Report on the Calibration Analysis for the San Diego County Hydrology Manual

Especially Prepared for:
County of San Diego

Prepared by:
Theodore V. Hromadka, II
Ph.D., Ph.D., Ph.D., P.E., P.H.
Licensed Geoscientist
Professor Emeritus, California State University
Professor, Dept. Mathematical Sciences,
United States Military Academy, West Point, NY

May 2007

Hromadka & Associates
Hydrologic, Earth and Atmospheric Sciences

Table of Contents

List of Tables

I. Introduction

In this report is presented a summary of the tasks accomplished in evaluating the performance of the new Hydrology Manual for the County of San Diego. At issue are the confidence levels achieved by the new Manual's methodology and how the many components used in the modeling relate to these confidence levels. Because the Manual contains mathematical models of various components and processes of the hydrologic cycle, as well as statistical properties of the rainfall and runoff observed in the County's watersheds, the ensemble of these components form a hydrologic model that is used in predicting runoff quantities associated with target return frequencies. The model can also be used in rainfall-runoff analysis of actual storm events, and hence there is good reason to use a model that reasonably approximates such rainfall-runoff observations. Thus, there are two goals linked to the resulting hydrologic model, namely, approximating the rainfall-runoff relationships as observed locally, and approximating runoff quantities of prescribed return frequencies at specified levels of confidence.

The course of the project changed during the test watershed phase of analysis where a single test watershed was to be evaluated using the current Manual's single area unit hydrograph (UH) design storm modeling approach. Because only a single watershed was to be tested, the choice of that test watershed became the subject of scrutiny due to the limited variety of stream gage records and watershed histories. A test watershed was eventually selected, and a flood frequency analysis completed after the gage record was adjusted for the effects of urbanization. However, due to the flood frequency curve shape observed and the effects of the skew used, it was decided that better information would be obtained by examining the sensitivity of the current modeling approach versus use of alternative and readily available methods for several of the modeling components (e.g., depth-area curves, loss function construct, S-Graphs, lag estimation procedure, among other possible considerations.) As such, the project evaluated the 40-square mile test watershed problem already contained in the Manual, and modeling results were re-generated for a variety of modeling component substitutions. These modeling results are contained in this report.

From the study effort, it was concluded that the current Manual meets or exceeds the 85-percent upper confidence level goal in the estimation of runoff quantities. The current modeling procedure is found to produce runoff estimates which approximately match or exceed analogous analysis results generated by use of the County of Orange procedures. These procedures were, in turn, calibrated to the COE LACDA procedures and calibration results.

Recommendations for future research are offered in the last section of this report which may be of possible interest for future consideration.

A) Acknowledgements

Many experts collaborated in this report's preparation and provided help and assistance during the course of the project. Leading the effort were several hydrology experts from the County of San Diego including but by no means limited to: Jim Zhu (P.E., Civil Engineer), Hung Tran (P.E., Senior Civil Engineer) and Cid Tesoro (P.E., Program Manager). Other contributing experts included but were by no means limited to: Wayne Chang, Bethany Espinosa, Rene Perez, Tory Walker, and Robert Whitley.

II. Testing the Original LACDA Calibration Effort

A) Update of 25 Years of Stream Gage Data

In the early 1980's, the United States Army Corps of Engineers (COE) Los Angeles District Office prepared a rainfall and runoff unit hydrograph method calibration study for the Los Angeles County Drainage Area (LACDA), encompassing drainage areas of hundreds of square miles and dozens of stream gages and rain gages. Subsequent to that regional study, local county flood control agencies utilized those data and results to create their county flood control hydrology design criteria and documented their procedures in Hydrology Manuals. For example, the County of San Bernardino published their Hydrology Manual in 1983. In 1985, the Orange County Environmental Management Agency (OCEMA) published a calibration of the unit hydrograph procedure, and subsequently produced the Orange County Hydrology Manual in 1986. The County of San Bernardino followed in these footsteps and updated their Hydrology Manual in 1986 to incorporate the calibration. In 1989, the County of Orange extended their calibration effort by examining lower confidence levels ("Investigation of Mitigation Needs for Changes in Duration Floodflows Due to Development", Williamson and Schmid, July 1989).

The stream gage data originally used in the LACDA regional calibration included data up through the early 1980's. Now that some quarter of a century of additional data is available, a comparison of new flood frequency curves and confidence levels is appropriate in order to determine whether or not the additional data has any effect on the original results.

B) Bulletin 17B Procedures

The procedures recommended for flood-frequency analysis by U.S. federal agencies and in general use in the United States are those given in [20], a reference which is commonly referred to as (Bulletin) 17B; also see [31], [25], [21], and [13]. The confidence levels computed following the procedure in 17B are not accurate, therefore in the original LACDA calibration, simulations were used to give confidence levels using the methodology described in [32]. Because of the substantial increase in computing power now available, the simulations of [32] can be recreated with much greater accuracy.

The development of the procedures given in Bulletin 17B is described in [13]. A prior discussion was that of [30], the title of which, "A Uniform Technique for Flood-Frequency Analysis" indicates the important goal of 17B: to provide a uniform technique which would facilitate comparisons among various areas of the United States. An indispensable reference is [28]. The procedure adopted can be thought of as consisting of the following steps:

(1) Choose a distribution to represent annual maximal discharges. The distribution chosen was the Log-Pearson III distribution. This choice is discussed in Appendix 14 of 17B; see also [3], [4] and the comprehensive [13].

(2) Estimate the skew parameter for this distribution. The skew of the distribution can be estimated by combining skew data from several stations [20] [pgs 10-

15]. The variability of skew estimates based on site data alone led to a 17B recommended procedure of using a map of skew values for the United States, which is discussed in [23], [24], and [29]. Since the skew estimators specified in 17B use either a regional skew or a combination of a regional skew and a site skew, it is difficult to precisely model the variability in these estimators because that requires a joint distribution for regional skews and nearby site skews and there is scant available information about such a distribution. Because of this, the original LACDA regional calibration and this update use the adopted site skew in the calculation, ignoring the variability inherent in choosing this skew. For this simplified model it is possible to compute confidence levels with high accuracy. (There have been attempts to include the effects of the variability of the skew estimator, e.g. [9], [34], [35], and [6], [10], [11], with, as would be expected, less accuracy and with corresponding greater complexity than in the simpler situation modeled here .)

(3) Calculate remaining parameters. Various procedures have been examined for computing the remaining parameters of the distribution; for example, maximum likelihood [8], and see [6], [10], [11]. Bulletin 17B uses the usual estimates for the mean and standard deviation of logs of the yearly maximal discharges. These estimates and the value of the skew give enough information to estimate the three parameters in the Pearson III density.

(4) Use the resulting distribution to estimate various statistical aspects of the maximal flood distribution. An explicit requirement in the development of the methodology of 17B was that the calculations involved be easy to perform. This was accomplished by an estimation involving only the computation of the mean and standard deviation of the relevant discharge data, as noted above, and by looking up a tabulated constant. Specifically, a flood frequency curve for the discharge Q is obtained, for the given skew, from the basic equation (1) of [20]

$$\log Q = \hat{\mu} + \hat{\sigma} K, \tag{1}$$

where $\hat{\mu}$ and $\hat{\sigma}$ are the usual estimators for the mean and standard deviation of the logarithms of the maximal annual discharges, and K is read from a table given in terms of exceedance probability and skew (which implicitly implies a skew value which is known exactly). [20], [16], [17]. The plot of the values of equation (1) for values of K corresponding to various exceedance probabilities gives the flood-frequency curve of 17B.

C) The Flood Frequency Curve

The underlying (mandated) hypothesis of 17B is that the series of yearly maximum discharges from a catchment at a gage have a Log-Pearson III distribution [15], [12], [2]. The density for the logarithms then has the form

$$f(t) = \frac{[(t-c)/a]^{b-1}}{|a|\Gamma(b)} e^{-[(t-c)/a]} \quad for \quad (t-c)/a > 0, \tag{2}$$

$\Gamma(b)$ the gamma function, and $f(t) = 0$ for $(t - c)/a < 0$. The parameter b is related to the skew γ, denoted by G in 17B, by $b = 4/\gamma^2$; the parameter a has the same sign as the skew. For the important case of zero skew the distribution can, by a limiting argument, be shown to be a normal distribution.

In terms of the scaled variable $x = (t - c)/a$, the density for $x > 0$ is

$$f(x) = \frac{x^{b-1}}{\Gamma(b)} e^{-x}. \tag{3}$$

The values of K in the table in [20] are given in terms of skew and exceedance probability p'. Here the probability $p = 1 - p'$, related to the T-year flood by $p = 1 - 1/T$, will be used instead of p'. The K values given in 17B can be calculated in two steps:

(1) Given a probability p, use the distribution in equation (3) to find a value K' with the property that a random variable X with this density has $Prob(X \leq K') = p$.

(2) Use the fact that X has mean b and standard deviation $b^{1/2}$ to scale K' and so obtain the value K of the 17B table,

$$K = \frac{K' - b}{\sqrt{b}}. \tag{4}$$

For the case of zero skew, K is the p^{th} percentile for a normal random variable with mean zero and standard deviation one.

It will be useful to indicate the dependence of K on the parameters γ and p by writing:

$$K = K(\gamma, p). \tag{5}$$

By considering $K(\gamma, p)$ for negative values of skew it can be shown that:

$$K(-\gamma, p) = -K(\gamma, 1 - p). \tag{6}$$

One consequence is that the 17B table values for negative skew are superfluous as they can be obtained from the values for positive skew by the use of equation (6), as was clear to the authors of 17B [16].

Suppose that an estimate of the value of the T-year flood y_p, for $p = 1 - 1/T$, is needed. Equation (1) relates $K(\gamma, p)$ to the random variable $(y_p - \hat{\mu})/\hat{\sigma}$. Let Y have the density (2) and suppose that the site has n years of yearly maximal discharge data. Let $\hat{\mu}_Y$ denote the sample mean of Y, i.e. for a sequence of independent random variables $Y_1, Y_2, \ldots Y_n$, each with the same distribution as Y,

$$\hat{\mu}_Y = \frac{1}{n}\sum_{i=1}^{n} Y_i$$

and let $\hat{\sigma}_Y$ denote the unbiased estimator

$$\hat{\sigma}_Y^2 = \frac{1}{n-1}\sum_{i=1}^{n}(Y_i - \hat{\mu}_Y)^2 \ .$$

It can be shown that:

$$\frac{y_p - \hat{\mu}_Y}{\hat{\sigma}_Y} \equiv \frac{z_p - \hat{\mu}_Z}{\hat{\sigma}_Z} \ \ for\ skew \geq 0, \tag{7}$$

and

$$\frac{y_p - \hat{\mu}_Y}{\hat{\sigma}_Y} \equiv \frac{\hat{\mu}_Z - z_{1-p}}{\hat{\sigma}_Z} \ \ for\ skew \leq 0. \tag{8}$$

Here $\equiv$ means that the two sides of the equivalence have the same probability distribution. The random variable Z has a gamma distribution with the density given in (3). Corresponding to y_p, z_p is the value satisfying $Prob(Z \leq z_p) = p$, and $\hat{\mu}_Z$ and $\hat{\sigma}_Z$ denote the sample estimates for the mean and standard deviation of Z, based on the same number n of data points as are in the record of the log discharge Y values. (Note the distributions in (7) and (8) are the same in the case of zero skew, where Z is normally distributed with mean zero and standard deviation one, by symmetry.)

