SANTA CLARA VALLEY WATER DISTRICT

SAN FRANCISQUITO CREEK HYDROLOGY REPORT

Prepared by

James Wang, PhD, PE
Engineering Unit Manager

Wendy Chang, PE
Associate Civil Engineer

Nahm Lee
Assistant Engineer

April, 2006

Revised December 2007

TABLE OF CONTENTS

SANTA CLARA VALLEY WATER DISTRICT

SAN FRANCISQUITO CREEK HYDROLOGY REPORT

1. INTRODUCTION

This report summarizes the work completed on the San Francisquito Creek hydrology. San Francisquito Creek is the northernmost creek within the Santa Clara County and is part of the Santa Clara Valley Water District's Lower Peninsula Watershed. Presented in the report are recommended design flows for San Francisquito Creek and its respective tributaries.

The following is a list of the major tributaries included in this study:

San Francisquito Creek Hydrology

- Bear Creek
- Los Trancos Creek
- San Francisquito Creek

2. HYDROLOGIC MODEL

This section provides a summary of the method and procedures developed and used for the hydrology study. Refer to the District's publication entitled "Hydrology Procedures (Saah et al, 2006)" for more detailed information.

2.1 Design Storm Precipitation

The design storm precipitation is obtained by using the weighted rainfall gage stations data which is checked against the District's Precipitation Global Regional Equations (Saah et al, 1996). The design storm precipitation is then applied to an appropriate rainfall-runoff model for estimating the flood runoff.

The following describes how the precipitation depth is obtained from the Global Regional Equations:

From the isohyetal map of mean annual precipitation (MAP), locate the specific location of the site and determine the MAP. Some interpolation may be required to obtain the MAP.

Given the mean annual precipitation for the ungaged site, the precipitation intensity is calculated as:

$$i_{T,D} = \frac{a_1 T^{a_2} \exp[SD_e^{\,2} / 2]}{D^{a_3}} \qquad \textbf{(1)}$$

Where:

$i_{T,D}$ = the predicted precipitation intensity in inches per hour or inches per day at return period T

T = return period in years as the recurrence intervals

D = duration D in hours or days

$[a_1, a_2, a_3]$ = model coefficients

SD_e = the standard deviation of the model residuals (random term).

The precipitation depth (in inches) $x_{T,D}$ can be obtained from the precipitation intensity $i_{T,D}$ by the following relation:

$$x_{T,D} = i_{T,D}.D \qquad (2)$$

For short duration rainfall of a 1% event (specifically for durations of 5 minutes, 10 minutes, up to as much as 24 hours), the model parameters $[a_1, a_2, a_3]$ and SD_e are as follows:

$$\begin{aligned} a_1 &= 0.2675 + 0.01199 \cdot MAP + 0.00002472 \cdot E \\ a_2 &= 0.167033 \\ a_3 &= 0.5853 - 0.004155 \cdot MAP - 0.000001096 \cdot E \end{aligned} \qquad (3)$$

SD_e=0.120039 for short duration data

Where:

MAP = the mean annual precipitation in inches
E = the elevation of the ungaged site in feet.

Note that the duration D for short duration rainfall analysis in Eq. (1) is in hours.

To illustrate the use of Eq. (1), the example of estimating the 24-hour, 100-year return period storm rainfall at the sub basin AA10 in the San Francisquito Creek watershed is used.
Given parameters are

T = 100-year
D= 24-hour (4)
SD_e=0.120039

And basin characteristics of sub basin AA10 are

E = 1126 feet
MAP =33.45 inches (5)

Substitute basin characteristics (5) into Eq. (3) then the coefficients are

$$\begin{aligned} a_1 &= 0.2675 + 0.01199 \times 33.45 + 0.00002472 \times 1126 = 0.696400 \\ a_2 &= 0.167033 \\ a_3 &= 0.5853 - 0.004155 \times 33.45 - 0.000001096 \times 1126 = 0.445081 \end{aligned}$$

