DIRECT RUNOFF HYDROGRAPH PARAMETERS VERSUS URBANIZATION

REFERENCES: Time of Concentration for Overland Flow
- The Effect of Urbanization on Floods of Different Recurrence Interval
- Instantaneous unit hydrographs, Peak Discharges and time Lages in in Urban Basins

TECHNICAL PAPER NO. 48

DIRECT RUNOFF HYDROGRAPH PARAMETERS VERSUS URBANIZATION

by

DAVID L. GUNDLACH

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DIRECT RUNOFF HYDROGRAPH PARAMETERS VERSUS URBANIZATION

By David L. Gundlach,[1] A.M. ASCE

Introduction

Various rainfall-runoff models are based on the development of unit hydrographs, loss rate functions, and routing criteria. With models of this type, characteristics used to define the unit hydrograph, loss rate, and routing criteria need to be modified to predict runoff that would occur because of future development within a watershed. Certain aspects of this problem, particularly changes in peak flow and lag time due to urbanization, have been treated previously (1,2,4,5,6). It is the aim of this note to present additional information regarding the modification of unit hydrograph characteristics due to increased urbanization and to introduce techniques which can be utilized in a practical solution.

Effects of Urbanization

A multiple regression analysis based on 15 flood hydrograph reconstitutions in the vicinity of Philadelphia, utilized in the preliminary report, "Metropolitan Chester Creek Basin, Pennsylvania," Department of the Army, Philadelphia District, Corps of Engineers, January 1976, resulted in the following expressions (see Table 1):

$$(TC + R) = 19.46\, I^{-0.40} \left(\frac{DA}{S}\right)^{0.24} \quad \ldots\ldots (1)$$

$$(TC) = 12.98\, I^{-0.42} \left(\frac{DA}{S}\right)^{0.27} \quad \ldots\ldots (2)$$

Eqs. 1 and 2 relate the direct runoff hydrograph parameters, TC and R to physiographic characteristics of the drainage basin, in which TC = the time from the end of effective rainfall to the inflection point on the recession limb of the hydrograph, in hours; R = the ratio of the discharge at the inflection point on the recession limb of the hydrograph to the rate of change of discharge at that point, in hours; I = the percentage of impervious surface within a watershed

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Note.—Discussion open until February 1, 1977. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 102, No. IR3, September, 1976. Manuscript was submitted for review for possible publication on October 23, 1975.

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(a measure of urbanization); DA = drainage area, in square miles; and S = the average channel slope between the points 10% and 90% of the distance upstream from the gage or outflow point to the watershed boundary, in feet per mile. Although the correlation was improved in subsequent work when specific physiographic and meteorological characteristics were combined (4), the information outlined in Table 1 is sufficient to illustrate the following techniques.

TABLE 1.—Results of Multiple Regression Analysis in Which Direct Runoff Hydrograph Characteristics, $TC + R$ and TC, are Related to Physiographic Characteristics of Drainage Basin

Equation number (1)	Standard error of estimate (2)	Correlation coefficient, $\bar{R}$ (3)	Coefficient of determination, $\bar{R}^2$ (4)
1	0.080	0.939	0.882
2	0.083	0.945	0.893

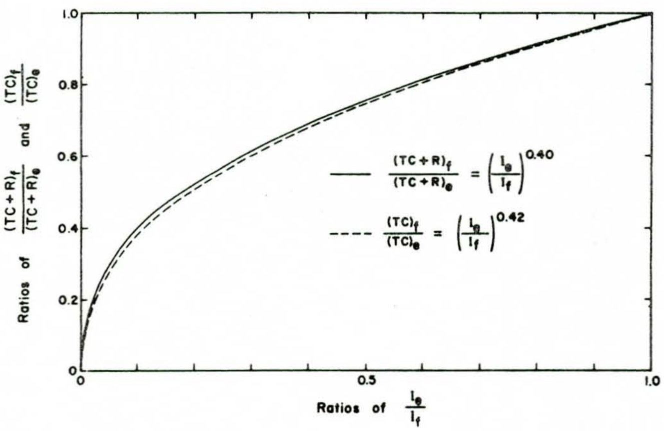

FIG. 1.—Effects of Changes in Imperviousness on Characteristics ($TC + R$) and (TC)

If development is predicted within one of the drainage basins in the study area and if drainage area and slope remain relatively constant with time, then it follows from Eqs. 1 and 2 that

$$\frac{(TC+R)_f}{(TC+R)_e} = \left(\frac{I_e}{I_f}\right)^{0.40} \quad \ldots\ldots (3)$$

and $$\frac{(TC)_f}{(TC)_e} = \left(\frac{I_e}{I_f}\right)^{0.42} \quad \ldots\ldots (4)$$

in which subscripts e and f refer to existing and future conditions, respectively. A graph of the left-hand terms of Eqs. 3 and 4 versus the change in imperviousness, I, is shown in Fig. 1. As indicated by the curves and indirectly by the Tracor Report (6), Eqs. 1 and 2 are significantly limited in range, particularly for practical application. Consider as first approximations the following two examples.

Example 1.—It is predicted that a pristine area, $I_e = 0\%$, will be developed to such an extent that at some time in the future it will be considered 100% impervious. In cases such as this where $I_e = 0\%$ initially the ratio $(I_e/I_f) = 0$ regardless of the value of I_f, and from Fig. 1 or Eq. 4, $(TC)_f = 0$. The preceding results are impractical even for very small values of $(TC)_e$.

TABLE 2.—Results of Multiple Regression Analysis in Which $K = 1.0 + 0.30\,I$

Equation number (1)	Standard error of estimate (2)	Correlation coefficient, $\bar{R}$ (3)	Coefficient of determination, $\bar{R}^2$ (4)
8	0.082	0.937	0.878
9	0.081	0.948	0.898

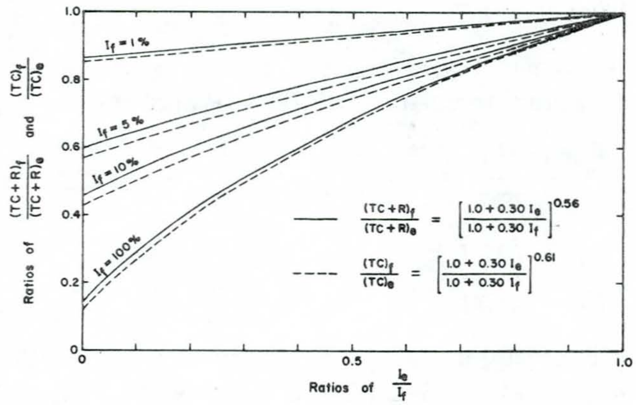

FIG. 2.—Effects of Changes in Imperviousness on $(TC + R)$ and (TC) where $K = 1.0 + 0.30\,I$

A reasonable estimate of $(TC)_f/(TC)_e$ for a condition similar to the preceding can be developed from the following formula proposed by Kerby (3):

$$t^{2.14} = \frac{2}{3}\frac{Ln}{\sqrt{S}} \quad \text{(5)}$$

in which t = the time of concentration for overland flow within a catchment area, in minutes; L = the length of flow, in feet; S = the slope of the surface, in feet per foot; and n = a retardance coefficient. In a situation where a dense grass covered surface will be completely paved, then

$$\frac{(t)_f}{(t)_e} = \left(\frac{n_f}{n_e}\right)^{0.47} \quad \text{(6)}$$

$$\text{or} \quad \frac{(t)_f}{(t)_e} = 0.18 \quad \text{(7)}$$

in which n_f and n_e are 0.02 and 0.80, respectively.

Example 2.—If only a small amount of development is predicted, $0\% < I_f \leq 5\%$, for a relatively pristine drainage area, then, in most cases, it is reasonable to assume that development may have little or no effect on the time of concentration. Under these circumstances $(TC)_f/(TC)_e$ should be equal to or nearly equal to 1.

First approximations such as given in Examples 1 and 2 were used to modify the regression expressions (Eqs. 1 and 2) as originally developed. In this case various transformations were tested until one of the general forms, $K = C_1 + C_2\, I$, proved applicable. The constants, C_1 and C_2, were varied until the initial approximations were reasonably satisfied and an optimum degree of correlation obtained. The modified relationships (see Table 2) are

$$(TC + R) = 17.01\, K^{-0.56} \left(\frac{\mathrm{DA}}{S}\right)^{0.24} \quad \text{(8)}$$

$$(TC) = 11.54\, K^{-0.61} \left(\frac{\mathrm{DA}}{S}\right)^{0.27} \quad \text{(9)}$$

in which $K = 1.0 + 0.30\, I$ . . . . (10)

The results are shown in Fig. 2 and can readily be compared with previous results. The ratios, future to existing, of $TC + R$ and TC now become

$$\frac{(TC + R)_f}{(TC + R)_e} = \left(\frac{1.0 + 0.30\, I_e}{1.0 + 0.30\, I_f}\right)^{0.56} \quad \text{(11)}$$

and $$\frac{(TC)_f}{(TC)_e} = \left(\frac{1.0 + 0.30\, I_e}{1.0 + 0.30\, I_f}\right)^{0.61} \quad \text{(12)}$$

From Fig. 2 it is apparent that $(TC + R)_f/(TC + R)_e \approx (TC)_f/(TC)_e$ such that a practical method of relating TC and R exists for a particular study area. From Eqs. 8 and 9

$$\frac{TC}{TC + R} = \frac{11.54\, K^{-0.61} \left(\frac{\mathrm{DA}}{S}\right)^{0.27}}{17.01\, K^{-0.56} \left(\frac{\mathrm{DA}}{S}\right)^{0.24}} \quad \text{(13)}$$

or $$\frac{TC}{TC + R} = 0.68\, K^{-0.05} \left(\frac{\mathrm{DA}}{S}\right)^{0.03} \quad \text{(14)}$$

For practical purposes, Eq. 14 becomes

$$\frac{TC}{TC + R} \approx 0.68 \quad \text{(15)}$$

A similar analysis yields

$$\frac{R}{TC + R} \approx 0.32 \quad \text{(16)}$$

Summary and Conclusions

Relationships presented in this paper can be used as a guide to compute the regional unit hydrograph parameters, *TC* and *R*, for existing and predicted values of imperviousness. Modified expressions, such as those developed, are applicable for all values of I and I_e/I_f.

