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A modified approach to flood prediction in urban watersheds

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A MODIFIED APPROACH TO
FLOOD PREDICTION IN URBAN WATERSHEDS

by
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APPROVAL PAGE

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ABSTRACT

A Modified Approach to Flood Prediction in Urban Watersheds

by

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The purpose of this dissertation was to develop a better method for dealing with the problems of flood prediction in Urban Watersheds. It has long been realized that urbanization activity such as increased imperviousness, drainage improvements, etc. increases runoff volumes. Therefore, traditional flood prediction methods using the Log Pearson III distribution underestimate flood frequencies when applied to urban watersheds without modification.

In attempt to compensate for the effects of urbanization on streamflow, previous workers usually employed regional analysis techniques involving a number of different watersheds at various degrees of urbanization. Results obtained by this approach leave room for improvement primarily due to heterogeneities in hydrologic characteristics of watersheds. In contrast, the method developed in this thesis characterizes a watershed using a time based analysis in which the basin response patterns are studied through as long a period as data exists.

The method proposed is based on the hypothesis that basin response to small storms after dry periods derives

mainly from impervious areas and hence provides a measure of the basin's state of development. By analyzing the peak flows resulting from drought period small storms over a long period of time, a trend equation may be established indicating the growth pattern of runoff contributed largely by impervious surfaces. This relationship in turn forms the basis for separating runoff components from pervious and impervious areas during major, wet period storms.

Next, the impervious surface runoff contribution is updated to present conditions equivalent flow by again applying the above trend equation, while the pervious surface contribution is updated by the ratio of the pervious surface in the present year to the pervious surface in the year in consideration.

Finally, the composite update ratios thus calculated are applied on the historic record of annual peak flows and the Log Pearson III technique applied to predict future floods.

The above ideas were illustrated using the Saddle River Basin in New Jersey. The maximum update ratio obtained was about 1.8 and the predicted floods increased in the range of 1.05 for the 100 year flood to 1.4 for the 2 year flood.

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This thesis is dedicated to my senior brother,

Ben Ofungwu

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Figure **Page**

CHAPTER 1

INTRODUCTION

1.1 General

Most hydrologic processes cannot be predicted or evaluated solely on a deterministic basis because it is not possible to quantify all their causative mechanisms. Due to the inherent randomness of these phenomena it becomes necessary to resort to statistical methods for analysis and predictions. Rainfall and streamflow are two examples that readily come to mind. Statistical methods offer a mechanism for reducing, organizing and presenting observed hydrologic data in a manner that facilitates their interpretation and utilization.

It has been stated above that hydrologic processes such as rainfall and streamflow are random variables because their causal mechanisms are uncertain and therefore their prediction can only be probabilistic. In nature, engineers and planners need to consider the effects of flooding in planning for land use and urban development, in designing dams, culverts, bridges, drainage systems, etc. Some sense of what might be expected is also necessary in establishing flood insurance rates.

If accurate knowledge of flood characteristics is available, then adequate storm sewers and other drainage structures can be constructed economically, while flooding hazards can be minimized through effective zoning in flood prone areas.

Some Common Probabilistic Models

To meet the above challenges, statisticians have developed a number of probabilistic models that may be applied to hydrologic data.

The more commonly used distributions include the Gumbel (also known as extreme value type I), normal (or Gaussian), Log Normal, gamma (Pearson III), Log gamma (Log Pearson III) and exponential distributions.

The Log Pearson III Distribution

Based on studies conducted at the Center for Research in Water Resources at the University of Texas at Austin, the United States Water Resources Council (now known as the U.S. Interagency Advisory Committee on Water Data) recommended the Log Pearson III distribution as the most accurate method for frequency analysis of flood flows. The recommended technique for application of this distribution is detailed in the Council's Guidelines for Determining Flood Flow Frequency, Bulletin 17 (revised as Bulletin 17B in September 1981).

The Gamma or Pearson Type 3 distribution is popular in hydrology because it has well known mathematical properties

(Hann, 1977) and because it is conveniently shaped. The three parameters of this distribution are simple functions of the mean, variance and skewness but it is usual to evaluate the continuous density function with frequency factors, K, which are tabulated in most hydrology texts.

It is possible to consider a hypothetical run-off calculation in which run-off equals the product of functions of several random variables such as rainfall, evaporation, infiltration, etc. From the Central Limit Theorem, if a random variable X results from the product of a large number of other random variables, then the distribution of the logarithm of X will approach the normal distribution since the logarithm of X comprises the sum of the logarithms of the contributory parameters.
This assumption of a multiplicative mechanism for run-off may be the basis for preference for Log Pearson III over the Pearson III distribution.

Constraints in the use of the Log Pearson III distribution

As stated in the Water Resources Council's Bulletin 17, a statistical analysis requires that the array of flood information to be analyzed represent an adequate time sample of random, homogeneous events. Broadly speaking, an array of annual maximum peak flows in a rural watershed may be considered a random sample. It can be assumed that random hydrologic processes of rainfall, infiltration, evaporation, etc. result in random flood flows.

This situation changes when the flow regime in the watershed is altered over time by development activity such as increased imperviousness, channelization, storm sewerage, flow diversions or reservoirs, etc.

The annual peak series becomes a mixed sample consisting of a random component as described above and a non-random component determined by urban development.

Application of the Log Pearson III distribution or other frequency analysis as originally formulated, in a developing watershed can therefore be expected to yield misleading results.

For instance, it is generally agreed that urbanization leads to increased run-off volumes. In consequence, floods that are predicted to have certain recurrence intervals are observed to occur much more frequently in practice.

Herein lies the focus of this research. As explained in the following section, a function is developed which is applied to the annual peak flows to account for the expected increases in streamflow resulting from urbanization, prior to frequency analysis using Log Pearson III .

1.2 Purpose and Proposal

The purpose of this study is to develop a methodology for modifying recorded annual peak flows in an urbanizing watershed so as to create a homogeneous data sample on which to perform flood frequency calculations. It is proposed to achieve homogeneity by establishing a common reference frame, namely present development conditions for all recorded annual peak flows. In other words, the focal question is "how would the current watershed respond to historic rainfall events?"

Since it is recognized that development activity changes the run-off generation mechanism within a watershed, transforming the historic record of annual peak flows to their present day equivalents before frequency analysis should lead to more realistic, though higher, results for the predicted flood flows.

The proposed approach involves a time based study of the basin in question. By focusing on one basin at a time, such variables as topography, soil type and loss coefficients, vegetative cover, surface storage, etc. that introduce much error in regional analysis, are no longer in contention.

For practical purposes, the hydrologic, meteorologic and physiographic characteristics of a basin remain invariant. Interest is therefore concentrated on the interrelationship between rainfall intensity, urban development and total run-off.

In summary, this concept of compensating for the effects of urbanization on flood flows within a basin by studying hydrologic relationships over time within the same basin rather than through regional relationships that attempt to link different hydrologic domains, constitutes the point of departure of this study from previous research. It is believed that eliminating many of the sources of error mentioned above assures greater accuracy in the final analysis.

CHAPTER 2

LITERATURE REVIEW

The problem of urban flood prediction has been studied since the ninteen-sixties. It was realized that early that urban development introduces changes in the flow regime within a river basin and many workers have tried to develop techniques for accommodating the effects of these changes on the flood prediction process.

Some of the more frequently cited studies are reviewed below in summary form:

2.1 Carter, R.W. (1961)

Carter's paper was titled Magnitude and Frequency of Floods in Suburban Areas.

He made the following assumptions:

1. The average rainfall-run-off coefficient of 0.3 as determined from rainfall-flood volume studies for storms in the Washington, D.C. area applies to flood peaks as well as to flood volumes.
2. The effect of increases in impervious area does not depend on the size of the flood.
3. Seventy-five percent of the rainfall volume on impervious surfaces reaches the stream channel.
4. The impervious area consists of many fairly small areas randomly distributed throughout the basin.

Using these assumptions, he obtained following equation:

$$K = (0.3+0.0045\ I)/0.30$$

Where I = percent impervious area

K = Factor by which flood peaks are increased to account for imperviousness, I.

Next, he related lag time, T, to the ratio $L/S^{0.5}$

Where L = Length from the gaging station to the rim of the basin measured along the principal channel.

S = Weighted slope of all stream channels in basin

For undeveloped basins he found:

$$T = 3.10\ (L/S^{.5})^{0.6}$$

For partly sewered basins with natural channels the relation becomes :

$$T = 1.20\ (L/S^{.5})^{0.6}$$

There was not sufficient data to define a relationship for completely sewered basins. See fig. 2.1.

Finally, he applied multiple regression techniques to relate mean annual flood (recurrence interval = 2.33 years) to the basin area, lag time and imperviousness (through K).

He obtained: $Q_n/K = 223\ A^{0.85}\ T^{-0.45}$

Where Q_n = mean annual discharge (cfs)

T = lagtime (hours)

A = Basin area (square miles)

K = as previously defined.

The average standard error was + or - 25% and from sample calculations he concluded that the ratio

$$Q_n\ \text{suburban} / Q_n\ \text{undeveloped}$$

has a maximum value of 1.8 for the Washington D.C. area.

Comment

Carter's assumption that the effect of imperviousness is independent of the flood size has been proven invalid by subsequent studies.

It is now known that as the flood size increases, the soil becomes saturated, infiltration tapers off and run-off is contributed almost equally by pervious and impervious areas. Secondly, only the mean annual discharge was considered. No information was available on other frequencies. Thirdly, it is uncertain to what extent the relations derived are applicable outside the study basins.

2.2 Anderson, Daniel G. (1968)

This study was titled: Effects of Urban Development on Floods in Northern Virginia.

As described in 2.1, Carter (1961) limited his effort to the mean annual discharge and ignored the effects of urbanization on floods of other return periods.

Anderson extended Carter's work by using an expanded data base (81 basins) and developed adjustment relationships for recurrence intervals ranging up to the 100 year flood.

He accepted Carter's equation for K but went further to use Dalrymple's (1960) concept of flood frequency ratios to define relationships for various recurrence intervals and various degrees of basin development.

First, he slightly modified the estimating equation for basin lag. For natural-rural basins:

$$T = 4.64\ (L/s^{.5})^{0.42}$$

For completely sewered basins:

$$T = 0.56\ (L/s^{.5})^{0.52}$$

For developed, partly channelled basins (interpreted to mean storm sewering of all small tributaries but either natural larger channels or moderate improvement by alignment and rough surfaced banks of rock or grass):

$$T = 0.9(L/s^{.5})^{0.50}$$

See figure 2.2.

Following from the modification of the lag equation, Carter's final relationship was in turn modified to :

$$Q_n/K = 230\ A^{0.82}\ T^{-0.48}$$

Next, using Dalrymple's (1960) ideas the flood sizes for various recurrence intervals were normalized by dividing by the mean annual discharge in order to obtain dimensionless frequency relations for comparative purposes. For natural basins, seven undeveloped basins were studied and median values of the ratio

$$\frac{\text{Flood size at stated return period}}{\text{Mean annual flood at 2.33 years return period}}$$

were selected.

For developed basins, there was not sufficient data to define equivalent dimensionless flood frequency relationships for various degrees of imperviousness.

Alternatively, it was assumed that the shape of a dimensionless frequency curve for impervious basins approaches the shape of a dimensionless rainfall-frequency relation as imperviousness approaches 100%.

The U.S. Weather Bureau rainfall-frequency relations (1955) were therefore used to establish the dimensionless ratios for 100% impervious basins. Table 2.1 shows the values obtained.

Table 2.1
Flood Frequency Dimensionless Ratios
for Rural and 100% Impervious Watersheds

Recurrence Interval (years)	Flood Frequency Ratios	
	Rural	100% Impervious
2.33 (mean annual)	1.0	1.0
10	2.2	1.45
25	3.3	1.80
50	4.4	2.0
100	5.5	2.2

To interpolate for basin conditions between the extremes of zero and 100%, this equation was used:

$$R_i = \frac{Rn + 0.01\ I(2.5\ R100 - Rn)}{1.00 + 0.015\ I}$$

Where R_i = dimensionless flood ratio for given % imperviousness

I = % imperviousness

R100 = dimensionless ratio for a 100% impervious basin

Rn = flood ratio for a natural basin

This relationship was used in defining the curves in fig. 2.3 which provide dimensionless ratios for various recurrence intervals.

Finally, combining all the relations developed in this analysis:

$$Qx = (Qm)(R) = 230\ K\ (R)\ A^{0.82}\ T^{-0.48}$$

Where Qx = the magnitude of a flood of x-year recurrence interval (cfs)

Qn = the mean annual flood (cfs)

R = Dimensionless flood frequency ratio from fig. 2.3

K = Coefficient of imperviousness as previously defined

A = Basin area (square miles)

T = Lag time (hours) from fig. 2.2

Comment

Anderson carried over the assumption that the effect of imperviousness is not affected by flood size from Carter's definition of the imperviousness factor K. However, at the end of his analysis, he concluded "A complete impervious

surface will increase the average size flood (i.e. mean annual flood) by a factor of 2.5, but impervious surface has a decreasing effect upon larger floods and has an insignificant effect upon the 100 year flood."
As will be shown in the results section of this study, Anderson's conclusion agrees to some extent with the results obtained here. It is shown in Table 5.9 that the 100 year flood is increased by only 5% due to urbanization effects. Secondly, Anderson's analysis is affected by the errors inherent in matching data from different hydrologic domains, as is usually the case for most regional studies.

2.3 Leopold, Luna B. (1968)

This study was titled: Hydrology for Urban Land Planning - A Guidebook on the Hydrologic Effects of Urban Land Use.
Leopold identified four interrelated but separable effects of land-use changes on the hydrology of an area, namely, changes in peak flow characteristics, changes in total run-off, changes in water quality and changes in the aesthetic appearance of the basin. He states, "Of all land use changes affecting the hydrology of an area, urbanization is the most forceful."

To quantify the effects of urbanization on peak flows, he assembled data from the reports of previous investigators (including Carter (1961) and Anderson (1968)). Then using this data, he established a series of curves indicating values of the ratio:

$$\frac{\text{mean annual discharge after urbanization}}{\text{mean annual discharge before urbanization}}$$

for different degrees of sewarage and imperviousness. See fig. 2.4 (page 32). In interpreting the results of previous studies, assumptions had to be made as to what was intended by descriptions such as "partly sewered" pertaining to basins. For instance in Carter's study it was assumed that "partly sewered meant 50% sewered and 20% impervious. One square mile was assumed as the standard planning unit and the data values extrapolated to this common denominator.

For the Brandywine Creek basin in Pennsylvania he related drainage basin area to average annual discharge and then using a regional flood frequency curve (relating the ratio of peak discharge to mean annual discharge for different return periods), he worked out the flood peaks for various return perioods for a 1.0 square mile unurbanized basin. This result was presented in the form of a frequency curve.

Finally, using the ratios established earlier in fig. 2.4, he sketched in frequency curves for different degrees of imperviousness and sewarage using his best judgement but guided by the principle that the larger floods are less susceptible to the effects of urbanization while the smaller and more frequent storms have the greatest increases.

2.4 Stankowski, Stephen J. (1974)

Stankowski's report was titled Magnitude and Frequency of Floods in New Jersey with Effects of Urbanization.
The most distinctive feature of this study was the formulation of a relationship between population density and percent impervious surface, that greatly facilitated the estimation of the percentage of a basin rendered impervious by urbanization. Instead of the standard but tedious procedure of measuring impervious surface from aerial photographs and land use maps, the estimated population density within the basin was simply applied to this equation:

$$I = 0.117\ D^{\,0.792-0.039\ \log D}$$

where I = Index of manmade impervious cover (as percent of total basin area)

D = Population density (persons/sq. mi.)

Table 2.2 gives a range of average percentages of impervious cover representing the effects of typical urban development activity in each land use category as estimated from general field observations and the reports of previous investigators.

Table 2.2
Impervious Land Area
<u>For Various Land Use Categories</u>

Land Use Category	Impervious Land Area (%)		
	Low	Intermediate	High
Single family residential	12	25	40
Multifamily residential	60	70	80
Commercial	80	90	100
Public and quasipublic	50	60	75
Conservational, open and Recreational	0	0	1

By using the intermediate values of percent imperviousness shown in Table 2.2 as weighting factors, the impervious area in each of the 567 municipalities in New Jersey was determined as the sum of the weighted proportions of land area in each land use category.
Municipal population density data for 1966 was then plotted against impervious area and the estimating equation given above was fitted to the plotted data.

Next, multiple regression analysis was used to develop relationships between flood discharges for various return periods and hydrologic characteristics of the basin. One hundred and three river basins were analyzed in this manner and the following relationships were found:

$$Q_2 = 25.6\ A^{0.89}\ S^{0.25}\ st^{-0.56}\ I^{0.25}$$

$$Q5 = 39.7\ A^{0.88}\ S^{0.26}\ st^{-0.54}\ I^{0.22}$$

$$Q_{10} = 54.0\ A^{0.88}\ S^{0.27}\ st^{-0.53}\ I^{0.20}$$

$$Q_{25} = 78.2\ A^{0.86}\ S^{0.27}\ st^{-0.52}\ I^{0.18}$$

$$Q_{50} = 104.0\ A^{0.85}\ S^{0.26}\ st^{-0.51}\ I^{0.16}$$

$$Q_{100} = 136.0\ A^{0.84}\ S^{0.26}\ st^{-0.51}\ I^{0.14}$$

Where Q_T = Peak discharge for T-year recurrence interval (cfs)

A = Drainage basin area (square miles)

S = Slope of main channel in feet per mile, defined as the average slope of the main channel between points 10% and 85% of the distance from the gaging station to the basin boundary

st = Surface storage index, in percent of drainage area occupied by lakes and swamps, increased by 1%

I = Percent imperviousness (minimum of 1%).

Comment

Stankowski's method suffers the setbacks usually associated with regional analyses in their attempts to aggregate river basins with very different hydrologic and physiographic characteristics. Additional error was introduced by the effort to link population density to imperviousness because situations frequently arise where actual land use characteristics of some communities deviate widely from what is considered typical.

The average standard errors for Stankowski's regression equations for Q_T ranged between 48% and 54%.

2.5 Sauer et al (1983)

Their paper was titled, <u>Flood Characteristics of Urban Watersheds in the United States.</u>

These investigators developed three sets of regression equations for estimating flood discharges in ungaged basins or basins with various degrees of urbanization and for various return periods.

Their analysis was based on two important descriptors. The first is an independent estimate of the equivalent rural discharge for the basin. For instance, using Stankowski's regression equations for New Jersey, the required equivalent rural discharge is obtained by setting imperviousness to 1%. The second important descriptor is what was referred to as the basin development factor (BDF), which is a measure of the extent of development of the drainage system in the basin. Two hundred and sixty nine gaged basins, at various degrees of urbanization, were analyzed.

For each basin, two sets of flood frequency estimates were defined. The first set relates to the basin in a rural condition and is obtained from previous, independent studies of urban flood characteristics such as Stankowski's report,

described above. The second set of flood frequency estimates pertains to the basin in an urbanized condition. For this second set, the flood frequency curve is determined in the usual manner using the Log Pearson III and recorded or synthetically derived annual peak flows.
Additional data is assembled for each basin including area, slope, surface storage and basin rainfall.

Before presenting the results of the regression analysis, some explanations are needed as to how the variable basin development factor (BDF) is determined. The BDF is a measure of the efficiency of the drainage system. First the basin is divided into upper, middle and lower thirds. Then, within each third four aspects of the drainage system are evaluated and each assigned a code as follows:

1. Channel improvements

If at least 50% of the main drainage channels and the principal tributaries (ie. those that drain directly into the main channel) have received some degree of improvement (such as straightening, enlarging, deepening or clearing), then a code of 1 is assigned. If not, a code of zero is given.

2. Channel linings

If at least 50% of the length of the main drainage channels and principal tributaries have been lined with an impervious material, such as concrete, then a code of 1 is assigned to this aspect. If it is less than 50% lined, zero is assigned.

3. Storm drains or sewers

These are enclosed drainage structures (usually pipes) frequently serving as secondary tributaries fed directly from streets and parking lots and emptying into open channels or enclosed culverts. If 50% or more of the secondary tributaries within the subarea (ie. a third) consists of storm drains, then a code of 1 is assigned. If not then zero will be the value.

