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A REVIEW OF RAINFALL-RUNOFF MODELING FOR STORMWATER MANAGEMENT

By

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Office of Surface Water Resources and Systems Analysis

Prepared for the
U.S. Geological Survey, Illinois District

Champaign, Illinois

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CONTENTS

I. INTRODUCTION

The objective of this review is to present available literature that provides an assessment of the comparative strengths and weaknesses in 1) hydrologic and hydraulic models used to estimate flood hydrographs in Illinois, 2) precipitation and rainfall-loss inputs to these models, and 3) frequency estimation methods using model results. The review is intended to promote a better understanding of the differences in available models, their application, model inputs, and resulting output. This knowledge will be beneficial for those who use models to evaluate stormwater management practices and effects of urban development, and may assist in the development of a set of guidelines for stormwater modeling in Illinois. It is not practical to review all the technical issues associated with the rainfall-runoff modeling of stormwater, but an attempt is made to cover most of the broad concerns about differences in model capabilities and modeling procedures.

The focus of this review is the application of comprehensive rainfall-runoff models that can generally be used on all types and sizes of watersheds. There is no attempt to evaluate special-purpose models such as urban stormwater models, about which there is a substantial amount of available literature. For these, the reader is referred to Kibler (1982) and McPherson and Zuidema (1977). Many of the concerns related to urban stormwater modeling also apply to the comprehensive rainfall-runoff models and are discussed herein.

Rainfall-runoff modeling may be used for a variety of purposes. A common use is for design purposes when complete hydrographs are needed and peak discharge values alone are insufficient. The use of relatively simple rainfall-runoff models has become common over the years for designing detention storage or for design projects in medium to large watersheds where channel and floodplain storage are important factors in evaluating the flood hydrograph (Pilgrim, 1986). Rainfall-runoff modeling may also be used as a management tool, for example, in the management of stormwater runoff for water quality and urban development.

In recent years, new demands have been placed on rainfall-runoff models that require more physically based or complex methods. Todini (1988) recognizes three such demands: the use of models for simulating long continuous records; the application of models to complex watersheds with a variety of land uses, soil types, and stormwater management facilities; and the transfer of the models for use on similar ungaged catchments. Todini (1988) indicates that these applications have stretched or surpassed the limits of several widely used models, and other models having greater detail and more complex structure are being applied. Use of more complex models brings with it the associated needs for greater amounts of data and improved methods of representing model inputs. But at present the level of model sophistication is much more developed than the available data necessary to support complex modeling (McPherson and Zuidema, 1977; Shafer and Skaggs, 1983; Pilgrim, 1986; Todini, 1988). The development of the appropriate improved model inputs is a topic of crucial concern.

Even with the improved abilities of models, the rainfall and rainfall-loss information used as inputs into the models remain primary sources of uncertainty in the modeling process. McPherson (1978) states that the degree of data network adequacy is the most important measure of the reliability of the simulation analysis, and far exceeds the normal issue of model accuracy. Advancements in the estimation of the precipitation distribution over the watershed and rainfall loss inputs are necessary before the existing models can be used to their capability (Berndtsson and Niemczynowicz, 1988). Fifteen years ago, Overton and Meadows (1976) stated: "There remains much uncertainty in stormwater modeling. There appear to be enough parametric models available which have been shown to be feasible conceptualizations of the stormwater runoff process. What is needed now is a continued and accelerated verification of the existing models and a follow-up regionalization of the parameters." Since this statement has been written there have been significant advances, particularly in sophisticated data gathering and management for use with rainfall-runoff models. However, the statement by Overton and Meadows remains an appropriate summary of the state of rainfall-runoff/stormwater modeling.

Topics Covered in the Review

As the term "rainfall-runoff model" suggests, the major input into the model is an estimate of rainfall, and the output is an estimate of runoff. The intermediate steps that transform rainfall to runoff are the model processes. Among the hydrologic processes typically modeled are: interception, infiltration, evapotranspiration, snowpack and snowmelt, retention and detention storages, soil water movement, percolation to ground water, overland flow, open channel flow, and subsurface flow (interflow and base flow). The processes of major concern, which are discussed in this review, are the rainfall losses (primarily infiltration) and channel flow routing. Many of the other processes, such as evapotranspiration, interception, and overland flow, have important roles in rainfall-runoff hydrology; but these could not all feasibly be covered within the framework of this review.

Additional related topics of foremost concern in the application of the rainfall-runoff models are the general characteristics of available models, the overall modeling approach, the estimation of watershed precipitation, and the frequency analysis associated with model output. The review of these topics is ordered as follows:

- Overview of Rainfall-Runoff Simulation Models
- Watershed Precipitation
- Rainfall Losses
- Channel Routing
- Flood Frequency Methodologies
- Modeling Approaches

Methodology — Literature Search and Review

A literature search was conducted on the technical issues pertaining to rainfall-runoff modeling for the analysis of flood peak discharge and volume. A search of keywords for their occurrence in a computerized library database, *Selected Water Resources Abstracts*, produced a list of over 400 citations. The relevance of the retrieved studies to the rainfall-runoff modeling topics reviewed in this report varies according to the keywords used in the search. Broad keywords, such as *rainfall-runoff modeling,* were not used to avoid retrieving thousands of citations not relevant to the literature review. When more specific keywords are used, such as the *Horton equation,* the retrieved references have high relevance, yet a great number of other pertinent studies may be missed. Thus, the procedure used for the literature search attempted to compromise between these approaches. Articles were selected from the list of retrieved citations and reviewed. Many additional articles, which were cited in other studies or were otherwise recommended, were also reviewed. The articles that form the basis of this literature review are referenced directly in the text.

An attempt was made to keep the scope of this literature review as wide as possible within the framework of the overall project. The material summarizes what many scientists have produced over a period of years. The authors have tried to categorize the problems associated with rainfall-runoff modeling, the results of some attempts at solutions, and, in some cases, the recommendations of the researchers, without expressing their own biases for or against any particular method. Although some aspects of rainfall-runoff modeling are dealt with in less detail than others, that decision was made based upon the perceived application of flood modeling methodologies in Illinois in the future.

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II. OVERVIEW OF RAINFALL-RUNOFF SIMULATION MODELS

Classification of Rainfall-Runoff Models

Models are normally characterized or classified to help describe and discuss their capabilities, strengths, and limitations. There is no universal method to characterize rainfall-runoff models, and models have been classified in several ways depending on the criteria of interest. Examples of classifications are given by Anderson and Burt (1985); Dooge (1977); Larson et al. (1982); Shafer and Skaggs (1983); and Todini (1988). For this review, four categories were chosen from the above references and are presented below: 1) Event and Continuous Simulation Models, 2) Conceptual and Hydrodynamic Models, 3) Lumped and Distributed Parameter Models, and 4) Models with Fitted, Physically Determined, or Empirically Derived Parameters.

One common classification scheme not included below is the differentiation between deterministic and stochastic models. But as Klemes (1978) points out, deterministic modeling is simply a category of stochastic modeling that disregards the uncertainties in the model, its parameters, and its inputs. Rainfall-runoff modeling has typically taken a deterministic approach, although James and Burges (1982a) detect an increase in the use of stochastic estimation.

1) Event and Continuous Simulation Models

Rainfall-runoff models are either event models or continuous simulation (CS) models. Event models typically estimate the runoff from an individual storm event, i.e., describing a relatively short period within the hydrologic record. Event models ordinarily evaluate a partial set of the hydrologic processes that affect the watershed: infiltration, overland and channel flow, and possibly interception and detention storage. Most event models use a constant time interval, whose value may typically range from minutes to several hours.

Continuous simulation models operate for a sustained period that includes both rainfall events and interstorm conditions. To legitimately evaluate the streamflow during interstorm periods, CS models should include additional hydrologic properties such as evapotranspiration, shallow subsurface flow, and ground-water flow. Also crucial to these models is an accounting of the soil moisture and how it relates to changes in infiltration. The CS time interval can be daily, hourly, subhourly, or variable. Models that provide only daily simulation are not ordinarily useful for stormwater applications.

When using a CS model, the initial conditions are normally set for a time well in advance of any storm under consideration. The antecedent conditions for a storm are calculated analytically as a part of the normal model operation and are sensitive to the input series of climatological data, not to the initial conditions. The calculation of antecedent conditions using a CS model is normally considered advantageous because it does not require subjective evaluations by the model user.

For an event model, the initial conditions (antecedent soil moisture, stream and reservoir levels, etc.) must either be subjectively assigned by the user, calibrated with some type of error-reduction procedure, or approximated by an external procedure. Of these three approaches, the first two are common. In the third approach, an external approximation of the antecedent conditions may be obtained using the preceding climatological time series and applying either a simple empirical procedure or a more complex, explicit accounting of hydrologic conditions. When an explicit accounting procedure and the past climatological record are used to estimate the initial conditions, the function of the event model can approach that of a CS model. Feldman (1979) suggests that an accounting of antecedent conditions for use in a detailed event model can be obtained by using a relatively simple CS model. The authors note that the reliability of any estimate of the antecedent condition is a function of the appropriateness of the moisture accounting procedures, not whether these procedures are accomplished internally (as with CS models), or calculated independently of the model.

In modeling a long period that contains a number of floods of various magnitudes, the application of CS models provides more opportunities to compare model results with observed runoff. A long calibration period with a variety of hydrological conditions increases confidence in model results (James and Burges, 1982b). Event models are typically applied to fewer storms, but increased confidence can be gained by calibrating the model to as many storms as possible.

2) Conceptual and Hydrodynamic Models

The categorization describes the types of equations used in the model to describe the hydrologic processes. These categories of models are identified as: 1) "black-box" or transfer functions, 2) conceptual models, and 3) hydrodynamic models. Black-box models rely upon a statistical correspondence between the model input (rainfall) and model output (runoff) without relation to any underlying physical processes. Conceptual models are described by Dooge (1977) as "models which are formulated on the basis of a simple arrangement of a relatively small number of elements, each of which is itself a simple representation of a physical relationship." This definition is sufficiently broad enough to include hydrodynamic models, but conceptual models usually represent an intermediate level of component sophistication. Hydrodynamic models -- sometimes also termed physically based models (Beven, 1989) -- are also simplifications of reality and have a certain amount of empiricism (Haan, 1988). However, these models are generally based on the most recent physics-based understanding of the hydrologic processes that control the runoff response in the watershed. One of the attributes of the physics-based processes, as explained by Beven (1985), is that they involve laws and principles that can be validated independently of the model. Another difference between conceptual and hydrodynamic models is how the parameters are estimated, which will be discussed later. For the purposes of the review, only the conceptual and hydrodynamic models are of interest.

In reality, the boundaries between conceptual and hydrodynamic models are fuzzy. Individual models will normally combine both conceptual and hydrodynamic components. Not all hydrologic properties can be represented by hydrodynamic components, which forces all models to have some conceptual and empirical aspects. The predominant manner in which the components are modeled results in the overall classification. For the above reasons, any discussion of differences between conceptual and hydrodynamic models is not absolute but falls along a continuum. Comments related to hydrodynamic models may also be attributed to a hydrodynamic component that is contained within a "conceptual" model.

3) Lumped and Distributed Parameter Models

The hydrologic parameters used in the rainfall-runoff models can be represented in either a lumped or distributed manner. The lumping method averages the total rainfall, its distribution over space, soil characteristics, overland flow conditions, etc. for the entire watershed, ignoring all flow-routing mechanisms that exist within it. The expectation is that any minor details of the rainfall-runoff process will be inconsequential, resulting in an "average" flood condition. Although certain lumped parameters may implicitly represent physical attributes of the hydrologic system, they cannot be expected to have any direct physical interpretation (Delleur, 1982; Troutman, 1985). Lumped models can be made to behave more like distributed parameter models by adopting a detailed database and dividing a watershed into very small subwatersheds (Nix, 1991).

Distributed parameters both describe the geographical variation of parameters across the watershed and discriminate between changes in the hydrologic processes that occur throughout the watershed. In a fully distributed model, the hydrology of each small element of the watershed is distinctly simulated to include the hydrologic interactions with bordering elements. In reality, parameters in the distributed models have to be lumped to a small degree to match the grid scale used for computations (Troutman, 1985). In addition, the fitting of distributed, hydrodynamic models to observed streamflow at present is usually accomplished through the simplification and calibration of certain parameters (Bathurst, 1986). Therefore, without a sufficiently detailed database, a distributed model effectively may deteriorate into a lumped system model (McPherson and Zuidema, 1977; Beven, 1989).

A third approach simulates the hydrologic processes for a discrete number of land use and soil types. A land use and soil type combination, termed a hydrologic response unit (HRU), may occur in numerous locations in the same watershed; however, the hydrologic response is modeled for this combination only once, and this response is assumed to be homogeneous for all locations having that HRU. The HRU parameter approach is used in many rainfall-runoff models (for example, HSPF, SWMM, and PRMS) that are commonly considered distributed parameter models. Depending on how the watershed is partitioned, either the hydrologic response from each HRU is assigned to individual elements throughout the watershed, or the responses from several HRUs are prorated and aggregated to represent the lumped response from a subwatershed.

Within the framework of any individual model, the level of distribution can be user-controlled. James and Robinson (1985) state that the appropriate extent to which a modeler chooses to distribute the parameters should depend upon the objectives of the study and the available data, time, and money.

Many studies (Larson et al., 1982; Beven, 1985; Wilcox et al., 1990) suggest that distributed parameter models are desirable because they have the greatest potential for use in describing land use change, water-quality modeling, and forecasting on ungaged watersheds. These advantages assume that the parameters of distributed models are more physically realistic than the lumped model parameters, which should be the case when the model is well designed (Troutman, 1985). Distributed parameters have the potential to be physically interpreted and, when this is the case, greater confidence can be placed in the use of the model for prediction of flows (James and Burges, 1982b; Troutman, 1985). One reason that distributed parameter models have not seen widespread use is the availability of detailed databases. Future improvements in data acquisition, including the application of geographical information systems (GIS), will likely lead to more extensive use of distributed and HRU parameter models (Toms, 1989).

4) Models with Fitted, Physically Determined, or Empirically Derived Parameters

Parameters for rainfall-runoff models can be 1) fitted through calibration, 2) determined from field measurements, or 3) empirically fixed. Fitted parameters, set in the calibration process, typically have no little or no physical interpretation. Physically determined parameters are derived from measurable watershed characteristics such as slope, channel width, hydraulic conductivity of soils, etc. Measured values may not always produce the best results when used directly in a model. Thus, some physically determined parameters may be adjusted during the calibration process and are not necessarily equal to the measured variables. But to maintain the physical relationship these parameters should be similar in magnitude and behavior to the measured values.

The use of fitted versus physically determined parameters is a major issue in the application of rainfall-runoff models. Fitted parameters are less likely to be consistent from one data set to another, and models that use these parameters are less appropriate for extrapolation (Larson, 1973; Gan and Burges, 1990). In general, lumped models and most conceptual models use fitted parameters. However, Larson et al. (1982) indicate that fitted parameters cannot reliably be transferred for use on ungaged watersheds. Thus, empirically derived parameter methods (described below) are often used with the lumped conceptual models for ungaged sites. Distributed and quasi-distributed conceptual models can use a combination of fitted, physically determined, and empirical parameters. Distributed hydrodynamic models primarily use measured or physically determined parameters, with some empirically derived parameters.

Empirically derived parameters are developed by the regression analysis of either fitted or physically determined parameters. Empirically derived parameters may vary in the amount of physical interpretation that can be associated with their values. This category of parameters includes the Soil Conservation Service (SCS) runoff curve numbers that were developed for estimating rainfall losses on ungaged watersheds. Many of these empirically fixed relationships are required for parameterization of selected components in all models, including the models that are more physically based.

Classification of Selected Models.

Eight widely recognized models were chosen specifically to give the reader helpful examples of classifications of models. Many other models could have been used for this example, and the inclusion of a model in this list should not be considered in any way as an endorsement. The classifications, which are presented below, are based on the common uses of a model and may not be representative in all cases. Most models have flexibility in their parameterization, and may be applied in a somewhat different manner than that described below.

HEC -1 Flood Hydrograph Package (U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1990): Conceptual, event model with fitted and/or empirical, lumped parameters

TR-20 (Soil Conservation Service Technical Release 20, 1978): Conceptual, event model with empirical, lumped parameters

HSPF -- Hydrologic Simulation Program - FORTRAN (Johanson et al., 1984): Conceptual, CS model with fitted and/or physical, HRU parameters

SWMM -- Storm Water Management Model (Huber et al., 1981): Conceptual, event model with fitted, HRU parameters

ANSWERS -- Areal Nonpoint Source Watershed Environment Response Simulation (Beasley and Huggins, 1982): Conceptual, event model with fixed, distributed parameters

SHE -- Systeme Hydrologique Europeen (Abbott et al., 1986b): Hydrodynamic, CS model with physical, distributed parameters

PRMS -- Precipitation-Runoff Modeling System (Leavesley et al., 1983): Conceptual, CS model with physical and fitted, HRU parameters

NWSRFS -- National Weather Service River Forecast System (NWS, 1983) Conceptual, CS model with fitted, lumped parameters

Model Calibration, Validation, and Verification

Dendrou (1982) identifies calibration, validation, and verification as the three crucial steps for the proper application of a model. Calibration is the process of modifying model parameters to reduce the error between the simulated streamflow and some portion of the observed flow record. Model validation tests the ability of the model to estimate runoff for periods outside that used to calibrate the model. Model verification, as defined by Dendrou (1982), investigates the range of conditions over which the model will produce acceptable results. In normal application of a model to a gaged watershed, calibration is often the only procedure of the three that is followed. Model validation and verification are often not considered practical. If essential information about these two procedures is to be obtained, then it is normally up to the model developers and researchers. Some explanation of model verification is especially important for applications to ungaged watersheds when calibration and validation cannot be achieved.

Associated with the procedures of calibration, validation, and verification are three separate issues involving model application: flexibility, divergence, and extrapolation. Model flexibility describes the capability of a model to calibrate for a variety of different watersheds and flow conditions. Model divergence defines the relative accuracy of the model between the calibrated period and the validated period. Model extrapolation is the use of a model to describe hydrologic conditions outside of the range used for calibration and validation. These three issues are discussed below in context with other concerns related to calibration, validation, and verification.

Calibration and Model Flexibility

Sorooshian (1983) identifies two major purposes of calibration: 1) "to obtain a conceptually realistic and unique parameter set which closely reflects our understanding of the physical system"; and 2) "to obtain a (any) parameter set which gives the best possible fit between the model-simulated and observed hydrographs." With a physically realistic model and unbiased model inputs, each of these two objectives should lead to the same parameter values (Troutman, 1985). But for most models the first objective is not obtainable; therefore, it has become common to give greater emphasis to the second objective.

