DEVELOPMENT OF A POLICY

FOR FLOOD CONTROL DESIGN AND PLANNING

T.V. Hromadka II[1], R.H. McCuen[2], R.J. Whitley[3], C.C. Yen[4]

Abstract

For southern California watersheds, as is the case for most watersheds in the United States, rainfall-runoff data are relatively sparse such that the calibration of a hydrologic model is uncertain. With the large number and types of hydrologic models currently available, the choice of the "best" hydrologic model to use is not clear. Because of the limited data, the hydrologic model must be simple in order to validate parameter values and submodel algorithms. Due to the uncertainty in stream gage data frequency analysis, a level of confidence (e.g. 85%) should be chosen to provide a level of protection against a specified flood return frequency (e.g. 100-year). Due to the calibrated model range and distribution of possible outcomes caused by the uncertainty in modeling parameter values, the use of a regionally calibrated model at an ungaged catchment needs to address the probability that the hydrologic model estimate of flood quantities (e.g. peak flow rates) achieves the level of protection for a specified flood level. In this paper, a design storm unit hydrograph model is developed and calibrated with respect to model parameter values and with respect to runoff frequency tendencies (design storm) in order to address each of these issues.

1 Director of Water Resources, Williamson and Schmid, Irvine, CA, and Research Associate, Princeton University, New Jersey

2 Professor, Department of Civil Engineering, University of Maryland, College Park, MD

3 Professor of Mathematics, University of California, Irvine, CA

4 Hydrologist, Williamson and Schmid, Irvine, CA

INTRODUCTION

Fundamental to the preparation of a policy statement for flood control are decisions related to the following: (1) the specific hydrologic model to adopt; (2) the selection of calibration data sets; (3) the selection of a desired level of flood protection (e.g., 100-year flood, or other); and (4) the selection of a level of certainty in achieving the level of flood protection. Each of these decision points, must be considered in formulating a policy. This paper provides a discussion of each of the points and the factors that were considered in formulating a flood control and drainage policy for a county in southern California.

In the selection of the hydrologic model, the need for both runoff peak flow rates and runoff volumes (for the testing of detention basins) require the selection of a model that produces a runoff hydrograph. The U.S. Army Corps of Engineers (COE) (1980) categorizes all hydrologic models into eight groupings of which three develop a runoff hydrograph; namely, single event (design storm), multiple discrete events, and continuous records (continuous simulation). These models can be further classified according to the submodels employed. For example, a unit hydrograph or a kinematic wave model may be used to represent the catchment hydraulics in a design storm model.

In a survey of hydrologic model usage by Federal and State governmental agencies and private engineering firms (U.S. Department of Transportation, Federal Highway Admin., Hydraulic Engineering Circular No. 19, October 1984), it was found that "practically no use is made of watershed models for discrete event and continuous hydrograph simulation."

In comparison, however, design storm methods were used from 24 to 34 times more frequently than the complex models by Federal agencies and the private sector, respectively. The frequent use of design storm methods appear to be due to several reasons: (1) design storm methods are considerably simpler to use than discrete event and continuous simulation models; (2) it has not been established, in general, that the more complex models provide an improvement in computational accuracy over design storm models; and (3) the level of complexity typically embodied in the continuous simulation class of models does not appear to be appropriate for the catchment rainfall-runoff data that are typically available. Consequently, the design storm approach is most often selected for flood control and drainage policies. For the choice of design storm to be used, the work of Beard and Chang (1979) and HEC (1975) provide fundamental reasons for developing a design storm using rainfalls of identical return frequency, adjusted for watershed area effects.

The next decision is whether to use the standard unit hydrograph method or a more recently advanced method, such as the kinematic wave method, to model catchment hydraulics. Again, it has not been clearly established that the kinematic wave approach (e.g., the overland flow plane concept) provides an improvement in modeling accuracy over the unit hydrograph approach that has been calibrated to local rainfall-runoff data. This is especially true for regions where both very small and large watersheds must be studied.

Finally, specific components of the design storm/unit hydrograph approach must be selected and specified in the policy statement; the components include the design storm, the loss rate function, catchment lag relationships, and unit hydrograph or S-graph development. Inherent in the choice of submodels is the ability to calibrate the model at two levels: (1) calibration of model parameters to represent local or regional catchment rainfall-runoff characteristics, and (2) calibration of the design storm to represent local rainfall intensity-duration-frequency characteristics. Beard and Chang (1979) note that in a hydrologic model, the number of calibration parameters should be as small as possible in order to correlate model parameters with basin characteristics. They also state that a regional study should be prepared to establish the loss rate and unit hydrograph characteristics, "and to compute from balanced storms of selected frequencies (storms having the same rainfall frequency for all durations) the resulting floods."

To calibrate the peak flow rates, flood frequency curves must be developed. Additionally, by the selection of a desired level of confidence in achieving the level of flood protection, such as 85 percent confidence in protecting for the T-year flood, confidence limits must be computed. Since the Water Resources Council Bulletin 17A and 17B procedures do not always achieve the desired confidence limits (Stedinger, 1983a, 1983b), a simulation procedure is used herein to determine the 85 percent confidence limits that are capable of stating limits below which individual flood protection can be provided. In this fashion, model uncertainty in appli-lications at ungaged catchments can be evaluated with respect to the chosen level of confidence in achieving flood protection at gaged sites.

CHOICE OF WATERSHED MODEL

In developing a flood control and drainage policy, the first, and possibly the most important question to answer is: What type of model should be used to form the basis for design calculations? To answer this question, the literature was reviewed extensively. Based on the research findings summarized in the following paragraphs, the design storm/unit hydrograph was selected because it appears to combine the accuracy needed with the simplicity that is necessary for practical reasons.

The literature contains several reports of problems in using complex models, especially in parameter optimization. Additionally, it has not been clearly established whether complex models, such as in the continuous simulation or discrete event classes of models, provide an increase in accuracy over a standard design storm unit hydrograph model.

There are only a few papers and reports in the literature that provide a comparison in hydrologic model performance. From these references, it appears that a simple unit hydrograph model provides as good as or better results than quasi-physically based (or QPB) or complex models (Loague and Freeze, 1985).

Schilling and Fuchs (1986) write "that the spatial resolution of rain data input is of paramount importance to the accuracy of the simulated hydrograph" due to "the high spatial variability of storms" and "the amplification of rainfall sampling errors by the nonlinear transformation" of rainfall into runoff. Their recommendations are that a model should employ a simplified surface flow model if there are many subbasins; a simple runoff coefficient loss rate; and a diffusion (zero inertia) or storage channel routing technique. Horberger, et al. (1985) writes that "Even the most physically based models...cannot reflect the true complexity and heterogeneity of the processes occurring in the field. Catchment hydrology is still very much an empirical science." In attempting to define the modeling processes by the available field data forms, Hornberger, et al. find that "Hydrological quantities measured in the field tend to be either integral variables (e.g., stream discharge, which reflects an integrated catchment response) or point estimates of variables that are likely to exhibit marked spatial and/or temporal variation (e.g., soil hydraulic conductivity)." Hence, the precise definition of the physics in a modeling sense becomes a problem that is "poorly posed in the mathematical sense." Typically, the submodel parameters cannot be estimated precisely due to the large associated estimation error. "Such difficulties often indicate that the structural complexity of the model is greater than is warranted on the basis of the calibration data set."

