate an areally-weighted average CN for the watershed.
5. Use Exhibit 13 to find the watershed yield in inches using the weighted CN from Step 4 and the total design storm rainfall depth from Step 2.
6. Enter this as a yield adjustment factor in the model data input file above for VCRat 2.2 or VCRat 2.6. In 2.6 the user can also enter the rainfall and curve number directly and the program will calculate the resultant yield following the assumption that the initial abstraction is 0.2 times the total abstraction.
7. As a check, the volume of the output hydrograph should equal the yield adjustment factor times the area of the tributary watershed after the units are converted.

VCRat 2.2 and VCRat 2.6 perform the yield adjustment by increasing or decreasing the hydrograph ordinates adjacent to the peak until the desired volume is achieved. The peak of the hydrograph is not

changed but flow values in the vicinity of the peak can be increased up to the peak if necessary to provide the desired volume. AMC II conditions best represents yields from District watersheds.

3.11.1 VCRat Yield Adjustment – Advanced Topics

The yield under the VCRat hydrograph is smaller than the watershed yield calculated through an NRCS CN approach except for areas of the County where the 100-yr 1-day rainfall depth is 6 inches or less. This is mainly in the vicinity of the City of Oxnard. If the hydrograph yield is adjusted for watershed in that area, the VCRat hydrograph volume can even be decreased slightly depending on the type of development. As the user are trying to match the yield from the NRCS CN method during yield adjustment, a small decrease in volume is considered acceptable.

In a related issue, recently it was noticed that the algorithm used to adjust the hydrograph volumes to match the calculated NRCS yield could result in small negative values at early and late times in the hydrograph. The VCRat 2.6 interface was then revised to check the yield adjusted hydrographs for negative values and reset them to 0 if they are found. This process only changes the hydrograph volumes by a small amount so has no effect on facility design.

As the VCRat hydrology model only has a simulation length of 24-hrs, some portion of the runoff hydrograph is truncated at the end of the run. Because the peak rainfall does not occur until 80% of the one-day run has occurred, for large watersheds with significant lag times the peak can occur relatively late and the truncation of the receding limb of the hydrograph includes more volume. This effect causes unrealistic shapes of hydrographs during yield adjustment. The District has found that it is not possible to create a realistic yield-adjusted hydrograph for watersheds that are bigger than 70 to 80 sq mi. If a hydrograph is needed for mainstem locations with large tributary areas it is necessary to use another hydrology model to provide it as discussed below in Section 5.

The CNs used to calculate the yield to be applied to the VCRat hydrographs were revised to vary with infiltration rate more smoothly than the previous CNs provided in the District's 2006 Manual. Additional land use categories were also added to reflect General Plan land use files provided by the County and Cities.

3.12 VCRAT 2.2 PROGRAM COMPLEXITIES

The flexibility of the VCRat 2.2 and 2.6 programs has led to a number of possibly unexpected results from historic models as described in the following subsections.

3.12.1 Machine Routing Specified But n Value or Sideslope Specified

A model can be prepared using the machine routing capability by not specifying a channel type in a subarea data input line but providing a channel length and slope. If the user enters a channel side slope and/or an n value, this information is ignored until the routing routine has enough flow to select the use of an open channel. At that point, any specified values are applied to the channel.

3.12.2 Stage-Storage-Discharge Curve

VCRat expects the first point on a stage-storage-discharge curve to correspond to 0.0 ac-ft. If it is not 0.0 ac-ft, the program cannot interpolate the stage-storage curve correctly and provides erroneous results for the maximum spill elevation reached during outflow. This error can also affect the length of spill and

peak outflow information provided in the output. These errors do not affect the actual basin outflow hydrograph and routing results in the program.

VCRat 2.2 does not require the reservoir spillway elevation to be entered in the input file. This can result in inaccurate spill durations provided in the output because the program defaults to a spillway elevation of 0 feet. The peak flow of the output hydrograph is not affected by this.

VCRat 2.2 does not require sequential points in the stage-storage-discharge data to increase or be the same value. This allows the user to enter elevations that are lower than the previously entered values without causing a fatal error. This results in inaccurate routing through the reservoir by the program.

VCRat 2.2 or 2.6 does not warn the user that not enough data points were provided in the stage-storage-discharge curve so the upper limit of storage was reached during the run. At the time-step where the storage is exceeded the discharge is reset to 0.0 cfs and remains there for the rest of the simulation. The easiest way to make sure this does not happen during a run is to review the outflow hydrographs to make sure they do not drop to 0.0 cfs during the run. The modeler can also compare the basin inflow volume to the outflow volume to make sure they are similar. If the outflow volume is much less than the inflow volume, it is possible it is due to the outflow being set to 0.0 cfs when the maximum storage is exceeded.

3.12.3 Incorrect Rainfall Mass Curve Information

In certain cases, an incorrect rainfall mass curve designation can be entered into the data input file for VCRat 2.2 and the program might still run and not indicate any problems. For example, a curve B97 can be specified which is not defined in VCRain.dat. VCRat 2.2, when using mass curves beginning with a letter between A-I, only checks the numerical value following the letter (in this case 97) and thus would use rainfall curve A97 in this example (J' Zone- 100-Yr) instead of B98 (K Zone-100-Yr) for this subarea. Currently, the VCRat 2.6 import program also uses the number following the letter designation to determine which mass curve to use for a subarea but adds an issue/warning message to let the user know that there may be an issue with the design storm specification.

3.12.4 Specifying Width for Road Routing

It is possible to enter a width while specifying routing type 3 (which by default in VCRat uses the velocity-slope-discharge data for 40' road). The effect in VCRat 2.2 or 2.6 is to force the flow to stay in the road and not have the program automatically change to a pipe or a channel when the flow is large enough. This allows the modeler to simulate channel overflows that occur as street flow at road crossings such as in the Tapo Canyon channel in Simi Valley. It does not matter what the specified width value is- the resulting flow will be the same because the program uses the default 40' street velocity/flow depth info embedded in the program.

3.12.5 Pipe Diameter Not Specified But N Value Specified

VCRat 2.2 ignores the n value when it is entered and uses the default value of 0.012 when pipe flow is specified for the channel type but no diameter is specified. If flow is large enough for the program to select a channel, the n value is used by the program at that point.

3.12.6 Channel Type 5 with Depth or Velocity Specified

VCRat 2.2 ignores any maximum depth or velocity information associated with channel type 6 entered by the user if channel type 5 is specified. VCRat 2.6 provides a warning message about this if the file is imported.

3.12.7 Natural Valley or Mountain Channel Routing but N Value or Width Specified

If natural valley or mountain channel routing is specified but an n value or width is specified, VCRat ignores this information during the run. The embedded Manning's "n" value for Mountain Channel Routing is higher than Valley Channel because it is assumed that Mountain Channels have cobble bottoms or possible vegetation in the channel. Therefore, for a given slope and flow the velocity will be higher in a Valley Channel and could lead to higher downstream peaks.

3.12.8 Pipe Routing Specified and Diameter Specified

If pipe routing is selected and a specified pipe diameter is given, this forces the program to use that pipe size for the routing. If the diameter is not adequate to carry the calculated flow, the program will force flow through the pipe under pressure, with unrealistic velocities that affect travel times and routing calculations. Pipe diameters should generally not be specified in the input file unless they have been verified to be adequate to carry the design storm flow under open channel conditions.

3.12.9 Clearing Hydrograph Bank

Clearing of a hydrograph should occur before any other operation for that node. This needs to be done when importing a hydrograph or after performing a split and before using the lateral channel letter for a new tributary. Otherwise any hydrograph data stored in the program memory in the lateral will be added to the new subarea information and the peak flow results will be not be correct for the new subarea.

3.12.10 Importing Historic Models into VCRat 2.6

VCRat 2.2 and previous version models allowed more flexibility in the input data than is currently allowed. Some historic models have areas and Tc's that are outside of the currently allowed ranges of 5 to 300 ac and 5 to 30 minutes, respectively. Channel routing data is sometimes inconsistent such as specifying a Manning's "n" value for street routing. When imported into VCRat 2.6, the program will identify any out-of-bound data and routing inconsistencies and generate a list for the user.

VCRat 2.6 was programmed to import a 2.2 input file and run the model as long as the user does not attempt to revise a subarea that contains any inconsistencies. If the subarea selected for editing, VCRat 2.6 will force the user to change the inconsistent parameters and save the changes before running the file. There are several ways to change the file back to the historic model version so that the same results can be obtained as follows:

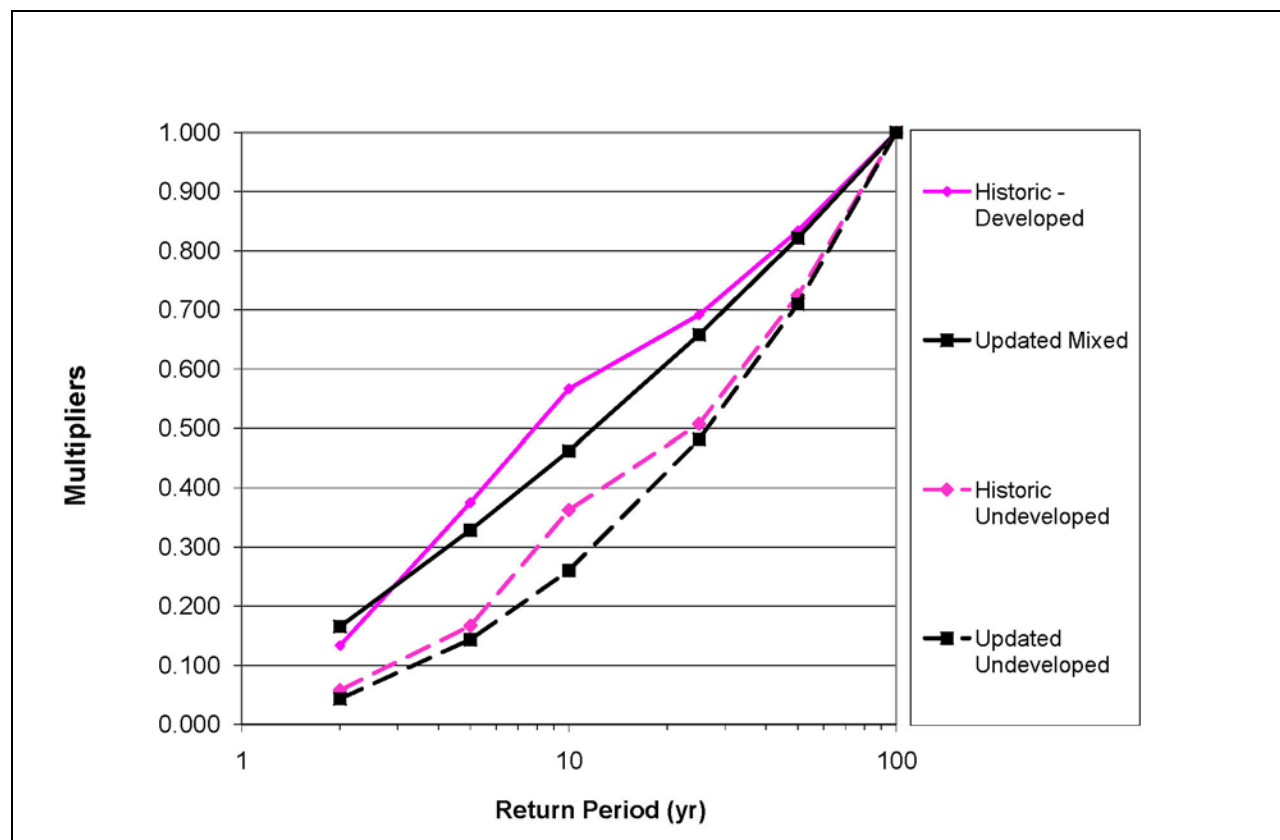
1. If a subarea has an area that is bigger than 300 ac, another subarea can be inserted above or below and the area can be split so that both input lines have the same Tc, soil, storm type, etc, and the total area matches the historic total.
2. For areas that are too small or Tc's that are outside the limits, revise the subarea with an acceptable number, Close VCRat 2.6, and open the *.vin input file with a text editor and change the affected data

back to match the historic file. Then reopen the file in the VCRat 2.6 program and run the model. Using a text editor to edit the VCRat 2.6 input file is also useful in limited cases where numerous stage-storage-discharge points must be entered into the file. When doing direct editing, however, it is always a good idea to save a backup copy in case the editing process somehow results in a corrupted file that will not execute.

3.12.11 Comparison of VCRat Results to Stream Gage Flow Frequency Data

As part of the Manual update, the design storm ratios or multipliers commonly used by hydrologists to estimate the other design storm peaks from a single model run were updated using stream gage flow frequency analysis results. Gage frequency results were separated into two categories based on land use, undeveloped and mixed (partially developed). The design storm ratios calculated from the frequency results were averaged for each category. The resultant ratios relative to the 100-yr storm are plotted in the following figure.

The data show that the updated ratios are relatively similar to the historic ratios except at the 10-yr storm level. It appears that the historic 10-yr ratios were adjusted upward to match the results from VCRat modeling. This has implications for design storm work because the 10-yr storm is commonly used to set the mitigation goal for detention facilities. If the VCRat 10-yr results are too high compared to stream gage design storm ratio, that means that mitigation facilities such as detention basins may not providing as much benefit as expected. If the VCRat model and associated design storm ratios are revised to be consistent with stream gage frequency analyses, mitigation facilities may have to be bigger.



3.12.12 NOAA Atlas 14 Updated Rainfall Intensities

The National Oceanic and Atmospheric Administration (NOAA) is currently working on an update of their Atlas 14 that provides design rainfall isohyetal maps for this area. Some jurisdictions use these maps to develop site-specific design rain data and prepare individualized hyetographs for use in design storm modeling. The District has no plans at this time to update the design storm mass curves used in the VCRat model or allow the use of Atlas 14 data to provide design rainfall.

3.12.13 Multiple Areal Reduction Calculations

It is sometimes necessary to apply areal reduction more than once to a drainage line to get final AR results at a study location. This occurs often when the study site is located downstream from a detention basin where the inflow to the basin has already had AR applied so that the basin outflow peak is correctly calculated. If the study site has a tributary area downstream of the dam that is several hundred acres in size, it is necessary to apply another AR factor appropriate for the net area downstream of the basin to the peak flows. In this case the AR factor is applied even if the net area is less than 640 ac because the total watershed area is greater than 640 ac. An example of how to do this is provided in [Appendix B](#).

Another issue is whether or not an AR curve developed for the unregulated watershed above the basin can be applied to peaks below the basin. Research by the District on this topic has indicated that if the inflow peak to the basin is significantly attenuated by the basin then an AR curve developed for the unregulated watershed is not appropriate for use in applying AR to the regulated subareas on the mainstem downstream of the basin. If the basin does not attenuate the inflow peak much then the AR curve developed for the unregulated watershed can be applied to the downstream subareas. It is relatively easy to do AR runs however, so when in doubt it is best to do additional AR runs to get the data points needed for the study.

3.12.14 Multiple Yield Adjustment Calculations

It is sometimes necessary to apply yield adjustments more than once to a drainage line to get final results at a study location. This occurs often when the study site is located downstream from a series of detention basins where the inflow to an upstream basin has already been “fattened” so that the basin outflow peak is correctly calculated. For the next downstream basin, the fattening factor is calculated by the following:

1. Calculate the yield of the outflow hydrograph from the upstream basin.
2. Calculate the yield of the tributary watershed downstream of the upper basin that provides flow to the downstream basin.
3. Sum the two yields, and convert that to inches by dividing the yield volume by the total tributary area upstream of the second basin. Use that as the fattening factor.

An example of how this is done is provided in [Appendix B](#).

3.12.15 Debris and Detention Basin Modeling Policy

The District’s policy is that only basins that were specifically designed to provide detention for the 100-yr storm, meeting all of the District’s design standards, can be included in hydrology models. Debris basins

that were primarily designed to capture sediment and result in emergency spillway flow for the 100-yr storm cannot be included in hydrology models even if they attenuate the inflow peak to some extent.

Detention basins are modeled by including stage-storage-discharge data in the model. The first discharge and storage data point must be a 0 for the interpolation routine to work correctly. More accurate results are obtained if the storage data points each represent no more than about 10% of the total basin volume up to the emergency spillway level. Points should be provided wherever the discharge and volume curves show large changes or discontinuities in slope. The design standard for detention basins are described in detail in the District's Debris and Detention Basins Manual (2005).

SECTION 4 HSPF DESIGN STORM MODELING

The work done to use continuous hydrology HSPF models for design storm modeling includes results for the Santa Clara Watershed (AQUA TERRA, 2009), and the Ventura River Watershed (Tetra Tech 2009 and VCWPD, 2010). The project to obtain design storm results for the Calleguas Creek Watershed Model is expected to be completed in 2011.

Many of the catchments in the Santa Clara and Ventura River watersheds been evaluated in those regional models and the results are contained in the reports posted on the District's website in the pages for each watershed. These results can be used for design purposes. If detailed modeling results are required, the HSPF model can be refined. For urban areas, a MRM model can also be developed. The MRM and HSPF models should provide flow peaks within 5% at coincident points.

The conceptual model of flow in the HSPF model is different from the VCRat model in that runoff from a subarea is assumed to be added to the top of the channel reach in addition to any flow from an upstream subarea and routed through the channel. This provides the opportunity to calibrate the peak and shape of the outflow hydrograph by adjusting the channel routing parameters in the model. However, this conceptual model provides somewhat shorter travel times than VCRat model routing due to steeper water surface slopes from the subarea flow being added to the channel in the model before it physically reaches the channel in reality.

4.1 CONTINUOUS MODELS IN DESIGN STORM MODELING

Hypothetically, the results from a calibrated continuous HSPF model could be used to provide annual peaks to be subjected to a flow frequency analysis to produce design storm peaks. In practice, the HSPF model results on Ventura County streams have resulted in annual maxima flow frequency analysis peaks that are not sufficiently close to results obtained from stream gage data to be used for design purposes. The main problem with the HSPF results has been concluded to be the underestimation of peak flows during relatively dry years. During those years, the storm cells that produce rain leading to the peaks seem to be of limited extent and often do not pass over the rain gage network. Therefore, too little rain is applied to the watershed in the model, resulting in the underestimated peaks and skewing the frequency analysis results. Some agencies (DuPage County, Illinois) have overcome this obstacle by using partial duration flow frequency analyses to obtain the 100-yr design storm peaks for floodplain mapping but this provides 2- to 10-yr peaks that are higher than those obtained through annual maxima analyses. Another issue with partial duration results is that the methodology is not considered to be a common practice in the use of Bulletin 17b (USGS, 1982) techniques and programs and therefore may not be acceptable for submitting design storm hydrology to FEMA.

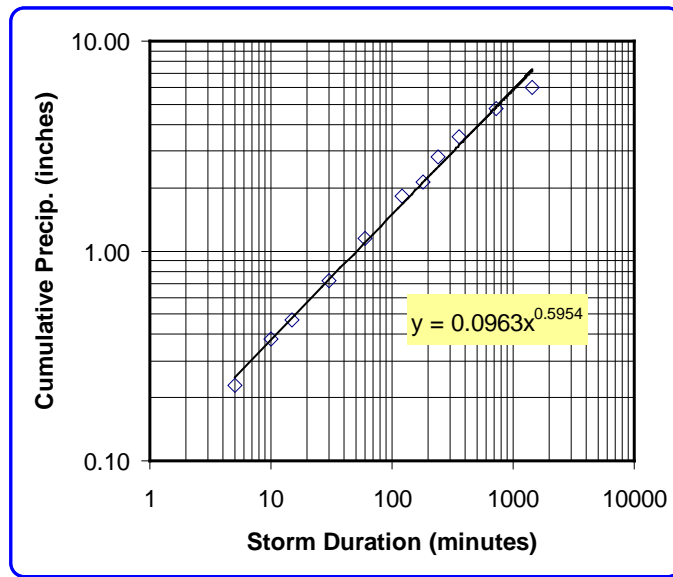
Because of this, in 2008 the HSPF model of the Santa Clara River was used to provide design storm peaks by applying a design storm hyetograph to the model just after some of the highest rainfall totals observed in recent history. The resultant model peaks matched the stream gage FFA results to within 10% or less. Because of this generally good calibration result, the Santa Clara model was used to provide design peaks for FEMA and USACE floodplain mapping studies in the river. The approach was subsequently used in the FEMA FIS for the Ventura River to provide design storm peaks from the HSPF model. Again, there was good agreement with the HSPF results and the stream gage FFA design peaks.

4.2 DEVELOPMENT OF DESIGN STORM HYETOGRAPHS

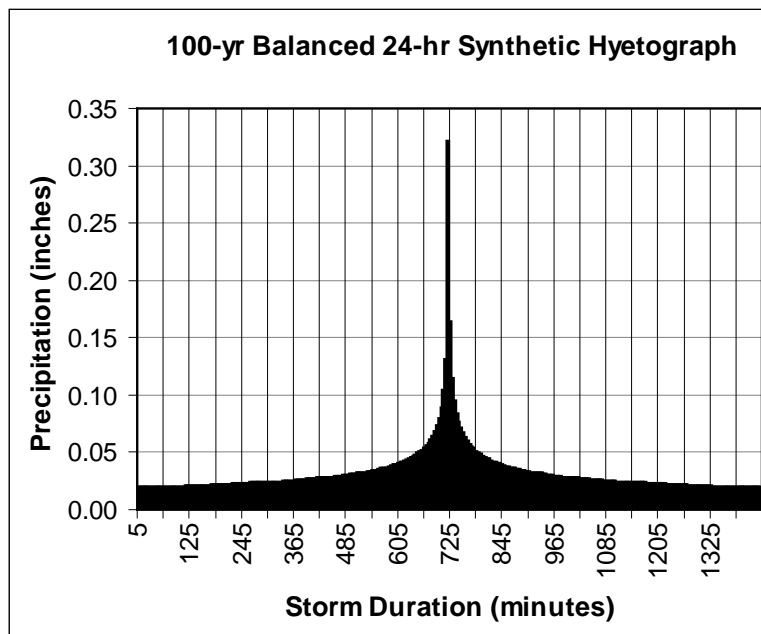
Design storm hyetographs are developed for input to the HSPF design storm model through the alternating block method as follows:

1. Perform a Pearson III Frequency Analysis of the rainfall data in the study area using the annual maxima data at intervals ranging from 5-minutes to 24-hours. The District makes these data available for each rain gage in their network.
2. Plot the depth-versus-duration data on a log-log plot and fit a power equation trendline through the results.
3. Establish the desired rainfall storm duration. For the relatively large Santa Clara and Ventura River watersheds, a 24-hour duration storm was used. For smaller watersheds, a shorter design storm may be more appropriate.
4. Establish a duration interval that divides equally into an hour. For District studies, a 5-minute interval is commonly used.
5. Tabulate the duration in increasing values of the interval.
6. Use the regression equation from Step 2 to calculate the rainfall depth for each interval.
7. Calculate the incremental rainfall depth for each time period by subtracting the cumulative rainfall at the previous time step from the cumulative rainfall for the current time step.
8. If the sum of the incremental values is larger than the 24-hour depth from the frequency analysis, reduce the incremental values by a constant factor for each interval so that the sum matches the 24-hour depth.
9. Distribute the incremental depth values. Use time blocks that correlate with the duration intervals. Assign the highest incremental depth to the central time block, and arrange the remaining incremental depth blocks in descending order, alternating between the upper and lower time blocks away from the central time block.

The resulting rain gage hyetograph ordinates are then used as input to the HSPF Model. For rain gages that only have daily records, the 24-hour value (resulting from a frequency analysis of the daily gage data) can be applied to the dimensionless distribution of an adjacent gage concluded to be a good surrogate for the gage of interest. The following figures shows the depth-versus-duration data and trendline for a District rain gage and the resultant hyetograph for gage 165 (Stewart Canyon) used in the Ventura River design storm modeling. A detailed example is shown in [Appendix C](#). A spreadsheet that can be used to provide the hyetograph is provided in [Appendix E](#).



Gage 165 (Stewart Canyon) Depth Versus Duration Data

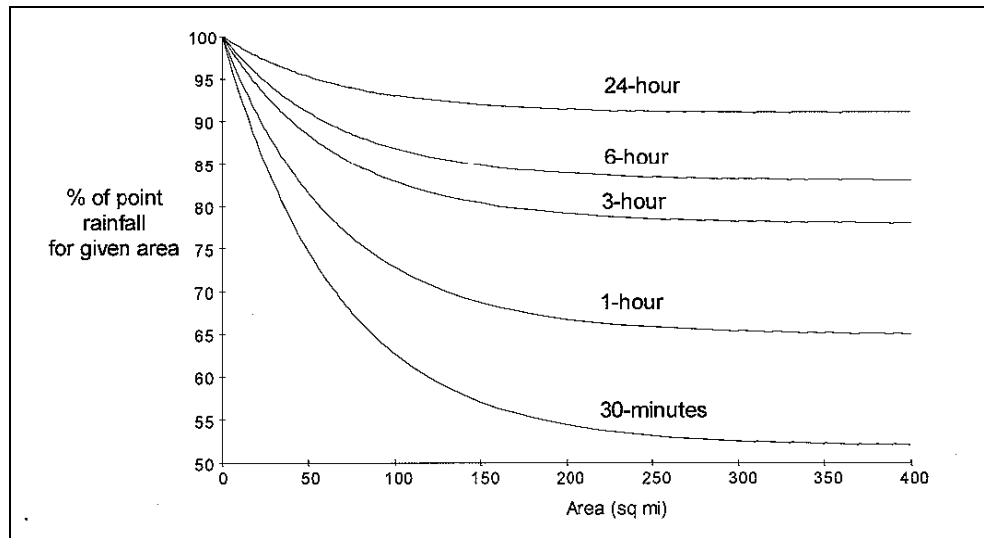


Gage 165 Design Storm Hyetograph

4.3 AREAL REDUCTION FACTORS

Areal reduction rainfall factors are used in the HSPF modeling to account for the limited spatial extent of the storm cells providing design storm intensities, similar to the VCRat approach. The factors are applied in the model run similar to the VCRat approach. The areal reduction factors are obtained from the HEC-

HMS model documentation or from a HEC-HMS model run using the AR option in the meteorological module.



HEC-HMS Areal Reduction Curves for Design Storms

4.4 DESIGN STORM PERIOD SELECTION

The design storm rainfall hyetographs should be applied to the model in the period immediately after a storm that provides saturation levels appropriate for the recurrence level of interest. For the 50-yr and greater recurrence interval storms in the Santa Clara and Ventura River watersheds, the period after the intense rains preceding January 10, 2005, was concluded to be an appropriate period to provide peaks for those design storms. In the Calleguas Creek watersheds, the biggest storms occurred in the late 1970's and early 1980's, so the design storm period should be set for a day after one of those storms. For a 10-yr storm, the design storm rain should be applied to a historic period that provided an approximate 10-yr peak.

4.5 MODEL PROCEDURE AND CALIBRATION

The steps in preparing and running a design storm HSPF model are as follows:

1. Run the calibration HSPF User Control Input (UCI) file for the entire watershed to get an initial state of the system at the beginning of the analysis period for the design storm (end of day January 9, 2005). Extract initial state data from model output for all subareas and reaches to set initial conditions to simulate runoff from January 10 through 31, 2005.
2. Modify calibration UCI for storm simulation, including changing to a 5-minute time step, and revising initial storages, the start time, rain data sets, and adjusted rainfall factors. The adjusted rain factors (MFACTs) will incorporate the calibrated continuous model MFACTs, areal reduction (AR) factors, and any adjustments with a calibration factor required to match design storm peaks from

stream gage frequency analyses. Therefore, the applied HSPF Rain Factor= Calibrated MFACT from continuous model * AR Factor * Calibration Factor).

3. Run the modified UCI. Multiple runs were needed to implement the appropriate AR factors for each site; AR factors for all sites upstream of a location of interest must be identical. For calibration sites, adjust calibration factor to calibrate/match 100-year peak flow within several percent.
4. For ungaged sites, evaluate results from gaged watersheds with similar land uses and hydrological conditions and apply calibration factors accordingly. Compare to previous modeling results for consistency if available.
5. Extract results for plotting and summary tables using WDMUtil or GenScn.

4.6 DESIGN STORM RATIOS AND HSPF RESULTS

Little work has been done to evaluate and calibrate the HSPF model design storm peaks for storms with recurrence intervals shorter than the 100-yr storm. For these storms, the soil saturation levels can vary and it is more difficult to select a historic period with saturation levels that will reliably provide the design storm peaks. Therefore, it is acceptable to use the District Design Storm Ratios to provide the peaks and hydrographs for design work from the HSPF model results.

4.7 INTERMEDIATE DISCHARGES

Because the HSPF models prepared by the District have been regional models, sometimes the subareas can be relatively large with relatively long channel reaches. It is often necessary to provide flow at intermediate points along the reach for hydraulic modeling purposes. When this occurs in the most upstream subarea of a stream where the variation in flow along the reach is not known, the District recommends the use of the USGS 94-4002 report equations (USGS, 1993). In that report, the equations recommended for estimating the design storm peaks for the 2-, 5-, 10-, 25-, 50-, and 100-yr storms are as follows:

$$Q_2 = 0.14 A^{0.72} p^{1.62}$$

$$Q_5 = 0.40 A^{0.77} p^{1.69}$$

$$Q_{10} = 0.63 A^{0.79} p^{1.75}$$

$$Q_{25} = 1.10 A^{0.81} p^{1.81}$$

$$Q_{50} = 1.50 A^{0.82} p^{1.85}$$

$$Q_{100} = 1.95 A^{0.83} p^{1.87}$$

Where A is the area of the tributary watershed in sq mi, and p is the mean annual rain in inches using a regional map referenced in the report.

If the Q100 is provided by an HSPF model for a subarea, the equation constant can be calculated and compared to the suggested 1.95 value. For most HSPF model subareas under evaluation, p is relatively constant from the regional map. With that assumption, the 100-yr peak discharge at an upstream location in an HSPF model subarea can be calculated as:

$$Q_{100_{us}} = Q_{100_{ds}} (A_{us} / A_{ds})^{0.83}$$

The 0.83 exponent on the equation implies that the cfs/area ratio increases in the upstream direction consistent with the concept of areal reduction. This approach can be used on other model results if necessary. If the subarea is downstream from another subarea, the intermediate discharges can be estimated by interpolating between the two HSPF model peaks using the peaks and the tributary areas.

Because the modified rational method model tends to have a higher degree of spatial resolution than the other models in use, a historic VCRat model can be used to provide intermediate flow results within a watershed if it follows all of the design criteria listed in the Hydrology Manual. This is done by calculating the ratio of the peak at the point of interest to the peak at the subarea outlet and applying that ratio to the HSPF subarea peak. The results are concluded to reflect timing differences within a watershed better than use of the USGS regression equations.

4.8 CHANNEL ROUTING AND TIMESTEP ISSUES

HSPF uses the Modified Puls method to do channel routing. This treats a channel reach like a linear reservoir and requires the generation of a stage-storage-discharge table (ftable) for each reach. The ftable is relatively easy to generate if there is a HEC-RAS model of the channel as it currently is an option in the RAS model version 4.0 to provide it. However, the District's experience with the Santa Clara and Ventura River design storm models has shown that there are a number of issues associated with this routing method as follows:

1. In order for this simplified channel routing method to work correctly, the model timestep must be longer than the travel time through the channel reach based on wave velocity. Otherwise there is not enough time for the translatory floodwave to travel through the reach in the duration of one timestep and the assumption that outflow is a function of storage inherent in the method is not valid. An approximation of the wave velocity in a natural channel is 1.5 times the main channel velocity. For the continuous HSPF model with 15-min (Ventura River model) or 1-hr (Santa Clara model) timesteps, the Modified Puls routing method was acceptable. However, for the 5-min timestep used in the design storm model, some of the model reaches were too long for the Modified Puls method to work as intended.
2. The long reaches relative to the 5-min timestep, and combined in some cases with significant areas of overbank storage reflected in the ftables used in the HSPF models, resulted in attenuation of the peaks for some reaches of greater than 15% in the design storm model. This is undesirable for a number of reasons, including: 1) the routing scheme in HSPF is probably not complex enough to adequately resolve peak attenuation for design and floodplain mapping purposes, especially if the timestep length affects the attenuation; and 2) the hydraulic engineer using the hydrology results often prefers flows that do not reflect hydraulic constrictions so that they can adjust the hydrographs as necessary based on their own engineering judgment or by inserting the hydrographs in an unsteady flow model.
3. In steep reaches the HEC-RAS model will often show the flow to be in the supercritical regime. However, it is known that in natural streams the channel conditions will be affected by high velocity supercritical flow to create scour holes or meanders to dissipate that energy and restore the stream to a more equilibrium situation.

Our analysis of the above factors has led to the following approaches in developing ftables for HSPF channel routing:

1. HEC-RAS runs to generate ftables should be done at subcritical conditions to reflect typical stream behavior in returning to equilibrium conditions.
2. HSPF model results should be evaluated to make sure peak flow attenuation in any reach is not excessive. Attenuation levels greater than 10% should be re-evaluated based on aerial photos and topography to ensure that the overbank flow conditions actually occur in the reach. If not, the ftable should be adjusted accordingly.
3. The recent option of performing unsteady state flow simulations with HEC-RAS makes it possible to transfer the unattenuated peak flow results from HSPF to the HEC-RAS model to more accurately model the attenuation and overbank storage. If this option is done, the HEC-RAS model used to create ftables for the HSPF model should be adjusted to limit the amount of overbank storage represented in the resultant ftables.
4. If a reduction in the timestep appears to affect the peak flow results for long reaches in the model, the modeler should consider subdividing that reach in the HSPF model.

4.9 URBAN RUNOFF PEAK REDUCTION FACTOR

The representation of storage within the urbanized watershed and its effect on design storm peaks has not been evaluated in detail in the District's models. Most cities in the County design their curb inlets to accept only 10-yr inflow, and runoff above that level is expected to be stored in the streets until it can be discharged into one of the District's jurisdictional channels. Some cities have required the building of detention facilities for new development to mitigate existing flooding problems and have developed storage in that manner.

The Ventura River HSPF model has two urbanized subareas with stream gages to use in calibrating the model. Most of the development in the watersheds occurred before detention basin requirements were implemented in the County so that storage in the subareas is mostly due to urban inlet constrictions and undersized drainage facilities. In order to calibrate the HSPF model peaks to match the FFA results, the 100-yr rain intensities from each watershed had to be reduced by a factor of 0.7.

Upon further review, it was observed that the use of the 0.7 factor applied to the rain also reduced the runoff volume from the subarea to unrealistically low levels. Therefore, an approach was developed as follows:

1. A rain factor of 0.7 was applied to match the FFA peak in the model for the developed watersheds.
2. The rain factor was reset to 1.0.
3. The ftable was adjusted to provide more storage for discharges and volumes above the 10-yr peak level based on the FFA results. For each additional foot of channel elevation above the 10-yr level, approximately 20% of an assumed extra volume was added to the existing Ftable.
4. The extra volume was adjusted up and down until the 100-yr peak matched the FFA result.
5. Then the extra volume number was checked to make sure it was in a reasonable range by converting it to a depth in inches. The additional volume for the two watersheds worked out to about 0.3 inches of storage across the watersheds, which seems reasonable based on the proportion of streets to the rest of the watershed.

Further work needs to be done to develop rainfall factors that can be used to calibrate HSPF design storm results for Cities like Simi Valley with large numbers of homeowner and regional detention basins. However, the 0.7 factor can be used as an initial assumption to account for the urban storage pending calibration of the Simi Valley HSPF model for design storm purposes.

4.10 HSPF HYDROGRAPH YIELDS

Because the HSPF design storm model is based on a calibrated continuous hydrology HSPF model that matches historic runoff volumes well, the hydrograph yields for the 100-yr storms have been comparable to the yields that would be calculated using the District's NRCS CN approach. Therefore, HSPF hydrographs can generally be used in other model work such as sediment transport studies or levee seepage analyses after checking their yields for consistency with NRCS results.

4.11 COMPARISON TO OTHER MODELS

It is recommended that the HSPF design storm flow results for ungaged watersheds be compared to other available model results to make sure that the numbers are consistent. Because the method is relatively new and has only been used on regional models, the comparison will help develop confidence in the use of the HSPF model results for design purposes. The District has done this comparison in all of the work they have done with HSPF design storm models.

Because the HSPF models developed so far have been regional in nature, the District's policy is to accept VCRat model results with refined subareas, flow patterns, or boundaries, for projects where they have been developed to calibrate the regional HSPF model results.

4.12 TRANSFORMING HYDROGRAPHS

If VCRat hydrographs are imported into another model, the District's approach is to extrapolate the VCRat hydrograph to 0 cfs using the last several points in the hydrograph, and then adjust the time-of-peak of the VCRat hydrograph so that it matches the other model time-of-peak at that location. The last step is to transform the VCRat hydrograph into a hydrograph with regular time-intervals to match the other model.

When using other model hydrographs in VCRat models, the regularly-spaced hydrograph time ordinates are adjusted so that the peaks occur at the same time and then the flows are interpolated to obtain the points required in the irregularly-spaced VCRat hydrograph discretization scheme. The various programs produce hydrographs with slightly different shapes but they are similar enough so that the results can be used for design purposes.

SECTION 5 HEC-HMS DESIGN STORM MODELING

5.1 HEC-HMS MODELS AND REPORTS

The biggest HEC-HMS modeling effort providing official model results for one of the District's watersheds was done by the USACE (2003) when they prepared a regional HMS model of the Calleguas watershed in support of the FEMA FIS update for that basin. This model presented results for the natural, existing, and future conditions and has been used for various studies in the watershed. The model used an S-Graph developed for Sespe Creek and design storm hyetographs using the alternating block technique. The resultant hydrographs are very peaky, similar to MRM hydrographs, and generally have yields that are less than would be expected based on NRCS Curve Number yield analyses for the watersheds. Therefore, the hydrographs should not be used for hydrology and floodplain mapping studies require accurate volumetric hydrographs. The model did not explicitly include urban hydraulic constraints and required the use of the standard areal reduction factors and also a runoff calibration factor to match the peaks provided by stream gage frequency analyses.

5.2 UNDEVELOPED WATERSHED S-GRAPH

The District performed a study in 2007 that calibrated hydrologic model parameters using the Snyder Unit Hydrograph and a constant loss rate in conjunction with the USACE lag method. From the results several S-Graphs were obtained that can be used to obtain design storm peaks and hydrographs for undeveloped watersheds greater than 5 sq mi in size. Although several partially-developed watersheds were included in the study, the results were inconclusive and therefore the method should not be used on developed or mixed use watersheds. The smallest undeveloped watershed in that study was Pole Ck at about 8.5 sq mi. This method should not be applied to developed watersheds or undeveloped watersheds less than 5 sq mi in area.

The S-Graphs are used in conjunction with a lag time, a constant loss rate, the SCS Type 1 storm distribution, the 100-yr 24-hr rainfall depth, and the SCS AMC II watershed yield to produce the design storm hydrograph.

5.2.1 Lag Equation

The lag time is calculated through the use of the USACE lag equation as follows:

$$T_1 = 24 * n (LL_c / S^{1/2})^m$$

Where:

T_1 = catchment lag (hr), defined as time from start of unit rainfall to 50% of total runoff volume.

L = length of the main stream from outlet to divide (mi)

L_c = length of main stream from outlet to the point closest to the watershed centroid (mi)

n = basin factor related to Manning's n for channels and overland flow, ranging from 0.015 for improved channels and uniform drainage areas to 0.200 for overland flow areas with substantial vegetation.

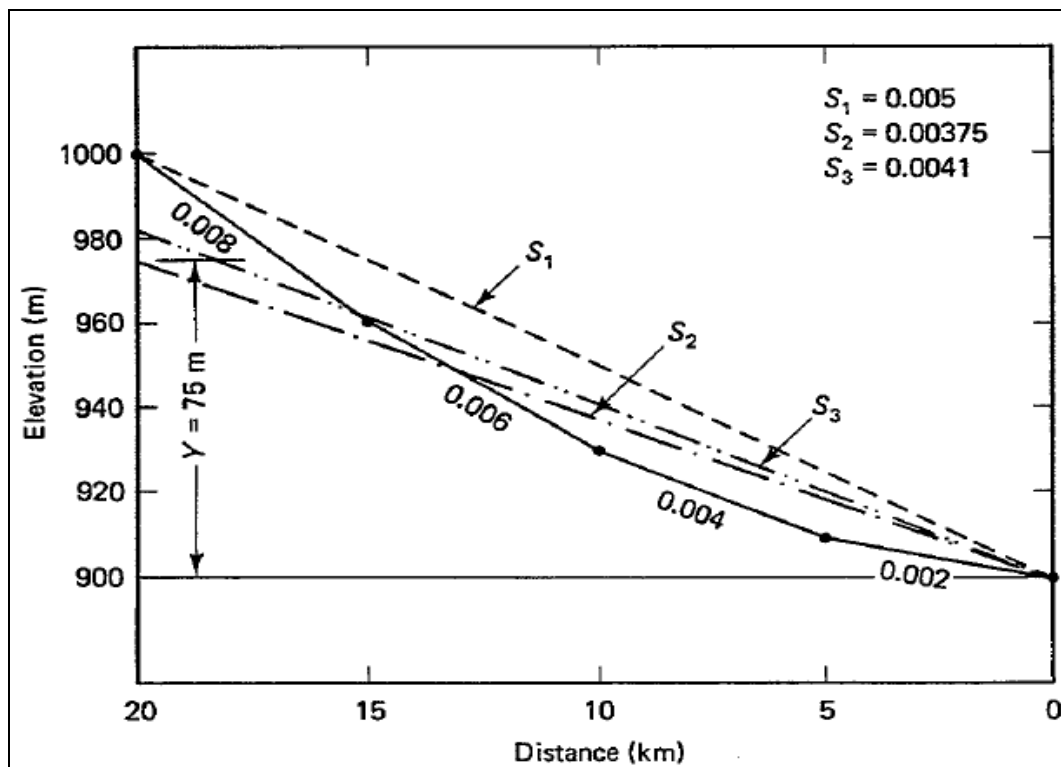
S = Overall slope between the headwaters and collection point in ft/mi

m = exponent, determined by regional flood reconstitution studies- Los Angeles District of USACE has determined the exponent for Southern California to be 0.38 based on analysis of historical storms.

This equation is commonly used to calculate lags for other unit hydrograph methodologies because it is assumed that the uncertainties associated with using the parameters to calculate the lag are greater than the uncertainties introduced by the different lag definitions.

5.2.2 Slope

There are several different definitions of catchment slope. Ponce (1989) defined the slope in the USACE lag equation as S_1 . He defined an S_3 slope as a weighted average slope that accounts for the fact that the travel time in different channel reaches does not vary linearly and therefore takes into account the basin response time. The figure below from Ponce (1989) shows the graphical representations of the slopes. The VCWPD study (2007) found that the S_1 slope did not always provide a lag that matched the optimized lag. In some cases the S_3 slope, yielding a slightly long lag, matched the optimized lag better.



Catchment Slope Depictions

The equation used in order to calculate the S_3 slope is:

$$S_3 = \left[\frac{\sum_{i=1}^n L_i}{\sum_{i=1}^n (L_i / S_i^{1/2})} \right]^2$$

The S3 slope is found by plotting the elevation vs. watercourse length and separating the curve into at least three reaches. The upper reach should be the steep upper portion of the stream, the middle reach should be a less steep foothill portion of the stream, and the most downstream reach is usually a flatter valley portion of the stream. The effect of using this approach for North Fork Matilija is that the S1 lag time using the COE lag equation was 1.31 hours and the S3 lag is 1.35 hours, a 3 % difference.

In some cases, the lag calculated with the S1 slope matched the optimized slope from the calibration better than lags calculated with the S3 slope. A number of shape and slope parameters were tested to find a combination that appeared to explain which slope should be used in the lag calculation. The best combination is as follows:

$$ER * S3 / (Lc/L)$$

Where ER is the elongation ratio defined as the diameter of a circle with an area equivalent to the watershed area divided by the length of the longest watercourse. As the watershed becomes longer and narrower, ER decreases. Physically the shape factor represents the ER times the change in elevation divided by the distance from the catchment outlet to a point opposite the catchment centroid. When this shape factor is less than 200, the S1 slope should be used in the lag calculation. When this shape factor is greater than or equal to 200, S3 slope should be used in the calculation.

5.2.3 Basin N Factors

The VCWPD study resulted in calibrated basin n factors as follows:

1. For undeveloped basins with S3 slopes greater than 150 ft/mi in the Santa Clara, Calleguas, and lower Ventura River watersheds, use a basin n factor of 0.045.
2. For steep S3 slope catchments of 300 ft/mi or greater in the upper Ventura River watershed, use a basin n factor of 0.055.
3. For undeveloped basins with S3 slopes less than or equal to 150 ft/mi in all watersheds, use a basin n factor of 0.035.

5.2.4 Design Storm Rainfall

The VCWPD study (2007) found that the rainfall distribution that provided reasonable calibration results is the SCS Type 1 rainfall distribution. The 24-hr 100-yr rainfall depth used with this distribution is obtained from the isohyetal maps provided in this manual. If the contour maps vary smoothly across the watershed, the value at the centroid of the watershed can be used. If the contours do not vary smoothly, a weighted average value should be calculated. This weighted average rainfall depth is applied in the meteorological module in the HMS model after specifying the SCS Type 1 rainfall distribution.

5.2.5 Design Storm Yields and Loss Rates

The VCWPD study (2007) found that the watershed yield calculated assuming Antecedent Moisture Condition II and obtaining the curve numbers and soil types for the undeveloped study watershed provides reasonable results in obtaining design storm peaks. Once the watershed yield is calculated, the constant loss rate in the HMS model is adjusted until the model yield matches the calculated 100-yr yield. No initial loss rate is necessary for the design storm.

5.2.6 S-Graph Selection.

For undeveloped watersheds, one of three S-Graphs can be selected based on the shape parameters calculated for each watershed as follows:

1. For long narrow watersheds with extensive headwater areas such as Ellsworth Barranca with a relatively low slope and small elongation ratio, use the Ellsworth Barranca S-Graph.
2. For round-shaped watersheds with a high elongation ratio and where the existence of several major tributaries with distinctly different lag times for major portions of the watershed (such as North Fork Matilija), choose the North Fork Matilija S-Graph.
3. For other watersheds, use the Canada Larga S-Graph that represents the average of the other four undeveloped watersheds included in the study. Tables of the S-Graph ordinates are provided in [Appendix C](#). HMS model files using the three S-Graphs and other parameters to produce design storm hydrographs are provided in [Appendix E](#).

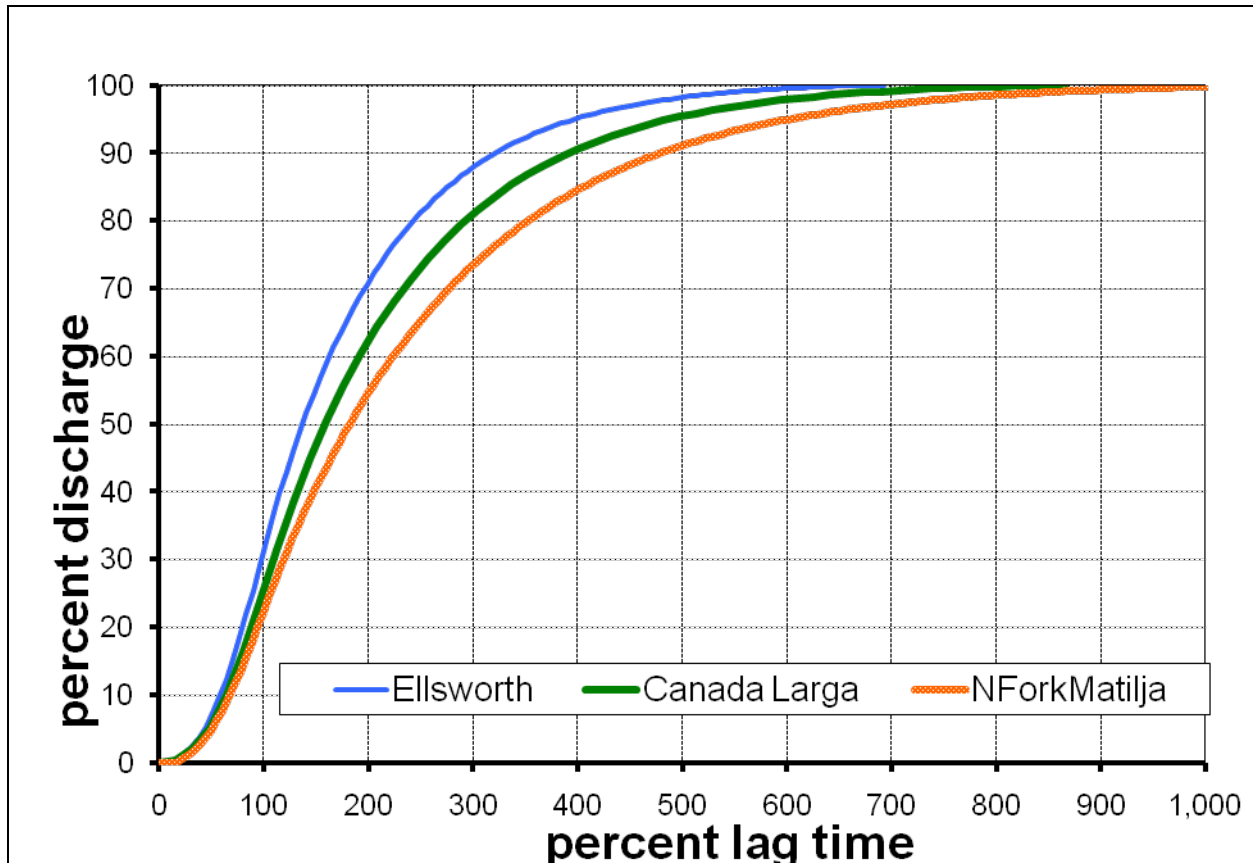
5.3 CALIBRATION TO OTHER MODELS

The results of the HMS model design storm should be checked against other HMS runs of similar watersheds, other model results, or available stream gage flow frequency analyses for similar watersheds. The peak cfs/ac ratio for the design storm should be within the range of other model or stream gage results for nearby watersheds.

5.4 GENERATION OF SNYDER HYDROGRAPH

The HMS model has been used to develop regional hydrographs for use in VCRat 2.6 modeling in limited applications. Due to the truncation of the VCRat 2.6 hydrograph at the end of the one day simulation length, it is difficult to perform a yield adjustment on hydrographs from watersheds greater than about 70 to 80 sq mi. For these watersheds, a lag is calculated, then the NRCS yield using soils and CN's is calculated, and a weighted average 100-yr 24-hr rainfall is obtained. These data are entered in an HMS

model with a Snyder C_p factor of about 0.50. The constant loss rate is adjusted until the hydrograph yield matches the NRCS yield. Then the C_p parameter is adjusted until the hydrograph peak matches the areally-reduced VCRat 2.6 peak at the location of interest. Finally, the hydrograph is translated so that the peak flow occurs at the same time as the VCRat peak, interpolated onto the VCRat variable time hydrograph spacing, and then the hydrograph is imported into the VCRat program for use in evaluating the downstream impacts on the District's facilities. An example of this is given in [Appendix C](#) and sample files are available in [Appendix E](#).



Undeveloped Watershed S-Graphs.

SECTION 6 DESIGN PROCEDURES AND TOPICS

The following sections provide information on topics affecting the design procedures. Several examples are provided in [Appendix B](#) to demonstrate the use of the design techniques to calculate peak flows.

6.1 MODELING HYDRAULIC CONSTRAINTS

VCRat does not have the capability to easily model hydraulic constraints in the drainage system such as 10-yr storm curb inlet limitations or undersized drainage pipes detaining flow and attenuating peaks, etc. Therefore, with limited exceptions, model results generally provide peaks assuming that all of the flow in the watershed can be discharged to the outlet in adequately sized pipes and channels. If the undersized pipes are included explicitly in the model, VCRat assumes that the flow is forced through under pressure, resulting in an unrealistic flow travel time and likely affecting the model peaks.

Providing peaks that are not attenuated by system deficiencies allows the hydraulic engineer to adjust the hydrographs as necessary to reflect constraints or show the impacts of improving the drainage system on the downstream peak. If it is important to accurately model the effect of hydraulic constrictions, there are a number of ways to use VCRat or other models to do this as follows:

1. For VCRat, apply the 10-yr rain mass curves in any developed subarea with 10-yr curb inlet limits. Adjust the yield to match the 100-yr yield. Evaluate the topography to identify any alternate routes for the street flow to reach the channel and model those as splits in the VCRat model. Evaluate detention due to backwater effects with stage storage curves and model those as basins in VCRat.
2. In HSPF, modify the ftables for flows above the 10-yr level to reflect the storage in the system. Add detention basins where necessary.
3. In HMS, add detention basins where necessary to reflect system storage.

6.2 VCRAT AND FLOODPLAIN ROUTING-BREAKOUT SIMULATION

Although the VCRat program has been upgraded over the years to provide limited routing capabilities, in some cases the program has been used to simulate floodplain routing and/or flow breakouts that occur as streetflow and confluence with the channel flow downstream of the breakout location. The District does not consider the VCRat program to be suitable to perform hydraulic routing where timing differences between the channel and breakout flow can reduce the channel peak flow significantly. If this occurs, the peak flows provided by the program should be transferred to a hydraulic program that provides routing methods that can adequately simulate complex flow patterns.

6.3 DISTRICT HYDROLOGY DATA AVAILABILITY

The streamflow, rainfall, evaporation, and hydrology study data available electronically at the District are available through the <http://www.vcwatershed.org/> website. This website allows the users to download formatted or text files of daily flows and rainfall for the District's gages. It also provides station information, and precipitation and stream frequency analysis results. The "Frequently Asked Questions" file shown on the website provides a detailed description of the available data and quality control codes. Limited hourly flows and rainfall are also available through the website.

If District data are needed for studies that are not currently available through the website, consultants can fill out the Data Request Form available on the website and email the request to HydroData@Ventura.Org. The Hydrology Section will review the request and provide a response and/or the data in a timely fashion. If data is requested requires extensive processing in advance of the District's normal schedule or in addition to the normal workload, costs for the processing will be charged according to the policy set in 2006 by the Ventura County Board of Supervisors.

6.4 DESIGN STUDY SUBMITTAL MATERIALS

Often hydrology studies are performed to demonstrate the effects of development and required mitigation strategies on the District's jurisdictional channels and are submitted to the Permits Section for review. At a minimum, the required submittal materials for hydrology studies for encroachment and watercourse permit reviews by the Permits or Hydrology Section should include the following:

1. Hydrology report with Table of Contents clearly indicating the page numbers of the report sections including executive summary, hydrologic model input and output results, Tc calculations, summary result tables, hydraulic calculations, routing results, and NPDES calculations.
2. Report should be stamped and if final, signed by a California Registered Civil Engineer
3. Statement of criteria used in the study and applicable agency (Example: Mitigate 100-yr developed condition peak flow to 10-year undeveloped peak flow as specified by the City of Moorpark).
4. Description of watershed and any special features in it, including undeveloped and developed hydrologic and hydraulic conditions.
5. Undeveloped and developed condition hydrology hardcopy maps showing subarea boundaries, node I.D.s used in modeling, Tc's, elevation contours, drainage system alignments and sizes, building locations, elevation datum, and other common map information included on engineering drawings. Maps should also be submitted electronic format.
6. Summary table of results clearly showing that the design has met the mitigation requirements.
7. Electronic files and/or hardcopies of Exhibits from VCWPD Hydrology Manual used in the study.
8. Electronic copies of the Tc Calculator and VCRat input and output files used in the study in addition to any hardcopy results provided.
9. Tc calculation documentation should show subarea boundaries and development type used in evaluation as well as various flowpaths and related subarea portions.
10. Hydraulic calculations used in study. If a proprietary detention basin routing program is used, provide hardcopy results. If non-proprietary, provide electronic files in addition to hardcopy output.
11. If resubmitting a package in response to comments, provide a cover letter clearly indicating how the originally reviewed report has been changed and page numbers of any additional requested items. A copy of any comments received from the District should be included in the package.

This list is not meant to be comprehensive, and other materials may be required by the District to fulfill the requirements of the permit review. A WORD document showing an example format and providing procedures for many of the topics discussed in this section are provided for consultant use as part of [Appendix E](#).

6.5 UPDATED MPD DATA AND DESIGN

Before the District began developing official watershed models the Cities had Master Plans of Drainage (MPDs) to evaluate their local storm drain systems and plan for system upgrades. The MPDs all use the District's VCRat model except for the City of Oxnard with their Cook's method (considered to be a simplified Rational Method approach). The Oxnard MPD prepared by Hawks Associates (2003) contains a comparison of peak flow using both methods. The results of the comparison are that the Cook's method yields peaks that are about the same or bigger than VCRat peaks for developed watersheds up to several hundred acres. VCRat peaks are bigger for larger watersheds and for undeveloped areas.

The District's policy is to allow the use of MPD results for small project areas as described in Section 6.6. The Cities of Thousand Oaks and Camarillo have recently issued updates to their MPDs. The District has commented on the Thousand Oaks MPD but has not reviewed the Camarillo MPD. Because of this, the District will accept the use of MPD results to evaluate small project areas at this time but the MPD models cannot be used to evaluate large projects or assess the downstream impacts of projects on the District's jurisdictional channels. When this is required, the District's official models should be used or if not available, the MPD should be updated to meet the District's requirements and calibrated to existing FFA data.

6.6 UNIT RUNOFF CALCULATIONS FOR SMALL PROJECTS

When development projects consist of small areas that do not meet the Hydrology Manual minimum subarea guidelines, it is possible to estimate the change in peak flows from the project area using pro-rated values based on the unit runoff. The steps are as follows:

1. Find the subarea that incorporates the project area in a city Master Drainage Plan or official District VCRat hydrology model.
2. Evaluate if the change in development for the project would affect the subarea T_c by more than 0.5 minutes (VCRat only accepts T_c 's rounded to the nearest minute). Unless the project is located in the upper portion of the subarea and would affect the overland flow portion of the T_c , the T_c is not likely to change. If the T_c does not change, the intensity used in the subarea peak flow calculation does not change. If the T_c does change, recalculate it to obtain the correct design storm intensity for use in peak flow calculations.
3. Find the C coefficient for the pre- and post-development conditions, and recalculate the peak flows from the project area only.
4. If the pre-or post-developed condition percent imperviousness matches that used in the model subarea, the peak can be obtained from the model result through pro-rating based on area.
5. Sometimes the increase in runoff is very small but the District has been concerned about the cumulative increase in flow due to numerous small projects and so has been asking for mitigation for any new project with very limited exceptions.
6. If a detention basin is selected to provide mitigation, the VCRat model can be used for its design. Simplified methods for designing detention areas for very small projects are provided in following sections.

An example of this method is provided in [Appendix B](#).

6.7 HYDROLOGIC MULTIPLIERS

Time of Concentration (T_c) values required for each subarea in the VCRat model are a function of rainfall intensity and therefore design storm recurrence interval. Because of this, they have to be recalculated for each design storm. Because hand T_c calculations are time-intensive, in the past the District developed a table of hydrologic design storm ratios or “multipliers” based on stream gage frequency analyses. The multipliers were used to estimate other design storm flows once the results from one design storm were available. Additionally, the 1992 Hydrology Manual only provided the 50-yr 24-hr rainfall design storm depths for use in yield calculations, and there was a need to estimate other design storm values using average design storm rainfall ratio multipliers.

Currently, the T_c calculator has been automated so that it is relatively easy to calculate T_c 's for all design storms of interest once the subarea T_c physical data have been entered into the calculator. The current Hydrology Manual now provides the 24-hr rainfall depths for the 10-, 25-, 50-, and 100-yr storms. Therefore, there is no longer a need to use the Hydrology Multipliers in design calculations using the VCRat model except in limited special cases. However, the HSPF and HEC-HMS model results still require the use of the multipliers. [Exhibit A-21](#) provides the updated multipliers developed by the District in a summary table.