Using calibration to achieve the "best-fit" parameters can lead to biased or unrealistic values (Troutman, 1985; Haan, 1988). If this happens, the parameter values may not produce reasonable results when applied to other flow records from that watershed or to other watersheds that may be very similar in nature to the calibrated watershed. Thus, the flexible model may in reality be "less accurate" in its inability to match the general hydrologic situation (see Todini, 1988).

The ability of a model to calibrate to specific watersheds and flow conditions is dependent not only on the appropriateness of the hydrologic assumptions written into the model, but also on the flexibility of the model parameters. Flexible models have parameters that are designed to be fitted during the calibration process. Black-box and simpler conceptual models are designed to have flexible, fitted parameters, whereas the more physically based models are designed to have less flexible fixed and measurable parameters. In many cases, the more physically based models may be less likely to calibrate as well, and simpler models may have either comparable or superior accuracy (Todini and Wallis, 1977). In this context, flexibility can be viewed as a model asset.

Having a good fit between predicted and observed streamflow for the calibration period is a necessary test of a model's applicability to a watershed, but it is insufficient because it does not guarantee that the model will properly simulate runoff for noncalibrated periods (Beven, 1989; Todini and Wallis, 1977). This viewpoint is echoed by Yen (1982):

> "Calibration has the potential to improve the modeling accuracy over the ranges of conditions for which calibration is performed. However, one should bear in mind that calibration is not verification of a model and that beyond the calibrated ranges the model reliability may be questionable unless assessed through some other means."

Troutman (1985) argues that, with a properly designed model, the physically based parameter values and calibrated values should be similar, if not the same. Achieving such model parameter values, thereby satisfying both accuracy and reliability criteria, is a highly desired goal in rainfall-runoff modeling.

Automatic Calibration. Manual Calibration, and Objective Functions. The adjustment of parameter values to minimize model errors can be accomplished by automatic or manual calibration. Automatic calibration routines attempt to minimize an objective function (that represents model error) by iterative adjustment of a specified set of parameters. In manual calibration the user normally adjusts one parameter at a time and examines the model output between each adjustment. Manual calibration can involve either a subjective evaluation of model results or the use of an objective function. The use of automatic procedures is desirable because it greatly reduces the amount of time devoted to calibration and provides an objective evaluation of the "best-fit" (WMO, 1975; Troutman, 1985). However, because model parameters may calibrate to unrealistic values, several studies (WMO, 1975; Thompson and Westphal, 1989; Franchini and Pacciani, 1991) have concluded that automatic calibration alone is rarely an adequate procedure for estimating model parameters, and that some amount of manual calibration is always needed. One benefit of manual calibration is that "the modeler gains a great deal of familiarity with the role and sensitivity of various parameters. Such familiarity should always be an essential element of applying any model" (Troutman, 1985). The World Meteorological Organization (1975) states that proper model calibration requires a skillful hydrologist who has familiarity with both the model and the watershed to which the model is applied. The information that the modeler/hydrologist gains through manual calibration can be fed back into determining appropriate parameters. "It is apparent that between an automatic calibration procedure and a procedure based on successive rational attempts, the latter is preferable as it is the only one which makes it possible to use prior knowledge of the nature of the watershed (Franchini and Pacciani, 1991)."

Another danger of using automatic calibration without an additional evaluation of results, is that the parameters may converge to values that are not hydrologically realistic. Because many parameters are highly interdependent, there may exist several combinations that provide similar reductions in the model error (Gan and Burges, 1990). Many of these parameter combinations could cause the model to be highly divergent (described below) and give poor results when used for prediction (WMO, 1975). Gan and Burges (1990) indicate that a good initial estimate of the model parameters is a significant factor in reducing the overall calibration effort and in developing a reliable set of parameters.

Various types of objective functions for use with automatic calibration have been investigated, particularly maximum likelihood functions (Troutman, 1985; Sorooshian and Gupta, 1983). Troutman (1985) indicates that an ordinary least-squares objective function may not result in physically realistic parameter values, and recommends the use of a maximum likelihood function. However, Gan and Burges (1990) conclude that there appears to be no "best" type of objective function. Objective functions may overemphasize a certain portion of the flow record, for example, providing good estimates of extreme flood events at the expense of accuracy for more frequent floods. For this reason, Troutman (1985) recommends that objective functions be based on the logarithms of the predicted and observed runoff.

Validation and Model Divergence

The validation process should be used to test a model's divergence over a wide range of hydrologic conditions. Divergence occurs when the model error for forecasted or predicted flows is significantly greater than the model error for calibrated conditions (Todini and Wallis, 1977). Todini and Wallis (1977) state that "a good fit to the calibration period may comfort the individual hydrologic model builder but it does not guarantee a model with minimum forecast divergence, and it imparts no substantial message to those concerned with comparative model quality." To properly validate a model and reduce the potential divergence, the modeling error should be fully evaluated. Examples of error analysis are provided in Troutman (1985), Melching et al. (1990), and Gan and Burges (1990). However, such a detailed validation procedure is frequently beyond the scope of most applications (Dendrou, 1982; Pilgrim, 1986).

Verification and Model Extrapolation

Model extrapolation occurs when a model is used to simulate conditions outside the range used for calibration. Extrapolation can include simulating runoff in ungaged watersheds, evaluating effects of land use changes, or predicting the response from extreme rainfall amounts (Larson, 1973; Gan and Burges, 1990). In these cases, observed streamflow records are not available to validate whether the model is performing acceptably. Verification investigates the model processes and parameters to ascertain the range of conditions over which the model will produce acceptable results. But there is typically little information available to help the user verify the circumstances when a model might be inappropriate for a particular problem (Dooge, 1977). For these reasons, James and Burges (1982a) feel it is desirable, if not a responsibility of model developers and researchers to supply this information to the model users.

The use of conceptual models that have fitted parameters for use with extrapolated conditions has been questioned by many investigators (Abbott et al., 1986a; Larson, 1973; Gan and Burges, 1990). "When calibrated to a given set of hydrological signals (time series), there is no guarantee that a conceptual model can predict accurately when it is used to extrapolate beyond the range of calibration or verification experience" (Gan and Burges, 1990). According to Larson (1973) and Gan and Burges (1990), the physical basis of the model processes and physical interpretation of the model parameters provide the best indication of a model's ability to extrapolate to different hydrologic conditions. Numerous investigators recommend the use of more physically based models and parameters for applications that require extrapolation (Abbott et al., 1986a; James and Burges, 1982a; Beven, 1985; Franchini and Pacciani, 1991).

Evaluation and Selection of Models

In choosing a model or modeling approach, the objective of the model application should be clearly defined. Several options are outlined in section VII: "Modeling Approaches." It should be stressed that all types of models have their own particular area of effectiveness in rainfall-runoff modeling, and their use depends upon the objective of a study and the desired accuracy (Dooge, 1977). McPherson (1978) advises that the models "should be selected on the basis of their suitability for solving defined problems. What is wanted from the simulation should be defined first, and the selection of techniques should follow, not lead, this decision." But Nix (1991) notes that matching the modeling objectives with an appropriate model is not necessarily a trivial task.

Discussions concerning the evaluation and selection of models are provided in selected sources (Woolhiser and Brakensiek, 1982; James and Burges, 1982b; Anderson and Burt, 1985). Within these discussions, a number of criteria are usually recommended for use in the model selection: 1) ease of model use, 2) model accuracy, 3) consistency of parameters, 4) sensitivity of output to changes in parameters, 5) theoretical limitations of the model, and 6) data limitations. These six criteria, in one form or another, appear to be widely received by the research community, though Linsley (1986) contends that greater emphasis should be placed on model accuracy and relatively less on simplicity or ease of use. Dooge (1977) notes that there is little quantitative information available from which to make an objective selection using these criteria, especially with respect to model sensitivity and limitations. Discussions are provided for only the first three criteria because of the lack of references that address the latter criteria.

Ease of use

A basic rule has long been expressed in modeling: Adopt the simplest model that will provide acceptable results. This advice is provided, not only to ease the potential amount of work, but also to reduce the potential for model misuse. McPherson (1978) states:

> "There is evidence of simulation failures that have been attributed to model inadequacies where the blame more properly belonged to improper handling by the user because of the insufficient comprehension of the complex processes involved. Thus, the first cardinal rule is that simulation techniques adopted should not exceed the level of mastery of such tools by the user."

Therefore, the user should become sufficiently familiar with the model to understand the hydrologic implications of the modeling. The ease of application will also depend upon the individual model user. The experience of the modeler (both in the use of the model and the knowledge of the watershed) is of paramount value in both the ease of model application and in the reliability of model results (WMO, 1975).

Normally, ease of application is most closely associated with model complexity. Both Abbott (1978) and Franchini and Pacciani (1991), in their comparisons of models, note that the simpler models generally require the least effort to apply. Of the seven models tested by Franchini and Pacciani (1991), the TANK model, which was considered to be conceptually weakest, required the least effort calibration and the Stanford and Sacramento models, considered to most accurately represent the hydrologic exchanges involved in runoff formation, required the greatest effort. Franchini and Pacciani indicate that for the more complex models, "the handling of many parameters which affect the behavior of these exchanges therefore remains without control and difficult to check, which greatly increases the number of attempts that must be made before acceptable results can be achieved."

Comparative Accuracy of Models

Five studies were identified in the literature that quantitatively compare the performance of different models on the same watershed (WMO, 1975; Franchini and Pacciani, 1991; Abbott, 1978; Melching et al., 1991; Loague and Freeze, 1985). These studies are discussed individually below. Similar comparative studies have also been conducted for evaluating urban runoff models (Heeps and Mein, 1974; Papadakis and Preul, 1973; Marsalek et al., 1975). From the results of these earlier urban runoff model comparisons, McPherson and Zuidema (1977) concluded: "comparisons of different models have provided varying results and are mostly inconclusive and therefore controversial." However, the authors believe the following conclusion can be drawn from the five studies described below: Most rainfall-runoff models, when properly applied, will predict streamflow with similar accuracy.

The study by the World Meteorological Organization (1975) attempted to compare simulations from a wide variety of rainfall-runoff models on six large hydrologically dissimilar watersheds. Models used in this comparison range from simple black-box linear system models to the more complex Stanford Watershed Model. The comparison of models was partially hampered by incomplete data sets, which made it difficult to apply many of the models. For the watersheds in wet climates, all of the models appeared to do a similar job. For dry climates, the models with explicit accounting of soil moisture (such as the Stanford and Sacramento watershed models) were considered superior.

Franchini and Pacciani (1991) provide a detailed descriptive comparison of seven conceptual models, ranging from a simple linear-reservoir conceptual model to the more complex Stanford Watershed Model. The models are applied to a watershed in Italy, and almost all the models calibrate to match the recorded discharges with very similar accuracy. Franchini and Pacciani indicate that, even though the accuracy of the models was similar, the simpler models required considerably less time to achieve good results.

Abbott (1978) applied three event models (HEC-1, SWMM, and MITCAT) and three CS models (HSPF, STORM, and SSARR) to a small urban watershed in California. The results indicated that each model calibrated with similar accuracy and, when applied to a separate portion of the available storm data, validated with similar accuracy. One significant aspect of this comparison is that, in the validation, the event models used an "average" antecedent moisture condition, yet predicted the hydrograph peaks and volumes with similar accuracy to the CS models. The lumped models' accuracy was similar to that of models having a greater distribution of parameters.

Melcbing et al. (1991) applied the HEC-1 lumped system model and the quasi-distributed RORB event model (Laurenson and Mein, 1985) to a central Illinois watershed, and compared the output reliability. The two models produced flood hydrograph predictions of equal reliability, both for calibration and and validation events. In conclusion, Melching (1991) notes that "the common assumption that distributed system (and quasi-distributed) models provide greater accuracy and reliability than simple lumped system models may be violated when the input data are not sufficiently distributed."

A widely cited study by Loague and Freeze (1985) compares predictions of a "quasi-physically based" model, a black-box model, and a simple conceptual model. This comparison indicates that, for several watersheds, the physically based model performs poorly, and that the simpler, less data-intensive models provide comparable or better predictions. However, the "quasi-physically based" model used by Loague and Freeze (1985) is a partial model that contains no subsurface flow components and predicts streamflow only via Horton overland flow. One of the watersheds to which Loague and Freeze applied this model has high rates of infiltration and a low probability of obtaining overland flow. Beven (1989) questions the validity of this application and thus the comparison of models. According to Beven (1989): "Loague and Freeze (1985) recognise the incongruence of the model and the data in this case, but it is surely and indictment of hydrological practice to even consider applying such a model to such a catchment."

Most of the additional information dealing with comparative accuracy is subjective, but also implies that most models calibrate with similar accuracy. Troutman (1985) states that the accuracy of a model is, to a large degree, a function of the data inputs into the model. The needed to provide accurate simulation results is normally insufficient, and many of the limitations of a model that might affect accuracy are usually not approached (McPherson and Zuidema, 1977; Shafer and Skaggs, 1983). The insufficiency of data partially explains why an increase in the level of model complexity does not translate into increased model accuracy (Beven, 1989; Anderson and Burt, 1985; Todini and Wallis, 1977).

Consistency of Model Parameterization

As discussed earlier, parameter consistency has been most directly related to the level of physical interpretation associated with the parameter (Troutman, 1985; Gan and Burges, 1990; James and Burges, 1982a; Larson, 1973; Beven, 1985). These studies indicate that distributed parameters have the greatest potential to be physically interpreted and consistent; when this is the case, greater confidence can be placed in the use of the model for prediction and extrapolation. Research has indicated that the "best-fit" calibration of a conceptual model may yield several distinct sets of parameters, each yielding similar calibration error (Melching et al., 1990). When this occurs, parameters lack consistency and direct physical interpretation.

Consistency is essential if parameters are to be regionalized (Troutman, 1985). Although the lack of parameter consistency may not affect the model's ability to simulate runoff under circumstances similar to those used for calibration, it will greatly impair the model's use for extrapolative conditions, which includes transferability of the parameters to ungaged sites. Examples of the difficulties in regionalizing inconsistent parameters are given by Magette et al. (1976) and Weiss and Ishii (1987).

Kundewicz (1986) sees regionalization as an emerging topic in hydrology and therefore expects a gradual movement toward the use of distributed, physically interpretable parameters in rainfall-runoff models. One approach for the development of consistent parameters is the calibration of model parameters based on multiple objective criteria, such as attempting to match observed patterns in soil moisture, overland flow, channel streamflow, and other measurable quantities. Haan (1988) and Yan and Haan (1991) report that parameter values estimated using multiple objectives are more likely to be reliable than parameter values based on one or two criteria such as flood peak, flood volume, or both.

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III. WATERSHED PRECIPITATION

Types of Precipitation Input

Historical precipitation is used to calibrate all types of rainfall-runoff models, either on an event or continuous simulation (CS) basis. However, when using model output to determine flow conditions for infrequent events, other types of precipitation input, such as design storms, are often used. The National Research Council (1988) identifies three basic classes of precipitation inputs that can be used in rainfall-runoff models to determine flow-frequency: 1) historical precipitation; 2) synthetic precipitation, including both a) standard design storms and b) transposed storms; and 3) stochastically generated precipitation. A summary of the use of these different precipitation classes for estimating infrequent events follows.

1) Historical Precipitation

Historical precipitation is commonly used in CS models to extend the total runoff record, which includes the annual series of peak flows. In this procedure, simultaneous rainfall and runoff records are used to calibrate the model. The rainfall record from an extended period is then used as model input, and the model simulates a corresponding sequence of runoff. The observed and simulated runoff records are normally used jointly to provide a longer-term annual flood series. The total length of the observed and simulated records of runoff is generally shorter than the return intervals of the flood peak (or flow volume) needed for either design or stormwater management. Therefore, standard frequency analysis or some other extrapolation technique is normally required to estimate the peak discharge and flow volume of the more infrequent flood events. Problems associated with frequency analysis of model output is discussed in section VI: Flood Frequency Methodologies. The greatest advantage in the use of historical precipitation to analyze stormwater response is that it presents a variety of scenarios of both antecedent conditions and precipitation intensity within the storm. This helps provide an understanding of the types of storms that are likely to result in severe flooding.

2a) Synthetic Precipitation / Design Storms

Design storms are synthetic rainstorms of a predetermined quantity, duration, temporal distribution, and frequency. When simulating rainfall-runoff using a design storm, the simulated peak discharge is often assumed to have a recurrence interval equal to that of the design-storm rainfall. This assumption is not necessarily valid (Voorhees and Wenzel, 1982; Niemczynowicz, 1984), because it does not account for the associated probabilities of the rainfall-loss processes and other antecedent factors such as streamflow and detention storage. But in cases where these are not major factors, soil moisture conditions can generally be chosen to provide a peak flow with the same return period as the design-storm rainfall (Wenzel and Voorhees, 1984).

Synthetic design storms were originally developed for use on urban and small rural watersheds where the time of concentration was generally less than 3 hours (Hicks, 1944; Jens, 1948; Keifer and Chu, 1957). In their landmark paper, Keifer and Chu (1957) indicate that the synthetic design storms are an appropriate input for watersheds where the hydrograph peak is most closely related to the rainfall intensity within the *critical period*, the period of most intense rainfall over a specified duration of the storm. This holds only when antecedent factors such as soil moisture, streamflow, and detention storage have a relatively small impact on flow levels. Keifer and Chu (1957) warn that:

> ". . . the rainfall and resulting runoff hydrograph should be analyzed in order to determine what other characteristics, besides the average rate of runoff falling within a given time period, would tend to affect the peak rate of runoff."

Feldman (1979) indicates that problems may occur in the application of synthetic design storms with watersheds for which a series of storm events may lead to the critical flood. For this reason, synthetic design storms are primarily applicable to watersheds where the storage effects are minimal, and become less appropriate for watersheds in which storage can affect peak flows. Despite these conceptual limitations, design storms are widely used for modeling infrequent flood events in all types of watersheds.

<u>Use of Appropriate Rate-Duration-Freauencv Data</u>. Under certain climatic situations, or with certain types of watersheds, severe floods will not generally occur in the same season in which rainfall intensity is greatest (Bell, 1968). In Illinois, for example, summer is frequently the season with the most intense rainfall. Yet for many watersheds, large floods may be caused by less intense rainfall that occurs when the antecedent soil moisture is wet. If the annual rainfall rate-duration-frequency curve is always used for design storms, then the initial antecedent conditions may have to be artificially reduced to produce floods with frequencies similar to the rainfall frequency. Some of the earliest applications of synthetic design storms recognized this seasonal difference in rainfall rates, and used frequency data developed only from storms within the typical flood season (Williams, 1943).