Equations (7) and (8), given in [32], are the key to improving the values used for K, because the statistic given in these equations gives the distribution of K (as a random variable) and depends only on the skew and not on the parameters a (except for its sign), or c.

D) Analysis of Bulletin 17B Confidence Intervals

One critical question to ask of any procedure for estimating a return frequency value for maximal discharge is: What protection is obtained by the use of this procedure? If this procedure were applied over and over again to independent samples from the given distribution, how often would the true (and unknown) T-year flood value y_p be less than or equal to that estimated by use of the procedure? Which is to say how often would the use of this estimate in flood protection measures actually provide protection against an occurrence of the T-year flood? In statistical terms, one refers to the one-sided confidence interval or the confidence level given by the procedure. The numerical value for this confidence level gives a good idea of the statistical risk posed by the use of the procedure.

In order to evaluate the confidence levels obtain in using the Bulletin 17B estimator for the T-year flood, the probabilities

$$Prob(z_p \le \hat{\mu}_Z + K(\gamma, p)\hat{\sigma}_Z) \tag{9}$$

are computed for representative values of γ, p, and the number n of data points used in computing the sample mean and standard deviation.

The values for negative skew can be obtained from those for positive skew as follows: suppose $\gamma < 0$, from (8)

$$Prob\left(\frac{y_p - \hat{\mu}_Y}{\hat{\sigma}_Y} \le K(\gamma, p)\right) \tag{10}$$

$$=Prob\left(\frac{\hat{\mu}_z - z_{1-p}}{\hat{\sigma}_Z} \le K(\gamma, p)\right) \tag{11}$$

$$=Prob\left(\frac{z_{1-p} - \hat{\mu}_Z}{\hat{\sigma}_Z} \ge -K(\gamma, p)\right). \tag{12}$$

Using (6)

$$=1\text{-}Prob\left(\frac{z_{1-p} - \hat{\mu}_Z}{\hat{\sigma}_Z} \le K(-\gamma, 1-p)\right). \tag{13}$$

By comparing with (7), the probability in (13) is seen to be that used to find the confidence level corresponding to $K(-\gamma, 1-p)$. Numerical examples will be given below.

The $p = 1 - 1/T$ values used in the Table 1 and Table 2 correspond to those used in the table of K values in [20]. The 17B table gives K values for skews in the range -9 to 9, but an examination of the skew values on the United States maps given in [20] shows that all the skews are in the range -2 to 2 and in fact nearly all (97 %) are in the range -1 to 1, and consequently the set of values {0, 0.5, 1} was chosen for the computations here. The extension to negative skews {−0.5, −1.0} will be discussed below. The computations were done for site record lengths of $n = 10$, which 17B considers to be the smallest number of site data points to which its procedure should be applied, and $n = 50$.

The confidence levels obtained when (1) is used were computed using equation (7). For each of the skew values under consideration and each of the record lengths numbers $n = 10$ and $n = 50$, one million sites each with n values were simulated; so for $n = 50$ and one of the γ values, fifty million gamma distributed random variables were simulated. A count was kept of the number of times $z_p \le \hat{\mu}_Z + \hat{\sigma}_Z K(\gamma, p)$ and the resulting percentages are the values in the table. The simulation was run a second time and the fractions obtained differed by at most .001, i.e. 0.1%. The program was checked by comparing percentiles for the sample mean $\hat{\mu}_Z$ of equation (7) with the theoretical percentile values and was further checked by comparing a selection of K values with the output from the program HEC-FFA as well as with the tabulated K values from 17B. In addition, selected values in the tables themselves were checked by a separate Mathematica program using a relative small (10,000) number of sites.

Example 1: For a skew of 0.5 and $n = 10$ site data points, Table 1 shows that the use of the 17B table of K values to produce the value of $\log(Q)$ of equation (1) for the $T = 100$ year flood $(p = 0.99)$ gives a value which the true 100-year flood will be less than or equal to 44% of the time, and so of course greater than 56% of the time. That the computed value does not give the desired protection 56% of the time may or may not be acceptable to the user, but as things stand in 17B this is not quantified.

Example 2: For a negative skew value of -0.5, $n = 10$ site data points, and the $T = 100$ year flood $(p = 0.99)$, the use of equation (13) and the entry for $p = 1 - 0.99 = 0.01$, shows that the true 100 year flood will be less than or equal to the computed value 100% − 55% = 45% of the time.

E) The Confidence Levels

One solution to the problem of the risk associated with the use of 17B flood frequency curves would be to adjust the constant K in equation (1) so that this equation provides a confidence level which is acceptable to the user. Appendix 9 of 17B gives formulas, for a given $0 < c < 1$, for estimators $U_{P,c}(X)$ and $L_{P,c}(X)$, applied to a sample "X" of data points at the site, which are supposed to have the property that

$$Prob\{U_{P,c}(X) \geq X_p^*\} = c \tag{14}$$

$$Prob\{L_{P,c}(X) \leq X_p^*\} = c \tag{15}$$

where X_p^* is the true but unknown value of the T-year flood, $p = 1 - \frac{1}{T}$, and the probabilities are in the sense of repeated sampling.

It is known ([27], [1]) that the confidence intervals for quantiles given in 17B are not satisfactory, and [27] gives an approximate formula, using [22], which is accurate for small skew [33]. The confidence levels for 17B were also checked by [18] in the same way as they will be checked here, but, because of the limited computer power available 30 years ago, the author at that time was forced to use small sample sizes of 1000. In the calculations here, one million sites are used together with a random number generator with a period of $2^{31} - 1$. For a site with a record of length 50, this generation of 50 million gamma distributed random variables has a total running time of about 30 seconds.

From the point of view of confidence levels, the probabilities of interest are the probabilities that the actual T-year value is less than or equal to an estimator, which is the value c in (14) but is the value $1 - c$ in (15). For consistency the values reported in Tables 3, 4, 5 and 6 labeled q are for the 17B computed probabilities that the T-year flood, reported in terms of $p = 1 - \frac{1}{T}$, will be less than or equal to that obtained by use of the 17B estimators. The formulas for the estimators require that $c \le 0.5$. For $c > 0.5$, $L_{P,1-c}(X)$ supplies the confidence level c while for $c \le 0.5$, $U_{P,c}(X)$ supplies the confidence level. One further point: Bulletin 17B notes that for zero skew the Pearson III distribution is a normal distribution and therefore precise confidence levels can be computed from a table of the non-central t-distribution. (Subroutines for the non-central t-distribution are available in the IMSL software library used for the calculations given here.) The result of the tests given in Tables 3, 4, 5 and 6 for $\gamma = 0$ are actually for $\gamma = 0.001$ so as to indicate how the accuracy in the formulas for confidence levels given in Appendix 9 of 17B varies with skew.

In Tables 3, 4, 5 and 6, the confidence level is indicated by q, the number of years of record by N, the T-year flood by $p = 1 - 1/T$, and the skew by γ. The table entry is the actual percent of the time the estimator is greater than or equal to the true value of the T-year flood.

Example 3: For a site with 10 data points and a skew of 0.5, Table 3 shows that 17B 50% confidence level for the 100-year flood actually is a 43.6% confidence level, i.e. it provides a procedure which gives an estimate greater than or equal to the 100-year flood not 50% of the time but 43.6% of the time.

The 17B confidence level c for negative skew $-\gamma$ and a T-year flood, $p = 1 - 1/T$, is equal to the confidence level c which 17B provides for positive skew for $1 - p$ and γ. This follows from (6), and, using the notation of section 9 of 17B with a slight modification to indicate the skews involved, the equation $K^{U}_{p,c}(\gamma) = -K^{L}_{1-p,c}(-\gamma)$. And the fact that if X has a Pearson III distribution, scaled to have mean zero and standard deviation one, with skew γ, then $-X$ has a Pearson III distribution with mean zero, standard deviation one, and skew $-\gamma$.

Example 4: For a site with 10 data points and a skew of -0.5, Table 5 shows that the 75% confidence level provided by 17B actually is an 82.0% confidence level.

F) Accurate Confidence Levels by Simulation

Using (7) and (8), simulations can deliver accurate values for constants $K(N,\gamma,c)$ for which

$$Prob(y_p \le \hat{\mu}_Y + \hat{\sigma}_Y K(N,\gamma,c)) = c$$

is approximately true with error in the fourth decimal. That this can be done is clear in principle but was impractical at the time 17B was written. To give an idea of the change in readily available computer power, a remark made in ([32]) can be used to estimate that a program which would have taken 40 hours using the personal computers and software of 20 years ago, would now take about 10 seconds. The computer programs for this paper were compiled with a Lahey/Fujitsu FORTRAN 95 compiler using the IMSL software library. The simulations involved for a specific T-year flood take, in the case of $N = 50$

data points, approximately three minutes. An accurate value for the constant K of (1) is obtained by a simulation of the right-hand side of (7) or (8) in which an empirical histogram is used to obtain percentiles corresponding to the desired K values. After the values of K have been calculated, they are tested using a completely different random number generator from that used in the simulation.

Tables 7, 8, 9 and 10 give the results of testing the K values obtained from simulations. Each value was tested twice, and the numbers reported in the Tables are the larger of the two test errors.

Example 5: Table 7 shows that for the 100-year flood, $N = 10$ and $\gamma = 0.5$, the simulation value of K gives a confidence level of 50.05% for the 50% level.

Example 6: Table 9 shows that for $N = 10$ and a skew of −0.5 the simulation value of K gives a confidence level of 49.9% for the 50% confidence level.

Example 7: A specific case involving hypothetical data: $N = 10$, $\gamma = 0.5$, $\hat{\mu} = 3.4$ (Logarithms base 10 are used so the corresponding discharge is $10^{3.4} = 2512$ cfs), and $\hat{\sigma} = 0.2$, is given in Table 11.

G) Updating the Stream Gage Records

A key element of the new analysis is to update the LACDA flood frequency estimates by including nearly a quarter century of additional stream gage data that post-dated the LACDA study itself. Five stream gages (Alhambra Wash, Compton Creek, Eaton Wash, Rubio Wash and Arcadia Wash) are the focus of this re-analysis and are tabulated in Tables 12-16. Table 17 provides hydrologic information relevant to these five stream gages. It is noted that other stream gages, such as Dominguez Channel, were considered in this re-analysis but since there was no additional data, the stream gage records could not be extended.