4. Curb and gutter streets

If 50% or more of a subarea is urbanized (ie. having residential, commercial and/or industrial development), and if 50% or more of the streets and highways in the subarea are constructed with curbs and gutters, then a code of 1 is assigned. If not a value of zero is assigned.

The BDF is the sum of the assigned codes. Therefore, with three subareas per basin and four drainage aspects per subarea, the maximum value for a fully developed drainage system would be 12. Conversely, for a totally undeveloped system, the BDF would equal zero. In the regression analysis BDF is represented as (13-BDF) in order to accommodate the possibility of zero BDF and not create a singularity condition.

The first set of regression equations obtained are presented below:

$$Q_2 = 2.35A^{.41}Sl^{.17}(RI_2+3)^{2.04}(St+8)^{-.65}(13\text{-}BDF)^{-032}IA^{.15}RQ_2^{.47}$$

$$Q_5 = 2.70A^{.35}Sl^{.16}(RI_2+3)^{1.86}(St+8)^{-.59}(13\text{-}BDF)^{-.31}IA^{.11}RQ_5^{.54}$$

$$Q_{10} = 2.99A^{.32}Sl^{.15}(RI_2+3)^{1.75}(St+8)^{-.57}(13\text{-}BDF)^{-.30}IA^{.9}RQ_{10}^{.58}$$

$$Q_{25} = 2.78A^{.31}Sl^{.15}(RI_2+3)^{1.76}(St+8)^{-.55}(13\text{-}BDF)^{-.29}IA^{.7}RQ_{25}^{.60}$$

$$Q_{50} = 2.67A^{.32}Sl^{.15}(RI_2+3)^{1.74}(St+8)^{-.53}(13\text{-}BDF)^{-.28}IA^{.9}RQ_{50}^{.62}$$

$$Q_{100} = 2.50A^{.29}Sl^{.15}(RI_2+3)^{1.76}(St+8)^{-.52}(13\text{-}BDF)^{-.28}IA^{.6}RQ_{100}^{.63}$$

$$Q_{500} = 2.27A^{.29}Sl^{.16}(RI_2+3)^{1.86}(St+8)^{-.54}(13\text{-}BDF)^{-.27}IA^{.5}RQ_{500}^{.63}$$

The second set of equations uses only the three most significant variables:

$$Q_2 = 13.2\ A^{0.21}\ (13\text{-}BDF)^{-0.43}\ RQ_2^{\ 0.73}$$

$$Q_5 = 10.6\ A^{0.17}\ (13\text{-}BDF)^{-0.39}\ RQ_5^{\ 0.78}$$

$$Q_{10} = 9.51\ A^{0.16}\ (13\text{-}BDF)^{-0.36}\ RQ_{10}^{0.79}$$

$$Q_{25} = 8.68\ A^{0.15}\ (13\text{-}BDF)^{-0.34}\ RQ_{25}^{0.80}$$

$$Q_{50} = 8.04\ A^{0.15}\ (13\text{-}BDF)^{-0.32}\ RQ_{50}^{0.81}$$

$$Q_{100} = 7.70\ A^{0.15}\ (13\text{-}BDF)^{-0.32}\ RQ_{100}^{0.82}$$

$$Q_{500} = 7.47\ A^{0.16}\ (13\text{-}BDF)^{-0.30}\ RQ_{500}^{0.82}$$

Where Q2 = 2 year urban peak discharge (cfs), etc.

RI2 = 2 year, 2 hr. rainfall intensity (inches)

RQ2 = 2 year pead discharge for equivalent rural basin

Sl = Main channel slope in ft./mi. measured between points 10% and 85% the length of the main channel, upstream from the study site

St = Basin surface storage, i.e. % of basin area occupied by lakes, reservoirs, swamps, etc.

A = Contributing drainage area (sq. miles)

BDF = Basin discharge factor

IA = % imperviousness

The third set of estimating equations is similar to the first except that surface storage is replaced by lag time. The average standard error for the first set of equations ranged from 38% (plus or minus) for Q_2 to 49% (plus or minus) for Q_{500}. The average standard error for the second set ranged from 43% to 52% but they are easier and faster to apply than the first set.

Comment

The work of Sauer et al involved enormous effort. They identified the basin development factor (BDF) and equivalent rural discharges (RQ) as the most significant variables, and gave small weight to the effect of imperviousness.

To a large extent, BDF may be viewed as imperviousness with a high degree of connectivity. Storm sewers are usually constructed of impervious material, channels are frequently lined with impervious material. Therefore, downplaying the importance of imperviousness may be unjustified.

The errors inherent in the methods used for estimating RQ are carried over and further compounded by relating basin characteristics from sites across the whole country. The resulting equations are said to be applicable nationwide. Also the urban peak discharge (Q) is obtained by applying the Log Pearson III distribution without modification on basins that have undergone urbanization, contrary to the recommendations of the Water Resources Council.

2.6 Army Corps of Engineers (1990)

In the hydrology appendix of an unpublished document titled <u>General Design Memorandum, Flood protection feasibility, Lower Saddle River, Bergen County, New Jersey</u>, the Army Corps described a method for urban flood prediction that deviates from previous research work in this field.
This seemed to be the first time that an attempt was made to account for the effects of urbanization by focusing on one basin and performing a time based analysis rather than the traditional approach of aggregating any number of basins in a regional analysis with a fixed time frame.

For their analysis, the Army Corps used the Generalized Stream Network option of the HEC-1 flood hydrograph package, which is essentially a sophisticated rainfall run-off model. The HEC-1 model was calibrated by reproducing the November 1977, May 1968 and May 1989 floods at the three USGS gages on the Saddle River at Ridgewood, Lodi and the Hohokus Brook at Hohokus.
The Clark unit hydrograph parameter Tc (time of concentration) and R (the storage coefficient) were

determined using the results of a regression analysis involving 13 gaged basins at various degrees of urbanization. The analysis related basin physical parameters such as area, slope, length, etc. to Clark unit hydrograph Tc and R. However, to eliminate drainage area as a variable and to improve correlation, all data for the thirteen gaged basins was transformed to a standard 10 square mile unit. The regression equations obtained for the 10 square mile Tc and R are:

$$Tc_{10} = 0.46\ L_{10}^{2.0051}\ S^{-0.4160}\ Rtimp^{-0.1021}$$

$$R_{10} = 1.369\ L_{10}^{1.4202}\ S^{-0.4758}\ Rtimp^{-0.0657}$$

Where L = Length of main channel

S = Slope of main channel in ft./mi. between points 10% and 85% upstream from basin outlet

Rtimp = Percent impervious area

It should be noted that Rtimp and S require no transformation because they are dimensionless and independent of basin size.

Rtimp was obtained from a grid cell data bank developed with the HEC HYDPAR utility file program and supplemented by information on imperviousness for each subarea based on Stankowski's Report 38.

The transformation equations used were:

$$L_{10} = \{[10 \text{ square miles/D.A.}]^{0.5}\} * L$$

$$R_{10}(\text{or } Tc_{10}) = \{[10 \text{ square miles/D.A.}]^{0.25}\} * R(\text{or } Tc_{10}$$

Where D.A. is the drainage area of the subbasin in sq. miles The actual Tc and R for each subbasin are then obtained simply by inverting the above transformation equations.

Next, eight floods between September 1938 and May 1989 were selected and their associated precipitation amounts obtained by Thiessen networks. Each flood was reproduced using the calibrated HEC-1 model and inputting the appropriate values of precipitation as well as the time based values of Tc and R (from the previously described regression equations).

After thus reproducing the historic flood peaks, the updated peak discharges under present urbanized conditions were developed by using the same storm precipitations and loss parameter values in the HEC-1 model, but substituting the 1990 Tc and R for the appropriate value of impervious surface (Rtimp), where 1990 is assumed to represent present conditions.

Finally for each modeled storm, the ratio

$$\frac{\text{updated peak discharge}}{\text{observed peak discharge}}$$

was calculated and plotted against time in years since 1924. The plotted points are shown in fig. 2.5 and this equation was fitted to the plotted data points:

$$\text{Ratio} = 10^{0.185872-0.002893t}$$

where t = time in years since January 1924.

After applying the appropriate updating ratio to each observed annual peak flow, the Log Pearson III distribution was then applied in the usual way to calculate flood magnitudes at various recurrence intervals.

Comment

Though logical and detailed, the Army Corp's methodology requires sophisticated HEC modeling procedures, which expertise is not readily available to everybody. Secondly, the approach started out as a one basin study but eventually recourse had to be made to a regression analysis relating several other basins for the purpose of obtaining representative values for unit graph Tc and R. The pitfalls of regional regression analyses have previously been highlighted.

Thirdly, some of the imperviousness factor (Rtimp) was derived using Stankowski's equation which relates impervious surface to population density whereas, as has been previously discussed, Stankowski's population density equation does not always give satisfactory results and its use inevitably added to the error component in the Army Corp's technique.

Conclusions

The following general conclusions may be reached regarding the present state of the art in urban flood prediction:

1. Regional analysis methods such as Stankowski's or Sauer's have high standard errors which derive primarily from heterogeneities in the hydrologic, physiographic and climatologic characteristics of the basins used in the regional studies. However, for ungaged watersheds, application of regional relationships may well be the most viable option in predicting future flows.

2. The Army Corps single station technique eliminates some of the error sources mentioned above, but requires sophisticated modeling procedure which might limit its applicability among potential users.

Therefore, the need still exists for a methodology that is free of the errors of regional analyses, that does not require highly specialised skills, and that can be implemented using readily available data.

Filling that need is the central purpose of this research as will be described in the following sections.

