Report No. FHWA-RD-78-88

RESEARCH ON THE EFFECTS OF URBANIZATION ON SMALL STREAM FLOW QUANTITY

December 1978

Final Report

Prepared for

FEDERAL HIGHWAY ADMINISTRATION

Offices of Research & Development

Washington, D. C. 20590

FOREWORD

This report describes the development of a simple technique to estimate the effects of urbanization on flood peaks in small streams. It was evaluated against results available for a limited number of basins and found to be reasonably accurate.

Research in urban hydrology is included in the Federally Coordinated Program of Highway Research and Development as part of Task 2 of Project 5H "Protection of the Highway from Hazards attributed to Flooding." Dr. Roy E. Trent is the Project Manager and Dr. D. C. Woo is the Task Manager.

This research was conducted by Resource Analysis, Inc., at Waltham, Massachusetts, for the Federal Highway Administration, Office of Research, Washington, D.C., under P.O. 7-3-0006.

Sufficient copies of this report are being distributed to provide a minimum of two copies to each FHWA regional office, one copy to each division office, and one copy to each State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-RD-78-88		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code / 8. Performing Organization Report No.
Research on the Effects of Urbanization on Small Stream Flow Quantity	5. December 1978 6. 8.
7. Author(s) B. M. Harley	
9. Performing Organization Name and Address Resource Analysis, Inc. 235 Wyman Street Waltham, Massachusetts 02154	10. Work Unit No. (TRAIS) 11. Contract or Grant No. P.O. 7-3-0006
12. Sponsoring Agency Name and Address U.S. Department of Transportation Federal Highway Administration Washington, D.C. 20590	13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

Contract Manager: D. C. Woo

16. Abstract

This study is a preliminary investigation into the feasibility of using simple techniques to evaluate the effects of urbanization on flood flows in small streams. A number of regression techniques and computer simulation techniques were evaluated, and a recommendation was made that a model similar to those developed by Carter and Anderson be adopted for further investigation. The technique was evaluated against results available for a number of basins located throughout the United States, and reasonable agreement noted.

17. Key Words	18. Distribution Statement
Hydrology, Urbanization, Small Streams	Document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
UNCLASSIFIED	UNCLASSIFIED	56	

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PREFACE

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The assistance of Dr. D. C. Woo of the Federal Highway Administration in the conception and supervision of this study is gratefully acknowledged.

METRIC CONVERSION FACTORS

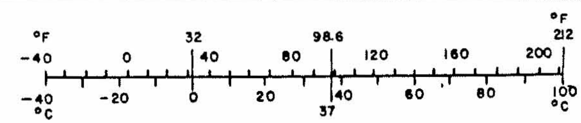
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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Section 1

SCOPE OF STUDY

1.1 Introduction

The goals of the study presented herein were to select a method which could be used to estimate the effect of urbanization on the flood peaks from rural watersheds and, secondly, to formulate a research program which would develop this method for nationwide use. The present study is thus a forerunner of a far more comprehensive study, and is designed to seek and evaluate some alternative flood peak estimation measures. The study was confined to watersheds with areas of twenty-five square miles or less, and analyzes the effect of urbanization of all degrees on flood peaks with recurrence intervals of ten, twenty-five, fifty, and one hundred years. The study was confined to evaluating the impact on the peak discharge rate only and does not include prediction of the total runoff hydrograph.

Several other constraints were also imposed. These included that the allowable error of estimate was to be plus or minus twenty percent, and that the approximate time for the analysis of a drainage structure would be about three person-days.

1.2 The Problem

Rapid urban development during the last three decades has greatly effected the natural drainage response of many formerly rural watersheds. This urbanization has had significant impact on highway drainage facilities, especially the great number of culvert accommodating small stream crossings. Many of these existing highway culverts are no longer adequate for the increased flood flows.

The replacements for these culverts and culverts installed in new highway construction should be designed to handle the increased flows which can be anticipated from forecast urbanization of the drainage basin. Although the design of drainage facilities on major drainage systems can make use of the many recently developed simulation techniques to evaluate the impact of planned urbanization on the response of the watershed, it is not generally economically feasible to deploy such techniques where the basins concerned are quite small.

The net hydrologic effect of urbanization is to increase the volume and peak rate of storm runoff due largely to the increased drainage density of the stream conveyance network, and to the decrease in pervious surface in the basin. The quantitative evaluation of this impact on small basins is quite difficult for the following reasons:

- The degree of urbanization is constantly changing, and this combined with the stochastic nature of storm events makes determination of the true flood frequency curve practically impossible even where field data exist.

- Very little field data exists in the actual discharge from small watersheds. Several test watersheds have been extensively instrumented, but their geographic extent is limited.

- There is also a lack of a single generalized method to estimate the flood flow from completely rural watersheds on a nationwide basis. However, reliable techniques do exist in various areas throughout the country.

Although it has been frequently suggested that a massive nationwide effort be devoted to the collection of data from many small watersheds, and that the resulting data be analyzed to yield generalized flood frequency formulae, this approach has a number of significant problems:

- The cost of a sufficiently detailed data measurement network is quite excessive.

- Since the degree of urbanization of many of the basins of interest changes quite slowly, the data collection effort would have to continue over quite a number of years.

- The analysis of time varying systems subjected to stochastic rainfall inputs is very difficult. It is difficult to reliably evaluate the shift in the flood frequency curve under such conditions.

Rather than embark on such a massive data collection program with potentially limited results, it is F.H.W.A.'s desire to determine if it is possible to develop an inexpensive method of estimating the increase in the peak flow as a result of urbanization in a basin. This technique would thus separate the increased flows due to development in the basin from the response of the natural watershed itself. In addition, any such technique should exhibit the following desirable characteristics:

- require only data that can be readily evaluated by the user from physical basin characteristics
- require a minimum of judgement factors so that the various individuals applying the method can obtain consistent results
- be based on sound hydrologic principles so that the relative importance and impact of the required data can be understood and evaluated by the design engineers
- be relatively easy to use and not require elaborate computational effort.

1.3 The Approach

The approach adopted in this preliminary study was quite straightforward. The first step involved a review of various existing methods for evaluating the impact of urbanization on the hydrologic response of the watershed. The methods covered ranged from hand computation techniques to complex computer-based mathematical models.

Following this initial review, it was determined that a method similar to that first suggested by Carter (1961) and expanded on by Anderson (1970) had many of the desirable characteristics. This method was modified as described in this report and applied to a number of basins throughout the United States. The sensitivity of the model to parameter inputs was evaluated.

In general, it was found that the revised method could be applied nationwide within the accuracy and time constraints of the present study. It is recommended that this technique serve as the basis of a further evaluation of the applicability of simple methods to estimate the net effects of urbanization or peak flood flows from small watersheds.

Section 2

REVIEW AND EVALUATION OF METHODS

2.1 General Review

The first phase of the study included a review of the existing methods for estimating the effects of urbanization on the peak flow response characteristics of small watersheds. Several of the references reviewed are listed in the Bibliography. In addition, an OASIS search of the National Weather Service and Library of Congress files was performed to expand the list of available publications.

The review of these publications concentrated on determining if a method or methods which included the desired characteristics for this study was available. The expected general form of a feasible method is as follows:

$$\Delta Q = f(K_B \cdot K_U \cdot K_T \cdot K_C) \quad (2.1)$$

where:

ΔQ = increase in flow due to urbanization

K_B = the set of basin parameters

K_U = the set of urbanization parameters

K_T = the set of recurrence interval parameters

K_C = the climatic factors

The review of the literature indicated that while hydrology is rapidly moving from the "rational formula" era to the use of relatively complex simulation models for evaluating the response characteristics of a given watershed, very few "generalized" studies have been made of the quantitative effects of urbanization. The majority of the techniques used in the literature are extremely site or region specific and do not include a sufficient level of breakdown of the various aspects of urbanization as indicated in Equation (2.1) above.

In addition to the review of essentially hand computation techniques, a review was also conducted of some of the more generalized computer models which have begun to be applied throughout the country in the past decade. Although these models are quite complex in operation, they are nearly all extremely simple in their underlying formulation of the various physical processes to be simulated. The models reviewed during this study included:

SWMM - the E.P.A.'s Storm Water Management Model

STORM - A model used by the Corps of Engineers primarily to study non-point source runoff

HEC-1 - The basic hydrograph routing and combining model of the Corps of Engineers

ILLUDAS - The Illinois Urban Drainage Simulation Model

MITCAT - The M.I.T. Catchment Model which is primarily used to simulate urban areas.

This review of these models indicated the following aspects of interest to the current study:

a. Although an analyst thoroughly familiar with one of the models could possibly apply it to a small basin in order to assess the impacts of urbanization within the time and budget constraints set by this study, these models are in general too complex for use under the desired goals.

b. Although many of the models were relatively linear in operation, they all indicated a very high degree of transferability between various locations throughout the nation. This transferability is achievable without extensive re-calibration in most cases, especially where small basins are involved and complications due to groundwater exchange are minor.

c. Several good data sets are available from model simulation studies of urbanized areas. Although these data sets are not quite so satisfying as having field data available, the accuracy of the discharge frequency curves is comparable to that achieved from analyzing actual streamflow records. In addition, several of these models have been exercised to estimate the effects of various degrees of urbanization on a basin.

