limited, and runoff-producing rainfall will cover a smaller fraction of the watershed as the size of the watershed increases. Therefore, where the storm is centered should become increasingly important with increasing watershed size.

On the other hand, the influence of varying the occurrence of maximum intensity within the storm duration is more or less a function of watershed size and becomes relatively less important with increasing watershed size.

Quantitative analysis of the relationships between thunderstorm rainfall and runoff illustrated here is extremely difficult for several reasons. One reason is that rainfall is not uniform in time or space, and rainfall input can only be estimated from rainfall measurements within certain limits of accuracy and precision. Also, channel abstractions may account for much, or all, of on-site runoff. For example, annual runoff from the 58-mile^2 Walnut Gulch watershed is only about 5 percent of summer rainfall (2).

The next step, therefore, would be to model a larger watershed (several square miles) by using KINEROS and simulated rainfall input. In a step-by-step process, by increasing watershed size and complexity, it should be possible to define the interrelationships between storm-cell properties and watershed characteristics. The test of these interrelationships, in each case, would be the comparison of simulated peak discharges and runoff volumes.

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Conceptual and Empirical Comparison of Methods for Predicting Peak-Runoff Rates

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A wide variety of hydrologic methods have been proposed by hydrologic design. Because peak-discharge methods are the most widely used, it is instructive to compare the methods that are used most frequently. The methods compared include the rational formula, the U.S. Geological Survey urban peak-discharge equations, and the Soil Conservation Service peak-discharge methods. In addition to a comparison of the methods by using data from 40 small urban watersheds, the methods are compared on the basis of their input requirements and the means by which channel systems are accounted for. These latter two comparison criteria appear to be more important in selecting a method than accuracy.

The adverse hydrologic effects of land-cover changes and the different design solutions that have been proposed to overcome these effects have led to a diverse array of hydrologic methods. Many state and local policies on floodplain management, erosion control, watershed planning, and storm-water management (SWM) require a specific hydrologic method for design. Such policies usually generate considerable controversy among hydrologists and design engineers because each hydrologic method has one or more disadvantages. More important, the different methods lead to different designs at the same location. The failure to specify a specific design method in the design component of a drainage or SWM policy often leads to significant difficulties in the review and approval process.

A number of studies have been undertaken to identify the best method (1,2). Most of the comparisons were limited in some respect. For example, some publications involved data obtained for a limited region, whereas others were based on a limited sample size. In some cases, the criteria for comparison were limited. In all cases, the comparisons were limited to empirical analyses. McCuen and others (3) concluded that (a) there is a noticeable lack of consistency in the structure and presentation of results of comparisons of hydrologic methods, (b) the literature does not accurately reflect the methods that are most frequently used in hydrologic design, and (c) the literature is often deficient in the description of the procedure and its accuracy, reproducibility, and the effort that is required to apply the method.

The most comprehensive comparison of hydrologic methods for predicting peak-flow frequencies was undertaken by the Hydrology Committee of the U.S. Water Resources Council (1); the study was undertaken as a pilot test, however, which was designed and conducted to aid in the design of and provide guidance for performing a conclusive nationwide test. The report concluded that a study involving considerably more data would be necessary to make conclusive statements about the accuracy of the methods. The sample size was much larger than the data base for any previous study involving a comparison of procedures. This suggests that until the funds are available to conduct a nationwide test, results based entirely on empirical analyses cannot be considered conclusive.

The objective of this paper is to compare hydrologic methods that are used for predicting peak-flow rates on the basis of structure, input, and calibration requirements as well as on the basis of the accuracy measured by fitting with data. A comparison of methods based on criteria such as structure and input requirements may be as valuable as a comparison based on measured data. After all, the studies involving a comparison of hydrologic methods based on a comparison of computed peak discharges with estimates obtained from flood-frequency analyses have not been conclusive.

CLASSIFICATION OF HYDROLOGIC MODELS

In order to select procedures from among the many that are currently in use, it is useful to first establish a classification scheme for categorizing procedures that have common distinguishing characteristics. By grouping similar procedures, one or more procedures can be selected to represent each category and then the procedures can be tested and compared. If the procedures selected are representative of those in the category, the results may be used to make generalized inferences about the procedures in that category. The categories in the classification scheme should be different by at least one significant element. It is hoped that procedures assigned to a category would be similar in important characteristics, and differences in these characteristics should be apparent when procedures assigned to different categories are compared.