6.8 TIME OF CONCENTRATION CALCULATIONS AND DESIGN EXAMPLES

Because hand calculations of the Time of Concentration (T_c) values required for each subarea in the VCRat model were time-intensive, previously the hydrologist was only required to calculate T_c 's for 20 percent of the subareas. The hydrologist could then use those results to estimate T_c 's for the other subareas if they were hydrologically-similar. In addition, T_c 's for other design storms were often assumed based on engineering judgment instead of recalculating them. The current T_c calculator program automatically does the iterations necessary for the calculation and therefore greatly decreases the time necessary for a single T_c . Also, the program makes it easy calculate T_c 's for all design storm levels once the T_c input data file is generated. Because of these factors, currently the hydrologist is required to calculate T_c 's for 75 percent of the developed subareas and 25 percent of the undeveloped subareas included in the hydrologic model. Engineering judgment can be used to estimate T_c 's for the other subareas as long as they are hydrologically similar in slope and development type to the subareas where T_c 's have been calculated directly. It is also possible to develop regression equations based on the calculated T_c 's to estimate the other T_c 's.

To illustrate use of the MRM for computing design flow rates, two detailed examples are presented in [Appendix B](#). The first example describes the recommended procedure for computing both a peak flow rate and runoff hydrograph considering the entire tributary drainage area as a single unit. This is appropriate only when channel routing is not necessary and only for watershed areas with a time of concentration between 5 and 30 minutes. The first example also demonstrates a technique for adjusting the volume of runoff associated with a storm hydrograph. Reasonable storm volumes are important for the design of pumping stations and detention basins.

Complete drainage areas with total times of concentration greater than 30 minutes must be subdivided into smaller subareas and a time of concentration computed for each. Runoff hydrographs are then computed for each subarea, routed, and combined with other subarea hydrographs to establish design flow rates. Because of the complex calculations required for the hydrograph routing, it is necessary to use the

VCRat program to model multiple subareas with channel routing. Example Two in [Appendix B](#) describes the necessary procedures for use in subarea definition and model development using VCRat. The Exhibits for the examples are provided in Appendix A.

6.9 BULKING FACTORS

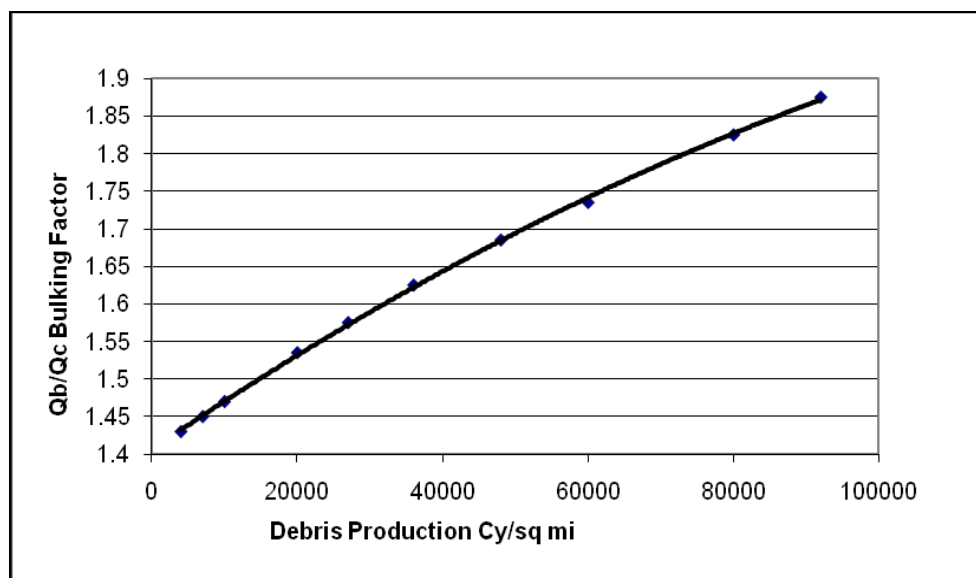
The hydrology model results generated by the VCRat model are generally considered clear water estimates except in limited cases where the model results have been calibrated to stream gages. Stream gages generally have a limited amount of sediment in the flow, such as the gages used to calibrate the District's official model of Calleguas Creek. Because of the sediment, the Calleguas model numbers are considered to be bulked to a limited extent. Bulking has also been applied to model results for smaller watersheds with known high sediment production such as the FIS 100-yr peak for Pole Creek where the clear water hydrology model peak was increased by a bulking factor of 1.5.

Bulking factors for watersheds such as Pole Creek or subject to frequent fires can be obtained by first calculating the expected sediment yield using the Scott and Williams (1978) equation or some similar method. Then the sediment production in cy/sq mi is used to obtain a bulking factor based on a historic curve shown below. The bulking factors obtained through this approach range from about 1.42 to 1.72 for the District's watersheds.

The District has an ongoing consultant study to make recommendations about the use of bulking factors in design storm hydrology. The preliminary conclusions are as follows:

1. The highest sediment concentrations ever measured on a mainstem was about 70,000 mg/l, which converts to a sediment volume of roughly four percent. Bulking factors on mainstems are generally small due to runoff from numerous watersheds, only some of which are high sediment producers due to fires or erosion-prone soils.
2. Bulking should be limited to jurisdictional channels downstream of small watersheds that are known high sediment producers or subject to frequent fires. In the case of Pole Creek, the gravel-sized chunks of shale that were carried in the flow deposited and blocked the channel in 2005 after a fire in 2003. The Pole Creek watershed is less than 9 sq mi in size.
3. Bulking should also be used for emergency projects designed to protect development from recent fires in the watersheds.

Specific recommendations and procedures for bulking are still under development at this time and will be included when the consultant project is finished. Until the District's policy is finalized, bulking will be applied to design hydrology results on a case-by-case basis after consultation with the District's Hydrology Section.



VCWPD Bulking Factors Used in Historic Studies

6.10 CHANGE IN RUNOFF PARAMETERS DUE TO FIRE

A related issue to the increase in peak due to bulking is the increase in runoff due to reduced infiltration and vegetation cover after fires. The District does not have a policy to increase C coefficients in VCRat or reduce basin n values and soil infiltration rates in HEC-HMS to account for this. The HSPF modeling done in the Santa Clara River model did a simple analysis of the increase in peaks due to fires in the Piru Creek and Sespe watersheds and found that the mean annual runoff volumes can increase by up to 20 percent, while the 100-yr design storm peak on the Sespe increased by about five percent when about a third of the watershed was modeled with burn conditions.

Specific recommendations and procedures for changing runoff parameters due to fires are still under development at this time and will be included when a consultant project is finished. Until the District's policy is finalized, fire effects will be modeled on a case-by-case basis after consultation with the District's Hydrology Section.

6.11 FLOW LIMITATION AGREEMENTS

The District's Rice Rd Drain jurisdictional channel in Oxnard has been subject to upstream development. Because the increase in runoff has increased the flooding potential in downstream areas, the District and the City of Oxnard developed an agreement that limited the runoff from new development to specified limits. Agreement FC-2-87-6A limited the peaks from new development for the 100-yr storm to 1 cfs/ac north of Fifth Street and to 0.72 cfs/ac for the portion of the watershed between Fifth and Emerson Avenue.

6.12 DEVELOPMENT MITIGATION CRITERIA

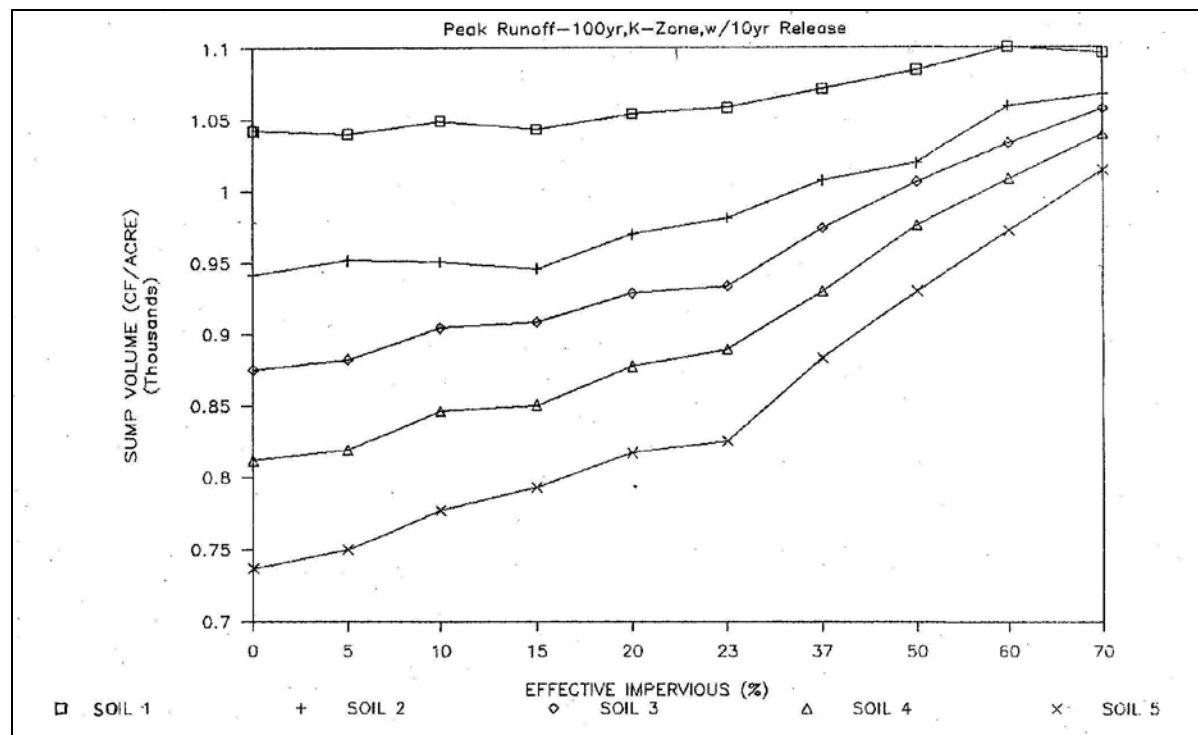
Information on development criteria is provided for convenience but it is suggested that the consultant verify their mitigation with the agency having jurisdiction over their proposed projects. Generally the

Cities and unincorporated County direct the consultant to design local drainage to accept the 10-yr developed condition peak and to check to see that the flow above that level up to the 100-yr peak could be conveyed in the street. One exception is the City of Moorpark that allows consultants to design the local drainage to convey the 50-yr peak.

To minimize the impacts of development on the existing drainage system, and in some cases to remedy existing drainage deficiencies, most Cities require that the consultant use basins to attenuate the 100-yr developed condition peak back to the 10-yr developed condition peak. For some channels with severe deficiencies, the mitigation goal has been the 10-yr undeveloped condition peak. For the unincorporated County, the mitigation is normally no increase in flow for any design storm from the 10-, 25-, 50-, or 100-yr frequency. However, these criteria should be confirmed with the agency having jurisdiction prior to any work on project design.

6.13 BYPASS BASIN DESIGN

In 1988 the District provided training to consultants and City engineers about detention basin design. As part of the training a Truncated Sump figure was provided to the trainees that provided design basin volume criteria for bypass basins located in the City of Oxnard. The figure indicated that the required basin volume to attenuate the developed 100-yr peak back to the 10-yr developed peak level was a maximum of about 1,000 cf/ac for most development and soil combinations. This mitigation volume was then adopted by a number of cities in Ventura County including Simi Valley, Thousand Oaks, Camarillo, and Oxnard. Studies submitted to the District for review are commonly designed by providing a basin with this volume, with outflow from the basin controlled by an orifice plate sized to limit the discharge to the 10-yr peak based on the assumed depth of the basin.

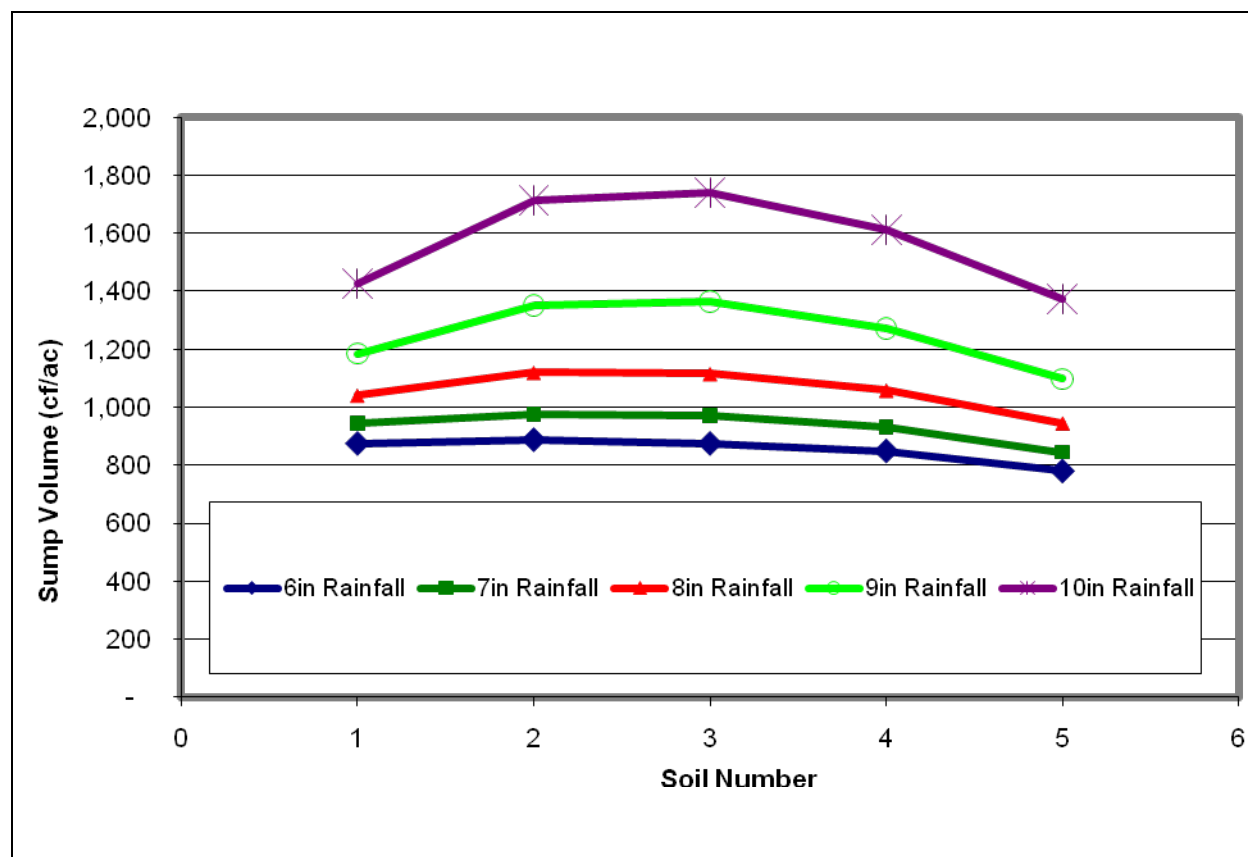


Truncated Sump Volume Figure, 1988

The Truncated Sump Figure discussed above presented results that were simplified because it did not show how the mitigation volume varied with Tc and watershed yield. The figure was only intended to be used for bypass basin design in the City of Oxnard where the VCRat hydrograph yield is generally very similar to the watershed yield calculated through the NRCS CN approach.

As the figure is often used in the County, the District recently updated the study to include these other factors to make it more useful for design purposes. The results of the study show that more volume is required for basins to mitigate the peaks for subareas with Tc's near 30 mins and with lower % impervious values. Both situations lead to broader hydrograph shapes that provide more volume above the mitigation level compared to hydrographs from subareas with short Tc's and high % impervious values.

The data in the following figure leads to the conclusion that low soil numbers and higher 100-yr 24-hr rainfall depths lead require more basin volume. Appendix D discusses the design procedure in more detail. A hydraulic analysis is required during design to make sure that the side channel weir structure used to transfer flow from the main channel to the bypass basin will work as intended during the design storm.



Bypass Basin Mitigation Volume, Long Tc, 10% Impervious. 100 to 10-yr Developed Case

6.14 FLOW-THROUGH BASIN DESIGN

The District has not attempted to produce a nomograph that could be used for flow-through basin design by consultants because there are so many parameters associated with routing of flow through a basin, including hydrograph shape, operating spillway design, low flow outlet design, 100-yr 24-hr rain depth, subarea % imperviousness, and Tc's and C coefficients for the VCRat program. However, in an attempt to make the design process simpler and more straightforward for basin designers, a study was completed that developed a method to design and evaluate basins using VCRat in conjunction with HEC-HMS.

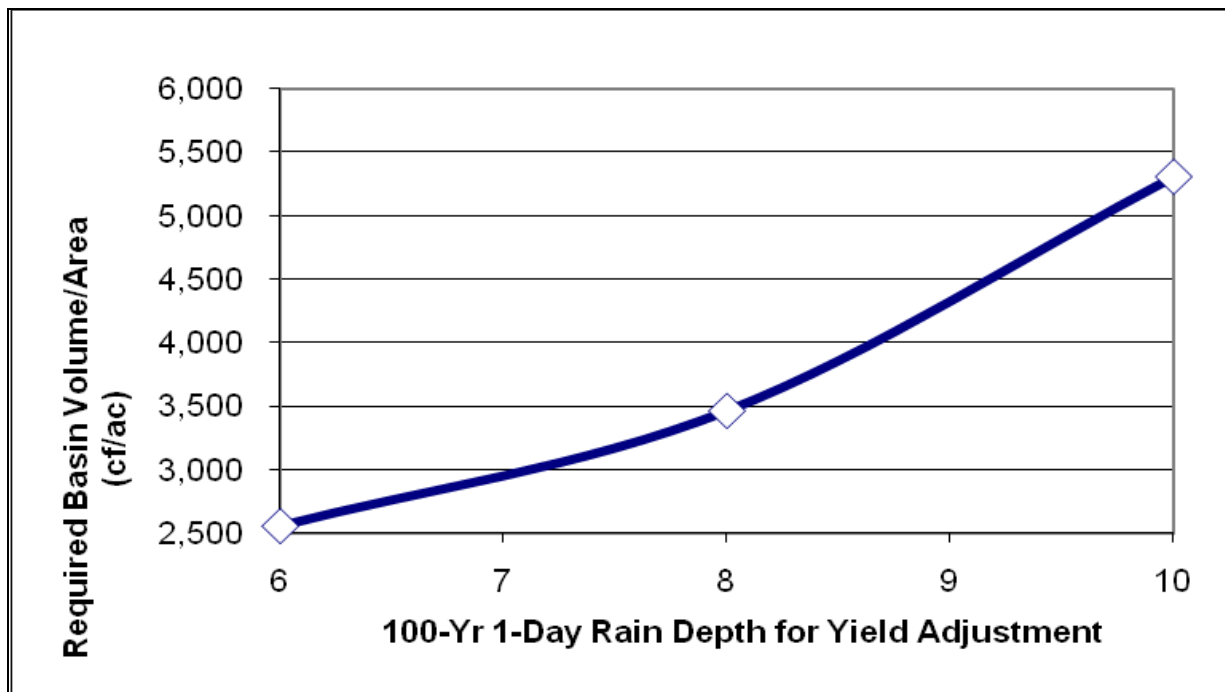
The design parameters selected for use in the study were the following:

1. Use as many criteria as possible from the County NPDES Technical Guidance Manual to design basin detention for stormwater quality purposes such as basin sideslope criteria (3H:1V), basin length to width ratio (2L:1W), and the low flow outlet orifice sizing equation.
2. To minimize the downstream impacts on natural channels due to erosion from the increase in runoff volume, the basin was provided with a low level outlet sized to detain flow until 20-24 hrs after the peak rainfall had occurred.
3. Assume the 100-yr developed condition was being mitigated back to the 10-yr developed condition in the study. This is the most common mitigation condition used in the County.
4. Evaluate the range of 100-yr 1-day rainfall values commonly found in developed areas- 6 in for Oxnard, 8 in for Simi Valley and Thousand Oaks, and 10 in for Ojai. The K Zone rainfall distribution was used in all runs to provide a better comparison of the results.
5. Minimize the depth of the basin because it is assumed that this is desirable for most projects because areas with flat slopes will require more grading if deeper basins are built. Most of the basins were assumed to be 5 ft deep for a subarea of 100 ac.
6. Because VCRat simulations only extend to 24 hrs, and one of the goals was to design a basin that would not outlet all of the flow until 24 hrs after the peak rainfall had occurred, it was necessary to export the VCRat hydrograph to HMS to do a multiple day run.
7. Each basin was assumed to be designed with a vertical standpipe with the top at $\frac{1}{2}$ the basin depth and sized to outlet the 10-yr developed condition peak with basin full. Each standpipe was assumed to have a low level outlet sized to empty the basin 20-24 hrs after the end of rainfall and sized using NPDES Technical Guidance Manual equations.

A detailed procedure and example is provided in [Appendix D](#). The study results showed that, similar to the Truncated Sump Figure update, the most conservative basin design is required for long Tc-small % impervious development combinations because the broader hydrograph shape contains more volume above the mitigation level. An evaluation of the regional detention basins contained in the District's official Calleguas Creek model showed that the average ratio of the operating volume at the time of peak outflow from the basins to the inflow hydrograph volume was about 0.15. In this study, the same ratio ranged from about 0.16 to 0.18.

The results of the study for a conservative combination of long Tc-small impervious area shows that the required basin volume per acre of development ranges from about 2,500 cf/ac in Oxnard to about 5,300 cf/ac in Ojai. This will require significantly more volume for basins to provide the desired mitigation

than has been provided in the past using the 1,000 cf/ac guideline. [Appendix D](#) provides an example of the design process. [Appendix E](#) provides copies of the files used in the examples.



Note: Data are for Long Tc-23% Impervious Development

Flow-Through Basin Volume Variation with Design Rain Depth

6.15 SIMPLIFIED BASIN DESIGN PROCEDURES FOR SMALL PROJECTS

For consultants or permit applicants that are only developing or modifying a small project area, the District has developed a set of simplified procedures to size detention facilities. The results of the procedures are relatively conservative when compared to designs using full basin routing analyses. Therefore, they are only used when the small size of the project does not warrant a more sophisticated analysis. Projects of 5 ac or more can be evaluated with the VCRat program, therefore, this method should be limited to projects that are less than 5 ac in size.

6.15.1 100-Yr Undeveloped Condition Peak Mitigation

Estimates of the detention volume required for projects to mitigate from the 100-yr developed to the 100-yr undeveloped condition can be obtained with a NRCS Curve Number evaluation of the additional impervious project area as follows:

1. Find 100-yr 1-day rainfall for project area from Appendix E maps or GIS shapefiles provided with Hydrology Manual.
2. Find CNs associated with undeveloped and impervious project conditions from Exhibit 14,
3. Find runoff depths in inches from Exhibit 13 or use equation provided on Exhibit 13.

4. Subtract to find net yield, subtract additional 0.5 inch for additional depression storage and abstractions so that result is more consistent with the District's detention basin routing results.
5. Multiply runoff depths in feet by impervious area of project in square feet to obtain the undeveloped and developed runoff volumes.
6. This volume will generally be larger than that obtained through a routing and design analysis of the basin using a yield-adjusted VCRat hydrograph because the routing effects generally reduce the required basin size. The volume may also be larger than that required by reducing the peak back to the 10-yr developed level below due to the CNs used.

An example calculation is provided in the following table, and the spreadsheet is provided in Appendix E.

Watershed Data	Undeveloped	Impervious
100-yr 1-d Rain (inch)	7.6	7.6
Soil Type	3	3
Land Use	Open Space	Imperv area
CN Exhibit 14	73	98
S = 1000/CN-10	3.70	0.20
Yield (inch) from Exhibit 13	4.46	7.36
Volume Calculation		
Yield Difference in		2.90
Additional Assumed Depression Storage Impervious Surfaces (inch)		0.50
Net Yield (inch)		2.40
Impervious Area (ac)		0.0500
Vol Increase - Max. Basin Size Req'd (cf)		436

6.15.2 10-Yr Developed Peak Mitigation Criteria

For small project areas with mitigation requirements that the 100-yr developed peak should be reduced to the 10-yr developed peak, the following approach has also been used:

1. Find the 100-yr 1-day rain depth for project site.
2. Find the 10-yr 1-day rain depth for project site
3. Subtract the two plus an additional 0.5 in for depression storage
4. Multiply by impervious area with units in cubic feet

The following table shows an example of the approach.

Volume Calculation	100-yr 24-Hr Depth	10-yr 24-Hr Depth
Rainfall (inch)	7.6	5
Yield (inch)		2.6
Depression Storage Impervious Surfaces (inch)		0.50
Net Yield (inch)		2.10
Impervious Area (ac)		0.0500
Vol Increase - Max. Basin Size Req'd (cf)		302.40

6.15.3 VCRat Hydrographs for Small Projects

The minimum subarea size in VCRat 2.6 is 5 ac to decrease the chances of the program being used to produce flows for numerous tiny subareas such as is commonly done for drainage design. However, for small developments where peak mitigation is required through basin design, it is possible to use the program to get a hydrograph. The procedure takes advantage of the fact that the VCRat results are linear with area, so that the hydrograph of a 10-ac subarea is identical to the hydrograph of a 100-ac subarea divided by 10 as long as the other model input parameters (T_c , % impervious, soil type) are identical. The procedure to get a hydrograph for a 1.3 ac site is as follows:

1. Per the previous MPD discussion, find the subarea T_c applicable to the project area from an existing model or calculate one. Set up a 2.6 model using the existing model rain zone and T_c and project area soil type and % imperviousness. For subarea size in the model, use the project area times a factor from 10 to 100. For this example, use a factor of 10 so the subarea size will be $1.3 \times 10 = 13$ ac.
2. “Fatten” the project area hydrograph using the yield adjustment procedure described in this manual.
3. Export the project area hydrograph in csv format so it can be imported into Excel. Divide the hydrograph ordinates by 10. The resultant hydrograph can be used in another program to do basin design for the project area.

An example of this technique is shown in [Appendix B-12](#).

6.16 AREALLY REDUCED FLOW RESULTS IN JUNCTION ANALYSES

If confluence information is requested in the VCRat model, the model will print out the flows at a confluence at the time the main line upstream of the junction is peaking, at the time the tributary is peaking, and at the time that the main line downstream of the junction is peaking. The flows provided in the model results fulfill mass balance requirements. However, if AR is required for the main line and perhaps the tributary, because the AR curves are nonlinear with area, the resultant flows will not be balanced. To preserve mass balance requirements for hydraulic junction analyses, use the following steps:

1. From main line point-of-view: Find the time of peak of the main line downstream of the junction, apply AR to that peak. Find the tributary flow at the main line time-of-peak, and apply tributary AR

factor to that flow. Upstream Net Main Line Flow is equal to the Downstream Main Line AR flow minus tributary AR flow.

2. From the tributary point-of-view: Find the time of tributary peak, apply AR to that peak. Find the main line flow downstream of the junction at the tributary time-of-peak, and apply main line AR factor to that flow. Upstream Net Main Line Flow is equal to the Downstream Main Line AR flow minus tributary AR peak.

The results are generally used in hydraulic analyses of junctions using VCRat model results and the most conservative case (generally when the tributary is peaking) governs the design. For HMS and HSPF models, because AR is applied in the model to the input rainfall, two model runs are required to provide the flow results for the two points-of-view, one for the junction and one for the main line.

6.17 SEDIMENT YIELD CALCULATION UPDATES FOR BASIN DESIGN

The District's Debris and Detention Basin Manual updated in 2005 describes the requirements for basin design for facilities to be maintained by the District. If a detention basin is located downstream of an undeveloped area that can produce sediment, it is generally required to have enough volume for 125% of the 100-yr storm sediment yield in addition to the volume that is required for flood storage and peak attenuation.

The regression equation used by the District to estimate design storm sediment yield volumes for basin design was developed by Scott and Williams (1978). Recently the District's basin sediment removal volume was used to evaluate and update the regression equation and develop equations with other parameters. The results show that the 1-day and 10-day rainfall that were important parameters in the Scott and Williams equation do not correlate as highly now. For basins with small watersheds of 1.6 sq mi or less, the maximum hourly intensity during a storm is a better predictor of yield. For large watershed basins, other studies have found that peak inflow is a better predictor. Because peak inflow is not measured at any of the District basins except for the new Pole Creek basin, 1-day rain is still used as a predictor of sediment yield in the updated equation for medium to large watersheds. The District has a draft report presenting the results of the analysis (2009). The results cannot be used for basin design but instead can be used for comparison purposes as the District finalizes the report findings.

6.18 NPDES REQUIREMENTS

NPDES requirements for projects are outlined in the Technical Guidance Manual available through the District's website. Those requirements have been developed through the use of national methodologies applied to Ventura County and also evaluations of local data. Most of the methodologies involve using a rational method approach to estimating runoff volumes based on design rainfall intensity data. The one flow-based peak method provided in the Technical Guidance Manual (TGM) allows the user to calculate the Stormwater Quality Design Flow (SQDF) peak from a model run that provides the 50-yr peak for the developed area by multiplying the peak times 0.10. The resultant peak that has to be treated by the NPDES system is close to the peak that would be expected from a 2-yr storm for an undeveloped subarea using the District's standard multipliers. The other methods provided in the Technical Guidance Manual result in Stormwater Quality Design Volumes (SQDV) that need to be treated to meet the NPDES requirements. The design numbers should be obtained from the TGM.

SECTION 7 REFERENCES

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VCWPD, 2003. Calleguas Creek Watershed Management Study Hydrology Model. March 2003.

APPENDIX A: EXHIBITS

EXHIBIT 1A. DESIGN STORM RAINFALL CONTOURS- 100-YR STORM

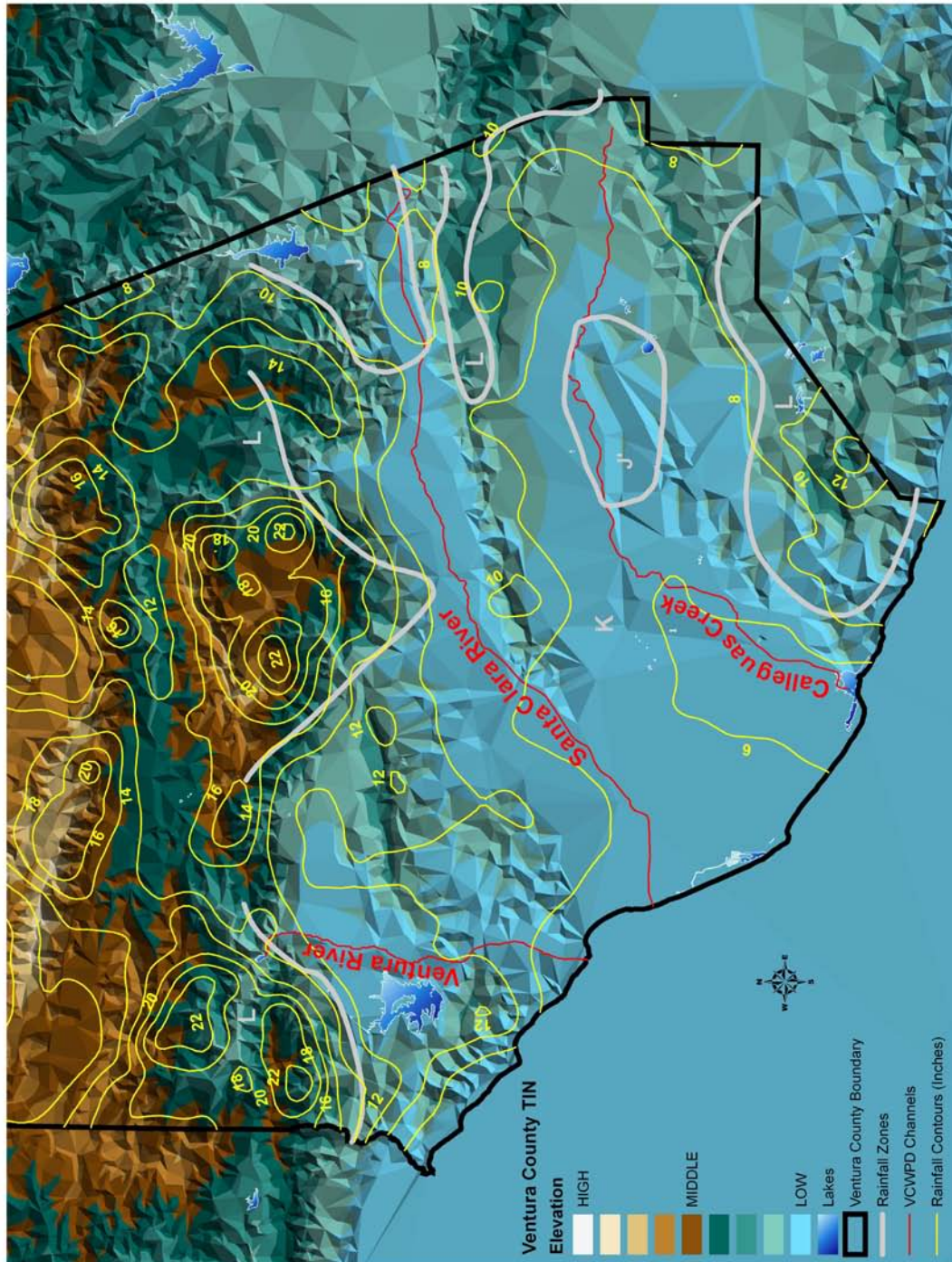


EXHIBIT 1B. DESIGN STORM RAINFALL CONTOURS- 50-YR STORM

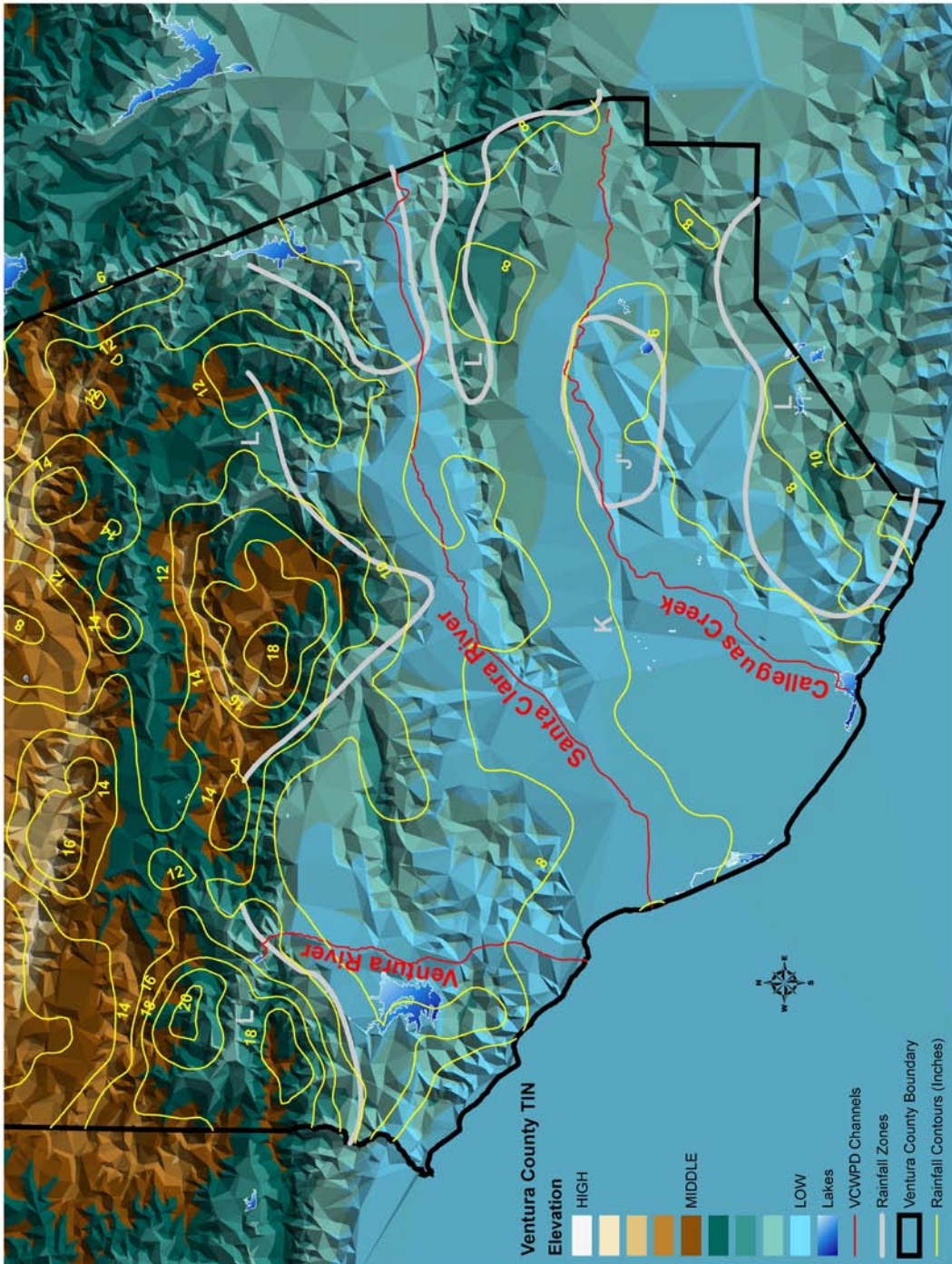


EXHIBIT 1C. DESIGN STORM RAINFALL CONTOURS- 25-YR STORM

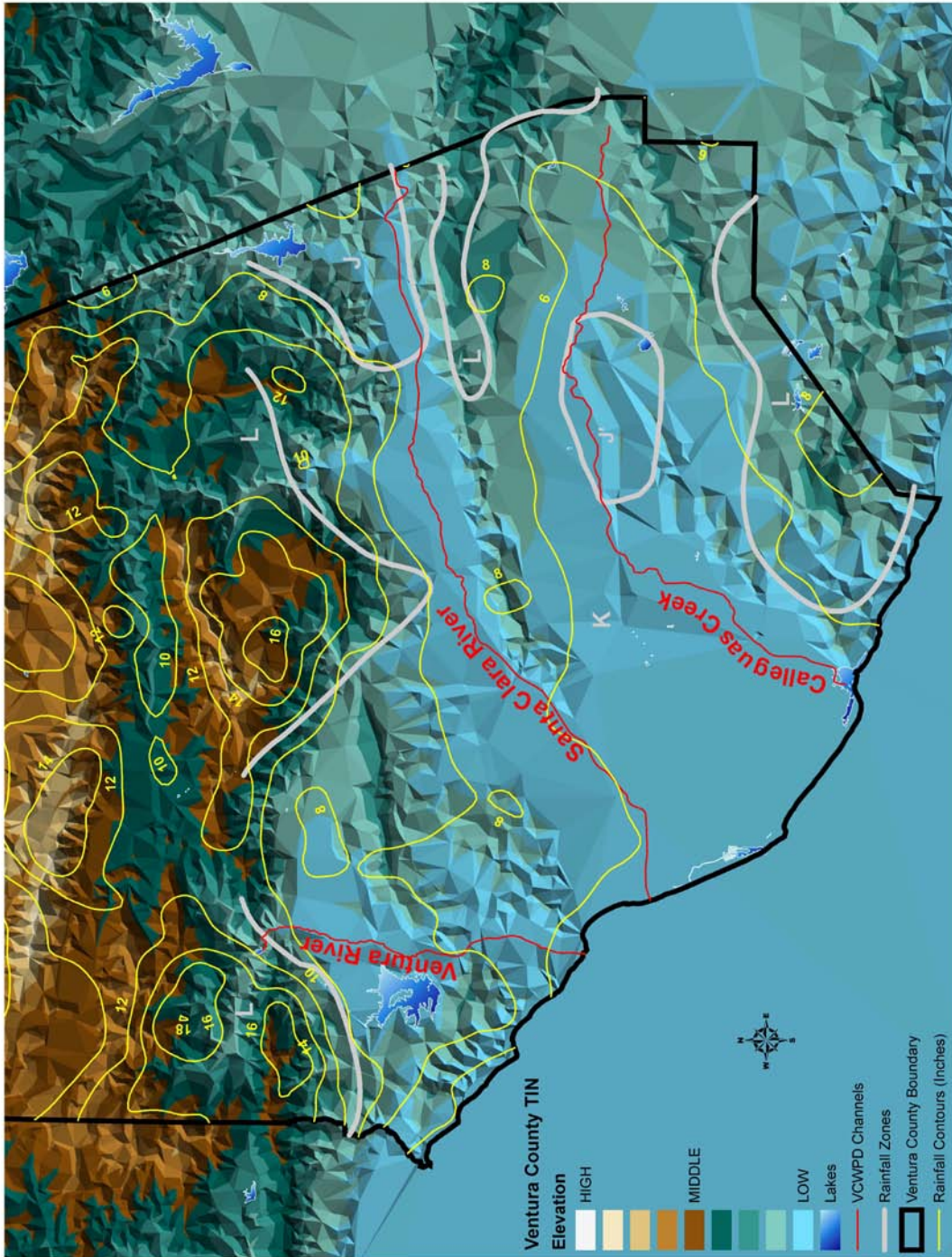


EXHIBIT 1D. DESIGN STORM RAINFALL CONTOURS- 10-YR STORM

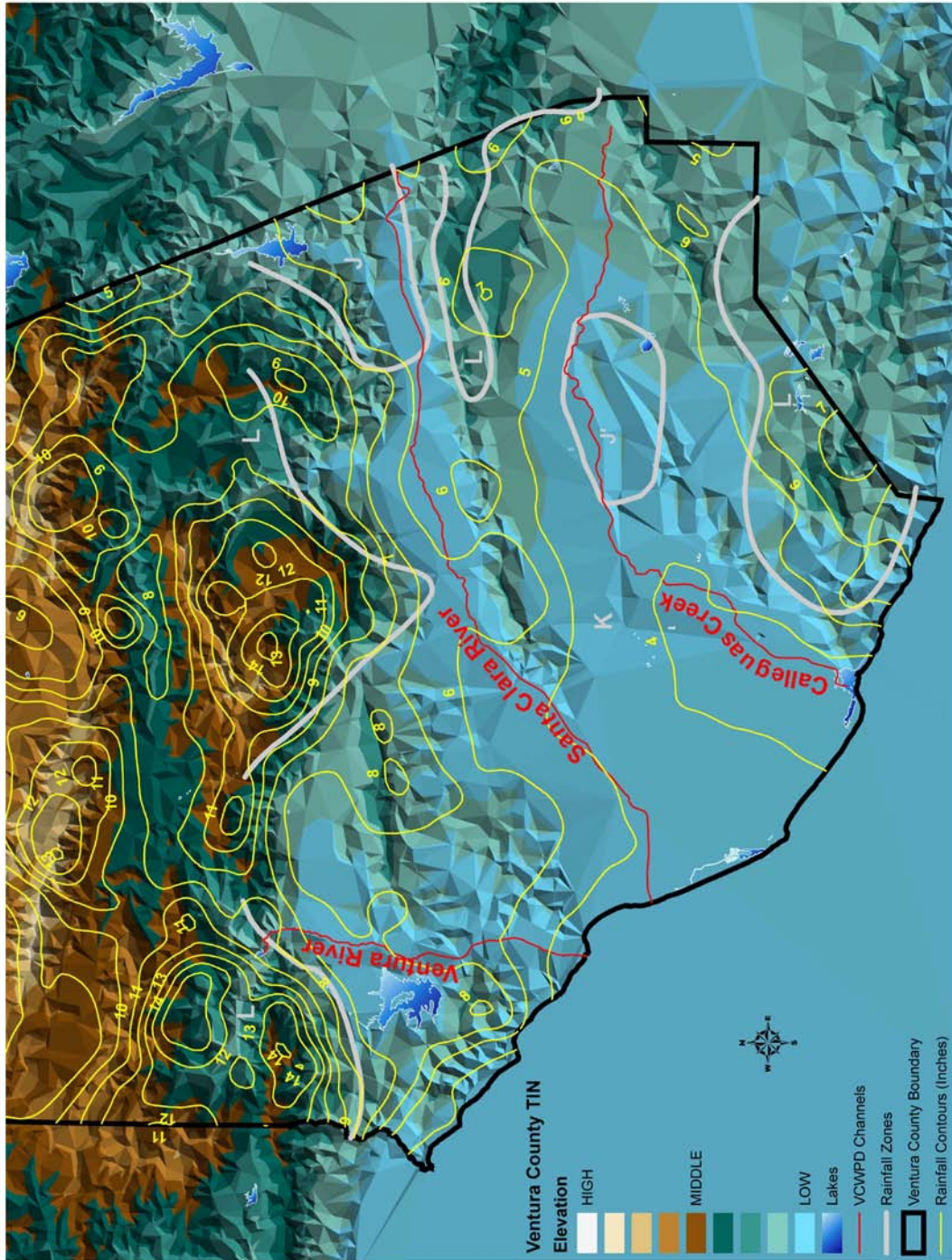


EXHIBIT 2. MAXIMUM RAINFALL INTENSITIES

Zone	J	Jp	K	L	J	Jp	K	L	J	Jp	K	L	J	Jp	K	L
Year	10	10	10	10	25	25	25	25	50	50	50	50	100	100	100	100
Cum. Rain (in.)	3.17	4.38	5.53	7.21	3.91	5.28	6.41	8.81	5.0	6.0	8.0	11.0	7.0	6.66	10.6	15.0
Tc (min)	Maximum Rainfall Intensity (in/hr)															
5	2.16	2.16	3.72	4.31	2.64	3.34	4.27	4.94	2.94	3.79	4.55	5.58	3.23	4.06	5.10	6.11
6	2.02	2.01	3.40	3.90	2.52	2.94	3.80	4.39	2.80	3.34	4.10	5.05	2.90	3.55	4.59	5.43
7	1.86	1.90	3.09	3.56	2.30	2.65	3.45	3.99	2.55	3.01	3.77	4.63	2.67	3.19	4.23	4.95
8	1.74	1.82	2.86	3.30	2.14	2.58	3.19	3.69	2.36	2.93	3.52	4.28	2.50	2.99	3.95	4.58
9	1.63	1.76	2.68	3.07	1.99	2.44	2.99	3.45	2.21	2.77	3.33	4.00	2.36	2.87	3.74	4.30
10	1.53	1.70	2.52	2.86	1.87	2.29	2.81	3.24	2.08	2.60	3.16	3.76	2.25	2.78	3.57	4.07
11	1.45	1.64	2.40	2.70	1.76	2.17	2.66	3.07	1.95	2.46	3.02	3.56	2.13	2.67	3.39	3.88
12	1.38	1.59	2.29	2.56	1.66	2.07	2.53	2.92	1.85	2.35	2.90	3.39	2.02	2.58	3.23	3.72
13	1.33	1.55	2.20	2.44	1.58	1.98	2.43	2.80	1.76	2.25	2.80	3.25	1.94	2.49	3.10	3.59
14	1.28	1.51	2.12	2.34	1.52	1.90	2.34	2.70	1.68	2.16	2.72	3.13	1.86	2.42	2.99	3.47
15	1.23	1.47	2.04	2.25	1.46	1.84	2.26	2.60	1.62	2.09	2.62	3.02	1.80	2.36	2.89	3.37
16	1.18	1.43	1.98	2.18	1.40	1.78	2.18	2.50	1.56	2.02	2.54	2.92	1.73	2.29	2.79	3.25
17	1.14	1.39	1.92	2.11	1.36	1.73	2.12	2.42	1.50	1.96	2.47	2.83	1.67	2.22	2.70	3.14
18	1.11	1.35	1.86	2.04	1.31	1.68	2.06	2.34	1.45	1.90	2.41	2.75	1.61	2.16	2.62	3.05
19	1.07	1.32	1.82	1.99	1.27	1.63	2.01	2.28	1.41	1.86	2.35	2.68	1.56	2.11	2.55	2.96
20	1.04	1.29	1.77	1.94	1.24	1.60	1.96	2.22	1.37	1.81	2.29	2.62	1.52	2.07	2.49	2.88
21	1.02	1.26	1.73	1.90	1.20	1.55	1.91	2.17	1.33	1.76	2.23	2.55	1.48	2.03	2.43	2.82
22	0.99	1.23	1.68	1.85	1.17	1.51	1.87	2.12	1.30	1.72	2.17	2.49	1.44	1.99	2.36	2.76
23	0.97	1.21	1.65	1.82	1.14	1.48	1.83	2.07	1.27	1.68	2.12	2.44	1.41	1.95	2.31	2.70
24	0.95	1.19	1.62	1.78	1.12	1.44	1.79	2.03	1.24	1.64	2.07	2.39	1.38	1.92	2.26	2.65
25	0.93	1.16	1.58	1.75	1.09	1.41	1.76	1.99	1.21	1.61	2.03	2.34	1.35	1.89	2.22	2.60
26	0.90	1.14	1.56	1.72	1.07	1.39	1.73	1.96	1.18	1.57	1.98	2.29	1.32	1.86	2.17	2.56
27	0.88	1.13	1.53	1.68	1.05	1.36	1.70	1.92	1.16	1.54	1.94	2.25	1.29	1.83	2.13	2.51
28	0.87	1.11	1.50	1.66	1.03	1.34	1.67	1.89	1.14	1.52	1.90	2.21	1.27	1.80	2.09	2.46
29	0.85	1.09	1.48	1.63	1.01	1.31	1.64	1.87	1.12	1.49	1.87	2.17	1.24	1.77	2.05	2.42
30	0.83	1.08	1.46	1.61	0.99	1.29	1.61	1.84	1.10	1.47	1.84	2.13	1.22	1.74	2.02	2.38

EXHIBIT 3. HYDROGRAPH RAINFALL INTENSITIES - J ZONE

- Not provided in 2010 Manual, see 2006 Manual

EXHIBIT 4. HYDROGRAPH RAINFALL INTENSITIES - K ZONE

- Not provided in 2010 Manual, see 2006 Manual

EXHIBIT 5. HYDROGRAPH RAINFALL INTENSITIES - L ZONE

- Not provided in 2010 Manual, see 2006 Manual

EXHIBIT 6A. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 1 (NRCS TYPE D)

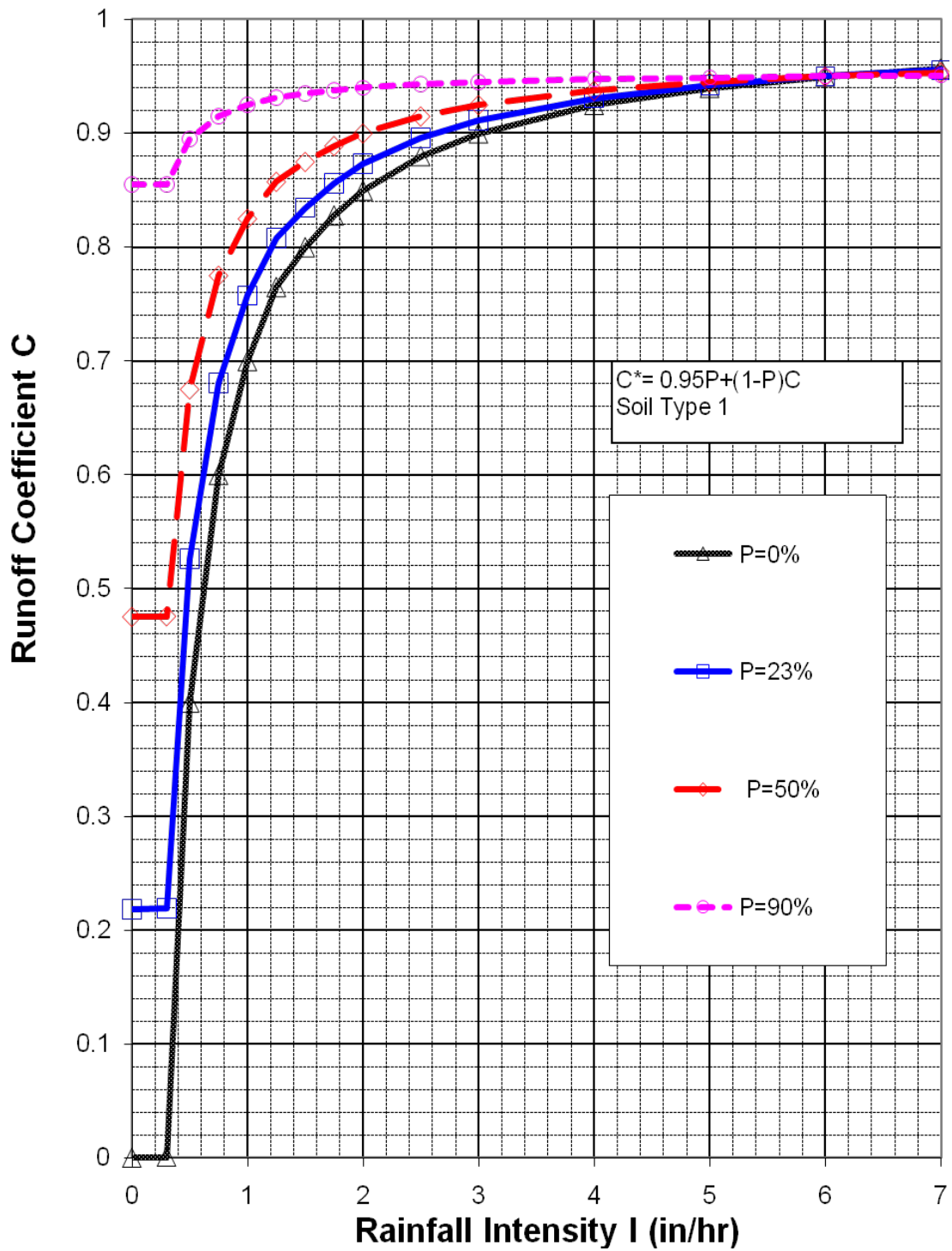


EXHIBIT 6B. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 2 (NRCS TYPE C)

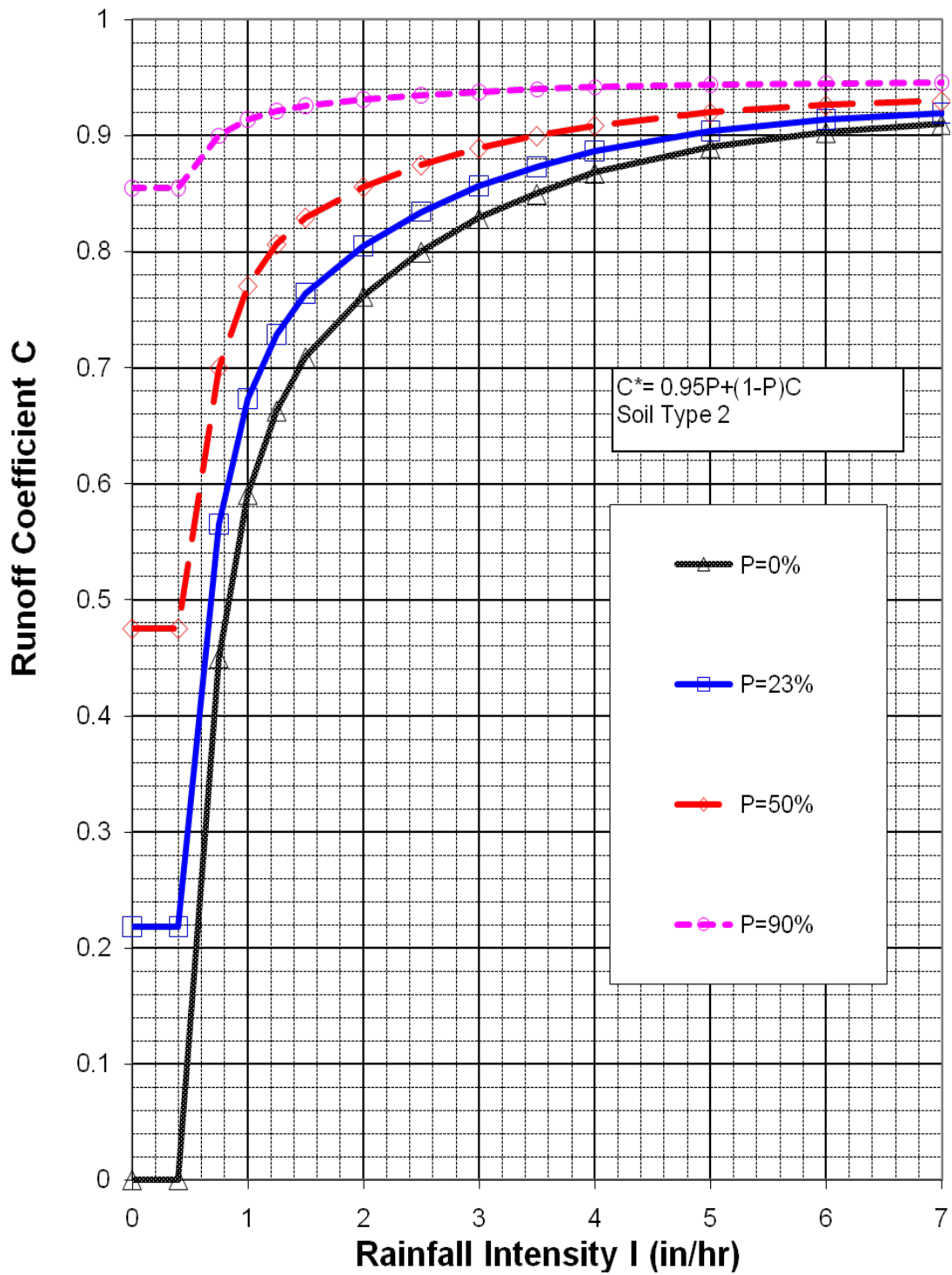


EXHIBIT 6C. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 3 (NRCS TYPE C)

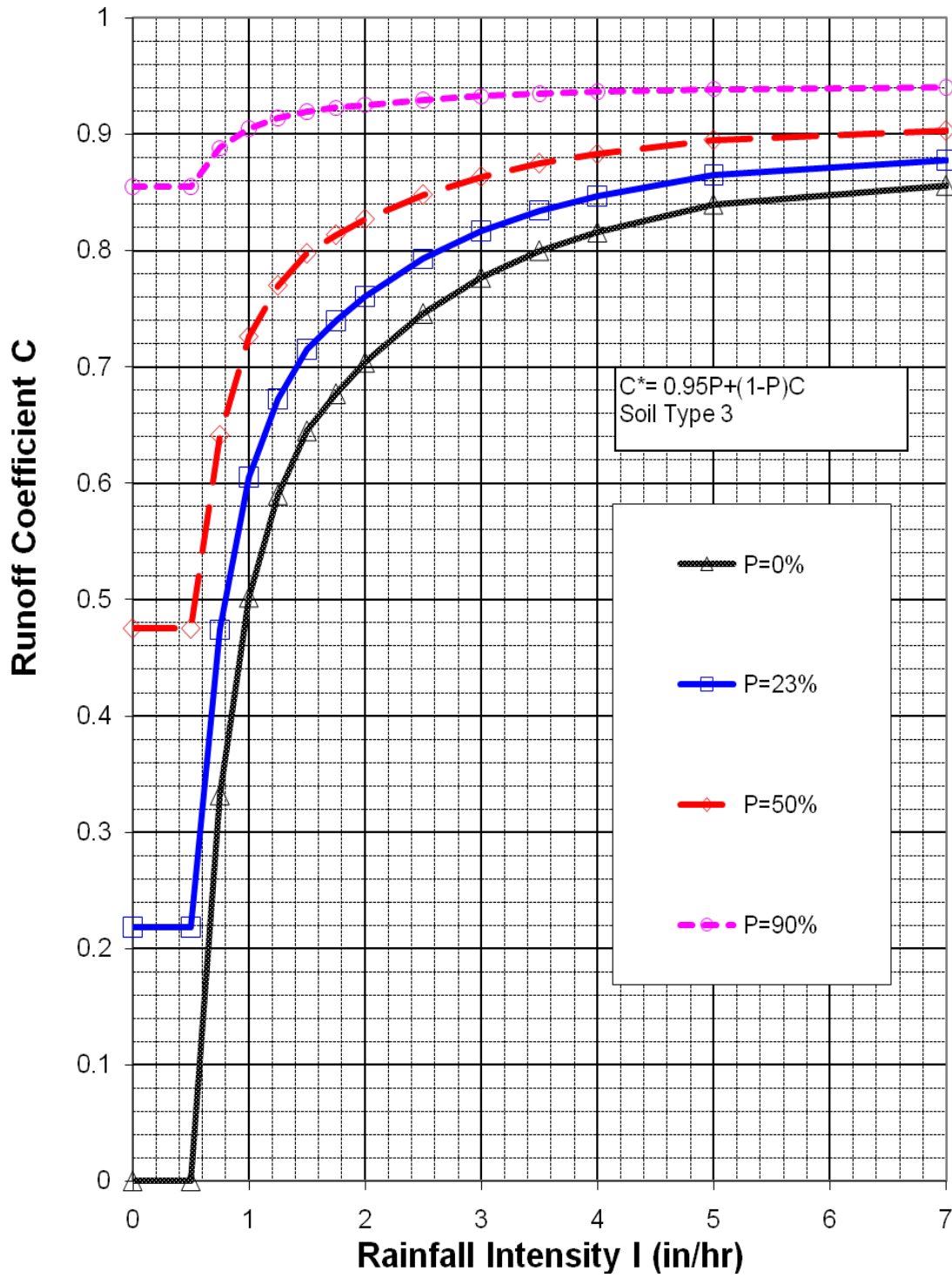


EXHIBIT 6D. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 4 (NRCS TYPE B)

