FLOOD INSURANCE STUDY

SANTA CLARA COUNTY, CALIFORNIA
AND INCORPORATED AREAS
VOLUME 2 OF 4

COMMUNITY NAME	COMMUNITY NUMBER
CAMPBELL, CITY OF	060338
CUPERTINO, CITY OF	060339
GILROY, CITY OF	060340
LOS ALTOS, CITY OF	060341
LOS ALTOS HILLS, TOWN OF	060342
LOS GATOS, TOWN OF	060343
MILPITAS, CITY OF	060344
MONTE SERENO, CITY OF	060345
MORGAN HILL, CITY OF	060346
MOUNTAIN VIEW, CITY OF	060347
PALO ALTO, CITY OF	060348
SAN JOSE, CITY OF	060349
SANTA CLARA, CITY OF	060350
SARATOGA, CITY OF	060351
SUNNYVALE, CITY OF	060352
SANTA CLARA COUNTY (UNINCORPORATED AREAS)	060337

REVISED: February 19, 2014

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
06085CV002B

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Select Flood Insurance Rate Map panels for the community contain information that was previously shown separately on the corresponding Flood Boundary and Floodway Map panels (e.g., floodways, cross-sections). Former flood hazard zone designations have been changed as follows:

Old Zone	New Zone
A1 through A30	AE
B	X
C	X

Part or all of this FIS may be revised and republished at any time. In addition, part of this FIS may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS components.

Initial Countywide FIS Effective Date: May 18, 2009
Revised Countywide FIS Effective Date: February 19, 2014

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3.0 ENGINEERING METHODS

For the flooding sources studied in detail in the community, standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this FIS. Flood events of a magnitude, which are expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood, which equals or exceeds the 1-percent-annual-chance flood in any 50-year period, is approximately 40 percent (4 in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this FIS. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for each flooding source studied in detail affecting the community.

Flood hydrographs and peak flow rates for the 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance floods for streams studied by detailed procedures were based on rainfall-runoff computations and regional regression equations developed by the SCVWD (Reference 10).

The regional regression equations are based on the frequency statistics of the records of 20 stream gages in Santa Clara County and the surrounding area. The parameters used in the regional regression equation are the drainage area of the basin mean annual precipitation, characteristic drainage lengths of the basin, and slope of the main drainage course of the basin. With these parameters, the statistics of the peak flow rate and 24-hour flow volume can be determined through use of the regression equations.

Drainage areas were broken down into smaller subbasins. The HEC-1 computer program (Reference 11) was used with the SCVWD’s 24-hour storm pattern and storm depth to produce subbasin hydrographs. For rural areas, the hydrographs were balanced using HEC-1 to reflect both the peak flow rate and 24-hour volume as predicted by the regional regression equations. For urban areas, the hydrographs were based on runoff coefficients from the SCVWD urban hydrology methodology (Reference 12). Actual storm drain capacities were included for routing these hydrographs.

The effects of channel and valley (overbank) storage on floodflow rates were determined by developing storage-discharge relationships for reaches of each stream. The storage-discharge relationships were developed by computing a series of water-surface profiles for various flow rates and determining the storage in the reach for each outflow rate. Flood hydrographs from the smaller subbasins were combined and routed downstream using the Modified Puls routing procedure. For reaches outside the limits of detailed study, storage-discharge relationships were generally obtained from normal depth computations, most of which were developed previously by the SCVWD.

Flood hydrographs for streams studied by approximate methods were calculated only when required to complete the detailed study analysis. Relative flood magnitudes for other streams studied by approximate methods were based on historic information, existing hydrologic analyses, available watershed information, and field observations.

City of Campbell

There is no hydrologic data available at this time.

City of Cupertino

Stevens Creek-Reservoir, with a capacity of 3,800 acre-feet, was built in 1936. The reservoir's principal purpose is water supply, and any flood-control benefits are incidental. Reservoir storage for each of the four recurrence intervals was determined with a coincidental frequency analysis of storage level and inflow flood hydrograph.

Channel flow rates generally increase downstream with increase in drainage area. The flow rate for Calabazas Creek is reduced by capacity restrictions of the channel sections and bridge sizes. Excess flows were routed overland to a downstream subbasin.

Two stream gages near Cupertino were considered to possess an adequate record to be included in the stations used to develop the regional regression equations (Reference 22). Permanente Creek (1955-1975) and Calabazas Creek (1946-1975) were used.

The results of the gage analysis on Calabazas Creek were compared to the predictions of the regional regression equations and flow values from the USACE. The three sets of predicted flow rates were almost identical for the 1-percent-annual-chance flood-recurrence interval at this location. A log-Pearson Type III analysis (Reference 19) of the Permanente Creek gage records compared favorably with the predictions from the regional regression equations.

City of Gilroy

Uvas and Chesbro Reservoirs, with capacities of 10,000 and 8,090 acre-feet, respectively, provide the only regulation on Uvas and Llagas Creeks. The principal function of the reservoirs is water supply. The dams were not constructed for flood-control purposes. However, Uvas Reservoir does provide an incidental flood-control benefit.

Modified Puls routings were performed for each of the recurrence intervals. An appropriate starting reservoir level for each recurrence interval was determined by a coincidental frequency analysis, which was performed by the SCVWD.

Four stream gages in the area were considered to possess an adequate record: Bodfish Creek (1960-1975), Coyote Creek near Gilroy (1961-1975), Uvas Creek above the reservoir (1962-1975), and Uvas Creek at Morgan Hill (1931-1957). These records were analyzed by the log-Pearson Type III method of analysis (Reference 23) and included in the stations used to develop the regional regression equations.

The attenuation caused by Uvas Reservoir is the reason the peak flow rates for Uvas Creek decrease with increase in drainage area below the dam. Flow rates on Miller Slough at the railroad and Uvas Creek below Thomas Road decrease due to a channel capacity restriction. The resulting channel overflows were routed with normal depth computations.

Hydrologic data for the restudy were taken from the original 1976 Santa Clara County FIS and the study conducted by the SCVWD in April 1991 (Reference 192). The 1-percent-annual-chance peak discharges in both studies were developed by the SCVWD using the urban hydrology methodology (Reference 18) and regional regression equations (Reference 19).

Lions, Llagas, and North and South Morey Creeks, West Branch Llagas Creek (downstream of Day Road), Llagas Overbank (Old Miller Slough), and Miller Slough

The revised hydrology resulted in increases in base flood peak discharges along lions, Llagas, North and South Morey, and West Branch Llagas Creeks and a decrease in base flood peak discharge along Miller Slough. The decrease in base flood peak discharge along Miller Slough resulted from the decrease in drainage area caused by the construction of channel improvements along lions, North and South Morey, and West Branch Llagas Creeks.

Uvas Creek

Uvas Creek is a perched channel leveed on both banks for nearly the entire reach from the railroad to Thomas Road. Creek flows that overtop or breach the levees travel away from the main channel, and may or may not re-enter the creek farther downstream depending on the effects of manmade impediments to flow. Non-engineered levees, which consist primarily of topsoil that supports vegetation including large trees, have been created by agricultural interests to protect farmland.

City of Los Altos

Channel flow rates generally increase downstream with increase in drainage area. The flow rates for Hale, Permanente, and Adobe Creeks are reduced by capacity restrictions of the channel sections and bridge sizes. Excess flows were routed overland, downslope to an adjacent subbasin.

Flow rates for the upper portions of Adobe Creek generally matched those used by the USGS for the FIS for the Town of Los Altos Hills (Reference 24).

Town of Los Altos Hills

In an open-file report (Reference 25), the USGS derived flood-frequency relations on the basis of streamflow records. Peak discharges were computed for several recurrence intervals, up to 50 years, by fitting the log-Pearson Type III distribution (Reference 26) to observed annual peak flows and correlating the peak discharges with climatological and topographical parameters. According to the report, the most significant parameters were the drainage area and the mean annual precipitation. Regional relations, derived by multiple regression analysis, were of the form

$$Q_T = KA^aP^b$$

where: Q_T = Peak discharge (in cubic feet per second) for a recurrence interval of T years

A =Drainage area (in square miles)

P = Mean annual precipitation (in inches)

K, a, and b = Constants

Estimates of discharge for the 2-, 5-, 10-, 25-, and 50-year floods were computed by application of these regional relations for 25 sites in Los Altos Hills. Estimates of the 1-percent and 0.2-percent-annual-chance floods at these sites were then obtained by logarithmic extrapolation. The discharge values for the 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance floods were adjusted for the effects of development by methods described in the open-file report (Reference 25).

Town of Los Gatos

The three dams that exist on Los Gatos Creek are Lexington, with capacity of 20,210 acre-feet; Austrian, with capacity of 6,280 acre-feet; and, Vasona, with capacity of 410 acre-feet. The principal function of all three dams is water supply. These dams were not constructed for flood-control purposes. Lexington, however, does provide significant flood-control benefit.

Modified Puls routings were performed for each of the four recurrence interval floods. Vasona and Austrian Dams were considered full to the spillway level. Lexington Dam was subjected to a coincidental frequency analysis (References 27 and 28). As Lexington possesses a large reservoir capacity, it is unlikely that, on the average, the reservoir would be at spillway level when a large flood occurred. The coincidental frequency analysis, performed by the SCVWD, predicted the most appropriate starting reservoir level for each of the four recurrence interval floods.

Two stream gages in the area were considered to possess an adequate record: Los Gatos Creek (1930-1944), located 0.5 miles downstream from Lexington Reservoir, and Saratoga Creek (1934-1975), located at Springer Avenue, 0.7 mile downstream from diversion dam. Both stream gage records were analyzed by the log-Pearson Type III method of analysis (Reference 22) and included in the stations used to develop the regional regression equations.

The results of the gage analysis on Los Gatos Creek were compared to the predictions of the regional regression equations. The comparison was made on the peak flow rates just upstream of Lexington Dam. The two sets of predicted flow rates were almost identical at this location.

The attenuation caused by Lexington and Vasona Reservoirs is the reason the peak flow rates for Los Gatos Creek do not continuously increase with the increase in drainage area.

City of Milpitas

Tidal elevations in San Francisco Bay were developed by the USACE, San Francisco District (Reference 29). The 1-percent-annual-chance tide level of 12 feet was coordinated with the USACE.

Elevations for floods of the selected recurrence intervals on San Francisco Bay are shown in Table 7, “Summary of Stillwater Elevations.”

The 1-percent-annual-chance peak discharges used in the restudy were developed by the SCVWD using its urban hydrology methodology (Reference 18) and regional regression equations.

Arroyo De Los Coches

Topography and land-features mapping upstream of old Piedmont Road was supplemented by proposed improvement plans for Los Coches prepared by the SCVWD in 1973 (not built).

Berryessa Creek

Upstream of this study's limits, at Montague Expressway, the flow rate in Berryessa Creek is reduced to 800 cfs by a capacity restriction. Additionally, spills totaling approximately 2,000 cfs occur upstream of Arroyo De Los Coches. The 1-percent-annual-chance peak discharges for reaches downstream of the confluence with Arroyo De Los Coches reflect this 2,000 cfs loss, which occurs upstream of the confluence.

Calera Creek

Topography upstream of Interstate Highway 680/North Park Victoria Drive was supplemented by landscape plans for Higuera Adobe Park supplied by the City of Milpitas.

City of Monte Sereno

There is no hydrologic data available at this time.

City of Morgan Hill

Chesbro Reservoir, with a capacity of 8,090 acre-feet, is the only regulation on Llagas Creek. The principal function of the reservoir is water supply. The dam was not constructed for flood-control purposes. However, Chesbro Reservoir provides an incidental flood-control benefit.

Coyote and Anderson Reservoirs regulate the Coyote Creek outflow. Coyote Reservoir has a capacity equal to 23,700 acre-feet, and Anderson Reservoir has capacity equal to 91,300 acre-feet. Coyote Reservoir is intended to act as a water-supply source for storage in Anderson Reservoir. The principal function of Anderson Reservoir is water supply for ground-water recharge and irrigation. However, both reservoirs provide an incidental flood-control benefit.

Modified Puls routings were performed for each of the four flood-recurrence intervals. An appropriate starting reservoir level for each flood-recurrence interval was determined by a coincidental frequency analysis, which was performed by the SCVWD.

Four stream gages in the area were considered to possess an adequate record (Reference 19): Fisher Creek (1963-1975), Uvas Creek at Morgan Hill (1931-1957), Uvas Creek above the Uvas Reservoir (1962-1975), and Coyote Creek at Madrone (1925-1935). These records were analyzed by the log-Pearson Type III method of analysis (Reference 23) and were included in the stations used to develop the regional regression equations.

The attenuation caused by Chesbro Reservoir is the reason the peak flow rates for Llagas Creek decrease with an increase in drainage area below the dam. Flow rates on West Little Llagas Creek at Monterey Highway and Llagas Road decrease due to capacity restrictions at existing culverts. The spill at Monterey Highway was routed with normal depth computations to Llagas Creek. The spill at Llagas Road weired over Llagas Road, into the overbank, and out of the West Little Llagas Creek watershed, into the Fisher Creek watershed.

The 1-percent-annual-chance peak discharges used for this restudy were determined using urban hydrology methodology (Reference 18) and regional regression equations developed by the SCVWD. The discharge values given in Table 6, Summary of Discharges, reflect existing conditions in the watershed and take into account attenuation of overbank storage.

<u>City of Mountain View</u>

Channel flow rates generally increase downstream with an increase in drainage area. The flow rates for Stevens, Hale, Permanente, and Adobe Creeks are reduced by capacity restrictions of the channel sections and bridge sizes. Excess flows were pounded, then routed overland and downslope to an adjacent subbasin.

The Permanente Creek stream gage near Mountain View (1955-1975) was considered to possess an adequate record to be included in the stations used to develop the regional regression equations. A log Pearson Type III analysis of the gage records matched the predictions from the regional regression equations (Reference 30). Flow values for Adobe Creek matched the routed flow values from the SCVWD regional equations from an ongoing study by the USGS (Reference 31).

Analyses were carried out to establish the peak elevation-frequency relationships for each flooding source studied in detail.

The 1984 USACE report (Reference 3) summarizes the results of a tidal stage-frequency restudy of San Francisco Bay. The tidal data, as well as other tidal parameters presented in the report, reflect only stillwater conditions. It does not consider the effects of wave height or wave runup on the 1-percent-annual-chance water-surface elevation. Based on this report, the 1-percent-annual-chance water-surface elevation for San Francisco Bay in the City of Mountain View is 11 feet NAVD.

Elevations for floods of the selected recurrence intervals on San Francisco Bay are shown in Table 7, "Summary of Stillwater Elevations."

The 1-percent-annual-chance peak discharges used in the restudy were developed by the SCVWD using its urban hydrology methodology and regional regression equations (Reference 18). The flow rates reflect existing conditions in the watershed and take into account attenuation of overbank storage.

The floodplain boundary, as determined by the hydrologic and hydraulic analyses, was delineated on a 1": 200' scale topographic map for the study area.

City of Palo Alto

Stream gages are located on San Francisquito Creek (1930-1941, 1951-1978) and on Matadero Creek at El Camino Real (1953-1978). Log-Pearson Type III analyses (Reference 23) were performed on the gage records. In addition, extensions to the record for San Francisquito Creek were done by the SCVWD (Reference 32), the USACE, and Stanford University (Reference 33). The extended records were combined with the up-to-date stream gage records, and a log-Pearson Type III analysis was performed.

An extension to the record (1915-1975) on Matadero Creek was developed by the SCVWD (Reference 34). The extended record was supplemented with up-to-date stream gage information, and a Log Pearson Type III analysis was performed to determine peak discharges for selected recurrence intervals.

Peak flood estimates for the streams studied by detailed methods were also developed using the SCVWD regional regression equations. These equations are based on the frequency statistics of the records of 20 stream gages in Santa Clara County and surrounding areas. The regression equations provide estimates of the peak discharge and 24-hour flow volume for selected frequency floods. The parameters used in the regional regression equation consist of the drainage area of the basin, mean annual precipitation, characteristic drainage lengths of the basin, and slope of the main drainage course of the basin. With these parameters, the statistics of the peak flow rate and 24-hour flow volume can be determined through use of the regression equations.

The final peak flood discharges for selected recurrence intervals for Matadero Creek were developed using the discharge estimates from the extended gage records and those from the regional frequency analysis by application of the weighting procedures specified in the U.S. Water Resources Council "Guidelines for Determining Flood Flow Frequency" (Reference 23).

Peak flow rates for San Francisquito Creek at the stream gage were determined by application of the same weighting procedures. The peak flow rates developed from the three extended records and those developed from the regional regression equations were used in the weighting. Peak flow rates for Matadero Creek were determined by the same weighting procedure. The peak flow rates developed from the extended record and those developed from application of the regional regression equations were used in the weighting for Matadero Creek.

Drainage areas were broken down into smaller subbasins. The HEC-1 computer program (Reference 35) was used with the SCVWD 24-hour storm pattern and storm depth to produce subbasin hydrographs. These hydrographs were balanced using HEC-1 to reflect both the peak flow rate and the 24-hour volume as predicted by the regional regression equations.

For the San Francisquito Creek and Matadero Creek watersheds, the HEC-1 results were compared to the extended gage record analyses. For unurbanized subbasins, the peak flow rates and 24-hour volumes were further adjusted to enable the HEC-1 rainfall-runoff model to produce a favorable comparison to the peak flow rate based on the extended gage record analysis. For urbanized subbasins, the peak flow rates and hydrographs were based on the SCVWD urban hydrology methodology (Reference 18) with consideration of local storm drain capacity in routing these hydrographs to the stream channels. Capacities of bridges, culverts, and stream channels, as well as the effects of channel and valley storage on floodflow rates, were considered in developing the final flow rates and hydrographs.

In the restudy of Matadero Creek, a hydrologic analysis was performed to evaluate the previous study results, because nine more years of gage record are now available (1976-1984). Two flood frequency distributions were determined using log-Pearson Type III distribution: one for the period of record through 1975 and the other for the period of record through 1984. Comparison of the two flood frequency distributions showed that the flood discharges for various recurrence intervals were within 2 percent of each other. It was, therefore, concluded that the hydrographs and discharges used in the 1979 study are still valid and should continue to be used.

A hydrologic storage routing analysis, using the HEC-1 computer program, was performed to determine the ponding elevation for the 1-percent-annual-chance flood within the study area. The required elevation discharge-rating curve for the Railroad Bridge was developed from hydraulic computations. The elevation-storage volume curve for the ponding area was established from the floodplain topographic map, which was developed based on the floodplain elevation data (References 36 and 10). The inflow hydrograph for the hydrologic storage routing included the split flow from the adjacent Barron Creek upstream of the railroad and local inflow through storm drains, in addition to the stream flow from the upstream channel of Matadero Creek.

For unurbanized basins in the Barron and Adobe Creek watersheds, the results of SCVWD's regional regression equations were used to balance peak flows and 24-hour volumes.

Flow rates and hydrographs for urban subbasins were based on the SCVWD urban hydrology methodology (Reference 18). Local storm drain capacity was included in routing these hydrographs to the stream channels.

The effects of channel and valley (overbank) storage on floodflow rates were determined by developing storage-discharge relationships for reaches of each stream. The storage-discharge relationships were developed by computing a series of water-surface profiles for various flow rates and determining the storage in the reach for each outflow rate. Flood hydrographs from the smaller subbasins were combined and routed downstream using the Modified Puls routing procedure. For reaches outside of the limits of detailed study, routings were based on the Muskingum method with velocity of flow estimated.

Capacities of bridges, culverts, and stream channels were considered in developing the final flow rates. The perched nature of the watercourses does not allow for a continuous water-surface elevation across the channel and adjoining overbanks. As floodwaters rise above the banks, they flow away from, then generally parallel to, the channel's alignment. Flows in excess of capacity were routed overland and recombined with channel flows, where appropriate. Also, overland flows from one watercourse could combine with overland or channel flows from another watercourse. Such combinations were accounted for in the hydrograph routings and used in determining the flow rates for the four recurrence intervals.

Channel flow rates generally increase downstream. However, at several points, the flow is restricted by the capacity of the channel and/or culverts. Flows in excess of the channel capacity were routed overland into an adjacent subbasin.

Tidal elevations in San Francisco Bay were developed by the USACE, San Francisco District (Reference 3).

The 1984 USACE report summarizes the results of a tidal stage frequency restudy of San Francisco Bay. This report does not consider the effects of wave height or wave runup on the 1-percent-annual-chance water-surface elevation. Based on this report, the 1-percent-annual-chance water-surface elevation for San Francisco Bay in the City of Palo Alto is 11 feet NAVD.

Stage data from the USACE study reflected a static water condition that included wind set and any other hydrologic action that tended to build up stage levels, but not wave action.

Elevations for floods of the selected recurrence intervals on San Francisco Bay are shown in Table 7, "Summary of Stillwater Elevations."

Hydrologic analyses for the restudy were carried out to establish peak discharge frequency relationships for floods of the selected recurrence interval. Discharges for the main channel of San Francisquito Creek were determined using the USACE HEC-1 computer program (Reference 189). Routing was performed using the modified-Puls routing method. Volume-discharge parameters were determined by a multiple-discharge HEC-2 analysis. The overbank flows of San Francisquito Creek were calculated by split-flow analysis in the USACE HEC-2 model (Reference 188) and routing methods using the USACE HEC-1 computer program (Reference 189). Discharges for the main channel and overbank areas of San Francisquito Creek are shown in Table 6, Summary of Discharges.

City of San Jose

Flow rates and hydrographs for urban subbasins were based on the SCVWD urban hydrology methodology (Reference 18). Local storm drain capacities were considered in routing these hydrographs to the stream channels.

The effects of channel and valley (overbank) storage on floodflow rates were determined by developing storage-discharge relationships for reaches of each stream. The storage-discharge relationships were developed by computing a series of water-surface profiles for various flow rates and determining the storage in the reach for each outflow rate. Flood hydrographs from the smaller subbasins were combined and routed downstream using the Modified Puls routing procedure. For reaches outside the limits of detailed study, routings were based on the Muskingum method, with velocity of flow estimated.

Capacities of bridges, culverts, and stream channels were considered in developing the final flow rates. The perched nature of most of the watercourses does not allow for a continuous water-surface elevation across the channel and adjoining overbanks. As floodwaters rise above the banks, they flow away from, then generally parallel to, the alignment of the channel. Flows in excess of capacity were routed overland and recombined with channel flows where appropriate. Also, overland flows from one watercourse could combine with overland or channel flows from another watercourse. Such combinations were accounted for in the hydrograph routings and used to determine the flow rates for the four recurrence intervals.

Five reservoirs exist in the Guadalupe River basin and two in the Coyote Creek basin. These reservoirs, their dates of construction, storage capacities, and drainage areas are listed in Table 5, "City of San Jose Reservoirs."

Table 5 – City of San Jose Reservoirs

Reservoir	Date Constructed	Storage (Acre-Feet)	Drainage Area (Sq/. Miles)
Guadalupe River			
Almaden	1936	1,790	11.90
Calero	1936	10,160	6.96
Guadalupe	1936	3,740	5.97
Elsman	1951	6,280	9.79

Reservoir	Date Constructed	Storage (Acre-Feet)	Drainage Area (Sq/. Miles)
Lexington	1952	20,210	37.00
Coyote River Basin			
Coyote	1936	23,700	120.00
Anderson	1950	91,280	195.00

The reservoirs were included in the hydrologic routings. Their initial storage levels were determined by coincidental frequency analyses (Reference 28).

Channel flow rates generally increase downstream. However, at several points, the flow is restricted by the capacity of the channel and/or culverts. Flows in excess of the channel capacity were routed overland into an adjacent subbasin.

Four stream gages near San Jose were considered to possess an adequate record to be included in the stations used to develop the regional regression equations (Reference 19). A log-Pearson Type III analysis (Reference 26) was performed on the gage records for Calabazas Creek (1946-1975), Saratoga Creek (1934-1975), Ross Creek (1940, 1942, 1944-1963, and 1965-1975), and Upper Penitencia Creek (1962-1975). The results of the gage analysis for Calabazas Creek and Ross Creek matched the flow rates predicted by the regional regression equations and urban hydrology methods.

The results of the gage analysis on Saratoga Creek were compared with the predictions of the regional regression equations. The comparison was made on the peak flow rates. The 2-percent, 1-percent, and 0.2-percent-annual-chance floodflow from the gage analysis were slightly more than 10 percent above the flows derived from the regional equations. The 10-percent-annual-chance floodflow value from the gage analysis was 33 percent lower than the flow value from the regional equations. Because of the 41-year gage record with unregulated flow for the Saratoga Creek watershed, it was decided to use the frequency analysis of the gage to determine flow values at that point.

Frequency results from the analysis of streamflow records did not match the results predicted using the regional regression equations at the Upper Penitencia Creek gage. A procedure involving weighted averages (Reference 26) was used to develop the discharges used in this study.

Stream gages have been in operation on the Guadalupe River since 1929 and on Coyote Creek near Madrone from 1902 to 1912 and again from 1916. The water-supply reservoirs constructed above the gage precluded the systematic analysis of the gage records.

Tidal elevations in San Francisco Bay were developed by the USACE, San Francisco District (Reference 26).

Flood elevations for the selected recurrence intervals on San Francisco Bay are shown in Table 7, "Summary of Stillwater Elevations."

The 1-percent-annual-chance peak discharges used for the restudy were determined using the USACE HEC-1 computer program (Reference 189) and procedures and parameters developed by the SCVWD (References 139-141). The HEC-1 model developed for the effective FIS was modified to reflect the changes in land used within the watershed area using the SCVWD procedures.

Because the lands that have been developed are located at the downstream end of the watershed, the peak discharge from these areas will precede the peaks from the upstream undeveloped areas. As a result, the land development was determined to have a minimal effect on the peak discharges within the study area. The 1-percent-annual-chance peak discharges determined for this study are shown in Table 6, Summary of Discharges.

<u>City of Santa Clara</u>

The effects of channel and valley (overbank) storage on floodflow rates were determined by developing storage-discharge relationships for each stream's reaches. The storage-discharge relationships were developed by computing a series of water-surface profiles for various flow rates and determining the storage in the reach for each outflow rate. Flood hydrographs for the smaller subbasins were combined and routed downstream using the Modified Puls routing procedure. For reaches outside of the limits of detailed study, routings were based on the Muskingum method, with velocity of flow estimated.

Capacities of bridges, culverts and stream channels were considered in developing the final flow rates. The perched nature of the watercourses does not allow for a continuous water-surface elevation across the channel and adjoining overbanks. As floodwaters rise above the banks, they flow away generally parallel to the channel's alignment. Excessive flows were routed overland and recombined with channel flows where appropriate. Also, overland flows from one watercourse could combine with overland flows from another watercourse. Such combinations were accounted for in the hydrograph routings and used to determine the flow rates for the four recurrence intervals.

Five reservoirs exist in the Guadalupe River basin. These five reservoirs, along with their dates of construction, storage capacities, and drainage areas, are listed in Table 5, City of San Jose Reservoirs.

All five reservoirs are operated for water-supply purposes. A flood-control pool is not available at any of these reservoirs, and only an incidental flood-control function is available.

The reservoirs were included in the hydrologic routings. Their initial storage levels were determined by coincidental frequency analyses (Reference 28).

Channel flow rates generally increase downstream. However, at several points, the flow is restricted by the capacity of the channel and/or culverts. Flows in excess of the channel capacity were routed overland into an adjacent subbasin.

Two stream gages near Santa Clara were considered to possess an adequate record to be included in the stations used to develop the regional regression equations (Reference 38). A log-Pearson Type -III analysis (Reference 26) was performed on the gage records for Calabazas Creek (1946-1975) and Saratoga Creek (1934-1975). The results of the gage analysis for Calabazas Creek matched the flow rates predicted by the regional regression equations and urban hydrology methods.

The results of the gage analysis on Saratoga Creek were compared to the predictions of the regional regression equations. The comparison was made on the peak flow rates. The 2-percent, 1-percent, and 0.2-percent-annual-chance floodflow from the gage analysis were slightly more than 10 percent above the flows derived from the regional equations. The 10-percent-annual-chance floodflow value from the gage analysis was 33 percent below the flow value from the regional equations. Because of the 41-year gage record with unregulated flow for the Saratoga Creek watershed, it was decided to use the frequency analysis of the gage to determine flow values at that point.

The flow rates for the Guadalupe River reflect only that portion of the total flood discharge that remains within the leveed channel. Santa Clara is subject to flooding from a spill from the Guadalupe River upstream of State Highway 17. These waters flow as sheetflow through the San Jose Airport into the City of Santa Clara.

City of Saratoga

Flow rates and hydrographs for urban subbasins were based on the SCVWD's urban hydrology methodology (Reference 27). Local storm drain capacity was included in routing these hydrographs to the stream channels.

For the original study, two stream gages near the City of Saratoga were considered to possess an adequate record to be included in the stations used to develop the regional regression equations (Reference 38). A log-Pearson Type III analysis (Reference 23) was performed on the gage records for Calabazas Creek (1946-1975), located at Rainbow Avenue, approximately 1 mile north of Prospect Road, and Saratoga Creek (1934-1975), located at Springer Avenue, 0.7 mile downstream of a diversion dam. The results of the gage analysis for Calabazas Creek matched the flow rates predicted by the regional regression equations and urban hydrology methods.

The results of the gage analysis on Saratoga Creek were compared to the predictions of the regional regression equations. The comparison was made on the peak flow rates. The 2-percent, 1-percent, and 0.2-percent-annual-chance floodflow from the gage analysis were slightly more than 10 percent above the flows derived from the regional equations. The 10-percent-annual-chance floodflow value from the gage analysis was 33 percent below the flow value from the regional equations. Because of the 41-year gage record with unregulated flow for the Saratoga Creek watershed, it was decided to use the frequency analysis of the gage to determine flow values at that point.

For the original study, flood hydrographs for streams studied by approximate methods were calculated only when required to complete the detailed study analysis. Relative flood magnitudes for other streams studied by approximate methods were based on historic information, existing hydrologic analyses, available watershed information, and field observations.

The 1-percent-annual-chance peak discharges used for this study were determined using the USACE HEC-1 computer program (Reference 35) and procedures and parameters developed by the SCVWD (References 39, 40, and 41). The HEC-1 model developed for the original FIS was modified to reflect the changes in land used within the watershed area using the SCVWD procedures. The data and parameters used included the following:

- Watershed areas were developed and provided by the SCVWD (Reference 42).

- Current land conditions were estimated from Santa Clara County aerial photographs (Reference 183).

- A 24-hour storm pattern as developed by the SCVWD and used for the effective FIS was used. Total storm rainfall ranged from 6.35 inches at the lower end of the watershed to 8.33 inches at the upper end (Reference 42). The storm distributions are based on 15-minute time steps (Reference 40).

- A constant infiltration loss rate ranging from 0.02 inch per hour for fully developed areas to 0.11 inch per hour for undeveloped watershed was used.

- The Clark unit-hydrograph option of HEC-1 was used.

- The Clark unit-hydrograph times of concentration perimeter was calculated using the SCVWD methodology for the areas that have developed subsequent to the effective FIS. This method analytically separates the impervious and pervious areas within developed watershed subareas. The model was not modified for the areas that remain undeveloped (Reference 39).

- The Clark routing (storage) coefficient was based on the SCVWD guidelines and the effective FIS (Reference 39).

- Peak discharges and runoff volumes for the undeveloped watershed subareas were determined using the SCVWD regression formula (Reference 41).

- HEC-l model results were adjusted to match the regression formula peak and volume values for the undeveloped subarea using the program's hydrograph balancing routine.

- HEC-1 storage routing methods were used to evaluate storm drain storage and ponding. The storm drain values were based on the SCVWD guidelines (Reference 39).

