APPENDIX C

Hydraulic and Hydrologic Report

HYDROLOGY AND HYDRAULIC STUDIES

This section summarizes the hydrologic and hydraulic analysis techniques used to determine design flow rates and water surface elevations. It provides information on the approach, methodology, and calibration of the models used to analyze and develop the flood management alternatives.

C-1.0 Watershed Hydrology

The purpose of hydrologic modeling on this project was to define design flow rates in San Luis Obispo Creek and its major tributaries for storms of various recurrence interval, ranging from the 2-year to the 100-year storm. This information will form the basis for the design and evaluation of flood management alternatives within the basin.

C-1.1 Hydrologic Modeling Approach

Questa's modeling approach has been to create a theoretical watershed runoff model using the U.S. Army Corps of Engineers' Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) computer modeling package. HMS is similar in computational ability to the old HEC-1 computer model but has a graphical user interface and allows for more detailed rainfall infiltration modeling and for greater GIS compatibility.

The model is composed of three components; watershed sub-basins, stream flow routing reaches, and modeled precipitation events. The watershed sub-basin component mimics the physical characteristics of the watershed including the relationship between precipitation and runoff. The flow routing component describes how flow moves from the upper reaches of the watershed to the mouth and determines the relative timing of this runoff. The precipitation component describes precisely how much rainfall occurs on each watershed sub-basin at each model time step.

The San Luis Obispo Creek Watershed above the mouth is approximately 217 square kilometers (84 square miles) in area. The topographic variability is quite impressive. Elevations vary from sea level to over 800 meters (2600 feet) along the crest of the Cuesta Ridge, in the Santa Lucia Mountains. No point in the watershed is more than 22 km (14 mi) from the coast. Storms coming off the Pacific Ocean are pushed over the mountains, tending to create widely varying rainfall patterns within the watershed. Precipitation in the lower Southeastern portions of the watershed can be less than half of that in the higher Northern portions. Flow in San Luis Obispo Creek can respond very quickly to short high intensity rainfall bursts. Floods in San Luis Obispo Creek tend to be of high magnitude and relatively short duration.

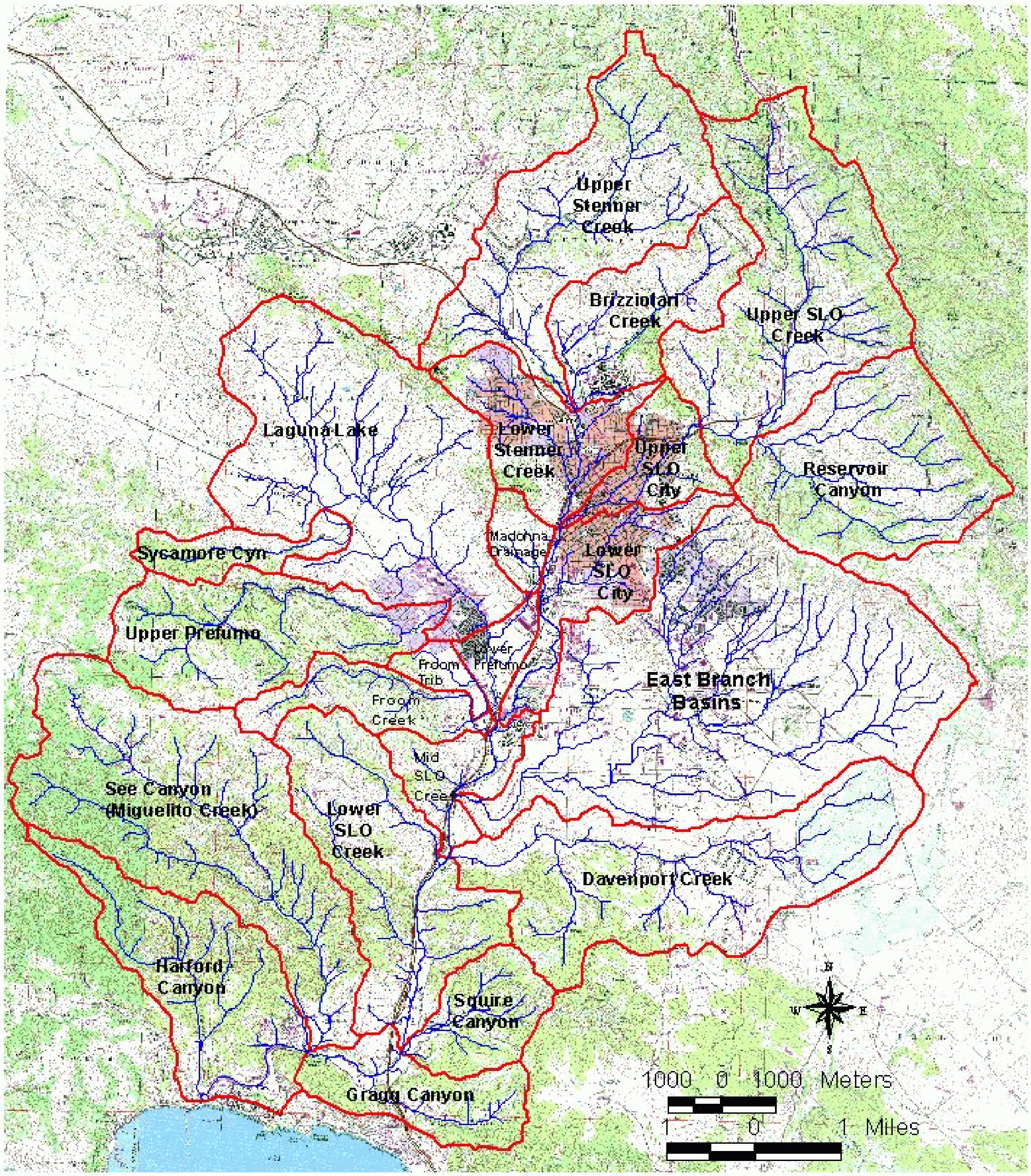

Figure C-1. Watershed Sub-Basin Boundaries

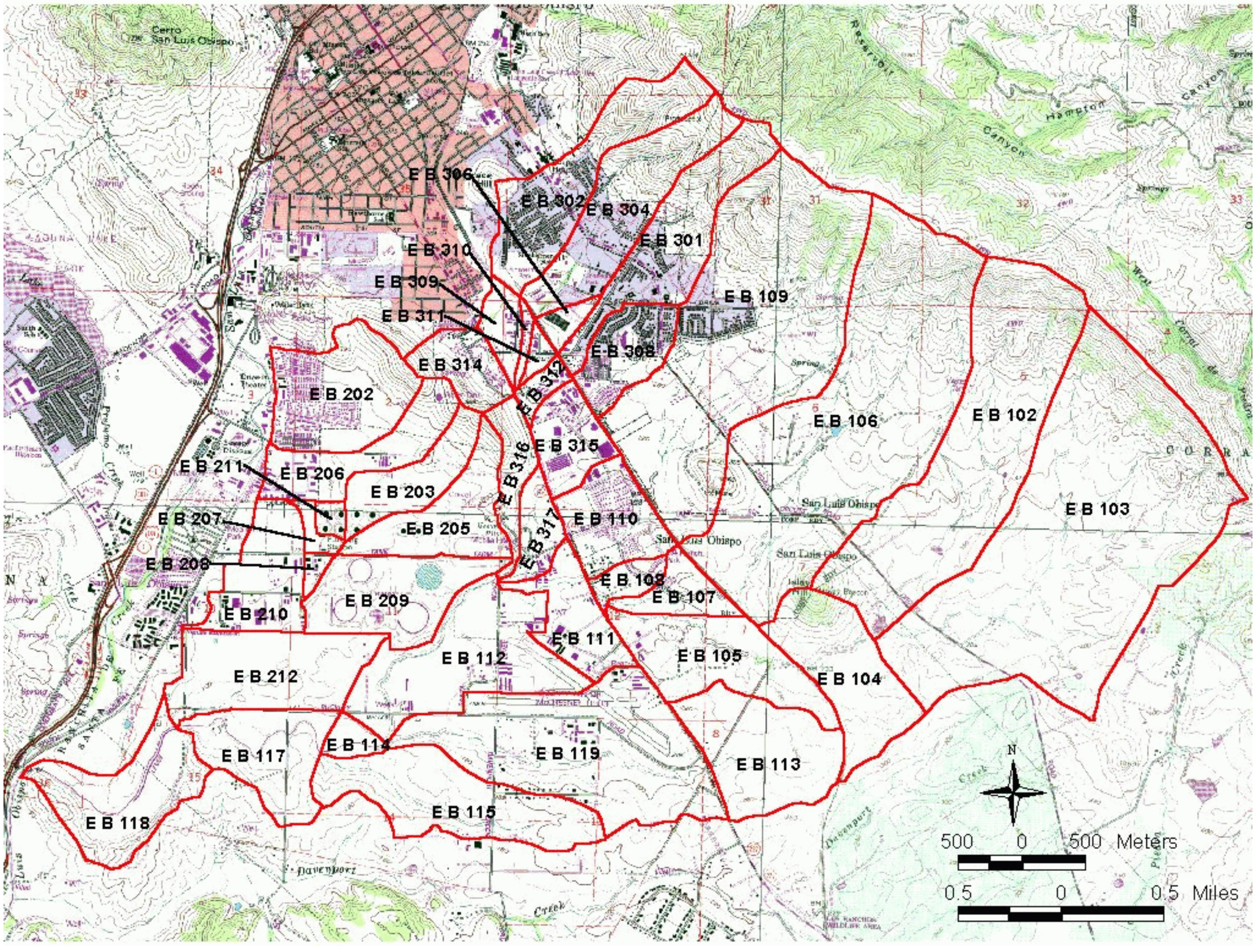

Figure C-2. Sub-basin delineation along the East Branch of San Luis Obispo Creek follows the *City of San Luis Obispo Storm Drain Master Plan* (Boyle Engineering Corporation, 1999).

C-1.2 Watershed Model

The watershed model was formed by splitting the watershed into 61 individual sub-basins (**Figure C-1**). To maintain consistency with the recently published *San Luis Obispo Storm Drainage Master Plan* (Boyle Engineering Corporation, 1999), basin boundaries within the watershed of the East Fork of San Luis Obispo Creek were taken from that report. The SCS loss-rate and the SCS unit hydrograph methods were used to determine runoff hydrographs from each of the sub-basins, based on a set of 24-hour design storms.

Loss-rate

In the SCS loss-rate method, infiltration properities of a basin are described by a runoff curve number. Curve numbers (CN) range from 1 to 100, with lower values denoting less runoff for a given precipitation total than higher values. The SCS curve number was typically calculated as a function of land use and soil hydrologic characteristics, according to Natural Resources Conservation Service (NRCS) recommendations outlined in Technical Report 55 (TR55) (Soil Conservation Service, 1975).