H) Adopted Skew

As part of the analysis, an “adopted” skew value is assumed, as is done in the 17B procedures. The adopted skew value used in the re-analysis is the value used in the original calibration report ([19]). It is noted that had the skew value not been ”adopted”, then additional uncertainty would be simulated and the confidence level estimates would increase in value above what is reported by this report. That is, by “adopting” a skew value (according to the 17B procedure and as done in the original calibration), it is presupposed that skew is known, which of course is not possible. These adopted skew values are found in Table 18.

I) Flood Frequency Curve Results

Using the simulation procedure described in the previous sections of this report, each of the five stream gage data sets were extended by directly including the latest quarter century of runoff peak flow rate data. The calibration analysis of 1987 had concluded that urbanization effects stabilized some 50 years ago and so further adjustment of the runoff data was not necessary beyond that which was done in the 1987 calibration effort. Using the adopted skew values established in the 1987 calibration effort, confidence level estimates were made for several return frequencies of storm events and for various confidence levels. A similar analysis was made for the original stream gage record data sets in order that a comparison could be made between the extended and un-extended data sets. In order to determine a generalized trend as to the effect on the prior calibration effort of 1987 by the inclusion of the extended record stream gage data, some ratios are used between the extended and un-extended data set results, and in order to develop trends across the five stream gage data sets, two types of weightings are used as discussed in the text below. The results of all of these analyses are summarized in the following tables.

Table 18 reviews the relevant statistics for the five gages for the two periods of record, extended and un-extended, and includes the adopted skew values.

In Table 19, the peak Q results are in two sets of tabulations, for the extended and un-extended records, respectively. This is a collection of smaller tables of peak Q values at each gage with the confidence levels and return frequencies listed. Table 19 is the underpinnings of the following tables.

Table 20 simply compares the peak Q estimates for the relevant confidence levels versus return frequency estimates by taking the ratio of extended/unextended record estimates. It is curious to note that the biggest change occurs at the 50% confidence.

Table 21 tries to come up with a single representative statistic to describe the overall change in estimates due to increased record length. This is done by two popular methods of weighting averages of the Table 20 ratios, using watershed area for one method, and record length for the other method. The two methods of weighting provide very similar results. Overall, adding more years of record (almost a quarter century) increases the 50% confidence results.

Table 22 compares the differences between the 50% confidence values and the other calculated confidence level values.

Table 23 tries to come up with another single representative statistic to describe the overall results of Table 22, by using the same two weighting methods used in Table 21. Again, the conclusions are very close between the two methods of weighting.

Table 24 examines whether or not the higher confidence level estimates “close in” on the 50% confidence results.

III. Test Watershed

A. General Information

The course of the project changed during the test watershed phase of analysis where a single test watershed was to be evaluated using the current Manual's single area unit hydrograph (UH) design storm modeling approach. Because only a single watershed was to be tested, the choice of that test watershed became the subject of scrutiny due to the limited variety of stream gage records and watershed histories. A test watershed was eventually selected, and a flood frequency analysis completed after the gage record was adjusted for the effects of urbanization. However, due to the flood frequency curve shape observed and the effects of the skew used, it was decided that better information would be obtained by examining the sensitivity of the current modeling approach versus use of alternative and readily available methods for several of the modeling components (e.g., depth-area curves, loss function construct, S-Graphs, lag estimation procedure, among other possible considerations.) As such, the project evaluated the 40-square mile test watershed problem already contained in the Manual, and modeling results were re-generated for a variety of modeling component substitutions.

B. Choice of Test Watershed

Working in conjunction with County personnel and following their recommendations, the test watershed was chosen. Located in San Diego County, this watershed consists of 7,817 acres with the Los Coches Creek watercourse running through the midst. Stream Gage 11022200 on Los Coches Creek provided stream gage data for the time period of 1983 to 2005. A detailed analysis of development over time within the watershed was also conducted to help determine whether certain variables needed to be adjusted over the length of record being studied.

C. Stream Gage 11022200

Stream Gage 11022200 is owned and operated by the USGS. The gage on Los Coches Creek was installed by the County of San Diego prior to 1983 and has been in continuous operation by the USGS since 11/20/83. This gage is a bubble gage which operates by pressure differential and records data continuously every 15 minutes. The data recorded from 1983 to 2005 was obtained from USGS NWIS in digital form.

D. Stream Gage Data

In order to measure the effects of urbanization on the data from Stream Gage 11022200, our test watershed characteristics were examined with respect to time. Since each annual peak flow rate would be different today if the identical storm happened now, an adjustment may be made to the data by multiplying each annual Q by a factor which is a function of time. The three key elements which were candidates for change were: lag time, unit hydrograph, and loss rates.

E. Lag Assumed Invariant

The watershed lag was assumed not to change since photo reconnaissance indicated small changes to main watercourse channelization over the time period of the stream gage record.

F. Unit Hydrograph Assumed Invariant

The unit hydrograph for this watershed was assumed to be invariant as well. This is due to the small changes in drainage patterns observed in the photo reconnaissance over the time period of the stream gage record.

G. Percent Impervious Versus Time

Of the three factors originally mentioned, only the loss rate appears to change significantly with time, which primarily is due to an increasing imperviousness. The percent of pervious area was plotted versus time and a linear regression was derived and applied to the years of record for Stream Gage 11022200 (i.e., years 1983-2005).

H. Stream Gage Data Adjustment Procedure

Using a Rational Method approach, there is a runoff coefficient, C_i, for the fraction of impervious area, and a runoff coefficient, C_p, for the remaining fraction of pervious area. The area weighted runoff coefficient is then

$$C = C_i A_i + C_p A_p , \tag{1}$$

where

$$A_i + A_p = 1. \tag{2}$$

It was assumed that C_i=1, thus

$$C = A_i + (1 - A_i) * C_p . \tag{3}$$

Three values for C_p were considered: {0.4, 0.6, 0.8}. The values of A_p and A_i were obtained by examination of the watershed and equation (2), respectively.

Next, an adjustment factor, K, was calculated as

$$K_j = \frac{A_i + (1 - A_i) * C_p}{A_{ij} + (1 - A_{ij}) * C_p} , \tag{4}$$

where K_j is the adjustment factor for year j, and the numerator of every K_j value is the latest condition, i.e. the year 2005 value.

Then, the adjusted Q for every year j is given as

$$Q_{adj(j)} = K_j * Q_j . \tag{5}$$

I. Log Pearson III Flood Frequency Curve Analysis

Since there are three values for C_p being considered, there are three synthetic stream gage records obtained. All of these were run through the FFC program to look at the sensitivity of the data with respect to the chosen C_p value. Examination of the data illustrates that there was very little sensitivity in this regard. It should be noted that for each of these FFC runs, the site skew value was used.

J. Evaluation of LACDA Watershed Single Area Unit Hydrograph Design Storm Models

Single area unit hydrograph models were prepared for the subject LACDA calibration watersheds for comparison purposes with the original modeling accomplished in the work leading up to the cited 1987 COE Calibration report. The parameters used were the same parameters used in the original calibration study and are listed in the cited COE report. The Urban S-Graph was used, which corresponds to the current Manual's S-Graph that was originally developed by the SCS some 35 years ago. The loss function used in the calibration was re-used but as a comparison, two of the watersheds were re-run using the SCS loss function as found in the current Manual. The effect of using the SCS loss function was a significant increase in peak flow rates over that obtained by using the two-parameter loss function of the COE report (1987).

The two watersheds were then re-run using the current Manual's depth-area curves. The result of using the Manual's depth-area curves instead of the 1943 Sierra-Madre event depth-area curves, as used in the cited 1987 calibration report, was a significant increase in peak flow rates. It is noted that the said 1987 COE calibration report compares the effect of using the current Manual's NOAA Atlas 2 depth-area curves versus the local severe storm Sierra-Madre event depth-area curves. It was concluded that the Sierra-Madre depth-area curve set was a significant element in modeling and that use of these curves better fit stream gage calibration efforts than using the NOAA Atlas 2 depth-area curves.

In all of these tests, the sum of travel times was used as the estimator for catchment Tc and not the COE lag regression equation. Estimates of lag using the COE lag equation resulted in decreases in catchment lag estimates which would result in an increase in peak flow rate estimates for these two test cases.

In these tests, the original rainfall T-year return frequency rainfall values were used so as to not obfuscate the sensitivity of modeling results by concurrent parameter variations.

In order to better demonstrate the sensitivity of modeling results to variations in modeling parameters, the 40 square mile example problem watershed found in the current Manual is examined with respect to the stated parameter and algorithm variations.

K. Single Area Design Storm Unit Hydrograph Analysis Using Current Manual as Baseline

The example problem presented in the Manual utilizing a 40 square mile watershed (Example 3.2 from the San Diego Workbook) was analyzed using various combinations of parameters in order to evaluate relative significance. Thus, three models were set up whose results are contained in Table 25.

IV. Conclusions and Recommendations

1. For any calibration effort, the target must be well defined. In our case, the design storm unit hydrograph procedure presented in the Manual includes several components that combine to produce a modeling response for the inputs of watershed size, runoff response factors of timing and arrival time distribution, loss rates, T-year rainfall return frequency, design storm pattern shape and characteristics, and depth-area curves. The "target" is categorized as two parts: (a) given actual storm rainfall, how well does the methodology reproduce storm runoff, and (b) estimating T-year peak flow rate and volume quantities. The first target is accommodated by the good performance that calibrated unit hydrograph models have in watersheds such as experienced in southern California. The second target requires a decision as to the confidence levels desired and the applicability of the Log-Pearson III distribution to local watersheds.

2. In the early 1980's, the U.S. Army Corps of Engineers (COE) prepared a detailed study of the Los Angeles Drainage Area (LACDA) where several stream gages were analyzed for past significant storm events and flood frequency estimates were determined for several watersheds. That COE study formed the underpinnings of the County of Orange Calibration study which was originally published by that County in 1985 and then re-published by the COE as a COE technical report in 1987. In the County of Orange calibration effort, an assumption was made that the LACDA watersheds were hydrometeorologically similar to the County of Orange watersheds, and that the rainfall-runoff relationships developed for the LACDA study were applicable to the County of Orange except where local soils and rainfall information were applicable. This assumption has also been applied to other Counties in California including San Bernardino, Kern, and San Joaquin, among others.

3. Because a quarter century of additional stream gage data are available since the conclusion of the LACDA study, the "target" was re-evaluated to see what changes would occur with the addition of the updated data. The results of this update analysis show that the rare return frequency estimates for peak flow rate increased with the addition of the latest data, but only by a few percent. Although not examined, it is plausible that rainfall return frequencies may also increase when the latest quarter century of rainfall data are included in the rainfall statistics. If the rainfall statistics were to increase as observed in the LACDA runoff data, then it is rational to assume that the original model calibration effort would apply directly. Furthermore, the new flood frequency curve values were higher than previously estimated in the LACDA study; however, the new values were still less than the original 85-percent confidence level estimates. That is, designs and plans based on the prior 85-percent confidence level estimates provided flood protection for the new flood frequency curve analysis results.