A number of schemes for classifying hydrologic methods have been developed. Classification schemes based on systems analysis concentrate on the three elements of the system black box: the input, transfer, and output functions. Systems are often characterized by the nature of the transfer function (i.e., model), which in systems theory is the function that transforms the input function into the output function. Systems are often categorized with the following sets of dichotomous terms: (a) deterministic versus stochastic, (b) static versus dynamic, (c) linear versus nonlinear, (d) lumped versus distributed, (e) time invariant versus time variant, and (f) conceptual versus empirical. Although these represent mutually exclusive categories, they are of limited value in classifying hydrologic models. They represent a limited scheme because there is wide variation in important characteristics of procedures that would fall within the same category. Thus, a classification scheme was developed that concentrated on the output function.

It is easiest to develop a classification scheme that concentrates on the hydrologic output. The classification system that is given in Table 1 identifies three forms of primary output: a peak discharge, a flood hydrograph, and a frequency curve; these outputs correspond to the three level-1 classes. After the primary output has been generated, a secondary output can be obtained. For example, when a peak-discharge formula is used, the frequency curve can be obtained by using the formula to compute an array of peak discharges for selected return periods. Similarly, a peak discharge for a selected return period can be obtained from a frequency curve obtained from multiple-event hydrograph analysis.

In addition to level 1, it is useful to define a second level. In level 2, the methods are separated on the basis of other factors, such as whether the method is based on calibration to measured data and the structure of the method. A third level of the classification scheme would consist of specific methods. For example, the Stanford watershed model is a continuous-record method, whereas the rational formula is an uncalibrated peak-discharge equation.

Peak-Discharge Methods

For many hydrologic designs the only output required is a peak discharge for a selected return period. Thus, one category of the classification scheme is labeled peak-discharge methods. Many methods have been proposed for such design problems. The required peak discharge can be evaluated directly or by constructing the frequency curve and taking the value from the curve. Peak-discharge methods can be classified as belonging to one of four subgroups: single return period, index flood, moment estimation, and uncalibrated. Except for the uncalibrated equations, the other three level-2 methods in this class require fitting of empirical coefficients; the coefficients are most often obtained by regression.

Single-Return-Period Method

Single-return-period methods use watershed and precipitation characteristics to predict the peak discharge for a specific return period; a separate equation is usually calibrated for each return period. Most often, the single-return-period methods have the following form:

$$Q_p = b_0 X_1^{b_1} X_2^{b_2} \ldots X_p^{b_p} \tag{1}$$

Table 1. System for classifying hydrologic models.

Primary Output	Secondary Output	Classification Level 1	Classification Level 2
Peak discharge	Frequency curve	Peak discharge	Single-return-period equations Index-flood method Moment estimation Uncalibrated equations
Flood hydrograph	Peak discharge and frequency curve	Single-event hydrograph	Calibrated unit graph Uncalibrated unit graph
Frequency curve	Peak discharge	Multiple-event hydrograph	Multiple event Continuous record

in which X_i ($i = 1,2,\ldots,p$) are the watershed and precipitation characteristics, b_j ($j = 0,1,2,\ldots,p$) are the coefficients, and p is the number of predictor variables used. A frequency curve can be derived by estimating the peak discharge with the equation for each of the necessary return periods. The U.S. Geological Service (USGS) state equations are examples of the single-return-period category.

Index-Flood Method

The index-flood method is based on a single prediction equation and a series of index ratios. The prediction equation, which is usually calibrated by using regression, relates the peak discharge for a selected, or index, return period to both watershed and precipitation characteristics. The form of the index-flood equation is usually the same as that for the single-return-period method. Although the index-flood equation is usually calibrated for a 2-yr return period, the 10-yr event is sometimes used. Peak-discharge estimates for other return periods are obtained by multiplying the estimated peak discharge of the index-return period by a constant, which depends on the specific return period. The index ratios are often obtained by regression; therefore, the expected peak discharges computed by the index-flood method are unbiased.

Moment-Estimation Methods

Moment-estimation methods usually relate the first three statistical moments, i.e., mean ($\bar{X}$), standard deviation (S), and skew (g), to watershed and precipitation characteristics; a separate prediction equation is used for each of the three moments. The peak discharge for a return period (p) is thus obtained by using the following relationship:

$$Q_p = \bar{X} + KS \qquad (2)$$

where

Q_p = peak discharge (ft^3/sec),
$\bar{X}$ = estimated mean value (ft^3/sec),
S = standard deviation (ft^3/sec), and
K = dimensionless value that is a function of both the skew and the return period.

In most cases, $\bar{X}$ and S are the mean and standard deviation of the logarithms of the annual maximum series; in such cases, it is necessary to use the antilogarithm of the Q_p computed with Equation 2. The frequency curve can be evaluated by computing the value of Q_p for selected return periods.