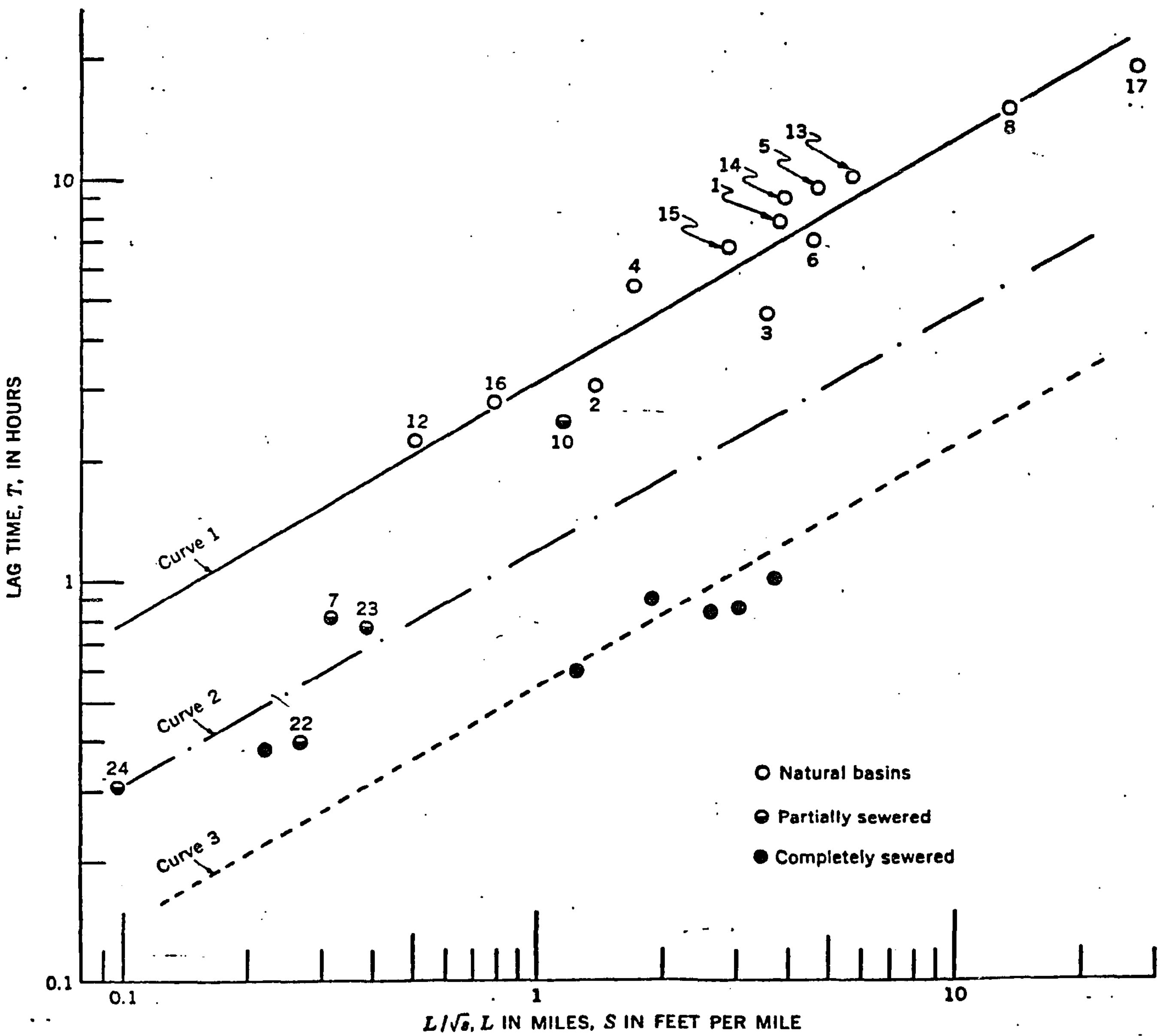

Figure 2.1 Effect of suburban development on lag time

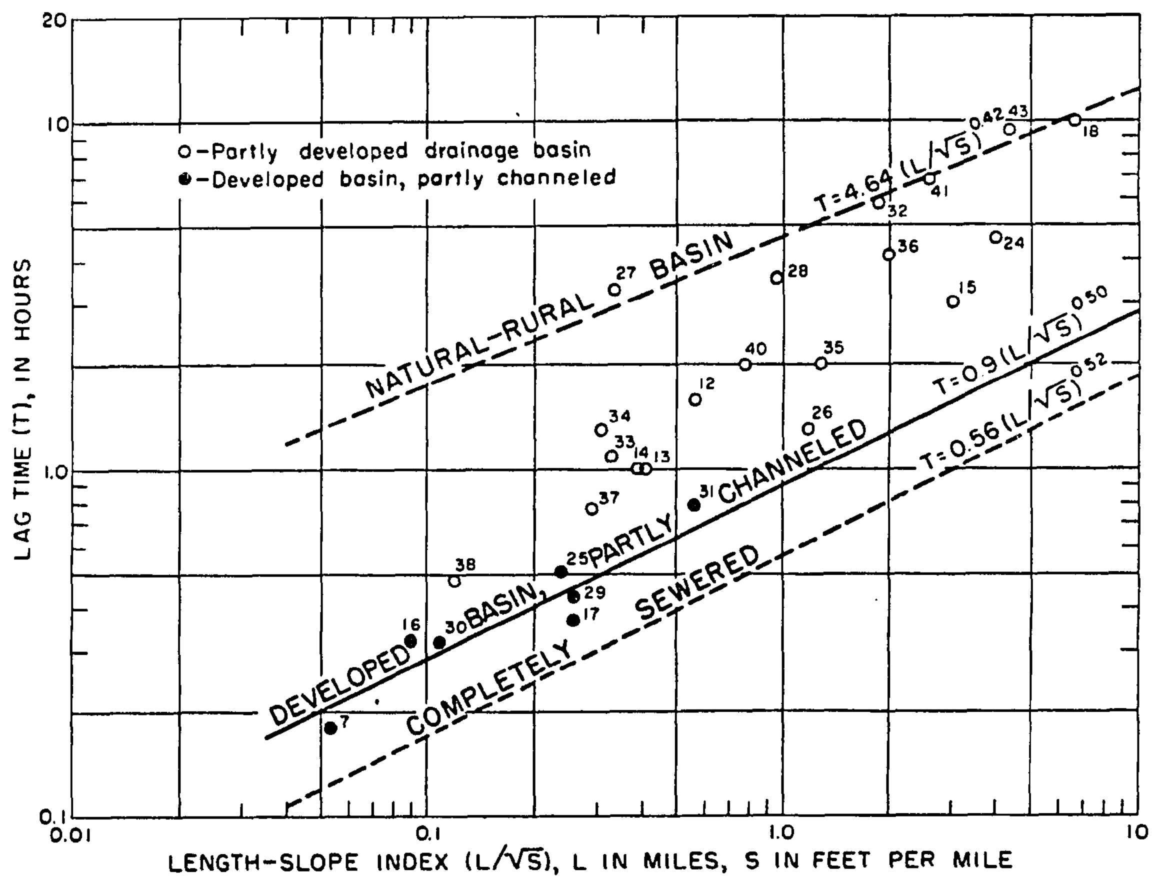

Figure 2.2 Lag time as a function of $(L/S^{0.5})$

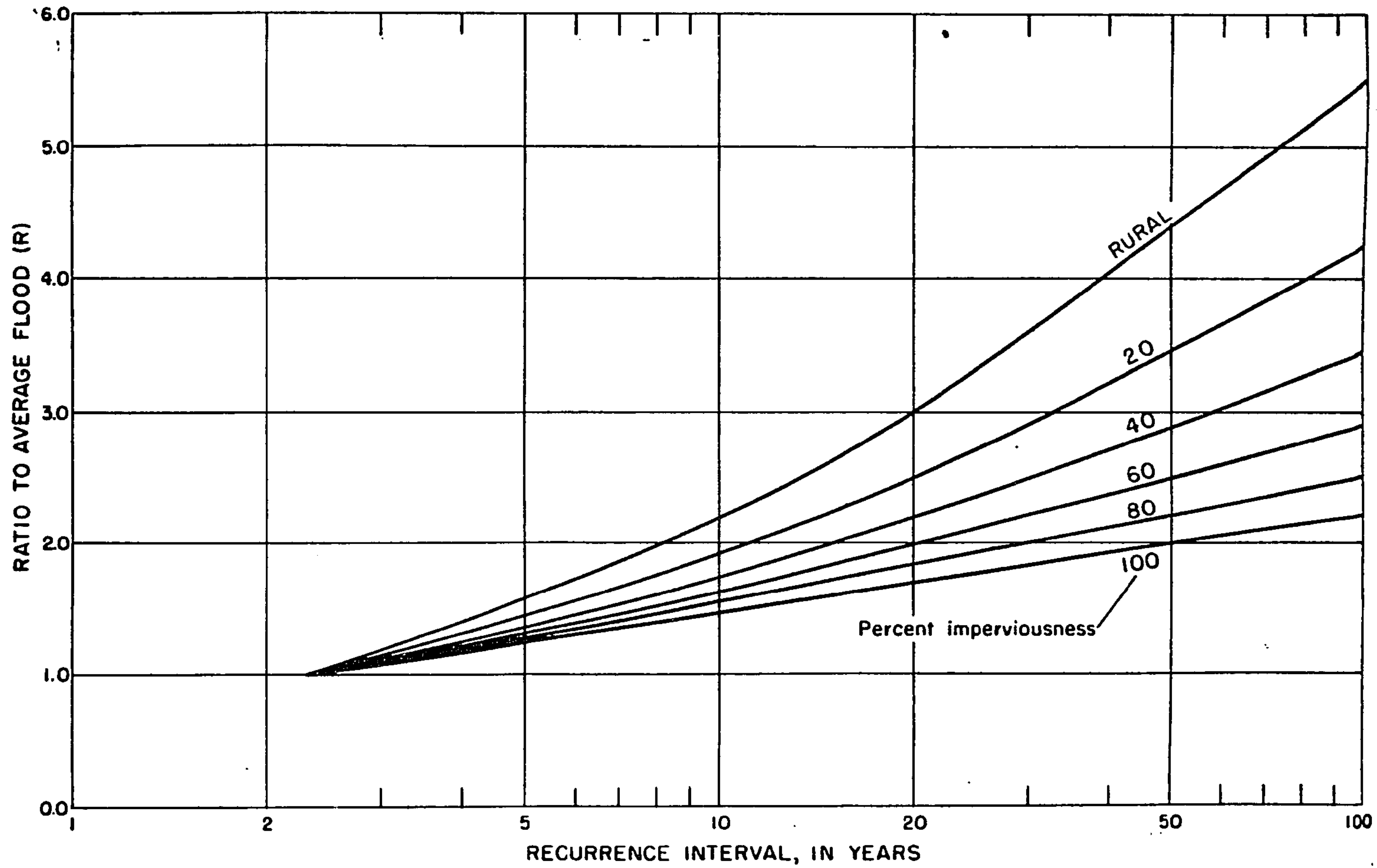

Figure 2.3 Flood frequency curves for selected degrees of imperviousnes

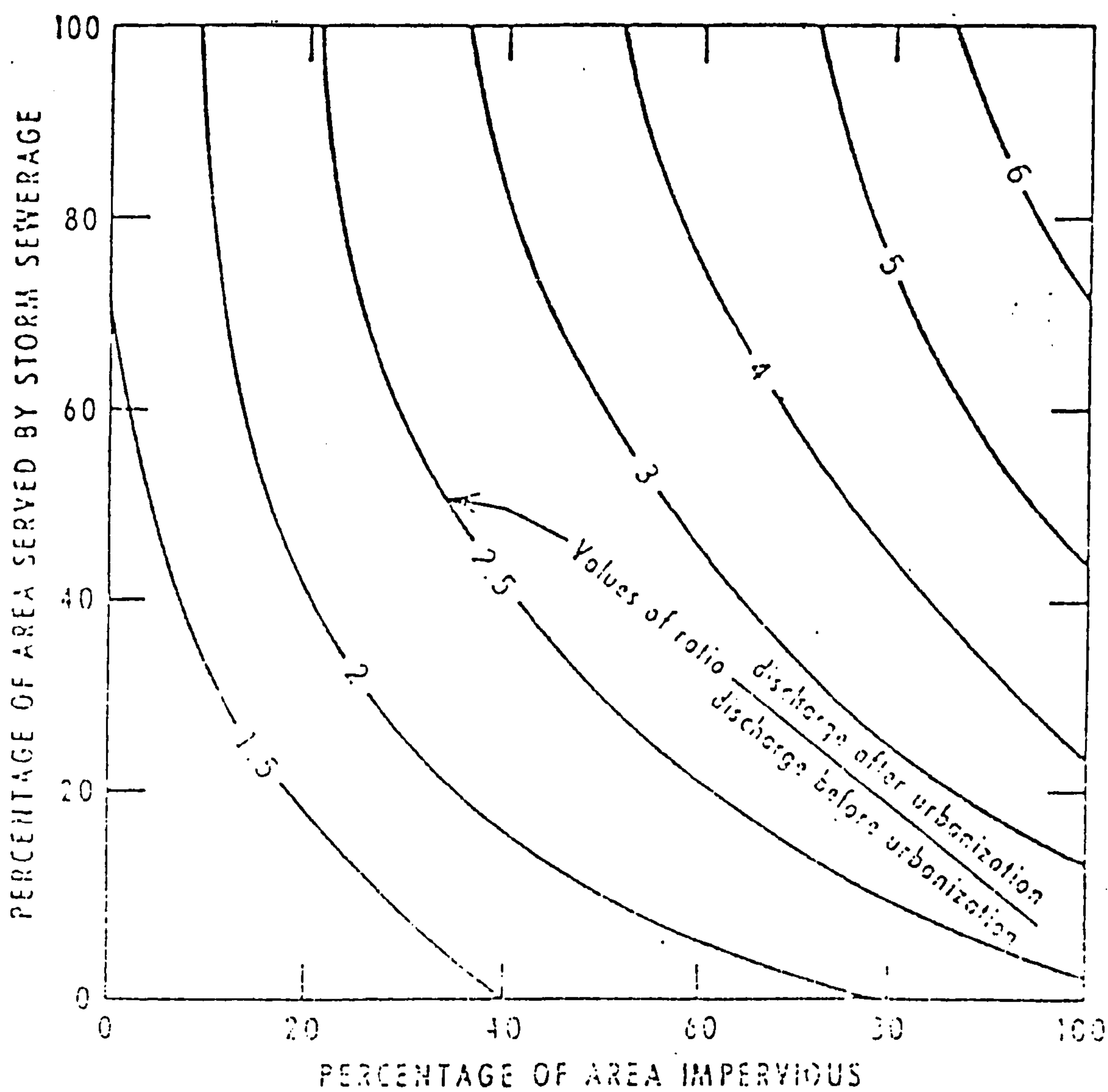

Figure 2.4 Effect of urbanization on mean annual flood for a 1-sq. mile drainage area

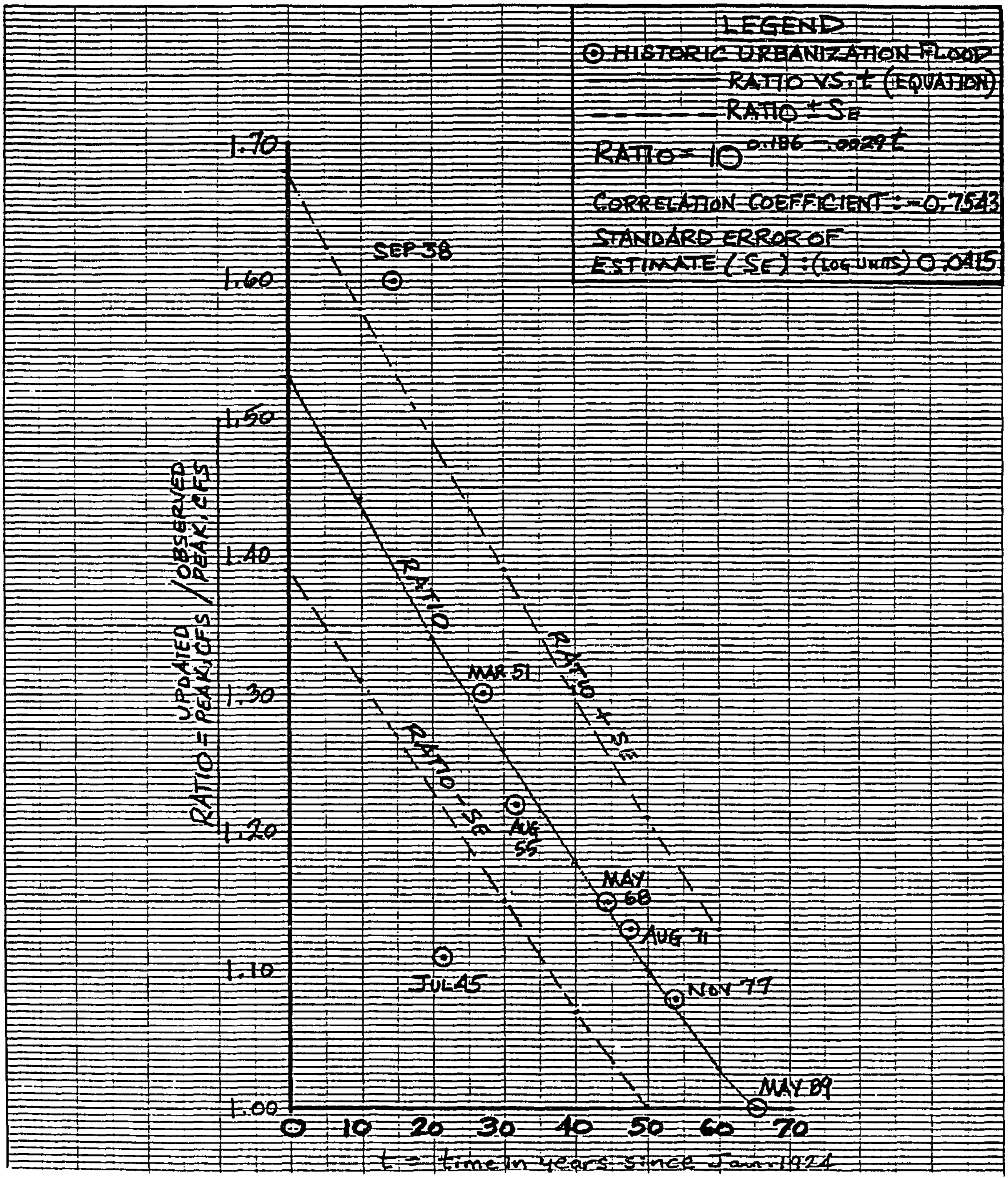

Figure 2.5 Update ratio versus time: Saddle River at Lodi,NJ

CHAPTER 3

STUDY AREA AND DATABASE DESCRIPTIONS

3.1 Study Area

The Saddle River basin is a hatchet shaped drainage area which lies within the eastern portion of the Passaic River basin. The stream gage of interest is the USGS gage number 01391500 located at Lodi in New Jersey at latitude $40^{o}53'25''$ and longitude $74^{o}04'51''$ The total drainage area upstream of the Lodi gage is 54.6 square miles. About 85% of this area lies in Bergen County, NJ while the remaining 15% is in Rockland County, NY. The Saddle River's headwaters originate in Rockland County and the river flows in a southerly direction to the Passaic River in Garfield and Wallington, NJ. Figure 3.1 is a map of the basin which also shows the approximate locations of the rainfall stations in and around the basin. Though it lies within the Passaic River basin, The Saddle River watershed is a hydrologically independent and distinct river basin.

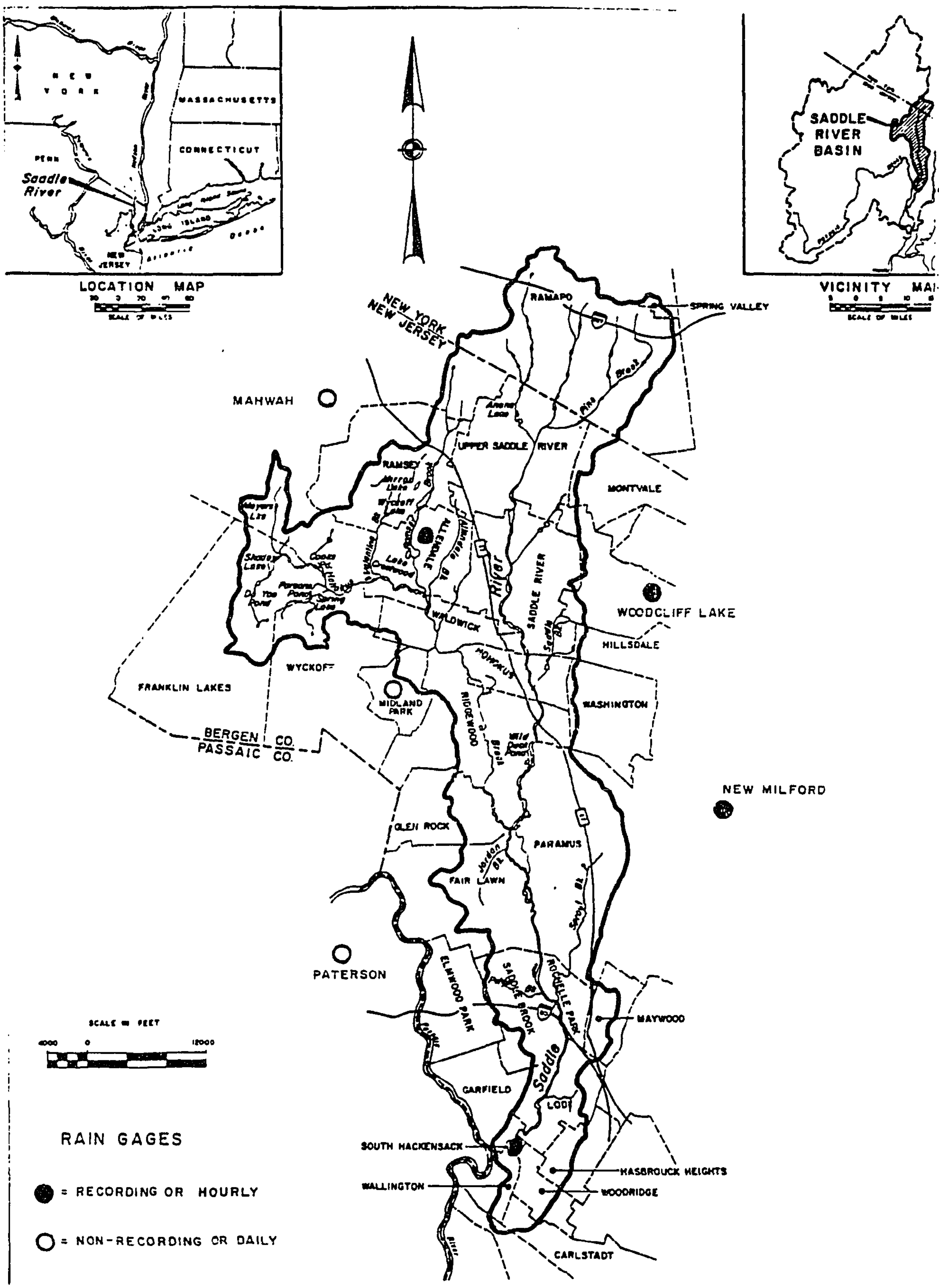

Figure 3.1 Map of the Saddle River Basin

Annual and Monthly Precipitation

From data compiled by the United States National Weather Service, the average annual precipitation in the Saddle River basin is estimated at 43.2 inches. The observed extreme annual rainfall amounts in the Passaic basin were 85.99 inches at Paterson, NJ in 1882 and 25.26 inches at Morristown, NJ in 1930.

The observed monthly extremes were 25.98 inches (September 1882) at Paterson and 0.02 inch (June 1949) at Plainfield and Jersey City, both also in New Jersey. Precipitation is distributed fairly uniformly throughout the year but on the average, rainfall amounts are greater during the summer months.

3.2 Database Description

One of the main advantages afforded by the single station approach is that the size of the required data base is greatly reduced. There is no need for topographic variables such as basin area, slope, and length, or physiographic descriptors such as soil type, vegetative cover, surface storage , etc. These characteristics of the basin may be considered invariant for most practical purposes and therefore do not have to be factored into the analysis. Rather, effort is focused on the hydrologic variables such as rainfall and streamflow, and indices of urbanization such as percent imperviousness.

3.2.1 Precipitation

Figure 3.1 shows the location of recording (i.e. hourly) and non recording (i.e. daily) rainfall stations within the vicinity of the Saddle River basin. Though reference was occasionally made to daily rainfall amounts recorded by daily gages such as Midland Park, and Mahwah, this study was based principally on hourly rainfall information as furnished by the recording gages at Woodcliff Lake, New Milford, Hackensack, Allendale and Little Falls. The model developed required reasonably accurate estimates of average rainfall intensity and maximum hourly rainfall amount and it is not possible to achieve this objective by using daily rainfall amounts with no idea of the time distribution. Precipitation averages were obtained by Thiessen Networks.

Data Sources

The United States Weather Bureau started operating hourly recording gages in 1939 and these records may be obtained in microfiche form from the National Climatic Data Center in Ashville, North Carolina. Alternatively, copies may be made at the Army Corps offices in Manhattan, New York, at Cornell University at Ithaca, New York or at Rutgers University, Busch campus library at Piscataway in New Jersey.

Unfortunately gaps in the hourly rainfall record occur frequently, diminishing the overall reliability of the data set. For instance, between 1940 and 1952, only two adjacent hourly recording stations, Woodcliff Lake and New Milford outside the eastern boundary of the Saddle River basin, were operational. Between 1953 and 1963, Allendale and Hackensack came online, greatly improving areal coverage of the basin. After 1963, the Hackensack gage was discontinued and Allendale stopped operating in 1973. In 1979, both Woodcliff Lake and New Milford were downgraded to daily stations, thereby essentially eliminating hourly rainfall coverage for the Saddle River Basin. Little Falls continued operation but this station is outside the basin and merely served to make estimations in conjunction with nearby daily stations. In view of the above difficulties, it was necessary to run multiple stepwise regressions on the assembled data set in order to provide an estimating relationship spanning the entire study period (1940-1990).

3.2.2 Streamflow

Published streamflow data usually indicates daily averages computed from USGS streamflow records. For this study however, it was required to use instantaneous hourly streamflow observations instead. In order to make a reasonably accurate estimate of the peak flow associated with a particular storm event, it is necessary to compare hourly rainfall information with matching hourly streamflow data, which clearly indicates antecedent base flow and actual instantaneous peak discharge. Depending on the shape of the flood hydrograph, the instantaneous maximum daily discharge may be substantially higher than the daily recorded minimum. Therefore averaging the daily maximum and minimum flows may lead to considerable underestimation of the actual peak response associated with a storm event.

Data Sources

The primary source of streamflow data is the United States Geologic Survey (USGS). For streams within the New Jersey area, the USGS office in West Trenton supplies the information. Hourly streamflow data for the earlier years (up to 1965 or so) is contained in files stored at the National Archives and access to these files requires an application to the USGS and a waiting period of about 2 weeks. Records exist for the Saddle River Lodi gage from 1964 onward, with the occasional gap in record but generally a reliable data set.

3.2.3 Percent Impervious Area

Impervious surface is believed to be primarily responsible for the effects on the flow regimen associated with the urbanization of a watershed. Storm drains or sewers and channel improvement works can actually be viewed as impervious surfaces with nearly 100% connectivity. Unfortunately, estimation of this variable is quite difficult. Not only are land use maps or aerial photographs scarce, especially for earlier years, taking the impervious surface measurements off these maps is a very tedious process. For this study, a couple of sets of aerial photographs, provided by Bergen County Planning Board, were used and supplemented by data from the Army Corps Study of the Saddle River Basin.

Data Sources

A major source of land use information is the USGS quadrangle maps set. However, these maps are drawn to a very small scale and are usually cluttered by topographic features. Also the maps are often unavailable for some years that are of interest. On the other hand, local county planning boards provide larger scale aerial photographs that depict land use exclusively. These are much easier to work with but again their availability is erratic, especially for the earlier years, and is dependent on the organizational ability and foresight of the individual planning boards.

CHAPTER FOUR

METHODOLOGY

4.1 Introduction

The objective of this dissertation is to develop a method for adjusting the historic record of annual peak floods to reflect changing land use conditions in a river basin.
The Log Pearson III distribution may then be applied to the modified discharges in the usual manner to predict flood magnitudes for various frequencies.

Development of the updating method referred to above is based on the concept that volume of run-off generated by a small storm after a period of drought is dependent on the land use condition within the basin.
Percent impervious area is the main parameter of interest, but storm sewerage, channel improvements and other basin development activities also increase run-off volume.
A small storm event occuring after a rainless period of time in a predominantly rural river basin produces very little run-off due to high infiltration rate. In other words, whatever run-off is generated by this type of small storm is largely contributed by the impervious portions of the basin.
As impervious surface percentage increases with time, the basin response to dry period small storms also increases.It should therefore be possible to characterize the basin by developing a basin response relationship to small storms

over time.This relationship forms the basis for calculation of run-off contributions resulting from impervious surface, starting from a selected base year and sequentially working up to the present.

The next step is an analysis of large storms in wet periods. Because soil moisture content is generally high in this situation, infiltration capacity is likely to be exceeded during large storms distributed over considerable time periods.In consequence, run-off is contributed by impervious as well as pervious surfaces, contrasting with the previous situation in which pervious ground contributes very little to total run-off.

Finally, composite adjustment ratios for annual recorded peak discharges are calculated, based on separation of pervious and impervious surface run-off contributions during large storms. Since run-off generated by small storms in dry spells is mainly due to impervious surface, the run-off contribution from impervious surface during large storms may be estimated by simple proportion.
Having obtained the impervious surface contribution,the remainder may be attributed to pervious surface.
The adjustment ratio for updating the impervious surface contribution is obtained from the time versus small storm response discussed earlier.

For the pervious surface contribution, the required ratio is

$$\frac{\%\text{ pervious surface in present year}}{\%\text{ pervious surface in year under consideration}}$$

Then, adding the two adjusted components yields the magnitude of run-off that might be expected if the storm and other metereological or hydrological conditions that caused the flood in question occured under present development conditions.

Implicit in the proposed method are the two assumptions of unit hydrograph theory that rainfall excesses of equal duration produce hydrographs with equivalent time bases, and that direct run-off ordinates for a storm of given duration are directly proportional to rainfall excess volumes. For the dry period small stroms, the principal generator of run-off is impervious surface. Therefore, it is only required to subtract a few hundredths of an inch to account for interception and detention storage. There is no need to consider infiltration losses. For the rainy period large storms, the focus in on the impervious surface run-off contribution and again infiltration is not a factor.
These concepts will be further clarified by sample calculations presented later.