For this study, the goal was to develop runoff curve numbers representing four separate watershed conditions: pre-European settlement, historic circa 1960 conditions, existing conditions, and future conditions assuming general plan build-out. An individual runoff curve number map was created for each of the four watershed conditions. While it is possible to model changes in land use by changing an "impervious surface" variable in the SCS method, rather than by changing the curve number itself, this technique was not used as part of this study. Changes in curve number were the only way that change in infiltration characteristics over time were modeled.

The soil map shown in **Figure C-3** is based on published NRCS data, and is applicable for all four watershed conditions. Land use was determined using a combination of USGS quadrangle maps, recent aerial photography, city and county general plan land use maps, and several GIS vegetation coverages for the watershed. A future conditions land-use map (**Figure C-4**) was created by merging the city and county general plan land use maps and correlating the land use categories in those maps with land use categories defined by the NRCS (Soil Conservation Service, 1975). In the few locations where city and county data overlapped, the city land use category superceded the county category unless the city category was "open space," where the county map was assumed to be more representative. In areas zoned "open space," "agriculture," and "rural land," vegetation maps were overlain on top of the zoning map to better characterize those areas. Since only existing conditions vegetation maps were available, this technique assumes that vegetation characteristics in the rural parts of the watershed have been and will remain fairly constant over time.

Existing conditions land use was determined by comparing the general plan land use categories with recent aerial photography. Where the general plan land use did not appear to represent existing conditions as interpreted from a current aerial photograph, the land use category was changed to be more appropriate. This was most common directly south of San Luis Obispo.

Some areas zoned suburban or rural residential appeared on the aerial photographs to have not yet achieved total buildout. These areas were given the mean curve number between the most extensive existing vegetation type in the area and the curve number representing future general plan conditions.

There were two special cases where the existing condition land use was significantly less developed than the general plan buildout, and where simple averaging of undeveloped and post-buildout curve numbers would not be representative. These areas were See Canyon's area of "rural residential" zoning and the area of "suburban" zoning in Squire canyon. For See Canyon, we assumed that good condition brush characterized 75% of the basin and rural residential 25%. We weighted the curve numbers for these two categories accordingly. For Squire Canyon, we assumed that the existing condition was similar to the much less dense rural residential category, with 2-acre lots, and used that SCS category. Where areas were partly zoned suburban and partly grassland, we assumed that a rural residential 2-ac lot zoning was representative of existing conditions.

A similar method was used to define circa 1963 land use. This time, instead of adjusting general plan build-out curve numbers based on recent aerial photography, the general plan conditions were modified using a 1963 USGS quadrangle map. Where conditions on the historic USGS quadrangle differed from the general plan, a best estimate of the 1960's land use was made.

For pre-European settlement conditions, a curve number of 67.2 (calibrated), representing the average for undeveloped sub-basins in the existing conditions model, was applied to all sub-basins that in the 1960 model contained significant development. Essentially, this represents removing the city of San Luis Obispo and replacing it with land use that currently exists outside of the city limits. Otherwise, the pre-European settlement model is identical to the circa 1960 model.

Sub-basin curve numbers ranged from 61 to 79 (calibrated) and were typically lower in the upper portions of the watersheds (**Table C-1**).

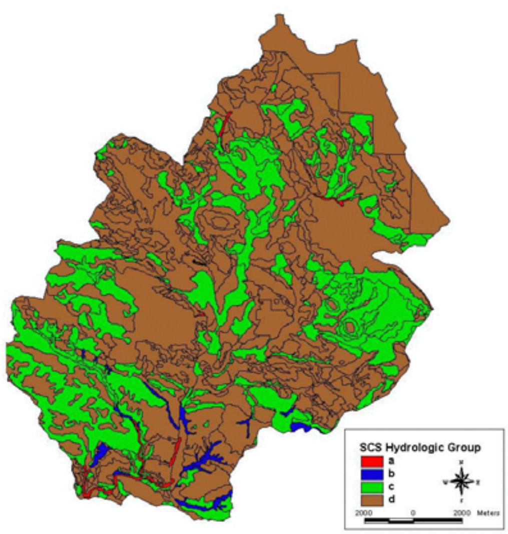

Figure C-3. Soil Hydrologic Groups.

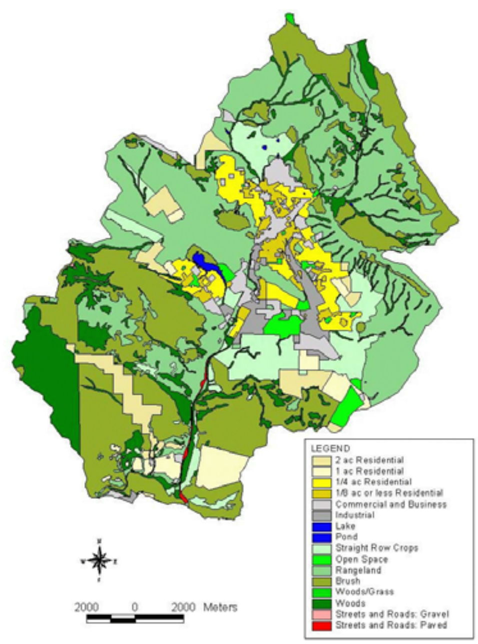

Figure C-4. NRCS Land Use Categories.

Table C-1. Loss Rate Parameters

Basin	Basin Area (km2)	Uncalibrated Existing Conditions SCS Curve Number	Calibrated Existing Conditions SCS Curve Number	Calibrated Future Conditions SCS Curve Number	Calibrated Historic 1965 Conditions SCS Curve Number	Calibrated Pre-European Settlement SCS Curve Number
Brizziolari Creek	7.28	83.7	71.2	71.2	70.7	67.2
Davenport Creek	17.97	77.1	65.6	65.6	65.9	65.9
Gragg Canyon	5.11	73.0	62.1	62.0	61.4	61.4
Froom Creek	7.57	76.1	64.7	64.8	67.8	67.2
Froom Tributary	1.12	82.2	69.9	74.3	67.8	67.2
Harford Canyon	11.09	70.3	59.7	60.1	59.5	59.5
Laguna Lake	21.69	83.4	70.9	70.9	70.7	67.2
Lower Prefumo	1.64	88.4	75.1	75.7	67.8	67.2
Lower SLO City	6.01	88.9	75.6	75.6	74.5	67.2
Lower SLO Creek	13.54	76.9	65.3	65.1	64.7	64.7
Lower Stenner	6.44	85.6	72.7	73.1	72.1	67.2
Madonna Drainage	1.56	82.7	70.3	70.3	67.8	67.2
Mid SLO Creek	3.57	81.5	69.3	69.3	67.8	67.2
Miguelito Creek	21.01	71.6	60.8	62.0	60.8	60.8
Reservoir Canyo	12.51	78.3	66.5	66.5	66.5	66.5
Squire Canyon	4.18	76.4	65.0	66.1	61.4	61.4
Sycamore Canyon	2.72	76.1	64.7	64.6	64.7	64.7
Upper Prefumo	10.46	75.8	64.4	64.4	64.1	64.1
Upper SLO City	3.45	87.5	74.4	74.5	74.0	67.2
Upper SLO Creek	17.15	78.6	66.8	66.8	66.8	66.8
Upper Stenner	15.01	80.5	68.5	68.5	68.3	67.2
E B 102	2.29	78.7	66.9	66.9	66.9	66.9
E B 103	4.193	79.3	67.4	67.4	67.4	67.2
E B 104	0.912	80.9	68.7	68.7	68.7	67.2
E B 105	0.837	85.8	72.9	72.9	72.0	67.2
E B 106	3.577	82.1	69.8	69.7	70.4	67.2
E B 107	0.282	85.4	72.6	72.6	71.4	67.2
E B 108	0.127	84.0	71.4	71.4	71.9	67.2
E B 109	2.968	82.7	70.3	70.3	71.0	67.2
E B 110	0.518	89.5	76.1	76.2	73.8	67.2
E B 111	0.458	92.8	78.9	78.9	74.8	67.2
E B 112	1.083	89.4	76.0	75.6	75.6	67.2
E B 113	0.909	89.0	75.6	75.6	75.5	67.2
E B 114	0.041	85.8	72.9	72.9	72.9	67.2
E B 115	1.031	86.3	73.4	73.5	75.5	67.2
E B 117	0.671	86.0	73.1	73.1	73.1	67.2
E B 118	1.054	82.0	69.7	69.7	69.8	67.2
E B 119	0.86	89.0	75.7	76.2	76.2	67.2
E B 202	0.86	86.8	73.8	74.6	73.0	67.2
E B 203	0.448	84.3	71.7	75.5	72.6	67.2
E B 205	0.606	83.7	71.1	73.7	75.4	67.2
E B 206	0.534	86.9	73.9	75.9	72.1	67.2
E B 207	0.085	84.1	71.5	80.3	76.8	67.2
E B 208	0.06	92.9	79.0	79.0	76.7	67.2
E B 209	0.751	84.2	71.6	68.1	78.1	67.2
E B 210	0.58	89.1	75.7	78.1	73.4	67.2
E B 211	0.06	84.0	71.4	80.7	78.2	67.2
E B 212	0.899	88.2	75.0	75.0	75.1	67.2
E B 301	0.904	84.9	72.2	72.6	72.4	67.2
E B 302	0.979	85.7	72.9	72.8	73.1	67.2
E B 304	0.907	85.8	72.9	72.9	72.2	67.2
E B 306	0.08	91.9	78.1	78.6	75.6	67.2
E B 308	0.433	85.9	73.0	73.8	73.6	67.2
E B 309	0.176	90.6	77.0	78.2	74.4	67.2
E B 310	0.049	87.9	74.7	78.7	70.1	67.2
E B 311	0.054	88.4	75.2	79.1	75.0	67.2
E B 312	0.106	88.9	75.5	79.0	72.7	67.2
E B 314	0.368	87.9	74.7	74.7	74.6	67.2
E B 315	0.365	92.9	78.9	78.9	75.4	67.2
E B 316	0.233	83.8	71.3	71.6	70.8	67.2
E B 317	0.238	90.5	76.9	76.7	75.5	67.2

The *initial abstraction* represents the amount of water temporarily stored in puddles, on plant stems, in the soil, etc., before runoff begins. It is related to the runoff curve number but can vary from this relationship depending on how recently the watershed experienced a significant rainfall event. For this study, the initial abstraction was initially assumed to follow an empirical relationship with the runoff curve number as described by **Equation C-1**.