Uncalibrated Equations

Although the single-return-period equation, index-flood method, and the moment-estimation method require fitting to measured hydrologic data, some peak-discharge methods are developed without fitting; these are called uncalibrated equations. The rational method and the TR-55 peak-discharge methods fall into this category. One would expect that the process of fitting would improve the accuracy of a method.

Single-Event Hydrograph Methods

Although unit hydrograph models can be distinguished on the basis of more than one criterion, it is useful to limit the separation criterion to whether or not calibration was required. Calibrated unit hydrographs, such as the HEC-1 model, should provide more accurate estimates of runoff hydrographs than uncalibrated methods. Uncalibrated unit hydrograph models are desirable because they are designed to be used at ungaged locations; the Soil Conservation Service (SCS) TR-20 model is an example of this class. Although unit hydrograph methods provide a storm hydrograph as the primary output, they can also provide either a peak discharge or a frequency curve. Because urbanization causes significant changes in the volumes and timing of runoff, hydrograph models are becoming more widely used, especially where policies require storage of runoff to compensate for the effect of land use changes on the excess runoff volumes and rates.

Multiple-Event Hydrograph Methods

Because the use of hydrograph methods has increased, there has been increased concern about some of the underlying assumptions. For example, the uncalibrated unit hydrograph models most often require acceptance of the assumption that the recurrence interval of the runoff equals that of the rainfall. Empirical evidence indicates that this is rarely the case. In addition, many argue that where data are available, the single-event hydrograph models make little use of the available data. Thus, multiple-event hydrograph models are sometimes recommended. Two subclasses of multiple-event models exist: multiple-event analysis and continuous-record analysis. A multiple-event analysis involves using the larger storm events of record as input to a conceptual hydrologic model. A frequency analysis is performed on the output to develop a frequency curve. A peak discharge can then be obtained from the frequency curve. This model type is based on at least two assumptions. First, it assumes that the largest rainfall event may not cause the greatest runoff; therefore, several storms for each year are used to ensure that the largest event for each year is included in the frequency analysis. Second, it assumes that the hydrometeorological data are sufficient input once the model has been fitted.

Continuous-record models are considered to be at least as accurate as the multiple-event models because they provide for continuous moisture accounting. On the other hand, they require considerably more technical expertise, time, and resources to calibrate and use. The continuous record of computed runoff is used to compute a frequency curve; because the peak-runoff rates are computed rather than measured, the frequency curve is often referred to as synthetic.

CRITERIA FOR COMPARISON OF PROCEDURES

In making design estimates of flood peak discharges, accuracy is often considered the most important criterion. To characterize properly the accuracy of a hydrologic model, it is necessary to identify and to quantify factors that influence the accuracy of the model. That is, accuracy must be separated into its fundamental components. Definitions of precision and bias are needed to define accuracy. Precision is a measure of the random variation in a set of repeated estimates when the procedure is identically evaluated more than once. Bias is a measure of the systematic error in a set of estimates; it measures the deviation of the central tendency of these estimates from the true value. Given these definitions, accuracy can be defined as a measure of the closeness of the predicted values to the true value of the quantity being evaluated; it considers both precision and bias.

In statistical analyses, the mean square error (MSE) is used as the measure of accuracy. Accuracy is a function of systematic and random error variation; accuracy can be separated into the precision and bias components as follows:

$$MSE = E(\hat{\theta} - \theta)^2 \tag{3a}$$

$$= E\left\{[\hat{\theta} - E(\hat{\theta})] + [E(\hat{\theta} - \theta)]\right\}^2 \tag{3b}$$

$$= E[\hat{\theta} - E(\hat{\theta})]^2 + E[E(\hat{\theta}) - \theta]^2 + 2E[\hat{\theta} - E(\hat{\theta})]\,[E(\hat{\theta}) - \theta]^2 \tag{3c}$$

where θ is any parameter, $\hat{\theta}$ is an estimate of θ, and E( ) denotes the expected value of the quantity enclosed in the parentheses or brackets. Given the above definitions for precision and bias and because $E[\hat{\theta} - E(\hat{\theta})] = 0$, MSE is the sum of the precision and the square of the bias:

$$MSE = E\left\{[\hat{\theta} - E(\hat{\theta})]^2\right\} + E[E(\hat{\theta}) - \theta]^2 \tag{4a}$$

$$= \text{precision} + \text{bias}^2 \tag{4b}$$

The variation of an estimated peak discharge from the true value can be represented by

$$(Y_{ijk} - Y_{0i}) = (Y_{ijk} - \overline{Y_{ik}}) + (\overline{Y_{ik}} - Y_{0i}) \tag{5}$$

where Y_{0i} is the true estimate on watershed i, Y_{ijk} is a value estimated by individual j on watershed i and by using procedure k, and $\overline{Y_{ik}}$ is the the mean of all estimates made on watershed i by using procedure k. The terms in the computational equation (Equation 5) correspond directly to the statistical definition of Equation 4.