<u>Temporal Distributions of Design Storms</u>. Design-storm hyetographs are usually based on either a geometric function of rainfall versus time (for example, the uniform and triangular distributions), or on temporal patterns based on the intensities of observed precipitation. The synthetic design-storm hyetographs that are developed using observed rainfall intensities can be representative of either "average" or "extreme" storm events. Most

of the rainfall distributions based on observed heavy rainfall -- such as the HEC-1 Standard Project Storm (U.S. Army Corps of Engineers, 1990), SCS Type-II storm (U.S. Soil Conservation Service, 1972), and Chicago method (Keifer and Chu, 1957) -- arrange the periods of most intense rainfall so that they are nested in the center (or other portion) of the storm. Marsalek (1978) questions the use of this type of temporal distribution, noting that: "all the values represented by a particular intensity-duration-frequency curve are implied to belong to the same storm, whereas such curves are typically obtained by a synthesis of data from a large number of storms." In other words, the distributions do not represent the precipitation profile that would be expected from any naturally occurring storm event.

The Huff temporal distributions (Huff, 1967; 1980) are based on Illinois rainfall data. Each Huff distribution represents the "average" hyetograph of a selected set of heavy rain events. Separate distributions are provided for point rainfall and areal coverage (50 to 400 square miles). The Huff curves attempt to maintain similar intrastorm temporal patterns of rainfall that are observed in actual heavy rainstorms, yet Marsalek (1978) notes that there is still "some degree of arbitrariness in the definition of these storms, particularly in the choice of the storm duration, which affects the magnitude of the rainfall intensities."

2b) Synthetic Precipitation / Transposed Storms

Transposed storms are historic precipitation events that have occurred outside of the watershed of interest, but within a region believed to have similar meteorological conditions. The methodology of transposed storms is based on the assumption that the transposed rainfall event is from the same population of storms that influence the watershed of interest. In this manner, there is a "substitution of space for time" (NRC, 1988). In ordinary applications, the temporal and spatial characteristics of rainfall from the transposed storm are duplicated as if that storm had occurred over the watershed of interest. But a major application of transposed storms is the estimation of the probable maximum precipitation (PMP), in which case the total precipitation in the observed storms is adjusted by the ratio of moisture potential between the watershed of interest and the original storm location.

Foufoula-Georgiou (1989) classifies the application of transposed storms as following either a *deterministic* or *probabilistic* approach. The two approaches are identical, except the latter examines attempts to identify a probability of occurrence to the storm. The fundamental interest in applying transposed storms is to determine the effect of heavy rainfall on flooding, and in most cases to determine the frequency of that flooding. For extreme events, the frequency of flooding may be difficult to estimate without a probabilistic evaluation of the storm's occurrence over the watershed. Among the many factors that may affect the probability of that storm occurring over a watershed are the average rainfall and areal coverage of the storm, the storm duration, the temporal distribution of the storm, the size of the watershed of interest, and the location of the storm center over the watershed (Foufoula-Georgiou, 1989).

Methodologies for estimating the probability of transposed storms based on these factors dates back to Alexander (1963). More thorough developments are given by Gupta (1972) and Foufoula-Georgiou (1989). But Foufoula-Georgiou (1989) indicates that these methodologies are not well developed and several difficulties remain in their application. Research by Foufoula-Georgiou and Wilson (1990) begins to better define the crucial storm characteristics that affect the probability of the extreme events.

The majority of applications of transposed storms are for determination of the PMP, which is a deterministic approach. Deterministic applications can also be useful for studying the general rainfall-runoff relationship of watersheds for heavy storms (Bradley and Potter, 1990). There have been relatively few applications of the probabilistic methods. The National Research Council (1988) cites two such applications by the Tennessee Valley Authority and the Yankee Atomic Electric Company in Massachusetts, used for estimating extreme floods on large watersheds. [Extreme floods are considered to be floods having a recurrence interval much greater than 100 years.] Richards and Westcott (1986) present a similar application for estimating a 1,000-year rainfall event in Massachusetts. Fontaine and Potter (1989) applied a general probabilistic approach (presented in NRC, 1988) to a Wisconsin watershed, and concluded that the conceptual framework of transposed storms needed to be further developed, and a method to estimate uncertainties be defined, before routine application would be possible. The most complete storm transposition application appears to be presented by Cluckie and Pessoa (1990), which uses radar-derived storm data with a transposed storm methodology for application in Great Britain.

The application of probabilistic storm transposition requires some information on the spatial distribution of the storms (Cluckie and Pessoa, 1990). This may limit the number of storms for which adequate data is available, especially for convective precipitation that occurs on a smaller spatial scale. Dawdy and Lettenmaier (1987) suggest that spatial variability may not be crucial for small watersheds, but indicate that the determination of which watersheds are "small enough" is a research topic in itself. The storm catalog published by the U.S. Army Corps of Engineers (1945-present) provides depth-area-duration information for storms covering large areas, although Richards et al. (1988) and Foufoula-Georgiou and Wilson (1990) indicate that this data is reconstructed, nondetailed, and can contain large errors. Future use of radar information, supplemented by more abundant raingage information, may be the most successful method for determining the spatial characteristics of a number of storms. A number of transposed storms will usually be needed for modeling any one watershed, so that a range of temporal and spatial patterns, rainfall depths, and corresponding antecedent watershed conditions can be simulated. The authors note that antecedent conditions can be a significant factor affecting the probability of the predicted flood resulting from transposed storms (or any other type of rainfall input.) Because transposed storms are historical events, the climatological data needed to approximate the antecedent conditions are ordinarily available.

Though the use of transposed storms would provide a set of extreme runoff events, the frequency of the runoff events would still not be known. This is complicated because the type of storm rainfall used to produce a peak runoff similar to the 100-year peak flow is likely to vary with the size of the drainage basin and other watershed characteristics. Another consideration is that the storm that produces the 100-year peak flow may not be the same storm that produces a 100-year flood volume. Guidelines would have to be developed to determine expected runoff frequencies for selected storms and particular types of watersheds.

3) Simulated Precipitation

Conceptually, simulated precipitation is the most desirable input for rainfall-runoff modeling for two reasons - it has the potential to address the spatial dynamics of storms; and, more importantly, the use of a long, stochastically generated series of rainfall avoids the need for analysis of the frequency of simulated runoff. The types of models that have been developed to simulate the rainfall process vary from purely stochastic models, which attempt to duplicate the statistical properties of observed rainfall, to multidimensional space-time models, which are also stochastic but have some conceptual parameters and a structure based on regularities in observed rainfall. Stochastic models simulate rainfall for either one or several locations in the watershed. Space-time models attempt to describe the rainfall at all points within the area.

Gupta and Waymire (1979) describe the basic structure of rainfall as composed of five spatial levels: 1) the synoptic area, 2) large meso-scale area (LMSA), 3) small meso-scale area (SMSA), 4) cell clusters, and 5) individual convective cells. Associated with each new level is an increase in rainfall intensity and a decrease in the spatial coverage and life span of the rainfall. The spatial coverage ranges from greater than 10,000 square kilometers (3,800 square miles) for the synoptic area to less than 30 square kilometers (10 square miles) for each convective cell; the respective life spans of the rainfall last from several days to less than one hour. The application of a particular model depends, to a great extent, on whether the level or resolution for which the model was developed matches the rainfall data available at a site as well as the types of rainfall that are likely to cause flood events. For example, models based just on convective rainfall will not be appropriate for use in watersheds where peak runoff occurs from cyclonic rainfall systems.

Purely Stochastic Precipitation Models. Most mathematical modeling of precipitation has dealt with the temporal processes at a point. Many of the earlier temporal stochastic models that were developed in the 1960s and 1970s analyze the daily rainfall process using simple, statistically independent processes (such as Poisson distributions) that have little temporal persistence. More recent investigations of temporal models have concentrated more on stochastic modeling of rainfall in convective cells and cell clusters, at either hourly or continuous time scales. Rainfall at these shorter time intervals is much more dependent in space and time, and its modeling has required development of a whole new set of cluster model processes. Many of the mathematical assumptions used in these temporal models are reviewed by Amorocho (1981), Waymire and Gupta (1981), and Waymire (1985). Some of the more recent investigations are briefly described by Georgakakos and Kavvas (1987).

Temporal models have also been applied to multiple locations through the use of spatial correlation (Bras et al., 1985; Franz et al., 1989). Neither of these two applications attempted to spatially correlate the temporal distribution from one location to another. Bras et al. (1985) spatially correlated the estimate of the total storm rainfall at each point to the total rainfall at all other locations; however, the intrastorm distribution is totally independent. Franz et al. (1989) provided a procedure to maintain the probability of rainfall occurring jointly between pairs of stations. In each case, the rainfall was spatially simulated using a lumped approach over large watersheds (greater than 1,000 square miles). The temporal resolution of rainfall for the two studies was 6 hours and 1 hour, respectively.

The use of simulated rainfall for the runoff modeling, especially with small- to medium-sized basins, requires small time increments of rainfall. However, very few of the stochastic models have a temporal resolution smaller than one hour. Stochastic rainfall models, like any other type of statistically based models, extrapolate extreme conditions only with a great deal of uncertainty (Franz et al., 1989). Most rainfall simulation models have problems with temporal aggregation, and cannot maintain the statistical relationships between long-term or short-term precipitation (Rodriguez-Iturbe et al., 1984). For this reason, Franz et al. (1989) do not recommend their model for smaller watersheds where short-term rainfall is of consequence:

> "... the method can be used for extrapolation to large floods in any situation involving the river flooding from a watershed large enough to satisfy the requirement that the flood is not significantly affected by any one hour of rainfall... in general the drainage area should be at least several hundred square miles."

Short-time period stochastic models are being developed that attempt to simulate the internal temporal structure within individual convective rainfall events (Garcia-Bartual and Marco, 1990; Kavvas and Herd, 1985). These models do not appear to be sufficiently well developed or tested for application in the near future. More importantly, they do not solve the concern pertinent to rainfall-runoff modeling, i.e., the development of a long-term simulated rainfall record. An alternative approach is the disaggregation of simulated daily rainfall into continuous rainfall, using a model such as that developed by Hershenhorn (Hershenhorn and Woolhiser, 1987) and applied by Econopouly et al. (1990). However, it is not clear from these studies whether the model maintains accurate relationships of rainfall intensity for short time increments.

Space-Time Models. The major trend in the modeling of rainfall has shifted to space-time models of mid-latitude cyclonic rainfall (Rodriguez-Iturbe et al., 1984). Georgakakos and Kawas (1987) report that new concepts are continually evolving, a fact that tends to support the contention that the development of these space-time models is still in its infancy (Franz et al., 1989). The WGR model, developed by Waymire et al. (1984), is one of the most complete models and simulates the cyclonic rainfall at a combination of the LMSA level with the smaller convective cell and cell-cluster levels. While the incorporation of meteorological features into these models makes them more realistic, it also makes them mathematically more complicated.

The structure, complexity, and parameterization of either the stochastic or space-time models depend greatly on the temporal scale of modeling and the type of rainfall event being modeled (Rodriguez-Iturbe et al., 1984; Valdes et al., 1985). Space-time models, like stochastic models, may have accuracy problems in the temporal and spatial aggregation of the simulated rainfall (Rodriguez-Iturbe et al., 1984; Valdes et al., 1985; Gupta and Waymire, 1990). Valdes et al. (1985) report that their model preserves the stochastic structure of rainfall for the temporal scale at which the parameters were estimated, but for a longer time scale "may produce synthetic values which do not reproduce the moment characteristic of the historical data." The model presented by Sivalapan and Wood (1987) has the opposite problem — it preserves the rainfall statistics for the longer, aggregated time periods but not the shorter periods for which the parameters were correlated. Nevertheless, the space-time models appear to present a more realistic simulation of precipitation patterns than do most of the stochastic models (Valdes et al., 1985).

In previous reviews (AGU Hydrology Section, 1984; Georgakakos and Kawas, 1987; NRC, 1988), it is stated that a great deal of research is needed before the space-time models can adequately represent infrequent storm events. Recent advances have not changed this overall evaluation; however, studies in the estimation of parameters for these models have started to fill a major void toward eventual application of such models.

Parameter Estimation for Space-Time Models. A major need for the practical application of space-time models is a standard methodology for estimating parameters, i.e., tuning the model to a specific location and type of rainfall (Rodriguez-Iturbe et al., 1987). The parameter estimation process can be extensive (Georgakakos and Kavvas, 1987), and any attempt at parameter estimation should involve a detailed understanding of the physics and statistics of the rainfall process (Islam et al., 1988). There are only a few studies addressing parameter estimation. Initial studies at parameterization for the nine-parameter WGR model have been accomplished for air-mass thunderstorms in Arizona (Islam et al., 1988) and tropical storms in the Atlantic Ocean (Valdes et al., 1990). In the first study, a dense raingage network of hourly rainfall was used for parameter evaluation. Valdes et al. (1990) used radar images to develop the spatial distribution. For each study, the success of parameter fitting was verified by the ability of the model to reproduce the main statistical properties of the original data set. Smith and Karr (1985) have also estimated parameters for their daily, air-mass thunderstorm model, for use in Virginia.

Valdes et al. (1990) state that a dense raingage network is needed to provide stable parameter estimates for space-time models, and attempts to use a normal raingage network for parameterization have resulted in significant estimation problems (Smith and Karr, 1985). Few locations have networks that can provide the spatial information required by the models, causing a major restriction in their application. Radar information can be used to provide spatial information, such as in the study by Valdes et al. (1990), but the evaluation of radar data produces large errors when estimating rainfall intensity (Krajewski and Georgakakos, 1985). Several recent investigations (Eddy, 1979; Kawas and Herd, 1985; Smith and Krajewski, 1987) have attempted to blend the reliable point values of rainfall offered by raingages with the spatial information provided by radar.

Sources of Error in Rainfall Estimation

The rainfall input into models is usually accepted as a measured or definite areal value. However, the rainfall input consists of measurements for only a limited number of points within the watershed. The use of these few values to represent conditions over an entire watershed provides for only a rough estimate of the observed conditions. The rainfall input into watershed models is often the largest source of error in the modeling process because point measurements fail to accurately represent the watershed rainfall (Berndtsson and Niemczynowicz, 1988; Schilling and Fuchs, 1986).

Three factors, which are major sources of error, are not accounted in the estimation of rainfall: 1) the temporal variation of rainfall at any point in the watershed, 2) the spatial rainfall distribution, and 3) the movement (direction and speed) of the rainfall

(Niemczynowicz, 1984). Failure to represent any of these factors affects model calibration, and consequently influences the entire rainfall-runoff modeling process. When the temporal and spatial distribution of rainfall is not represented, the model's ability to calibrate to observed streamflow is reduced, the model may calibrate to unrealistic parameter values, and the ability to model a range of conditions in a complex watershed is compromised (Troutman, 1983). The latter effect defeats the major advantage of using distributed or quasi-distributed models. Because the precipitation input is the most basic component in the modeling process, it is unlikely that the accuracy of rainfall-runoff modeling efforts will dramatically improve until these errors are addressed in some manner (Berndtsson and Niemczynowicz, 1988).

Temporal Distribution of Rainfall

The effects of the temporal distribution of rainfall on model output have been demonstrated by numerous investigators (Aron, 1989; Dawdy and Bergmann, 1969; James and Robinson, 1985; Knapp and Terstriep, 1981; Lambourne and Stephenson, 1987; Paine, 1989; Schilling, 1984; Wei and Larson, 1971; Wenzel and Voorhees, 1984; Wilson et al., 1979). Model output for small watersheds, which are particularly responsive to differences in rainfall intensity, will show the greatest differences. Studies by Wenzel and Voorhees (1984) and Knapp and Terstriep (1981) indicate that a 30 to 50% variation in peak flows, caused by using different design-storm hyetographs, is common for watershed sizes less than 10 square miles. The disparity in flood peaks is due to the differences in the peak rainfall intensity between selected design storms, which can be as great as 400% (SCS Type-II compared to the 24-hour Huff distribution), as well as to differences in the amount of precipitation that precedes the peak intensity. Using different temporal distributions will cause variation in the total runoff volume estimated by the rainfall-runoff model, but the difference between the estimated volumes is unlikely to exceed 10% (Wilson et al., 1979).

For watersheds with minimal storage effects, Keifer and Chu (1957) indicate that the flood peak is closely related to the total rainfall occurring within the most intense, *critical period* of a storm; and that temporal variations within this critical period have little impact on the estimate of the flood peak. The duration of this critical period is dependent on watershed characteristics such as drainage area, slope, and channel length. Simulations by Knapp and Terstriep (1981) appear to support Keifer and Chu (1957), but only for cases when all antecedent conditions leading into the critical period of the storm are the same. Most design storms have fixed durations and, for a large number of applications, a portion of the storm will precede the critical period. The total rainfall that precedes the critical period can be a significant factor in determining the flood peak because it modifies the antecedent moisture conditions. When the critical period of most intense rainfall occurs later in the design storm, the rainfall preceding the critical period is increased and flood peaks become greater (see examples in Marsalek, 1978; Wenzel and Voorhees, 1984; and Knapp and Terstriep, 1981).

Delleur (1982) states that design storms with a particular frequency will likely produce different flood peaks than those resulting from an actual storm ~ just by nature of the differences in temporal distribution. Runoff simulations performed by Marsalek (1978) compared runoff peaks on a small, hypothetical urban watershed using both design storms and an annual series of actual storm events as model input. Peak flows simulated using the . Chicago design storm were up to 50% greater than those simulated using the observed storms. The Huff design storm simulated by Marsalek (1978) also overestimated peak flow but to a lesser degree. A similar study by Beaudoin et al. (1983) produced variable results depending on the frequency of the flood being considered.

Spatial Distribution of Rainfall

The spatial distribution of rainfall can easily cause an even greater difference on the modeled runoff than the temporal distribution, even on small watersheds (Niemczynowicz, 1988). The additional information supplied by the spatial distribution of rainfall can improve the runoff estimates in two ways: 1) by more accurately estimating the depth of rainfall over the watershed and 2) by distributing that depth of rainfall over various portions of the watershed.

Error in Estimating Watershed Rainfall Amount bv Using One Gage. The error in estimating total watershed rainfall from one or a small number of raingages can be substantial, and is usually one of the major sources of error in the runoff modeling process. The peak outflow simulated using only one gage in the watershed may frequently differ from the peak flow estimated using the "true" spatial rainfall by over 50%, even on small watersheds less than 10 square miles (Colyer, 1982; Niemczynowicz, 1988; Schilling, 1984). In simulation studies over a 3-square-mile watershed, Schilling and Fuchs (1986) report that errors caused by using a single raingage were normally greater than 20% and frequently greater than 50%. Furthermore, Schilling and Fuchs indicate that typical modeling techniques will propagate the difference in modeled versus observed rainfall: "On the average, a rainfall depth of 30% results in a volume error of 60% and a peak flow error of 80%."