The review of the general body of literature indicates that the majority of techniques proposed fit into a number of broad categories. These can be listed as:

a. Frequency Analysis of Streamflow - These are techniques developed to statistically analyze streamflow data which exist in a watershed. Since the constraints set for this study preclude extensive data collection on the part of the designer, methods based on this technique were generally of little use.

b. Empirical Formulae - Although a number of such techniques exist, e.g., the Rational Formula, their ability to handle the changes in watershed response due to urbanization is extremely limited.

c. Regression Techniques - These methods relate effects such as peak flow, annual runoff, etc. with causal factors such as watershed characteristics, and precipitation frequencies through statistical correlation. The methods are extensions of empirical formulae usually based on more data and more sophisticated methods of analysis. A number of techniques have been presented in the literature which meet many of the goals of this study.

d. Hydrograph Synthesis - These are the most complex methods and are the basis for the detailed simulation models. However, the effort required to apply these techniques is generally more than that available under the constraints set for this study.

2.2 Selected Methods

The net result of the evaluation performed in this phase of the study was that a method similar to that developed by Carter (1961) and subsequently extended by Anderson (1970) could prove to be a feasible tool to adapt as a nationwide method. Anderson's method had been extensively applied in conjunction with the MITCAT model to basins in Fairfax and Henrico Counties, Virginia and had indicated that it possessed the ability to track the changes in watershed response with increasing urbanization. The input parameters for the basic model included the following:

- Basin Area
- Basin Shape - Stream Length Only
- Slope - Stream Slope Only
- Measures of Urbanization - % Imperviousness
  - % Sewered
- Recurrence Intervals - Ranges from 2.3 to 100 Years
- Climatic Factors - Included as a Single Constant

A similar procedure was developed by Stankowski (1974) for use in the New Jersey area.

RAI experience with the strengths and weaknesses of Anderson's method indicated that it could form the basis of a nationwide model once several characteristics had been added. A study of the relationships of Q_{100} to Q_{10} for a number of basins which RAI had modeled using detailed models indicated that the recurrence frequency ratios used by Anderson did not change appreciably for several basins scattered through the country. Similarly, the procedures used to estimate basin lag time gave reasonably good answers on the basins tested. In both of these situations, the minimal non-linearity in the response of small basins makes such transferability possible.

Section 3

A FEASIBLE METHOD

3.1 The Basin Method

As noted in Section 2, a method similar to that used in Virginia by Anderson (1970) appeared to offer many of the desirable characteristics required for a feasible nationwide technique. Anderson's basic relation is of the form:

$$\overline{Q} = 230\ K \cdot A^{0.82}\ T^{-0.48} \qquad (3.1)$$

where:

$\overline{Q}$ = mean annual flood (cfs)

A = watershed area (square miles)

T = lag time (hours)

K = coefficient related to degree of impervious cover in the basin

230 = factor related to rainfall and soil conditions in the northern Virginia area - determined by regression analysis

In addition, the t-year peak discharge can be estimated from the relationship:

$$Q_t = R \cdot \overline{Q} \qquad (3.2)$$

where:

R = flood frequency ratio - function of recurrence frequency and degree of imperviousness

It is possible to define the effect of urbanization on the peak discharges for a given recurrence interval in the following form:

$$\hat{Q}_t = \frac{Q_t^D}{Q_t^N}$$

$$= \frac{230\ K_D \cdot A^{0.82} \cdot T_D^{-0.48} \cdot R_D}{230\ K_N \cdot A^{0.82} \cdot T_N^{-0.48} \cdot R_N}$$

$$= \frac{K_D}{K_N} \cdot \frac{R_D}{R_N} \cdot \left(\frac{T_D}{T_N}\right)^{-0.48} \qquad (3.3)$$

where:

$\hat{Q}_t$ = ratio of urbanized to rural peak flow at the t-year recurrence interval

D, N = subscripts reflecting developed and rural values

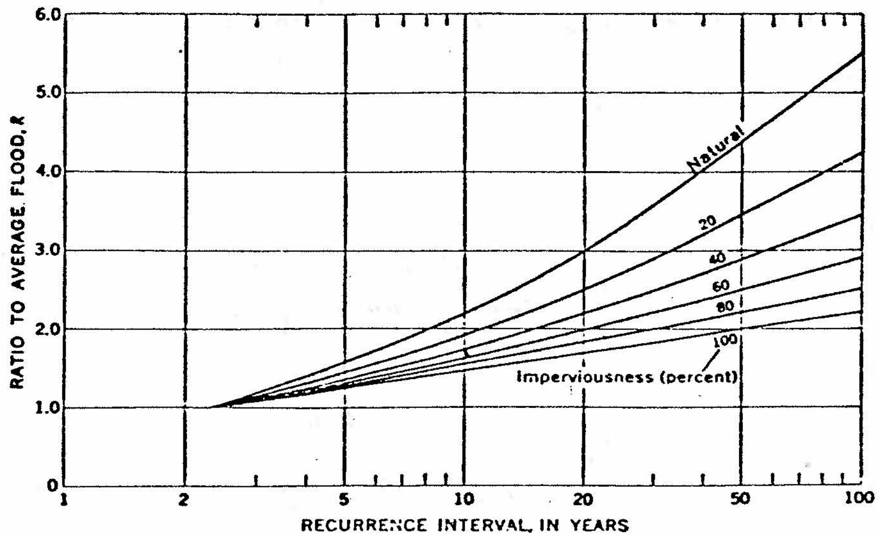
The factors used in Equation (3.3) are readily derived from available basin characteristics and standard charts. The R factors can be evaluated from the chart shown in Figure 3.1, and using the appropriate imperviousness ratios. The imperviousness coefficient, K, first proposed by Carter (1961) is defined by:

$$K = 1.00 + 0.015\ I \qquad (3.4)$$

where:

I = percentage of basin covered by impervious surfaces

The lag times, T, are related to the level of development of the basin and the main stream length and slope. These were defined by Anderson to be:



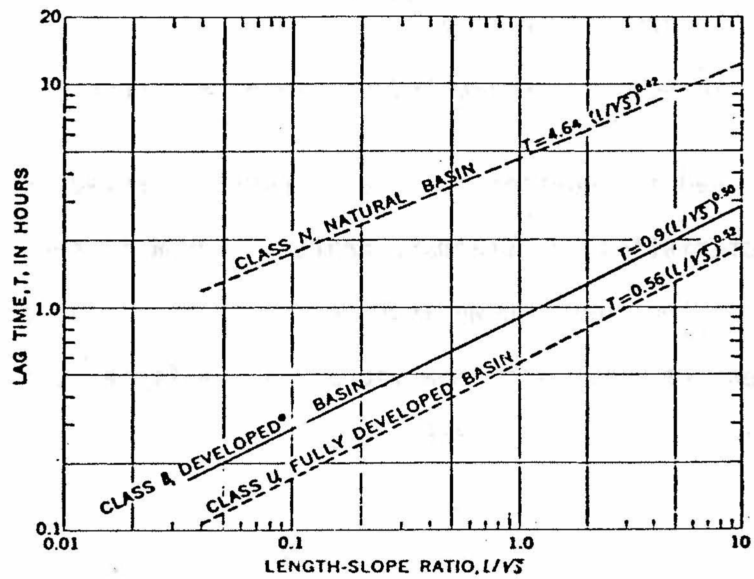


Figure 3.1: Anderson's Figures
(From: Anderson (1970))

$$T_N = 4.64\ (L/\sqrt{S})^{0.42}, \text{ for undeveloped basins (Class N)} \qquad (3.5)$$

$$T_B = 0.9\ (L/\sqrt{S})^{0.50}, \text{ for developed basins (Class B)} \qquad (3.6)$$

$$T_U = 0.56\ (L/\sqrt{S})^{0.52}, \text{ for fully developed basins (Class U)} \qquad (3.7)$$

where:

L = distance (miles) along the primary water course for the stream mouth to the boundary of the watershed

S = average slope (feet/mile) of the primary water course between points located at 10 and 85 percent of the length, L, upstream of the stream mouth

These ratios are of the general form:

$$\log T = f\ (\log\ (L/\sqrt{S})) \qquad (3.8)$$

which is widely used by many investigators in determining the lag times of a watershed. A fourth class of basin, P, was defined in this study to enable the hydrologist to evaluate situations where the urbanization lies somewhere between the undeveloped case (Class N) and the limit of development represented by Class B. The percentage of the basin subject to storm sewers is the factor used to select the lag time for such basins. The governing equation is as follows:

$$T_P = T_N + (T_B - T_N) \cdot * (S_W/100) \qquad (3.9)$$

where:

S_W - percentage of basin considered sewered

3.2 Modifications

Several modifications were made to the basic method to enable it to be applied to areas of the country other than northern Virginia. These modifications are as follows:

Runoff Factor

The factor, '230', used in the basic method is related to the response of small watersheds to the rainfall regime of Virginia. An analysis of the relative rainfall depths of the fifty-year and one hundred-year two-hour durations to the twenty-five year depth indicated that these ratios were fairly constant throughout the nation. It was thus decided to modify this factor as follows:

$$H = 230 \cdot \frac{E}{E_V} \qquad (3.10)$$

where:

H = runoff factor

E = direct runoff for the twenty-five year, two-hour duration storm at the basin site for undeveloped conditions (ins)

E_V = direct runoff for northern Virginia conditions under similar conditions (ins)

Several infiltration computation techniques were applied to watersheds which had recently been simulated in Fairfax County, Virginia as part of a Comprehensive Drainage Study. (Parsons, Brinckerhoff, Quade & Douglas, 1973/4). These techniques included the SCS Curve Number method (SCS, 1969), Hortons technique (Chow, 1964) and Holtans Method (USDA, 1975). Although it was found, as expected, that the direct runoff varied considerably from basin to basin, it appears that it is possible to reduce Equation (3.10) to the following simplified form

$$H = 200 \cdot E \tag{3.11}$$

for the purposes of this initial methodological review.