City of Sunnyvale

The hydrologic analysis for Stevens Creek was based on 12 years of records available from the SCVWD (References 43, 44, and 45). The records available since construction of Stevens Dam were separated into two categories: (1) spill plus releases plus local inflow and (2) releases plus local inflow. Published records of Stevens Creek Reservoir storage were used to determine if the reservoir was full during the event that produced the annual maximum peak discharge (Reference 45). Seventeen spill-years were statistically analyzed using the methods described above. The rationale for using the 17 spill-years for analysis was based on the probability that flood-producing discharge could be expected to be generated from the area above the reservoir and the improbability of flood-producing discharges from reservoir releases plus local inflow below the dam.

A standard project flood was computed for the gage location, and the frequencies of the recorded annual maximums due to spills were obtained from the full data log-Pearson Type III analysis (Reference 26). These frequencies were adjusted to reflect the frequency of spill events. A frequency-flow rate curve was then constructed using the adjusted log Pearson Type III data.

For reaches downstream of the gaging station, hydrographs for the 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance floods were developed using rainfall-runoff computations. These computations were based on the unit hydrograph-loss rate method of hydrograph generation. Unit hydrographs were developed from a regional parametric "S" graph, while loss rates were developed from hydrograph reconstitutions of major events on the gage record. Rainfall amounts and temporal distributions were based on a statistical analysis of the 71-year record at the San Jose recording rain gage (Reference 46). Rainfall amounts were transposed to the basin by use of the ratio of normal annual precipitation in the basin to that at the rain gage.

No stream gage records are available on Calabazas Creek, Sunnyvale East Channel, and Sunnyvale West Channel. The drainage basins were broken into 15 smaller subbasins and rainfall-runoff computations were used to develop 10-

percent, 2-percent, 1-percent, and 0.2-percent-annual-chance flood hydrographs for each subbasin characteristic and regional loss rate functions.

Subbasin hydrographs were combined and routed downstream. The combining and routing operations considered the capacity of the storm drainage system in each subbasin, the capacities of the channels, the velocities of flow in the channels, the points and magnitudes of overflows from the channels, and the path and velocity of overland flows. Overland flows were caused by waters being unable to get into the storm drainage system and by overflows from the channels.

The Sunnyvale West Channel is a closed conduit in its upper portions. All flows in excess of the capacity of the pipe system travel downslope parallel to the channel until they pond north of U.S. Highway 101 and are slowly dissipated by the storm drainage system. Lower portions of the channel are subjected to backwater effects from San Francisco Bay.

Frequency-discharge-drainage area curves for the Sunnyvale East and West Channels show an erratic behavior pattern caused by locations of points of major inflows from the storm drainage system and locations of restricted channel capacity where overflows from the channel occur.

The 1-percent-annual-chance peak discharges used in the restudy were developed by the SCVWD using urban hydrology methodology and regional regression equations. The flow rates reflect existing conditions in the watershed, take into account attenuation of overbank storage, and consider the effects of storm drainage and pump systems in the area.

The city is served by independent storm drainage systems that intercept significant drainage areas and prevent flows from entering Sunnyvale East and West Channels. These flows are pumped directly into Guadalupe Slough.

<u>Santa Clara County (Unincorporated areas)</u>

Two stream gages near San Jose were considered to possess an adequate record to be included in the stations used to develop the regional regression equations (Reference 19). A log-Pearson Type III analysis (Reference 203) was performed on the gage records for Calabazas Creek (1946-1975) and Upper Penitencia Creek (1962-1975). The results of the gage analysis for Calabazas Creek matched the flow rates predicted by the regional regression equations and urban hydrology methods.

Six stream gages in the southern portion of the county were considered to possess an adequate record (Reference 19): Bodfish Creek (1960-1975), Coyote Creek near Gilroy (1961-1975), Coyote Creek at Madrone (1925-1935), Fisher Creek (1963-1975), Uvas Creek at Morgan Hill (1931-1957), and Uvas Creek above the reservoir (1962-1975). These records were analyzed by the log-Pearson Type III method of analysis (Reference 16) and were included in the stations used to develop the regional regression equations.

Frequency results from the analysis of streamflow records did not match the results predicted using the regional regression equations at the Upper Penitencia Creek gage. A procedure involving weighted averages (Reference 203) was used to develop the discharges used in this study.

Stream gages have been in operation on the Guadalupe River since 1929 and on Coyote Creek near Madrone from 1902 to 1912 and again from 1916. The water-supply reservoirs constructed above the gage precluded the systematic analysis of the gage records.

Peak discharge-drainage area relationships for the streams studied in detail are shown in Table 6, Summary of Discharges.

Channel flow rates generally increase downstream. However, at several points, the flow is restricted by the capacity of the channel and/or culverts. Flows in excess of the channel capacity were routed overland into an adjacent subbasin. A decrease in the flow rate on some streams resulted due to attenuation in the adjacent floodplain or an upstream reservoir.

Capabilities of bridges, culverts, and stream channels were considered in developing the final flow rates. The perched nature of most of the watercourses does not allow for a continuous water-surface elevation across the channel and adjoining overbanks. As floodwaters rise above the banks, they flow away from, then generally parallel to, the alignment of the channel. Flows in excess of capacity were routed overland and recombined with channel flows where appropriate. Also, overland flows from one watercourse could combine with overland or channel flows from another watercourse. Such combinations were accounted for in the hydrograph routings and used in determining the flow rates for the four recurrence intervals.

Five reservoirs exist in the Guadalupe River basin and two in the Coyote Creek basin. These reservoirs, their dates of construction, storage capacities, and drainage areas are shown in Table 5, City of San Jose Reservoirs.

There are also reservoirs in the southern part of the county. Chesbro Reservoir, with a capacity equal to 8,090 acre-feet, regulates Llagas Creek; and Uvas Reservoir, with a capacity equal to 10,000 acre-feet, regulates Uvas Creek.

Modified Puls routings were performed for each of the four flood-recurrence intervals. Appropriate starting reservoir level for each flood-recurrence interval was determined by a coincidental frequency analysis that was performed by the SCVWD.

The attenuation caused by Chesbro and Uvas Reservoirs is the reason the peak flow rates for Llagas and Uvas Creeks, respectively, decrease with an increase in drainage area below the dam. Flow rates decrease with an increase in drainage area, due to capacity restrictions caused by channel or bridge sizes.

Tidal elevations in San Francisco Bay were developed by the USACE (Reference 3). Elevations for the 1-percent-annual-chance recurrence interval flood on San Francisco Bay are shown in Table 7, Summary of Stillwater Elevations.

Flood hydrographs for streams studied by approximate methods were calculated only when required to complete the detailed study analyses. Relative flood magnitudes for other streams studied by approximate methods were based on historic information, existing hydrologic analyses, available watershed information, and field observation.

Coordination efforts for floodflow values and drainage areas for the southern portion of the county involved three separate agencies: the USACE, the SCVWD, and the USGS. No agency objected to the routed flow values as determined for existing conditions for this study.

Peak discharge-drainage area relationships for Santa Clara County streams are shown in Table 6.

As part of this restudy the following flooding sources were studied: Alamitos Creek, from the percolation pond to approximately 800 feet upstream of the Almaden Expressway; Watsonville Road Overflow Area, from its convergence with Llagas Creek to its divergence from West Little Llagas Creek; East Little Llagas Creek, from its confluence with Llagas Creek to the confluence of Madrone Channel and West Little Llagas Creek; Madrone Channel, from its confluence with East Little Llagas Creek to approximately 1.02 miles upstream of East Main Avenue; Middle Avenue Overflow Area, from its convergence with Llagas Creek to its divergence from West Little Llagas Creek; San Tomas Aquino Creek, from just upstream of Old Mountain View Aviso Road to just upstream of Monroe Avenue in the City of Santa Clara; Tennant Creek, from its confluence with East Little Llagas Creek to approximately 0.27 mile upstream of Fountain Oaks Drive; Uvas Creek, from the railroad to approximately Thomas Road; Uvas Creek - East Overbank above Highway 101, from Highway 101 to approximately 2,600 feet upstream; Uvas Creek - South Spill, from Bloomfield Avenue to approximately 3,450 feet upstream; West Branch Llagas Creek, from the NRCS, formerly the SCS, PI-566 interceptor project at Day Road to approximately 2,500 feet upstream of Coolidge Avenue; West Branch Llagas Creek - Lower Split, from the NRCS, formerly the SCS, PLS66 to approximately 650 feet upstream of Golden Gate Avenue; West Branch Llagas Creek - Middle Split, from approximately 2,200 feet downstream of Highland Avenue to Highland Avenue; West Branch Llagas Creek - Upper Split, from Highland Avenue to approximately 1,050 feet upstream of Coolidge Avenue; and West Little Llagas Creek, from its confluence with East Little Llagas Creek to approximately 0.35 mile upstream of Llagas Road.

The 1-percent-annual-chance peak discharges used for the restudy were determined using urban hydrology methodology and regional regression equations developed by the SCVWD. The discharge values shown in Table 6, Summary of

Discharges, for the restudied flooding sources reflect existing conditions in the watershed and take into account attenuation of overbank storage. Although new discharges were not computed for Alamitos, San Tomas Aquino, Uvas, and West Branch Llagas Creeks, the channel capacity has been recomputed.

As part of this restudy, Calabazas Creek was studied from the northern corporate limit at Prospect Avenue to Wardell Road, and Prospect Creek was studied from the confluence with Calabazas Creek to Prospect Avenue. Only a portion of Calabazas Creek is located in Santa Clara County. Prospect Creek is located entirely in the City of Saratoga.

In addition, an approximate total length of 1.5 miles of shallow flooding due to overtopping of Calabazas Creek was analyzed.

The 1-percent-annual-chance peak discharges used for this restudy were determined using the USACE HEC-1 computer program (Reference 189) and procedures and parameters developed by the SCVWD (References 39-41). The HEC-1 model developed for the previous FIS for Santa Clara County was modified to reflect the changes in land used within the watershed area using the SCVWD procedures. The data and parameters used included the following:

1. Watershed areas were developed and provided by the SCVWD (Reference 42).

2. Land conditions were estimated from Santa Clara County aerial photographs (Reference 183).

3. A 24-hour storm pattern as developed by the SCVWD, and used for the previous FIS, was used. Total storm rainfall ranged from 6.35 inches at the lower end of the watershed to 8.33 inches at the upper end (Reference 42). The storm distributions were based on 15-minute time steps (Reference 40).

4. A constant infiltration loss rate ranging from 0.02 inch per hour for fully developed areas to 0.11 inch per hour for undeveloped watershed was used.

5. The Clark unit-hydrograph option of HEC-1 was used.

6. The Clark unit-hydrograph times of concentration perimetry was calculated using the SCVWD methodology for the areas that have developed subsequent to the previous FIS. This method analytically separates the impervious and pervious areas within developed watershed subareas. The model was not modified for the areas that remain undeveloped (Reference 39).

7. The Clark routing (storage) coefficient was based on the SCVWD guidelines and the previous FIS (Reference 39).

8. Peak discharges and runoff volumes for the undeveloped watershed subareas were determined using the SCVWD regression formula (Reference 41).

9. The HEC-1 model results were adjusted to match the regression formula peak and volume values for the undeveloped subarea using the hydrograph-balancing routine in the program.

10. HEC-1 storage routing methods were used to evaluate storm drain storage and ponding. The storm drain values were based on the SCVWD guidelines (Reference 39).

Because the lands that have been developed are located at the downstream end of the watershed, the peak discharge from these areas will precede the peaks from the upstream undeveloped areas. As a result, the land development was determined to have a minimal effect on the peak discharges within the study area.

Revisions were made to reflect the effects of revised hydrology and the construction of a flood-control project in the Lower Llagas Creek watershed within the City of Gilroy and the unincorporated areas of Santa Clara County, California. The study was conducted by the SCVWD and issued by FEMA as a LOMR, dated August 31, 1995. The flood-control project consisted of the following:

- Channel improvements to West Branch Llagas Creek, including the reach formerly known as Ronan Channel, from its confluence with Llagas Creek to an interceptor channel just upstream of Day Road;

- Channel improvements to and realignment of Llagas Creek from approximately 1,625 feet upstream of Bloomfield Avenue to approximately 900 feet above its confluence with West Branch Llagas Creek;

- Channel improvements to and realignment of the entire reaches of North and South Morey Creeks; and

- Channel improvements to Lions Creek from its confluence with West Branch Llagas Creek to approximately 1,100 feet upstream of its confluence with an interceptor channel extending from approximately 2,700 feet east to approximately 200 feet east of Geri Lane.

The levee system constructed along Llagas Creek serves only to contain the base flood and does not eliminate any SFHAs inundated by other flooding sources.

The revised hydrology resulted in increases in base flood peak discharges along Lions, Llagas, North and South Morey, and West Branch Llagas Creeks and a decrease in base flood peak discharge along Miller Slough. The decrease in base flood peak discharge along Miller Slough resulted from the decrease in drainage area caused by the construction of channel improvements along Lions, North and South Morey, and West Branch Llagas Creeks.

Discharge-frequency relationships for the Pajaro River have been published in reports developed by the USACE, San Francisco District (References 199-200). A statistical analysis of stream-gage records for the Pajaro River produced discharge values similar to those determined by the USACE. The 1-percent-annual-chance peak discharge used in this restudy for this watercourse is from the USACE analysis.

Hydrologic methodology used by the USACE to develop a 1-percent-annual-chance peak discharge for the Pajaro River was based on statistical analysis of streamflow and precipitation records and runoff characteristics. The USGS stream-gaging station at Chittenden was used for the Pajaro River restudy.

A summary of the drainage area-peak discharge relationships for the streams studied by detailed methods is shown in Table 6, "Summary of Discharges."

New Hydrologic Analyses Included in This Revision

Santa Clara County includes Approximate Zone A and Detailed Zone AE studies. In this study, San Tomas Aquino Creek stream reach totaling 3,037 feet (0.575 mi) and an 8.27 square miles subbasin analyzed. In addition, two other streams, Coyote Creek and Upper Penitencia Creek, were analyzed, totaling 1.15 and 1.12 miles respectively, covering subbasin areas of 312.99 and 23.15 square miles.

According to the "USGS Water – Resources Investigation 77-21 (WRI 77-21) Magnitude and Frequency of Floods in California" (Reference 211), the most recent version of statewide regression equations, California is divided into six regions. Santa Clara County is located entirely in the WRI 77-21 - Region Central Coast area and, therefore, discharges for this study were computed using regression equations developed under this WRI.

Drainage area magnitude was calculated using GIS tools. The mean annual precipitation was calculated using the Mean Annual Precipitation Map from the Santa Clara County Drainage Manual, (Reference 212). The altitude index was calculated using the DEM derived from contours provided by the SCVWD, and following procedures outline in the WRI 77-21 to determine the elevation at the selected location for each basin.

The USGS National Streamflow Statistics (NSS) tool (Reference 213) was used to calculate the estimates for the peak discharge using the regional regression equations per WRI 77-21. The NSS input/output file is Santa Clara Hydrology.nss.

Peak discharges were calculated at selected recurrence intervals from the WRI 77-21 regression equations (Reference 211), adjusted values by urbanization, the SCVWD Hydrology Report (Reference 214), and the SCVWD 2003 regression equations (Reference 215)

Table 6 – Summary of Discharges

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
ADOBE CREEK					
Above Railroad (At El Camino Real)	8.50	1,350	2,500	2,700[1]	2,700[1]
At East Charleston Road	9.30	1,400[1]	1,400[1]	1,400[1]	1,400[1]
At East Meadow Drive	10.40	1,350	1,350	1,350	1,350
At Edith Road	6.86	1,000	1,830	2,140	2,700
At El Monte Avenue	5.14	690	1,340	1,700	2,370
At corporate limits	6.16	890	1,650	1,920	2,400
At Foothill Expressway	6.90	1,070	2,120	2,320	2,690
At Middlefield Road	9.30	1,020[1]	1,020[1]	1,020[1]	1,020[1]
At Moody Road	4.30	590	1,150	1,430	1,930
At Old Altos Road	6.55	960	1,760	2,050	2,490
At Pine Lane	7.00	1,110	2,150	2,360	2,730
At Railroad	8.50	1,350	1,450[1]	1,450[1]	1,450[1]
At U.S. Highway 101	13.50	1,660	1,780	1,780	1,780
At Van Buren Road	7.25	1,060	1,890	2,220	2,810
Below Alma Street	9.20	1,450	1,700	1,700	1,750
Below Purissima Creek	6.10	1,040	1,980	2,200	2,510
ALAMITOS CREEK					
Downstream of confluence with Arroyo Calero	28.60	2,150	5,180	6,750	11,000
Downstream of confluence with Golf Creek	37.40	3,530	7,020	8,680	12,700
Downstream of confluence with Greystone Creek	33.80	2,940	6,200	7,800	11,800
Downstream of confluence with Randol Creek	31.60	2,660	5,800	7,380	11,400
Upstream of confluence with Arroyo Calero	16.20	1,430	3,580	4,750	7,900

[1]*Decrease in flow rate based on capacity restrictions*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
ALAMITOS CREEK, continued					
Upstream of confluence with Guadalupe River	38.00	3,630	7,180	8,860	12,900
ALAMITOS CREEK BY-PASS CHANNEL	[1]	[1]	[1]	3,250	[1]
ALAMITOS CREEK OVERFLOW AREA	[1]	[1]	[1]	140	[1]
ARROYO CALERO					
Downstream of confluence with Santa Teresa Creek	11.60	1,020	1,820	2,180	3,010
Upstream of confluence with Alamitos Creek	12.40	1,180	1,980	2,330	3,110
Upstream of confluence with Santa Teresa Creek	9.60	660	1,120	1,320	1,770
ARASTRADERO CREEK					
At Page Mill Road	1.13	140	300	360	460
ARROYO DE LOS COCHES					
At confluence with Berryessa Creek	4.00	[1]	[1]	1,420	[1]
BARRON CREEK					
At El Camino Real	2.60	270	270	270	270
At Foothill Expressway	1.54	176	364	453	640
At Foothill Expressway	1.80	320	630	760	1,100
At Laguna Avenue	1.80	180[1]	180[1]	180[1]	180[1]
At Lower Fremont Road	0.80	96	208	268	390
At mouth	3.10	320	430	430	430
At Ramona Street	2.80	320	430[2]	430[2]	430[2]
At Railroad	2.80	320	675	675	675
At Upper Fremont Road	0.26	32	77	98	143
Downstream of El Camino Real	2.60	270	270	270	270
Upstream of Barron Creek Diversion	1.80	[1]	[1]	740	[1]
Upstream of Fabian Way	2.90	[1]	[1]	250	[1]
Upstream of Laguna Avenue	1.80	[1]	[1]	160[3]	[1]
Upstream of Railroad	2.80	320	820	920	1,080

[1]*Data not available*

[2]*Decrease in flow rate based on capacity restrictions*

[3]*Discharge decrease due to Barron Creek Diversion*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
BERRYESSA CREEK					
At confluence with Calera Creek	21.50	[1]	[1]	3,600[2]	[1]
At confluence with Sierra Creek	7.70	1,230	2,250	2,580	3,090
At confluence with Tularcitos Creek	17.00	[1]	[1]	2,500[2]	[1]
At confluence with Wrigley Ditch	19.10	[1]	[1]	2,000[2]	[1]
At Morrill Avenue	7.70	1,230	1,700[1]	1,750[2]	1,800[1]
At Piedmont Road	4.50	[1]	[1]	1,600	[1]
Downstream of confluence with Arroyo De Los Coches	15.10	[1]	[1]	2,000[2]	[1]
Downstream of Montague Expressway	8.80	800[2]	800[2]	800[2]	800[2]
CALABAZAS CREEK					
Above Prospect Road	4.40	[1]	[1]	1,800	[1]
Above Railroad and Prospect Creek	2.90	[1]	[1]	1,140	[1]
At Coffin Road	20.80	3,000	4,100	4,600	5,800
At El Camino Real	13.70	2,090[3]	2,290[3]	2,340[3]	2,360[3]
At Grant Road	4.10	1,200	1,600	1,800	2,300
At Interstate Highway 280	11.60	1,950	2,490	2,700	3,360
At Junipero Serro	11.20	2,000	2,700	3,100	3,900
At Kifer Road	17.00	2,600	3,600	4,000	5,200
At Lawrence Expressway	12.30	2,100	3,000	3,300	4,200
At Rainbow Drive	4.50	750	1,070	1,310	1,370
Below La Mar Court	10.10	1,740	2,500	2,830	3,740
Below Miller Avenue	10.10	1,670	2,050	2,210	2,670
Below Tantau Avenue/Upstream of Pruneridge Avenue	11.60	1,700[2]	1,900[2]	1,950[2]	2,000[2]
Downstream of confluence with Rodeo Creek	6.70	1,170	1,700	1,950	2,610

[1]*Data not available*

[2]*Decrease in flow rate based on capacity restrictions*

[3]*Flow rate accounts for upstream channel spills*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
CALABAZAS CREEK, continued					
Downstream of Prospect Road	4.30	750[1]	1,000[1]	1,180[1]	1,220[1]
Downstream of U.S. Highway 101	19.10	2,760[2]	3,200[3]	4,780[3]	5,510[3]
Through box culvert at Miller Avenue	10.10	1,400[4]	1,550[4]	1,600[4]	1,600[4]
Upstream of Benton Street	13.30	2,100[2]	2,170[5]	2,170[5]	2,200[5]
Upstream of Kifer Road	17.10	2,550[2]	2,820[2]	3,000[2]	3,340[2]
Upstream of Lawrence Expressway	13.30	2,050[2]	2,310[2]	2,370[2]	2,540[2]
Upstream of Pomeroy Avenue	13.60	2,190[2]	2,200[2]	2,200[2]	2,200[2]
Upstream of U.S. Highway 101	19.10	2,760[2]	3,020[2]	3,200[2]	3,550[2]
Upstream of State Highway 237	20.50	3,010[2]	3,420[2]	5,000[2]	5,100[2]
CALERA CREEK					
At confluence with Berryessa Creek	2.90	[6]	[6]	920	[6]
Upstream of Interstate Highway 680	2.40	[6]	[6]	850	[6]
CANOAS CREEK					
At Blossom Hill Road	12.50	1,320	1,390	1,400	1,420
At Capitol Expressway	17.60	1,850	1,910	1,960	2,000
At confluence with Guadalupe River	18.60	1,900[4]	1,950[4]	1,970[4]	2,000[4]
At Cottle Road	4.60	480	500	510	530
At Santa Teresa Boulevard	7.40	780	810	830	850
Upstream of Nightingale Drive	18.60	1,990	2,250	2,350	2,500
CONCEPCION DRAINAGE					
At Alto Verde Lane	0.17	22	51	68	102
COYOTE CREEK					
At Interstate Highway 280	246.00	3,880	10,180	12,630	14,700
At U.S. Geological Survey gage near Edenvale	229.00	4,050	10,940	13,670	14,700[4]

[1]*Slow rate reflects upstream capacity restriction*

[2]*Flow rate accounts for upstream channel spills*

[3]*Flow influenced by spill from adjoining watercourse*

[4]*Decrease in flow rate based on capacity restrictions*

[5]*Flow reduction due to bridge or channel capacity restriction*

[6]*Data not available*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
At U.S. Geological Survey gage near Madrone	193.00	4,500	12,000	15,000	24,000
Downstream of Anderson Reservoir	192.70	4,500	11,000	15,000	23,500
Downstream of confluence with Berryessa Creek	313.00	7,300	10,500	12,800	15,000
Downstream of confluence with Silver Creek	291.00	6,200	10,300	12,500	15,000
Downstream of Silver Creek Diversion	239.00	4,000	10,680	13,330	14,700
Upstream of confluence with Fisher Creek	205.00	4,410	12,010	14,830	16,400[1]
Upstream of confluence with Silver Creek	248.00	3,790	9,920	11,400[1]	11,400[1]
Upstream of Silver Creek Diversion	233.00	4,000	10,680	13,330	14,700
DAVES CREEK					
At Los Gatos Creek	0.50	130	230	270	370
EAST LITTLE LLAGAS CREEK					
Approximately 1,500 feet upstream of Sycamore Avenue	6.20	[2]	[2]	2,211	[2]
At confluence of Church Creek	21.40	[2]	[2]	5,355	[2]
At confluence of San Martin Creek	18.90	[2]	[2]	3,712	[2]
At U.S. Highway 101	8.00	700	1,200	1,300	1,700
At Tenant Creek confluence	14.00	[2]	[2]	2,881	[2]
Upstream of Seymour Avenue	6.20	330	430	460	490
EAST PENITENCIA CREEK					
Downstream of Trimble Road	1.60	280	340[1]	340[1]	340[1]
Upstream of confluence with Lower Penitencia Creek	1.70	480	970[3]	1,080[3]	1,280[3]
Upstream of Trimble Road	1.60	280	400	450	540

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Data not available*

[3]*Increase in flow rate due to spills from neighboring subbasins*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
FISHER CREEK					
At confluence with Coyote Creek	15.00	700[1]	700[1]	700[1]	700[1]
At Kalana Avenue	5.80	470	960	1,130	1,500
At Miramonte Avenue	2.80	300	600	710	930
At Richmond Avenue	8.60	450	700	700	700
At Willow Springs Road	1.60	270	460	560	810
Downstream of Bailey Avenue	13.00	1,000	1,810	2,160	2,950
Upstream of Bailey Avenue	11.20	620	900	900	900
Upstream of Railroad	15.00	1,260	2,310	2,560	3,530
FISHER CREEK OVERBANK					
500 feet downstream of Richmond Avenue	8.60	250	630	900	1,540
At Bailey Avenue	11.20	220[2]	680	970	1,670
GUADALUPE RIVER					
At Blossom Hill Road	53.20	3,500	8,500	11,500	19,000
At Coleman Avenue	151.00	7,000	13,500[1]	15,500[1]	15,500[1]
At Hedding Street	153.00	7,500	9,800[1]	9,800[1]	9,800[1]
At Hobson Avenue	152.00	7,000	11,400[1]	11,400[1]	11,400[1]
At Interstate Highway 280	95.00	6,000	7,000[1]	7,000[1]	7,000[1]
At Malone Road	90.00	5,600	11,500	11,900[1]	11,900[1]
At Railroad	92.10	5,800	10,900[1]	10,900[1]	10,900[1]
Downstream of confluence with Canoas Creek	88.60	5,500	11,000	12,800	12,800
Downstream of confluence with Los Gatos Creek	150.00	7,000[1]	10,000[1]	10,000[1]	10,000[1]
Downstream of confluence with Ross Creek	65.20	4,500	9,000	12,500	20,000
Downstream of State Highway 17	154.00	7,500	12,000[1]	13,000[1]	17,000[1]
Upstream of confluence with Canoas Creek	70.00	4,500	9,500	12,000[1]	12,000[1]

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Flow rate reduction due to attenuation in the floodplain*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
HALE CREEK					
At Berry Avenue	3.30	510	1,020	1,120	1,580
At confluence with Permanente Creek	4.40	710	880	900	960
At Cuesta Drive/North Springer Road	3.70	595	750	760	810
At Foothill Expressway	3.10	460	970	1,060	1,490
At Interstate Highway 280	0.75	101	218	284	440
At Rosita Avenue	3.60	595	700[1]	700[1]	700[1]
At Summer Hill Avenue	1.37	177	370	472	735
LIONS CREEK					
Upstream of West Branch Llagas Creek	2.60	[2]	[2]	1,840	[2]
LLAGAS CREEK					
At Rucker Avenue	57.00	4,900[3]	9,700[3]	10,200[3]	12,700[3]
At Railroad	27.50	2,200	3,900	5,300	8,500
Downstream of Buena Vista Creek	60.40	5,200	10,400	11,000	11,500[1]
Downstream of Chesbro Reservoir	19.20	900	3,100	3,900	6,000
Downstream of East Little Llagas Creek	56.80	5,000	9,800	10,400	12,900
Downstream of Hayes Creek	26.90	1,800	3,800	4,800	7,500
Downstream of Leavesley Road	67.00	5,200[4]	5,200[4]	5,200[4]	5,200[4]
Downstream of Live Oak Creek	63.70	5,500	9,700	9,800	10,300
Downstream of Machado Creek	23.90	1,400	3,600	4,500	7,000
Downstream of Panther Creek	62.10	5,300	9,700[1]	9,800[1]	10,100[1]
Downstream of Princevalle Drain	87.70	[2]	[2]	18,800	[2]
Downstream of West Branch Llagas Creek	84.80	[2]	[2]	17,800	[2]
Upstream of East Little Llagas Creek	29.80	2,500	4,300	5,400	8,600
Upstream of Jones Creek	103.60	[2]	[2]	18,800	[2]
Upstream of Panther Creek	60.70	5,200	9,400[1]	9,400[1]	9,400[1]

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Data not available*

[3]*Flow rate reduction due to attenuation in the floodplain*

[4]*Decrease in flow with increase in area is result of spill*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
LOS GATOS CREEK					
At Leigh Avenue	50.20	1,680	6,510	7,440	11,340
At Meridian Avenue	51.20	1,770	6,620	7,570	11,500
At Park Road	44.00	1,580	6,140	6,990	10,630
At State Highway 17	48.80	1,540[1]	6,370	7,300	11,200
Below Lexington Dam	37.00	1,610	5,850	6,650	9,630
Below Vasona Dam	44.10	1,550	6,100	6,950	10,600
Upstream of confluence with Guadalupe River	54.80	2,130	7,000	7,980	11,900
LOWER PENITENCIA CREEK					
At Capitol Avenue	4.00	740	1,200	1,210	1,220
At confluence with Berryessa Creek	26.70	2,550	3,700	3,700	3,700
At Nimitz Freeway	27.70	1,750[2]	3,500[2]	3,500[2]	3,500[2]
At Redwood Avenue	5.20	850	1,150[3]	1,150[3]	1,150[3]
At South Main Street	3.70	700[3]	1,120[3]	1,120[3]	1,120[3]
Downstream of confluence with Berryessa Creek	26.70	2,550	2,600[2]	2,600[2]	2,600[2]
Downstream of confluence with East Penitencia Creek	3.70	800	1,670	2,150	2,840
Downstream of Trimble Road	2.00	320	1,060[4]	1,510[4]	1,620[4]
MADRONE CHANNEL					
At East Dunne Avenue	1.40	[5]	[5]	600	[5]
Upstream of East Little Llagas Creek	3.20	[5]	[5]	1,200	[5]
MATADERO CREEK					
Above confluence with Arastradero Creek	1.44	194	392	506	690
Approximately 270 feet upstream of U.S. Highway 101	8.50	[5]	[5]	2,800	[5]
At Alma Street	9.40	1,380	2,000[2]	2,000[2]	2,000[2]

[1]*Flow rate reduction due to attenuation in reservoirs*

[2]*Decrease in flow rate based on capacity restrictions*

[3]*Reduction in flood rate due to storage behind railroad*

[4]*Increase in flow rate due to spills from neighboring subbasins*

[5]*Data not available*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
MATADERO CREEK, continued					
At corporate limits	3.39	402	795	970	1,300
At El Camino Real	7.60	1,100	2,100	2,280	2,690
At Louis Road	9.40	1,380	1,500[1]	1,500[1]	1,500[1]
At Middlefield Road	9.40	1,380	1,900[1]	1,500[1]	1,900[1]
At Railroad	9.10	[2]	[2]	2,435	[2]
At U.S. Highway 101	13.60	1,660	1,775	1,775	1,775
Below confluence with Arastradero Creek	2.70	325	660	790	1,030
Downstream of Foothill Expressway	5.60	[2]	[2]	1,900	[2]
Downstream of Park Boulevard	7.50	[2]	[2]	2,700	[2]
Downstream of U.S. Highway 101	15.80	[2]	[2]	3,100	[2]
Upstream of Railroad	9.10	1,220	2,170	2,520	2,810
MILLER SLOUGH					
At U.S. Highway 101	1.80	[2]	[2]	760	[2]
MIDDLE ROAD OVERFLOW AREA					
At convergence with Llagas Creek	[2]	[2]	[2]	39	[2]
At divergence from West Little Llagas Creek	[2]	[2]	[2]	658	[2]
NORTH MOREY CREEK					
Upstream of Lions Creek	1.00	[2]	[2]	485	[2]
PAJARO RIVER					
At U.S. Highway 101	522	[2]	[2]	30,500	[2]
PERMANENTE CREEK					
At confluence with Hale Creek	13.50[3]	780[4]	1,650[4]	1,780[4]	1,980[4]
At El Camino Real	14.30[3]	1,150	1,310	1,310	1,310
At Railroad	15.20[3]	1,270	1,470	1,600	1,600
Downstream of confluence with Hale Creek	13.50[3]	1000[1]	1000[1]	1000[1]	1000[1]
Downstream of East Charleston Road	16.10[5]	1,390	1,400[1]	1,400[1]	1,400[1]