To assess the precision of a hydrologic model, it would be necessary to make an estimate of the random error. In a strict sense, this requires repeated measurements. True repetition is not possible in hydrology, and thus a true measure of precision is not obtainable. A best estimate of precision can be obtained by using estimates of peak discharge made by different hydrologists. The variation of these estimates is a measure of the random variation. Because it is not a true estimate of precision, it is termed reproducibility. The term $(Y_{ijk} - \overline{Y_{ik}})$ represents the reproducibility of a procedure and is evaluated by repeated use of procedure k on the same watershed by different hydrologists; as such, it is as close as one can come to replication in hydrology. It is intended to provide an answer to the question, "How well can I expect to agree with other hydrologists?" Because replication is usually not available, only accuracy and bias are assessed.

By using the separation-of-variation concept of Equation 5, accuracy equals the variation of the predicted values from the true values. Because the true value differs for each watershed, it is necessary to standardize the differences when the accuracy of a method is evaluated. Thus, the accuracy is evaluated in the form of a standardized standard error:

$$A = \left\{[1/(n-1)] \sum_{i=1}^{n} [(Y_{ijk} - Y_{0i})/Y_{0i}]^2\right\}^{0.5} \tag{6}$$

The true peak discharge (Y_{0i}) is never known. For purposes of comparing hydrologic models where gaged data are available, the flood frequency analysis estimate of a particular exceedence probability can be used as the best estimate (1).

The term $(\overline{Y_{ik}} - Y_{0i})$ is the difference between the mean of all estimates on watershed i by using procedure k and the true value. It represents the systematic error variation of the procedure and identifies either overestimation or underestimation; a zero value indicates no systematic error. The bias of procedure k is estimated by

$$B_k = \left[\sum_{i=1}^{n} (Y_{ijk} * Y_{0i})^2 \Big/ \sum_{i=1}^{n} Y_{0i}^2\right] - 1.0 \tag{7}$$

COMPARISON OF MODEL STRUCTURES AND INPUT REQUIREMENTS

The classification system of Table 1 is based on the primary output. This separation also represents different levels of design requirements. For example, it would not be practical to use a continuous-record model to design storm drain inlets. Similarly, it would not be rational to use an empirical peak-discharge equation to perform real-time flood forecasting. Because of the interest in comparing peak-discharge methods, the remainder of this study will focus on these methods; it is, however, important to recognize the classification system of Table 1 to maintain a proper perspective.

Most of the peak-discharge methods require quite similar input. The drainage area is a major input to most of the equations. An index of the rainfall depth is usually required; this is most often obtained from a curve of rainfall intensity, duration, and frequency for the site. When a rainfall depth or intensity is required, such as with the rational and SCS graphical methods, the product of the rainfall and drainage area reflects a supply of available water for runoff. The actual supply is a function of the return period, which is required by the peak-discharge methods. The reduction of rainfall supply to the volume of direct runoff is usually controlled by a runoff index that is primarily a function of land use; some methods use other factors such as slope or soil type in reducing the rainfall supply to a runoff volume. The slope and length are other watershed characteristics that serve as input to many peak-discharge methods.

Calibrated Equations

Single-Return-Period Equations

The single-return-period equations, which have the structure of Equation 1, are nonlinear multiplicative because the variables have nonunit exponents; thus, the relative change in Q_p due to a change in any of the variables depends on the value of the variables. In this sense, the model is nonlinear. For example, the three-parameter USGS urban peak-discharge equations (4) have the following form:

$$Q_p = b_0 A^{b_1} (13 - BDF)^{b_2} RQ_T^{b_3} \tag{8}$$

where A is the drainage area in square miles; BDF is the basin development factor, which represents the degree of land and channel development; RQ_T is the peak discharge obtained from the USGS equation for rural watersheds within a state; and b_i (i = 0,1,2,3) is the fitting coefficient dependent on the return period (T).

A separate equation is provided for the return periods of 2, 5, 10, 25, 50, 100, and 500 yr. The change in Q_p due to a change in A for Equation 8 is given by

$$\partial Q_p/\partial A = b_0 b_1 A^{b_1 - 1} (13 - BDF)^{b_2} RQ_T^{b_3} \tag{9}$$

The rate of change for the USGS urban equations is nonlinear. It should be evident that the actual slopes of the relationships between Q_p and A for the models will depend on the values of both the variables and the coefficients.