Substitue the numerical parameters (4) and the coefficients into Eq. (1), then

$$i_{T,D} = \frac{0.696400 \times 100^{0.167033} \exp[0.120039^2 / 2]}{24^{0.445081}} = 0.367915 \quad \textit{inches per hour}$$

Apply Eq. (2) with this precipitaion intensity value, the precipitation depth of the 100-year, 24-hour storm rainfall for sub basin AA10 is therefore 8.83 inches as shown below.

$$x_{T,D} = 0.367915 \times 24 = 8.83 \;\; \textit{inches}$$

For sub basin AA10, the weighted rainfall gage stations depth is 8.88 inches which is close to the precipitation depth obtained from the Global equations. Refer to section 4 for further discussions of rainfalls.

2.2 Flood Flow Regression Model

Since the early 1970s, the District has utilized regional regression and correlation techniques to estimate design flows at ungaged locations. The regional regression equations are formulas consisting of flow information such as values from gaged stations as dependent variables and measurable watershed characteristics as independent variables. The application of these equations on ungaged locations will result in estimates of flood flow from any watershed for 1% or 10% design flows. Regional regression equations usually apply to rural watershed, and they are generally used for an initial flow rate range check.

The District updated the regional regression equations in 2003 to include data through 2000 based on historical data from Santa Clara County and Santa Cruz County. The outcomes of this study are documented in a report entitled *Development of Regional Regression Equations to Calculate Flood Quantiles in Santa Clara County* (Saah et al, 2003). The updated regional regression equations to estimate 24-hr peaks and 1-day volumes for both the 1% and 10% quantiles are as shown below:

FOR 24-hr PEAKS in cfs:

$$Q_{1\%} = 11.22 \times A^{0.954} \times MAP^{1.03}$$
$$Q_{10\%} = 2.985 \times A^{0.988} \times MAP^{1.173}$$

Where:

A = the watershed area in square miles
MAP = the mean annual precipitation in inches

FOR 1-day VOLUMES in cfs :

$$V_{1\%} = 2.254 \times A^{0.964} \times MAP^{1.187}$$
$$V_{10\%} = 0.895 \times A^{0.933} \times MAP^{1.244}$$

Where:

A = the watershed area in square miles
MAP = the mean annual precipitation in inches

Note: The 24-hr volume may be approximated by multiplying the 1-day volume by a factor of 1.15.

2.3 Loss Rates

The loss rates for the rural and pervious parts of sub-watersheds are calculated using the Soil Conservation Service (SCS) Curve Number (CN) method. For all impervious areas, loss rates are assumed to be minimal. The hydrologic soil type, the Antecedent Moisture Content (AMC) and the ground cover are defined for each sub-watershed and are used to determine the respective Initial Abstraction (Ia) and the CN values.

The Antecedent Moisture Conditions are further calibrated for various watershed conditions and various frequencies of occurrences using the flood volumes obtained from the regression equations as well as the rainfall-runoff model. Based on this information, the Ia and CN values for the updated AMC are calculated and input into the HEC-1 model. A map of hydrologic soil types for each watershed is included in the appendices.

2.4 Clark's Synthetic Hydrograph Parameters (Tc and R)

The unit hydrograph parameters applicable to this study are calculated and presented for each creek. Rural, pervious and impervious parts of each sub-watershed are considered separately. The Time of Concentration (Tc) is calculated using Kirby Hathaway's formula:

$$Tc = 0.01377[(L * n)^{0.47} S^{-0.235}]$$

Where:

L=length of overland flow in feet (ft)
n=Manning's watershed roughness coefficient
S=average slope in ft/ft

The Routing Coefficient (R) is calculated based on an acceptable routing indicator: R/(Tc+R). This indicator directly impacts the peaking characteristics of hydrographs. For rural and pervious sub-sub-watersheds, the indicator is ranging between 0.5 and 0.9 based on the calibration process with regression quantiles. For impervious sub-sub watersheds, the indicator is generally ranging between 0.1 and 0.5.