Surface Storage Factor

Initial applications of the method indicated that the technique was deficient in handling areas where some of the watershed was covered by lakes or swamps. A review of the literature (Chow, McCuen, Stankowski and others) indicated that factors of the following general form were used to represent the effect of surface storage on peak flows.

$$F_s = (S_f)^{-0.5} \tag{3.12}$$

where:

F_s = surface storage factor

S_f = percentage of drainage basin occupied by lakes or swamps plus 1 percent (to handle the situation where area lakes/swamps is zero)

A typical use of such a term is illustrated by Stankowski (1974).

Population Index

The basic method requires the user to estimate the percentage imperviousness of the watershed, and the percent of the basin which is considered to be sewered. In certain situations it may be desirable to allow the user to express the level of urbanization of a basin in the form of the present or future population density in the area. An analysis of the population density for the basins used in this study indicated that the following equations can be used to estimate the percent impervious and percent sewered.

$$I = 0.250\ P^{0.554} \qquad (3.13)$$

$$S_W = 0.262\ P^{0.608} \qquad (3.14)$$

where:

P = population density in people per square mile

I = percent imperviousness of basin

S_W = percent of basin which is sewered

Both of these equations are somewhat simpler in form that previous ones proposed in the literature, for example, in Stankowski (1974). However, their accuracy is consistent with the requirements and sensitivity of the proposed technique, as will be shown later.

Proposed Method

As a result of the modifications suggested above, the basic method to estimate the required peak flood frequency can be expressed as follows:

$$Q_t = 200 \cdot E \cdot K \cdot R \cdot A^{0.82} \cdot T^{-0.48} \cdot S_f^{-0.5} \qquad (3.15)$$

where each of the terms has been defined previously.

3.3 Applications

A number of basins for which data were readily available were used in the initial applications of the proposed method. The significant parameters of each of these basins is shown in Table 3.1. It will be noted that these basins cover a relatively wide geographic area, and range from quite heavily developed urbanized basins to essentially undeveloped watersheds. Each of these basins had been previously analyzed using either field data or extensive computer modeling. As a result, flood frequency curves for the levels of urbanization shown were available.

The modified technique described in Section 3.2 was applied to each of these basins and its ability to estimate the absolute peak flood frequency evaluated. In order to permit rapid application of the technique to numerous basins, a computer program was written to evaluate Equation (3.15) for each of the basins listed in Table 3.1. This program used as input variables the following:

Basin Area (A)
Main Stream Length (L) and Slope (S)
Imperviousness Percentage
Sewered Percentage
Surface Storage Percentage
SCS Curve Number
Rainfall depth, 25-year, 2-hour storm

The program then used functional representations of Anderson's curves shown in Figure 3.1 to compute the remaining variables required for application of Equation (3.15). The response of the basins was then evaluated for recurrence periods of 10, 25, 50 and 100 years and for

TABLE 3.1: TEST BASIN CHARACTERISTICS

Name	Location	Area (Acres)	Area (Sq.Mi.)	Stream Length (Miles)	Slope (ft/ft)	Pop.* Density (people/ sq. mi.)	Impervious† (%)	Sewered† (%)	Surface Storage (%)	SCS Curve Number
Baldwin	Long Island	1041	1.6	3.09	0.0027	16000	42	95	-	50
West-End	Henrico, Va	112	0.2	0.19	0.012	2290	12.6	30.7	10	70
Mountain	Henrico, Va	594	0.9	1.48	0.0053	107	5.9	4.8	-	70
Stankowski	N.J.	1920	3.0	3.0	0.0028	3900	2.5	-	-	65
Castro Valley	California	3526	5.5	4.81	0.04	6000	27	60	-	85
Upham Brook	Henrico, Va	25408	39.7	12.4	0.003	3800	22	40	2	70
Espiritu Santo	Puerto Rico	19084	29.8	3.8	0.014	100	5	5	-	95
Q. Suspiro	Puerto Rico	1279	2.0	2.2	0.0135	500	10	5	30	85

Note:

*Estimated from available data

†Values used in previous analysis or models.

present conditions (Class P), pervious (Class N) and expected future maximum development levels of the basin (Classes B and U). These classes were defined by Anderson as:

Class N - Basins with natural channel systems

Class B - Small tributaries sewered, main channels remain in natural stage

Class U - All channels completely sewered

The application of this program permitted a rapid evaluation of the total response characteristics of the watershed. The comparison between the "true" values and the estimated ones is shown in Table 3.2. The predicted flood frequency curves for most of the test watersheds are shown in Figures 3.2 to 3.7.

The following general observations on the applicability of the technique can be made.

a. It appears that the general technique is able to adequately estimate the peak flood response from each of the basins selected. There appears to be no consistent bias in the results towards over or under prediction.

b. The information shown in Figures 3.2 to 3.7 is of significant interest to the design engineer in that it not only shows the present and future anticipated peak values, but also indicates the increase which would be possible in the basin were it to undergo complete urbanization to Class B for example.

TABLE 3.2

COMPARISON OF ESTIMATED AND "ACTUAL" PEAK FLOWS

Basin	Frequency	Actual	Estimated*	Error (%)	Source of "Actual" Flows
Baldwin	2	369	367	- 0.5	MITCAT Model
	5	510	473	- 7.3	
West-End	50	99	104	5.1	MITCAT Model
	100	141	128	- 9.2	
Mountain	50	279	201	-27.9	MITCAT Model
	100	316	250	-20.9	
Stankowski	2	300	270	-10.0	Stankowski, 1974
	5	430	383	-10.9	
	10	560	547	- 2.3	
	25	750	771	2.8	
	50	890	974	9.4	
	100	1090	1178	8.1	
Castro Valley	5	1200	982	-18.2	Various Models HEC-1, SWMM, STORM, MITCAT
	10	1500	1398	- 6.8	
	25	2100	1960	- 6.7	
	100	3100	2974	- 4.1	
Upham Brook	50	7074	7916	11.9	MITCAT Model and
	100	8740	9607	9.9	
Espiritu Santo	10	24000	22093	- 8.0	MITCAT Model USGS Frequency Curves
	25	33000	32669	- 1.0	
	50	40000	43073	7.7	
	100	51000	53476	4.9	
Q. Suspiro	10	520	451	-13.3	MITCAT Model
	100	930	1060	14.0	

*Applying Equation (3.15)