Moment-Estimation Method

Thomas and Benson (5) derived the empirical coefficients of equations for predicting the mean and standard deviation of the logarithms of the annual peak-flow series; the regression equations for the skew coefficients were not statistically significant. Equations were derived for four regions of the United States. The regression equations for the mean ($\overline{X}$) and standard deviation (S) for rural watersheds in the eastern regions are

$$\overline{X} = 0.00264A^{1.01}P^{1.58} \tag{10}$$

$$S = 0.0142A^{0.99}P^{0.85} \tag{11}$$

in which P is the mean annual precipitation in inches. Skew coefficients for ungaged sites can best be estimated by averaging station values within the hydrologic vicinity of the ungaged site. McCuen (6) concluded that for the United States a mean skew value of zero was reasonable. In this case, the value of K of Equation 3 becomes the standardized normal variate and can be obtained from any basic textbook on statistical methods.

Equations 10 and 11 require only the drainage area and the mean annual precipitation to obtain an estimate of the peak discharge. In this respect, the input requirements are easier to obtain than those for the other methods described. Therefore, one would expect the accuracy to be less; nevertheless, because they were derived by regression, the estimates should be unbiased. The equations are nonlinear multiplicative in structure, although the exponents for the drainage area are nearly equal to unity.

Index-Flood Method

The index-flood method requires the calibration of both the equation for the index return period and the ratios between the peak-flow rates for other return periods and the index return period. The structure of the index-flood equation is usually nonlinear multiplicative; watershed and precipitation characteristics are used as predictor variables. As an example, Trent (7) provided the following index-flood equation for estimating the 10-yr peak discharge from small rural watersheds:

$$Q_p = b_0 A^{b_1} R^{b_2} DH^{b_3} \tag{12}$$

in which R is an iso-erodent factor, defined as the mean annual rainfall kinetic energy times the annual maximum 30-min rainfall intensity, and DH is the difference in feet of the elevation of the main channel between the most distant point on the watershed boundary and the design point. The coefficients b_i (i = 0,1,2,3) are a function of the hydrophysiographic zone. The estimated peak dicharge must be modified when the surface water storage in lakes, swamps, and ponds exceeds 4 percent. The 2-yr peak discharge is estimated by multiplying the 10-yr peak (Q_{10}) by the index ratio of 0.41. The 100-yr peak (Q_{100}) can be estimated by

$$Q_{100} = 1.64Q_{10}^{1.029} \tag{13}$$

If the index ratios are obtained by regression, the index-flood method should provide unbiased estimates of the peak discharge. Nevertheless, the accuracy of the estimates for return periods other than the index return period can be no greater than that obtained by the single-return-period equations; in most cases, the accuracy of the index-flood method will be less because the ratio represents a single fitting coefficient. For the single-return-period equation of the same return period, several coefficients are available for fitting.

Uncalibrated Equations

The three methods discussed earlier, i.e., single-return-period equations, index-flood method, and moment estimation, require calibration; that is, the methods are fitted to peak-flow rates obtained from flood frequency analyses. Past empirical studies have indicated that the nonlinear multiplicative structure provides the greatest accuracy. Thus, this structure is usually chosen for these methods.

Uncalibrated equations are most often based on a conceptual framework. Therefore, the model structure is not simply chosen; instead, the structure is the result of the conceptual framework. Thus, the structure of uncalibrated equations shows wider variation than that of the calibrated methods.

Rational Formula

The rational formula is the most widely used hydrologic equation. It has the following form:

$$Q_p = CiA \tag{14}$$

where

- Q_p = peak discharge (ft³/sec),
- C = runoff coefficient,
- i = rainfall intensity (in./hr), and
- A = drainage area (acres).

The form of the rational method results from the underlying conceptual framework. The method assumes a constant rainfall of intensity i for a duration of t_c (hr); thus, the total rainfall depth is it_c. The product of the drainage area and the total rainfall depth is the volume of rainfall in inches that is available for runoff. The runoff coefficient (C) determines the proportion of the rainfall volume that appears as runoff. Conceptually, the runoff hydrograph for the rational method is triangular with a time base of $2t_c$, a time to peak of t_c, and a volume of runoff of $CiAt_c$; thus, 50 percent of the runoff lies under the rising limb of the runoff hydrograph.