The rainfall distribution represented by a single point tends to have a shorter duration and greater intensity than precipitation over an entire watershed (Berndtsson and Niemczynowicz, 1988). For example, figure 1 compares the hyetographs at two raingages with the average rainfall that may be computed for the watershed. The composite rainfall for the total watershed rainfall is likely to be longer and less intense than the rainfall at any one point. An overestimation of rainfall intensity over the watershed occurs when either one of

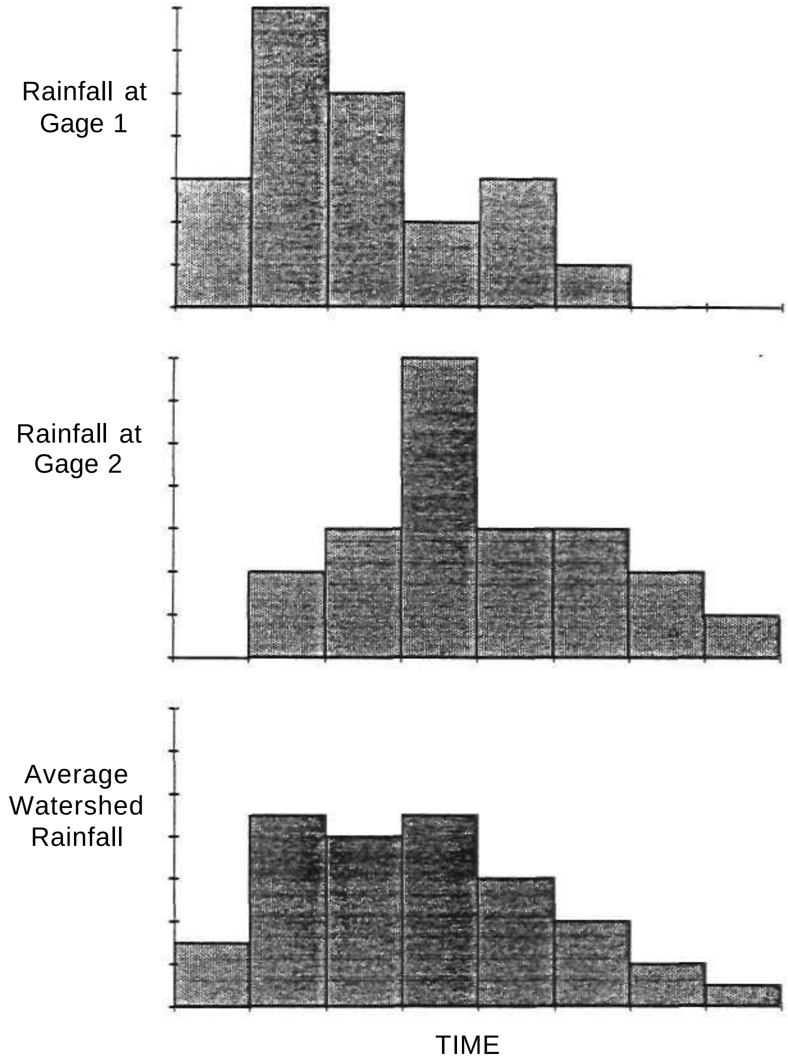

Figure 1. Comparison of Point Rainfall and Watershed Rainfall Estimates

these precipitation records is assumed to represent the total watershed rainfall. Therefore, in failing to account for either spatial distribution of rainfall or storm movement, the average rainfall intensity over the watershed is usually overestimated for large storms (Troutman, 1983). This is especially true for the high-intensity storms that result in infrequent flood events for small- and medium-sized watersheds. Also, if high precipitation is measured at any one gage, there is an increased probability that the measured rainfall is greater than the areal average . Conversely, if low precipitation is measured at the gage, there is an increased probability that it will underestimate the areal average.

When using a single raingage, a model with unbiased or "correct" parameter values will tend to overpredict large storm events and underpredict small storm events (Troutman, 1983). However, following normal calibration, model parameters are not unbiased but are adjusted to compensate for the bias in the precipitation measurements. The resulting, biased parameters will tend to overpredict rainfall losses for large storm events and underpredict rainfall losses for small storm events. The biased infiltration parameters will cause the underestimation of design floods and other large, infrequent flood events (Troutman, 1983; Yen, 1982). Thomas (1987) indicates that the annual series of flood events developed by using these biased parameters is likely to have less variability than the observed annual maxima. Troutman (1986) discusses a few techniques that may be used in the modeling process to reduce the bias in model parameters.

Bradley and Potter (1990) examine the effect that modeling errors, caused by using a single raingage, have on the estimates of flood frequency. In this study, a 13-year streamflow record was simulated using a dense raingage network, and then a number of different 13-year flow records were simulated using a selected raingage from within the network. Frequency distributions were fitted to the annual flood series for each simulated record. The 50-year peak flow estimates computed using the single raingage records ranged from 40% lower to 88% higher than that estimated using the dense raingage network. The standard deviation of the 50-year flood estimate was 25%. Bradley and Potter conclude that this is a fairly small error when compared to other modeling errors, such as the error in estimating infrequent floods from short streamgage records, and recommend the use of long-record raingages for simulation. Bradley and Potter note, however, that dense raingage data are likely needed for accurate model calibration.

Distribution of Rainfall Within a Storm. The effect of intrastorm variation in rainfall amount is often neglected because the spatial distribution of rainfall is not usually known. A common viewpoint is that the effects of areal differences in rainfall over a watershed are usually diffused as the flows are routed downstream, and therefore do not need to be modeled. This may be true for some medium to large watersheds. For example, in a study that modeled the Goose Creek watershed in eastern Illinois (47 square miles), Beven and Hornberger (1982) indicated that spatial rainfall differences affected the timing of the peak flow, but they didn't have much effect on the peak discharge. However, many other studies reach the opposite conclusion, especially when modeling urban watersheds. Wilson et al. (1979) state that "even in cases when the total depth of rainfall is not in serious error, the spatial distribution of the input, when not observed, may lead to large discrepancies in the volume of the runoff output." A study by Schilling (1984), which involves the modeling of an urban storm sewer catchment, indicates that the runoff process tends to amplify rather than diffuse the errors resulting from spatial rainfall differences. Each of the above studies focused on watersheds of less than 30 square miles. In a more theoretically based study, Milly and Eagleson (1988) state that the modeling of the spatial variability of rainfall should be even more critical for larger watersheds, because the likelihood that the storm may cover only a portion of the watershed is increased. Milly and Eagleson also concluded that one result of including spatial variability, relative to uniform rainfall, will be an increase in the total amount of surface runoff.

Unlike when dealing with the temporal distribution of storms, there are few examples on how to model spatially distributed rainfall. Areal reduction factors, such as those developed by Hershfield (1961) and Huff (1990), give no indication of how rainfall will vary within an actual storm, and in fact show much less areal variation than actual storms (Niemczynowicz and Jonsson, 1981). These areal reduction factors are estimated from a fixed point in the raingage network (Huff, 1990), which is not necessarily near the storm center (the point having greatest accumulation of rainfall). The fixed-point reduction factors are useful for design-storm applications because they represent the ratio between areal and point rainfall having the same frequency. But in reality, any storm - particularly convective storms -- will have areas within the storm that have significantly greater rainfall than the areal average, while other areas have little or no rainfall. The areal reduction factors given by Huff (1967), for "extreme storm gradients" in Illinois, are more typical of the reduction that might be expected from a storm center. Terstriep et al. (1988) reference two other studies (Osborn et al., 1980; Woodley et al., 1975), which provide storm-centered reduction factors for sites in Arizona and Florida, respectively. Eventually, with the application of radar images, there should be acceptable examples or "scenarios" of spatial variations to use in modeling. But for the present, a limited number of scenarios of storm spatial patterns may be available in records from dense raingage networks.

Storm Movement

Several studies (Ngirane-Katashaya and Wheater, 1985; Niemczynowicz, 1988; James and Robinson, 1985; Richardson and Julien, 1989; Sargent ,1981; Yen and Chow, 1969) analyze the effect of storm movement on peak runoff. Niemczynowicz (1988), in particular, strongly recommends the use of storm movement data as a source of significant spatial rainfall information. The first two studies, the most comprehensive ones, indicate that increases in peak runoff will result if the storm is moving in a downstream direction. This is especially true for urban watersheds as a result of their rapid response to rainfall. For most other storm orientations, stationary storms were shown to produce similar or slightly higher runoff peaks than moving storms. Ngirane-Katashaya and Wheater (1985) simulated an increase in peak flows of nearly 40% on a 7-square-mile urban watershed, assuming an optimal speed and orientation of a design storm. Niemczynowicz (1988), using multiple-gage historical rainfall and a variety of simulated speeds and directions, reported maximum changes in simulated peak flows of approximately 30% when modeled over an urban watershed. Both Richardson and Julien (1989) and Yen and Chow (1969) used a rainfall simulator over a laboratory model of a watershed, and reported storm movement effects remarkably similar to the runoff simulation models, with peak flow increases of 25 to 40% above stationary rainfall.

Niemczynowicz (1988), using recorded rainfall in Sweden, estimates that storm speed and direction information is as valuable as the information that may be gained by increasing the number of gages in a watershed. This is especially true if regional information indicates prevailing storm directions (Shearman, 1977). These studies indicate that storm speed and direction can be estimated from data on high-altitude winds. This wind information is available at most major airports and at National Weather Service offices.

Summary and Remarks

Each of the different types of precipitation modeling described above have their uses in one of several approaches to rainfall-runoff modeling (see Section VII, Modeling Approaches). The temporal and spatial characteristics of these four types of rainfall input are major determinants in the estimation of flood peak and flood volume, and these rainfall characteristics can be significantly different. The temporal character of transposed storms and the simulated precipitation either duplicates or mimics historical rainfall. Design storms have a temporal character not representative of a natural event. Transposed storms and space-time simulations contain a description of the rainfall spatial differences. Design storms typically have no information on spatial variability. If the watershed has more than one raingage, the historical rainfall input will provide some spatial variation. Stochastic simulation models usually provide only point rainfall, although there have been some multiple-location applications.

The long-term simulation of precipitation conceptually provides a very desirable input for defining infrequent flood events, regardless of whether CS or event modeling is used. However both stochastic and space-time simulation models have problems with the temporal aggregation/disaggregation of simulated rainfall. The stochastic simulation models appear to be closer to application, yet need better development of short time-increment precipitation for use on small- and medium-sized watersheds.

Deterministic applications of transposed storms can be useful for studying the general rainfall-runoff relationship of watersheds for heavy storms. Storm transposition generally requires spatial rainfall information, and dense raingage networks or radar imaging will likely be needed to identify these characteristics in smaller convective storms. The existing storm catalog (U.S. Army Corps of Engineers, 1945-present) provides spatial and temporal information for large storms, but may lack sufficient detail for applications to smaller watersheds. If the database of heavy rainfall becomes more complete and more detailed it should provide a valuable source of scenarios to evaluate the effect of spatial variability on watershed response. Research is needed to determine the watershed sizes for which spatial information may not be required. Probabilistic evaluations of transposed storms are likely necessary to estimate the frequency of the resulting flooding, particularly for extreme storms. The conceptual framework to estimate the probability of transposed storms requires greater study, but the National Research Council (1988) considers this technique to be promising for estimating probabilities of extreme floods. Procedures could be developed to use transposed storms with both CS and event models.

The literature illustrates that temporal and spatial distributions can have significant effects on flood hydrographs. But under what modeling circumstances will information on the spatial variability of rainfall be beneficial? Relatively few studies are available to help us understand how the flood estimates for certain types of watersheds might respond to spatial differences in rainfall and storm movement. It is likely that in some watersheds, neglecting to model the spatial variability may lead to poor model results; in other cases, the expected differences in runoff may not be great, and the potential increase in accuracy may be economically unjustified (Amorocho, 1981). It could be argued, understandably, that neglecting spatial influences may be reasonable when using design storms, or when modeling watersheds on a lumped-parameter basis. However, spatially inaccurate rainfall information also impairs modeling by affecting the calibration process, leading to poor estimates of the model parameters (Troutman, 1983).

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IV. RAINFALL LOSSES

Types of Rainfall Losses

The term "rainfall loss" refers to that portion of the total rainfall that fails to *directly* result in storm runoff. The rainfall losses are the primary determinant of the amount and distribution of runoff that result from an individual storm. This estimation of the rainfall loss is considered by many to be the most complex, and possibly least studied, component in the rainfall-runoff process. McPherson (1978) and Aron (1982) have considered rainfall losses to be the weakest link in the proper estimation of runoff; and others, such as Yen (1982) and Pilgrim (1986), have stressed that the study of rainfall losses should be given considerably more attention.

Rainfall losses can be divided into three processes: 1) the interception of rainfall by plants, 2) the retention and storage of water in depressions, and 3) infiltration of water into the ground. Together, interception and depression storage are significant factors in the overall water budget and, according to Goldman et al. (1990), could account for the major losses in up to 80% of observed storm events. However, infiltration is frequently the only process of the three to be simulated in rainfall-runoff models, especially with event models. There are two reasons for this type of treatment: 1) infiltration is the major rainfall loss during heavy accumulations of rainfall, and 2) the interactions between the various rainfall loss processes are not easily separated except through the use of detailed field studies (McPherson, 1978).

Failure to include interception and depression storage as processes in the model can produce significant errors, especially when modeling runoff from light to moderate rainfall. For example, depression storage can be substantial in agricultural watersheds, with maximum storage values up to 0.40 inch on deeply plowed fields (Goldman et al., 1990). Interception storage, generally less than 0.20 inch, is greatest in forested watersheds. As the rainfall quantity increases, the ratio of the interception and depression storages to the potential amount of infiltration decreases. Therefore, errors caused by incomplete modeling will be comparatively smaller for heavy rainstorms than for other rainfall events. These errors may reduce the potential accuracy for calibrating the watershed infiltration, regardless of which interception equation is used. Despite these potential errors, this review has chosen to focus on the infiltration process because of its widespread impact on all types of modeling.

Determination of Initial Conditions

Rainfall losses are highly dependent upon soil moisture conditions preceding the rainfall event, and these soil moisture conditions must be well-explained to properly model the loss-rate. All continuous simulation (CS) models have some type of accounting system to appraise soil moisture, which is then used as a variable in the estimation of infiltration. Evapotranspiration (ET) is by far the largest factor affecting the estimation of soil moisture. Daily values of potential ET are typically estimated either using daily pan evaporation data with a monthly adjustment factor, or with physically based or empirical equations that use meteorological variables. Saxton et al. (1974) and Allen and Pruitt (1986) suggest that any of these approaches appears to be reasonably accurate. In a more detailed evaluation, Jensen et al. (1990) indicate that the physically based equations are generally more accurate, and that pan evaporation data can lead to variable results: "Evaporation pan data are relatively easy to obtain and can be very reliable if the evaporation site is maintained in a suitable and consistent manner. Evaporation data collected in poorly maintained locations will not produce estimates as accurate as those based on good meteorological data." Daily and weekly ET values may vary significantly from average amounts. The use of either monthly averages of potential ET, or average values from year to year, will likely produce large errors in the estimated soil moisture, which will indirectly result in errors in the estimated infiltration.

Soil moisture is usually not simulated when using event models. Normally, initial conditions for event models are set in the calibration process by using an objective function or are subjectively adjusted by the user. A less commonly used alternative is to relate soil moisture to an antecedent precipitation index (API), which is used to relate the amount of cumulative precipitation preceding the storm event (typically for a 5- to 15-day period ahead of the storm) to the initial value of an infiltration parameter. Goldman et al. (1990) indicate that APIs are unlikely to adequately represent the soil moisture prior to a rainfall event because they ignore the effect of ET between storm events. Soil moisture measurements are rarely available to produce the initial conditions, although Goldman et al. (1990) present a good argument for their use. Explicit soil moisture models also exist that could be used in conjunction with the event models to provide antecedent conditions, but this approach requires additional modeling effort and is rarely used.

Equations for Estimating Infiltration

Eight common methods for predicting infiltration or rainfall losses in hydrologic modeling are identified in table IV-1. A list of stormwater or hydrologic models, referenced earlier in the literature review, in which these methods appear is also provided.

The Richards equation, a complex partial differential equation, provides the most accepted theory on how water moves into and through soil (Richards, 1931). But the Richards equation cannot easily be applied in modeling and therefore is normally approximated. Both

Table IV-1. List of Common Infiltration Equations and the Models That Use Them

Infiltration Method	Models
1) Green-Ampt equation	PRMS, HEC-1
2) Philip equation	---
3) HEC Exponential Loss Rate equation (HEC)	HEC-1
4) Initial/Uniform Loss Rate Function (IUF)	HEC-1
5) Horton equation	SWMM
6) Holtan equation	USDAHL-74, ANSWERS, and HEC-1
7) SCS runoff curve number	HEC-1, TR-20
8) HSPF infiltration equation	HSPF

the Green-Ampt and Philip methods are based on simplifications to the Richards equation. The basic assumption used in both methods is that infiltration occurs when water is ponded above the soil.

The HEC, IUF, Horton, Holtan, and SCS methods are simplified algebraic equations, most of which are developed from the empirical analysis of field data. The SCS method is different from the others in that it was designed for use with daily precipitation values and has no conceptual basis for use in describing intrastorm rainfall losses. The HSPF infiltration equation is based on the Philip equation, but has undergone extensive modifications (Hydrocomp, 1976) which qualify it as more of an empirical procedure.

A major difference between the equations listed in table IV-1 is the type of independent variables used to estimate the infiltration rate (table IV-2). Infiltration can be predicted either as a function of precipitation intensity, time, soil moisture storage, or a combination thereof.

Infiltration rates vary temporally, both between storms and within a storm, and spatially. Several studies (Tricker, 1981; Berndtsson, 1987) indicate that soil moisture storage is the most basic factor in determining the variability in infiltration rates, particularly the differences that occur spatially and between different storms. The effect of soil moisture storage can be described either implicitly (such as parameters related to accumulated rainfall

Table IV-2. Independent Variables Used in the Infiltration Equations

Method	Independent Variables
Green-Ampt equation	storage (explicit), time
Philip equation	storage (explicit), time
HEC Exponential Loss Rate equation	precipitation intensity, storage (implicit)
Initial/Uniform Loss Rate Function	precipitation intensity
Horton equation	time
Holtan equation	storage (explicit)
SCS runoff curve number	precipitation intensity
HSPF infiltration equation	storage (explicit)

loss) or explicitly as a volume of water in the soil. The World Meteorological Organization (1975) identifies the use of explicit moisture accounting as a particularly important component in rainfall-runoff models, and logic implies this is especially true for CS models. It appears that all of the widely available CS models use infiltration equations based on explicit soil moisture storage.