Eq. C-1

$$I_a = 0.2\left(\frac{1000}{CN} - 10\right)$$

Values of initial abstraction ranged from 0.48 to 0.69 in, but were adjusted down 50 percent after model calibration. Because the purpose of the modeling is to predict the runoff from relatively large design storm events, and because the most intense rainfall in the design storm occurs 12 hours after the storm begins, the initial abstraction is usually "filled" long before the most intense design rainfall occurs. This makes initial abstraction a less important variable for our purposes than the curve number. It would be more important if the purpose of the modeling was to predict peak flow rates from less intense, shorter duration storms.

Hydrograph Transformation

The SCS unit hydrograph method was used to transform excess rainfall into runoff at the outlet of any given basin.

Lag time is the difference in time between the center of mass of excess rainfall and the time at which flow from that sub-basin peaks. It is the only required input parameter for the SCS unit hydrograph transformation. Lag time is often calculated as a function subbasin geometry according to the following form:

Eq. C-2

$$T_{lag} = C_t * \left(\frac{(L * L_{ca})}{\sqrt{S}}\right)^m$$

where:

C_t = empirical coefficient
= 24*N where N is a basin roughness coefficient (Nolte and Associates, 1977)
L = the maximum flow length in a basin, in mi.
S = the average slope along the maximum flow length pathway
L_{ca} = the distance from the basin outlet to the centroid
m = lag exponent.

For the sub-basins in the East Branch of San Luis Obispo Creek, lag parameters were taken from the *City of San Luis Obispo Storm Drain Master Plan* (Boyle Engineering Corporation, 1999). For the remaining basins, two sets of coefficients were used. For the two urbanized basins in the watershed–Upper SLO City and Lower SLO City–coefficients derived by the US Army Corps of engineers for 100% urbanized watersheds in the Tulsa Oklahoma area were used (Boss International, 1999). These are $C_t = 0.59$ and $m = 0.30$. For all other basins outside the East Branch watershed, coefficients derived by Riverside County, California for foothill areas were used. Here, $C_t = 0.72$ (i.e. $N = 0.03$) and $m = 0.38$. Time lag for each of the sub-basins is listed in **Table C-2**.

As part of the model verification process, the unit hydrograph used by George S. Nolte and Associates (1977) was substituted for the SCS Unit Hydrograph Method. The difference in peak flow rates and timing was negligible–on the order of 2-3 percent.

Base Flow

Base flow from each sub-basin was determined by looking at the daily-average flow rates at the stream gauge that operated on San Luis Obispo Creek near Avila until 1986. A conservative estimate was made by assuming that base flow in the creek during a large storm would be similar to the base flow in the creek that was observed over the week following the storm of March 2, 1983. The average base flow for this time period, omitting days when rainfall occurred, was approximately 14 cms (500 cfs). Divided over the upstream area of 207 km^2 (80 mi^2) this gives an average base flow rate of 0.067 cms/km^2 (6.3 cfs/mi^2), which was then applied to each sub-basin.

This base flow rate is significantly higher than the long term average winter-season flow rate in San Luis Obispo creek, and is intended to represent the base flow in the creek during a series of wet storms. It is much greater than any likely wintertime releases from the City of San Luis Obispo Water Reclamation Facility, which discharges into San Luis Obispo Creek downstream from the Prado Road Bridge.

C-1.3 Flow Routing

Runoff from individual sub-basins is routed through the system using the Muskingum-Cunge 8-point routing technique. This technique uses a rough approximation of a channel cross section, including the floodplain, along with representative roughness values, to evaluate the effects of channel and floodplain storage on the flood hydrograph as it passes downstream through the reach.

Highway 101 crosses San Luis Obispo Creek at two locations near the upstream city limits, once just below Cuesta Park, and once just above Cuesta Park. These culverts have been observed to cause ponding upstream of the respective highway embankments during large storms, which could cause a significant amount of attenuation of flood peak flow rates. The backwater behind each of these culverts was modeled using reservoir routing techniques available in HEC-HMS. A computer hydraulic model was created for each of these culverts using HEC-RAS (as described in Section B-2 of this appendix). A flow versus upstream water surface elevation curve for each culvert was obtained from the model. Elevation versus water surface area curves were obtained from the LIDAR survey flown as part of this project for the lower culvert and from the 1994 City of San Luis Obispo 10-m DEM for the upper culvert. Note that the 10-m DEM is more accurate than the 100-ft DEM currently supported by the City. It was flow by Golden State Aerials in 1994, has a 10 meter horizontal spacing between points, and has a vertical accuracy on the order of 0.6 to 0.76 m (2 to 2.5 ft) (Baragona, pers. comm, 2001). The LiDAR accuracy is on the order of 0.15 m (0.5 ft).

Another important routing area was Laguna Lake in the Prefumo Creek watershed. Laguna Lake was modeled as a reservoir using the Modified Puls method. The stage elevation curve for the lake was obtained from a combination of an existing 10-m Digital Elevation Model (DEM) of the San Luis Obispo Creek watershed and an aerial laser topographinc (LIDAR) survey performed as part of the WMP. (See the WMP for more details). The stage-discharge curve for the reservoir, which empties into lower Prefumo Creek through two 2.13 m x 3.05 m (7 ft x 10 ft) concrete box culverts and one 2.13 m x 4.27 m (7 ft x 14 ft) concrete box culvert under Madonna Road, was obtained by setting up a HEC-RAS backwater model of the culvert and stream system in that area.
Some of Laguna Lake's flood storage volume would likely already be used at the start of a peak 24-hour rainfall event. A conservative starting water surface elevation for the 10-year, 25-year, 50-year, and 100-year storms was obtained by developing a separate simplified rainfall-runoff model of the watershed above the lake and then running an 8-day storm corresponding to the desired recurrence interval through the watershed and lake on an hourly time increment. The simplified model used a constant infiltration rate of 0.13 in/hr, which was reported by George S. Nolte and Associates (1977) to be appropriate for long-term detention analysis. The highest lake elevation from the given design 8-day storm was used as the starting water surface elevation for the 24-hour design storm. The rainfall depth for the 8-day storms was obtained by a statistical analysis of the each year's highest 8-day precipitation total as recorded at the San Luis Obispo Cal Poly rain gage, for the 1948 to 2001 water years. The highest total, 21.8 in, occurred from January 19 to January 26, 1969. The statistical results, as fit to a Gamma probability distribution function, are shown in **Figure C-5**. The precipitation pattern for the 8-day storm was based on the January 19 to 26, 1969 storm as recorded at the Huasna, California gage (the only hourly gage record currently available for that storm). This gage is located approximately 30 km (20 mi) southeast of San Luis Obispo.

C-1.4 Precipitation

The 24-hour design storm precipitation was based on *NOAA Atlas II, Precipitation-Frequency Atlas of the Western United States*. Because of the significant topographic variation within the watershed, two separate 24-hour design storms for each recurrence interval were synthesized, one for the lower portions of the watershed (those basins with a mean elevation below 200 meters) and one for the upper portions (mean elevation 200 meters or greater), based on typical depth-duration-frequency numbers taken from the *NOAA Atlas II* (National Oceanic and Atmospheric Administration, 1973). **Table C-3** lists the depth-duration-frequency values used for developing the design storms. **Figure C-6** shows the basins with mean elevations above 200 meters.

Table C-3. Design Depth-Duration-Frequency Values

For Basins Below 200-m in mean elevation:

	Rainfall (mm) at various durations and frequencies				
Duration	**100-year**	**50-year**	**25-year**	**10-year**	**2-year**
5 min	11.7	10.5	9.3	7.9	5.0
10 min	18.2	16.3	14.4	12.2	7.7
15 min	23.1	20.7	18.2	15.5	9.8
1 hr	40.5	36.3	32.0	27.2	17.1
2 hr	55.2	49.6	45.0	37.7	24.0
3 hr	69.1	62.1	57.4	47.7	30.5
6 hr	101.6	91.4	86.4	71.1	45.7
12 hr	135.9	128.3	115.6	94.0	64.8
24 hr	170.2	165.1	144.8	116.8	83.8

For Basins Above 200-m in mean elevation:

	Rainfall (mm) at various durations and frequencies				
Duration	**100-year**	**50-year**	**25-year**	**10-year**	**2-year**
5 min	12.2	11.2	10.0	8.6	5.6
10 min	19.0	17.4	15.5	13.4	8.7
15 min	24.1	22.0	19.7	16.9	11.0
1 hr	42.2	38.6	34.5	29.7	19.3
2 hr	59.5	54.9	48.2	41.5	27.5
3 hr	76.0	70.5	61.2	52.7	35.2
6 hr	114.3	106.7	91.4	78.7	53.3
12 hr	158.8	146.1	125.7	106.7	73.7
24 hr	203.2	185.4	160.0	134.6	94.0