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Data not available*

[3]*Decrease in flow rate due to storage along channel*

[4]*High flows affected by Permanente Diversion*

[5]*High flows diverted to Stevens Creek*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
PERMANENTE CREEK, continued					
Downstream of Miramonte Avenue	8.9[6]	370	760	890	1,030
Downstream of Permanente Road	3.40	760	1,260	1,480	1,960
Downstream of Portland Avenue	8.10	1,340	2,050	2,050	2,050
Downstream of U.S. Highway 101	15.80[2]	1,350	1,400[1]	1,400[1]	1,400[1]
Upstream of confluence with Hale Creek	9.20[2]	440[3]	840[3]	980[3]	1,110[3]
Upstream of Interstate Highway 280	7.60	1,250	2,160	2,570	3,480
Upstream of Portland Avenue	8.10	1,340	2,220	2,700	3,440
Upstream of Tributary, 700 feet upstream of Interstate Highway 280	3.90	860	1,460	1,720	2,310
Upstream of U.S. Highway 101	15.80[2]	1,350	2,250[4]	4,000[4]	7,100[4]
PERMANENTE DIVERSION					
At confluence with Stevens Creek	8.90[5]	1,230	1,280	1,390	1,550
At Grant Road	8.60	1,200	1,240[1]	1,340[1]	1,490[1]
Downstream of Carmel Terrace	8.20	1,075[1]	1,075[1]	1,075[1]	1,075[1]
Downstream of Diversion Structure	8.10	1,190	1,610	1,610	1,610
PROSPECT CREEK					
Upstream of confluence with Calabazas Creek	1.40	[6]	[6]	635	[6]
PURISSIMA CREEK					
At corporate limits	1.25	147	320	402	588
At Interstate Highway 280	0.30	37	82	104	153
At Viscaino Road	0.70	88	182	227	320
SAN FRANCISQUITO CREEK					
At Alma Street	40.60	4,350	7,050	8,280	9,850[1]
At U.S. Geological Survey gage	37.10	4,050	6,700	7,860	10,500
Downstream of Chaucer Road	41.60	4,350	6,000[1]	6,000[1]	6,200[1]

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Decrease in flow rate due to storage along channel*

[3]*High flows affected by Permanente Diversion*

[4]*Flow influenced by spill from adjoining watercourse*

[5]*Low flows continue down Permanente Creek*

[6]*Data not available*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
SAN FRANCISQUITO CREEK, continued					
Downstream of Middlefield Road	41.60	4,350	6,350[1]	6,690[1]	7,410[1]
Near Pasteur Drive	39.10	4,200	6,850	8,070	10,400
Upstream of Middlefield Road	41.60	43,50	7,100	8,330	9,850[1]
SAN FRANCISQUITO CREEK-OVERFLOW					
At Chaucer Street	[2]	[2]	[2]	563	[2]
At Middlefield Road	[2]	[2]	[2]	752	[2]
Combined Middlefield/Chaucer Overflows	[2]	[2]	[2]	1,080	[2]
SANTA TERESA CREEK					
Upstream of confluence with Arroyo Calero	2.00	360	700	860	1,240
SAN TOMAS AQUINO CREEK					
At Cabrillo Avenue	22.50	2,560[3]	2,920[3]	2,920[3]	2,920[3]
At confluence with Saratoga Creek	39.10	5,900	8,300	9,100	11,000
At El Camino Real	22.20	3,570	3,610	3,610	3,610
At Homestead Road	21.50	3,450[3]	3,450[3]	3,450[3]	3,450[3]
At Pruneridge Avenue	20.40	3,460	3,820[3]	3,820[3]	3,820[3]
At Saratoga and Los Gatos Roads	2.50	620	990	1,140	1,480
At Stevens Creek Boulevard	19.60	3,300	3,820[3]	3,820[3]	3,820[3]
At U.S. Highway 101	41.80	5,900	8,300	9,100	11,000
At U.S. Highway 237	45.10	5,900	8,300	9,100	11,000
Downstream of Railroad	39.30	5,900	8,300	9,100	11,000
Upstream of Westmont Avenue	8.27	2,000	2,900	3,200	4,077[4]
Near Bicknell and Quito Roads	2.80	670	1,050	1,230	1,580
Near Old Adobe and Quito Roads	3.10	730	1,150	1,350	1,720
SARATOGA CREEK					
At confluence with San Tomas Aquino Creek	16.60	2,700	3,750	4,100	4,800
At El Camino Road	16.40	2,700	3,750	4,100	4,800

[1]*Decrease in flow rate based on capacity restrictions* [2]*Data not available*

[3]*Flow reduction due to bridge or channel capacity restriction* [4] *Logarithm extrapolation*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
SARATOGA CREEK, continued					
At Herriman Avenue	10.10	1,550	3,020	3,750	4,630
At Homestead Road	15.80	2,700	3,750	4,100	4,800
At Kiely Boulevard	15.90	2,700	3,750	4,100	4,800
At Stevens Creek Boulevard	14.50	2,500	3,500	3,900	4,600
At U.S. Geological Survey gage at Springer	9.20	1,350	2,750	3,490	4,450
At Railroad	11.10	1,760	3,230	3,950	4,800
Downstream of Benton Street	16.20	2,700	3,750	4,100	4,800
Downstream of Kiely Boulevard	15.90	2,700	3,750	4,100	4,800
Downstream of Warburton Avenue	16.50	2,700	3,750	4,100	4,800
SILVER CREEK					
At confluence with Coyote Creek	43.50	2,550	2,650	2,670	2,750
At intersection of King and McKee Roads	36.20	2,000[1]	2,000[1]	2,000[1]	2,000[1]
At Interstate Highway 680	35.20	2,210	2,400	2,400	2,400
At Ocala Avenue	27.10	1,530	2,000[2]	2,000[2]	2,000[2]
Downstream of confluence with Thompson Creek	22.00	2,080	3,200	3,600	4,300
Downstream of Cunningham Avenue	26.20	1,420[2]	2,150[2]	2,580[2]	2,600[2]
Downstream of confluence with Miguelita Creek	40.60	2,300	2,300	2,300	2,300
Downstream of confluence with North Babb Creek	33.70	1,500[1]	1,500[1]	1,500[1]	1,500[1]
Downstream of confluence with South Babb Creek	31.10	1,940	2,600	2,700	2,700
SMITH CREEK					
At Railroad	0.80	200	370	440	610
At Wedgewood Avenue	0.70	160	300	350	480
Below Smith Creek Drive	0.50	125	230	280	390

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Flow rate reduction due to storage in Lake Cunningham*

[3]*Data not available*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
SOUTH BABB CREEK					
At Clayton Road	3.70	390	760	890	1,150
At confluence with Silver Creek	4.00	200[1]	200[1]	200[1]	200[1]
Downstream of White Road	3.90	390[1]	390[1]	390[1]	390[1]
Upstream of Clayton Road	3.70	[2]	[2]	890	[2]
Upstream of Lochner Drive	3.80	400	550[1]	550[1]	550[1]
Upstream of White Road	3.90	400	570[1]	570[1]	570[1]
SOUTH MOREY CREEK					
Upstream of Lions Creek	1.30	[2]	[2]	420	[2]
STEVENS CREEK					
At Crittenden Lane	36.40	2,350[3]	2,350[3]	2,350[3]	2,350[3]
At Homestead Road	21.00	1,110[4]	4,530	5,570	7,470
At Interstate Highway 280	20.00	1,110[4]	4,460	5,460	7,310
At Stevens Creek Boulevard	19.60	1,110[4]	4,430[4]	5,430	7,240
At U.S. Geological Survey gaging station No. 262	18.80	1,200	2,800	5,400	7,000
At U.S. Highway 101	36.40	3,030	5,550	5,750	5,950
Downstream of Interstate Highway 280	20.10	1,110	4,460	5,460	7,310
Downstream of Junipero Serra	20.90	1,550	3,200	5,580	7,650
Downstream of Stevens Creek Dam	17.30	1,140	4,440	5,280	6,940
Downstream of Railroad	34.30	2,750	5,350[3]	5,350[3]	5,350[3]
Upstream of Junipero Serra	20.20	1,500	3,150	5,500	7,500
Upstream of Permanente Diversion	24.20	1,750	3,600	6,000	8,200
Upstream of Railroad	34.30	2,750	6,110	7,360	9,610
SUNNYVALE EAST CHANNEL					
Downstream of Caribbean Drive	6.10	[2]	[2]	1,100	[2]

[1]*Decrease in flow rate based on capacity restrictions*

[2]*Data not available*

[3]*Flow reduction due to bridge or channel capacity restriction*

[4]*Decrease in flow rate due to storage along channel*

[5]*Flow rate reduction due to storage in Lake Cunningham*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
SUNNYVALE WEST CHANNEL					
Downstream of Highway 237	2.87	[1]	[1]	360	[1]
TENNANT CREEK					
Approximately 1,250 feet upstream of Hill Avenue	[1]	[1]	[1]	420	[1]
Downstream of Maple Avenue	4.30	[1]	[1]	650	[1]
Upstream of confluence with East Little Llagas Creek	5.60	[1]	[1]	2,015	[1]
THOMPSON CREEK					
2,000 feet downstream of Aborn Road	17.30	1,440	2,550	3,000	3,700
At Aborn Road	14.70	1,440	2,350	2,700	3,250
At Quimby Road	18.00	1,480	1,900[2]	1,900[2]	1,900[2]
Downstream of Yerba Buena Creek	8.90	1,060	1,750	1,950	2,400
UPPER PENITENCIA CREEK					
At Capitol Avenue	23.00	1,350[2]	1,350[2]	1,350[2]	1,350[2]
At confluence with Coyote Creek	23.90	1,110	1,110	1,110	1,110
At Gridley Street	22.20	1,460	3,050	3,600	4,950
Upstream of North Jackson Avenue	23.15	1,350[2]	1,350[2]	1,350[2]	1,350[2]
At King Road	23.00	960[2]	960[2]	960[2]	960[2]
At Mabury Avenue	23.00	1,050[2]	1,050[2]	1,050[2]	1,050[2]
At Upper Penitencia Road	22.20	1,460	2,810[2]	2,950[2]	2,950[2]
At U.S. Geological survey gage at Dorel Road	21.10	1,400	2,940	3,600	5,170
UVAS CREEK					
At confluence with Bodfish Creek	50.30	[1]	[1]	10,910	[1]
At confluence with Little Arthur Creek	37.30	[1]	[1]	8,500	[1]
At downstream face of Watsonville Road Bridge	46.70	[1]	[1]	10,360	[1]
At Thomas Road	69.10	[1]	[1]	14,000	[1]
At Railroad	72.70	[1]	[1]	5,200[3]	[1]

[1]*Data not computed*

[2]*Decrease in flow rate based on capacity restrictions*

[3]*Decrease in flow with increase in area is result of spill*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
UVAS CREEK, continued					
At U.S. Highway 101	71.60	[1]	[1]	8,000[2]	[1]
At Uvas Road	30.50	[1]	[1]	7,800	[1]
Downstream of Hecker Pass Road	65.10	[1]	[1]	13,550	[1]
Downstream of Santa Teresa Boulevard	68.00	[1]	[1]	14,000	[1]
UVAS CREEK – EAST OVERBANK ABOVE HIGHWAY 101					
Approximately 1,200 feet above U.S. Highway 101	[3]	[1]	[1]	2,200	[1]
At U.S. Highway 101	[3]	[1]	[1]	1,100	[1]
UVAS CREEK – EAST OVERBANK ABOVE RAILROAD					
At downstream limit of flooding	[3]	[1]	[1]	3,200	[1]
At upstream limit of flooding	[3]	[1]	[1]	2,100	[1]
WATSON ROAD OVERFLOW AREA					
At convergence with Llagas Creek	[1]	[1]	[1]	447	[1]
At divergence from West Little Llagas Creek	[1]	[1]	[1]	97	[1]
WEST BRANCH LLAGAS CREEK					
Downstream of divergence from West Branch Llagas Creek – East Split	5.60	[1]	[1]	160	[1]
Upstream of divergence from West Branch Llagas Creek – East Split	5.60	[1]	[1]	1,400	[1]
WEST BRANCH LLAGAS CREEK – LOWER SPLIT					
At Day Road Interceptor (NRCS PL566)	[3]	[1]	[1]	1,200	[1]
WEST BRANCH LLAGAS CREEK – MIDDLE SPLIT					
Downstream of Highland Avenue	[3]	[3]	[3]	80	[3]

[1]*Data not available*

[2]*Decrease in flow with increase in area is result of spill*

[3]*Flooding due to spill – drainage area not applicable*

Table 6 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
WEST BRANCH LLAGAS CREEK – UPPER SPLIT					
Upstream of Highland Avenue	[1]	[1]	[1]	200	[1]
WEST LITTLE LLAGAS CREEK					
1,000 feet upstream of Wright Avenue	1.50	[3]	[3]	188[2]	[3]
At Fourth Street	3.00	[3]	[3]	900[2]	[3]
At U.S. Highway 101	8.00	[3]	[3]	1,080[2]	[3]
Downstream of Edmundson Avenue	6.00	[3]	[3]	1,269	[3]
Downstream of Monterey Highway	5.60	[3]	[3]	813[2]	[3]
Downstream of Railroad	6.00	[3]	[3]	460[2]	[3]
Upstream of Llagas Avenue	1.00	[3]	[3]	1,702[2]	[3]
Upstream of Monterey Highway	5.60	[3]	[3]	1,936	[3]
Upstream of Seymour Avenue	6.20	[3]	[3]	1,770[2]	[3]
WILDCAT CREEK					
Above Portos Drive	2.00	480	810	960	1,230
At Saratoga and Los Gatos Roads	1.10	310	500	570	740
Below Douglas Lane	1.60	430	710	840	1,070

[1]*Flooding due to spill – drainage area not applicable*

[2]*Data not computed*

[3]*Decrease in flow rate based on capacity restrictions*

Elevations for floods of the selected recurrence intervals on San Francisco Bay are shown in Table 7, "Summary of Stillwater Elevations."

Table 7 – Summary of Stillwater Elevations

Flooding Source and Location	Elevation (feet) (NAVD) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
MAYFIELD SLOUGH				
At Embarcadero Road	10.0	[1]	10.5	10.8
SAN FRANCISCO BAY				
At confluence of Guadalupe Slough and Coyote Creek	[1]	[1]	10.8	[1]
At crossing of Railroad And Alviso Slough	[1]	[1]	11.3	[1]
At Milpitas	[1]	[1]	11.4	[1]
At Mountain View	10.2	[1]	10.7	11.0
At Palo Alto	9.9	[1]	10.5	10.8
At Sunnyvale	3.7	[1]	10.7	[1]

[1]*Data Not Available*

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the flood elevations of the selected recurrence intervals. Users should be aware that flood elevations shown on the FIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. For construction and/or floodplain management purposes, users are encouraged to use the flood elevation data presented in this FIS report in conjunction with the data shown on the FIRM.

For studies performed before the 2009 effective FIS, flood elevations were computed using the USACE HEC-2 step-backwater computer program (Reference 3), supplemented by hand calculations and special computer programs where required.

For each community within Santa Clara County that had a previously printed FIS report, the hydraulic analyses described in those reports have been compiled and are summarized below.

City of Campbell

There is no hydraulic data available at this time.

City of Cupertino

Cross sections for backwater analysis were located at close intervals upstream and downstream from bridges, culverts, and other hydraulically significant features in order to establish the backwater effect of such structures in areas that are urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions. Topographic data for channel cross sections were obtained from existing plans and topographic mapping, supplemented with aerial photogrammetric and field survey data, as necessary, for Calabazas Creek (References 47 and 48), Permanente Creek (Reference 39), and Stevens Creek (Reference 50).

Roughness factors (Manning’s “n”) for hydraulic computations were assigned on the basis of field inspection of floodplain areas and are shown in Table 8, Manning’s “n” Values.

Areas subject to sheetflow flooding were delineated using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

City of Gilroy

Limited areas of Gilroy are subject to sheetflow flooding, which is shallow overland flooding that is generally less than 3 feet deep and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area.

Cross sections for backwater analysis were located at small intervals upstream and downstream from bridges, culverts. and other hydraulically significant features in order to establish the backwater effect of such structures in areas that are urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions. Topographic data for channel cross sections were obtained from existing plans and topographic mapping, supplemented with aerial photogrammetric and field survey data, as necessary (References 51 through 58).

Roughness factors (Manning's "n") for the hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

The hydraulic analyses for the restudy were conducted using the USACE HEC-2 computer program (Reference 193). Water-surface elevations were determined using the HEC-2 computer program and BFEs were developed.

Lions, Llagas, and North and South Morey Creeks, West Branch Llagas Creek (downstream of Day Road), Llagas Overbank (Old Miller Slough), and Miller Slough

The revised hydrology resulted in increases in base flood peak discharges along lions, Llagas, North and South Morey, and West Branch Llagas Creeks and a decrease in base flood peak discharge along Miller Slough. The decrease in base flood peak discharge along Miller Slough resulted from the decrease in drainage area caused by the construction of channel improvements along lions, North and South Morey, and West Branch Llagas Creeks.

The base flood is contained within the identified channel banks along Llagas Creek, from approximately 950 feet downstream of Pacheco Pass Highway to approximately 80 feet upstream of Pacheco Pass Highway; West Branch Llagas Creek, from its confluence with Miller Slough to just downstream of Leavesley Road, from the railroad to Church Street, and from approximately 950 feet upstream to approximately 1,650 feet upstream of Farrell Avenue; the entire reaches of North and South Morey Creeks; and Miller Slough, from its confluence with West Branch Llagas Creek to its upstream limit.

Because the base flood is contained within the identified channel banks, the regulatory floodway has been removed along West Branch Llagas Creek, from approximately 950 feet upstream of Farrell Avenue to approximately 1,000 feet downstream of Day Road, and along the entire reaches of Lions and North and South Morey Creeks.

The SFHA and regulatory floodway have been removed along Llagas Overbank from approximately 2,100 feet downstream of Pacheco Pass Highway to Pacheco Pass Highway. The SFHAs have been removed along the entire reaches of North and South Morey Creeks, the channelized reach of West Branch Llagas Creek, and Lions Creek within the City of Gilroy corporate limits.

Because the base flood is contained within the identified channel banks, Flood Profile Panels have been removed for lions and North and South Morey Creeks and Miller Slough. Additionally, Cross Sections A through D along Lions Creek, A through F along North Morey Creek, A and B along South Morey Creek, and A through H along West Branch Llagas Creek (downstream of Day Road) have been deleted from the Floodway Data Table.

<u>Uvas Creek</u>

Uvas Creek is a perched channel leveed on both banks for nearly the entire reach from the railroad to Thomas Road. Creek flows that overtop or breach the levees travel away from the main channel, and may or may not re-enter the creek farther downstream depending on the effects of manmade impediments to flow. Non-engineered levees, which consist primarily of topsoil that supports vegetation including large trees, have been created by agricultural interests to protect farmland.

Levees that did not satisfy FEMA freeboard requirements or structural soundness criteria (i.e., the levee was not certified by a responsible agency) were assumed failed. Several HEC-2 models were developed to study the levee failures. Levees were failed in sections (i.e., from one bridge embankment to the next upstream bridge embankment). Levee failure considered the levee as removed to natural grade. For Uvas Creek and its overbank areas, several flooding scenarios were possible depending upon various levee failure modes. The right and left levees were failed independently of each other and modeled accordingly. The flood hazard zones and BFEs on the FIRM reflect the "with levee" condition between the levees and the sectional levee failures in the overbanks. The impact on flooding of each levee failure mode was investigated and the worst-case flood-hazard delineations were mapped.

Cross sections and overbank elevations for Uvas Creek were taken photogrammetrically from aerial photographs dated October 22, 1990.

Between cross sections, the flood-hazard delineations were based on USGS topographic maps (Reference 194) and field investigations.

The starting water-surface elevation for the HEC-2 analysis for Uvas Creek was taken from the “Uvas Creek Levee” study prepared by the SCVWD in April 1991 (Reference 192).

Dimensions of hydraulic structures were field measured. Culverts and bridges were modeled using bridge routines in accordance with the HEC-2 computer program (Reference 193).

All analyses were conducted based on subcritical flow.

During the base flood, flows from Uvas Creek will leave the main flow path. It was determined that if the flows are confined to the main flow path, the computed rise in water-surface elevation due to the increase in discharge would exceed 1 foot. Therefore, a floodway was not calculated along this reach of Uvas Creek.

The hydraulic analyses were conducted using the USACE HEC-2 computer program (Reference 193). Water-surface elevations were determined using the HEC-2 computer program, and BFEs were developed.

A floodway was developed for this portion of Uvas Creek.

As a result of the flood-control project, the BFEs have increased along Uvas Creek from approximately 1,800 feet downstream to approximately 2,550 feet upstream of Thomas Road. The 1-percent-annual-chance flood is contained by the levee system along the left bank of Uvas Creek and the right channel bank from approximately 300 feet downstream to approximately 2,550 feet upstream of Thomas Road. However, the 1-percent-annual-chance flood is not contained within the channel from approximately 1,800 feet downstream to approximately 300 feet downstream of Thomas Road.

West Branch Llagas Creek (upstream of Day Road) and West Branch Llagas Creek – East Split

West Branch Llagas Creek flows easterly out of Hayes Valley, becoming a perched channel as it passes between residential properties and Highland Avenue. Old railroad flat cars are used to bridge the creek for driveways in this area. At Highland Avenue, the creek turns southward, flowing into broad cultivated fields to the west of Monterey Highway. In this stretch the creek is little more than a drainage ditch, which local farmers have realigned and filled to accommodate operations. Eventually, the floodplain is intercepted at Day Road by the NRCS, formerly the SCS, P1.566 project.

Between the mouth of Hayes Valley and Coolidge Avenue, perched-channel capacity is not sufficient to contain the 1-percent-annual-chance flood discharge. Consequently, flows spill to the north and south of the creek. Northerly spills flow parallel to Highland Avenue in a topological depression and eventually rejoin creek flows at Highland Avenue. Southerly spills flow in a southeasterly direction as shallow overland flow, rejoining the creek downstream of Highland Avenue.

At Highland Avenue the flow splits, with the majority discharging down the main creek channel and a small portion flowing through a depression in Highland Avenue to the east. Downstream of Highland Avenue, the entire discharge is passed to a broad floodplain bounded to the east by Monterey Highway and to the west by higher ground elevations. Upstream of Day Road, higher ground to the center of the floodplain splits the flow again, with most flow passed to a broad floodplain along Monterey Highway to the interceptor and a lesser percentage remaining in the creek channel.

The floodway along the main creek channel and in areas where the water-surface profile is continuous is established using equal-conveyance reduction. Floodways are not established above Highland Avenue because the channel is perched and the area north of Highland Avenue is already developed with very low-density residential housing.

The floodplain boundary delineations were developed based on existing conditions in the watershed where flows break out of the main channel and do not return to West Branch Llagas Creek. The floodway boundary delineation was developed based on the assumption that no breakouts occur along the study reach.

Cross sections and overbank elevations for West Branch Llagas Creek and West Branch Llagas Creek - East Split were taken photogrammetrically from aerial photographs dated October 22, 1990. Between cross sections, the flood-hazard delineation was based on USGS topographic maps (Reference 194) and field investigations.

The starting water-surface elevation for the HEC-2 analyses for West Branch Llagas Creek and West Branch Llagas Creek - East Split were determined using critical depth.

Dimensions of hydraulic structures were field measured. Culverts and bridges were modeled using bridge routines in accordance with the HEC-2 computer program (Reference 193).

All analyses were conducted based on subcritical flow. Areas of shallow flooding were identified based on normal-depth calculations.

City of Los Altos

Limited areas of Los Altos are subject to sheetflow flooding, which is shallow overland flooding, generally less than 3 feet deep, and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area.

Cross sections were obtained from existing plans (References 59 and 60), topographic mapping (References 61-64), aerial photogrammetric (Reference 65), and field survey data, as necessary.

Roughness coefficients (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Starting water-surface elevations were obtained for Adobe Creek and Stevens Creek using normal depth computations. For Hale Creek, the starting elevations used were confluence elevations at Permanente Creek. For Permanente Creek, critical depth was used for starting elevations. For Permanente Diversion, backwater was calculated 4,400 feet downstream of the limit of study from estimates of water-surface elevations in Stevens Creek.

Areas subject to sheetflow flooding were developed using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

For approximate stream reaches studied along Adobe Creek, Hale Creek, Permanente Creek, Stevens Creek, and Heney Creek, flood levels were established according to the professional judgment of engineers familiar with the region, taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

Town of Los Altos Hills

Cross sections were located above and below all culverts and at approximately 600- to 800-foot intervals throughout the stream reaches. A total of 113 stream cross sections were obtained in the field in addition to road cross sections at each culvert.

Roughness factors (Manning's "n") for these computations were assigned on the basis of field inspections of the stream channels and the floodplains. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Starting elevations were developed by the slope-conveyance method.

It was determined that for Purissima, Matadero, and Arastradero Creeks, as well as Manuella and Robleda Drainages, the 1-percent-annual-chance flooding is contained in their channels.

Flood profiles were computed on the basis of full hydraulic efficiency of the channels and structures, without consideration for the effect of obstructions from accumulations of sediment and debris. Such obstructions are commonly the cause of flooding in local areas, but the frequency of occurrence of such obstructions is unpredictable.

Areas subject to sheetflow flooding were delineated using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

For streams studied by approximate methods, the elevations of the 1-percent-annual-chance flood were determined by the slope-area method and from information on previous flooding provided by local officials and residents.

Town of Los Gatos

Cross sections for backwater analysis were located at close intervals upstream and downstream from bridges and culverts and other hydraulically significant features in order to establish the backwater effect of such structures. All bridges and culverts were measured to determine channel geometries at flow restrictions.

Topographic data for channel cross sections were obtained from existing plans and topographic mapping (References 65-75), supplemented with aerial photogrammetric (References 74) and field survey data, as necessary. Cross sections for Los Gatos Creek were supplied by the SCVWD (Reference 76).

Roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Due to the size of the culverts and channels within Los Gatos, Daves Creek was not found to be a source of flooding in Los Gatos. Therefore, no profiles are presented for it.

Critical depth analysis was used to determine starting water-surface elevations for San Tomas Aquino Creek, starting at Quito Road in Los Gatos; Smith Creek, through railroad; and Los Gatos Creek, through a drop structure downstream of the study reach. The starting water-surface elevations for Daves Creek were based on the average depth at the confluence point with Los Gatos Creek in conjunction with hand calculations.

Elevations for streams studied by approximate methods were determined using normal depth calculations and available data in conjunction with topographic information.

Limited areas of Los Gatos are subject to sheetflow flooding, which is shallow, overland flooding, generally less than 3 feet deep, and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area.

City of Milpitas

Cross sections for backwater analysis were located at small intervals upstream and downstream from bridges, culverts, and other hydraulically significant features in order to establish the backwater effect of such structures in areas that are either urbanized or potentially subject to development. All bridges and culverts were measured, to determine channel geometries at flow restrictions.

Topographic data for channel cross sections were obtained from existing plans (Reference 77) and topographic mapping (Reference 78), supplemented with aerial photogrammetric (Reference 79) and field survey data, as necessary.

Roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

The starting water-surface elevations for Lower Penitencia Creek were based on water-surface elevations from the USACE report for Coyote Creek (Reference 80). Starting water-surface elevations for Berryessa Creek were based on the water-surface elevations for Lower Penitencia Creek. Water-surface elevations for Calera Creek were based on water-surface elevations on Berryessa Creek. Starting water-surface elevations for East Penitencia Creek were based on water-surface elevations on Lower Penitencia Creek.

Many areas of Milpitas are subject to sheetflow flooding, which is shallow overland flooding that is generally less than 3 feet deep and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area.

The hydraulic analyses used for sheetflow flooding were based on surveyed and photogrammetric elevations (Reference 79) field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

Areas subject to tidal inundation include all areas lower than the 1-percent-annual-chance tide (Reference 5) that are not protected by an adequate, maintained levee system. Wave runup due to tsunami events was also considered. However, based on previous studies (Reference 81) wave runup in the Milpitas area is not as significant an event as the 1-percent-annual-chance tidal elevation of 11.4 feet NAVD, for insurance purposes.

Flooding for creeks studied by approximate methods was established according to the professional judgment of engineers familiar with the region, taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

Improvements to Calera Creek, designed to contain the 1-percent-annual-chance flood, have led to the elimination of this creek from the study as a detailed study reach.

The NRCS, formerly the SCS, report summarizes the results of a restudy of Upper Penitencia Creek and the overland flooding associated with the overtopping of the channel banks. Based on this report, the 1-percent and 0.2-percent-annual-chance recurrence interval flood elevations and floodplain boundary delineations have been revised.

The USACE report summarizes the results of a tidal stage-frequency restudy of San Francisco Bay. The tidal data, as well as other tidal parameters presented in the report, reflect only "still" water conditions. It does not consider the effects of wave height or runup on the 1-percent-annual-chance water-surface elevation. Based on this report, the 1-percent-annual-chance water-surface elevation for San Francisco Bay in the City of Milpitas is 11.4 feet NAVD.

The hydraulic analyses for the restudy were conducted using the USACE HEC-2 computer program (Reference 188). Water-surface elevations were determined using the HEC-2 computer program, and BFEs were then developed.

Cross sections and overbank elevations for Berryessa Creek, Arroyo De Los Coches, and Calera Creek were taken photogrammetrically by Pugh-Nolte in 1990. For mapping purposes, Sheet 21 of the 1":500' scale County of Santa Clara Cadastral Map was used as the base map. A topographic computer model was created from digitized points for mapping purposes. This was also at a scale of 1":500'.

Arroyo De Los Coches

The starting water-surface elevation for the HEC-2 analysis for Arroyo De Los Coches was based on the peak water-surface elevation at the confluence with Berryessa Creek using the HEC-2 model for Berryessa Creek prepared under this revision -

Manning's "n" values were determined by field observation. Right overbank Manning's "n" values were based on field observation and modified for overbank urban conditions using Hejl's method (Reference 196).

Dimensions of hydraulic structures were field measured by Nolte and Associates Consulting Engineers. Culverts and bridges were modeled using bridge routines in accordance with USACE guidelines (Reference 188).

Because the study stream is a natural channel, all analyses were conducted based on subcritical flow.

A floodway analysis was not conducted for Arroyo De Los Coches.

The floodplain boundary, as determined by the hydrologic and hydraulic analyses, was delineated on a 1":500' scale topographic map for the study area.

Levees that did not satisfy FEMA freeboard requirements were assumed failed. Several HEC-2 models were developed to study the levee failures. Levees were failed in sections, i.e., from one bridge embankment to the next upstream bridge embankment. The right and left levees were failed independently of each other and modeled accordingly. The flood hazard zones and BFEs on the FIRMs reflect the with-levee condition between the levees and the sectional levee failures in the overbanks.

Berryessa Creek

The starting water-surface elevation for the HEC-2 analysis for Berryessa Creek was taken from the 1988 FIS for the City of Milpitas (Reference 126) at the confluence with Penitencia Creek.

Manning's "n" values were determined by field observation. Right overbank Manning's "n" values were based on field observation and modified for overbank urban conditions using Hejl's method (Reference 196). Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Dimensions of hydraulic structures were field measured by Nolte and Associates Consulting Engineers. Culverts and bridges were modeled using bridge routines in accordance with USACE guidelines (Reference 188).

Because the study stream is a natural channel, all analyses were conducted based on subcritical flow.

A floodway analysis was not conducted for Berryessa Creek.