The HEC, IUF, Horton, and SCS methods do not use any measure of soil moisture storage. The Horton method uses the cumulative time of rainfall during the storm event as an empirical substitute of the soil moisture storage to determine the intrastorm variation of infiltration. Aron (1982) identifies the absence of a soil moisture variable as one of the major problems of applying the Horton equation from storm to storm. Tracy and Marino (1987) indicate that the two equations that are partially based on precipitation intensity (the SCS and HEC methods) may be unreliable when used for large storm events . Tracy and Marino explain that these methods should be applied when the rainfall intensities used for prediction are in the same range as that used for calibration. If the calibration and prediction rates are different, application of these equations may not be valid. One validation test of these methods by Tracy and Marino (1987) indicated that a 50% increase in precipitation intensity above the calibrated value resulted in poor predictions of infiltration.

Comparative Accuracy of the Infiltration Equations in Calibration

Numerous studies have compared the infiltration techniques by calibrating the parameters to field or laboratory measurements of intrastorm runoff (Berndtsson, 1987; Haverkamp et al., 1987; Idike et al., 1980; Maheshwari et al., 1988; Rawls et al., 1976; Skaggs et al., 1969; Swartzendruber and Young, 1974; Tracy and Marino, 1987). The results of these comparisons are discussed below. Of the eight infiltration equations cited only the HSPF equation was not compared in these studies, but its accuracy is expected to be similar to the Holtan equation. This comparison between the HSPF and Holtan methods is based on the observation that the rainfall loss for these methods uses a similar function of soil moisture within a given control depth. The Holtan equation has an additional term, f_c, which is the constant infiltration rate.

Table IV-3 indicates that the more empirical Horton and Holtan equations showed somewhat better calibrating abilities than the more physically based Green-Ampt and Philip equations. It should be noted, however, that the relative differences in the calibration accuracy between these four methods are usually neither great nor always clear-cut. Given the information from these studies, the four equations all appear to adequately model the intrastorm infiltration process. Because the Green-Ampt and Philip equations have the greater physical basis, they are often believed to more genuinely describe the infiltration

Table IV-3. Comparative Studies between Different Infiltration Equations

Source	Equation Comparison
Tracy and Marino (1987)	Horton, Philip >> SCS, HEC, IUF
Skaggs et al. (1969)	Holtan, Horton > Green-Ampt, Philip
Berndtsson (1987)	Horton > Philip
Haverkamp et al. (1987)	Philip = Green-Ampt
Idike et al. (1980)	Green-Ampt (Mein-Larson) > Holtan
Swartzendruber and Young (1974)	Horton, Holtan > Philip, Green-Ampt
Maheshwari et al. (1988)	Horton > Philip
Rawls et al. (1976)	Horton > Philip, Green-Ampt, Holtan

Notes:
= The equations have similar accuracy.
> The equations on the left are slightly more accurate than the equations on the right.
» The equations on the left are decidedly more accurate than the equations on the right.

process (Wilcox et al., 1990); however, the above comparisons indicate that this does not relate to improved accuracy. The reason why the more physically based approaches do not ordinarily provide better accuracy may be due to three factors:

1) Both techniques are designed to estimate infiltration during ponded conditions. These methods tend to overestimate infiltration during the early part of the storm, prior to ponding (Rawls et al., 1976; Tracy and Marino, 1987). Several modifications to the Green-Ampt approach have been developed to estimate infiltration prior to ponding (Mein and Larson, 1973; Smith and Parlange, 1978).

2) These physically based infiltration techniques may lose much of their physical representation when calibrated to soils that have macro-channels such as worm holes, root channels, and other cracks (Gish and Starr, 1983).

3) The parameter values set during the calibration process can be greatly affected by the spatial variability of both rainfall and infiltration, and to a lesser degree on errors in the measurement of precipitation and streamflow (Tracy and Marino, 1987). For this reason, it is difficult to obtain good, consistent values for the infiltration parameters. Even the parameters for methods designed to use measurable soil properties are usually altered in the calibration process.

According to the tests by Tracy and Marino (1987), the SCS and uniform-rate equations are likely to calibrate poorly to intrastorm infiltration. In particular, the uniform infiltration rate was the least accurate method that they tested. The HEC equation calibrated well but gave poor results when simulating the response to rainfall that was more intense than that used for calibration. Tracy and Marino (1987) warn that both the SCS and HEC methods may provide poor results when applied to rainfall intensities that are not in the same range as that used for calibration.

Transferability (Regionalization) of Parameters

Much of the discussion dealing with consistency of parameters, presented earlier in this review, applies most directly to the rainfall loss processes. Part of the problem with many of the infiltration equations is that calibration is required to attain reliable parameters, yet many of the model applications are for ungaged watersheds where calibration is not possible. Wilcox et al. (1990) point out that the "popularity of the SCS curve number method lies in the fact that it is simple to use, does not require calibration, and is purported to give reliable results." Wilcox et al. identify the development of a methodology not constrained by calibration to be one of the goals of rainfall loss modeling.

It is generally agreed that the transferability of a method will most likely be successful when the parameters can either be determined from measurable soil and land use properties or otherwise maintain physically realistic properties (Larson, 1973; Collis-George, 1977; Wilcox et al., 1990). These are the same characteristics that allow models to objectively evaluate the effects due to changes in land use. In addition, having physically realistic parameters acts as a control factor for avoiding biased model results. Yet the purpose of most rainfall-loss calibrating procedures has almost always been to come up with parameters that result in the best fit, not necessarily parameters that are physically realistic (Sorooshian, 1983). The development of regional parameters using physically realistic values is seen as a necessary step if watershed models are to be reliably applied to ungaged sites.

Several methodologies have been developed to relate rainfall loss parameters to soil and watershed characteristics. For example, Ewing and Mitchell (1986) developed a method to predict parameters for the Holtan equation from measurable soil properties. Applications by Dinicola (1990) indicate that the HSPF infiltration parameters, which are often considered fitted parameters, may maintain physically realistic values and apply in a regional context.

The Horton equation, which also calibrates well for specific storms, does not include parameters that are likely to have regional transferability. This is because the Horton equation considers infiltration only in terms of a function of time rather than of the soil moisture storage (Aron, 1982). Heeps and Mein (1974) state that calibration is the only adequate technique to develop antecedent conditions for the Horton method. There are relatively few studies dealing with regional parameterization of the Philip equation, but there is some indication that the sorptivity parameter is not easily predictable (Sharma et al., 1980).

Numerous studies in the past decade have been devoted to developing parameters for the Green-Ampt equation using soil texture information (Brakensiek and Onstad, 1977; McCuen et al., 1981; Rawls and Brakensiek, 1983). These studies provide objective techniques to describe the effect of land use, particularly agricultural management practices, on the infiltration capacity. For the most part, these parameter estimation techniques are shown to be reliable. Springer and Cundy (1987) note that the texture-based parameters may not give the same results as field-measured parameters. However, they also state that, because that study did not calibrate to observed streamflow conditions, it is uncertain which set of parameters provides the most accurate modeling results. Cundy and Hawkins (1987) indicate that parameters based solely on field measurements generally underestimate the total infiltration.

Both Van Mullem (1991) and Wilcox et al. (1990) compared model runoff using two noncalibrated rainfall-loss methods - the SCS curve number method and the soil texture Green-Ampt parameters. The results of Van Mullem (1991) indicate that the Green-Ampt method more accurately predicted runoff volume than the curve number. Van Mullem recommends the Green-Ampt model because it is physically based, has measurable parameters that can be easily estimated from soils data, is appropriate for evaluating changes in runoff resulting from agricultural practices and urbanization, and can use objective estimates of antecedent soil moisture. Wilcox et al. (1990) found that the Green-Ampt and curve number methods gave comparable results. "While this may not seem like a great accomplishment, it should be understood that the curve number method uses a mature, well-tested methodology, while the estimation of Green-Ampt parameters is a young process and will likely improve with time" (Wilcox et al., 1990). The U.S. Soil Conservation Service is evaluating the use of the Green-Ampt model and working to better quantify factors affecting the equation's parameters (Miller and Cronshey, 1989). Having an extensive knowledge base of applications for various watersheds is an important factor in the development of reliable parameters.

Methods in Regionalization

In any regionalization analysis, it should be noted that calibrated parameters are highly influenced by model input errors, such as those introduced by precipitation variability. These errors can make it difficult to establish regionalized parameters even in areas that have a homogeneous infiltration character (Troutman, 1985). Typically, a regionalization process begins by calibrating a set of parameter values for each watershed. A regression analysis is then performed to predict the calibrated parameters values from various watershed characteristics, which produces a "regression-upon-regression" approach. Any errors introduced in the calibration will likely be compounded in this type of regression procedure. These errors may be particularly great if a "best-fit" calibration is used and leads to some unrealistic parameter values (Todini, 1988). The authors of this review recommend that parameter regionalization should rely upon an objective function that attempts to minimize the total error between observed and estimated streamflow for all stations, instead of attempting to match the "optimal" parameter values calibration for each watershed.

More subjective approaches to regionalization can be used to define model parameters based on knowledge of the hydrologic responses between different soil types and land use. For example, in the calibration procedure, the model user can limit the value of the infiltration parameter for a particular soil to an acceptable range, congruent with the values for all other soils. In this manner a subjective analog is established between the understood hydrologic response and the model parameters, and a consistent hydrologic response between the various soil types is maintained. This approach was used by Dinicola (1990) to develop regional parameters in applying the HSPF model near Seattle, Washington. This type of approach requires the model user to have considerable knowledge of both the sensitivity of model parameters and the rainfall-runoff hydrology of the watersheds being modeled.

Spatial Variability of Infiltration

In addition to the problems of defining infiltration parameters is the spatial variability of infiltration expected within individual soil types. The factor that most greatly affects this spatial variability is the difference in soil moisture between sites (Berndtsson, 1987; Tricker, 1981). Owe et al. (1982), for example, indicate that soil moisture differences within a given soil type can vary by as much as 25%. Additional factors affecting spatial differences within an individual soil type are the amount of (and differences in) the vegetation cover (Berndtsson, 1987), the presence of roots and cracks in the soils (Gish and Starr, 1983), and the thickness of the litter layer at the surface (Tricker, 1981; Merzougui and Gifford, 1987). But frequently a large portion of the spatial variability cannot be explained by these factors, and must be considered random (Sharma et al., 1980; Tricker, 1981; Gish and Starr, 1983).

The variability of infiltration within a soil type may be described by the coefficient of variation (CV), which is the standard deviation of the sample of measurements divided by the mean of the sample. Information on the expected CV values is available from only a limited number of field studies (NCRC, 1979; Hawkins, 1981; Aboulabbes et al., 1985; Ben-Hur et al., 1987; Rogowski et al., 1987). These studies indicate that CV values are typically greater than 50%, ranging from under 20% on several types of soils to 185% for a compacted clay soil (Rogowski et al., 1987). The infiltration data given by the North Central Regional Committee (1979) provides information for selected agricultural soils in Illinois and the Midwest, typically having CV ranges from 30-50%. Several investigations suggest that the variability of infiltration on any given soil type displays a log-normal distribution (Sharma et al., 1980; Sivalapan and Wood, 1986; Ben-Hur et al., 1987; Haan, 1988).

Because of the expected variation in infiltration, considerable errors in estimating both infiltration and runoff can result when the infiltration rate for a given soil type is simply represented using a set of "average" parameter values (Sivalapan and Wood, 1986; Haan, 1988;

Sharma et al., 1980). Both Bresler and Dagan (1983) and Springer and Cundy (1987) indicate that a single effective parameter set for describing the infiltration response of a field with spatially varying parameters does not exist. Therefore a distributed approach is needed to avoid this error (Springer and Cundy, 1987; Haan, 1988). When using distributed-parameter models, infiltration parameters can be spatially varied using either Monte Carlo or geostatistical techniques (Sharma et al., 1980). Lumped models offer little opportunity to provide spatial variations in the infiltration process, and the straight delineation of large hydrologically homogeneous areas may lead to an inaccurate representation of the hydrologic processes within the basin. According to Troutman (1985), this is a major reason why lumped models usually have biased, physically unrealistic parameters.

The analysis by Bresler and Dagan (1983) indicates that the variance of model results due to spatial variability may be much larger than that related to model method. In other words, it may be far better to use good infiltration data in a poor model than to use poor data in a sophisticated model. Not considering the variability at all leads to biased results (Sivalapan and Wood, 1986). However, developing deterministic information on the spatial variability of infiltration is not economically pragmatic (Smith, 1983). Several other options are available. The problem of spatial variability can be reduced by using estimates of infiltration parameters that are based on soil texture (Springer and Cundy, 1987). However, the texture-based techniques do not usually provide the same level of variation in infiltration variation expected over an area (Goldman et al., 1990; Loague and Gander, 1990; Springer and Cundy, 1987). Random variation can be used in the modeling, and Sivalapan and Wood (1986) indicate that this is appropriate for all but the small watersheds. The HSPF model provides an additional approach — to implicitly account for infiltration variability by arbitrarily varying the soil moisture status in the watershed (Johanson et al., 1984; Larson et al., 1982).

Summary and Remarks

Comparative studies by Tracy and Marino (1987) and others indicate that certain equations more accurately simulate intrastorm variation of infiltration. The major difficulty in comparing rainfall losses between the different models is that their parameters are developed in varying manners. An adequate comparison between models and their output is unlikely to be achieved until the rainfall loss parameters can be estimated from similar data. Bresler and Dagan (1983) have indicated that the parameterization and manner in which infiltration is spatially represented are more important than which infiltration equation is used. The equations that explicitly represent the status of soil moisture, needed for use in CS models, are among the methods which provide accurate calibration. Some of these equations may be adopted for use in event models as a first step toward uniformity.

The development of regionalized rainfall loss parameters for use at ungaged sites is of paramount importance. It is also likely to be a difficult process because of limited information on both field measurements of infiltration and acceptable parameters used in models. Precautions should be taken in the calibration process at gaged sites to develop parametric values that have regional applicability. When using event models, it should be emphasized that the development of reliable, regionally applicable parameters is likely only when these parameters are related to an explicit evaluation of the antecedent soil moisture conditions, which requires the preceding climatological record.

Rainfall loss equations identified as having the greatest potential for regionalization are: 1) modifications to the Green-Ampt equation, 2) the Holtan equation, and 3) the HSPF equation. Regionalization of HSPF parameters for specific watersheds in northeastern Illinois is currently being analyzed by the U.S. Geological Survey-Illinois District. Applications of both the Holtan and Green-Ampt equations are currently restricted in number. But an increase in the available information for parameter development of the Green-Ampt method is likely to result from the ongoing applications by the U.S. Soil Conservation Service and uses of the texture-based technique to estimate the Green-Ampt parameters. The authors also note that a great number of newly developed rainfall-runoff models use the Green-Ampt method to estimate infiltration.

When using distributed models, the spatial variation of infiltration should be considered. This can be partially accomplished by using the texture-based parameter estimation methods. However, additional consideration using an estimated random distribution may well improve the simulation results as compared to assuming uniformity.

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V. CHANNEL ROUTING

Channel Routing Methods

Channel flow routing is used to predict flood discharge at various locations along a water conveyance system. The end-user of channel routing may need flood discharge information to size outlet works or flood stage information to determine flood insurance requirements. Channel routing models are classified into two general categories: hydrologic and hydraulic models. Hydrologic models are based on the principle of conservation of mass and a second function, which usually defines a relationship between stage, storage, and discharge. Hydraulic models are based on the principles of conservation of mass and conservation of momentum.

Hydrologic Routing

Most hydrologic routing methods are based on a storage-stage-discharge relationship and the conservation of mass equation, usually expressed in the form:

$$I - O = \Delta S / \Delta t \tag{1}$$

where At is a given time interval, I and O are the average inflow and outflow over that time interval, and AS is the change in storage in the reach. Hydrologic routing methods include reservoir storage routing (modified Puls, level pool reservoir), Muskingum, and working R and D (U.S. Army Corps of Engineers, 1990). For reservoir routing, the stage-discharge relationship is usually computed from information on the channel control, such as by estimating spillway coefficients. In most natural channels, the stage-discharge relationship must usually be calibrated from observed discharge. An exception is the Muskingum-Cunge method, whose parameters are physically based values. In all cases, the storage-stage-discharge relationship used in hydrologic routing models must be single valued and represent average hydrograph shapes and speeds.

Other hydrologic routing methods are based on lagging average hydrograph values, such as the straddle-stagger and Tatum methods. These average lag methods are purely empirical, being limited to conditions where the inflow-outflow relationship is calibrated using observed values, and give best results when applied to slowly fluctuating rivers (Fread, 1985). Discussions of all these routing methods are included in Fread (1985) and U.S. Army Corps of Engineers (1990). Table V-1 lists selected rainfall-runoff models that include the hydrologic routing methods described above. Among available computer programs, the HEC-1 Flood Hydrograph Package contains the greatest variety of hydrologic routing methods.

Table V-1. List of Common Channel Routing Methods and Models That Use Them

Routing Method	Model
Modified Puls	HEC-1
Level Pool Reservoir	HEC-1, FEQ, TR-20
Muskingum	HEC-1, NWS
Muskingum-Cunge	HEC-1
Average Lag	HEC-1, NWS
Kinematic-Wave	HEC-1, HSPF, ANSWERS, PRMS
Diffusion-Wave	FEQ, SHE, DAFLOW, CONROUT
Dynamic-Wave	DWOPER, FEQ, EXTRAN (SWMM), BRANCH, UNET, HYDRAUX

Hydraulic Routing

Hydraulic routing models can be one-, two-, or three-dimensional. Flow routing analyses for modeling floods in Illinois have generally used the one-dimensional flow equations. Two- and three-dimensional models, cited in the literature, are presently applied to complex unsteady flow conditions, such as those associated with oceans, bays, estuaries, or lake circulation. This review will therefore focus on the one-dimensional models.

Most hydraulic routing models are numerical solutions to the one-dimensional form of the conservation of mass and momentum equations (Saint Venant equations). These are:

$$dA/dt + dQ/dx = q \qquad (2)$$

$$(1/A)\, dQ/dt + (1/A)\, d(Q^2/A) / dx + g\,(dy/dx) - g(S_f - S_O) = 0 \qquad (3)$$

where q is the lateral inflow rate, A is the cross-sectional area of flow, t is time, Q is the flow rate, y is the flow depth, x is the distance along the channel, g is the gravitational acceleration, S_f is the friction slope, and S_o is the channel bottom slope. The derivation of these equations can be found in a number of sources, for example, Chow (1959). Assumptions inherent in the derivation by Chow (1959) are:

1. The flow is one-dimensional (primarily in one direction).
2. The water surface elevation is horizontal at any given cross section.
3. The flow is open channel (hydrostatic pressure is dominant).
4. The channel bottom slope is small.
5. The physical reach characteristics are fixed over time.
6. The friction coefficient for uniform flow (Manning's n) applies for gradually varied flow.