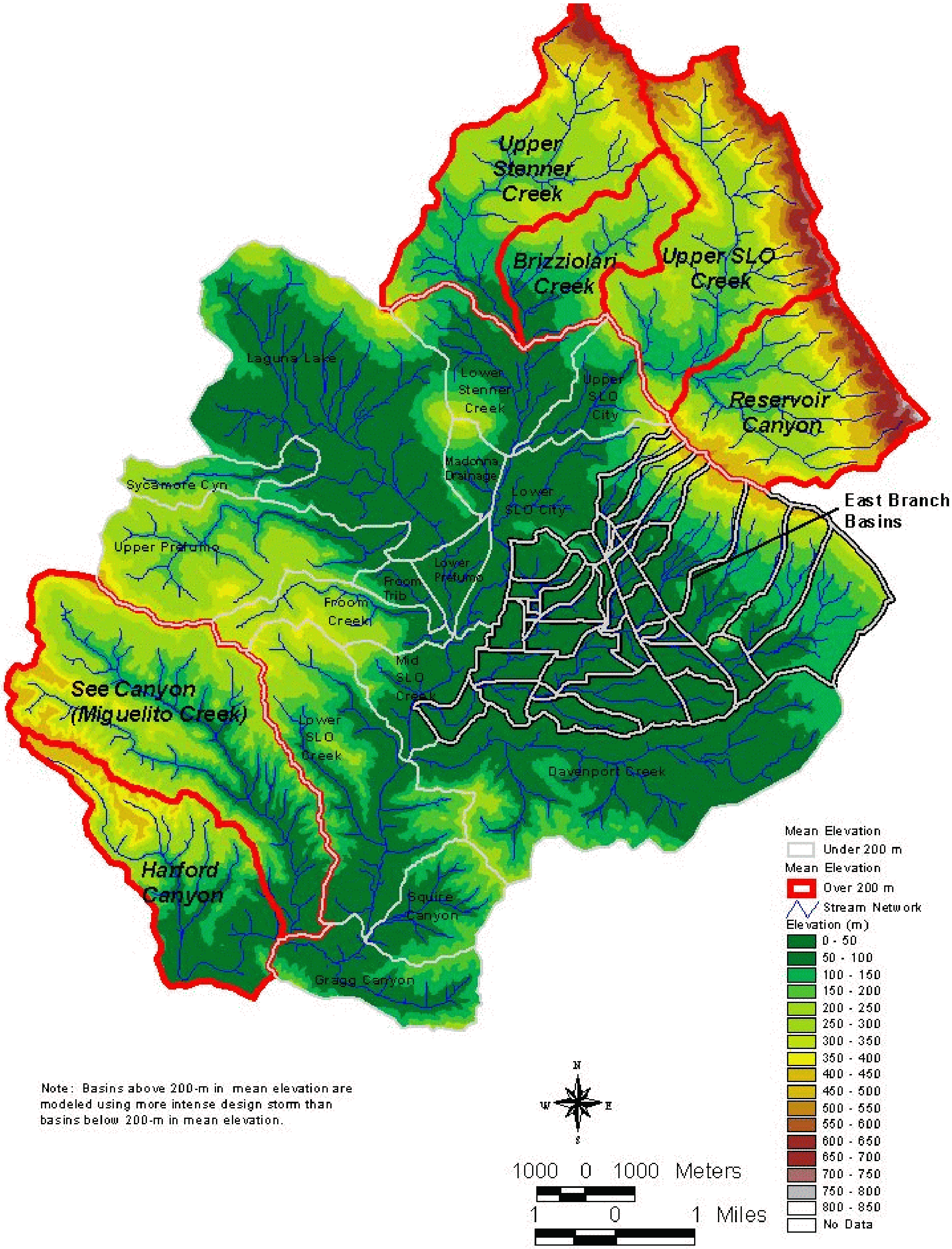

Figure C-6. Higher Intensity Design Storm Locations

Hypothetical design storms generated using this method give precipitation distributions that are appropriate for individual points but not for large areas. For areas much larger than a few square kilometers, the fact that the storm must travel from one portion of the basin to the next prevents the most intense rainfall from occurring all at once. In other words, while it may be raining heavily at point A, at the same time, it is only lightly raining at point B, and the totals at point B may never reach those of point A during that particular storm event. The further apart A is from B, the more pronounced this effect. Because both A and B contribute flow to the lower portions of the creek, flow rates there are lower than if the storm at A was occurring simultaneously at B.

To account for this phenomenon, a correction factor must be applied to the design storms derived using NOAA data. This factor reduces the storm precipitation based on the area of upstream contributory watershed. While there is a fairly simple way to handle this in HEC-1, the current version of HMS does not include this ability. Consequently, we derived four different design storms, each of which would give a conservative approximation of this effect for a selected set of points along the stream system. The depth-area curve used to make the reduction was taken from *NOAA Atlas II* (**Figure C-7**).

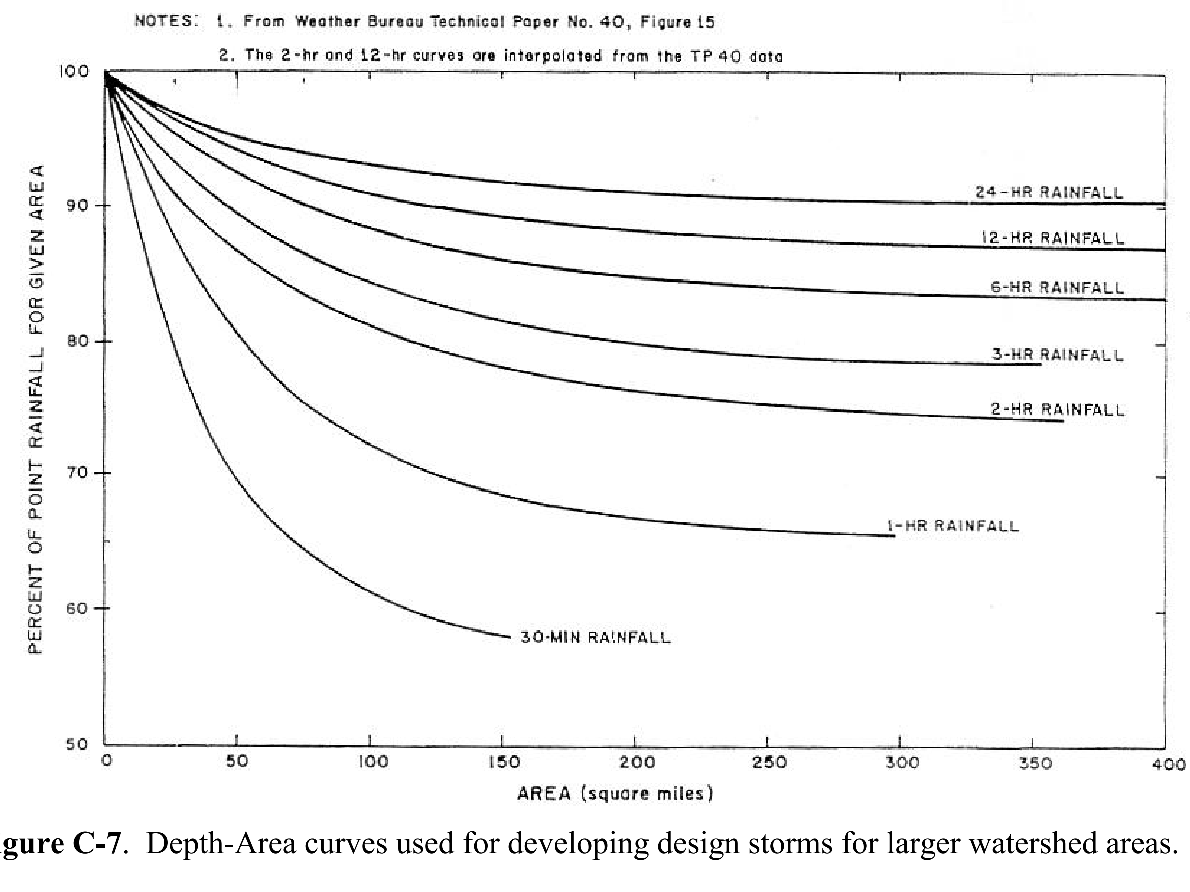

Figure C-7. Depth-Area curves used for developing design storms for larger watershed areas.

The first storm size was set equal to the area of each individual sub-basin. This storm (Storm A) is appropriate along all tributaries of San Luis Obispo Creek before their confluence with San Luis Obispo Creek, as well as for Upper San Luis Obispo Creek before the confluence with Stenner Creek. The second storm (Storm B) was given a size based on the combined area of the Stenner and San Luis Obispo Creek basins above the Stenner/San Luis Obispo Creek confluence (61.8 km^2, 23.9 mi^2). It is appropriate for computing flow in San Luis Obispo Creek from the confluence with Stenner Creek downstream to the confluence with the East Branch of San Luis Obispo Creek. The third storm (Storm C) was given a size based on the combined area of the Main Stem of San Luis Obispo Creek and the East Branch of San Luis Obispo Creek at the Main Stem/East Branch confluence (133.9 km^2, 51.7 mi^2). This storm is appropriate for computing flow in San Luis Obispo Creek from the East Branch confluence downstream to the mouth of the Gragg Canyon tributary. The fourth storm (Storm D) was given a size equal to the entire San Luis Obispo Creek watershed above the confluence with Miguelito Creek (See Canyon, 174.7 km^2, 67.5 mi^2). It is appropriate to use this storm from the mouth of the Gragg Canyon Tributary downstream to the mouth of San Luis Obispo Creek. The depth-area reduction factors used for each storm size are listed in **Table C-4**.

Table C-4. Depth-Area Reduction Factors

Duration	Storm A (12 km^2)	Storm B (62 km^2)	Storm C (133 km^2)	Storm D (175 km^2)
5 min	0.94	0.79	0.69	0.66
10 min	0.94	0.79	0.69	0.66
15 min	0.94	0.79	0.69	0.66
1 hr	0.97	0.88	0.8	0.77
2 hr	0.98	0.91	0.85	0.82
3 hr	0.99	0.94	0.9	0.88
6 hr	0.99	0.96	0.93	0.91
12 hr	1.00	0.96	0.94	0.92
24 hr	1.00	0.97	0.95	0.94

C-2 Hydraulic Model

Project flood management alternatives were analyzed using the U.S. Army Corps of Engineers Hydraulic Engineering Center–River Analysis System (HEC-RAS) version 3.0. HEC-RAS is a one-dimensional hydraulic computer modeling system that is used to predict flood water surface elevations at approximately evenly spaced cross-sections, oriented perpendicular to the predominate flow direction and distributed throughout the modeled reach. The predicted water surface elevations are then compared to the elevation of the top of channel banks and of the floodplain (and buildings) to determine flood break-out points and outline the extent and depth of flood water for various flood flow recurence intervals (i.e. 10-year, 100-year flows).

C-2.1 Data Requirements

The input requirements for the model include stream flow rates, the geometry of various hydraulic structures such as bridges and culverts, topographic information along a set of relatively evenly spaced cross-sections oriented perpendicular to the predominant flow direction, channel roughness estimates (such as flow resistance) along each cross-section, and a water surface elevation at the downstream boundary of the model.

Section C-1 of this appendix describes the rainfall-runoff modeling methods used to define stream flow rates used in this study. Field surveys and as-built drawings were used to define the hydraulic structures such as bridges and culverts.

The topographic information for this project was obtained using LIDAR technology. LIDAR is a system where a laser beam mounted on an aircraft is shot at the ground from the air. The signal produced when the laser beam hits the ground can be used to measure the distance from the aircraft and the ground. This, combined with a global positioning system (GPS) receiver on the aircraft and some post-processing that corrects for signal returns coming from objects not directly on the ground surface can be used to produce a map of ground spot elevations. The raw LIDAR points, which for our survey were spaced approximately 2-meters apart, are then used to create a gridded surface map (at 5-meter spacing for this project) of the channel and floodplain topography.

