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ABSTRACT

A brief review of previous work is presented. Different methods for quantifying urbanization are discussed. A stepwise multiple regression technique was used to select the best parameter of urbanization. The rainfall and flood events from nine urban watersheds in the Denver Metropolitan region were analyzed. Unit hydrographs were derived from the measured floods on these watersheds. The unit hydrograph parameters were correlated with storm and physical watershed parameters. It was found that the changes in the unit hydrograph in the urban region were related to the decrease in the watershed response time. The best way of defining the response time was the lag time. The lag time was found to be sensitive to the increase in the hydraulic capacity to the decrease in the ratio of pervious watershed and the shape of the watershed.

Chapter 1
INTRODUCTION

There are many problems related to hydrology which plague the present day city engineer. One of these is related to flood hydrology and the design of storm water drainage. Water courses through the urban region which previously were apparently of adequate capacity chronically suffer the symptoms of under capacity after urbanization has taken place.

Accompanying this frequent flooding problem is the tendency for the water courses to deteriorate, become unsightly dumping grounds and to become the beginnings of urban blight.

The problems of sedimentation--both channel scour and channel degradation at various points along the remaining drainageways--are in turn related to the higher incidence of floods in the urban channel reaches. To document these facts, the U. S. Geological Survey has undertaken a program to measure the floods on urban watersheds throughout the country. The American Society of Civil Engineers has acted as an agency to assemble and disseminate data and results of the information obtained from urban floods. There have been several research projects which have developed correlations between the flood hydrograph parameters and various physical features unique to the urban watershed.

Chapter 2
RESUMÉ OF PREVIOUS WORK

RATIONAL FORMULA

One of the early developments in urban flood hydrology occurred after the enactment of the Arterial Drainage Act of Ireland in 1842 (dealing with urban drainage). The first Commissioner for Drainage was Thomas James Mulvaney. According to Biswas (1970), Mulvaney was responsible for the planning, design and construction of various urban drainage, navigation and harbor projects. According to Dooge(1957), it was William Mulvaney (the younger brother of Drainage Commissioner Mulvaney) who first proposed the use of the Rational Formula in 1851:

$$Q = CIA \ . \tag{1}$$

According to Biswas, Mulvaney correctly realized the importance of the time of concentration in applying the Rational Formula.

In 1889, Kuichling discussed the use of the rational formula in connection with the design of storm drainage in Rochester, New York. Ramser (1927) defined the time of concentration for small simple agricultural watersheds as the time interval between the low flow stage and the maximum flow stage. Later Kirpich (1940) empirically related Ramser's Time of Concentration, Tc, to the watershed variables, channel length, L, and slope, s, for some small Pennsylvania watersheds:

$$Tc = 0.0013 \ \frac{L}{\sqrt{s}}^{.77} \tag{2}$$

The importance of the watershed response time (the time of concentration) and the channel length - slope parameter were recognized early in the development of hydrology.

GEOMORPHOLOGY

The research work of Horton (1945) was a natural outgrowth of the earlier work on the formation of the flood hydrograph from the watershed runoff. These watershed properties were commonly used:

a) The watershed area;
b) The length of the longest channel;
c) The slope of the channel.

Horton developed many concepts which formed the basis of modern geomorphology. The basis for Horton's concept was the ordering of stream channels beginning with the most elementary channels in the headwater region. The most elementary channel branch is given order number one. When two first order channels join, a second order channel is formed. Horton found that simple geometric relationships developed between the number of channels of the different order numbers, the length of channel of a particular order number and the watershed size were all related to the stream order number. It can be inferred from Horton's work that the drainage density of the watershed has an important bearing on the characteristics of the watershed which have a bearing on the flood hydrograph Horton found that the ratio of the number of channels of a particular order number was related to the number of channels in the next lower order. This he defined as the Bifurcation Ratio:

$$R_b = \frac{N_u}{N_u + 1} \ ,$$

where R_b is the Bifurcation Ratio,

N_u is the number of channels of order of u

N_{u+1} is the number of channels of order u+1 (the next higher order).

Since the bifurcation ratio tends to be preserved as the more complex drainage patterns evolve, the number of channels of a given order, N_u, can be computed using Horton's Law of Stream Numbers:

$$N_u = R_b^{\,k-u} \ ,$$

Where k is the order number of the trunk segment at the outlet of the watershed. This law has been verified by a number of researchers. The application is rather impractical for any natural watershed of appreciable size because of the laborious procedure required to obtain the data. Furthermore, the task is influenced by the quality and consistency with which the cartographer prepared the map. The urban region superimposes a new channel pattern upon the original consequent stream pattern. It is possible that some of the resultant urban flood hydrograph characteristics can be explained using principles of geomorphology.