4. Because the County of Orange procedure is analogous in many ways to the new San Diego County procedures, and because it may be assumed that the County of Orange and the County of San Diego regions are hydrometeorologically similar, the basis of the County of Orange Hydrology Manual may be assumed to also be applicable to the

County of San Diego. Indeed, many features and specifics of the new Manual can be found in the County of Orange Manual. Of course, as stated before, local trends in loss rates and rainfall would be applicable in the respective Manuals.

5. Because of the relatively small increase in high frequency runoff estimates with the update of the LACDA stream gage analysis, the current County of Orange Manual still accommodates, at the 85-percent confidence level, the updated flood frequency curve values. Consequently, the departures found between the County of Orange and the County of San Diego Manuals represent departures with the LACDA information used in the original calibration. The key departures between Manuals are as follows:

a) S-graphs available: The County of Orange provides for four types of S-Graphs. The San Diego Manual provides for one S-Graph (see their Figure 4-4).

b) Lag time estimation: The County of Orange provides for a more rigorous estimation of lag time based on the sum of travel times in the main channel linkage network, whereas the San Diego Manual allows the use of the original COE lag equation. (The County of Orange calibration effort showed an increase in correlation between the use of their sum of travel times versus actual catchment lag values resolved from rainfall-runoff data). The Manual does include, however, provisions for sums of travel time (see their Section 4.3.1.2).

c) Local rainfall and loss rate trends: Of course, the local data would be used.

d) Loss rate formula: The County of Orange uses a two-parameter loss function whereas the County of San Diego uses the so-called SCS loss function (see their Section 4.3.3). The SCS loss function was not developed in tandem with the other components found in the Manual and so the applicability of that SCS loss function to the other components of the Manual are not well known. The SCS loss function exhibits a strong response to design storm timing among other effects which the County of Orange loss function does not.

e) Depth-Area curves: The County of Orange uses the 1943 Sierra-Madre thunderstorm event from the COE analysis of that event in Los Angeles. The County of San Diego uses the 1972 published NOAA depth-area curve set (see their Figure 4-2) that were not developed using southern California data.

6. The several departures mentioned above all tend to increase runoff estimates for the County of San Diego method over the values obtained from the County of Orange method. (Of course, this is setting aside the issue that loss rates and rainfall trends differ between these two Counties. Otherwise, rainfall-runoff trends should be similar because of the similarity between these two Counties.) The Depth-Area curves used by the County of San Diego produce significantly higher runoff quantities of peak flows than do the Sierra-Madre curves. Lag estimates tend to be significantly smaller than the sum of travel times, causing an increase in peak flow rate estimates. Using the single S-Graph tends to produce greater runoff estimates than the other S-Graphs found in the County of

Orange Manual which have more storage effects incorporated. The use of the SCS loss function tends to produce smaller loss rates than does the two-parameter function used by the County of Orange, resulting in higher peak flow rates. All of these departures aim in the same direction as far as being conservative in runoff estimates.

7. To demonstrate the impact of the above departures, the example problem presented in the Manual (40 square mile watershed) was analyzed using various combinations of the mentioned departures in order to evaluate relative significance. These comparisons demonstrate that all of these departures may be significant.

8. From the above analysis, it is possible to conclude that the data and analysis considered in this study show that the San Diego Manual's methodology achieves the 85-percent or higher level of confidence in estimating peak flow rates and runoff quantities.

9. Recommendations are offered as follows:

a) The County may wish to further develop their stream gage and rain gage network and appropriately handle these new data.

b) Once an appropriate record is developed, the County may wish to re-evaluate their Manual's performance and again analyze the confidence level achieved by their methodology.

c) The County may wish to study their modeling responses and relate that response set to their risk tolerance in order to further refine their Manual's goals in modeling estimates.

V. References

[1] Ashkar, F. and Bob´ee, B (1988), Confidence intervals for flood events under a Pearson 3 or Log-Pearson 3 distribution, *Water Resources Bulletin*, *24* 639-650

[2] Beard, L.R.(1962), Statistical Methods in Hydrology, *U. S. Army Engineer District, Corp of Engineers*, Sacramento, California, 74 pp

[3] Bob´ee, B. (1975), The Log Pearson Type 3 Distribution and Its Application in Hydrology, *Water Resources Research*, *14* 365-369

[4] Bob´ee, B. and Robitaille, R. (1977), The Use of the Pearson Type 3 Distribution and Log Pearson Type 3 Distribution Revisited, *Water Resources Research*, *14* 427-443

[5] Cohn, T.A., Grifis, V.W., and Stedinger, J.R.(2004), Log-Pearson type 3 quantile estimators with regional skew information and low outlier adjustment, *Water Resources Research*, *40*, W07503, doi: 1029/2003WR002697

[6] Cohn, T.A., Lane, W.L., and Baier, W.G. (1997), An algorithm for computing moment-based flood quantile estimates when historical flood information is available, *Water Resources Research*, *33(9)*, 2089-2096

[7] Cohn, T.A., Lane, W.L., and Stedinger, J.R. (2001), Confidence intervals for EMA flood quantile estimates, *Water Resources Research*, *37(6)* 1695-1706

[8] Condie, R. (1977), The log Pearson type 3 distribution: the T-year event and its asymptotic standard error by maximum likelihood theory, *Water Resources Research*, *13* 987-991

[9] Chowdhury, J.U. and Stedinger, J.D. (1991) Confidence intervals for design floods with estimated skew coefficients, *ASCE Journal of Hydraulic Engineering.*, *11* 811-831

[10] England, J.F., Jr., Jarrett, R.D., and Salas, J.D. (2003) Data-based comparisons of moments estimators using historical and paleoflood data, *Journal of Hydrology*, *278(4)*, 172-196

[11] England, J.F., Jr., Salas, J.D., and Jarrett, R.D. (2003) Comparisons of two moments-based estimators that utilize historical and paleoflood data for the log Pearson type III distribution, *Water Resources Research*, *39(9)*, 1243, doi: 10.1029/2002WR001791

[12] Foster, H.A. (1924), Theoretical frequency curves and their applications to engineering problems, *Transactions of the American Society of Civil Engineers*, *87* 142-173; discussion 174-203

[13] Griffis, V.W., and Stedinger, J.R, Evolution of flood frequency analysis with bulletin 17, (submitted *Journal of Hydrology*, July 2005)

[14] Griffis, V.W., Stedinger, J.R., and Cohn, T.A. (2004), LP3 quantile estimators with regional skew information and low outlier adjustments, *Water Resources Research*, *40*, pp.W07503, doi: 1029/2003WR002697

[15] Hall, L.S. (1921), The probable variations in yearly run-off as determined from a study of California streams, *Transactions of the American Society of Civil Engineers*, *84* 191-231; discussion 215-257

[16] Harter, R.H. (1969), A new table of percentage points of the Pearson type III distribution, *Technometrica*, *11* 177-186
10

[17] Harter, R.H. (1971), More percentage points of the Pearson type III distribution, *Technometrica*, *13* 203-204

[18] Hardison, C.H. (1976), Confidence limits for flood frequency curves computed from Pearson type III populations *Journal of Research U.S.G.S.*, *4* 545-547

[19] Hromadka, T.V. et.al.(November 1987), Derivation of a Rainfall-Runoff Model to Compute N-year Floods for Orange County Watersheds, *publication U.S. Army Corp of Engineers, Los Angeles Division*

[20] Interagency Committee on Water Data (1982), Guidelines for determining flood flow frequency, *Bulletin 17B*, 28pp. Hydrol. Subcomm., Washington D.C., March

[21] Committee on Techniques for Estimating Probabilities of Extreme Floods, Water Science and Technology Board, Commission on Physical Sciences, Mathematics, and Resources, Nation Research Council (1988), Estimating Probabilities of Extreme Floods, *National Academy Press*, Washington D.C

[22] Kite, G.W. (1975), Confidence limits for design events, *Water Resources Research*, *11* 48-53

[23] McCuen, R.H. (1979), Map Skew, *Journal of Water Resources, Planning and Management Division, American Society of Civil Engineers (now) Journal of Water Resources*, *105* 265-277 [with Closure, *107*, 582 (1981)]

[24] McCuen, R.H. (2001), Generalized flood skew: map versus watershed skew, *Journal of Hydrologic Engineering*, *64* 293-299

[25] Maidment, D.R., Editor in Chief (1993), Handbook of Hydrology, *McGraw-Hill*, New York

[26] Reis, D.S., Stedinger, J.R., and Martins, E.S. (2005) Bayesian generalized least squares regression with application to log Pearson 3 regional skew estimation, *Water Resources Research*, *41*, *W10419, doi: 10.1029/2004WR003445*

[27] Stedinger, J.R. (1983), Confidence Intervals for Design Events, *Journal of Hydraulic Engineering*, *109* 13-27

[28] Stedinger, J.R. and England, J.R., Jr., Key documents for discussion of evolution of Bulletin 17B and proposed changes (2005), *ftp://ftp.usbr.gov/jengland/HFAWG/*

[29] Tasker, G.D. and Stedinger, J.R. (1986), Estimating Generalized Skew withWeighted Least Squares Regression, *Journal of Water Resources, Planning and Management Division*, *112* 225-237

[30] Thomas, W.O. (1985), A Uniform Technique for Flood-Frequency Analysis, *Journal of Water Resources, Planning and Management Division*, *111* 321-337

[31] Uncertainty of Discharge-Probability Functions (1996). Chapter 4 in Manual No. 1110-2-1619, *Department of the Army, U.S. Army Corps of Engineers*, Washington D.C. (Available online as EM 1110-2-1619 from www.usace.army.mil/inet/usace-docs/eng-manuals)

[32] Whitley, R.J. and Hromadka II, T.V. (1986), Computing Confidence Intervals for Floods I, *Microsoftware for Engineers*, *2*,138-150

[33] Whitley, R.J. and Hromadka II, T.V. (1986), Computing Confidence Intervals for Floods II, *Microsoftware for Engineers*, *2*,151-158

[34] Whitley, R.J. and Hromadka II, T.V. (1997), Chowdhury and Stedinger's approximate confidence intervals for design floods for a single site, *Stochastic Hydrology and Hydraulics*, *11* 51-63

[35] Whitley, R J. and Hromadka II, T.V. (1999), Approximate confidence intervals for design floods for a single site using a neural network, *Water Resources Research*, *35* 203-209

[36] Williamson and Schmid (1989), Investigation of Mitigation Needs for Changes in Duration Floodflows Due to Development, County of Orange publication

Table 1: Confidence Levels for 17B Flood Frequency Curve, N=10

n=10	γ=0.0	γ=0.5	γ=1.0
p=0.0001	56	56	60
p=0.0005	56	56	59
p=0.001	56	56	59
p=0.002	56	56	59
p=0.005	55	56	59
p=0.01	55	55	58
p=0.05	54	54	57
p=0.25	52	52	52
p=0.50	50	49	49
p=0.75	48	47	45
p=0.95	46	44	42
p=0.99	45	44	42
p=0.995	45	43	41
p=0.998	44	43	41
p=0.999	44	43	41
p=0.9995	44	43	41
p=0.9999	44	43	41