The floodplain boundary, as determined by the hydrologic and hydraulic analyses, was delineated on a 1":500' scale topographic map for the study area.

Levees that did not satisfy FEMA freeboard requirements were assumed failed. Several HEC-2 models were developed to study the levee failures. Levees were failed in sections, i.e., from one bridge embankment to the next upstream bridge embankment. The right and left levees were failed independently of each other and modeled accordingly. The flood hazard zones and BFEs on the FIRMs reflect the with-levee condition between the levees and the sectional levee failures in the overbanks.

Calera Creek

The starting water-surface elevation for the HEC-2 analysis for Calera Creek was based on the peak water-surface elevation at the confluence with Berryessa Creek using the HEC-2 model for Berryessa Creek prepared under this revision.

Hydraulic structure dimensions were field measured by Nolte and Associates Consulting Engineers. Culverts and bridges were modeled using bridge routines, in accordance with USACE guidelines (Reference 188).

Because the study stream is a natural channel, all analyses were conducted based on subcritical flow.

A floodway analysis was not conducted for Calera Creek.

The floodplain boundary, as determined by the hydrologic and hydraulic analyses, was delineated on a 1":500' scale topographic map for the study area.

Levees that did not satisfy FEMA freeboard requirements were assumed failed. Several HEC-2 models were developed to study the levee failures. Levees were failed in sections, i.e., from one bridge embankment to the next upstream bridge embankment. The right and left levees were failed independently of each other and modeled accordingly. The flood hazard zones and BFEs on the FIRMs reflect the with-levee condition between the levees and the sectional levee failures in the overbanks.

City of Monte Sereno

There is no hydraulic data available at this time.

City of Morgan Hill

Cross sections for backwater analysis were located at close intervals upstream and downstream from bridges, culverts, and other hydraulically significant features in order to establish the backwater effect of such structures in areas presently urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions. Topographic data for channel cross sections were obtained from existing plans and topographic mapping, at a scale of 1":1,200', with a contour interval of 2 feet; supplemented with aerial photogrammetric and field survey data, as necessary (References 82, 83, and 84, respectively).

Roughness coefficients (Manning’s “n”) for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning’s “n” Values.

A number of areas in Morgan Hill are subject to sheetflow flooding, which is shallow overland flooding generally less than 3 feet deep and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area. These areas were determined using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

Flood elevations for creeks studied by approximate methods were established according to the professional judgment of engineers familiar with the region, taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

Water-surface elevations for the restudy were computed through the use of the USACE HEC-2 computer program (Reference 37). Cross-section data were obtained from field surveys and digitized photo contact prints (Reference 184). Between cross sections, the 1-percent-annual-chance floodplain boundaries were interpolated using topographic mapping at a scale of 1”:200’ (Reference 185). Dimensions of hydraulic structures were determined by field survey.

Starting water-surface elevations were determined for West Little Llagas and Tenant Creeks and Madrone Channel using backwater elevations from East Little Llagas Creek, and for the Watsonville Road Overflow Area using the known water-surface elevation at its convergence with Llagas Creek.

Channel roughness factors (Manning’s “n”) were based on field investigation and comparison of field notes to Chow’s “Open Channel Hydraulics” and USGS Water Supply Paper 1849, “Roughness Characteristics of Natural Channels” (References 147 and 186).

Manning’s “n” values for flooded urban areas were determined using an abstract paper entitled “A Method for Adjusting Values of Manning’s Roughness Coefficient for Flooded Urban Areas,” by H.R. Hejl, Jr.

The floodway along Tennant Creek was determined using the USACE HEC-2 computer program (Reference 184) and the equal-conveyance-reduction method.

During the base flood, flows from West Little Llagas Creek, Madrone Channel, and the Watsonville Road Overflow Area will leave the main flow path. It was determined that if the flows are confined to the main flow path, the computed rise in water-surface elevation due to the increase in discharge would exceed 1 foot.

Therefore, a floodway was not calculated along these reaches of West little Llagas Creek, Madrone Channel, and the Watson Road Overflow Area.

City of Mountain View

Topographic data for channel cross sections were obtained from existing plans and topographic mapping that were supplemented with aerial photogrammetric and field survey data, as necessary (References 31, 85-91).

Cross sections for the backwater analysis were located at small intervals upstream and downstream from bridges and culverts and other hydraulically significant features in order to establish the backwater effects of such structures in areas presently urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions.

Roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Starting water-surface elevations for Stevens and Permanente Creeks were taken based on mean higher high tide for South San Francisco Bay. As a result of channel improvements along Stevens Creek, several areas that were previously identified as experiencing shallow flooding were removed from the SFHA. Permanente Creek was started at critical depth as it flows under El Camino Real. The Hale Creek starting water-surface elevation was based on the calculated water-surface elevation of Permanente Diversion was estimated at Stevens Creek, 1,800 feet downstream of the study limit.

The hydraulic analyses used for areas subject to sheetflow flooding were based on surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of, the sheetflow flooding investigations.

The hydraulic analyses for the restudy were conducted using the USACE HEC-2 computer program (Reference 188). Water-surface elevations were determined utilizing the HEC-2, and BFEs were then developed.

Cross sections were developed by Nolte and Associates using the SCVWD's 1":200' scale orthophoto plans, dated September 14, 1992. Nolte survey crews provided additional information. Supplemental information for the upstream end of the study was provided to Nolte by the City of Mountain View in the form of 1":200' scale topographic maps.

The starting water-surface elevation for the HEC-2 analysis for Permanente Creek was the mean high tide water-surface elevation for San Francisco Bay Area, as presented in a 1984 USACE report (Reference 3).

Manning's "n" values were based on field investigation and comparison, as well as field notes to the Chow and Barnes references (References 186-187). Specific creeks and roughness factors are shown on Table 8, Manning's "n" Values.

Dimensions of hydraulic structures were field measured by Nolte staff. Culverts and bridges were modeled using bridge routines in accordance with USACE guidelines (Reference 188).

Since the study stream is a natural channel, all analyses were conducted based on subcritical flow.

A floodway analysis was not conducted for Permanente Creek.

The floodplain boundary, as determined by the hydrologic and hydraulic analyses, was delineated on a 1":200' scale topographic map for the study area.

Levees that did not satisfy FEMA freeboard requirements were assumed failed. Levees were failed in sections; for example, from one bridge embankment to the next upstream bridge embankment. Several HEC-2 models were developed to study the levee failures. The flood hazard zones and BFEs on the FIRMs reflect the with levee conditions for the channel between the levees and the sectional levee failures in the overbanks.

A separate flood profile has been prepared for the West Overbank area in the vicinity of Amphitheatre Parkway. The West Overbank profile represents the potential flooding in the west overbank due to a failure of the west levee along Permanente Creek. Flooding in the west overbank north of Amphitheatre Parkway is controlled by the tidal effects of San Francisco Bay. Flooding between Amphitheatre Parkway and Charleston Road is the result of levee failure. In the event of tidal flooding from the Bay; however, the area may be subject to tidal flooding up to elevation 10.7 feet NAVD.

In the event of a west levee failure between Amphitheatre Parkway and Charleston Road, it is expected that 1-percent-annual-chance flows will overtop Charleston Road to the west and cause flooding in the area roughly bounded by the Bayshore Freeway to the south, the East Bayshore Parkway to the west, Charleston Road to the north, and Landings Drive to the east. This area has been designated as Zone AE elevation 11 feet NAVD; however, it is also subject to tidal flooding from San Francisco Bay.

City of Palo Alto

The hydraulic analyses used for areas subject to sheetflow flooding were based on surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

Areas subject to tidal inundation include all areas lower than the 1-percent-annual-chance tide (Reference 92), which are not protected by an adequate, maintained levee system. Wave runup due to tsunami events was also considered. However, based on a previous study (Reference 81), wave runup in the Palo Alto area is not as significant an event for insurance purposes as the 1-percent-annual-chance tidal elevation of 10.5 feet NAVD. Tidal elevations were found to control the downstream portions of Adobe, Matadero, and San Francisquito Creeks studied by approximate methods.

Topographic data for channel cross sections were obtained from existing plans and topographic mapping, supplemented with aerial photogrammetric and field survey data, as necessary (References 85, 93-102).

Cross sections for the backwater analysis were located at small intervals upstream and downstream of bridges, culverts, and other hydraulically significant features in order to establish the backwater effect of such structures in areas presently urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions.

Channel roughness factors (Manning's "n") for hydraulic computations were chosen using engineering judgment and based on field observations of the streams and floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Starting water-surface elevations for San Francisquito Creek, downstream of Bayshore Freeway, were based on the slope-area method. Starting water-surface elevations for Adobe, Barron, and Matadero Creeks were based on ponding elevations within the Palo Alto Flood Basin (References 103 and 104).

The flood profiles shown reflect the results of the backwater analysis based on subcritical flow for the channels. Limited sections of channels in Palo Alto may maintain supercritical flow, resulting in lower water surfaces in portions of the channel. Supercritical flow effects were not included in the profile, but were considered in any cases where such effects could alter any spill from the channel or floodplain.

Some areas in Palo Alto are subject to sheetflow; that is, shallow overland flooding, generally less than 3 feet deep, and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area.

Hydrologic, hydraulic, and topographic information, in addition to other materials, were obtained from the City of Palo Alto, the SCVWD, the Cities of Menlo Park and East Palo Alto, Santa Clara and San Mateo Counties, the California Department of Transportation, and GSN.

Analyses of the restudied hydraulic characteristics were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along San Francisquito Creek. Starting water-surface elevations were determined by the slope-area method. Water-surface elevations were computed using the USACE HEC-2 computer program (Reference 188).

Channel cross sections were obtained from aerial photographs and topographic maps (References 94-95). Modifications to existing cross-section information were based on SCVWD as-built drawings (Reference 104) and field surveys.

In the overbank area, the 1-percent-annual-chance flood boundary has been delineated using a topographic map at a scale of 1”:3,600’, with a contour interval of 1 foot (Reference 36). Approximate floodplain boundaries have been delineated in the overbank area up to the extent of the San Francisquito Creek overflow flooding in February 1998.

No floodways were computed for San Francisquito Creek because of the perched nature of the channel, which results in the overflows constantly flowing away from the channel and into fully developed land areas.

<u>City of San Jose</u>

Cross sections for backwater analysis were located at small intervals upstream and downstream from bridges and culverts and other hydraulically significant features in order to establish the backwater effect of such structures in urbanized areas or areas potentially subject to development. All bridges and culverts were measured in order to determine channel geometry at flow restrictions.

Topographic data for channel cross sections were obtained from existing plans and topographic mapping, supplemented with aerial photogrammetric and field survey data, as necessary (References 105-125).

Reach lengths for Coyote Creek were based on unpublished USACE information. Alamitos Creek and Guadalupe River reach lengths were based on the SCVWD strip topography (References 105-110).

Roughness factors (Manning’s “n”) for hydraulic computations were assigned on the basis of field inspection of floodplain areas.

The roughness factors for Coyote Creek were obtained from unpublished USACE information, which was based on calibration with 1969 flooding high-water marks. Specific creeks and roughness factors are listed on Table 8, Manning’s “n” Values.

Starting water-surface elevations for Calabazas Creek and the Guadalupe River were based on the mean higher high water of 4.7 feet for San Francisco Bay.

Canoas, Los Gatos, and Ross Creeks elevations are from the Guadalupe River. The slope-area method was used to determine the starting water-surface elevations for Coyote Creek and San Tomas Aquino Creek.

Silver, Fisher, and Upper Penitencia Creeks elevations were based on Coyote Creek. South Babb, Miguelita, and Thompson Creeks elevations are from Silver Creek.

The Sierra Creek starting water-surface elevation was based on Berryessa Creek, which was based on Lower Penitencia Creek, part of the City of Milpitas FIS (Reference 126).

The Arroyo Calero starting water-surface elevation was based on Alamitos Creek. The Alamitos Creek elevation was obtained from SCVWD Improvement Plans (Reference 38) for the approximate-study reach of the Guadalupe River upstream of Blossom Hill Road.

For those streams (South Babbs Creek, Canoas Creek, the Guadalupe River, Ross Creek, Silver Creek, and Thompson Creek) shown as "1-percent-annual-chance flood discharge contained in channel," the profiles show only the water-surface elevations within the channel and do not always reflect the elevation of shallow flooding areas adjacent to the channel. The shallow overbank flooding is due to the 1-percent-annual-chance flood, which affects most of the city. For this reason, only the 10-percent and 1-percent-annual-chance flood profiles are shown.

Areas of San Jose subject to sheetflow flooding (shallow overland flooding, generally less than 3 feet deep, and characterized by unpredictable flow paths) were determined by using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations. The water-surface elevations of flooding in these areas were essentially independent of those along the adjacent stream channels and were affected principally by obstructions in the flooded area.

Due to the perched condition of Coyote Creek and the Guadalupe River below State Highway 17, the swale area between them was modeled separately. Also, the perched condition and limited capacity of Fisher Creek between Richmond and Bailey Avenues resulted in development of models for the east and west overflows. Results for the east overflow model indicated average flooding depths were less than 3 feet. Thus, it was not necessary to draw profiles or determine BFEs. However, large areas and significant flooding depths for the west Fisher Creek overflow (Fisher Creek Overbank) made it necessary to draw profiles, delineate zones, and determine BFEs.

Areas subject to tidal inundation include all areas lower than the 1-percent-annual-chance tide (Reference 3) that are not protected by an adequate, maintained levee system. Wave runup due to tsunami events was also considered. However, based on previous studies (Reference 81), wave runup in the San Jose area is not as significant an event for insurance purposes as the 1-percent-annual-chance tidal elevation.

Two areas were identified as highly susceptible to significant changes in water-surface elevation if overland flows are concentrated by floodplain development. These areas are the steep foothills and alluvial valley floor of the Evergreen area and the swale between the Guadalupe River and Coyote Creek north of Trimble Road.

The NRCS, formerly the SCS, report summarizes the results of a restudy of Upper Penitencia Creek and the overland flooding associated with the overtopping of the channel banks. Based on this report, the 1-percent and 0.2-percent-annual-chance recurrence interval flood elevations and flood boundary delineations have been revised.

The USACE report summarizes the results of a tidal stage-frequency restudy of San Francisco Bay. The tidal data, as well as other tidal parameters presented in the report, reflect only "still" water conditions. The report does not consider the effects of wave height or runup on the 1-percent-annual-chance water-surface elevations. Based on this report, the 1-percent-annual-chance water-surface elevations for San Francisco Bay in the City of San Jose have increased from 9.85 to 10.85 and 11.85 NAVD.

At some locations along San Francisco Bay, the tide gage data supplied by the National Ocean Survey were an estimate of the high-water elevation associated with a particular storm event. Therefore, some of the computed BFEs were lower than what would be expected during the 1-percent-annual-chance flood event. The USACE, using gage elevation values with a high degree of confidence and engineering judgment, published its "adopted" 1-percent-annual-chance stillwater elevations. This created a smooth transition of the 1-percent-annual-chance flood elevations throughout the bay.

As part of this study, Calabazas Creek was studied from the northern corporate limit at Prospect Avenue to Wardell Road. Prospect Creek was studied from the confluence with Calabazas Creek to Prospect Avenue.

In addition, an approximate total length of 1.5 miles of shallow flooding due to overtopping of Calabazas Creek was analyzed.

Water-surface elevations were computed through the use of the USACE HEC-2 computer program (Reference 188). Channel and overbank cross sections were determined from all surveyed cross sections and topographic mapping provided by the SCVWD (References 60, 142-146, and 183). The Manning's "n" roughness values were established based on field observations and USACE and USGS guidelines (References 147-148).

The floodplain and floodway boundaries, as determined by the hydrologic and hydraulic analyses, were delineated on horizontal-scale Santa Clara County base mapping at a scale of 1”:6,000’ (Reference 197).

Where the calculated average depth was greater than 1 foot, BFEs were determined. Flood plain boundaries were defined based on the hydraulic model, as determined by subcritical flow analyses. In channel reaches where supercritical flow conditions could occur, the BFEs are based on critical depth.

Where average depth of flow in the split-overflow areas is less than 1 foot, the floodplain area is designated Zone X (shaded).

Floodways were determined using the HEC-2 computer program and the equal-conveyance reduction method. The floodway widths are based on limiting the rise in water-surface or energy-gradeline elevations to 1 foot due to encroachment. The floodway analyses are based on containing all split-flow discharges.

Floodways were not determined for the area on Calabazas Creek from immediately upstream of the railroad to Saratoga- Sunnyvale Road because the entire overflow could not be contained without causing a water-surface rise of more than 1 foot.

Alamitos Creek

The cross-section data for the streams used in the hydraulic analyses were determined using photogrammetrical methods. Water-surface elevations of the floods of the selected recurrence intervals were computed using the USACE HEC-2 computer program (Reference 188). Between cross sections, the 1-percent-annual-chance floodplain boundaries were interpolated using topographic mapping at a scale of 1”:6,000’, with a contour interval of 1 foot (Reference 191). The topography was not included as part of the base map but was extrapolated from the cross-section data.

Channel roughness factors (Manning’s “n”) were chosen based on engineering judgment and field observations. The overbank “n” values were adjusted to consider the effects of flooded urban areas on the basis of the density of the buildings on the floodplain (Reference 196).

The levees along Alamitos Creek did not meet the freeboard requirement set forth by FEMA to allow them to be certified as providing protection from the 1-percent-annual-chance flood. Therefore, a levee failure analysis was performed and the overbanks flooded. Just upstream of Golf Creek, floodwaters in the west overbank, which are a result of a west levee failure, cross over the Almaden Expressway and form a separate flow path. This area has been designated the Alamitos Creek Overflow Area.

These floodwaters flow into Golf Creek and ultimately return to Alamitos Creek.

Just above its confluence with Golf Creek, Alamitos Creek splits into two flow paths. A separate flow path has been constructed to the east of Alamitos Creek and has been designated the Alamitos Creek Overflow Channel.

No floodways were computed for Alamitos Creek.

Berryessa Creek

The cross-section data for the streams used in the hydraulic analyses were determined using photogrammetrical methods. Dimensions of hydraulic structures were field measured by the study contractor. The starting water-surface elevation was determined from the 1988 FIS for the City of San Jose, California, at the confluence with Sierra Creek. Water-surface elevations of the floods of the selected recurrence intervals were computed using the USACE HEC-2 computer program (Reference 188). Between cross sections, the 1-percent-annual-chance floodplain boundaries were interpolated using topographic mapping at a scale of 1”:2,400’, with a contour interval of 1 foot. The topography was not included as part of the base map but was extrapolated from the cross-section data.

Channel roughness factors (Manning’s “n”) were chosen based on engineering judgment and field observations. The overbank “n” values were adjusted to consider the effects of urbanized development on the floodplain (Reference 196). Specific creeks and roughness factors are shown on Table 8, Manning’s “n” Values.

The levees along this portion of the study did not meet FEMA’s levee criteria. Failure of the levees, therefore, was assumed in this analysis. Some overtopping of the levees occurred with the levees in place. BFEs placed within Berryessa Creek reflect the with-levee-in-place condition.

Spill areas that were assigned BFEs were analyzed using the USACE HEC-2 computer program. The remaining spills (Zone AO) were traced using normal-depth calculations. A separate profile was prepared by Michael Baker Jr., Inc., from the study contractor’s data for the far-east overbank area to more clearly show the depth of flow in the area. This profile has been entitled “Berryessa Creek - East Overbank Spill” and is contained in this FIS.

No floodways were computed for Berryessa Creek.

South Babb Creek

The cross-section data for the streams used in the hydraulic analyses were determined using photogrammetrical methods. Nolte and Associates survey crews provided additional topographic information for selected areas. Dimensions of hydraulic structures were field measured by the study contractor. The starting water-surface elevation was determined from the backwater of Silver Creek. Water-surface elevations of the floods of the selected recurrence intervals were computed using the USACE HEC-2 computer program (Reference 188). Between cross sections, the 1-percent-annual-chance floodplain boundaries were interpolated using topographic mapping at a scale of 1”:2,400’, with a contour interval of 1 foot. The topography was not included as part of the base map but was extrapolated from the cross-section data.

Much of the lower study reach, from the confluence with Silver Creek to Clayton Road, is flowing supercritically. Spills occur at Lochner Drive and upstream of Lochner Drive due to the culvert constriction at Lochner Drive and overtopping of the creek banks. Historical flooding documentation shows the Lochner Drive spill flowing down Candler, Sienna, Murtha, and Lochner Drives. This area has been mapped as Zone AO (1 foot). Normal-depth calculations were prepared to determine the average depth of flooding. At White Road, the flow spill splits with the majority flowing over White Road and traveling along Murtha and Warrington Drives. The remainder will break off north along White Road and spill over in various locations.

In the left overbank, a spill occurs at Lochner Drive, travels along Mount Vista Drive, crosses White Road, and flows down Markingdon Avenue to Silver Creek.

Zone AO designations were determined by using normal-depth calculations and historical flooding data as a guide.

No floodways were computed for South Babb Creek.

Upper Silver Creek

The flow rates for Upper Silver Creek were determined based on the urban hydrology methodology and regional regression equations developed by the SCVWD (Reference 18). The rates reflect existing conditions in the watershed and take into account attenuation of overbank storage.

The cross-section data for the streams used in the hydraulic analyses were determined using photogrammetrical methods. Nolte and Associates survey crews provided additional topographic information for selected areas. Dimensions of hydraulic structures were field measured by the study contractor. The starting water-surface elevation was determined from the backwater elevation of Coyote Creek. Water-surface elevations of the floods of the selected recurrence intervals were computed using the USACE HEC-2 computer program (Reference 188). Between cross sections, the 1-percent-annual-chance floodplain boundaries were interpolated using topographic mapping at a scale of 1":2,400', with a contour interval of 1 foot. The topography was not included as part of the base map but was extrapolated from the cross-section data.

The culvert constriction at Yerba Buena Road causes a spill to occur. The spill flows in both directions perpendicular to the channel, with the majority flowing to the northeast along Yerba Buena Road. Flows in this area pond to an elevation of 170 feet on the southeast side of Yerba Buena Road. The spill traveling to the southwest ponds under the Highway 101 crossing to an elevation of 167 feet.

Another spill occurs in the vicinity of Cross Section S. This spill matches the historical flooding information obtained from the SCVWD during the course of the study. Flow depths for this area were determined using normal-depth calculations.

No floodways were computed for Upper Silver Creek.

City of Santa Clara

Cross sections for the backwater analysis were located at small intervals upstream and downstream from bridges and culverts and other hydraulically significant features in order to establish the backwater effect of such structures in areas that are urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions.

Topographic data for cross sections were obtained from existing plans and topographic mapping, supplemented with aerial photogrammetric and field survey data, as necessary (References 124, 127-140). Planned channel improvement projects for San Tomas Aquino and Calabazas Creeks from Guadalupe Slough to Bayshore Freeway were considered to be in place for the study.

Roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Starting water-surface elevations for San Tomas Aquino Creek downstream of Bayshore Freeway were based on the slope-area method. The starting water-surface elevations for Saratoga Creek were based on the water-surface elevations of San Tomas Aquino Creek. The starting water-surface elevations for Calabazas Creek were based on mean high water in San Francisco Bay. The starting water-surface elevations for the Guadalupe River and Guadalupe Slough were based on elevations on San Francisco Bay.

The profiles show only the water-surface elevations within the channel on all watercourses and do not always reflect the elevation of shallow flooding areas adjacent to the channel. The only shallow overbank flooding is due to the 1-percent-annual-chance flood, which affects most of the city. For this reason, only the 10-percent and 1-percent-annual-chance flood profiles are shown. No flood profile is included for San Tomas Aquino Creek through the culvert under San Tomas Expressway, because the 1-percent-annual-chance flood exceeds the culvert capacity.

Due to spills from Calabazas Creek upstream of Santa Clara in the City of Cupertino, there are no major spills from the creek within Santa Clara. There are spills from the channel at Lawrence Expressway, Benton Street, and Pomeroy Avenue. However, the resulting sheetflow remains less than 1 foot deep; therefore, these areas are included as Zone X (shaded) areas without an SFHA designation. Because no SFHAs were defined due to direct flooding from Calabazas Creek, the flood profiles are not included in this study.

The flood hazard areas adjacent to Lawrence Expressway are due to floodwater entering Santa Clara from the City of Sunnyvale. Spills from Calabazas Creek upstream of Santa Clara contribute to these flood areas.

Similarly, a spill from the Guadalupe River enters Santa Clara near the San Jose Airport. This sheetflow is directed northward adjacent to the levee along the river. Due to the river levee, the sheetflow elevations are not affected by the water-surface elevations within the river. Therefore, no water-surface profiles for the Guadalupe River have been included in this study.

The hydraulic analyses used for sheetflow flooding were based on surveyed and photogrammetric elevations (Reference 125), field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

The cross-section data for the streams used in the restudied hydraulic analyses were determined using photogrammetrical methods. Water-surface elevations of the floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater program (Reference 188). Between cross sections, the 1-percent-annual-chance floodplain boundaries were interpolated using topographic mapping at a scale of 1”:6,000’, with a contour interval of 1 foot (Reference 191). The topography was not included as part of the base map, but was extrapolated from the cross-section data.

There are five reaches along San Tomas Aquino Creek where the levees did not have the freeboard required to certify them as providing protection from the 1-percent-annual-chance flood. The five reaches are consecutive, and are separated by the embankments of Tasman Drive, The Great America Parking Lot Crossing, Agnew Road, Mission College Boulevard, Highway 101, and Scott Boulevard. A separate levee failure analysis was performed for each reach. Different reach failure combinations were not considered.

No floodways were computed for San Tomas Aquino Creek.

City of Saratoga

Topographic data for channel cross sections were obtained from existing plans and topographic mapping (References 47, 69, and 141), supplemented with aerial photogrammetric (Reference 74) and field-survey data, as necessary.

Cross sections were located at small intervals upstream and downstream from bridges and culverts and other hydraulically significant features to establish the backwater effect of such structures in areas presently urbanized or potentially subject to development. All bridges and culverts were measured to determine channel geometries at flow restrictions.

Limited areas of the City of Saratoga are subject to sheetflow flooding, which is shallow, overland flooding, generally less than 3 feet deep, and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those among the adjacent streamway and are affected principally by obstructions in the flooded area.

Starting water-surface elevations for Wildcat, Saratoga, and Calabazas Creeks were based on normal-depth analysis downstream of the study limit. Critical-depth analysis, starting at Quito Road in Los Gatos, was used to determine the starting water-surface elevation for San Tomas Aquino Creek.

For this study, the following data and parameters were used:

- Channel and overbank cross sections were determined from all surveyed cross sections and topographic mapping provided by the SCVWD (References 142-146).

- The Manning's "n" roughness values were established based on field observations and USACE and USGS guidelines (References 147 and 148).

- The HEC-2 special culvert and bridge routines were used to analyze the channel road crossings. In accordance with USACE guidelines, contraction and expansion coefficients of 0.1 and 0.3 were used for open-channel sections. Contraction coefficients at culverts and bridges ranged from 0.3 to 0.5, depending on configuration. An expansion coefficient of 0.5 was used at bridges. HEC-2 special bridge and culvert routines were used to model the existing road crossings. All culverts and bridges were analyzed based on the as-built plans or surveyed dimensions, and were assumed to be unobstructed (References 148-153).

- The downstream starting water-surface elevation for Prospect Creek was based on the HEC-2 slope-area method. For Calabazas Creek, the model was started approximately 800 feet downstream of the limit of study using the water-surface elevation from the SCVWD HEC-2 model (Reference 144).

- Supercritical flow conditions can occur in some channel reaches. In accordance with FEMA guidelines, subcritical analyses were conducted to determine BFEs for all stream reaches.

- Split-flow routines were used to determine discharges for overbank-flow paths that are hydraulically separated from the main channel. Split flows were based on a weir coefficient of 2.6.

- A separate HEC-2 analysis was performed to determine the depth of the 1-percent-annual-chance flood in the overbank areas, and the calculated depths were less than 1 foot. Therefore, the areas are designated Zone X (shaded) on the FIRM.

- A multiple-discharge HEC-2 analysis was conducted to determine the discharge in the 48-inch-diameter bypass culvert that conveys a portion of the Prospect Creek discharge directly to Calabazas Creek from upstream of Arroyo de Arguello.

- Floodways were determined by HEC-2 modeling methods limiting the rise in water-surface elevation to a maximum of 1 foot. Equal reduction on each side of the channel was used where possible. A floodway has not been defined for Calabazas Creek from immediately upstream of the railroad to Saratoga-Sunnyvale Road, because the full 1-percent-annual-chance flood discharge cannot be contained to pass through the culvert under the railroad and in the channel without causing a water-surface rise of greater than 1 foot.

- Because the calculation indicated that the peak discharge in Calabazas Creek would result in overtopping of the railroad, the FEMA levee policy has been applied to the railroad embankment. For the maximum upstream water-surface elevation and split flow, the embankment was assumed to be in place. For the worst-case downstream floodplain, the embankment was assumed not to exist.

- The levee policy was also applied to a masonry wall located in the overflow area downstream of the railroad tracks between Calabazas Creek and Saratoga Sunnyvale Road. To determine the downstream floodplain, the wall was assumed not to exist. To determine the overflow east of Saratoga-Sunnyvale Road, the wall was assumed to be in place.

- The downstream limits of the study for the overflow areas were:

- Where the overflow returns to the Calabazas Creek channel downstream of Prospect Road. It should be noted that the channel downstream of Prospect Road was not part of this study.
- The Route 85 Freeway. At these points, the overflow will enter the depressed freeway section.

Channel roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

City of Sunnyvale

Cross sections for backwater analysis were located at small intervals upstream and downstream from bridges and culverts and other hydraulically significant features to establish the backwater effect of such structures in areas presently urbanized or potentially subject to development. Construction plans and/or as-built plans from the SCVWD were used to determine cross sections, in whole or in part, for the four watercourses included in the study (References 154-161). USGS quadrangle maps (Reference 170) and field measurements were used to supplement these available data. Additional field measurements were required, as subsidence and/or siltation have recently occurred in the City of Sunnyvale area. As such, older plans and as-built plans were of questionable accuracy.

Channel roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

Tidal elevations for the 10-percent, 2-percent, and 1-percent-annual-chance tidal floods for San Francisco Bay were extrapolated from existing U.S. Coast and Geodetic Survey data (Reference 20). Starting water-surface elevations in the bay concurrent with 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance flood events on the streams studied were all set at mean higher high water. The effects of tsunami-induced flooding were considered and were found to be insignificant in the southern end of San Francisco Bay (Reference 162).

Flooding originating from San Francisco Bay controls water-surface elevations in the lower portions of the Sunnyvale West Channel.

Stevens Creek was found not to be a source of flooding to the City of Sunnyvale.

Areas subject to sheetflow flooding were delineated using surveyed elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were located as part of the sheetflow flooding investigations.

The hydraulic analyses for the restudy were conducted using the USACE HEC-2 computer program (Reference 188). Water-surface elevations were determined using the HEC-2 analysis and BFEs were then developed.

Cross sections were developed by Nolte and Associates through digitization of photo contact prints dated 1991. Contour mapping was then developed from the digitized cross sections, which were spaced an average of 500 feet apart. Topographic information between the cross sections was based on interpolation. Nolte and Associates survey crews provided additional topographic information for selected areas.

Starting water-surface elevations for the HEC-2 analyses for Sunnyvale East and West Channels were the mean high-tide water-surface elevation for the San Francisco Bay area, as presented in the USACE report entitled "San Francisco Bay, Tidal Stage vs. Frequency Study," dated October 1984 (Reference 3).

The USACE report summarizes the results of a tidal stage-frequency restudy of San Francisco Bay. The tidal data, as well as other tidal parameters presented in the report, reflect only stillwater conditions. The data do not consider the effects of wave height or runup on the 1-percent-annual-chance water-surface elevation. Based on this report, the 1-percent-annual-chance water-surface elevation for San Francisco Bay in the City of Sunnyvale is 10.7 feet NAVD.