In a similar vein, Beard and Chang (1979) write that in their study of 14 urban catchments, complex models such as continuous simulation typically have 20 to 40 parameters and functions that must be derived from recorded rainfall-runoff data. "In as much as rainfall data are for scattered point locations and storm rainfall is highly variable in time and space, available data are generally inadequate in this region for reliably calibrating the various interrelated functions of these complex models." Additionally, "changes in the model that would result from urbanization could not be reliabily determined." They write that the application "of these complex models to evaluating changes in flood frequencies usually requires simulation of about 50 years of streamflow at each location under each alternative watershed condition."

The introduction of a paper by Sorooshian and Gupta (1983) provides a brief review of some of the problems reported by other researchers in attempting to find a "true optimum" parameter set for complex models, including the unsuccessful two man-year effort by Johnston and Pilgrim (1976) to optimize parameters for a version of the Boughton model.

In the extensive study by Loague and Freeze (1985), three event-based rainfall-runoff models (a regression model, a unit hydrograph model, and a kinematic wave quasi-physically based model) were used on three data sets of 269 events from three small upland catchments. In that paper, the term "quasi-physically based" is used for the kinematic wave model. The three catchments were 25 acres, 2.8 mi^2, and 35 acres in size, and were extensively monitored with rain gage, stream gage, neutron probe, and soil parameter site testing. For example, the 25 acre site contained 35 neutron probe access sites, 26 soil parameter sites (all equally spaces), an on-site rain gage, and a stream gage. The QPB model utilized 22 overland flow planes and four channel segments. In comparative tests between the three modeling approaches to measured rainfall-runoff data, it was concluded that all models performed poorly and that the QPB performance was only slightly improved by calibration of its most sensitive parameter, hydraulic conductivity. They write that the "conclusion one is forced to draw... is that the QPB model does not represent reality very well; in other words, there is considerable model error present. We suspect this is the case with most, if not all conceptual models currently in use." Additionally, they state, "the fact that simpler, less data intensive models provided as good or better predictions than a QPB is food for thought."

It is noteworthy to consider the results of a study (HEC, 1979) where the Hydrocomp HSP continuous simulation model was applied to the West Branch of the Dupage River in Illinois. "It took one person six months to assemble and analyze additional data, and to learn how to use the model. Another six months were spent in calibration and long-record simulation." It was concluded that "Discharge frequency under changing urban conditions is a problem that could be handled by simpler, quicker, less costly approaches requiring much less data, e.g., design storms or several historical events used as input to a single-event model, or a continuous model with a less complex soil-moisture accounting algorithm."

The complex model parameter optimization problem has not been resolved. For example, Gupta and Sarooshian (1983) write that "even when calibrated under ideal conditions (simulation studies), it is often impossible to obtain unique estimates for the parameters." Troutman (1982) also discussed the often cited difficulties with the error in precipitation measurements "due to the spatial variability of precipitation." This source of error can result in "serious errors in runoff prediction and large biases in parameter estimates by calibration of the model."

Because it is still not clear whether there is a significant advantage in using a watershed model that is more complex or physically based than a design storm unit hydrograph approach, the design storm unit hydrograph method is proposed for use in the flood control runoff hydrograph model.

NONLINEAR KINEMATIC WAVE METHOD VERSUS LINEAR UNIT HYDROGRAPH METHOD

The dominant method used in runoff hydrograph development for representing catchment runoff response is the unit hydrograph (UH). The kinematic wave overland flowplane concept (KW) is an alternative to the UH approach. HEC (1982) provides a description and comparison of these two alternatives. The relative usage of KW by 1983 was indicated by Cermak and Feldman (1983) who write that "actual applications by Corps field offices have been few to nonexistent. Even at HEC the KW approach has not been utilized in any special assistance projects."

Use of KW implies a non-linear response, whereas the UH implies a linear response. Nash and Sutcliffe (1970) write that "the UH assumption of a linear time invariant relationship cannot be tested because neither the input (effective rainfall) nor output (storm runoff) are unequivocally defined." Although the watershed response is often considered to be mathematically nonlinear, the nonlinearity of the total watershed response has not been shown to be exactly described as a KW. Indeed, a diffusion hydrodynamic model, DHM (Hromadka and Yen, 1986), provides another nonlinear watershed response that includes an additional term in the governing St. Venant flow equations and that may differ significantly in response from a KW model (e.g., overland flow planes with KW channel routing). There are an infinity of nonlinear mathematical representations possible as a combination of surface runoff and channel routing analogs; therefore, merely claiming that the response of a watershed model can be classified as 'nonlinear' is not a proof that the model represents the true response of the catchment.

Given that the KW analog is only used to obtain an approximation to the catchment response, the KW approach does not appear to provide significantly better computational results (for floods of interest in flood control design and planning) than the commonly used UH method. Dickinson et al. (1967) noted that "in the range of discharges normally considered as flood hydrographs, the time [of concentration] remained virtually constant. In other words, in the range of flood interest, the nonlinear effect approached linearity." An explanation was advanced that "at low discharges, the mean velocity may vary considerably with discharge. However, for higher discharges contained within banks, the mean velocity in the channel remains approximately constant."

In actual travel time measurements of flows from a 96-acre catchment using a radioactive tracing technique, Pilgrim (1976) noted that although the flood runoff process "is grossly nonlinear at low flows, linearity is approximated at high flows." Pilgrim also stated that "simple nonlinear models fitted by data from events covering the whole range of flow may give gross errors when used to estimate large events." It is noted that overbank flow was one of the factors for linearity in this study.

In HEC (1978), six models, plus two variants of one of these models and a variant of another, were calibrated and tested on a 5.5 mi^2 urban catchment in Castra Valley near Oakland, Calibornia. Both single event and continuous simulation models based on both UH and KW techniques were used in the test. The study concluded that for this watershed "the more complex models did not produce better results than the simple models...." An examination of the test results between the KW and HEC-1 UH models did not show a clear difference between the methods.

It is of interest that Singh (1977) concluded, "if one is not very confident in estimates of watershed infiltration, then in some circumstances linear models may have an advantage over nonlinear models in runoff peak predictions because they do not amplify the input errors." That is, the uncertainty in effective rainfall quantities may be magnified by a nonlinear model; consequently, there is an advantage in using a linear model when there are errors in the loss rate and precipitation estimates.

Because it is not evident whether the nonlinear KW method for modeling surface runoff provides an improvement in accuracy over the linear UH based hydrologic models, the UH model is proposed for use with a design storm. The UH approach is simpler to apply, and there is less chance that the UH approach will be incorrectly applied.

Design Storms

Beard (1975) provides an in-depth study of the use of design runoff hydrographs for flood control studies. "Hypothetical floods consists of hydrographs of artificial flood flows...that can be used as a basis for flood-control planning, design, and operation decisions or evaluations." Beard (1975) notes that the balanced storm concept is an important argument for not using a historic storm pattern or sequence of storm patterns (e.g., continuous simulation or discrete event modeling) as "No one historical flood would ordinarily be representative of the same severity of peak flow and runoff volumes for all durations of interest." Indeed, should a continuous simulation study be proposed such that the "project is designed to regulate all floods of record, it is likely that one flood will dictate the type of project and its general features, because the largest flood for peak flows is also usually the largest-volume flood." Hence, a continuous simulation model of say 40 years of data can be thought of as a 40 year duration design storm with its own probability of re-occurrence, which typically reduces for modeling purposes to simply a single or double day storm pattern.