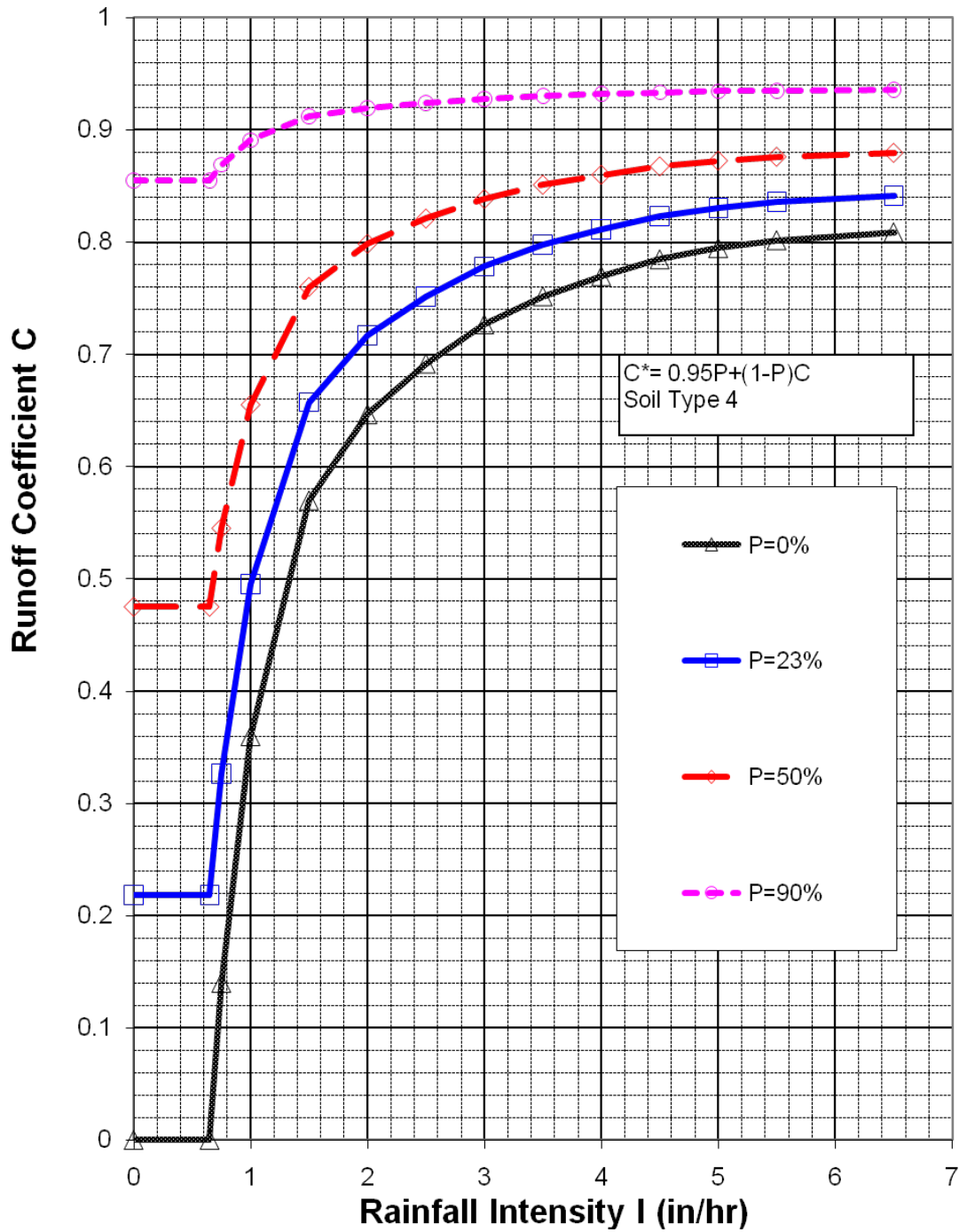


EXHIBIT 6E. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 5 (NRCS TYPE B)

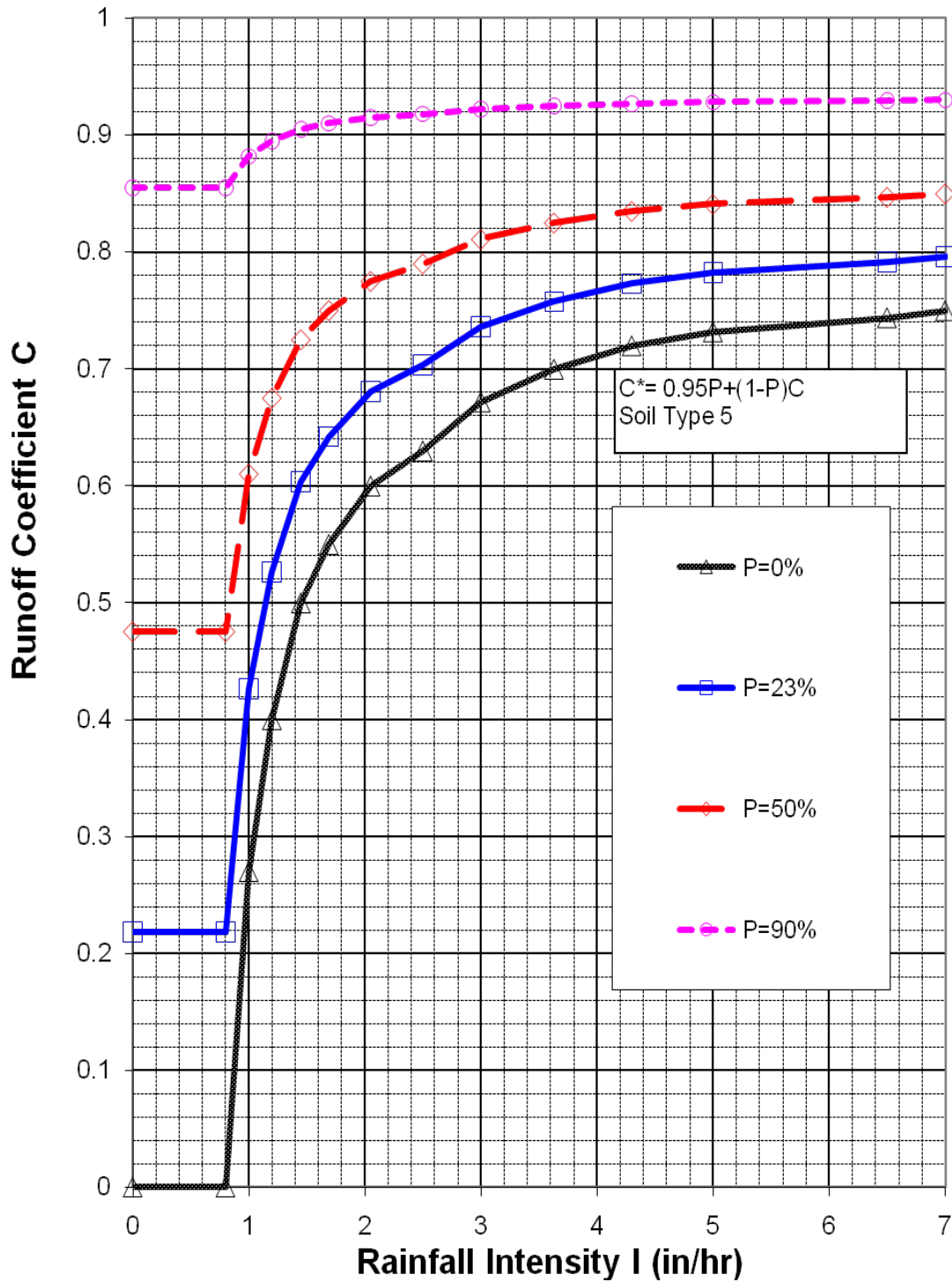


EXHIBIT 6F. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 6 (NRCS TYPE A)

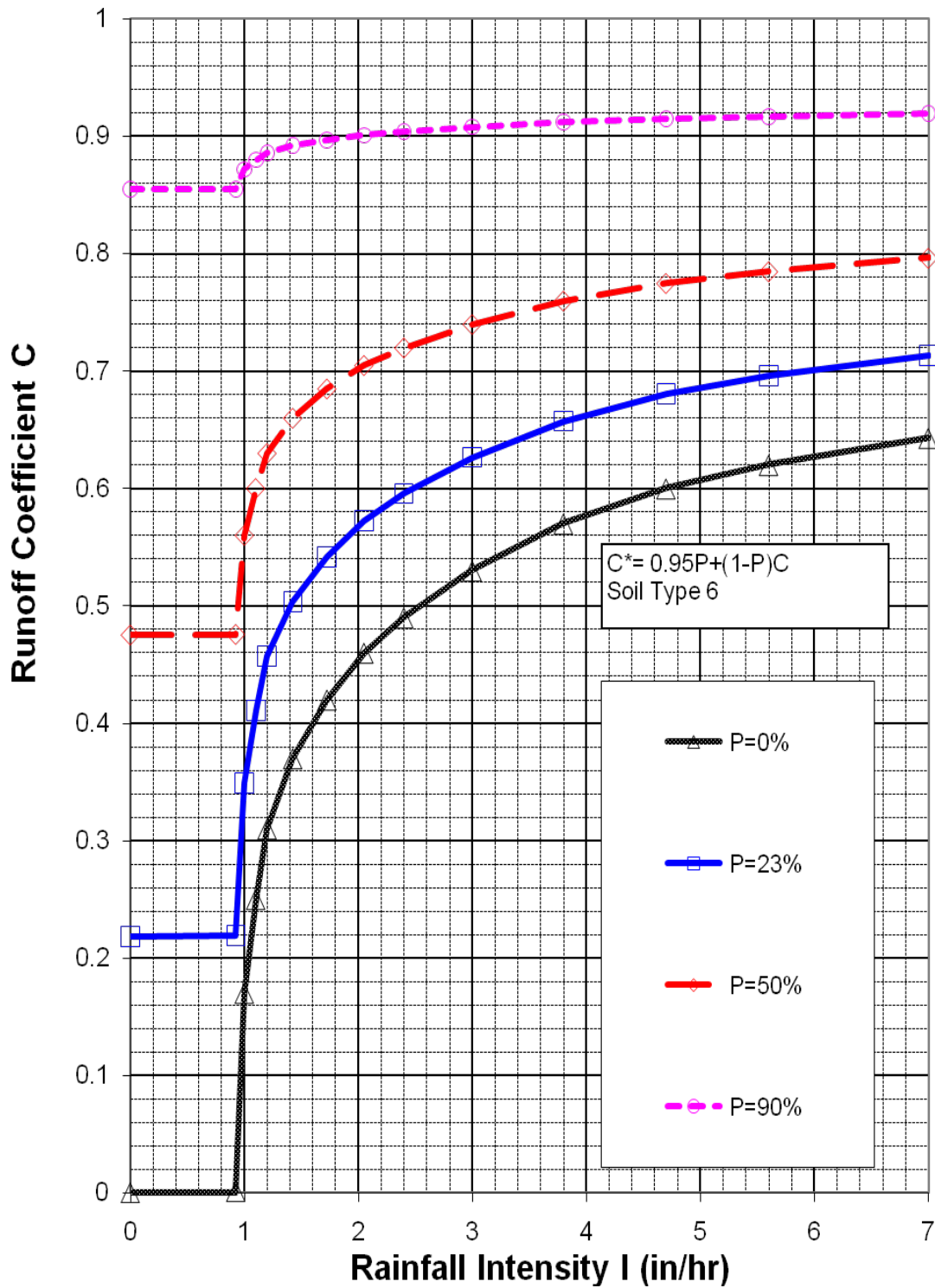


EXHIBIT 6G. RUNOFF COEFFICIENT CURVE- SOIL NUMBER 7 (NRCS TYPE A)

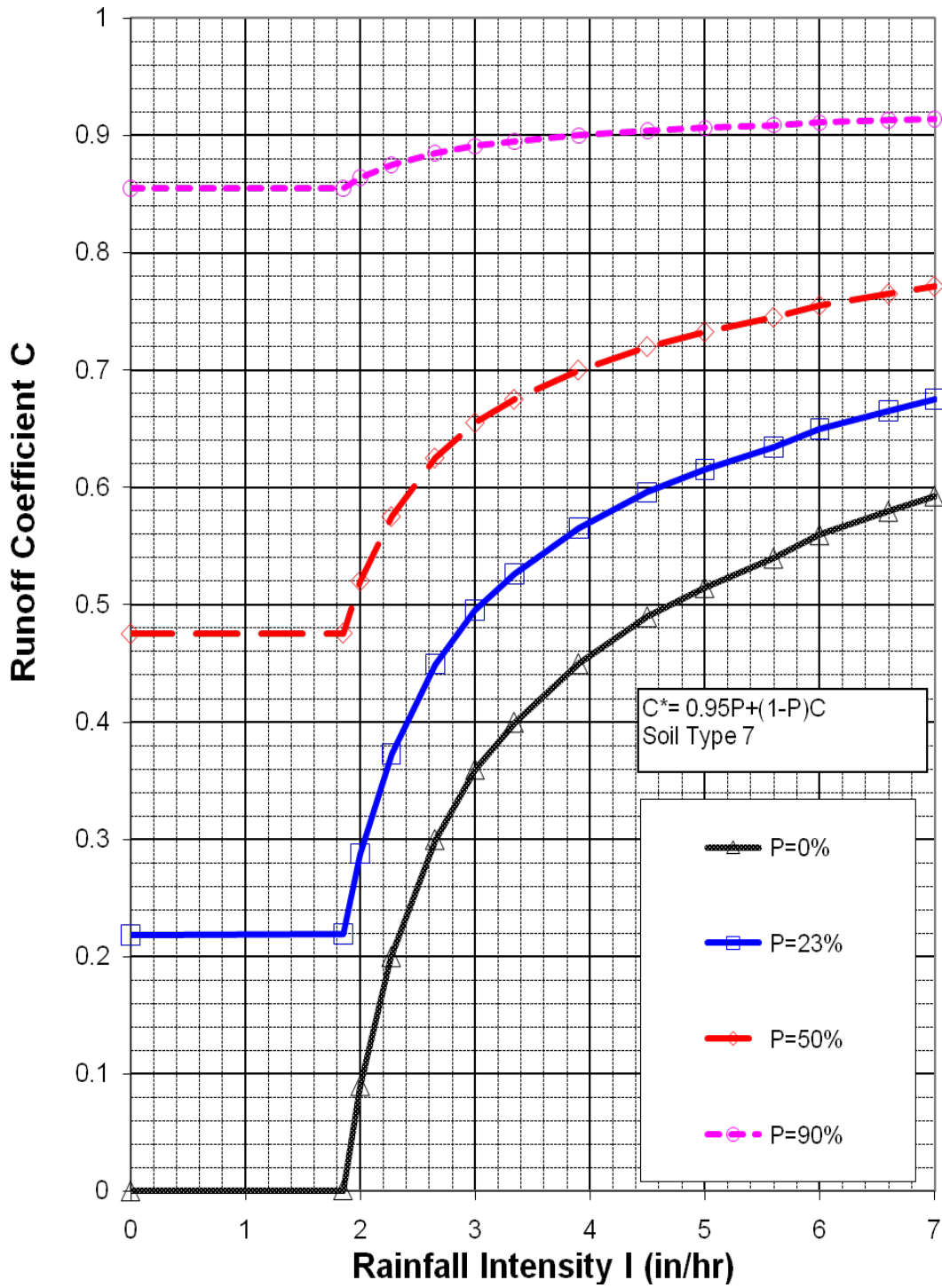


EXHIBIT 7. SLOPE CORRECTION CURVE FOR MOUNTAIN CHANNEL SCOUR VELOCITY CHECK

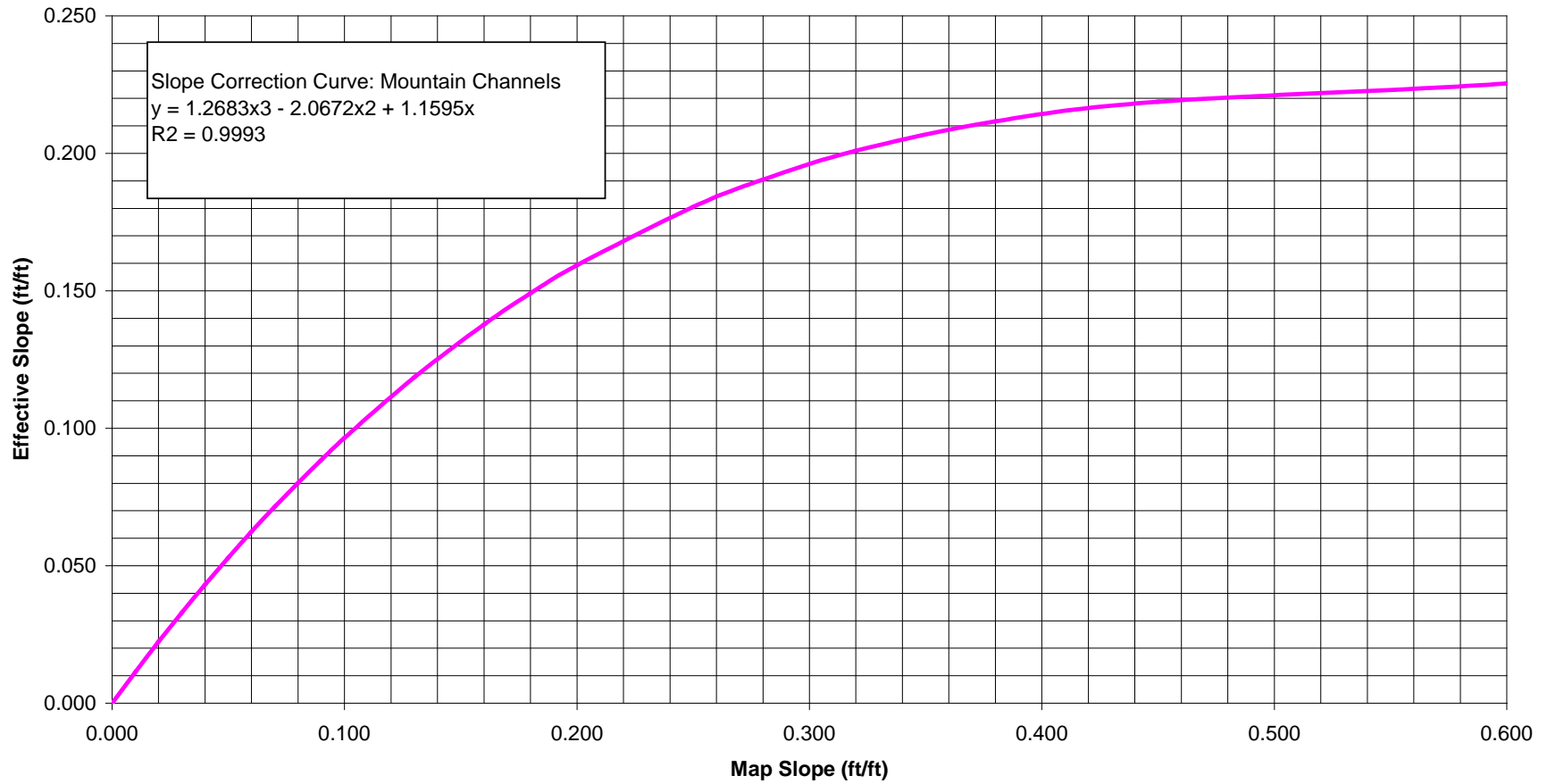
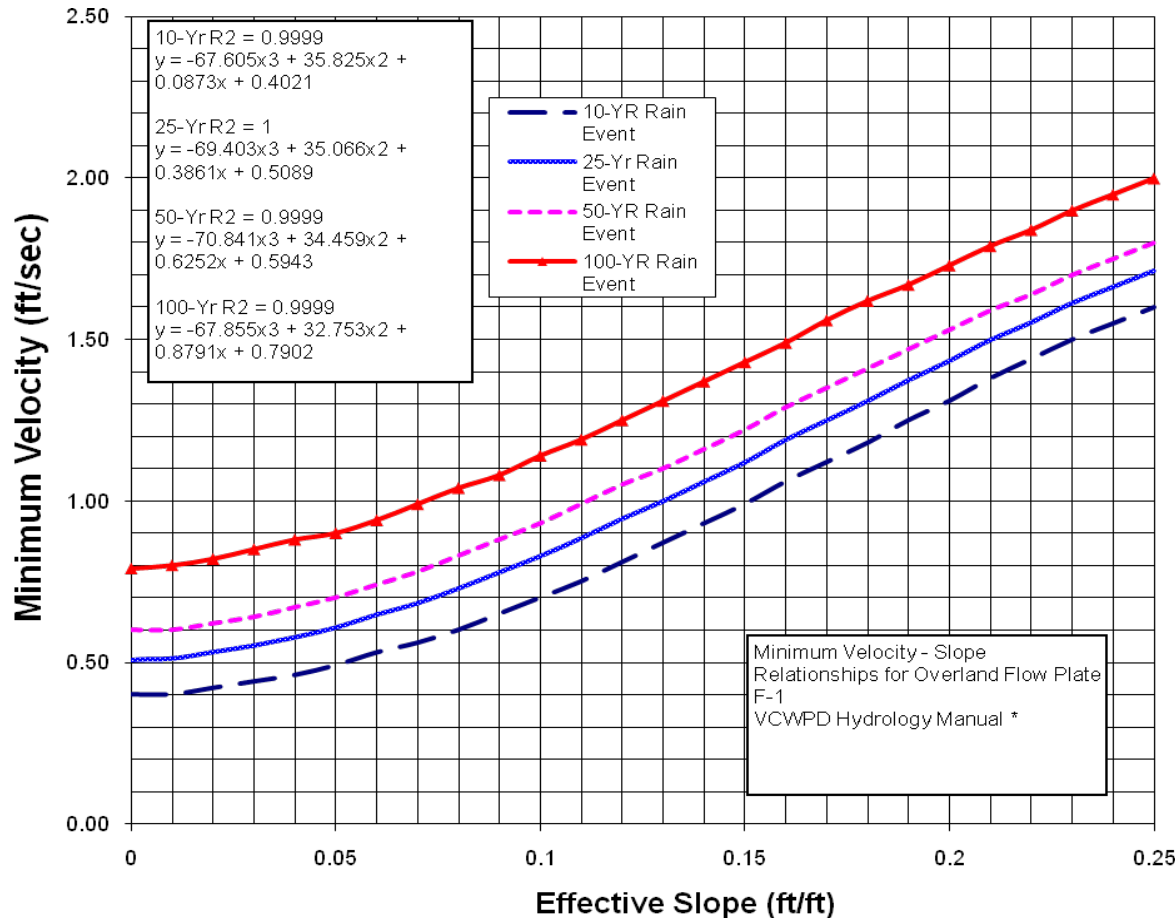


EXHIBIT 8. MINIMUM VELOCITY-SLOPE RELATIONSHIPS, OVERLAND FLOW



Note: Commercial and Industrial Areas: For 100- and 50-Yr storms, minimum velocity is 1.0 fps; for 25- and 10-Yr storms, minimum velocity is 0.5 fps.

EXHIBIT 9. VELOCITY-DISCHARGE-SLOPE RELATIONSHIPS, NATURAL MOUNTAIN CHANNELS

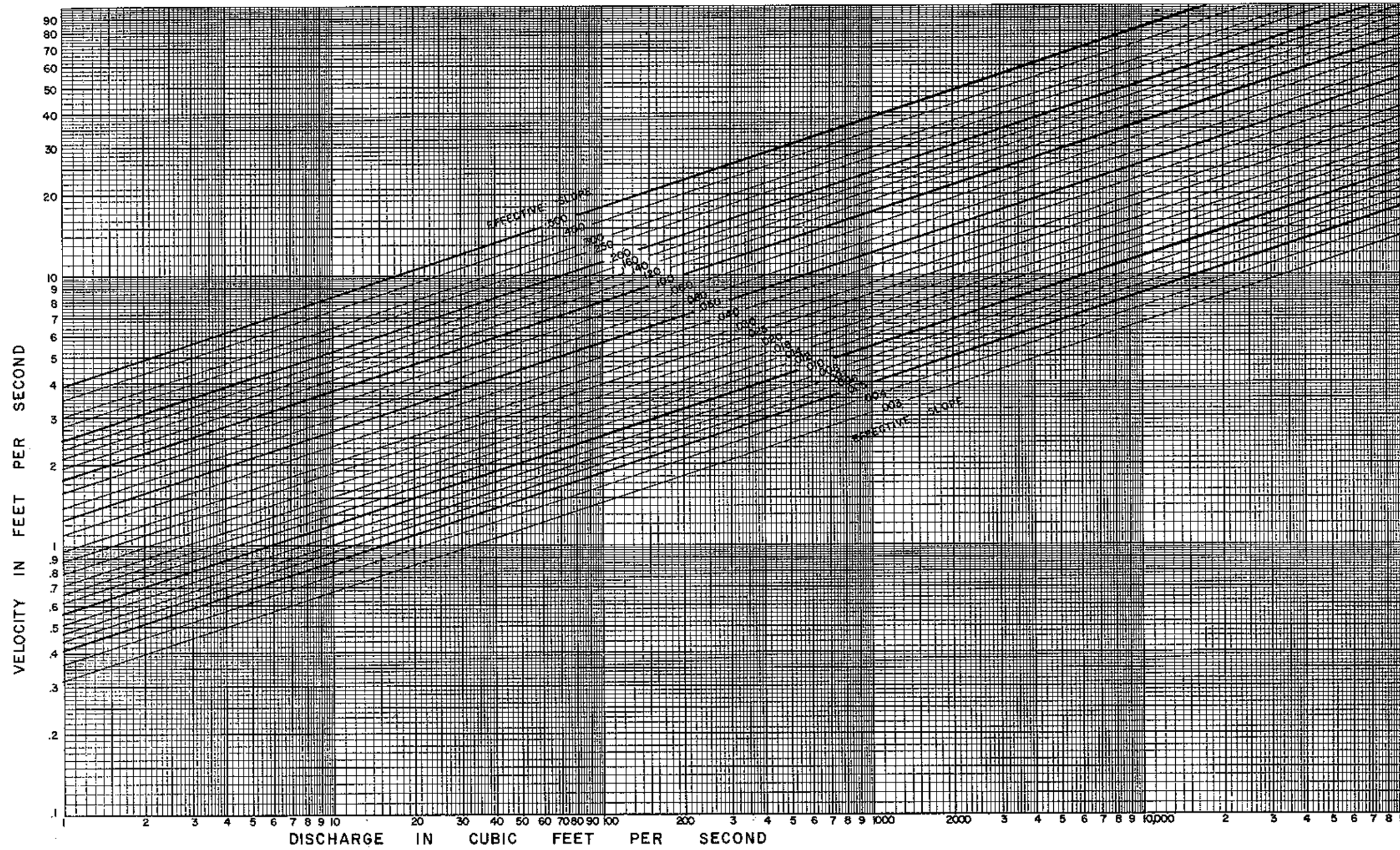


EXHIBIT 10. VELOCITY- DISCHARGE- SLOPE RELATIONSHIPS, NATURAL VALLEY CHANNELS

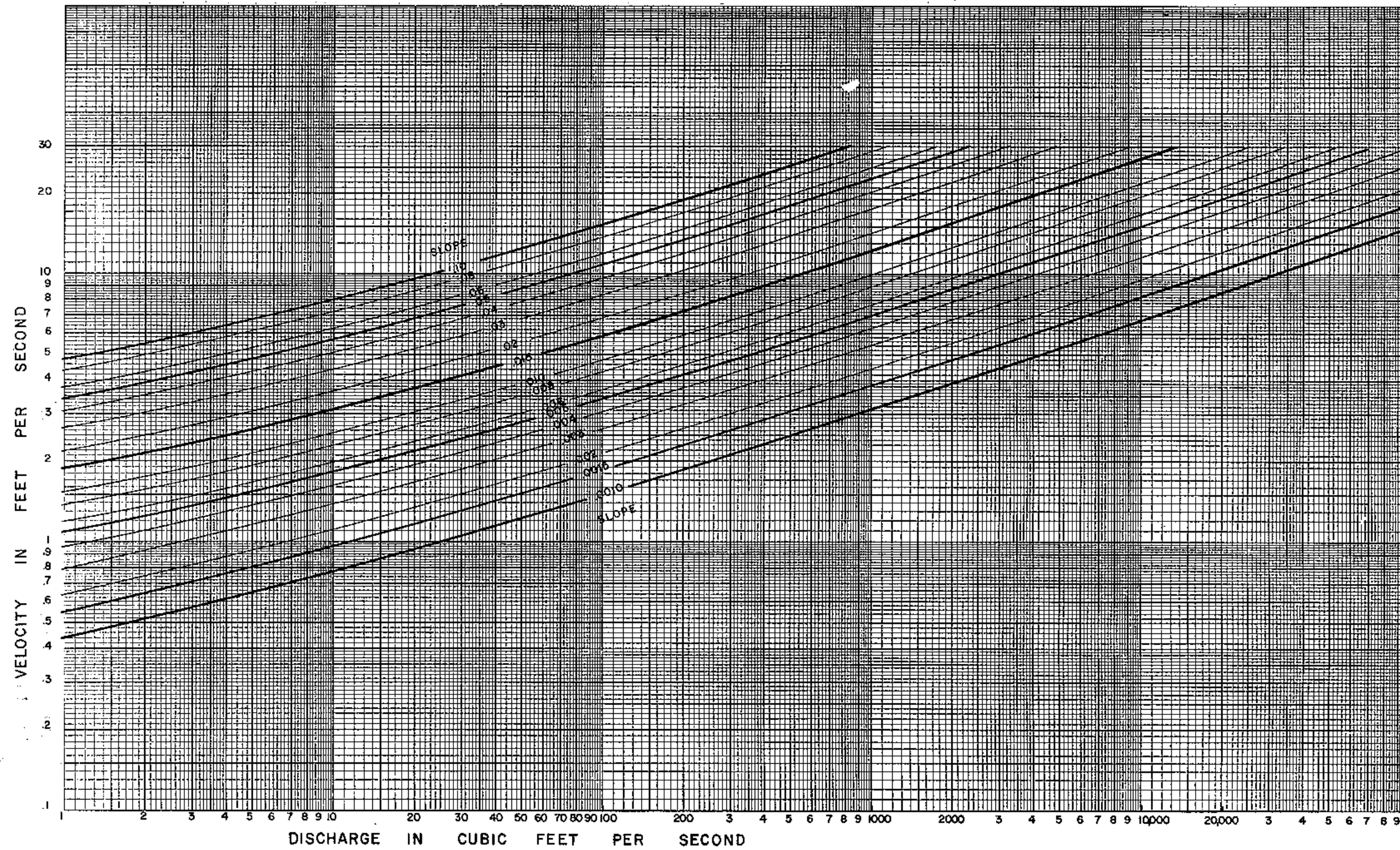
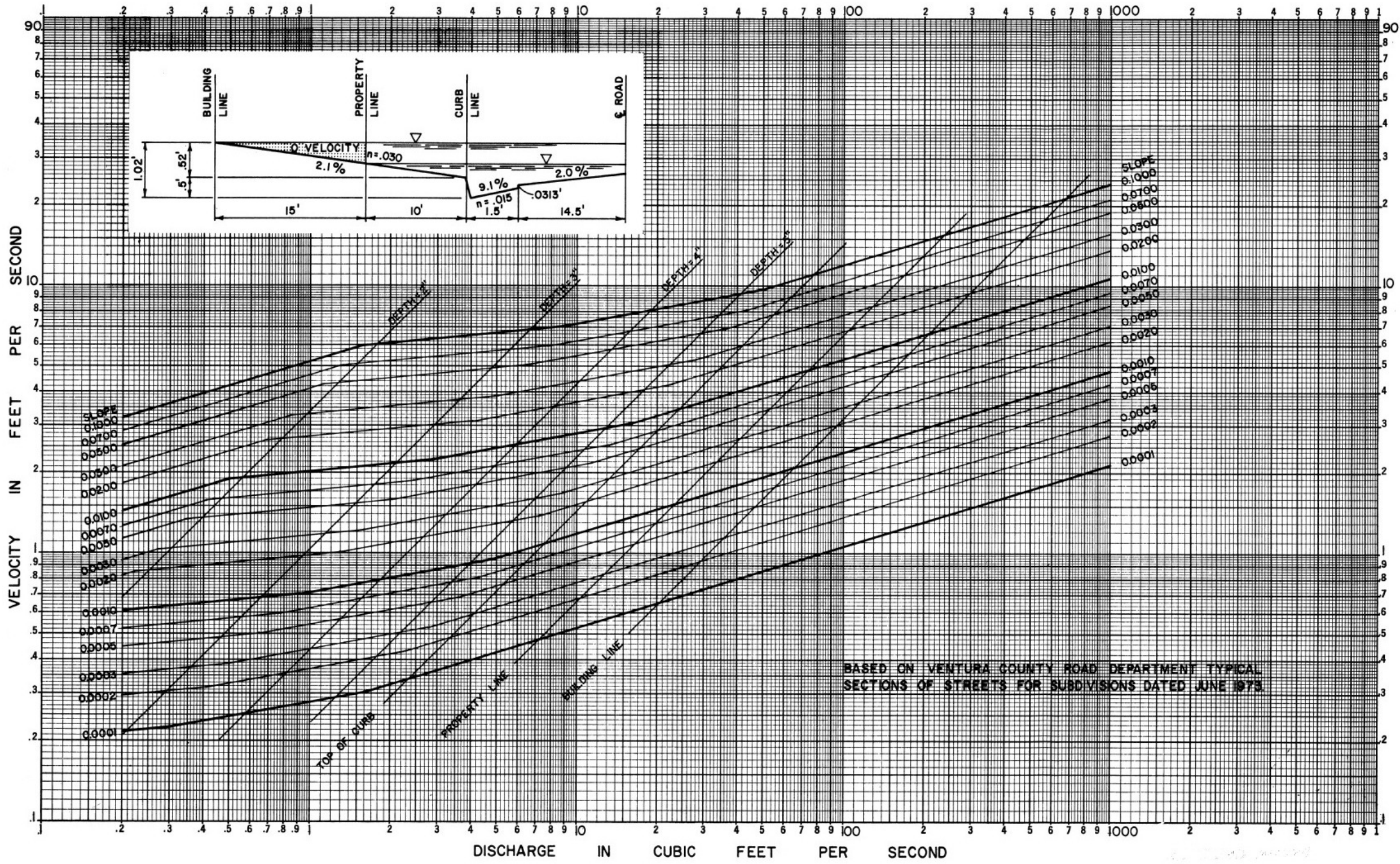


EXHIBIT 11A. VELOCITY- DISCHARGE- SLOPE RELATIONSHIPS, 32' ROAD WIDTH – 6" CURB



BASED ON VENTURA COUNTY ROAD DEPARTMENT TYPICAL SECTIONS OF STREETS FOR SUBDIVISIONS DATED JUNE 1973

EXHIBIT 11B. VELOCITY- DISCHARGE- SLOPE RELATIONSHIPS, 32' ROAD WIDTH – 8" CURB

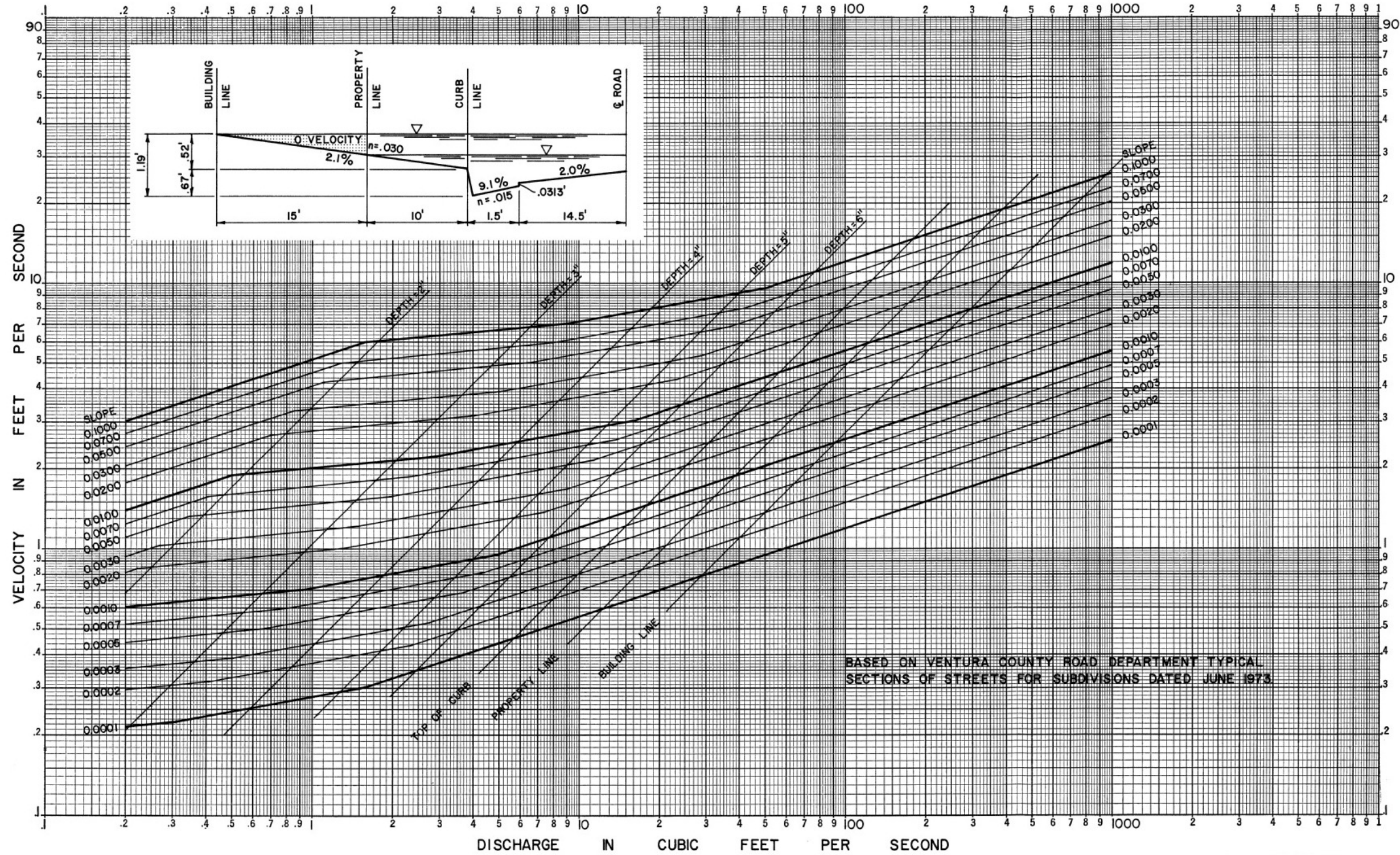


EXHIBIT 11C. VELOCITY- DISCHARGE- SLOPE RELATIONSHIPS, 40' ROAD WIDTH – 6" CURB

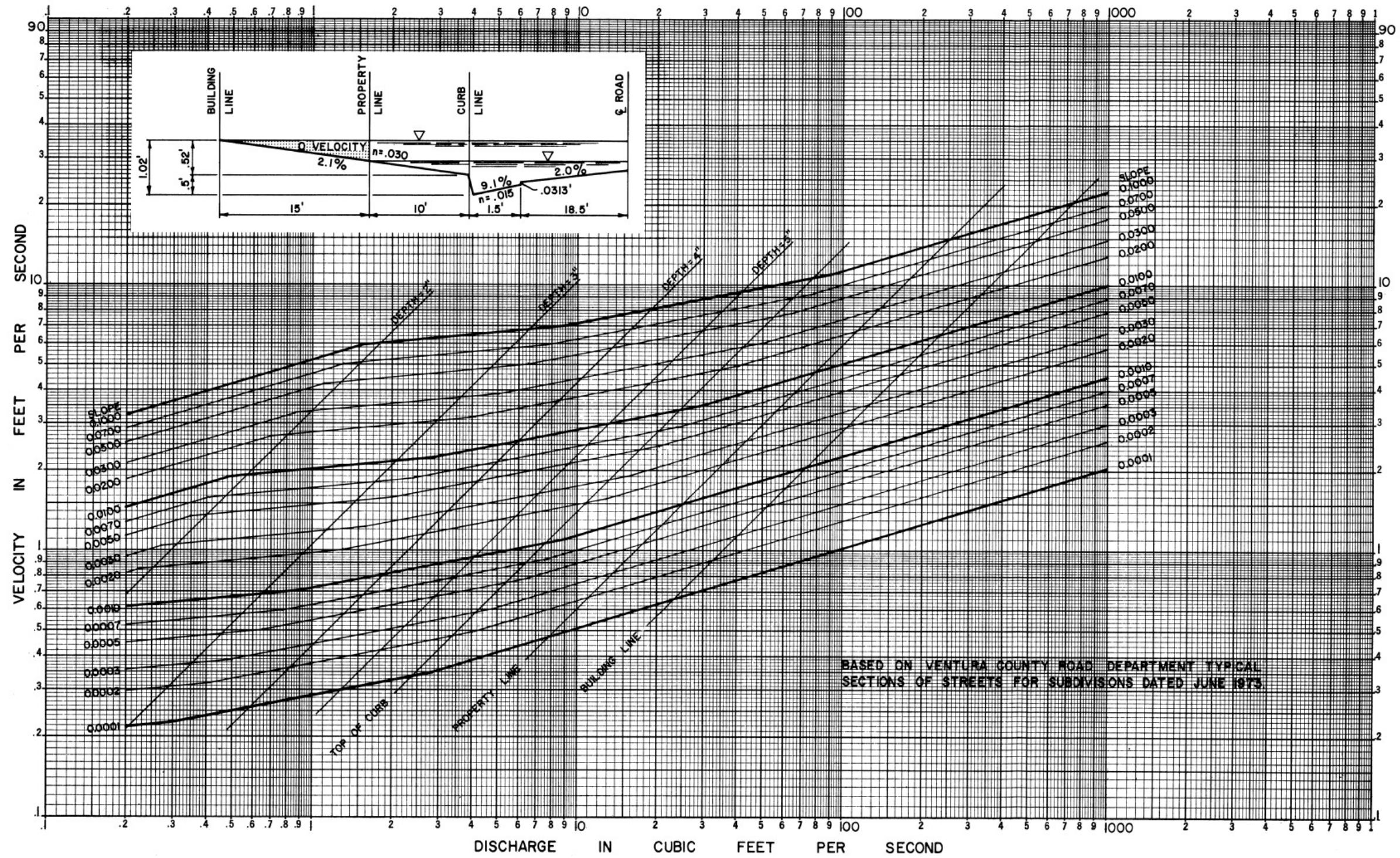


EXHIBIT 11D. VELOCITY- DISCHARGE- SLOPE RELATIONSHIPS, 40' ROAD WIDTH – 8" CURB

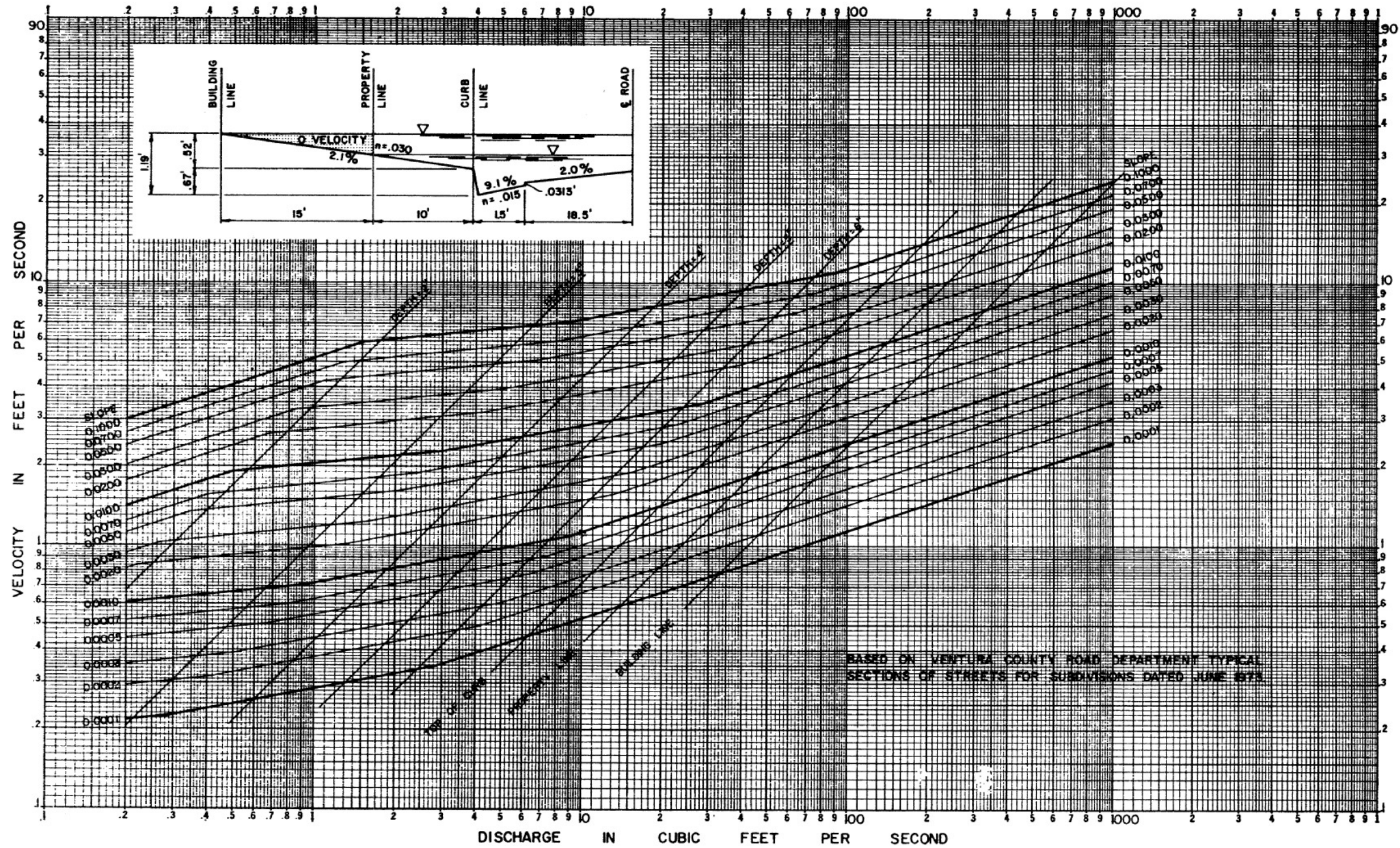


EXHIBIT 12A. WAVE VELOCITY- RECTANGULAR CHANNEL

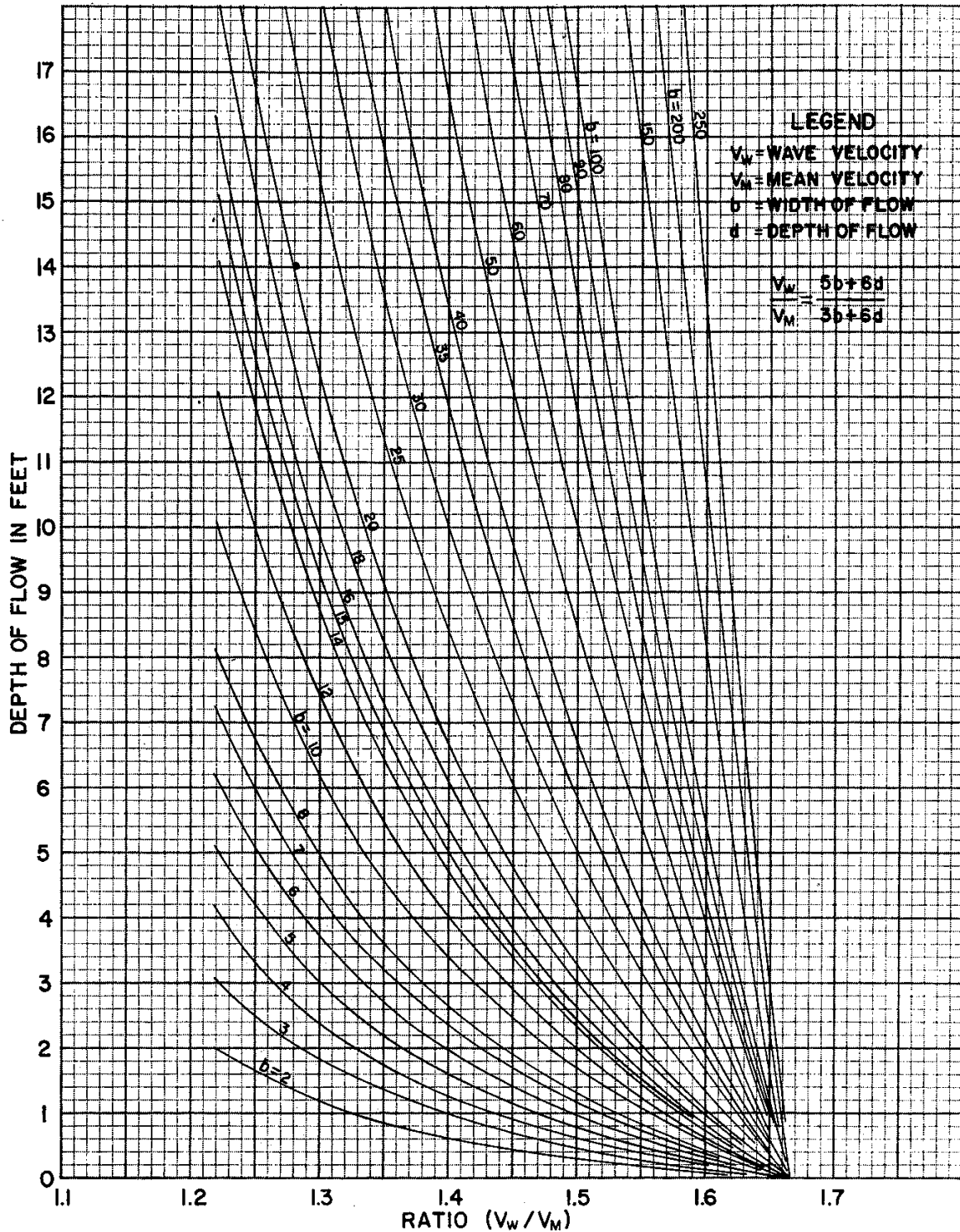
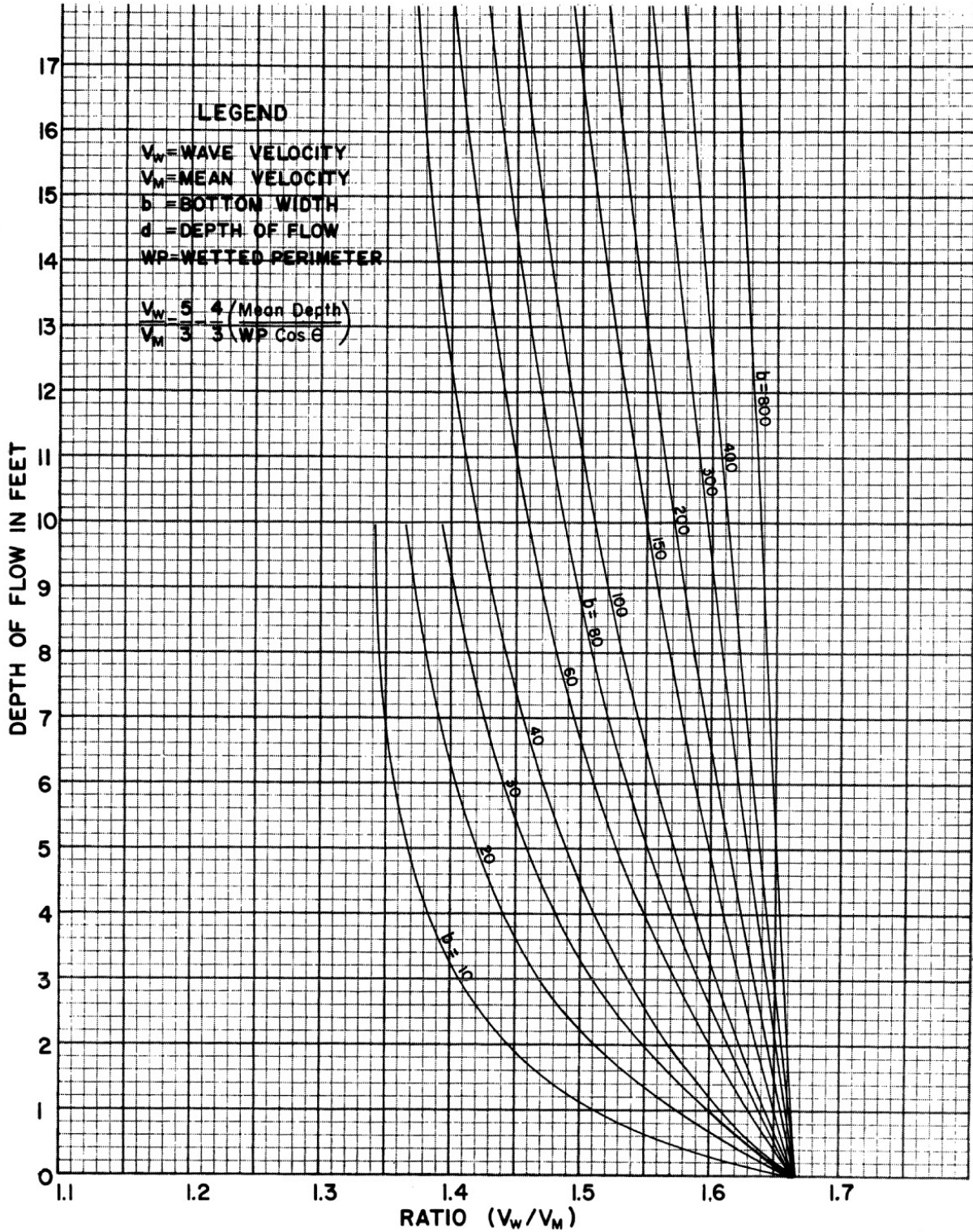


EXHIBIT 12B. WAVE VELOCITY- TRAPEZOIDAL CHANNEL 1.5:1 SIDESLOPE



%Q	%V	%Vw	%Q	%V	%Vw	%Q	%V	%Vw	%Q	%V	%Vw	%Q	%V	%Vw
0	0	--												
1	30	43	21	78	110	41	95	129	61	105	137	81	111	137
2	37	53	22	79	111	42	96	130	62	105	137	82	112	137
3	42	61	23	81	112	43	96	130	63	106	137	83	112	137
4	46	66	24	82	113	44	97	131	64	106	137	84	112	137
5	50	70	25	83	115	45	97	131	65	106	137	85	112	137
6	53	74	26	84	116	46	98	132	66	107	137	86	112	137
7	55	78	27	85	117	47	98	132	67	107	137	87	113	137
8	57	81	28	86	118	48	99	133	68	107	137	88	113	137
9	59	84	29	86	119	49	100	133	69	108	137	89	113	137
10	62	88	30	87	120	50	100	133	70	108	137	90	113	137
11	64	91	31	88	121	51	101	134	71	108	137	91	113	137
12	66	93	32	89	122	52	101	134	72	109	137	92	113	137
13	68	95	33	90	123	53	101	135	73	109	137	93	113	137
14	69	97	34	90	124	54	102	135	74	109	137	94	114	137
15	71	99	35	91	125	55	102	136	75	110	137	95	114	137
16	72	101	36	92	126	56	103	136	76	110	137	96	114	137
17	74	103	37	93	126	57	103	136	77	110	137	97	114	137
18	75	105	38	93	127	58	104	136	78	111	137	98	114	137
19	76	107	39	94	128	59	104	136	79	111	137	99	114	137
20	77	108	40	95	128	60	105	137	80	111	137	100	114	137

Note:

1. In %Q column find %Q based on pipe-full Q
2. On same line as %Q, find %V and %VW; These are percentages of mean velocity in pipe flowing full
3. Because pipe-full Q is not maximum Q due to boundary resistance, appears table only goes up to depth/diameter = 0.82

EXHIBIT 12C. WAVE VELOCITY- CIRCULAR PIPE

EXHIBIT 13. DIRECT RUNOFF FOR VARIOUS CURVE NUMBERS- NRCS

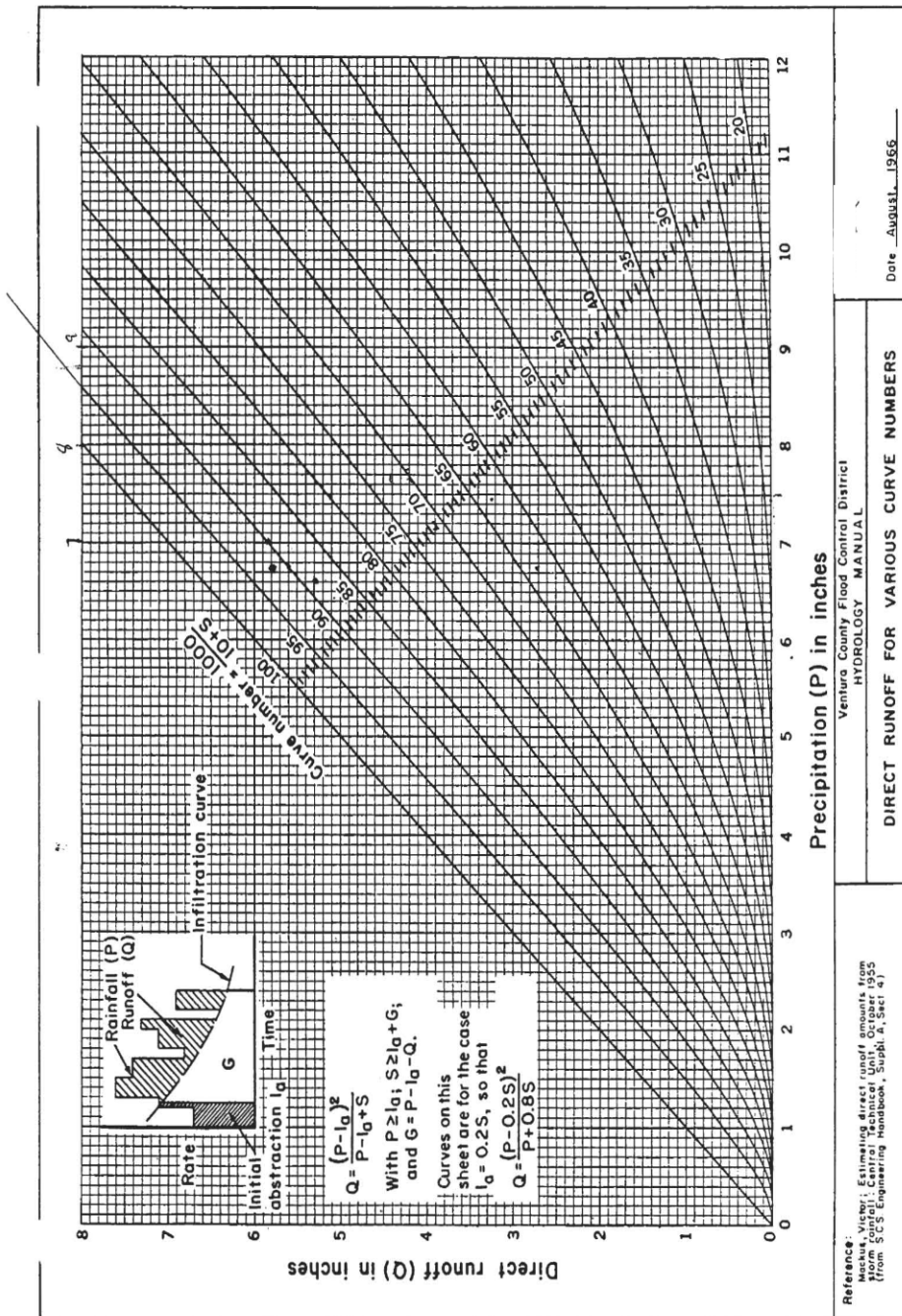


FIGURE 8

EXHIBIT 14A. AMC II NRCS CURVE NUMBERS FOR UNDEVELOPED LAND

UNDEVELOPED LAND USE AND CONDITION		% Impervious		HYDROLOGIC SOIL GROUP AND VCWPD NUMBERS						
				A (1), (2)		B		C		D (3)
		Effective	Average	7	6	5	4	3	2	1
Poor: Less than 50% Cover										
Fair: From 50% to 75% Cover										
Good: More Than 75% Cover										
Grassland (Annual Grass)	Poor	0	0	46	57	60	63	68	72	76
"	Fair	0	0	21	42	47	53	60	66	70
"	Good	0	0	-	-	41	47	54	59	64
Open Brush (Sagebrush, Flattop Buckwheat)	Poor	0	0	31	51	55	60	66	70	75
"	Fair	0	0	22	40	44	49	54	58	61
"	Good	0	0	-	-	33	39	46	51	56
Big Brush (Scrub Oak, Manzanita, Ceanothis)	Fair	0	0	23	39	42	46	51	54	59
"	Good	0	0	-	-	29	34	41	46	51
Chamise (Narrow Leaf Chaparral)	Fair	0	0	21	43	48	55	63	68	75
"	Good	0	0	-	-	44	49	55	60	64
Oak Savannah (Sparse Oaks & Annual Grass)	Poor	0	0	34	53	57	62	67	71	-
"	Fair	0	0	22	41	45	51	57	61	-
Orchard	Poor	0	0	42	56	59	62	65	67	71
Woodland	Fair	0	0	-	-	35	39	43	47	-
Pinon & Juniper	Fair	0	0	-	-	43	48	54	58	62
Forest	Fair	0	0	22	41	45	50	56	60	64
Pasture or Range	Poor	0	0	61	76	78	81	84	87	89
"	Fair	0	0	40	61	65	71	77	81	84
"	Good	0	0	29	52	57	64	71	76	80
NOTE: WPD MODIFIED RATIONAL METHOD USES SOIL TYPES 1-7 AND EFFECTIVE IMPERVIOUS PERCENTAGE IN VCRat MODEL										
Note (1)	Curve numbers for soil types 6 and 7 not all available									
Note (2)	For CNs<30, ensure that $P-0.2*S > 0$									
Note (3)	Curve numbers for soil type 1 not all available									
Reference:	Boyle, 1967. Revised Hydrologic Analysis, Zone II except Pasture from NRCS TR-55 Table 2-2c. For other land use types see TR-55									

Exhibit 14b. AMC II NRCS Curve Numbers for Developed Land

DEVELOPED LAND USE	Condition (1)	% IMPERVIOUS		HYDROLOGIC SOIL GROUP (5)						
		EFFEC- TIVE	AVER- AGE	A		B		C		D
				7	6	5	4	3	2	1
Open Spaces, Lawns, Parks, Golf Courses, Cemeteries, etc.	Good	0	0	29	52	57	64	71	76	80
"	Fair	0	0	42	61	65	71	77	81	84
Residential 1 ac. Lot	-	10	20	45	62	66	71	76	80	84
Residential 1/2 ac. Lot	-	13	25	45	65	68	73	78	81	85
Residential 1/3 ac. Lot	-	15	30	48	67	70	75	79	82	86
Residential 1/4 ac. Lot	-	19	38	53	70	73	77	81	84	87
Residential 1/5 ac. Lot	-	23	47	59	74	77	80	84	86	89
Residential 1/6 ac. Lot	-	28	56	66	79	81	84	86	88	90
Residential 1/8 ac. Lot	-	32	65	72	83	84	87	89	90	92
Residential - Condos	-	37	69	74	84	86	88	90	92	93
Industrial Unpaved Yards, etc.	-	36	72	77	86	87	89	91	92	93
Commercial & Business	-	50	85	88	90	91	93	93	95	95
Industrial Parks, Paved Parking, etc.	-	70	93	93	94	95	96	96	97	97
Parking Lots, Roofs, Driveways, Paved Streets with Curbs & Drains	-	90	100	98	98	98	98	98	98	98
Public Facilities & Institutions; Includes Schools, Government Centers, Military Bases, etc. (2)	-	23	47	59	74	77	80	84	86	89
Transportation and utilities (3)	-	70	93	79	87	88	90	91	92	93
Newly graded/under construction - No veg.	-	0	0	71	83	85	88	90	92	94
Paved Streets with open ditches including right-of-way (3)	-	70	93	79	87	88	90	91	92	93
Gravel streets including right-of-way	-	0	0	71	82	84	86	88	90	91
Dirt street including right-of-way	-	0	0	66	79	81	83	86	88	89
Natural desert landscaping- native vegetation	-	0	0	55	72	75	79	83	86	88
Farmsteads- buildings, lanes, driveways, and surrounding lots (2)	-	23	47	51	69	72	76	80	83	86
Agriculture- Straight Row + Crop Residue Cover on >5% of surface	Good	0	0	57	72	74	77	80	83	85
Agriculture- Straight Row + Crop Residue Cover on <5% of surface	Poor	0	0	64	78	80	83	86	88	90

DEVELOPED LAND USE	Condition (1)	% IMPERVIOUS		HYDROLOGIC SOIL GROUP (5)						
		EFFEC- TIVE	AVER- AGE	A		B		C		D
				7	6	5	4	3	2	1
Agriculture- Straight Row Good	Good	0	0	60	75	77	80	84	86	89
Agriculture- Straight Row Poor	Good	0	0	65	79	81	84	87	89	91
Strawberries, 36" beds on 48" centers, beds covered with plastic (4)	-	72	72	90	94	94	95	96	96	97
Fallow - Bare Soil or Newly Graded Lands	-	0	0	71	83	85	88	90	92	94
Fallow - with crop residue cover on >5% of surface	Good	0	0	68	80	82	84	87	88	90
Orchard or Tree Farm, 50/50 woods-grass	Poor	0	0	39	60	64	69	75	79	83
Orchard or Tree Farm, 50/50 woods-grass	Fair	0	0	26	48	53	59	67	72	77
Orchard or Tree Farm, 50/50 woods-grass	Good	0	0	21	42	47	54	61	66	72
NOTE: WPD MODIFIED RATIONAL METHOD USES SOIL TYPES 1-7 AND EFFECTIVE IMPERVIOUS PERCENTAGE IN VCRat MODEL										
Note (1)	Poor is < 50% cover; Fair is from 50 to 75% cover; Good is >75% cover; also consider density of canopy and vegetative cover and degree of surface roughness									
Note (2)	% Impervious and CNs assumed same as residential 1/5 ac lots									
Note (3)	Assumed same as industrial parks									
Note (4)	Calculated assuming planted on 200'x208' parcel with 8' road along one boundary.									
Note (5)	TR-55 Notes: CNs developed using average % imperviousness with CN=98, pervious areas equivalent to open space in good condition. Greater than 30% impervious area considered directly connected.									
Reference:	TR-55 Manual Table 2-2. For other land use types, see TR-55 Manual.									