Manning's "n" values were based on field investigations and comparison of field notes to the Chow and Barnes references (References 186-187).

Dimensions of hydraulic structures were field measured by Nolte and Associates staff. Culverts and bridges were modeled using bridge routines in accordance with USACE guidelines (Reference 188).

All analyses were conducted based on subcritical flow.

A floodway analysis was not conducted for Sunnyvale East or West Channels.

The floodplain boundary, as determined by the hydrologic and hydraulic analyses, was delineated on a 1":200' scale topographic map for the study area.

Levees that did not satisfy FEMA requirements were assumed failed. Several HEC-2 models were developed to study the levee failures. Levees were failed in sections, i.e., from one bridge embankment to the next upstream bridge embankment. The east and west levees for each channel were failed independently of each other and models were developed to reflect the appropriate expansion and contraction of flows through the failed section. The flood hazard zones and BFEs on the FIRMs reflect the "with-levee" condition in the channel between the levees and the sectional levee failures in the overbanks.

Along Sunnyvale East Channel, there are several reaches where the overbank BFEs are higher than the BFEs shown between the levees. The "with-levee" BFEs reflected in the channel between the levees include the effects of split flow in the HEC-2 modeling. This split flow in the "with-levee" HEC-2 model overtops the levee and does not return to Sunnyvale East Channel. When the levees are failed to reflect the overbank elevations, the effective flow area is increased due to the removal of the subject levee reach so the split flow does not occur to the same degree and, thus, the discharge in these areas is greater than in the "with levee" HEC-2 model.

Santa Clara County (Unincorporated areas)

Cross sections for backwater analysis were located at small intervals upstream and downstream from bridges and culverts and other hydraulically significant features in order to establish the backwater effect of such structures in areas presently urbanized or potentially subject to development. All bridges and culverts were measured in order to determine channel geometry at flow restrictions. Topographic data for channel cross sections were obtained from existing plans and topographic mapping, supplemented with aerial photogrammetric and field survey data, as necessary (References 37, 55, 57-58, 107, 109-112, 114-123, 139, 171-175, and 204-208).

Reach lengths for Coyote Creek were based on unpublished USACE information. Alamitos Creek and the Guadalupe River reach lengths were based on the SCVWD strip topography (References 37, 107, 109, 139, and 204-205).

Roughness factors (Manning's "n") for hydraulic computations were assigned on the basis of field inspection of floodplain areas. Specific creeks and roughness factors are listed on Table 8, Manning's "n" Values.

The starting water-surface elevations for Calabazas Creek and the Guadalupe River were based on the mean higher high water of 4.7 feet for San Francisco Bay.

The slope-area method was used to determine the starting water-surface elevations for Coyote Creek.

The starting water-surface elevations for Silver, Fisher, and Upper Penitencia Creeks were taken at the confluence with Coyote Creek. South Babb and Thompson Creek elevations were taken at the confluence with Silver Creek.

Starting water-surface elevations for Fisher Creek Overbank were based on weir flow back into the channel over the levee.

Canoas Creek and Los Gatos Creek starting water-surface elevations were taken at the confluence with the Guadalupe River.

The starting water-surface elevation for Santa Teresa Creek was taken at the confluence with Arroyo Calero.

The starting water-surface elevation for Arroyo Calero was based on Alamitos Creek. The Alamitos Creek elevation was obtained from SCVWD improvement plans (Reference 209).

Starting water-surface elevations for Miller Slough were taken at the confluence with Ronan Channel. For West Branch Llagas Creek and Lions Creek, the starting water-surface elevations were taken at the confluence with Miller Slough.

Starting water-surface elevations for North and South Morey Creeks were taken at the confluence with Lions Creek.

The starting water-surface elevations for Llagas Overbank and Ronan Channel were taken from the confluence with Llagas Creek.

The starting water-surface elevations for Uvas Creek, West Little Llagas Creek, and East Little Llagas Creek were calculated from normal depth.

Starting water-surface elevations for Stevens and Permanente Creeks were based on mean higher high tide for southern San Francisco Bay.

Due to the perched condition of Llagas Creek below Rucker Avenue and a low swale between Llagas Creek and the South Valley Freeway, this area was modeled separately. Llagas Creek channel was modeled as usual, but all overflow to the west was added to the Llagas Overbank. Large areas and significant flooding depths made it necessary to draw profiles, delineate zones, and determine BFEs for this overflow area. A unique situation exists near the confluence of Ronan Channel and Llagas Creek. High backwater elevations upstream of State Highway 152 in Llagas Creek and lower water-surface elevations in the overflow area cause reverse flow up Ronan Channel and down Old Miller Slough.

High levees along the Gilroy Sewage Treatment Plant and the City Dump force most of the overbank flow back toward Llagas Creek. The creek itself has a very limited capacity and responds to this additional flow by overtopping the east bank, causing shallow flooding in low areas east of the creek.

Due to the extreme meandering nature of streams in the study area, stream distances will not always agree between maps and profiles.

A number of areas in Santa Clara County are subject to sheetflow; that is, shallow overland flooding, generally less than 3 feet deep, and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area. These areas were determined using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect a point were evaluated as part of the sheetflow flooding investigations.

For those streams shown as "1-percent-annual-chance flood discharge contained in channel," the profiles show only the water-surface elevations within the channel and do not always reflect the elevation of shallow flooding areas adjacent to the channel. The shallow overbank flooding is due to the 1-percent-annual-chance flood, which affects most of the city. For this reason, only the 10-percent and 1-percent-annual-chance flood profiles are shown.

Due to the perched condition of Coyote Creek and the Guadalupe River below State Highway 17, the swale area between them was modeled separately. Also, the perched condition and limited capacity of Fisher Creek between Richmond and Bailey Avenues resulted in development of models for the east and west overflows. Results for the east overflow model indicated that average flooding depths wee less than 3 feet. Thus, it was not necessary to draw profiles. However, large areas and significant flooding depths for the west Fisher Creek overflow (Fisher Creek Overbank) made it necessary to draw profiles, delineate zones, and determine BFEs.

Areas subject to tidal inundation include all areas lower than the 1-percent-annual-chance tide (Reference 203) that are not protected by adequate, maintained levee system. Wave runup due to tsunami events was also considered. However, based on previous studies (Reference 81), wave runup affecting the unincorporated areas near San Jose is not as significant an event for insurance purposes as the 1-percent-annual-chance tidal elevation.

Analysis for streams studied by approximately methods was based on historic information, an existing report (Reference 210), and field observations.

Existing levees along Uvas Creek and Llagas Creek were not analyzed for levee stability. However, a failure of the Uvas Creek levee could result in shallow overflow, especially between Miller Avenue and Thomas Road north of Uvas Creek.

The NRCS, formerly the SCS, report summarizes the results of a restudy of Upper Penitencia Creek and the overland flooding associated with the overtopping of the channel banks. Based on this report, the 1-percent and 0.2-percent-annual-chance recurrence-interval flood elevations and flood boundary delineations have been revised.

The USACE report summarizes the results of a tidal-stage-frequency restudy of San Francisco Bay. The tidal data, as well as other tidal parameters presented in the report, reflect only "still" water conditions. The report does not consider the effects of wave height or runup on the 1-percent-annual-chance water-surface elevations. Based on this report, the 1-percent-annual-chance water-surface elevations for San Francisco Bay in the unincorporated areas of Santa Clara County have increased from 9.85 to 10.85 and 11.85 NAVD.

At some locations along San Francisco Bay, the tide-gage data supplied by the National Ocean Survey were an estimate of the high-water elevation associated with a particular storm event. Therefore, some of the computed BFEs were lower than what would be expected during the 1-percent-annual-chance flood event. The USACE, using gage-elevation values with a high degree of confidence and engineering judgment, published its "adopted" 1-percent-annual-chance stillwater elevations. This created a smooth transition of the 1-percent-annual-chance flood elevations throughout San Francisco Bay.

Alamitos Creek, East Little Llagas Creek, Madrone Channel, Middle Avenue Overflow Area, San Tomas Aquino Creek, Tennant Creek, Uvas Creek, Uvas Creek - East Overbank Above Highway 101, Uvas Creek - South Spill, Watsonville Road Overflow Area, West Branch Llagas Creek, West Branch Llagas Creek - Lower Split, West Branch Llagas Creek - Middle Split, West Branch Llagas Creek - Upper Split, and West Little Llagas Creek

As part of this restudy the following flooding sources were studied:

Alamitos Creek, from the percolation pond to approximately 800 feet upstream of the Almaden Expressway; Watsonville Road Overflow Area, from its convergence with Llagas Creek to its divergence from West Little Llagas Creek; East Little Llagas Creek, from its confluence with Llagas Creek to the, confluence of Madrone Channel and West Little Llagas Creek; Madrone Channel, from its confluence with East Little Llagas Creek to approximately 1.02 miles upstream of East Main Avenue; Middle Avenue Overflow Area, from its convergence with Llagas Creek to its divergence from West Little Llagas Creek; San Tomas Aquino Creek, from just upstream of Old Mountain View Aviso Road to just upstream of Monroe Avenue in the City of Santa Clara; Tennant Creek, from its confluence with East Little Llagas Creek to approximately 0.27 mile upstream of Fountain Oaks Drive; Uvas Creek, from the railroad to approximately Thomas Road; Uvas Creek - East Overbank above Highway 101, from Highway 101 to approximately 2,600 feet upstream; Uvas Creek - South Spill, from Bloomfield Avenue to approximately 3,450 feet upstream; West Branch Llagas Creek, from the NRCS, formerly the SCS, PI-566 interceptor project at Day Road to approximately 2,500 feet upstream of Coolidge Avenue; West Branch Llagas Creek - Lower Split, from the NRCS, formerly the SCS, PLS66 to approximately 650 feet upstream of Golden Gate Avenue; West Branch Llagas Creek - Middle Split, from approximately 2,200 feet downstream of Highland Avenue to Highland Avenue; West Branch Llagas Creek - Upper Split, from Highland Avenue to approximately 1,050 feet upstream of Coolidge Avenue; and West Little Llagas Creek, from its confluence with East Little Llagas Creek to approximately 0.35 mile upstream of Llagas Road.

Cross-section data were obtained from field surveys and digitized photo contact prints (Reference 184). Between cross sections, the 1-percent-annual-chance floodplain boundaries for East and West Little Llagas and Tennant Creeks, Madrone Channel, and Middle Avenue and Watsonville Road Overflow Areas were interpolated using topographic mapping at a scale of 1”:200’ (Reference 185); for San Tomas Aquino Creek, using topographic mapping at a scale of 1”:6,000’, with a contour interval of 1 foot (Reference 191); for Uvas Creek (downstream of Thomas Road), Uvas Creek - East Overbank above Highway 101, Uvas Creek - South Spill, West Branch Llagas Creek, West Branch Llagas Creek-Lower Split, West Branch Llagas Creek-Middle Split, and West Branch Llagas Creek-Upper Split, using USGS topographic maps (Reference 196); and for Uvas Creek (upstream of Hecker Pass Highway), using topographic mapping at a scale of 1”:100’, with a contour interval of 2 feet (Reference 198). The topography for Alamitos and San Tomas Aquino Creeks was not included as part of the base trap but was extrapolated from the cross-section data. For Uvas Creek (downstream of Thomas Road), Uvas Creek - East Overbank above Highway 101, Uvas Creek - South Spill, West Branch Llagas Creek - Lower Split, West Branch Llagas Creek - Middle Split, and West Branch Llagas Creek - Upper Split, base maps were the 1”:500’ scale County of Santa Clara cadastral maps. Topographic data were not provided on the cadastral maps. Dimensions of hydraulic structures were determined by field survey.

Channel roughness factors (Manning’s “n”) were based on field investigation and comparison of field notes to the Chow and Barnes references (References 147-153, 181, 184-186, 191, and 196-198). Manning’s “n” values for flooded urban areas were determined using an abstract paper entitled “A Method for Adjusting Values of Manning’s Roughness Coefficient for Flooded Urban Areas” by H.R. Hejl, Jr. Specific flooding sources and roughness factors are listed below.

The levees along Alamitos, San Tomas Aquino, and Uvas Creeks from the railroad to approximately 1,500 feet downstream of Thomas Road did not meet the levee requirements set forth by FEMA to allow the levees to be certified as providing protection from the 1-percent-annual-chance flood. Therefore, a levee-failure analysis was performed and the overbanks flooded.

Spill areas that were assigned BFEs were analyzed using the USACE HEC-2 computer program. The remaining spills (Zone AO) were traced using normal-depth calculations.

The floodways along East Little Llagas, Tennant, Uvas, and West Branch Llagas Creeks and West Branch Llagas Creek - Lower Split were determined using the USACE HEC-2 computer program (Reference 37) and the equal-conveyance-reduction method. The floodway widths were based on limiting the rise in water-surface elevations to 1 foot due to encroachment. The floodway on Uvas Creek determined under this restudy extends from Hecker Pass Highway to just downstream of Uvas Reservoir. The West Branch Llagas Creek floodway was based on full discharge in the creek except in the area of the Lower Split where a split floodway was determined.

During the base flood, flows from Alamitos Creek, Madrone Channel, Middle Avenue Overflow Area, San Tomas Aquino Creek, Uvas Creek - East Overbank above Highway 101, Uvas Creek - South Spill, Watsonville Road Overflow Area, West Branch Llagas Creek - Lower Split, West Branch Llagas Creek - Middle Split, West Branch Llagas Creek - Upper Split, and West Little Llagas Creek will leave the main flow path. It was determined that if the flows are confined to the main flow path, the computed rise in water-surface elevation due to the increase in discharge would exceed 1 foot. Therefore, floodways were not calculated along these flooding sources.

Calabazas Creek

As part of this restudy, Calabazas Creek was studied from the northern corporate limit at Prospect Avenue to Wardell Road, and Prospect Creek was studied from the confluence with Calabazas Creek to Prospect Avenue. Only a portion of Calabazas Creek is located in Santa Clara County. Prospect Creek is located entirely in the City of Saratoga.

In addition, an approximate total length of 1.5 miles of shallow flooding due to overtopping of Calabazas Creek was analyzed.

Subsequent to the original study, additional flood-protection measures have been constructed, including the following:

- Channel excavation and relocation between Saratoga-Sunny vale Road and the railroad.

- The channel immediately upstream of the railroad has been relocated for a length of approximately 100 feet.

Rock riprap slope protection has been installed over approximately 100 feet of channel starting approximately 100 feet upstream of the railroad.

Water-surface elevations were computed through the use of the USACE HEC-2 computer program (Reference 188). The following data and parameters were used:

1. Channel and overbank cross sections were determined from all surveyed cross sections and topographic mapping provided by the SCVWD (References 142-146).

2. The Manning's "n" roughness values were established based on field observations and USACE and USGS guidelines (References 147-148).

3. The HEC-2 special culvert and bridge routines were used to analyze the channel road crossings. In accordance with USACE guidelines, contraction and expansion coefficients of 0.1 and 0.3 were used for open-channel sections. Contraction coefficients at culverts and bridges ranged from 0.3 to 0.5, depending on configuration. An expansion coefficient of 0.5 was used at bridges. HEC-2 special bridge and culvert routines were used to model the existing road crossings. All culverts and bridges were analyzed based on the as-built plans or surveyed dimensions, and were assumed to be unobstructed (References 148-153 and 181).

4. The downstream starting water-surface elevation for Calabazas Creek was started approximately 800 feet downstream of the limit of study using the water-surface elevation from the SCVWD HEC-2 model (Reference 144).

5. Supercritical flow conditions can occur in some channel reaches. In accordance with FEMA guidelines, subcritical analyses were conducted to determine BFEs for all stream reaches.

6. Split-flow routines were used to determine discharges for overbank-flow paths that are hydraulically separated from the main channel. Split flows were based on a weir coefficient of 2.6.

A separate HEC-2 analysis was performed to determine the depth of the 1-percent-annual-chance flood in the overbank areas, and the calculated depths were less than 1 foot. Therefore, the areas were designated Zone X (shaded) on the FIRM.

7. A multiple-discharge HEC-2 analysis was conducted to determine the discharge in the 48-inch diameter bypass culvert that conveys a portion of the Prospect Creek discharge directly to Calabazas Creek from upstream of Arroyo de Arguello.

8. Floodways were determined by HEC-2 modeling methods limiting the rise in water-surface elevation to a maximum of 1 foot. Equal reduction on each side of the channel was used where possible. A floodway has not been defined for Calabazas Creek from immediately upstream of the railroad to Saratoga-Sunnyvale Road because the full 1-percent-annual-chance flood discharge cannot be contained to pass through the culvert under the railroad and in the channel without causing a water-surface rise of greater than 1 foot.

9. Because the calculation indicated that the peak discharge in Calabazas Creek would result in overtopping of the railroad, the FEMA levee policy has been applied to the railroad embankment. For the maximum

upstream water-surface elevation and split flow, the embankment was assumed to be in place. For the worst-case downstream floodplain, the embankment was assumed not to exist.

10. The levee policy was also applied to a masonry wall located in the overflow area downstream of the railroad tracks between Calabazas Creek and Saratoga-Sunnyvale Road. To determine the downstream floodplain, the wall was assumed not to exist. To determine the overflow east of Saratoga-Sunnyvale Road, the wall was assumed to be in place.

11. The downstream limits of the study for the overflow areas were:

- Where the overflow returns to the Calabazas Creek channel downstream of Prospect Road. It should be noted that the channel downstream of Prospect Road was not part of this restudy.

- The Route 85 Freeway. At these points, the overflow will enter the depressed freeway section.

The floodplain and floodway boundaries, as determined by the hydrologic and hydraulic analyses, were delineated on horizontal-scale Santa Clara County base mapping at a scale of 1”:500’ (Reference 197).

Where the calculated average depth was greater than 1 foot, BFEs were determined. Floodplain boundaries were defined based on the hydraulic model, as determined by subcritical flow analyses. In channel reaches where supercritical flow conditions could occur, the BFEs were based on critical depth.

Where average depth of flow in the split-overflow areas was less than 1 foot, the floodplain area was designated Zone X (shaded).

Floodways were determined using the HEC-2 computer program and the equal-conveyance reduction method. The floodway widths were based on limiting the rise in water-surface or energy gradeline elevations to 1 foot due to encroachment. The floodway analyses were based on containing all split-flow discharges.

Floodways were not determined for the area on Calabazas Creek from immediately upstream of the railroad to Saratoga-Sunnyvale Road because the entire overflow could not be contained without causing a water-surface rise of more than 1 foot.

<u>Lions, Llagas, and North and South Morey Creeks, West Branch Llagas Creek (downstream of Day Road), Llagas Overbank (Old Miller Slough), and Miller Slough</u>

The base flood is contained within the identified channel banks along Llagas Creek, from approximately 950 feet downstream of Pacheco Pass Highway to approximately 80 feet upstream of Pacheco Pass Highway; West Branch Llagas Creek, from its confluence with Miller Slough to just downstream of Leavesley Road, from the railroad to Church Street, and from approximately 950 feet upstream to approximately 1,650 feet upstream of Farrell Avenue; the entire reaches of North and South Morey Creeks; and Miller Slough, from its confluence with West Branch Llagas Creek to its upstream limit.

Because the base flood is contained within the identified channel banks, the regulatory floodway has been removed along West Branch Llagas Creek, from approximately 950 feet upstream of Farrell Avenue to approximately 1,000 feet downstream of Day Road, and along the entire reaches of Lions and North and South Morey Creeks.

The SFHA and regulatory floodway have been removed along Llagas Overbank from approximately 2,100 feet downstream of Pacheco Pass Highway to Pacheco Pass Highway. The SFHAs have been removed along the entire reaches of North and South Morey Creeks, the channelized reach of West Branch Llagas Creek, and Lions Creek within the City of Gilroy corporate limits.

Because the base flood is contained within the identified channel banks, Flood Profile Panels have been removed for Lions and North and South Morey Creeks and Miller Slough. Additionally, Cross Sections A through D along Lions Creek, A through F along North Morey Creek, A and B along South Morey Creek, and A through H along West Branch Llagas Creek (downstream of Day Road) have been deleted from the Floodway Data Table.

Pajaro River

An analysis of the hydraulic characteristics of flood hazards from the source studied was carried out to provide estimated flood elevations of the selected recurrence intervals.

Cross-section data for the backwater analysis were obtained from field surveys and supplemented with existing plans and topographic maps (Reference 201). Bridges, culverts, and other backwater causing obstructions were surveyed to obtain elevation data and structural information.

Channel and overbank roughness factors (Manning's "n") used in the hydraulic computations were based on engineering judgment and field observations of the stream and overbank.

Water-surface elevations of floods for the 1-percent-annual-chance peak discharge were computed using the USACE HEC-2 computer program (Reference 202). Hand calculations were performed when the computer modeling was not applicable.

The starting water-surface elevation for the Pajaro River was based on USACE Floodplain Information studies (References 199-200).

Uvas Creek

Revisions were made to reflect the results of a study of Uvas Creek conducted by the SCVWD and issued by FEMA as a LOMR, dated April 18, 1991. The LOMR applied to the portion of Uvas Creek from approximately 2,000 feet downstream of Thomas Road to Santa Teresa Boulevard. Revisions were made to show the effects of the following:

- The construction of a new Thomas Road bridge; and
- The elevation of the intersection of Miller Avenue and Uvas Park Drive.

The hydraulic analyses were conducted using the USACE HEC-2 computer program (Reference 101). Water-surface elevations were determined using the HEC-2 computer program and BFEs were developed.

A floodway was developed for this portion of Uvas Creek.

As a result of the flood-control project, the BFEs have increased along Uvas Creek from approximately 1,800 feet downstream to approximately 2,550 feet upstream of Thomas Road. From approximately 300 feet downstream to approximately 2,550 feet upstream of Thomas Road, the 1-percent-annual-chance flood is contained by the levee system along the left bank of Uvas Creek and the right channel bank. However, from approximately 1,800 feet downstream to approximately 300 feet downstream of Thomas Road, the 1-percent-annual-chance flood is not contained within the channel.

Roughness factors (Manning's "n") used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the streams and floodplain areas. Roughness factors for all streams studied by detailed methods are shown in Table 8, "Manning's "n" Values".

New Hydraulic Analyses Included in This Revision

For the January 2012 study in Santa Clara County, field survey data for Upper Penitencia Creek Reach 2 and Reach 2 Overflow were collected by Harned Surveying and Engineering, Inc. in November 2010 and included structure geometric data and surveyed contraction and expansion cross sections and cross sections at every 1,000 feet. This data was supplemented with 1-foot contour data provided Santa Clara County for all study reaches. Topographic data provided by Santa Clara County was derived from aerial photogrammetric surveys performed in 2006. All invert elevations, culvert diameters, and bridge geometries for Upper Penitencia Creek Reach 2 and Reach 2 Overflow were measured in the field by Harned Surveying and Engineering, Inc.

Roughness factors (Manning's "n" values) for all flooding sources were chosen by engineering judgment and were based on inspection of the aerial photography.

Manning's "n" roughness values for all reaches in are provided below in Table 8.

The downstream boundary condition for all reaches in the January 2012 study was based on the known water-surface elevation. The slope was measured as the bed slope between the two downstream cross sections.

For the streams studied by detailed methods, water-surface profiles for each reach were computed in HEC-RAS version 4.1 (Reference 216) for the 10-percent-, 2-percent-, 1-percent-, and 0.2-percent-annual-chance flood events.

The HEC-RAS hydraulic models were executed under the assumption of subcritical flow to produce the most conservative water-surface elevations.

Table 8 – Manning's "n" Values

Stream Name	Roughness Values Channel	Overbank
Adobe Creek	0.015 – 0.050	0.035 – 0.070
Alamitos Creek	0.022 – 0.050	0.020 – 0.210
Arroyo Calero	0.050	0.050
Berryessa Creek	0.015 – 0.035	0.025 – 0.050
Calabazas Creek	0.015 – 0.050	0.040 – 0.050
Calera Creek	0.025 – 0.060	0.025 – 0.035
Canoas Creek	0.017 – 0.050	0.030
Coyote Creek	0.025 – 0.057	0.030 – 0.114
East Little Llagas Creek	0.020 – 0.035	0.045
Fisher Creek	0.027 – 0.040	0.035 – 0.045
Fisher Creek Overbank	0.030	0.030
Guadalupe River	0.028 – 0.050	0.020 – 0.100
Hale Creek	0.015 – 0.045	0.050 – 0.060
Lions Creek	0.025	0.030 – 0.080
Llagas Creek	0.025 – 0.050	0.025 – 0.050
Llagas Overbank	0.025 – 0.040	0.025 – 0.050
Los Gatos Creek	0.045	0.045
Madrone Channel	0.014 – 0.035	0.020- 0.040
Middle Avenue Overflow Area	0.045	0.045
Miguelita Creek	0.030	0.030
Miller Slough	0.025 – 0.050	0.025 – 0.070
North Morey Creek	0.020 – 0.030	0.022 – 0.035
Pajaro River	0.080	0.040 – 0.050
Permanente Creek	0.015 – 0.070	0.030 – 0.080
Permanente Diversion	0.015 – 0.030	0.020 – 0.030

Table 8 – Manning's "n" Values, continued

Stream Name	Roughness Values Channel	Roughness Values Overbank
Ross Creek	0.018 – 0.030	0.030
San Tomas Aquino Creek	0.015 – 0.040	0.040
San Tomas Aquino Creek – Reach 2	0.025 – 0.045	0.06-0.09
Sierra Creek	0.015 – 0.030	0.035 – 0.045
Silver Creek	0.015 – 0.035	0.020 – 0.040
South Babb Creek	0.015 – 0.050	0.020 – 0.050
South Morey Creek	0.020 – 0.030	0.022 – 0.065
Stevens Creek	0.015 – 0.050	0.035 – 0.065
Sunnyvale East Channel	0.020 – 0.028	0.03
Sunnyvale West Channel	0.027	0.035
Tennant Creek	0.035	0.040 – 0.080
Thompson Creek	0.020 – 0.050	0.020 – 0.040
Upper Penitencia Creek	0.017 – 0.040	0.020 – 0.040
Upper Penitencia Creek – Reach 2	0.035 – 0.045	0.055 – 0.1
Upper Penitencia Creek – Reach 2 Overflow	0.045	0.055 – 0.1
Upper Silver Creek	0.016 – 0.065	0.025 – 0.040
Uvas Creek	0.016 – 0.065	0.025 – 0.070
Uvas Creek – East Overbank Above Highway 101	0.045	0.045
Uvas Creek – South Spill	0.020	0.045 – 0.120
Watsonville Road Overflow Area	0.040	0.040
West Branch Llagas Creek	0.016 – 0.100	0.020 – 0.055
West Branch Llagas Creek – East Split	0.024 – 0.035	0.050 – 0.045
West Branch Llagas Creek – Lower Split	0.024 – 0.035	0.045
West Branch Llagas Creek – Middle Split	0.035	0.045
West Branch Llagas Creek – Upper Split	0.035	0.045
West Little Llagas Creek	0.030 – 0.050	0.040 – 0.186

The hydraulic analysis for this revision was based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway is computed (Section 4.2), selected cross section locations are also shown on the Flood Boundary and Floodway Map (published separately).

All elevations are referenced to the North American Vertical Datum of 1988 (NAVD88). Elevation reference marks (ERMs) used in this study, and their descriptions, are shown on the FIRM. ERMs shown on the FIRM represent those used during the preparation of this and previous FIS reports. The elevations associated with each ERM were obtained and/or developed during FIS production to establish vertical control for determination of flood elevations and floodplain boundaries shown on the FIRM. Users should be aware that these ERM elevations might have changed since the publication of this FIS. To obtain up-to-date elevation information on National Geodetic Survey (NGS) ERMs shown on this map, please contact the NGS at:

NGS Information Services

NOAA, N/NGS12

SSMC-3, #9202

1315 East-West Highway

Silver Spring, Maryland 20910-3282

(301) 713-3242

www.ngs.noaa.gov

Map users should seek verification of non-NGS ERM monument elevations when using these elevations for construction or floodplain management purposes.

Qualifying bench marks within a given jurisdiction that are cataloged by the National Geodetic Survey (NGS) and entered into the National Spatial Reference System (NSRS) as First or Second Order Vertical and have a vertical stability classification of A, B, or C are shown and labeled on the FIRM with their 6-character NSRS Permanent Identifier.

Bench marks cataloged by the NGS and entered into the NSRS vary widely in vertical stability classification. NSRS vertical stability classifications are as follows:

- Stability A: Monuments of the most reliable nature, expected to hold position/elevation well (e.g., mounted in bedrock)
- Stability B: Monuments which generally hold their position/elevation well (e.g., concrete bridge abutment)
- Stability C: Monuments which may be affected by surface ground movements (e.g., concrete monument below frost line)
- Stability D: Mark of questionable or unknown vertical stability (e.g., concrete monument above frost line, or steel witness post)

In addition to NSRS bench marks, the FIRM may also show vertical control monuments established by a local jurisdiction; these monuments will be shown on the FIRM with the appropriate designations. Local monuments will only be placed on the FIRM if the community has requested that they be included, and if the monuments meet the aforementioned NSRS inclusion criteria.

To obtain current elevation, description, and/or location information for bench marks shown on the FIRM for this jurisdiction, please contact the Information Services Branch of the NGS at (301) 713-3242, or visit its Web site at www.ngs.noaa.gov.

It is important to note that temporary vertical monuments are often established during the preparation of a flood hazard analysis for the purpose of establishing local vertical control. Although these monuments are not shown on the FIRM, they may be found in the Technical Support Data Notebook associated with this FIS and FIRM. Interested individuals may contact FEMA to access this data.

Levee Hazard Analysis

Some flood hazard information presented in prior FIRMs and in prior FIS reports for Santa Clara County and its incorporated communities was based on flood protection provided by levees. Based on the information available and the mapping standards of the NFIP at the time that the prior FISs and FIRMs were prepared, FEMA accredited the levees as providing protection from the 1-percent-annual-chance flood. For FEMA to continue to accredit the identified levees as providing protection from the base flood, the levees must meet the criteria of 44 CFR 65.10, titled “Mapping of Areas Protected by Levee Systems.”

On August 22, 2005, FEMA issued Procedure Memorandum No. 34 - Interim Guidance for Studies Including Levees. The purpose of the memorandum was to help clarify the responsibility of community officials or other parties seeking recognition of a levee by providing information identified during a study/mapping project. Often, documentation regarding levee design, accreditation, and the impacts on flood hazard mapping is outdated or missing altogether. To remedy this, Procedure Memorandum No. 34 provides interim guidance on procedures to minimize delays in near-term studies/mapping projects, to help our mapping partners properly assess how to handle levee mapping issues.

While 44 CFR 65.10 documentation is being compiled, the release of more up-to-date FIRM panels for other parts of a community or county may be delayed. To minimize the impact of the levee recognition and certification process, FEMA issued Procedure Memorandum No. 43 - Guidelines for Identifying Provisionally Accredited Levees (PALs) on March 16, 2007. These guidelines will allow issuance of preliminary and effective versions of FIRMs while the levee owners or communities are compiling the full documentation required to show compliance with 44 CFR 65.10. The guidelines also explain that preliminary FIRMs can be issued while providing the communities and levee owners with a specified timeframe to correct any maintenance deficiencies associated with a levee and to show compliance with 44 CFR 65.10.

FEMA contacted the communities within Santa Clara County to obtain data required under 44 CFR 65.10 to continue to show the levees as providing protection from the 1-percent-annual-chance flood.

FEMA understood that it might take time to acquire and/or assemble the documentation necessary to fully comply with 44 CFR 65.10. Therefore, FEMA put forth a process to provide the communities with additional time to submit all the necessary documentation. For a community to avail itself of the additional time, it had to sign an agreement with FEMA. Levees for which such agreements were signed are shown on the final effective FIRM as providing protection from the 1-percent-annual-chance flood and labeled as a PALs. Communities have 2 years from the date of FEMA's initial coordination to submit to FEMA final accreditation data for all PALs. Following receipt of final accreditation data, FEMA will revise the FIS and FIRM as warranted.