Hydraulic routing models are classified by the number of terms included in the momentum equation. Kinematic-wave models assume that acceleration and pressure effects are negligible. Equation 3 then reduces to:

$$S_f - S_o = 0 \qquad (4)$$

Generally, Manning's equation is used with the kinematic-wave models to estimate the stage-discharge relationship. Diffusion-wave models include pressure effects but neglect acceleration. Equation 3 then reduces to:

$$S_f - S_o + dy/dx = 0 \qquad (5)$$

Diffusion-wave routing models have an advantage over kinematic models of being able to describe variable stage-discharge relationships, including backwater effects. Full dynamic-wave models include all terms in the momentum equation. Table V-1 lists some of the currently available one-dimensional hydraulic routing models. Most of the diffusion- and dynamic-wave methods are in stand-alone models and are not included in any of the comprehensive rainfall-runoff models.

Flow Routing Error Considerations

Flow routing errors can arise through the application of a model unsuited for the physical problem or through the uncertainty in parameter estimates used in a suitable model. Complex channel configurations that can present modeling difficulties, both in model selection and parameter estimation, include channels with floodplain interactions, variable roughness, lateral inflows, backwater effects, and networks.

Channel and Floodplain Interactions

Several researchers have studied and reported on the interaction of channel and floodplain flows (Smith, 1978; Bhowmik and Demissie, 1982; DeLong, 1986; Kiely et al., 1989; McKeogh and Kiely, 1989). As the flow overtops the channel and inundates the surrounding floodplain variations can occur in the hydraulic properties due to the change in roughness and geometrical shape. Bhowmik and Demissie (1982) reported that the floodplain carrying capacity can vary from a few percent to greater than 80% of the total flow. Generally, the carrying capacity of the floodplain increases with the return period of the flood (Bhowmik and Demissie, 1982).

Application of any type of one-dimensional model to a channel with floodplain interaction will require that the channel and floodplain be represented by a composite section. Bhowmik and Demissie (1982) list the following three methods to determine a composite section: (1) assume the entire cross section behaves as a single channel with a single roughness value, (2) assume the main channel conveys the flow and the floodplain acts as a storage reservoir, and (3) assume the floodplain and channel behave as separate conveyance channels. DeLong (1986) represented a channel with floodplains through variation of the momentum coefficients and conveyance with flow depth.

If the channel meanders, the floodplain flow path will be shorter than the channel flow path and the floodplain bottom slope will be steeper. Small meandering streams can have a channel flow path that is twice the length of the floodplain flow path. DeLong (1986) reported that the ratio of the channel length to the floodplain length can have a significant effect on the shape and timing of the flood peak. Using a laboratory flume, Smith (1978) found that the flow from a floodplain without a channel could be greater than the combined flow from a floodplain with a meandering channel.

Floodplain flow can present significant difficulty to kinematic and hydrologic storage routing models because of the hysteresis loop of the rating curve. Kinematic-type methods require a single-valued rating curve, whereas diffusion-wave and full dynamic-wave methods can use looped rating curves (Fread, 1985).

Changes in Roughness

Roughness can increase, decrease, or fluctuate with increasing discharge depending on the channel and overbank characteristics (Fread, 1989). Generally, hydraulic and hydrologic storage routing methods can be formulated to account for roughness variation with stage or discharge. Available routing models may allow the user to vary roughness with discharge. Hydrologic lagging methods can indirectly account for the roughness variation through the parameter calibration process.

Flow routing models are usually formulated as event methods, although many have been successfully applied in continuous simulation (CS) modeling studies. Most available routing models do not have "built-in" routines to account for seasonal variations in roughness due to the state of vegetative growth or ice formation. Flood sequence, a factor arising during CS studies, can also have a pronounced effect on the vegetative resistance (roughness) if it becomes deformed during antecedent flood events (Fread, 1989). Long-term changes in hydraulic conveyance can also occur during CS studies spanning long periods of record. Since extreme floods are by definition rare, channel conditions expected at the time of the flood event should be used in model studies. Aldridge (1989) reported that the average water surface elevation for a given stream was increased by 0.49 meters (1.6 feet) for the 10-year flood and 0.91 meters (3.0 feet) for the 100-year flood due to an 8-year growth of trees. Similarly, the discharge capacity of a channel was reduced by approximately 70% (from 716 to 227 cubic meters per second) due to the deposition of sediment terraces and the vegetative growth over an 18-year period (Aldridge, 1989).

Time and Distance Steps

Time step (dt) and distance step (dx) must be selected to ensure numerical accuracy of the results and, in the case of hydraulic models, to ensure numerical stability. If dt or dx are too large, routing models (either hydrologic or hydraulic) are likely to produce inaccurate results. Hydraulic models, in particular, may be unstable and fail to reach a solution. Additionally, the maximum allowable step size used within hydraulic models is dependent on the manner in which the Saint Venant equations are represented and solved (Fread, 1985). Excessively small step sizes should not affect model accuracy, but will increase computer computational costs.

Selection of appropriate time and distance steps for use in unsteady flow routing models, with considerations of numerical accuracy and stability, have been investigated by Fread, 1974; Weinmann and Laurenson, 1979; Ponce and Theurer, 1982; Fread, 1985; Husain et al., 1988; and Ponce et al., 1990. The appropriate time step (dt) for most models can be determined by dividing the time of rise of the inflow hydrograph (Tr) by a constant. Husain et al. (1988) recommend that the Tr/dt value range between 5 and 20, depending on the shape of the hydrograph. A value of 5 can be used for uniform inflow hydrographs and 20 can be used for sharply peaked hydrographs. The appropriate distance step (dx) is related to the type of model chosen. Weinmann and Laurenson (1979) discuss step selection for kinematic models. Ponce and Theurer (1982) give recommendations for the Muskingum-Cunge method, and Husain et al. (1988) discuss distance step selection for a full dynamic-wave model (DWOPER). The Tennessee Valley Authority's experience with unsteady flow modeling using the Simulated Open Channel Hydraulics model (SOCH), an explicit full dynamic-wave model, found that the Courant condition provided a good estimate for the time step when the distance step was selected (Granju and Lowe, 1988).

Distance step selection for steady flow models is discussed in Davidian (1984). In brief, the total reach length should be divided into a series of subreaches, each of which is approximately uniform in geometry and roughness. By definition, there is no time step selection in a steady-state analysis.

Flow Routing Model Comparisons

The applicability and accuracy of hydraulic flow routing models (kinematic-, diffusion-, and dynamic-wave) is related to the magnitudes of the individual terms in the momentum equation. When inertia and the pressure gradient are negligible, a kinematic-wave model may suffice; when inertia is negligible and the pressure gradient is significant, the diffusion-wave model may suffice (Weinmann and Laurenson, 1979).

The magnitudes of the terms in the momentum equation vary with the shape of the inflow hydrograph and the channel properties (Weinmann and Laurenson, 1979). Ferrick (1985) used a nondimensional form of the momentum equation to quantify significant terms for a given flood wave and channel type. Weinmann and Laurenson (1979) compared the dynamic-, diffusion-, and kinematic-wave solutions by varying the bed slope term for two hypothetical channel configurations with similar geometrical properties. Weinmann and Laurenson's results are given below.

Variable	Channel 1	Channel 2
Bed slope term	2.0×10^{-3}	0.2×10^{-3}
Error in computed peak flow due to diffusion-wave approximation	<0.1%	1.4%
Error in computed peak flow due to kinematic-wave approximation	0.2%	24.6%

The kinematic-wave approximation is sufficiently accurate for moderately steep slopes (Weinmann and Laurenson, 1979). Theoretically, the kinematic-wave method can only model flood wave translation; in practice, however, errors inherent in the finite difference solution technique introduce attenuation and dispersion during flood routing (Ponce et al., 1978).

Hydrologic storage routing methods should provide acceptable accuracy in situations where the kinematic-wave method will suffice (Weinmann and Laurenson, 1979). In applications involving highly nonprismatic channels, however, the modified Puis method may give better results than a kinematic-wave method by approximating the channel as a cascade of storage reservoirs (Katopodes and Schamber, 1983). In applications of the cascading reservoir method, a steady-state backwater model is typically used to determine the storage-discharge relationship in each of the reservoirs (U.S. Army Corps of Engineers, 1990b). The combined use of the modified Puls and backwater models is capable of simulating the rising and falling portions of a flood wave by using different storage levels in the cascade of reservoirs, thus approximating a looped storage-outflow effect over the total river reach (U.S. Army Corps of Engineers, 1990b).

Diffusion-wave models are applicable to a wider range of hydraulic problems than are kinematic-wave models (Fread, 1985). The Muskingum-Cunge method, although a hydrological method, was shown to be a diffusion type of routing method through the numerical formulation of the routing equation and the derivation of the routing coefficients

(Cunge, 1969; Koussis, 1980). Younkin and Merkel (1988) investigated the application of the diffusion-type routing (Muskingum-Cunge method) to Soil Conservation Service studies, and report that this method should satisfy accuracy requirements in over 80% of the SCS field conditions. Narrow, deep channels were more suited to the diffusion-type routing than wide, shallow channels (Younkin and Merkel, 1988).

Full dynamic-wave models can accurately simulate the widest variety of wave types and waterway characteristics (Fread, 1985). These models are needed in the presence of backwater effects arising from downstream disturbances, relatively small gradients, or significant lateral inflows. However, mere application of a dynamic-wave approach does not mean the real channel system has been adequately modeled. As pointed out by Weinmann and Laurenson (1979), full dynamic-wave models "do not guarantee accurate modeling as they are based on the assumptions made in the formulation of the Saint Venant equations."

Flow Routing Model Selection

Several researchers have investigated the types of conditions for which the various routing methods can be properly applied (Ponce et al., 1978; Weinmann and Laurenson, 1979; Katopodes and Schamber, 1983; Ferrick, 1985; Choi and Kang, 1990). Choi and Kang (1990) presented a figure that depicted the applicable ranges for kinematic-, diffusion-, and full dynamic-wave models. This figure was based on the bottom slope S_o and a dimensionless acceleration number G_w (G_w = 0.0001 g (dv/dt), where dv/dt is the local acceleration). Ferrick (1985) combined the Saint Venant equations to form a dimensionless system equation from which he could determine an appropriate hydraulic routing model. Ponce et al. (1978) formulated numerical criteria to aid in the selection of hydraulic models. Book et al. (1982) reported good agreement between their analysis and the criteria presented by Ponce et al. (1978). Fread (1985) listed these criteria as:

$$TrS_o^{1.6}/(q_o^{0.2}n^{1.2}) \geq 0.014 \qquad \text{(kinematic-wave method)} \qquad (6)$$

$$TrS_o^{1.15}/(q_o n)^{0.3} \geq 0.0003 \qquad \text{(diffusion-wave method)} \qquad (7)$$

where Tr is the time of rise of the inflow hydrograph (hours), q_o is the unit-width reference discharge, and n is Manning's roughness coefficient. Routing errors should be less than approximately 5% by adhering to these criteria (Fread, 1985).

Hydrologic storage routing methods can be used, given careful parameter selection, when the kinematic-wave approximation is felt to give acceptable accuracy (Weinmann and Laurenson, 1979; Katopodes and Schamber, 1983). Generally, the kinematic-wave approximation can be used in situations involving slowly rising hydrographs and moderately steep channels (Fread, 1985). Kinematic-wave models and hydrologic models cannot simulate backwater effects and are generally restricted to single-valued depth-discharge relationships. However, the combined use of a steady-state backwater model with the modified Puls hydrologic routing can approximate the effects of backwater can looped storage-outflow effects (U.S. Army Corps of Engineers, 1990b).

Diffusion-wave routing is applicable to a wider range of problems than kinematic-wave routing. As seen in the above criteria, the diffusion-wave method can simulate a more rapidly rising hydrograph or a shallower bottom slope than the kinematic-wave method. Younkin and Merkel (1988) investigated a diffusion-type routing method (Muskingum-Cunge) for use in the SCS field studies and found that diffusion routing satisfied accuracy requirements in over 80% of these studies. The HEC-1 User's Manual (U.S. Army Corps of Engineers, 1990a) recommends the Muskingum-Cunge method over the kinematic-wave method for channel flow routing, since it is an approximate diffusion routing method and can model peak discharge attenuation.

Full dynamic-wave models should be used when the criteria for kinematic- and diffusion-wave models cannot be met, as in the case of a rapidly rising hydrograph, shallow bottom slope, or significant lateral inflow (Fread, 1985). Full dynamic-wave models can simulate backwater effects, downstream control, and flow reversals (Book et al., 1982). These models can also accurately simulate the widest range of routing problems.

Fread (1985) lists the following guidelines to aid in the selection of a flow routing model: model accuracy, accuracy required for the application, type and availability of required data, available computational resources, extent of flood wave information desired, model familiarity by the user, availability and documentation of an existing model, and time available for model development. These guidelines, in conjunction with one of the numerical criteria presented at the beginning of this section, can be used to determine an acceptable model for the given study.

Recently, due to the availability of dynamic-wave models, studies using simpler methods were commonly perceived as inherently less accurate than those using a complete dynamic-wave model (Ferrick, 1985). However, Ferrick (1985) states that "more complete equations may not yield more accurate river wave simulations." Weinmann and Laurensen (1979) have shown that the simpler routing methods can produce routing results practically equivalent (within 0.2%) to dynamic-wave models when used in studies amenable to the simpler technique. In a discussion of river quality models, Krenkel and Novotny (1979) suggest first clearly defining the problem, then choosing the simplest model that provides acceptable accuracy.

Peak Stage Estimation

Peak stage can be estimated by any of the hydrologic or hydraulic models listed previously. Hydraulic models compute stage during the solution process, whereas the hydrologic models generally compute stage indirectly from a rating curve. An alternative method for computing stage is based on the assumption that steady-state flow conditions exist. It is assumed that during peak flow conditions, the discharge hydrograph tends to flatten out and the flow will be approximately steady (Davidian, 1984). Steady-state analyses are limited to applications where inundation duration, peak attenuation, and flow routing information are not needed.

Steady-state water surface profiles are generally calculated by the standard step method, which solves the conservation of energy equation in a stepwise manner at each cross section. Commonly used steady-state models are HEC-2 (U.S. Army Corps of Engineers, 1982) WSP2 (U.S. Department of Agriculture, 1976) and WSPRO (U.S. Department of Transportation, 1986). Historically, floodplain inundation studies have used steady-state models. Steady-state conditions have often been applied because of the computational difficulties associated with unsteady flow analysis. This practice was commonly followed with little knowledge of the resulting accuracy (Franz, 1990). The advent of high-speed computers, however, has made the analysis of unsteady flows by full dynamic-wave equations more practical.

Thompson and James (1988) and Thompson (1989) compared the peak stage estimates of a one-dimensional steady-state model to those of a two-dimensional vertically averaged model for channels having constrictions. The water surface profile computed using HEC-2 was compared to that computed using the FESWMS-2DH two-dimensional model (Froehlich, 1989) for steady flow conditions. For the simulated steady-state conditions, the FESWMS-2DH model computed a higher, localized backwater effect at the constriction than did the HEC-2 model. In the channel reach, which was not influenced by the constriction, Thompson and James (1988) reported acceptable accuracy for the HEC-2 model. Comparisons between one- and two-dimensional unsteady flow routing models were not found in the literature. Because two-dimensional models require considerably more effort to develop and calibrate than one-dimensional models, there has been limited application of the two-dimensional models by the practicing community.

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VI. FLOOD FREQUENCY METHODOLOGIES

The objective of flood frequency analysis is to estimate the magnitude of extreme flow events corresponding to a specified recurrence interval or exceedance probability. Quantile estimates are used in planning or decision processes relating to hydraulic works or flood alleviation programs. Many of the decisions have economic implications and any mis-specification of a particular design flood may incur significant losses. Although the use of frequency analysis applies to both low and high flows, this review will focus only on high flows.

The state of flood frequency analysis is perhaps best summarized by Kumar and Chander (1987):

> "Statistical flood frequency analysis has probably been one of the most active areas of hydrological research for the last thirty or more years ... With such a voluminous research material available, one would like to ask whether significant progress has been made in understanding or arriving at more acceptable methodologies for frequency analyses. The answer to this question is rather difficult."

Matalas and Wallis (1973) list the general assumptions taken in estimating the probabilities of hydrologic events as 1) the sample of the observed flows are sufficiently large and independently distributed in time, 2) the flows follow a specified distribution function, 3) the estimates of the distribution parameters are unbiased, and 4) no operational decisions influence the results. According to Matalas and Wallis, when dealing with small samples of streamflow sequences taken from apparently skewed distributions, there is seemingly a one-to-one correspondence between the number of hydrologists and the number of best engineering judgements of the estimates of flood quantiles.

Both Kumar and Chander (1987) and Condie and Lee (1982) identify two sources of error in frequency analysis: 1) the identification of the "proper" frequency distribution and 2) selection of the most appropriate parameter estimation procedure for the distribution. When applying flood frequency analysis to simulated flows, the uncertainty in rainfall-runoff modeling introduces another possible source of error. Thomas (1987) compared the flood-frequency estimates based on observed and model-generated peak flows. The model used in this study is documented by Dawdy et al. (1972), who concluded that there were statistically significant differences between the observed and estimated flood-frequency estimates. In particular, there was a tendency to overestimate the lower recurrence interval floods and underestimate the higher recurrence interval floods. An earlier study by Clement (1984) yielded similar results using a smaller database and the same model.

In the most common application of rainfall-runoff models, the model parameters are estimated by using a short record of streamflow. The model is then used to generate synthetic streamflow records from the longer record of precipitation. However, even if the model fits well visually and balances the overall water budget, it may not preserve the distribution of the observed sample (which may not be a representative sample of the true populations or may not extend the frequency distribution beyond the record of observation). However, a detailed discussion of the problems associated with using streamflow generation models in flood frequency analysis is beyond the scope of this study.

The topics of this section are organized under three major headings: a) problems associated with the identification of the parent distribution to which the data belongs, b) problems associated with the selection of the parameter estimation technique, and c) types of data series used in flood frequency analysis. Obviously those three categories do not cover all aspects of flood frequency analysis and methodology. Cunnane (1987), in his review of flood frequency analysis, includes an additional procedure where data were classified as having either an "at-site" or "regional" origin. Since most of the problems associated with the "at-site" approach are relevant to the regional analysis (such as selection of a distribution and parameter estimation technique), the regional flood frequency analysis will not be discussed in detail, but several studies will be referenced for different regional analysis procedures. Finally, it should be noted that the choice of a distribution, estimation technique, or the type of data used are all interrelated. For example, the "best" estimator for a particular distribution may not be suitable for another distribution.