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Time of concentration for overland flow

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Estimating the time of concentration of rainfall within a catchment area, or the critical time of supply, gives the designing engineer the greatest difficulty of all the variable factors in hydrology formulas, such as the rational formula. The method used, even by experienced designers of drainage systems, has been to estimate the time of concentration by a guess. The guess is usually smaller than the actual time of concentration to make sure that the system will be adequate for the runoff.

In the design of facilities with many small drainage areas, such as an urban roadway, it is necessary to estimate the time of concentration for each drainage area. Because the time of concentration has to be computed repeatedly, and because a small variation in this time makes a large difference in the discharge, a more convenient method to determine the time of concentration is needed. Such a method is here presented. It is an expansion of charts by Gail A. Hathaway, Past President ASCE ("Design of Drainage Facilities", ASCE *Transactions*, vol. 110, pp. 697-730, 1945).

The variables needed to compute the time of concentration for a catchment area are its length, slope and surface retardance of flow. All these variables can be computed from the survey field notes normally taken for designing.

The length, L, is the distance from the extremity of the catchment area in a direction parallel to the slope until a defined channel is reached. The units of L are in feet. It is considered that overland flow will become channel flow within 1,200 ft in all cases and less in most cases. If channelized flow occurs in a catchment area, the time of concentration will be the time of overland flow plus the time within the channel.

The slope S is the difference in elevation between the extreme edge of the catchment area and the point in question, divided by the horizontal distance between the two points. The units are in feet per foot.

The retardance coefficient, n, is the average surface retardance value of the overland flow. The following values should be used for computing n:

Type of Surface	Value of n
Smooth impervious surface	0.02
Smooth bare packed soil	0.10
Poor grass, cultivated row crops or moderately rough bare surface	0.20
Pasture or average grass	0.40
Deciduous timberland	0.60
Conifer timberland, deciduous timberland with deep forest litter or dense grass	0.80

As stated by Mr. Hathaway, "The rate of overland flow . . . is a function of the product of nL; hence, any combination of n and L values that gives the same product will result in the same rate of discharge." And "The discharge rate . . . is also a function of the quotient $\frac{S^{0.25}}{L^{0.50}}$." In utilizing these factors, it is found that

$$L'' = \frac{Ln}{4\sqrt{S}} \quad \text{........(1)}$$

when converting the actual length, L, of a catchment area to a length, L'', required to obtain correct discharge by use of his supply curve charts.

In plotting L'' against the time of concentration t, from his supply curve charts, the empirical equation is:

$$L'' = \frac{t^{2.14}}{2.66} \quad \text{........(2)}$$

Combining Eqs. 1 and 2

$$t^{2.14} = \frac{2Ln}{3\sqrt{S}} \quad \text{.......(3)}$$

After computing the retardance coefficient, the slope and the length, the nomograph for Eq. 3, Fig. 1, can be used for estimating the time of concentration for overland flow.

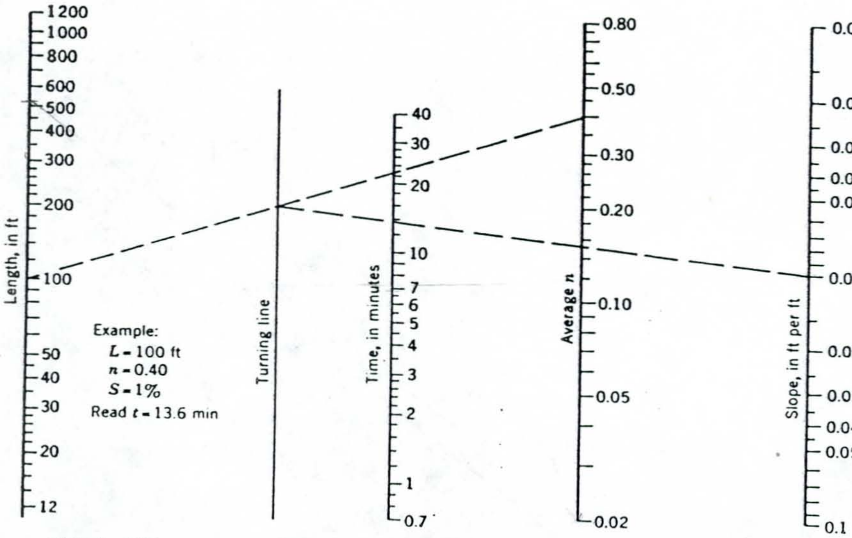

FIG. 1. Nomograph for determining time of concentration for overland flow.

The Effect of Urbanization on Floods of Different Recurrence Interval

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Studies have shown that the urbanization of a catchment can drastically change the flood characteristics of a river. Published results are synthesized to show the general relationship between the increase in flood flows following urbanization and both the percentage of the basin paved and the flood recurrence interval. In general, (1) floods with a return period of a year or longer are not affected by a 5% paving of their catchment, (2) small floods may be increased by 10 times by urbanization, (3) floods with a return period of 100 yr may be doubled in size by a 30% paving of the basin, and (4) the effect of urbanization declines, in relative terms, as flood recurrence intervals increase.

The development of an urban area within a catchment is a drastic change of land use, and it has major effects on the functioning of the hydrological cycle during flood conditions. When large areas of land are rendered impervious by roads, footpaths, roofs, and parking areas, the area in which rainfall can infiltrate into the soil is reduced, depression and interception storage of precipitation may be reduced, and overland flow can take place readily on the relatively smooth impermeable surfaces. The construction of an urban storm water drainage system invariably increases the drainage density of the catchment and so reduces the time necessary for overland flow to reach a drainage line. Moreover, well-designed and well-graded sewer systems are normally efficient channels in which water velocities are usually in excess of those in natural channels; therefore the drainage from a large area can be more rapidly conducted to the main river channel. The net effects of these changes are that a higher proportion of rainfall is translated into runoff, this runoff occurs more quickly, and flood flows are therefore higher and 'flashier' than was the case in the catchment before urbanization. There have been a number of studies that have measured these changes for individual catchments or groups of catchments, and these have been reviewed by *Moore and Morgan* [1969], *Costin and Dooge* [1973], and *Hollis* [1974]. Reports of research at Charlotte, North Carolina [*Martens,* 1968], Sacramento Creek, California [*James,* 1965], and Colma Creek, California [*Yücel,* 1974], have indicated that the effect of urbanization is greatest for small floods and as the size of the flood and its recurrence interval increase, so the effect of urbanization diminishes. The explanation of this finding is that during severe and prolonged rainstorms a rural catchment may become so saturated and its channel network so extended that it responds hydrologically as if it were an impervious catchment with a dense network of surface water drains and so it produces floods of a type and size similar to those of its urban counterpart. Moreover, it seems probable that some throttling of flow takes place in surface water drains during severe storms, which attenuates the very largest discharges. Despite the large number of studies of individual catchments and regions, there have been few attempts to generalize the results of these catchment studies. The *American Society of Civil Engineers Task Force on Effect of Urban Development on Flood Discharges* [1969] report discussed intuitively the hydrological processes modified by urbanization and then listed relevant articles with bibliographical notes and abstracts. This short paper critically examines *Leopold*'s [1968] major synthesis of work in this field, some of

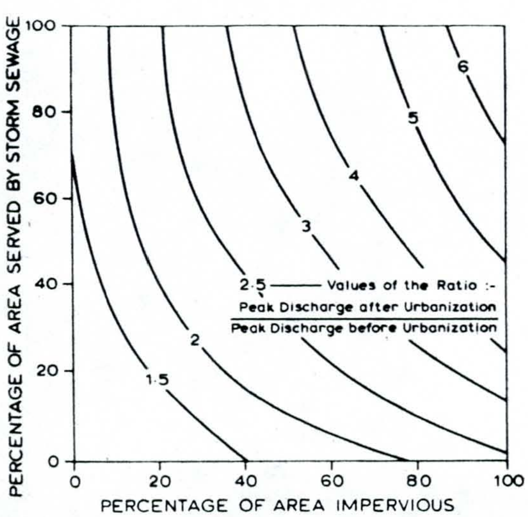

Fig. 1*a*. Effect of urbanization on mean annual flood for a 1-mi² drainage area (after *Leopold* [1968]).

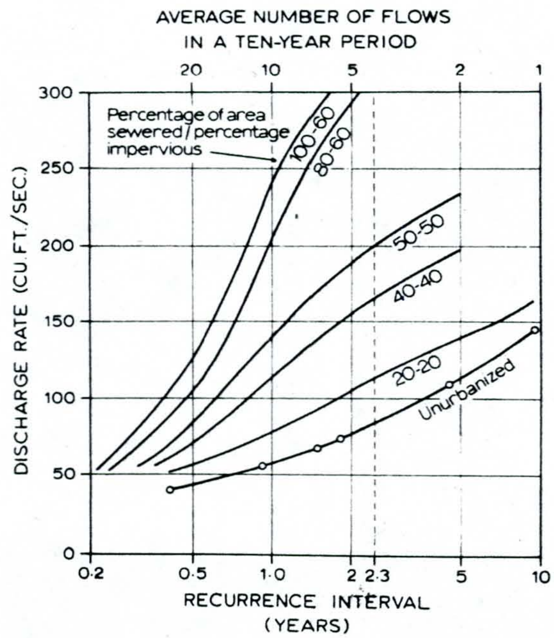

Fig. 1*b*. Flood frequency curves for a 1-mi² basin in various states of urbanization (after *Leopold* [1968]).