2.5 Urban Hydrology

In 1996, an urban hydrology procedure (Wang and Saah 1996) was developed which addressed the impact of urban growth on flood flows. This procedure accounts for the effects on runoff due to two major urban changes: increased imperviousness and increased channelization. Increased imperviousness reduces the overland flow travel time and thus increases the volume of flow. Increased channelization addresses the impact of conveyance through gutters and storm drains together with the increased storage capability of these facilities. Imperviousness represents coverage from streets, buildings and other lot coverage. The coverage from streets in urban residential areas ranges from 2% to 25%, while for other land uses the value can be as high as 95% of the total lot area. The concept of "Equivalent Street" is obtained from the land use requirements for ratio of streets as a part of the total urban sub-watershed. Based on this equivalent street concept, the length and width of streets in an urbanized area are defined and, hence, the unit hydrograph parameters are calculated. The following are the formulas used in this study to calculate unit hydrograph parameters as input to HEC-1.

The following describes how equivalent street length and time of concentration are obtained in urban areas:

Equivalent street length (all measurements are in ft or square ft) is calculated by the equation:

$$L_{st}=A_{st}/W_{st}$$

Where:

L_{st} = the equivalent street length
A_{st} = area of streets, (from land use guidelines)
W_{st} = width of streets (from traffic guidelines)

Overland flow length is calculated by the equations:

$$L_i = (A_i - A_{st})/2L_{st}$$
$$L_p = A_p /2L_{st}$$

Where:

L_i = length of overland flow of impervious area
A_i = impervious area, (from land use guidelines)
L_p = length of overland flow of pervious area
A_p = pervious area

The impervious length of overland flow is given by the equation $L_{imp} = L_i + L_{Cb}$ where L_{Cb} is the length of flow to the first catch basin (normally less than 300 ft), and L_i is defined above. The pervious length is given by the equation $L_{perv} = L_p + L_{imp}$.

The Time of Concentration (Tc) is calculated separately for pervious and impervious areas using Kirby-Hathaway's formula as defined in Section 2.4. Time of Concentration for impervious areas is calculated as:

$$Tc_{imp} = Tc_i + Tc_{cb}$$

Where:

Tc_{imp} = Time of Concentration for the impervious area
Tc_i = Time of Concentration of overland flow over impervious area
Tc_{cb} = Time of Concentration of flow length to first catch basin

Time of concentration for pervious areas is calculated as:

$$Tc_{perv} = Tc_p + Tc_{imp}$$

Where:

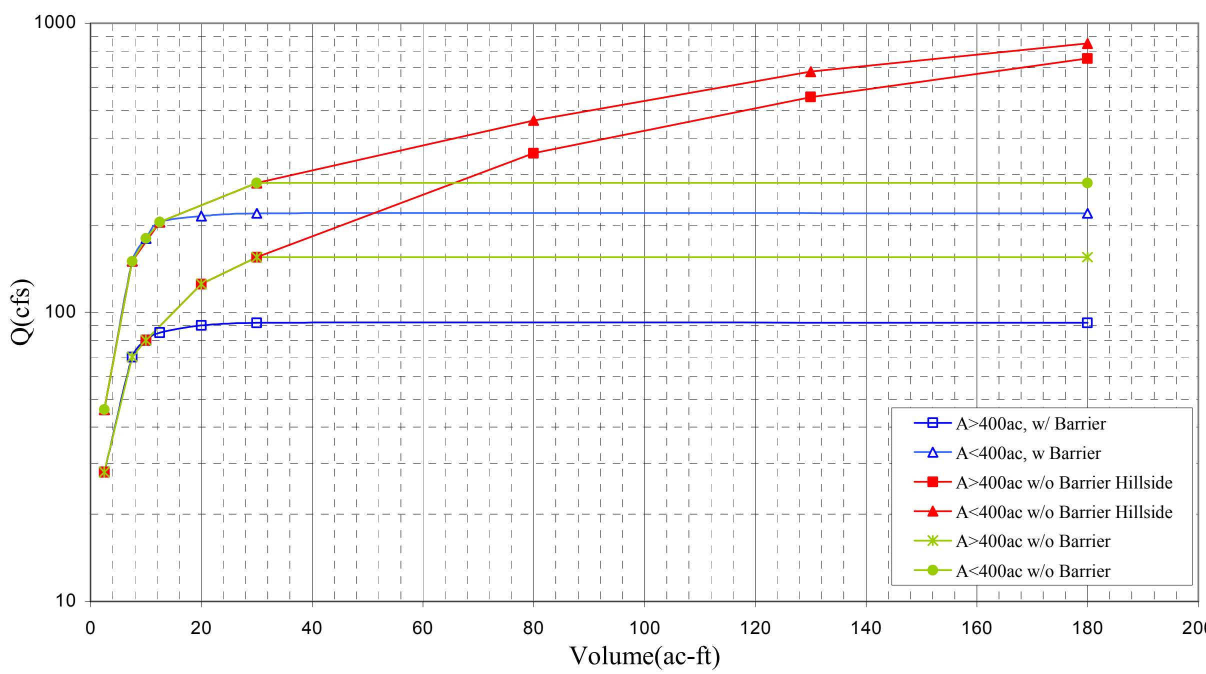
Tc_{perv} = Time of Concentration for pervious area
Tc_p = Time of Concentration of overland flow over pervious area
Tc_{imp} = Time of Concentration of the impervious length of overland flow

2.6 Routing Procedures

Most of the flood waters from urban areas are conveyed to flood control channels via storm drain systems. Storage-discharge rating curves based on Santa Clara Valley's storm drain system for average condition are presented in Figure 1. It shows the general unitized (prorated to one square mile of an area) storage-discharge rating curves that may be applied to valley urban areas. However, to minimize the impact from diversified design frequencies and/or criteria for the existing storm drain system, it is recommended that more detailed analysis of the storm drain storage-discharge relationship be performed for the specific project if a higher degree of accuracy is desired. The combined pervious and impervious inflow hydrographs for a study area are routed through the storm drains using the modified puls routing method. The storage-discharge relationship for that area is calculated from a unitized relationship and applied to obtain the outflow from the storm drain system. The storage routing usually consists of two types of boundary conditions: namely, "with barrier" (eg. berms, levees, houses and etc.) and "without barrier" conditions. The "with barrier" conditions can be found in the lower parts of a watershed, where the lay of the land has flat or mild slopes (generally less than 0.02). Here, the urban runoff can only reach the creeks through the storm drain system, without the possibility of overland connections. The "without barrier" condition generally exists at the upper part of a watershed where slopes are steep (generally greater than 0.02), or in areas without flood control improvements. Runoff water from these areas without barriers normally finds its overland course and eventually reaches the creek channels.

For channel routing, Muskingum-Cunge Routing method was used. Refer to Appendix A-9 for the channel routing parameters applied.

Figure 1. General Unitized Storage-Discharge Rating Curve (prorated to one square mile of an area)



3. DESIGN HYDROGRAPH ESTIMATION FOR RURAL AND URBAN WATERSHEDS

The hydrologic modeling tool adopted for this study is the HEC-1 Flood Hydrograph Package developed by the U.S. Army Corps of Engineers (1990).

The HEC-1 model has several optional procedures to simulate the various components of the rainfall-runoff process in a watershed. Based on previous modeling studies of rural and urban watersheds around the Santa Clara Valley, the approach adopted by the District in the use of the HEC-1 model for rainfall-runoff modeling is summarized as follows:

- The land use is based on the generalized land use information from Association of Bay Area Governments (ABAG 1999), Santa Clara County Parcel Data (2001), and Santa Clara County Ortho Photos (2001).
- The SCS curve number is calculated based on Hydrologic Soil Groups from Soil Survey of San Mateo County by United States Department of Agriculture, Natural Resources Conservation Service (1998) and Santa Clara County published by the United States Department of Agriculture, Soil Conservation Service.
- The longest flow path is defined as a sum of the main channel length and overland flow length. The slope for the basin is calculated using the elevation difference divided by the longest length.
- A watershed boundary is delineated using the 7.5-minute Digital Elevation Model available from USGS. For rural areas, the boundary follows the contour lines. For urban areas, the boundary follows the street, storm drain system and the contour.
- *SCVWD Maps of Flood Control Facilities and Limits of 1% Flooding* is utilized for geometric elements of the channel sections.