FEMA coordinated with the USACE, the local communities, and other organizations to compile a list of levees that exist within Santa Clara County. Table 9, "List of Structures Requiring Flood Hazard Revisions" lists all levees shown on the FIRM, to include PALs, for which corresponding flood hazard revisions were made.

Approximate analyses of "behind levee" flooding were conducted for all the levees in Table 9 to indicate the extent of the "behind levee" floodplains. The methodology used in these analyses is discussed below.

The approximate levee analysis was conducted using information from existing hydraulic models, where applicable, and USGS topographic maps.

Approximate levee analysis for the City of Mountain View was also conducted using information from 2007 LIDAR Contour data.

The extent of the 1-percent-annual-chance flood in the event of levee failure was determined. Normal-depth calculations were used to estimate the BFE if detailed topographic or representative cross section information was available. The remaining BFEs were estimated from effective FIRM maps. The 1-percent-annual-chance floodplain boundary was traced along the contour line representing the estimated BFE. Topographic features such as highways, railroads, and high ground were used to refine approximate floodplain boundary limits. The 1-pecent-annual-chance peak flow and floodplain widths and depth (assumed at 1 foot) were used to ensure the floodplain boundary was not overly conservative.

Table 9 – List of Structures Requiring Flood Hazard Revisions

Community	Flood Source	Levee Inventory ID (Lat./Long. Coordinates. ; FIRM panel)	USACE Levee
City of Campbell City of San Jose	San Tomas Aquino Creek	P110 (-121.99, 37.272; -121.977, 37.277 06085C0238H)	No
City of Campbell City of San Jose	San Tomas Aquino Creek	P111 (-121.991, 37.272; -121.978, 37.276 06085C0238H)	No
Town of Los Gatos	Los Gatos Creek	P32 (-121.961, 37.257; -121.964, 37.252 06085C0239H)	No
City of Milpitas	Berryessa Creek	P152 (-121.886, 37.411; -121.884, 37.41 06085C0067H)	No
City of Mountain View	Permanente Creek	P136 (-122.085, 37.433; -122.086, 37.423 06085C0037H)	No
City of Mountain View	Permanente Creek	P137 (-122.087, 37.421; -122.087, 37.417 06085C0037H)	No
City of Mountain View	Permanente Creek	P139 (-122.086, 37.435; -122.087, 37.425 06085C0037H)	No
City of Mountain View	South San Francisco Bay	P102 (-122.085, 37.435; -122.068, 37.435 06085C0037H)	No
City of Mountain View	South San Francisco Bay	P126 (-122.098, 37.436; -122.086, 37.435 06085C0036H / 06085C0037H)	No

Table 9 – List of Structures Requiring Flood Hazard Revisions, continued

Community	Flood Source	Levee Inventory ID (Lat./Long. Coordinates. ; FIRM panel)	USACE Levee
City of Sunnyvale	Sunnyvale West Channel	P164 (-122.026, 37.407; -122.02, 37.412 06085C0045H)	No
City of Sunnyvale	Sunnyvale West Channel	P166 (-122.026, 37.407; -122.021, 37.411 06085C0045H)	No

Several levees within Santa Clara County and its incorporated communities meet the criteria of 44 CFR 65.10, titled "Mapping of Areas Protected by Levee Systems." Table 10, "List of Certified and Accredited Levees" lists all levees shown on the FIRM that meet the requirements of 44 CFR 65.10 and have been determined to provide protection from the 1-percent-annual-chance flood.

Table 10 – List of Certified and Accredited Levees

Community	Flood Source	Levee Inventory ID (Lat./Long. Coordinates. ; FIRM panel)	USACE Levee
City of Gilroy	Uvas Creek	P0 (-121.601, 37.007; -121.567, 36.988 06085C0638H / 06085C0639H / 06085C0752H)	Yes
City of Gilroy	West Branch Llagas Creek	P101 (-121.559, 37.009; -121.541, 37.006 06085C0643H)	No
City of Milpitas	Berryessa Creek	P146 (-121.914, 37.446; -121.909, 37.442 06085C0058H)	No
City of Milpitas	Berryessa Creek	P148 (-121.901, 37.438; -121.893, 37.434 06085C0059H / 06085C0067H)	No
City of Milpitas	Berryessa Creek	P158 (-121.909, 37.442; -121.907, 37.44 06085C0058H)	No

Table 10 – List of Certified and Accredited Levees, continued

City of Milpitas	Coyote Creek	P79 (-121.925, 37.453; -121.915, 37.396 06085C0058H / 06085C0066H / 06085C0068H)	Yes
City of Milpitas	Lower Penitencia Creek	P52 (-121.921, 37.454; -121.914, 37.446 06085C0058H)	No
City of Mountain View	Stevens Creek	P141 (-122.068, 37.435; -122.069, 37.408 06085C0037H)	No
City of Mountain View	Stevens Creek	P143 (-122.068, 37.422; -122.069, 37.408 06085C0037H)	No
City of Palo Alto	Matadero Creek	P132 (-122.133, 37.424; -122.12, 37.436 06085C0017H / 06085C0036H)	No
City of Palo Alto	Matadero Creek	P134 (-122.133, 37.424; -122.122 37.435 06085C0017H / 06085C0036H)	No
City of San Jose	Coyote Creek	P145 (-121.927, 37.448; -121.924, 37.446 06085C0058H)	Yes
City of San Jose	Coyote Creek	P24 (-121.924, 37.447; -121.915, 37.396 6085C0058H / 06085C0066H / 06085C0068H)	Yes
City of San Jose City of Santa Clara	Guadalupe River	P177 (-121.941, 37.396; -121.933, 37.374 06085C0064H / 06085C0068H / 06085C0231H)	Yes
City of San Jose	Guadalupe River	P181 (-121.967, 37.419; -121.932, 37.374 06085C0064H / 06085C0065H / 06085C0068H / 06085C0231H)	Yes

Table 10 – List of Certified and Accredited Levees, continued

Community	Flood Source	Levee Inventory ID (Lat./Long. Coordinates. ; FIRM panel)	USACE Levee
City of San Jose	Thompson Creek	P57 (-121.808, 37.324; -121.795, 37.314 06085C0258H)	No
City of San Jose	Thompson Creek	P58 (-121.808, 37.324; -121.797, 37.316 06085C0258H)	No
City of Santa Clara	Calabazas Creek	P168 (-121.986, 37.413; -121.987, 37.389 06085C0063H / 06085C0065H)	No
City of Santa Clara	Calabazas Creek	P170 (-121.987, 37.407; -121.987, 37.389 06085C0063H / 06085C0065H)	No
City of Santa Clara	San Tomas Aquino Creek	P172 (-121.98, 37.416; -121.968, 37.384 06085C0063H / 06085C0064H / 06085C0065H)	No
City of Santa Clara	San Tomas Aquino Creek	P174 (-121.981, 37.416; -121.969, 37.384 06085C0063H / 06085C0064H / 06085C0065H)	No
City of Santa Clara	Guadalupe River	P176 (-121.968, 37.418; -121.941, 37.395 06085C0064H / 06085C0065H)	Yes

3.3 Vertical Datum

All FISs and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised FISs and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), many FIS reports and FIRMs are being prepared using NAVD88 as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to NAVD88. Structure and ground elevations in the community must, therefore, be referenced to NAVD88. It is important to note that adjacent communities may be referenced to NGVD29. This may result in differences in BFEs across the corporate limits between the communities.

The conversion factor from NGVD29 to NAVD88 was 2.85 for all streams in Santa Clara County.

As noted above, the elevations shown in the FIS report and on the FIRM for Santa Clara County are referenced to NAVD88. Ground, structure, and flood elevations may be compared and/or referenced to NGVD29 by applying a standard conversion factor.

The BFEs shown on the FIRM represent whole-foot rounded values. For example, a BFE of 102.4 will appear as 102 on the FIRM and 102.6 will appear as 103. Therefore, users that wish to convert the elevations in this FIS to NGVD29 should apply the stated conversion factor(s) to elevations shown on the Flood Profiles and supporting data tables in the FIS report.

For more information on NAVD88, see Converting the National Flood Insurance Program to the North American Vertical Datum of 1988, FEMA Publication FIA-20/June 1992, or contact the Spatial Reference System Division, National Geodetic Survey, NOAA, Silver Spring Metro Center, 1315 East-West Highway, Silver Spring, Maryland 20910 (Internet address http://www.ngs.noaa.gov).

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance flood elevations; delineations of the 1-percent and 0.2-percent-annual-chance floodplains; and 1-percent-annual-chance floodway. This information is presented on the FIRM and in many components of the FIS, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community. For the stream studied in detail, the 1-percent and 0.2-percent-annual-chance floodplains have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale and a contour interval as shown on Table 11, "Topographic Map Information."

The 1-percent and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM (published separately). On this map, the 1-percent-annual-chance floodplain boundary corresponds to the SFHA (Zones A and AE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1-percent and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM (published separately).

Flood boundaries for creeks studied by approximate methods were established according to the professional judgment of engineers familiar with the region taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

Small areas within the flood boundaries may lie above the flood elevations and, therefore, not be subject to flooding; owing to limitations of the map scale, such areas are not shown.

For the January 2012 study in Santa Clara County, new flood zones were developed and mapped for the detailed study reaches described in Section 2.1. Flood zones for these studies were delineated using the 1-foot contour data obtained from Santa Clara County. Baker's RiverSystems was used to post-process the model data from HEC-RAS and generate draft floodplain boundaries. The draft floodplain boundaries were reviewed by an engineer and model modifications were made where appropriate. Final floodplain boundaries were derived from HEC-GeoRAS (References 217 and 218) and manual adjustment of automated floodplain output using engineering judgment. Flood profiles were created from HEC-RAS using RASPLOT software (Reference 215)

Table 11 – Topographic Map Information

Community	Scale	Contour Interval	Reference
City of Cupertino	1” : 1,200’	2 foot	47, 49
City of Gilroy	1” : 1,200’	2 foot	51
City of Los Altos	1” : 600’	2 foot	61-63, 163
	1” : 1,200’		
Town of Los Altos Hills	1” : 600’	2 foot	164
	1” : 24,000’	5, 10, 20, & 40 foot	165-168
Town of Los Gatos	1” : 600’	2 foot	68
	1” : 1,200’	2 foot	69
City of Milpitas	1” : 600’ (original)	2 foot	77
	1” : 1,200’ (original)	2 foot	78
	1” : 500’ (restudy)	*	*
City of Morgan Hill	1” : 1,200’ (original)	2 foot	83
	1” : 200’ (restudy)	*	185
City of Mountain View	1” : 480’ (original)	2 foot	87
	1” : 600’ (original)	2 foot	86
	1” : 1,200’ (original)	2 foot	85
	1” : 200’ (restudy)	*	*
City of Palo Alto	1” : 600’ (original)	2 foot	85, 97, 99
	1” : 1,200’ (original)	2 foot	85, 97, 99
	1” : 3,600’ (restudy)	1 foot	36
City of San Jose	1” : 6,000’		169
City of Saratoga	1” : 600’	2 foot	141
	1” : 1,200’	2 foot	47
City of Sunnyvale	1” : 24,000’	5, 20, & 40 foot	170
Santa Clara County (Unincorporated areas)	1” : 6,000’	*	54, 57-58, 171-176
	1” : 12,000’	*	

*Data not available

City of Campbell

No FIS available.

City of Cupertino

Limited areas of Cupertino are subject to sheetflow; that is, shallow overland flooding that is generally less than 3 feet deep and characterized by unpredictable flow paths. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent streamway and are affected principally by obstructions in the flooded area.

City of Gilroy

Areas subject to sheetflow flooding were delineated using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels, or where runoff in excess of storm drain capacity would collect and pond, were evaluated as part of the sheetflow flooding investigations.

City of Los Altos

For stream channels designated as “1-percent-annual-chance flood discharge contained in channel,” the 1-percent-annual-chance flood boundaries are based on the existing channel alignment and right-of-way.

For those streams studied by approximate methods, boundaries were determined using a Santa Clara County plat map at a scale of 1”:6,000’ (Reference 177).

Town of Los Altos Hills

For streams studied by approximate methods, the boundary of the 1-percent-annual-chance flood was delineated by slope-conveyance procedures using field cross sections and information on previous flooding provided by local officials and residents.

Town of Los Gatos

Daves Creek was not found to be a source of flooding; therefore, no boundaries were determined for its studied segment.

Areas subject to sheetflow flooding were delineated using surveyed elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheetflow flooding investigations.

Flood boundaries for creeks, which were studied by approximate methods, were established according to the professional judgment of engineers familiar with the region, taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

No boundaries were delineated for the segment of Smith Creek that was studied by approximate methods, due to the existence of a culvert, which contains the 1-percent-annual-chance floodflow.

City of Milpitas

Flood boundaries for creeks studied by approximate methods were established according to the professional judgment of engineers familiar with the region taking into account flood elevations estimated from available data, existing hydrologic and hydraulic analyses, correlations with similar streams, and field observations.

City of Monte Sereno

No FIS available.

City of Morgan Hill

Approximate flood boundaries in some portions of the study were taken from FEMA's Flood Hazard Boundary Map (Reference 178).

City of Mountain View

Approximate 1-percent-annual-chance floodplain boundaries in some portions of the study area were taken directly from the FIRM for the City of Mountain View (Reference 179).

City of Palo Alto

For stream channels designated as "1-percent-annual-chance flood discharge contained in channel," the 1-percent-annual-chance flood boundaries are based on the existing channel alignment and right-of-way.

The floodplain boundaries for areas subject to sheetflow and ponding (Zones AO and AH) were based on surveyed and photogrammetric elevations, and were delineated to include areas with average flood depths greater than 1 foot for the 1-percent-annual-chance flood. The water-surface elevations of flooding in these areas are essentially independent of those along the adjacent stream channel and are affected principally by obstructions in the flood areas.

City of San Jose

Shallow flooding and approximate boundaries were delineated using the aforementioned maps.

For stream channels designated as "1-percent-annual-chance flood discharge contained in channel," the 1-percent-annual-chance flood boundaries are based on the existing channel alignment and right-of-way.

Approximate 1-percent-annual-chance floodplain boundaries in some portions of the study area were taken directly from the Flood Hazard Boundary Map (Reference 180).

As part of the 2012 update, new hydraulic analyses were performed on portions of San Tomas Aquino Creek Reach 2, Upper Penitencia Creek Reach 2, Upper Penitencia Creek Reach 2 Overflow, and on breakout overflows from Coyote Creek.

City of Santa Clara

Between cross sections, the boundaries were developed photogrammetrically, using aerial photos at a scale of 1":12,000' (Reference 125). In areas studied by approximate methods, maps at a scale of 1":480' were used (Reference 129). Sheetflow flooding was delineated photogrammetrically, using aerial photos (Reference 125).

For stream channels designated as "1-percent-annual-chance flood discharge contained in channel," the 1-percent-annual-chance flood boundaries are based on the existing channel alignment and right-of-way.

City of Saratoga

Areas subject to sheet-flow flooding were delineated using surveyed and photogrammetric elevations, field investigations by experienced engineers, and hand calculations based on normal depths. Areas where overflow from channels or runoff in excess of storm drain capacity would collect and pond were evaluated as part of the sheet-flow flooding investigations.

The floodplain and floodway boundaries along Calabazas and Prospect Creeks, as determined by the hydrologic and hydraulic analyses, were delineated on horizontal-scale Santa Clara County base mapping at a scale of 1":500' (Reference 181),

Where the calculated average depth was greater than 1 foot, BFEs were determined. In accordance with FEMA guidelines, floodplain boundaries were defined based on the hydraulic model, as determined by subcritical flow analyses. In channel reaches where supercritical flow conditions could occur, the BFEs are based on critical depth.

Where average depth of flow in the split-overflow areas is less than 1 foot, the floodplain area is designated Zone X (shaded). The floodplain boundaries are a composite of the worst-case condition.

City of Sunnyvale

In general, most of the City of Sunnyvale is designated as Zone X (shaded) on the FIRMs. Due to the limited capacity of the storm drainage system, the 1-percent-annual-chance flood will subject significant portions of the city to shallow sheetflow as floodwaters in excess of the storm drain capacity flow down the streets. Sheetflow areas (Zone AO) delineated in this study are those areas where the water would be approximately 1 to 1.5 feet deep during the 1-percent-annual-chance flood. Greater depths would occur during the 0.2-percent-annual-chance flood. Because all of the developed area in the City of Sunnyvale would be subject to some shallow sheetflow during a 1-percent-annual-chance flood, those areas not in Zone AO or other SFHAs (Zones AE, AH, and VE), were given a shaded Zone X designation. However, several areas experience more severe flooding conditions than does most of the community due to the nature of the topography. These areas are all located between SH-237 and Bayshore Freeway on both the Sunnyvale East Channel and Sunnyvale West Channel. This flooding is a result of overflow of the channels plus the effect of sheetflow moving across the community toward the bay, and occurs where there are topographic lows in the land.

Santa Clara County (Unincorporated areas)

Shallow flooding and approximate boundaries were delineated using the cited maps.

Approximate flood boundaries in some portions of the study area were taken from the Flood Hazard Boundary Map (Reference 182).

For stream channels designated as “1-percent-annual-chance flood discharge contained in channel,” the 1-percent-annual-chance flood boundaries are based on the existing channel alignment and right-of-way.

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 1-percent-annual-chance flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as a minimum standard that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this FIS were computed for certain stream segments on the basis of equal conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections. The computed floodways are shown on the revised FIRM (published separately). In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

As shown on the FIRM (published separately), the floodway boundaries were determined at cross sections; between cross sections, the boundaries were interpolated. In cases where the floodway and 1-percent-annual-chance flood boundaries are close together, only the floodway boundary has been shown.

<u>City of Campbell</u>

No FIS available.

<u>City of Cupertino</u>

Floodway limits were always calculated to be at or inside the 1-percent-annual-chance floodplain limits. This resulted in some floodway limits being located within the banks of the existing channels. These types of floodway limits must be regarded as minimum criteria, as considerations of velocities of flow and the slopes of banks could often yield a more prudent setback to allow for bank sloughing. The FIRM (published separately), however, shows no encroachment within the natural channel, which is in accordance with Federal Insurance Administration guidelines.

Unlike the typical floodway cross sections shown in Figure 1, some of the channels in Cupertino have no overbank areas that allow a continuous water surface across the channel and the overbanks. In general, overbank areas are lower than the bank elevations, so that once water overtops the channel banks, it flows along its own path as sheetflow. Floodways are not applicable in sheetflow areas; therefore, no floodway was computed. Forcing floodwaters to stay within channel banks during the 1-percent-annual-chance flood by placing theoretical encroachments could lead to worsening of downstream overflow and subject new properties to flooding.

Floodways were developed only for Stevens and Permanente Creeks. The floodway on Stevens Creek is broken at Stevens Creek Boulevard and to the north. In this area, a floodway could not be drawn that would meet FEMA guidelines due to weir flow on the east side of the floodplain at Stevens Creek

Boulevard. A floodway was not developed for Calabazas Creek as the channel has no adjoining overbank area and floodflow in excess of capacity leaves the channel and flows independently as sheetflow.

City of Gilroy

Floodway limits were always calculated to be at or inside the 1-percent-annual-chance floodplain limits. This resulted in some floodway limits being located within the banks of the existing channels. These types of floodway limits must be regarded as minimum criteria, as considerations of velocities of flow and the slopes of banks could often yield a more prudent setback to allow for bank sloughing.

Unlike the typical floodway cross sections shown in Figure 1, a portion of the channels in Gilroy have no overbank areas that allow a continuous water surface across the channel and the overbanks. In general, overbank areas are lower than the bank elevations, so that once water overtops the channel banks, it flows along its own path as sheetflow. The following describes the floodways in Gilroy:

Lions Creek

Upstream of the confluence with North Morey Channel, the floodway was based on equal-conveyance reduction. Downstream of this confluence, the floodway was set at the 1-percent-annual-chance flood boundary. Any encroachment in the downstream area would result in a break in the floodway with floodwaters being transferred to West Branch Llagas Creek.

Miller Slough

Miller Slough is an undersized channel that cuts through the plains of an alluvial valley running through a rather densely developed section of the City of Gilroy. Floodways were found not to be applicable for Miller Slough. Unlike the typical cross section shown in Figure 1, the cross sections along Miller Slough will not allow a continuous water surface across the channel and adjoining overbanks. Two overbank conditions generally exist:

1. The overbanks may be level with the top of the channel bank and only after the cross section is extended from 200 to 300 feet beyond the channel does it begin to slope away from the channel.

2. The channel banks are perched from floodwaters overtopping the banks and depositing sediment, with overbanks sloping away from the channel.

These topographic conditions result in sheetflow areas caused by the channel overflows and a separate water surface within the channel.

Forcing floodwaters to stay within the channel banks during the 1-percent-annual-chance flood by placing encroachments parallel to the channel causes flooding problems, which did not exist prior to the designation of the floodway. In addition, most of the overbank areas are developed with many structures either along or close to the channel. Filling in the gaps between these existing structures would augment downstream overflows and could cause new overflows.

Because of the existing development, undersized channel capacity, and inadequate carrying capacity in a reasonable floodway width, floodways are not applicable for Miller Slough.

North and South Morey Creek

The floodway for South Morey Creek was determined by equal-conveyance reduction. In areas of perched channel, the floodway was placed at the limits of that portion of the 1-percent-annual-chance floodplain where there was a continuous water surface across the channel and adjoining overbanks. Because the floodway prevented the stream-to-stream interchange of floodwaters, the flow rates used in floodway determinations were different from those used in the floodplain delineations.

Upstream of the study limit, it was not possible to contain the total flow rate with a rise in water surface of 1 foot or less. An overflow across Morey Avenue was the result.

Ronan Channel

The floodway for Ronan Channel is defined by the channel banks because the channel is designed to contain the 1-percent-annual-chance flood

Uvas Creek

The floodway for Uvas Creek upstream of Thomas Road was determined by equal-conveyance reduction methods except where limited by channel banks. Downstream of Thomas Road, the channel capacity falls well below that necessary to convey the 1-percent-annual-chance flood and extensive overtopping of the existing levee occurs. The significant shallow flooding of areas adjacent to Uvas Creek downstream of Thomas Road is the result of a spill over the south bank immediately upstream of Monterey Highway and numerous spills over the north bank between Thomas Road and Monterey Highway.

Floodways for Uvas Creek from 1,100 feet downstream of Thomas Road to the limit of study at Monterey Highway were not applicable under FEMA standards. However, raising of the water-surface elevation downstream of Thomas Road through construction of a levee or extensive filling for development would cause overtopping of the levee upstream of Thomas

Road has the potential to cause new flooding problems and/or increase the severity of existing problems due to building location and density.

West Branch Llagas Creek

The floodway for West Branch Llagas Creek is generally based on equal-conveyance reduction. This method was used upstream of the high mounds that are north of the lake in Las Animas Park. Downstream of this point, the western floodway boundary was set at the west bank of the channel, and there was no eastern floodway boundary. This unique situation is caused by the high mounds acting as barriers to the flow and necessitates that the floodway be discontinuous near the lake. This discontinuity results in a spillage of floodwaters from the floodway on West Branch Llagas Creek into the floodplain for Miller Slough.

City of Los Altos

The floodway boundaries were determined at cross sections; between cross sections, the boundaries were interpolated. The FIRM (published separately) shows the floodway boundaries determined for the City of Los Altos. The 1-percent-annual-chance floodplain boundaries are not shown for reaches in which the boundaries are not significantly different from the floodway boundaries.

Unlike the typical floodway cross sections shown in Figure 1, a portion of the channels in Los Altos have no overbank areas that allow a continuous water surface across the channel and the overbanks. In general, overbank areas are lower than the bank elevations, so that once water overtops the channel banks, it flows along its own path as sheetflow. Floodways are not applicable in sheetflow areas; therefore, no floodway was computed. Forcing floodwaters to stay within channel banks during the 1-percent-annual-chance flood by placing theoretical encroachments could lead to worsening of downstream overflow and subject new properties to flooding.

A floodway was developed only for Adobe Creek. Floodways were not developed for Hale or Permanente Creeks or for the Permanente Diversion, as the channels have no adjoining overbank areas and floodflows in excess of capacity leave the channel and flow independently as sheetflow. For Stevens Creek, the 1-percent-annual-chance flood is well within the banks of the channel, and floodway limits would more appropriately be set using considerations of velocity of flow and slopes of channel banks to produce a setback allowing for bank sloughing.

Town of Los Altos Hills

Floodways were not computed for Barron Creek and Adobe Creek upstream from O’Keefe Lane because floodway calculations indicated supercritical flow and resultant high velocities. Floodways were not computed for Arastradero, Matadero, and Purissima Creeks because the 1-percent-annual-chance flood is contained in the channel. No floodway was computed for Concepcion Drainage or

Hale Creek due to the fully developed nature of their 1-percent-annual-chance floodplain.

City of Los Gatos

No floodway has been delineated for the impoundment area created by Vasona Dam, on Los Gatos Creek, because within the confines of such an impoundment, conveyance is undefined; therefore, a floodway is not appropriate.

City of Milpitas

Unlike typical floodway cross sections, the channels in Milpitas have no overbank areas that allow a continuous water surface across the channel and the overbanks. In general, overbank areas are lower than the bank elevations, so that once water overtops the channel banks, it flows along its own path as sheetflow. Floodways are not applicable in sheetflow areas; therefore, no floodway data table is included. Forcing floodwaters to stay within channel banks during the 1-percent-annual-chance flood by placing imaginary encroachments could lead to worsening of downstream overflow and subject new properties to flooding.

Due to the extensive sheetflow in the floodplains and the extent of urbanization in the community, no floodways were designated on the channels studied in Milpitas.

City of Monte Sereno

No FIS available.

City of Morgan Hill

Five breaks occur in the floodway along West Little Llagas Creek. Four are the result of inadequate carrying capacity of culverts and perked channels. At Llagas Road, Hale Avenue, Wright Avenue, Monterey Highway near 4th Street, and Monterey Highway near Watsonville Road, breaks in the floodway with attendant shallow overflows are necessary, as the entire flow rate cannot be contained in a floodway.

The undersized channel near Spring Avenue limits the flow that may be contained within a floodway, which includes the channel plus adjacent floodplain. A break in the floodway must occur at this point.

City of Mountain View

Unlike typical floodway cross sections, the channels in Mountain View have no overbank areas that allow a continuous water surface across the channel and the overbanks. In general, overbank areas are lower than the banks, so that once water overtops the channel banks, it flows along its own path as sheetflow. Floodway are not applicable in sheetflow areas.

Forcing floodwaters to stay within channel banks during the 1-percent-annual-chance flood by placing imaginary encroachments parallel to the channels is not reasonable because most of the areas along the channels are totally developed, with many structures along or close to the channel. Filling in the gaps between structures is unreasonable and upstream encroachments could lead to a worsening of some downstream overflows or could cause new overflows, thus subjecting new properties to overflow from the channels.

Due to the sheetflow in the floodplains and the extent of urbanization in the community, no floodways were designated on the channels studied in Mountain View.

City of Palo Alto

Channels in Palo Alto generally have no overbank areas that allow a continuous water surface across the channel and the overbanks. Overbank areas are typically lower than the channel bank elevations, so that once water overtops the channel banks, it flows along a separate path as sheetflow. Floodways are not applicable in sheetflow areas.

Due to the extensive sheetflow in the floodplains and the extent of urbanization in the community, no floodways were designated on the channels studied in Palo Alto.

It is not appropriate to delineate floodways for tidal water bodies; therefore, no floodway is presented for San Francisco Bay.

City of San Jose

Many perched channels in San Jose have no overbank areas that allow a continuous water surface across the channel and the overbanks. Instead, overbank areas are lower than the channel bank elevations; therefore, once water overtops the channel banks, it flows along a separate path as sheetflow. Floodways are not applicable in sheetflow areas.

Forcing floodwaters from perched channels to stay within the channel banks during the 1-percent-annual-chance flood by placing imaginary encroachments parallel to the channels is not reasonable as most of the areas along the channels are totally developed, and many structures area along or close to the channel. Filling in the gaps between structures is unreasonable, and upstream encroachments could lead to a worsening of some downstream overflows or could cause new overflows and subject new properties to overflow from the channels.

Floodways were designated for the following creeks:

Alamitos Creek

For the valley channel reach from approximately 2,000 feet upstream of the Guadalupe River confluence to Camden Avenue, floodways were based on equal-conveyance reduction. From the confluence with the Guadalupe River to a point 2,000 feet upstream, the floodways approximated the 1-percent-annual-chance flood boundaries and included the existing channel and percolation ponds. The reach through the SCVWD percolation ponds has depths in excess of 4 feet. For the reach of Alamitos Creek from upstream of McKean Road to Bertram Road, the floodway was computed on the basis of equal conveyance reduction; however, due to hazardous velocities in the area, the floodway was made coincident with the 1-percent-annual-chance flood boundary.

Coyote Creek

Floodway boundaries for Coyote Creek were based on equal-conveyance reduction unless the channel was perched. For perched channel reaches, floodways were designated where the flow rate could be increased to obtain up to a 1-foot rise in water surface without causing flooding that did not exist during the 1-percent-annual-chance flood. An exception to these methods for designating floodways occurred through a reach of quarry approximately 3 miles downstream of Anderson Dam. Floodways for the quarry reach were based on the limit of effective flow for the east floodway boundary and the top of the levee (the 1-percent-annual-chance flood boundary) for the west floodway boundary. Floodways could not be designated from Sinclair Freeway (Interstate Highway 280) downstream to the Silver Creek confluence or from 2,500 feet upstream of Trimble Road to San Francisco Bay due to the perched channel condition. For the upstream reach of Coyote Creek, the floodway was delineated to preserve the volume-discharge relationship as much as possible with the minimum effect on the overall floodway. This was necessary to include the effect of the percolation ponds in this area. In cases in which a percolation pond could not be excluded from the floodway in its entirety, the floodway was delineated to include the entire pond. The ponds are either completely within the floodway or completely out.

Fisher Creek and Fisher Creek Overbank

The floodway for Fisher Creek was based on equal-conveyance reduction for valley channel reaches. For perched channel reaches, the floodway was designated where the flow rate could be increased up to a rise in water surface of 1 foot without causing flooding that did not exist during the 1-percent-annual-chance flood. From a point approximately 500 feet upstream of Richmond Avenue to the upstream limit of study at Tilton Avenue, floodways were based on equal-conveyance reduction. In two separate reaches downstream of Richmond Avenue, it was necessary to maintain storage volumes so as to minimize changes in downstream flooding. The first reach included the separately modeled Fisher Creek Overbank floodway, which contained the east and west overflows from the spill 500 feet upstream of Richmond Avenue. This reach terminated at Hailey Avenue. The second reach, requiring storage, extended from approximately 1,500 feet downstream of Bailey Avenue to a point approximately 3,000 feet upstream of the Coyote Creek confluence. For both of these reaches, the designated floodways approximated the 1-percent-annual-chance flood boundaries. Floodway data are not presented for cross sections A through I on Fisher Creek because the floodway was based on storage considerations rather than conveyance.

City of Santa Clara

Channels in Santa Clara generally have no overbank areas that allow a continuous water surface across the channel and the overbanks. Overbank areas are typically lower than the channel bank elevations, so that once water overtops the channel banks, it flows along a separate path as sheetflow. Floodways are not applicable in sheetflow areas.

It is not necessary to delineate floodways parallel to the channels because most of the areas along the channels are totally developed with many structures along or close to the channel. Filling the gaps between structures is unreasonable; upstream encroachments could lend to a worsening of some downstream overflows or could cause new overflows thus subjecting new properties to overflow from the channels.

In general, most of the City of Santa Clara is designated as a Shaded Zone X (0.2-percent-annual-chance flood). The limited capacity of the storm drainage system will subject almost the entire city to shallow sheetflow during the 1-percent-annual-chance flood as floodwaters in excess of the storm drain capacity flow down the streets.

Due to the extensive sheetflow in the floodplains and the extent of urbanization in the community, no floodways were designated on the channels studied in Santa Clara.