TABLE 1. Increase in Flood Discharges After the Urbanization of a Rural Catchment

Reference	Values of the Ratio: Flood Discharge After Urbanization to Flood Discharge Before Urbanization	Flood Recurrence Interval, years	Percentage of Basin Paved
Bigwood and Thomas [1955]			
Basin 1	3	2.33	(20)
Basin 2	3	2.33	(20)
Carter [1961]	1.8	2.33	12
Wiitala [1961]	3.0	2.33	25
James [1965]	1.4	2.33	10[a]
	1.3	5.00	10[a]
	1.2	10.00	10[a]
	1,2	25.00	10[a]
	1.1	100.00	10[a]
	1.1	200.00	10[a]
Crawford and Linsley [1966]	20	0.1[b]	6.7
	13	0.5[b]	6.7
	1.6	3.0[b]	6.7
Espey et al. [1966]			
38th Street	3.2	2.33	21
	5.9	2.33	50
23rd Street	4.4	2.33	27
	6.0	2.33	50
Wilson [1966]	1.9	2.33	9[c]
	2.2	2.33	11[c]
	2.8	2.33	18[c]
	3.6	2.33	27[c]
Anderson [1967][d]	2.86	2.33	20
	2.35	25.00	20
	2.24	50.00	20
	2.20	100.00	20
	3.85	2.33	50
	2.61	25.00	50
	2.36	50.00	50
	2.20	100.00	50
Kinosita and Sonda [1969]	2[e]	(100)[f]	44.3
Curtis, Lee, and Thomas (reported by *American Society of Civil Engineers Task Force on Effect of Urban Development on Flood Discharges* [1969])	1.5	.10	(15)
	1	100(+)	(15)
U.S. Geological Survey study of Little Sugar Creek, North Carolina (reported by *American Society of Civil Engineers Task Force on Effect of Urban Development on Flood Discharges* [1969])	1.6	2.3	15
	1.3	10	15
	1.2	20	15
Shaw and Waller [1973]	10	1(+)	(20)
Hammer [1973]	2.5	1.50	25
	2.2	2.33	25
	2.0	5.00	25
	1.9	10.00	25
	1.8	20.00	25
	1.7	50.00	25
	4.3	1.50	50
	3.5	2.33	50
	3.0	5.00	50
	2.8	10.00	50
	2.6	20.00	50
	2.5	50.00	50
Putnam (cited by *Hammer* [1973])	3.3	1.5	25
	2.9	2.33	25
	2.6	5.00	25
	2.4	10.00	25
	2.2	20.00	25
	2.0	50.00	25
	4.2	1.5	50
	3.7	2.33	50
	3.2	5.00	50
	2.9	10.00	50
	2.6	20.00	50
	2.3	50.00	50
Hollis [1974]	1.0	20.00	16.6

Parentheses indicate figures assumed from qualitative descriptions contained in the papers. A plus in parentheses indicates that the figure is a conservative underestimate.

[a]Watershed condition was 22% urban including 10% of the basin actually paved and 17% of the tributaries improved.

[b]Estimated from gaging station records published by *Crippen and Waananen* [1969].

[c]Only the percentage of the basin with storm sewers and improved channels was given. Following the work of *James* [1965], 0.45 of this area was assumed to be paved.

[d]All the ratios refer to a basin with natural channels, sewered tributaries, and a $L/(s)^{1/2}$ ratio of 1.0, where L is the length of the basin in miles along the main channel and s is the slope in feet per mile between points 10 and 85% of the distance along the main channel in the basin.

[e]Data is taken from the mimeographed paper circulated at the symposium.

[f]The rainfall was the heaviest on record in Tokyo.

the shortcomings of the work are discussed, and Leopold's methodology is extended so as to derive a family of curves for the relationships between the increase in flood peaks following a given amount of urbanization, the percentage of the catchment paved, and the flood recurrence intervals.

Hydrology for Urban Land Planning—A Guidebook on the Hydrologic Effects of Urban Land Use is *Leopold*'s [1968] synthesis and generalization of the studies of *Anderson* [1967], *Martens* [1968], *Wilson* [1966], *Carter* [1961], *James* [1965], *Wiitala* [1961], and *Espey et al.* [1966] into 'terms that the planner can use to test alternatives during the planning process' [*Leopold,* 1968, p. 1]. Two of the main graphs from Leopold's work are shown in Figure 1, but some of the data used by Leopold are projections by other authors beyond the observed empirical evidence. Figure 1*a* relates the percentage of the catchment paved and sewered to the posturbanization increase in the rural mean annual flood from a 1-mi² basin. This is valuable in that it depicts urban-induced increases in floods of between 1.5 and 6 times and so dispels the idea of a single ratio which is 'the effect of urbanization.' However, the figure is of limited value for the urban planner because it cannot be used directly for a catchment of more than 1 mi² nor for a flood with a long return period, and therefore it may be of little help in the evaluation of flood alleviation schemes.

Figure 1*b* is also a useful and bold synthesis 'based on minimal data and [requiring] corroboration or revision as additional field data become available' [*Leopold,* 1968, p. 10]. In a planning situation it is deficient in several respects. First, it does not include extreme floods that cause significant damage. Second, *Leopold*'s [1968, p. 10] view that 'the curves representing urbanized conditions . . . converge at low flow values . . . [because] . . . the most frequent flows are decreased in number because low flows in an urban area are not sustained by ground water as in a natural basin' must be reviewed in the light of published research. Natural base flows undoubtedly fall as a result of reduced groundwater levels following reduced percolation after urbanization. *Sawyer* [1963], for instance, showed a 2% reduction in base flow in a Long Island stream following development, and *James* [1965] found that base flow in Morrison Creek, California, fell by 30% after the paving of 45% of the catchment. However, the actual low flows and modal flows in a river may not exhibit a similar decline. *Crippen and Waananen* [1969] found that the Sharon Creek near San Francisco changed after urban development from an ephemeral stream with a flow on only 20 d/yr into a perennial stream. As a result of the paving of 16.6% of the catchment of the Canon's Brook, Harlow, England, the modal flow has risen from 2 to 5 ft³/s, and the frequency of floods in the range 40–100 ft³/s has increased from around 8 to 27 per annum. The reason for these increases in the frequency and magnitude of small floods is that paved surfaces produce runoff and a rise in streamflow from very modest rainstorms which would be completely absorbed by soil storage if they were to occur over a rural catchment. This evidence contradicts Leopold's opinion that 'the most frequent flows are . . . increased by smaller ratios than would be the mean annual flood' and supports the finding of *Hammer* [1973] that 'the effect of urbanization on peak discharges decreases with . . . [an increase in] . . . recurrence interval in relative terms.' A divergence of the lines at low flows in Figure 1*b* would reflect this conclusion.

The most valuable aspect of *Leopold*'s [1968] paper is the synthesis of results from a large number of studies and reports to produce a series of diagrams for practical application. Following this lead, the salient data from relevant articles on the effect of urbanization on floods have been collated in Table 1 in an attempt to express the ratio of flood peak after urbanization to that before urbanization as a function of the percentage of the catchment paved and the return period of the flood. As far as possible only empirically derived figures have been included, and where it was necessary to make estimates, comments have been appended. When the value of the ratio of peak discharge after urbanization to that before urbanization from Table 1 is plotted against the percentage of the catchment paved and the recurrence interval of the rural flood, the result (Figure 2) is in accord with the suggestion made earlier that the effect of urbanization on floods declines as flood magnitudes increase because of the reduced importance of interception, depression storage, and infiltration in a rural catchment undergoing a very severe rainstorm. Lines representing the extent of the increase in flood flows after urbanization have been fitted by eye to the data in Figure 2, and there is general agreement between the lines and the points. Deviations from the interpolated lines were inevitable because the catchments vary in size, morphology, and geology, and they are in widely differing climatic environments from the semiarid parts of the United States to Japan and western Britain. The variations in

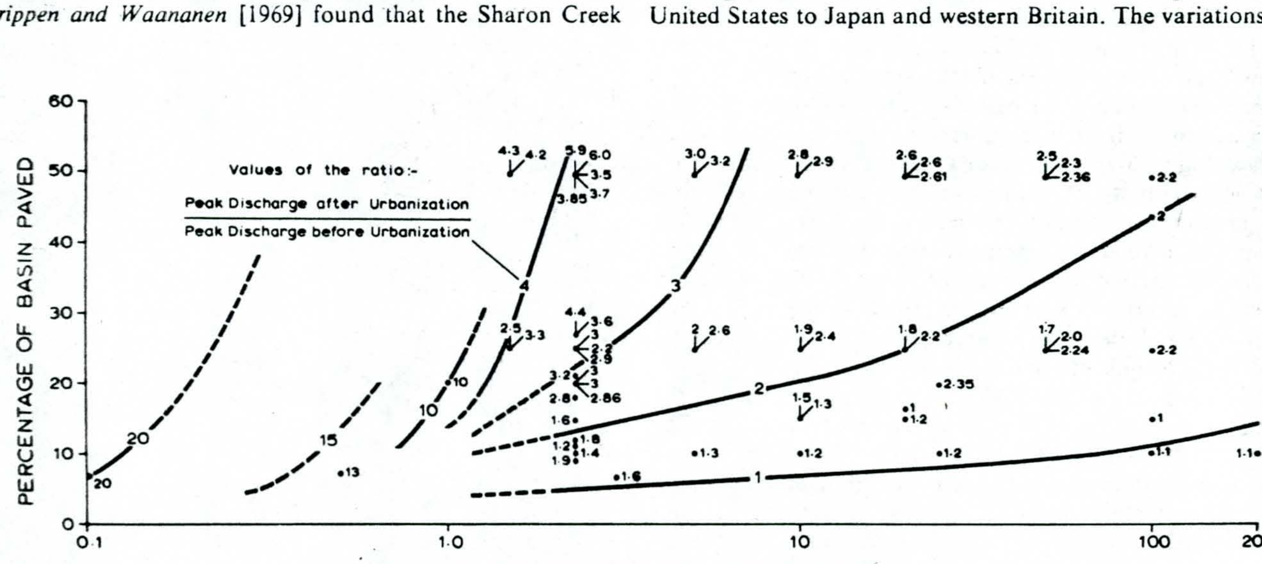

Fig. 2. Effect of urbanization on flood peaks. (Data taken from Table 1.)