RAINFALL TO RUNOFF FREQUENCY RELATIONSHIPS

The association between return frequency of rainfalls and the return frequency of runoff is not clear. However, some studies have been reported in the literature that suggest relating the two frequency curves. Rose and Hwang (1985) used a rainfall-based frequency curve design storm pattern with the HEC-1 Flood Hydrograph package and developed an "Equivalent Frequency" to relate rainfall to runoff frequencies. Bell (1968) showed in a plot of return period of gross rainfall to return period of flood peak, approximately "the same number of points fall on each side of the 45° line for the full range of values, indicating that, on the average, the same return period applies to both rainfall and associated floods. The average 100-year flood, for example, corresponds with the average 100-year rainfall for the watersheds considered."

The above statement does not apply to all watersheds or regions, but a study of the Bell's results (1968) do provide a general study that can be compared with the results from the design storm model calibration effort used for the Los Angeles, California, area, where a direct relationship between rainfall and runoff frequency curves appears reasonable.

MODEL SELECTION

Of the eight model types and over 100 specific models available, a design storm/unit hydrograph model is selected for this particular application. Some of the reasons are as follows: (1) the design storm approach--the multiple discrete event and continuous simulation categories of models have not been clearly established to provide better predictions of flood flow frequency estimates for evaluating the impact of urbanization and for design flood control systems than a calibrated design storm model; (2) the unit hydrograph method--it has not been shown that the kinematic wave modeling technique or other QPB method provides a significantly better representation of a watershed hydrologic response than a model based on unit hydrographs (locally calibrated or regionally calibrated) that represent free-draining catchments; (3) model usage--the model has been used extensively nationwide and has proved genrally acceptable and reliabile; (4) parameter calibration--the model used in this application is based on a minimal number of parameters, giving relatively higher accuracy in calibration of model parameters to rainfall-runoff data and the design storm to local flood flow frequency tendencies; (5) calibration effort--the model does not require for calibration either a large data base or time requirements; (6) application effort--the model does not require excessive computation for application; (7) acceptability--the model uses algorithms that are accepted in engineering practice; (8) model flexibility for planning--data handling and computational sub-models can be coupled to the model (e.g., channel and basin routing) resulting in a highly flexible modeling capability; (9) model certainty evaluation--the certainty of modeling results can be readily evaluated as a distribution of possible outcomes over the probabilistic distribution of parameter values.

RUNOFF HYDROGRAPH MODEL PARAMETERS

The design storm unit hydrograph model developed for Orange County, California, is based upon several parameters, namely, two loss rate parameters (a phi index coupled with a fixed percentage), an S-graph, catchment lag, storm pattern (shape, location of peak rainfalls, duration), depth-area (or depth-area-duration) adjustment, and the return frequency of rainfall.

Loss Function

The loss function, f(t), used in the model is defined by

$$f(t) = \begin{cases} \bar{Y}\, I(t), & \text{for } \bar{Y}\, I(t) < Fm \\ Fm, & \text{otherwise} \end{cases} \tag{1}$$

where $\bar{Y}$ is the low loss fraction, I(t) is the design storm rainfall intensity at storm time t, and Fm is a maximum loss rate defined by:

$$Fm = \sum ap_j \, Fp_j \tag{2}$$

where ap_j is the actual pervious area fraction with a corresponding maximum loss rate of Fp_j; the infiltration rate for impervious areas is set at zero.

The use of a constant percentage loss rate $\bar{Y}$ in Eq. 1 is reported in Scully and Bender (1969), Williams et al. (1980), and Schilling and Fuchs (1986). The use of a phi index (ϕ - index) method in effective rainfall calculations is also well-known (e.g., Kibler, 1982).

The low loss rate fraction is estimated from:

$$\bar{Y} = 1-Y \tag{3}$$

where Y is the catchment yield computed by:

$$Y = \sum a_j \, Y_j \tag{4}$$

In Eq. 4, Y_j is the yield corresponding to the catchment area fraction a_j and is estimated using the SCS curve number (CN) by

$$Y_j = \frac{(P_{24} - Ia)^2}{(P_{24} - Ia + S)\, P_{24}} \tag{5}$$

where P_{24} = the 24-hour T-year precipitation depth; Ia is the initial abstraction computed by Ia = 0.2S, and S = (1000/CN)-10. From the above relationships, the los loss fraction, $\bar{Y}$, acts as a fixed loss rate percentage, whereas Fm serves as an upper bound to the possible values of $f(t) = \bar{Y}\, I(t)$.

Values for Fm are based on the actual pervious area cover percentage (a_p) and a maximum loss rate for the pervious area, F_p. Values for F_p are developed from rainfall-runoff calibration studies of several significant storm events for several watersheds within the region under study.

S-Graph

The S-graph representation of the unit hydrograph (e.g., McCuen and Bondelid, 1983; Chow and Kulandalswamy, 1982; Mays and Coles, 1980) can be used to develop unit hydrographs corresponding to various watershed lag estimates. The S-graph was developed by rainfall-runoff calibration studies of several storms for several watersheds in the Los Angeles area. By averaging the storm event S-graphs for all storm events on each watershed studied, a representative S-graph was developed for each watershed. By comparing the representative S-graphs, a regional S-graph was derived to represent the regional S-graph to be used for design work in Orange County, California.

Lag

Fundamental to any hydrologic model is a catchment timing parameter. For the model, watershed lag is defined as the time from the beginning of effective rainfall to that time corresponding to 50-percent of the S-graph ultimate discharge. To estimate catchment lag, it is assumed that lag is related to the catchment time of concentration (Tc) as calculated by a sum of normal depth flow calculated travel times; i.e., a mixed velocity method (e.g., Beard and Chang, 1979; McCuen, et al., 1984). To correlate lag to Tc estimates, lag values measured from calibrated S-graphs for individual watersheds in Los Angeles were plotted against Tc estimates. A least-squares best fit line gives the estimator:

$$\text{lag} = 0.80\,Tc \qquad (6)$$

Design Storm Pattern

A 24 hour duration design storm composed of nested 5-minute unit intervals (with each principal duration nested within the next longer duration) was adopted as part of the policy. The storm pattern provides equal return frequency rainfalls for any storm duration. Such a storm pattern construction was reported by HEC (1982), while HEC used a nested central-loaded design storm pattern, the policy statement requires a rear-loaded distribution since analysis of measured storm events suggested that rear-loaded patterns were more common for major storm events.

<u>Runoff Hydrograph Model</u>

The model produces a time distribution of runoff Q(t)given by the standard convolution integral representation of:

$$Q(t) = \int_{0}^{t} e(s)\ u(t-s)\ ds \qquad (7)$$

where Q(t) is the catchment flow rate at the point of concentration; e(s) is the effective rainfall intensity; and u(x) is the unit hydrograph developed from the regionalized S-graph. In Eq. 7 e(s) represents the time distribution of the 24-hour duration design storm pattern modified according to depth-area effects and then further modified according to the loss function definition of Eq. 1.

PARAMETER CALIBRATION

Considerable rainfall-runoff calibration data have been prepared by the Corps of Engineers COE for use in their flood control design and planning studies. Much of this information has been prepared during the course of routine flood control studies in Orange County and Los Angeles County, but additional information has been compiled in preliminary form for ongoing COE studies for the massive Santa Ana River project (Los Angeles County Drainage Area, or LACDA). The watershed information available includes rainfall-runoff calibration results for three or more significant storms for each watershed. The calibration provided optimized estimates for the watershed S-graphs, the lag formula, and the loss rate at the peak rainfall intensities. A total of 12 watersheds were considered in detail for our study. Seven of the watersheds are located in Los Angeles County while the other five catchments are in Orange County (Fig. 1). Several other local watersheds were also considered in light of previous COE studies that resulted in additional estimates of loss rates, S-graphs, and lag values. Table 1 provides a summary of both the data obtained from the COE studies and watershed data assumed for catchments considered hydrologically similar to the COE study catchments.