4. SAN FRANCISQUITO CREEK EXISTING HYDROLOGY

San Francisquito Creek drains the eastern part of Santa Cruz Mountains between Kings Mountain and Russian Ridge. The watershed is a funnel shape, with the upper portion stretching along the San Andreas Fault in the north—south direction. The lower part of the watershed, east of Highway 280, is narrow and mostly urban.

Tributaries to San Francisquito Creek are:

1. Bear Creek
2. Alambique, Martin, Sausal and Corte Madera Creeks. These creeks are located in San Mateo County and drain into Searsville Lake.
3. Los Trancos Creek forms the northern boundary of Santa Clara County. It joins San Francisquito Creek at a location downstream of Searsville Lake, approximately one-half mile north of Highway 280.

Searsville Lake was built in 1892 with top of elevation at 338 feet, and retrofitted in 1928 with top of elevation at 342 feeet. Based on information obtained from a meeting on March 8, 2006 with Philippe Cohen, director of Jasper Ridge Conservatory, the dam used to have 1500 Ac-Ft storage in 1892. However, due to years of sediment from Corte Madera Creek, its storage has significantly reduced to about 150 Ac-Ft.

The average slopes of the tributary creeks range from 100 to 160 feet/mile, whereas the slope of the lower portion of San Francisquito creek downstream of Alpine Road ranges from 10 to 40 feet/mile.

The San Francisquito Creek Basin has warm dry summers and mild wet winters. The mean annual precipitation ranges from 14.5 inches near the Bay to 41 inches near Skeggs Point in the Santa Cruz Mountains. The highest point in the watershed is on Borel Hill (Elevation 2,570). The distribution of vegetative cover follows the mean annual precipitation pattern. It ranges from forest cover on top of the Santa Cruz Mountains where rainfall is highest to pasture land at lower elevations. The flat land is mainly urban with densities ranging from very low residential in the upper watershed to high density residential and commercial in the lower watershed.

There are many rainfall stations that collectively represent the precipitation patterns on the watershed. These stations are listed below:

Gage Station Number	Gage Station Name	24 hrs 1% Rainfall Depth (inches)	24 hrs 10% Rainfall Depth (inches)
15	San Gregorio 2 SE, San Mateo County Recording	7.63	4.70
17	Pilarcitos, San Mateo County Recording	10.13	6.49
24	Dahl Ranch, S.C.V.W.D. Recording	6.67	4.85
129	Palo Alto Reclamation Plant, S.C.V.W.D. Recording	3.37	2.50

The weighted rainfall from the surrounding precipitation stations were applied to the rainfall-runoff model. These rainfalls are closely in comparisons with the estimates obtained from the Global Equations. The rainfall input range for 24-hr duration varies from 10.1 inches (1%) and 6.5 inches (10%) for sub-basin AA12 to 4.8 inches (1%) and 3.2 inches (10%) for sub-basin O.

Appendix A contains tables and figures that represent the hydrologic characteristics and design flows for San Francisquito Creek.