For this project, the grid produced from the LIDAR was not dense enough to fully characterize the channel bed. Even the 2-meter spacing between raw points was not sufficient in certain locations to fully define the channel. Consequently, a second LIDAR flight, this time with a raw point spacing of less than 1 meter, was performed in the spring of 2000 to densify the channel. Raw points from this and from the original LIDAR survey were used to develop the surface used for hydraulic modeling between channel banks. Outside of the channel banks, the original 5-meter grid was used

Because the post-processing that corrects for vegetation and buildings can remove a significant number of points at some locations where the stream channel bed is obscured by dense vegetation, it was necessary for us to directly inspect the raw point coverage to determine where the LIDAR survey had resulted in a dense point coverage in the channel bed. We drew our cross-sections at locations where the raw data points existed all the way across the stream bed. Also, since bridges can obscure the channel bed from the LIDAR instrumentation, we augmented the LIDAR survey with physical surveys taken in the field at all bridges in the study reach (with the exception of those bridges along the East Branch of San Luis Obispo Creek, where information was taken directly from the HEC-RAS model developed for the area by Boyle Engineering Corporation as part of the Airport Area Specific Plan).

Channel roughness was estimated in the field by comparing published roughness values for various photographed channels with the condition of the local channel (from bank-top to bank-top). Roughness in the floodplain areas outside the stream banks was estimated by creating a map of representative regions using digital orthophotography of the site and coding these representative regions with appropriate values based on published Corps of Engineers guidelines (U.S. Army Corps of Engineers, 2001). Where buildings provide significant obstruction to flood flow, especially through the downtown district of San Luis Obispo and on the east side of Higuera street south of downtown, very high roughness values on the order of 1.0 to 2.0 were used to represent the composite effect of bed roughness across streets and lawns and the obstructing effects of the buildings. Streets in built-up areas that run parallel to the creek channel were coded with low roughness values in order to represent the increased flood flow conveyance these zones provide. **Table C-5** shows the typical roughness values used in the model.

Table C-5. Typical Manning's Roughness Values.

Land Use	Roughness Value
Overbank Areas	
Typical Built up Areas	0.07-0.15
Fields	0.035
Orchard	0.06
Riparian Scrub/Forest	0.09-0.1
Suburban Areas	0.06
Open Streets	0.025
Upland Woodland/Chaparral	0.07
Downtown SLO Commercial Buildings *	1.0-2.0
Stream Channels	
San Luis Obispo Creek Through City	0.045-0.065
San Luis Obispo Creek Below City Limits	0.06-0.07
San Luis Obispo Creek at Avila Golf Course	0.03-0.045
Stenner Creek	0.05-0.065
Brizziolari Creek	0.055-0.06
Prefumo Creek	0.06-0.07

* Downtown commercial buildings were coded with extremely high roughness to effectively block all flow from being conveyed through them. Overbank flow in those areas was allowed to travel down individual streets, which were coded with a roughness of 0.025.

The downstream boundary condition for the model was taken as the highest recorded tide at Port SanLuis, approximately 1.6 km (1 mi) west of the mouth of San Luis Obispo Creek. This water surface elevation was observed on January 18, 1973, during one of the largest storms on record for the region. It is approximately 0.73 meters (2.40 ft) above Mean Higher High Water (MHHW) at this location. A sensitivity analysis performed on this variable showed that the downstream boundary only influenced the model significantly for several hundred meters upstream of the mouth and had no impact on the model above the coffer dam upstream of the Avila Golf Course, about 2 km above the mouth.

C-2.2 Flow Splits

There are several points in the watershed where flow splits out of the main channel and spills across a roadway or berm, leaving the main channel for a significant distance. Specifically, this occurs on Stenner Creek above Foothill Boulevard and again at Murray and Santa Rosa Streets, and on San Luis Obispo Creek across Highway 101 at several locations in the vicinity of Madonna Road. At these specific locations, some of the assumptions made in producing a 1-dimensional model are violated, and a different modeling technique must be used. We used the broad crested weir equation to calculate the amount of flow lost from the main channel at these locations. A separate reach was defined in the HEC-RAS model for the overflow areas until they finally meet up with a modeled creek reach downstream of the breakout point.

C-2.3 Undercity Culvert

In Downtown San Luis Obispo, San Luis Obispo Creek runs for about 370 m (1200 ft) through a completely enclosed structure referred to here as the undercity culvert. According to a U.S. Army Corps of Engineers report (U.S. Army Corps of Engineers, 1985), the culvert has a capacity of 127 cms (4500 cfs). This is not sufficient to pass even a 25-year flow event, according to our hydrology model results. Flow in San Luis Obispo Creek was observed in1973 to split out of the channel upstream of the culvert and to re-enter the channel over 1 km (0.6 mi) downstream, along Higuera Street.

To model the undercity culvert in HEC-RAS, the capacity determined by the Corps of Engineers was assumed to represent the condition just before flow spills out of the channel immediately upstream of the culvert. For flow rates less than the culvert capacity, the culvert was modeled as a rectangular box whose dimensions and characteristics were calibrated so that a 127-cms (4500-cfs) flow just overtopped the channel at the upstream end. For flow rates greater than the culvert capacity, the model was made more stable by simply removing the culvert and modeling an overland flow rate equal to the total design flow minus the 127-cms (4500-cfs) culvert capacity.

C-3 Model Calibration

Regardless of the amount of detail incorporated into the model, calibration against real data must occur before results can be verified and used reliably. Calibration of the hydrologic and hydraulic models was performed using NEXRAD radar rainfall totals and high water marks observed for the storm of March 9 to 11, 1995.

C-3.1 Calibration Storm

One of the challenges of modeling the rainfall along California's Central Coast is the strong orographic influence the Coast Ranges have on precipitation totals. While rainfall for the March 1995 storm was recorded at numerous rain gauges throughout the basin, only six rain gauges in the immediate vicinity of the San Luis Obispo Creek Watershed recorded rainfall on the15-minute (or shorter) time intervals necessary for the hydrology model (**Figure C-8**). The difference between the lowest and highest rainfall total for the March 1995 storm was just over 100% of the lowest gauge total. These gauges were deemed insufficient to fully characterize the magnitude of the storm in certain parts of the watershed, especially where orographic effects would have acted to increase precipitation beyond what the valley floor experienced.

Figure C-9 shows cumulative rainfall at each of six recording rainfall gauges in the watershed. Peak recorded 24-hour totals ranged from 9.39 cm (3.69 in) at the Cuesta Ridge gauge to 21.56 cm (8.49 in) at the Santa Margarita Booster gauge, just north of the northern watershed boundary near the crest of Cuesta Ridge, while peak 48-hour totals ranged from 13.20 cm (5.20 in) at the Cuesta Ridge gauge to 29.76 cm (11.71 in) at the Santa Margarita Booster gauge. The rainfall totals at the county-maintained Cuesta Ridge gauge were significantly lower than at any of the other gauges and are likely in error–especially considering the much higher totals recorded a few miles away at the Santa Margarita Booster gauge. The Cuesta Ridge data were not used in any technical analysis. The next lowest totals were at the SoCal Gas gauge, near the San Luis Obispo Airport, with a 12.12 cm (4.77 in) 24 hr-total and a 14.29 cm (5.62 in) 48-hour total. Because of the wide variability in precipitation totals from gauge to gauge and because of uncertainty in the reliability of the county-maintained Cuesta Ridge gauge (and, by extension, at the other county-maintained gauge at Davis Peak), a more detailed method of modeling rainfall for the March 1995 storm was required.

To provide a more complete picture of rainfall for the March 1995 Storm, archival NEXRAD meteorologic radar information for the time period in question was used to develop a detailed set of rainfall information, on 15-minute time steps, for each basin in the watershed model. The meteorologic analysis, performed by NEXRAIN corporation, involved calibrating radar return information with gauged rainfall intensities so that the NEXRAIN dataset was consistent with gauged information. Gauges outside the San Luis Obispo Creek watershed were used for this rainfall calibration process. Data was first computed on a 2-km by 2-km grid, and then averaged by sub-basin. Totals for the peak 24-hour period ranged from 16.81 cm (6.62 in) for the Davenport Creek sub-basin to 33.20 cm (13.11 in) for the Harford Canyon sub-basin. A complete 48 hour period was not covered by the NEXRAIN dataset. The entire NEXRAIN dataset can be found in the HEC-HMS hydrology model, which is published on CD along with this document.