TABLE I. WATERSHED CHARACTERISTICS

	Watershed Geometry						Calibration Results			
Watershed Name	Area (mi^2)	Length (mi)	Length of Centroid (mi)	Slope (ft/mi)	Percent Impervious (%)	Tc (Hrs)	Storm Date	Peak F_p (inch/hr)	Lag (hrs)	Basin factor
Alhambra Wash[1]	13.67	8.62	4.17	82.4	45	0.89	Feb.78 Mar.78 Feb.80	0.59,0.24 0.35,0.29 0.24	0.62	0.015
Compton2[1]	24.66	12.69	6.63	13.8	55	2.22	Feb.78 Mar.78 Feb.80	0.36 0.29 0.44	0.94	0.015
Verdugo Wash[1]	26.8	10.98	5.49	316.9	20	--	Feb.78	0.65	0.64	0.016
Limekiln[1]	10.3	7.77	3.41	295.7	25	--	Feb.78 Feb.80	0.27 0.27	0.73	0.026
San Jose[2]	83.4	23.00	8.5	60.0	18		Feb.78 Feb.80	0.20 0.39	1.66	0.020
Sepulveda[2]	152.0	19.0	9.0	143.0	24	--	Feb.78 Mar.78 Feb.80	0.22,0.21 0.32 0.42	1.12	0.017
Eaton Wash[1]	11.02[4] (57%)	8.14	3.41	90.9	40	1.05	---	---	---	0.015[7]
Rubio Wash[1]	12.20[5] (3%)	9.47	5.11	125.7	40	0.68	---	---	---	0.015[7]
Arcadia Wash[1]	7.70[6] (14%)	5.87	3.03	156.7	45	0.60	---	---	---	0.015[8]
Compton1[1]	15.08	9.47	3.79	14.3	55	1.92	---	---	---	0.015[8]
Dominguez[1]	37.30	11.36	4.92	7.9	60	2.08	---	---	---	0.015[8]
Santa Ana Delhi[3]	17.6	8.71	4.17	16.0	40	1.73	---	---	---	0.053[9] 0.040[10]
Westminster[3]	6.7	5.65	1.39	13	40		---	---	---	0.079[9] 0.040[10]
El Modena-Irvine[3]	11.9	6.34	2.69	52	40	0.78	---	---	---	0.028[9]
Garden Grove-Wintersberg[1]	20.8	11.74	4.73	10.6	64	1.98	---	---	---	---
San Diego Creek[1]	36.8	14.2	8.52	95.0	20	1.39	---	---	---	---

Notes 1: Watershed Geometry based on review of quadrangle maps and LACFCD strom drain maps.
2: Watershed Geometry based on COE LACDA Study.
3: Watershed Geometry based on COE Reconstitution Study for Santa Ana Delhi and Westminster Channels (June, 1983)
4: Area reduced 57% due to several debris basins and Eaton Wash Dam reservoir, and groundwater recharge ponds.
5: Area reduced 3% due to debris basin.
6: Area reduced 14% due to several debris basins.
7: 0.013 basin factor reported by COE (subarea characteristics, June, 1984).
8: 0.015 basin factor assumed due to similar watershed values of 0.015.
9: Average basin factor computed from reconstitution studies
10: COE recommended basin factor for flood flows.

The key catchments utilized for the calibration of the model design storm are Alhambra, Arcadia, Compton1 (at 120th Street), Compton2 (at Greenleaf), Dominguez, Eaton, and Rubio Washes. Although other watersheds were considered in the study (see Table 1) for the population of the parameter value distributions, the seven key catchments were considered similar to the region where the model is intended for use (the valley area of Orange County) and are used to develop flood frequency estimates.

Four of the seven catchments have been fully urbanized for 20 to 30 years, with efficient interior storm drain systems draining into a major concrete channel. Additionally, all of the major storm events have occurred during the gage record of relatively constant urbanization; hence, the gage record could have been assumed to be essentially homogeneous. Nevertheless, adjustments were made in this study to account for the urbanization effects in order to ensure accurately calibrated parameter values.

Peak Loss Rate, F_p

From Table 1, several peak rainfall loss rates that include, when appropriate, two loss rates for double-peak storms are tabulated. The range of values of all F_p estimates lie between 0.30 and 0.65 inch/hour with the highest value occurring in Verdugo Wash, which has substantial open space in the foothill areas. Except for Verdugo Wash, F_p is in the range from 0.2 to 0.60, which is a variation in values of the order noted for Alhambra Wash alone. Figure 2 shows a histogram of F_p values for the several watersheds. It is evident from the figure that 88 percent of F_p values are between 0.20 and 0.45 inch/hour, with 77 percent of the values falling between 0.20 and 0.40 inch/hour. Consequently, a regional mean value of F_p equal to 0.30 inch/hour is recommended for design in Orange County; this value contains nearly 80 percent of the F_p values, for all watersheds and for all storms, within 0.10 inch/hour.

S-Graph

Each of the watersheds listed in Table 1 has S-graphs developed for each of the storms where peak loss rate values were developed. For example, Fig. 3 shows the S-graphs developed for Alhambra Wash. By averaging the S-graph ordinates (developed from rainfall-runoff data), an average watershed S-graph was obtained. By combining the several watershed average S-graphs (Fig. 4) into a single plot, a regional S-graph is obtained. Indeed, the variation in S-graphs for a single watershed for different storms (see Fig. 3) is of the same order of magnitude of variation seen between the several catchment averaged S-graphs.

In order to quantify the effects of variations in the S-graph due to variations in storms and in watersheds (i.e., for ungaged watersheds not included in the calibration data set), the scaling of Fig. 5 was used where the variable "X" signifies the average value of an arbitrary S-graph as a linear combination of the steepest and flattest S-graphs obtained. That is, all the S-graphs (all storms, all catchments) lie between the Feb. 1978 storm Alhambra S-graph (X = 1) and the San Jose S-graph (X = 0). To approximate a particular S-graph of the sample set,

$$S(X) = X\, S_1 + (1-X)\, S_2 \qquad (8)$$

where S(X) is the S graph as a function of X, and S_1 and S_2 are the Alhambra (Feb. 1978 storm) and San Jose S graphs, respectively. Figure 6 shows the population distribution of X where each watershed is weighted equally in the total distribution (i.e., each watershed is represented by an equal number of X entries). Table 2 lists the X values obtained from the Fig. 5 scalings of each catchment S-graph. In the table, an "upper" and "lower" X-value that corresponds to the X coordinate at 80 percent and 20 percent of ultimate discharge values, respectively, is listed. An average of the upper and lower X values is used in the population distribution of Fig. 6.

TABLE 2. CATCHMENT S-GRAPH X-VALUES

WATERSHED NAME	STORM	X(UPPER)	X(LOWER)	X(AVG)
Alhambra	Feb. 78	1.00	1.00	1.00
	Feb. 80	0.95	.60	0.78
	Mar. 78	0.70	.80	0.75
Limekiln	Feb. 78	0.50	0.80	0.65
	Feb. 80	0.80	1.00	0.90
Supulveda	AVG.	0.90	0.80	0.85
Compton	AVG.	0.90	1.00	0.95
Westminster	AVG.	0.60	0.60	0.60
Santa Ana Delhi	AVG.	0.80	1.00	0.90
Urban	AVG.	0.90	0.80	0.85

Catchment Lag

In Table 2, the Urban S-graph, which represents a regionalized S-graph for urbanized watersheds in valley type topography, has an associated X value of 0.85. When the Urban S-graph is compared to the standard SCS S-graph, a striking similarity is seen (Fig. 7). Because the new Urban S-graph is a near duplicate of the SCS S-graph, it was assumed that catchment lag (COE definition) is related to the catchment time of concentration, Tc, as is typically assumed in the SCS approach.

