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DEVELOPMENT OF HYDROLOGY MANUALS FOR SOUTHERN CALIFORNIA COUNTIES:
I. MODEL SELECTION
II. MODEL CALIBRATION

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Abstract

For southern California watersheds, as is the case of 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 the two papers, a design storm unit hydrograph model is selected (paper I); developed and calibrated with respect to model parameter values and with respect to runoff frequency tendencies (paper II) in order to address each of these issues.

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I. MODEL SELECTION

INTRODUCTION

Fundamental to the preparation of a policy statement for the development of flood control system design values are the decisions of (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) Hydrologic Engineering Center (HEC) Training Document (TD) No. 11, (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 which is typically available. Consequently, the design storm approach is most often selected for flood control and drainage policies (considerations in the choice of modeling approach are contained in the literature review).

The next decision is whether to use the standard unit hydrograph method or the more recently advanced 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.

For the choice of design storm to be used, the work of Beard and Chang (1979) and HEC ("Hypothetical Floods", 1975) provide fundamental reasons for developing a design storm using rainfalls of identical return frequency, adjusted for watershed area effects.

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."

In order to facilitate these two calibration requirements, the runoff hydrograph model should be as simple as possible. For example, by using a loss rate defined as a fixed percentage of rainfall ($\bar{Y}$) such that the losses do not exceed a maximum value of Fm (phi index), then the design storm pattern shape and location of the peak rainfall are essentially removed as variables in the calibration of the design storm. The parameters for a single area unit hydrograph model are S-graph, lag, and loss rate values of $\bar{Y}$ and Fm. Here, $\bar{Y}$ is essentially $\bar{Y}$ = (1- yield), and Fm serves as a phi index loss function.

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 (e.g., Stedinger, 1983), a simulation procedure is used to determine the 85 percent confidence limits. In this fashion, model uncertainty in applications at ungaged catchments can be evaluated with respect to the chosen level of confidence in achieving flood protection at gaged sites.

LITERATURE REVIEW

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.

A criterion for complex and simple models is given by Beard and Chang (1979) as the "difficulty or reliability of model calibration....Perhaps the simplest type of model that produces a flood hydrograph is the unit hydrograph model"...and..."can be derived to some extent from physical drainage features but fairly easily and fairly reliably calibrated through successive approximations by relating the time distribution of average basin rainfall excess to the time distribution of runoff."

In comparison, the "most complicated type of model is one that represents each significant element of the hydrologic process by a mathematical algorithm. This is represented by the Stanford Watershed Model and requires extensive data and effort to calibrate."

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, see the work of Loague and Freeze (1985)) or complex models.

In their paper, Beard and Chang (1979) write that in the case of the unit hydrogragh model, "the function of runoff versus rainfall excess is considered to be linear, whereas it usually is not in nature. Also, the variations in shapes of unit hydrographs are not derivable directly from physical factors. However, models of this general nature are usually as representative of physical conditions as can reasonably be validated by available data, and there is little advantage in extending the degree of model sophistication beyond validation capability." It is suggested that "if 50 yr-100 yr of streamflow were available for a specified condition of watershed development, a frequency curve of flows for that condition can be constructed from a properly selected set of flows."

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. Hornberger, 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 emperical 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."

Schilling and Fuchs (1986) note that errors in simulation occur for several reasons, including:

"1. The input data, consisting of rainfall and antecedent conditions, vary throughout the watershed and cannot be precisely measured.

2. The physical laws of fluid motion are simplified.

3. Model parameter estimates may be in error."

By reducing the rainfall data set resolution from a grid of 81 gages to a single catchment-centered gage in an 1,800 acre catchment, variations in runoff volumes and peak flows "is well above 100% over the entire range of storms implying that the spatial resolution of rainfall has a dominant influence on the reliability of computed runoff." It is also noted that "errors in the rainfall input are amplified by the rainfall-runoff transformation" so that "a rainfall depth error of 30% results in a volume error of 60% and a peak flow error of 80%."