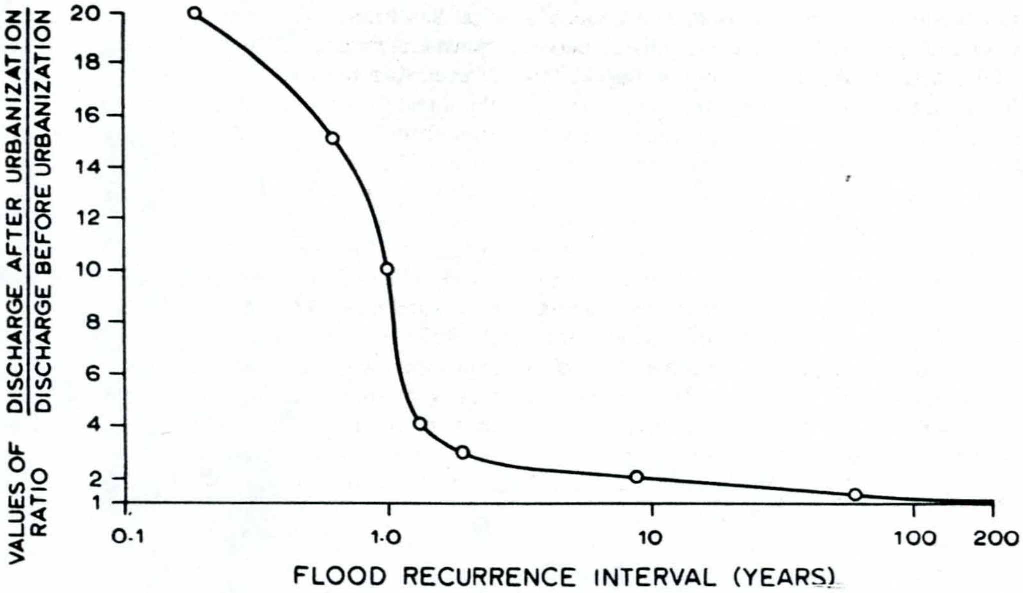

Fig. 3. Effect on flood magnitudes of paving 20% of a basin (based on Figure 2).

geology are probably of considerable significance, for the imposition of paved surfaces on a permeable catchment will cause much greater changes than a similar type of development on a catchment which is already impermeable by virtue of its lithology. Moreover, the type and degree of urbanization vary between catchments in a way that cannot be fully expressed by a simple measure such as 'percentage of the catchment paved.' The position of the development in the catchment [*Yücel*, 1974] and the degree of improvement of the drainage network [*Espey et al.*, 1966] have been shown to be particularly important additional measures of urbanization. As more results are published, it should be possible to extend this type of synthesis to include other important explanatory variables and so make the results more directly applicable to individual catchments.

In spite of these reservations, Figure 2 suggests that (1) floods with a return period of 1 yr or more are not appreciably affected by a 5% paving of their catchment area, (2) small floods may be increased by a factor of 10 or more depending upon the degree of urbanization, (3) floods with a return period of 100 yr may be doubled in size by the complete urbanization of a catchment if that urbanization results in at least a 30% paving of the basin, and (4) the effect of urbanization declines in relative terms as flood recurrence intervals increase (Figure 3). Figure 3 shows that the relative increase, but not necessarily the absolute increase, in floods brought about by urbanization declines as recurrence intervals lengthen. Moreover, a flood with a return period of around 150 yr is not materially affected by urban development. In view of the considerable uncertainties involved in estimating the magnitude of a 150-yr flood, the effect of urbanization is so small as to be within the bounds of the error in prediction for such a flood.

The ideas of *Leopold* [1968] have been developed in this paper, and it has been found that urbanization does not affect floods of different recurrence intervals to the same extent. Whilst small frequent floods are increased many times by urbanization, large rare floods that are likely to cause severe damage are not significantly affected by the construction of urban areas within a catchment area.

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(Received July 16, 1974;
accepted December 3, 1974.)

Hydrological Sciences—Bulletin—des Sciences Hydrologiques, XIX, 2 6/1974

INSTANTANEOUS UNIT HYDROGRAPHS, PEAK DISCHARGES AND TIME LAGS IN URBAN BASINS

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Revised MS. received 26 November 1973

ABSTRACT

The effects of urbanization of a basin on the runoff have been investigated in the past by the use of linear conceptual models in which the time lag appears as an important parameter. However, in this approach the effects of noise in the data, of sampling rate, of errors due to the lack of synchronization between the effective rainfall and runoff on the instantaneous unit hydrograph do not become readily apparent. A case in which the cumulative effects of these factors are predominant is presented as an example of the possible difficulties which might be encountered in the analysis of urban hydrological data by the unit hydrograph method. The disadvantages of relating the peak discharge, the time to peak discharge and the time lag to the physiographic characteristics alone have been discussed. Alternative regression relationships which involve storm characteristics along with the physiographic characteristics to estimate the peak discharge, time to peak discharge and time lag have been presented.

RÉSUMÉ

On a étudié jadis les effets de l'urbanisation des bassins sur le ruissellement au moyen de modèles conceptuels linéaires. Néanmoins, dans cette méthode, les effets du bruit dans les données, de l'intervale de discrétisation et des erreurs dues au manque de synchronisation entre les mesures des pluies effectives et du ruissellement ne sont pas apparents. Le cas où les effets cumulatifs de ces facteurs deviennent prédominants est présenté comme example des difficultés que l'on peut rencontrer dans l'analyse de données hydrologiques urbaines par la méthode de l'hydrogramme unitaire. On a discuté les inconvénients des relations de corrélation entre le débit maximum, le temps de montée, le temps de réponse comme variables indépendantes et les caractéristiques physiographiques du bassin comme variables dépendantes. Les relations proposées incluent aussi des caractéristiques météorologiques comme variables dépendantes.

INTRODUCTION AND PROBLEM STATEMENT

The effects of urbanization on the hydrology of basins are being investigated by using several different techniques: the choice of technique being dictated in part by the problem to be solved. A review of literature indicates that the effects of urbanization have been investigated (1) by using unit hydrograph methods, (2) by the use of 'structure imitating' or 'component models' such as the Stanford watershed model, (3) by analysing the magnitudes and times to peak flows, (4) by analysing the time lag variations in basins of different degrees of urbanization and finally (5) by analysis of frequencies of annual maximum floods. By their very nature, the methods of analysis outlined in (1) and (2) above are more general than the other methods. The present paper deals with some aspects of investigating the effects of urbanization on run-off by using the instantaneous unit hydrograph (IUH), and also of analysis of magnitudes and time to peak of annual maximum floods. Problems which may arise in using the average time lag as an indicator of the effects of urbanization are also discussed.

Data Used in the Analysis and their Treatment

Rainfall and Runoff Data

Data from eight urban and five rural basins in Indiana and in Texas were used in the study. The locations of the basins and some of their physiographic characteristics are shown in Table 1. For some of these basins the physiographic characteristics such as area, stream slopes, etc., were obtained from topographic maps of the US Geological Survey (USGS) or they were available from the USGS Offices. The percentage of impervious areas for the Indiana basins were estimated from aerial photographs of the basins by a sampling method. The establishment of two gauging stations in the Purdue Swine Farm basin and the corresponding data collection were started as a part of this study (Sarma *et al.*, 1969).

TABLE 1

Physiographic characteristics of the basins

Number	Name	Area A (mi^2)	Length of stream L (mi)	Mean basin slope S (ft/mi)	Percentage impervious area U (%)	No. of storms used for analysis	Average time lag $\bar{T}_4$ (h)
1	Ross Ade (upper)[a]	0·0455	0·6133	112·0	38.0	21	0·21
2	Ross Ade (lower)[a]	0·6125	2·1765	112·0	37.4	10	0·61
3	Purdue Swine Farm (upper)[a]	0·2776	0·6439	3·9	21·3	4	0·300
4	Purdue Swine Farm (lower)[a]	0·4562	1.067	3·40	13·3	2	0·400
5	Pleasant Run (Arlington)[b]	7·580	3·822	14·26	10·5	13	3·57
6	Pleasant Run (Brookville)[b]	10·10	5·644	12·67	15·5	12	3·72
7	Little Eagle Creek[b]	19·31	11·10	23·23	2·1	8	9·34
8	Lawrence Creek[b]	2·86	1·705	32·21	0	12	2·05
9	Bear Creek[c]	7·00	3·864	39·07	0	5	5·44
10	Bean Blossom Creek[c]	14·60	6·44	32·74	0	7	6·56
11	Waller Creek (38th Street)[d]	2·31	4·371	47·0	27·0	17	1·44
12	Waller Creek (23rd Street)[d]	4·13	5·23	47·0	37·0	12	1·58
13	Wilbarger Creek[d]	4·61	3·17	45·94	0	8	2·89

a: West Lafayette, Indiana; b: Indianapolis, Indiana; c: Indiana; d: Austin, Texas.