Catchment Tc values are estimated by subdividing the watershed into subareas with the initial subarea less than 10 acres and a flow-length of less than 1000 feet. Using a Kirpich formula, an initial subarea Tc is estimated, and a Q is calculated. By subsequent routing downstream of the peak flowrate (Q) through the various conveyances (using normal depth flow velocities) and adding estimated successive subarea contributions, a catchment Tc is estimated as the sum of travel times analogous to a mixed velocity method.

Lag values are developed directly from available COE storm event calibration data, or by using "basin factor" calibrated from neighboring catchments (see Fig. 1). The COE standard lag formula is:

$$\text{lag (hours)} = 24\ n \left(\frac{L\ L_{ca}}{s^{0.5}} \right)^{0.38} \qquad (9)$$

where L is the watershed length in miles; L_{ca} is the length to the centroid along the watercourse in miles; s is the slope in ft/mile; and n is the basin factor.

Because Eaton Wash, Rubio Wash, Arcadia Wash and Alhambra Wash are all contiguous (see Fig. 1), have similar shapes, slopes, development patterns, and drainage systems, the basin factor of $n = 0.015$ developed for Alhambra Wash was also used for the other three neighboring watersheds. Then the lag was estimated using Eq. 9.

Compton Creek has two gages, and the $n = 0.015$ calibrated for Compton2 was also used for the Compton1 gage. The Dominguez catchment, which is contiguous to Compton Creek, is also assumed to have a lag calculated from Eq. 9 using $n = 0.015$.

The Santa Ana-Delhi and Westminster catchment systems of Orange County have lag values developed from prior COE calibration studies. Figure 8 provides a summary of the local lag versus Tc data. A least-squares best fit results in

$$\text{lag} = 0.72\,Tc \qquad (10)$$

McCuen et al. (1984) provide additional measured lag values and mixed velocity Tc estimates which, when lag is modified according to the COE definition, can be plotted with the local data such as shown in Fig. 9. A least-squares best fit results in:

$$\text{lag} = 0.80\,Tc \qquad (11)$$

In comparison, McCuen (1982) gives standard SCS relationships between lag, Tc, and time-to-peak (Tp) which, when modified to the COE lag definition, results in:

$$\text{lag} = 0.77\,Tc \qquad (12)$$

Adopting a lag of 0.80Tc as the estimator, the distribution of (lag/Tc) values with respect to Eq. 11 is shown in Fig. 10.

PARAMETER VERIFICATION

For each of the watersheds in the study, the calibrated model including the three parameter distributions of loss rate F_p values, S-graph, and lag w5re used to simulate the response to the severe storm of March 1, 1983, which was not included in the calibration set of storms. The March 1, 1983, storm was a multi-peaked storm event. Selected results for each of the Los Angeles watersheds are shown in Table 3.

The values for parameters used in the modeling results of Table 3 are F_p = 0.30 inch/hour; pervious cover = actual value; Urban S-graph; measured gage rainfall and storm pattern; and computed lag values from Eq. 11. Two sets of values for the low loss fraction, $\bar{Y}$, were used; namely (i) $\bar{Y}$ estimated from Eq. 3, and (ii) $\bar{Y}$ calibrated by taking $\bar{Y}$ equal to 1 - (measured storm runoff volume)/(measured storm rainfall). This second value of $\bar{Y}$ was calibrated (rather than using Eq. 3) due to the obvious variation in rainfall intensities over the watershed for the March 1 storm. Figure 11a provides a comparison between measured and modeled runoff hydrographs. Figure 11b shows the point rainfalls recorded at various gage locations.

TABLE 3. MARCH 1, 1983, PEAK FLOW ESTIMATES

Watershed	Time (Hours)	Recorded Q(CFS)	Regional Model			Calibrated to Low Loss Fraction, $\bar{Y}$		
			Std. Time	Modeled Q(CFS)	Relative Error	Std. Time	Modeled Q(CFS)	Relative Error
Arcadia[1]	0615	1460	0655	1830	25	0655	1740	+19
	0845	3660	0900	3490	5	0900	3500	-4
	1215	4110	1215	1275	69	1250	1260	-69
	2000	1340	1925	3550	165	1925	3530	+163
Eaton[2,4]	0619	1780	0720	2005(+150)	13	0715	2320(+150)	+30
	0904	4300	0925	5420(+150)	26	0925	5510(+150)	+28
	1219	5430	1300	2570(+600)	53	1300	2830(+600)	-48
	1934	3080	1950	5160(+750)	68	1950	5310(+750)	+72
Rubio[3]	0630	1500	0700	2610	74	0700	2450	+63
	0900	3760	0905	5980	59	0905	5990	+59
	1215	2520	1245	2200	-13	1245	2110	-16
	1930	3520	1930	5920	68	1930	5900	+68
Alhambra	0615	2290	0710	2405	5	0710	2450	+7
	0830	7010	0915	6460	8	0915	6460	-8
	1200	5300	1255	1700	68	1255	2550	-52
	1900	5250	1940	5300	1	1940	5320	+1
Compton2	0645	3380	0645	1290	62	0645	1237	-63
	0815	4620	0940	4160	-10	0940	4100	-11
	1200	2430	1200	2460	1	1200	2320	-5
	1845	847	1955	970	14	2010	920	+9
Dominguez	0910	9830	0925	6915	30	0925	7600	-23
	1206	6180	1240	4180	32	1240	5000	-19
	1900	2400	1955	1735	28	1955	2090	-13
	2400	1700	2440	1905	12	2440	2370	+39

Notes: 1 - Area reduced 14% due to Debris Basins
2 - Area reduced 57% due to Eaton Dam
3 - Area reduced 3% due to Debris Basins
4 - ( ) added to modeled Q's to account for Eaton Wash Dam outflow (per LACFCD 1983 Storm Report)

FLODD FREQUENCY ANALYSIS

Adjustments for the Effects of Urbanization

In order to calibrate the design storm of the model, flood frequency curve development was required. Seven of the Los Angeles County stream gage records show only minor effects due to urbanization and, therefore, required only a small amount of adjustment in order to develop a homogeneous gage record. The near homogeneity of the record is due to two major factors: (1) the watersheds have been fully urbanized for the last several decades, and (2) most of the major storms occurred during the period of full urbanization. The hydrologic stream gage adjustment procedure used for this study is presented in a paper by McCuen, Yen, and Hromadka (1986).

Development of Confidence Intervals

The estimation of the T-year flood is a basic problem in hydrology due to the uncertainty in the estimate caused by the uncertain estimation of parameters of the flood distribution. This uncertainty can have a significant effect on the flood design value, and its quantitication is an important aspect of evaluating the risk involved in a chosen level of flood protection. For liability reasons, the policy statement was formulated to address this source of uncertainty. In the past, model calibration has been based on the discharge estimate obtained from the curve, which would assume 50 percent protection for the T-year event. To incorporate the sampling uncertainty into the calibration and, therefore, the policy, a decision based on legal considerations was made to use the discharge from the 85 percent upper confidence interval for the T-year flood as the criterion for calibrating the hydrologic model.