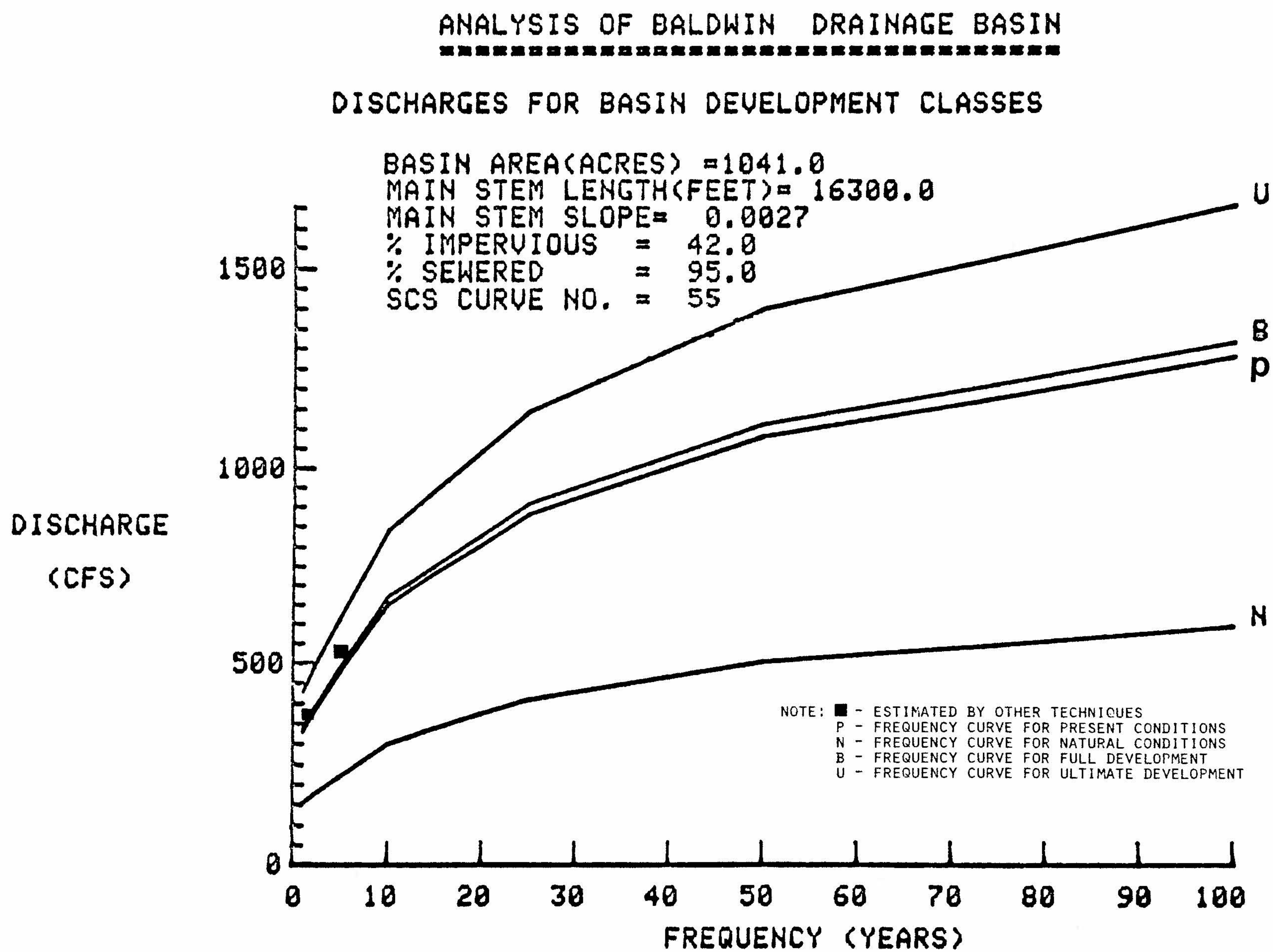


Figure 3.2 - Estimated Discharge Frequency Curves for Baldwin Creek

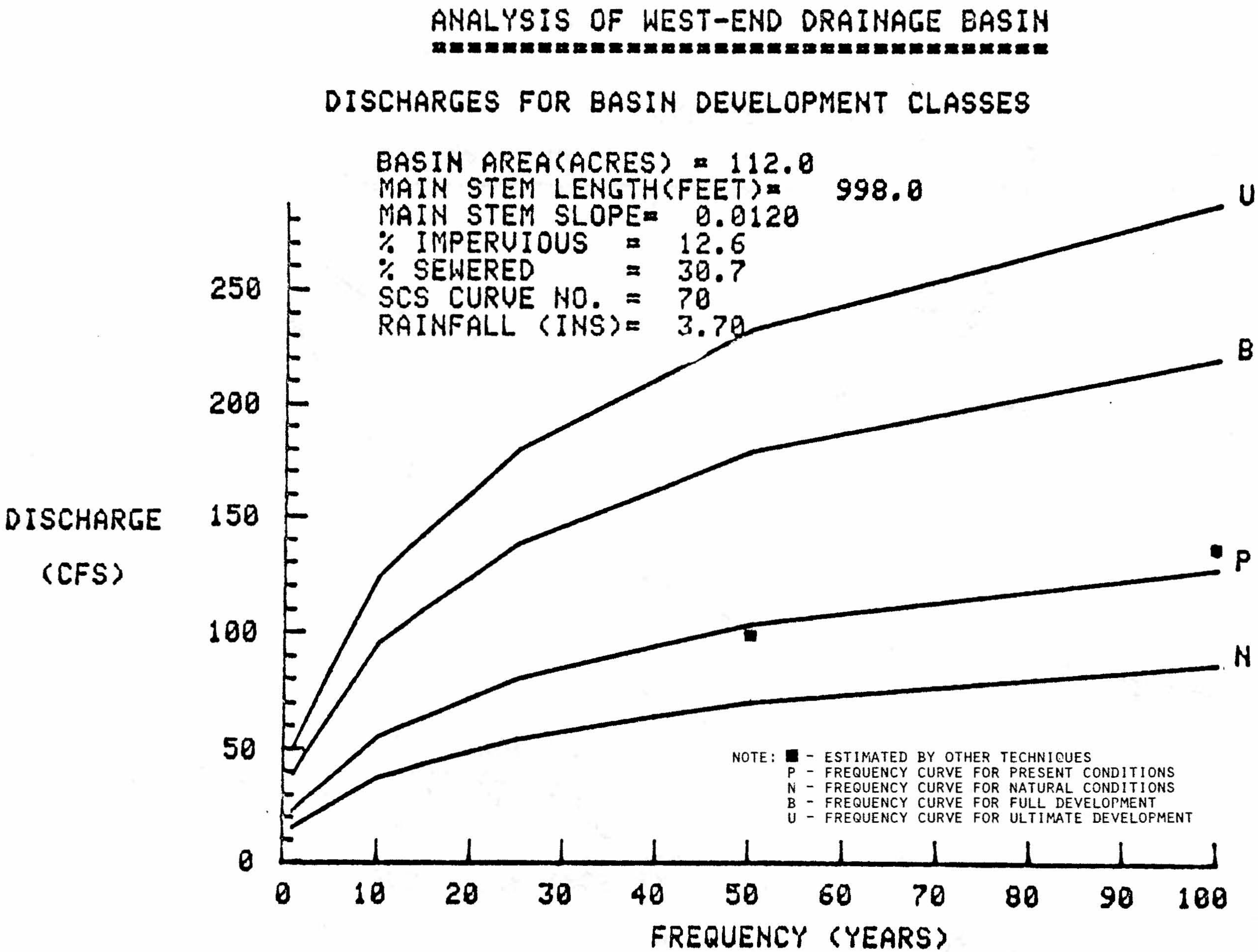


Figure 3.3 - Estimated Discharge Frequency Curves for West End Basin

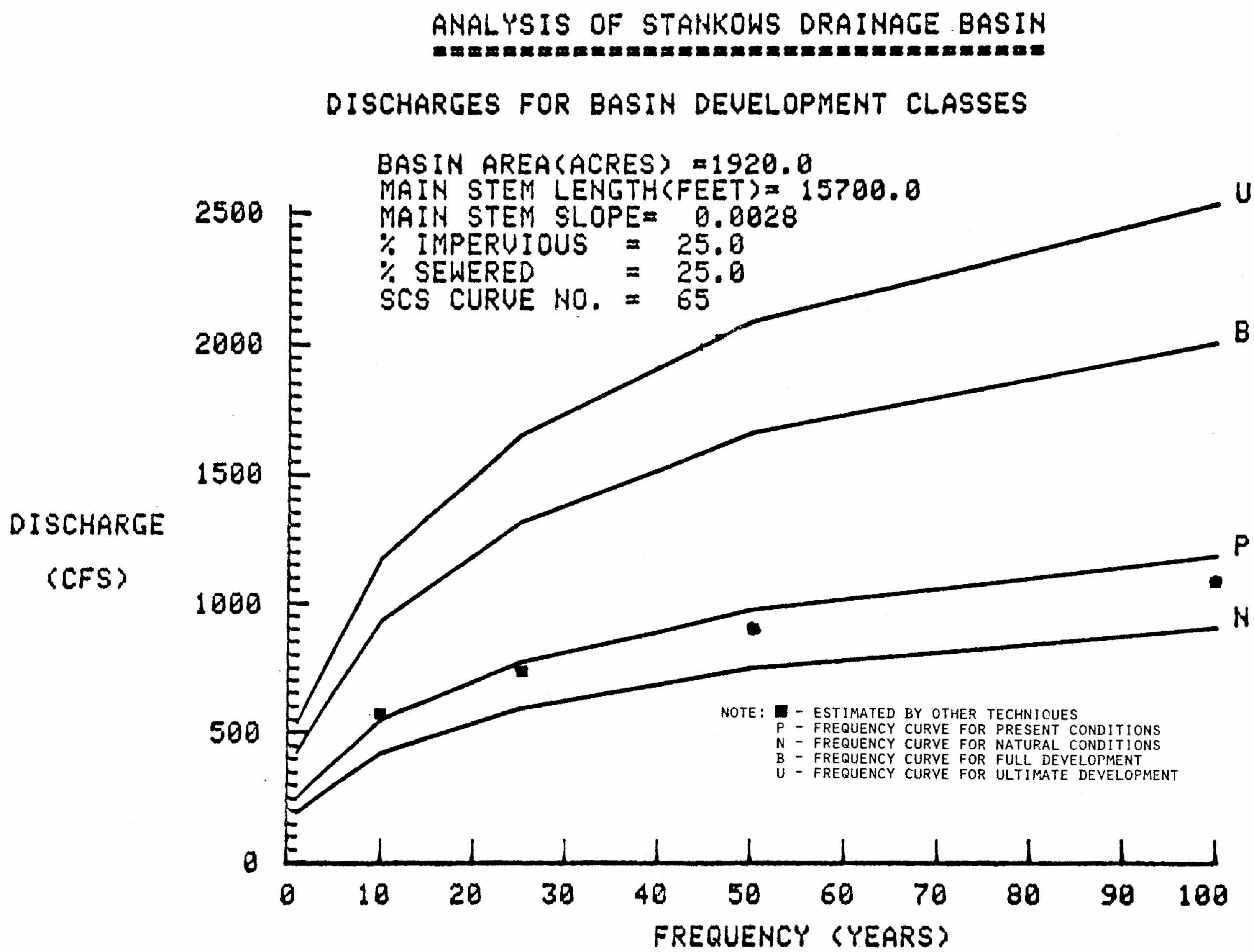


Figure 3.4 - Estimated Discharge Frequency Curves for Stankowski's Watershed

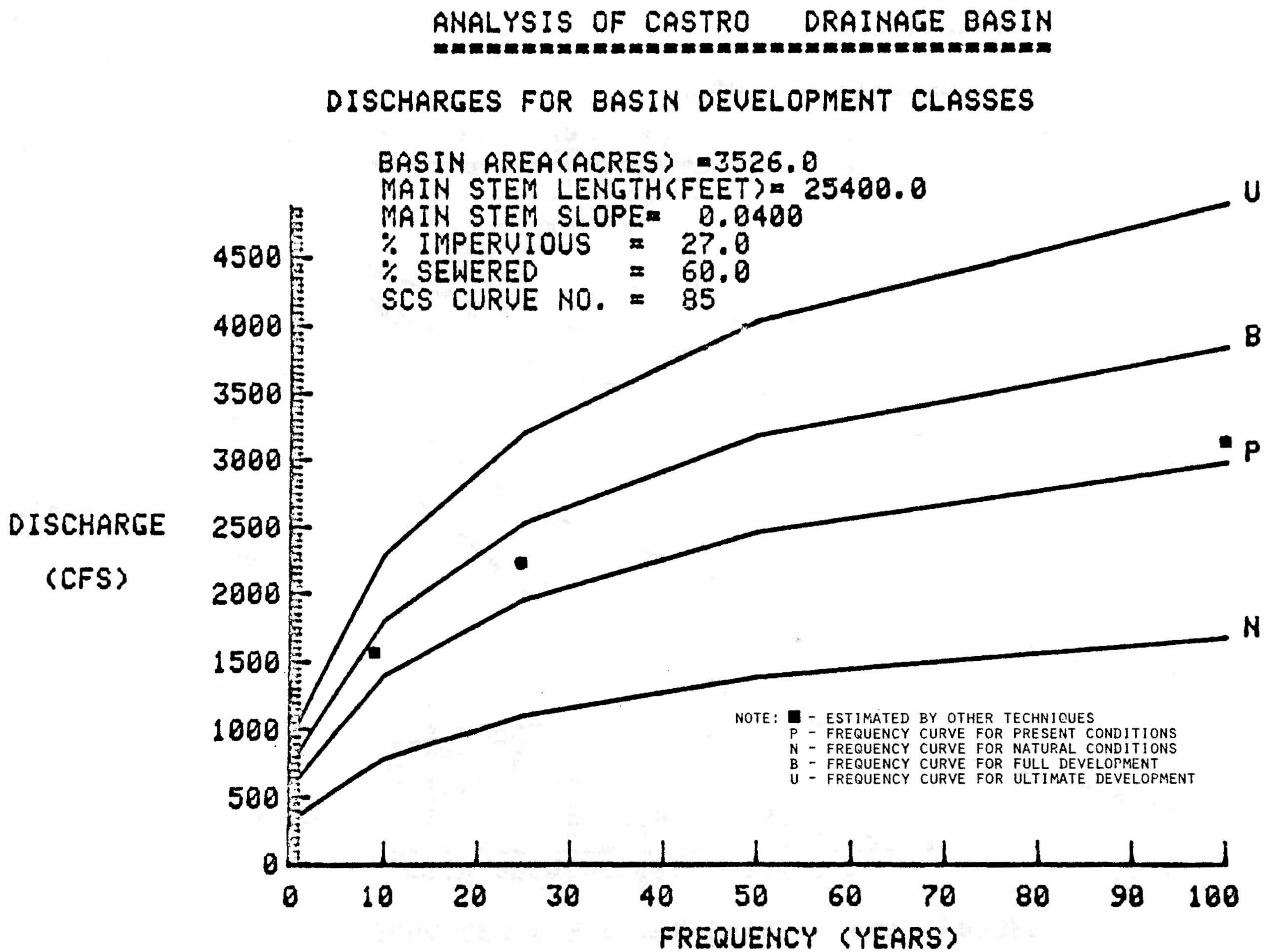


Figure 3.5 - Estimated Discharge Frequency Curves for Castro Valley

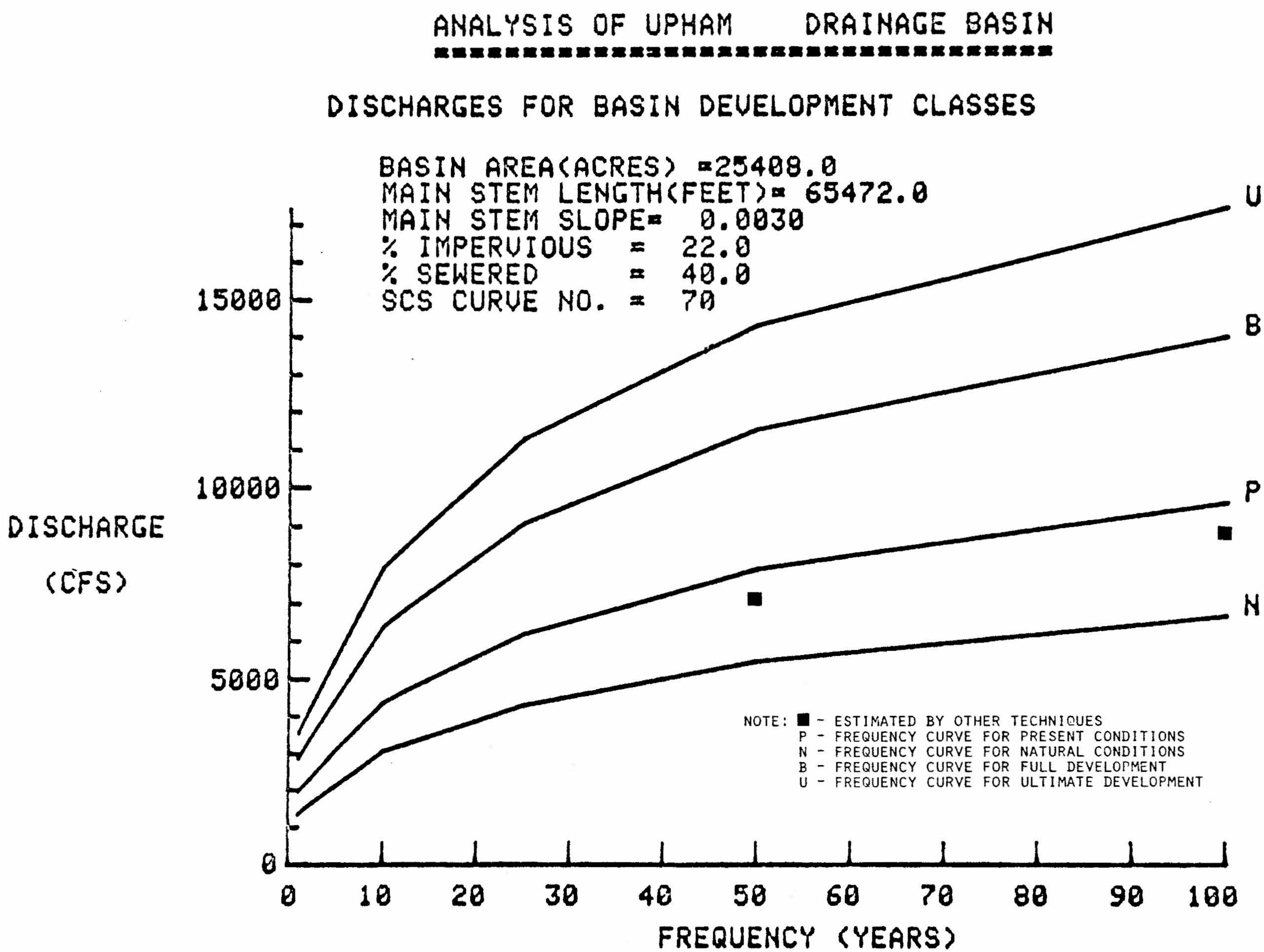


Figure 3.6 - Estimated Discharge Frequency Curves for Upham Brook

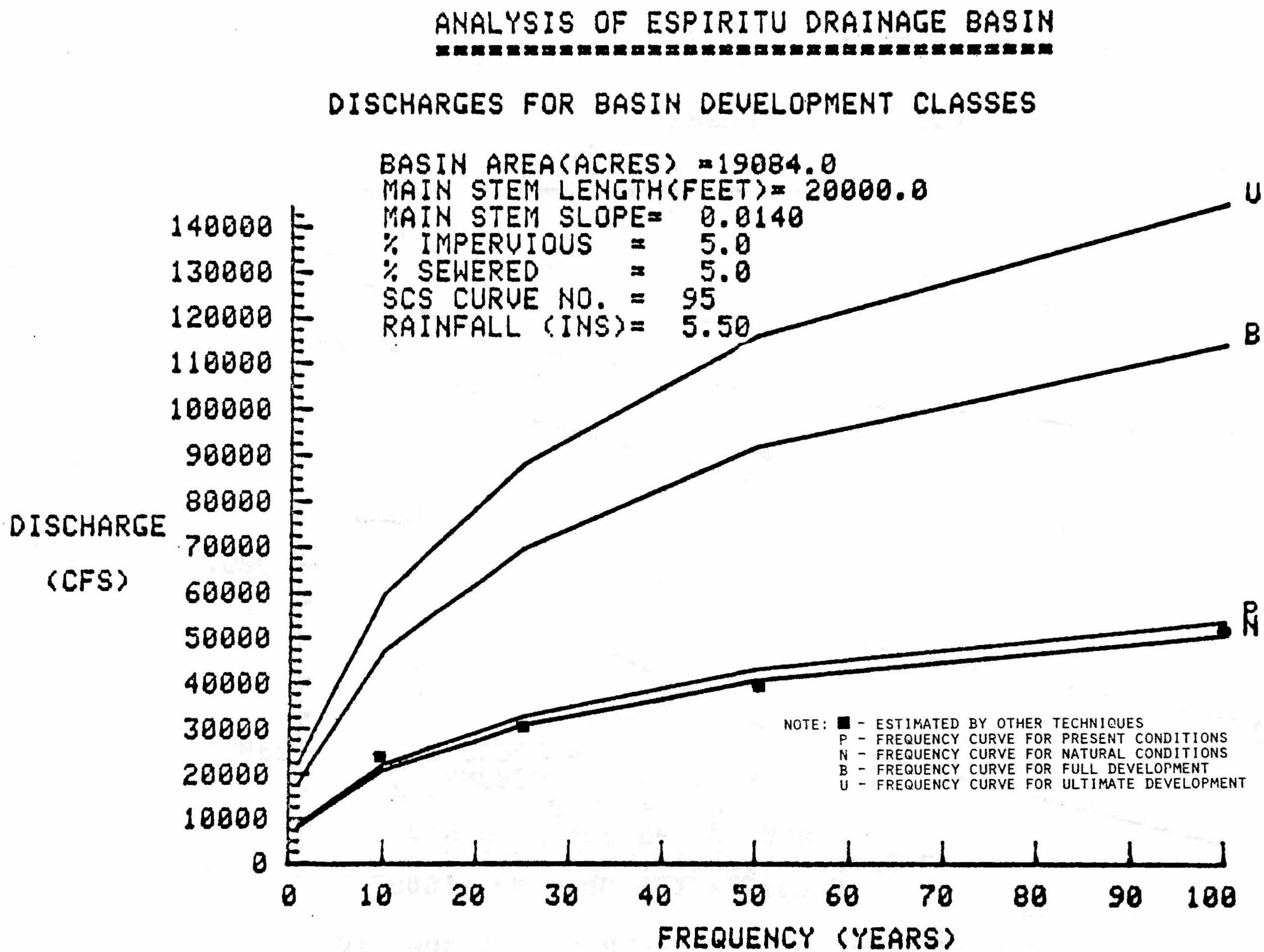


Figure 3.7 - Estimated Discharge Frequency Curves for Espiritu Santo Basin

3.4 Sensitivity of Method

In order that the proposed technique be feasible for operational use, it is essential that the predicted peak discharge not be unduly sensitive