City of Saratoga

Floodways were not determined for the area on Calabazas Creek from immediately upstream of the railroad to Saratoga-Sunnyvale Road because the entire overflow could not be contained without causing a water-surface rise of more than 1 foot (Reference 4).

Prospect Creek

Floodways along Prospect Creek were determined using the HEC-2 computer program and the equal-conveyance-reduction method. The floodway widths are based on limiting the rise in water-surface or energy-gradeline elevations to 1 foot due to encroachment. The floodway analyses are based on containing all split-flow discharges.

City of Sunnyvale

Due to existing flood-control measures in the City of Sunnyvale, flooding that occurs is primarily sheetflow in nature; according to FEMA criteria, the establishment of a floodway is not required.

Santa Clara County (Unincorporated areas)

Some of the channels in Santa Clara County have no overbank areas that allow a continuous water surface across the channel and the overbanks. In general, overbank areas are lower than the bank elevations; therefore, when water overtops the channel banks, it flows along its own path as sheetflow. Floodways are not applicable in sheetflow areas.

Forcing floodwaters from perched channels to stay within the channel banks during the 1-percent-annual-chance flood by placing imaginary encroachments parallel to the channels is not reasonable as most of the areas along the channels are totally developed, and many structures area along or close to the channel. Filling in the gaps between structures is unreasonable, and upstream encroachments could lead to a worsening of some downstream overflows or could cause new overflows and subject new properties to overflow from the channels.

Due to the extensive sheetflow in the floodplains and the extent of urbanization in the community, no floodways were designated on Calabazas Creek, Canoas Creek, the Guadalupe River, Miller Slough, the downstream potion of Permanente Creek, Silver Creek, South Babb Creek, Thompson Creek, and Upper Penitencia Creek.

Floodways, where applicable on streams flowing through Santa Clara County, are described as follows:

Alamitos Creek

For the valley channel reach from approximately 2,000 feet upstream of the Guadalupe River confluence to Camden Avenue, floodways were based on equal-conveyance reduction. From the confluence with the Guadalupe River to a point 2,000 feet upstream, the floodways approximated the 1-percent-annual-chance flood boundaries and included the existing channel and percolation ponds. The reach through the SCVWD percolation ponds has depths in excess of 4 feet. For the reach of Alamitos Creek from upstream of McKean Road to Bertram Road, the floodway was computed on the basis of equal conveyance reduction; however, due to hazardous velocities in the area, the floodway was made coincident with the 1-percent-annual-chance flood boundary.

Calabazas Creek

Floodways along Calabazas Creek were determined using the HEC-2 computer program and the equal-conveyance-reduction method. The floodway widths are based on limiting the rise in water-surface or energy-gradeline elevations to 1 foot due to encroachment. The floodway analyses are based on containing all split-flow discharges.

Floodways were not determined for the area on Calabazas Creek from immediately upstream of the railroad to Saratoga-Sunnyvale Road because the entire overflow could not be contained without causing a water-surface rise of more than 1 foot (Reference 1).

Coyote Creek

Floodway boundaries for Coyote Creek were based on equal-conveyance reduction unless the channel was perched. For perched channel reaches, floodways were designated where the flow rate could be increased to obtain up to a 1-foot rise in water surface without causing flooding that did not exist during the 1-percent-annual-chance flood. An exception to these methods for designating floodways occurred through a reach of quarry approximately 3 miles downstream of Anderson Dam. Floodways for the quarry reach were based on the limit of effective flow for the east floodway boundary and the top of the levee (the 1-percent-annual-chance flood boundary) for the west floodway boundary. Floodways could not be designated from Sinclair Freeway (Interstate Highway 280) downstream to the Silver Creek confluence or from 2,500 feet upstream of Trimble Road to San Francisco Bay due to the perched channel condition. For the upstream reach of Coyote Creek, the floodway was delineated to preserve the volume-discharge relationship as much as possible with the minimum effect on the overall floodway. This was necessary to include the effect of the percolation ponds in this area. In cases in which a percolation pond could not be excluded from the floodway in its entirety, the floodway was delineated to include the entire pond. The ponds are either completely within the floodway or completely out.

East Little Llagas Creek

The carrying capacity of the culvert at U.S. Highway 101 limits the flow that may be contained within a floodway. A break in the floodway is necessary to allow an overflow.

Fisher Creek and Fisher Creek Overbank

The floodway for Fisher Creek was based on equal-conveyance reduction for valley channel reaches. For perched channel reaches, the floodway was designated where the flow rate could be increased up to a rise in water surface of 1 foot without causing flooding that did not exist during the 1-

percent-annual-chance flood. From a point approximately 500 feet upstream of Richmond Avenue to the upstream limit of study at Tilton Avenue, floodways were based on equal-conveyance reduction. In two separate reaches downstream of Richmond Avenue, it was necessary to maintain storage volumes so as to minimize changes in downstream flooding. 'The first reach included the separately modeled Fisher Creek Overbank floodway, which contained the east and west overflows from the spill 500 feet upstream of Richmond Avenue. This reach terminated at Hailey Avenue. The second reach, requiring storage, extended from approximately 1,500 feet downstream of Bailey Avenue to a point approximately 3,000 feet upstream of the Coyote Creek confluence. For both of these reaches, the designated floodways approximated the 1-percent-annual-chance flood boundaries. Floodway data are not presented for cross sections A through I on Fisher Creek because the floodway was based on storage considerations rather than conveyance.

Lions Creek

Upstream of the confluence with North Morey Channel, the floodway was based on equal-conveyance reduction. Downstream of this confluence, the floodway was set at the 1-percent-annual-chance flood boundary. Any encroachment in the downstream area would result in a break in the floodway with floodwaters being transferred to West Branch Llagas Creek.

Llagas Creek and Llagas Overbank

Floodways in the Llagas Creek basin were unusual in that the overflow floodway was closely related to floodway determined in the channel. Floodway boundaries in the perched channel areas were the 1-percent-annual-chance floodplain boundaries with the flow rate increased to obtain a 1-foot rise without causing flooding that did not exist during the 1-percent-annual-chance flood. Where the entire 1-percent-annual-chance floodflow rates could not be contained, floodways were delineated in these spill and overflow areas. Where the channel was not perched, equal-conveyance reduction was used to determine floodways. The reach of stream between Church Avenue and U.S. Highway 101 has no floodway on the east overbank due to the fact that a floodway could not be drawn to meet current standards, as land adjacent to the channel is not subject to flooding. Finally, the quarry area, approximately 2,500 feet west of Monterey Road adjacent to Llagas Creek, is inundated by the 1-percent-annual-chance flood due to low banks at the upstream end. The water reenters the channel on the downstream end of the quarry. Therefore, through the quarry area, the floodway boundary coincides with the 1-percent-annual-chance flood boundary, thus eliminating spilling into the quarry.

North and South Morey Creek

The floodway for South Morey Creek was determined by equal-conveyance reduction. In areas of perched channel, the floodway was placed at the limits of that portion of the 1-percent-annual-chance floodplain where there was a continuous water surface across the channel and adjoining overbanks. Because the floodway prevented the stream-to-stream interchange of floodwaters, the flow rates used in floodway determinations were different from those used in the floodplain delineations.

Upstream of the study limit, it was not possible to contain the total flow rate with a rise in water surface of 1 foot or less. An overflow across Morey Avenue was the result.

Ronan Channel

The floodway for Ronan Channel is defined by the channel banks because the channel is designed to contain the 1-percent-annual-chance flood.

Stevens Creek

The floodway on Stevens Creek is broken at Stevens Creek Boulevard and to the north. In this area, a floodway could not be drawn due to weir flow on the eastern side of the floodplain at Stevens Creek Boulevard.

Uvas Creek

The floodway for Uvas Creek upstream of Thomas Road was determined by equal-conveyance reduction methods except where limited by channel banks. Downstream of Thomas Road, the channel capacity falls well below that necessary to convey the 1-percent-annual-chance flood and extensive overtopping of the existing levee occurs. The significant shallow flooding of areas adjacent to Uvas Creek downstream of Thomas Road is the result of a spill over the south bank immediately upstream of Monterey Highway and numerous spills over the north bank between Thomas Road and Monterey Highway.

Floodways for Uvas Creek from 1,100 feet downstream of Thomas Road to the limit of study at Monterey Highway were not applicable under FEMA standards. However, raising of the water-surface elevation downstream of Thomas Road through construction of a levee or extensive filling for development would cause overtopping of the levee upstream of Thomas Road has the potential to cause new flooding problems and/or increase the severity of existing problems due to building location and density.

West Branch Llagas Creek

The floodway for West Branch Llagas Creek is generally based on equal-conveyance reduction. This method was used upstream of the high mounds that are north of the lake in Las Animas Park. Downstream of this point, the western floodway boundary was set at the west bank of the channel, and there was no eastern floodway boundary. This unique situation is caused by the high mounds acting as barriers to the flow and necessitates that the floodway be discontinuous near the lake. This discontinuity results in a spillage of floodwaters from the floodway on West Branch Llagas Creek into the floodplain for Miller Slough.

West Little Llagas Creek

The floodway for West Little Llagas Creek is based on equal-conveyance reduction. However, four breaks occur in the floodway. These are the result of inadequate carrying capacity of culverts. At Llagas Road, Monterey Road near Fourth Street, and Monterey Road near Watsonville Road, breaks in the floodway with attendant shallow overflows occur, as the entire flow rate cannot be contained in a floodway.

The undersized channel and separate water surface near Spring Avenue limits the flow, which may be contained within a floodway (the channel plus the adjacent floodplain). A break in the floodway must occur at this location.

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 1-percent-annual-chance flood by more than 1 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 1, “Floodway Schematic.”

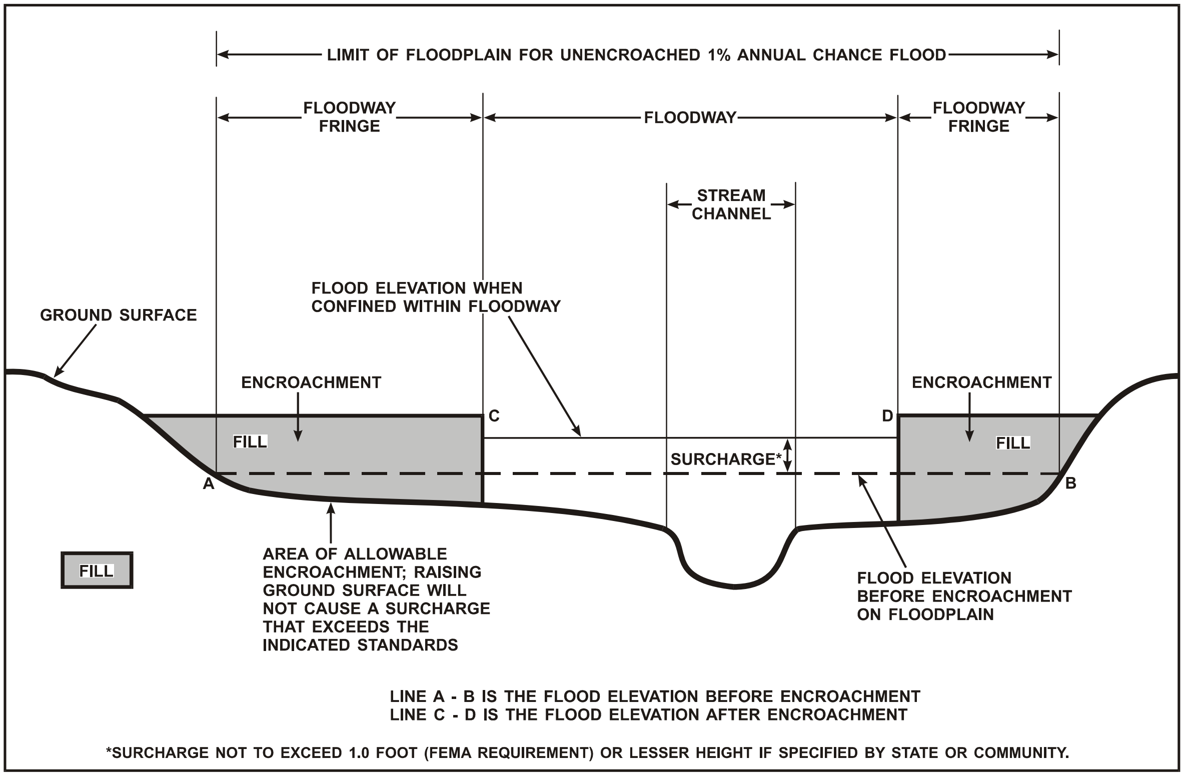

Figure 1 – Floodway Schematic

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Adobe Creek								
A	33,288	65	400	5.8	124.6	124.6	124.6	0.0
B	33,623	45	320	7.3	126.9	126.9	126.9	0.0
C	34,181	45	310	7.5	130.3	130.3	130.3	0.0
D	34,923	30	220	10.5	136.6	136.6	137.0	0.4
E	35,420	40	240	9.7	142.1	142.1	142.1	0.0
F	36,223	80	480	4.8	145.8	145.8	146.7	0.9
G	36,733	90	910	2.4	157.1	157.1	157.9	0.8
H	37,351	60	460	4.8	157.2	157.2	158.1	0.9
I	37,715	55	290	7.6	160.2	160.2	160.3	0.1
J[2]	38,871							
K	39,361	115	500	4.4	180.1	180.1	181.0	0.9
L	39,938	110	560	3.9	183.5	183.5	184.5	1.0
M	40,398	65	340	6.5	185.6	185.6	186.6	1.0
N	41,178	110	870	2.5	195.5	195.5	196.5	1.0
O	41,913	75	430	5.1	196.6	196.6	197.6	1.0
P	42,483	60	270	8.1	199.7	199.7	200.6	0.9
Q	42,928	90	410	5.4	204.0	204.0	204.7	0.7
R	43,338	70	310	7.1	208.1	208.1	208.6	0.5
S	43,843	90	360	6.1	212.4	212.4	213.4	1.0
T	44,238	50	260	8.5	216.7	216.7	217.2	0.5

[1]Feet above Tide Gates
[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

ADOBE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Alamitos Creek								
A	2,445 [1]	80	730	12.1	202.9	202.9	203.7	0.8
B	3,200 [1]	150	1,180	7.5	211.8	211.8	212.2	0.4
C	25,640 [2]	413	985	4.7	352.7	352.7	352.7	0.0
D	26,510 [2]	663	2,722	0.6	365.3	365.3	365.3	0.0
E	27,460 [2]	593	2,473	1.9	375.1	375.1	375.1	0.0
F	28,820 [2]	290	881	5.3	385.2	385.2	385.2	0.0
G	29,300 [2]	174	719	6.5	392.2	392.2	392.2	0.0
H	30,700 [2]	163	483	9.7	404.8	404.8	404.8	0.0
I	31,900 [2]	131	477	9.9	418.4	418.4	418.4	0.0
J	32,920 [2]	127	682	6.9	427.6	427.6	427.6	0.0
Arroyo Calero								
A	100 [3]	90	510	4.6	286.2	286.2	286.9	0.7
B	670 [3]	55	330	7.1	289.8	289.8	289.9	0.1
C	1,280 [3]	55	290	8.0	293.9	293.9	293.9	0.0
D	1,890 [3]	60	280	8.3	300.6	300.6	300.7	0.1
E	2,488 [3]	65	360	6.5	305.7	305.7	305.8	0.1
F	3,087 [3]	60	300	7.8	308.5	308.5	309.0	0.5
G	3,587 [3]	90	530	2.5	314.1	314.1	314.1	0.0
H	4,186 [3]	45	140	9.4	318.0	318.0	318.0	0.0

[1]Feet above confluence with Alamitos Creek Percolation Pond
[2]Feet above confluence with Guadalupe River
[3]Feet above confluence with Alamitos Creek

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

ALAMITOS CREEK - ARROYO CALERO

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Calabazas Creek								
A	55,195	30	153	7.4	323.8	323.8	323.8	0.0
B	55,395	26	158	7.2	325.8	325.8	325.8	0.0
C	55,600	29	180	6.3	327.6	327.6	327.7	0.1
D	55,925	30	170	6.7	330.1	330.1	330.3	0.2
E	56,060	30	186	6.1	331.3	331.3	331.4	0.1
F	56,165	25	144	7.9	331.9	331.9	331.9	0.0
G	56,455	28	166	6.9	335.1	335.1	335.1	0.0
H	56,710	23	160	7.1	337.2	337.2	337.2	0.0
I	56,960	29	190	6.0	339.2	339.2	339.2	0.0
J	57,120	26	176	6.5	340.2	340.2	340.2	0.0
K	57,185	25	99	11.5	340.8	340.8	340.8	0.0
L	57,226	20	151	7.5	342.8	342.8	342.8	0.0

[1]Feet above confluence with Guadalupe Slough

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

CALABAZAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Coyote Creek								
A	76,025	175	3,000	4.3	46.6	46.6	46.6	0.0
B	78,450	175	2,450	5.3	47.5	47.5	47.5	0.0
C	79,935	300	4,060	3.2	49.6	49.6	50.3	0.7
D	80,900	135	890	14.8	50.8	50.8	51.2	0.4
E	81,970	130 [3]	1,320	9.5	57.7	57.7	58.0	0.3
F	82,500	175 [3]	1,620	7.7	59.7	59.7	59.8	0.1
G	83,360	265 [3]	2,440	5.1	61.4	61.4	61.6	0.2
H	83,700	295 [3]	2,830	4.4	62.0	62.0	62.3	0.3
I	84,570	250 [3]	2,180	5.7	62.2	62.2	62.7	0.5
J	85,400	155	1,730	7.2	63.1	63.1	64.1	1.0
K	86,400	205	2,420	5.2	67.2	67.2	68.2	1.0
L	87,000	345	3,890	3.2	70.6	70.6	71.1	0.5
M	88,200	355	4,030	3.1	72.4	72.4	73.4	1.0
N	88,800	280	2,450	5.1	73.2	73.2	74.0	0.8
O	89,360	130	1,510	8.3	74.5	74.5	75.3	0.8
P	90,000	230	2,070	6.0	77.7	77.7	78.6	0.9
Q	91,030	140	1,660	7.5	79.6	79.6	80.2	0.6
R	92,020	110	1,690	7.4	82.1	82.1	83.0	0.9
S	92,480	235	2,560	4.9	83.0	83.0	84.0	1.0
T	93,600	340	3,460	3.6	84.7	84.7	85.2	0.5
U	94,000	335	3,330	3.8	84.7	84.7	85.4	0.7
V	94,620	345	3,360	3.7	84.9	84.9	85.8	0.9
W - Z [2]								

[1]Feet above confluence with San Fransisco Bay

[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

COYOTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Coyote Creek (Continued)								
AA	105,700	130	1,750	7.6	97.3	97.3	98.0	0.7
AB	106,400	210	3,420	3.9	99.2	99.2	99.7	0.5
AC	107,300	200	3,160	4.2	99.9	99.9	100.4	0.5
AD	107,800	210	3,810	3.5	100.1	100.1	100.9	0.8
AE	108,900	240	3,250	4.1	101.1	101.1	102.0	0.9
AF	109,400	160	2,270	5.9	101.5	101.5	102.3	0.8
AG	110,100	215	3,350	4.0	102.7	102.7	103.5	0.8
AH	111,200	230	2,690	5.0	104.1	104.1	104.8	0.7
AI	112,600	575	3,980	3.3	106.2	106.2	106.8	0.6
AJ	113,655	625 [2]	5,220	2.6	107.5	107.5	108.4	0.9
AK	114,700	320	2,450	5.4	109.3	109.3	110.3	1.0
AL	115,600	245	2,640	5.0	111.4	111.4	112.3	0.9
AM	116,535	125 [2]	1,440	9.3	117.4	117.4	117.5	0.1
AN	117,100	130	1,760	7.6	120.0	120.0	120.2	0.2
AO	117,900	210	3,350	4.0	120.9	120.9	121.9	1.0
AP	118,700	170	2,410	5.5	121.5	121.5	122.4	0.9
AQ	119,412	150	2,080	6.4	123.5	123.5	124.0	0.5
AR	120,720	145	2,220	6.0	126.4	126.4	126.9	0.5
AS	121,690	165	2,690	5.0	127.6	127.6	128.0	0.4
AT	122,720	225	2,800	4.8	128.1	128.1	128.6	0.5
AU	123,980	230	1,980	6.7	129.6	129.6	129.9	0.3
AV	124,900	285	2,500	5.3	131.0	131.0	131.3	0.3
AW	125,940	240	2,250	5.9	131.8	131.8	132.0	0.2
AX	126,900	295	2,640	5.0	133.2	133.2	133.8	0.6
AY	127,760	305	2,520	5.3	135.0	135.0	135.6	0.6
AZ	128,200	285	2,380	5.6	135.7	135.7	136.6	0.9

[1]Feet above confluence with San Fransisco Bay

[3]Floodway lies entirely outside county limits

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

COYOTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Coyote Creek (Continued)								
BA	129,290	210	2,040	6.5	143.8	143.8	143.8	0.0
BB	129,540	175	1,870	7.1	144.4	144.4	144.4	0.0
BC	130,150	215	2,020	6.6	145.7	145.7	145.7	0.0
BD	131,190	155	1,550	8.6	148.4	148.4	148.4	0.0
BE	132,200	90 [2]	1,180	11.3	151.0	151.0	151.0	0.0
BF	133,050	185	3,040	4.4	156.6	156.6	156.7	0.1
BG	133,930	240	2,420	5.5	156.5	156.5	156.9	0.4
BH	135,090	365	3,740	3.6	157.2	157.2	157.6	0.4
BI	135,875	330	3,230	4.1	157.6	157.6	158.2	0.6
BJ	136,450	195	2,050	6.5	157.8	157.8	158.3	0.5
BK	137,090	80	910	15.0	158.0	158.0	158.4	0.4
BL	137,550	150	1,930	7.1	162.1	162.1	162.7	0.6
BM	138,830	150	1,470	9.3	163.9	163.9	164.5	0.6
BN	139,500	245	2,810	4.9	167.2	167.2	167.5	0.3
BO	140,700	305	2,990	4.6	169.5	169.5	170.0	0.5
BP	141,100	320	3,190	4.3	170.6	170.6	170.9	0.3
BQ	141,500	400	3,610	3.8	171.2	171.2	171.5	0.3
BR	142,170	255	1,990	6.9	172.8	172.8	173.0	0.2
BS	142,740	215	1,900	7.2	175.7	175.7	175.9	0.2
BT	143,500	210	2,020	6.8	177.9	177.9	178.4	0.5
BU	145,400	130	1,260	10.8	186.4	186.4	186.6	0.2
BV	146,500	155 [2]	1,700	8.7	191.6	191.6	191.9	0.3
BW	147,500	150 [2]	1,940	7.7	194.5	194.5	194.7	0.2
BX	148,500	100 [2]	1,530	9.7	196.6	196.6	196.8	0.2
BY	149,500	185 [2]	2,970	5.0	199.3	199.3	199.6	0.3
BZ	150,500	105	1,460	10.2	200.1	200.1	200.5	0.4

[1]Feet above confluence with San Fransisco Bay
[3]Floodway lies entirely outside county limits

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

COYOTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Coyote Creek (Continued)								
CA	151,500	170	2,580	5.8	202.5	202.5	203.1	0.6
CB	151,865	170	2,450	6.1	202.7	202.7	203.3	0.6
CC	152,500	170	2,500	5.9	207.0	207.0	207.0	0.0
CD	153,400	250	2,950	5.0	207.9	207.9	208.1	0.2
CE	154,500	225	2,230	6.5	208.6	208.6	208.8	0.2
CF	155,300	220 [2]	2,750	5.4	209.6	209.6	209.7	0.1
CG	156,000	255 [2]	2,250	6.6	210.2	210.2	210.2	0.0
CH	157,100	250 [2]	2,910	5.1	210.6	210.6	211.3	0.7
CI	157,600	195 [2]	2,110	7.0	210.9	210.9	211.5	0.6
CJ	158,430	185	2,270	6.6	213.5	213.5	213.7	0.2
CK	159,500	235	2,100	7.1	214.4	214.4	214.5	0.1
CL	160,750	700	3,825	3.9	220.1	220.1	220.9	0.8
CM	161,200	895	8,270	1.8	220.9	220.9	221.5	0.6
CN	162,200	870	10,200	1.4	221.1	221.1	221.7	0.6
CO	163,200	630	3,730	4.0	221.8	221.8	221.8	0.0
CP	164,200	265	2,690	5.5	226.5	226.5	226.5	0.0
CQ	165,200	490	5,700	2.6	227.9	227.9	228.0	0.1
CR	166,200	530	8,220	1.8	228.1	228.1	228.3	0.2
CS	166,700	430	4,500	3.3	228.1	228.1	228.3	0.2
CT	167,200	195	1,190	12.5	229.7	229.7	229.7	0.0
CU	167,610	510 [2]	3,100	4.8	234.3	234.3	234.3	0.0
CV	168,500	825 [2]	7,760	1.4	235.0	235.0	235.0	0.0
CW	169,650	610 [2]	4,930	3.0	235.3	235.3	235.3	0.0
CX	170,900	120 [2]	920	16.1	238.7	238.7	238.7	0.0
CY	171,745	230	3,040	4.9	247.9	247.9	247.9	0.0
CZ	172,170	270	3,860	3.8	248.2	248.2	248.2	0.0

[1]Feet above confluence with San Fransisco Bay

[3]Floodway lies entirely outside county limits

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

COYOTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Coyote Creek (Continued)								
DA	173,244	235	3,080	4.8	248.7	248.7	248.7	0.0
DB	174,253	340	2,800	5.3	248.8	248.8	249.6	0.8
DC	175,210	490	2,650	5.6	250.8	250.8	251.2	0.4
DD	176,192	410	3,140	4.7	252.1	252.1	252.8	0.7
DE	176,800	390 [2]	3,510	4.2	254.5	254.5	255.2	0.7
DF	177,729	225	1,710	8.7	256.0	256.0	256.0	0.0
DG	178,795	160	2,230	6.6	263.7	263.7	263.7	0.0
DH	179,265	255	2,360	6.3	263.9	263.9	263.9	0.0
DI	180,237	240	2,640	5.6	263.9	263.9	264.7	0.8
DJ	181,180	255	2,290	6.5	265.1	265.1	265.7	0.6
DK	182,830	235 [2]	1,930	7.6	269.7	269.7	269.7	0.0
DL	183,352	260	1,510	9.8	269.7	269.7	270.2	0.5
DM	184,530	570	4,310	3.4	272.1	272.1	272.4	0.3
DN	185,420	495	2,720	5.5	272.3	272.3	273.2	0.9
DO	186,524	375	2,200	7.1	275.2	275.2	275.9	0.7
DP	187,170	520	2,630	5.7	281.0	281.0	281.5	0.5
DQ	188,743	355	2,090	7.2	286.6	286.6	286.9	0.3
DR	189,772	390	3,650	4.1	288.2	288.2	289.0	0.8
DS	190,803	420	1,570	9.6	294.3	294.3	294.3	0.0
DT	191,828	785 [2]	5,300	2.8	300.5	300.5	300.9	0.4
DU	192,785	760	6,500	2.3	301.0	301.0	301.5	0.5
DV	194,055	1,040	4,500	3.3	301.7	301.7	302.5	0.8
DW	195,140	555	3,130	4.8	305.8	305.8	306.8	1.0
DX	196,248	2,090	29,000	0.5	311.0	311.0	311.6	0.6
DY	197,122	2,130	24,200	0.6	311.7	311.7	312.0	0.3
DZ	198,022	2,280	20,250	0.7	317.3	317.3	317.9	0.6

[1]Feet above confluence with San Fransisco Bay

[3]Floodway lies entirely outside county limits

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

COYOTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Coyote Creek (Continued)								
EA	199,110	1,970	28,700	0.5	318.0	318.0	318.6	0.6
EB	200,243	1,260	11,400	1.3	318.5	318.5	319.0	0.5
EC	201,200	830	8,300	1.8	324.7	324.7	325.2	0.5
ED	202,421	1,340	7,850	1.9	329.3	329.3	330.2	0.9
EE	203,305	1,180	11,750	1.3	335.6	335.6	336.1	0.5
EF	204,350	390	1,830	8.2	338.7	338.7	339.2	0.5
EG	205,365	405 [2]	2,450	6.1	341.6	341.6	342.3	0.7
EH	206,375	535 [2]	2,210	6.8	344.6	344.6	345.6	1.0
EI	207,600	500	2,710	5.5	348.4	348.4	349.4	1.0
EJ	208,635	350	2,470	6.1	355.1	355.1	355.1	0.0
EK	209,695	415	2,180	6.9	357.4	357.4	357.6	0.2
EL	210,707	330	2,100	7.1	360.9	360.9	361.7	0.8
EM	211,725	315	2,230	6.7	364.9	364.9	365.2	0.3
EN	212,631	225	1,250	12.0	369.6	369.6	369.6	0.0
EO	213,840	260	1,740	8.6	374.0	374.0	374.7	0.7
EP	214,790	190	1,420	10.6	376.4	376.4	376.9	0.5
EQ	215,960	150	1,190	12.6	381.2	381.2	381.2	0.0
ER	216,950	130	980	15.3	385.2	385.2	385.2	0.0
ES	217,520	150	1,350	11.1	388.8	388.8	389.6	0.8
ET	217,832	155	1,480	10.1	390.8	390.8	390.8	0.0
EU	218,185	170	1,830	8.2	393.8	393.8	394.7	0.9
EV	218,582	410	2,830	5.3	394.6	394.6	395.6	1.0
EW	218,900	400	3,360	4.5	395.5	395.5	396.2	0.7
EX	219,625	260	1,800	8.3	397.7	397.7	398.7	1.0
EY	220,370	145	1,110	13.5	402.7	402.7	402.7	0.0

[1]Feet above confluence with San Fransisco Bay

[3]Floodway lies entirely outside county limits

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

COYOTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
East Little Llagas Creek								
A	3,225	348	1,200	4.5	264.6	264.6	264.6	0.0
B	5,621	82	535	6.9	269.6	269.6	270.6	1.0
C	7,679	99	467	7.9	275.4	275.4	275.4	0.0
D	10,149	73	371	10.0	283.9	283.9	283.9	0.0
E	14,352	65	265	8.3	299.1	299.1	300.1	1.0
F	15,397	71	423	5.2	303.7	303.7	303.7	0.0
G	16,471	73	364	6.1	305.3	305.3	305.5	0.2

[1] Feet above confluence with Llagas Creek

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

EAST LITTLE LLAGAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Fisher Creek								
A - I [2]								
J	18,068	170	200	5.6	268.7	268.7	269.0	0.3
K	18,850	110	270	4.2	271.0	271.0	271.9	0.9
L	19,644	85	180	6.3	274.2	274.2	274.2	0.0
M	20,771	90	260	4.2	277.7	277.7	278.7	1.0
N	21,649	50	170	6.4	279.5	279.5	280.1	0.6
O	22,619	110	290	3.9	284.4	284.4	284.7	0.3
P	23,615	125	330	3.4	287.0	287.0	288.0	1.0
Q	24,472	115	420	1.7	292.0	292.0	292.9	0.9
R	24,920	85	280	2.5	292.2	292.2	293.1	0.9
S	25,950	85	200	3.5	295.2	295.2	296.1	0.9
T	26,560	160	400	1.8	299.2	299.2	299.9	0.7
U	27,404	130	290	2.4	300.2	300.2	300.8	0.6
V	28,200	75	230	3.1	302.5	302.5	303.0	0.5
W	28,950	460	450	1.6	305.5	305.5	306.5	1.0
X	29,370	100	300	1.9	306.6	306.6	307.4	0.8
Y	30,201	135	230	2.4	309.6	309.6	310.6	1.0
Z	31,120	75	200	2.8	311.1	311.1	312.1	1.0
AA	31,810	50	150	3.7	312.3	312.3	313.2	0.9
AB	32,810	45	130	4.3	315.7	315.7	316.7	1.0
AC	33,732	180	290	1.9	320.0	320.0	320.7	0.7
AD	34,715	255	180	3.1	324.0	324.0	325.0	1.0
AE	35,728	75	130	4.3	328.2	328.2	329.1	0.9
AF	36,640	90	180	3.1	331.6	331.6	332.5	0.9
AG	37,550	100	130	1.7	334.6	334.6	334.6	0.0