Schilling and Fuchs (1986) also write that "it is inappropriate to use a sophisticated runoff model to achieve a desired level of modeling accuracy if the spatial resolution of rain input is low" (in their study, the raingage densities considered for the 1,800-acre catchment are 81-, 9-, and a single centered gage).

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. "Inasmuch 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."

Garen and Burges (1981) noted the difficulties in rainfall measurement for use in the Stanford Watershed Model, because the K1 parameter (rainfall adjustment factor) and UZSN parameter (upper level storage) had the dominant impact on the model sensitivity. This is especially noteworthy because Dawdy and O'Donnell (1965) concluded that insensitive model coefficients could not be calibrated accurately. Thus, they could not be used to measure physical effects of watershed changes.

Using another complex model, Mein and Brown (1978) write that on "the basis of several tests with the Boughton model it is concluded that for this model at least, relationships derived between any given parameter value and measureable watershed characteristics would be imprecise; i.e., they would have wide confidence limits. One could not be confident therefore in changing a particular parameter value of this model and then claiming that this alteration represented the effect of some proposed land use change. On the other hand, the model performed quite well in predicting flows with these insensitive parameters, showing that individual parameter precision is not a prerequisite to satisfying output performance."

According to Gburek (1971),"...a model system is merely a researcher's idea of how a physical system interacts and behaves, and in the case of watershed research, watershed models are usually extremely simplified mathematical descriptions of a complex physical situation...until each internal submodel of the overall model can be independently verified, the model remains strictly a hypothesis with respect to its internal locations and transformations..." (also quoted in McPherson and Schneider, (1974)).

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 cited above.

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" or QPB 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 spaced), 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, "the fact that simpler, less data intensive models provided as good or better predictions than a QPB is food for thought."

Based on the literature, the main difficulty in the use, calibration, and development of complex models appears to be the lack of precise rainfall data and the high model sensitivity to (and magnification of) rainfall measurement errors. Nash and Sutcliff (1970) write that

"As there is little point in applying exact laws to approximate boundary conditions, this, and the limited ranges of the variables encountered, suggest the use of simplified empirical relations."

It is noteworthy to consider the HEC Research Note No. 6 (1979) where the Hydrocomp HSP continuous simulation model was applied to the West Branch DuPage River in Illinois. Personnel from Hydrocomp (R. Linsley, N. Crawford, co-authors of the Stanford Watershed Model, principals), HEC, and COE participated in this study which started with a nearly complete hydrologic/meteorologic data base. "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." This time allocation applies to only a 28.5 mi^2 basin. The quality of the final model is indicated by the average absolute monthly volume error of 32.1% and 28.1% for calibration and verification periods, respectively. Peak flow rate average absolute errors were 26% and 36% for calibration and verification periods, respectively. 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 discusses 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 (e.g., the Stanford Watershed Model or Hydrocomp HSP has been in operation for over 20 years) whether there is a significant advantage in using a watershed model 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.

Nonlinearity: Use of a Nonlinear Kinematic Wave Method or a Linear Unit Hydrograph Method

The dominant method used in runoff hydrograph development for representing catchment runoff response is the unit hydrograph (UH). The next most frequently used method is the kinematic wave overland flowplane concept (KW). HEC TD#15 (1982) provides a description and comparison of these two alternatives. The relative usage of KW by 1983 is indicated in 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." The relatively small usage of KW were then explained as being due to the slack in hydrologic studies and due to unfamiliarity with the technique.

Watt and Kidd (1975) write that in the comparison of so-called 'physically-based' or 'black-box' modeling types (e.g., UH or n-linear reservoirs) the differences are not clear. For example, "except for certain 'ideal' laboratory catchments, the flow does not conform to the sheet-flow model but instead occurs in many small rivulets...The choice is then between a 'black-box' model and a 'physically-based' model which is based on a physical situation quite different than the actual field situation, i.e., a 'black-box' model."