Table 2: Confidence Levels for 17B Flood Frequency Curve, N=50

n=50	γ=0.0	γ=0.5	γ=1.0
p=0.0001	52	53	55
p=0.0005	52	53	55
p=0.001	52	53	55
p=0.002	52	53	55
p=0.005	52	53	55
p=0.01	52	53	55
p=0.05	52	52	54
p=0.25	51	51	51
p=0.50	50	50	49
p=0.75	49	49	48
p=0.95	48	48	46
p=0.99	48	47	46
p=0.995	48	47	46
p=0.998	48	47	46
p=0.999	48	47	45
p=0.9995	48	47	45
p=0.9999	48	47	45

Table 3: Confidence Levels from 17B, N=10, *p*=0.99

n=10	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	0.9	8.6	21.6	44.9	71.0	88.5	99.3
γ=0.5	2.7	11.9	23.8	43.6	66.9	85.1	98.8
γ=1.0	4.9	14.6	25.0	41.6	61.9	79.8	97.7

Table 4: Confidence Levels from 17B, N=50, *p*=0.99

n=50	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	1.0	9.4	23.4	47.8	73.4	89.4	99.0
γ=0.5	2.7	13.3	26.5	47.0	69.3	85.1	97.9
γ=1.0	5.6	17.1	28.9	45.9	64.6	79.7	95.3

Table 5: Confidence Levels from 17B, N=10, *p*=0.01

n=10	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	0.7	11.5	29.0	55.1	78.4	91.4	99.1
γ=0.5	0.4	9.4	26.8	55.6	82.0	94.8	99.8
γ=1.0	0.3	8.2	26.1	58.5	86.4	97.4	100.0

Table 6: Confidence Levels from 17B, N=50, *p*=0.01

n=50	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	1.0	10.6	26.7	52.5	76.5	90.6	99.0
γ=0.5	0.5	8.3	24.2	52.6	79.7	93.6	99.7
γ=1.0	0.3	7.4	24.0	54.6	82.4	95.2	99.8

Table 7: Confidence Levels from Simulation, N=10, *p*=0.99

n=10	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	1.00	10.04	25.02	50.03	75.05	90.03	98.99
γ=0.5	1.01	10.05	25.05	50.05	75.08	90.09	99.02
γ=1.0	0.99	9.98	24.98	50.06	75.08	90.08	99.01

Table 8: Confidence Levels from Simulation, N=50, *p*=0.99

n=50	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	1.01	10.04	25.04	50.04	75.04	90.01	98.99
γ=0.5	1.01	10.05	25.06	50.03	75.01	89.99	99.01
γ=1.0	0.99	9.98	24.95	50.04	75.05	89.99	98.99

Table 9: Confidence Levels from Simulation, N=10, *p*=0.01

n=10	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	1.02	10.03	25.01	49.96	75.03	90.02	98.99
γ=0.5	0.98	9.96	24.98	50.10	75.08	90.03	98.98
γ=1.0	1.02	10.04	25.04	50.10	75.06	90.06	99.04

Table 10: Confidence Levels from Simulation, N=50, *p*=0.01

n=50	q=0.01	q=0.10	q=0.25	q=0.50	q=0.75	q=0.90	q=0.99
γ=0.0	1.02	10.04	25.06	50.07	75.05	89.98	99.02
γ=0.5	0.99	10.02	25.02	50.02	74.96	89.96	98.98
γ=1.0	0.99	9.94	25.05	50.08	75.05	90.04	99.02

Table 11: Flood Frequency Curve (17B) cfs vs 50% values, N=10, skew=0.5, sample mean=3.4, sample standard deviation=0.2

T-year	T=2	T=5	T=10	T=25	T=50	T=100	T=200	T=500
17B FFC	2420	3645	4620	6055	7280	8650	10190	12515
50% curve	2425	3720	4770	6335	7690	9220	10955	13600

Table 12: Alhambra Wash (F81D-R) Stream Gage Data

F81D-R ALHAMBRA WASH above Klingerman Street

Season	Daily CFS			Total Runoff	Peak Flow	
	Maximum	Minimum	Mean	(Acre-feet)	Date	CFS
1929-30	N.D.	0	N.D.	635	14-Mar	1,870
1930-31	226	0	2.1	1,480	3-Feb	1,530
1931-32	220	0	2.7	1,940	31-Jan	1,120
1932-33	418	0	2.3	1,680	19-Jan	1,850
1933-34	1,770.00	0	8	5,820	1-Jan	4,890
1934-35	219	0	3.3	2,380	5-Jan	2,280
1935-36	144	0	2	1,420	12-Feb	1,700
1936-37	309	0	5.4	3,880	15-Mar	2,470
1937-38	997	0	7.6	5,520	2-Mar	5,010
1938-39	288	0	4.1	2,990	5-Jan	2,480
1939-40	130	0	2.4	1,730	1-Feb	1,280
1940-41	219	0	7.8	5,650	3-Mar	2,080
1941-42	193	0	2.5	1,810	10-Dec	2,320
1942-43	893	0	8.4	6,070	4-Mar	4,480
1943-44	454	+	5.6	4,100	22-Feb	1,860
1944-45	199	0.1	3.1	2,250	11-Nov	2,220
1945-46	342	0.1	4.1	3,000	22-Dec	1,600
1946-47	345	0.1	5.2	3,800	13-Nov	3,810
1947-48	155	0.1	2.8	2,040	24-Mar	2,670
1948-49	95	0.2	2.8	2,020	17-Dec	758
1949-50	254	0.2	4.3	3,090	6-Feb	1,630
1950-51	106	0.2	3.3	2,360	11-Jan	1,620
1951-52	594	0.2	12.5	9,040	16-Jan	3,810
1952-53	228	0.1	4.5	3,240	15-Nov	3,140
1953-54	369	0.2	5.2	3,770	13-Feb	2,410
1954-55	185	0.2	4.2	3,020	18-Jan	1,890
1955-56	1,100.00	0.3	7.6	5,520	26-Jan	4,550
1956-57	242	0.6	6.1	4,440	23-Feb	3,090
1957-58	544	0.3	12.8	9,270	19-Feb	4,830
1958-59	279	0.2	4.2	3,020	6-Jan	3,170
1959-60	200	0.1	3.8	2,720	11-Jan	1,710
1960-61	153	0.3	2.5	1,790	5-Nov	1,480
1961-62	382	0.1	9.1	6,270	12-Feb	2,560
1962-63	359	0.1	4	2,880	16-Mar	2,210
1963-64	196	0.2	4	2,870	21-Jan	2,210
1964-65	339	0.1	6.4	4,610	9-Apr	3,730
1965-66	686	0.3	10.7	7,740	24-Nov	3,520
1966-67	662	0.4	12.2	8,820	22-Jan	3,550
1967-68	398	0.4	6.5	4,740	8-Mar	3,480
1968-69	999	0.4	17	12,300	6-Feb	3,980
1969-70	486	0.3	5.3	1,871	28-Feb	3,430
1970-71	648	0.4	7.1	2,601	29-Nov	4,040

1971-72	449	0.3	2.5	3,000	24-Dec	2,000
1972-73	555	0.3	12.6	9,110	11-Feb	4,450
1973-74	813	0.3	7.9	5,720	7-Jan	4,330
1974-75	429	0.3	5.6	4,070	4-Dec	6,000
1975-76	274	0.3	5.3	3,790	5-Feb	1,820
1976-77	252	0.3	6	4,340	22-Oct	1,770
1977-78	695	0.3	17	11,927	1-Mar	5,950
1978-79	836	0.3	10.5	7,614	27-Mar	4,484
1979-80	1,240.00	0.3	18.4	13,051	16-Feb	6,660
1980-81	196	0.1	5.1	3,720	19-Mar	2,750
1981-82	371	0.2	6	4,317	17-Mar	2,410
1982-83	1,050.00	0.1	17.8	12,941	1-Mar	7,010
1983-84	235	0.4	3.7	2,715	25-Dec	2,480
1984-85	260	0.3	4.9	3,543	19-Dec	3,050
1985-86	329	0.3	9.2	6,633	8-Mar	4,130
1986-87	177	0.6	3.6	2,579	2-Oct	5,670
1987-88	386	0.6	7	5,048	4-Dec	4,500
1988-89	226	0.9	5.2	3,570	21-Dec	1,410
1989-90	530	0.9	4.8	3,483	17-Feb	2,010
1990-91	452	0.6	7.6	5,437	1-Mar	2,700
1991-92	570	0.7	13.8	10,008	12-Feb	6,340
1992-93	796	1	20.5	14,810	7-Dec	5,880
1993-94	260	0.5	7.1	5,157	24-Mar	3,000
1994-95	875	0.2	14.3	10,380	10-Mar	8,080
1995-96	462	0.4	7	5,071	31-Jan	8,110
1996-97	279	0.3	8.7	6,260	15-Jan	2,640
1997-98	727	0.6	20.2	14,660	6-Feb	7,770
1998-99	142	0.3	6.1	4,400	28-Nov	3,500
1999-00	306	0.4	8.5	6,170	21-Feb	4,480
2000-01	404	0.6	10.2	7,380	11-Jan	3,220
2001-02	325.6	0.8	7.5	5,457.40	24-Nov	6,153
2002-03	881.7	0.5		9,652.40	15-Mar	5,980