4.2 Data Analysis

4.2.1 Small Storms

The Saddle River Basin upstream of the USGS gage at Lodi was selected for demonstration of the proposed methodology.

The study period is 1940 - 1990. 1940 was chosen as the starting year because hourly rainfall data became available from that date.

Data on small storms occuring within the basin was collected based on the following criteria:

1. The storm should in general occur during the dry months of May to November.
2. Preferrably, the storm should be preceded by 7 to 10 days without rainfall. Alternatively, hourly streamflow records were examined to determine that a preceeding storm had completely run off.

About 300 storms satisfying the above restrictions were analysed by the Thiessen method to obtain precipitation amount, duration, average intensity, and maximum hourly rainfall for the drainage basin.

Hourly streamflow data associated with the above storms were obtained from the USGS office in Trenton.

Finally, from the 300 available, 97 storms that satisfied the additional requirements below were selected for further study:

1. Hourly rainfall data should be available for each storm.
2. The rainfall should be reasonably steady and uniform basinwide.
3. Matching hourly or bi-hourly streamflow data should exist.
4. Precipitation amount should not exceed 1.0 inch.

The resulting data set is shown in Table C.1 in Appendix 3. The effective precipitation amount, maximum hourly precipitation and average intensity were multiplied by 100 in order to minimize scale effects during the regression analysis. Percent impervious surface was taken obtained from land use photographs of Bergen County, and supplemented by data from a study of the Saddle River Basin by the Army Corps of Engineers.

Multiple stepwise regressions were run on the data in Table 1, excluding percent impervious surface.

The dependent variable was the basin response (peakflow - baseflow), while the other variables were the predictors.

A second case where time was replaced by impervious surface while the other variables remained the same was then run.

Further, to assist in evaluation of results, the data in Table 1 was compressed to 63 observations between 1953 and 1973, this period representing the best data coverage for hourly rainfall within the Saddle River Basin, and the regressions repeated.

4.2.2 Large Storms

Data was collected on approximately 100 major storms meeting the following requirements:

1. The storm should occur during the rainy months, characterized by high soil moisture levels and high base flows.
2. Eligible large storms occuring during the summer months should be large enough to simulate the wet ground conditions required above.
3. The storms should be in excess of 1.0 inch.
4. The duration should in general not exceed 12 hours inorder to enable identification of a distinct peak discharge from hourly streamflow records.
5. Storms having significant snowfall components were excluded.
6. Storm should be reasonably steady and uniform.

Some of these storms are presented later in Table 5.4.

4.2.3 Impervious Surface

Using two sets of aerial photographs of the Saddle River Basin for 1970 and 1980, obtained from Bergen County Planning Board, and supplemented by data from the Army Corps' study of the Basin, percent impervious areas for the following years were established:

Year	Percent Impervious Area
1938	6.0
1945	8.2
1951	10.0
1955	11.2
1968	14.8
1971	16.5
1977	20.4
1990	28.1

This data plotted as a bilinear curve with 1968 as the separation point, indicating slower development up to 1968 and an accelerated development pace from that year onward. From the bilinear curve, the following relationships may be used to interpolate percent imperviousness for individual years:

Up to 1968: $I = (0.3 * X) + 6.0$

On from 1968: $I = (0.6 * X) - 3.1$

Where I = Percent imperviousness

X = Years since 1938

These relationships were used in obtaining the impervious areas in Table C.1 in Appendix 3.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 Results

5.1.1 Regression Analysis

Due to gaps in the hourly rainfall record as explained in Section 3.2.1, it was necessary to perform regression analyses in order to obtain relationships linking peak flow with rainfall and urbanization indices.

Using the SAS stepwise procedure at the 99% significance level, the following equation was obtained for the 97 observations data set (1940-80) with time as the urbanization index:

$$DP = 0.0555\ IT^{1.0106}\ BF^{0.545}\ TL^{0.7723}\ DL^{0.4303} \qquad (5.1)$$

Correlation coefficient = **0.8854**

Average prediction error = **15 cfs**

With the SAS MAXR procedure for the best 5-variable model, this relationship emerged:

$$DP = 0.0491\ IT^{0.6958}\ BF^{0.5617}\ TL^{0.7914}\ IH^{0.314}\ DL^{.3866} \qquad (5.2)$$

Where IT = Average effective intensity (in/hr)*100

BF = Baseflow (cfs)

TL = Number of years since 1939

DL = Effective rainfall duration (hours)

IH = Maximum hourly precipitation (in/hr)*100

DP = Peak - Baseflow (cfs)

Correlation coefficient = **0.8864**

Average prediction error = **15 cfs**

Table 5.1 compares the observed and predicted values of DP. When the regression was performed on the 63 most reliable(i.e. representing the best areal data coverage for hourly rainfall) observations, this result was obtained:

$$DP = 0.0031\ AT^{0.2508}\ BF^{0.4914}\ TL^{1.8606}\ IH^{0.6373} \qquad (5.3)$$

Where AT = Effective rainfall amount (in * 100)

Other variables as previously defined.

Correlation coefficient improves to **0.9547**

Average prediction error reduces to **4 cfs**

Table 5.2 compares observed and predicted values of DP for the 63 observations data set.

A second set of equations resulted when time was replaced by percent impervious surface in the regression data set.

For 98 observations between 1940 and 1980:

$$DP = 0.0028\ IT^{.5662}\ BF^{0.5414}\ IP^{2.1514}\ IH^{0.3630}\ DL^{.4078} \quad (5.4)$$

Where IP = Percent impervious surface

Other variables as previously defined.

Correlation coefficient = **0.9350**

Average prediction error = **6 cfs**

The observed and predicted values of DP are compared in Table 5.1.

Similarly for 63 observations (1953-1973):

$$DP = 0.0002\ AT^{0.8570}\ BF^{0.4032}\ IP^{3.4841}\ DL^{-0.4981} \quad (5.5)$$

Coefficient of correlation = **0.9608**

Average prediction error = **2 cfs**

Table 5.2 compares the observed and predicted values for this model.

From all the above results, the following may be inferred:

1. The 63 observations equations generally estimate more accurately than the 97 observations relationships.
This is because the period represented by the 63 observations had better data coverage as earlier explained.

2. The equations with percent impervious surface rather than time seem superior.
Impervious surface and basin response have a direct cause effect relationship, whereas time is indirectly related to peak flows through impervious surface and other drainage improvement practices that result from urban development.

While due regard was given to the above considerations, it was decided to adopt the time based relationship for 97 observations (1940-1980).
This decision was based on the following reasons:
1. Previous studies indicate that the rate of development within the Saddle River Basin was not uniform during the study period 1940-1990. Development was more rapid in the sixties and seventies . Therefore, although the equations with 63 observations served to demonstrate that the accuracy of the model depends on the reliability of the data, they are not sufficiently representative of the study period to warrant adoption.
2. Impervious surface is a very important variable in run-off generation. But storm sewerage, channel lining, etc improve drainage efficiency and hence increase run-off volume too.
Since it is quite difficult to quantify these other factors, using time to represent the sum total of all development effects seems to be the more practical alternative.

In summary, this regression equation will be used in further analysis as the best available estimating relationship across the entire period of study:

$$DP = 0.0555\ IT^{1.0106}\ BF^{0.5450}\ TL^{0.7723}\ DL^{0.4303} \qquad (5.1)$$

Before proceeding to work out update ratios, an effort was made to improve the prediction accuracy of the selected equation and reduce the average error or bias.
A numerical search procedure for optimizing non linear equations was employed for this purpose using the above regression coefficients as initial estimates.
After a number of iterations, there did not seem to be any net improvement in prediction accuracy. The effort was therefore discontinued.

TABLE 5.1 COMPARISON OF OBSERVED AND PREDICTED DP FOR 97 OBSERVATIONS (SEE TABLE C.1 FOR DATA ON THE ASSOCIATED STORMS)

Peak-Baseflow (Observed)	Peak - Baseflo (Predicted) with time	Peak-Baseflow (Predicted) with %imperv.
16	9	14
19	12	17
96	63	70
60	27	35
77	49	57
53	31	34
20	20	23
104	61	61
46	27	28
52	26	24
89	72	72
73	68	60
24	31	26
41	51	45
33	53	48
69	74	61
70	95	76
20	23	21
106	141	127
85	79	68
70	94	75
24	36	32
116	186	138
74	135	103
48	67	57
35	64	52
19	20	21
83	113	99
51	76	64
27	47	41
36	45	36
90	166	129
74	122	97
72	126	108
100	154	119
34	57	52
35	53	46
23	34	32

TABLE 5.1(CONTD) COMPARISON OF OBSERVED AND PREDICTED DP FOR 97 OBSERVATIONS

Peak - Baseflow (Observed)	Peak - Baseflo (Predicted) with time	Peak-Baseflow (Predicted) with %imperv.
100	105	87
75	64	60
51	100	82
21	27	24
60	59	50
57	62	63
116	70	102
89	71	94
53	62	67
99	122	108
67	102	89
23	41	43
70	75	69
42	54	49
138	150	134
120	155	136
66	69	63
140	128	109
32	28	29
121	130	118
150	230	199
40	57	59
133	76	81
87	66	68
47	45	49
66	65	70
106	110	103
67	67	65
52	45	42
67	73	69
177	223	201
250	364	321
130	141	133
200	137	139
142	74	75
168	134	132
186	212	214

TABLE 5.1(CONTD) COMPARISON OF OBSERVED AND PREDICTED DP FOR 97 OBSERVATIONS

Peak - Baseflow (Observed)	Peak - Baseflo (Predicted) with time	Peak-Baseflow (Predicted) with %imperv.
110	84	84
163	106	123
276	303	295
233	212	217
383	309	303
156	107	128
317	237	284
140	92	104
224	208	235
150	95	117
685	494	529
450	286	318
257	269	210
200	144	188
263	268	368
72	56	92
570	382	560
327	224	343
200	102	158
539	389	594
607	538	777
381	234	378

TABLE 5.2 COMPARISON OF OBSERVED AND PREDICTED DP FOR 63 OBSERVATIONS

Peak - Baseflow (Observed)	Peak - Baseflo (Predicted) with time	Peak-Baseflow (Predicted) with %imperv.
35	35	37
19	12	14
83	57	61
51	40	43
27	28	30
36	29	30
90	87	90
64	66	67
72	73	75
100	93	92
34	39	39
35	35	37
23	23	25
100	72	73
75	45	47
51	69	67
21	21	22
60	44	44
57	46	46
116	86	85
89	93	86
53	53	51
99	111	99
67	89	83
23	41	38
70	77	70
42	53	49
138	129	118
120	138	126
66	76	71
140	125	109
32	30	30
121	117	109
150	198	174
40	61	62
133	79	79
87	73	72
47	51	51

TABLE 5.2(CONTD) COMPARISON OF OBSERVED AND PREDICTED DP FOR 63 OBSERVATIONS

Peak - Baseflow (Observed)	Peak - Baseflo (Predicted) with time	Peak-Baseflow (Predicted) with %imperv.
66	68	67
106	118	111
67	83	79
52	60	55
67	84	81
177	223	203
250	342	205
130	142	134
200	190	178
142	94	88
168	150	147
186	221	214
110	101	101
163	122	122
276	332	337
233	240	246
383	393	400
156	134	156
317	271	293
140	143	157
224	245	289
150	137	166
685	590	680
450	366	429
257	234	296

5.1.2 Update Ratios_(Standard Method)

The regressions described in the previous section were intended to identify the most significant variables and the relative importance of each.

It may also be recalled that a time versus peak flow relationship is required for use in calculating the update ratios for large storms.

The selected estimating equation is:

$$DP = 0.0555\ IT^{1.0106}\ BF^{0.5450}\ TL^{0.7723}\ DL^{.4303} \quad (5.1)$$

This relationship will be used to compare the increases in peak flow (DP) with time (TL).

However, inorder to establish a common reference frame, it is necessary to standardize the values of the other significant variables:

1. BASE FLOW

As surface run-off increases with increasing impervious surface, infiltration decreases and ultimately, a decrease in base flow might be expected. In this case, the base flows recorded for the Saddle River Basin between 1940 and 1990 did not indicate any downward trend.

Rather, it fluctuated in much the same manner in 1990 as it did in 1940.

Perhaps this might be explained by the fact that groundwater flow is slower than surface flow by several orders of magnitude. Therefore it might take a period of time for reduced infiltration volumes to significantly impact on the groundwater reservoir, in the absence of other factors such as pumpage.

It may also be the case that the recharge areas for the aquifer systems within the Saddle River Basin have been relatively unaffected by urbanization.

In the absence of any significant trend in baseflow, simple statistical frequency was used to select a characteristic base flow. From a frequency analysis, 30 cfs was found to be the modal value of base flow, with 35 cfs as the next most frequent value.

It was therefore decided to adopt **30 cfs** as the characteristic base flow for comparative purposes.

2. RAINFALL AMOUNT

This variable is represented by intensity (IT) and duration in the estimating equation.

1 inch is considered a reasonable standard. Later in this report, it will be necessary to match large storm durations with small storm durations, and a small storm amount under 1 inch spread over, say 9 hours may not generate significant run-off under drought conditions.

Having fixed rainfall amount, duration may then be allowed to vary between 3 hours and 12 hours and the intensity determined for each duration by division.

With the variables base flow and intensity thus standardized, the true time versus peak flow relationship will then emerge, unclouded by variations in base flow or intensity.
In this form, the relationship may be applied in calculating the update ratios.

For 3 hours rainfall duration, the estimating relationship reduces to :

$$DP = 19.6453\ TL^{0.7723} \qquad (5.6)$$

Where as previously defined:

DP = Peak - Base flow

TL = Time in years since 1939

The required incremental ratio for impervious surface run-off contribution is now obtained by subtracting the estimated DP in the year under consideration from the DP in 1990 and dividing by the DP in the year in question.
For instance:

1942 $DP = (19.6453)(3)^{0.7723}$

$= 46$ cfs

1990 $DP = (19.6453)(51)^{0.7723}$

$= 409$ cfs

Incremental Ratio = (409 - 46)/ 46

= 7.8913

For 6 hours duration of rainfall, the estimating equation becomes:

$$DP = 13.1512\ TL^{0.7723} \qquad (5.7)$$

To obtain the required incremental ratio for, say **1950**, this expression is used:

$$\frac{(13.1512)(51)^{0.7723} - (13.1512)(11)^{0.7723}}{(13.1512)(11)^{0.7723}}$$

$$= 2.2619$$

Similarly for 9 hours duration, the estimating equation is:

$$DP = 10.3845\ TL^{0.7723} \qquad (5.8)$$

And again considering **1950** the incremental ratio is calculated to be: **2.2727**

Incremental ratios for other durations may be worked out in the same manner. The ratios for 3 hours, 6 hours, and 9 hours are compared in Table 5.3 See Appendix 1 for more sample calculations for incremental ratios. Also see

Table A.1 in Appendix 1 for incremental ratios for all durations between 3 hours and 9 hours.

While the actual values of DP are significantly influenced by the rainfall duration, it can be observed from Table 5.3 that the incremental ratios for each year do not differ much between the different durations.

Calculations for the final composite adjustment ratios are now illustrated below(also see Appendix 1 for detail explanations):

1945 Major storm = 2.19 in ---- 6 hours ---- 420 cfs

Small storm = 1.00 in ---- 6 hours ---- 52 cfs

Percent impervious surface in 1945 = 8.2 %

Percent impervious surface in 1990 = 28.1 %

From the above data,impervious surface contribution to major storm peak discharge in 1945 is:

(2.19 / 1.00) * 52

= 114 cfs

Pervious surface contribution:

420 - 114 = 306 cfs

For the 6 hour duration storm above, the incremental ratio for impervious surface contribution = **4.2692**(see Table 5.3).

Therefore updated impervious surface run-off contribution becomes:

$$(1 + 4.2692)(114) = 601 \text{ cfs}$$

And updated pervious surface contribution in 1990 is:

$$\frac{(100\% - 28.1\%)}{(100\% - 8.2\%)} * 306$$

$$= 240 \text{ cfs}$$

Total updated run-off:

$$240 + 601 = 841 \text{ cfs}$$

Update Ratio:

$$841/420 = 2.0016$$

Update Ratios for other years between 1942 and 1990 may be calculated using the steps outlined above. See Appendix 1 for more sample calculations.

The results are presented in Table 5.4

The criteria used in selecting large storms were described earlier in the data analysis section. The data shown in

Table 5.4 represent the range of high base flows generally associated with the rainy months.

Inorder to eliminate the minor fluctuations evident in Table 5.4, a smoothing exponential relationship was applied:

$$R = (\ 0.9224\ )(\ 10\ )^{0.0056\ T}$$

Where R = Update Ratio

T = Time in years from 1990 with 1940 as base.

Correlation coefficient = 0.9238

Standard error = 8% (+ or -)

The above exponential equation was obtained by fitting the raw update ratios to the following semi-log linear model:

$$\text{Log } R = A + (B * T)$$

Where R = Update Ratio

A = Log of intercept coefficient(i.e. take its anti log in order to transform to linear space)

B = Slope coefficient (in linear space)

T = Years from 1990

The smoothed update ratios are presented in Table 5.5, and the Log Pearson III distribution may now be applied on the modified historic peak flows to calculate the various flood frequencies and the results obtained compared with those of previous studies.