Hack (1957) studied streams in Pennsylvania and Virginia and later extended his findings to a wide variety of rivers around the world. He found a consistent relationship between the longest channel and the drainage area:

$$L = kA^n \tag{3}$$

where L is the length of the longest collector in miles,

A is the watershed area in square miles,

k is a coefficient varying from 1 to 2.5 with an average of 1.4,

n is an exponent which varies from 0.6 to 0.7 with an average value of 0.6.

These results were based on observations on natural watersheds in which the channel systems were free to evolve. In the case of an urban watershed, parts or sometimes all of the channel systems are fixed and therefore they may not be free to evolve into other networks. The superposition of the street network over the watershed has a great deal to do with the final shape and extent of the watershed as well as channel network and length of the channels.

The drainage density is defined as the ratio of the channel length per unit area:

$$D_d = \frac{L_s}{A} \quad (4)$$

where D_d is the total length stream channels per unit of watershed area.

L_s is the total length of all channels both ephemeral and perennial. There is a practical limitation to the evaluation of the drainage density. The only way that all of the ephemeral drainage channels can be identified is by a detailed survey in the field or by careful analysis of aerial photographs. A great deal of the details required to determine the drainage density are lost on topographic maps of scale 1:24000 (7-1/2 minute quadrangle sheets). Even with maps of the scale 1:24000, the task of determining the drainage density is laborious.

As originally conceived and used by Horton (1945) and later by Langbein (1947), the channels were defined by the blue lines shown on the topographic sheets. This practice resulted in some inconsistency in the results depending upon the season of the year and the relative wetness or dryness of the year in which the maps were prepared. During the time when flood runoff is occuring, many depressions or otherwise ephemeral channels are also part of the active channel system. Therefore, when attempting to establish relationships between flood hydrograph parameters and the channel network system it is perfectly valid to consider these depressions and ephemeral channels as part of the drainage system. Carlston (1963) extended the channel networks into all depressions and drainage ways suggested by upslope "V" shaped interruption in the contour lines.

The preparation of the extended channel network in a watershed of any appreciable size is laborious. In order to reduce the task to acceptable magnitude, Balayo (1967) using the technique described by Carlston obtained estimates of the drainage density using the extended channels on sample blocks in the catchment. Standard 4-centimeter square blocks were selected at random on the watershed map. All channels were extended and the length of the extended channel network was determined. The average drainage density was computed for five sample blocks. The data for the sixth block was entered and the new average drainage density was computed. The procedure was repeated adding one block at a time until the change in the average was less than one percent. The average drainage density was then adopted as the drainage density for the entire watershed. Using the method of obtaining estimates of the extended channel networks, Carlston (1963) found this regression equation for relating the unit area mean annual flood (Q peak for Tr = 2.33 years) and the drainage density:

$$\frac{Q_{2.33}}{\text{Drainage Area}} = 1.3\ D_d^{\ 2}\ .$$

Wolman and Miller (1960) found that the channel capacity developed in a watershed tended to equal to the value of recurrence interval of 2.33 years. This value is used as a standard or normal value in geomorphic processes.

In general Carlston found that the drainage density for the Appalachian watersheds he studied varied from 3.0 to 9.0 miles per square mile. The highly permeable sandy watersheds always had lower values of the drainage density. When the rainfall readily infiltrates into the watershed, overland flow is not available for development of the drainage network. In addition, as the watershed develops a channel network, the flood peak discharge increases.

In a complementary part of the investigation on the influence of channel networks on the runoff hydrograph, Carlston (1963) found that the unit area base flow was inversely related to the drainage density:

$$\frac{Q_{base}}{\text{Drainage Area}} = 14 D_d^{\ -2}\ .$$

Since the base flow is supplied from the groundwater storage, it is logical for the unit area base flow to be greatest under those conditions when the surface runoff is least efficient. The drainage density is directly related to the surface runoff drainage efficiency (considering the watershed slope, channel slope, area and roughness to be constant).

It is assumed that the channel networks and consequently the drainage density evolved naturally without man-made restrictions. In the urban environment, the channel network in existence before the landscape was urbanized is drastically altered. In some cases the major drainageways remain, but these are altered. The overbank areas are reduced, channels are straightened; sometimes the roughnesses are removed. The result is deeper flowing water, higher velocities and a hydraulically more efficient channel.