Given annual series of homogeneous stream gage data for the fully urbanized watersheds considered, a statistical analysis is needed to develop the confidence intervals for the flood design values of the several T-year return frequency floods (e.g., 2-year, 5-year,···, 100-year). The Water Resources Council Bulletin 17B (1981) provide (in the case of the flood distribution whose logarithm is normally distributed) confidence intervals for the T-year flood by the use of the non-central student's t-distribution. The more general case, following the guidelines of Bulletin 17B, is when the logarithms of the flood distribution has a Pearson III distribution with a non-zero skew parameter. The case of non-zero skew is more complicated than the case of lognormally distributed floods, which is the case of zero skew (Bobee, 1979, 1973; Condie, 1977; Hardison, 1976; Kite, 1975, 1977; Stedinger, 1983a). In an important paper, Stedinger (1983a) shows that the confidence intervals for quantiles that are given in the U.S. Water Resources Council guidelines are not satisfactory. He uses a formula due to Kite (1975) and derives an expression for confidence intervals that he shows is satisfactory in several simulations.

Following the above developments, confidence intervals for the flood design values can be determined by simulation. The simulation gives an approximate sampling distribution, and so simulation can be used to obtain confidence intervals of various levels, which allows one to quantify the magnitudes and probabilities of various possible errors due to sampling.

Statistical Model Development

Zero skew. Consider the case where the logarithm of the maximum annual discharge (x) has a normal distribution. For the T-year flood, the exceedance probability p is:

$$p = 1/T \tag{13}$$

and the p-th quantile of x, which is denoted as y_p, is:

$$P(x \leq y_p) = p \tag{14}$$

It is y_p that we want to estimate for a given value of p. If the mean μ and standard deviation σ of x were known, then since $(x-\mu)/\sigma$ has a N(0,1) distribution, i.e., a normal distribution with mean 0 and standard deviation 1,

$$(y_p - \mu)/\sigma = z_p \tag{15}$$

where z_p is the p-th quantile for an N(0,1) distribution. Of course, μ and σ are not actually known; we only have estimates for them, $\hat{\mu}$ and $\hat{\sigma}$, based on m data points for m years of gage data. It follows that

$$(y_p - \hat{\mu})/\hat{\sigma} = ([(\mu - \hat{\mu})/\sigma] + z_p)/(\hat{\sigma}/\sigma). \tag{16}$$

The random variable in brackets in Eq. 16 is distributed as $Z/\sqrt{m}$, where Z has a N(0,1) distribution, and the denominator of Eq. 16 is distributed as $\sqrt{W}$, where W has a chi-squared distribution with m-1 degrees of freedom, divided by m-1, which is independent of Z. Thus Eq. 16 can be written as:

$$(1/\sqrt{m})[(Z + z_p\sqrt{m})/\sqrt{W}]. \qquad (17)$$

The random variable in brackets in Eq. 17 has a non-central t-distribution, with non-centrality parameter $\delta = z_p\sqrt{m}$; the special case $\delta = 0$ is the student's t-distribution (e.g., Breiman, 1973).

Since the distribution of Eq. 16 can be written in terms of the known non-central t-distribution, confidence intervals of y_p can be developed. For flood control study purposes, the confidence interval is an upper, one-sided interval. The choice of the value T, for the T-year flood, and the number m of data points (years of gage record) determine the non-centrality parameter δ. The other quantity that must be specified is a probability p' for the one-sided confidence interval. If $t_{p'}$ is the p'-th quantile of this non-central t-distribution, then

$$P(y_p \leq \hat{\mu} + \hat{\sigma}(t_{p'}/\sqrt{m})) = p' \qquad (18)$$

giving the one-sided 100p' percent confidence interval for y_p.

Non-zero skew. Let the logarithm of the yearly peak discharge x have a Pearson type II distribution with density function:

$$f(x) = (1/|a|\Gamma(b))[(x-c)/a]^{b-1} \exp-[(x-c)/a] \qquad (19)$$

where, in the case of positive "a", the density is given by Eq. 19 for $x > c$ and is zero for $x < c$, while in the case of negative "a" the density is given by Eq. 19 for $x < c$ and is zero for $x > c$. Computing the mean μ, standard deviation σ, and skew γ from Eq. 19 gives (see, for example Hall, 1984)

$$\mu = c + ab$$
$$\sigma^2 = a^2 b \qquad (20)$$
$$\gamma^2 = 4/b,$$

where "a" has the same sign as γ.

In following the guidelines of Bulletin 17B, the skew coefficient γ is estimated either from a map of regional skew or from a large pool of data from that region. Consequently, the error in estimating γ is of an entirely different kind than that which arises in estimating μ and σ by means of m data points for the specific area for which the T-year flood is being estimated. Typically, the station skew differs considerably from the regional skew (resulting in significantly different flood frequency statistics) and a weighted average is developed. However, there is substantial uncertainty as to what the "true" skew is. To proceed with the development, it is assumed that the skew γ is given "exactly". What is thereby analyzed is that part of the variability in the estimate of the T-year flood that arises from the uncertainty in the estimation of μ and σ; this ignores the variability that comes from the uncertainty in the estimate of γ. (As an example of the effects of the choice of skew, estimates for Q_{100} in Orange County varied by more than 60 percent depending on whether a station or regional skew is used.)

The form of the density Eq. 19 shows that the random variable

$$Z = (x-c)/a \tag{21}$$

has the one parameter density

$$g(x) = (1/\Gamma(b))x^{b-1}e^{-x} \tag{22}$$

for $x>0$ and 0 for $x<0$; i.e., Z has a gamma distribution with shape parameter b and scale parameter 1.

Introducing the constant K_p defined by

$$y_p = \mu + \sigma K_p. \tag{23}$$

then from Eq. 20,

$$(y_p - \mu)/\sigma = (1/\sqrt{b})[(y_p - c)/a]-\sqrt{b} \text{ if } a>0$$
$$(y_p - \mu)/\sigma = (1/\sqrt{b})[(y_p - c)/a]+\sqrt{b} \text{ if } a<0 \qquad (24)$$

If t_q is the 100q-th percentile for the gamma distribution Z, then the choice q=p gives the first equation below for $a>0$, while the second equation follows from the choice q=1-p:

$$P((y_p - \mu)/\sigma \leq (t_p-b)/\sqrt{b}) = p \text{ if } a>0$$
$$P((y_p - \mu)/\sigma \leq (b-t_{1-p}) /\sqrt{b}) = p \text{ if } a>0. \qquad (25)$$

That is,

$$K_p = (t_p-b)/\sqrt{b} \text{ for } a>0,$$
$$K_p = (b-t_{1-p})/\sqrt{b} \text{ for } a<0. \qquad (26)$$

As in the case of zero skew,

$$(y_p - \hat{\mu})/\hat{\sigma} = \{[(\mu - \hat{\mu})/\sigma] + K_p\}/[\hat{\sigma}/\sigma] \qquad (27)$$

This Eq. 27 shows that the joint distribution of the two random variables in brackets must be found. A substantial simplification can be obtained by writing both of these random variables in term of the Z of Eq. 21.