TABLE 5.3 PARTIAL INCREMENTAL RATIOS FOR IMPERVIOUS SURFACE RUN-OFF CONTRIBUTION

YEAR	3HOUR DURATION	6HOUR DURATION	9HOUR DURATION
1942	7.8913	7.8387	8.0000
43	6.1754	6.2105	6.2000
44	5.0147	4.9565	5.0000
45	4.2436	4.2692	4.2683
46	3.6477	3.6441	3.5957
47	3.1735	3.1515	3.1538
48	2.8224	2.8056	2.7895
49	2.5259	2.5128	2.5410
50	2.272	2.2619	2.2727
51	2.0522	2.0444	2.0423
52	1.88003	1.8842	1.8800
53	1.7086	1.7129	1.7000
54	1.5723	1.5849	1.5714
55	1.4491	1.4464	1.4545
56	1.3371	1.3419	1.3226
57	1.2350	1.2276	1.2268
58	1.1414	1.1406	1.1386
59	1.0553	1.0602	1.0571
60	0.9854	0.9885	0.9817
61	0.9112	0.9161	0.9115
62	0.8507	0.8514	0.8462
63	0.7860	0.7908	0.7851
64	0.7331	0.7342	0.7280
65	0.6831	0.6810	0.6744
66	0.6360	0.6310	0.6364
67	0.5853	0.5930	0.5882
68	0.5434	0.5480	0.5429
69	0.5037	0.5055	0.5000
70	0.4659	0.4652	0.4694
71	0.4301	0.4346	0.4305
72	0.4007	0.3980	0.3935
73	0.3679	0.3700	0.3671
74	0.3366	0.3366	0.3333
75	0.3067	0.3110	0.3091
76	0.2821	0.2804	0.2780
77	0.2546	0.2569	0.2558
78	0.2282	0.2287	0.2273
79	0.2065	0.2070	0.2067
80	0.1821	0.1861	0.1803

TABLE 5.3(CONTD.) PARTIAL INCREMENTAL RATIOS FOR IMPERVIOUS SURFACE RUN-OFF CONTRIBUTION

YEAR	3HOUR DURATION	6HOUR DURATION	9HOUR DURATION
81	0.1619	0.1610	0.1613
82	0.1393	0.1417	0.1368
83	0.1205	0.1230	0.1192
84	0.0995	0.1004	0.1020
85	0.0820	0.0830	0.0800
86	0.0651	0.0661	0.0640
87	0.0460	0.0498	0.0485
88	0.0302	0.0301	0.0286
89	0.0149	0.0148	0.0141
90	0.0000	0.0000	0.0000

TABLE 5.4 UPDATE RATIOS DATA SET

DATE	RAINFALL AMOUNT (IN)	EFFECTIVE DURATION (HOURS)	BASE FLOW (CFS)	PEAK-BASEFLOW (CFS)	UPDATE RATIO
03-03-42	1.40	9	52	223	2.0141
08-24-45	2.19	6	74	420	2.0016
04-25-45	1.30	6	84	231	2.0960
06-08-47	1.40	6	130	463	1.4589
04-05-47	2.20	6	140	850	1.3712
05-13-48	1.60	6	93	520	1.4582
04-01-48	1.45	5	156	540	1.4405
01-05-49	1.80	6	104	661	1.3705
03-19-51	1.61	7	104	474	1.4265
05-11-52	1.40	5	104	484	1.4325
05-25-52	2.15	6	93	767	1.3546
03-03-53	1.23	6	84	372	1.4394
01-24-53	1.20	6	141	384	1.4046
09-10-54	4.50	8	47	1225	1.4000
02-06-55	1.15	6	54	354	1.4066
04-04-57	2.10	6	141	654	1.3714
04-06-58	1.90	3	146	964	1.3160
03-06-59	2.00	5	61	739	1.3150
01-03-60	1.10	4	109	471	1.2983
04-16-61	1.40	3	183	687	1.2892
01-09-64	1.70	5	47	643	1.2516
02-13-66	1.75	5	68	670	1.2474
05-11-67	1.05	5	104	356	1.2434
04-24-68	2.00	8	40	560	1.2216
03-24-69	3.00	6	73	1472	1.0937

TABLE 5.4 (CONTD.) UPDATE RATIOS DATA SET

DATE	RAINFALL AMOUNT (IN)	EFFECTIVE DURATION (HOURS)	BASE FLOW (CFS)	PEAK-BASEFLOW (CFS)	UPDATE RATIO
03-19-71	1.20	4	93	497	1.1935
05-14-72	1.60	3	104	830	1.1668
07-13-72	2.20	4	123	1197	1.1111
02-03-72	1.20	4	55	475	1.2014
03-26-73	1.15	3	96	564	1.1757
03-21-74	1.45	3	86	714	1.1643
04-03-75	1.20	3	104	647	1.1322
04-01-76	2.60	5	118	1582	1.0836
03-04-77	1.79	5	69	931	1.0656
03-13-77	1.65	5	78	805	1.0770
01-08-78	1.91	4	104	1066	1.0703
05-14-78	1.65	5	98	902	1.0537
01-24-79	2.20	6	180	1370	1.0220
03-21-80	3.00	6	82	1758	1.0260
04-28-80	3.10	5	167	2203	1.0167
03-18-83	2.60	6	87	1343	1.0280
03-27-83	2.00	5	150	1650	1.0015
02-15-84	1.20	4	118	836	1.0191
09-26-85	4.00	6	76	2044	1.0202
03-30-87	2.20	6	80	1200	1.0111
04-03-87	2.80	9	135	2185	0.9949
05-18-88	1.80	6	74	819	1.0116
05-16-89	3.10	9	140	2240	0.9987

TABLE 5.5 UPDATED ANNUAL PEAK FLOWS

WATER YEAR	DATE	OBSERVED PEAK (CFS)	UPDATE RATIO	UPDATED PEAK (CFS)
1924	04-07-24	1280	1.7576	2250
25	02-12-25	980		1722
26	02-26-26	741		1302
27	09-02-27	1630		2865
28	07-07-28	829		1457
29	02-08-29	903		1587
30	04-08-30	418		735
31	04-24-31	549		965
32	03-29-32	686		1206
33	11-20-32	1320		2320
34	03-06-34	850		1494
35	10-01-34	614		1079
36	03-12-36	1720		3023
37	05-15-37	1060		1863
38	09-22-38	1680		2953
39	12-06-38	760		1336
40	03-15-40	1380	1.7576	2425
41	02-08-41	1030	1.7351	1787
42	08-10-42	820	1.7128	1404
43	12-31-42	1020	1.7128	1747
44	04-25-44	998	1.6692	1666
45	07-23-45	3500	1.6479	5768
46	05-28-46	1100	1.6267	1789
47	04-06-47	1010	1.6059	1622
48	11-09-47	830	1.6059	1333
49	12-31-48	1030	1.5853	1633
50	03-24-50	452	1.5450	698
51	03-31-51	2530	1.5252	3859
52	06-02-52	1740	1.5056	2620
53	03-14-53	1860	1.4863	2765
54	09-12-54	1270	1.4673	1863
55	08-19-55	2200	1.4485	3187
56	10-16-55	1530	1.4485	2216
57	11-02-56	795	1.4299	1137
58	02-28-58	1760	1.3935	2453
59	03-07-59	806	1.3757	1109
60	09-13-60	1190	1.3581	1616
61	02-26-61	952	1.3407	1276

TABLE 5.5 (CONTD.) UPDATED ANNUAL PEAK FLOWS

WATER YEAR	DATE	OBSERVED PEAK (CFS)	UPDATE RATIO	UPDATED PEAK (CFS)
1962	03-13-62	1670	1.3235	2210
63	03-07-63	824	1.3065	1077
64	01-10-64	702	1.2898	905
65	02-08-65	1490	1.2733	1897
66	09-22-66	1600	1.2570	2011
67	03-07-67	800	1.2409	993
68	05-29-68	3330	1.2250	4042
69	03-25-69	1540	1.2093	1862
70	04-03-70	2130	1.1938	2543
71	09-12-71	3770	1.1785	4443
72	06-19-72	2240	1.1634	2606
73	02-03-73	3210	1.1485	3687
74	12-21-73	2940	1.1485	3377
75	07-14-75	2720	1.1192	3044
76	07-01-76	2440	1.1049	2696
77	02-25-77	3130	1.0907	3414
78	11-09-77	4500	1.0907	4908
79	01-21-79	2890	1.0630	3072
80	04-10-80	2470	1.0493	2592
81	05-12-81	1900	1.0359	1968
82	01-04-82	1980	1.0226	2025
83	04-16-83	2550	1.0095	2574
84	04-05-84	3350	1.0000	3350
85	09-27-85	2120	1.0000	2120
86	01-26-86	1850	1.0000	1850
87	04-04-87	2320	1.0000	2320
88	10-28-87	1630	1.0000	1630
89	05-17-89	2380	1.0000	2380
90	05-17-90	2620	1.0000	2620

5.1.3 Update Ratios (Alternative Approach)

The updating method outlined in section 5.2. requires knowledge of percent impervious area within the basin for each year of the study period. Estimating impervious areas from aerial photographs and land use maps is a tedious and time consuming process, and may discourage would be users of the proposed method.

An alternative approach was therefore devised that greatly reduces the labour involved in calculating impervious areas.

This method requires knowledge of the impervious area for the present year only, rather than for every year of the period of interest as was the case in the previous calculations. Further, obtaining the present year impervious area does not present a major problem because most county or municipal planning boards have on record good approximations of present land use conditions.

The information thus collected from government establishments may be supplemented by a study of the most recent set of aerial photographs or land use maps available.

The proposed alternative method is illustrated using the data for 1945 in section 5.2. reproduced below:

1945 Major storm produces 2.19 inches in 6 hours and yields 420 cfs direct run-off

From equation 5.7, 1 inch small storm of duration 6 hours is estimated to yield 52 cfs

Percent impervious area in 1990 = 28.1 %

From Table 5.3, partial incremental ratio for impervious surface run-off contribution is **4.2692.** Therefore, actual ratio of impervious surface contribution in 1990 to impervious surface contribution in 1945 is:

1 + 4.2692 = **5.2692**

The assumption is made that this ratio is a fair approximation of the ratio of impervious areas between 1990 and 1945.

Therefore, **[^{1990}impervious area]/[^{1945}imp.area]**

= 5.2692

Given the impervious area in 1990(assumed to be the present year), the impervious area for 1945 may be estimated as:

28.1% / 5.2692 = 5.33%

The ratio of pervious areas in 1990 and 1945 becomes:

(100% - 28%)/(100% - 5.33%) = 0.7595

And equivalent pervious area run-off contribution in 1990 now is:

0.7595 * 306(see section 5.2.) = 232 cfs

Equivalent impervious area run-off contribution remains **601 cfs** (see section 5.2.).

Total updated run-off in **1990 = 232 + 601 = 833 cfs**

And update ratio = **833 / 420 = 1.9843**

This compares favourably with the update ratio of 2.0016 obtained using the previous method. A comparable value of update ratio has therefore been obtained without any knowledge of impervious area in 1945. See Appendix 1 for more examples and explanations regarding this approach.

Table 5.6 compares the previously calculated and estimated (using the alternative approach) values of update ratios. The two sets of values can be seen to agree closely. The estimated update ratios shown in Table 5.6 are smoothed values. See Appendix 1 for raw values of the estimated ratios as well as the smoothing exponential relationship.

The values of impervious (and hence pervious) areas for various years used in Table 5.6 were obtained as explained in Section 4.2.3.

TABLE 5.6 COMPARISON OF CALCULATED AND ESTIMATED UPDATE RATIOS

YEAR (X)	PERVIOUS(90) / PERVIOUS(X) CALCULATED	PERVIOUS(90) / PERVIOUS(X) ESTIMATED	UPDATE RATIO CALCULATED	UPDATE RATIO ESTIMATED
1942	0.7748	0.7422	1.7128	1.7003
43	0.7773	0.7482	1.6909	1.6793
44	0.7798	0.7546	1.6692	1.6585
45	0.7832	0.7595	1.6479	1.6380
46	0.7858	0.7653	1.6267	1.6178
47	0.7875	0.7712	1.6059	1.5978
48	0.7901	0.7762	1.5853	1.5781
49	0.7927	0.7815	1.5650	1.5586
50	0.7954	0.7868	1.5450	1.5393
51	0.7989	0.7919	1.5252	1.5203
52	0.8007	0.7966	1.5056	1.5015
53	0.8034	0.8021	1.4863	1.4829
54	0.8061	0.8071	1.4673	1.4646
55	0.8097	0.8123	1.4485	1.4465
56	0.8115	0.8170	1.4299	1.4286
57	0.8143	0.8227	1.4116	1.4110
58	0.8170	0.8276	1.3935	1.3935
59	0.8198	0.8329	1.3757	1.3763
60	0.8217	0.8377	1.3581	1.3593
61	0.8236	0.8445	1.3407	1.3425
62	0.8264	0.8477	1.3235	1.3259
63	0.8293	0.8528	1.3065	1.3096
64	0.8331	0.8580	1.2898	1.2934
65	0.8360	0.8633	1.2733	1.2774
66	0.8380	0.8684	1.2570	1.2616
67	0.8409	0.8736	1.2409	1.2460
68	0.8449	0.8784	1.2250	1.2306
69	0.8509	0.8840	1.2093	1.2154
70	0.8560	0.8896	1.1938	1.2004
71	0.8621	0.8946	1.1785	1.1856
72	0.8684	0.8996	1.1634	1.1709
73	0.8747	0.9049	1.1485	1.1564
74	0.8800	0.9104	1.1338	1.1421
75	0.8877	0.9158	1.1192	1.1280
76	0.8932	0.9211	1.1049	1.1141
77	0.9021	0.9267	1.0907	1.1003

TABLE 5.6(CONTD.) COMPARISON OF CALCULATED AND ESTIMATED UPDATE RATIOS

YEAR (X)	PERVIOUS(90 / PERVIOUS(X) CALCULATED	PERVIOUS(90 / PERVIOUS(90 ESTIMATED	UPDATE RATIO CALCULATED	UPDATE RATIO ESTIMATED
78	0.9078	0.9322	1.0768	1.0867
79	0.9159	0.9372	1.0630	1.0733
80	0.9218	0.9422	1.0493	1.0600
81	0.9301	0.9486	1.0359	1.0469
82	0.9362	0.9537	1.0226	1.0340
83	0.9436	0.9589	1.0095	1.0212
84	0.9523	0.9655	1.0000	1.0086
85	0.9587	0.9710	1.0000	1.0000
86	0.9664	0.9763	1.0000	1.0000
87	0.9756	0.9818	1.0000	1.0000
88	0.9849	0.9887	1.0000	1.0000
89	0.9917	0.9946	1.0000	1.0000
90	1.0000	1.0000	1.0000	1.0000

5.1.4 Flood Frequency Calculations

Applying the guidelines set out in United States Water Resources Council Bulletin #17, flood magnitudes for various return periods were computed using first the historic annual series, then the updated annual peak flows using various updating methods and the results compared with one another.

Historic Annual Peak Flows

Computed station skew coefficient = **-0.09355**

Generalised skew coefficient = **0.40000**

Weighted skew = 0.56*(-0.09355) + 0.44*0.40

= **0.1236**

Log Q = Mean(log peak) + K * STD(log peak)

Where Mean(log peak) = **3.162105**

STD(log peak) = **0.243142**

K is obtained from Appendix 3 of Bulletin 17 (excerpts are included in Appendix 6)

Using the above equation, the following values were obtained:

2 year flood -------- Q2 = **1436 cfs**

5 year flood -------- Q5 = **2318 cfs**

10 year flood -------- Q10 = **2997 cfs**

25 year flood -------- Q25 = **3962 cfs**

50 year flood -------- Q50 = **4758 cfs**

100 year flood -------- Q100 = **5620 cfs**

500 year flood -------- Q500 = **7915 cfs**

Updated Annual Peaks (Standard Method)

Computed station skew coefficient = **-0.117245**

Generalised skew coefficient = **0.400000**

Weighted skew = 0.56*(-0.117245) + 0.44 * 0.40

= **0.1103**

Mean(log peak) = **3.302889**

STD (log peak) = **0.195181**

Now using the same procedure as above, the following values were obtained:

2 year flood ----- Q2 = **1992 cfs**

5 year flood ----- Q5 = **2924 cfs**

10 year flood ----- Q10 = **3591 cfs**

25 year flood ----- Q25 = **4486 cfs**

50 year flood ----- Q50 = **5190 cfs**

100 year flood ----- Q100 = **5925 cfs**

500 year flood ----- Q500 = **7777 cfs**

Updated Annual Peaks (Alternative Method)

The update ratios given in tabel 5.7 were applied on the observed annual peaks shown in Table 5.6 and the updated peaks thus obtained used in the following calculations.

Computed Station Skew Coefficient = **- 0.112924**

Generalized Skew Coefficient = **0.400**

Weighted Skew = (0.56)*(-0.112924) + 0.44 * 0.40 = **0.1128**

Mean (log peak) = **3.302591**

STD (log peak) = **0.1963432**

Using: Log Q = Mean (log peak) + K * STD (log peak)

these values were obtained:

Q2 = 1990 cfs

Q5 = 2929 cfs

Q10 = 3601 cfs

Q25 = 4506 cfs

Q50 = 5219 cfs

Q100= 5964 cfs

Q500= 7846 cfs

Table 5.7

Comparison of Predicted Flood Magnitudes

Using Standard and Alternative Updating Methods

Flood Freq	Standard Method cfs	Alternative Method cfs
Q2	1992	1990
Q5	2924	2929
Q10	3591	3601
Q25	4486	4506
Q50	5190	5219
Q100	5925	5964
Q500	7777	7846

Again, there is good agreement between the two sets of values.

Stankowski's Method

Contributing Basin Drainage Area = **54.60 sq miles**

Main channel slope = **16.60 ft/mile**

Surface storage index = **5 %**

Impervious surface (present) = **28.1 %**

Impervious surface (rural) = **1.00 %**

$Q2(urban) = 25.6\ (54.6)^{0.89}\ (16.6)^{0.25}\ (5)^{-0.56}\ (28.1)^{0.25}$

= **1699 cfs**

$Q2(rural) = 25.6\ (54.6)^{0.89}\ (16.6)^{0.25}\ (5)^{-0.56}\ (1.0)^{0.25}$

= **738 cfs**

$Q5(urban) = 39.7\ (54.6)^{0.88}\ (16.6)^{0.26}\ (5)^{-0.54}\ (28.1)^{0.22}$

= **2432 cfs**

Q5(rural) = **1168 cfs**

$Q10(urban) = 54.0\ (54.6)^{0.88}\ (16.6)^{0.27}\ (5)^{-0.53}\ (28.1)^{0.20}$

= **3235 cfs**

Q10(rural)= **1660 cfs**

$Q25(urban) = 78.2\ (54.6)^{0.86}\ (16.6)^{0.27}\ (5)^{-0.52}\ (28.1)^{0.18}$

= **4111 cfs**

Q25(rural)= **2255 cfs**

$Q50(urban) = 104\ (54.6)^{0.85}\ (16.6)^{0.26}\ (5)^{-0.51}\ (28.1)^{0.16}$

= **4855 cfs**

Q50(rural)= **2847 cfs**

$Q100(urban) = 136\ (54.6)^{0.84}\ (16.6)^{0.26}\ (5)^{-0.51}\ (28.1)^{0.14}$

=**5706 cfs**

Q100(rural)=**3577 cfs**

Sauer's Method

BDF (urban) = **10** (assumed)

BDF (rural) = **0**

RQ is equivalent to Q(rural) from Stankowski's equations

$$Q2(urban) = 13.2\ (54.6)^{0.21}\ (13-10)^{-0.43}\ (738)^{0.73}$$

$$= \mathbf{2365\ cfs}$$

$$Q2(rural) = 13.2\ (54.6)^{0.21}\ (13-0)^{-0.43}\ (738)^{0.73}$$

$$= \mathbf{1259\ cfs}$$

$$Q5(urban) = 10.6\ (54.6)^{0.17}\ (13-10)^{-0.39}\ (1168)^{0.78}$$

$$= \mathbf{3366\ cfs}$$

$$Q5(rural) = \mathbf{1900\ cfs}$$

$$Q10(urban) = 9.51\ (54.6)^{0.16}\ (13-10)^{-0.36}\ (1660)^{0.79}$$

$$= \mathbf{4249\ cfs}$$

$$Q10(rural) = \mathbf{2506\ cfs}$$

$$Q25(urban) = 8.68\ (54.6)^{0.15}\ (13-10)^{-0.34}\ (2255)^{0.80}$$

$$= \mathbf{5241\ cfs}$$

$$Q25(rural) = \mathbf{3183\ cfs}$$

$$Q50(urban) = 8.04\ (54.6)^{0.15}\ (13-10)^{-0.32}\ (2847)^{0.81}$$

$$= \mathbf{6475\ cfs}$$

$$Q50(rural) = \mathbf{4050\ cfs}$$

$$Q100(urban) = 7.70\ (54.6)^{0.15}\ (13-10)^{-0.32}\ (3577)^{0.82}$$

$$= \mathbf{8096\ cfs}$$

$$Q100(rural) = \mathbf{5064\ cfs}$$

Army Corps Of Engineers' Method

Using the set of updated annual peaks obtained by HEC modeling, the ARMY CORPS obtained the following statistics:

Mean log = **3.2585**

STD log = **0.2174**

Weighted skew = **0.1000**

The flood magnitudes were then obtained using the procedures outlined above:

Q2 = **1800 cfs**

Q5 = **2750 cfs**

Q10 = **3500 cfs**

Q25 = **4500 cfs**

Q50 = **5370 cfs**

Q100 = **6280 cfs**

Q500 = **8730 cfs**

These results are compared in Tables 5.8 and 5.9.

TABLE 5.8 PREDICTED FLOODS FOR VARIOUS MAGNITUDES

RETURN PERIOD (YEARS)	OBSERVED ANNUALPEAK (CFS)	UPDATED PEAK (THESIS)	UPDATED PEAK (ARMY)
2	1436	1992	1800
5	2318	2924	2750
10	2997	3591	3500
25	3962	4486	4500
50	4758	5190	5370
100	5620	5925	6280
500	7915	7777	8730
STANDARD ERROR		8 %	10 %

TABLE 5.9 EFFECT OF URBANIZATION ON PREDICTED FLOODS

RETURN PERIOD (YEARS)	QUP/QOB (THESIS)	QUP/QOB (ARMY CORPS)
2	1.3871	1.2535
5	1.2614	1.1864
10	1.1982	1.1678
25	1.1323	1.1358
50	1.0908	1.1286
100	1.0543	1.1174
500	0.9826	1.1030

Where QOB refers to the historic record of annual peaks

QUP refers to the updated annual peak flows

The ratio Qurban / Qrural may be obtained for Stankowski's and Sauer's methods using the procedures outlined in Section 5.1.4. It was found that the value of this ratio ranged from 2.3 for the 2 year flood to 1.6 for the 100 year flood, in Stankowski's case. For Sauer's method, the range was 1.9 to 1.6.