M Data Missing
* Record Incomplete
E Estimate
N.D. Not Determined
** Record not Computed
+ Less than 0.05 Acre Feet or Less

http://ladpw.org/wrd/report/0203/runoff/peak.cfm

Table 13: Compton Creek (F37B-R) Stream Gage Data

F 37B-R COMPTON CREEK near Greenleaf Drive

Season	Daily CFS			Total Runoff	Peak Flow	
	Maximum	Minimum	Mean	(Acre-feet)	Date	CFS
1927-28	*	0	*	1230*	5-Mar	240*
1928-29	197	0	3.1	2,270	10-Mar	924
1929-30	144	0	3.5	2,520	14-Mar	580
1930-31	137	+	3.3	2,400	26-Apr	678
1931-32	248	0	4.4	3,220	31-Jan	757
1932-33	166	0	2.4	1,780	19-Jan	740
1933-34	372	0	3.5	2,560	1-Jan	960
1934-35	301	0	5.7	4,170	8-Apr	850
1935-36	143	0	4	2,920	12-Feb	824
1936-37	559	0	*	*	6-Feb	1,220
1937-38	986.0E	*	*	*	2-Mar	N.D.
1938-39	837	0	7.1	5,150	25-Sep	2,150
1939-40	256	10	7.4	5,340	3-Feb	1,630
1940-41	544	1	22.7	16,400	23-Dec	2,660
1941-42	236	3	10.1	7,280	10-Dec	1,730
1942-43	752	0.8	11.8	8,560	22-Jan	2,050
1943-44	739	2.3	15.6	11,290	20-Feb	2,370
1944-45	363	4.4	12.7	9,210	11-Nov	3,010
1945-46	362	2.6	11	7,960	23-Dec	2,010
1946-47	474	4.1	13.9	10,080	12-Nov	2,930
1947-48	170	0.6	7.9	5,740	24-Mar	1,410
1948-49	282	0.1	5.1	3,660	17-Dec	2,710
1949-50	433	+	6.6	4,820	6-Feb	2,830
1950-51	209	+	4.9	3,550	10-Jan	1,790
1951-52	661	0.1	14.7	10,650	18-Jan	3,220E
1952-53	220	0.1	5.6	4,020	15-Nov	2,380
1953-54	797	0.1	7.5	5,410	13-Feb	3,600
1954-55	374	0.1	8.4	6,080	18-Jan	2,710
1955-56	2,090.00	0.2	12.7	9,240	26-Jan	4,910
1956-57	286	+	5.6	4,070	11-May	1,780
1957-58	1,100.00	+	16	11,610	19-Feb	4,640
1958-59	449	0	4.6	3,330	6-Jan	4,320
1959-60	463	0	6.3	4,590	11-Jan	3,220
1960-61	204	+	2.7	1,960	5-Nov	1,640
1961-62	1,060.00	0.1	14.5	10,520	19-Feb	4,550
1962-63	576	+	8.8	6,400	10-Feb	3,310
1963-64	212	+	4.7	3,440	6-Nov	2,430
1964-65	424	0	7.4	5,390	9-Apr	2,630
1965-66	809	+	10.8	7,800	29-Dec	3,250
1966-67	765	+	11.8	8,560	7-Nov	4,650
1967-68	1,120.00	+	9.4	6,850	7-Mar	3,690
1968-69	1,040.00	0	16.6	12,010	20-Jan	5,890

1969-70	275	0.2	4.4	3,150	16-Jan	1,960
1970-71	609	0.4	11.7	8,500	29-Nov	2,930
1971-72	622	0.4	6.8	4,940	27-Dec	6,000
1972-73	473	0.2	12.2	8,830	14-Nov	4,300
1973-74	810	0.3	10	7,210	4-Jan	3,140
1974-75	677	0.2	9.1	6,550	4-Dec	8,690
1975-76	285	0.1	4.6	3,270	9-Feb	2,470
1976-77	542	0	7.2	5,220	17-Aug	1,970
1977-78	688	0	20	14,471	1-Mar	3,620
1978-79	559	+	12.3	8,888	27-Mar	2,410
1979-80	*	*	*	*	16-Feb	4,780
1980-81	440	0.1	6.4	4,658	1-Mar	2,970
1981-82	237	0.3	6.3E	4,647E	1-Jan	2,720
1982-83	1,010.00	0.4	21.9	16,720	28-Jan	6,020
1983-84	277	0.3	5.4	3,893	24-Nov	2,380
1984-85	458	0.1	7.4	5,354	19-Dec	4,110
1985-86	*	*	*	*		*
1986-87	187	0.4	4	2,935	17-Nov	1,670
1987-88	443	0.3	8	5,826	4-Dec	2,980
1988-89	258	0.6	5.9	4,254	21-Dec	1,990
1989-90	755	0.2	5.4	3,887	17-Feb	2,500
1990-91	527	0.5	9.1	6,586	19-Mar	3,940
1991-92	510	0.1	15.5	11,228	20-Mar	4,640
1992-93	717	0.1	21.8	15,760	6-Jan	5,240
1993-94	290	0.2	6	4,315	30-Nov	2,680
1994-95	1,120.00	0	15.8	11,440	4-Jan	7,660
1995-96	627	0.5	8	5,792	31-Jan	3,410
1996-97	402	0.7	10.1	7,300	9-Dec	2,510
1997-98	826	0.7	26.9	19,500	6-Feb	7,040
1998-99	384	0.2	9	6,540	8-Nov	2,420
1999-00	611	0	7.6	5,480	5-Mar	6,150
2000-01	525	0	10.6	7,710	11-Jan	3,250
2001-02	402.8	0	7.1	15,512.20	24-Nov	2,519
2002-03	997.2	0		8,881	15-Mar	4,750

M Data Missing
* Record Incomplete
E Estimate
N.D. Not Determined
** Record not Computed
\+ Less than 0.05 Acre Feet or Less

http://ladpw.org/wrd/report/0203/runoff/peak.cfm

Table 14: Eaton Wash (F318-R) Stream Gage Data

F318-R EATON WASH at Lotus Drive

Season	Daily CFS			Total Runoff	Peak Flow	
	Maximum	Minimum	Mean	(Acre-feet)	Date	CFS
1956-57	201	0	0	2,400	23-Feb	1,760
1957-58	368	0.1	0.1	7,460	19-Feb	2,700
1958-59	245	0.1	0.1	2,850	6-Jan	3,480
1959-60	186	+	+	2,420	12-Jan	1,090
1960-61	123	0.1	0.1	1,590	26-Nov	1,200
1961-62	598	0.1	0.1	6,880	11-Feb	1,950
1962-63	311	0.3	0.3	2,980	9-Feb	1,230
1963-64	227	0.1	0.1	3,050	20-Nov	2,360
1964-65	254	0.2	0.2	3,760	9-Apr	2,150
1965-66	605	0.3	0.3	8,990	29-Dec	2,290
1966-67	548	0.3	0.3	8,670	24-Jan	2,100
1967-68	318	0.3	0.3	4,040	8-Mar	2,390
1968-69	1,860.00	0.3	0.3	M		M
1969-70	M	M	M	M		M
1970-71	M	M	M	M		M
1971-72	M	M	M	M		M
1972-73	M	M	M	M		M
1973-74	592	0.3	0.3	4,870	7-Jan	1,530
1974-75	480	0.5	0.5	4,870	4-Dec	3,000
1975-76	275	0.4	0.4	3,980	11-Sep	2,660
1976-77	206	0.4	0.4	3,650	23-Oct	1,820
1977-78	914	0.4	0.4	21,425	10-Feb	5,810
1978-79	335	0.3	0.3	7,156	21-Feb	2,630
1979-80	1,460.00	0.1	0.1	27,991	16-Feb	5,240
1980-81	203	0.3	0.3	3,937	19-Mar	1,630
1981-82	377	0.4	0.4	5,453	17-Mar	3,060
1982-83	1,570.00	0.5	0.5	28,952		N.D.
1983-84	191	0.4	0.4	3,307	25-Dec	1,930
1984-85	199	0.4	0.4	4,258	19-Dec	2,460
1985-86	313	0.4	0.4	4,827	31-Jan	1,730
1986-87	178	0.1	0.1	1,782	2-Oct	1,400
1987-88	317	0	0	3,048	17-Jan	4,950
1988-89	172	0.1	0.1	2,134	15-Dec	1,150
1989-90	383	0.1	0.1	2,289	17-Apr	1,310
1990-91	331	0	0	3,948	28-Feb	1,850
1991-92	757	0	0	10,304	12-Feb	3,900
1992-93	664	0	0	21,580	7-Dec	5,090
1993-94	159	0	0	2,122	24-Mar	2,580
1994-95	954	0	0	14,500	11-Mar	5,330
1995-96	551	0.1	0.1	5,734	31-Jan	5,090
1996-97	236	0.1	0.1	4,630	12-Jan	1,010
1997-98	1,070.00	0.1	0.1	14,050	23-Feb	4,650

1998-99	136	0.2	0.2	1,990	28-Nov	1,430
1999-00	247	0.1	0.1	3,720	21-Feb	2,490
2000-01	352	0.2	0.2	4,680	11-Jan	1,760
2001-02	236.4	0.2	2.3	1,685.10	24-Nov	4,059
2002-03	557.8	0.1		5,352.60	15-Mar	3,030

M Data Missing
* Record Incomplete
E Estimate
N.D. Not Determined
** Record not Computed
+ Less than 0.05 Acre Feet or Less

http://ladpw.org/wrd/report/0203/runoff/peak.cfm

Table 15: Rubio Wash (F82C-R) Stream Gage Data

F82C-R RUBIO WASH at Glendon Way

Season	Daily CFS			Total Runoff	Peak Flow	
	Maximum	Minimum	Mean	(Acre-feet)	Date	CFS
1930-31	107	0	1.5	1,110	3-Feb	1,690
1931-32	124	0	2.1	1,490	27-Nov	798
1932-33	234	0	1.5	1,110	16-Jan	1,510
1933-34	684	0	3.6	2,580	31-Dec	2,070
1934-35	134	0	2.4	1,770	17-Oct	1,680
1935-36	81	0	1.8	1,280	22-Feb	1,370
1936-37	186	0	3.9	2,800	27-Dec	1,180
1937-38	802	0	5.8	4,180	2-Mar	2,400E
1938-39	250	0	3.3	2,370	5-Jan	1,720
1939-40	122	0	2.4	1,270	7-Jan	1,000
1940-41	200	0	8.1	5,890	3-Mar	1,940
1941-42	130	0	2.1	1,530	10-Dec	1,200
1942-43	697	0	6.2	4,520	4-Mar	2,780
1943-44	393	0	4.4	3,190	22-Feb	1,930
1944-45	152	0	2.1	1,540	11-Nov	1,780
1945-46	244	0	2.5	1,840	22-Dec	1,630
1946-47	233	0	3.2	2,300	13-Nov	2,650
1947-48	91	0	1.5	1,080	24-Mar	2,090
1948-49	59	0	1.5	1,080	30-Oct	530
1949-50	161	0	2.3	1,690	6-Feb	1,060
1950-51	80	0	1.4	1,010	11-Jan	2,290
1951-52	335	0	7.3	5,300	16-Jan	3,020
1952-53	133	0	2	1,460	15-Nov	2,200
1953-54	288	+	3.4	2,490	19-Jan	2,310
1954-55	126	+	2.6	1,870	18-Jan	1,290
1955-56	639	0	4	2,880	26-Jan	1,970
1956-57	199	+	3.2	2,290	23-Feb	2,980
1957-58	286	0.1	7.7	5,610	19-Feb	2,740
1958-59	218	0.2	2.8	2,030	6-Jan	2,780
1959-60	135	0.2	2.5	1,820	11-Jan	985
1960-61	117	0.2	1.8	1,270	6-Nov	902
1961-62	281	0.1	5.7	4,120	20-Jan	1,200
1962-63	246	0.1	2.4	1,760	9-Feb	1,180
1963-64	136	0.2	2.6	1,870	21-Jan	1,570
1964-65	164	0.1	2.8	2,030	9-Apr	2,040
1965-66	466	0.1	6.4	4,650	24-Nov	2,300
1966-67	344	0.2	7.2	5,220	3-Dec	2,040
1967-68	343	0.2	4	2,930	8-Mar	2,460
1968-69	712	0.2	11.4	8,220	25-Jan	2,890
1969-70	**	**	**	**	28-Feb	2,540
1970-71	**	**	**	**	29-Nov	3,700
1971-72	**	**	**	**	24-Dec	1,240