However, 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 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 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 in 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 writes 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.

Beven (1979) proposed to place limits on the nonlinearity associated to KW by the specification of a constant flow velocity for catchment runoff for large floods. He proposes "a nonlinear channel system at low flows and a linear system at high flows into a single model." Hence for flood flows of interest in flood control planning and design, Beven's model would reduce to a linear representation of the catchment hydraulics.

A physical test of the KW concept was provided by Hjelmfelt and Burwell (1984), who studied a set of 40 similar erosion plots and the net response to storm events. Due to the large variability in measured runoff quantities from the plots, however, it was concluded that a criterion for a valid rainfall-runoff model "is that it predicts the mean runoff for each event." The KW method did not fulfill this objective.

In HEC Technical Paper No. 59 (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 Castro Valley near Oakland, California. 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 with Singh (1977) concluded that "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 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

HEC (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. These floods represent classes of floods of a specified or implied range of severity." Such "floods are ordinarily derived from rainfall or snowmelt or both, with ground conditions that are appropriate to the objectives of the study, but they can be derived from runoff data alone, usually on the basis of runoff volume and peak-flow frequency studies and representative time sequences of runoff."

In complex watershed systems that include catchment subareas, and channel and basin routing components, Beard (1975) writes that "it is usually necessary to simulate the effects of each reservoir on downstream flows for all relevant magnitudes of peaks and volumes of inflows. Here it is particularly important that each hypothetical flood has a peak flow and volumes for all pertinent durations that are commensurate in severity, so that each computed regulated flow will have a probability or frequency that is comparable to that of the corresponding unregulated flow...In the planning of a flood control project involving storage or in the development of reservoir operation rules, it is not ordinarily known what the critical duration will be, because this depends on the amounts of reservoir space and release in relation to flood magnitude. When alternate types of projects are considered, critical durations will be different, and a design flood should reflect a degree of protection that is comparable for the various types of projects."

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.

Beard and Chang (1979) write that for design storm construction, "it is generally considered that a satisfactory procedure is to construct an approximately symmetrical pattern of rainfall with uniform areal distribution having intensities for all durations corresponding to the same recurrence interval and for that location and size of area" (i.e., depth-area effects).

The nested design storm concept is developed in detail in HEC TD#15 (1982), including the use of depth-area adjustments.

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) shows 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, but study results of Bell (1968) do provide a general study for comparison to 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 over 100 models available, a design storm/unit hydrograph model (i.e. "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 provides a significantly better representation of 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 generally acceptable and reliable; (4) parameter calibration--the "model" used in this application is based on a minimal number of parameters, giving 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 large data or time requirements for calibration; (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 submodels 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.

Reliability of Flood Frequency Confidence Limits

The following are embodied in the U.S. Water Resources Council's well-known Bulletin 17B: the Federal guideline for the choice of the statistical distribution to be used to fit a stream gage annual series; the methods for computing parameter estimates of sample mean, standard deviation, and computed skew; regional skew values; and the methods for computing confidence intervals. The reaction to the use of this distribution and in the accuracy of the approximation methods (especially for the computation of confidence intervals) are the topic of several papers including Wallis and Wood (1985), Bear (1978), U.S. Dept. of Transportation Federal Highway Administration Hydraulic Engrg. Circular No. 19 (1984), Hardison (1976), Kite (1975), and Stedinger (1983), among others. One major concern is the accurate computation of confidence intervals corresponding to the Log Pearson III distribution with an "exact" skew (assumed). Stedinger (1983) writes that "Confidence intervals constructed using the U.S. Water Resources Council guidelines, Bulletins 17A and 17B, often did not achieve the desired confidence level." To compute the accurate confidence intervals, numerical simulation can be used (e.g., Hardison (1976)) for the flood frequency analysis. Further details of this procedure are contained in a subsequent section.