Comment

From the foregoing calculations, the following observations may be made:

1. For regional adjustment methods as typified by Sauer's and Stankowski's works, the ratio

Q(urban)/Q(rural)

seems to vary between **1.5 and 2.5**, while single station analyses yield the range **1.0 - 1.5** for the ratio **Qup / Qob** (see page 84)

2. Urbanization affects the more frequent and generally smaller storms the most, while the larger storms with higher return periods are less susceptible.

3. The 500 year flood predicted from records of 100 years or less should be viewed with reservation and is usually omitted by most investigators.

5.2 DISCUSSION OF RESULTS

5.2.1 General

The methods described in this study basically apply the hydrologic expreriences of the Saddle River Basin in estimating the consequences of urbanization on streamflows within the basin. The equations developed pertain to the Saddle River Basin only and on this account are independent of topographic or physiographic variables such as size or shape of basin, slope, soil type and infiltration characteristics, vegetal cover, etc.

While the equations developed apply only to the Saddle River Basin, the method is applicable to any basin, regardless of location, size, shape or other characteristics. The only requirements are availability of hourly rainfall and streamflow records, an estimate of impervious area for the present year (at least) and sufficient level of development to significantly increase streamflows. It is expected that the equations will have the same general form, but different coefficients for different basins.

5.2.2 Model Format

A. Urbanizaton index

Several variants of the relationships linking increases in annual peak flows to precipitation and time or to precipitation and impervious area, were investigated. The equations with impervious area rather than time as the urbanization index generally seemed to yield higher update ratios. Also the model developed from a limited data set (1953 - 1973) generally led to higher update ratios. See Appendix B for update calculations using these equations.

The equations with impervious area require estimates of this variable for each year of the study period, a situation in which errors in the estimation process can cumulate and diminish the prediction accuracy of the equations.

Using a data set that is limited to a particular interval within the study period, say for reasons of improved data coverage or availability, is ill-advised because that interval may not be representative of the development rate through the entire period. In view of the above considerations, it is recommended that the time based model be adopted, developed from a data set extending through as long a period as possible.

Because time is only indirectly related to increases in streamflow through such urbanization indices as imperviousness, the correlation coefficient or standard error of the time based model may be slightly lower or higher (respectively) than for the equation with impervious area. However, the elimination of much of the labour and error of impervious surface estimation compensates adequately for the slight losses in accuracy.

B. Base flow

As recorded by the USGS gage at Lodi, the Saddle River did not indicate any significant upward or downward trends in dry weather base flows. The most frequently occurring dry weather base flow within the data set was therefore adopted as the characteristic base flow for use in the estimating equations. This situation may differ in other basins. If a trend in drought period base flow is detected, then it will be recessary to establish that trend and use baseflows corresponding to the trend equation for each year, rather than a fixed base flow for all year.

C. Rainfall amount

One inch was adopted as a standard amount. It was assumed that a rainfall event of greater than 1.0 inch may violate the requirement that most of the run-off be contributed by impervious areas. It is possible to use 0.5 inch or 0.75 inch instead of 1.0 inch. Such modifications do not substantially affect the results, as demonstrated in Appendix B. The adopted rainfall amount is merely a standard unit for comparison and as long as it remains constant, there will not be any appreciable influence on the indicated growth pattern of peak flows.

5.2.3 Range of Update Ratios Obtained

Using the adopted time based estimating equation, the maximum update ratio obtained was about 1.80. The assumed base year for the updating process was 1940 (from which year hourly rainfall data became available), and it was assumed the state of development within the basin prior to 1940 was not significant. Using the other miscellaneous models detailed in Appendix B, the maximum update ratio seemed to be in the neighborhood of 3. As explained in Appendix B, these other equations are flawed in a number of respects and the results produced by their application should be regarded with a lesser degree of confidence. Shown in Table 5.10 is a comparison of the theoretical update ratios and the incremental ratios of actual floods resulting from roughly equivalent storm events and antecedent conditions.

TABLE 5.10 COMPARISON OF ACTUAL INCREASES IN PEAK FLOWS WITH CALCULATED UPDATE RATIOS

DATE	AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW(CFS) DP	ACTUAL INCREASE IN DP	CALCULATD UPDATE RA -TIO(1990
08-24-45 03-13-77	2.19 1.65	6 5	74 78	420 805	1.9166	1.6479
05-13-48 05-14-78	1.60 1.65	6 5	93 98	520 902	1.7346	1.5853
01-05-49 01-08-78	1.80 1.91	6 4	104 104	661 1066	1.6127	1.5650
05-11-52 02-15-84	1.40 1.20	5 4	104 118	484 836	1.7273	1.5056
05-25-52 03-18-83	2.15 2.60	6 6	93 87	767 1343	1.7510	1.5056
09-10-54 09-26-85	4.50 4.00	8 6	47 76	1225 2044	1.6686	1.4673
04-06-58 03-27-83	1.90 2.00	3 5	146 150	964 1650	1.7116	1.3935
02-13-66 03-04-77	1.75 1.79	5 5	68 69	670 931	1.3793	1.2570
03-24-69 03-21-80	3.00 3.00	6 6	73 82	1472 1758	1.1943	1.2093
03-21-74 02-15-84	1.45 1.20	3 4	104 118	714 836	1.1710	1.1376

The calculated update rations are with reference to 1990 and the prevailing conditions are not always exactly equivalent, so the comparisons can only be approximate. However, the calculated update ratios can be seen to match actual incremental ratios reasonably closely.

CHAPTER 6

CONCLUSIONS AND SUGGESTIONS

6.1 Conclusions

Presented in this report is a method for dealing with the problems of urban flood frequency prediction that break with the traditional regional approach. It was shown how relationahips could be established that are tailored to the specific hydrologic experiences of a basin with rainfall, stream flow and land use information that is readily available from government agencies.

Given a reliable data set, it is possible to accurately estimate the effects of basin development on stream flows. Having quantified the urbanization influences on the flow regime, update ratios may then be calculated and applied to the historic record of annual peak discharges in order to obtain a homogenous data set of present year equivalent flows. Flood frequency calculations may then be made in the usual manner using the Log Pearson III distribution.

In the standard updating method proposed, it is required to know the ratio of pervious area in the present year (assumed to be 1990) to the pervious area in the year under consideration, for all years of the study period. To eliminate the tedium of estimating impervious surface from land use maps for so many years, an alternative approach was presented which approximates the above ratio of pervious areas as a function of the ratio of small storm basin response in the present year to the equivalent peak flow in the year in question. All that is now required is an estimate of the impervious area in the present year. This latter approach is more elegant and was shown to produce results that compare favorably with those of the previous method.

Sophisticated modeling techniques are not required for the proposed method but it is possible to refine the estimating relationships using HEC or other suitable rainfall runoff models.

6.2 Suggestions

A. Hydrology

Much can be achieved in hydrologic study and modeling if a reliable record of hourly rainfall and matching hourly streamflow data is available. Such a data record provides variables like maximum hourly rainfall, average effective intensity, effective rainfall amount, instantaneous peak discharge, etc. These are precise descriptors of the rainfall run-off mechanism and may be applied to build models of any desired form.

The results obtained in this study were somewhat restricted by gaps in the houly rainfall reacord for some key stations within the Saddle River Basin. Operations of these stations were delayed or discontinued at various times within the study period for unknown reasons, forcing increased reliance on subjective judgement in determining the true areal and temporal distribution of rainfall. Perhaps budgetary constraints should be held responsible, but it is strongly believed that a couple of hourly rainfall stations strategically located within the basin serves a far greater purpose than a large number of daily rainfall stations in the same vicinity.

Daily stations merely record the cumulated rainfall amount within a 24-hour period. It is difficult in the circumstances to make determinations regarding the continuity or actual intensity of the storm events.

Similarly, instantaneous hourly streamflow data give a more accurate picture of the basin response to a storm than the usual published daily mean flows.

B. Land Use

Traditionally, hydrologists have relied on regional flood frequency equations to account for the effects of urbanization on streamflows. Percentage imperviousness, main channel slope, basin area, etc were merely plugged into these regional equations to obtain predicted future floods. As has been earlier described, errors are inevitable when basins of significantly different hydrologic characteristics are lumped together in regional analyses.

This study calls for increased committment on the part of county planning boards or other river basin authorities in formulating relationships appropriate to the basin in question. For instance, starting from a convenient base year, percent impervious area within the basin can be estimated using land use maps, published records, field inspections, etc. Subsequently, information on development and imperviousness can be collected and collated by examining building permits, observing actual construction activity, studying property tax records, etc.

Once this data base is established , obtaining a fair estimate of impervious area for any year should no longer be the laborious task that it normally is. Using the steps outlined in this study, the historic record of annual peak flows may then be transformed to present day equivalents and the Log Pearson III technique applied in the usual manner to predict future floods. The update ratios may be revised every five years or so to keep pace with continuing development.

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APPENDIX 1
SAMPLE CALCULATIONS AND TABLES

Partial Incremental Ratios

This represents the increase in dry period small storm run-off between the year in consideration and the present year (assumed to be 1990). Since dry period run-off is mainly due to impervious surface, this ratio also represents the increase in impervious surface run-off contribution during large storms.

The adopted estimating equation is:

$$DP = 0.0555 IT^{1.0106}\ BF^{0.545}\ TL^{0.7723}\ DL^{0.4303} \qquad \text{(A-1)}$$

where DP = Peak - baseflow (cfs) (Reference to small dry period storms)

IT = Average effective intensity (in/hr)*100 #

BF = Baseflow (CFS)

TL = number of years since 1939.

DL = Effective duration (hours)

#Multiplication by 100 simply served to reduce scale effects during the regression analysis.As explained in previous sections, it was decided to normalize the values of DP by assuming 30 cfs and 1 inch as standard values for dry period baseflow and dry period, small storm rainfall amount respectively.

For a rainfall duration of 3 hours,

intensity (IT) = (1.0/3) * 100 = 33.3333 in/hr.

Plugging this value, 30 cfs for baseflow and 3 hours for duration into equation A-1, the following equation is obtained:

$$DP = 19.6453 * TL^{.7723} \qquad (A-2)$$

Suppose we now consider 1942. TL becomes 3 years and we have:

$$DP = 19.6453 * 3^{.7723} = 46 \text{ cfs}$$

For 1990, TL = 51 years. Therefore:

$$DP = 19.6453 * 51^{0.7723} = 409 \text{ cfs}$$

So, small storm run-off incremental ratio between 1942 and 1990 is calculated as:

(409-46)/46 =7.8913

For the following year (that is 1943), TL now is 4 years.

DP becomes:

$$DP = 19.6453 * 4^{0.7723} = 57 cfs$$

DP for 1990 remains constant at 409 cfs. Therefore, incremental ratio between 1943 and 1990 is :

$$(409 - 57)/57 = 6.1754$$

Incremental ratios for all other years up to 1990 may be similarly obtained.

Next consider a different rainfall duration, say 6 hours. Intensity now becomes:

$$(1.0/6) * 100 = 16.6667 \text{ in/hr.}$$

Again plugging this value in addition to 30 cfs for base flow and 6 hours for duration, into equation A-1, we obtain:

$$DP = 13.1512 * TL^{0.7723} \qquad (A-3)$$

Again considering 1942, TL = 3 years and

$$DP = 13.1512 * 3^{0.7723} = 31 cfs.$$

For 1990, TL = 51 years and

$$DP = 13.1512 * 51^{0.7723} = 274 cfs$$

Incremental ratio = (274-31)/31 = 7.8387

For 1943, TL = 4 years and plugging this into equation A-3, we obtain:

DP = 13.1512 /4 0.7723 = 38 cfs.

DP is constant at 274 cfs . Therefore incremental ratio:

(274-38)/38 = 6.2105

Incremental ratios for 6-hour rainfall duration may be similarly obtained for all other years up to 1990. Further for all other desired durations the incremental ratios can be calculated by plugging the duration, 30 cfs for baseflow and 100/(duration) for intensity into equation A-1, and then following the steps outlined above. Table A-1 presents incremental ratios for durations between 3 and 9 hours for all years between 1942 and 1990.

TABLE A.1 PARTIAL INCREMENTAL RATIOS FOR IMPERVIOUS SURFACE RUN-OFF CONTRIBUTION FOR VARIOUS DURATIONS

YEAR	3 HOUR	4 HOUR	5 HOUR	6 HOUR	7 HOUR	8 HOUR	9 HOUR
1942	7.8913	7.8718	7.9412	7.8387	7.9286	7.9231	8.0000
1943	6.1754	6.2083	6.0698	6.2105	6.1429	6.2500	6.2000
1944	5.0147	4.9655	4.9608	4.9565	4.9524	4.9487	5.0000
1945	4.2436	4.2424	4.2414	4.2692	4.2083	4.2727	4.2683
1946	3.6477	3.6133	3.6061	3.6441	3.6296	3.6400	3.5957
1947	3.1735	3.1687	3.1644	3.1515	3.1667	3.2182	3.1538
1948	2.8224	2.8022	2.8000	2.8056	2.7879	2.8033	2.7895
1949	2.5259	2.5306	2.5349	2.5128	2.5211	2.5152	2.5410
1950	2.2720	2.2642	2.2688	2.2619	2.2468	2.2676	2.2727
1951	2.0522	2.0619	2.0400	2.0444	2.0488	2.0526	2.0423
1952	1.8803	1.8595	1.8679	1.8842	1.8736	1.8642	1.8800
1953	1.7086	1.7031	1.7143	1.7129	1.7174	1.7294	1.7000
1954	1.5723	1.5630	1.5763	1.5849	1.5773	1.5778	1.5714
1955	1.4491	1.4539	1.4516	1.4464	1.4510	1.4421	1.4545
1956	1.3371	1.3378	1.3385	1.3419	1.3364	1.3434	1.3226
1957	1.2350	1.2323	1.2353	1.2276	1.2321	1.2308	1.2268
1958	1.1414	1.1358	1.1408	1.1406	1.1368	1.1481	1.1386
1959	1.0553	1.0595	1.0541	1.0602	1.0661	1.0714	1.0571
1960	0.9854	0.9771	0.9869	0.9855	0.9841	0.9829	0.9817
1961	0.9112	0.9116	0.9119	0.9161	0.9084	0.9174	0.9115
1962	0.8507	0.8503	0.8424	0.8514	0.8519	0.8560	0.8462
1963	0.7860	0.7835	0.7882	0.7908	0.7857	0.7984	0.7851
1964	0.7331	0.7300	0.7371	0.7342	0.7361	0.7313	0.7280

TABLE A.1(CONTD.) PARTIAL INCREMENTAL RATIOS FOR IMPERVIOUS SURFACE RUN-OFF CONTRIBUTION FOR VARIOUS DURATIONS

YEAR	3 HOUR	4 HOUR	5 HOUR	6 HOUR	7 HOUR	8 HOUR	9 HOUR
1965	0.6831	0.6796	0.6796	0.6810	0.6779	0.6812	0.6744
1966	0.6360	0.6321	0.6344	0.6310	0.6340	0.6338	0.6364
1967	0.5853	0.5872	0.5916	0.5930	0.5823	0.5890	0.5882
1968	0.5434	0.5446	0.5431	0.5480	0.5432	0.5467	0.5429
1969	0.5037	0.5043	0.5050	0.5055	0.5060	0.5065	0.5000
1970	0.4659	0.4661	0.4686	0.4652	0.4706	0.4684	0.4694
1971	0.4301	0.4298	0.4340	0.4346	0.4286	0.4321	0.4305
1972	0.4007	0.4008	0.4009	0.3980	0.3966	0.3976	0.3935
1973	0.3679	0.3676	0.3694	0.3700	0.3661	0.3728	0.3671
1974	0.3366	0.3359	0.3333	0.3366	0.3369	0.3410	0.3333
1975	0.3067	0.3057	0.3047	0.3110	0.3089	0.3107	0.3091
1976	0.2821	0.2815	0.2827	0.2804	0.2821	0.2818	0.2781
1977	0.2546	0.2536	0.2562	0.2569	0.2563	0.2541	0.2558
1978	0.2282	0.2270	0.2308	0.2287	0.2315	0.2340	0.2273
1979	0.2065	0.2056	0.2063	0.2070	0.2077	0.2083	0.2067
1980	0.1821	0.1809	0.1829	0.1861	0.1848	0.1837	0.1803
1981	0.1619	0.1611	0.1603	0.1610	0.1628	0.1658	0.1613
1982	0.1393	0.1382	0.1386	0.1417	0.1416	0.1429	0.1368
1983	0.1205	0.1197	0.1218	0.1230	0.1211	0.1208	0.1192
1984	0.0995	0.1019	0.1014	0.1004	0.1013	0.1048	0.1020
1985	0.0820	0.0813	0.0819	0.0830	0.0823	0.0841	0.0800
1986	0.0651	0.0646	0.0629	0.0661	0.0638	0.0691	0.0640
1987	0.0460	0.0453	0.0483	0.0498	0.0460	0.0498	0.0485
1988	0.0302	0.0298	0.0305	0.0301	0.0288	0.0311	0.0286
1989	0.0149	0.0147	0.0133	0.0148	0.0121	0.0175	0.0141
1990	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Composite Update Ratios

A. Standard Method

Example 1 Suppose it is given that a major rainy period storm event in 1945 produced 2.19 in of rainfall in 6 hours and generated a direct run-off of 420 cfs. Suppose it is also given that the impervious area within the basin in 1945 is about 8.2% while in 1990 that figure increased to 28.1% It is required to estimate what factor to apply to 420 cfs in order to obtain the equivalent run-off that might be generated if this 2.19 in. storm occured in 1990.

Considering equation A-3, TL = 6 years for 1945.

Therefore: $DP = 13.1512 * 6^{0.7723} = 52$ cfs.

This implies that a dry period small storm of 1 in. amount and 6 hour duration in 1945 generates approximately 52 cfs of direct run-off (mostly from impervious area). Therefore impervious area contribution to the major storm direct run-off of 420cfs may be estimated by simple proportion as:

$$(2.19/1.00) * 52 = 114\text{cfs.}$$

Pervious ground contribution is obtained by difference:

$$420-114 = 306 \text{ cfs.}$$

From Table A-1, the partial incremental ratio (for impervious surface run-off contribution) for a 6-hour duration storm in 1945 is 4.2692. Therefore, equivalent impervious surface run-off contribution in 1990 is:

$$(1 + 4.2692) * 114 = 601 \text{ cfs.}$$

Equivalent pervious surface contribution in 1990 is calculated as:

$$(1990 \text{ pervious area}/1945 \text{ pervious area}) * 306 =$$

$$[(100\%-28.1\%)/(100\%-8.2\%)] * 306 = 240 \text{ cfs.}$$

Total updated run-off in 1990:

$$240 + 601 = 841 \text{ cfs.}$$

Therefore, update ratio for 1945:

$$841/420 = 2.0016.$$

Example 2 Suppose a major rainy season storm in 1958 produced 1.90 inches in 3 hours and generated a direct run-off of 964 cfs. It is known that the impervious area in 1958 is approximately 12% and increased to 28.1% in 1990.

Since the major storm duration in this case is 3 hours, we refer to equation A-2. TL = 19years(that is 1939 to 1958). Therefore:

$$DP = 19.6453 * 19^{0.7723} = 191 \text{ cfs.}$$

This is the estimated value of run-off generated by a dry period small storm of 1 in amount and 3-hour duration in 1958. Since this run-off is mostly produced by impervious area, the impervious area contribution to the major storm run-off of 964 cfs may be estimated by proportion:

$$(1.90/1.00) * 191 = 363 \text{ cfs.}$$

Pervious gound contribution becomes :

$$964-363 = 601 \text{ cfs.}$$

From Table A-1, partial incremental ratio for a 3-hour storm in 1958 is 1.1414. Therefore equivalent impervious surface run-off contribution in 1990 is:

$$(1 + 1.1414) / 363 = 777\text{cfs.}$$

Equivalent pervious ground contribution in 1990 is :

[(100 %-28.1%) / (100%-12%)] * 601 = 491 cfs.