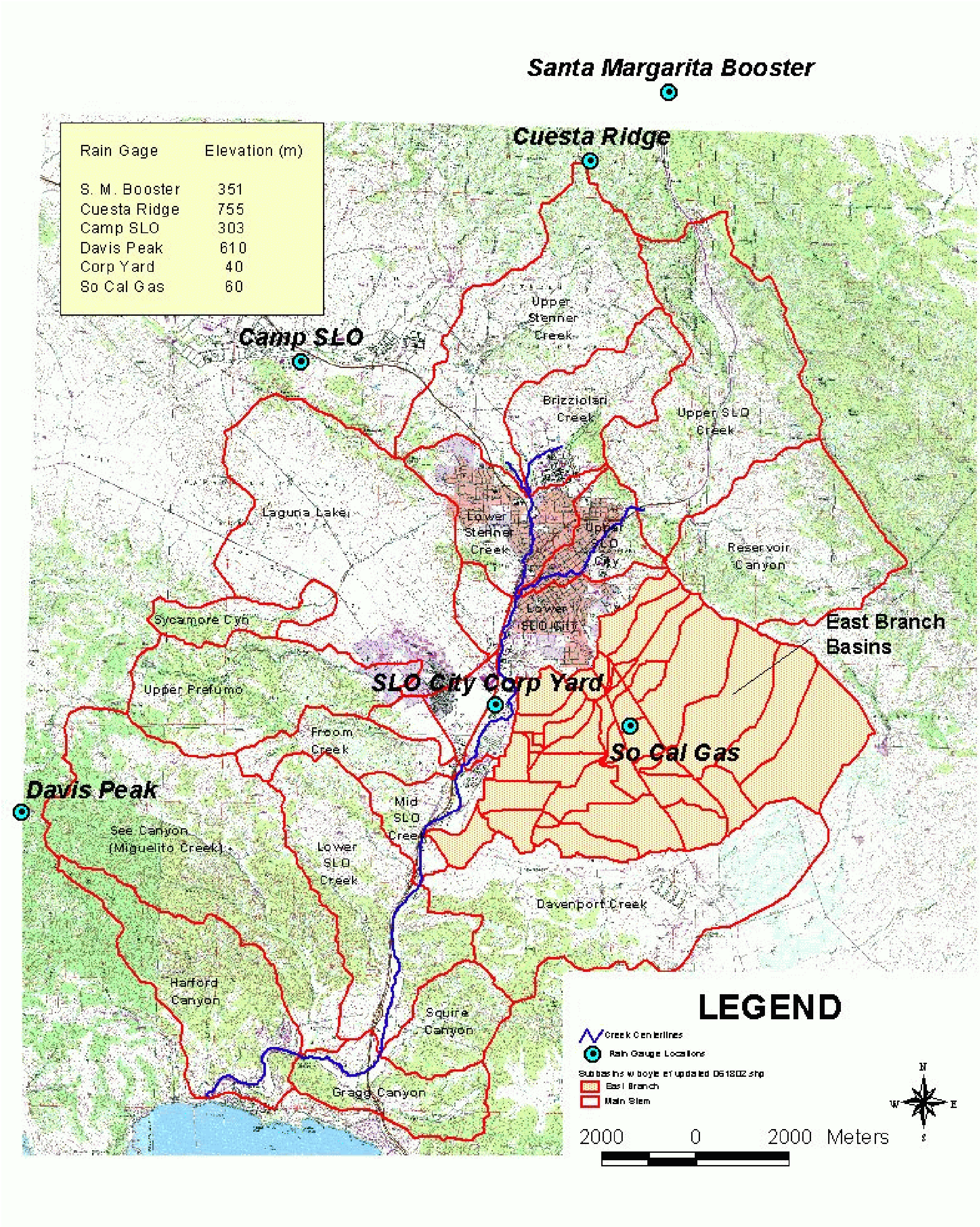

Figure C-8. Precipitation gauge Locations

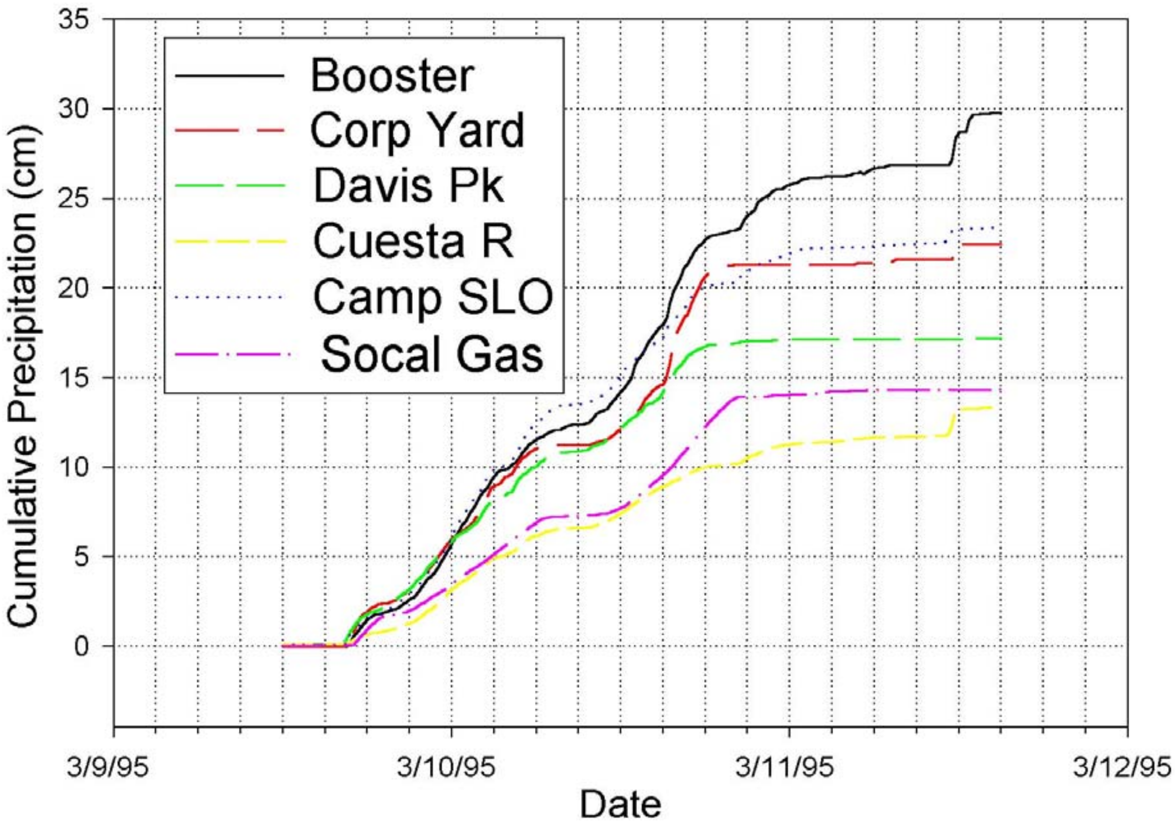

Figure C-9. Rain gauge record, March 9 to 11, 1995.

C-3.2 High Water Marks

Historically, at least two stream gauges existed in the San Luis Obispo Creek Watershed that would have been capable of recording flood peaks. One was located on lower San Luis Obispo Creek near Avila, and the other was located on Upper San Luis Obispo Creek, in San Luis Obispo. Unfortunately, both of these gauges were put out of service in 1992. Since that time, the city of San Luis Obispo has re-installed a gauge on Upper San Luis Obispo Creek. However, there is no gauge record for the 1995 water year.

The best records available for describing the effects of the March 1995 storm are in the form of high water marks surveyed at various points throughout the basin. Some of these high water marks were surveyed immediately after the storm, while others were derived later based on photographs taken near the flood peak. A summary of the available marks is shown in **Table C-6**

.

Table C-6. Hydrology Model Calibration Points.

Observed Mark Location	Source	NAVD 88 Elevation (m)	Reliability [1]	Back-Calculated Flow (m^3/s)	Uncalibrated HMS Flow (m^3/s)	Calibrated HMS Flow (m^3/s)
Sycamore Mineral Springs ~St 3342	Colleen Snyder, Sycamore Employee	9.21-9.36	A	450-500	655	571
Ontario Road ~St 4370	Dan Erdman, County Engineer	9.88	A	490	655	571
Below Sycamore Mineral Springs [2]	Church Water Consultants	multiple points	A	n/a	n/a	n/a
Caltrans Yard, ~St 15646	Caltrans Employee/City Survey	47.67	A	210	293	268
McNamera, ~St 16712	Property Owner/City Survey	53.33	A	325	271	247
Nipomo Bridge, 17431	Photo/LIDAR	58.4	B	140	161	148
Dana Street, ~St 17180	Photo/LIDAR	55.8	B	143	161	148
Upper Stenner Creek @ Radio Tower	Cal Poly Student Survey	N/A	C	62	64	58
Stenner Creek 300m above Nipomo	Cal Poly Student Survey	N/A	C	78	118	106
Brizziolari Creek Above Cal Poly	Cal Poly Student Survey	N/A	C	26	33	30

[1] Reliability is used here to denote the quality of the survey used to determine the high water mark. For an "A" rating, the datum (the mark must be correctly known and have been surveyed professionally. For a "B" rating, the location of the mark is precisely known, bu the elevation of the nearest surface visible on the LIDAR survey is used as a vertical datum. Neither the precise location nor elevatic of the "C" marks is known. These were taken from a senior project prepared by a Cal Poly student in 1995. Some manipulati of the data in the student report was required to allow the data to be used for this study

[2] Points surveyed by Church Water Consultands were too numerous to back-calculate flows individually. They were used as model validation. See Figure B-1

C-3.3 Calibration Technique and Results

Because no reliable stream gauge data was available for the March 1995 storm, the best way to check the results of the rainfall-runoff model against reality was to use the hydraulic model to back-calculate flow rates from recorded high water marks, and to then check whether the rainfall-runoff model produced these flow rates for the March 1995 storm. This raises the question of how we could computed reliable flow rates using the hydraulic model that itself had not been checked against reality. The reality check for the hydraulic model came from trying to make high water marks for any given region consistent with one another. This was accomplished by adjusting channel roughness assumptions until the high water marks produced consistent flow rates.

Without any calibration, the rainfall-runoff model gave fairly high runoff results **(Table C-6).** To achieve the best fit possible, the SCS curve number parameter was reduced by 15% across the entire model. The 15% reduction was applied to all basins of all watershed models, including the pre-European settlement model, the 1965 conditions model, the existing conditions model, and the future conditions model.

Figure C-10 shows the position of a set of high water marks taken on San Luis Obispo Creek near the lower San Luis Bay Drive Bridge with respect to the modeled water surface elevation for the March 1995 flow (after calibration). **Figure C-11** shows observed and calibrated high water model results on San Luis Obispo Creek near the confluence with Stenner Creek. The agreement between the high water marks from this data set and the modeled water surface is relatively good. The most error between the predicted and observed water surface within the City of San Luis Obispo occurred at the Marsh Street Bridge, where the observed flood elevation was approximately 0.6 m (2 ft) above the modeled flood elevation. The most likely reason for this discrepancy is the tendency for the Marsh Street Bridge to collect debris during a large storm event. Due to their unpredictable nature, the HEC-RAS model does not account for debris blockages (In general, bridges known to be prone to debris blockage should be monitored during large storm events, and any debris blocking the bridge opening should be removed.) It is likely that debris raised the flood elevation at the Marsh Street Bridge above the level that would have occurred if no debris had been present.