Identification of Distributions

Current methods of frequency analysis of extreme events in hydrology are based on the assumption that the sample of flood observations belong to a population having a known probability density function (Adamowski 1989; Condie and Lee, 1982; Matalas and Wallis, 1973). Ideally, one could identify the distribution on the basis of physical laws governing floods, and then relate the distribution to the physical process. Unfortunately, this has not been possible, and the solution of identifying the parent distribution for flood frequency analysis still remains empirical, which relies on the best-fit of the data.

For the purpose of flood hydrology, the hydrologist's interest lies in the upper tail of the flood distribution. Kuczera (1982) emphasizes the importance of using robust flood frequency models. Robustness, according to Kuczera, is an estimator property that is now receiving well-merited attention from hydrologists (Cunnane 1987). In this context, the postulated parent distribution and the estimator are considered jointly. The distribution/estimator is robust in the flood frequency context if it estimates medium and high return period quantiles with low bias and low standard error, even if the sample is drawn from a distribution other than the assumed one.

Rival candidate distributions for modeling flood series give markedly different quantile values, especially at higher recurrence intervals. This is supported and demonstrated by several researchers (Cunnane, 1985; Matalas and Wallis, 1973; Matalas et al., 1975; Slack et al., 1975; Wallis et al., 1974; 1976).

To address this problem and to achieve some degree of uniformity, attempts have been made in various countries to find the underlying frequency distribution of floods. No real success has been achieved and, as a result, distributions are usually chosen by their ability to mimic the data better. There are no statistical tests of goodness-of-fit powerful enough to discriminate between the available distributions (Matalas and Wallis, 1973; Condie and Lee, 1982; Kumar and Chander, 1987). Thus, there is no real knowledge of the error introduced by the choice of an inappropriate distribution.

One of the most commonly used recommendations on flood frequency analysis has been made by the U.S. Water Resources Council, or USWRC (1967). The USWRC recommendation proposed the use of Log Pearson Type 3 (LP3) distribution as the base method, with provisions for departures from the base method, where justified. Within the context of flood frequency analysis, LP3 is probably the most analyzed and criticized method, both on the basis of being imposed arbitrarily (Adamowski, 1989; Thomas, 1985) and of its questionable validity (Wallis and Wood, 1985). Many studies have tried to compare the properties of LP3 and other distributions, as well as the parameter estimation technique recommended by the USWRC. The parameter estimation techniques will be summarized in the following sections.

The USWRC's decisions have also come under attack after they had reiterated their earlier findings in their two reviews (1976, 1981 revised). Wallis and Wood (1985) have strongly advocated the re-evaluation of the USWRC recommendations in light of recent studies, stating that failure to do so may amount to "professional negligence."

Separation Effect

Actually, most of the controversy on LP3 distribution does not rise from its *de facto* choice, but rather from the difficulties experienced in estimating parameters. The LP3 distribution cannot explain the "separation" of skewness of annual maxima series (Matalas et al., 1975). Similar conditions were reported in Italian annual maxima data by Rossi et al. (1984) and in British annual maxima data by Beran et al. (1986). Matalas et al. (1975) concluded that "the relationship between the mean and the standard deviation of regional estimates of likeness for historical flood sequences is not compatible with the relations desired from several well-known distribution functions" (Normal, Gumbel, Log-Normal, Pearson, Weibull, Pareto, and Uniform). There are also objections to the suggested use of regional skew maps by USWRC (1981). Landwehr et al. (1978) and Singh (1981) questioned the validity of log-skew maps. Again Matalas et al. (1975) and Wallis et al. (1977) showed that the existing "separation" in the regions of the United States could not be explained by LP3 or regional skew maps. These discussions underscore the potential problems of using sample or weighted skew values in estimating the parameters of LP3 distribution.

Bounds in the Distribution

Another problem associated with the selection of a probability distribution for annual maximum flood series is the existence of upper or lower bounds in the distribution under certain circumstances. The three parameter probability distributions are formed basically by adding a shift or location parameter to their two-parameter counterparts. The location parameter serves as a bound for the random variable being fitted (Rao, 1981), assuming a different value for each combination of the mean, variance, and skewness coefficient of the random variable. Moreover, the bound becomes negative in certain instances for log-normal, Weibull, and Pearson distributions. Hydrologic variants cannot be negative unless transformed, and the imposition of an upper bound implies that there is a "maximum certain flood flow," which is not physically meaningful. Probability distributions having negative lower tails, like the Gumbel, have long been used in flood flow frequency analysis. In such cases, since the quantiles in the upper tail of the distribution are of interest, the fact that a portion of the distribution lies in the negative region will not be readily perceived unless a complete investigation of the distribution is made (Rao, 1981). An extensive analysis of upper and lower bounds for the log-normal, Pearson Type 3, Weibull, LP3, and Gamma distributions are given by Rao (1981) for various combinations of variance and skewness coefficients.

The problem of upper bounds, particularly with LP3 distribution, is discussed by several researchers (Matalas and Wallis, 1973; Bobee, 1975; Condie 1977; and Phien and Jivajirajah, 1984). Use of the LP3 distribution with an observed sample from a negatively skewed population requires an upper bound which, as stated before, has no physical meaning (Bobee, 1975). Phien and Jivajirajah (1984) consider the LP3 distribution unsuitable for those cases when the skew of the population is negative.

Type of Data Series Used in Flood Frequency Analysis

There are essentially two different approaches to the problem of flood analysis. One corresponds to the analysis of the annual flood series (AFS), and the other to the analysis of the partial duration series (PDS) (Todorovic, 1978). In the AFS method, the data sample consists of a series of annual maximum flood flows obtained from a continuous record of discharge, regardless of how small these maximum flow values are. On the other hand, PDS values are obtained by retaining only the hydrograph peaks that exceed a certain threshold level. Since the dissatisfaction with statistical flood frequency analysis has arisen mostly from the use of short samples of AFS (Cunnane, 1987), the PDS method has been investigated in this study as an alternative to the AFS method.

"One of the main problems in constructing a feasible stochastic flood model based on AFS is the analytic insufficiency and inadequacy inherent in the use of empirical procedures. Most of these procedures are somewhat *ad hoc* on theoretical and physical grounds. On the other hand, stochastic flood models based on PDS have a solid theoretical base" (Todorovic, 1978).

The number of exceedances of a threshold level X_o in an interval of time [0, *t*] is a random variable. Generally speaking, the choice of the base level depends on the particular engineering problem under consideration. It is also stipulated by a desire to make these exceedances mutually independent events. It seems intuitively clear that if the truncation level X_o is sufficiently high, the assumption of stochastic independence becomes physically plausible. In practice, the base is usually chosen so that on the average no more than two or three exceedances for each year are included (Langbein, 1949; Dalrymple, 1960; Todorovic, 1978). The most important feature of the result of the hydrograph truncation procedure is the empirically established property process (Todorovic, 1978). This particular property of the exceedance series is explained by the light density of events and by the Poisson nature of precipitation events (Todorovic and Yevjevich, 1969). Its theoretical explanations can be found in certain works by Leadbetter (e.g., Cramer and Leadbetter, 1967).

Ashkar and Rousselle (1983) also have analyzed the use of PDS models in flood frequency analysis. They warn about limitations in using PDS:

> "Water Resources Council (1976) defined the partial flood series as a sequence of flood events separated by at least as many days as five plus the natural logarithm of the drainage area in square miles. This in addition to the arbitrarily imposed requirement that the intermediate flows between two consecutive flood peaks must drop below 75% of the lower of the two separate maximum daily flows. The purpose of these restrictions imposed on the interarrival time between two successive flood events is to minimize the stochastic dependence between flood exceedances.
>
> .. . The requirement that the time between two consecutive flood peaks be at least equal to 5 days (plus an additional factor related to the drainage area!) may therefore constitute an acute violation of one of the underlying hypotheses of the Poisson process, namely, that the lower bound of interarrival time is 0 and not 5. This violation may render the Poisson model inapplicable in certain situations.
>
> We suggest therefore that the guidelines of the Water Resources Council be followed only when statistical tests show that successive exceedances are correlated."

Ashkar and Rousselle (1983) also suggest that several truncation levels be tested to minimize the dependence between exceedances. They conclude that both the Poisson distribution as a model for flood frequency and the exponential distribution as a model for flood magnitude, once found applicable with a certain truncation level, should remain so with any higher level of truncation. A great degree of freedom is left to the engineer. What is important is that the truncation level should be sufficiently high to satisfy the Poisson model.

In their more recent study, Ashkar and Rousselle (1987) introduced the R-curve approach, where R is the sample mean/sample variance used as a function of the threshold level, as an aid in making a better choice of the threshold level. This was tested only by using hydrographs obtained from snowmelt.

Smith (1987) proposed an estimation procedure using only the largest 10-20% of flood peaks in quantiles. This method used a generalized Pareto procedure based on Picklands' (1975) theorem for annual peak quantile estimation, and was applied to the Potomac River. The results of the application are not conclusive (due to the lack of other testing). This method cannot be classified a PDS method but rather enters a subgroup of partial series called "censoring." In this approach, the number of events exceeding a certain threshold level (called historic information) within a historic time span is known.

For a good description of censored samples and possible benefits of employing them, refer to Condie and Lee (1982). Censoring can be used not only to remove data below a given threshold, but also provides for the use of paleoflood information:

> "Occasionally at the site, there may have been some large flood in the past whose value is known, and it is also known that this flood was the largest in some historic time span, Y_t years. Although the number of fully specified floods has only increased by 1, there is also the knowledge that in the intervening years when no record is available, the maximum annual floods were all less than the historic flood whose value is known. Such a record can be considered as censored sample from a postulated frequency distribution and analyzed accordingly" (Condie and Lee, 1982).

Stedinger and Cohn (1986) categorized the historical information as "censored data" where the magnitudes of historical flood peaks are known, and "binomial data" where only threshold exceedance information is available. Historic information can be available in the form of "out of bank floods," "extreme floods in the gaged record," or "extreme floods prior to the gaged record."

The application of the maximum likelihood estimator (MLE) and censored sample theory to flood frequency analysis seems to have been first made by Leese (1973). Condie and Lee (1982) extended the concept to the more flexible three parameter log-normal distribution. Stedinger and Cohn (1986) performed a Monte Carlo study using MLEs and adjusted-moment method with the two parameter log-normal and LP3 distributions. The MLEs used were both for "censored" and "binomial" data. The results emphasized the usefulness of historical information and the flexibility, efficiency, and robustness of the MLEs.

Parameter Estimation Procedures

As was discussed earlier, one of the important components of flood frequency analysis is the estimation of the parameters of the postulated probability distribution. More than a half century ago, considerable argument existed concerning the relative merits of moment estimates versus MLEs. One of the colorful chapters in the history of statistical theory was the arguments of Pearson, an advocate of moment estimates, and Fisher, an advocate of MLEs (Matalas and Wallis, 1973). Over the years, much of the argument has vanished with the general acceptance that, at least from the point of view of efficiency, MLEs are better than moment estimates. However, MLEs have not become dominant in hydrology, probably due to the USWRC recommendations in 1967 for using the method of moments to the logarithms of data, for the fitting of the LP3 distribution. Perhaps another reason is the formidable computations needed to obtain MLEs.

The method of fitting proposed by the USWRC has been compared extensively with other methods of estimation. These studies have tested the performance of several methods of moments and MLEs with different probability distribution functions. Due to the large number of publications in this area, only the most interesting references will be mentioned. Quite surprisingly, several authors have different conclusions on the use of certain methods of estimations, and the limited timeframe of this study did not permit a detailed investigation of the possible sources of such discrepancies.

Parameter Estimation for Annual Flow Series

To follow the discussion of evaluating the parameter estimation procedures better, it may be useful to make a legend of several methods used by different researchers. The following list of estimation procedures are applicable to AFS. Similar procedures exist for PDS and will be presented later.

MIM: Method of indirect moments. [USWRC method deriving moments from the logarithms of the flood flows.]

MDM: Method of direct moments. [Bobee's method (1975) deriving moments directly from the observed flood flows.]

MLE: Maximum likelihood estimators.

MMM: Method of mixed moments. [Moments of X (observed flows) and moments of $Y = \ln X$ are mixed in various combinations. Examples of different MMM techniques are given by Nozdryn-Plotnicki and Watt (1979).]

PWM: Method of probability-weighted moments (Greenwood et al., 1979; Landwehr et al, 1979).

Matalas and Wallis (1973) compared moment estimates and MLEs on the basis of the Pearson Type 3 distribution. Comparison was made between the parameter values of the distribution and the variate values at specified probability levels. When solutions were possible, MLEs yielded less biased and less variable solutions than the comparable moment estimates. These results are even more pronounced as the probability level becomes greater than N/(N+1), where N is the sample size. They also mention encountering several difficulties in obtaining both MIM and MLE estimates when using the Pearson Type 3 distribution and therefore suggest considering other distributions, especially if MLEs are of interest.

Bobee (1975) is probably the first to attempt to change the parameter estimation method proposed by the USWRC. This method, sometimes referred to as the method of direct moments (MDM) applies the method of moments directly to the observed data to determine the moments. This method preserves the moments of the observed data, rather than the logs of the observed data, and thus it is impossible to determine the variance of estimates of the parameters. The variance of estimates can be achieved by Monte Carlo techniques to compare moment estimates and MLEs for the parameters of the Pearson Type 3 distribution (Matalas and Wallis, 1973).

Condie (1977) derived MLEs for LP3 distribution from the logarithmic likelihood function. He analyzed 37 Canadian rivers by this method and compared them using moment analysis. They reported markedly superior results by MLE, in terms of the standard error of estimate, but the asymptotic nature of the results must always be borne in mind. The moment method used here was a slightly modified version of Bobee's (1975) method. Condie did not perform any Monte Carlo analysis to test the bias in the expected value of the T-year event for a given sample size. Also, a good discussion on the possibility of biased estimates of the T-year event with different values of the shape parameter is given.

The same data set of 37 Canadian rivers was used by Nozdryn-Plotnicki and Watt (1979) to assess the fitting techniques for the LP3 distribution. For each sample, parameters and specific quantiles were estimated by using MLE, MIM, and MDM, and all three methods were poor in terms of estimating the distribution parameters. The estimates were generally highly biased and exhibited large sampling variances. There were indications, however, that the parameter estimates derived by MLEs were more stable than those of the other two methods. For those samples without an MLE solution, the other methods also yielded "wild" solutions.

Phien and Hira (1983) compared seven parameter estimation methods for the LP3 distribution, using MIM, MDM, MLE, and four different MMM methods. Six sets of parameter values were used representing a variety of Canadian rivers, with sample sizes varying from 20 to 80. For each sample size 100 replications were used. The results indicate that the MLE and two of the MMM methods are clearly superior to the other methods. This study analyzed only the relative errors of the parameter estimates, and did not evaluate the performance of these methods in terms of the standard error of the T-year event.

Arora and Singh (1987) and Jain and Singh (1987) compared the performance of parameter estimation methods with Gumbel's Extreme Value Type 1 (EV1) distribution. Arora and Singh (1987) concluded that: a) MLE generally provided the most efficient quantile estimates, followed closely by maximum entropy estimates (ENT), which are relatively easier to solve; b) method of moments resulted in poor estimates; and c) for small samples, PWM poorly estimated the quantiles, but the efficiency of PWM improved relative to MLE with increasing sample size and also resulted in nearly unbiased quantile estimates. Jain and Singh (1987) concluded that MLE was the most accurate estimate, followed by ENT, PWM, and method of moments, and that any one of these could be recommended for general use. However, ENTs would be preferable because of convergence properties. Greenwood et al. (1979) and Landwehr et al. (1979) found that PWM, in general, compared favorably with MLE and method of moments. Phien et al. (1987) evaluated the performance of four MMMs by using the generalized gamma distribution and 45 annual flood data sets.

Parameter Estimation for Partial Duration Series

All the above-mentioned studies have used annual flood series. The investigation of the past research on partial duration series showed that the level of comparative analysis of estimation methods for PDS (including censored data) are not as detailed as for AFS. In most partial duration series, the model is assumed to be Poissonian (Todorovic, 1978; Ashkar and Rousselle, 1983; 1987). Ashkar and Rousselle (1983) found that both the Poisson distribution, as a model for flood frequency, and the exponential distribution, as a model for flood magnitude, were applicable with a certain truncation level.

Parameter Estimation for Series with Censored Data

For the analysis with historic information, Condie and Lee (1982) treated the censored samples as a three-parameter log-normal population. They developed an MLE function and historically weighted moments to determine the parameters. Although both methods show bias to some degree, the MLEs were substantially less biased, within the limits of the experiment. Condie (1986) also analyzed the asymptotic standard error of estimate of the T-year flood using the same method as Condie and Lee (1982) and showed that including historic information can improve the estimation. Stedinger and Cohn (1986) made a Monte Carlo study employing the two-parameter log-normal distribution with an MLE and the USWRC's adjusted-moment estimator. The results indicated that the MLE performed well even when floods (with historical and paleoflood information) were drawn from other than the assumed log-normal distribution. Phien and Fang (1989) investigated the effects of censoring on MLEs obtained from samples drawn from the general extreme-value distribution. They concluded that for sample sizes larger than 60, the bias in the T-year event due to low censoring levels may be considered negligible.

Beyond LP3 Distribution

The volume of study performed on alternate parameter estimation methods for flood frequency distributions is an indicator of hydrologists' general discontent with the USWRC estimation procedures. However, the first step in this quest should be a better understanding of the underlying distribution. We should not expect to get statistically meaningful inferences by testing the parameter estimation performance drawn from a population with a different distribution. This is why Kuczera's (1982) concept of "robust" models should be employed more diligently in flood frequency analysis.

Despite all the dissatisfaction with the LP3 distribution, it still remains the *de facto* distribution. However, there has been an increasing interest in the search for other methods or other distributions than LP3. Some researchers suggest using transformation methods to enable a good fit with the data without the problems associated with identifying the underlying distribution. Jain and Singh (1986) compared various transformation methods, concluding that power transformation suggested by Chander et al. (1978) was better than most other methods.

The thought that flood populations may consist of two or more sub-populations led to the development of mixed distributions. Potter (1958) was one of the first to discuss the evidence of two or more distinct populations of peak runoff, and the proposed possible climatic causes for the multiple populations (Hirschboeck, 1987). Singh and Sinclair (1972) and Singh (1983) presented a methodology simulating mixed distributions in hydrologic samples. The method of mixed distributions proposed in these studies adequately explains the condition of separation. Waylen and Woo (1982) looked at the differences between rainfall and snowmelt-generated floods to examine the problem of mixed distributions in hydrologic data.

It is desirable to postulate the mixed distributions by identifying the cause of the different sub-populations. Otherwise, there are no valid reasons to expect that an observed flood series would conform to a chosen statistical distribution. "Variations in storm types, antecedent soil moisture conditions, and channel and floodplain storage cause variations in the basin response and flood peaks. Therefore the flood series can hardly be expected to follow a particular theoretical distribution" (Singh, 1987). Unfortunately, not much work is available on the applications of these distributions.