For all basins located outside West Lafayette, Indiana, the total runoff values were obtained from the USGS gauging station stage records and the corresponding stage–discharge curves. The rainfall values were obtained from the ESSA records for raingauges located in the basin itself or from the records of the raingauge located nearest to the basin. All the data from the West Lafayette basins were acquired as a part of this study.

Base Flow Separation and Determination of Excess Rainfall

In a hydrograph, let A and B respectively represent the points at which the direct runoff begins and ends (Fig. 1). The point A is easily located in isolated storms and the value of the maximum discharge Q_m is determined by drawing a horizontal line through the point A. The point B is then located on the recession limb of the hydrograph such that the discharge at B in excess of that at A, Q_B, is one hundredth of the peak discharge Q_m. The points A and B are joined by a straight line which represents the base flow separation line. The ordinates of the direct runoff hydrograph are then obtained by subtracting the ordinates below the straight line AB from the corresponding ordinates of the total runoff hydrograph. The peak of the direct runoff hydrograph is designated by Q_p.

The excess rainfall was estimated by assigning the rainfall which occurred before the time of commencement of direct runoff to be the initial abstraction. The beginning time or rise of runoff was taken as the beginning time of excess rainfall. The ordinates of the excess rainfall hyetograph were obtained by multiplying the corresponding ordinates of the total rainfall hyetograph by the ratio of the volume of direct runoff to the volume of the total rainfall. Thus, the time distributions of the excess rainfall and of the total rainfall are geometrically similar after the time of rise of hydrograph (Fig. 1).

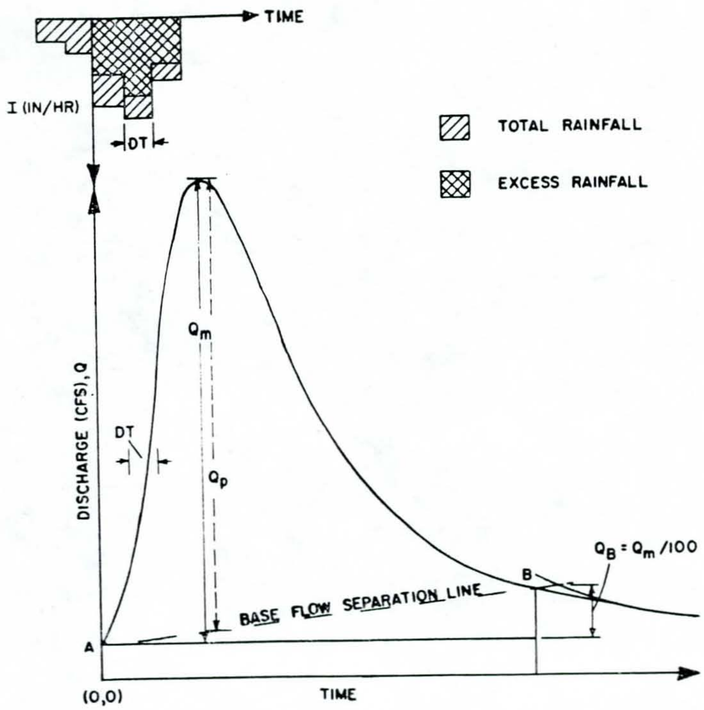

Fig. 1 — Baseflow separation and determination of excess rainfall distribution.

The Determination of the Instantaneous Unit Hydrographs for Urban Basins

Conceptual models such as the Nash model or the single linear reservoir model have been used to simulate the effective rainfall–direct runoff process in urban basins (Viessman, 1968; Willeke, 1962; Sarma *et al.*, 1973) and in the study of effects of urbanization on basin hydrology (Rao *et al.*, 1972). However, the IUH for the effective rainfall–direct runoff process can be obtained by the use of Fourier transforms (Blank *et al.*, 1971). The advantages of computation of the IUH by the Fourier or other transforms are, (1) the usual conceptualizations about the system (for example considering it to be a series of reservoirs and channels) are eliminated, and (2) the behaviour of the system and the effects of noise in input and output can be determined by an examination and analysis of the IUH (Rao and Delleur, 1971). The method of determination of the IUH by using the Fourier transforms as well as the details of estimation of the IUH by using transforms such as the Z-transform are available elsewhere (Blank *et al.*, 1971; Delleur and Rao, 1971a, b). The convolution integral [equation (1)] is the basic model assumed in these analyses:

$$Y(t) = \int_0^t x(\tau)\, h(t-\tau)\, \mathrm{d}\tau \qquad (1)$$

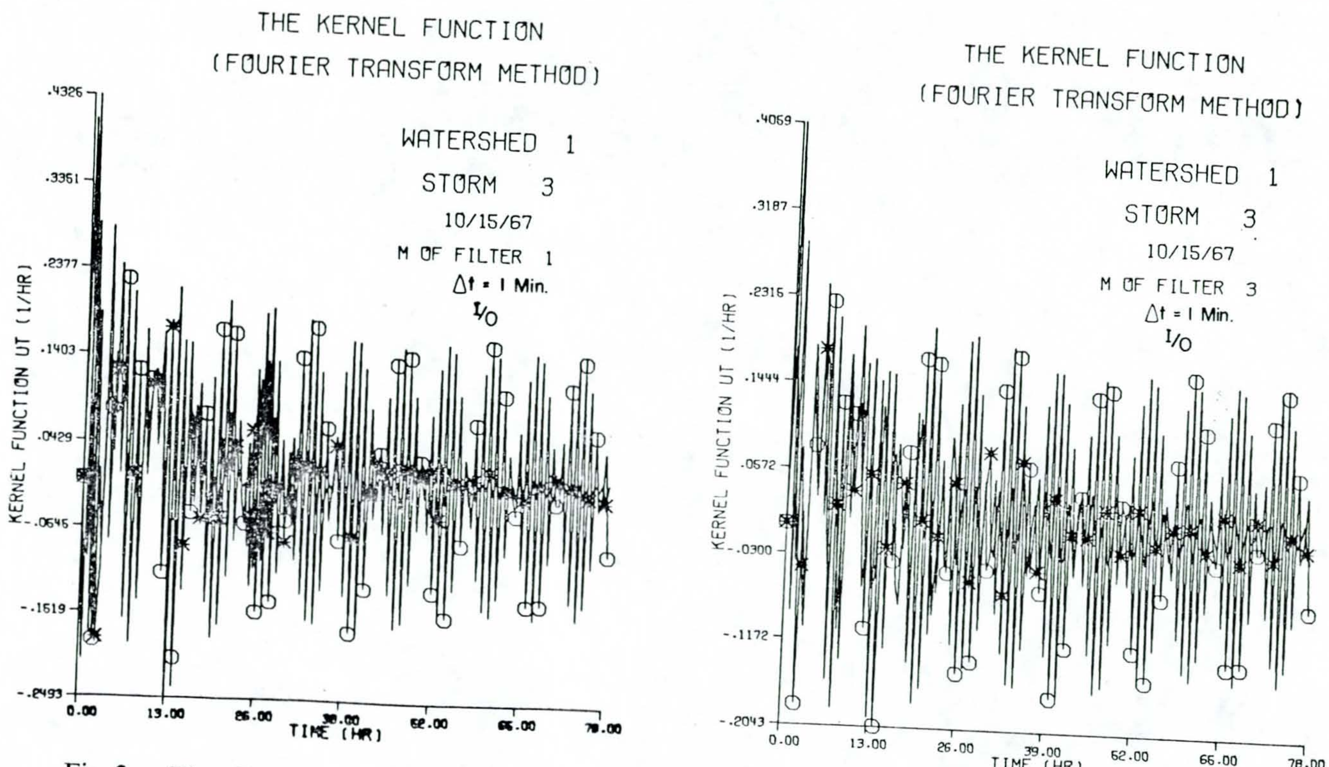

Fig. 2 — The effect of changing the discretization rate and of filtering on the IUH. Circles and stars refer to IUH from unfiltered and filtered input and output data. MJF was used for filtering.