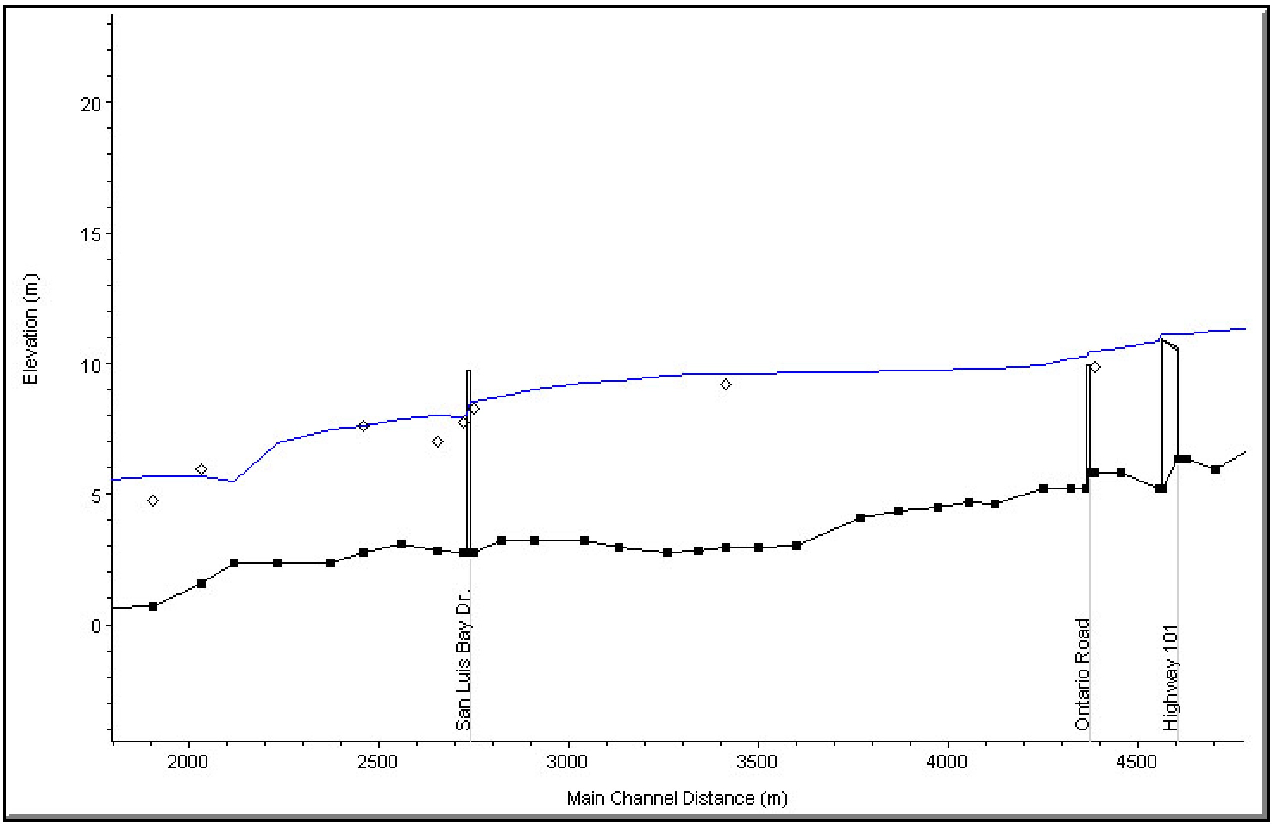

Figure C-10. Observed high water marks (black diamonds) compared with calibrated modeled water surface (blue line) for March 11, 1995 storm, near Avila Beach.

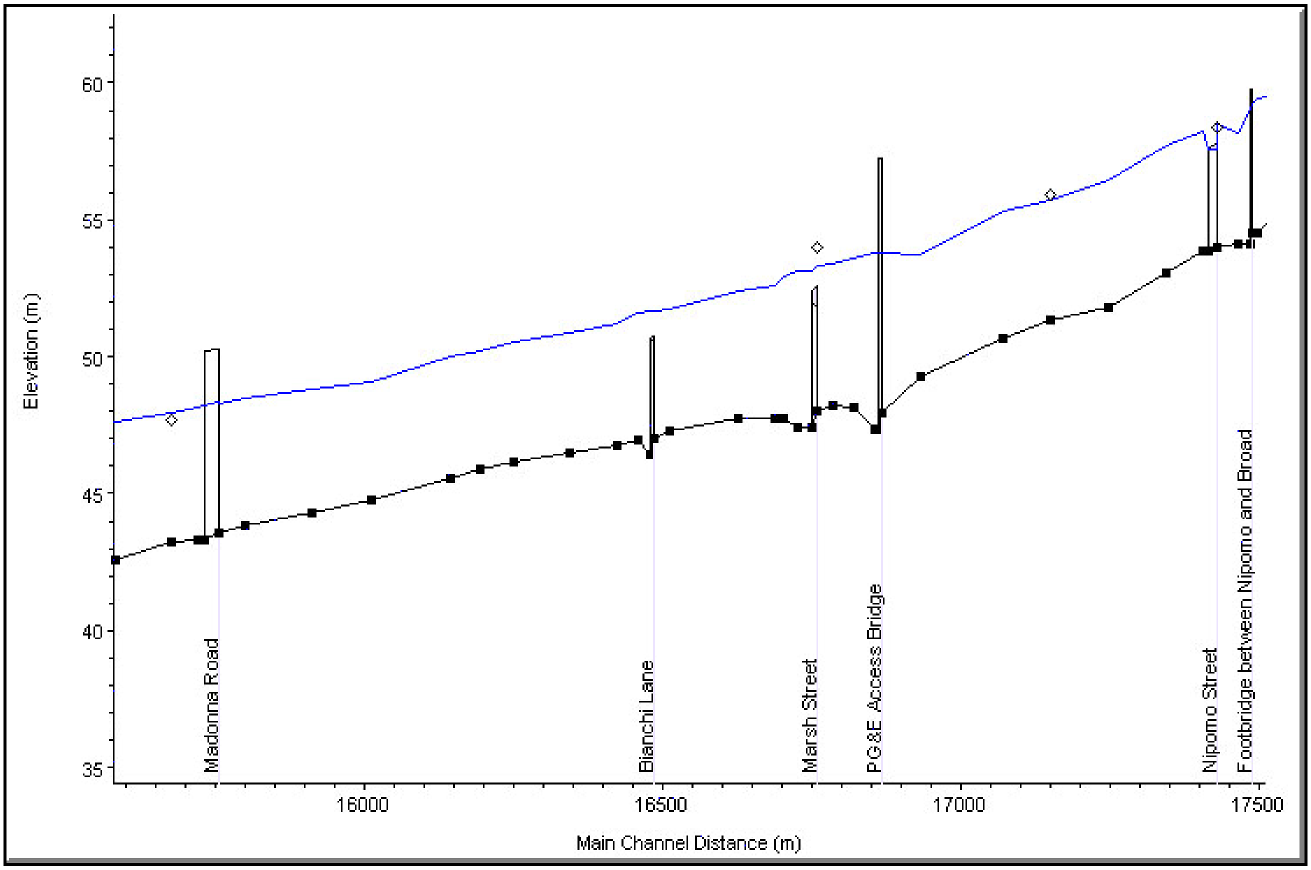

Figure C-11. Observed High Water Marks (black diamonds) compared with calibrated modeled water surface (blue line) for March 11, 1995 storm, within San Luis Obispo City Limits.

C-4 Results of the Hydrologic and Hydraulic Modeling

The results of the hydrologic model at various locations in the watershed are shown in **Table C-7**. Modeled water surface elevation profiles are included in at the end of this Appendix and are numbered **CP-1** (Appendix C/Profile #) through **CP-24**.

One of the objectives of this study was to evaluate the impact that development within the upper areas of the watershed has had on flood flow rates lower in the watershed. Typically, increasing impervious surface areas within a watershed increases flood risk downstream. To test this, design precipitation events were run through each of the four models (i.e. prehistoric, historic 1963, existing, and general plan buildout conditions). The results are shown in **Table C-8**. These results show very little change in peak flow rates from prehistoric conditions to existing conditions. This is primarily due to the presence of the two crossings of Highway 101 over San Luis Obispo Creek at Cuesta Park, above San Luis Obispo. The highway embankment at these locations acts as a dam, holding back the highest storm peaks. Were it not for the highway, increases in impervious surface throughout the watershed would likely have caused an increase of between about 4 and 7 percent, depending on recurrence interval and location in the watershed. Most of this effect likely occurred fairly early in this century, at least before the 1960's. However, the construction of the two Highway 101 crossings of San Luis Obispo Creek at Cuesta Park has essentially negated this increase.

Table C-7. Selected Hydrology Model Results for Existing Watershed Conditions

				Flow Rates (m^3/s)					
Creek	**Station**	**Description**	**Storm Size**	**Q100**	**Q50**	**Q25**	**Q10**	**Q2**	**1995 Flow**
SLO	20627	Cuesta Park	A	176	166	147	123	71	136
SLO	19319	At California Boulevard	A	190	179	160	133	79	148
SLO	16935	At Stenner Creek Confluence	B	354	319	274	220	119	247
SLO	15583	At Meadow Creek Confluence	B	378	341	292	231	127	268
SLO	12148	At Prefumo Creek Confluence	B	433	389	333	258	142	323
SLO	11897	At Froom Creek Confluence	B	444	398	342	264	145	333
SLO	10182	At E. Branch Confluence	C	538	476	412	309	165	468
SLO	9159	At Davenport Creek Confluence	C	596	525	455	338	179	516
SLO	4929	At Squire Creek Confluence	C	624	548	478	353	185	559
SLO	4554	At Gragg Canyon Confluence	C	632	555	485	357	186	570
SLO	3131	At Miguelito Creek Confluence	D	671	589	506	374	194	639
SLO	214	At Harford Canyon Confluence	D	686	603	513	376	191	669
Prefumo	1906	Laguna Lake Outlet	A	58	48	40	28	17	49
Prefumo	1385	At Drainage from Madonna Plaza	A	60	50	42	29	18	51
Prefumo [1]	432	At Calle Joaquin	A	71	62	51	35	21	56
Stenner		Above Brizziolari Creek	A	106	93	76	59	30	58
Stenner	2449	At Brizziolari Creek Confluence	A	166	146	120	93	48	85
Stenner	976	At Garden Creek Confluence	A	206	181	149	115	58	106
Brizziolari	n/a	Entire Sub-basin	A	70	62	51	40	21	30
Orcutt	2416	At Orcutt Road	A	6.2	5.5	4.6	3.4	1.7	2.8
Orcutt	1079	At Broad Street	A	12	11	9	6.9	3.5	5.2
Orcutt	583	At Confluence with Acacia Creek	A	15	14	11	8.6	4.3	6.6
Acacia	1877	At Orcutt Road	A	11	10	8.2	6.2	3.2	5.9
Acacia	1593	At Broad Street	A	40	35	29	22	11	22
Acacia	489	At Confluence with East Branch	A	42	37	30	23	11	23
East Branch SLO	6685	Above Acacia Creek Confluence	A	115	101	83	61	29	81
East Branch SLO	5984	Below Acacia Creek Confluence	A	154	136	112	83	40	107
East Branch SLO	4040		A	178	157	130	96	47	116
East Branch SLO	3425	Below Airport Tributary Confluence	A	186	164	136	100	49	120
East Branch SLO	1834	Below Tank Farm Creek Confluence	A	212	187	155	114	56	136
East Branch SLO	740	At Mouth	A	215	189	158	116	57	139

[1] Includes "Froom Tributary" basin, which during low flow drains to Froom Creek.
Inclusion of this basin results in conservative flow estimate where Prefumo Creek crosses under U.S. 101.

A more detailed analysis of these results and their flood management implications is available in

Table C-8. Impact of Changes in Land Use and Watershed Development on Flow Rates

Estimated Pre-Settlement Conditions	Flow Rates (m^3/s) Q10	Q100	% Change from Pre-Settlement Q10	Q100
SLO Creek Below Stenner Conf.	235	430	n/a	n/a
SLO Creek at Mouth	360	690	n/a	n/a
1963 Conditions				
SLO Creek Below Stenner Conf.	218	352	-7.2%	-18.1%
SLO Creek at Mouth	375	685	4.2%	-0.7%
Existing Conditions				
SLO Creek Below Stenner Conf.	220	354	-6.4%	-17.7%
SLO Creek at Mouth	376	686	4.4%	-0.6%
Existing Conditions, Discounting Detention at 101[1]				
SLO Creek Below Stenner Conf.	248	448	5.5%	4.2%
SLO Creek at Mouth	385	726	6.9%	5.2%
General Plan Buildout Conditions [2]				
SLO Creek Below Stenner Conf.	220	354	-6.4%	-17.7%
SLO Creek at Mouth	378	685	5.0%	-0.7%

[1] Currently Highway 101 at Cuesta Park provides some flood protection. These runs ignore this protection.
[2] Assumes Highway 101 at Cuesta Park is in its existing configuration. It may be possible to augmen the protection provided by the highway embankment. See Flood Management Alternatives Section

Section 5.4 of the Waterway Management Plan Report.