The runoff coefficient is usually obtained from a table and is defined in terms of the land use. Some tables provide for selection of the value on the basis of return period and slope; the value of C increases for the less frequent events and with increasing slope. Some tables provide a range of C-values for each land use; although this permits the designer to select a value that reflects on-site conditions, it also leads to a lack of reproducibility. Poor reproducibility often creates difficulties between those proposing site development and those who are responsible for approving site-development plans. The rainfall intensity of Equation 14 is a function of the return period, the location, and the storm duration; the storm duration is most commonly taken as the time of concentration, although it has been shown that the critical storm duration may actually be shorter than the time of concentration (8). The value of i is obtained from a curve of rainfall intensity, duration, and frequency for the location. The relationship between the intensity and time of concentration (t_c) can be represented by an equation of the following form:

$$i = d_0 t_c^{d_1} \tag{15}$$

in which d_0 and d_1 are empirical coefficients that reflect both the location and the units of i and t_c. The time of concentration is a function of the slope, length, and land cover; the value of t_c has also been shown to be a function of rainfall intensity (9,10), although most methods for estimating t_c are independent of i. When t_c is a function of i, an iterative solution is necessary because i is also a function of t_c.

In summary, the basic input data required to use the rational method are the drainage area, the watershed slope, the hydraulic length, the return period, a nominal statement of the land cover, a table of C-values, and a curve of rainfall intensity, duration, and frequency for the site location. The drainage area, slope, and hydraulic length are obtained from either a site survey or a commercially available topographic map. For cases of nonhomogeneous land cover, the slope, length, and land cover are obtained for each flow segment to compute t_c.

SCS TR-55 Graphical Method

The graphical method is quite similar in concept and structure to the rational formula and has the following form (11):

$$Q_p = q_u AQ/640 \tag{16}$$

where

- Q_p = peak discharge (ft^3/sec),
- q_u = unit peak discharge [ft^3/($mile^2$ • in.) of direct runoff],
- A = drainage area (acres), and
- Q = direct runoff (in.).

The unit peak discharge, which is obtained from Figure 5-2 of TR-55 (11), is a function of the time of concentration measured in hours. The runoff volume (Q) is a function of the SCS runoff curve number (CN) and the 24-hr rainfall depth (P_{24}) in inches. The curve number is a function of the land use, cover condition, and SCS soil type; CN is obtained from a table. The value of P_{24} is a function of location and return period and is obtained from a volume-duration-frequency curve for the site location. The input requirements for the SCS graphical method are the drainage area, the watershed slope, the hydraulic length, the return period, a nominal statement of the land cover and condition, the soil type, a table of CN-values, the location, and the volume-duration-frequency curve for the location.

The graphical method of Equation 16 has a linear multiplicative structure, even though the equation for computing the runoff volume Q is nonlinear. The curve relating the unit peak discharge and the time of concentration is also nonlinear; actually, the structure of the curve of q_u versus t_c is quite similar to the structure of the intensity-duration-frequency curve used with the rational formula. It is evident that the rational method and the graphical method are almost identical in both structure and input requirements. The structures are classed as linear multiplicative because peak discharge is linearly related to each of the variables defined in the equation. For example, a change in the drainage area of 1 acre causes the same relative change in Q_p regardless of the value of A. The two methods are multiplicative as opposed to being additive because the peak discharge is obtained by multiplying the values of the input variables.

The graphical method was formulated from numerous runs of the SCS TR-20 program (12). The TR-20 program uses a curvilinear unit hydrograph to compute the runoff hydrograph and thus the peak discharge. This curvilinear unit hydrograph has 37.5 percent of the volume under the rising limb. The time to peak of the unit hydrograph is two-thirds of the time of concentration. The runoff volume is a nonlinear function of the precipitation. It should be evident that conceptual differences exist between the graphical and the rational methods despite their use of similar input.

Summary

It should be evident that the peak-discharge methods differ little in either their structure or their input requirements. The input usually consists of the drainage area, a precipitation characteristic, and one or more watershed characteristics. The main difference between methods, at least with respect to input requirements, is the number of predictor variables used. The accuracy of prediction does not appear to improve when variables are added beyond the drainage area, the precipitation index, a land use index, and a watershed characteristic such as the slope.

The structures of the methods are also quite similar. Although linear multiplicative structures are often used for the uncalibrated equations, the other peak-discharge methods usually rely on nonlinear multiplicative form, which is a more flexible structure. For the uncalibrated methods, the linear multiplicative structure is used because empirical evidence indicates a wide range of values for the exponents. For example, for estimating peak discharges in Iowa, Lara (13) reported exponents for the drainage-area variable from 0.42 to 0.70. Sauer and others (4) reported values from 0.15 to 0.41 for nationwide urban peak-discharge equations. For estimating floods in Maryland, Walker (14) reported values from 0.8585 to 0.947. Trent (7) reported values from 0.23 to 1.31. It is evident that the empirical coefficients are highly variable and depend on factors such as location, type of watershed (urban or rural), and the range of watershed sizes.