Total equivalent run-off in 1990:

777 + 491 = 1268 cfs.

Update ratio = 1268/964 = 1.3160.

Update ratios were similarly obtained for about 50 large storms between 1942 and 1990 and presented in Table 5.4 Table 5.5 contains smoothed update ratios in column 4.

B. Alternative method

In the foregoing update caculations, it is required to estimate the ratio of pervious area in 1990 to pervious area in the year in consideration in order to update the pervious area run-off contribution. This calls for estimation of impervious area (and hence pervious area) for each year of the study period, a very tedious process. In the proposed alternative method, the assumption is made that ratio of equivalent small storm run-off volumes in 1990 and the year in consideration is a good approximation of the ratio of impervious areas between 1990 and the year in question.

For instance, using example 1, the ratio:

$$\frac{\text{Equivalent small storm run-off in 1990}}{\text{Small storm run-off in 1945}} = \frac{601}{114} = 5.2719$$

This value is the same as adding unity to the incremental ratio in Table 5.4 for six hour duration in 1945, allowing for round-off errors.Therefore,

$$\frac{\text{1990 impervious area}}{\text{1945 impervious area}} = 5.2719 \text{ (approx)}$$

Given that impervious area in 1990 is 28.1%, the impervious area is estimated to be:

$$\frac{28.1000}{5.2719} = 5.33\%$$

The ratio $\frac{\text{1990pervious area}}{\text{1945pervious area}}$ becomes:

$$[(100\%-28.1\%)/(100\%-5.33\%)] = 0.7595$$

Refering to example 1, the updated pervious surface run-off contribution is:

$$0.7595 * 306 = 232 \text{ cfs.}$$

Total updated run-off = 232 + 601 = 833 cfs.

Update ratio = 833/420 = 1.9843.

The difference from the previously obtained value is :

$$2.0016 - 1.9843 = 0.0173$$

which is close enough for practical purposes.

Similarly, considering example 2 the assumption is made

that: $\frac{\text{1990impervious area}}{\text{1958impervious area}} = (1 + 1.1414) = 2.1414$

So, impervious area in 1958 = 28.1% / 2.1414 = 13.1% (approx

The ratio $\frac{\text{1990pervious area}}{\text{1958pervious area}}$ is now estimated as:

$$[100\%-28\%]/[100-13.1\%] = 0.8276$$

Updated pervious surface contribution becomes:

$$0.8276 * 601 = 497 \text{ cfs.}$$

Total updated run-off = 777 + 497 = 1274 cfs.

Therefore the update ratio = 1274/964 = 1.3220.

The difference from previously obtained update ratio is:

$$1.220 - 1.3160 = 0.006$$

which is well within acceptable limits. Table A2 gives the raw update ratios. This equation was applied to smooth the values:

$$R = 0.9361\ (10)^{0.0054T}$$

Where T is number of years since 1990

R is the update ratio.

The correlation coefficient for this equation is 0.9397 and the standard error is + or - 7%.
See Section 5.1.2, page 66 for suggestions on how to obtain the above smoothing relationship.

It should be noted that from table A1 that the partial incremental ratios for each year do not differ much between different durations. Therefore, no matter what rainfall duration is being considered for any year, the estimated ratio of pervious areas using the above alternative approach remains nearly the same.

TABLE A.2 UPDATE DATA SET FOR ALTERNATIVE UPDATING METHOD

DATE	PRECIP AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW (CFS)	UPDATE RATIO CALCULTD	UPDATE RATIO SMOOTHED
03-03-42	1.40	9	52	223	2.0012	1.7003
08-24-45	2.19	6	74	420	1.9836	1.6380
04-25-45	1.30	6	84	231	2.0883	
04-05-47	1.40	6	130	463	1.4422	1.5978
05-13-48	1.60	6	93	520	1.4462	1.5781
04-01-48	1.45	5	156	540	1.4295	
03-19-51	1.61	7	104	474	1.4215	1.5203
05-11-52	1.40	5	104	484	1.4298	1.5018
03-03-53	1.23	6	84	372	1.4390	1.4829
01-24-53	1.20	6	141	384	1.4491	
09-10-54	4.50	8	47	1225	1.3912	1.4646
02-06-55	1.15	6	54	354	1.4068	1.4465
04-04-57	2.10	6	141	654	1.3770	1.4110
04-06-58	1.90	3	146	964	1.3225	1.3935
03-06-59	2.00	5	61	739	1.3229	1.3763
01-03-60	1.10	4	109	471	1.3069	1.3593
04-16-61	1.40	3	183	687	1.3016	1.3425
01-09-64	1.70	5	47	643	1.2629	1.2934
02-13-66	1.75	5	68	670	1.2395	1.2616
05-11-67	1.05	5	104	356	1.2797	1.2460
04-24-68	2.00	8	40	560	1.2372	1.2306
03-19-71	1.20	4	93	497	1.2070	1.1856
05-14-72	1.60	3	104	830	1.1807	1.1709
02-03-72	1.20	4	55	475	1.2132	
03-26-73	1.15	3	96	564	1.1873	1.1564

TABLE A.2(CONTD.) UPDATE DATA SET FOR ALTERNATIVE METHOD

DATE	PRECIP AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW (CFS)	UPDATE RATIO CALCULTD	UPDATE RATIO SMOOTHED
03-21-74	1.45	3	104	714	1.1754	1.1421
04-03-75	1.20	3	104	647	1.1434	1.1280
06-01-76	2.6	5	118	1582	1.0611	1.1141
03-04-77	1.79	5	69	931	1.0789	1.1003
03-13-77	1.65	5	78	805	1.0889	
01-08-78	1.91	4	104	1066	1.0821	1.0867
05-14-78	1.65	5	98	902	1.0665	
01-24-79	2.20	6	180	1370	1.0355	1.0733
03-21-80	3.00	6	82	1758	1.0384	1.0600
04-28-80	3.1	5	167	2203	1.0297	
02-18-83	2.60	6	87	1343	1.0364	1.0212
03-27-83	2.00	5	150	1650	1.0126	
02-15-84	1.20	4	118	836	1.0264	1.0068
09-26-85	4.00	6	76	2044	1.0264	1.0000
03-30-87	2.20	6	80	1200	1.0143	1.0000
04-03-87	2.80	9	135	2185	0.9997	
05-18-88	1.80	6	74	819	1.0129	1.0000
05-16-89	3.10	9	140	2240	1.0003	1.0000

APPENDIX 2
SENSITIVITY CACULATIONS

A. Other rainfall amounts

The small storms data set shown in Table 1.1 contains rainfall amounts generally limited to 1.0 inch. It was assumed that even in drought periods small strom amounts greater than 1 inch might generate significant volumes of run-off from pervious ground, in addition to impervious areas. This contradicts the requirement that run-off be contributed almost solely by impervious areas. In caculating update ratios, it was first necessary to estimate the relative increase in DP (i.e., small storm peak flow - base discharge) over the years, and to make this comparison, rainfall amount was arbitrarily fixed at 1 inch. As long as it stays fixed, other rainfall amounts may be substituted, such as 0.5 or 0.75 inch without appreciably affecting the final update ratios. For instance, suppose 0.5 inch is chosen, instead of 1.0 inch. Again, considering example 1 in Appendix 1:

rainfall intensity = 0.5 / 6hr.* 100 = 8.333 inch/hr.

See Section 4.2.1, page 46 for explanations regarding the multiplication by 100.

Substituting this value as well as 30 cfs for base flow and 6 hrs. for duration into equation A1, the following results:

$DP = 6.5215 * TL^{0.7723}$In 1945 TL is 6 years which yields DP = 26 cfs.

In 1990 TL is 51 which produces DP = 136 cfs.

Therefore, partial incremental ratio for small storm basin response is:

$$(136 - 36) / 26 = 4.2308.$$

Impervious surface run-off contribution :

$$2.19/0.50 * 26 = 114 \text{ cfs.}$$

Pervious ground contribution = 420 - 114 = 306 cfs.

Total updated run-off :

$$(1+4.2308)(114) + [71.9\%/91.8\%] * 306 = 836 \text{ cfs.}$$

Update ratio = 836/420 = 1.9904.

This update ratio is almost the same as the 2.0016 obtained using 1.0 inch rainfallamount.

B. Other Models

In chapter 5 a number of other estimating equations were presented. These other equations were not as satisfactory as the adopted equation due to the reasons discussed in chapter 5 but for comparative purposes, two were aplied in calculating update ratios.

Time based squation for 63 values between 1953 and 1973

Equation 5.3 in chapter 5 was obtained using the SAS stepwise procedure at the 99% confidence level, but does not contain the duration variable. So, the SAS MAXR procedure for best 4 variable model was substituted:

$$DP = 0.0043\ AT^{0.8866}\ BF^{0.4328}\ TL^{1.8268}\ DL^{-0.5646} \qquad (B\text{-}1)$$

where AT = effective rainfall amount (inches x 100) and the other variables as previously described.

This equation was developed with data between 1953 and 1973, the period which had the most reliable hourly rainfall data coverage for the basin. However, in order to update annual peak flows to 1990, it was necessary to extrapolate equation B-1 through the study period 1940-1990. This resulted in under estimation of DP during the earlier years and overestimation for the latter years, and ultimately to higher values for update ratios as illustrated below:

Again, consider example 1 from appendix 1. This time, for 1.0 inch small storm, base flow of 30 cfs and 6 hour duration, equation B-1 yields:

$$DP_{1945} = 11 \text{ cfs (cf 52 cfs previously obtained)}$$

Similarly, small storm run-off incremental ratio is calculated to be 48.864.

Following the same procedures as in example 1,

Impervious surface run-off contribution:

$$2.19/1.0*11 = 24 \text{ cfs}$$

$$\text{Pervious contribution} = 420 - 24 = 396 \text{ cfs}$$

Total updated run-off:

$$(24)(1+48.864)+[71.9\%/91.8\%]*396 = 1507\text{cfs}$$

Update ratio = 1507/420 = 3.5877

The update ratios obtained in this manner, using the same data set as in Table 5.4 are presented in Table B-1. The update ratios were smoothed using the exponential equation:

$$R = 1.0529\ (10)^{0.0098T}$$

Where R = update ratio

T = years from 1990

As can be seen, the equation converges to 1.0529 rather than 1.0.

TABLE B.1 UPDATE RATIOS DATA SET USING EQUATION 5.3

DATE	PRECIP AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW (CFS)	UPDATE RATIO CALCULTD	UPDATE RATIO SMOOTHD
03-03-42	1.40	9	52	223	5.5131	3.1102
08-24-45	2.19	6	74	420	3.5877	2.9594
04-25-45	1.30	6	84	231	3.7568	
04-05-47	1.40	6	130	463	2.3366	2.8288
05-13-48	1.60	6	93	520	2.3368	2.7657
04-01-48	1.45	5	156	540	2.3228	
03-19-51	1.61	7	104	474	2.3657	2.5852
05-11-52	1.40	5	104	484	2.4187	2.5276
03-03-53	1.23	6	84	372	2.4098	2.4712
01-24-53	1.20	6	141	384	2.3351	
09-10-54	4.50	8	47	1225	2.3118	2.4160
02-06-55	1.15	6	54	354	2.3786	2.3621
04-04-57	2.10	6	141	654	2.3088	2.2579
04-06-58	1.90	3	146	964	2.1637	2.2075
03-06-59	2.00	5	61	739	2.1841	2.1582
01-03-60	1.10	4	109	471	2.1255	2.1101
04-16-61	1.40	3	183	687	2.1427	2.0630
01-09-64	1.70	5	47	643	2.0371	1.9284
02-13-66	1.75	5	68	670	1.9790	1.8433
05-11-67	1.05	5	104	356	2.0916	1.8022
04-24-68	2.00	8	40	560	1.9706	1.7620
03-19-71	1.20	4	93	497	1.8839	1.6466
05-14-72	1.60	3	104	830	1.7900	1.6099
02-03-72	1.20	4	55	475	1.8946	
03-26-73	1.15	3	96	564	1.8088	1.5740

TABLE B.1(CONTD.) UPDATE RATIOS DATA SET USING EQUATION 5.3

DATE	PRECIP AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW (CFS)	UPDATE RATIO CALCULTD	UPDATE RATIO SMOOTHED
03-21-74	1.45	3	104	714	1.7716	1.5389
04-03-75	1.20	3	104	647	1.6601	1.5045
06-01-76	2.6	5	118	1582	1.3811	1.4441
03-04-77	1.79	5	69	931	1.4399	1.4385
03-13-77	1.65	5	78	805	1.4745	
01-08-78	1.91	4	104	1066	1.4400	1.4064
05-14-78	1.65	5	98	902	1.3873	
01-24-79	2.20	6	180	1370	1.2675	1.3750
03-21-80	3.00	6	82	1758	1.2677	1.3443
04-28-80	3.1	5	167	2203	1.2389	
02-18-83	2.60	6	87	1343	1.2315	1.2563
03-27-83	2.00	5	150	1650	1.1434	
02-15-84	1.20	4	118	836	1.1843	1.2283
09-26-85	4.00	6	76	2044	1.1732	1.2009
03-30-87	2.20	6	80	1200	1.0987	1.1479
04-03-87	2.80	9	135	2185	0.9997	
05-16-89	3.10	9	140	2240	1.0170	1.0769

Impervious area based equation for 98 values (1940-1980)

When time was replaced by impervious area for the 98 observations in the data set between 1940 and 1980, equation 5.4 (see chapter 5) was obtained:

$$DP = 0.0028\ IT^{.5662}\ BF^{.5414}\ IP^{2.1514}\ IH^{.3630}\ DL^{.4078}$$

where IP = percent impervious area

IH = maximum hourly precipitation (in/hr) x 100

This model was extended to 1990 and used to calculate update ratios. In applying this equation, steady rainfall was assumed, implying that average intensity (IT) approximately equals maximum hourly rainfall (IH).

Following the steps outlined in appendix A, it was found that the calculated incremental ratios were again much higher than could be justfied by examining actual dry period discharges within the study period. For instance, considering example 1 in appendix A , DP for 1 in. small storm in 1945 of 6 hour duration , and base flow 30 cfs is calculated to be 46 cfs, while the incremental ratio becomes 13.1480.

So, the impervious run-off contribution is:

$$2.19/1.0*46 = 101 \text{ cfs}$$

Pervious surface run-off contribution is:

$$420 - 101 = 319 \text{ cfs}$$

Total updated runoff:

$$(1+13.148)(101)+[71.9\%/91.8\%] * 319 = 1679 \text{ cfs}$$

Update ratio = 1679/420 = 3.9976.

This value is nearly double that obtained using the equivalent time based equation and, is not borne out by comparison with observed stream flows.

Theoretically, the time based equation should yield higher update ratios because time represents a summation of all urbanization effects on run-off, of which imperviousness is only one.

Equation 5.4 assigns a high coefficient of 2.1514 to the impervious area variable. Therefore, errors in estimation of impervious area through the years may have been blown up and resulted in the reverse situation obtained above.

The update ratios obtained using equation 5-4 are shown in Table B-2. The ratios were smoothed using the equation:

$$R = 1.135\ (10)^{0.0095T}$$

Where R = update ratio

T = years from 1990

The closure here is also not good.

_TABLE B.2 UPDATE RATIOS DATA SET USING EQUATION 5.4

DATE	PRECIP AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW (CFS)	UPDATE RATIO CALCULTD	UPDATE RATIO SMOOTHD
03-03-42	1.40	9	52	223	3.9111	3.2434
08-24-45	2.19	6	74	420	3.9977	3.0374
04-25-45	1.30	6	84	231	4.2553	
04-05-47	1.40	6	130	463	2.6521	2.9073
05-13-48	1.60	6	93	520	2.6780	2.8444
04-01-48	1.45	5	156	540	2.5877	
03-19-51	1.61	7	104	474	2.6662	2.6638
05-11-52	1.40	5	104	484	2.6776	2.6061
03-03-53	1.23	6	84	372	2.7608	2.5497
01-24-53	1.20	6	141	384	2.6613	
09-10-54	4.50	8	47	1225	2.6630	2.4946
02-06-55	1.15	6	54	354	2.7160	2.4406
04-04-57	2.10	6	141	654	2.6498	2.3361
04-06-58	1.90	3	146	964	2.4304	2.2856
03-06-59	2.00	5	61	739	2.5009	2.2361
01-03-60	1.10	4	109	471	2.4408	2.1877
04-16-61	1.40	3	183	687	2.4505	2.1404
01-09-64	1.70	5	47	643	2.4040	2.0045
02-13-66	1.75	5	68	670	2.3595	1.9187
05-11-67	1.05	5	104	356	2.5421	1.8771
04-24-68	2.00	8	40	560	2.4246	1.8365
03-19-71	1.20	4	93	497	2.2725	1.7199
05-14-72	1.60	3	104	830	2.1321	1.6827
02-03-72	1.20	4	55	475	2.2909	
03-26-73	1.15	3	96	564	2.1641	1.6463

TABLE B.2(CONTD.) UPDATE RATIOS DATA SET USING EQUATION 5.4

DATE	PRECIP AMOUNT (IN)	DURATI -ON (HOUR)	BASE FLOW (CFS)	PEAK-BASE FLOW (CFS)	UPDATE RATIO CALCULTD	UPDATE RATIO SMOOTHED
03-21-74	1.45	3	104	714	2.1211	1.6106
04-03-75	1.20	3	104	647	1.9637	1.5758
06-01-76	2.6	5	118	1582	1.5944	1.5417
03-04-77	1.79	5	69	931	1.6662	1.5083
03-13-77	1.65	5	78	805	1.7170	
01-08-78	1.91	4	104	1066	1.6691	1.4757
05-14-78	1.65	5	98	902	1.5982	
01-24-79	2.20	6	180	1370	1.4262	1.4438
03-21-80	3.00	6	82	1758	1.4309	1.4125
04-28-80	3.1	5	167	2203	1.3838	
02-18-83	2.60	6	87	1343	1.3744	1.3228
03-27-83	2.00	5	150	1650	1.2399	
02-15-84	1.20	4	118	836	1.2908	1.2942
09-26-85	4.00	6	76	2044	1.2844	1.2626
04-03-87	2.80	9	135	2185	1.0798	1.2120
05-16-89	3.10	9	140	2240	1.0305	1.1601

APPENDIX 3

TABLE C.1 SMALL STORMS DATA SET

DATE	EFFECTIVE PRECIP IN * 100	MAX HOUR PRECIP IN/HR*100	EFFECTIVE DURATION HOURS	AVERAGE INTENSITY IN/HR*100	BASEFLOW CFS	PEAK - BASEFLOW CFS	YEARS SINCE 1939	IMPERVIOUS SURFACE %
10-10-41	48	16	4	12	13	16	2	7.0
06-01-42	28	10	4	7	39	19	3	7.2
07-17-42	100	84	2	69	19	96	3	7.2
09-09-42	42	18	3	14	58	60	3	7.2
10-17-42	78	29	3	26	54	77	3	7.2
10-20-44	84	22	6	14	21	53	5	7.8
11-10-44	33	18	3	11	25	20	5	7.8
07-02-45	46	32	2	23	52	104	6	8.2
10-23-45	24	11	3	8	58	46	6	8.2
04-25-46	30	6	5	6	52	52	7	8.5
08-07-46	72	28	4	18	52	89	7	8.5
09-21-46	60	36	2	30	31	73	7	8.5
10-08-48	36	13	3	12	21	24	9	9.0
11-19-48	45	29	2	21	25	41	9	9.0
07-06-49	52	24	3	17	25	33	10	9.3
08-29-49	93	39	3	31	15	69	10	9.3