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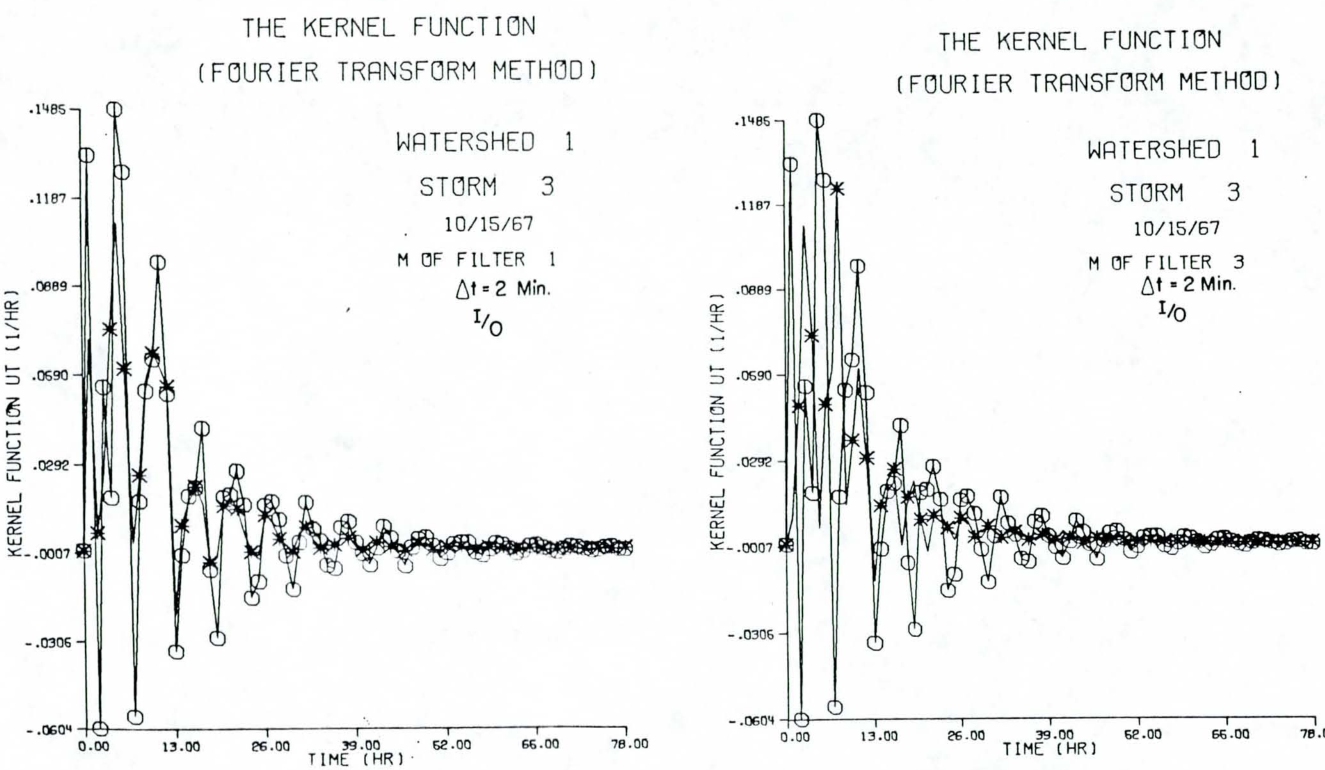

Fig. 3 — The effect of changing the discretization rate and of filtering on the IUH. Circles and stars refer to IUH from unfiltered and filtered input and output data. MJF was used for filtering.

In equation (1), $Y(t)$ is the direct runoff, $x(t)$ is the rainfall excess, and $h(t)$ is the impulse response or the IUH.

The instantaneous unit hydrographs computed by using the Fourier or other transform and the convolution integral, in which only the linearity between the input and output is assumed, can be shown to be similar to the exponentially decaying IUH obtained by the single linear reservoir model, for partly impervious small basins of areas less than about 5 mi^2 (Rao *et al.*, 1972). This aspect strengthens the case for use of a simple model such as the single linear reservoir model for the analysis of data from small basins, although the use of a single value of reservoir coefficient, which is supposed to represent the basin's response, yields erroneous results.

If the input and output are corrupted by noise, then the IUH obtained by the transform method may be highly oscillatory as shown in Fig. 2. This oscillatory behaviour may be due to (1) erroneous synchronization of rainfall and runoff data in time, (2) the errors introduced in the measurement itself and (3) inappropriate discretization rate. If the sampling interval is increased, then the high frequency oscillations observed in the IUH's shown in Fig. 2 are replaced by the low frequency oscillations of Fig. 3. Further increases in the sampling rate will further reduce the oscillations in the IUH but the resolution of the IUH will also steadily diminish.

These high frequency oscillations of the IUH may be eliminated by using low pass filters. For example, a low pass filter which appears to be suitable is the modified Jenkins filter (MJF) whose Fourier transform is given in equation (2).

$$w'(f) = w_0 + 2 \sum_{i=1}^{M} (-1)^i \, w_i \cos 2\pi f i \Delta t \tag{2}$$

In equation (2), M is the order (length) of the filter, f is the frequency and w_i are the weights given by equations (3) and (4):

$$w_0 = 1 - \frac{1}{M+1} \tag{3}$$

$$w_i = w_{-i} = -\frac{1}{M+1}\left(\tfrac{1}{2} + \tfrac{1}{2}\cos\frac{i\pi}{M+1}\right) \quad i = 1, \ldots, M \tag{4}$$

The transforms of the input and output functions can be filtered separately to eliminate the effects of noise in the data and the filtered inputs and outputs may then be used to obtain the response functions. Although no rules can be given to specify the length of the filter, M, usually a filter of length M less than 3 is sufficient to eliminate most of the effects of noise. As filtering the data usually entails loss of data (if M is the length of filter, then $2M$ data points are lost by filtering, if MJF is used) increasing the filter length beyond a 'minimum' length is not advisable. The IUH obtained after filtering the input and output data by using filters of lengths M equal to 1 and 3 are also shown in Figs. 2 and 3. As it can be seen, the filter of length $M = 3$ is not more effective than the filter of length M equal to 1.

The foregoing discussion is primarily intended to indicate the instabilities which may arise in the analysis of data from small urban basins. The sampling intervals used in the measurement of runoff from these basins must necessarily be small, of the order of a few minutes. Otherwise it would be impossible to measure these events meaningfully. If the rainfall and runoff data measured at these small time intervals are not synchronized properly, or if they are corrupted by noise then the IUH computed by using such data might show some oscillations. Similar comments are valid for the unit hydrographs computed by the matrix inversion methods (Newton and Vinyard, 1967).

These observations will not be apparent if conceptual models such as the Nash model are used for analysis of effects of urbanization on runoff (Rao *et al.*, 1972) although in such cases the parameters of the conceptual models do reflect the effects of noise corrupted data.

The IUH provides an effective tool for estimating the direct runoff hydrographs from given rainfall excess hyetographs. Often, the part of the runoff hydrograph of most interest is the peak discharge and its time of occurrence. These quantities are of particular importance in the design of urban drainage. It may, therefore, be of practical interest to develop relations to estimate directly the peak discharge and the time to peak in terms of a measure of the amount of urban development and other basin and storm characteristics. This is the objective of the results presented in the remainder of the paper.

Peak Flows and Time to Peak Flows

The facts that the magnitudes of peak flows are increased and the times to peak flows are reduced for the more urban basins are well known. The diminished infiltration and the improved conveyance provided in urban basins have been recognized as the basic reasons for this effect. Carter (1961) in his study of changes in magnitude and frequency of peak floods caused by urbanization assumed that the average rainfall–runoff coefficient of 0·3, which was obtained by rainfall–flood volume studies for basins near Washington, D.C. was applicable to peak floods also. Furthermore, the effect of changes in the impervious areas was assumed to be independent of the size of the flood and that 75 per cent of the volume of rain falling on the impervious surfaces reaches the stream. Based on these assumptions, Carter related the annual peak flood $\bar{Q}$ to the area of the basin, A in square miles, and to the average time lag $\bar{T}_4$ in hours, (see Table 2 and Fig. 6) by

$$\bar{Q} = \left(\frac{0{\cdot}30 + 0{\cdot}0045\,U}{0{\cdot}30}\right) a_0\, A_1^{a_1}\, \bar{T}_4^{a_2} \tag{5}$$

where U is the percentage of impervious area and a_0, a_1 and a_2 are regression coefficients. Carter concluded that the effects of sewer construction, channel improvement, and other features were more significant than the effects of changes in percentage of impervious areas. In a similar analysis, Anderson (1968) has presented charts to estimate the magnitudes of the peak floods with recurrence intervals up to 100 years for basins with varying degrees of urban or suburban development.

Stall and Smith (1961) compared the unit hydrographs for two basins in Champaign, Illinois, one of which was completely rural and the other urban with 38·1 per cent of impervious area. Although the mean basin slopes, stream channel shapes and areas of both basins were generally similar, the unit hydrograph peak discharge for the urban basin was

TABLE 2

Definitions of time lag

Symbol	Definition*
T_1	Time from centroid of excess rainfall to the peak of direct runoff hydrograph.
T_2	Time from beginning of continuous excess rainfall to the peak of direct runoff hydrograph.
T_3	Time from beginning of continuous excess rainfall to the centroid of direct runoff hydrograph.
T_4	Time from the centroid of excess rainfall to the centroid of direct runoff hydrograph.
T_5	Time from the centroid of excess rainfall to the mid-volume of direct runoff.

* See Fig. 6.

about four times that for the rural one. In another study of the effects of urbanization on runoff which was based on unit hydrographs, Espey *et al.* (1966) concluded that the peak discharge from urban basins was about 50 per cent higher than the peak discharge from rural ones.

The effect of urbanization on the peak discharge depends not only on basin area, and on the percentage of impervious area, as previous authors have found, but most importantly on the magnitude and the duration of the rainfall. Although in some of the previous work, this dependence of peak flow on rainfall characteristics has been implicitly assumed, it has not been explicitly considered. Consequently, an equation of the form

$$Q_p = c_0 A^{c_1}(1+U)^{c_2} P_E^{c_3} T_R^{c_4} \tag{6}$$

was fitted by multiple regression analysis to the data of peak discharge of direct runoff for the basins listed in Table 1. In equation (6), Q_p is the peak direct runoff in cusecs, U is the fraction of impervious area of the basin, P_E is the magnitude of effective precipitation in inches, T_R is the duration of effective precipitation in hours and c_1, c_2, etc. are constants. The equation which resulted from this analysis is given below:

$$Q_p = 484{\cdot}1\, A^{0{\cdot}723}(1+U)^{1{\cdot}516}\, P_E^{1{\cdot}113}\, T_R^{-0{\cdot}403} \tag{7}$$

The correlation coefficient, the standard error of estimate and the coefficient of determination for equation (7) are respectively 0·9844, 0·4025 cusecs and 0·969. It is interesting to note that the peak discharge is affected most by the urbanization factor. The magnitude of effective rainfall and its duration also significantly affect the peak discharge.