EXHIBIT 15. ZONE FOURTH DAY STORM TOTALS AND MASS CURVE IDS

Rainfall Zone	100-Year Storm	50-Year Storm	25-Year Storm	10-Year Storm
J	7.00	5.0	3.91	3.17
J' (J prime)	6.66	6.0	5.28	4.38
K	10.60	8.0	6.41	5.53
L	15.00	11.0	8.81	7.21
VCRat 2.5 Model Mass Curve IDs				
J	D96 (Note 1)	I50	I25	I10
J' (J prime)	A97	J50	J25	J10
K	B98	K50	K25	K10
L	C99	L50	L25	L10

Note 1: For VCRat 2.2, the J and J prime distributions use the same mass curve Ids of A97, J50, J25, and J10 in the VCRain.dat file and the modeler is required to insert the correct mass curve data required for the run in the appropriate location.

EXHIBIT 16. VCRATP.FOR AREAL REDUCTION DATA FROM FORTRAN PROGRAM

Area (ac)	Area (sq mi)	Factor	Area (ac)	Area (sq mi)	Factor
0	0	1.0000	19,200	30	0.7850
800	1.25	0.9450	22,400	35	0.7730
1,600	2.5	0.9250	25,600	40	0.7625
3,200	5	0.8950	28,800	45	0.7520
6,400	10	0.8600	32,000	50	0.7420
9,600	15	0.8350	48,000	75	0.7020
12,800	20	0.8150	64,000	100	0.6700
16,000	25	0.8000			

EXHIBIT 17. 100-YR HYDROGRAPH DISCRETIZATION

Time Period Minutes	Point Spacing Minutes	Number of Points
0 - 1000	100	11
1000 - 1100	50	2
1100 - 1130	10	3
1130 - 1300	1	170
1300 - 1400	10	10
1400 - 1460	20	3
1460 - 1500	40	1
		Total 200

EXHIBIT 18. VCRAIN.DAT FILE FOR VCRAT 2.2

C coefficients and mass curves for J'10, 50, 100, J25, K10, 25, 50, 100, L10, 25, 50, and 100. For VCRat 2.2, the J and J prime distributions use the same mass curve Ids of A97, J50, J25, and J10 in the VCRain.dat file and the modeler is required to insert the correct mass curve data required for the run in the appropriate location.

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EXHIBIT 19. VCRAIN.DAT FILE FOR VCRAT 2.6