Let $X_1, \cdots, X_m$ be independent, each with a Pearson III distribution and let $Z_j = (X_j-c)/a$. Then

$$\hat{\mu} = (1/m) \Sigma X_j = (a/m)\{\Sigma ((X_j-c)/a)\} + c = a\hat{\mu}_Z + c \qquad (28)$$

where $\hat{\mu}_Z$ is the estimator for the mean of Z. The customary estimator for $\hat{\sigma}^2$ is:

$$\frac{\hat{\sigma}^2}{\sigma^2} = (1/ma^2b)\,\Sigma\,(X_j-\hat{\mu})^2 = (1/mb)\,\Sigma(Z_j-\hat{\mu}_Z)^2 = \hat{\sigma}_Z^2/b \qquad (29)$$

where $\hat{\sigma}_Z^2$ is the customary estimator for the variance $\sigma_Z{}^2$ of Z. Using $\sqrt{(\hat{\sigma}^2/\sigma^2)}$ as the estimator for $\hat{\sigma}/\sigma$, Eq. 29 shows that this estimator is:

$$\hat{\sigma}/\sigma = \hat{\sigma}_Z/\sqrt{b} \qquad (30)$$

Similarly, it follows that:

$$(\mu-\hat{\mu})/\sigma = (b-\hat{\mu}_Z)/\sqrt{b} \text{ for } a>0,$$
$$(\mu-\hat{\mu})/\sigma = (\hat{\mu}_Z-b)/\sqrt{b} \text{ for } a<0. \qquad (31)$$

Combining Eqs. 26, 27, 29, and 31 we obtain the final equation set:

$$(y_p-\hat{\mu})/\hat{\sigma} = (t_p-\hat{\mu}_Z)/\hat{\sigma}_Z \text{ for } a>0,$$
$$(y_p-\hat{\mu})/\hat{\sigma} = (\hat{\mu}_Z-t_{1-p})/\hat{\sigma}_Z \text{ for } a<0. \qquad (32)$$

So confidence intervals for y_p can be obtained if, for Z having the distribution of Eq. 22, we know the distribution of

$$(\delta-\hat{\mu}_Z)/\hat{\sigma}_Z. \qquad (33)$$

Confidence intervals are then determined by simulation. Details of the simulation procedure and the computer code can be obtained in Whitley and Hromadka (1986a,b).

POLICY STATEMENT OBJECTIVE

In order to calibrate the design storm for the model so that the peak flow rate estimates represent local flood frequency tendencies, an objective criterion for flood protection must be specified. That is, not only is a 100-year flood, for example, selected as a targeted level of flood protection, but also a level of confidence in estimating (or exceeding) the 100-year flood must be specified. This specification for a hydrologic model becomes part of the local agency's policy statement for flood control.

Flood potentials and flood insurance rates are usually based on an estimated value for a specific exceedance probability, with the 100-year event quite common. A flood estimate obtained from a flood frequency curve represents the best, or most likely, estimate. Recognizing that the flood estimate from the curve is only an estimate, there will be some error distribution about the estimate. If we assume that the sampling distribution about the flood frequency curve for any exceedence probability is symmetrical, then there would be a 50 percent chance that for any one location the flood frequency estimate would be exceeded when the same procedure is applied. To reduce an individual's chance of being flooded when the procedure is applied, an estimate obtained from the upper one-sided confidence interval can be used. Thus, the policy must state both the exceedance probability and the level of confidence to be used in providing flood protection for a community.

In this application, the policy statement objective is that an 85 percent upper confidence in design flow estimates be achieved:

$$P(Q_m^T \geq Q_T) = 0.85 \qquad (34)$$

where Q_m^T is the T-year design storm unit hydrograph model peak flow rate, and Q_T is the "true" T-year flood flow rate. From the simulation of the adjusted stream gage data, several confidence levels in stream gage data for the Q_{100} estimates are listed in Table 4. Estimates of the T-year flood at a 85 percent confidence level are provided in Table 5.

TABLE 4. SIMULATION RESULTS IN ESTIMATING Q_{100}

record number of years	Watershed Name	Confidence Level Q(cfs) 25%	50%	75%	85%	95%
54	ALHAMBRA	7659	8280	9025	9488	10391
28	ARCADIA	4429	5029	5820	6354	7496
27	COMPTON1	5563	6227	7092	7669	8891
56	COMPTON2	7769	8455	9282	9798	10807
16	DOMINGUEZ	18262	21199	25388	28428	35526
28	EATON	6307	7066	8053	8712	10100
54	RUBIO	4667	4979	5348	5575	6012

TABLE 5. T-YEAR FLOOD ESTIMATES

Watershed Name	At 85% Confidence Level, T-Year Flood 2	5	10	25	50	100
ALHAMBRA	3100	4543	5611	7078	8250	9488
ARCADIA	1610	2568	3330	4431	5351	6354
COMPTON1	2296	3473	4365	5606	6606	7669
COMPTON2	2899	4364	5489	7079	8386	9798
DOMINGUEZ	7702	12313	15820	20639	24454	28428
EATON	2566	3875	4882	6302	7463	8712
RUBIO	2152	3026	3627	4401	4986	5575

In the development of the confidence limit values of Tables 4 and 5, the computed skew used for each station is the Bulletin 17B recommended weighted average of a regional skew and station skew. Here, the station skew used is developed from the adjusted annual series (adjusted for urbanization effects), and the regional skew used is based on regionalization of the adjusted station skews (7 gages).

DESIGN STORM CALIBRATION

For each of the seven catchments, values for Fm ($Fm = a_p F_p$), $\bar{Y}$ as a function of curve number and P_{24}, and lag were developed (see Table 6). Because lag varied only slightly for T-year storm estimates of Tc, a constant lag is used for each catchment.

The objective function, χ, that was used to calibrate the design storm is:

$$\chi_T(t) = \frac{\sum_{i=1}^{m} (\hat{Q}_i^t - Q_i^T)\, A_i / Q_i^T}{\sum_{i=1}^{m} A_i} \qquad (35)$$

where $\chi_T(t)$ is the objective function considered at the T-year flood; $\hat{Q}_i^t$ is the "model" peak flow estimate for catchment i using a t-year design storm; Q_i^T is the 85 percent confidence level for the T-year flood computed from the adjusted stream gage data for catchment i and the simulation procedure (see Table 5); and A_i is the area of catchment i. Consequently, the objective function provides an area weighted fit of relative errors in achieving the desired flood control level of protection.

The design storm of the model is calibrated for the T-year flood protection level by finding the t that satisfies

$$\chi_T(t) = 0 \qquad (36)$$

Figure 17 shows the variation in $\chi_T(t)$ as a function of t, for T = 2-, 5-, 10-, 25-, 50-, and 100-year events.

TABLE 6. MODEL PARAMETER VALUES

Watershed	a_p[1]	F_p(in/hr)	F_m(in/hr)	CN[2]	Low Loss Fraction ($\bar{Y}$) 2 yr	5 yr	10 yr	25 yr	50 yr	100 yr	LAG(hrs)[3]
Arcadia	0.55	0.30	0.165	56	0.53	0.45	0.41	0.37	0.35	0.32	0.48
Eaton	0.60	0.30	0.180	56	0.57	0.50	0.46	0.41	0.39	0.37	0.84
Rubio	0.60	0.30	0.180	56	0.56	0.49	0.44	0.40	0.38	0.35	0.54
Alhambra	0.55	0.30	0.165	56	0.53	0.46	0.43	0.39	0.36	0.34	0.71
Compton1	0.45	0.30	0.135	56	0.47	0.44	0.40	0.37	0.35	0.33	1.54
Compton2	0.45	0.30	0.135	56	0.47	0.44	0.40	0.37	0.35	0.33	1.78
Dominguez	0.40	0.28	0.112	61	0.43	0.37	0.34	0.31	0.29	0.27	1.66

Notes:

1) Map measured value
2) AMC II
3) LAG = 0.8Tc

CONCLUSIONS

New technologies like the computer spawn new alternatives for public policies, such as a flood control policy. Computers have made more sophisticated design models possible, thus increasing the number and type of alternative hydrologic models to choose from. Therefore, model selection must be one element of policy formulation. A review of the literature does not suggest that the more complex computer based models improve the accuracy of hydrologic designs; thus, the widely used design storm method still appears to be the best choice and was selected for the Orange County (California) flood control policy.