Given that the structures and input requirements are similar, is there any reason to believe that one method is any better than the others? The major difference certainly is that which exists between the calibrated and the uncalibrated methods. Calibrated methods should provide unbiased estimates; they will also provide more accurate estimates when the test watersheds are similar to those used in calibrating the equation. Accuracy can be expected to decrease significantly as the coefficients become less applicable to the watersheds being tested.

Among the calibrated methods (i.e., single return period, index flood, and moment estimation), the major difference lies in the number of coefficients calibrated. For example, the single-return-period method required 24 coefficients for the 3-parameter models for the 2-, 5-, 10-, 25-, 50-, and 100-yr peaks. The moment-estimation technique would probably require fewer because there are only three equations to calibrate. The index flood would require one set for the index flood and at least five for the index ratios. Although the larger number of coefficients should lead to greater accuracy for the single-return-period method, the increased accuracy may not be statistically significant. Nevertheless, the independent calibration of single-return-period equations for different return periods may not preserve the skew of the individual station frequency curves. The moment-estimation method would be expected to best replicate the shape characteristics of the frequency curve.

ACCOUNTING FOR FLOW IN CHANNEL SYSTEMS IN MAKING PEAK-DISCHARGE ESTIMATES

Hydrograph and multiple-event methods almost always include one or more input variables or parameters that reflect flow in channel systems. For example, the SCS TR-20 model uses the channel length and convex method routing coefficient to reflect channel characteristics. Channel system characteristics are not handled in such a direct manner with most of the peak-discharge methods. The uncalibrated equations such as the rational formula and SCS TR-55 graphical method do not include specific variables to reflect channel characteristics; nevertheless, flow in channel systems can be partly accounted for by including channel-flow characteristics in the computation of the time of concentration. This indirect method of accounting for channel characteristics limits the potential accuracy of the methods for watersheds where channel flow is significant. The uncalibrated equations should not be used where channel storage effects are significant. That is, where flow rates are significantly affected by channel characteristics, adjustment of the time of concentration may not be adequate for handling the effects of channel characteristics on peak-discharge rates.

Conceptually, the product of the intensity and the drainage area in the rational formula represents the supply rate of water; the runoff coefficient represents the portion of the supply rate that is converted into direct runoff; the proportion (1 - C) represents the losses due to interception and other overland flow processes, such as depression storage and infiltration. When the time of concentration is adjusted to reflect channel runoff, it is not totally reasonable that the shape of the intensity-duration-frequency curve, from which the value of i is obtained, reflects the sensitivity of peak discharge to channel characteristics. A similar argument can be made for the graphical method. If the effect of channel characteristics is accounted for in the time of concentration, it is not totally reasonable that the shape of the unit peak-discharge curve of TR-55 reflects the sensitivity of peak discharge to channel characteristics.

The three types of peak-discharge methods that usually require calibration most often account for flow in channel systems differently than do the uncalibrated equations. Specifically, the single-return-period equations, the moment-estimation methods, and the index-flood methods are often calibrated by using data obtained from stream gages. In such cases, the log Pearson type III estimates of the peak discharge and the statistical moments of the annual maximum series reflect the effects of the channel system. Thus, the values of the fitting coefficients reflect the channel system. For example, the three-parameter USGS urban equation (Equation 8) contains four coefficients that are directly affected by the channel characteristics of the urban watersheds that were used to calibrate the model. Also, the coefficients in the equations that are used to estimate RQ_T contain fitting coefficients that reflect the channel characteristics of the rural watersheds that were used to calibrate the models for predicting RQ_T.

The point of this discussion is that methods calibrated with data obtained from stream gages should be expected to perform differently from those in which the characteristics of the channel system must be reflected indirectly, such as through the time of concentration. When models calibrated with data from stream gages are compared with peak discharges obtained from stream-gage records, one would expect such models to perform better, in terms of accuracy and bias, than models that were not calibrated. Similarly the calibrated models might not perform as well as the uncalibrated models when the models are compared with data obtained from watersheds where channel systems are nonexistent or minor. In summary, it is important for a model to be used under conditions similar to those used in the development or calibration of the model.