1972-73	410	0	7.0*	5,041*	11-Feb	3,166
1973-74	460	0.2	5.5	3,950	7-Jan	1,985
1974-75	328	0.3	4.5	3,240	4-Dec	3,180
1975-76	373	0.2	4.1	2,920	10-Sep	2,070
1976-77	180	0.1	4.4	3,187	23-Oct	2,610
1977-78	531	0	12.9	9,340	10-Feb	*
1978-79	176	0	8.4	6,056	21-Feb	2,680
1979-80	781	0	11.8	8,372	29-Jan	4,594
1980-81	205	0	4.3	3,108	1-Mar	1,754
1981-82	186	0	4	2,890	17-Mar	1,650
1982-83	620	0.1	12.6	9,079	2-Mar	4,560
1983-84	165	0.1	2.8	1,976	25-Dec	1,680
1984-85	154	0.1	3.5	2,543	19-Dec	1,610
1985-86	212	0.1	6.1	4,445	8-Mar	2,090
1986-87	153	0.2	3.6	2,580	2-Oct	2,790
1987-88	246	0	4.3	3,113	4-Dec	3,620
1988-89	123	0.1	2.9	2,122	15-Dec	783
1989-90	341	0.3	4.5	3,249	16-Jan	1,560
1990-91	355	0	4.9	3,513	1-Mar	1,840
1991-92	287	0	5.7	4,115	12-Feb	2,540
1992-93	323	0	7.9	5,726	14-Jan	3,660
1993-94	105	0	2.3	1,640	24-Mar	1,970
1994-95	707	0	9.4	6,777	11-Mar	4,610
1995-96	656	0	7.5	5,464	31-Jan	5,010
1996-97	156	0	3.9	2,790	15-Jan	1,180
1997-98	438	0	9.1	6,590	6-Feb	4,030
1998-99	79	0	2.2	1,560	28-Nov	2,430
1999-00	218	0.1	5.6	4,030	21-Feb	2,710
2000-01	249	0.6	5.7	4,120	11-Jan	1,670
2001-02	186.3	0.4	3	2,187.30	24-Nov	3,553
2002-03	0	0		0	16-Mar	2,550

M	Data Missing
*	Record Incomplete
E	Estimate
N.D.	Not Determined
**	Record not Computed
+	Less than 0.05 Acre Feet or Less

http://ladpw.org/wrd/report/0203/runoff/peak.cfm

Table 16: Arcadia Wash (F317-R) Stream Gage Data

F317-R ARCADIA WASH below Grand Avenue

Season	Daily CFS			Total Runoff	Peak Flow	
	Maximum	Minimum	Mean	(Acre-feet)	Date	CFS
1956-57	108	0.1	1.8	1,340	23-Feb	1,184
1957-58	212	0.1	4.6	3,330	1-Feb	1,932
1958-59	127	0.2	1.9	1,360	6-Jan	1,270
1959-60	101	0.3	1.7	1,220	27-Apr	593
1960-61	69	+	1.1	831	5-Nov	570
1961-62	408	0.1	4.7	3,400	11-Feb	1,480
1962-63	153	0.2	2.1	1,510	9-Feb	600
1963-64	120	0.1	2.2	1,620	20-Nov	1,340
1964-65	153	0.1	3.1	2,270	9-Apr	1,460
1965-66	267	0.1	4.7	3,430	29-Dec	1,270
1966-67	283	0.3	6.3	4,560	22-Jan	1,260
1967-68	M	M	M	M		M
1968-69	M	M	M	M		M
1969-70	M	M	M	M		M
1970-71	M	M	M	M		M
1971-72	M	M	M	M		M
1972-73	M	M	M	M		M
1973-74	279	0.3	4	2,910	7-Jan	931
1974-75	207	0.3	3.2	2,290	4-Dec	2,560
1975-76	167	0.3	3.6	2,600	11-Sep	1,400
1976-77	119	0.2	2.9	2,121	23-Oct	1,320
1977-78	355	0.2	9.4	6,823	10-Feb	4,110
1978-79	128	0.2	4.5	3,263	27-Mar	1,290
1979-80	633	0	9.9	7,025	29-Jan	3,280
1980-81	104	0.4	2.8	1,991	29-Jan	1,050
1981-82	208	0.4	4.3	3,137	17-Mar	2,470
1982-83	435	0.4	10.8	7,824	1-Mar	4,110
1983-84	121	0	3.2	2,354	1-Oct	1,430
1984-85	137	0.1	4.7	3,399	19-Dec	1,420
1985-86	211	0	8.4	6,116	8-Mar	1,760
1986-87	172	0.1	3.5	2,530	2-Oct	2,410
1987-88	284	0.1	5.4	3,915	17-Jan	4,360
1988-89	114	0.1	3.7	2,521	21-Dec	507
1989-90	728	0.1	3.5	2,505	17-Apr	1,330
1990-91	228	0.1	5	3,598	28-Feb	2,120
1991-92	301	0.1	11.1	8,043	12-Feb	3,190
1992-93	586	0.3	17.3	12,560	17-Jan	2,720
1993-94	239	0	6.4	4,661	19-Mar	1,360
1994-95	480	0.1	11.1	8,032	11-Mar	2,740
1995-96	405	0.4	5.2	3,764	20-Feb	1,560
1996-97	206	0.5	6.3	4,540	26-Jan	1,430
1997-98	489	0.6	13.3	9,640	6-Feb	2,850

1998-99	151	0.5	4.2	3,020	26-Jan	1,040
1999-00	162	0.1	4.3	3,150	21-Feb	1,750
2000-01	240	0.2	6	4,320	11-Jan	1,380
2001-02	161.3	0.6	2.6	1,904.00	24-Nov	2,712
2002-03	0	0		0	15-Mar	2,120

M Data Missing
* Record Incomplete
E Estimate
N.D. Not Determined
** Record not Computed
+ Less than 0.05 Acre Feet or Less

http://ladpw.org/wrd/report/0203/runoff/peak.cfm

Table 17: Hydrologic Stream Gage Information

	Area (sq.mi)	Length of longest watercourse (mi)	Slope (ft/mi)	Time of Concentration (hrs)	Lag (hrs)
Alhambra Wash	13.67	8.62	82.40	0.89	0.62
Compton Creek	24.76	12.69	13.80	2.22	0.94
Eaton Wash	11.02	8.14	90.90	1.05	0.54
Rubio Wash	12.20	9.47	125.70	0.68	0.63
Arcadia Wash	7.70	5.87	156.70	0.60	0.41

Table 18: Statistical Parameters

Alhambra

	1929-2003	1929-1983
N	74	54
mean	3.502	3.462
standard deviation	0.213	0.199
skew	-0.089	-0.133
adopted skew*	0.2	0.2
watershed area (square miles)	15.2	15.2

Compton

	1927-2003	1927-1984
N	74	56
mean	3.46	3.432
standard deviation	0.216	0.218
skew	-0.436	-0.522
adopted skew*	0.1	0.1
watershed area (square miles)	22.6	22.6

Eaton

	1956-2003	1956-1984
N	47	28
mean	3.386	3.375
standard deviation	0.213	0.193
skew	0.105	0.225
adopted skew*	0.2	0.2
watershed area (square miles)	22.8	22.8

Rubio

	1930-2003	1930-1983
N	74	54
mean	3.318	3.299
standard deviation	0.193	0.187
skew	-0.461	-0.599
adopted skew*	-0.2	-0.2
watershed area (square miles)	10.9	10.9

Arcadia

	1956-2003	1956-1984
N	47	28
mean	3.207	3.167
standard deviation	0.22	0.22
skew	-0.187	0.146
adopted skew*	0.1	0.1
watershed area (square miles)	8.5	8.5

*** adopted skew found in *Derivation of a Rainfall Runoff Model to Compute N-year Floods for Orange County Watersheds, November 1987***

Table 19: Flow Rate Results

Extended Record

Alhambra	50%	65%	85%	95%
T=2	3126	3196	3318	3440
T=5	4783	4917	5166	5425
T=10	6031	6234	6641	7018
T=25	7782	8097	8697	9347
T=50	9210	9629	10433	11315
T=100	10747	11288	12333	13490

Compton	50%	65%	85%	95%
T=2	2861	2925	3039	3151
T=5	4352	4471	4689	4916
T=10	5453	5629	5956	6305
T=25	6960	7228	7736	8288
T=50	8164	8515	9188	9922
T=100	9438	9885	10744	11692

Eaton	50%	65%	85%	95%
T=2	2394	2461	2581	2702
T=5	3666	3797	4045	4312
T=10	4626	4824	5206	5629
T=25	5973	6282	6890	7579
T=50	7073	7485	8301	9242
T=100	8258	8789	9855	11098

Rubio	50%	65%	85%	95%
T=2	2110	2153	2226	2298
T=5	3038	3107	3232	3362
T=10	3645	3740	3917	4103
T=25	4401	4535	4789	5059
T=50	4954	5121	5438	5781
T=100	5499	5702	6086	6503

Arcadia	50%	65%	85%	95%
T=2	1597	1644	1726	1810
T=5	2467	2556	2723	2903
T=10	3113	3245	3500	3782
T=25	4003	4207	4608	5062
T=50	4720	4989	5523	6138
T=100	5481	5824	6513	7316

Unextended Record

Alhambra	50%	65%	85%	95%
T=2	2854	2924	3048	3171
T=5	4249	4381	4627	4889
T=10	5279	5475	5847	6253
T=25	6701	7001	7581	8227
T=50	7846	8240	9012	9880
T=100	9066	9570	10562	11693

Compton	50%	65%	85%	95%
T=2	2682	2752	2877	3002
T=5	4124	4258	4507	4772
T=10	5191	5389	5768	6179
T=25	6658	6964	7555	8214
T=50	7836	8238	9026	9913
T=100	9085	9598	10611	11763

Eaton	50%	65%	85%	95%
T=2	2337	2415	2556	2702
T=5	3446	3593	3880	4204
T=10	4259	4478	4916	5427
T=25	5375	5710	6397	7222
T=50	6271	6711	7627	8750
T=100	7222	7783	8963	10436

Rubio	50%	65%	85%	95%
T=2	2019	2065	2145	2223
T=5	2869	2943	3078	3221
T=10	3421	3522	3713	3917
T=25	4104	4246	4518	4815
T=50	4601	4777	5116	5491
T=100	5089	5301	5711	6169

Arcadia	50%	65%	85%	95%
T=2	1457	1512	1613	1717
T=5	2255	2361	2570	2809
T=10	2848	3008	3331	3714
T=25	3668	3917	4433	5060
T=50	4328	4658	5349	6209
T=100	5030	5453	6350	7486

Table 20: Change in Confidence Level Estimates $(\alpha_{x,T,g})$ Due to Added Years of Records