Figure 4 shows the observed peak flow values plotted against the corresponding values computed by equation (7). The 90 per cent confidence limits are also shown. The values computed by equation (7) are seen to be in good agreement with the observed values.

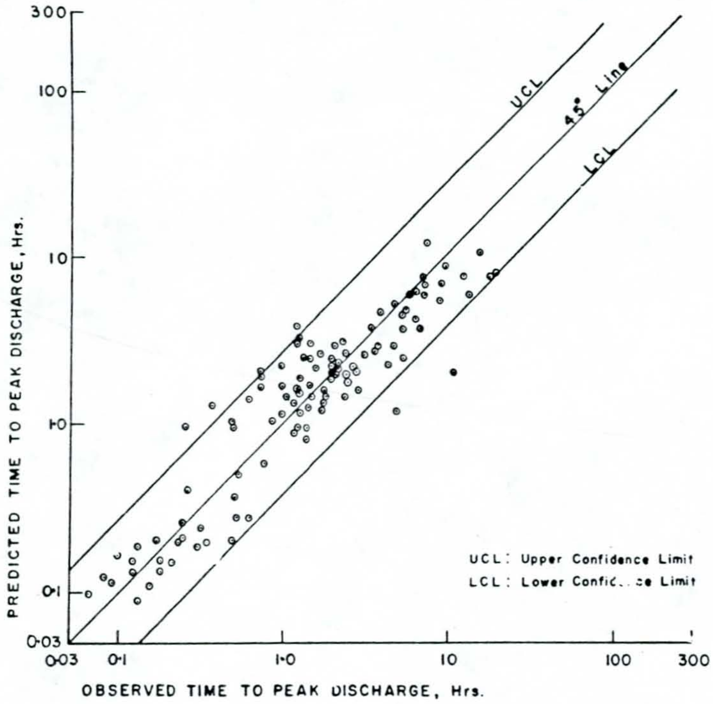

Fig. 4 — Observed and predicted peak discharge values.

By a similar regression analysis, the time to peak was related to A, $(I+U)$, P_E and T_R to obtain

$$T_p = 0{\cdot}775\, A^{0{\cdot}323}(1+U)^{-1{\cdot}285}\, P_E^{-0{\cdot}195}\, T_R^{0{\cdot}634} \tag{8}$$

The correlation coefficient, standard error of estimate and the coefficient of determination for equation (8) are respectively 0·93, 0·509 (h) and 0·865. The time to peak discharge increases with the area of the basin and the duration of effective rainfall whereas it decreases with the urbanization factor and the magnitude of effective rainfall. Of the four variables in equation (8), the urbanization factor and the duration of rainfall excess have a much stronger effect on time to peak than the other two variables. The values computed by equation (8) are plotted against the corresponding observed values in Fig. 5 along with the 90 per cent confidence limits. The agreement between the observed and computed values is good.

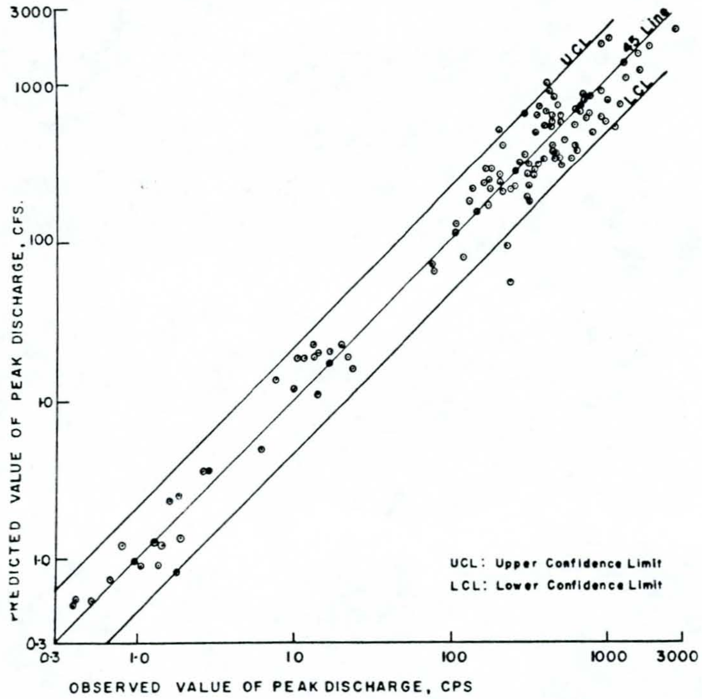

Fig. 5 — Observed and predicted times to peak discharge.

Time Lag

Various definitions of the time lag are used by different investigators. These definitions are summarized in Table 2 and are schematically represented in Fig. 6.

Carter (1961) related the average time lag, $\overline{T}_4$ (h), to the length of the main stream, L (mi), and to the weighted slope of the main stream, s_w, by the regression relationship given in equation (9), in which a_0 and a_1 are regression coefficients:

$$\overline{T}_4 = a_0(L/\sqrt{s_w})^{a_1} \tag{9}$$

The average time lag for sewered catchments was shown by Carter to be less than the average time lag for unsewered catchments. In comparing his results with those of Snyder (1958), Carter observed that the time of concentration T_c in Snyder's study was expressed by equation (9) and that the coefficients a_0 and a_1 had different values, although the values of the exponent

a_1 were almost equal. It must be noted that Snyder's analysis was conducted with data from fully sewered catchments whereas Carter's analysis was based on partially sewered and unsewered catchments. Carter concluded that the average time lag values of the partially sewered basins and completely sewered basins were 66 per cent and 85 per cent less than the time lag values for natural basins. Wiitala (1961) applied equation (9) for two basins, one of which was in a rural condition and the other was completely sewered with about 25 per cent impervious cover. The value of average time lag for the urban basin was found by Wiitala to be about 70 per cent less than the average time lag for the rural one.

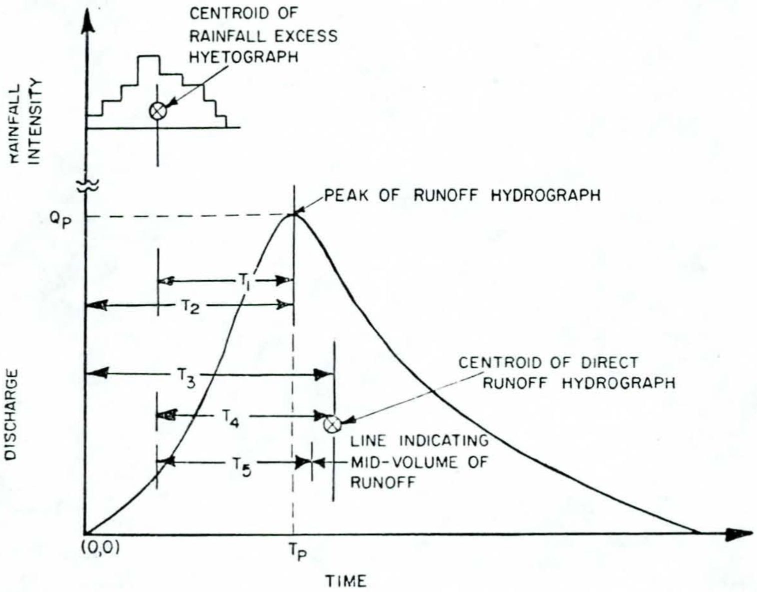

Fig. 6 — Different definitions of time lag.

Linsley *et al.* (1958) related the average time lag $\bar{T}_3$ to the length of the main stream L, the distance along the main stream from the basin outlet to a point opposite the centre of gravity of the basin in miles, and $\bar{S}_0$ the mean basin slope by equation (10), in which a_0 and a_1 are regression coefficients obtained from an analysis of nonurbanized basins in California:

$$\bar{T}_3 = a_0 \left(\frac{L L_{ca}}{\sqrt{\bar{S}}} \right)^{a_1} \tag{10}$$

In his study of unit hydrographs from urban basins Eagleson (1962) compared the values of average time lag $\bar{T}_1$ of urban basins with the results presented by Linsley *et al.* and found that the average time lag from urban basins was *less* than the time lag values obtained from an analysis of data from mountainous foothill and valley areas. Many other studies, among which those of Van Sickle (1962), Espey *et al.* (1966), Espey (1968) include the time lag as an important parameter.

Although the average time lag value has been used in several previous studies, its use in runoff prediction from urban basins can lead to erroneous results. The dependence of average time lag value on various physiographic characteristics cannot be clearly brought out by regression analysis although several previous investigators have used regression analysis for this purpose. To illustrate the points mentioned above equation (11) was fitted by multiple regression analysis to the data from the basins listed in Table 1:

$$\bar{T}_4 = c_0 A^{c_1} L^{c_2} \bar{S}^{c_3} (1+U)^{c_4} \tag{11}$$

Some of the values of $\bar{T}_4$ are shown in Table 1 and others can be found in Sarma *et al.* (1969). Equations (12), (13), and (14) in Table 3 represent the expressions for $\bar{T}_4$ obtained by regression analysis with gradual deletion of the variables which were not significant at the 1 per cent level. The linear correlation coefficients and other measures associated with equations (12)–(14) are also presented in Table 3.