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VCRAIN.DAT v2.5
VERSION 200604
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RUNOFF 1 0.000 0.001 0.400 0.600 0.700 0.765 0.800 0.828 0.850 0.880 0.900 0.925 0.940 0.950
SOILCRV 020
INTENSI 1 0.000 0.400 0.750 1.000 1.250 1.500 2.000 2.500 3.000 3.500 4.000 5.000 6.000 7.000
RUNOFF 1 0.000 0.000 0.450 0.591 0.663 0.709 0.762 0.800 0.829 0.850 0.868 0.890 0.903 0.910
SOILCRV 030
INTENSI 1 0.000 0.500 0.750 1.000 1.250 1.500 1.750 2.000 2.500 3.000 3.500 4.000 5.000 7.000
RUNOFF 1 0.000 0.000 0.332 0.502 0.590 0.645 0.677 0.704 0.746 0.777 0.800 0.816 0.840 0.856
SOILCRV 040
INTENSI 1 0.000 0.650 0.750 1.000 1.500 2.000 2.500 3.000 3.500 4.000 4.500 5.000 5.500 6.500
RUNOFF 1 0.000 0.000 0.140 0.360 0.570 0.647 0.692 0.727 0.752 0.770 0.785 0.795 0.802 0.809
SOILCRV 050
INTENSI 1 0.000 0.800 1.000 1.200 1.450 1.690 2.050 2.500 3.000 3.630 4.300 5.000 6.000 7.000
RUNOFF 1 0.000 0.000 0.270 0.400 0.500 0.550 0.600 0.630 0.672 0.700 0.720 0.732 0.744 0.750
SOILCRV 060
INTENSI 1 0.000 0.925 1.000 1.100 1.200 1.425 1.725 2.050 2.400 3.000 3.800 4.700 5.600 7.000
RUNOFF 1 0.000 0.001 0.170 0.250 0.310 0.370 0.420 0.460 0.490 0.530 0.570 0.600 0.620 0.643
SOILCRV 070
INTENSI 1 0.000 1.850 2.000 2.270 2.650 3.000 3.340 3.900 4.500 5.000 5.600 6.000 6.600 7.000
RUNOFF 1 0.000 0.001 0.090 0.200 0.300 0.360 0.400 0.450 0.490 0.515 0.540 0.560 0.580 0.593
MASSCRV J 10 I10 3.17
TIME 1 0 100 200 300 400 500 600 700 800 900 1000 1050 1100 1110
RAIN 1 0 0.017 0.069 0.156 0.278 0.433 0.632 0.848 1.105 1.398 1.726 1.903 2.089 2.127
TIME 15 1120 1130 1135 1140 1145 1147 1148 1149 1150 1151 1152 1153 1154 1155
RAIN 15 2.168 2.216 2.246 2.286 2.340 2.370 2.392 2.428 2.464 2.500 2.536 2.572 2.584 2.594
TIME 29 1160 1170 1180 1190 1210 1230 1260 1300 1340 1400 1440 1500
RAIN 29 2.632 2.684 2.719 2.753 2.817 2.876 2.954 3.039 3.103 3.159 3.170 3.170
MASSCRV J 25 I25 3.91
TIME 1 0 100 200 300 400 500 600 700 800 900 1000 1050 1100 1110
RAIN 1 0.000 0.022 0.085 0.192 0.340 0.532 0.765 1.042 1.360 1.722 2.125 2.343 2.572 2.620
TIME 15 1120 1130 1135 1140 1145 1147 1148 1149 1150 1151 1152 1153 1154 1155
RAIN 15 2.677 2.745 2.786 2.833 2.887 2.920 2.952 2.996 3.040 3.084 3.128 3.172 3.186 3.198
TIME 29 1160 1170 1180 1190 1210 1230 1260 1300 1340 1400 1440 1500
RAIN 29 3.241 3.300 3.347 3.389 3.469 3.542 3.640 3.747 3.827 3.897 3.910 3.910
MASSCRV J 50 I50 5.00

```

APPENDIX A

EXHIBITS

TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0	0.028	0.111	0.251	0.446	0.698	1.005	1.368	1.786	2.261	2.791	3.077	3.377	3.439
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	3.508	3.585	3.628	3.676	3.735	3.769	3.804	3.853	3.902	3.951	4.000	4.049	4.066	4.081
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	4.133	4.199	4.257	4.313	4.419	4.516	4.645	4.786	4.891	4.983	5.000	5.000		
MASSCRV J	100	D96	7.00												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.210	0.420	0.641	0.883	1.167	1.482	1.841	2.302	2.859	3.542	3.952	4.463	4.593
TIME	15	1120	1130	1131	1132	1133	1134	1135	1136	1137	1138	1139	1140	1141	1142
RAIN	15	4.751	4.887	4.908	4.929	4.949	4.970	4.991	5.012	5.033	5.053	5.074	5.095	5.122	5.148
TIME	29	1143	1144	1145	1146	1147	1148	1149	1150	1151	1152	1153	1154	1155	1156
RAIN	29	5.174	5.202	5.236	5.270	5.305	5.339	5.350	5.360	5.424	5.592	5.655	5.689	5.716	5.733
TIME	43	1157	1158	1159	1160	1161	1162	1163	1164	1165	1166	1167	1168	1169	1170
RAIN	43	5.751	5.772	5.785	5.802	5.817	5.833	5.846	5.861	5.874	5.886	5.896	5.908	5.910	5.932
TIME	57	1171	1172	1173	1174	1175	1176	1177	1178	1179	1180	1181	1182	1183	1184
RAIN	57	5.943	5.952	5.962	5.972	5.982	5.991	6.000	6.011	6.018	6.028	6.035	6.041	6.049	6.056
TIME	71	1185	1186	1187	1188	1189	1190	1191	1192	1193	1194	1195	1196	1197	1198
RAIN	71	6.062	6.070	6.076	6.083	6.092	6.099	6.108	6.116	6.123	6.132	6.140	6.148	6.156	6.164
TIME	85	1199	1200	1201	1202	1203	1204	1205	1206	1207	1208	1209	1210	1211	1212
RAIN	85	6.172	6.180	6.190	6.199	6.209	6.218	6.227	6.237	6.246	6.256	6.265	6.275	6.281	6.287
TIME	99	1213	1214	1215	1216	1217	1218	1219	1220	1221	1222	1223	1224	1225	1226
RAIN	99	6.294	6.300	6.306	6.313	6.319	6.325	6.332	6.338	6.344	6.350	6.357	6.363	6.369	6.376
TIME	113	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240
RAIN	113	6.382	6.388	6.395	6.401	6.407	6.414	6.420	6.426	6.432	6.439	6.445	6.451	6.458	6.464
TIME	127	1241	1242	1243	1244	1245	1246	1247	1248	1249	1250	1251	1252	1253	1254
RAIN	127	6.469	6.474	6.480	6.485	6.490	6.495	6.501	6.506	6.511	6.517	6.522	6.527	6.532	6.538
TIME	141	1255	1256	1257	1258	1259	1260	1261	1262	1263	1264	1265	1266	1267	1268
RAIN	141	6.543	6.548	6.553	6.559	6.564	6.569	6.574	6.580	6.585	6.590	6.595	6.601	6.606	6.611
TIME	155	1269	1270	1271	1272	1273	1274	1275	1276	1277	1278	1279	1280	1281	1282
RAIN	155	6.616	6.622	6.626	6.630	6.634	6.638	6.643	6.647	6.651	6.655	6.659	6.664	6.668	6.672
TIME	169	1283	1284	1285	1286	1287	1288	1289	1290	1291	1292	1293	1294	1295	1296
RAIN	169	6.676	6.680	6.685	6.689	6.693	6.697	6.702	6.706	6.710	6.714	6.718	6.723	6.727	6.731
TIME	183	1297	1298	1299	1300	1310	1320	1330	1340	1350	1360	1370	1380	1390	1400
RAIN	183	6.735	6.739	6.744	6.748	6.779	6.811	6.832	6.853	6.884	6.905	6.926	6.947	6.968	6.979
TIME	197	1420	1440	1460	1500										
RAIN	197	6.989	6.998	6.999	7.000										
MASSCRV J'	10	J10	4.38												
TIME	1	0	276	512	768	926	1072	1084	1088	1092	1104	1108	1112	1116	1120
RAIN	1	0.000	0.140	0.340	1.160	1.760	2.584	2.677	2.718	2.754	2.856	2.888	2.931	2.970	3.003
TIME	15	1128	1136	1140	1144	1148	1152	1153	1154	1156	1158	1160	1163	1165	1167

APPENDIX A

EXHIBITS

RAIN	15	3.072	3.160	3.208	3.277	3.360	3.504	3.540	3.561	3.590	3.612	3.621	3.658	3.676	3.692
TIME	29	1170	1180	1200	1215	1236	1254	1276	1305	1340	1375	1430	1500		
RAIN	29	3.714	3.784	3.896	3.940	4.020	4.080	4.140	4.200	4.260	4.340	4.360	4.380		
MASSCRV J'	25	J25	5.28												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.044	0.088	0.176	0.290	0.502	0.836	1.250	1.707	2.200	2.746	3.080	3.476	3.546
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	3.628	3.727	3.784	3.856	3.934	4.004	4.007	4.022	4.050	4.066	4.224	4.278	4.300	4.316
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	4.373	4.443	4.532	4.594	4.726	4.840	4.998	5.166	5.192	5.271	5.280	5.280		
MASSCRV J'	50	J50	6.00												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.050	0.100	0.200	0.330	0.570	0.950	1.420	1.940	2.500	3.120	3.500	3.950	4.030
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	4.123	4.235	4.300	4.382	4.470	4.550	4.553	4.570	4.602	4.620	4.800	4.861	4.886	4.904
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	4.969	5.049	5.150	5.220	5.370	5.500	5.680	5.870	5.900	5.990	6.000	6.000		
MASSCRV J'	100	A97	6.66												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.200	0.400	0.610	0.840	1.110	1.410	1.752	2.190	2.720	3.370	3.760	4.246	4.370
TIME	15	1120	1130	1131	1132	1133	1134	1135	1136	1137	1138	1139	1140	1141	1142
RAIN	15	4.520	4.650	4.670	4.690	4.709	4.729	4.749	4.769	4.789	4.808	4.828	4.848	4.873	4.898
TIME	29	1143	1144	1145	1146	1147	1148	1149	1150	1151	1152	1153	1154	1155	1156
RAIN	29	4.923	4.949	4.982	5.014	5.047	5.080	5.090	5.100	5.161	5.320	5.380	5.413	5.438	5.455
TIME	43	1157	1158	1159	1160	1161	1162	1163	1164	1165	1166	1167	1168	1169	1170
RAIN	43	5.472	5.492	5.504	5.520	5.534	5.550	5.562	5.576	5.589	5.600	5.610	5.621	5.623	5.644
TIME	57	1171	1172	1173	1174	1175	1176	1177	1178	1179	1180	1181	1182	1183	1184
RAIN	57	5.654	5.663	5.672	5.682	5.691	5.700	5.709	5.719	5.726	5.735	5.742	5.748	5.755	5.762
TIME	71	1185	1186	1187	1188	1189	1190	1191	1192	1193	1194	1195	1196	1197	1198
RAIN	71	5.768	5.775	5.781	5.788	5.796	5.803	5.811	5.819	5.826	5.834	5.842	5.849	5.857	5.865
TIME	85	1199	1200	1201	1202	1203	1204	1205	1206	1207	1208	1209	1210	1211	1212
RAIN	85	5.872	5.880	5.889	5.898	5.907	5.916	5.925	5.934	5.943	5.952	5.961	5.970	5.976	5.982
TIME	99	1213	1214	1215	1216	1217	1218	1219	1220	1221	1222	1223	1224	1225	1226
RAIN	99	5.988	5.994	6.000	6.006	6.012	6.018	6.024	6.030	6.036	6.042	6.048	6.054	6.060	6.066
TIME	113	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240
RAIN	113	6.072	6.078	6.084	6.090	6.096	6.102	6.108	6.114	6.120	6.126	6.132	6.138	6.144	6.150
TIME	127	1241	1242	1243	1244	1245	1246	1247	1248	1249	1250	1251	1252	1253	1254
RAIN	127	6.155	6.160	6.165	6.170	6.175	6.180	6.185	6.190	6.195	6.200	6.205	6.210	6.215	6.220
TIME	141	1255	1256	1257	1258	1259	1260	1261	1262	1263	1264	1265	1266	1267	1268
RAIN	141	6.225	6.230	6.235	6.240	6.245	6.250	6.255	6.260	6.265	6.270	6.275	6.280	6.285	6.290
TIME	155	1269	1270	1271	1272	1273	1274	1275	1276	1277	1278	1279	1280	1281	1282

APPENDIX A

EXHIBITS

RAIN	155	6.295	6.300	6.304	6.308	6.312	6.316	6.320	6.324	6.328	6.332	6.336	6.340	6.344	6.348
TIME	169	1283	1284	1285	1286	1287	1288	1289	1290	1291	1292	1293	1294	1295	1296
RAIN	169	6.352	6.356	6.360	6.364	6.368	6.372	6.376	6.380	6.384	6.388	6.392	6.396	6.400	6.404
TIME	183	1297	1298	1299	1300	1310	1320	1330	1340	1350	1360	1370	1380	1390	1400
RAIN	183	6.408	6.412	6.416	6.420	6.450	6.480	6.500	6.520	6.550	6.570	6.590	6.610	6.630	6.640
TIME	197	1420	1440	1460	1500										
RAIN	197	6.650	6.658	6.659	6.660										
MASSCRV K	10	K10	5.53												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.100	0.210	0.350	0.520	0.735	0.980	1.290	1.670	2.140	2.792	3.154	3.547	3.646
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	3.754	3.864	3.934	4.014	4.107	4.149	4.179	4.214	4.254	4.324	4.424	4.489	4.509	4.524
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	4.594	4.689	4.762	4.830	4.961	5.060	5.175	5.305	5.400	5.490	5.530	5.530		
MASSCRV K	25	K25	6.408												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.100	0.235	0.407	0.623	0.893	1.215	1.605	2.061	2.523	3.198	3.619	4.138	4.251
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	4.367	4.508	4.588	4.678	4.776	4.821	4.844	4.915	4.986	5.057	5.128	5.200	5.224	5.241
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	5.315	5.426	5.518	5.596	5.726	5.835	5.973	6.135	6.253	6.364	6.408	6.408		
MASSCRV K	50	K50	8.000												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.100	0.280	0.510	0.810	1.180	1.640	2.175	2.770	3.470	4.340	4.784	5.311	5.426
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	5.560	5.701	5.776	5.886	6.020	6.080	6.111	6.146	6.195	6.290	6.400	6.490	6.520	6.540
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	6.620	6.730	6.830	6.930	7.100	7.240	7.420	7.640	7.800	7.950	8.000	8.000		
MASSCRV K	100	B98	10.600												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.450	0.940	1.500	2.060	2.650	3.270	3.920	4.600	5.350	6.220	6.730	7.267	7.407
TIME	15	1120	1130	1131	1132	1133	1134	1135	1136	1137	1138	1139	1140	1141	1142
RAIN	15	7.536	7.706	7.723	7.740	7.760	7.781	7.801	7.816	7.838	7.859	7.880	7.902	7.928	7.953
TIME	29	1143	1144	1145	1146	1147	1148	1149	1150	1151	1152	1153	1154	1155	1156
RAIN	29	7.978	8.004	8.038	8.072	8.106	8.140	8.225	8.310	8.327	8.480	8.565	8.599	8.624	8.646
TIME	43	1157	1158	1159	1160	1161	1162	1163	1164	1165	1166	1167	1168	1169	1170
RAIN	43	8.665	8.682	8.698	8.716	8.729	8.742	8.755	8.774	8.783	8.795	8.805	8.817	8.834	8.840
TIME	57	1171	1172	1173	1174	1175	1176	1177	1178	1179	1180	1181	1182	1183	1184
RAIN	57	8.852	8.863	8.873	8.884	8.894	8.904	8.913	8.922	8.932	8.942	8.952	8.962	8.971	8.981
TIME	71	1185	1186	1187	1188	1189	1190	1191	1192	1193	1194	1195	1196	1197	1198
RAIN	71	8.991	9.000	9.010	9.020	9.030	9.039	9.049	9.058	9.068	9.078	9.088	9.097	9.107	9.117

APPENDIX A

EXHIBITS

TIME	85	1199	1200	1201	1202	1203	1204	1205	1206	1207	1208	1209	1210	1211	1212
RAIN	85	9.126	9.136	9.144	9.153	9.161	9.170	9.178	9.186	9.195	9.203	9.212	9.220	9.228	9.237
TIME	99	1213	1214	1215	1216	1217	1218	1219	1220	1221	1222	1223	1224	1225	1226
RAIN	99	9.246	9.254	9.262	9.271	9.280	9.288	9.296	9.305	9.314	9.322	9.330	9.339	9.348	9.356
TIME	113	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240
RAIN	113	9.364	9.373	9.382	9.390	9.398	9.407	9.415	9.423	9.432	9.440	9.448	9.457	9.465	9.473
TIME	127	1241	1242	1243	1244	1245	1246	1247	1248	1249	1250	1251	1252	1253	1254
RAIN	127	9.482	9.490	9.498	9.507	9.515	9.523	9.532	9.540	9.548	9.557	9.565	9.573	9.582	9.590
TIME	141	1255	1256	1257	1258	1259	1260	1261	1262	1263	1264	1265	1266	1267	1268
RAIN	141	9.598	9.607	9.615	9.623	9.632	9.640	9.648	9.655	9.662	9.670	9.678	9.685	9.692	9.700
TIME	155	1269	1270	1271	1272	1273	1274	1275	1276	1277	1278	1279	1280	1281	1282
RAIN	155	9.708	9.715	9.722	9.730	9.738	9.745	9.752	9.760	9.768	9.775	9.782	9.790	9.798	9.805
TIME	169	1283	1284	1285	1286	1287	1288	1289	1290	1291	1292	1293	1294	1295	1296
RAIN	169	9.812	9.820	9.828	9.835	9.842	9.850	9.858	9.865	9.872	9.880	9.887	9.895	9.902	9.910
TIME	183	1297	1298	1299	1300	1310	1320	1330	1340	1350	1360	1370	1380	1390	1400
RAIN	183	9.918	9.925	9.932	9.940	10.002	10.065	10.128	10.190	10.237	10.283	10.330	10.377	10.423	10.470
TIME	197	1420	1440	1460	1500										
RAIN	197	10.535	10.600	10.600	10.600										
MASSCRV L	10	L10	7.21												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.100	0.220	0.390	0.600	0.875	1.240	1.700	2.240	2.890	3.700	4.162	4.698	4.821
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	4.965	5.115	5.198	5.281	5.368	5.418	5.449	5.490	5.558	5.638	5.768	5.808	5.828	5.843
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	5.905	6.023	6.123	6.220	6.390	6.520	6.710	6.900	7.040	7.160	7.210	7.210		
MASSCRV L	25	L25	8.808												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.092	0.233	0.453	0.748	1.117	1.603	2.198	2.906	3.729	4.690	5.278	5.893	6.023
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	6.167	6.345	6.435	6.526	6.639	6.692	6.719	6.800	6.881	6.963	7.046	7.131	7.156	7.175
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	7.265	7.406	7.526	7.635	7.802	7.966	8.198	8.439	8.617	8.758	8.808	8.808		
MASSCRV L	50	L50	11.000												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.080	0.250	0.540	0.950	1.450	2.100	2.880	3.820	4.880	6.090	6.765	7.507	7.657
TIME	15	1120	1130	1135	1140	1145	1147	1148	1149	1150	1151	1152	1153	1154	1155
RAIN	15	7.820	8.000	8.103	8.220	8.350	8.410	8.445	8.500	8.565	8.670	8.835	8.910	8.950	8.975
TIME	29	1160	1170	1180	1190	1210	1230	1260	1300	1340	1400	1440	1500		
RAIN	29	9.066	9.228	9.364	9.498	9.740	9.950	10.240	10.550	10.780	10.950	11.000	11.000		
MASSCRV L	100	C99	15.000												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110

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RAIN	1	0.000	0.720	1.500	2.300	3.120	3.960	4.820	5.750	6.710	7.800	9.070	9.780	10.550	10.720
TIME	15	1120	1130	1131	1132	1133	1134	1135	1136	1137	1138	1139	1140	1141	1142
RAIN	15	10.891	11.093	11.113	11.133	11.158	11.182	11.207	11.232	11.256	11.279	11.302	11.326	11.359	11.392
TIME	29	1143	1144	1145	1146	1147	1148	1149	1150	1151	1152	1153	1154	1155	1156
RAIN	29	11.424	11.457	11.491	11.525	11.559	11.593	11.695	11.796	11.898	12.000	12.102	12.136	12.168	12.192
TIME	43	1157	1158	1159	1160	1161	1162	1163	1164	1165	1166	1167	1168	1169	1170
RAIN	43	12.217	12.241	12.260	12.277	12.296	12.313	12.330	12.348	12.364	12.380	12.396	12.413	12.428	12.444
TIME	57	1171	1172	1173	1174	1175	1176	1177	1178	1179	1180	1181	1182	1183	1184
RAIN	57	12.460	12.477	12.492	12.506	12.521	12.536	12.550	12.563	12.576	12.588	12.602	12.616	12.631	12.645
TIME	71	1185	1186	1187	1188	1189	1190	1191	1192	1193	1194	1195	1196	1197	1198
RAIN	71	12.659	12.674	12.688	12.702	12.721	12.740	12.753	12.766	12.780	12.793	12.806	12.819	12.832	12.846
TIME	85	1199	1200	1201	1202	1203	1204	1205	1206	1207	1208	1209	1210	1211	1212
RAIN	85	12.859	12.872	12.885	12.898	12.910	12.923	12.936	12.949	12.962	12.974	12.987	13.000	13.012	13.025
TIME	99	1213	1214	1215	1216	1217	1218	1219	1220	1221	1222	1223	1224	1225	1226
RAIN	99	13.038	13.050	13.062	13.075	13.088	13.100	13.112	13.125	13.138	13.150	13.162	13.175	13.188	13.200
TIME	113	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240
RAIN	113	13.212	13.225	13.238	13.250	13.262	13.273	13.285	13.297	13.308	13.320	13.332	13.343	13.355	13.367
TIME	127	1241	1242	1243	1244	1245	1246	1247	1248	1249	1250	1251	1252	1253	1254
RAIN	127	13.378	13.390	13.402	13.413	13.425	13.437	13.448	13.460	13.472	13.483	13.495	13.507	13.518	13.530
TIME	141	1255	1256	1257	1258	1259	1260	1261	1262	1263	1264	1265	1266	1267	1268
RAIN	141	13.542	13.553	13.565	13.577	13.588	13.600	13.610	13.621	13.632	13.642	13.652	13.663	13.674	13.684
TIME	155	1269	1270	1271	1272	1273	1274	1275	1276	1277	1278	1279	1280	1281	1282
RAIN	155	13.694	13.705	13.716	13.726	13.736	13.747	13.758	13.768	13.778	13.789	13.800	13.810	13.820	13.831
TIME	169	1283	1284	1285	1286	1287	1288	1289	1290	1291	1292	1293	1294	1295	1296
RAIN	169	13.842	13.852	13.862	13.873	13.884	13.894	13.904	13.915	13.926	13.936	13.946	13.957	13.968	13.978
TIME	183	1297	1298	1299	1300	1310	1320	1330	1340	1350	1360	1370	1380	1390	1400
RAIN	183	13.988	13.999	14.010	14.020	14.110	14.200	14.290	14.380	14.450	14.520	14.590	14.660	14.730	14.800
TIME	197	1420	1440	1460	1500										
RAIN	197	14.900	15.000	15.000	15.000										
MASSCRV T	01	T01	8.00												
TIME	1	0	100	200	300	400	500	600	700	800	900	1000	1050	1100	1110
RAIN	1	0.000	0.100	0.280	0.510	0.810	1.180	1.640	2.175	2.770	3.470	4.340	4.784	5.311	5.426
TIME	15	1120	1130	1131	1132	1133	1134	1135	1136	1137	1138	1139	1140	1141	1142
RAIN	15	5.560	5.701	5.716	5.731	5.746	5.761	5.776	5.798	5.820	5.842	5.864	5.886	5.913	5.940
TIME	29	1143	1144	1145	1146	1147	1148	1149	1150	1151	1152	1153	1154	1155	1156
RAIN	29	5.966	5.993	6.020	6.050	6.080	6.111	6.146	6.195	6.290	6.400	6.490	6.520	6.540	6.556
TIME	43	1157	1158	1159	1160	1161	1162	1163	1164	1165	1166	1167	1168	1169	1170
RAIN	43	6.572	6.588	6.604	6.620	6.631	6.642	6.653	6.664	6.675	6.686	6.697	6.708	6.719	6.730
TIME	57	1171	1172	1173	1174	1175	1176	1177	1178	1179	1180	1181	1182	1183	1184
RAIN	57	6.740	6.750	6.760	6.770	6.780	6.790	6.800	6.810	6.820	6.830	6.840	6.850	6.860	6.870
TIME	71	1185	1186	1187	1188	1189	1190	1191	1192	1193	1194	1195	1196	1197	1198

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RAIN	71	6.880	6.890	6.900	6.910	6.920	6.930	6.938	6.947	6.956	6.964	6.972	6.981	6.990	6.998
TIME	85	1199	1200	1201	1202	1203	1204	1205	1206	1207	1208	1209	1210	1211	1212
RAIN	85	7.006	7.015	7.024	7.032	7.040	7.049	7.058	7.066	7.074	7.083	7.092	7.100	7.107	7.114
TIME	99	1213	1214	1215	1216	1217	1218	1219	1220	1221	1222	1223	1224	1225	1226
RAIN	99	7.121	7.128	7.135	7.142	7.149	7.156	7.163	7.170	7.177	7.184	7.191	7.198	7.205	7.212
TIME	113	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240
RAIN	113	7.219	7.226	7.233	7.240	7.246	7.252	7.258	7.264	7.270	7.276	7.282	7.288	7.294	7.300
TIME	127	1241	1242	1243	1244	1245	1246	1247	1248	1249	1250	1251	1252	1253	1254
RAIN	127	7.306	7.312	7.318	7.324	7.330	7.336	7.342	7.348	7.354	7.360	7.366	7.372	7.378	7.384
TIME	141	1255	1256	1257	1258	1259	1260	1261	1262	1263	1264	1265	1266	1267	1268
RAIN	141	7.390	7.396	7.402	7.408	7.414	7.420	7.426	7.431	7.436	7.442	7.448	7.453	7.458	7.464
TIME	155	1269	1270	1271	1272	1273	1274	1275	1276	1277	1278	1279	1280	1281	1282
RAIN	155	7.470	7.475	7.480	7.486	7.492	7.497	7.502	7.508	7.514	7.519	7.524	7.530	7.536	7.541
TIME	169	1283	1284	1285	1286	1287	1288	1289	1290	1291	1292	1293	1294	1295	1296
RAIN	169	7.546	7.552	7.557	7.563	7.568	7.574	7.580	7.585	7.590	7.596	7.602	7.607	7.612	7.618
TIME	183	1297	1298	1299	1300	1310	1320	1330	1340	1350	1360	1370	1380	1390	1400
RAIN	183	7.624	7.629	7.634	7.640	7.680	7.720	7.760	7.800	7.825	7.850	7.875	7.900	7.925	7.950
TIME	197	1420	1440	1460	1500										

EXHIBIT 20A. TC CALCULATION FORM FOR HAND CALCULATIONS- PART 1

VENTURA COUNTY WATERSHED PROTECTION DISTRICT
TIME OF CONCENTRATION CALCULATION FORM

PROJECT _____ PROJECT NO. _____ RAINFALL ZONE _____
 SUBAREA _____ SOIL No. _____ COMPUTED _____ DATE _____
 TOTAL AREA _____ ac _____ in/hr _____ RAINFALL FREQUENCY _____
 TYPE DEVELOPMENT _____ TYPE DEVELOPMENT _____ SHEET _____ OF _____

ASSUMPTION NO.	Tc (min)	INTENSITY (c/f/ft)	C-coeff	AREA (acres)	Q (cfs)
1					
2					

FINAL Tc = _____ min.

IDENT NO.	PLAN AREA	% OF AREA	Q-Trial 1 (cfs)	Q-Trial 2 (cfs)
a			0.0	0.0
b			0.0	0.0
c			0.0	0.0
d			0.0	0.0
e			0.0	0.0
TOTAL	0.0	0%	0.0	0.0

TRIAL NO.	Length (ft)	Elev. Top	Elev. Bot	SLOPE	EFFEC SLOPE	VELOCITY (ft/s)	LENGTH (ft)	TIME (min)
1								
2								

CONVEYANCE	TRIALS
LOWLAND	1 0.00
NATURAL CHAN.	2 0.00
STREET	3 0.00
PIPE	
RECT CHANNEL	
TRAP CHANNEL	
TOTAL	0.0

MOUNTAIN OVERLAND FLOW VELOCITY CHECK	
VEL CHECK	VELOCITY > 6.0 f/s
LENGTH (ft)	ELEVATION TOP BOT Δ

MOUNTAIN OR VALLEY CHANNEL ROUTING

REACH	LENGTH (ft)	ELEVATION TOP	ELEVATION BOT	Δ	MAP SLOPE	EFFEC SLOPE	TOP Q (cfs)	BOT Q (cfs)	TOP VELOCITY (ft/s)	BOT VELOCITY (ft/s)	AVE VELOCITY (ft/s)	WAVE TIME (min)
Trial 1												
Trial 2												

STREET TRIAL 1 WIDTH _____ CURB HEIGHT _____												
REACH	LENGTH (ft)	ELEVATION TOP	ELEVATION BOT	Δ	MAP SLOPE	EFFEC SLOPE	TOP Q (cfs)	BOT Q (cfs)	TOP VELOCITY (ft/s)	BOT VELOCITY (ft/s)	AVE VELOCITY (ft/s)	WAVE TIME (min)
1												
2												
3												
Sum												

STREET TRIAL 2 WIDTH _____ CURB HEIGHT _____												
REACH	LENGTH (ft)	ELEVATION TOP	ELEVATION BOT	Δ	MAP SLOPE	EFFEC SLOPE	TOP Q (cfs)	BOT Q (cfs)	TOP VELOCITY (ft/s)	BOT VELOCITY (ft/s)	AVE VELOCITY (ft/s)	WAVE TIME (min)
1												
2												
3												

$d^{8/3} = \frac{Q_n}{S^{1/2} \times 0.463} = \frac{ft}{ft} = d = \frac{ft}{ft}$ USE $d = \frac{ft}{ft}$
 $Q_{full} = K_d^{8/3} \times S^{1/2} = \frac{0.463 \times d^{8/3} \times S^{1/2}}{0.012} = \frac{ft^3}{s} = cfs$
 $AREA = \frac{Q_{full}^2}{4} = \frac{ft^6}{ft^2} = ft^4$ $V_n = \frac{Q_{full}}{AREA} = \frac{ft^3/s}{ft^4} = \frac{ft}{s} = ft/sec$

REACH	LENGTH (ft)	ELEVATION TOP	ELEVATION BOT	SLOPE	Q (cfs)	Gauge DIA (ft)	Norm (ft)	n	FULL PIPE K	% Q	V	Vw	TIME (min)
1													
2													

RECTANGULAR-TRAPEZOIDAL CHANNEL													
REACH	LENGTH (ft)	Type	Width (ft)	ELEVATION TOP	ELEVATION BOT	Δ	SLOPE	Steslope	Ht (ft)	IV	N Value	Q (cfs)	TIME (min)
1			2					0			0.015		
2													
3													
Normal Depth (ft) _____ VELOCITY _____ TIME _____													

EXHIBIT 20B. TC CALCULATION FORM FOR HAND CALCULATIONS- PART 2

VENTURA COUNTY WATERSHED PROTECTION DISTRICT
TIME OF CONCENTRATION CALCULATION FORM

PROJECT _____	PROJECT NO. _____	RAINFALL ZONE _____
SUBAREA _____	SOIL No. _____	COMPUTED _____
	Infil. Rate _____ in/hr	RAINFALL FREQUENCY _____
TOTAL AREA _____ ac	% IMPERVIOUS _____	TYPE DEVELOPMENT _____
		SHEET _____ OF _____

Subarea Figure showing:

1. Subarea boundary
2. Scale
3. North Arrow
4. Topography or elevations used for slope calculations
5. Development
6. Flowpaths and associated subarea portions with labels

EXHIBIT 21. DESIGN STORM RATIOS

Category (1)	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr
Precipitation	0.43	0.61	0.73	0.88	1.00	1.11	Not Analyzed	Not Analyzed
Undeveloped-HMS & HSPF	0.043	0.144	0.262	0.484	0.711	1.000	1.345	1.952
Developed HMS & HSPF	0.166	0.330	0.464	0.660	0.882	1.000	1.191	1.502
Undeveloped-VCRat (2)	0.043	0.144	0.362	0.484	0.711	1.000	1.345	1.952
Developed VCRat (2)	0.166	0.330	0.567	0.660	0.882	1.000	1.191	1.502
Casitas Dam Outflow	0.005	0.030	0.048	0.110	0.143	1.000	1.191	1.448
Coyote Ck below Dam	0.005	0.100	0.200	0.400	0.580	1.000	1.191	1.416
Piru Ck Below Dam	0.031	0.042	0.061	0.136	0.805	1.000	1.183	1.463

Note (1): Ratios cannot be used for watersheds with detention basins or water storage dams affecting more than 10% of the area except for those developed specifically for dam outflow (Casitas and Piru).

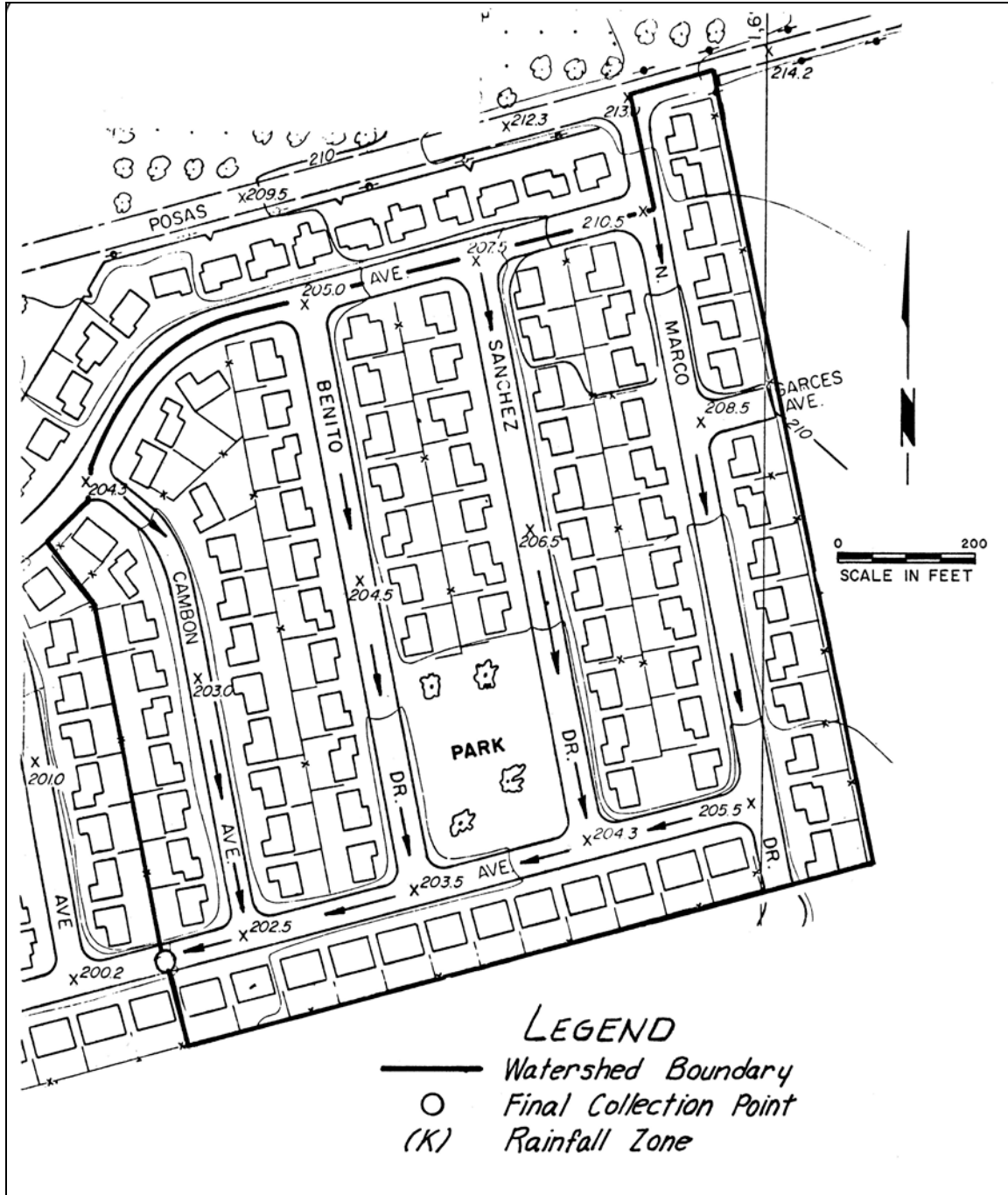
Note (2): VCRat ratios provided for reference only as current practice is to run the model using the correct Tc's and rainfall for all storms required for design studies. In rare cases it may be necessary to use the multipliers in VCRat studies such as estimating 10-yr peaks from Soil Type 7 in the J' zone.

APPENDIX B – VCRAT METHODS AND EXAMPLES

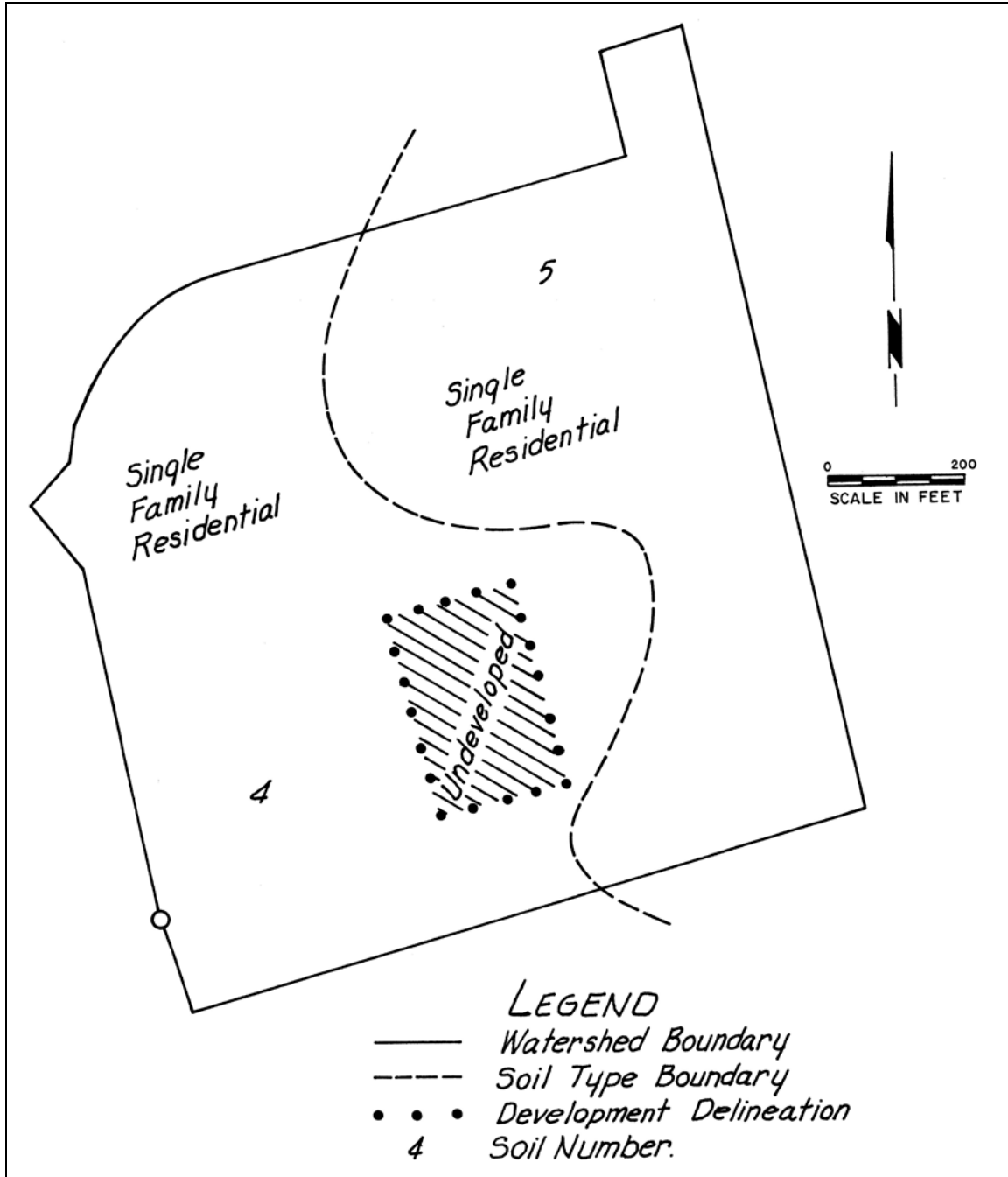
B-1. TC EXAMPLE 1- LAS POSAS-FORTUNA TRACT

1. Subarea Boundaries- Carefully outline the drainage boundary on a topographic map. With reference to other larger scale maps, known improvement plans, and planned improvements, decisions governed by topography alone must be adjusted to reflect an ultimate drainage boundary. The study watershed used in this example is shown in Exhibit B-1.1
2. Field Verification- Verify boundary correctness by a field trip to the area and incorporate any changes on the map.
3. Rainfall and Soil Zones- From the hydrologic maps and GIS files in Appendix E, determine the dominant rainfall intensity zones for the study watershed. Indicate this zone on the watershed map. From the same hydrologic map, evaluate the soil types of the project watershed using an overlay or the provided GIS shapefile.
4. Future Land Use- Determine anticipated ultimate development from best sources available (current zoning, Planning Department master plans, development trends in surrounding area). Undeveloped, single family residential, and commercial development types describe most areas with occasional classifications of heavy industrial or large estate lots.
5. Percent Imperviousness- Calculate percent impervious values for the development. Assign effective imperviousness percentages to each development type using the values in Exhibit 14a&b.

B-1.1 Example 1- Las Posas-Fortuna Tract Watershed Map



B-1.2 Soils and Development Map



6. Watershed Area- Planimeter or use GIS to calculate the area associated with each soil and development type combination and indicate as shown in Appendix B-1.3.
7. Composite Runoff Coefficient- Calculate composite runoff coefficient curve as shown in Appendix B-1.3, by first listing the appropriate runoff coefficients for selected rainfall intensities from Exhibits 6a-g. The composite runoff coefficient is weighted according to the drainage area of each soil and development classification. Plot composite runoff coefficient curve as shown in Appendix B-1.4.
8. Compute the time of concentration for the watershed using the form for hand calculations shown in Exhibit 20 or use the Tc Calculator Program described above. This trial-and-error hand computation includes the following general procedures:
 - a. Using engineering judgment, select a time of concentration for the project watershed. From the appropriate rainfall intensity-duration table, Exhibits 2a-5d, select the peak period intensity for the duration equal to the assumed time of concentration. Using the selected rainfall intensity, find an appropriate runoff coefficient “C” from the composite runoff coefficient curve from Exhibits 6a-g. Solve $Q=CIA$ for an initial peak flow estimate.
 - b. Divide the project watershed into several contributing parts by streets, topographic barriers, or fence lines, and proportion the initial flow rate to the various parts by area.
 - c. Start at most remote point in the project watershed, select an appropriate parcel, and assign an overland flow velocity (Exhibit 8). Using street velocity-discharge-slope graphs (Exhibits 11a-d), pipe or channel hydraulic tables (not provided- use Manning’s Equation), compute flow velocities from collection point to collection point gathering discharge on the selected path to the point of concentration. Average velocity is increased to wave velocity in streets and natural channels by a factor of 1.5. Pipe and trapezoid channel wave velocities are provided in Exhibits 12a-c.
 - d. Compute and sum travel times associated with lot, street, and pipe or channel flow. Compare to assumed time of concentration. If original assumption was incorrect, select a revised time of concentration, compute a new peak flow rate, distribute as before, and determine revised travel time.
 - e. When the computed travel time equals the estimated time of concentration within 0.5 minutes, the associated discharge represents the desired peak flow rate.

A detailed discussion of the Tc calculation procedures is provided in the Tc Calculator Manual provided with the program.

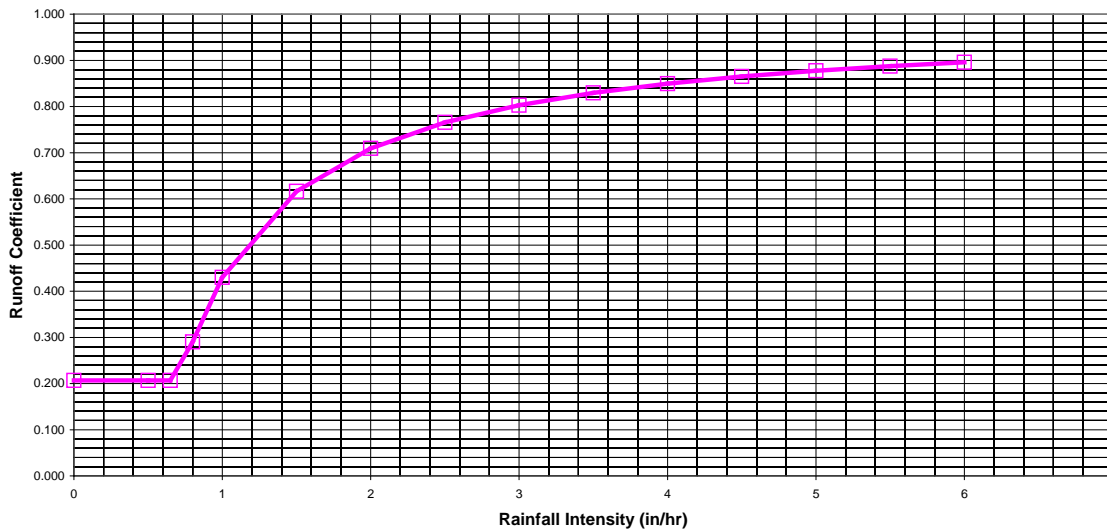
B-1.3 Composite Runoff Coefficient Curve Data

	Soil Number						
	4			5			
	Undeveloped	Single Family 1/4 ac. lots	Commercial	Undeveloped	Single Family 1/4 ac. lots	Commercial	
	Effective Imperviousness (%)						
	0	23	-	-	23	-	
	Land Use Area (ac)						Total Area
	1.3	12.2	-	-	10.5	-	24.0
	Percent of Total Area (%)						Total Percent
	5.4	50.8	-	-	43.8	-	100.0
	Pervious Area Infiltration Rate (in/hr)						
	0.65			0.80			
Intensity (in/hr)	Runoff Coefficients						Composite Runoff Coeff.
0	0.000	0.219	-		0.219	-	0.207
0.5	0.000	0.219	-		0.219	-	0.207
0.65	0.000	0.219	-		0.219	-	0.207
0.8	0.188	0.363	-		0.219	-	0.290
1	0.350	0.488	-		0.373	-	0.430
1.5	0.567	0.655	-		0.578	-	0.616
2	0.675	0.738	-		0.681	-	0.710
2.5	0.740	0.788	-		0.742	-	0.765
3	0.783	0.822	-		0.783	-	0.803
3.5	0.814	0.846	-		0.813	-	0.829
4	0.838	0.863	-		0.835	-	0.849
4.5	0.856	0.877	-		0.852	-	0.865
5	0.870	0.888	-		0.865	-	0.877
5.5	0.882	0.898	-		0.877	-	0.887

6	0.892	0.905	-	0.886	-	0.896
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Sample Calculation for Intensity = 2.0 in/hr:

$$\text{Composite } C = (0.675 * 5.4 / 100) + (0.738 * 50.8 / 100) + (0.681 * 43.8 / 100) = 0.710$$



B-1.4 Composite Runoff Coefficient Curve

B-2 EXAMPLE 1 (CONT.) - CALCULATING A RUNOFF HYDROGRAPH

If only an instantaneous peak flow rate is desired at the reference point, the peak flow can be computed with $A=CIA$ using the appropriate intensity and runoff coefficient. However, if flood routing is necessary, or a flood-detention basin is to be sized for a small development, a runoff hydrograph will need to be computed. The procedure continues as follows.

1. Using an appropriate tabulation of rainfall intensities selected from 2a-5d, tabulate intensity for selected storm times as shown in Exhibit B-2.1.
2. Using the composite runoff coefficient curve for the watershed, tabulate as shown in Exhibit B-2.1 the runoff coefficient for each intensity value.

3. Compute the flow rate for each storm time using the rational equation and plot the resulting runoff hydrograph as shown in Exhibit B-2.3.
4. Calculate the area under the hydrograph and convert to runoff volume in inches as shown in Exhibit B-2.2, and compare with the 24-hour rainfall amount at the centroid of the watershed as shown in the isohyetal maps in Appendix E. Expected runoff volume can be estimated from nearby raingage and streamflow records if available. Expected volume can also be estimated from synthetic rainfall-runoff relationships such as the Soil Conservation Service method for computing runoff using 24-hour rainfall and an appropriate runoff curve number as described in Section 4.
5. If the percentage of runoff is less than the expected volume from that watershed, the hydrograph should be adjusted using a technique similar to that shown in Exhibit B-2.2. The adjustment procedure is as follows:
 - a. Compute a runoff coefficient curve adjustment factor by determining the ratio of desired yield to actual yield.
 - b. Adjust the runoff coefficient curve using the adjustment factor, and plot a new curve as shown in Exhibit B-2.1.
 - c. Using the peak flow determined above, compute $C \cdot I$ in cfs/acre.
 - d. From the adjusted runoff coefficient curve, find a combination of C and I such that their product equals the value determined in c., above.
 - e. From the appropriate intensity-duration curve, select a revised duration corresponding to the average rainfall intensity determined in d. and use as a revised time of concentration for the watershed.
 - f. As shown in Exhibit B-2.1, tabulate adjusted rainfall intensities, adjusted runoff coefficients, and compute an adjusted runoff hydrograph.
6. Plot the adjusted hydrograph and check its runoff volume as shown in Exhibit B-2.3

B-2.1 Hydrograph and Hydrograph Adjustment Calculations

K Zone 10-Year Recurrence Storm								
Storm Time (min)	Rainfall Intensity (in/hr) Tc=15 min	Runoff Coefficient C	Q=CIA (cfs) A=24 ac	Incremental Hydrograph Volume (ac-ft)	Rainfall Intensity (in/hr) Tc=22 min	Adjusted Runoff Coeff C(P)	Qadj = Cadj I A (cfs) A=24 ac	Incremental Hydrograph Volume (ac-ft)
0	0.00	0.207	0.00		0	0.256	0.00	
200	0.07	0.207	0.35	0.05	0.07	0.256	0.43	0.06
400	0.10	0.207	0.50	0.12	0.10	0.256	0.62	0.14
600	0.15	0.207	0.74	0.17	0.15	0.256	0.92	0.21
900	0.28	0.207	1.39	0.44	0.28	0.256	1.72	0.55
1000	0.39	0.207	1.93	0.23	0.39	0.256	2.40	0.28
1050	0.43	0.207	2.13	0.14	0.43	0.256	2.65	0.17
1100	0.47	0.207	2.33	0.15	0.47	0.256	2.89	0.19
1110	0.55	0.207	2.73	0.03	0.53	0.256	3.26	0.04
1120	0.63	0.207	3.12	0.04	0.61	0.256	3.75	0.05
1130	0.66	0.212	3.36	0.04	0.65	0.263	4.11	0.05
1140	0.82	0.304	5.99	0.06	0.77	0.377	6.98	0.08
1145	0.97	0.409	9.52	0.05	0.87	0.508	10.60	0.06
1150	1.28	0.534	16.42	0.09	1.12	0.663	17.82	0.10
1152	1.83	0.678	29.77	0.06	1.53	0.841	30.89	0.07
1154	2.04	0.714	34.96	0.09	1.68	0.886	35.72	0.09
1156	2.02	0.712	34.51	0.10	1.68	0.883	35.61	0.10
1158	1.98	0.706	33.54	0.09	1.68	0.876	35.31	0.10
1160	1.95	0.700	32.77	0.09	1.67	0.869	34.83	0.10
1162	1.86	0.683	30.51	0.09	1.63	0.848	33.18	0.09
1166	1.31	0.546	17.15	0.13	1.53	0.677	24.86	0.16
1170	0.66	0.212	3.36	0.06	1.39	0.263	8.79	0.09
1175	0.53	0.207	2.63	0.02	0.64	0.256	3.94	0.04
1180	0.48	0.207	2.38	0.02	0.54	0.256	3.32	0.03
1200	0.40	0.207	1.98	0.06	0.40	0.256	2.46	0.08
1225	0.30	0.207	1.49	0.06	0.33	0.256	2.03	0.08
1300	0.20	0.207	0.99	0.13	0.20	0.256	1.23	0.17
1400	0.09	0.207	0.45	0.10	0.09	0.256	0.55	0.12
1440	0.06	0.207	0.30	0.02	0.06	0.256	0.37	0.03
			Total				Total	
			Vol (ac-ft)	2.74			Vol (ac-ft)	3.33

B-2.2 Hydrograph Volume Adjustment Steps

Hydrograph Volume Adjustment

1. Watershed Yield

Areally Weighted CN=73.5

From Contour Map, 10Yr Rainfall= 4.3 inches

From SCS Direct Runoff Figure, Runoff = 1.7 inches

Watershed Yield = $1.7/4.3=0.395$ or 0.40

2. Hydrograph Volume, $T_c=15$ min= 2.74 ac-ft

Yield (24 ac) = 1.370 inches

3. Desired Yield Adjustment Factor

$P=\text{Desired Yield}/\text{Actual Yield} = 1.7/1.37 = 1.24$

4. Calculate (CI) for Peak Q

$CI= Q/A= 2.04 \text{ in/hr} * 0.714 = 1.46 \text{ cfs/ac}$

5. Increase C Coefficients By Adjustment Factor

6. Find intensity with increased C that gives same CI

Max C= 0.886024

$I= C*I/C=1.46/0.886= 1.644 \text{ in/hr}$

7. Find T_c on K Zone 10-Yr Intensity Table with peak intensity

$T_c= 22$ min

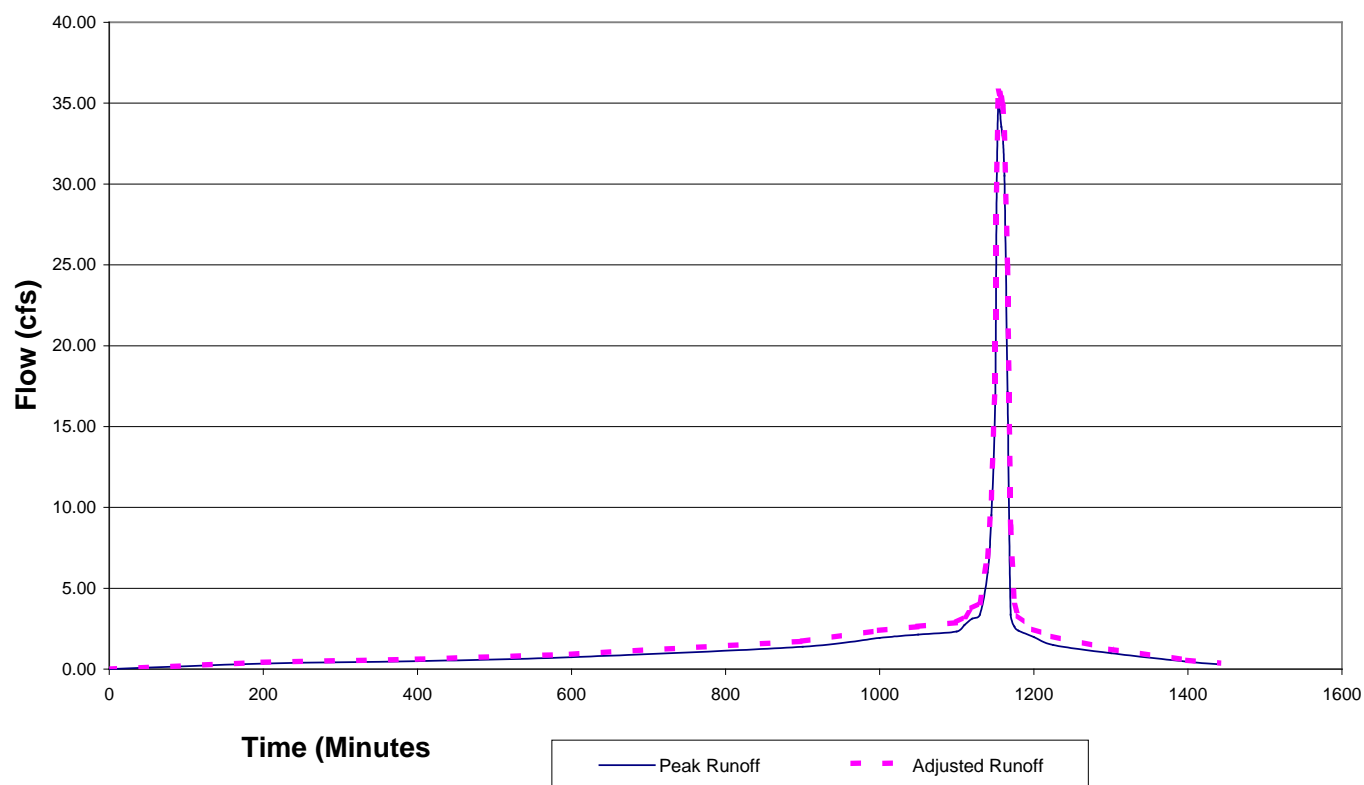
8. Recalculate Q's Using

C from adjusted runoff coefficient curve

I for $T_c= 22$ min

Area as above (24 ac)

B-2.3 Runoff Hydrograph with Adjustment



B-3 EXAMPLE 2 - VCRAT PROGRAM USE FOR MULTIPLE SUBAREAS

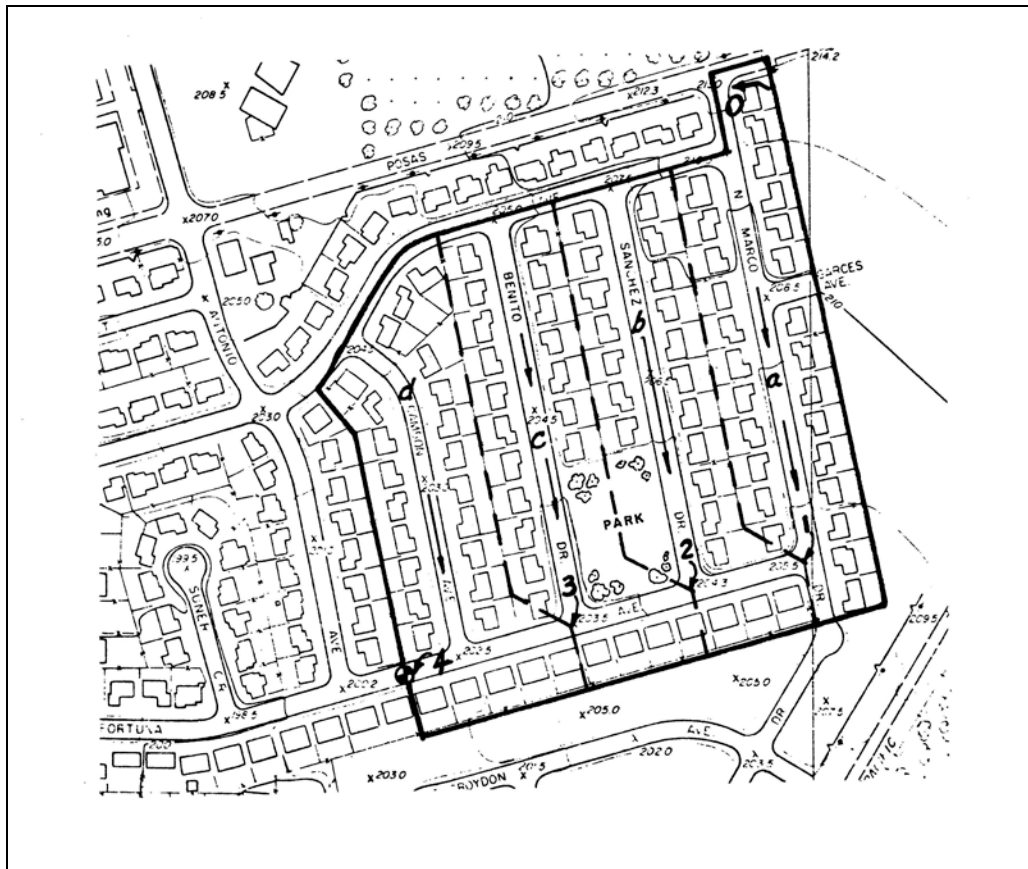
1. Watershed Delineation- Carefully outline the drainage boundary on a topographic map. With reference to other larger scale maps, known improvement plans, and planned improvements, decisions governed by topography alone must be adjusted to reflect an ultimate drainage boundary.
2. Field Verification- Verify drainage boundary map with a field trip to the area and incorporate any changes on the map.
3. Subarea Delineation- Divide the watershed into subareas ranging from 40 to 80 acres in size. Where areas tributary to the main collector channel exceed the maximum size, a separate lateral channel is delineated and the contributing area is also subdivided.
4. Beginning with the point of concentration of the hydraulically most remote subarea, assign a number representing each point in the drainage system where a Q is to be determined. The main channel is designated the "A" -line and subareas are numbered sequentially in VCRat 2.2 (i. e., 1A, 2A, 3A, etc.).
5. Laterals are assigned a letter from B to F preceded by the next number in sequence (i.e., 11B, 12B, 14C, etc.).

6. At a lateral-main line confluence, the next number in sequence is followed by letters representing both lines, with the primary channel listed first (i. e. I6AB).
7. Using the hydrologic maps or GIS files in Appendix E, prepare a soils overlay of the watershed. Place the soils map over the watershed and determine which soil type is dominant in each subarea.
8. Determine anticipated ultimate development from best sources available (current zoning, Planning Department master plans, development trends in surrounding area). Undeveloped, single family residential and commercial development should encompass most areas, with occasional classifications of heavy industrial or large estate lots. Prepare land-use map of watershed.
9. Assign an impervious percentage to each development using the general classifications in Exhibit 15. When a subarea has more than one development type, a weighted average imperviousness should be determined.
10. Planimeter or use GIS to calculate the area of each subarea and check by totaling all subareas and comparing with the total watershed area.
11. From provided maps or shapefiles, determine the appropriate rainfall intensity zone.
12. Using the form and the hand-calculation procedures described above or Tc calculator program, compute time of concentration for all subareas.
13. Determine the length and average slope of conveyance channels between collection points.
14. Sufficient watershed data is now available to code the input for VCRat 2.2 or VCRat 2.5. A detailed description of input data for each column is included in below. An input data line is needed for each subarea and confluence with sufficient data listed for the computation of a runoff hydrograph and flood routing of each hydrograph through the indicated conveyance channel. To clear the hydrograph memory at the start of a run, a letter G should be placed in Column 64, Row 1, in the first 006 data line for each VCRat 2.2 job. Place a number 1 in Column 63 in each row where confluence flow and timing data are required, and in Column 62 in each row where a printed hydrograph is desired. A page heading data line, is needed for each job that will clearly identify the job, and for each point at which a hydrograph has been specified.
15. Data input for the seven runoff coefficient curves are included as input for the program in the default file VCRAIN.DAT. If composite runoff coefficient curves are needed for a watershed, they should be calculated and entered in the VCRAIN.DAT file.

Computer input and output for an example watershed are provided [below](#).

NOTE: The VCRat programs perform complex channel routing calculations with wave velocity so that it is not possible to reproduce the results of the VCRat model by hand at this time. If several subareas are modeled and require channel routing, the hydrologist must obtain the VCRat program from the District in order to perform the design study correctly.

B-3.1 Multiple Subarea Watershed- Las Posas



VENTURA COUNTY WATERSHED PROTECTION DISTRICT
 TIME OF CONCENTRATION
 TC Program Version: 2.6.2008.11
 Project: Las Posas Manual Example
 Date: 5/27/2010 1:52:09 PM
 Engineer: TMB
 Consultant: VCWPD

S U M M A R Y O F C O M P U T A T I O N S

Watershed Name: Las Posas

Name	Zone	Storm	Soil	Area (acres)	TC (min)
Single Family and Par	K	10	4.00	24.0 / 24	15.003 / 15

Watershed Name: Las Posas

Sub-Area Name: Single Family and Park
 Tc: 15.003 Minutes
 DATA FOR SUB AREA 1

SUB AREA TIME OF CONCENTRATION: 15.003 min. = 15 min.

SUB AREA INPUT DATA

 Sub Area Name: Single Family and Park
 Total Area (ac): 24
 Flood Zone: 3
 Rainfall Zone: K
 Storm Frequency (years): 10
 Development Type: Residential
 Soil Type: 4.00
 Percent Impervious: 23
 SUB AREA OUTPUT

Intensity (in/hr): 2.044
 C Total: 0.720
 Sum Q Segments (cfs): 35.31
 Q Total (cfs): 35.31
 Sum Percent Area (%): 100.0
 Sum of Flow Path Travel Times (sec): 900.17
 Time of Concentration (min): 15.003

DATA FOR FLOW PATH 1

Flow Path Name: Overland
 FLOW PATH TRAVEL TIME (min): 5.7681
 Flow Type: Overland
 Length (ft): 140
 Top Elevation (ft): 214
 Bottom Elevation (ft): 213
 Contributing Area (acres): 1.2
 Percent of Sub-Area (%): 5.0
 Overland Type: Valley
 Development Type: Residential
 Map Slope: 0.0071
 Effective Slope: 0.0071
 Q for Flow Path (cfs): 1.77
 Avg Velocity (ft/s): 0.40
 Passed Scour Check: N/A

DATA FOR FLOW PATH 2

Flow Path Name: Subarea A
 FLOW PATH TRAVEL TIME (min): 5.5823
 Flow Type: Street
 Length (ft): 1040
 Top Elevation (ft): 213
 Bottom Elevation (ft): 205.5
 Contributing Area (acres): 4.32
 Percent of Sub-Area (%): 18.0
 Street Width (ft): 32
 Curb Height (in): 6
 Map Slope: 0.0072
 Q for Flow Path (cfs): 6.36
 Q Top (cfs): 1.77
 Q Bottom (cfs): 8.12
 Velocity Top (ft/s): 1.68
 Velocity Bottom (ft/s): 2.46
 Avg Velocity (ft/s): 2.07
 Wave Velocity (ft/s): 3.11

DATA FOR FLOW PATH 3

Flow Path Name: Subarea B
 FLOW PATH TRAVEL TIME (min): 1.1873
 Flow Type: Street

Length (ft): 250
Top Elevation (ft): 205.5
Bottom Elevation (ft): 204.3
Contributing Area (acres): 6.48
Percent of Sub-Area (%): 27.0
Street Width (ft): 32
Curb Height (in): 6
Map Slope: 0.0048
Q for Flow Path (cfs): 9.53
Q Top (cfs): 8.12
Q Bottom (cfs): 17.65
Velocity Top (ft/s): 2.11
Velocity Bottom (ft/s): 2.57
Avg Velocity (ft/s): 2.34
Wave Velocity (ft/s): 3.51

DATA FOR FLOW PATH 4

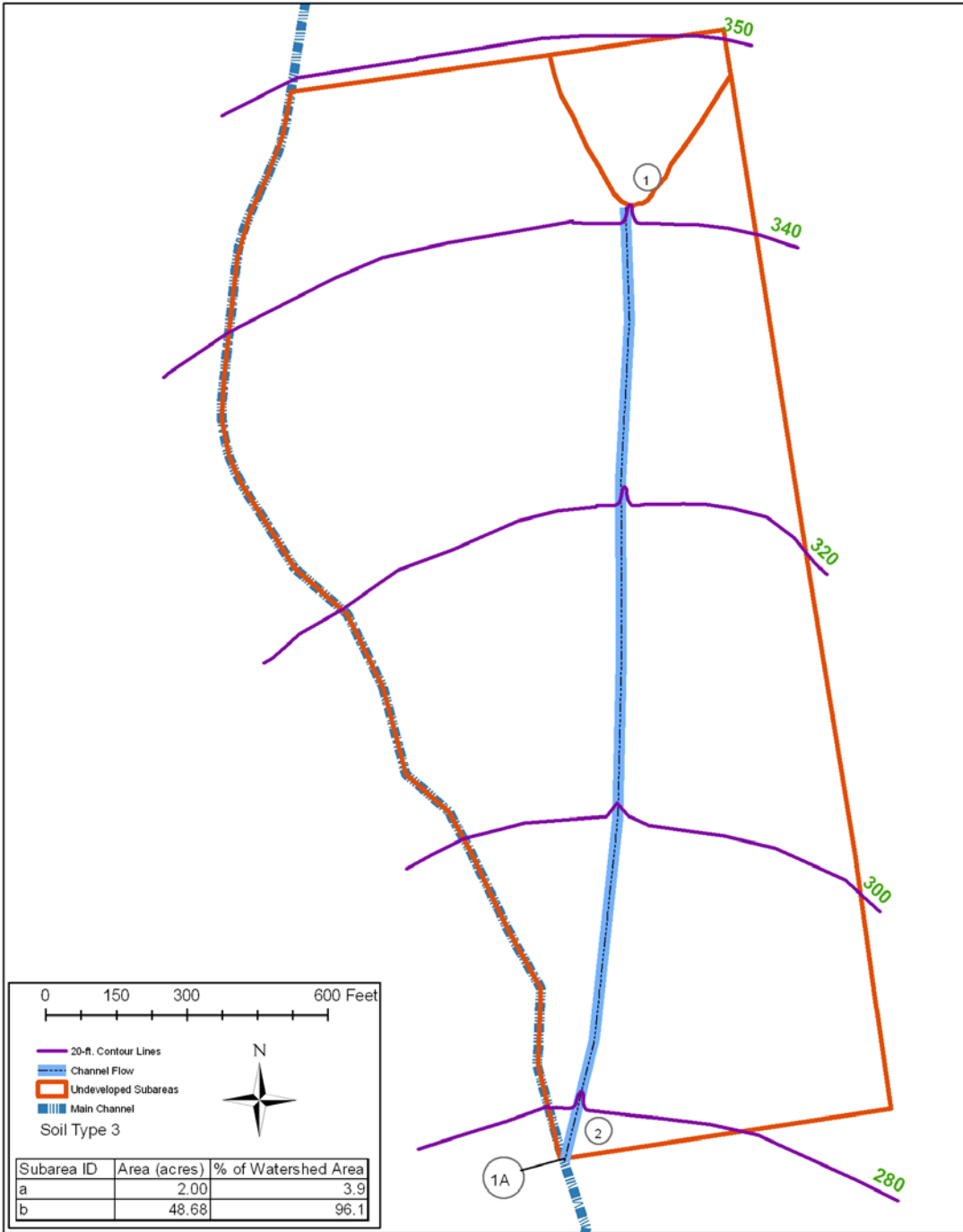
Flow Path Name: Subarea C
FLOW PATH TRAVEL TIME (min): 1.1988
Flow Type: Street
Length (ft): 250
Top Elevation (ft): 204.3
Bottom Elevation (ft): 203.5
Contributing Area (acres): 5.76
Percent of Sub-Area (%): 24.0
Street Width (ft): 32
Curb Height (in): 6
Map Slope: 0.0032
Q for Flow Path (cfs): 8.47
Q Top (cfs): 17.65
Q Bottom (cfs): 26.13
Velocity Top (ft/s): 2.20
Velocity Bottom (ft/s): 2.43
Avg Velocity (ft/s): 2.32
Wave Velocity (ft/s): 3.48

DATA FOR FLOW PATH 5

Flow Path Name: Subarea D
FLOW PATH TRAVEL TIME (min): 1.2663
Flow Type: Street
Length (ft): 370
Top Elevation (ft): 203.5
Bottom Elevation (ft): 201.2
Contributing Area (acres): 6.24
Percent of Sub-Area (%): 26.0
Street Width (ft): 32
Curb Height (in): 6
Map Slope: 0.0062
Q for Flow Path (cfs): 9.18
Q Top (cfs): 26.13
Q Bottom (cfs): 35.31
Velocity Top (ft/s): 3.12
Velocity Bottom (ft/s): 3.37
Avg Velocity (ft/s): 3.25
Wave Velocity (ft/s): 4.87

B-4 EXAMPLE 3 UNDEVELOPED WATERSHED- CLARKE BARRANCA

B-4.1 Watershed Map



K:\PR\hydrology\VCRATManual\2006Manual_Figures\TCEexampleFigs

01/09/2006

B-4.2 Tc Calculator Results

VENTURA COUNTY WATERSHED PROTECTION DISTRICT
 TIME OF CONCENTRATION
 TC Program Version: 2.6.2008.11
 Project: Clark Barranca - Tc Example 3
 Date: 5/27/2010 1:52:09 PM
 Engineer:
 Consultant: VCWPD

S U M M A R Y O F C O M P U T A T I O N S

Watershed Name: Watershed

Name	Zone	Storm	Soil	Area (acres)	TC (min)
Clark Barranca	K	100	3.00	50.6 / 51	12.186 / 12

Watershed Name: Watershed

Sub-Area Name: Clark Barranca
 Tc: 12.186 Minutes
 DATA FOR SUB AREA 1

SUB AREA TIME OF CONCENTRATION: 12.186 min. = 12 min.

SUB AREA INPUT DATA

Sub Area Name: Clark Barranca
 Total Area (ac): 50.6
 Flood Zone: 2
 Rainfall Zone: K
 Storm Frequency (years): 100
 Development Type: Undeveloped
 Soil Type: 3.00
 Percent Impervious: 0
 SUB AREA OUTPUT

Intensity (in/hr): 3.230
 C Total: 0.788
 Sum Q Segments (cfs): 128.72
 Q Total (cfs): 128.72
 Sum Percent Area (%): 100.0
 Sum of Flow Path Travel Times (sec): 731.17
 Time of Concentration (min): 12.186

DATA FOR FLOW PATH 1

Flow Path Name: Overland Area A
 FLOW PATH TRAVEL TIME (min): 8.5319
 Flow Type: Overland
 Length (ft): 424
 Top Elevation (ft): 350
 Bottom Elevation (ft): 340
 Contributing Area (acres): 1.97
 Percent of Sub-Area (%): 3.9
 Overland Type: Valley
 Development Type: Undeveloped
 Map Slope: 0.0236
 Effective Slope: 0.0236

Q for Flow Path (cfs): 5.01
Avg Velocity (ft/s): 0.83
Passed Scour Check: YES
Scour Velocity (ft/sec): 3.20

DATA FOR FLOW PATH 2

Flow Path Name: Valley Channel
FLOW PATH TRAVEL TIME (min): 3.6543
Flow Type: Natural Channel
Length (ft): 2040
Top Elevation (ft): 340
Bottom Elevation (ft): 280
Contributing Area (acres): 48.63
Percent of Sub-Area (%): 96.1
Overland Type: Valley
Map Slope: 0.0294
Effective Slope: 0.0294
Q for Flow Path (cfs): 123.71
Q Top (cfs): 5.01
Q Bottom (cfs): 128.72
Velocity Top (ft/s): 3.62
Velocity Bottom (ft/s): 8.79
Avg Velocity (ft/s): 6.20
Wave Velocity (ft/s): 9.30

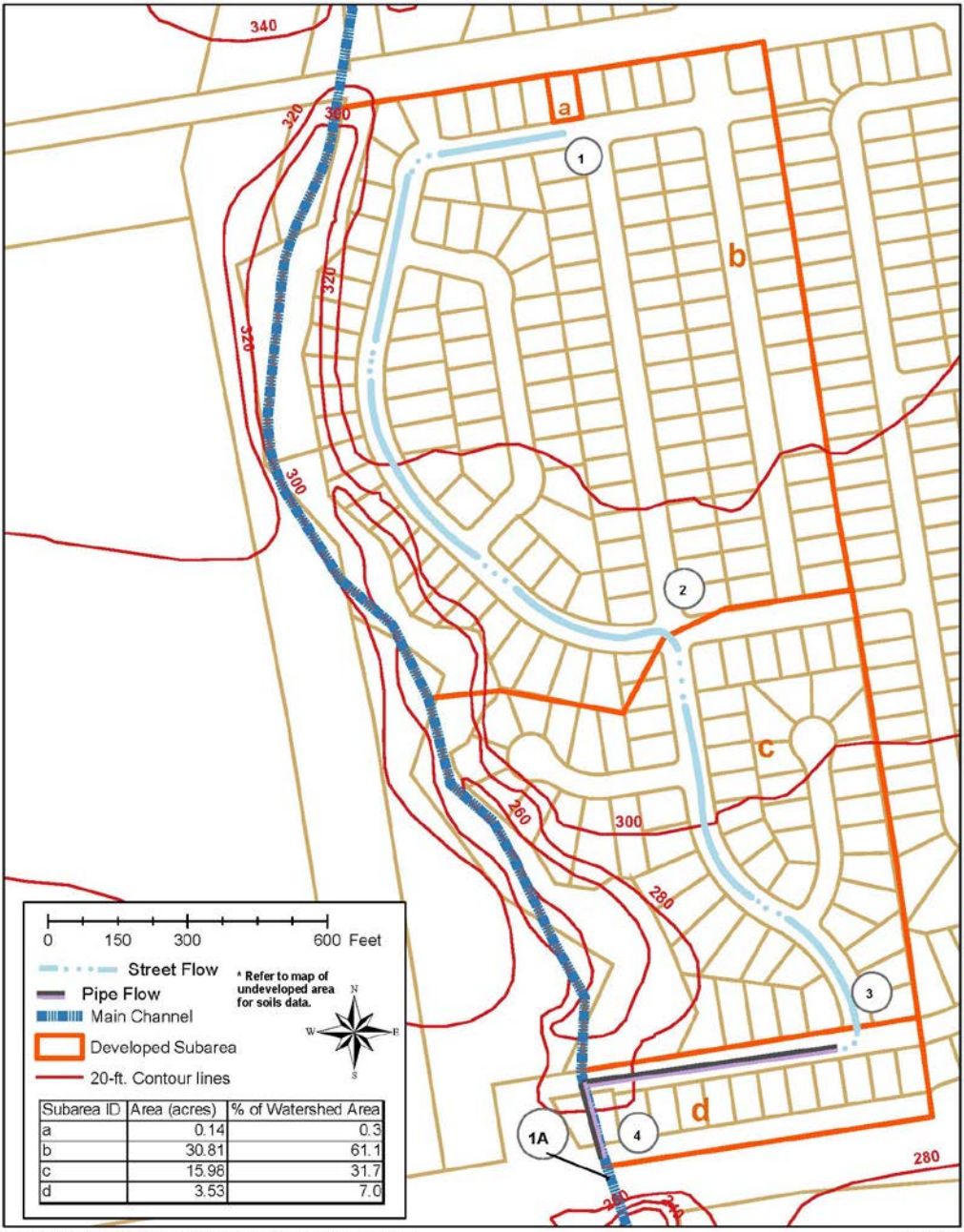
B-5 EXAMPLE 4 DEVELOPED WATERSHED- CLARKE BARRANCA

B-5.1 Aerial Map



K:\PR\hydrology\VCRATManual\2006Manual\Figures\TCExampleFigs 01/09/2007

B-5.2 Parcel Map



K:\PR\Hydrology\VCRAT\Manual\2006Manual_Figures\TCExampleFigs

01/09/2007

B-5.3 Tc Calculator Results

VENTURA COUNTY WATERSHED PROTECTION DISTRICT
 TIME OF CONCENTRATION
 TC Program Version: 2.6.2008.11
 Project: Clark Barranca - Tc Example 4
 Date: 5/27/2010 1:52:09 PM
 Engineer: TMB
 Consultant: VCWPD

S U M M A R Y O F C O M P U T A T I O N S

Watershed Name: Clark Barranca

Name	Zone	Storm	Soil	Area (acres)	TC (min)
Clark Barranca	K	100	3.00	50.7 / 51	8.291 / 8

Watershed Name: Clark Barranca

Sub-Area Name: Clark Barranca
 Tc: 8.291 Minutes
 DATA FOR SUB AREA 1

SUB AREA TIME OF CONCENTRATION: 8.291 min. = 8 min.

SUB AREA INPUT DATA

Sub Area Name: Clark Barranca
 Total Area (ac): 50.65
 Flood Zone: 2
 Rainfall Zone: K
 Storm Frequency (years): 100
 Development Type: Residential
 Soil Type: 3.00
 Percent Impervious: 50
 SUB AREA OUTPUT

Intensity (in/hr): 3.953
 C Total: 0.882
 Sum Q Segments (cfs): 176.62
 Q Total (cfs): 176.62
 Sum Percent Area (%): 100.0
 Sum of Flow Path Travel Times (sec): 497.44
 Time of Concentration (min): 8.291

DATA FOR FLOW PATH 1

Flow Path Name: Overland Area A
 FLOW PATH TRAVEL TIME (min): 2.0317
 Flow Type: Overland
 Length (ft): 100
 Top Elevation (ft): 347
 Bottom Elevation (ft): 345
 Contributing Area (acres): 0.15
 Percent of Sub-Area (%): 0.3
 Overland Type: Valley
 Development Type: Residential
 Map Slope: 0.0200
 Effective Slope: 0.0200

Q for Flow Path (cfs): 0.52
Avg Velocity (ft/s): 0.82
Passed Scour Check: N/A

DATA FOR FLOW PATH 2

Flow Path Name: Street Area B
FLOW PATH TRAVEL TIME (min): 4.4400
Flow Type: Street
Length (ft): 1750
Top Elevation (ft): 345
Bottom Elevation (ft): 310
Contributing Area (acres): 30.92
Percent of Sub-Area (%): 61.0
Street Width (ft): 32
Curb Height (in): 6
Map Slope: 0.0200
Q for Flow Path (cfs): 107.82
Q Top (cfs): 0.52
Q Bottom (cfs): 108.34
Velocity Top (ft/s): 1.82
Velocity Bottom (ft/s): 6.94
Avg Velocity (ft/s): 4.38
Wave Velocity (ft/s): 6.57

DATA FOR FLOW PATH 3

Flow Path Name: Street Area C
FLOW PATH TRAVEL TIME (min): 1.3622
Flow Type: Street
Length (ft): 925
Top Elevation (ft): 310
Bottom Elevation (ft): 290
Contributing Area (acres): 16.04
Percent of Sub-Area (%): 31.7
Street Width (ft): 32
Curb Height (in): 6
Map Slope: 0.0216
Q for Flow Path (cfs): 55.93
Q Top (cfs): 108.34
Q Bottom (cfs): 164.28
Velocity Top (ft/s): 7.15
Velocity Bottom (ft/s): 7.94
Avg Velocity (ft/s): 7.55
Wave Velocity (ft/s): 11.32

DATA FOR FLOW PATH 4

Flow Path Name: Pipe Area D
FLOW PATH TRAVEL TIME (min): 0.4567
Flow Type: Pipe
Length (ft): 600
Top Elevation (ft): 290
Bottom Elevation (ft): 280
Contributing Area (acres): 3.54
Percent of Sub-Area (%): 7.0
Initial Pipe Diameter (in): 48
Calculated Pipe Diameter (in): 48
Used Pipe Diameter (in): 48
Manning's N: 0.012
Map Slope: 0.0167
Q for Flow Path (cfs): 12.34
Q Top (cfs): 164.28

Q Bottom (cfs): 176.62
Avg Velocity (ft/s): 17.90
Wave Velocity (ft/s): 21.89

-

B-6 VCRAT 2.2 PROGRAM INPUT DATA

VCRat 2.2 program input data must be included in the order shown except for the data marked as optional in the following list:

- a. System Data.
- b. Page Heading Data.
- c. System Data.
- d. Hydrograph Header Data. (Optional)
- e. Hydrograph Data (Optional)
- f. System Data.
- g. Location Data.
- h. Reservoir Routing and Yield Adjustment Data
- i. System Data.
- j. Runoff Coefficient Curve Data.
- k. Rainfall Mass Curve Header Data.
- l. Rainfall Mass Curve Data.
- m. System Data.

VCRat 2.6 requires the same data but the Windows interfaces formats the input file for the modeler.

B-6.1 Input Data Preparation

Page Heading Input Data (Code 005)- Required

Column	Entry	Description
1-3	005	Input code number.
4-9	integer	Job number to identify watershed or drain. For VCWPD models, this is the 5-digit project number associated with each jurisdictional channel.
10-14	integer	Identification point which must be identical to the most upstream point on the location layout sheet.
15-16	alphabetic	Drain label which must be identical to those indicated on location layout sheet Column.
17-80	alphanumeric	Description to be printed.

The end of the 005 input data is specified by entering 999 in the first three columns of the line after the last 005 data.

Hydrograph Data Header (Code 007) - Required

This line is required for all locations where the user wants import a hydrograph into the model.

Column	Entry	Description
1-3	007	Input code number.
4-9	integer	Job number to identify watershed or drain. For VCWPD, this is the 5-digit project number associated with each redline channel.
10-15	alphanumeric	Location number of point where hydrograph to be read in.
16-23	integer	Drainage area tributary to location in acres. If omitted the total watershed area of the model will be incorrect.
24-26	integer	Number of points used to define rising limb of hydrograph (point number of peak Q).
27-30	integer	Time of peak flow in minutes.
31-38	integer	Peak flow rate in cubic feet per second.
39-41	integer	Total number of points used to define hydrograph.
42-44	1	If first day of four-day design storm.
	2	If second day of four-day design storm.
	3	If third day of four-day design storm.
	4	If fourth day of four-day design storm.

Hydrograph Data (Code 008) Input

This input is required after 007 input data are provided.

Column	Entry	Description
1-3	008	Input code number.
4-6	integer	Maximum cumulative point number on each input line.
7-11	integer	Selected storm time in minutes.
12-20	integer	Instantaneous flow rate in cubic feet per second corresponding to storm

		time listed in Column 7-11.
20-24	integer	Selected storm time.
25-32	integer	Instantaneous flow rate corresponding to storm time listed in Column 20-24.
33-37	integer	Selected storm time.
38-45	integer	Instantaneous flow rate corresponding to storm time listed in Column 33-37.
46-50	integer	Selected storm time.
51-58	integer	Instantaneous flow rate corresponding to storm time listed in Column 46-50.
59-63	integer	Selected storm time.
64-71	integer	Instantaneous flow rate corresponding to storm time listed in Column 59-63.
<p>Selected storm times may vary between 0 and 1500 minutes up to a maximum 200 points and must be listed chronologically. If default J through L zone rainfall mass curves are used, all hydrograph points for times described in the Exhibit 17 must be used.</p> <p>If A through I or T storm patterns are used, times to define a hydrograph must be the same times used to define the cumulative storm rainfall curve.</p>		

The end of the 007 and 008 input is specified by entering 999 in the first three columns of the last 008 data line.

Location Data (Code 006) - Required

This data input line generally follows 005 input data, or 007 and 008 input lines if a hydrograph has been imported for use in the simulation. A line with 999 in the first three columns precedes the first 006 line signifying the end of the header data lines or hydrograph import section.

Column	Entry	Description
1-3	006	Input code number.
4-9	integer	Job number to identify watershed or drain.
10-14	integer	Location number or model ID of point in the watershed where a calculation is to be made. Location numbers must be listed in the sequence calculations are to be made and numbered sequentially from upstream to downstream without gaps.

Column	Entry	Description
15	A	If hydrograph in main line of the drainage system located in primary storage.
	B	If hydrograph in main line of the drainage system located in primary storage.
	C	If hydrograph in Lateral C located in primary storage.
	D	If hydrograph in Lateral D located in primary storage.
	E	If hydrograph in Lateral E located in primary storage.
	F	If hydrograph in Lateral F located in primary storage.
		The alphabetic code identifies the drain where flow rate is to be modified by: (1) addition of a hydrograph from a lateral or subarea; (2) separation of flow by junction with a relief drain; (3) flood routing and channel storage effects. A letter must be listed on each location data input line.
16	B	If hydrograph in principal Lateral B to be combined with hydrograph in the main line.
	C	If hydrograph in Lateral C to be combined with hydrograph in Line A or B.
	D	If hydrograph in Lateral D to be combined with hydrograph in Lines A, B or C.
	E	If hydrograph in Lateral E to be combined with hydrograph in Lines A, B, C, or D.
	F	If hydrograph in Lateral F to be combined with hydrograph in Lines A, B, C, D, or E.
		For relief drains (See Column 53-59.) the alphanumeric code indicates the lateral receiving flow separated from the primary drain.
17	integer	Place 0 in this column when one of seven standard runoff coefficient curves is used.
18	integer	When one of the seven standard runoff coefficient curves are used, place a number from 1 through 7 in this column.
19	integer	Place 0 in this column when one of seven standard runoff coefficient curves is used.
		When a composite runoff coefficient curve is used, any integer designation between 002 through 199 may be used in columns 17-19.
20-22	integer	Effective imperviousness of subarea in percent.
23-26	integer	Subarea tributary drainage in acres.
27-28	integer	Time of concentration in minutes. An arbitrary time for a zero acre subarea

Column	Entry	Description
		in Column 23-26 must be shown if flood routing through a reach with no tributary area is desired- 99 is typically used.
29	alphabetic	If storm pattern D through I for other than design storm rainfall to be used.
	J (or A)	If J or J' zone design storm rainfall pattern to be used. (For 100-year frequency rainfall use A; also for areal reduction runs.)
	K (or B)	If K zone design storm rainfall pattern to be used. (For 100-year frequency rainfall use B.)
	L (or C)	If L zone design storm rainfall pattern to be used. (For 100-year frequency rainfall use C, also for areal reduction runs.)
	T	If design thunderstorm rainfall pattern to be used. Also used for simulations to evaluate areal reduction for K Zone 50-yr storms
30-31	integer	If A through I storm pattern to be used, an identification number between 1 and 99 must also be used.
	01	If thunderstorm (convective storm) design rainfall pattern to be used. Also used for simulations to evaluate areal reduction for K zone 50-yr storms.
	10	If 10 - year frequency rainfall for J, K, L zone rainfall to be used.
	25	If 25 - year frequency rainfall for J, K, L zone rainfall to be used.
	50	If 50 - year frequency rainfall for J, K, L zone rainfall to be used.
	97	If 100-year frequency rainfall for A zone rainfall to be used.
	98	If 100-year frequency rainfall for B zone rainfall to be used.
	99	If 100-year frequency rainfall for C zone rainfall to be used.
32	integer	Specify the type of channel to use for routing to next confluence point
	1	If mountain channel reach to be used. (See Exhibit 9.)
	2	If natural valley channel reach to be used. (SeeExhibit 10)
	3	If typical 40-foot road width, 8-inch curb street to be used. (SeeExhibit 11d)
	4	If circular pipe to be used, or if street flow is undesirable.
	5	If rectangular or trapezoidal channel base width or depth to be specified. (Trapezoidal section side slope can be specified as well as Manning's n values.)
	6	If trapezoidal channel side slope, maximum peak velocity, and either width or depth to be specified. The trapezoidal channel (6) may involve composite lining with unlined bottom to facilitate channel percolation, or for economic or other design

Column	Entry	Description
		considerations. In steeper areas where scour of the channel bottom could occur, the system routes flow by adjusting channel slope as necessary (drop structures assumed) to not exceed the specified maximum velocity at peak flow rate in Cols 76-77. Specified values would be: (1) maximum velocity and maximum depth; or (2) maximum velocity and base width.
	blank	If the system is to begin flood routing in a street section and change from street to pipe section when flow depth reaches the property line, from pipe to rectangular channel when pipe diameter of 8 feet is exceeded, and from the hydraulically most efficient rectangular section to a maximum depth of 13 feet when that depth is reached. [This change in conveyance is also automatic if either street (3) or pipe (4, with no size listed) is specified].
33-37	integer	Length of channel reach between subarea collection points in feet.
38-43	numeric	Slope of drain in feet per foot. Decimal is not used but is assumed to be between columns 37 and 38.
44-46	numeric	If trapezoidal channel (5 or 6 in Column 32). Computed as horizontal projection of channel wall divided by depth in feet per foot. Decimal is not used but is assumed to be between columns 44 and 45.
	blank	If rectangular section.
47-52	numeric	If specified circular pipe diameter in feet is to be used (4 in Column 32). Decimal is not used but is assumed to be between columns 50 and 51. If 3 is specified in Column 32, then any width in these fields will force the flow to stay in the street no matter how large the flow is.
	numeric	If specified bottom width of rectangular channel in feet is to be used (5 in Column 32).
	numeric	If specified bottom width of trapezoidal channel in feet is to be used (6 in Column 32).
	blank	For channel types 1, 2, and 3, or if the user wishes the program to size the appropriate channel for types 4, 5, or 6.
53-59	integer	Peak flow rate in cubic feet per second (no decimal) to remain in main line or lateral listed in Column 15 at junction with relief drain specified (3 or 4 in Column 60).
	integer	Flow rate to be used to as the basis to calculate a percent used to proportion hydrograph and transfer excess to relief drain (1 in Column 60).
	integer	Percent (no decimal) to be applied to hydrograph ordinate to proportion the hydrograph and transfer excess to relief drain (2 in Column 60).
60	1	Hydrograph in drain listed in Column 15 to be proportioned on percentage basis with residual flow transferred to relief drain listed in Column 16.

Column	Entry	Description
		Percentage is calculated by dividing flow number in Columns 53-59 by peak flow calculated by program. Hydrograph remaining in drain will have peak flow listed in Column 53-59.
	2	Hydrograph in drain listed in Column 15 to be proportioned on percentage basis such that hydrograph remaining in drain has percentage of total flow specified in Column 53-59 with excess flow transferred to relief drain listed in Column 16.
	3	Hydrograph in drain listed in Column 15 to be separated such that all flow up to peak flow rate listed in Column 53-59 remains in the drain with excess flow transferred to relief drain listed in Column 16.
	4	Hydrograph in drain listed in Column 15 to be separated such that only flow above base value (equal to peak minus specified value in Columns 53-59) remains in the drain with all other flow transferred to relief drain listed in Column 16.
61	1	If hydrograph for all four days of a four-day design storm to be computed.
	2	If hydrograph for second, third, and fourth days of four-day design storm to be computed.
	3	If hydrograph for third and fourth days of four-day design storm to be computed.
	blank	If hydrograph for only fourth day (maximum day rainfall) of four-day design storm to be computed, or hydrograph for thunderstorm, or other selected storm rainfall to be computed.
62	1	If hydrograph printout only desired. Requires 005 input data containing location for hydrograph
	2	If hydrograph printout plus output hydrograph file desired.
63	1	Confluence Q printout listing peak flow and time for primary drain and lateral and combined peak and time at downstream end of confluence.
64	A	If hydrograph stored in Line A to be erased.
	B	If hydrograph stored in Line B to be erased.
	C	If hydrograph stored in Line C to be erased.
	D	If hydrograph stored in Line D to be erased.
	E	If hydrograph stored in Line E to be erased.
	F	If hydrograph stored in Line F to be erased.
	G	If all hydrographs stored in system to be erased (beginning of model).

Column	Entry	Description
65	1	If project description heading at beginning of printout and heading for hydrograph printout sheets desired.
	2	If end of job- last data input line.
66	A,B,C,D,E,F	Causes hydrograph data input from Column 15 to be read into main line or lateral listed here.
67	blank	If area reduction not to be computed.
	1	If main line flow to be recomputed using area reduction factor for total drainage area and initially computed drain sizes.
	2	If Lateral B flow to be recomputed using area reduction factor for total drainage area and initially computed drain sizes.
	3	If Lateral C flow to be recomputed using area reduction factor for total drainage area and initially computed drain sizes.
	4	If Lateral D flow to be recomputed using area reduction factor for total drainage area and initially computed drain sizes.
	5	If Lateral E flow to be recomputed using area reduction factor for total drainage area and initially computed drain sizes.
	7	If area reduction to be set at 1.0.
68-70	blank	If minimum n of .014 to be used for channel bottom- channel type 5 or 6.
	numeric	If higher n value for rougher channel bottom to be used for channel type 5 or 6. Decimal is not used but is assumed to be between columns 67 and 68.
71-73	blank	If minimum n of .014 to be used for channel side walls- types 5 and 6.
	numeric	If higher n value for rougher surface to be used for types 5 and 6. Decimal is not used but is assumed to be between columns 37 and 38.
74-75	blank	If rectangular or trapezoidal channel maximum depth 13 feet to be used.
	integer	If rectangular or trapezoidal channel, maximum channel depth in whole feet other than 13 feet to be used.
76-77	integer	Maximum velocity in whole feet per second when trapezoidal channel (6) specified in Column 32.

B-6.2 Rainfall and Runoff Coefficient File Data Input

Runoff Coefficient Curve Data (Code 009) – Required

These data are entered in a separate rainfall and runoff coefficient data file. Currently the default filename used in VCRat 2.2 is vcrain.dat.

Column	Entry	Description
1-3	009	Code number.
4-6	integer	Soil type number specified by Code 010 through 070.If composite runoff coefficient curve is available, use numbers between 002 and 199.
7	1	If runoff coefficient curve is used.
	2	If infiltration rate (loss rate) curve is used.
8-10	integer	Total number of points (maximum of 14) used to define runoff coefficient curve. This number must appear on all input lines.
11-13	integer	Maximum cumulative point number appearing on each input line.
14-17	numeric	Runoff coefficient as decimal corresponding to rainfall intensity listed in Column 18-22. The initial point on first input line must be 0.000. If decimal is not included it is assumed to be between the first and second columns.
18-22	numeric	Rainfall intensity in inches per hour corresponding to point value listed in Column 14-17. The initial point on first input line must be 00.000. If decimal is not included it is assumed to be between the second and third columns
23-26	numeric	Runoff coefficient corresponding to rainfall intensity listed in Column 27-31. If decimal is not included it is assumed to be between the first and second columns
27-31	numeric	Rainfall intensity corresponding to point value listed in column 23-26. If decimal is not included it is assumed to be between the second and third columns
32-35	numeric	Runoff coefficient corresponding to rainfall intensity listed in Column 36-40.
36-40	numeric	Rainfall intensity corresponding to point value listed in Column 32-35. If decimal is not included it is assumed to be between the second and third columns
41-44	numeric	Runoff coefficient corresponding to rainfall intensity listed in Column 45-49. If decimal is not be included it is assumed to be between the first and second columns
45-49	numeric	Rainfall intensity corresponding to point value listed in Column 41-44. If decimal is not included it is assumed to be between the second and third columns
50-53	numeric	Runoff coefficient corresponding to rainfall intensity listed in Column 54-58. If decimal is not be included it is assumed to be between the first and second columns

Column	Entry	Description
54-58	numeric	Rainfall intensity corresponding to point value listed in Column 50-53. If decimal is not included it is assumed to be between the second and third columns
59-76	numeric	Repeat procedure described above for Columns 14-58.

These data are currently included in the VCRAIN.DAT file that is the default runoff coefficient and rainfall mass curve file for VCRat 2.2. VCRat 2.2 and 2.5 use this file to generate a model and rainfall combined input file that is used by the FORTRAN VCRat calculation engine.

Rainfall Mass Curve Header Data (Code 010) – Required

These data currently follow the 009 data entered in a rainfall and runoff coefficient file. Currently the default file used in VCRat 2.2 is vcrain.dat.

Column	Entry	Description
1-3	010	Code number for first line of rainfall hydrograph data line.
4	alphabetic	If storm pattern D through I for other than design storm rainfall to be used.
	J (or A)	If J zone design storm rainfall pattern to be used. (For 100-year frequency rainfall only use A.)
	K (or B)	If K zone design storm rainfall pattern to be used. (For 100-year frequency rainfall only use B.)
	L (or C)	If L zone design storm rainfall pattern to be used. (For 100-year frequency rainfall only use C.)
	T	If design thunderstorm rainfall pattern to be used. Also used for simulations to evaluate areal reduction
5-6	integer	If A through I storm pattern to be used, an identification number between 1 and 99 must also be used.
	01	If thunderstorm (convective storm) design rainfall pattern to be used. Also used for simulations to evaluate areal reduction
	10	If 10 - year frequency rainfall for J, K, L zone rainfall to be used.
	25	If 25 - year frequency rainfall for J, K, L zone rainfall to be used.
	50	If 50 - year frequency rainfall for J, K, L zone rainfall to be used.
	97	If 100-year frequency rainfall for A zone rainfall to be used.
	98	If 100-year frequency rainfall for B zone rainfall to be used.
	99	If 100-year frequency rainfall for C zone rainfall to be used.

Column	Entry	Description
7-9	Integer	Number of points in the hydrograph contained in the following 011 data lines

In VCRat 2.2, J and J' zones use the same zone designations in the VCRAIN.DAT file and the modeler has to use the correct rain mass curve data to get the right results.

Rainfall Mass Curve Data (Code 011)– Required

These data currently follow the 010 data entered in a rainfall and runoff coefficient file. Currently the default file used in VCRat 2.2 is vcrain.dat.

Column	Entry	Description
1-3	011	Code number for lines following 010 line of rainfall mass curve data.
4	J, K, L, A, B, or C	Must match zone description on 010 line preceding 011 line
5-6	integer	Design frequency of storm, must match frequency entered on 010 line preceding 011 line.
7-9	Integer	Total number of points in the rainfall mass curve contained in the 011 data lines
10-12	Integer	Subtotal of mass curve ordinates contained in this and preceding 011 lines – generally 7 points per line.
13-17	Integer	Time in minutes of first mass curve point. First point must be 0000
18-22	Numeric	Cumulative rainfall corresponding to preceding time entry. If decimal is not entered it is assumed to occur between columns 19 and 20.
23-26	Integer	Time in minutes of next mass curve point
27-31	Numeric	Cumulative rainfall corresponding to preceding time entry. If decimal is not entered it is assumed to occur between columns 19 and 20.
32-72	Numeric	Remaining hydrograph times and cumulative rainfall pairs until the total number of pairs matches the entry in columns 10-12.

More than one rainfall mass curve can be included in a rainfall and runoff coefficient file. When all desired rainfall mass curves are included, the last line of the file should have 999 entered in the first three columns.

B-6.3 Reservoir Routing Information

The data input lines can follow any 006 in a VCRat 2.2 input file except the last one. Decimal points must be included with all numbers on any of these data lines. A dummy node must be used in a regular reservoir routing. A dummy node must be used after fattening. A confluence must be used in a split flow or bypass reservoir. The possible data input lines for reservoir routing include the following:

- a. 110 (required for areal reduction, fattening and/or reservoir routing)
- b. 111 (required for areal reduction, fattening and/or reservoir routing)
- c. 112 (required for reservoir routing)
- d. 113 (required for reservoir routing)
- e. 114 (required for reservoir routing)
- f. 115 (required for channel routing directly below the reservoir)
- g. 116 (required for channel routing directly below the reservoir)
- h. 110 (required for fattening and/or reservoir routing)

Start of Reservoir Routing, Areal Reduction, and Fattening Data Input Line (110)

Required to signify beginning and ending of reservoir routing and/or fattening. Can be preceded by a 006, 114, or 116 data line.

Column	Entry	Description
1-3	110	Code number for lines following 006 subarea line with dummy information.

Reservoir Routing, Areal Reduction, and Fattening Data Input Line (111)

Required after a 110 data input line. The numeric entries MUST contain a decimal point.

Column	Entry	Description
1-3	111	Code number for reservoir and hydrograph information.
4-11	numeric	Spillage elevation- usually corresponds to emergency spillway invert elevation. (elevation in feet) Used in reservoir routing only. This numeric entry MUST be in the form #####.##
	blank	No reservoir routing.
12-22	numeric	Hydrograph adjustment or areal reduction factor. Each point on the hydrograph will be multiplied by this adjustment factor. This numeric entry MUST be in the form #####.#####

Column	Entry	Description
	blank	Hydrograph will not be adjusted.
23-30	numeric	Runoff factor. (inches) Used in fattening only. This numeric entry MUST be in the form #####.##
31-38	numeric	Drainage area tributary to location. (acres) Overrides drainage area in node directly below the reservoir. This entry is the same as column16-23 on a 007 card for a read in hydrograph. This numeric entry MUST be in the form #####.##
	blank	No drainage area change.

Reservoir Routing Data- Stage Information (112)

Required if reservoir routing is performed. The reservoir stage (elevation in feet) information is entered on this data line. Up to 10 numeric entries can be placed on each 112 line, and up to five 112 lines are allowed. Elevation values must be larger than zero. The numeric entries MUST contain a decimal point and be in the form #####.##. VCRat 2.2 does not check to make sure the values are entered in ascending order.

Column	Entry	Description
1-3	112	Card code number.
4-11	numeric	The reservoir stage. (elevation in feet)
12-19	numeric	The reservoir stage. (elevation in feet)
20-27	numeric	The reservoir stage. (elevation in feet)
28-35	numeric	The reservoir stage. (elevation in feet)
36-43	numeric	The reservoir stage. (elevation in feet)
44-51	numeric	The reservoir stage. (elevation in feet)
52-59	numeric	The reservoir stage. (elevation in feet)
60-67	numeric	The reservoir stage. (elevation in feet)
68-75	numeric	The reservoir stage. (elevation in feet)
76-83	numeric	The reservoir stage. (elevation in feet)

Reservoir Routing Data- Storage Information (113)

Required after a 112 line. The reservoir storage (volume in acre-feet) information is entered in this line corresponding to the elevations entered on the 112 data lines. Up to 10 numeric entries can be placed on

each 113 line, and up to five 113 lines are allowed. The numeric entries **MUST** contain a decimal point and be in the form #####.###. VCRat 2.2 does not check to make sure the values are entered in ascending order.

Column	Entry	Description
1-3	113	Card code number.
4-11	numeric	The reservoir storage (acre-feet)
12-19	numeric	The reservoir storage (acre-feet)
20-27	numeric	The reservoir storage (acre-feet)
28-35	numeric	The reservoir storage (acre-feet)
36-43	numeric	The reservoir storage (acre-feet)
44-51	numeric	The reservoir storage (acre-feet)
52-59	numeric	The reservoir storage (acre-feet)
60-67	numeric	The reservoir storage (acre-feet)
68-75	numeric	The reservoir storage (acre-feet)
76-83	numeric	The reservoir storage (acre-feet)

Reservoir Routing Data- Discharge Information (114)

Required after a 113 line. The reservoir discharge (cfs) information is entered in this line corresponding to the elevations entered on the 112 lines. Up to 10 numeric entries can be placed on each 114 line, and up to five 114 lines are allowed. The numeric entries **MUST** contain a decimal point and be in the form #####.###. VCRat 2.2 does not check to make sure the values are entered in ascending order.

Column	Entry	Description
1-3	114	Card code number.
4-11	numeric	The reservoir discharge (cfs)
12-19	numeric	The reservoir discharge (cfs)
20-27	numeric	The reservoir discharge (cfs)
28-35	numeric	The reservoir discharge (cfs)
36-43	numeric	The reservoir discharge (cfs)
44-51	numeric	The reservoir discharge (cfs)
52-59	numeric	The reservoir discharge (cfs)
60-67	numeric	The reservoir discharge (cfs)

68-75	numeric	The reservoir discharge (cfs)
76-83	numeric	The reservoir discharge (cfs)

Start of Channel Routing, Areal Reduction, and Fattening Data Input Line (115)

Required after a 114 data line at the end of reservoir routing or a 111 data line after areal reduction and fattening to signify that channel routing below the reservoir or upstream subarea will be performed next.

Column	Entry	Description
1-3	115	Code number for line following 114 end of reservoir routing or 111 end of areal reduction/fattening data input.

Channel Routing Below Reservoir Data Input Line (116)

Required after a 115 data line to provide channel routing information. Dummy information for a subarea should be provided on this line along with the routing information. Please see the explanation for the routing data required for a 006 data line for the required parameters.

End of Channel Routing, Areal Reduction, and Fattening Data Input Line (110)

Required after a 111, 114, or 116 line to signify the end of reservoir and/or channel routing and/or areal reduction and fattening information.

Column	Entry	Description
1-3	110	Code number for line following 111, 114 or 116 data input.

Example data input lines with added column numbers (not used in model input file)

Column Numbering

```

0          10          20          30          40          50          60          70
1234567890123456789012345678901234567890123456789012345678901234567890
*****
006 15031 056B 020          099B98          1
110
111 1130.5          1.00000          4.50
112 1110.          1112.          1114.          1116.          1118.          1119.          1120.          1121.          1122.          1123.
112 1124.          1125.          1126.          1127.          1128.          1129.          1130.          1131.          1132.          1133.
113 0.0          2.5          5.4          8.9          12.0          15.1          17.4          19.9          22.5          25.2
    