COMPARISONS WITH MEASURED DATA

Because the peak-discharge methods are so widely used, comparisons of the methods are of special interest. Two studies have recently been undertaken that illustrate the concepts discussed earlier. The Water Resources Council (WRC) study (1) compared nine methods on 70 rural watersheds. Because the USGS single-return-period equations for each state, the rational method, and the SCS graphical method are widely used, it is of interest to compare the results from the WRC study for these three methods. The mean bias of the 100-yr event was 10, 80, and 75 percent, respectively, for the three methods. The interquartile ranges of the percentage of deviation from the gage estimate were 45, 180, and 165 percent, respectively. Given that the USGS equations are intended for rural watersheds and were calibrated from similar data, it is not surprising that these equations produced the smallest bias and the smallest dispersion. The rational and graphical methods were intended to be used on watersheds that were smaller than most of the watersheds included in the study. In addition, these two methods are used mostly to estimate inlet peak discharges rather than peaks on streams of significant channel storage where gages would likely be located. Thus, the large positive biases and large ranges should not have been unexpected.

Rawls, Wong, and McCuen (15) compared several peak-discharge methods on 40 small urban watersheds. The watersheds included in this data set had drainage areas less than 4,000 acres. The methods included the USGS urban equations (Equation 8), the rational method (Equation 14), and the graphical method (Equation 16). By using the bias and accuracy statistics of Equations 6 and 7, the three methods resulted in bias values of -0.11, -0.49, and -0.07, respectively, and accuracy values of 0.66, 0.68, and 1.17, respectively, for the 100-yr events. Because the data base that was used for testing was part of the data base used to calibrate the USGS equation, the bias and accuracy values of -0.11 and 0.66 can be used as standards of comparison. The graphical method is relatively unbiased, whereas the rational method still tended to overpredict. The graphical method showed somewhat greater scatter (i.e., poorer accuracy) than that of the USGS equations, whereas the scatter for the rational method was comparable with that of the USGS equation. The low bias of the graphical method indicates that, on the average, the method provides reasonable agreement to peak discharges computed by using log Pearson type III analyses. The higher variability in comparison with that of the USGS method probably results because it was not calibrated from such data. The tendency of the rational method to overpredict may indicate that it was applied to watersheds that are too large; that is, the conceptual framework and runoff coefficients may not be applicable to watersheds as large as 4,000 acres.

DISCUSSION AND CONCLUSIONS

Despite empirical studies comparing peak-discharge methods, the debate continues over which method should be used in design. Although the comparison studies have suggested that calibrated equations are relatively unbiased and have the smallest error variation, all of the studies have avoided defining what represents a significant difference. Furthermore, the WRC study (1) suggested that the sample sizes used in these comparison studies were inadequate for making conclusive statements. If the empirical evidence is inadequate, it is possible to combine the results of the empirical studies with a rational analysis of the conceptual framework, structure, and input requirements of the methods.

When the peak-discharge methods are compared on the basis of their input requirements, there is little difference; drainage area is usually the most important input variable; a rainfall characteristic and a time characteristic are other common, important input variables. The methods also differ little in structure. Although the methods calibrated are usually nonlinear, the variation in the coefficients from one empirical study to another is sufficiently large that the results do not suggest that a linear structure is unreasonable.

The greatest difference between the methods is their conceptual framework. The calibrated equations emphasize channel characteristics, whereas the uncalibrated equations emphasize surface-runoff characteristics. The input variables for the calibrated methods are often similar to those for the uncalibrated equations, but the fitting coefficients provide a conceptual mechanism for incorporating channel characteristics into the estimated peak discharges. Although the uncalibrated equations can attempt to account for channel flow by modifying the time of concentration, the use of Manning's equation for computing channel velocities cannot totally reflect channel-storage characteristics. Thus, for watersheds where the flow in channels is significant, the calibrated methods have a distinct advantage.

The uncalibrated methods also differ conceptually among themselves. For example, although both the graphical and the rational methods are based on unit hydrograph concepts, the rational method assumes a much larger portion of flow within the rising limb of the hydrograph than the graphical method (i.e., 50 percent versus 37.5 percent). Thus, one would expect that the rational method would be more appropriate for small watersheds where the land cover conditions cause a rapid response. The graphical method appears to be more appropriate for slightly larger watersheds where surface runoff storage effects are more evident.

Where it is necessary to formulate design standards as part of stormwater management or drainage policies, how does this rational comparison provide insight concerning which method to select? Both the empirical evidence and the rational analysis suggest that a single-return-period equation should be used where a peak discharge is needed on a stream having significant storage. If an entire frequency curve is required, the moment estimation may be preferable. For small watersheds where surface runoff dominates, the uncalibrated equations may be preferred. Selection of the uncalibrated equation should depend on the similarity of the watershed characteristics to the characteristics of the site. For small inlet areas, the rational method may be preferred; selection of this method, however, would assume that the watershed response is rapid. Thus, the rational method may not be appropriate for low sloped areas such as coastal watersheds.

To summarize, in formulating policy adequate consideration should be given to the agreement between the conceptual framework of the design method and the characteristics of the design problem for which the policy is intended.

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