TABLE C.1 (CONTD.) SMALL STORMS DATA SET

DATE	EFFECTIVE PRECIP IN * 100	MAX HOUR PRECIP IN/HR*100	EFFECTIVE DURATION HOURS	AVERAGE INTENSITY IN/HR*100	BASEFLOW CFS	PEAK - BASEFLOW CFS	YEARS SINCE 1939	IMPERVIOUS SURFACE %
08-03-50	84	23	4	21	34	70	11	9.6
10-23-50	24	8	4	6	25	20	11	9.6
11-04-50	105	59	3	35	34	106	11	9.6
11-20-50	50	34	2	25	30	85	11	9.6
07-19-51	52	28	2	26	34	70	12	10.0
09-15-51	20	12	2	10	34	24	12	10.0
09-27-51	63	63	1	63	40	116	12	10.0
10-24-51	46	46	1	46	40	74	12	10.0
10-02-52	32	19	2	16	40	48	13	10.2
08-09-53	34	18	2	17	30	35	14	10.5
10-25-53	18	10	3	6	18	19	14	10.5
10-28-53	81	39	3	27	26	83	14	10.5
11-23-53	40	24	2	20	30	51	14	10.5
06-13-54	20	11	2	10	41	27	15	10.8
07-25-54	20	20	1	20	18	36	15	10.8
08-03-54	123	46	3	41	22	90	15	10.8
10-29-54	48	25	2	24	47	74	15	10.8
06-20-55	52	31	2	26	39	72	16	11.2
07-06-55	53	53	1	53	26	100	16	11.2
08-06-56	24	15	2	12	35	34	17	11.4
08-21-56	42	7	6	7	35	35	17	11.4
10-23-56	25	6	5	5	33	23	17	11.4

TABLE C.1 (CONTD.) SMALL STORMS DATA SET

DATE	EFFECTIVE PRECIP IN * 100	MAX HOUR PRECIP IN/HR*100	EFFECTIVE DURATION HOURS	AVERAGE INTENSITY IN/HR*100	BASEFLOW CFS	PEAK - BASEFLOW CFS	YEARS SINCE 1939	IMPERVIOUS SURFACE %
11-18-56	60	13	4	13	54	100	17	11.4
12-09-56	40	9	5	7	57	75	17	11.4
06-26-57	40	20	2	20	35	51	18	11.7
09-16-57	15	5	3	5	30	21	18	11.7
10-18-57	28	14	2	14	26	60	18	11.7
09-21-58	32	12	4	8	43	57	19	12.0
08-05-59	69	20	4	17	30	116	20	12.3
10-07-59	35	35	1	35	24	89	20	12.3
06-21-61	36	10	6	6	43	53	22	12.7
08-20-61	45	24	2	21	35	99	22	12.7
10-02-61	55	14	4	13	35	67	22	12.7
11-07-61	19	9	3	6	35	23	22	12.7
11-14-61	30	15	2	13	35	70	22	12.7
06-12-62	18	9	2	9	35	42	23	13.0
09-17-62	72	29	3	24	27	138	23	13.0
06-15-63	60	21	3	20	38	120	24	13.3
07-14-63	40	13	3	12	22	66	24	13.3
08-13-63	30	30	1	30	30	140	24	13.3
11-01-63	20	5	5	4	22	32	24	13.3
11-23-63	64	18	4	16	33	121	24	13.3
11-08-63	64	35	2	32	45	150	24	13.3
11-19-64	50	13	5	10	14	40	25	13.7
09-28-64	75	24	5	15	11	133	25	13.7

TABLE C.1 (CONTD.) SMALL STORMS DATA SET

DATE	EFFECTIVE PRECIP IN * 100	MAX HOUR PRECIP IN/HR*100	EFFECTIVE DURATION HOURS	AVERAGE INTENSITY IN/HR*100	BASEFLOW CFS	PEAK - BASEFLOW CFS	YEARS SINCE 1939	IMPERVIOUS SURFACE %
10-01-65	44	14	4	11	17	87	26	14.0
11-22-65	30	8	5	6	22	47	26	14.0
11-27-65	32	12	4	8	28	66	26	14.0
12-25-65	44	24	2	22	21	106	26	14.0
08-14-66	44	26	2	22	8	67	27	14.2
09-04-66	16	16	1	16	12	52	27	14.2
10-16-66	39	13	3	13	18	67	27	14.2
06-18-67	64	38	2	32	31	177	28	14.5
10-25-67	96	58	2	48	36	250	28	14.5
11-23-67	52	15	4	13	41	130	28	14.5
05-11-68	40	18	2	16	48	200	29	14.9
07-19-68	12	12	1	12	46	142	29	14.9
09-06-68	60	16	4	15	30	168	29	14.9
09-11-68	80	24	4	20	41	186	29	14.9
10-07-68	44	11	4	11	23	110	29	14.9
11-07-68	60	19	5	12	25	163	29	14.9
09-17-69	93	31	3	31	42	276	30	15.5
11-19-69	65	22	3	21	45	233	30	15.5
07-10-70	70	65	1	65	25	383	31	16.0
10-15-70	84	19	6	14	15	156	31	16.0
10-22-70	81	41	3	27	33	317	31	16.0
11-04-70	40	19	2	17	19	140	31	16.0

TABLE C.1 (CONTD.) SMALL STORMS DATA SET

DATE	EFFECTIVE PRECIP IN * 100	MAX HOUR PRECIP IN/HR*100	EFFECTIVE DURATION HOURS	AVERAGE INTENSITY IN/HR*100	BASEFLOW CFS	PEAK - BASEFLOW CFS	YEARS SINCE 1939	IMPERVIOUS SURFACE %
05-08-71	76	20	4	19	38	224	32	16.6
08-26-72	15	15	1	15	40	150	33	17.2
09-18-72	85	85	1	85	33	685	33	17.2
10-28-72	52	52	1	52	30	450	33	17.2
07-15-73	24	24	1	24	46	257	34	17.8
11-12-74	20	20	1	20	46	200	35	18.3
09-12-75	54	29	2	27	46	263	36	19.0
09-16-76	20	7	3	6	30	72	37	19.5
09-10-76	100	67	2	50	27	570	37	19.5
06-01-77	70	21	3	20	39	327	38	20.3
10-01-77	19	19	1	19	24	200	38	20.3
07-15-78	72	49	2	48	30	539	39	20.8
08-31-78	65	65	1	65	50	607	39	20.8
09-19-78	48	27	2	24	40	381	39	20.8

APPENDIX 4

HOURLY RAINFALL DATA FOR SELECTED SMALL STORMS

Thiessen Networks

The earliest hourly rainfall stations in the Saddle River Basin are New Milford and Woodcliff Lake (see figure 3.1, page 34). For these two stations, the drainage area was partitioned as follows:

Station	Enclosed Area(sq. mi.)	Proportion of total area
Woodcliff Lake	36.8	0.67
New Milford	17.8	0.33
	54.6	1.00

Allendale and Hackensack stations started operating in 1953. (See fig. 3.1).The drainage area was partitioned into Thiessen Polygons with the following areas:

Station	Enclosed Area(sq. mi.)	Proportion of total area
Woodcliff Lake	6.8	0.12
New Milford	8.7	0.16
Hackensack	8.7	0.16
Allendale	30.4	0.56
	54.6	1.00

The Hackensack gage stopped operating in 1963 and the remaining three stations were partitioned as follows:

Station	Enclosed Area(sq. mi.)	Proportion of total area
Woodcliff Lake	6.80	0.12
New Milford	17.40	0.32
Allendale	30.40	0.56
	54.6	1.00

The Allendale gage stopped in 1973, and the thiessen network thus reverted to the initial format with only Woodcliff Lake and New Milford gages in operation.

Given below is hourly rainfall data for some selected storms. Note that hourly rainfall is supplemented in some cases by daily rainfall records from neighbouring stations.

TABLE D.1 *HOURLY RAINFALL DATA FOR SOME SMALL STORMS

		A M HOUR ENDING												P M HOUR ENDING												
STATN	DATE	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	TOT.
Wdclf	10-10-41								06	08	16	12	10	01												53
N Mlf	10-10-41							02	10	12	16	10	08	01												59
Wdclf	07-02-45															27	15	08	02	03	03	03				61
N Mlf	07-02-45															41	15	07	03	02	03	03				74
Wdclf	10-08-48			03	04	04	02	14	15	16	02	01	01				03	02	03							70
N Mlf	10-08-48			02	06	07	06	10					40				01	02	01							75
Wdclf	10-28-53	08	08	02	08	24	40	16	10	02													02	04	04	120
N Mlf	10-28-53	10	06	02	12	22	32	26	08													01	03	03	03	128
Alld	10-28-53	02	02	02	14	20	40	07	03	05														02	03	100
Hack	10-28-53	07	06	02	15	46	39	25	10													03	02	01	02	158
Wdclf	07-06-55																					66	82	08		156
N Mlf	07-06-55																						38	18	09	65
Alld	07-06-55																						70	16	01	87
Hack	07-06-55																				02			13	05	20

* Values (in inches) were multiplied by 100
Wdclf = Woodcliff Lake : N Mlf = New Milford
Alld = Allendale : Hack = Hackensack

TABLE D.1 *HOURLY RAINFALL DATA FOR SOME SMALL STORMS

		A M HOUR ENDING												P M HOUR ENDING												
STATN	DATE	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	TOT.
Wdclf	08-05-59			03	02	14	13	20	15	18	07	02														94
N Mlf	08-05-59			04	01	11	17	11	02	16	07	01														70
Alld	08-05-59		01	03		10	23	17	10	25	05	01														95
Hack	08-05-59				06	03	17	17	13		03	04	01													64
Wdclf	10-16-66				04	07	16	17	11																	55
N Mlf	10-16-66				03	11	11		03																	28
Alld	10-16-66				-	-	-	-	-																	-
Wdclf	10-15-70					+	+	40	03	05	16	30	09		02						08					113
N Mlf	10-15-70							17	13	02	14	14	05	08	03	01										77
Alld	10-15-70						25	20	10	06	15	14	07	09	05							05				116
Wdclf	07-15-73			09	03	13				01	01	02	10	01												40
N Mlf	07-15-73			05	24	06	40					01	04	12						02	38	01	08	48	48	145
Alld	07-15-73			11	30								07	05	02					02		06	01	09	05	55
Wdclf	06-01-77										21	19	10	05	16	13		01								85
N Mlf	06-01-77										39	25	05	08	22	03										102

* Values (in inches) were multiplied by 100 : + = Cumulated values
Wdclf = Woodcliff Lake : N Mlf = New Milford
Alld = Allendale : Hack = Hackensack

APPENDIX 5

TABLE E.1 *HOURLY RAINFALL DATA FOR SOME MAJOR STORMS

		A M HOUR ENDING												P M HOUR ENDING												
STATN	DATE	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	TOT.
Wdclf	08-24-45			02	03	04	13	13	19	11	05	19	11	16	23	16	29	21	06	06	08	08				233
N Mlf	08-24-45			04	05	05	14	13	32	06	05	18	10	14	16	15	26	24	05	10	13	10	01	01		247
Wdclf	04-01-48			05	21	25	16	38	33	03		04	01	01				01								149
N Mlf	04-01-48			04	17	22	15	46	39	06	01	04	03		01		02									160
Wdclf	09-11-54	16	14	14	30	40	82	62	34	30	16	22	04	06	03											379
N Mlf	09-11-54	23	11	18	34	56	98	50	120	56	20	20	06	06	03	01										522
Alld	09-11-54	14	08	13	25	30	70	57	63	69	21	15	05													390
Hack	09-11-54	12	10	31	32	79	63	105	90	55	10	27	05	03												522
Wdclf	04-16-61	01				01	05	03	04	12	15	04	25	36	65	04	02									177
N Mlf	04-16-61						03	04	04	03	14	02	13	28	69	06	01	01		01						149
Alld	04-16-61					01	09	01	04	11	10	07	23	40	27	07										140
Hack	04-16-61					04	04	04	05	05	10	04	20	35	50	02	01									144
Wdclf	01-09-64						01		01	02	07	12	12	16	19	22	13	10	07	02	03					127
N Mlf	01-09-64								02	02	02	10	07	13	15	20	12	04	03	01	02					93
Alld	01-09-64						02	03	01	02	08	16	20	34	36	38	30	30	07	05	01	01			02	236

* Values (in inches) were multiplied by 100
Wdclf = Woodcliff Lake : N Mlf = New Milford : Alld = Allendale

TABLE E.1 *HOURLY RAINFALL DATA FOR SOME MAJOR STORMS

		A M HOUR ENDING												P M HOUR ENDING												
STATN	DATE	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	TOT.
Wdclf	04-24-68								03		02	12	17	21	06	02	02	15	33	21	21	10	01	24	05	195
N Mlf	04-24-68									01	01	08	19	21	16	01	01		20	37	33	12	05	20	10	205
Alld	04-24-68											08	27	29	04	03	04	25	35	+	+	+	+	+	118	253
Wdclf	07-13-72						17	32	49	80	54	15	10	02	01		02									262
N Mlf	07-13-72			01		01	25	33	75	99	57	02														293
Alld	07-13-72							01	31	06	62	32	22	12	03	02	01									172
Wdclf	05-14-78					02	02						05	33	35	19	16	04	15	24	20	03	05	02	03	188
N Mlf	05-14-78															07	14	04	12	12	12	02	02	02	01	68
Wdclf	01-08-78										01	04	02	09	08	02	04	06	08	23	03	01	08	02	03	84
	01-09-78	01	39	07	44	09	04	11	07	04																126
N Mlf	01-08-78												03	09	06	01	08	03	14	23	01		04	01		73
	01-09-78	03	32	16	31	03	05	07	07	03						01										108

* Values (in inches) were multiplied by 100 : + = Cumulated values
Wdclf = Woodcliff Lake : N Mlf = New Milford
Alld = Allendale : Hack = Hackensack

APPENDIX 6

ABBREVIATED TABLE OF K VALUES

The Table on pages 137 and 138 give values of frequency factor, K, for selected exceedance probabilities and skew coefficients.

P = Exceedance Probability

G = Skew Coefficient

P	G =0.0	G =0.1	G =0.2	G =0.3	G =0.4	G =0.5	G =0.6
0.9999	-3.71902	-3.50703	-3.29921	-3.09631	-2.89907	-2.70836	-2.52507
0.9995	-3.29053	-3.12767	-2.96698	-2.80889	-2.65390	-2.50257	-2.35549
0.9990	-3.09023	-2.94834	-2.80786	-2.66915	-2.53261	-2.39867	-2.26780
0.9980	-2.87816	-2.75706	-2.63672	-2.51741	-2.39942	-2.28311	-2.16884
0.9950	-2.57583	-2.48187	-2.38795	-2.29423	-2.20092	-2.10825	-2.01644
0.9900	-2.32635	-2.25258	-2.17840	-2.10394	-2.02933	-1.95472	-1.88029
0.9800	-2.05375	-1.99973	-1.94499	-1.88959	-1.83361	-1.77716	-1.72033
0.9750	-1.95996	-1.91219	-1.86360	-1.81427	-1.76427	-1.71366	-1.66253
0.9600	-1.75069	-1.71580	-1.67999	-1.64329	-1.60574	-1.56740	-1.52830
0.9500	-1.64485	-1.61594	-1.58607	-1.55527	-1.52357	-1.49101	-1.45762
0.9000	-1.28155	-1.27037	-1.25824	-1.24516	-1.23114	-1.21618	-1.20028
0.8000	-0.84162	-0.84611	-0.84986	-0.85285	-0.85508	-0.85653	-0.85718
0.7000	-0.52440	-0.53624	-0.54757	-0.55839	-0.56867	-0.57840	-0.58757
0.6000	-0.25335	-0.26882	-0.28403	-0.29897	-0.31362	-0.32796	-0.34198
0.5704	-0.17733	-0.19339	-0.20925	-0.22492	-0.24037	-0.25558	-0.27047
0.5000	0.0	-0.01662	-0.03325	-0.04993	-0.06651	-0.08302	-0.09945
0.4296	0.17733	0.16111	0.14472	0.12820	0.11154	0.09478	0.07791
0.4000	0.25335	0.23763	0.22168	0.20552	0.18916	0.17261	0.15589
0.3000	0.52440	0.51207	0.49927	0.48600	0.47228	0.45812	0.44352
0.2000	0.84162	0.83639	0.83044	0.82377	0.81638	0.80829	0.79950
0.1000	1.28155	1.29178	1.30105	1.30936	1.31671	1.32309	1.32850
0.0500	1.64485	1.67279	1.69971	1.72562	1.75048	1.77428	1.79701
0.0400	1.75069	1.78462	1.81756	1.84949	1.88039	1.91022	1.93896
0.0250	1.95996	2.00688	2.05290	2.09795	2.14202	2.18505	2.22702
0.0200	2.05375	2.10697	2.15935	2.21081	2.26133	2.31084	2.35931
0.0100	2.32635	2.39961	2.47226	2.54421	2.61539	2.68572	2.75514
0.0050	2.57583	2.66965	2.76321	2.85636	2.94900	3.04102	3.13232
0.0020	2.87816	2.99978	3.12169	3.24371	3.36566	3.48737	3.60872
0.0010	3.09023	3.23322	3.37703	3.52139	3.66608	3.81090	3.95567
0.0005	3.29053	3.45513	3.62113	3.78820	3.95605	4.12443	4.29311
0.0001	3.71902	3.93453	4.15301	4.37394	4.59687	4.82141	5.04718

P	G =0.7	G =0.8	G =0.9	G =1.0	G =1.1	G =1.2	G =1.3
0.9999	-2.35015	-2.18448	-2.02891	-1.88410	-1.75053	-1.62838	-1.51752
0.9995	-2.21328	-2.07661	-1.94611	-1.82241	-1.70603	-1.59738	-1.49673
0.9990	-2.14053	-2.01739	-1.89894	-1.78572	-1.67825	-1.57695	-1.48216
0.9980	-2.05701	-1.94806	-1.84244	-1.74062	-1.64305	-1.55016	-1.46232
0.9950	-1.92580	-1.83660	-1.74919	-1.66390	-1.58110	-1.50114	-1.42439
0.9900	-1.80621	-1.73271	-1.66001	-1.58838	-1.51808	-1.44942	-1.38267
0.9800	-1.66325	-1.60604	-1.54886	-1.49188	-1.43529	-1.37929	-1.32412
0.9750	-1.61099	-1.55914	-1.50712	-1.45507	-1.40314	-1.35153	-1.30042
0.9600	-1.48852	-1.44813	-1.40720	-1.36584	-1.32414	-1.28225	-1.24028
0.9500	-1.42345	-1.38855	-1.35299	-1.31684	-1.28019	-1.24313	-1.20578
0.9000	-1.18347	-1.16574	-1.14712	-1.12762	-1.10726	-1.08608	-1.06413
0.8000	-0.85703	-0.85607	-0.85426	-0.85161	-0.84809	-0.84369	-0.83841
0.7000	-0.59615	-0.60412	-0.61146	-0.61815	-0.62415	-0.62944	-0.63400
0.6000	-0.35565	-0.36889	-0.38186	-0.39434	-0.40638	-0.41794	-0.42899
0.5704	-0.28516	-0.29961	-0.31368	-0.32740	-0.34075	-0.35370	-0.36620
0.5000	-0.11578	-0.13199	-0.14807	-0.16397	-0.17968	-0.19517	-0.21040
0.4296	0.06097	0.04397	0.02693	0.00987	-0.00719	-0.02421	-0.04116
0.4000	0.13901	0.12199	0.10486	0.08763	0.07032	0.05297	0.03560
0.3000	0.42851	0.41309	0.39729	0.38111	0.36458	0.34772	0.33054
0.2000	0.79002	0.77986	0.76902	0.75752	0.74537	0.73257	0.71915
0.1000	1.33294	1.33640	1.33889	1.34039	1.34092	1.34047	1.33904
0.0500	1.81864	1.83916	1.85856	1.87683	1.89395	1.90992	1.92472
0.0400	1.96660	1.99311	2.01848	2.04269	2.06573	2.08758	2.10823
0.0250	2.26790	2.30764	2.34623	2.38364	2.41984	2.45482	2.48855
0.0200	2.40670	2.45298	2.49811	2.54206	2.58480	2.62631	2.66657
0.0100	2.82359	2.89101	2.95735	3.02256	3.08660	3.14944	3.21103
0.0050	3.22281	3.31243	3.40109	3.48874	3.57530	3.66073	3.74497
0.0020	3.72957	3.84981	3.96932	4.08802	4.20582	4.32263	4.43839
0.0010	4.10022	4.24439	4.38807	4.53112	4.67344	4.81492	4.95549
0.0005	4.46189	4.63057	4.79899	4.96701	5.13449	5.30130	5.46735
0.0001	5.27389	5.50124	5.72899	5.95691	6.18480	6.41249	6.63980