```

```

113  28.0   31.0   34.1   37.4   40.9   44.5   48.3   52.4   56.6   60.5
114   0.0   30.0  100.0  195.0  310.0  360.0  400.   430.   463.   492.
114 520.   545.   570.   594.   616.   639.   660.   713.   871.  1088.
115
116 15031  056B 020      099B982 1100002277
110

```

B-6.4 Running VCRat 2.2

Copy the input files (model input and rainfall and runoff coefficient input) to a directory containing the VCRat 2.2 programs. Run program VCRat 2.exe and enter the following data:

1. Model input data filename
2. Rainfall and runoff coefficient mass curve data file
3. Output model data filename.

If the program does not run, the following sections discuss some common error messages that occur.

B-6.5 VCRat 2.2 Error Checking

VCRat 2.2 is designed to edit runoff coefficient curves, rainfall mass curves, and hydrograph data input, and to verify the consistency of subarea data and various computation instructions. It checks input data prior to hydrologic computations. When errors are encountered, further processing is terminated and error messages are produced.

Runoff Coefficient Curve and Rainfall Data Mass Curve Editor (PROGRAM F0601M)

Error messages produced by this program are of the following form:

CURVE NAME ERROR NO.

Error Number	Description
1	The code (Column 1-3) on first line of a mass curve not 009 (runoff coefficient curve) or 010 (rainfall mass curve).
2	Number in Column 7 on the first line outside the range 1-5.

Error Number	Description
3	Total number of points indicated for the curve exceeds 14 (runoff coefficient curve) or 199 (rainfall mass curve).
4	On lines following the first input line of a curve either (a) The input line code (Column 1-3) not equal to 009 (runoff coefficient curve) or 011 (rainfall mass curve), (b) The curve number in Column 4-6 does not match or (c) The curve number in Column 7 does not match.
5	The number of points in a line exceeds the total number indicated on the header line.
6	The data input lines are out of sequence.
7	Initial data point not zero, negative runoff coefficient curve number, or points on rainfall mass curve not in chronological or cumulative sequence.
8	The curve number (Column 4-6) listed in the first header line, not (a) 002 through 199 for runoff coefficient curve, (b) A through I in Column 4 and 01 through 99 in Column 5-6, for selected storm rainfall mass curve, (c) J through L in Column 4 and 10, 25 or 50 in Column 5-6 for standard design storm rainfall curve, or (d) T in Column 4 and 01 in Column 5-6 for thunderstorm (convective storm) rainfall mass curve.

Subarea and Hydrograph Data Editor (PROGRAM F06010)

Error messages produced by this program are of the following form:

LOCATION NAME ERROR NO.

Error Number	Description
1	The line code (Column 1-3) of a subarea data input line not 006.
2	Subarea data not in ascending sequence (hydrograph input may be specified on the same line, and hydrograph modification may be specified on the preceding line) or a thunderstorm specified in Column 67 and Line A not specified in Column 15.
3	Column 15 (primary storage location), Column 17-19 (runoff coefficient curve) or Column 29-31 (rainfall mass curve) contains all blanks or an invalid value.
4	Secondary hydrograph is specified and (a) alphanumeric character other than A through F listed in Column 16, (b) hydrograph computation is also specified, (c) read hydrograph also specified, (d) number in Column 60 outside the range 0-4, (e) number in Column 63 outside range 0-1, or (f) negative number in Column 53-59.

Error Number	Description
5	Hydrograph input and hydrograph computation both indicated.
6	Hydrograph computation specified and (a) time of concentration not listed in Column 27-28 or hydrograph input, (b) confluence output also specified, (c) hydrograph modification also specified, (d) control Q also specified, or (e) negative number in Column 20-28. Read hydrograph specified with confluence output, hydrograph modification or control Q specified also results in error 6 printout.
7	Routing errors with (a) number in Column 32 outside range 0-6, (b) 1-6 specified in Column 32 with slope and channel length not specified in Column 33-43 or, (c) 6 specified in Column 32 and combination of Column 47-52, and Column 74-75, Column 47-52 and Column 76-77, or Column 74-75 and Column 76-77 not specified, (d) number specified in Column 74-75 outside range 0-29, or (e) negative number in Column 33-52 or Column 68-77.
8	Location input line instruction errors with (a) number in Column 61 inconsistent or outside range 0-3, (b) number in both Column 61 and Column 67 other than 0, 1, 2, 3, 5, 7 specified in Column 67, (c) other than alphabetic code A through G specified in Column 64, or (d) negative number in Column 65.
9	Hydrograph header input line errors with (a) input line code (column 1-3) not 007, (b) missing job or location number in Column 4-14, (c) drain A through F not specified in Column 15, (d) number in Column 24-26 outside range 1-200, (e) number in Column 27-30 outside range 1-1500, (f) negative number in Column 16-23 or Column 31-38, (g) number in Column 39-41 outside range 1-200, or (h) number in Column 42-44 outside range 1-4.
10	Hydrograph data input line errors with (a) input line code (Column 1-3) not 008, (b) number of points on input line less or greater than total number indicated on header input line, or (c) data input lines not in sequence.
11	Hydrograph error affecting the associated subarea data set.
12	Initial hydrograph data point not zero, or points on hydrograph not in chronological sequence.
13	Total hydrograph data points not equal to number of points specified in header line.
14	The line code (Column 1-3) not 005 on first page heading input line.
15	Hydrograph output specified and (a) the associated page heading line is missing, (b) the line code (Column 1-3) not 005 on associated page heading line, or (c) the location name on the associated page heading line and location point line not identical.
16	More than 1000 lines of input data submitted for job. (This limitation applies to VCRat 1.0. VCRat 2.2 has been recompiled to allow up to 5,000 subareas.)
18	End of job not specified by number 2 in Column 65.

Hydrologic Computation (PROGRAM F0601A)

Error messages produced by this program are of the following form:

PAGE HEADING

PROCESSING DISCONTINUED AT LOCATION NAME

ERROR NO.

Error Number	Description
2	Device failure has occurred while reading from file. Program should be restarted.
3	The name on a runoff coefficient curve or rainfall mass curve does not match the one called for on a location point line or the number of points exceeds the maximum allowable points.
4	Failure in search of table for partially full pipe sections when computing wave velocities.
5	Hydrograph input indicated and the referenced hydrograph not in the input stream.
6	The number or time of points for an input hydrograph not identical with points used in rainfall mass curve.
7	The specified time of concentration is greater than the interval between zero time and the first time specified on the rainfall mass curve.
8	Failure in search for bottom width of trapezoidal channel.
9	Failure in search for depth while computing wave velocities in trapezoidal channel.
10	Failure in search for area reduction factor during computation of subarea hydrograph.
11	Failure in search for runoff rate during computation of subarea hydrograph.
12	Failure in hydrograph routing due to magnitude of channel flow rate.
99	Recycle past the beginning of the job during thunderstorm computations attempted, or more than 1,000 location point and page heading input lines were submitted. (This limitation applies to VCRat 1.0. VCRat 2.2 has been recompiled to allow up to 5,000 subareas.)

B-6.6 Computation Procedures

Storm Rainfall Relationships

The system interpolates rainfall mass curve data entered as data input and constructs a system storage table of accumulated total rainfall by one minute increments from zero rainfall to the time of the last point indicated by input data.

Design storm rainfall mass curve data included in vcrain.dat are:

Fourth Day Total, Inches

Rainfall Zone	100-Year Storm	50-Year Storm	25-Year Storm	10-Year Storm
J	7.00	5.0	3.91	3.17
J' (J prime)	6.66	6.0	5.28	4.38
K	10.60	8.0	6.41	5.53
L	15.00	11.0	8.81	7.21

The average rainfall intensity for any specified duration at a specified storm time is calculated by subtracting from the cumulative storm rainfall at the specified storm time from the cumulative storm rainfall at the earlier storm time or determined by the specified duration. Rainfall intensity in inches per hour is calculated as the incremental cumulative storm total, times 60 (minutes in an hour) divided by the specified duration (time of concentration in minutes). These results are shown in Plates XX through XX.

For the design storm, rainfall intensities are reduced if computations for other than the fourth day (maximum day) rainfall are specified. The reduction factor is 0.10, 0.40, 0.35, and 1.00 for first through fourth day, respectively. Thunderstorm rainfall intensities are reduced by a reduction curve coded into the program by to decrease rainfall intensity with increasing watershed size.

VCRatP.for data from FORTRAN PROGRAM

ac	sq mi	Factor	ac	sq mi	Factor
0	0	1.0000	19,200	30	0.7850
800	1.25	0.9450	22,400	35	0.7730
1,600	2.5	0.9250	25,600	40	0.7625
3,200	5	0.8950	28,800	45	0.7520
6,400	10	0.8600	32,000	50	0.7420
9,600	15	0.8350	48,000	75	0.7020
12,800	20	0.8150	64,000	100	0.6700
16,000	25	0.8000			

B-7 AREAL REDUCTION EXAMPLE**B-7.1 VCRat 2.2 AR Input File**

The VCRat2.2 input file format for an Areal Reduction run is as follows with AR input in shaded yellow lines:

```

005 8822 1A SUDDEN BARRANCA UPDATE TO 1982 STUDY Q100F DL/JL 3/03; AREALLY REDUCED
005 8822 8ABSUDDEN BARRANCA AT FOOTHILL RD Q100F
005 8822 13ABSUDDEN BARRANCA AT TELEGRAPH RD Q100F
005 8822 18ACSUDDEN BARRANCA AT SANTA PAULA FREEWAY Q100F
005 8822 23ABSUDDEN BARRANCA AFTER JCT W/ PIPE IN JASPER AND EAST TELE.Q100F
005 8822 30ABSUDDEN BARRANCA AFTER JCT. W/ TR 3198 Q100F
999
999
006 8822 1A 040 0 7612B981 1550005940 G1 7
006 8822 2A 030 0 5810B981 1730004040
006 8822 3A 020 2908B981 1250003200
006 8822 4A 020 8211B981 600003330
006 8822 5A 020 099B98
006 8822 6B 020 10 7215B984 650000170
006 8822 7B 030 10 4014B98
006 8822 8AB030 B985 1300002750 11
006 8822 9A 040 23 4108B985 0950002750
006 8822 10A 040 23 31 8B985 950001700
006 8822 11A 040 00 0706B98
006 8822 12B 040 23 3713B984 150000800
006 8822 13AB040 B984 2500002500 11
006 8822 14A 040 10 2107B98
006 8822 15C 040 23 5010B984 2500002000
006 8822 16C 030 35 5312B98
006 8822 17C 010 23 4108B984 1200001200
006 8822 18AC010 B985 1580002200 11
006 8822 19A 030 20 2211B985 1975001500
006 8822 20A 030 23 3211B98
006 8822 21B 030 23 5511B984 1500001400
006 8822 22B 030 23 4912B98
006 8822 23AB030 B985 1200001330 11
006 8822 24C 030 099B98
006 8822 25A 030 099B98
006 8822 26A 030 23 0813B985 337003233
006 8822 27A 030 23 0513B985 472001688
006 8822 28A 030 23 0613B985 165001688
006 8822 29B 030 23 3613B98
1234567890123456789012345678901234567890123456789012345678901234567890(Not used in run)
006 8822 30AB030 B985 1050002630200 800 11 1035035
006 8822 31A 030 23 1608B98 2

```

Where “1” in column 65 prints the heading at the beginning of the printout and headings where hydrograph printout sheets are desired. In column 67 at node 1A (first 006 line , beginning of 006 lines), any number from 1 to 7 is necessary to perform Areal Reduction. In column 64, “G” clears the all the hydrographs stored in the system. Areal reduction runs with or without the column 64 command.

By putting a “1” on column 67 on any node, including dummies and confluences, the program recognizes that areal reduction is needed therefore it will re-run to re-calculate the peak flow. The program then uses the embedded rainfall reduction factors in the VCRATP.for program to obtain an Areal Reduction Factor for Rain (ARr) and applies that to the Zonal rainfall mass curve to reduce the intensities at that location. For this example, areal reduction was set for node 30AB. The program ran regularly until it reaches node 30AB and recognized a command for areal reduction. Therefore, VCRat 2.2 ran again to accommodate for areal reduction. The output should repeat, like the example below. The highlighted section in the input above shows where areal reduction is calculated. The highlighted section in the output below shows how the program runs regularly until 30AB where the areal reduction command was set, which re-calculate to accommodate for areal reduction. The AR factor for flow (ARq) is calculated as the areally-reduced flow/unareally-reduced flow or in this case $1,467/1,583=0.9267$.

3) Another characteristic of an AR run is that the flow values up to the point where AR has been specified (1 in col 67) cannot be used to calculate ARq values because the ARr value is only good at the node point specified. Also, the flow values below where AR was specified are not valid model results and cannot be used for design or in ARq calculations.

4) If a hydrograph is printed at the node where AR is requested, the reduction factor printed at the top of the hydrograph is the ARr value for the node location, not the ARq value. The ARq value has to be hand calculated.

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VCRAT METHODS

B-7.2 VCRat2.2 AR Output File

VENTURA COUNTY FLOOD CONTROL DISTRICT
 MODIFIED RATIONAL METHOD HYDROLOGY / PC 2.21-950
 SUDDEN BARRANCA UPDATE TO 1982 STUDY Q100F DL/JL 3/03

LOCATION	SUBAREA	SUBAREA	TOTAL	TOTAL	CONV	CONV	CONV	CONV	CONV	CONTROL	SOIL	RAIN	PCT				
AREA	Q	AREA	Q	TYPE	LNGLTH	SLOPE	SIZE	Z	Q	NAME	TC	ZONE	IMPV				
8822	1A	76.	182.	76.	182.	1	1550.	0.05940	0.00	0.00	0.	40	12	B98	0.00		
8822	2A	58.	166.	134.	309.	1	1730.	0.04040	0.00	0.00	0.	30	10	B98	0.00		
8822	3A	29.	99.	163.	299.	1	1250.	0.03200	0.00	0.00	0.	20	8	B98	0.00		
8822	4A	82.	235.	245.	423.	1	600.	0.03330	0.00	0.00	0.	20	11	B98	0.00		
8822	5A	0.	0.	245.	415.	0	0.	0.00000	0.00	0.00	0.	20	99	B98	0.00		
8822	6B	72.	174.	72.	174.	4	650.	0.00170	6.00	0.00	0.	20	15	B98	0.10		
8822	7B	40.	95.	112.	262.	0	0.	0.00000	0.00	0.00	0.	30	14	B98	0.10		

* CONFLUENCE Q'S *																	
8822	8A	TA 1161	QA	415.	QAB	666.	QB	250.	8822	8B	TB 1158	QB	262.	QBA	649.	QA	386.
* 8822 8AB TAB 1160 QAB 668. QA 412. QB 256. *																	

LOCATION	SUBAREA	SUBAREA	TOTAL	TOTAL	CONV	CONV	CONV	CONV	CONV	CONTROL	SOIL	RAIN	PCT				
AREA	Q	AREA	Q	TYPE	LNGLTH	SLOPE	SIZE	Z	Q	NAME	TC	ZONE	IMPV				
8822	8AB	112.	262.	357.	668.	5	1300.	0.02750	8.00	0.00	0.	30	0	B98	0.00		
8822	9A	41.	131.	398.	733.	5	950.	0.02750	8.00	0.00	0.	40	8	B98	0.23		
8822	10A	31.	99.	429.	786.	5	950.	0.01700	8.00	0.00	0.	40	8	B98	0.23		
8822	11A	7.	25.	436.	780.	0	0.	0.00000	0.00	0.00	0.	40	6	B98	0.00		
8822	12B	37.	90.	37.	90.	4	150.	0.00800	3.50	0.00	0.	40	13	B98	0.23		

* CONFLUENCE Q'S *																	
8822	13A	TA 1159	QA	780.	QAB	864.	QB	84.	8822	13B	TB 1155	QB	90.	QBA	734.	QA	645.
* 8822 13AB TAB 1159 QAB 864. QA 780. QB 84. *																	

LOCATION	SUBAREA	SUBAREA	TOTAL	TOTAL	CONV	CONV	CONV	CONV	CONV	CONTROL	SOIL	RAIN	PCT				
AREA	Q	AREA	Q	TYPE	LNGLTH	SLOPE	SIZE	Z	Q	NAME	TC	ZONE	IMPV				
8822	13AB	37.	90.	473.	864.	4	2500.	0.02500	6.50	0.00	0.	40	0	B98	0.00		
8822	14A	21.	71.	494.	859.	0	0.	0.00000	0.00	0.00	0.	40	7	B98	0.10		
8822	15C	50.	143.	50.	143.	4	2500.	0.02000	3.50	0.00	0.	40	10	B98	0.23		
8822	16C	53.	145.	103.	267.	0	0.	0.00000	0.00	0.00	0.	30	12	B98	0.35		
8822	17C	41.	151.	144.	404.	4	1200.	0.01200	5.75	0.00	0.	10	8	B98	0.23		

* CONFLUENCE Q'S *																	
8822	18A	TA 1162	QA	859.	QAC	1172.	QC	313.	8822	18C	TC 1158	QC	395.	QCA	1196.	QA	800.
* 8822 18AC TAC 1160 QAC 1216. QA 842. QC 373. *																	

SUBAREA	SUBAREA	TOTAL	TOTAL	CONV	CONV	CONV	CONV	CONV	CONTROL	SOIL	RAIN	PCT					

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LOCATION	AREA	Q	AREA	Q	TYPE	LNPTH	SLOPE	SIZE	Z	Q	NAME	TC	ZONE	IMPV
8822	18AC	144.	395.	638.	1216.	5	1580.	0.02200	10.00	0.00	0.	10	0	B98 0.00
8822	19A	22.	62.	660.	1249.	5	1975.	0.01500	10.00	0.00	0.	30	11	B98 0.20
8822	20A	32.	90.	692.	1291.	0	0.	0.00000	0.00	0.00	0.	30	11	B98 0.23
8822	21B	55.	155.	55.	155.	4	1500.	0.01400	4.00	0.00	0.	30	11	B98 0.23
8822	22B	49.	131.	104.	276.	0	0.	0.00000	0.00	0.00	0.	30	12	B98 0.23

 * CONFLUENCE Q'S *
 * 8822 23A TA 1162 QA 1291. QAB 1499. QB 207. 8822 23B TB 1157 QB 276. QBA 1268. QA 992. *
 * 8822 23AB TAB 1161 QAB 1504. QA 1268. QB 236. *

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNPTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
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VENTURA COUNTY FLOOD CONTROL DISTRICT
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SUDDEN BARRANCA UPDATE TO 1982 STUDY Q100F DL/JL 3/03 STORM DAY 4

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNPTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
8822	23AB	104.	276.	796.	1504.	5	1200.	0.01330	12.00	0.00	0.	30	0	B98 0.00
8822	24C	0.	0.	0.	395.	0	0.	0.00000	0.00	0.00	0.	30	99	B98 0.00
8822	25A	0.	0.	796.	1497.	0	0.	0.00000	0.00	0.00	0.	30	99	B98 0.00
8822	26A	8.	20.	804.	1512.	5	337.	0.03233	10.00	0.00	0.	30	13	B98 0.23
8822	27A	5.	13.	809.	1518.	5	472.	0.01688	10.00	0.00	0.	30	13	B98 0.23
8822	28A	6.	15.	815.	1526.	5	165.	0.01688	10.00	0.00	0.	30	13	B98 0.23
8822	29B	36.	91.	36.	91.	0	0.	0.00000	0.00	0.00	0.	30	13	B98 0.23

 * CONFLUENCE Q'S *
 * 8822 30A TA 1163 QA 1525. QAB 1582. QB 57. 8822 30B TB 1154 QB 91. QBA 913. QA 822. *
 * 8822 30AB TAB 1162 QAB 1583. QA 1515. QB 68. *

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNPTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
8822	30AB	36.	91.	851.	1583.	5	1050.	0.02630	8.00	2.00	0.	30	0	B98 0.00

VENTURA COUNTY FLOOD CONTROL DISTRICT
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SUDDEN BARRANCA UPDATE TO 1982 STUDY Q100F DL/JL 3/03 STORM DAY 4

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNPTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
8822	1A	76.	169.	76.	169.	1	1550.	0.05940	0.00	0.00	0.	40	12	B98 0.00
8822	2A	58.	155.	134.	286.	1	1730.	0.04040	0.00	0.00	0.	30	10	B98 0.00
8822	3A	29.	93.	163.	272.	1	1250.	0.03200	0.00	0.00	0.	20	8	B98 0.00
8822	4A	82.	220.	245.	385.	1	600.	0.03330	0.00	0.00	0.	20	11	B98 0.00
8822	5A	0.	0.	245.	378.	0	0.	0.00000	0.00	0.00	0.	20	99	B98 0.00
8822	6B	72.	162.	72.	162.	4	650.	0.00170	6.00	0.00	0.	20	15	B98 0.10

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8822  7B  40.  89.  112.  245.  0  0.  0.00000  0.00  0.00  0.  30  14  B98  0.10
*****
*                                     CONFLUENCE Q'S                                     *
* 8822  8A  TA 1161 QA  378. QAB  611. QB  233.  8822  8B  TB 1158 QB  245. QBA  596. QA  350. *
*                                     8822  8AB TAB 1160 QAB  613. QA  375. QB  239.                                     *
*****
*                                     CONFLUENCE Q'S                                     *
* 8822  13A TA 1159 QA  718. QAB  796. QB  78.  8822  13B TB 1155 QB  84. QBA  672. QA  588. *
*                                     8822  13AB TAB 1159 QAB  796. QA  718. QB  78.                                     *
*****
*                                     CONFLUENCE Q'S                                     *
* 8822  18A TA 1162 QA  792. QAC  1084. QC  292.  8822  18C TC 1158 QC  371. QCA  1111. QA  740. *
*                                     8822  18AC TAC 1160 QAC  1129. QA  779. QC  349.                                     *
*****
*                                     CONFLUENCE Q'S                                     *
* 8822  23A TA 1162 QA  1198. QAB  1392. QB  194.  8822  23B TB 1157 QB  258. QBA  1164. QA  906. *
*                                     8822  23AB TAB 1161 QAB  1395. QA  1175. QB  220.                                     *
*****

```

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
8822	8AB	112.	245.	357.	613.	5	1300.	0.02750	8.00	0.00	0.	30	0	B98 0.00
8822	9A	41.	123.	398.	673.	5	950.	0.02750	8.00	0.00	0.	40	8	B98 0.23
8822	10A	31.	93.	429.	723.	5	950.	0.01700	8.00	0.00	0.	40	8	B98 0.23
8822	11A	7.	24.	436.	718.	0	0.	0.00000	0.00	0.00	0.	40	6	B98 0.00
8822	12B	37.	84.	37.	84.	4	150.	0.00800	3.50	0.00	0.	40	13	B98 0.23
8822	13AB	37.	84.	473.	796.	4	2500.	0.02500	6.50	0.00	0.	40	0	B98 0.00
8822	14A	21.	66.	494.	792.	0	0.	0.00000	0.00	0.00	0.	40	7	B98 0.10
8822	15C	50.	134.	50.	134.	4	2500.	0.02000	3.50	0.00	0.	40	10	B98 0.23
8822	16C	53.	136.	103.	250.	0	0.	0.00000	0.00	0.00	0.	30	12	B98 0.35
8822	17C	41.	142.	144.	380.	4	1200.	0.01200	5.75	0.00	0.	10	8	B98 0.23
8822	18AC	144.	371.	638.	1129.	5	1580.	0.02200	10.00	0.00	0.	10	0	B98 0.00
8822	19A	22.	58.	660.	1160.	5	1975.	0.01500	10.00	0.00	0.	30	11	B98 0.20
8822	20A	32.	84.	692.	1198.	0	0.	0.00000	0.00	0.00	0.	30	11	B98 0.23
8822	21B	55.	145.	55.	145.	4	1500.	0.01400	4.00	0.00	0.	30	11	B98 0.23
8822	22B	49.	122.	104.	258.	0	0.	0.00000	0.00	0.00	0.	30	12	B98 0.23
8822	23AB	144.	371.	638.	1129.	5	1580.	0.02200	10.00	0.00	0.	10	0	B98 0.00
8822	23B	22.	58.	660.	1160.	5	1975.	0.01500	10.00	0.00	0.	30	11	B98 0.20
8822	23B	32.	84.	692.	1198.	0	0.	0.00000	0.00	0.00	0.	30	11	B98 0.23
8822	23B	55.	145.	55.	145.	4	1500.	0.01400	4.00	0.00	0.	30	11	B98 0.23
8822	23B	49.	122.	104.	258.	0	0.	0.00000	0.00	0.00	0.	30	12	B98 0.23

VENTURA COUNTY FLOOD CONTROL DISTRICT

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VCRAT METHODS

MODIFIED RATIONAL METHOD HYDROLOGY / PC 2.21-950

STORM DAY 4

LOCATION	SUBAREA	AREA	SUBAREA	Q	TOTAL	AREA	TOTAL	Q	CONV	TYPE	CONV	LNGLTH	CONV	SLOPE	CONV	SIZE	CONV	Z	CONTROL	SOIL	Q	NAME	TC	RAIN	PCT	IMPV	
8822	23AB	104.		258.	796.		1395.	5			1200.	0.01330		12.00	0.00		0.	30	0	B98			0	B98	0.00		
8822	24C	0.		0.	0.		371.	0			0.	0.00000		0.00	0.00		0.	30	99	B98			99	B98	0.00		
8822	25A	0.		0.	796.		1388.	0			0.	0.00000		0.00	0.00		0.	30	99	B98			99	B98	0.00		
8822	26A	8.		19.	804.		1402.	5			337.	0.03233		10.00	0.00		0.	30	13	B98			13	B98	0.23		
8822	27A	5.		12.	809.		1407.	5			472.	0.01688		10.00	0.00		0.	30	13	B98			13	B98	0.23		
8822	28A	6.		14.	815.		1415.	5			165.	0.01688		10.00	0.00		0.	30	13	B98			13	B98	0.23		
8822	29B	36.		86.	36.		86.	0			0.	0.00000		0.00	0.00		0.	30	13	B98			13	B98	0.23		

CONFLUENCE Q'S																											
*	8822	30A	TA	1163	QA	1415.	QAB	1467.	QB	53.	8822	30B	TB	1154	QB	86.	QBA	829.	QA	744.	*						
*	8822 30AB TAB 1163 QAB 1467. QA 1415. QB 53.																										

LOCATION	SUBAREA	AREA	SUBAREA	Q	TOTAL	AREA	TOTAL	Q	CONV	TYPE	CONV	LNGLTH	CONV	SLOPE	CONV	SIZE	CONV	Z	CONTROL	SOIL	Q	NAME	TC	RAIN	PCT	IMPV	
8822	30AB	36.		86.	851.		1467.	5			1050.	0.02630		8.00	2.00		0.	30	0	B98			0	B98	0.00		
8822	31A	16.		50.	867.		1466.	0			0.	0.00000		0.00	0.00		0.	30	8	B98			8	B98	0.23		

B-8 – VCRAT 2.2 AND VCRAT 2.6 DATA FILES**B-8.1 VCRat 2.2 Partial Input File**

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005 48651 99E IMPORTED HYDROGRAPH FN=SAMPLE1.DAT VCRat 2 VER. 2.2
005 48651 105A OUTFLOW HYDROGRAPH AT THIS POINT
005 48651 170AB THIS HYDROGRAPH IS ADJUSTED FOR AREAL REDUCTION & FATTENING
005 48651 171AE INFLOW HYDROGRAPH TO DETENTION BASIN
005 48651 172A DETENTION BASIN ROUTING
005 48651 173A DATA LINE ALWAYS NEEDED IF END OF FILE FOLLOWS
999
007 48651 99E 4220 441158 3582200 4
008 5 0 0 100 91 200 216 300 288 400 343
008 10 500 387 600 433 700 502 800 604 900 756
008 15 1000 1047 1050 1410 1100 1804 1110 2087 1120 2245
008 20 1130 2451 1131 2494 1132 2532 1133 2568 1134 2601
008 25 1135 2633 1136 2669 1137 2707 1138 2744 1139 2782
008 30 1140 2820 1141 2857 1142 2890 1143 2923 1144 2956
008 35 1145 2988 1146 3022 1147 3057 1148 3096 1149 3138
008 40 1150 3177 1151 3191 1152 3209 1153 3298 1154 3428
008 45 1155 3502 1156 3546 1157 3570 1158 3582 1159 3582
008 50 1160 3568 1161 3545 1162 3514 1163 3477 1164 3437
008 55 1165 3398 1166 3362 1167 3329 1168 3300 1169 3279
008 60 1170 3266 1171 3258 1172 3253 1173 3249 1174 3245
008 65 1175 3242 1176 3238 1177 3234 1178 3230 1179 3225
008 70 1180 3220 1181 3219 1182 3218 1183 3216 1184 3214
008 75 1185 3212 1186 3210 1187 3211 1188 3217 1189 3251
008 80 1190 3289 1191 3301 1192 3300 1193 3293 1194 3285
008 85 1195 3277 1196 3266 1197 3247 1198 3220 1199 3214
008 90 1200 3206 1201 3194 1202 3181 1203 3141 1204 3089
008 95 1205 3044 1206 3006 1207 2972 1208 2941 1209 2913
008100 1210 2888 1211 2864 1212 2839 1213 2815 1214 2790
008105 1215 2766 1216 2741 1217 2717 1218 2693 1219 2669
008110 1220 2645 1221 2625 1222 2605 1223 2585 1224 2566
008115 1225 2546 1226 2527 1227 2508 1228 2490 1229 2472
008120 1230 2455 1231 2438 1232 2422 1233 2406 1234 2390
008125 1235 2373 1236 2356 1237 2340 1238 2324 1239 2309
008130 1240 2294 1241 2279 1242 2264 1243 2250 1244 2235
008135 1245 2221 1246 2207 1247 2194 1248 2180 1249 2167
008140 1250 2154 1251 2140 1252 2128 1253 2115 1254 2102
008145 1255 2089 1256 2077 1257 2065 1258 2053 1259 2041
008150 1260 2024 1261 2010 1262 1997 1263 1985 1264 1975
008155 1265 1964 1266 1954 1267 1944 1268 1933 1269 1922
008160 1270 1911 1271 1900 1272 1889 1273 1878 1274 1867

```

APPENDIX B

VCRAT METHODS

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008165 1275    1856 1276    1845 1277    1833 1278    1822 1279    1810
008170 1280    1798 1281    1787 1282    1775 1283    1763 1284    1751
008175 1285    1740 1286    1728 1287    1717 1288    1706 1289    1695
008180 1290    1685 1291    1675 1292    1665 1293    1656 1294    1647
008185 1295    1638 1296    1629 1297    1621 1298    1613 1299    1605
008190 1300    1598 1310    1546 1320    1453 1330    1361 1340    1271
008195 1350    1178 1360    1080 1370    988 1380    908 1390    845
008200 1400    795 1420    736 1440    676 1460    625 1500    570
999
006 48651 99E 010          B98                      G1A
006 48651 100A 020 23 4910B983 650 0140
006 48651 101A 020 40 2310B984 400 0100
006 48651 102A 020 14 62 8B98
006 48651 103A 020          099C99
-----ADDITIONAL SUBAREAS FROM 103A TO 169B-----
006 48651 169B 030015 2709B98
006 48651 170AB010          B98                      1
110
111          .877          6.0
110
006 48651 171AE030          B98                      1
-----RESERVOIR ROUTING SEQUENCE-----
006 48651 172A 030          099B98                      1
110
111 642.5
112 634.50 640.00 640.50 641.99 642.06 642.16 642.27 642.40 643.48
113 0.00 314.59 415.21 615.98 817.07 1018.61 1320.32 1522.33 1723.57
114 0.00 1905.00 2320.00 2830.00 3540.00 5550.00 5860.00 6870.00 8875.00
115
116 48651 172A 030          099B986 12000001 2 85          020022 20
110
006 48651 173A 030          099B98                      1 2

```

B-8.2 VCRat 2.2 Partial Output File

VENTURA COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY / PC 2.21-950
SAMPLE CALCULATION FN=SAMPLE1.DAT VCRat 2 VER. 2.2

LOCATION	SUBAREA		TOTAL	TOTAL	CONV	CONV	CONV	CONV	CONV	CONTROL	SOIL	STORM DAY 4	
	AREA	Q	AREA	Q	TYPE	LNTH	SLOPE	SIZE	Z			RAIN	PCT
48651 99E	4220.	3582.	4220.	3582.	0	0.	0.00000	0.00	0.00	0.	10	0	B98 0.00
48651 100A	49.	153.	49.	153.	3	650.	0.01400	0.00	0.00	0.	20	10	B98 0.23
48651 101A	23.	73.	72.	212.	4	400.	0.01000	4.75	0.00	0.	20	10	B98 0.40
48651 102A	62.	215.	134.	406.	0	0.	0.00000	0.00	0.00	0.	20	8	B98 0.14
48651 103A	0.	0.	134.	406.	0	0.	0.00000	0.00	0.00	0.	20	99	C99 0.00

APPENDIX B

VCRAT METHODS

```

-----ADDITIONAL OUTPUT FOR SUBAREAS 104A THROUGH 168B-----
48651 169B 27. 84. 264. 812. 0 0. 0.00000 0.00 0.00 0. 30 9 B98 0.15
48651 170AB 264. 812. 2317. 4057. 0 0. 0.00000 0.00 0.00 0. 10 0 B98 0.00
*****
* HYDROGRAPH FATTENED AT 170AB *
* INCOMING HYDROGRAPH PEAK = 4056.80 INCOMING HYDROGRAPH VOLUME = 898.16 AC.FT. *
* HYDROGRAPH ADJUSTMENT FACTOR = 0.87700 RUNOFF FACTOR = 6.00 IN. *
* ADJUSTED HYDROGRAPH PEAK = 3557.82 ADJUSTED HYDROGRAPH VOLUME = 787.69 AC.FT. *
* ADJ/FATTENED HYDROGRAPH PEAK = 3557.82 ADJ/FATTENED HYDROGRAPH VOLUME = 1157.95 AC.FT. *
*****
48651 170AB 0. 812. 2317. 3558. 0 0. 0.00000 0.00 0.00 0. 0 0 0 0.00
48651 171AE 4220. 3582. 6537. 6812. 0 0. 0.00000 0.00 0.00 0. 30 0 B98 0.00
48651 172A 0. 0. 6537. 6812. 0 0. 0.00000 0.00 0.00 0. 30 99 B98 0.00
*****
* RESERVOIR ROUTING AT 172A *
* INCOMING HYDROGRAPH PEAK = 6811.68 INCOMING HYDROGRAPH VOLUME = 2879.38 AC.FT. *
* HYDROGRAPH ADJUSTMENT FACTOR = 1.00000 *
* RESERVOIR INFLOW PEAK = 6811.68 TIME OF PEAK = 1171 VOLUME UNDER INFLOW HYDROGRAPH = 2879.38 AC.FT. *
* MAXIMUM ELEVATION = 642.03 TIME = 1249 SPILLAGE ELEVATION = 642.50 DIFFERENCE = -0.47 *
* NO SPILLAGE *
* RESERVOIR OUTFLOW PEAK = 3273.38 TIME OF PEAK = 1252 VOLUME UNDER OUTFLOW HYDROGRAPH = 2428.98 AC.FT. *
*****
48651 172A 0. 0. 6537. 3273. 6 1200. 0.00100 85.00 2.00 0. 30 99 B98 0.00
VENTURA COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY / PC 2.21-950
SAMPLE CALCULATION FN=SAMPLE1.DAT VCRat 2 VER. 2.2 STORM DAY 4
SUBAREA SUBAREA TOTAL TOTAL CONV CONV CONV CONV CONV CONTROL SOIL RAIN PCT
LOCATION AREA Q AREA Q TYPE LNGTH SLOPE SIZE Z Q NAME TC ZONE IMPV
48651 173A 0. 0. 6537. 3273. 0 0. 0.00000 0.00 0.00 0. 30 99 B98 0.00
MODIFIED RATIONAL METHOD HYDROLOGY / PC 2.21-950
OUTFLOW HYDROGRAPH AT THIS POINT
HYDROGRAPH AT 48651 105A STORM DAY 4 REDUCTION FACTOR = 1.000
TIME Q TIME Q TIME Q TIME Q TIME Q
0 0. 100 7. 200 8. 300 9. 400 9.
500 10. 600 10. 700 11. 800 12. 900 17.
1000 27. 1050 38. 1100 43. 1110 62. 1120 62.
1130 85. 1131 87. 1132 88. 1133 91. 1134 94.
1135 96. 1136 97. 1137 100. 1138 103. 1139 106.
1140 109. 1141 113. 1142 117. 1143 121. 1144 128.
1145 136. 1146 146. 1147 155. 1148 165. 1149 198.
1150 236. 1151 244. 1152 317. 1153 363. 1154 384.
1155 398. 1156 406. 1157 384. 1158 356. 1159 352.
1160 282. 1161 238. 1162 211. 1163 181. 1164 158.
1165 134. 1166 114. 1167 97. 1168 84. 1169 79.

```

APPENDIX B

VCRAT METHODS

1170	70.	1171	65.	1172	58.	1173	56.	1174	53.
1175	51.	1176	49.	1177	44.	1178	44.	1179	42.
1180	40.	1181	40.	1182	38.	1183	37.	1184	36.
1185	36.	1186	36.	1187	36.	1188	35.	1189	35.
1190	35.	1191	35.	1192	34.	1193	34.	1194	35.
1195	35.	1196	34.	1197	34.	1198	35.	1199	34.
1200	35.	1201	34.	1202	33.	1203	32.	1204	32.
1205	31.	1206	29.	1207	29.	1208	28.	1209	28.
1210	27.	1211	26.	1212	26.	1213	26.	1214	26.
1215	25.	1216	25.	1217	25.	1218	25.	1219	25.
1220	25.	1221	25.	1222	25.	1223	25.	1224	25.
1225	25.	1226	25.	1227	25.	1228	25.	1229	25.
1230	25.	1231	25.	1232	25.	1233	25.	1234	25.
1235	25.	1236	25.	1237	24.	1238	25.	1239	24.
1240	24.	1241	24.	1242	24.	1243	24.	1244	24.
1245	24.	1246	24.	1247	24.	1248	24.	1249	24.
1250	24.	1251	24.	1252	24.	1253	24.	1254	24.
1255	24.	1256	24.	1257	24.	1258	24.	1259	24.
1260	24.	1261	24.	1262	23.	1263	23.	1264	22.
1265	22.	1266	21.	1267	20.	1268	20.	1269	19.
1270	19.	1271	19.	1272	18.	1273	18.	1274	18.
1275	18.	1276	18.	1277	18.	1278	18.	1279	18.
1280	18.	1281	18.	1282	18.	1283	18.	1284	17.
1285	18.	1286	18.	1287	18.	1288	17.	1289	17.
1290	18.	1291	18.	1292	17.	1293	17.	1294	18.
1295	17.	1296	17.	1297	17.	1298	18.	1299	17.
1300	17.	1310	12.	1320	11.	1330	10.	1340	10.
1350	9.	1360	8.	1370	8.	1380	8.	1390	8.
1400	8.	1420	7.	1440	7.	1460	5.	1500	5.

 * HYDROGRAPH FATTENED AT 170AB *
 * HYDROGRAPH ADJUSTMENT FACTOR = 0.87700 RUNOFF FACTOR = 6.00 IN. *
 * ADJUSTED HYDROGRAPH PEAK = 3557.82 ADJUSTED HYDROGRAPH VOLUME = 1157.95 AC.FT. *

ADJUSTED HYDROGRAPH AFTER FATTENING

HYDROGRAPH AT 48651		170AB		STORM DAY 4		ADJUSTMENT FACTOR = 0.877			
TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	133.	200	157.	300	193.	400	225.
500	249.	600	288.	700	341.	800	403.	900	527.
1000	773.	1050	1002.	1100	1331.	1110	1438.	1120	1556.
1130	1726.	1131	1746.	1132	1766.	1133	1786.	1134	1807.
1135	1828.	1136	1849.	1137	1872.	1138	1895.	1139	1918.
1140	1942.	1141	1967.	1142	1993.	1143	2021.	1144	2049.
1145	2082.	1146	2116.	1147	2152.	1148	2191.	1149	2246.
1150	2304.	1151	2353.	1152	2445.	1153	2533.	1154	2622.

APPENDIX B

VCRAT METHODS

1155	2717.	1156	2799.	1157	2871.	1158	2945.	1159	3006.
1160	3085.	1161	3152.	1162	3228.	1163	3301.	1164	3356.
1165	3400.	1166	3442.	1167	3475.	1168	3501.	1169	3524.
1170	3542.	1171	3554.	1172	3558.	1173	3551.	1174	3536.
1175	3508.	1176	3469.	1177	3420.	1178	3361.	1179	3293.
1180	3218.	1181	3138.	1182	3062.	1183	2984.	1184	2907.
1185	2829.	1186	2749.	1187	2672.	1188	2598.	1189	2530.
1190	2468.	1191	2408.	1192	2350.	1193	2293.	1194	2239.
1195	2188.	1196	2139.	1197	2093.	1198	2050.	1199	2013.
1200	1979.	1201	1945.	1202	1911.	1203	1878.	1204	1846.
1205	1815.	1206	1784.	1207	1755.	1208	1728.	1209	1701.
1210	1677.	1211	1653.	1212	1631.	1213	1610.	1214	1589.
1215	1570.	1216	1551.	1217	1534.	1218	1517.	1219	1500.
1220	1484.	1221	1469.	1222	1454.	1223	1439.	1224	1425.
1225	1412.	1226	1399.	1227	1386.	1228	1374.	1229	1363.
1230	1352.	1231	1341.	1232	1331.	1233	1320.	1234	1310.
1235	1301.	1236	1291.	1237	1282.	1238	1272.	1239	1263.
1240	1253.	1241	1244.	1242	1234.	1243	1225.	1244	1216.
1245	1206.	1246	1197.	1247	1188.	1248	1179.	1249	1171.
1250	1163.	1251	1154.	1252	1146.	1253	1138.	1254	1130.
1255	1122.	1256	1115.	1257	1107.	1258	1100.	1259	1093.
1260	1086.	1261	1080.	1262	1073.	1263	1066.	1264	1060.
1265	1053.	1266	1046.	1267	1040.	1268	1033.	1269	1027.
1270	1021.	1271	1014.	1272	1008.	1273	1002.	1274	995.
1275	989.	1276	983.	1277	978.	1278	972.	1279	966.
1280	961.	1281	956.	1282	950.	1283	944.	1284	939.
1285	934.	1286	928.	1287	923.	1288	917.	1289	912.
1290	907.	1291	901.	1292	896.	1293	891.	1294	886.
1295	881.	1296	877.	1297	872.	1298	867.	1299	863.
1300	858.	1310	804.	1320	748.	1330	699.	1340	655.
1350	606.	1360	556.	1370	514.	1380	477.	1390	445.
1400	419.	1420	372.	1440	337.	1460	309.	1500	279.

```

*****
*
*                RESERVOIR ROUTING AT 172A                *
* INCOMING HYDROGRAPH PEAK = 6811.68 INCOMING HYDROGRAPH VOLUME = 2879.38 AC.FT. *
* HYDROGRAPH ADJUSTMENT FACTOR = 1.00000 *
* RESERVOIR INFLOW PEAK = 6811.68 TIME OF PEAK = 1171 VOLUME UNDER INFLOW HYDROGRAPH = 2879.38 AC.FT. *
* MAXIMUM ELEVATION = 642.03 TIME = 1249 SPILLAGE ELEVATION = 642.50 DIFFERENCE = -0.47 *
* NO SPILLAGE *
* RESERVOIR OUTFLOW PEAK = 3273.38 TIME OF PEAK = 1252 VOLUME UNDER OUTFLOW HYDROGRAPH = 2428.98 AC.FT. *
*****

```

B-8.3 VCRat 2.6 Partial Input File

```

PROGRAM VCRAT2.6PROJ
PROJECT 1 SAMPLE1
DESCRIP SAMPLE CALCULATION FN=SAMPLE1.DAT VCRat 2 VER. 2.2
JOB 48651 99 100
OUTPUT C:\Program Files\VCWPD Hydrology\VCRat 25\SAMPLE1.out
HYDROFILE C:\Program Files\VCWPD Hydrology\VCRat 25\SAMPLE1.hyd
TCOUTPUT C:\Program Files\VCWPD Hydrology\VCRat 25\SAMPLE1
SUBAREA 99 E 4
DESCRIP SAMPLE CALCULATION FN=SAMPLE1.DAT VCRat 2 VER. 2.2
MISCPARAMS N
IMPORT 4220 99E N
SUBAREA 100 A 0
SUBPARAMS 49 23 020 K 10
MISCPARAMS N
ROUTING 3 650 0.01400
SUBAREA 101 A 0
SUBPARAMS 23 40 020 K 10
MISCPARAMS N
ROUTING 4 400 0.01000 -1.00
SUBAREA 102 A 0
SUBPARAMS 62 14 020 K 8
MISCPARAMS N
SUBAREA 103 A 1
MISCPARAMS N
-----ADDITIONAL SUBAREAS BETWEEN 103A AND 169B-----
SUBAREA 169 B 0
SUBPARAMS 27 15 030 K 9
MISCPARAMS N
SUBAREA 170 A 2 B
DESCRIP THIS HYDROGRAPH IS ADJUSTED FOR AREAL REDUCTION & FATTENING
MISCPARAMS Y N
RESERVOIR 170 A NORES
ADJFACT 0.87700
FATRUN 6.00
SUBAREA 171 A 2 E
DESCRIP INFLOW HYDROGRAPH TO DETENTION BASIN
MISCPARAMS Y N
SUBAREA 172 A 1
DESCRIP DETENTION BASIN ROUTING
MISCPARAMS Y
RESERVOIR 172 A 642.50 643.48
ADJFACT 0.00000
STAGE 1 634.50 640.00 640.50 641.99 642.06 642.16 642.27 642.40
STORAGE 1 0.00 314.59 415.21 615.98 817.07 1018.61 1320.32 1522.33
OUTFLOW 1 0.00 1905.00 2320.00 2830.00 3540.00 5550.00 5860.00 6870.00
STAGE 9 643.48
STORAGE 9 1723.57
OUTFLOW 9 8875.00
ROUTING2 6 1200 0.00100 85
CHANLDATA2 2.00000 0.022 0.020
MAXVEL2 20 0
SUBAREA 173 A 1
DESCRIP DATA LINE ALWAYS NEEDED ONLY IF END OF FILE FOLLOWS
MISCPARAMS Y

```

APPENDIX B

VCRAT METHODS

B-8.4 VCRat 2.6 Partial Output File

(Note- # next to channel type indicates program has automatically changed conveyance type to accommodate flows)

Ventura County Watershed Protection District
 Modified Rational Method Hydrology Program (VCRat v2.6)

Modified Rational Model Results Report

Job: 48651 Project: SAMPLE1

Project Description

SAMPLE CALCULATION FN=SAMPLE1.DAT VCRAT2 VER. 2.2

VCRat version: 2.6.2009.7
 VCRain version: 200703
 DOS EXE version: PC 2.2-200809

Job: 48651 Project: SAMPLE1

Page: 2

Model Results

SUBAREA DATA AND RESULTS							ACCUMULATED DATA			ROUTING AFTER ACCUMULATION									
NODE ID	SOIL TYPE	RAIN ZONE	TC (MIN)	% IMP	AREA (AC)	FLOW (CFS)	AREA (AC)	FLOW (CFS)	TIME (MIN)	CHANNEL TYPE	LENGTH (FT)	SLOPE (FT/FT)	SIZE (FT)	H:V (Z)	N VALUES		VEL (FT/S)	DEPTH (FT)	
99E : SAMPLE CALCULATION FN=SAMPLE1.DAT VCRat 2 VER. 2.2																			
99E : Import Hydro: 99E																			
99E	---	---	--	--	4220	3582	4220	3582	1158	-----	----	-----	---	----	-----	----	----	---	---
100A	020	K100	10	23	49	153	49	153	1154	40' ROAD	650	0.01400	---	----	-----	----	----	---	---
101A	020	K100	10	40	23	73	72	212	1157	PIPE	400	0.01000	4.75	----	-----	----	----	---	---
102A	020	K100	8	14	62	215	134	406	1156	-----	----	-----	---	----	-----	----	----	---	---
103A	---	---	--	--	---	---	134	406	1156	-----	----	-----	---	----	-----	----	----	---	---
-----ADDITIONAL OUTPUT BETWEEN 103A AND 169B-----																			
169B	030	K100	9	15	27	84	264	812	1160	-----	----	-----	---	----	-----	----	----	---	---
170AB: THIS HYDROGRAPH IS ADJUSTED FOR AREAL REDUCTION & FATTENING																			

APPENDIX B

VCRAT METHODS

```

170AB --- --- -- -- 264 812 2317 4057 1172 -----
*****
* INCOMING HYDROGRAPH PEAK (cfs): 4056.80 VOLUME (acre-ft): 898.16 *
* HYDROGRAPH ADJUSTMENT FACTOR: 0.87700 *
* ADJUSTED HYDROGRAPH PEAK (cfs): 3557.82 VOLUME (acre-ft): 787.69 *
* RUNOFF FACTOR(in): 6.00 *
* FATTENED HYDROGRAPH PEAK (cfs): 3557.82 VOLUME (acre-ft): 1157.95 *
*****
170A --- --- -- -- --- --- 2317 3558 -----
171AE: INFLOW HYDROGRAPH TO DETENTION BASIN
171AE --- --- -- -- 4220 3582 6537 6812 1171 -----

```

Ventura County Watershed Protection District
 Modified Rational Method Hydrology Program (VCRat v2.6)

Job: 48651 Project: SAMPLE1

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Model Results

SUBAREA DATA AND RESULTS						ACCUMULATED DATA			ROUTING AFTER ACCUMULATION										
NODE ID	SOIL TYPE	RAIN ZONE	TC (MIN)	% IMP	AREA (AC)	FLOW (CFS)	AREA (AC)	FLOW (CFS)	TIME (MIN)	CHANNEL TYPE	LENGTH (FT)	SLOPE (FT/FT)	SIZE (FT)	H:V (Z)	N CHNL	VALUES SIDES	VEL (FT/S)	DEPTH (FT)	
172A : DETENTION BASIN ROUTING																			
172A						6537	6812	1171											

*	INCOMING HYDROGRAPH PEAK (cfs):					6811.68	VOLUME (acre-ft):			2879.38		*							
*	NO HYDROGRAPH ADJUSTMENT																		
*	NO HYDROGRAPH FATTENING																		
*	RESERVOIR INFLOW: PEAK (cfs):					6811.68 @ 1171	VOLUME (acre-ft):			2879.38		*							
*	MAXIMUM ELEVATION: STAGE (ft):					642.03 @ 1249	VOLUME (acre-ft):			730.89		*							
*	EMERGENCY SPILLWAY: ELEV (ft):					642.50	VOLUME (acre-ft):			1540.96		*							
*	DIFFERENCE: IN STAGE (ft):					-0.47	IN VOLUME (acre-ft):			810.07		*							
*	NO SPILL EXPECTED. PERCNT OF VOLUME REMAINING TO SPILLWAY: 52.6%																		
*	TOP OF DAM: ELEV (ft):					643.48	VOLUME (acre-ft):			1723.57		*							
*	DIFFERENCE IN STAGE (ft):					-1.45	IN VOLUME (acre-ft):			992.68		*							
*	NO OVERTOP EXPECTED. PERCNT OF VOLUME REMAINING TO TOP OF DAM: 57.6%																		
*	RESERVOIR OUTFLOW: PEAK (cfs):					3273.38 @ 1252	VOLUME (acre-ft):			2428.98		*							

172A						6537	3273	1252		CHANNELV	1200	0.00100	85	2.00	0.020	0.022	20	0	
173A : DATA LINE ALWAYS NEEDED ONLY IF END OF FILE FOLLOWS																			
173A						6537	3273	1256											

Warning Messages

TYPE	ERR NO	PROCEDURE	LOCATION	MESSAGE
------	--------	-----------	----------	---------

APPENDIX B

VCRAT METHODS

NO MESSAGES OR WARNINGS DETECTED

Ventura County Watershed Protection District
 Modified Rational Method Hydrology Program (VCRat v2.6)

Job: 48651 Project: SAMPLE1

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Hydrograph Printouts

HYDROGRAPH PRINTOUT AT: 105A

DESCRIPTION: OUTFLOW HYDROGRAPH AT THIS POINT

TOTAL AREA TO HYDROGRAPH: 134 acres
 HYDROGRAPH PEAK: 406 cfs
 TIME OF PEAK: 1156 minutes
 HYDROGRAPH VOLUME: 39.42 acre-ft

TIME (min)	FLOW (cfs)	TIME (min)	FLOW (cfs)	TIME (min)	FLOW (cfs)	TIME (min)	FLOW (cfs)	TIME (min)	FLOW (cfs)
0	0.	100	7.	200	8.	300	9.	400	9.
500	10.	600	10.	700	11.	800	12.	900	17.
1000	27.	1050	38.	1100	43.	1110	62.	1120	62.
1130	85.	1131	87.	1132	88.	1133	91.	1134	94.
1135	96.	1136	97.	1137	100.	1138	103.	1139	106.
1140	109.	1141	113.	1142	117.	1143	121.	1144	128.
1145	136.	1146	146.	1147	155.	1148	165.	1149	198.
1150	236.	1151	244.	1152	317.	1153	363.	1154	384.
1155	398.	1156	406.	1157	384.	1158	356.	1159	352.
1160	282.	1161	238.	1162	211.	1163	181.	1164	158.
1165	134.	1166	114.	1167	97.	1168	84.	1169	79.
1170	70.	1171	65.	1172	58.	1173	56.	1174	53.
1175	51.	1176	49.	1177	44.	1178	44.	1179	42.
1180	40.	1181	40.	1182	38.	1183	37.	1184	36.
1185	36.	1186	36.	1187	36.	1188	35.	1189	35.
1190	35.	1191	35.	1192	34.	1193	34.	1194	35.
1195	35.	1196	34.	1197	34.	1198	35.	1199	34.
1200	35.	1201	34.	1202	33.	1203	32.	1204	32.
1205	31.	1206	29.	1207	29.	1208	28.	1209	28.
1210	27.	1211	26.	1212	26.	1213	26.	1214	26.
1215	25.	1216	25.	1217	25.	1218	25.	1219	25.
1220	25.	1221	25.	1222	25.	1223	25.	1224	25.
1225	25.	1226	25.	1227	25.	1228	25.	1229	25.
1230	25.	1231	25.	1232	25.	1233	25.	1234	25.
1235	25.	1236	25.	1237	24.	1238	25.	1239	24.
1240	24.	1241	24.	1242	24.	1243	24.	1244	24.
1245	24.	1246	24.	1247	24.	1248	24.	1249	24.

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1250	24.	1251	24.	1252	24.	1253	24.	1254	24.
1255	24.	1256	24.	1257	24.	1258	24.	1259	24.
1260	24.	1261	24.	1262	23.	1263	23.	1264	22.
1265	22.	1266	21.	1267	20.	1268	20.	1269	19.
1270	19.	1271	19.	1272	18.	1273	18.	1274	18.
1275	18.	1276	18.	1277	18.	1278	18.	1279	18.
1280	18.	1281	18.	1282	18.	1283	18.	1284	17.
1285	18.	1286	18.	1287	18.	1288	17.	1289	17.
1290	18.	1291	18.	1292	17.	1293	17.	1294	18.
1295	17.	1296	17.	1297	17.	1298	18.	1299	17.
1300	17.	1310	12.	1320	11.	1330	10.	1340	10.
1350	9.	1360	8.	1370	8.	1380	8.	1390	8.
1400	8.	1420	7.	1440	7.	1460	5.	1500	5.

B-9 YIELD ADJUSTMENT EXAMPLE

This section provides an example of how to do a yield adjustment in VCRat to make sure a detention basin design hydrograph has a reasonable volume. It is necessary to do this because MRM hydrographs from VCRat generally have volumes that are less than expected from an NRCS CN analysis except where the 100-yr 24-hr rainfall depth is about 6 in or less such as in the vicinity of Oxnard. The section describes the procedure using ArcMap or some similar GIS program. Other programs can also be used, or hand techniques. The procedure is the following:

1. Use VCRatX.X to produce an inflow hydrograph for the design storm and tributary watershed at the detention basin location. For this example the yield is calculated for the Mt Sinai Basin location node 55 in the official 100-yr Calleguas Model with residential and undeveloped land uses.

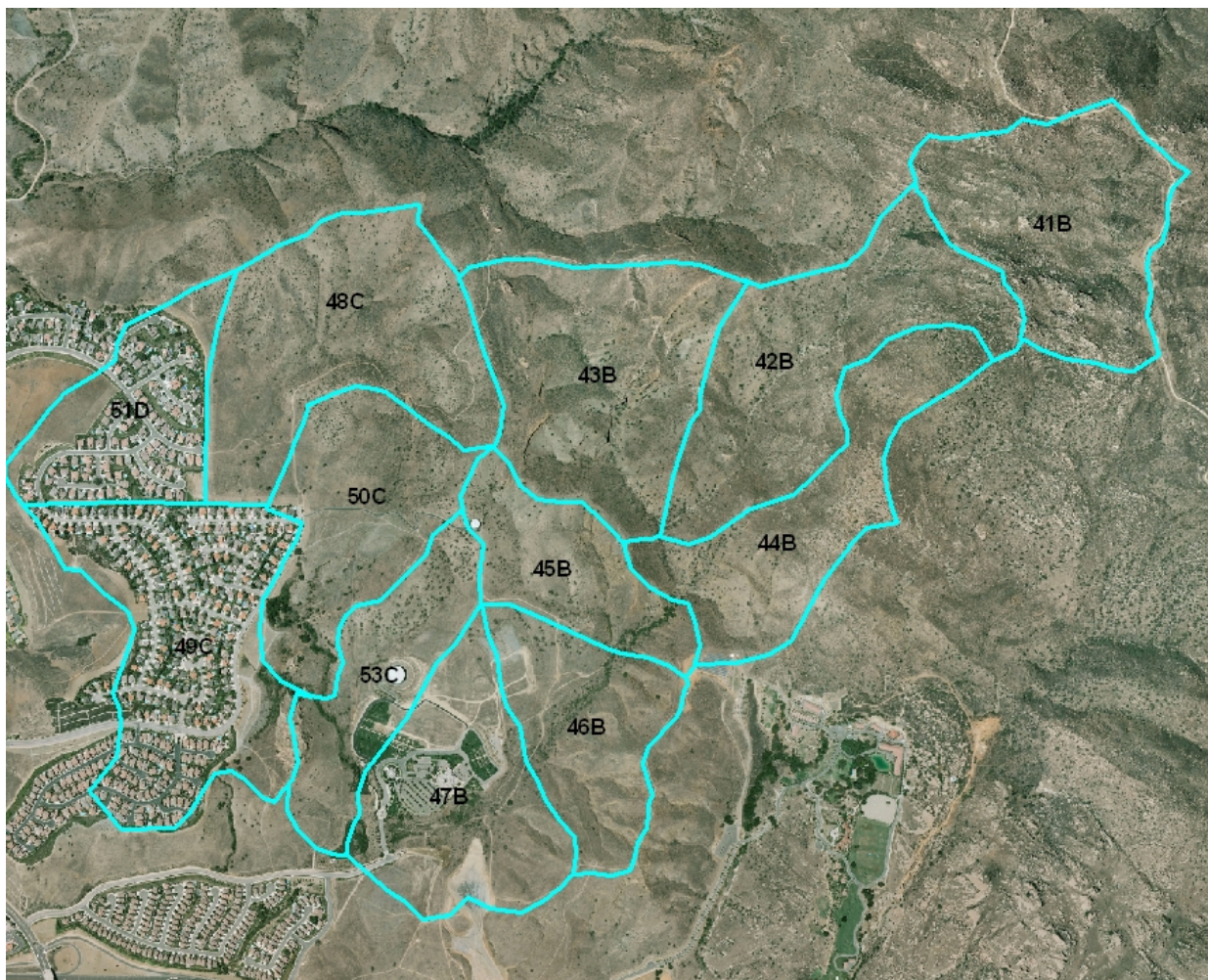


Figure B-9.1 Mt Sinai Basin Watershed Subareas

2. Use the shapefiles or maps in Appendix E to find the average soil type in the watershed by clipping the soils map with the watershed boundary, recalculating the clipped areas, and finding the weighted average soil type (1.45 for this example). If the rainfall contours have regularly spaced intervals across the watershed, find the total average design storm rainfall depth (N-yr 24-hour) at the

watershed centroid. If the contours are irregularly spaced, used GIS techniques to assign an area to each contour and calculate the weighted average rainfall.