to parameter uncertainty. To this end, a parameter-by-parameter analysis was carried out. The results are tabulated as follows:

Parameter	Relative Sensitivity	Change in $\hat{Q}$ for 10% Change
E - Direct Runoff	High	10%
K - Imperviousness Factor	f(I)	-
R - Frequency Ratio	f(I)	-
A - Area	Known	-
T - Lag Time	f(L, S, S_W)	-
L - Stream Length	Known	-
S - Stream Slope	Known	-
S_f - Surface Storage	High	70%
I - % Imperviousness	Low	1%
S_W - % Sewered	Moderate	8%
P - Population Density	Moderate	5%

An example of the effect of changes in the basic parameters in the case of the example basin used by Stankowski is shown in Figure 3.8.

In general, it is felt that the method is not unduly sensitive to input parameters except for:

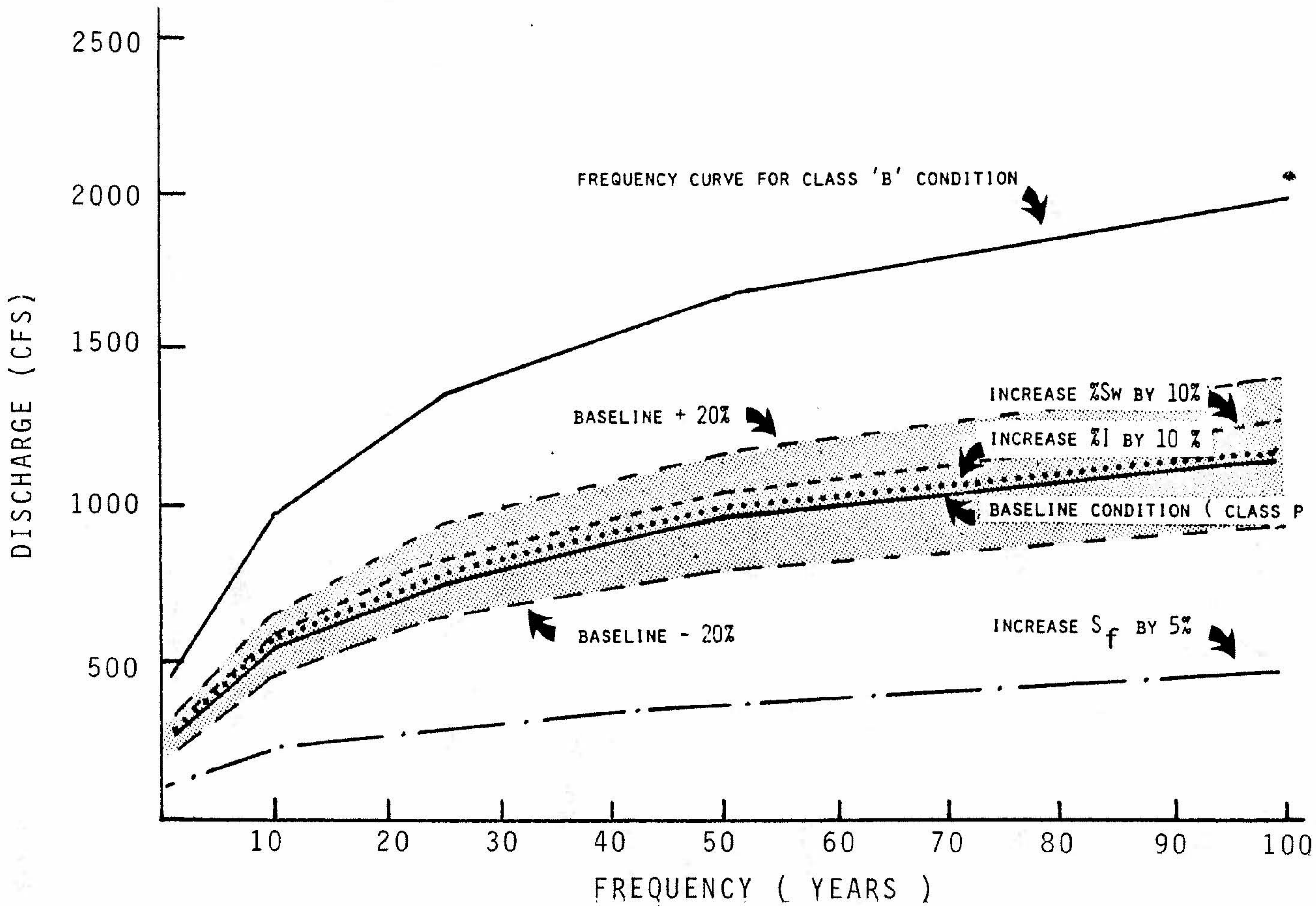


FIGURE 3.8 SENSITIVITY TO INPUT PARAMETERS

1. The rainfall, infiltration index (or cover complex), etc., which ultimately yield the factor, 'E'.

2. The surface storage index.

3.5 Evaluating ΔQ

As point out in Section 1, the goal of this study is not to determine the absolute value of the peak discharge following urbanization, but rather to determine the change in discharge which results from any such urbanization. This goal removes the sensitivity of the results to the parameter E which has been found difficult to estimate. A ratio approach can be developed from the following equation:

$$\tilde{Q}_t = \frac{K_D}{K_N} \cdot \frac{R_D}{R_N} \cdot \left(\frac{T_D}{T_N}\right)^{-0.48} \cdot \left(\frac{S_{fD}}{S_{fN}}\right)^{-0.5} \qquad (3.16)$$

where:

$\tilde{Q}_t$ = ratio of flow after urbanization to the natural flow in the system

The step-by-step methodology necessary to use this process is as follows:

1. Determine the population density, P_D, from population forecasts, etc.

2. Determine the basin imperviousness from:

$$I_D = 0.25\ P_D^{0.554} \qquad (3.17)$$

3. Determine the percent of the basin sewered from:

$$S_{WD} = 0.262\ P_D^{0.608} \qquad (3.18)$$

4. Determine K_D from:

$$K_D = 1.00 + 0.015\ I_D \tag{3.19}$$

5. Determine R_D from Figure 3.9a

6. Determine the basin stream length, L, and slope, S from basin plans.

7. Compute the Length-Slope Ratio = $L/\sqrt{S}$

8. Use Figure 3.9b to determine the lag time for this basin. If S_{WD} is greater than zero, then determine the lag time from:

$$T_D = T_N + (T_B - T_N) * S_{WD}/100 \tag{3.20}$$

where:

T_B and T_N are determined from Figure 3.9 or Equations (3.5) and (3.6)

9. Estimate the surface storage index, S_{fD}, by planimetery from the watershed maps.

10. Repeat steps 1 - 9 for the existing level of development (subscripted N).

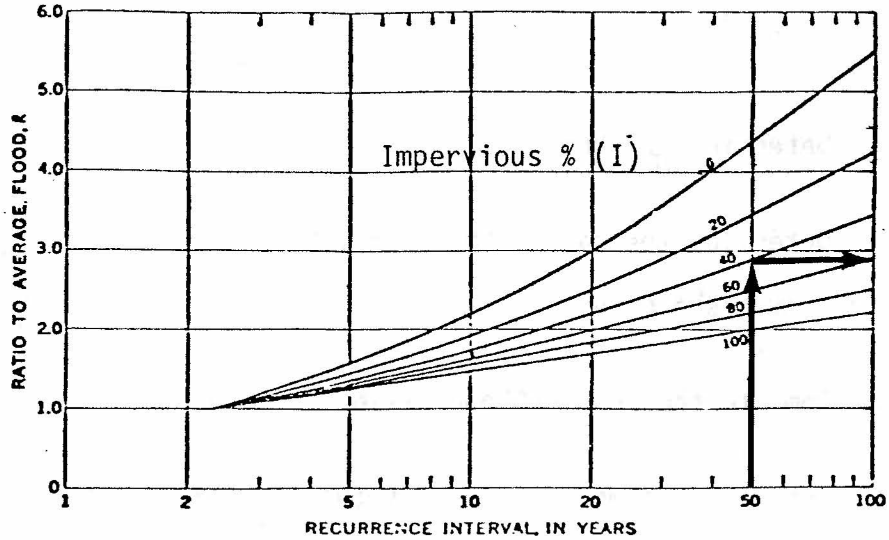
11. Compute $\tilde{Q}_t$ from Equation (3.16) above.

12. Estimate Q_{tN} (the pre-urbanization peak flood discharge at the t-year recurrence interval) from existing techniques.

13. Evaluate the urbanized peak flood flow, Q_{tD} from:

$$Q_{tD} = \hat{Q}_t \cdot Q_{tN} \tag{3.21}$$

a. Flood Frequency Ratios



b. Length-Slope Ratio ($L/\sqrt{S}$)

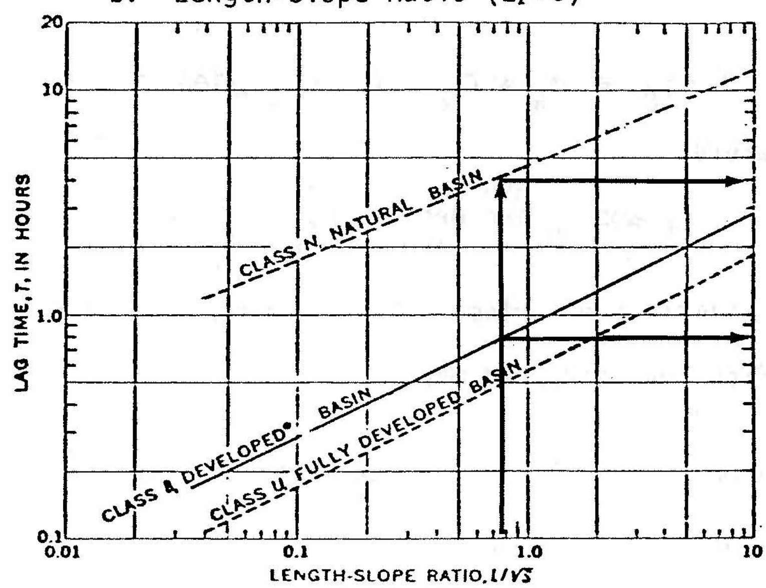


Figure 3.9

(Adapted from Anderson (1970))

It should be noted that multipliers for other frequencies can be estimated by simply re-evaluating the R_D and R_N factors of Step 5 above. All the other parameters remain constant.

An alternative approach involves using the basic equation to evaluate the change in the peak discharge with increase in urbanization. This difference can be expressed as:

$$\Delta Q = 200E\ A^{0.82}\ S_f^{-0.5}\left[K_D R_D T_D^{-0.48} - K_N R_N T_N^{-0.48}\right] \quad (3.22)$$

where the various terms have been defined previously.

The approach required to evaluate this procedure follows the general guidelines presented for the ratio method above. These steps can be summarized as follows:

1. Determine the basin area (A), stream length (L) and stream slope (S) from basin maps.

2. Compute the Length-Slope Ratio = $L/\sqrt{S}$.

3. Evaluate the area (A_s) occupied by surface water in the basin.

4. Compute the surface storage index, $S_f = A_s/A * 100 + 1$.

5. Estimate the effective runoff E (inches) from a twenty-five-year two-hour storm applied to the basin under pre-development conditions.

6. Determine the existing population density, P_N.

7. Compute the existing basin imperviousness from:

$$I_N = 0.25 P_N^{0.554} \tag{3.23}$$

8. Compute the percent of the basin considered sewered from:

$$S_{WN} = 0.262 P_N^{0.608} \tag{3.24}$$

9. Compute K_N from:

$$K_N = 1.0 + 0.015 I_N \tag{3.25}$$

10. Determine R_N from Figure 3.9a

11. Use Figure 3.9b to determine the existing lag time, T_N, in the basin. If S_{WN} is greater than zero, then determine an adjusted lag time using:

$$T_N = T_N + (T_B - T_N)*S_{WN}/100 \tag{3.26}$$

where T_N and T_B are determined from Figure 3.9 or Equations (3.5) and (3.6)

12. Repeat steps 6 - 11 for the forecast level of development/ population (subscripted D).

13. Compute the net increase in peak discharge rate using Equation (3.22).

Although this approach is quite similar to the ratio approach proposed above, it does involve direct use of the terms E, A and S_f to which the basic underlying equation is most sensitive. If used in this mode, the model is required to predict both the absolute magnitude of the basin's response in addition to the net increase due to urbanization. Present feelings are that the technique needs to be carefully tested on a wider range of basins before it can be reliably used in this manner.

3.6 Example Application

The ratio method is applied to a basin with the following characteristics:

$A = 3$ square miles

$L = 3$ miles

$S = 15$ feet/mile

$S_f = 1.0$

This is the same basin used by Stankowski as his example basin. Suppose the population density were to increase from 500 to 10,000 people per square mile. What is the change in the fifty-year discharge? The computational steps are indicated below.

Step	Variable	Rural	Developed
1	P	500	10,000
2	I	7.8	41.1
3	S_W	11.5	70.8
4	K	1.12	1.62
5	R	4.1	2.9
6	L, S	see above	
7	$L/\sqrt{S}$	0.77	0.77
8	T*	3.63	1.73
9	S_f	1	1

*Note: For this step, initially estimate $T_N = 4.0$, $T_B = 0.8$ from Figure 3.9

The factor $\tilde{Q}_t$ can then be computed to be

$$\tilde{Q}_{50} = 1.46 \qquad (3.27)$$

The method thus predicts that the fifty-year flood peak after urbanization will exceed that from the natural basin by 46 percent for this particular watershed. Stankowski's Figure 14 indicates that the discharge for the natural watershed would be 750 cfs. The estimated discharge after urbanization would thus be 1,095 cfs, indicating a net increase in discharge of 345 cfs.

In contrast, Stankowski's method would indicate that the after-urbanization discharge would be about 970 cfs. The difference between the two estimates of the after-urbanization flow is 13 percent, which can be considered within the allowable tolerance.

A direct application of Equation (3.22) to estimate the increase in peak discharge from the same basin indicates the following:

25-year, 2 hour rainfall = 3.2 inches

S.C.S. Curve number (CN) = 65

Direct Runoff, E = 0.6 inches

Applying Equation (3.22) and using the parameters derived on page 38 we find the net increase in flow to be

$$\Delta Q = 336 \text{ cfs}$$

This compares closely with the value of 345 cfs estimated using the ratio technique.

It will also be noted, that although the estimated flow rates after urbanization are within 13% of the "correct" value (i.e., Stankowski's estimate), the predicted increases are approximately 57% larger than those estimated by Stankowski.

It could also be noted, however, that this difference in flow estimate in this case is less than that required to change the subsequent selĕction of a culvert from among the standard sizes available. This insensitivity of this chosen culvert size to differences in the estimate of peak flow rates is obviously a function of basin size, and decreases with increase in flow to be handled. It has also been found, however, that the estimation technique is relatively better for larger basins, although the exact improvement in performance was not determined for this study.