Alhambra

	50%	65%	85%	95%
T=2	1.10	1.09	1.09	1.08
T=5	1.13	1.12	1.12	1.11
T=10	1.14	1.14	1.14	1.12
T=25	1.16	1.16	1.15	1.14
T=50	1.17	1.17	1.16	1.15
T=100	1.19	1.18	1.17	1.15

Compton

	50%	65%	85%	95%
T=2	1.07	1.06	1.06	1.05
T=5	1.06	1.05	1.04	1.03
T=10	1.05	1.04	1.03	1.02
T=25	1.05	1.04	1.02	1.01
T=50	1.04	1.03	1.02	1.00
T=100	1.04	1.03	1.01	0.99

Eaton

	50%	65%	85%	95%
T=2	1.02	1.02	1.01	1.00
T=5	1.06	1.06	1.04	1.03
T=10	1.09	1.08	1.06	1.04
T=25	1.11	1.10	1.08	1.05
T=50	1.13	1.12	1.09	1.06
T=100	1.14	1.13	1.10	1.06

Rubio

	50%	65%	85%	95%
T=2	1.05	1.04	1.04	1.03
T=5	1.06	1.06	1.05	1.04
T=10	1.07	1.06	1.06	1.05
T=25	1.07	1.07	1.06	1.05
T=50	1.08	1.07	1.06	1.05
T=100	1.08	1.08	1.07	1.05

Arcadia

	50%	65%	85%	95%
T=2	1.10	1.09	1.07	1.05
T=5	1.09	1.08	1.06	1.03
T=10	1.09	1.08	1.05	1.02
T=25	1.09	1.07	1.04	1.00
T=50	1.09	1.07	1.03	0.99
T=100	1.09	1.07	1.03	0.98

Note: $\alpha_{x,T,g}$ = Ratio of Peak Flow Rate estimate at confidence level *x*, return frequency *T*, gage *g* for the extended record versus the unextended record

Table 21: Weighted Average of $\alpha_{x,T,g}$ Values

Weighted Average (watershed size)

	50%	65%	85%	95%
T=2	1.06	1.06	1.05	1.04
T=5	1.08	1.07	1.06	1.05
T=10	1.08	1.08	1.06	1.05
T=25	1.09	1.09	1.07	1.05
T=50	1.10	1.09	1.07	1.05
T=100	1.11	1.10	1.08	1.05

$$\frac{\sum_g \alpha_{x,T,g} A_g}{\sum_g A_g}$$

Weighted Average (years of record)

	50%	65%	85%	95%
T=2	1.07	1.06	1.05	1.05
T=5	1.08	1.07	1.06	1.05
T=10	1.09	1.08	1.07	1.05
T=25	1.10	1.09	1.07	1.05
T=50	1.10	1.09	1.07	1.05
T=100	1.11	1.10	1.08	1.05

$$\frac{\sum_g \alpha_{x,T,g} N_g}{\sum_g N_g}$$

Notes:

1. A_g **=area of watershed,** ***g***
2. N_g **=number of years of record for gage,** ***g***

Table 22: Relationship Between Confidence Level Estimates versus 50% Confidence Level Estimate $(\beta_{x,T,g})$

Extended Record $(\beta_{x,T,g})$ Values

Alhambra	50%	65%	85%	95%
T=2	1.00	1.02	1.06	1.10
T=5	1.00	1.03	1.08	1.13
T=10	1.00	1.03	1.10	1.16
T=25	1.00	1.04	1.12	1.20
T=50	1.00	1.05	1.13	1.23
T=100	1.00	1.05	1.15	1.26

Compton	50%	65%	85%	95%
T=2	1.00	1.02	1.06	1.10
T=5	1.00	1.03	1.08	1.13
T=10	1.00	1.03	1.09	1.16
T=25	1.00	1.04	1.11	1.19
T=50	1.00	1.04	1.13	1.22
T=100	1.00	1.05	1.14	1.24

Eaton	50%	65%	85%	95%
T=2	1.00	1.03	1.08	1.13
T=5	1.00	1.04	1.10	1.18
T=10	1.00	1.04	1.13	1.22
T=25	1.00	1.05	1.15	1.27
T=50	1.00	1.06	1.17	1.31
T=100	1.00	1.06	1.19	1.34

Rubio	50%	65%	85%	95%
T=2	1.00	1.02	1.05	1.09
T=5	1.00	1.02	1.06	1.11
T=10	1.00	1.03	1.07	1.13
T=25	1.00	1.03	1.09	1.15
T=50	1.00	1.03	1.10	1.17
T=100	1.00	1.04	1.11	1.18

Arcadia	50%	65%	85%	95%
T=2	1.00	1.03	1.08	1.13
T=5	1.00	1.04	1.10	1.18
T=10	1.00	1.04	1.12	1.21
T=25	1.00	1.05	1.15	1.26
T=50	1.00	1.06	1.17	1.30
T=100	1.00	1.06	1.19	1.33

Unextended Record $(\beta_{x,T,g})$ Values

Alhambra	50%	65%	85%	95%
T=2	1.00	1.02	1.07	1.11
T=5	1.00	1.03	1.09	1.15
T=10	1.00	1.04	1.11	1.18
T=25	1.00	1.04	1.13	1.23
T=50	1.00	1.05	1.15	1.26
T=100	1.00	1.06	1.17	1.29

Compton	50%	65%	85%	95%
T=2	1.00	1.03	1.07	1.12
T=5	1.00	1.03	1.09	1.16
T=10	1.00	1.04	1.11	1.19
T=25	1.00	1.05	1.13	1.23
T=50	1.00	1.05	1.15	1.26
T=100	1.00	1.06	1.17	1.29

Eaton	50%	65%	85%	95%
T=2	1.00	1.03	1.09	1.16
T=5	1.00	1.04	1.13	1.22
T=10	1.00	1.05	1.15	1.27
T=25	1.00	1.06	1.19	1.34
T=50	1.00	1.07	1.22	1.40
T=100	1.00	1.08	1.24	1.44

Rubio	50%	65%	85%	95%
T=2	1.00	1.02	1.06	1.10
T=5	1.00	1.03	1.07	1.12
T=10	1.00	1.03	1.09	1.14
T=25	1.00	1.03	1.10	1.17
T=50	1.00	1.04	1.11	1.19
T=100	1.00	1.04	1.12	1.21

Arcadia	50%	65%	85%	95%
T=2	1.00	1.04	1.11	1.18
T=5	1.00	1.05	1.14	1.25
T=10	1.00	1.06	1.17	1.30
T=25	1.00	1.07	1.21	1.38
T=50	1.00	1.08	1.24	1.43
T=100	1.00	1.08	1.26	1.49

Note: $(\beta_{x,T,g})$ **= (*x* confidence level estimate / 50% confidence level estimate) at return frequency *T*, for gage *g***

Table 23: Weighted Average of $(\beta_{x,T,g})$ Values

Extended Record

Weighted Average (watershed size)

	50%	65%	85%	95%
T=2	1.0000	1.0245	1.0677	1.1107
T=5	1.0000	1.0301	1.0862	1.1456
T=10	1.0000	1.0358	1.1044	1.1769
T=25	1.0000	1.0429	1.1257	1.2173
T=50	1.0000	1.0480	1.1415	1.2463
T=100	1.0000	1.0529	1.1568	1.2744

$$\frac{\sum_g \beta_{x,T,g} A_g}{\sum_g A_g}$$

Weighted Average (years of record)

	50%	65%	85%	95%
T=2	1.0000	1.0237	1.0655	1.1070
T=5	1.0000	1.0289	1.0826	1.1393
T=10	1.0000	1.0342	1.0999	1.1685
T=25	1.0000	1.0409	1.1196	1.2062
T=50	1.0000	1.0458	1.1345	1.2333
T=100	1.0000	1.0504	1.1487	1.2594

$$\frac{\sum_g \beta_{x,T,g} N_g}{\sum_g N_g}$$

Unextended Record

Weighted Average (watershed size)

	50%	65%	85%	95%
T=2	1.0000	1.0287	1.0800	1.1321
T=5	1.0000	1.0357	1.1039	1.1785
T=10	1.0000	1.0425	1.1254	1.2190
T=25	1.0000	1.0511	1.1531	1.2711
T=50	1.0000	1.0573	1.1731	1.3093
T=100	1.0000	1.0632	1.1920	1.3459

$$\frac{\sum_g \beta_{x,T,g} A_g}{\sum_g A_g}$$

Weighted Average (years of record)

	50%	65%	85%	95%
T=2	1.0000	1.0278	1.0773	1.1274
T=5	1.0000	1.0342	1.0992	1.1700
T=10	1.0000	1.0406	1.1193	1.2077
T=25	1.0000	1.0487	1.1452	1.2562
T=50	1.0000	1.0545	1.1638	1.2915
T=100	1.0000	1.0600	1.1815	1.3254

$$\frac{\sum_g \beta_{x,T,g} N_g}{\sum_g N_g}$$

Notes:

1. A_g=area of watershed, g
2. N_g=number of years of record for gage, g

Table 24: Difference in Weighted Averages of Values: (Extended Record) – (Unextended Record)

Weighting by Watershed Area

	50%	65%	85%	95%
T=2	0.0000	-0.0043	-0.0123	-0.0214
T=5	0.0000	-0.0056	-0.0177	-0.0329
T=10	0.0000	-0.0067	-0.0210	-0.0421
T=25	0.0000	-0.0082	-0.0274	-0.0539
T=50	0.0000	-0.0093	-0.0315	-0.0630
T=100	0.0000	-0.0103	-0.0353	-0.0715

Weighting by Years of Record

	50%	65%	85%	95%
T=2	0.0000	-0.0041	-0.0118	-0.0204
T=5	0.0000	-0.0053	-0.0166	-0.0308
T=10	0.0000	-0.0063	-0.0195	-0.0392
T=25	0.0000	-0.0078	-0.0256	-0.0500
T=50	0.0000	-0.0087	-0.0293	-0.0581
T=100	0.0000	-0.0096	-0.0328	-0.0660

Table 25: Model Results Based of the San Diego Workbook Example 3.2

Model	**A**	**B**	**C**
Area (sq mi)	40	40	40
SCS CN	85		
Fp (in/hr)		***0.29***	***0.29***
Low loss fraction		***0.303***	***0.303***
Return Frequency	100-year	100-year	100-year
P_6 (in)	3.0	3.0	3.0
P_{24} (in)	5.5	5.5	5.5
Lag (hr)	1.74	1.74	1.74
S-graph	SCS	***Urban***	***Urban***
	SD	SD	Sierra
Depth-Area Factor	Manual	Manual	Madre
5-m	0.730	0.730	***0.475***
30-m	0.730	0.730	***0.510***
1-hr	0.830	0.830	***0.053***
3-hr	0.915	0.915	***0.865***
6-hr	0.940	0.940	***0.940***
24-hr	0.958	0.958	***0.963***
Peak Runoff (cfs)	18,544	16,471	13,027
Total Volume (af)	7,726	8,121	7,983

Note: Values are different from Model A