Soil Data Summary

SOIL NO.	AREA, AC.	WTD SOIL
1	713.8	0.785
2	19.2	0.042
2	38.5	0.085
2	39.6	0.087
3	59.3	0.196
3	9.3	0.031
3	2.3	0.008
7	27.9	0.214
Total	909.8	1.447

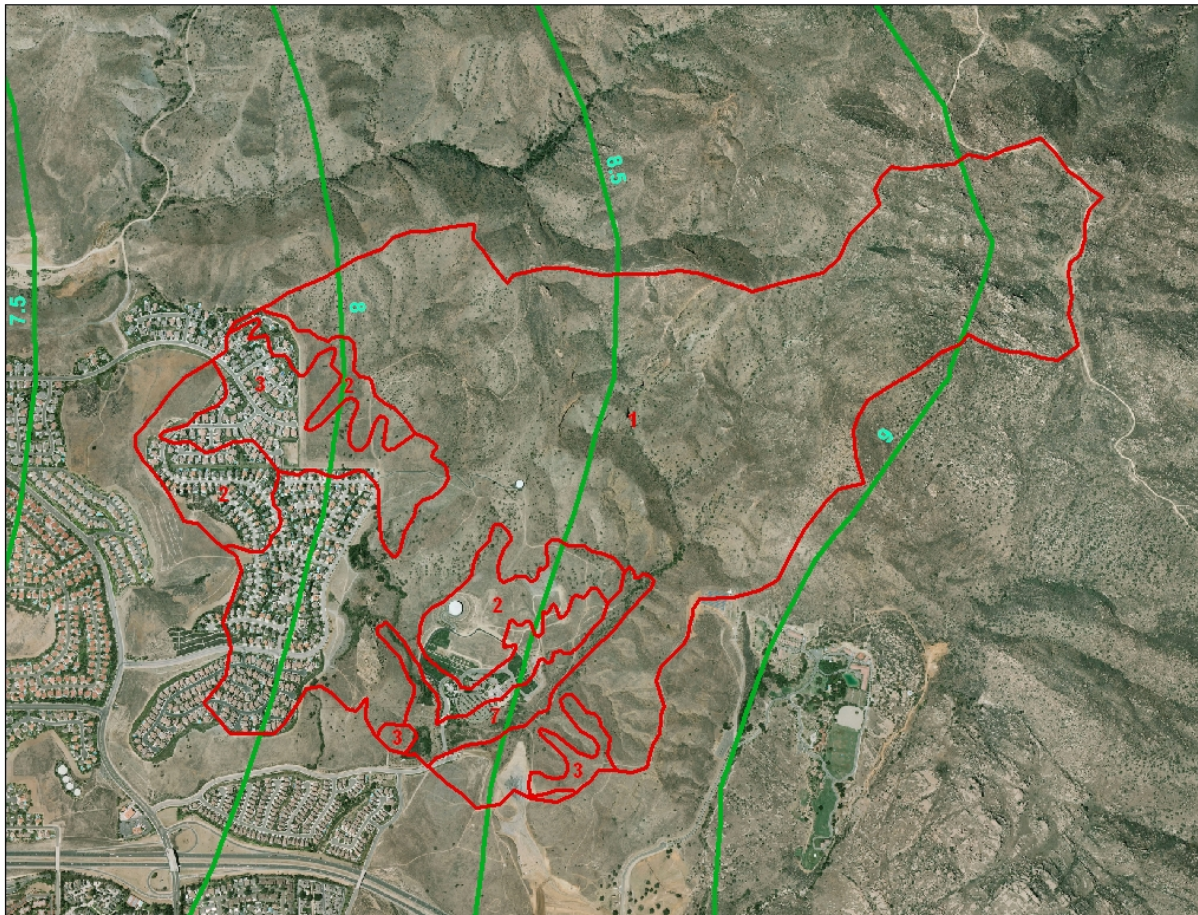


Figure B-9.2 Soils and 100-yr 24-hr Rain Map

- Clip the General Plan land use shapefile with the watershed boundary to obtain a list of land uses and associated areas. Assign appropriate NRCS CNs from Exhibits 14a and b to each land use and use

linear interpolation to calculate CNs for the areally weighted soil type for each land use. Calculate the areally-weighted CN for the watershed.

Land Use General	Land Use Specific	Exhibit 14 Land Use	Area Ac.	CN Soil 1	CN Soil 2	CN Soil 1.45	Wtd CN
OPEN SPACE AND RECREATION	Community Park	Open Space-Park	54	80	76	77.79	4.62
OPEN SPACE AND RECREATION	Undeveloped	Open Brush Fair	548	61	58	59.34	35.85
OPEN SPACE AND RECREATION	Open Space (1 du/40 ac)	Open Brush Fair	13	61	58	59.34	0.82
RESIDENTIAL 5 - 11.99 UNITS PER ACRE	Medium Density (4.5 to 15 du/ac)	Residential 1/8 ac lot	11	92	90	90.89	1.06
RESIDENTIAL 5 - 11.99 UNITS PER ACRE	Low Density (2 to 4.5 du/ac)	Residential 1/2 ac lot	282	86	82	83.79	26.05
Sum			907				68.40

- Use Exhibit 13 or the NRCS equation to find the watershed yield in inches using the weighted CN from Step 3 and the total design storm rainfall depth from Step 2.

	CN AMC II Yield
100-yr 24-hr Precip. P (in)	8.4
Wtd CN:	68.40
Potential Abstraction $S = (1000/CN) - 10$:	4.62
Initial Abstraction $I_a = 0.2S$:	0.92
Yield $= (P - 0.2S)^2 / (P + 0.8S)$: (in)	4.62

- Enter this as a yield adjustment factor in the model data input file above for VCRatX.X. In 2.6 the user can also enter the rainfall and curve number directly and the program will calculate the resultant yield following the assumption that the initial abstraction is 0.2 times the total abstraction. The yield information is entered by editing the reservoir routing portion in the appropriate subarea editing window.

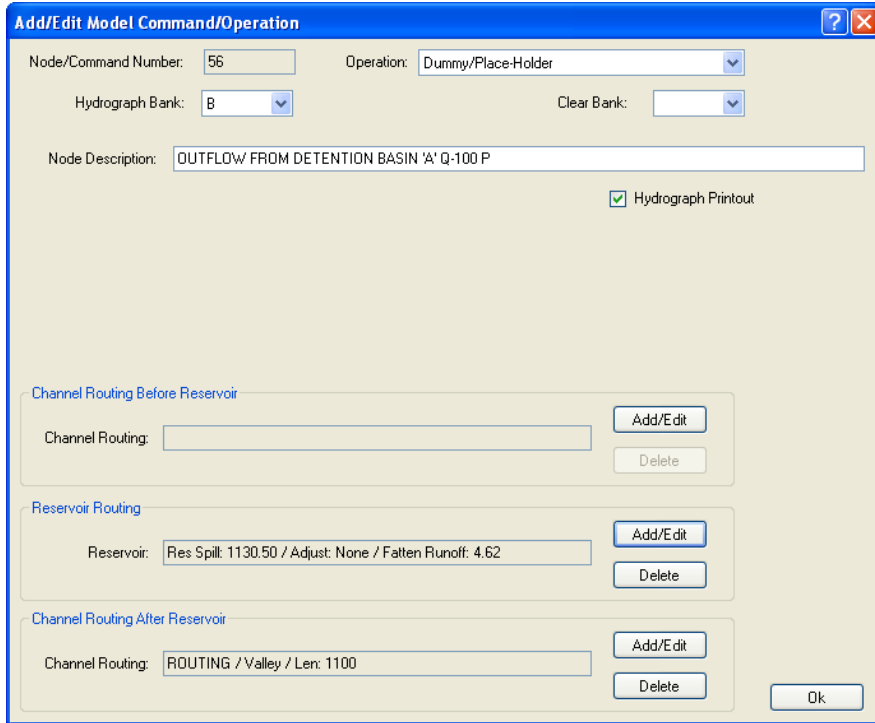


Figure B-9.3 Subarea Editing Window, VCRat2.6

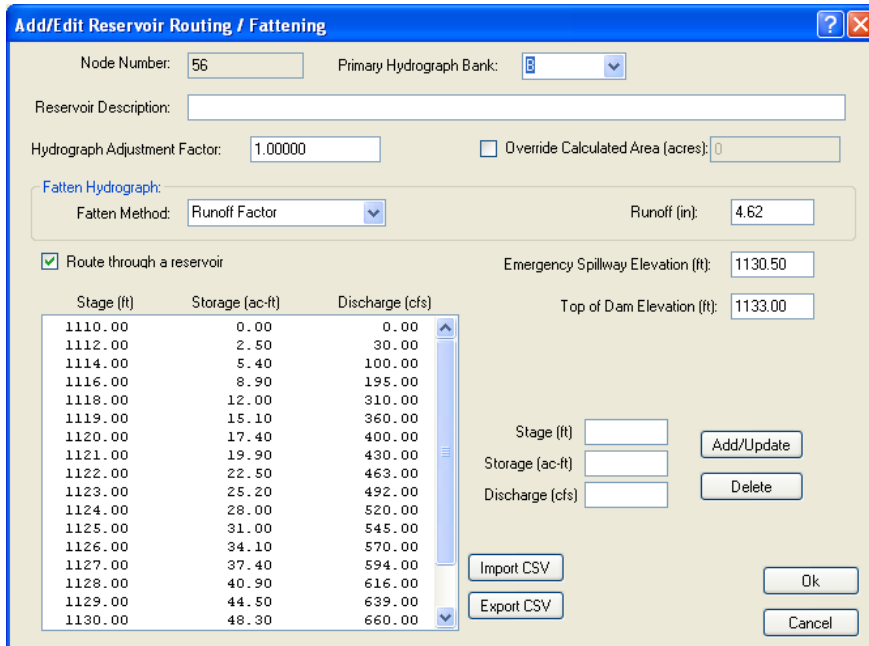


Figure B-9.4 Reservoir Routing Editing Window, VCRat2.6

- As a check, the volume of the output hydrograph should equal the yield adjustment factor times the area of the tributary watershed after the units are converted. Sample input and output files for a VCRat2.2 run are shown below.

- If desired, GIS techniques can be used to find each unique soil number and land use polygon, assigning CNs to the results, and calculating the average weighted CN for the watershed. Comparisons of the detailed approach and using the weighted soil number approach described above have shown relatively small differences between the two methods. It takes additional work to assign CNs to the unique combinations and therefore is not required for District studies.

B-9.1 Sample Yield Adjustment File, VCRat2.2

```

005 15031 1A CALLEGUAS CRK.Q100P YIELD ADJUSTMENT EXAMPLE 09/2010
005 15031 22A ARROYO SIMI AT KUEHNER DR.(HEADWATERS) Q100P (MDP '88)
005 15031 53C INFLOW TO DETENTION BASIN 'A' NORTH W/ AR APPLIED 100P
005 15031 56B OUTFLOW FROM DETENTION BASIN 'A' Q-100 P
005 15031 58B WHITE OAK CHL.PRIOR TO JCT.W/HUMMINGBIRD Q100P
005 15031 87BCWHITE OAK CHL.AFTER JCT.W/HUMMINGBIRD Q100P
005 15031 88B WHITE OAK PRIOR JCT.W/ARROYO SIMI Q-100P
005 15031 96ABARROYO SIMI AFTER JCT.W/WHITE OAK CHANNEL Q100P
005 15031 97A LAST POINT IN YIELD ADJUSTMENT EXAMPLE
999
007 15031 51D 55 141110 30200 4
008 5 0 0 100 3 200 3 300 4 400 4
008 10 500 4 600 4 700 4 800 5 900 8
008 15 1000 12 1050 18 1100 20 1110 30 1120 29
008 20 1130 30 1131 30 1132 30 1133 30 1134 30
008 25 1135 30 1136 30 1137 30 1138 30 1139 30
008 30 1140 30 1141 30 1142 30 1143 30 1144 30
008 35 1145 30 1146 30 1147 30 1148 30 1149 30
008 40 1150 30 1151 30 1152 30 1153 30 1154 30
008 45 1155 30 1156 30 1157 30 1158 30 1159 30
008 50 1160 30 1161 30 1162 30 1163 30 1164 30
008 55 1165 30 1166 30 1167 30 1168 30 1169 30
008 60 1170 29 1171 27 1172 26 1173 25 1174 24
008 65 1175 21 1176 21 1177 20 1178 20 1179 19
008 70 1180 17 1181 18 1182 17 1183 17 1184 17
008 75 1185 16 1186 16 1187 16 1188 16 1189 17
008 80 1190 16 1191 16 1192 16 1193 16 1194 16
008 85 1195 16 1196 16 1197 16 1198 16 1199 16
008 90 1200 14 1201 16 1202 15 1203 15 1204 15
008 95 1205 12 1206 13 1207 13 1208 13 1209 12
008100 1210 11 1211 11 1212 11 1213 11 1214 11
008105 1215 11 1216 11 1217 12 1218 11 1219 11
008110 1220 12 1221 12 1222 12 1223 11 1224 11
008115 1225 12 1226 12 1227 11 1228 11 1229 12
008120 1230 11 1231 11 1232 11 1233 11 1234 11
008125 1235 11 1236 11 1237 11 1238 11 1239 11
008130 1240 11 1241 11 1242 11 1243 11 1244 11
008135 1245 11 1246 11 1247 11 1248 11 1249 11
008140 1250 11 1251 11 1252 11 1253 11 1254 11
008145 1255 11 1256 11 1257 11 1258 11 1259 11
008150 1260 11 1261 11 1262 10 1263 10 1264 10
008155 1265 10 1266 9 1267 9 1268 9 1269 9
008160 1270 8 1271 8 1272 8 1273 8 1274 8
008165 1275 8 1276 8 1277 8 1278 8 1279 8
008170 1280 8 1281 8 1282 8 1283 8 1284 8
008175 1285 8 1286 8 1287 8 1288 8 1289 8
008180 1290 8 1291 8 1292 8 1293 8 1294 8
008185 1295 8 1296 8 1297 8 1298 8 1299 8
008190 1300 8 1310 4 1320 4 1330 4 1340 4
008195 1350 3 1360 3 1370 3 1380 3 1390 3
008200 1400 3 1420 2 1440 2 1460 0 1500 0
007 15031 54C 383 481162 706200 4
008 5 0 0 100 5 200 13 300 17 400 20
008 10 500 23 600 27 700 32 800 40 900 59
008 15 1000 100 1050 156 1100 222 1110 252 1120 282
    
```

008 20 1130	312 1131	315 1132	319 1133	323 1134	326
008 25 1135	330 1136	334 1137	338 1138	342 1139	347
008 30 1140	351 1141	355 1142	358 1143	361 1144	365
008 35 1145	369 1146	373 1147	377 1148	381 1149	386
008 40 1150	391 1151	397 1152	420 1153	455 1154	491
008 45 1155	527 1156	575 1157	617 1158	651 1159	676
008 50 1160	693 1161	703 1162	706 1163	703 1164	695
008 55 1165	682 1166	664 1167	643 1168	619 1169	596
008 60 1170	574 1171	552 1172	532 1173	519 1174	505
008 65 1175	492 1176	478 1177	465 1178	453 1179	441
008 70 1180	429 1181	418 1182	408 1183	400 1184	398
008 75 1185	396 1186	394 1187	391 1188	389 1189	387
008 80 1190	384 1191	382 1192	379 1193	377 1194	374
008 85 1195	371 1196	369 1197	366 1198	364 1199	361
008 90 1200	359 1201	356 1202	353 1203	350 1204	346
008 95 1205	343 1206	340 1207	336 1208	333 1209	329
008100 1210	326 1211	323 1212	319 1213	316 1214	313
008105 1215	309 1216	306 1217	303 1218	300 1219	296
008110 1220	291 1221	287 1222	283 1223	279 1224	275
008115 1225	271 1226	268 1227	264 1228	261 1229	257
008120 1230	254 1231	251 1232	248 1233	246 1234	243
008125 1235	240 1236	238 1237	235 1238	233 1239	229
008130 1240	222 1241	217 1242	212 1243	208 1244	204
008135 1245	201 1246	198 1247	196 1248	194 1249	192
008140 1250	190 1251	188 1252	187 1253	185 1254	184
008145 1255	182 1256	181 1257	180 1258	178 1259	177
008150 1260	176 1261	175 1262	174 1263	173 1264	172
008155 1265	171 1266	169 1267	168 1268	167 1269	165
008160 1270	164 1271	162 1272	161 1273	159 1274	158
008165 1275	156 1276	154 1277	153 1278	151 1279	150
008170 1280	149 1281	147 1282	146 1283	145 1284	143
008175 1285	142 1286	141 1287	140 1288	139 1289	138
008180 1290	137 1291	136 1292	135 1293	134 1294	133
008185 1295	132 1296	132 1297	131 1298	130 1299	130
008190 1300	129 1310	122 1320	110 1330	100 1340	93
008195 1350	87 1360	81 1370	75 1380	70 1390	66
008200 1400	63 1420	58 1440	54 1460	50 1500	46

999

006 15031 001A 010	099B98				G1 7
006 15031 002A 010010	8313B981	115000800			
006 15031 003A 010000	7611B98				
006 15031 004A 010000	5113B981	40000500			
006 15031 005B 010000	4512B981	50000800			
006 15031 006AB010	B981	220000600			1
006 15031 007A 010000	6011B981	30000300			
006 15031 008B 010000	5914B981	205003000			
006 15031 009B 010000	6410B98				
006 15031 010B 010000	8113B981	60003000			
006 15031 011B 010000	5212B981	170000500			
006 15031 012AB010	B981	60000250			1
006 15031 013A 010005	8212B981	110000180			
006 15031 014B 010000	6314B98				
006 15031 015B 010000	6312B981	90001500			
006 15031 016B 010000	2710B981	90000450			
006 15031 017B 010002	6913B985	76000130	000 800		
006 15031 018AB010	B985	20000200	000 1000		1
006 15031 019A 050005	3809B985	100000200	000 1000		
006 15031 020B 010007	7113B981	90000750			
006 15031 021AB010	B985	90000100	000 1400		1
006 15031 022A 050050	6912B985	50000100	000 1400		1
006 15031 023B 010000	7512B981	280002000			
006 15031 024B 010000	13117B981	70000200			
006 15031 025C 010000	6814B981	110000150			
006 15031 026C 010009	8412B981	45000200			
006 15031 027C 010007	2411B98				

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006	15031	082CD010	B985	12000015	100	1000			1
006	15031	083D 010000	5212B984	255000200					
006	15031	084D 040023	6411B98						
006	15031	085CD010	B985	180000020	100	1300			1
006	15031	086C 040020	7412B98						
006	15031	087BC010	B98						11
006	15031	088B 070023	4210B985	20000060	000	1800			1
006	15031	089C 050023	2809B984	60000020					
006	15031	090C 050023	8514B985	22000730	200	400		035035	
006	15031	091BC010	B985	40000060	000	1800			
006	15031	092B 040023	4912B985	45000060	000	1800			
006	15031	093C 020011	7313B984	32000012					
006	15031	094C 040023	9413B98						
006	15031	095BC010	B98						1
006	15031	096AB010	B985	22500010		2200			11
006	15031	097A 010	099B98						1 2□

APPENDIX B

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B-9.2 Partial Sample Yield Adjustment Output File, VCRat2.2

```

VENTURA COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY / PC 2.21-952
CALLEGUAS CRK.Q100P YIELD ADJUSTMENT EXAMPLE 09/2010
STORM DAY 4
SUBAREA SUBAREA TOTAL TOTAL CONV CONV CONV CONV CONTROL SOIL RAIN PCT
LOCATION AREA Q AREA Q TYPE LNGTH SLOPE SIZE Z Q NAME TC ZONE IMPV
*****
* CONFLUENCE Q'S
* 15031 52C TC 1160 QC 725. QCD 755. QD 30. 15031 52D TD 1120 QD 30. QDC 153. QC 123.
* 15031 52CD TCD 1160 QCD 755. QC 725. QD 30.
*****
SUBAREA SUBAREA TOTAL TOTAL CONV CONV CONV CONV CONTROL SOIL RAIN PCT
LOCATION AREA Q AREA Q TYPE LNGTH SLOPE SIZE Z Q NAME TC ZONE IMPV
15031 52CD 55. 30. 330. 755. 1 1550. 0.30200 0.00 0.00 0. 20 0 B98 0.00
15031 53C 52. 119. 382. 861. 0 0. 0.00000 0.00 0.00 0. 20 16 B98 0.00
15031 54C 383. 706. 383. 706. 0 0. 0.00000 0.00 0.00 0. 20 0 B98 0.00
*****
* CONFLUENCE Q'S
* 15031 55B TB 1166 QB 1024. QBC 1688. QC 664. 15031 55C TC 1162 QC 706. QCB 1637. QB 931.
* 15031 55BC TBC 1165 QBC 1704. QB 1022. QC 682.
*****
SUBAREA SUBAREA TOTAL TOTAL CONV CONV CONV CONV CONTROL SOIL RAIN PCT
LOCATION AREA Q AREA Q TYPE LNGTH SLOPE SIZE Z Q NAME TC ZONE IMPV
15031 55BC 383. 706. 911. 1704. 4 443. 0.06460 7.00 0.00 0. 20 0 B98 0.00
15031 56B 0. 0. 911. 1704. 0 0. 0.00000 0.00 0.00 0. 20 99 B98 0.00
*****
* HYDROGRAPH FATTENED AT 56B
* INCOMING HYDROGRAPH PEAK = 1703.73 INCOMING HYDROGRAPH VOLUME = 302.05 AC.FT.
* HYDROGRAPH ADJUSTMENT FACTOR = 1.00000 RUNOFF FACTOR = 4.62 IN.
* ADJUSTED HYDROGRAPH PEAK = 1703.73 ADJUSTED HYDROGRAPH VOLUME = 302.05 AC.FT.
* ADJ/FATTENED HYDROGRAPH PEAK = 1703.73 ADJ/FATTENED HYDROGRAPH VOLUME = 350.58 AC.FT.
*****
* RESERVOIR ROUTING AT 56B
* INCOMING HYDROGRAPH PEAK = 1703.73 INCOMING HYDROGRAPH VOLUME = 350.58 AC.FT.
* HYDROGRAPH ADJUSTMENT FACTOR = 1.00000
* RESERVOIR INFLOW PEAK = 1703.73 TIME OF PEAK = 1165 VOLUME UNDER INFLOW HYDROGRAPH = 350.58 AC.FT.
* MAXIMUM ELEVATION = 1132.43 TIME = 1182 SPILLAGE ELEVATION = 1130.50 DIFFERENCE = +1.93
* SPILLED FROM 1169 TO 1220 FOR 52 MINUTES
* RESERVOIR OUTFLOW PEAK = 963.53 TIME OF PEAK = 1182 VOLUME UNDER OUTFLOW HYDROGRAPH = 331.85 AC.FT.
*****
15031 56B 0. 0. 911. 964. 2 1100. 0.02277 0.00 0.00 0. 20 99 B98 0.00

```

B-10 MULTIPLE YIELD ADJUSTMENT EXAMPLE

It is infrequently necessary to apply yield adjustments more than once to a drainage line to get final results at a study location. This occurs most often when the study site has a series of detention basins where the inflow to an upstream basin has already been “fattened”. In this example, assume that a regional basin is needed at node 95BC downstream of the Mt Sinai Basin shown in Section B-8. For the next downstream basin, the fattening factor is calculated by the following:

1. Calculate the yield of the outflow hydrograph from the upstream basin. The VCRat output file at node 56B in Section B-8.2 shows that the basin outflow yield was 331.85 af for a tributary area of 911 ac.
2. Calculate the yield of the tributary watershed downstream of the upper basin that provides flow to the downstream basin. Use the same approach described in Section B-8 for the area downstream of 56B to 95BC.

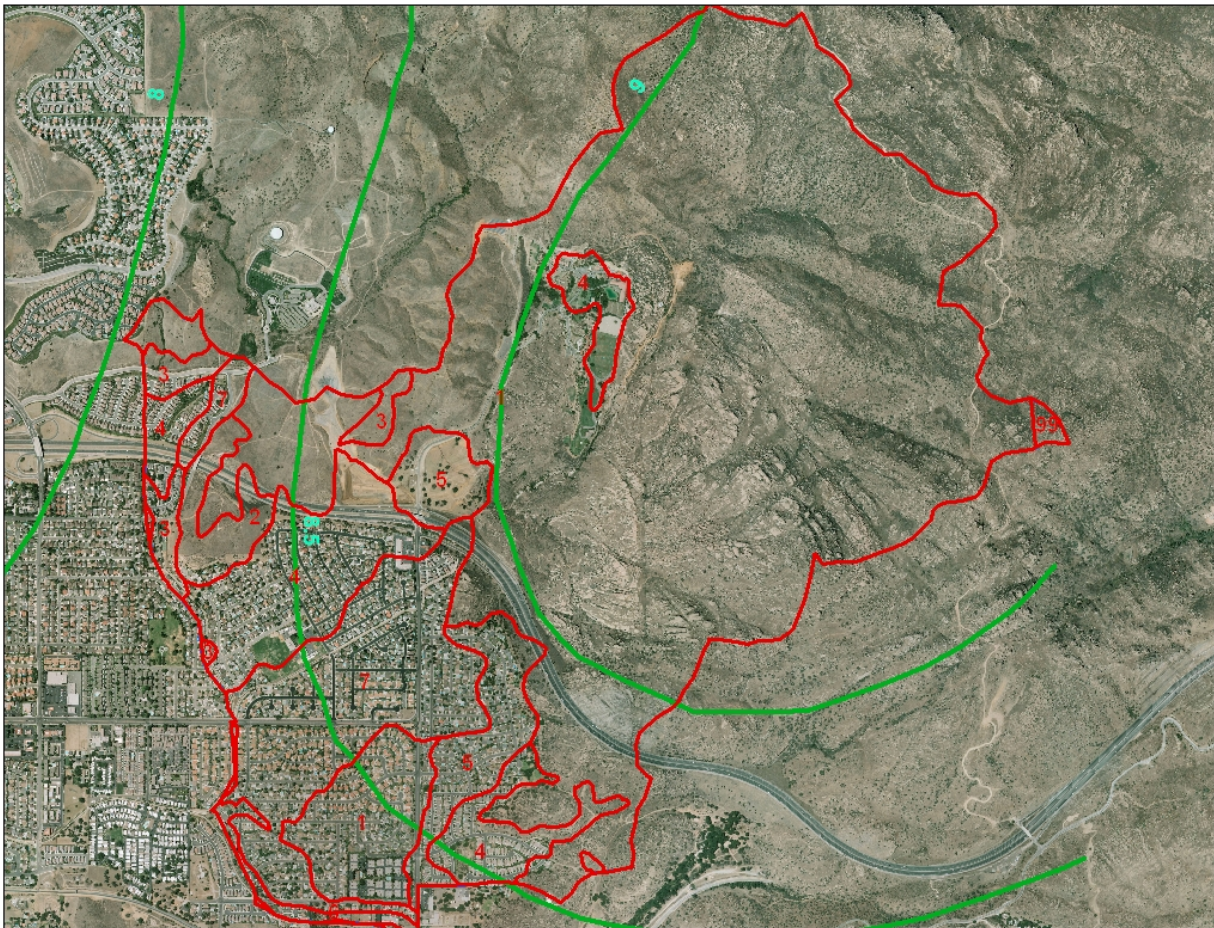


Figure B-10.1 Soil and 100-yr 24-hr Rain Map, Nodes 57 through 95, Calleguas VCRat Model

Soil Data Summary

	Summary	Wtd Soil
1	1,248.90	0.702
2	32.87	0.037
3	32.07	0.054
4	180.22	0.405
5	85.29	0.240
6	13.74	0.046
7	187.24	0.736
Total	1,780	2.22

Calculated Weighted CN AMC II			Wtd Soil Type				2.22
Land Use General	Land Use Specific	Exhibit 14 Land Use	Area Ac.	CN Soil 2	CN Soil 3	CN Soil 2.22	Wtd CN
COMMERCIAL	Commercial General	Comm	3	95	93	93.44	0.170
INDUSTRIAL	Light Industrial	Industrial	5	97	96	96.22	0.267
OPEN SPACE AND RECREATION	Community Park	Park	81	76	71	72.10	3.293
OPEN SPACE AND RECREATION	Open Space (1 du/40 ac)	Open Brush Fair	1,038	58	54	54.88	32.002
PUBLIC FACILITIES AND INSTITUTIONS	Elementary School	Public	13	86	84	84.44	0.631
RESIDENTIAL <= 1 UNIT PER ACRE	None provided	Residential 1/ac	74	80	76	76.88	3.191
RESIDENTIAL >= 12 UNITS PER ACRE	High Density (15 to 30 du/ac)	Condo	19	92	90	90.44	0.950
RESIDENTIAL 1 - 4.99 UNITS PER ACRE	Medium Density Residential (4 DU/AC)	Residential 1/4 ac lot	287	84	81	81.66	13.176
RESIDENTIAL 5 - 11.99 UNITS PER ACRE	Low Density (2 to 4.5 du/ac)	Residential 1/3 ac lot	144	82	79	79.66	6.433
RESIDENTIAL 5 - 11.99 UNITS PER ACRE	Medium Density (4.5 to 15 du/ac)	Residential 1/8 ac lot	70	90	89	89.22	3.514
TRANSPORTATION AND UTILITIES	Roadway	Transportation	46	92	91	91.22	2.334
		SUMS	1,780				65.96

	Yield Calculations
Precip. P (in) =	9
Wtd CN:	65.96
Potential Abstraction S =(1000/CN) -10:	5.16
Initial Abstraction I _a =0.2S:	1.03
Yield =(P-0.2S)2/(P+0.8S): (in)	4.84

- Sum the two yields, and convert that volume in af to inches by dividing the yield volume by the total tributary area upstream of the second basin and converting the units. Use 4.68 in as the fattening factor in the VCRat simulations. You will also have to apply the correct AR factor to the hydrograph applicable for the additional 1,780 ac below the upstream basin. This topic is explored more in the next example.

Combined Yield			
Watershed	Area Ac.	Volume af	Inches
Mt Sinai Outflow	911	331.85	4.37
Below Mt Sinai	1,780	717.34	4.84
Total	2,691	1,049	4.68

B-11 MULTIPLE AREAL REDUCTION EXAMPLE

It is sometimes necessary to apply areal reduction more than once to a drainage line to get final AR results at a study location. This occurs often when the study site is located downstream from a detention basin where the inflow to the basin has already had AR applied so that the basin outflow peak is correctly calculated. As an example the AR factor will be calculated for the tributary watershed ending at node 440A below Runkle Dam in Simi Valley where an AR factor was previously applied at 424A prior to basin routing. The watershed is shown below.

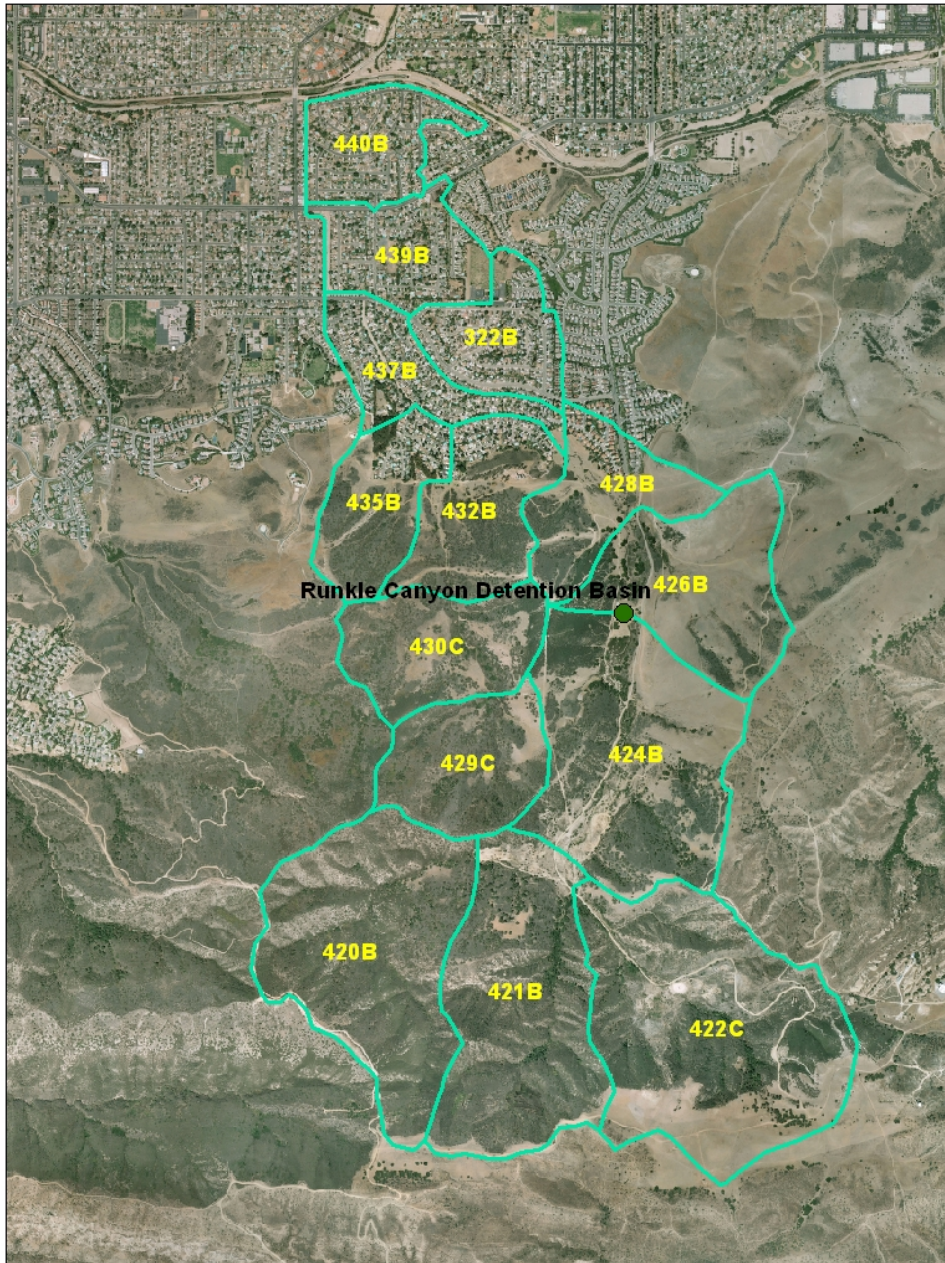


Figure B-11.1 Runkle Basin Watershed

The procedure is as follows:

1. If AR was not applied to the model at the upstream location because it was less than 600 ac in size, the AR factor should be based on the total tributary area at the location of interest. In this case it is only necessary to remove the basin routing, clear the hydrograph, and import a hydrograph of the basin outflow in the AR model. Then do an AR run for the project location.
2. If the tributary area to the basin is greater than 600 ac, do an AR run down to the basin. An example of the VCRat 2.2 AR input file for the Runkle Basin is provided.

```
005 15031 419A RUNKLE AR EXAMPLE RUN424AR.99I Q100P,9/2010
005 15031 424A RUNKLE CANYON DAM INFLOW Q100,AR=0.9277,Y=3.6
005 15031 425A LAST POINT
999
999
006 15031 419A 010 099B98 G1 7
006 15031 420A 020 0 22812B981 13000062
006 15031 421A 010 0 21011B98
006 15031 422B 020 0 28416B981 4750063
006 15031 423AB020 B981 30000037
006 15031 424A 020 0 23216B98 1 1
006 15031 425A 010 099B98 1 2□
```

3. At node 424A corresponding to the basin, the 954 ac tributary area has a non-AR peak of 2,213 cfs and an AR peak of 2,053 cfs, with a corresponding AR factor of $2053/2213=0.9277$.

4. Use this in the run for the full watershed with the following input file.

```
005 15031 419A RUNKLE AR EXAMPLE RUN440.99I Q100P,9/2010
005 15031 424A RUNKLE CANYON DAM INFLOW Q100,AR=0.92,Y=3.6,VVM/DDT/LS,6/97
005 15031 441A LAST POINT BEFORE ARROYO SIMI SUBAREAS
999
999
006 15031 419A 010 099B98 G1 7
006 15031 420A 020 0 22812B981 13000062
006 15031 421A 010 0 21011B98
006 15031 422B 020 0 28416B981 4750063
006 15031 423AB020 B981 30000037
006 15031 424A 020 0 23216B98 1
110
111 1086.5 0.9277 3.6
112 1071.75 1072.00 1073.00 1074.00 1075.00 1080.00 1086.00 1088.00 1090.00 1092.00
112 1094.00 1096.00
113 0.00 0.73 4.09 7.72 11.54 33.67 66.96 80.32 94.22 107.31
113 118.42 128.22
114 0.00 0.00 3.90 51.00 52.00 56.00 61.00 386.00 994.00 1805.00
114 2792.00 3942.00
110
006 15031 425A 010 099B985 130000423 300 2000 035035
006 15031 426A 020000 13814B985 140000321 300 2000 035035
006 15031 427A 010 099B98
006 15031 428A 030012 8714B985 240000205 150 750
006 15031 429B 020000 9913B981 162000545
006 15031 430B 020000 10313B981 190000316
006 15031 431B 010 099B984 70003570
006 15031 432A 030008 10014B98
006 15031 433AB010 B985 65000225 125 800 1
006 15031 434A 010 099B98
006 15031 435A 020008 8213B985 230000196 000 1000
006 15031 436A 010 099B98
006 15031 437A 040023 6912B985 165000122 200 1000
006 15031 438A 010 099B98
006 15031 439A 030021 7912B985 150000150 200 1200
```

006 15031 440A 040024 7110B98
 006 15031 441A 010 099B98

1 2□

5. Create a basin outflow hydrograph that can be imported to another VCRat 2.2 run. An easy way to do this is to import the VCRat 2.2 input file to VCRat 2.6, run the model, and then export the outflow hydrograph at 424B in VCRat 2.2 format. Then replace the subarea and basin info down to 424A with the imported hydrograph. Make sure the project number and node location is added to the header of the exported hydrograph. An example input file is shown below.

```

005 15031 419A RUNKLE AR EXAMPLE RUN440AR.99I Q100P,9/2010
005 15031 441A LAST POINT BEFORE ARROYO SIMI SUBAREAS
999
007 15031 424A      0 551169    1492200  4
008  5  0      0 100      0 200      0 300      1 400      3
008 10 500     13 600     22 700     30 800     41 900     51
008 15 1000    54 1050    56 1100    59 1110    60 1120    61
008 20 1130   159 1131   177 1132   194 1133   212 1134   229
008 25 1135   246 1136   263 1137   280 1138   297 1139   314
008 30 1140   331 1141   348 1142   365 1143   382 1144   410
008 35 1145   440 1146   470 1147   500 1148   530 1149   560
008 40 1150   593 1151   626 1152   661 1153   700 1154   742
008 45 1155   786 1156   834 1157   885 1158   941 1159  1000
008 50 1160  1082 1161  1160 1162  1233 1163  1299 1164  1358
008 55 1165  1407 1166  1445 1167  1473 1168  1488 1169  1492
008 60 1170  1487 1171  1474 1172  1456 1173  1433 1174  1406
008 65 1175  1375 1176  1341 1177  1306 1178  1269 1179  1233
008 70 1180  1196 1181  1160 1182  1124 1183  1090 1184  1056
008 75 1185  1023 1186   993 1187   971 1188   950 1189   929
008 80 1190   908 1191   888 1192   869 1193   850 1194   832
008 85 1195   814 1196   797 1197   780 1198   764 1199   748
008 90 1200   733 1201   719 1202   705 1203   691 1204   678
008 95 1205   666 1206   653 1207   642 1208   630 1209   619
008100 1210   609 1211   598 1212   588 1213   578 1214   569
008105 1215   559 1216   550 1217   541 1218   533 1219   524
008110 1220   516 1221   508 1222   500 1223   492 1224   485
008115 1225   478 1226   470 1227   463 1228   457 1229   450
008120 1230   444 1231   438 1232   431 1233   426 1234   420
008125 1235   414 1236   409 1237   404 1238   399 1239   394
008130 1240   390 1241   385 1242   383 1243   380 1244   378
008135 1245   375 1246   373 1247   371 1248   368 1249   366
008140 1250   363 1251   361 1252   358 1253   356 1254   354
008145 1255   351 1256   349 1257   347 1258   344 1259   342
008150 1260   340 1261   338 1262   335 1263   333 1264   331
008155 1265   329 1266   326 1267   324 1268   322 1269   320
008160 1270   317 1271   315 1272   313 1273   311 1274   308
008165 1275   306 1276   304 1277   302 1278   299 1279   297
008170 1280   295 1281   292 1282   290 1283   288 1284   285
008175 1285   283 1286   281 1287   278 1288   276 1289   274
008180 1290   271 1291   269 1292   267 1293   265 1294   262
008185 1295   260 1296   258 1297   256 1298   253 1299   251
008190 1300   249 1310   230 1320   210 1330   192 1340   174
008195 1350   157 1360   143 1370   130 1380   118 1390   108
008200 1400    99 1420    84 1440    73 1460    65 1500    61
999
006 15031 419A 010      099B98      G1 7
006 15031 420A 010      099B98
006 15031 421A 010      099B98
006 15031 422A 010      099B98
006 15031 423A 010      099B98
006 15031 424A 010      B98      A A
006 15031 425A 010      099B985 130000423 300 2000      035035
006 15031 426A 020000 13814B985 140000321 300 2000      035035
006 15031 427A 010      099B98
    
```

006	15031	428A	030012	8714B985	240000205	150	750		
006	15031	429B	020000	9913B981	162000545				
006	15031	430B	020000	10313B981	190000316				
006	15031	431B	010	099B984	70003570				
006	15031	432A	030008	10014B98					
006	15031	433AB010		B985	65000225	125	800		1
006	15031	434A	010	099B98					
006	15031	435A	020008	8213B985	230000196	000	1000		
006	15031	436A	010	099B98					
006	15031	437A	040023	6912B985	165000122	200	1000		
006	15031	438A	010	099B98					
006	15031	439A	030021	7912B985	150000150	200	1200		
006	15031	440A	040024	7110B98					1
006	15031	441A	010	099B98				1	2□

- The tributary area for the hydrograph should be set to 0 so the 2nd AR factor is only calculated for the net area below the basin down to the proposed project location. Turn on the AR with a 1 in column 67 of the VCRat 2.2 data input line for the project location, in this case subarea 440A. The results show that the non-AR peak is 2,247 cfs and the AR peak is 2,136 cfs, for an AR factor of 0.9507 for the 828 ac. watershed downstream of Runkle Basin.

B-12 UNIT RUNOFF EXAMPLE

Assume that a church in Simi Valley wishes to expand on its 2.71 ac lot as shown below. The parcel is located in node 439B of the District’s official Calleguas model.

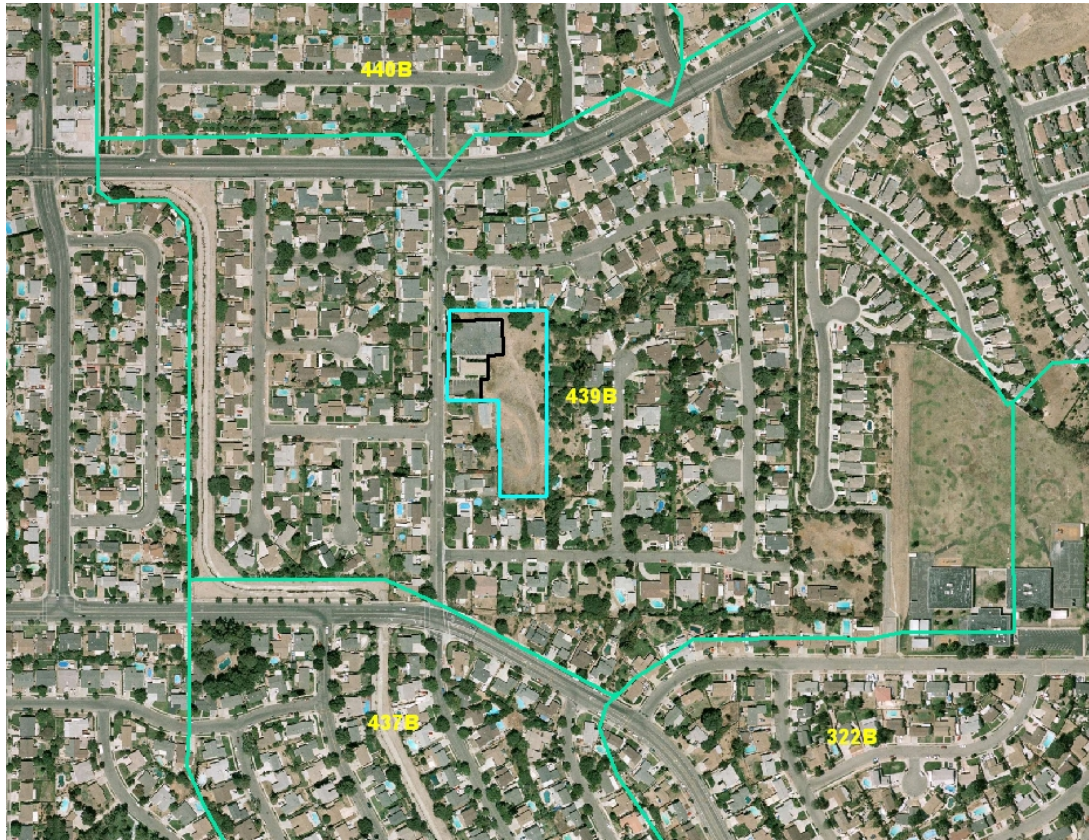


Figure B-12.1 Simi Valley Watershed

The parcel is 2.71 ac in size and approximately 0.73 ac is impervious in the existing condition. The pervious area is unirrigated grasses with some trees. The project will add a 1 ac senior center complex to the site.

The VCRat2.2 data for this subarea are as follows:

Area Ac.	Q100 cfs	Soil	Tc Min	RainZone	Eff. Imperv Frac.
79	196	3	12	B98 (K100)	0.21

As the development is located in the middle of the subarea, the Tc of the overall subarea should not be affected (usually the Tc would change only if the development affects the overland flowpath or changes the imperviousness of the subarea by more than 5%).

The existing effective impervious fraction of 0.21 indicates that it was selected from Exhibit 14b for 1/5 ac lots and decreased slightly from 0.23 to reflect the open spaces in the primarily low density residential subarea. The existing development site has an impervious fraction of $0.73/2.71=0.27$ which means the effective imperviousness of the parcel is likely similar to the overall subarea. Obtain the K100 rainfall

intensity of 3.23 in/hr for a Tc of 12 mins from Exhibit 2. Assuming that the P=23% curve applies to the overall parcel and P=0% for the new complex site, the C coefficients from Exhibit 6c are 0.870 and 0.845 respectively. Calculate the existing Q100 for the overall parcel and proposed senior complex as follows:

$$Q100e = 2.71 \text{ ac} * 3.23 \text{ in/hr} * 0.870 = 7.62 \text{ cfs}$$

$$Q100e = 1.0 \text{ ac} * 3.23 \text{ in/hr} * 0.845 = 2.73 \text{ cfs}$$

The additional development will increase the impervious fraction of the entire parcel to $1.73/2.71=0.64$. From Exhibit 14b, this is similar to high density residential development. Assuming the new project will incorporate measures to route the runoff across pervious areas and reduce the effective impervious area, the effective impervious fraction could be as low as 0.32 according to the Exhibit. The revised C coefficient for the parcel with the new center from Exhibit 6c is about 0.88. The proposed condition Q100s are then:

$$Q100p = 2.71 \text{ ac} * 3.23 \text{ in/hr} * 0.88 = 7.70 \text{ cfs, an increase of about 1\%}.$$

$$Q100e = 1.0 \text{ ac} * 3.23 \text{ in/hr} * 0.94 = 3.04 \text{ cfs}$$

While the new development does not increase the peak much, the volume of runoff from the site may increase due to the impervious area. The City of Simi Valley has historically required 1,000 cf/ac of detention for their mitigation. Using the approach outlined in [Section 6.15.2](#), the estimated detention volume required for mitigation is a conservative volume of approximately 7,000 cf. In this case, it would probably be more cost effective to use the techniques shown in Appendix D to design a bypass or flow-through basin.

	10 yr	100-yr
100-yr 1-d Rain in	4.75	7.2
Soil Type	3	3
Land Use	Open Space	Roof
Volume Calculation		
Yield Difference in		2.45
Depression Storage on Driveways, Patios in.		0.50
Net Yield		1.95
Impervious Area ac		1.00
Vol Increase CF- Max Basin Size Req'd		7078.50

A hydrograph for the project site can be developed using VCRat even though the project size is too small following the method discussed in [Section 6](#) as shown in Appendix B-12.

For many cities the project mitigation level is often the developed Q10 condition. If using MDPs to evaluate the site, the developed Q10 can be calculated from the info provided in the MDP 10-yr run using the approach given above. For the District's official Calleguas model, the subarea Q10 for 439B is obtained through the use of special design storm ratios developed for this study and provided on the

District's Calleguas Watershed webpage. A Q10 Tc can be back-calculated for the subarea by using VCRat and adjusting the Tc until the peak matches the official Q10. For this 79 ac subarea, the official 10-yr peak is 68.25 cfs. A 30-min Tc provides approximately this peak in VCRat 2.6, which corresponds to a peak intensity of 1.46 in/hr from Exhibit 2. From Exhibit 6c, the Q10p for the new complex with a P=90% is 0.92. The proposed Q10, which is the maximum-allowed outflow level from the detention basin is then:

$$Q10p = 1.46 \text{ in/hr} * 1 \text{ ac} * 0.92 = 1.34 \text{ cfs}$$

B-13 SMALL AREA HYDROGRAPH EXAMPLE

As discussed in Section 6, the minimum subarea size in VCRat 2.6 is 5 ac to decrease the chances of the program being used to produce flows for numerous tiny subareas such as is commonly done for interior drainage design. However, for small developments where peak mitigation is required through basin design, it is possible to use the program to get a hydrograph. The procedure takes advantage of the fact that the VCRat results are linear with area, so that the hydrograph of a 10-ac subarea is identical to the hydrograph of a 100-ac subarea divided by 10 as long as the other model input parameters (Tc, % impervious, soil type) are identical. The procedure to get a hydrograph for the 1.0 ac development pad to use for detention basin design as presented in Section B-11 is as follows:

1. Set up a VCRat 2.6 model using the existing model rain zone and Tc and project area soil type and % imperviousness. For subarea size in the model, use the project area times a factor from 10 to 100. For this example, use a factor of 10 so the subarea size will be 1.0 x 10 = 10 ac. Per Exhibit 14b, impervious surfaces have an effective imperviousness of 90%.

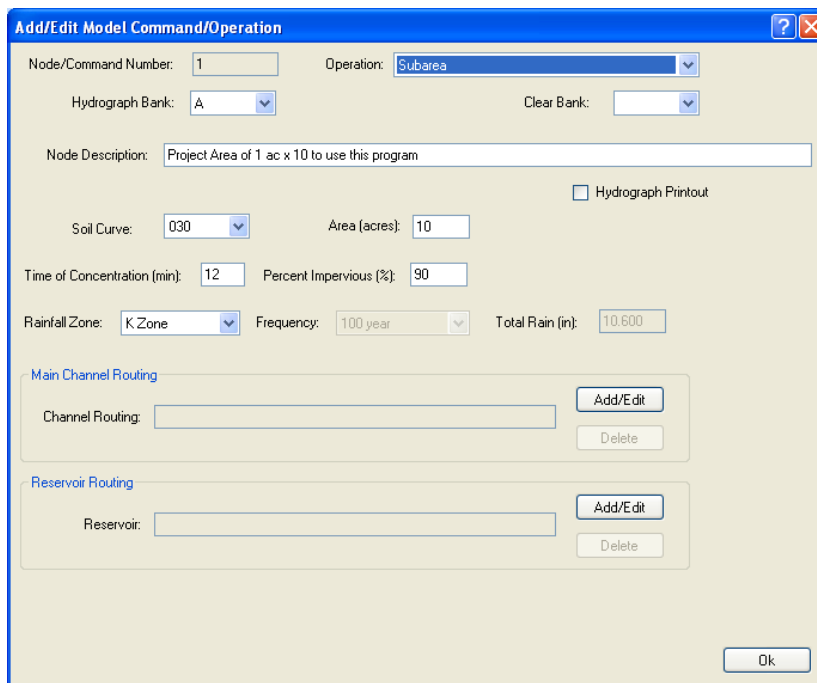


Figure B-13.1 VCRat2.6 Subarea Edit Window

2. “Fatten” the project area hydrograph in VCRat 2.6 using the yield adjustment procedure described in this manual. The impervious area has a CN of 98 with a 100-yr 24-hr rain depth of 7.2 in. In this case there is so much runoff from the impervious surface in the original calculation that the yield adjustment actually results in a volume decrease as shown in the model output.

```

2A : Do Fattening Here, Use CN=98 and Rain=7.2 in, print out hydrograph
2A --- --- -- -- --- --- 10 30 1154 -----
2A : Fattening for design hydrograph
*****
* INCOMING HYDROGRAPH PEAK (cfs): 30.16 VOLUME (acre-ft): 7.30
* NO HYDROGRAPH ADJUSTMENT
* RUNOFF FACTOR(in): 6.96 TOTAL RAIN(in): 7.20 SCS Curve: 98
* FATTENED HYDROGRAPH PEAK (cfs): 30.16 VOLUME (acre-ft): 5.81
*****
    
```

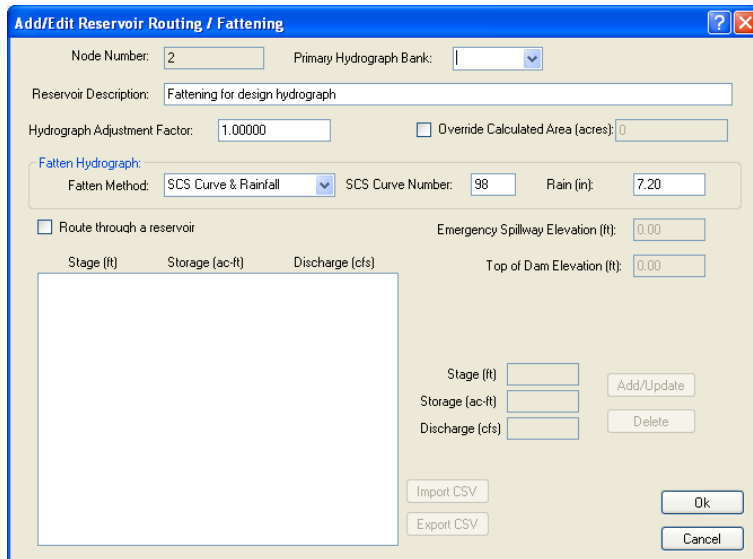


Figure B-13.2 VCRat2.6 Subarea Reservoir Routing Editing Window

3. Export the project area hydrograph in csv format so it can be imported into Excel. Divide the hydrograph ordinates by 10. The resultant hydrograph with a peak of approximately 3 cfs reduced from 30 cfs can be used in VCRat or another program to do basin design for the project area.

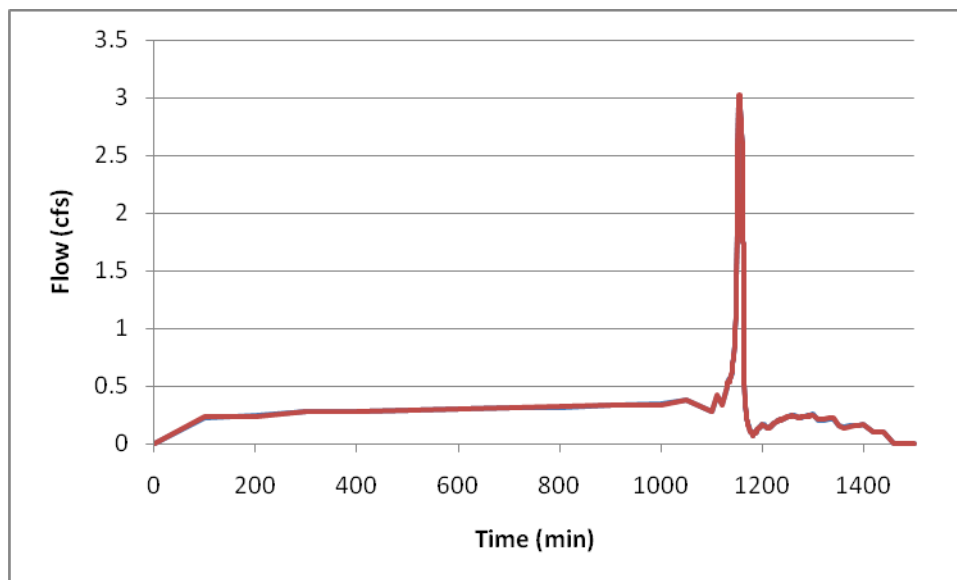


Figure B-13.3 Small Subarea Hydrograph After Export/Split

4. Another option is to use the split feature of VCRat and leave 10% of the flow ($10\% \times 10ac = \text{flow from 1 ac}$) in the mainline. Then export the mainline hydrograph for use in basin design. Whichever method is used, the hydrographs should be identical.
5. Comments received from consultants indicate that they are confused about why the T_c does not have to be increased when the area is increased. However, the process is only taking advantage of the linearity of the $Q=CIA$ equation used in VCRat. Because the “apparent” effect of the increase in area is removed at the end by dividing the hydrograph by the same factor, it is not necessary to change any of the other input parameters when calculating the subarea outflow.