Overall, it is felt that the technique is of potential application in addressing some of the problems caused by rapid urbanization of small watersheds. Some of the issues which require further consideration are discussed in Section 4.

Section 4

FURTHER RESEARCH PROGRAM

4.1 Introduction

The investigation carried out under this contract indicated that it is quite feasible to use a relatively simple technique to estimate the impact of urbanization on small basins which are located throughout the nation. The areas where the technique was tested ranged from semi-tropical conditions in Puerto Rico, through several mid-Atlantic coastline areas to a relatively dry region of California.

It will, no doubt, have been noted that in most of the basins tested, the simple technique was being evaluated against discharge values computed using a detailed simulation model of the basins rather than against direct field meansurements. This approach has been discussed earlier, and its advantages and problems presented. A significant point which has impact with respect to the application of the technique by local agencies is that the investigators were intimately familiar with the basins, their underlying parameters and response characteristics. It was thus perhaps somewhat easier to apply the simple technique to these basins and with a greater degree of confidence in the results than if the basin studied was a completely unstudied one.

As a result of this first phase, it is felt that the technique developed promises to be a worthwhile tool for use in the design of structures in small drainage basins. Prior to the technique being released for general use, it is suggested that a further set of research tasks be carried out to further develop the proposed method.

4.2 Tasks for Future Research

The proposed method indicated that the factors to which it was most sensitive were:

a. The Soil Cover Complex Curve Number

b. The Rainfall Rate Used

c. The Surface Storage Factor

The method of computing a $\tilde{Q}$ rather than an absolute Q to reflect the changes due to urbanization somewhat reduces the impact of the uncertainty in estimating the Curve Number and the Rainfall Rates. Although a less sensitive technique for handling the impact of distributed surface storage would be desirable, the technique can be operationally used with careful computation of the surface storage.

At this time it is recommended that the following tasks/goals be included in any further extension of this research effort.

Task A: Data Collection: In order to verify the applicability of this technique on a nationwide basis, it is recommended that the F.H.W.A. compile a basic data base of three to four basins in each state. The data for these basins should be supplied by the Department of Transportation (or other appropriate agency) of the state, and should reflect the degree of accuracy required for the technique. The basins selected should be those which are less than twenty-five square miles in area, be subject to possible urbanization, and have been studied recently in the context of culvert design, etc.

Task B: Application of Technique: The method presented in this report should be applied to each of the data sets collected in Task A. A critical analysis should be made with respect to the operation of the technique in the following areas:

1. Were the % Impervious and % Sewered estimated from the population estimates adequate in light of the detailed knowledge of each of the basins.
2. Was the ΔQ predicted correct in the area of both the immediate forecast population and in the estimated maximum/ultimate development of the basin.
3. Would the estimation errors have led to the selection of a culvert size which was too big or too small. This evaluation has to make allowance for such topics as the standard culvert sizes used by individual states, design safety factors, etc.
4. What difficulties were experienced in applying the technique following the manner suggested in this report.

Task C: Modification of Technique: The results of the test applications under Task B should be tabulated and carefully reviewed and discussed by a review panel. The panel members should include representatives of the F.H.W.A., a couple of state agencies and the research team. This panel will make recommendations as to the following items:

1. Is the basic methodology satisfactory for the proposed use in terms of fundamental design accuracy?

2. Which basin parameters should be handled in a different fashion if required?

3. Is the method of computation presented herein satisfactory, or should some other technique be developed? With the advent of desktop and pocket calculators, many of the requirements for reducing the solution procedure to a single nomograph are relaxed. This aspect may be dealt with by the review panel also.

The recommendations of the panel will be included in a revised technique and guidance for application.

Task D: Report on Application: The revised technique will then be applied to the data sets collected in Task A. This application will be similar to Task B, but should also include sensitivity studies to determine some limits of confidence in the technique.

Task E: Development of Guidance Manual: The final step before releasing the technique for line agency use is the development of a manual which will assist the user in applying the technique. It is recommended that this manual include not only the description of the basic technique but also the following:

1. Step-by-step procedure to use the technique.
2. A number of example studies, showing the various ways in which the technique can be applied.

3. A set of simple nomographs (at least) to convert population estimates to % Impervious Cover and % Basin Sewered estimates. Other possible charts include:

   Estimation of Recurrence Factor, R

   Estimation of Basin Lag Time, T

4. A listing of the source of underlying data such as SCS Curve Numbers for various soil classifications, standard rainfall rates for suggested design frequencies, etc. Although these data are typically available in other publications also, it is recommended that this manual be self-contained in so far as possible.

4.3 Time and Cost of Future Research

A preliminary evaluation of the time frame and budget required to carry out the proposed extension of the program has been made. This estimate is based on the following assumptions:

a. A small research team is involved, and that inter-agency coordination, etc. are not unduly time delaying.

b. The basic basin data would be gathered by a combination of local state agencies and a member of the research team. It is felt that using data from about twenty states should be sufficient at this time.

c. No major computational effort is required.

d. The final report will follow standard F.H.W.A. format.

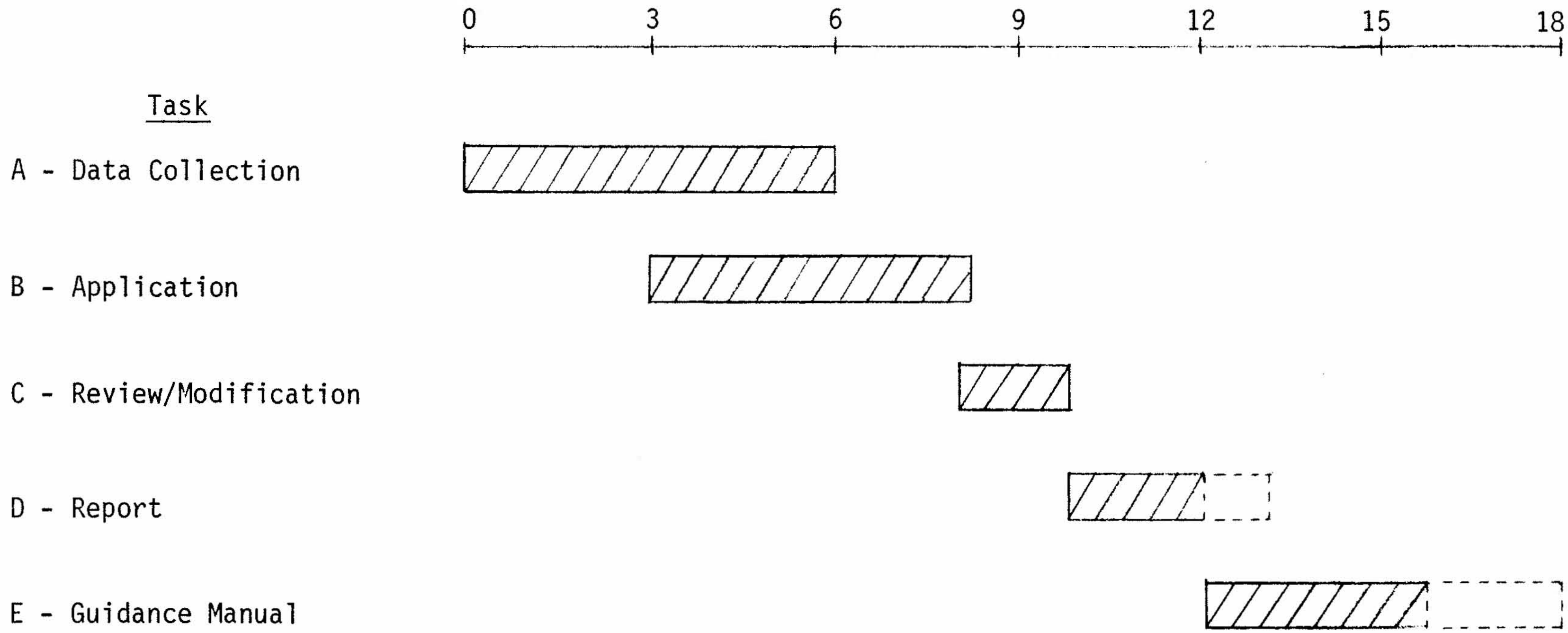
The estimated level of effort is shown in Table 4.1, and a suggested schedule bar chart is attached as Figure 4.1.

Table 4.1

PROPOSED LEVEL OF EFFORT

Task	Level of Effort (person months)	Travel Costs	Other Direct Expenses
A - Data Collection	6	$3000	$1000
B - Application	4	-	2000
C - Review/Modification	4	500	500
D - Report on Application	2	250	500
E - Guidance Manual	4	250	2000
TOTALS	20	$4000	$6000

Note: This estimate does not include level of effort of personnel outside the research team. Thus, the time of F.H.W.A. supervisory personnel, state agencies, etc. has not been included.



Note: [dashed box] - Indicates FHWA review and final document preparation time

Figure 4.1 Schedule

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☆ U. S. GOVERNMENT PRINTING OFFICE : 1979—630-087/2492

FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

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1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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