TABLE 3

Results of multiple correlation between $\bar{T}_4$ and A, $(I+U)$, L, S

Equation	Equation No.	Correlation coefficient R	Standard error of estimate (h)	Coefficient of determination (R^2)
$\bar{T}_4 = 0{\cdot}78\ A^{0{\cdot}496} L^{0{\cdot}073} S^{-(0{\cdot}075)} (1+U)^{-1{\cdot}289}$	(12)	0·930	0·481	0·864
$\bar{T}_4 = 0{\cdot}780\ A^{0{\cdot}542} S^{-(0{\cdot}081)} (1+U)^{-1{\cdot}210}$	(13)	0·929	0·471	0·864
$\bar{T}_4 = 0{\cdot}803\ A^{0{\cdot}512} (1+U)^{-1{\cdot}433}$	(14)	0·927	0·469	0·859

An inspection of equations (12)–(14) reveals the fact that a regression relationship involving only the area of the basin and the urbanization factor is as effective as a relationship which includes the length of the stream and the mean basin slope. This is not surprising as basin area and stream length are highly correlated and the man-made changes render the stream channel slope to be meaningless in many sewered basins, as the sewers are usually designed for an approximately standard flow velocity.

The single linear reservoir theory (Chow, 1964) indicates that the reservoir constant K should be equal to the time lag, $\bar{T}_4$, and that the IUH corresponding to this conceptual model is given by

$$h(t) = \frac{1}{K}\,\mathrm{e}^{-t/K} \tag{15}$$

The results of regenerating the direct runoff hydrograph by using the average time lag $\bar{T}_4$ $(=K)$, for a storm on basin 1 are shown in Fig. 7a. The regeneration made use of equations (15) and (1), and the observed rainfall excess. The storage constant was obtained from equation (12) (method 1) or was assumed to be equal to the average time lag for the 21 storms on basin 1. Figure 7a shows that taking the storage constant equal to the average time lag did not result in even a good regeneration. Other results similar to that presented in Fig. 7a by Sarma *et al.* (1969) also indicate that the prediction using equation (12) for the estimation of the average time lag was unsatisfactory.

The poor regeneration performance obtained when the average time lag, $\bar{T}_4$, was used led to an investigation of the variation of T_4 with the meteorological characteristics as well as the physiographic characteristics. Regression relationships in the form of equations (16) and (17) were formulated and the data presented in Table 1 were used to obtain equations (18) and (19) shown in Table 4.

$$T_4 = c_0 A^{c_1}(1+U)^{c_2} P_E^{c_3} T_R^{c_4} \tag{16}$$

$$T_4 = c_0 L^{c_1}(1+U)^{c_1} P_E^{c_3} T_R^{c_4} \tag{17}$$

A better prediction could be obtained using equation (18) or (19) for T_4 and by using this value for the storage constant K, since it accounts for the variation of the time lag with the storm characteristics. Nevertheless, it was found that the regeneration could be improved further by making K slightly different from T_4. Two values of K are proposed: K_1 which yields a

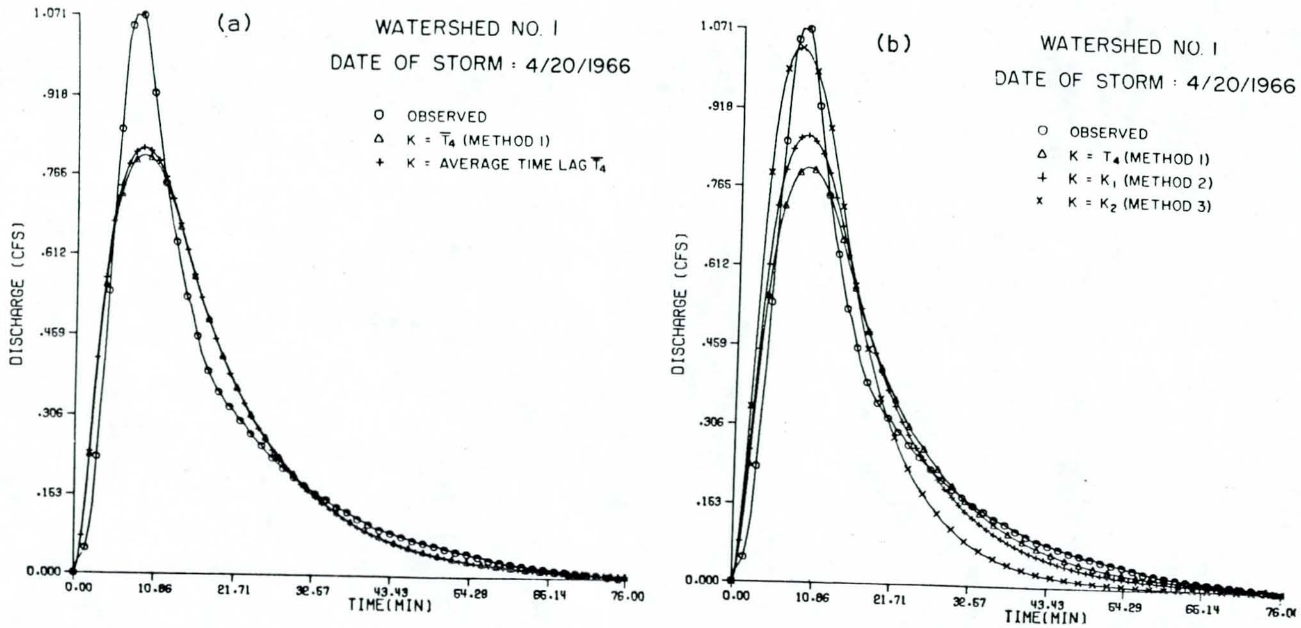

Fig. 7 — (a) Regeneration performance of the single linear reservoir model with the reservoir constant K being equal to the average time lag $\overline{T}_4$ given by equation (12). (b) Regeneration performance of the single linear reservoir model with K being equal to T_4 [equation (18)], K_1 [equation (21)], and K_2 [equation (22)].

computed hydrograph so that the sum of the squares of the deviations between the observed and regenerated hydrograph ordinates is a minimum, and K_2 which minimizes the deviation between the peak values and between the time of peaks for the observed and regenerated hydrographs as given by Z in equation (20), where

$$Z = \left[\left(\frac{Q_{po} - Q_{pc}}{Q_{po}}\right)^2 + \left(\frac{T_{po} - T_{pc}}{T_{po}}\right)^2\right]^{1/2} \tag{20}$$

Q_{po} and Q_{pc} are respectively the magnitudes of the peak discharges of the observed and of the regenerated hydrographs whereas T_{po} and T_{pc} are the times corresponding to Q_{po} and Q_{pc}.

TABLE 4

Results of multiple correlation between T_4, K_1, and K_2 with other variables

Equation	Equation No.	Correlation coefficient R	Standard error of estimate (h)	Coefficient of determination (R^2)
$T_4 = 0{\cdot}831\, A^{0{\cdot}458} (1+U)^{-1{\cdot}66}\, P_E^{0{\cdot}267}\, T_R^{0{\cdot}371}$	(18)	0·923	0·506	0·851
$T_4 = 0{\cdot}731\, L^{0{\cdot}943} (1+U)^{-4{\cdot}303}\, P_E^{-2{\cdot}114}\, T_R^{0{\cdot}238}$	(19)	0·933	0·473	0·781
$K_1 = 0{\cdot}887\, A^{0{\cdot}49} (1+U)^{-1{\cdot}683}\, P_E^{-0{\cdot}24}\, T_R^{0{\cdot}294}$	(21)	0·909	0·555	0·827
$K_2 = 0{\cdot}788\, A^{0{\cdot}409} (1+U)^{-2{\cdot}06}\, P_E^{-0{\cdot}15}\, T_R^{0{\cdot}156}$	(22)	0·857	0·614	0·735

Equations (21) and (22) in Table 4 give the results of correlating K_1 and K_2 to the basin area, A in square miles, the fraction of imperviousness U, the excess precipitation P_E in inches and the rainfall duration T_R in hours. The predicted values of the runoff hydrographs obtained by using K_1, K_2 and T_4 as the single linear reservoir constant are compared to the observed values in Fig. 7b. A comparison of Figs. 7a and 7b indicates the advantages gained by using different values of the time lag for different storms.

Conclusions

(1) The Fourier transform provides an efficient method of estimation of the IUH in urban basins. Possible computational instabilities may be controlled by the proper choice of the discretization interval and by digital filtering.

(2) The theoretical IUH obtained by the Fourier transform method has the same appearance as that of the single linear reservoir for areas of less than about 5 mi^2.

(3) The peak discharge is seen to depend principally on two basin characteristics, the area and the fraction of imperviousness, and on two storm characteristics, the amount of rainfall excess and the rainfall duration.

(4) The time lag is seen to depend on the same two basin and rainfall characteristics as in (3) above.

(5) The time lag is not a unique characteristic as it varies from storm to storm.

(6) For proper regeneration and prediction performance in small basins (area less than 5 mi^2) by the single linear reservoir method, the reservoir constant should not be taken as the average time lag. The dependence of the reservoir constant on both the basin and storm characteristics should be taken into account as the reservoir constant changes from storm to storm.

ACKNOWLEDGEMENTS

The results presented herein were obtained as a part of research entitled 'The Effect of Urbanization in Small Watersheds' supported jointly by the Office of Water Resources Research under project OWRR-B-012-IND, by the Indiana Department of Natural Resources and by Purdue University. The authors wish to express their gratitude to Dr. D. Blank who contributed greatly to the development of the Fourier transform method of evaluation of the instantaneous unit hydrograph, to Dr. P.B.S. Sarma who did the calculations for the correlation equations of Tables 3 and 4, and to Drs. J.F. McLaughlin, Head of the School of Civil Engineering and D. Wiersma, Director of the Water Resources Research Center at Purdue University for their substantial assistance in the administration of the research.

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