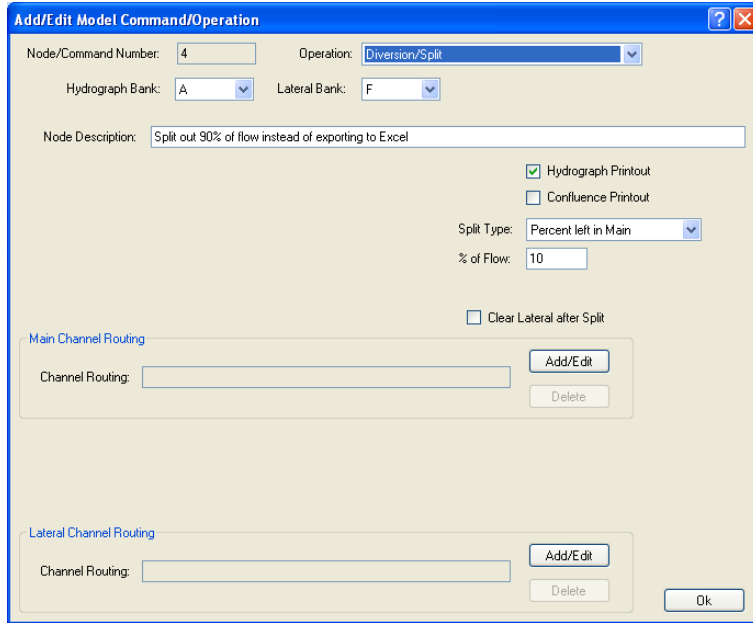


Figure B-13.4 VCRat2.6 Diversion/Split Editing Window

APPENDIX C HSPF AND HMS DESIGN STORM MODELING

C-1 DISTRICT RAIN GAGE FREQUENCY ANALYSIS FILE

PRECIPITATION DEPTH-DURATION (SHORT TERM) - MAXIMUM ANNUAL PRECIPITATION (IN)

VENTURA COUNTY WATERSHED PROTECTION DISTRICT

STATION #	STATION NAME	ELEV	LATITUDE	LONGITUDE	TOWNSHIP RANGE	COUNTY
VC 165	STEWART CANYON	960	34:27:38	119:14:50	T4N R23W SEC 1	VENTURA

M=MINUTES,H=HOURS,D=DAYS,W YR=WATER YEAR

WATER YR	5M	15M	30M	1H	2H	4H	6H	12H	24H
1957	0.09	0.20	0.40	0.73	1.08	1.45	1.75	2.36	2.84
1958	0.26	0.34	0.43	0.58	1.00	1.46	1.90	2.66	2.96
1959	0.27	0.47	0.71	1.09	1.35	2.00	2.70	3.40	3.45
1960	0.04	0.11	0.21	0.41	0.69	1.19	1.58	2.02	2.05
1961	0.10	0.26	0.48	0.72	1.12	1.28	1.28	1.58	2.40
1962	0.11	0.23	0.44	0.78	0.92	1.74	2.23	2.84	4.74
1963	0.24	0.46	0.56	0.88	1.29	2.44	2.76	3.20	3.94
1964	0.20	0.40	0.46	0.63	0.80	1.10	1.35	1.62	2.26
1965	0.10	0.15	0.20	0.44	0.48	0.84	1.17	2.05	3.41
1966	0.16	0.45	0.48	0.82	1.59	2.32	2.93	3.82	4.67
1967	0.18	0.25	0.42	0.53	0.90	1.53	1.86	3.50	4.25
1968	0.15	0.24	0.35	0.38	0.66	1.18	1.57	2.70	3.35
1969	0.34	0.53	0.85	1.16	1.63	2.77	3.60	4.48	7.44
1970	0.12	0.19	0.32	0.56	0.87	1.39	1.80	2.84	3.60
1971	0.09	0.21	0.31	0.58	1.11	2.01	2.98	4.25	5.06
1972	0.07	0.15	0.18	0.35	0.58	1.10	1.51	2.28	2.56
1973	0.15	0.35	0.58	1.02	1.94	2.86	3.25	3.88	4.57
1974	0.09	0.12	0.21	0.45	0.61	1.05	1.30	2.15	4.23
1975	0.10	0.25	0.47	0.87	1.59	2.54	3.14	4.59	4.81
1976	0.15	0.30	0.53	0.78	1.13	1.72	2.13	2.83	2.84
1977	0.12	0.32	0.47	0.68	1.18	1.52	1.77	2.14	2.15
1978	0.13	0.32	0.65	1.07	1.88	2.82	3.42	3.72	5.42
1979	0.08	0.16	0.30	0.57	0.97	1.55	2.00	2.76	3.01
1980	0.22	0.47	0.59	0.94	1.66	2.59	3.51	5.09	5.72
1981	0.15	0.28	0.43	0.71	1.13	1.45	1.67	2.30	2.78
1982	0.08	0.16	0.25	0.39	0.63	0.89	0.94	1.53	1.58
1983	0.35	0.57	0.77	1.15	1.50	2.30	3.03	3.46	4.28
1984	0.07	0.17	0.32	0.62	1.06	1.25	1.27	1.81	2.50
1985	0.08	0.21	0.35	0.59	1.01	1.31	1.32	1.80	2.08
1986	0.14	0.34	0.51	0.74	1.22	2.20	2.92	3.52	4.00
1987	0.12	0.35	0.59	0.85	1.06	1.27	1.38	1.54	1.61
1988	0.20	0.46	0.61	0.76	1.08	1.23	1.32	1.92	2.33
1989	0.05	0.14	0.28	0.49	0.81	1.05	1.26	1.66	2.09
1990	0.10	0.31	0.48	0.70	0.87	1.37	1.93	2.33	2.78
1991	0.11	0.18	0.27	0.42	0.81	1.35	1.86	2.92	4.24
1992	0.25	0.51	0.62	0.84	1.24	1.63	2.10	2.35	4.18
1993	0.17	0.34	0.55	0.92	1.37	1.88	2.21	3.01	5.15
1994	0.09	0.17	0.28	0.48	0.85	1.44	1.87	2.23	2.23
1995	0.19	0.48	0.72	1.19	2.18	3.85	4.35	6.92	8.68
1996	0.22	0.31	0.41	0.64	0.96	1.78	2.24	3.47	5.29
1997	0.10	0.16	0.24	0.44	0.84	1.44	2.16	2.60	2.96
1998	0.16	0.24	0.40	0.64	1.12	1.92	2.16	2.92	4.28
1999	0.16	0.24	0.32	0.44	0.56	0.80	1.12	1.76	1.96
2000	0.12	0.28	0.36	0.56	0.80	1.32	1.60	2.00	2.80
2001	0.12	0.32	0.52	0.84	1.32	2.32	2.76	3.84	4.96
2002	0.12	0.28	0.44	0.84	1.16	1.56	1.60	1.60	1.60
2003	0.12	0.24	0.40	0.72	1.24	2.16	3.04	4.20	4.64
2004	0.13	0.21	0.31	0.60	0.97	1.75	2.38	3.23	3.42
2005	0.15	0.40	0.65	1.08	1.87	2.29	2.93	3.80	7.09
2006	0.08	0.20	0.28	0.49	0.85	1.57	2.17	3.39	3.88
2007	0.10	0.24	0.34	0.43	0.65	0.76	0.98	1.25	1.63

APPENDIX C

HSPF AND HMS DESIGN STORM MODELING

2008 0.13 0.25 0.47 0.89 1.61 2.80 3.73 5.08 5.29

PRECIPITATION DEPTH-DURATION (SHORT TERM) - FREQUENCY TABLE AND STATISTICS
VENTURA COUNTY WATERSHED PROTECTION DISTRICT

STATION #	STATION NAME	ELEV	LATITUDE	LONGITUDE	TOWNSHIP RANGE	COUNTY
VC 165	STEWART CANYON	960	34:27:38	119:14:50	T4N R23W SEC 1	VENTURA

M=MINUTES,H=HOURS,D=DAYS,W YR=WATER YEAR

RETURN PERIOD IN YEARS	5M	15M	30M	1H	2H	4H	6H	12H	24H
2	0.13	0.26	0.40	0.64	1.01	1.56	1.95	2.64	3.35
5	0.19	0.39	0.59	0.95	1.50	2.32	2.90	3.92	4.98
10	0.23	0.47	0.72	1.15	1.82	2.81	3.52	4.76	6.04
20	0.27	0.55	0.84	1.34	2.12	3.28	4.10	5.55	7.04
25	0.28	0.57	0.87	1.40	2.21	3.42	4.28	5.79	7.36
40	0.31	0.62	0.95	1.52	2.41	3.72	4.66	6.30	8.00
50	0.32	0.65	0.98	1.58	2.50	3.86	4.83	6.54	8.30
100	0.36	0.72	1.09	1.75	2.77	4.29	5.37	7.26	9.22
200	0.39	0.79	1.20	1.92	3.04	4.71	5.89	7.96	10.11
500	0.43	0.88	1.33	2.13	3.38	5.23	6.54	8.85	11.24
1000	0.47	0.95	1.44	2.31	3.65	5.65	7.07	9.56	12.14
10000	0.58	1.16	1.77	2.84	4.50	6.96	8.70	11.76	14.94
PMP	1.16	2.34	3.56	5.70	9.03	13.97	17.47	23.63	30.01
MEAN	0.143	0.288	0.438	0.702	1.112	1.719	2.150	2.908	3.693
CLOCK HR. COR.	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
CALCULATED SKEW	1.279	0.654	0.507	0.454	0.755	0.976	0.651	1.085	0.960
REGIONAL SKEW	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200
SKEW USED	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200
KURTOSIS	4.757	2.727	2.947	2.502	3.347	4.148	2.834	4.940	4.293
YEARS OF RECORD	52	52	52	52	52	52	52	52	52
LAST YEAR RECORD	2008	2008	2008	2008	2008	2008	2008	2008	2008
RECORD YEAR	1983	1983	1969	1995	1995	1995	1995	1995	1995
RECORD MAXIMUM	0.35	0.57	0.85	1.19	2.18	3.85	4.35	6.92	8.68
NORMALIZED MAX	3.083	2.420	2.638	2.154	2.765	3.332	2.715	3.573	3.222
CALC. COEF. VAR	0.471	0.405	0.357	0.323	0.348	0.372	0.377	0.386	0.419
REGN. COEF. VAR	0.475	0.475	0.475	0.475	0.475	0.475	0.475	0.475	0.475
COEF. VAR USED	0.475	0.475	0.475	0.475	0.475	0.475	0.475	0.475	0.475

PEARSON TYPE III DISTRIBUTION USED

PROBABLE MAXIMUM PRECIPITATION (PMP) ESTIMATED ON 15 STANDARD DEVIATIONS

WHERE YEARS OF RECORD IS SMALL (LESS THAN 15 YEARS) - RESULTS ARE NOT DEPENDABLE

C-2 DESIGN STORM RAINFALL HYETOGRAPH

Data from frequency analysis are used to fit a trendline using a power equation.

	Duration	Cum. Precip.
	(min)	(in)
Points	(x)	$y=ax^{**}b$
1	5	0.36
2	10	0.58
3	15	0.72
4	30	1.09
5	60	1.75
6	120	2.77
7	180	3.24
8	240	4.29
9	360	5.37
10	720	7.26
11	1440	9.22
Note: Cells in Yellow Estimated from Trend		

The equation of the trendline is used to generate the rainfall intensities to build the hyetograph.

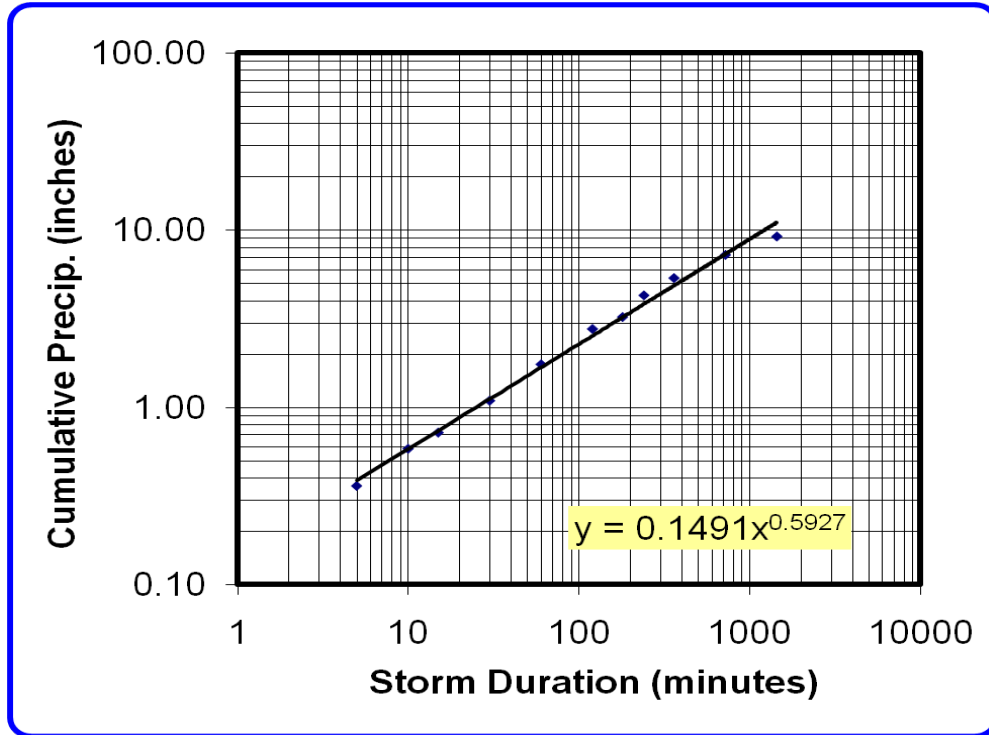


Figure C-2.1 Rain Data Regression Results

A spreadsheet with example calculations is provided in Appendix E.

C-3 UNDEVELOPED WATERSHED HEC-HMS S-GRAPH DATA

The following table provides the S-Graphs developed for three types of undeveloped watersheds- relatively long narrow ones, ones with rounded shapes, and others that do not clearly fit either category.

Ellsworth		Canada Larga		NForkMatilija			
% time	% Q	% time	% Q	% time	% Q	% time	% Q
0.00	0.00	0.00	0.00	0.00	0.00	605.56	95.14
6.41	0.06	16.04	0.41	5.56	0.03	611.11	95.30
12.82	0.26	32.07	1.96	11.11	0.13	616.67	95.45
19.23	0.69	48.11	5.09	16.67	0.33	622.22	95.59
25.64	1.38	64.14	9.97	22.22	0.67	627.78	95.74
32.05	2.37	80.18	16.38	27.78	1.16	633.33	95.87
38.46	3.67	96.22	23.77	33.33	1.81	638.89	96.01
44.87	5.31	112.25	31.36	38.89	2.63	644.44	96.14
51.28	7.30	128.29	38.44	44.44	3.64	650.00	96.26
57.69	9.64	144.32	44.81	50.00	4.84	655.56	96.39
64.10	12.32	160.36	50.51	55.56	6.24	661.11	96.51
70.51	15.29	176.40	55.64	61.11	7.82	666.67	96.62

APPENDIX C

HSPF AND HMS DESIGN STORM MODELING

Ellsworth		Canada Larga		NForkMatilja			
% time	% Q	% time	% Q	% time	% Q	% time	% Q
76.92	18.49	192.43	60.24	66.67	9.58	672.22	96.73
83.33	21.88	208.47	64.36	72.22	11.48	677.78	96.84
89.74	25.42	224.50	68.06	77.78	13.50	683.33	96.95
96.15	29.03	240.54	71.39	83.33	15.63	688.89	97.05
102.56	32.67	256.57	74.37	88.89	17.85	694.44	97.15
108.97	36.24	272.61	77.05	94.44	20.11	700.00	97.25
115.38	39.66	288.65	79.45	100.00	22.40	705.56	97.34
121.79	42.90	304.68	81.61	105.56	24.67	711.11	97.43
128.21	45.96	320.72	83.54	111.11	26.87	716.67	97.52
134.62	48.86	336.75	85.28	116.67	29.00	722.22	97.61
141.03	51.61	352.79	86.84	122.22	31.08	727.78	97.69
147.44	54.21	368.83	88.24	127.78	33.09	733.33	97.77
153.85	56.67	384.86	89.49	133.33	35.05	738.89	97.85
160.26	59.00	400.90	90.62	138.89	36.95	744.44	97.93
166.67	61.21	416.93	91.63	144.44	38.79	750.00	98.00
173.08	63.30	432.97	92.54	150.00	40.59	755.56	98.08
179.49	65.28	449.01	93.35	155.56	42.32	761.11	98.15
185.90	67.15	465.04	94.08	161.11	44.01	766.67	98.21
192.31	68.92	481.08	94.74	166.67	45.65	772.22	98.28
198.72	70.60	497.11	95.33	172.22	47.24	777.78	98.35
205.13	72.19	513.15	95.85	177.78	48.79	783.33	98.41
211.54	73.70	529.19	96.33	183.33	50.29	788.89	98.47
217.95	75.12	545.22	96.75	188.89	51.75	794.44	98.53
224.36	76.47	561.26	97.13	194.44	53.16	800.00	98.58
230.77	77.75	577.29	97.48	200.00	54.54	805.56	98.64
237.18	78.96	593.33	97.78	205.56	55.87	811.11	98.69
243.59	80.10	609.36	98.06	211.11	57.17	816.67	98.75
250.00	81.19	625.40	98.31	216.67	58.42	822.22	98.80
256.41	82.22	641.44	98.53	222.22	59.64	827.78	98.85
262.82	83.19	657.47	98.73	227.78	60.83	833.33	98.89
269.23	84.11	673.51	98.91	233.33	61.98	838.89	98.94
275.64	84.98	689.54	99.07	238.89	63.10	844.44	98.98
282.05	85.81	705.58	99.21	244.44	64.18	850.00	99.03
288.46	86.59	721.62	99.34	250.00	65.24	855.56	99.07
294.87	87.33	737.65	99.46	255.56	66.26	861.11	99.11
301.28	88.03	753.69	99.56	261.11	67.26	866.67	99.15
307.69	88.69	769.72	99.66	266.67	68.22	872.22	99.19

APPENDIX C

HSPF AND HMS DESIGN STORM MODELING

Ellsworth		Canada Larga		NForkMatilja			
% time	% Q	% time	% Q	% time	% Q	% time	% Q
314.10	89.32	785.76	99.74	272.22	69.16	877.78	99.23
320.51	89.91	801.80	99.82	277.78	70.07	883.33	99.27
326.92	90.48	817.83	99.88	283.33	70.95	888.89	99.30
333.33	91.01	833.87	99.95	288.89	71.81	894.44	99.34
339.74	91.51	849.90	100.00	294.44	72.64	900.00	99.37
346.15	91.99	865.94	100.00	300.00	73.45	905.56	99.40
352.56	92.44			305.56	74.23	911.11	99.43
358.97	92.87			311.11	75.00	916.67	99.46
365.38	93.28			316.67	75.74	922.22	99.49
371.79	93.66			322.22	76.45	927.78	99.52
378.21	94.02			327.78	77.15	933.33	99.55
384.62	94.37			333.33	77.83	938.89	99.58
391.03	94.69			338.89	78.49	944.44	99.60
397.44	95.00			344.44	79.13	950.00	99.63
403.85	95.30			350.00	79.75	955.56	99.66
410.26	95.57			355.56	80.35	961.11	99.68
416.67	95.83			361.11	80.93	966.67	99.70
423.08	96.08			366.67	81.50	972.22	99.73
429.49	96.32			372.22	82.05	977.78	99.75
435.90	96.54			377.78	82.59	983.33	99.77
442.31	96.75			383.33	83.11	988.89	99.79
448.72	96.95			388.89	83.61	994.44	99.81
455.13	97.14			394.44	84.10	1000.00	99.83
461.54	97.32			400.00	84.58	1005.56	99.85
467.95	97.48			405.56	85.04	1011.11	99.87
474.36	97.65			411.11	85.49	1016.67	99.89
480.77	97.80			416.67	85.92	1022.22	99.90
487.18	97.94			422.22	86.35	1027.78	99.92
493.59	98.08			427.78	86.76	1033.33	99.94
500.00	98.20			433.33	87.16	1038.89	99.95
506.41	98.33			438.89	87.54	1044.44	99.97
512.82	98.44			444.44	87.92	1050.00	99.99
519.23	98.55			450.00	88.28	1055.56	100.00
525.64	98.65			455.56	88.64		
532.05	98.75			461.11	88.98		
538.46	98.85			466.67	89.32		
544.87	98.93			472.22	89.64		

Ellsworth		Canada Larga		NForkMatilja			
% time	% Q	% time	% Q	% time	% Q	% time	% Q
551.28	99.02			477.78	89.96		
557.69	99.10			483.33	90.26		
564.10	99.17			488.89	90.56		
570.51	99.24			494.44	90.85		
576.92	99.31			500.00	91.13		
583.33	99.37			505.56	91.40		
589.74	99.43			511.11	91.66		
596.15	99.49			516.67	91.92		
602.56	99.54			522.22	92.17		
608.97	99.59			527.78	92.41		
615.38	99.64			533.33	92.64		
621.79	99.68			538.89	92.87		
628.21	99.73			544.44	93.09		
634.62	99.77			550.00	93.31		
641.03	99.81			555.56	93.52		
647.44	99.84			561.11	93.72		
653.85	99.88			566.67	93.92		
660.26	99.91			572.22	94.11		
666.67	99.94			577.78	94.29		
673.08	99.97			583.33	94.47		
679.49	100.00			588.89	94.65		
685.90	100.00			594.44	94.82		
692.31	100.00			600.00	94.98		

Note: Data also provided in Appendix E HEC-HMS Version 3.2 Files

C-4 HEC-HMS OR HSPF HYDROGRAPH TRANSFORMATION

The following figure shows the results of transforming an HMS hydrograph into a truncated VCRat hydrograph. The procedure is done with the spreadsheet provided in Appendix E. The process involves determining the relative peak times of the hydrographs and then adjusting the time ordinates. The adjusted file does not have to be transformed to the variable VCRat spacing because VCRat2.6 will do this after the hydrograph is imported.

The HMS or HSPF hydrograph will be truncated because the VCRat simulation length is 1,440 minutes. The yield of the transformed hydrograph will be too small once it is imported into VCRat but the volume of the leading limb of the hydrograph is most important if the hydrograph is needed for basin design. The truncated hydrograph cannot be used to evaluate the time it takes to drain the basin after the rainfall peak has occurred. Other hydrograph conversion spreadsheets are provided that convert VCRat variable timing hydrographs into 1- or 5-min intervals.

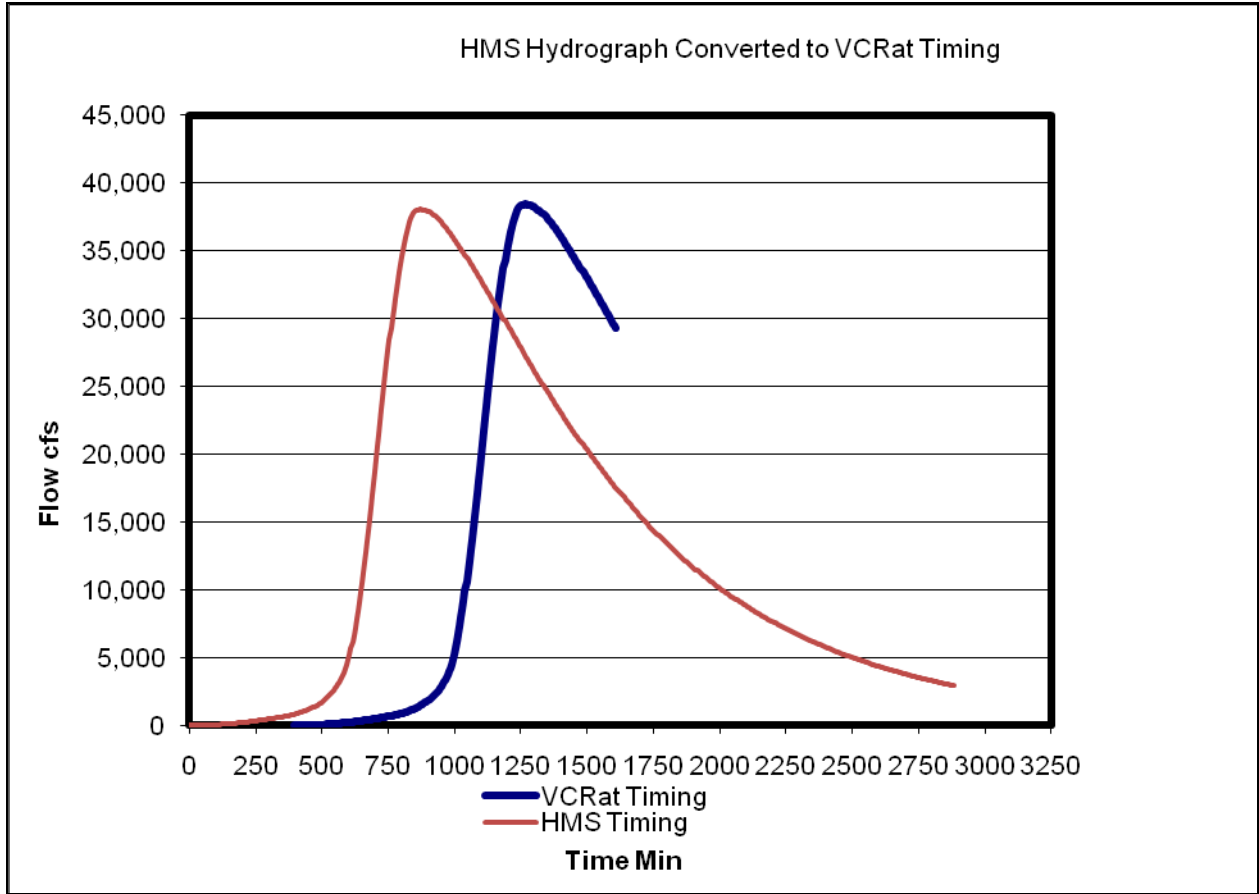


Figure C-4.1 HMS Hydrograph Converted to VCRat Timing

APPENDIX D DETENTION BASIN DESIGN**D-1 TRUNCATED SUMP UPDATE STUDY EXAMPLE FILES**

The Truncated Sump or Bypass Basin analysis presented in the report showed that the required basin volume varies with development type, time of concentration, mitigation goal, and 24-hour rain depth used to fatten the design hydrograph. The procedure for performing a hydrology run to obtain the necessary bypass basin volume is as follows:

1. Establish mitigation goal, commonly the 10-yr developed condition.
2. Calculate the Tc's for the 10- and 100-yr proposed development.
3. Create a VCRat2.6 hydrology input file and enable the option to use different storm frequencies in the run.
4. Add a subarea to the input file using the 10-yr storm for rainfall.
5. In the next data input line, clear the hydrograph and add the same subarea info with the 100-yr storm. Fatten the hydrograph.
6. Run the model and obtain the 10-yr storm.
7. Add a split to the model and split all flow above the 10-yr peak level into the bypass basin.
8. The hydrograph summaries at the bottom of the output file will provide the volume of the flow diverted to the bypass basin.

For design of bypass basins, it is necessary to perform a detailed hydraulic analysis to show that the side channel weir or other diversion structure will function adequately. Appendix E provides a sample hydrology file for performing the above operations.

D-2 FLOW THROUGH BASIN DESIGN

D-2.1 Basin Design Procedure

This procedure provides a method for sizing a basin and outlet works for project sites that are too small to be evaluated with standard Hydrology Section tools. In general, this procedure applies to projects that are less than 10-ac in size, although some of the techniques described here may be useful for larger projects. The variables associated with basin design include the storm interval, subarea size, T_c , soil type, % impervious value, required mitigation level, basin configuration and depth, outlet works assumptions, and n-yr 1-day rainfall used to adjust the hydrograph yield. The number of variables makes it difficult to generate nomographs or simplified relationships that can be used in design. Therefore, a simplified methodology and examples are provided here to demonstrate how basin design can be done in accordance with Hydrology Section requirements. The following sections provide information on different steps in the procedure.

D-2.1.1 Project Size

This method should be applied to projects less than 5-ac in size that cannot be evaluated with VCRat 2.6 or other hydrology section tools.

D-2.1.2 Mitigation Criteria

Obtain the mitigation criteria from the jurisdiction. A common mitigation level is to reduce the developed condition Q100 peak to the developed condition Q10 peak. Other mitigation levels include the undeveloped condition Q10 or Q100.

D-2.1.3 Project Area T_c 's

The project area T_c 's are found through the following steps:

1. Find the subarea that incorporates the project area in a Master Drainage Plan or official hydrology study. If both sources are available, use the subarea with the longest T_c for the evaluation. The 100-yr T_c and the T_c associated with the mitigation level are needed. Once the appropriate subarea is identified, extract the subarea information such as T_c , area, % imperviousness, soil, etc.
2. Determine if the T_c will change in the developed condition and recalculate the T_c if necessary. If the project area represents the overland flow path in the T_c calculation, the T_c may decrease. Perform a T_c calculation to check this. If the project is not an overland flow path, the T_c will likely stay the same as shown in the MDP or other model and does not need to be recalculated.
3. Calculate the T_c for the mitigation condition if it is different from those provided above. If the T_c calculation from the MDP or official hydrology study is not available, the peak mitigation flow can be estimated using the Hydrology Section standard multipliers. Then VCRat can be used to find the T_c that provides that peak flow by changing the T_c until the VCRat peak matches the calculated multiplier peak.

D-2.1.4 Project Area Peak Flows and Hydrographs

The project area flow are obtained through the following:

1. Calculate the developed condition peak flow using a C coefficient that represents the developed condition effective impervious value and the rainfall intensity associated with the developed condition Tc. Calculate the mitigation condition peak flow using the C coefficient that represents the mitigation condition development type and the rainfall intensity associated with the mitigation condition Tc.
2. If the user needs a hydrograph for basin design, input the required developed condition information into a VCRat 2.6 model run but multiply the subarea by a factor to get the modeled subarea into the 10-300 ac range. Because the rational method results are linearly related to area, the user can export the output VCRat hydrograph ordinates and divide by the same factor used to increase the area to get a hydrograph for use in basin design. Fatten the hydrograph in VCRat to make sure it has the right yield using the 100-yr 1-day rainfall at the project site and the appropriate Curve Number (Exhibit 14) associated with the development and soil types.
3. Reduce the hydrograph so that it represents the actual subarea size using the split flow option in VCRat (Percent left in main). For example, if the original project area is 2 ac, this could be increased to 20 or 200 ac for the run, the Percent Left in Main would be 10 or 1%, respectively. At this time the entry into the field must be an integer so make sure the factor used allows this when converting the hydrograph back to the right volume and peak for the area.
4. Add a placeholder data input line with reservoir routing after the split flow line and enter the stage-storage-discharge information calculated from the next section. Run the model and check to see if the basin meets the mitigation requirements. Alternatively, the basin design hydrograph can be exported from VCRat and imported into HEC-HMS after interpolating it to 1-min intervals and HEC-HMS can be used for basin routing modeling.

D-2.2 Basin Design

The basin design methodology outlined in this section is obtained mainly from the NPDES dry detention basin methodology outlined in the Stormwater Technical Guidance Manual (July 2002) available on the VCWPD website. The dry detention basin guidelines include the following:

1. Basin side-slopes are minimum 3H to 1V.
2. Basin length is twice the basin width
3. Max depth of basin is calculated at $0.1 * (\text{Bottom Area})^{0.5}$

For the purposes of this study, it is expected that basins will be designed to drain by gravity, and that a project site slope of 5% is the maximum that could be expected. Therefore, the depth of the basin is further limited so that Basin Depth/Basin Top Length ratio is less than or equal to 5% so that the basin could be designed with a minimum of above ground dam construction. For simplicity, the outlet works of the basins are assumed consist of the following:

1. One small orifice at the bottom of a circular riser that is designed discharge stored flow from the basin in 24 hours or less using the NPDES methodology or a number of small orifices in the riser to discharge stored flow in 24 hours or less
2. Weir flow into the top of a circular riser.
3. Once the height of the flow above the riser top is equal to the riser radius (submerged weir), it is assumed that the riser inflow is limited to the discharge through the culvert outlet with entrance, exit, and friction losses in the pipe. A figure given below illustrates this approach.

The design approach for the basin consists of the following steps:

1. Use the T_c and other hydrologic parameters from above to generate a VCRat run that provides the mitigation peak and developed condition peak and fattened hydrograph volume. Increase the project area by a factor to get the VCRat area in the range of 10-300 acres and then divide the results by the factor to get the project area peak, hydrograph ordinates, and hydrograph volume.
2. Find preliminary ratio of required basin volume to inflow volume from the attached summary table. As a first guess, a ratio of 0.10 is generally adequate except in high rainfall areas and for projects with long T_c 's (20-30 minutes).
3. Multiply ratio times inflow volume to obtain an initial guess of the required basin volume (SQDV in subsequent calculations).
4. Calculate maximum depth of basin based on NPDES requirements: $D_{max}=0.1*(\text{basin bottom area})^{0.5}$
5. Estimate required basin depth by rounding depth to nearest foot
6. Divide basin depth by calculated basin top length to get land slope necessary to provide gravity flow at outlet. If calculated slope is greater than 5%, reduce depth and recalculate slope until slope is less than 5%.
7. Calculate diameter of orifice required to outlet flow in 24 hrs or less with the following equation:
 - b. For single orifice outlet control or single row of orifices at the basin bottom surface elevation (see Figures 5-6), use the following equation based on the SQDV (ft^3) and depth of water above orifice centerline D_{BS} (ft) to determine total orifice area (in^2):

$$\text{Total orifice area} = (\text{SQDV}) + [(60.19)(D_{BS}^{0.5})(T)]$$

Where $T=24$ hrs and SQDV = basin volume calculated above.

The DesignExample.xls spreadsheet calculates the stage-volume curve at 0.5 ft intervals. The DesignExample.xls spreadsheet calculates the stage-discharge curve at 0.5 ft intervals based on one orifice at the bottom of the basin for long-term drainage, weir flow over the riser top, and a pipe outlet that controls the outflow. The riser top elevation is assumed to be located at $\frac{1}{2}$ the basin maximum depth in the preliminary design phase. These data can be entered into VCRat2.6 run to see if the basin meets the mitigation requirements. If it does not, redesign the outlet or increase the volume until the requirements are met.

Example:

A 9-ac site in Simi Valley will be converted from open space to an industrial development. The mitigation condition is assumed to be to build a detention basin that will reduce the developed condition Q100 to the developed condition Q10. The 2000 Condition Calleguas Model includes this site in a 40-ac subarea. Based on a review of topo for this subarea, the project is not part of the longest or overland flowpath used to calculate the T_c so the developed condition T_c does not need to be revised.

Model Output Evaluation

Calleguas 2000 Condition 100-yr Results

Soil Type 3; Tc 11 min; K Zone; Average Impervious = 23%.

Subarea peak flow is 113 cfs; or 2.825 cfs/ac

Calleguas 2000 Condition 10 yr Results

Q10 with Tc = 13 mins is 68 cfs, or 1.7 cfs/ac.

Use parameters from Calleguas subarea Soil Type 3; K Zone; Average Impervious = 23%

(See 2000ConditionCalleguasModelExample.vin, .out; attached output).

Using 2006 Hydrology Manual

Exhibit 14a,b show 70% effective impervious for industrial sites. CN for soil type 3 is 96.

Exhibit 2 shows K Zone rainfall intensities of:

Tc=11 min, 100-yr storm, I=3.39 in/hr

Tc=13 min, 10-yr storm, I=2.20 in/hr

Exhibit 6c shows C coefficients for Soil Type 3

If I=3.39in/hr, C(70%)= 0.90

If I=2.2in/hr, C(70%)=0.88

Project Area Developed Condition Q100= $0.90 * 3.39 \text{ in/hr} * 9 \text{ ac} = 27.5 \text{ cfs}$

Project Area Developed Condition Q100= $0.88 * 2.20 \text{ in/hr} * 9 \text{ ac} = 17.4 \text{ cfs}$

Preparation of VCRat Model for Project Area

See VCRAT2.5 files SimiValleyDesignExample.vin, .out

Since 9 ac project area is too small for VCRat run, analyze a 90 ac subarea and divide the results by factor of 10 or use split flow option in VCRat to get correct hydrograph peak and volume.

Enter subarea info for mitigation peak flow in model.

The screenshot shows the 'Add/Edit Model Command/Operation' dialog box for a subarea. The 'Node/Command Number' is 1 and the 'Operation' is 'Subarea'. The 'Hydrograph Bank' is 'A' and the 'Clear Bank' is also 'A'. The 'Node Description' is '10-yr ind dev peak Tc=13 min project= 9 ac factor=10'. The 'Soil Curve' is '030' and the 'Area (acres)' is '90'. The 'Time of Concentration (min)' is '13' and the 'Percent Impervious (%)' is '70'. The 'Rainfall Zone' is 'K Zone', the 'Frequency' is '10 year', and the 'Total Rain (in)' is '5.530'. There is a checked box for 'Hydrograph Printout'. Below these fields are sections for 'Main Channel Routing' and 'Reservoir Routing', each with an 'Add/Edit' and 'Delete' button. An 'Ok' button is at the bottom right.

Figure D-2.1 VCRat2.6 Subarea Edit Window

Enter Developed Condition Q100 parameters in model:

The screenshot shows the 'Add/Edit Model Command/Operation' dialog box for a subarea. The 'Node/Command Number' is 3 and the 'Operation' is 'Subarea'. The 'Hydrograph Bank' is 'A' and the 'Clear Bank' is also 'A'. The 'Node Description' is '100yr ind w/ fat Tc 11 Soil 3 project=9ac factor=10'. The 'Soil Curve' is '030' and the 'Area (acres)' is '90'. The 'Time of Concentration (min)' is '11' and the 'Percent Impervious (%)' is '70'. The 'Rainfall Zone' is 'K Zone', the 'Frequency' is '100 year', and the 'Total Rain (in)' is '10.600'. There is a checked box for 'Hydrograph Printout'. Below these fields are sections for 'Channel Routing Before Reservoir', 'Reservoir Routing', and 'Channel Routing After Reservoir', each with an 'Add/Edit' and 'Delete' button. The 'Reservoir' field contains the text 'No Reservoir / Adjust: None / Fatten SCS: 96 Rain: 8.00'. An 'Ok' button is at the bottom right.

Figure D-2.2 VCRat2.6 Subarea Edit Window

Find data for yield adjustment of hydrograph (fattening).

GIS shapefile in hydrology manual shows 100-yr 1-day rainfall at project site is 8 inches. Exhibit 14a,b shows CN for soil type 3, industrial land use is 96. Enter this in reservoir routing portion of subarea:

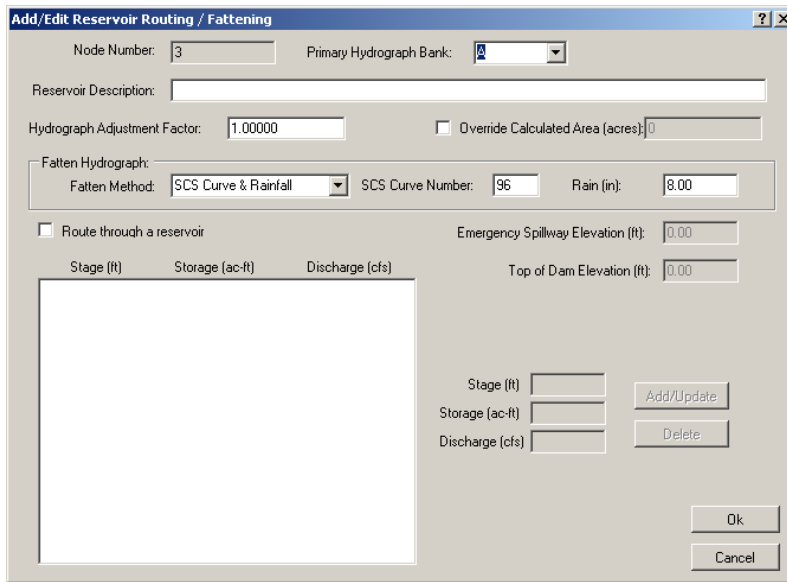


Figure D-2.3 VCRat2.6 Subarea Reservoir Routing Edit Window

Use split flow option in VCRat2.6 to reduce hydrograph peak and volume to represent project area only:

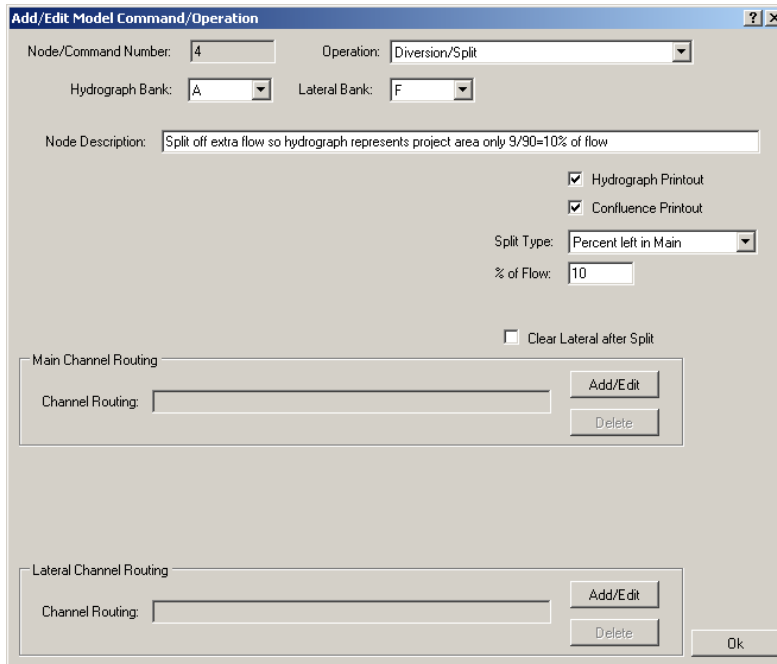


Figure D-2.4 VCRat2.6 Subarea Edit Window

VCRat Model Results (see attached)

90 Ac Subarea Results

Mitigation Q10 peak for 90 ac subarea= 175 cfs

Q100 developed condition peak for 90 ac subarea= 275 cfs

Hydrograph volume before fattening: 53.08 ac-ft

Hydrograph volume after fattening: 56.39 ac-ft

Hydrograph volume remaining in line A after splitting off 90%= 56.39/10=5.64 af

9 Ac Project Area Results

Mitigation Q10 peak for 9 ac project area = 175/10= 17.5 cfs

Q100 developed condition peak 9 ac project area= 27.5 cfs.

Hydrograph volume remaining in line A after splitting off 90%= 56.39/10=5.64 af

Calculate preliminary basin design info based on results:

From 20001 VC Stormwater Quality Management Program		
Basin Design Criteria		Highlighted cells require user input
Dmax- Basin Max Depth ft	$0.1 \cdot A^{.5}$	
A= Area sf		basin bottom width x basin bottom length
L/W		length to width ratio of 2:1 or greater
Bott Width ft		Minimum Bottom width 30 ft
Z ratio, sideslopes		sideslopes 3:1 H:V desired
Basin vol af	0.56394	.10*inflow vol
Basin Vol cf	24565.23	
Vol/ac Ratio	2729.47	
Sideslopes H:V	3	

Assumed bott Width ft	40	User Input
Calc bott Length ft	80	
Calculated Basin Depth Dmax	5.657	
Assumed Depth ft	5	User Input
Calc Top Width ft	70	
Calc Top Length ft	110	
Calculated Land Slope	0.045455	Basin Assumed Depth/Basin Top Length
Calc Vol w/ Assumed Depth cf	26125	SQDV

Single Orifice Outlet to meet 24hr drawdown time at 1/2 Depth

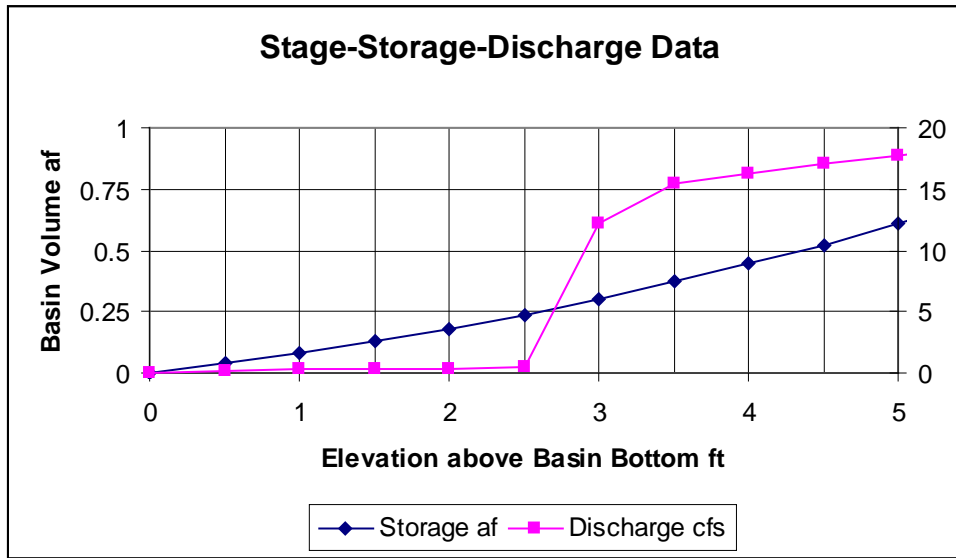
Db _s ft	4.95	Depth of water above centerline of the bottom perforation (assumed to be 0.05 ft above bottom)
c	0.6	orifice coefficient = 0.6
g ft/sec ²	32.2	gravity 32.2
T drawdown hrs	24	
Total Orifice Area sq in	8.128644	Orif Area sq in=(Basin Vol cf)/(60.19*Db _s ^{0.5} *T)
Orifice Area sf	0.056449	
Orifice Diam ft	0.13408	D=sqrt(4*A/pi)
Max Discharge cfs full depth	0.604717	c*a*sqrt(2gh)
Discharge Time for max Q sec	43,202	Basin Volume/Orifice Discharge
Discharge Time for max Q hrs	12.0	

Stage-Storage-Discharge Calculations

Design a basin with the following assumptions:

1. Single orifice of 0.0564 sf area at bottom of basin as above
2. Circular riser of 1.5 ft radius with top elevation of 2.5 ft above basin bottom
3. Culvert outlet 50 ft long with elevation drop of 1 ft (slope=0.02 ft/ft) with entrance, exit, and friction losses included in calcs. Outlet controls flow with radius of 0.75ft.

Based on these assumptions, the stage storage discharge data are:



These data are then inserted into the VCRat2.6 run as follows:

Add/Edit Reservoir Routing / Fattening

Node Number: Primary Hydrograph Bank:

Reservoir Description:

Hydrograph Adjustment Factor: Override Calculated Area (acres):

Fatten Hydrograph:
Fatten Method:

Route through a reservoir

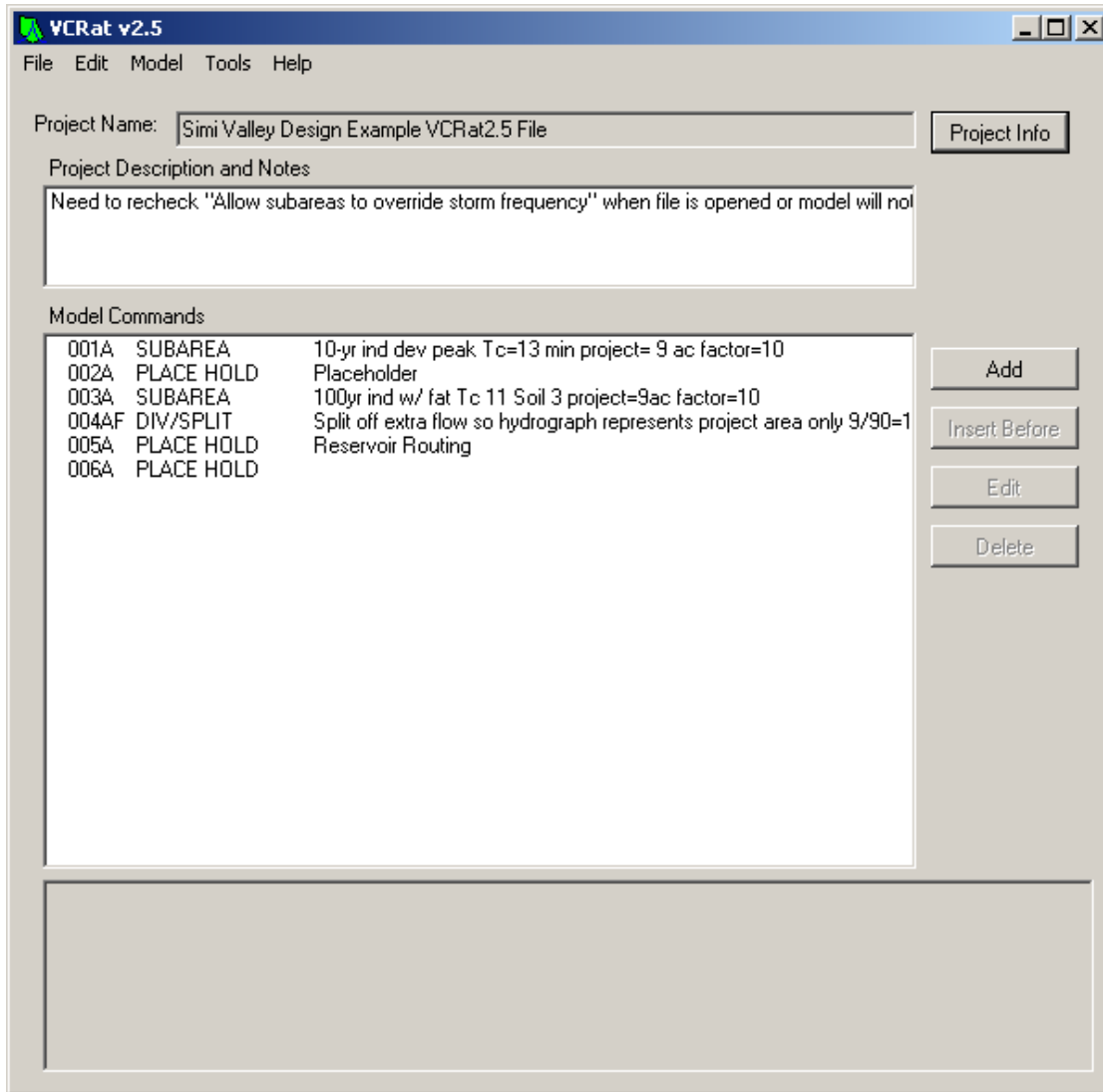
Emergency Spillway Elevation (ft):
Top of Dam Elevation (ft):

Stage (ft)	Storage (ac-ft)	Discharge (cfs)
0.00	0.00	0.00
0.50	0.04	0.19
1.00	0.08	0.27
1.50	0.13	0.33
2.00	0.18	0.38
2.50	0.24	0.43
3.00	0.30	12.13
3.50	0.37	15.37
4.00	0.44	16.21
4.50	0.52	17.00
5.00	0.61	17.75
5.50	0.70	18.48
6.00	0.80	19.18
6.50	0.90	19.85
7.00	1.01	20.50
7.50	1.13	21.13
8.00	1.26	21.74

Stage (ft) Add/Update
Storage (ac-ft) Delete
Discharge (cfs)

Ok
Cancel

The full model input file is:



The model output file is provided in Appendix E. A summary of the results are:

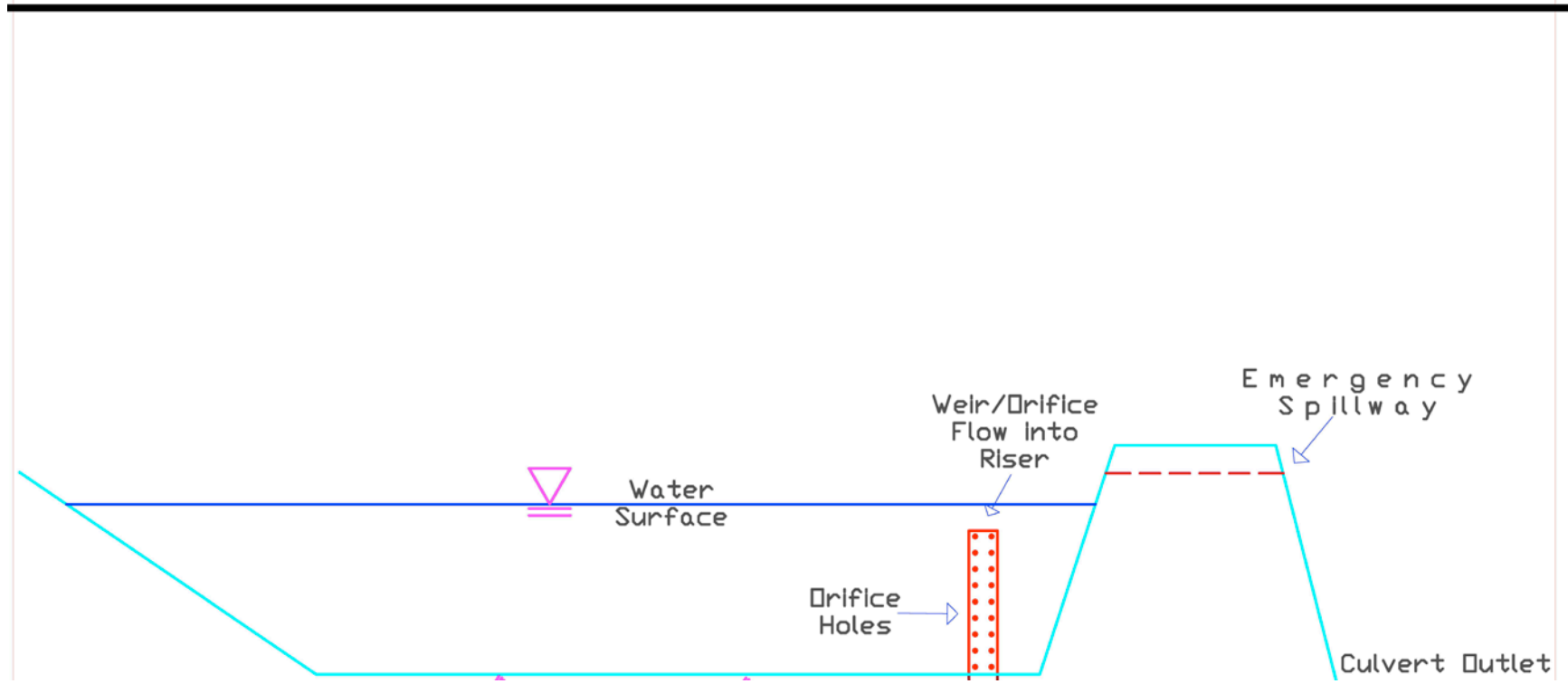
```

INCOMING HYDROGRAPH PEAK (cfs):      27.55          VOLUME (acre-ft):      5.64      *
NO HYDROGRAPH ADJUSTMENT
NO HYDROGRAPH FATTENING
RESERVOIR INFLOW:    PEAK (cfs):      27.55 @ 1154  VOLUME (acre-ft):      5.64      *
MAXIMUM ELEVATION:  STAGE (ft):      4.10 @ 1162  VOLUME (acre-ft):      0.46      *
EMERGENCY SPILLWAY: ELEV (ft):      5.00          VOLUME (acre-ft):      0.61      *
    DIFFERENCE:    IN STAGE (ft):      -0.90          IN VOLUME (acre-ft):      0.15      *
NO SPILL EXPECTED.  PERCNT OF VOLUME REMAINING TO SPILLWAY:  25.2%    *
TOP OF DAM:         ELEV (ft):      6.00          VOLUME (acre-ft):      0.80      *
DIFFERENCE    IN STAGE (ft):      -1.90          IN VOLUME (acre-ft):      0.34      *
NO OVERTOP EXPECTED.  PERCNT OF VOLUME REMAINING TO TOP OF DAM:  43.0%    *
RESERVOIR OUTFLOW:  PEAK (cfs):      16.37 @ 1162  VOLUME (acre-ft):      5.16      *
    
```

The results show that with this design, the maximum depth in the basin is expected to be 4.1 ft with a volume of 0.46 af.



Cross-Section View



APPENDIX D

DETENTION BASIN DESIGN

Ventura County Watershed Protection District
 Modified Rational Method Hydrology Program (VCRat v2.6)

VCRat version: 2.6.2009.7
 VCRain version: 200703
 DOS EXE version: PC 2.2-200604

Ventura County Watershed Protection District
 Modified Rational Method Hydrology Program (VCRat v2.5)

Job: 1 Project: 1000 Oaks Watershed Detention Study

Page: 2

Model Results

SUBAREA DATA AND RESULTS							ACCUMULATED DATA			ROUTING AFTER ACCUMULATION								
NODE ID	SOIL TYPE	RAIN ZONE	TC (MIN)	% IMP	AREA (AC)	FLOW (CFS)	AREA (AC)	FLOW (CFS)	TIME (MIN)	CHANNEL TYPE	LENGTH (FT)	SLOPE (FT/FT)	SIZE (FT)	H:V (Z)	N VALUES		VEL (FT/S)	DEPTH (FT)
															CHNL	SIDES		
1A : Calleguas 2000 Condition 10-yr results																		
1A : Clearing Hydrograph Bank: A																		
1A	030	K10	13	23	40	68	40	68	1154	-----	----	-----	---	----	-----	-----	--	--
2A : Placeholder																		
2A	---	---	--	--	---	---	40	68	1154	-----	----	-----	---	----	-----	-----	--	--
3A : Calleguas 2000 Condition Model incorporating project area 100-yr																		
3A : Clearing Hydrograph Bank: A																		
3A	030	K100	11	23	40	113	40	113	1154	-----	----	-----	---	----	-----	-----	--	--
4A : Placeholder																		
4A	---	---	--	--	---	---	40	113	1154	-----	----	-----	---	----	-----	-----	--	--

Model output from VCRat2.6 run using Calleguas 2000 Condition Model data for subarea incorporating project area.

APPENDIX D

DETENTION BASIN DESIGN

VCRat2.6 Model Output for Design Example

Ventura County Watershed Protection District
 Modified Rational Method Hydrology Program (VCRat v2.6)

Job: 1 Project: 1000 Oaks Watershed Detention Study
 Model Results

SUBAREA DATA AND RESULTS							ACCUMULATED DATA			ROUTING AFTER ACCUMULATION											
NODE ID	SOIL TYPE	RAIN ZONE	TC (MIN)	% IMP	AREA (AC)	FLOW (CFS)	AREA (AC)	FLOW (CFS)	TIME (MIN)	CHANNEL TYPE	LENGTH (FT)	SLOPE (FT/FT)	SIZE (FT)	H:V (Z)	N VALUES		VEL (FT/S)	DEPTH (FT)			
1A : 10-yr ind dev peak Tc=13 min project= 9 ac factor=10																					
1A : Clearing Hydrograph Bank: A																					
1A	030	K10	13	70	90	175	90	175	1154	-----	----	-----	---	----	-----	----	---	----			
2A : Placeholder																					
2A	---	---	--	--	---	---	90	175	1154	-----	----	-----	---	----	-----	----	---	----			
3A : 100yr ind w/ fat Tc 11 Soil 3 project=9ac factor=10																					
3A : Clearing Hydrograph Bank: A																					
3A	030	K100	11	70	90	275	90	275	1154	-----	----	-----	---	----	-----	----	---	----			

* INCOMING HYDROGRAPH PEAK (cfs):							275.47		VOLUME (acre-ft):			53.08		*							
* NO HYDROGRAPH ADJUSTMENT *																					
* RUNOFF FACTOR(in):							7.52		TOTAL RAIN(in):			8.00		SCS Curve: 96		*					
* FATTENED HYDROGRAPH PEAK (cfs):							275.47		VOLUME (acre-ft):			56.39		*							

3A	---	---	--	--	---	---	90	275	----	-----	-----	---	----	-----	----	---	----	-----			
4AF: Split percent																					
4AF: Control/Diversion: 10% to channel A																					

* Combined Peak:							275 @ 1154		Q in A:			28		Q in F:		248		*			
* Peak in A:							28 @ 1154		Q in F:			248		Combined Q:		275		*			
* Peak in F:							248 @ 1154		Q in A:			28		Combined Q:		275		*			

4AF	---	---	--	--	---	---	90	28	1154	-----	----	-----	---	----	-----	----	---	----			
5A : Reservoir Routing																					
5A	---	---	--	--	---	---	90	28	1154	-----	----	-----	---	----	-----	----	---	----			

* INCOMING HYDROGRAPH PEAK (cfs):							27.55		VOLUME (acre-ft):			5.64		*							
* NO HYDROGRAPH ADJUSTMENT *																					
* NO HYDROGRAPH FATTENING *																					
* RESERVOIR INFLOW: PEAK (cfs):							27.55 @ 1154		VOLUME (acre-ft):			5.64		*							
* MAXIMUM ELEVATION: STAGE (ft):							4.10 @ 1162		VOLUME (acre-ft):			0.46		*							
* EMERGENCY SPILLWAY: ELEV (ft):							5.00		VOLUME (acre-ft):			0.61		*							
* DIFFERENCE: IN STAGE (ft):							-0.90		IN VOLUME (acre-ft):			0.15		*							
* NO SPILL EXPECTED. PERCENT OF VOLUME REMAINING TO SPILLWAY: 25.2% *																					
* TOP OF DAM: ELEV (ft):							9.00		VOLUME (acre-ft):			0.80		*							

APPENDIX D

DETENTION BASIN DESIGN

```

*          DIFFERENCE    IN STAGE (ft):    -4.90          IN VOLUME (acre-ft):    0.34          *
*          NO OVERTOP EXPECTED.    PERCENT OF VOLUME REMAINING TO TOP OF DAM:    43.0%          *
*          RESERVOIR OUTFLOW:    PEAK (cfs):    16.37 @ 1162    VOLUME (acre-ft):    5.16          *
*****

```

```

5A  ---  ---  --  --  ---  ---  90  16  1162  -----  ----  -----  ---  ---  ---  ---  ---  ---
6A  ---  ---  --  --  ---  ---  90  16  1162  -----  ----  -----  ---  ---  ---  ---  ---  ---

```

APPENDIX E – HYDROLOGY FILES**E-1 10-YR RAINFALL PDF MAP****E-2 25-YR RAINFALL PDF MAP****E-3 50-YR RAINFALL PDF MAP****E-4 100-YR RAINFALL PDF MAP****E-5 10-YR RAINFALL SHAPEFILES****E-6 25-YR RAINFALL SHAPEFILES****E-7 50-YR RAINFALL SHAPEFILES****E-8 100-YR RAINFALL SHAPEFILES****E-9 RAINFALL ZONES SHAPEFILES****E-10 UPDATED SOILS MAP SHAPEFILES****E-11 EXAMPLE VCRAT2.6 AND 2.2 FILES****E-12 DESIGN RAINFALL FREQUENCY AND HYETOGRAPH FILES****E-13 HEC-HMS DESIGN STORM MODEL INPUT FILES****E-14 HYDROGRAPH TRANSFORMATION SPREADSHEETS****E-15 PERMIT REPORT SUBMITTAL EXAMPLE****E-16 TC EXAMPLE FILES****E-17 FLOW-THROUGH BASIN DESIGN SPREADSHEET****E-18 FLOW-THROUGH BASIN MODEL FILES****E-19 SMALL AREA BASIN DESIGN SPREADSHEET****E-20 EXAMPLE AR FILES- VCRAT 2.2****E-21 VCRAIN.DAT FILES FOR VCRAT 2.2**

E-22 YIELD ADJUSTMENT EXAMPLE FILES

E-23 MULTIPLE AREAL REDUCTION FACTOR FILES FOR VCRAT 2.2

E-24 BYPASS BASIN VCRAT2.6 FILES

E-25 SMALL AREA HYDROGRAPH FILES

E-26 VCRAT2.2 PROGRAM

APPENDIX F ERRATA

Date	Page(s)	Description
June 15, 2011	3-16	Revise text to clarify that C is not calculated with $(I-f)/I$ as f varies with intensity uniquely for each soil type
June 15, 2011	A-8 to A-14	Revise C Coefficient vs Intensity figures in Appendix A to reflect the information about the variable f in the calculation