[1] Feet above confluence with Coyote Creek
[2] No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

FISHER CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Fisher Creek Overbank								
A	450	350	1,700	0.5	256.2	256.2	257.2	1.0
B	1,000	1,470	3,320	0.3	256.2	256.2	257.2	1.0
C	1,625	2,050	4,440	0.4	256.2	256.2	257.2	1.0
D	2,470	2,575	4,190	0.3	256.3	256.3	257.2	0.9
E	3,320	1,980	1,600	0.9	256.3	256.3	257.3	1.0
F	3,980	2,050	1,460	1.0	257.4	257.4	257.6	0.2
G	4,740	1,800	1,830	0.8	257.7	257.7	257.9	0.2
H	5,740	920	2,720	0.5	260.0	260.0	260.2	0.2
I	6,251	960	880	2.0	260.1	260.1	260.2	0.1
J	7,580	630	410	3.2	263.8	263.8	263.9	0.1
K	8,520	925	880	1.5	266.3	266.3	267.0	0.7
L	8,960	600	680	1.8	267.8	267.8	268.1	0.3

[1]Feet above confluence with Fisher Creek

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

FISHER CREEK OVERBANK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Llagas Creek								
A	16,130	70	800	8.1	177.2	177.2	178.1	0.9
B	21,200	175	1,400	5.1	187.5	187.5	188.3	0.8
C	22,100	210	1,360	5.2	188.5	188.5	189.2	0.7
D	22,860	235	1,250	5.7	189.4	189.4	190.1	0.7
E	23,610	195	1,060	6.7	190.5	190.5	191.0	0.5
F	24,654	190	2,230	3.2	198.8	198.8	199.7	0.9
G	25,595	190	1,820	3.9	199.1	199.1	200.0	0.9
H	26,690	185	1,720	3.9	199.5	199.5	200.3	0.8
I	27,457	370	1,860	3.6	199.8	199.8	200.7	0.9
J	28,320	140	1,000	6.8	200.4	200.4	201.2	0.8
K	29,087	190	1,050	6.4	201.6	201.6	202.2	0.6
L	29,845	85	670	10.1	207.4	207.4	208.1	0.7
M	30,349	85	1,020	6.6	210.4	210.4	211.4	1.0
N	31,115	405	2,230	4.4	211.1	211.1	212.1	1.0
O	31,770	155	1,280	8.0	211.1	211.1	212.0	0.9
P	32,390	145	1,210	8.5	211.9	211.9	212.7	0.8
Q	33,123	145	1,290	8.0	213.4	213.4	214.2	0.8
R	33,715	145	1,300	7.9	214.2	214.2	215.0	0.8
S	34,585	175	1,440	7.2	215.5	215.5	216.3	0.8
T	35,545	155	1,470	7.0	216.7	216.7	217.5	0.8
U	36,383	170	1,590	6.5	217.7	217.7	218.6	0.9
V	37,135	135	1,210	8.5	218.2	218.2	219.0	0.8
W	38,070	145	1,220	8.4	219.6	219.6	220.5	0.9
X	39,090	180	1,220	8.4	225.4	225.4	226.0	0.6
Y	39,660	400	1,966	5.2	227.3	227.3	227.9	0.6
Z	40,585	925	2,480	4.1	228.3	228.3	229.0	0.7
AA	41,440	860	2,220	4.6	228.9	228.9	229.9	1.0
AB	42,265	845	2,170	4.8	230.4	230.4	231.1	0.7

[1]Feet above confluence with Pajaro River

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

LLAGAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Llagas Creek (Continued)								
AC	43,090	1,205	3,170	3.3	231.3	231.3	232.2	0.9
AD	43,790	1,085	2,820	3.7	233.2	233.2	234.2	1.0
AE	44,530	820	1,590	6.6	235.1	235.1	235.5	0.4
AF	45,330	510	2,400	4.4	237.8	237.8	238.8	1.0
AG	45,740	525	2,240	4.7	238.8	238.8	239.8	1.0
AH	46,435	270	1,030	10.2	240.3	240.3	240.3	0.0
AI	47,240	685	2,690	3.9	241.5	241.5	242.5	1.0
AJ	47,890	770	2,440	4.3	242.5	242.5	243.5	1.0
AK	48,435	590	1,840	5.7	244.1	244.1	244.8	0.7
AL	49,280	1,160	3,870	2.7	246.1	246.1	246.8	0.7
AM	49,980	985	1,780	5.8	246.3	246.3	247.1	0.8
AN	50,725	1,050	2,640	3.9	247.5	247.5	248.4	0.9
AO	51,460	635	1,720	3.0	250.9	250.9	251.0	0.1
AP	52,370	110	770	7.0	251.2	251.2	251.4	0.2
AQ	53,055	110	700	7.7	252.9	252.9	253.0	0.1
AR-AS[2]								
AT	55,820	80	560	9.5	261.0	261.0	261.5	0.5
AU	56,565	135	680	7.8	263.1	263.1	263.5	0.4
AV	57,175	160	810	6.5	265.7	265.7	265.7	0.0
AW	58,010	155	620	8.5	266.8	266.8	266.9	0.1
AX	58,965	145	840	6.3	269.2	269.2	269.2	0.0
AY	59,965	130	610	8.7	270.6	270.6	270.6	0.0
AZ	60,925	145	690	7.7	273.3	273.3	273.3	0.0
BA	61,670	130	710	7.5	275.3	275.3	275.4	0.1
BB	62,525	110	460	11.5	277.4	277.4	277.4	0.0
BC	63,300	125	630	8.4	280.6	280.6	280.6	0.0
BD	64,280	90	510	10.4	285.0	285.0	285.0	0.0

[1]Feet above confluence with Pajaro River

[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

LLAGAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Llagas Creek (Continued)								
BE	65,270	110	620	8.5	288.8	288.8	288.8	0.0
BF	66,000	95	490	10.8	289.8	289.8	290.0	0.2
BG	66,975	140	800	6.6	293.8	293.8	294.2	0.4
BH	67,760	105	610	8.7	296.3	296.3	296.3	0.0
BI	68,220	55	360	14.7	296.7	296.7	296.7	0.0
BJ	68,935	95	650	8.2	300.8	300.8	300.8	0.0
BK	69,510	90	530	10.0	301.3	301.3	301.3	0.0
BL	70,125	75	520	10.2	303.9	303.9	303.9	0.0
BM	70,580	122	690	7.7	306.0	306.0	306.0	0.0
BN	71,440	143	710	6.9	307.9	307.9	308.0	0.1
BO	72,149	140	890	5.5	310.0	310.0	310.0	0.0
BP	72,655	170	790	6.2	310.2	310.2	310.5	0.3
BQ	73,363	140	740	5.3	311.2	311.2	312.0	0.8
BR	74,200	170	740	6.6	312.6	312.6	313.6	1.0
BS	74,970	150	810	6.0	319.3	319.3	320.3	1.0
BT	75,865	150	870	5.6	321.9	321.9	322.9	1.0
BU	76,480	105	690	7.1	325.0	325.0	325.3	0.3
BV	77,090	100	670	7.3	327.7	327.7	328.4	0.7
BW	78,310	94	620	7.9	331.9	331.9	332.5	0.6
BX	78,640	85	710	6.9	334.0	334.0	334.3	0.3
BY	79,590	150	1,110	4.4	341.3	341.3	341.9	0.6
BZ	80,320	130	910	5.4	343.8	343.8	344.5	0.7
CA	81,115	100	580	8.3	349.7	349.7	349.7	0.0
CB	81,980	135	870	5.5	355.9	355.9	355.9	0.0
CC	82,670	125	790	5.8	358.9	358.9	359.2	0.3
CD	83,230	120	1,010	4.6	360.4	360.4	361.4	1.0
CE	83,535	125	840	5.5	361.3	361.3	362.1	0.8

[1]Feet above confluence with Pajaro River

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

LLAGAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Llagas Creek (Continued)								
CF	84,140	140	1,020	4.5	364.2	364.2	364.4	0.2
CG	85,690	160	1,230	3.7	372.1	372.1	372.3	0.2
CH	86,430	125	1,290	3.5	377.7	377.7	378.4	0.7
CI	86,970	110	1,110	4.1	377.9	377.9	378.6	0.7
CJ	87,915	130	980	4.0	380.4	380.4	380.8	0.4
CK	88,860	80	570	6.8	388.0	388.0	388.1	0.1
CL	89,815	100	550	7.1	390.9	390.9	391.9	1.0
CM	90,400	120	580	6.7	394.8	394.8	395.0	0.2
CN	91,530	140	760	5.1	402.0	402.0	402.0	0.0
CO	91,935	130	930	4.2	403.3	403.3	403.9	0.6
CP	92,735	95	580	6.7	405.1	405.1	405.9	0.8
CQ	93,345	110	680	5.7	409.2	409.2	409.3	0.1
CR	93,920	85	550	7.1	411.9	411.9	412.1	0.2
CS	94,495	90	560	7.0	414.9	414.9	415.7	0.8
CT	94,970	110	730	5.3	417.1	417.1	418.1	1.0
CU	95,590	115	690	5.7	419.8	419.8	420.5	0.7
CV	96,230	80	520	7.5	422.6	422.6	423.5	0.9
CW	96,850	145	830	4.7	426.5	426.5	426.8	0.3
CX	97,440	65	440	8.9	428.8	428.8	429.1	0.3
CY	98,230	125	400	9.8	434.8	434.8	434.8	0.0
CZ	98,695	130	880	4.4	437.7	437.7	438.2	0.5
DA	99,300	120	640	6.1	439.5	439.5	440.0	0.5
DB	99,720	110	570	6.8	443.8	443.8	443.8	0.0
DC	100,220	85	440	8.9	447.3	447.3	447.8	0.5
DD	101,120	90	600	6.5	451.9	451.9	452.8	0.9

[1]Feet above confluence with Pajaro River

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

LLAGAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Llagas Overbank								
A [2]								
B	7,430	850	5,750	1.2	185.1	185.1	186.1	1.0
C	8,235	995	3,370	2.0	185.3	185.3	186.2	0.9
D	8,980	600	1,600	4.2	186.0	186.0	186.9	0.9
E	9,775	895	1,780	2.7	188.2	188.2	189.1	0.9
F	10,380	785	1,310	3.7	189.3	189.3	190.2	0.9
G	11,030	895	3,080	1.5	191.7	191.7	192.4	0.7
H	11,880	600	980	4.7	192.3	192.3	192.9	0.6
I	12,400	575	1,390	3.3	193.7	193.7	194.4	0.7
J	13,855	620	1,270	3.7	196.0	196.0	196.9	0.9
K	15,250	680	1,600	2.9	198.2	198.2	198.9	0.7
L	17,555	475	1,475	3.2	199.5	199.5	200.4	0.9
M	18,085	690	1,890	2.5	200.4	200.4	201.2	0.8
N	18,810	600	3,380	1.4	202.5	202.5	203.3	0.8
O - W [2]								

[1]Feet above confluence with Llagas Creek
[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

LLAGAS OVERBANK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Los Gatos Creek								
A	35,859	108	700	9.9	250.7	250.7	250.8	0.1
B	36,856	90	895	7.8	260.9	260.9	261.0	0.1
C	37,744	120	1,115	6.2	263.3	263.3	264.0	0.7
D	38,618	306	2,608	2.7	265.8	265.8	266.4	0.6
E	39,215	263	1,733	4.0	266.3	266.3	266.8	0.5
F	39,918	152	1,077	6.5	270.4	270.4	270.5	0.1
G	40,766	115	927	7.5	274.8	274.8	274.8	0.0
H	41,409	117	906	7.7	278.0	278.0	278.0	0.0
I	43,900	N/A	7,530	0.9	N/A	N/A	302.3	N/A
J	45,300	435	1,100	6.4	303.1	303.1	303.1	0.0
K	45,700	90	352	7.7	304.6	304.6	304.6	0.0
L	46,300	545	910	7.7	305.9	305.9	306.7	0.8
M	46,700	95	520	13.4	310.1	310.1	310.2	0.1
N	47,300	120	640	10.9	316.0	316.0	316.1	0.1
O	47,700	80	640	10.9	318.6	318.6	319.4	0.8
P	48,300	110	800	8.7	323.3	323.3	323.9	0.6
Q	48,900	75	710	9.8	329.3	329.3	329.7	0.4
R	49,500	95	770	9.1	332.1	332.1	332.1	0.0
S	49,900	120	1,210	5.8	333.0	333.0	333.6	0.6
T	50,500	90	780	9.0	333.3	333.3	334.2	0.9

[1]Feet above confluence with Guadalupe River

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

LOS GATOS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Pajaro River								
A	111,700	400	9,120	3.3	142.4	142.4	143.4	1.0
B	113,663	329	7,938	3.8	143.6	143.6	144.6	1.0
C	114,438	484	9,428	3.3	144.4	144.4	145.3	0.9
D	118,150	665	10,773	2.9	145.6	145.6	146.5	0.9
E	119,325	484	7,951	3.9	146.3	146.3	147.1	0.8
F	120,088	450	9,751	3.2	147.2	147.2	148.1	0.9

[1]Feet above Pacific Ocean

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

PAJARO RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Permanente Creek								
A - L [2]								
M	16,730	50	220	11.7	294.7	294.7	294.8	0.1
N	17,290	50	220	11.7	305.0	305.0	305.0	0.0
O	18,240	35	150	11.5	326.1	326.1	326.1	0.0
P	18,762	25	140	12.3	336.8	336.8	336.8	0.0
Q	19,460	20	170	10.1	358.3	358.3	358.7	0.4
R	20,300	55	270	6.4	376.1	376.1	376.2	0.1
S	20,910	50	300	5.7	382.4	382.4	382.7	0.3
T	21,375	40	160	10.8	394.8	394.8	394.8	0.0
U	21,770	15	180	8.2	411.4	411.4	411.4	0.0
V	22,830	55	260	5.7	426.7	426.7	426.7	0.0
W	23,240	45	210	7.0	434.6	434.6	435.3	0.7
X	23,850	40	200	7.4	445.9	445.9	446.8	0.9
Y	24,120	40	180	8.2	452.4	452.4	452.4	0.0
Z	24,580	55	400	3.7	475.4	475.4	475.4	0.0
AA	25,210	30	130	11.3	490.5	490.5	490.5	0.0
AB	26,400	25	120	12.3	540.5	540.5	540.5	0.0

[1]Feet above confluence of Permanente Diversion Channel with Stevens Creek
[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

PERMANENTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Prospect Creek								
A	388 [1]	28	130	4.9	325.2	325.2	325.2	0.0
B	658 [1]	25	117	5.4	326.1	326.1	326.5	0.4
C	1,078 [1]	35	125	5.1	328.7	328.7	328.9	0.2
D	1,513 [1]	24	66	9.6	336.3	336.3	336.3	0.0
E	1,913 [1]	21	82	7.8	344.8	344.8	344.8	0.0
F	2,087 [1]	21	75	8.4	346.9	346.9	346.9	0.0
G	2,192 [1]	33	103	6.2	348.9	348.9	348.9	0.0
H	2,290 [1]	20	121	5.3	353.5	353.5	353.5	0.0
I	2,395 [1]	21	100	6.4	353.5	353.5	353.9	0.4
Santa Teresa Creek								
A	310 [2]	30	190	4.5	316.7	316.7	317.0	0.3
B	910 [2]	55	140	6.1	321.9	321.9	322.2	0.3

[1]Feet above confluence with Calabazas Creek

[2]Feet above confluence with Arroyo Calero

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

PROSPECT CREEK - SANTA TERESA CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
San Tomas Aquino Creek								
A	1,420	25	150	8.4	317.4	317.4	317.4	0.0
B	1,635	30	140	9.0	321.0	321.0	321.2	0.2
C	2,030	40	130	9.7	329.2	329.2	329.2	0.0
D	2,280	45	210	6.0	332.9	332.9	333.4	0.5
E	2,535	55	260	4.8	341.4	341.4	341.4	0.0
F	2,675	45	240	5.3	341.5	341.5	342.4	0.9
G	2,900	30	120	10.3	343.1	343.1	343.1	0.0
H	3,175	30	190	6.5	345.8	345.8	346.7	0.9
I	3,530	55	280	4.4	351.9	351.9	352.7	0.8
J	3,945	30	190	6.5	357.3	357.3	357.5	0.2
K	4,710	30	160	7.7	363.0	363.0	363.6	0.6
L	5,085	30	110	11.2	369.4	369.4	369.4	0.0
M	5,435	30	140	8.8	374.0	374.0	374.4	0.4
N	5,545	35	130	9.5	375.3	375.3	375.6	0.3
O	5,725	25	130	9.1	378.2	378.2	378.2	0.0
P	6,450	50	380	3.1	391.8	391.8	392.3	0.5
Q	6,650	40	240	4.9	391.8	391.8	392.6	0.8
R	7,290	90	180	6.6	400.1	400.1	401.1	1.0
S	7,845	50	160	7.4	413.3	413.3	413.6	0.3
T	8,275	40	180	6.6	416.3	416.3	417.1	0.8
U	9,105	20	100	11.8	435.1	435.1	435.1	0.0
V	9,975	40	120	9.5	449.9	449.9	449.9	0.0

[1]Feet above Pollard Road

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

SAN TOMAS AQUINO CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Saratoga Creek								
A	6,604	74	458	8.6	315.5	315.5	315.5	0.0
B	6,922	52	349	11.2	320.5	320.5	320.5	0.0
C	7,446	65	430	9.1	324.5	324.5	324.9	0.4
D	7,851	55	320	12.2	327.0	327.0	327.0	0.0
E	8,295	50	350	11.0	331.7	331.7	331.7	0.0
F	8,519	60	420	9.1	334.7	334.7	335.0	0.3
G	8,865	50	330	11.6	337.9	337.9	337.9	0.0
H	9,290	45	330	11.6	341.6	341.6	341.6	0.0
I	9,829	60	330	11.5	346.1	346.1	346.8	0.7
J	10,190	50	330	11.5	352.4	352.4	352.4	0.0
K	10,525	60	480	7.9	357.5	357.5	357.6	0.1
L	10,940	55	410	9.3	359.9	359.9	360.0	0.1
M	11,340	65	460	8.3	362.6	362.6	363.3	0.7
N	12,122	55	330	11.5	371.8	371.8	372.6	0.8
O	12,917	50	330	11.5	381.7	381.7	382.5	0.8
P	13,192	55	520	7.3	388.1	388.1	388.1	0.0
Q	13,547	50	330	11.4	388.8	388.8	389.7	0.9
R	14,300	60	410	9.1	398.5	398.5	398.7	0.2
S	15,055	55	420	8.9	407.8	407.8	407.8	0.0
T	15,745	50	440	8.5	416.1	416.1	416.2	0.1
U	16,660	85	520	7.1	427.4	427.4	427.4	0.0
V	17,505	35	310	11.9	438.7	438.7	439.1	0.4
W	17,905	65	460	7.8	446.4	446.4	446.4	0.0
X	18,675	85	510	7.1	455.7	455.7	456.4	0.7
Y	19,335	55	360	10.0	467.1	467.1	467.8	0.7
Z	19,815	55	390	9.3	475.7	475.7	476.7	1.0

[1]Feet above Pollard Road

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

SARATOGA CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Saratoga Creek (Continued)								
AA	20,400 [1]	75	520	6.8	483.8	483.8	484.5	0.7
AB	21,170 [1]	65	440	8.1	494.8	494.8	494.8	0.0
AC	21,470 [1]	75	640	5.5	505.2	505.2	505.2	0.0
AD	22,030 [1]	45	260	13.4	508.6	508.6	508.6	0.0
AE	22,135 [1]	30	270	12.9	514.5	514.5	514.5	0.0
Smith Creek								
A	159 [2]	10	50	7.9	255.1	255.1	255.1	0.0
B	524 [2]	10	40	9.9	256.0	256.0	256.0	0.0
C	1,500 [2]	30	170	2.1	271.4	271.4	271.9	0.5
D	1,850 [2]	25	90	3.9	271.6	271.6	272.1	0.5
E	2,300 [2]	25	50	6.5	276.1	276.1	276.2	0.1
F	2,725 [2]	35	90	3.6	279.7	279.7	279.7	0.0
G	3,000 [2]	25	70	4.3	282.1	282.1	282.1	0.0
H	3,500 [2]	20	40	7.6	286.1	286.1	286.1	0.0
I	3,700 [2]	25	40	7.0	288.8	288.8	288.8	0.0
J	4,095 [2]	35	80	3.5	297.1	297.1	297.1	0.0
K	4,250 [2]	20	70	3.4	298.4	298.4	298.6	0.2
L	4,465 [2]	25	90	2.7	301.2	301.2	301.6	0.4
M	4,685 [2]	25	90	2.7	303.8	303.8	304.2	0.4

[1]Feet above Pollard Road
[2]Feet above Union Pacific Railroad

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

SARATOGA CREEK - SMITH CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Stevens Creek								
A	38,510	90	780	7.2	252.2	252.2	252.2	0.0
B	39,410	90	500	11.2	255.4	255.4	255.4	0.0
C	39,885	65	570	9.8	259.7	259.7	259.9	0.2
D	40,800	60	710	7.7	264.4	264.4	264.5	0.1
E	41,573	80	730	7.5	269.5	269.5	269.7	0.2
F	42,400	60	460	12.0	272.9	272.9	273.4	0.5
G-I [2]								
J	45,882	205	1,630	3.3	299.6	299.6	300.0	0.4
K	46,910	115	500	10.9	302.2	302.2	302.2	0.0
L	47,710	85	450	12.1	307.6	307.6	307.6	0.0
M	48,710	110	580	9.4	317.4	317.4	318.4	1.0
N	49,610	110	550	9.9	327.8	327.8	327.9	0.1
O	50,310	325	1,230	4.4	331.4	331.4	332.1	0.7
P	51,110	260	790	6.9	336.5	336.5	337.3	0.8
Q	51,711	240	1,900	2.9	344.6	344.6	344.6	0.0
R	52,510	105	470	11.6	346.6	346.6	346.6	0.0
S	53,310	90	580	9.4	351.8	351.8	352.8	1.0
T	54,110	75	450	12.1	360.9	360.9	360.9	0.0
U	54,910	105	470	11.6	367.4	367.4	367.4	0.0
V	55,710	65	390	14.0	372.4	372.4	372.4	0.0
W	56,510	105	660	8.3	385.6	385.6	385.6	0.0
X	57,310	75	530	10.3	394.2	394.2	394.2	0.0
Y	58,110	80	500	10.9	399.6	399.6	399.7	0.1
Z	58,710	75	620	8.8	404.9	404.9	405.9	1.0
AA	59,110	110	1,050	5.1	413.6	413.6	414.1	0.5
AB	59,710	75	700	7.5	416.2	416.2	417.0	0.8
AC	59,910	110	1,140	4.6	418.3	418.3	419.0	0.7
AD	60,710	70	500	10.6	421.7	421.7	422.6	0.9

[1]Feet above Inboard Levees

[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

STEVENS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Tennant Creek								
A	431	190	406	5.0	293.9	293.5 [2]	293.5	0.0
B	2,001	256	570	3.5	300.6	300.6	301.5	0.9
C	3,944	290	765	2.6	307.6	307.6	308.5	0.9
D	6,106	170	413	4.9	315.7	315.7	316.2	0.5
E	9,385	182	361	1.8	326.2	326.2	326.8	0.6
F	11,458	47	157	2.7	335.1	335.1	336.1	1.0
G	13,507	120	107	3.9	343.2	343.2	343.4	0.2
H	16,857	71	312	1.3	361.7	361.7	362.2	0.5

[1]Feet above confluence with East Little Llagas Creek

[2]Elevation computed without consideration of flooding controlled by East Little Llagas Creek

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

TENNANT CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Uvas Creek								
A	17,405	181	1,644	8.5	209.4	209.4	209.4	0.0
B	18,150	278	1,969	7.1	211.2	211.2	211.2	0.0
C	19,090	194	1,855	7.5	212.6	212.6	213.1	0.5
D	19,700	195	2,020	6.9	213.7	213.7	213.9	0.2
E	20,185	165	2,380	6.1	214.5	214.5	214.7	0.2
F	20,925	221	2,788	5.0	215.3	215.3	215.4	0.1
G	21,555	244	3,440	4.1	215.4	215.4	215.7	0.3
H	22,415	380	4,663	3.0	215.5	215.5	216.0	0.5
I	22,885	306	2,898	4.8	215.7	215.7	216.1	0.4
J	23,315	140	3,194	4.4	215.9	215.9	216.5	0.6
K	23,705	65	2,657	5.3	216.1	216.1	216.7	0.6
L	24,310	150	5,842	2.4	216.7	216.7	217.3	0.6
M	24,985	460	11,722	1.2	216.8	216.8	217.4	0.6
N	25,785	190	6,557	2.1	216.8	216.8	217.4	0.6
O	26,610	300	4,913	2.8	216.9	216.9	217.5	0.6
P	27,240	445	6,204	2.3	217.1	217.1	217.6	0.5
Q	28,035	505	4,580	3.0	217.4	217.4	218.2	0.8
R	28,925	520	4,190	3.2	217.4	217.4	218.3	0.9
S	29,950	295	1,430	9.5	219.5	219.5	220.3	0.8
T	30,540	205	1,390	9.7	222.4	222.4	222.4	0.0
U	31,200	225	1,500	9.0	224.4	224.4	224.5	0.1
V	31,730	270	2,200	6.2	225.6	225.6	226.1	0.5
W	32,175	220	1,070	12.7	225.6	225.6	225.6	0.0
X	32,970	190	1,140	11.9	231.2	231.2	231.2	0.0
Y	33,610	205	1,690	8.0	234.9	234.9	235.0	0.1
Z	34,120	175	1,350	10.0	235.5	235.5	235.9	0.4

[1] Feet above Union Pacific Railroad

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

UVAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Uvas Creek (Continued)								
AA	34,660	165	1,240	10.9	236.7	236.7	237.5	0.8
AB	35,355	225	1,720	7.9	239.2	239.2	240.1	0.9
AC	35,770	160	980	13.8	240.3	240.3	240.4	0.1
AD	36,460	165	1,270	10.7	244.2	244.2	245.2	1.0
AE	36,900	127	1,376	9.8	245.9	245.9	246.5	0.6
AF	37,629	375	3,656	3.7	249.0	249.0	249.7	0.7
AG	39,650	385	2,447	4.5	254.6	254.6	255.1	0.5
AH	40,075	472	2,231	4.9	256.0	256.0	256.8	0.8
AI	40,950	269	1,960	5.6	261.2	261.2	262.2	1.0
AJ	41,485	235	1,873	5.8	265.0	265.0	265.8	0.8
AK	42,417	589	4,464	2.4	266.8	266.8	267.8	1.0
AL	43,339	400	1,314	8.3	268.3	268.3	268.7	0.4
AM	43,884	650	3,436	3.2	272.1	272.1	273.0	0.9
AN	45,389	766	3,131	3.5	277.7	277.7	278.7	1.0
AO	46,652	995	3,408	3.2	282.8	282.8	283.8	1.0
AP	47,640	498	2,379	4.6	287.9	287.9	288.4	0.5
AQ	48,157	308	2,407	4.5	290.6	290.6	291.1	0.5
AR	49,013	110	1,357	7.6	295.2	295.2	295.5	0.3
AS	49,521	75	1,016	8.4	301.2	301.2	301.4	0.2
AT	50,363	300	2,487	3.4	303.7	303.7	304.6	0.9
AU	51,346	384	2,197	3.9	305.2	305.2	306.2	1.0
AV	52,860	175	1,242	6.8	313.8	313.8	314.6	0.8
AW	53,932	488	1,717	4.9	321.1	321.1	322.1	1.0
AX	54,987	590	1,634	5.2	326.1	326.1	326.9	0.8
AY	55,523	375	1,582	5.4	330.3	330.3	330.3	0.0
AZ	56,529	305	1,535	5.5	335.8	335.8	336.5	0.7

[1]Feet above Union Pacific Railroad

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

UVAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Uvas Creek (Continued)								
BA	57,260	262	1,417	6.0	338.2	338.2	339.2	1.0
BB	57,989	231	1,472	5.8	341.9	341.9	342.8	0.9
BC	58,883	119	1,035	8.2	346.8	346.8	347.8	1.0
BD	59,991	114	1,024	8.3	351.4	351.4	352.0	0.6
BE	60,910	170	1,124	6.9	356.6	356.6	357.1	0.5
BF	61,708	205	1,362	5.7	363.4	363.4	363.6	0.2
BG	63,746	420	3,069	2.5	374.2	374.2	375.2	1.0
BH	64,754	273	2,373	3.3	376.9	376.9	377.9	1.0
BI	65,831	300	1,506	5.2	384.4	384.4	384.7	0.3
BJ	66,885	223	1,658	4.7	391.9	391.9	392.1	0.2
BK	67,865	171	1,622	4.8	398.3	398.3	398.6	0.3

[1]Feet above Union Pacific Railroad

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

UVAS CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
West Branch Llagas Creek								
A - D[2]								
E	11,034	609	809	2.4	235.2	235.2	236.2	1.0
F	12,014	310	655	3.0	238.2	238.2	239.2	1.0
G	12,885	317	499	3.9	241.5	241.5	242.4	0.9
H	13,498	425	718	2.7	243.9	243.9	244.9	1.0
I	14,123	525	684	2.9	246.9	246.9	247.8	0.9
J	15,801	300	691	2.8	250.3	250.3	251.2	0.9
K	16,882	450	520	2.8	254.3	254.3	255.2	0.9
L	17,752	500	745	1.9	257.1	257.1	257.8	0.7
M	19,293	89	211	6.8	259.0	259.0	259.9	0.9
West Branch Llagas Creek East Split								
A[2]								
B	8,452	250	437	3.8	227.8	227.8	228.4	0.6
C	9,440	276	286	5.9	231.0	231.0	231.8	0.8
D	10,045	750	1,097	1.5	233.4	233.4	234.4	1.0
E	11,034	636	839	2.3	235.2	235.2	236.2	1.0

[1]Feet above confluence with Miller Slough

[2]No floodway determined

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

WEST BRANCH LLAGAS CREEK - WEST BRANCH LLAGAS CREEK EAST SPLIT

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Wildcat Creek								
A	3,815	35	240	3.9	330.6	330.6	330.6	0.0
B	4,308	35	100	9.3	334.0	334.0	334.0	0.0
C	4,903	35	130	7.0	341.6	341.6	341.6	0.0
D	5,304	30	100	9.1	347.3	347.3	347.3	0.0
E	6,014	45	170	5.4	358.8	358.8	358.8	0.0
F	6,468	55	120	7.6	361.4	361.4	361.4	0.0
G	6,770	65	310	2.9	372.7	372.7	372.8	0.1
H	7,354	45	120	7.6	377.7	377.7	377.7	0.0
I	8,033	40	100	9.1	388.1	388.1	388.1	0.0
J	8,920	50	210	4.2	400.4	400.4	400.6	0.2
K	9,235	45	100	8.8	402.7	402.7	402.7	0.0
L	9,746	55	110	7.6	409.8	409.8	409.8	0.0
M	10,065	55	200	4.2	417.6	417.6	418.0	0.4
N	10,670	30	70	8.1	421.9	421.9	421.9	0.0
O	11,160	50	280	2.0	434.8	434.8	434.8	0.0
P	11,992	45	80	7.1	448.4	448.4	448.4	0.0
Q	12,107	70	410	1.4	456.0	456.0	456.0	0.0
R	12,992	15	50	11.4	475.2	475.2	475.2	0.0
S	13,487	40	200	2.8	493.7	493.7	493.7	0.0
T	14,142	35	70	8.1	502.6	502.6	502.6	0.0

[1]Feet above Quito Road

TABLE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

SANTA CLARA COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

WILDCAT CREEK