C-5 Comparison with Previous Studies

One of the motivating factors for the San Luis Obispo Waterway Management Plan (WMP) has been the frequent flooding that has occurred on San Luis Obispo Creek. It is believed that previous studies have inadequately predicted the relatively frequent occurrence of flooding in the area, especially in the Mid-Higuera area and along Stenner Creek.

C-5.1 1974 Corps of Engineers/Nolte/FEMA Study.

Since the 1970's, the definitive study on flow in the San Luis Obispo Creek watershed has been the 1974 U.S. Army Corps of Engineers floodplain study of San Luis Obispo Creek (U.S. Army Corps of Engineers, 1974). This study was updated in 1977 by George S. Nolte and Associates to predict flow rates at recurrence intervals other than the 100-year event. The Nolte study was used by FEMA for its Flood Insurace Study of the area (FEMA 1978).

The Corps/Nolte/FEMA study involved the construction of a theoretical watershed model similar in nature to that used for the current study. As in the current study, the Corps/Nolte/FEMA study split the watershed into a set of small sub-basins. A theoretical equation was used to predict rainfall losses for each sub-basin. Then a unit hygrograph was used to translate the rainfall excess (that not lost using the loss equation) into a runoff hydrograph. The hydrograph was then routed downstream from the outlet of each sub-basin in a similar way to the model described in this report.

The precipitation model used in the Corps/Nolte/FEMA study was very different than that used in the WMP, however. Instead of modeling a specific design precipitation event at each recurrence interval (i.e. a 10-year or 100-year 24-hour design storm) as was done for the WMP, the Corps/Nolte/FEMA study used an actual recorded rainfall event (in this case, the January 19, 1973 event) to define a storm that theoretically represented the maximum precipitation possible for a given part of the watershed. The process involved defining precipitation contours for the 1973 event, which was centered over the Irish Hills near the Prefumo Creek watershed, and then developing a way to re-center the storm over any given basin. The temporal distribution for the storm was determined from two recording rain gauges and was computed on 15-minute intervals.

The runoff occurring from the theoretical maximum possible precipitation event (which was derived from but different than the 1973 event), when centered over a given basin, was termed the standard project flood (SPF). The SPF has no direct relationship with a given recurrence interval. To develop such a relationship, a second watershed model was developed for the nearby Arroyo Grande Creek watershed, which at that time had a gauge with a 28-year record (prior to the construction of Lopez Dam) that had been analyzed statistically to determine a 100-year flow rate. At that gauge, the statistically-determined 100-year flood event was 63% of the SPF. This fraction was then assumed to apply to San Luis Obispo Creek watershed. The 100-year flow rate for any given basin in the San Luis Obispo Creek Watershed was found by multiplying the SPF for that basin by 0.63.

To determine flow rates at more frequent recurrence intervals, the Nolte study used a regional regression analysis of six nearby watersheds to define a set of regional flood frequency curves, which state the ratio of the 50-, 25-, and 10-year events to the 100-year event as a function of drainage area. These relationships were used to define flow rates at recurrence intervals other than the 100-year event in the San Luis Obispo Creek Watershed.

Flow rates from the Nolte study were used by FEMA to develop a backwater hydraulic model of San Luis Obispo Creek and tributaries within the City Limits of San Luis Obispo. The results of this model were used to develop the current FEMA flood plain map. This model was very similar conceptually to the HEC-RAS model employed by the current study

(WMP) to develop flood water surface elevations and flood plain information. However, advances in computer technology allow the current (WMP) model to use additional, more tightly spaced cross sections and more detailed floodplain topography and roughness information.

C-5.2 1999 U.S. Army Corps of Engineers Statistical Analysis of Local Stream Gauges

Serious flooding throughout the Central Coast of California in 1995 and 1997 prompted the U.S. Army Corps of Engineers to perform a flood frequency study at certain local gauges in 1999 as part of a larger study of San Luis Obispo and Monterey Counties. This study applied traditional flood frequency statistical analysis at several gauges in the watershed. The results are listed in **Table C-9**.

C-5.3 1999 U.S. Army Corps of Engineers Regional Statistical Analysis

After performing their analysis of specific gauges (Section B-5.2), the U.S. Army Corps of Engineers performed a regional flood frequency analysis using stream gauge data at various locations along the Central California Coast. This study resulted in a set of equations that predict flow rates at given recurrence intervals as a function of drainage area, mean annual rainfall, length of time of concentration, and length of "blue line" streams within the sub-basin on the appropriate USGS quadrangle map (U.S. Army Corps of Engineers, 1999b). The results at a few select points within the San Luis Obispo Creek watershed are listed in **Table C-9**. In general, this method resulted in lower flow rates than the analysis of specific gauges within the San Luis Obispo Creek watershed (Section B-5.2).

C-*5.4 Discussion of Differences From Previous Studies*

While the WMP model generally shows higher flow rates at all recurrence intervals than the previous studies (with the possible exception of the Corps of Engineers individual gauge analysis), the most important differences occur for frequent (i.e. 25-year or shorter) recurrence interval storms (**Table C-9**). The WMP model shows on the order of twice the flow rate from the Corps/Nolte/FEMA model at the 10-year event, while the difference is far less at the 100-year event. The one exception to the WMP results being higher than the Corps/Nolte/FEMA results is on San Luis Obispo Creek just above the confluence with Stenner Creek (point 2 in Table B-9). This occurs because the Corps/Nolte/FEMA model did not consider the detention provided by the Highway 101 culverts at Cuesta Park (Section C-1.3). In general, since the current model results in higher flow rates for frequent storms than the previous USACE/Nolte/FEMA model, its use will result in a more conservative flood management design.

The Corps of Engineer's individual gauge analysis is difficult to interpret. It shows a greater flow rate at the 100-year event on Stenner Creek than on lower San Luis Obispo Creek near Avila. In general, its results are higher than the current hydrology model results except at the gauge near Avila. It appears likely that the Avila Gauge may have mis-recorded high flow rates and should be dismissed. The fact that the Avila gauge was used by the Corps of Engineers for its regional regression analysis could help explain why the regional regression analysis predicts lower flow rates than either the current model or the Corps/Nolte/FEMA model. Additionally, work Questa performed for the County of San Luis Obispo (Questa Engineering Corporation, 2000) identified an error in one of the other gauge records used in the Corps gauge analysis (The Main Street gauge on Santa Rosa Creek in Cambria appears to have missed the peak of the crucial March 1995 flood event). Because of the uncertainties associated with the gauge record, further application of the 1999 Corps of Engineers hydrology studies to the San Luis Obispo Watershed should be undertaken only cautiously.

In summary, the WMP model generally predicts higher flow values than the other studies. Its use would consequently be expected to result in relatively conservative flood management designs. In any case, the development and calibration procedures for the WMP model used the most current technology and data available and should represent the most accurate and complete flow and flood plain information of any of the studies reviewed here.

Table C-9. Comparison of Modeled Flow Results with Other Studies

		Upstream Drainage Area		10-Year Flow		100-Year Flow	
		sq. mi	sq. km	m^3/s	cfs	m^3/s	cfs
Questa/Zone 9 Model							
1	SLO Creek Above City Limits (above Res. Cyn)	6.6	17.1	64	2300	117	4100
2	SLO Creek Above Stenner Creek Confluence	12.8	33.2	133	4700	190	6700
3	Stenner Creek Above Brizziolari Creek Confluence	5.8	15.0	59	2100	106	3700
4	Stenner Creek Above SLO Creek Confluence	11.1	28.7	115	4100	206	7300
5	SLO Creek At Squire Canyon	70	181.3	353	12500	624	22000
FEMA Flood Insurance Study							
1	SLO Creek Above City Limits (above Res. Cyn)	–	–	–	–	–	–
2	SLO Creek at Higuera Street (above Stenner conf.) [1]	12.6	32.6	71	2500	221	7800
3	Stenner Creek Above Brizziolari Creek Confluence [1]	5.7	14.8	31	1100	102	3600
4	Stenner Creek at Broad Street (above SLO conf.) [1]	10.8	28.0	59	2100	190	6700
5	SLO Creek above See Canyon [2]	64.6	167.3	119	4200	561	19800
Corps of Engineers Analysis of Individual Gage Record [3]							
1	SLO Creek "Near San Luis Obispo" (above Res. Cyn) [3]	5.27	13.6	46	1640	167	5900
2	SLO Creek Above Stenner Creek Confluence	–	–	–	–		–
3	Stenner Creek at Cal Poly (above Briz. conf.) [3]	5.5	14.2	76	2680	282	9950
4	Stenner Creek Above Confluence with SLO Creek	–	–	–	–		–
5	Lower San Luis Obispo Creek Near Avila [3]	67.7	175.3	146	5140	272	9620
Corps of Engineers Regional Regression Equation							
1	SLO Creek "Near San Luis Obispo" (above Res. Cyn) [3]	5.27	13.6	–	–	130	4608
2	SLO Creek Above Stenner Creek Confluence [4]	13.03	33.7	42	1500	136	4800
3	Stenner Creek at Cal Poly (above Briz. conf.) [3]	5.5	14.2	–	–	210	7427
4	Stenner Creek Above SLO Creek Confluence [4]	11.01	28.5	40	1400	108	3800
5	Lower San Luis Obispo Creek Near Avila [3]	67.7	175.3	–	–	485	17131

[1] Federal Emergency Management Agency. Flood Insurance Study: City of San Luis Obispo, California. October 1978.

[2] George S. Nolte and Associates. Flood Control and Drainage Master Plan for the San Luis Obispo Creek Watershed. 1977.

[3] U. S. Army Corps of Engineers, Los Angeles District. Part II Discharge-Frequency Analysis: Report on Hydrologic Analysis of San Luis Obispo, Santa Rosa, and Arroyo Grande Creeks.October 1999.

[4] U. S. Army Corps of Engineers, Los Angeles District. Part I Regional Discharge-Frequency Analysis: Interim Report on Hydrologic Analysis of San Luis Obispo, Santa Rosa, and Arroyo Grande Creeks. June 1999.

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