5. SAN FRANCISQUITO CREEK 50- YEAR FUTURE HYDROLOGY

For 50-year Future hydrology, it is assumed that sub-basins downstream of I-280 will be fully developed. Followings are the assumptions made on the urbanization with the value of percent of imperviousness:

- If the existing sub-basin presently consists of urban and rural area, the rural area will be assumed fully developed in 50 years, and the percent of imperviousness of the then rural area will be increased to the same level as the urban area:

  a. Sub-basin F will be divided into two sub-basins, namely F0 and F1. Sub-basin F0 (upstream of I-280) will have the same landuse as that of the existing condition. Sub-basin F1 (downstream of I-280) will be fully urbanized with 50% imperviousness.

  b. Sub-basin G6 will be fully urbanized with a 50% imperviousness.

  c. Sub-basin H has 80% rural area at present. Those rural areas will be developed into urban area with 60% imperviousness.

  d. Sub-basin J has 50% rural area at present. Those rural areas will be developed into urban area with 50% imperviousness.

- If existing sub-basin is already fully urbanized, then the current percent of imperviousness will assume to be increased by 10% as shown in the table below.

Sub-basin	Existing Impervious Area Ratio	50 years Future Plan
I	85%	95%
K	95%	100%
L	85%	95%
M0	55%	65%
M1	90%	100%
N	60%	70%
O	65%	75%

6. COMPARISON BETWEEN EXISTING AND 50- YEAR FUTURE HYDROLOGY

Following Table presents the comparison between the existing and future 100-year flowrates for San Francisquito Creek:

Location	Drainage Area (mi^2)	Existing (cfs)	50 Years Future (cfs)
Bear Creek u/s San Francisquito Creek	11.85	3,200	3,200
San Francisquito Creek u/s Lake Searsville	14.65	4,100	4,100
San Francisquito Creek d/s Lake Searsville	14.65	4,100	4,100
San Francisquito Creek d/s Bear Creek	26.50	7,300	7,300
San Francisquito Creek u/s Los Trancos Creek	29.61	7,600	7,700
Los Trancos Creek u/s San Francisquito Creek	7.65	1,200	1,200
San Francisquito Creek d/s Los Trancos Creek	37.26	8,800	8,900
San Francisquito Creek @ USGS 11164500	37.62	8,800	8,900
San Francisquito Creek @ El Camino Real	41.20	9,200	9,400
San Francisquito Creek @ US 101	44.55	9,300	9,500
San Francisquito Creek @ Palo Alto Airport of Santa Clara County	46.17	9,400	9,600

The 50-year future flow rate for San Francisquito Creek downstream of Los Trancos Creek increases about 100 cfs (from 8,800 cfs to 8,900 cfs), which is about 1%; the 50 year future flow rate for San Francisquito Creek at Palo Alto Airport of Santa Clara County increases about 200 cfs (from 9,400 cfs to 9,600 cfs), which is about 2%. The increase in 50-year future flow rate is considered insignificant. The main reason for the small increase is attributed to the storm drain system in the urban areas normally throttles the flow due to its limited conduit capacity. Flood waters routed through such a storm drain system often result in lagging of hydrographs due to changes in timing and magnitude of flow.

REFERENCES

Saah A.D., J. Wang, W. Chang and Nahm Lee, 2006. *Hydrology Procedures*, SCVWD, San Jose, CA.

Saah A.D., G.Q. Tabios, J. Wang and W. Chang, 2003. *Development of Regional Regression Equations to calculate Flood Quantiles in Santa Clara County*, SCVWD, San Jose, CA.

Saah, A. D. 1978. *Hydrologic Documentation for the Northwest Flood Control Zone,* Santa Clara County, SCVWD, San Jose CA.

Saah, A., G. Q. Tabios, W. Chang and B.S. Jaber, 1996. *Precipitation-Depth-Duration-Frequency Analysis and Modeling for Santa Clara County and Vicinity*, San Jose, CA. SCVWD.

United States Army COE, 1990. *Hydrologic Engineering Center, HEC-1 Flood Hydrograph Package, User's Manual*, Davis, CA. September.

United States Department of Agriculture, 1968. *Soil Conservation Services, Soil of Santa Clara County*, Berkeley, CA. August.

Wang, J. and A.D. Saah, 1996. *Development of Urban Hydrology Procedures for Urban Watersheds*, SCVWD, San Jose, CA. July.