A second element of policy development is the calibration of the model. Recognizing the ever increasing concern about liability and public responsibility for flood damages, the existing practice for incorporating the uncertainty into flood control policies is not optimum. An alternative that gives more consideration to public liability is outlined, with a case study used to show how the uncertainty was handled for a county in southern California.

A hydrology model for flood control design and planning has been developed that addresses the issues of uncertainty in stream gage data and statistics, and the uncertainty in model estimates. Because the coupling of these two uncertainties has not been addressed in the literature nor considered in the usual policy making procedures in flood control, it is suggested that it is important for flood control policy statements to include the specification of stream gage confidence levels and confidence in modeling predictions.

REFERENCES

1. Beard, L. and Chang, S., Urbanization Impact of Streamflow, J. Hydraulic Div., ASCE, Vol. 105, HY6, 647-659, 1979.

2. Bell, F., Estimating Design Flood From Extreme Rainfall, Hydrology Papers, Colorado State University Fort Collins, Colorado, No. 29, June 1968.

3. Bobee, B., Comment on "The log Pearson type 3 distribution: The T-year event and its asymptotic standard error" by R. Condie, Water Res. Research, Vol. 15, 189-190, 1979.

4. Brieman, L., *Statistics: With a View Towards Applications*, Houghton-Mifflin, Boston, 1973.

5. Cermak, R. and Feldman, A., Urban Hydrologic Modeling Using HEC-1/Kinematic Wave, Presented at the 19th Annual AWRA Conference, October 9-13, 1983, San Antonio, Texas.

6. Chow, V. and Kulandaiswamy, V., The IUH of General Hydrologic Systems Model, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 108, No. HY7, 830-844, 1982.

7. Condie, R., The log Pearson type 3 distribution: The T-year event and its asymptotic standard error by maximum likelihood theory, Water Res. Research, Vol. 13, 987-991, 1977.

8. Dickinson, W., et al, An Experimental Rainfall-Runoff Facility, No. 25, Hydrology Papers, Colorado State University, Fort Collins, Colorado, September, 1967.

9. Gupta, V., Sorooshian S., Uniqueness and Observability of Conceptual Rainfall-Runoff Model Parameters: The Percolation Process Examined, Water Res. Research, Vol. 19, 269-276, 1983.

10. Hall, M. J., *Urban Hydrology*, Elseviewa London, 1984.

11. Hardison, C., Confidence limits for flood frequency curves computed from samples from Pearson type III populations, J. of Research U.S.G.S. 4, 545-547, 1976.

12. Horberger, G. M., Beven, B. J., Cosby, B. J., and Sappington, D.E., Shenandoah Water Shed Study: Calibration of A Topography-Based, Variable Contributing Area Hydrological Model to a Small Forested Catchment, Water Res. Research, Vol. 21, 1841-1850, 1985.

13. Hromadka II, T. V., and Yen, C. C., A Diffusion Hydrodynamic Model: (DHM), Advances in Water Resources, Vol. 9, No. 3, 118-170, 1986.

14. Johnston P. and Pilgrim, D., Parameter Optimization for Watershed Models, Water Res. Research, Vol. 12, 477-486, 1976.

15. Kibler, D. F., Desk-Top Methods for Urban Stormwater Calculation, AGV Wat. Res. Moro. No.7, (1982).

16. Kite, G., Confidence Limits for Design Events, Water Res. Research, Vol. 11, 48-53, 1975.

17. Loague, K. and Freeze, R., A Comparison of Rainfall-Runoff Modeling Techniques on Small Upland Catchments, Water Res. Research, Vol. 21, 229-248, 1985.

18. Mays, L., and Coles, L., Optimization of Unit Hydrograph Determination, Hydraulics Div., ASCE, Vol. 106, No. HY1, 85-97, 1980.

19. McCuen, R. and Bondelid T., Estimating Unit Hydrograph Peak Rate Factors, Irrigation and Drainage Engineering, ASCE, Vol. 109, No. IR2, 238-250, 1983.

20. McCuen, R. H., Yen, C. C., and Hromadka II, T. V., Adjusting Stream Gage Data for Urbanization Effects, Microsoftware for Engineers, in-press (1986).

21. Nash, J., Sutcliffe, J., Riber Flow Forecasting Through Conceptual Models Part 1 - A Discussion of Principles, Journal of Hydrology, Vol. 10, 282-290, 1970.

22. Pilgrim, D., Travel Times and Nonlinearity of Flood Runoff From Tracer Measurements on a Small Watershed, Water Res. Research, Vol. 12, 487-496, 1976.

23. Rose, F., Hwang, G., A Study of Differences Between Streamflow Frequency and Rainfall Frequency for Small Rural Watersheds, 1985 International Symposium on Urban Hydrology, Hydraulic Infrastructures and Water Quality Control, University of Kentucky, July 23-25, 1985.

24. Schilling, W. and Fuchs, L., Errors in Stormwater Modeling-A Quantitive Assessment, Hydraulic Div., ASCE, Vol. 112, No. HY2, 111-123, 1986.

25. Scully, D. and Bender, D., Separation of Rainfall Excess from Total Rainfall, Water Res. Research, Vol. 5, 877-883, 1969.

26. Singh, K. P., Unit Hydrographs - A Comparative Study, Water Resour. Bull. 12,381, 1976.

27. Sorooshian, S. and Gupta, V., Automatic Calibration of Conceptual Rainfall-Runoff Models: The Question of Parameter Observability and Uniqueness, Water Res. Research, Vol. 19, 260-268, 1983.

28. Stedinger, J., Confidence Intervals for Design Events, Hydraulic Div., Vol. 109, No. HY1, 13-27, 1983a.

29. Stedinger, J., Design Events with Specified Flood Risk, Water Res. Research, Vol. 19, 511-522, 1983b.

30. Troutman, B., An Analysis of Input in Perception-Runoff Models Using Regression with Errors in the Independent Variables, Water Res. Research, Vol. 18, 947-964, 1982.

31. U.S. Army Corps of Engineers, The Hydrologic Engineering Center, Continuous Hydrologic Simulation of the West Branch Dupage River Above West Chicago: An Application of Hydrocomp's HSP, Research Note No. 6, 1979.

32. U.S. Army Corps of Engineers, The Hydrologic Engineering Center, Hydrologic Analysis of Ungaged Watersheds Using HEC-1, Training Document No. 15, April, 1982.

33. U. S. Department of Transportation, Federal Highway Administration, Hydrology, Hydraulic Engineering Circular No. 19, October, 1984.

34. U.S. Army Corps of Engineers, The Hydrologic Engineers Center, Testing of Several Runoff Models on an Urban Watershed, Technical Paper No. 59, 1978.

35. Whitley, R. and Hromadka II, T.V., Computing Confidence Intervals for Floods, I, Microsoftware for Engineers, Vol. 2, No. 3, 138-150, 1986.

36. Whitley, R. and Hromadka II, T.V., Computing Confidence Intervals for Floods, II, Microsoftware for Engineers, Vol. 2, No. 3, 151-158, 1986.

37. Williams, D.W. et al, TRRL and Unit Hydrograph Simulations Compared with Measurements in an Urban Catchment, Journal of Hydrology, 48 (1980) 63-70, Elsevier Scientific Publishing Company, Amsterdam-Printed in The Netherlands.