Another distribution that has aroused interest is the Wakeby distribution (Houghton, 1978a; Wallis and Wood, 1985). The Wakeby distribution is defined in an inverse way:

$$x = -a\,(1-F)^{b} + c\,(1-F)^{-d} + e$$

where F is a uniform (0,1) variable. The five-parameter Wakeby distribution has two distinct advantages (Kumar and Chander, 1987): 1) the left- and right-hand side of the distribution can be modeled separately (parameters "a" and "b" govern the low-flow tail, while the parameters "c", "d," and "e" govern the high-flow tail); and 2) the distribution can explain some of the separation effects not explained by most other distributions. Although it is a five-parameter distribution, its parameters can be estimated without going into the higher moments, such as skewness coefficient. The Wakeby distribution parameters can be estimated by using the incomplete means algorithm suggested by Houghton (1978b) or the more elegant probability weighted moment approach suggested by Greenwood et al. (1979).

Regional Analysis

Regional flood frequency analysis is performed either when data is limited or when the model does not have enough degrees of freedom with data from a single location. Regional estimating procedures use all records in the region, and require an assumption about regional homogeneity of catchments with respect to flood statistics, which may or may not be justified.

Cunnane (1987) summarizes the regional flood frequency procedures under three groups: index flood method, empirical Bayesian method, and the two-component extreme value method (TCEV). The index flood method, once the standard USGS approach, is based upon a dimensionless flood variate X, where X = Q/Qindex. Use of any random variable for Qindex has been discussed by Stedinger (1983) and Smith (1989), and a modification of the index flood method was pursued by Rossi et al. (1984). Lettenmaier and Potter (1985) tested several estimation methods with the flood index method, which was based on regional probability-weighted moments (PWM). They concluded that for high region mean coefficient of variation, improvements will come from improved at-site estimators. Stedinger (1983) used PWM with the index flood method in log space, but the correlation among concurrent flood flows in a region was shown to limit the accuracy of the estimation of a distribution's moments. Boes et al. (1989) proposed the use of a Weibull model, but there a few applications of this model.

In the Bayesian analysis, it is assumed that there is an unknown super population of flows from which the parameters of individual basins are drawn. These parameters are then known realizations of the super population, which can be pooled to form some estimate of the regional population. By combining the regional or super population information with the site-specific information using Bayes' theorem, it is possible to arrive at a posterior distribution of quantile events (Kumar and Chander, 1987). The Bayesian method, in giving a posterior distribution of parameters, allows legitimate subjective probability statements to be made about parameters and quantiles that hold even using a noninformative prior distribution (not based on region flood information). This is one of its major advantages (Cunnane, 1987).

The TCEV method is based on a four-parameter distribution for annual maximum floods (Rossi et al., 1984), in which parameters are estimated using a combined regional data set of standardized annual maximum values. The model assumes that flood peaks do not all come from a single parent distribution but rather that the most extreme events and the more ordinary ones come from different distributions. This allows separate consideration of different physical mechanisms of flood production as was done successfully by Waylen and Woo (1982). In the application to Italian flood data, quoted by Rossi et al. (1984), the different physical mechanisms could not be identified uniquely with season, unlike the Waylen and Woo (1982) case, and the parameters of the separate component distributions could not be estimated independently.

A good overview of regional flood frequency analysis is made by Potter (1987). He has emphasized the lack of effort on understanding the physical processes producing the floods, and that much of the attention has been focused only on the statistical analysis of flood data.

Final Remarks

One thing is evident from this review: there is substantial discontent with the USWRC recommendations. Both the criteria for selection of distribution and parameter estimation methods need revision. There are great discrepancies in the results of using MLEs with various distributions. The effects of postulating a distribution for a sample other than the actual distribution to which the population belongs should be investigated more thoroughly. This is especially important with the increased emphasis on applying flood frequency methods to urban watersheds. The question of whether the distributions used for natural watersheds are valid for urban watersheds needs to be answered, this assuming, of course, that the application of these distributions to natural watersheds is valid.

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VII. MODELING APPROACHES

The choice of a particular rainfall-runoff model, the types of input, and modeling approach are functions of both the desired products of the modeling effort and the complexity of the watershed. Five approaches are identified for quantifying infrequent flood events and their frequencies. These approaches are based on a combination of precipitation inputs into the hydrologic model and frequency analysis. The first three approaches (frequency analysis, continuous modeling with historical precipitation, and design storm modeling) are commonly used. The last two approaches are presented in the literature as alternatives but will require additional development before their application. The authors have provided some general, subjective guidelines on when the given approaches may be appropriate, since this information is not readily available in the literature. Pilgrim (1986) believes that greater research is required to provide designers with adequate guidelines for choosing a modeling approach.

Frequency Analysis of Gaging Records

Frequency analysis of the available streamflow record is normally preferred to modeling when a streamgage record is available and the watershed conditions have remain relatively constant. In some cases, rainfall-runoff modeling may be an alternative, depending on the length of the streamgage record, length of the precipitation record, and accuracy of the flood estimates provided by the model (Pilgrim, 1986). McPherson and Zuidema (1977) indicate that the period of record should span at least ten years to support flood frequency analysis. Once the flow record is fairly long, there will be less error associated with frequency analysis than with simulation. If available, a regional flood frequency methodology can be used for ungaged sites and, at times, may even be applicable to a gaged site (Pilgrim, 1986). Frequency analysis of recorded streamflow has some limitations: it only predicts flood peak and therefore is not a useful technique when a complete storm hydrograph is needed for design, and it is also not appropriate for evaluating changing watersheds. Concerns associated with frequency analysis were presented earlier in Section VI, Flood Frequency Methodologies.

Historical Rainfall Used as Input into Continuous Simulation Models

Continuous simulation (CS) modeling is increasingly used for modeling complex or changing watersheds that have a streamgage record. By using historical precipitation, the effect of varying temporal and spatial distributions of rainfall can be examined. Application of CS models also provides for the estimation of antecedent moisture conditions in the watershed. Ordinarily, the historical rainfall provides useful information on frequent storm events but often lacks a sufficient number of extreme storms from which to estimate infrequent flooding such as a 100-year flood event. Therefore, frequency analysis or some other extrapolation technique is normally required to estimate the peak discharge and flow volume of these more infrequent flood events.

Most CS studies use relatively complex models, which require greater modeling cost and effort. Complex models may provide a potential advantage by better describing the hydrologic processes that result in streamflow, but most of this potential is lost without an adequate precipitation network and streamgage records with which to properly calibrate model parameters and check model predictions and parameter consistency (McPherson, 1978; Troutman, 1985). Sufficient precipitation and streamgage records both improve accuracy and provide the user with a greater degree of confidence in the modeling results. James and Burges (1982) and Bradley and Potter (1990) suggest that intense gaging activities in a watershed for a period of only two or three years would provide enormous benefits for calibrating models, and should be carried out whenever possible. The costs for streamflow and precipitation gages, though not minor, can be relatively less than the total cost of complex simulation. Without the use of gaging records or a dependable regional parameterization, the uncertainties in model accuracy and the additional costs of CS modeling may outweigh the potential benefits of applying these models. Approaches to provide reliable regional parameters for complex modeling are being studied (for example, Dinicola, 1990).

Finally, there is some evidence that the flood frequency distributions developed from CS output differ from the distribution of observed floods. The model user should be aware that the simulated flood events may have less variability than the observed floods (Thomas, 1982; 1987). Sherwood (1990) employs a variance inflation technique that adjusts the frequency estimates for simulated peak flows to more accurately reflect the observed frequency. Lichty and Liscum (1978) apply an adjustment factor to modeled flood quantiles that is determined from watershed characteristics and regional climate factors.

Design Storms Used as Input into Event Models

This methodology uses a probabilistic estimate of precipitation amount as input into the rainfall-runoff model and assumes that the frequency of the resulting model output is equal to the frequency of the precipitation amount. There are two major problems with this assumption. First, the frequency of the model output is affected by interrelated factors such as the seasonality of the storm, temporal rainfall distribution, storm duration, and antecedent moisture conditions. In Illinois, research has attempted to determine values for these secondary factors that, for a particular type of basin, will produce a peak runoff value with similar frequency to the precipitation input (Weiss and Ishii, 1987; Singh, 1982; Knapp and Terstriep, 1981). As yet, these studies have not produced comprehensive modeling guidelines, particularly in the area of rainfall losses. Examination of rainfall loss guidelines, and regionalization of parameters, are areas of research that require continued attention (Pilgrim, 1986).

The second major problem in the design storm approach is that, for any one storm, the storm volume return period is not necessarily the same as the peak discharge return period. For example, a short-duration storm may produce the peak flow but not the peak discharge. This is a major problem when modeling watersheds with detention storage (Marsalek, 1978; Wenzel, 1982). Marsalek (1978) indicates that for small watersheds the storms that result in high peak runoff rarely produce maximum runoff volume. At present, changing the duration of the design storm when using the Huff distributions is the only method for examining the effect of different rainfall scenarios on runoff volume and peak discharge (Knapp and Terstriep, 1981).

Design storms are often applied in a lumped manner over the watershed. Thus, they are usually inadequate for use with complex watersheds where the spatial distribution of precipitation can have a significant effect on the estimate of peak flows. The eventual development of an alternative rainfall input that has space-time complexities will address some of these weaknesses associated with the use of design storms. But despite its other problems, the design storm methodology is likely to be the standard approach in the foreseeable future for use with ungaged watersheds that do not require a complex analysis (Pilgrim, 1986).

Transposed Storms Used as Input into Continuous Simulation or Event Models

The use of transposed storms, though not fully developed at present, appears to be an encouraging storm methodology when dealing with either event or CS modeling. The development of a transposed storm methodology for use in rainfall-runoff modeling is specifically recommended by the National Research Council (1988). Though this recommendation is directed primarily toward extreme flooding, with recurrences in excess of 100 years, the approach is believed to be applicable to normal design applications. Transposed storms provide the opportunity to model the response of a watershed to a variety of infrequent, observed storms. Application of transposed storms may be a possible alternative for the estimation of infrequent events, in lieu of frequency analysis or other extrapolative measures.

A major problem with the transposed storm approach is the identification of recurrence intervals likely to be associated with the runoff from each particular storm (NRC, 1988). The frequency relationship will change from one watershed to another, and therefore a pool of a number of transposed storms may be needed to cover a wide range of hydrologic conditions. Experience in using these storms would produce guidelines that identify which storms are likely to produce the largest floods in particular types of watersheds. Because the transposed storms can be severe events that produce infrequent flood events, the use of this method should reduce the reliance on flood frequency distributions for extrapolating extreme events.

Antecedent conditions for each transposed storm must be pre-specified. For CS models this can be accomplished by using the historical meteorological record preceding the storm of interest as model input. Guidelines for setting antecedent conditions in event models would have to be established for each storm.

Simulated Rainfall Used as Input into Continuous Simulation Models

Simulated rainfall is seen as the most conceptually desirable input for CS modeling. The approach has the potential to address the spatial dynamics of storms and, because the simulated series of rainfall can be very long, the modeled runoff series may be sufficiently long to no longer require frequency analysis. However, the use of precipitation simulation for normal applications is many years away and would require considerable development before it can be a feasible approach for stormwater modeling. The development of appropriate stochastic models is likely to precede the evolution of space-time models for rainfall-runoff modeling purposes. Among the shortcomings of the stochastic precipitation mentioned in the literature are: 1) the availability of high-resolution precipitation data (both in the temporal and spatial scales), 2) accuracy of the temporal aggregation techniques in the models, and 3) accuracy in extrapolating to infrequent storm conditions.

Problems in Comparing Modeling Approaches

Precipitation is the driving process in rainfall-runoff modeling, and any differences in its representation will show up in model output. Two common approaches in modeling are the use of design storms in event models and the use of historical rainfall in CS models. These approaches, considerably different in concept, should not be expected to produce comparable estimates (Pilgrim, 1986), making overall model comparison difficult. Historical storms could be simulated with event models to provide some level for model comparison, but the problem of estimating infrequent events would still remain. Design storms could be simulated by CS models, except that the estimate of initial conditions would be questionable at best. Only the transposed storm and simulated precipitation methodologies seem to offer the potential for comparing output from the different models for severe flooding.

The broad concerns that affect the accuracy of rainfall-runoff models apply to all the different modeling approaches (except frequency analysis). Both the event/design storm and historical simulation approaches are plagued by 1) inadequate representation of the precipitation process, and 2) inconsistent rainfall-loss parameters. The shortcoming due to inadequate precipitation can only be overcome with more complete rainfall networks or the application of radar-based precipitation estimates (with sufficient ground-truth information). The possible development of consistent rainfall loss parameters for use on a regional scale should also apply to all types of modeling — although the actual parameter values will vary depending on which rainfall loss function is chosen. Output will differ slightly from one model to another, even with standardized rainfall loss parameters, but the comparisons of model performance and expected output will be more meaningful.

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VIII. SUMMARY

A literature search was conducted on the technical issues pertaining to rainfall-runoff modeling for the analysis of flood peak discharge and volume. The technical literature found in this search was selectively reviewed and summarized in this review. The review assesses the strengths and weaknesses of: 1) hydrologic and hydraulic methods used to estimate flood hydrographs, 2) precipitation and rainfall-loss inputs to models, and 3) the frequency estimation methods used with the model results. The review focuses on application of the comprehensive rainfall-runoff models, which can estimate hydrographs of infrequent flooding events for a wide variety of watersheds. Most of the issues facing rainfall-runoff modeling in Illinois are universal in scope. Thus, many of the findings in this review are also applicable to model use in other locations.

Rainfall-runoff models can be classified as either event models or continuous simulation (CS) models. Event models estimate runoff for a relatively short period, such as the runoff associated with an individual storm event, whereas CS models predict streamflow for sustained periods that include both rainfall events and interstorm conditions. The model's representation of the hydrologic processes may range from simple conceptual functions to physics-based equations. The physical basis or complexity of the processes dictate the type of parameters for the model. Model parameters may be fitted through calibration, measured from field properties, or fixed by using an empirical procedure. These differences in model characteristics influence 1) the applicability of a particular model for a given location or set of conditions, 2) the accuracy of the model's simulation of recorded streamflow, and 3) the reliability of the model's extrapolation of either the runoff response from extreme rainfall or prediction of runoff in ungaged catchments. Model selection should be based on the suitability for solving a defined problem, but the literature does not provide procedures that judge or predict this.

Two major issues in model application are 1) the model's accuracy in predicting the flood hydrographs and 2) the model's ability to properly simulate hydrologic conditions outside the range used for model calibration. Comparative evaluations indicate that most rainfall-runoff models, when properly applied, will predict streamflow with similar accuracy, and that the degree of model accuracy is more directly associated with the adequacy of the rainfall data network. Objective comparisons are not available concerning a model's ability to predict runoff for either extrapolated hydrologic conditions or when parameters are transferred for use on ungaged watersheds. However, a preponderance of the available literature contends that parameters need to be consistent, and maintain physically realistic values, if models are to be confidently applied in these cases. Parametric consistency is less likely when the parameter values are lumped or are fully dependent on calibration. The importance of parameterization, stressed throughout the literature, is also reflected in this review.

Inadequate representation of precipitation is the greatest source of error in the modeling process, and the lack of appropriate rainfall data adversely affects both the accuracy of predictions that use the data and, more importantly, the estimation of the model parameters. Biased parameters impair the predictive abilities of the models, regardless of what modeling approach is used to estimate flood peaks and volumes for design purposes. Errors in future modeling efforts may selectively be reduced through placement of additional precipitation gages in areas likely to be intensively modeled. Radar imagery may eventually provide the spatial information on rainfall needed to improve modeling accuracy, and is a promising topic of applied research. Other technical advances, such as geographic information systems, may also provide more sophisticated data for use in model applications.

Four types of rainfall inputs can be used with rainfall-runoff models to develop design flood hydrographs: historical rainfall, design storms, transposed storms, and simulated precipitation. Of these, historical rainfall and design storms are most commonly used, but the two approaches are conceptually dissimilar and likely to produce different flood frequency estimates. Stochastic and space-time precipitation simulation models may eventually provide the ideal input for rainfall-runoff models but presently have drawbacks that limit their applicability — especially for use in modeling floods in small to medium watersheds. Transposed storms may provide an alternative methodology for modeling infrequent floods, and it is believed that antecedent conditions for these storms could be derived to make this rainfall input compatible with both event and CS models. The application of transposed storms is believed to have considerable potential in flood modeling.

Rainfall losses, the primary determinant of the amount and distribution of runoff that results from an individual storm, are probably the least studied and understood aspects of the rainfall-runoff process. Most of the infiltration equations, for which comparative information is available, calibrate with similar accuracy to measured infiltration. However, studies suggest that a few of the more frequently used equations (SCS curve number and HEC methods) have less accuracy, especially when applied to greater rainfall intensities than those used for calibration. The Horton equation, though accurate in calibration, uses parameters that are unlikely to be consistent between storms or different watersheds. Despite the potential shortcomings of some equations, the choice of the infiltration method appears to be less important than the availability of good infiltration data and the development of consistent parameters. Methods to estimate infiltration parameters based on soil texture information may provide the needed parameter consistency. The development of regionalized rainfall loss parameters is a primary topic requiring further research. Spatial variability of infiltration, another major source of error in modeling, cannot be properly accounted using normal lumped parameters, according to several studies. Alternative indices, modeling techniques, or both may be necessary to develop acceptable lumped parameter values.

Complex channel configurations, the interaction between channel and floodplain flow, and changes in roughness with flow depth can all have a significant impact on flood peaks. Not all channel complexities can be perfectly described by available hydraulic routing methodologies; however the full dynamic wave models provide the most complete analytical techniques. Simpler methods, such as the diffusion-wave, kinematic-wave, and some hydrologic routing procedures, may often provide a level of accuracy similar to the dynamic wave models. In these cases, the simpler models are recommended because of their fewer data requirements and ease of application. Conditions for the appropriate application of these different routing models are reviewed.

Frequency analysis is applied to simulated runoff and to recorded streamflow from gaged sites in the same manner. Several studies have observed, however, that series of model-generated flood peaks are often statistically different than corresponding recorded values. Extra prudence is therefore recommended when using model-generated flows. Errors in frequency analysis generally result from the use of an incorrect distribution or parameter estimation techniques. A great amount of literature addresses these types of errors. It may be particularly difficult to properly fit ordinary frequency distributions to floods from complex watersheds such as those in urban areas. The use of partial duration series or censoring of data in annual flood series may provide alternative approaches for the frequency analysis of complex watersheds, but these methods have not been fully developed and had only limited application. Therefore, further research is needed concerning the use of 1) censored data or partial duration series for frequency analysis with complex watersheds and 2) more robust parameter estimators for flood frequency distributions.