Urbanization often obliterates entirely the secondary channel networks. These are replaced by a network of roadside ditches, or curb and gutter networks. The curb and gutter network is relatively smooth and the alignment is straight. These relatively deep straight hydraulically efficient channel networks decrease the transit time of a flood wave in the channel network system.

CHANNEL SLOPE

The channel slope in the watershed relates the rate with which the potential energy of the streamflow is consumed in friction losses, turbulence and kinetic energy. Kirpich's (1940) relationship for the time of concentration contained a slope term in the length-slope parameter. Likewise USBR (1965) enlarging on Snyder's (1938) work with the unit hydrograph found that the channel slope appeared in the lag time relationships. Dempster (1974) found that the length-slope parameter was a significant parameter in the regression model predicting the peak discharge of the T-year flood. The past researchers have found various ways of defining the channel slope.

In the typical natural watershed, the stream channels increase in size proceeding in the downstream direction because the watershed area contributing the flood runoff increases. The increase in channel size may also be attributed to the decrease in the average stream velocity proceeding in the downstream direction. Usually the channel gradients are greatest in the headwaters region and progressively flatten in the downstream direction.

For practical reasons, it is desirable to devise a single measure of channel slope to represent the slope of the whole watershed channel. The problem arises then to define the slope such that the defined slope bears the most meaningful relationship to the flood characteristics of the watershed. The channel selected to represent the whole watershed is usually the longest collector in the watershed; although the most significant channel is probably the channel having the longest transit time. The simplest slope expression is the fall in the watershed between the headwaters and the outlet divided by the length of the longest collector:

$$S_1 = H/L \ .$$

This definition may be faulty because too great emphasis may be placed on the steep slopes in the headwaters region which are hydraulically quite far removed from the outlet. Another method of defining the average channel slope was described by Reich (1962) and later incorporated in the Colorado State University small watershed flood data file, Laurenson et al. (1963). The slope quantity is the slope of a straight line joining the elevation of the outlet on the profile of the mainstream with the average elevation of the actual stream profile. Nash and Shaw (1966) have given this equation for finding this particular slope:

$$S_2 = \frac{2\Sigma L_i Z_i}{(\Sigma L_i)^2} \ ,$$

where L_i is the distance along the mainstream between successive contours,

Z_i is the average elevation above the outlet for each reach of length L_i.

These are shown on Fig. 1. It is apparent that the area under the stream profile diagram is equal to the area under the straight slope line.

A simpler definition of the stream slope is given in Laurenson et al. (1963) which had been suggested earlier by Benson (1959). The greatest bias is placed on the 75 percent of the channel length (longest channel extended to the watershed divide) which, in most watersheds, collects the majority of the flood runoff:

$$S_3 = \frac{\text{Elevation at } 0.85L - \text{Elevation at } 0.1L}{0.75L}$$

Data on these various methods of defining stream slope and other watershed parameters have been assembled in the Small Watershed Data File at Colorado State University. Using the data assembled, several types of multivariate analyses were made to attempt to select a significant difinition of the various pertinent watershed parameters. Yevjevich et al. (1972) reported that for the data in the CSU flood data file, it appeared that the third definition of channel slope listed previously was the more satisfactory way of defining channel slope for a natural watershed.

Whatever definition of channel slope is employed, the effect of the slope is that the watershed response time is inversely related to the square root of the slope. This was demonstrated by Kirpich (1940), USBR (1965), Taylor and Schwarz (1952) and many others.

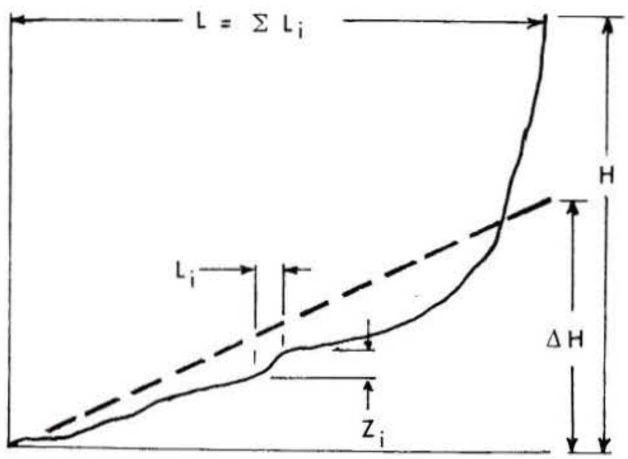

Fig. 1. Definition of Average Channel Slope

Straight line is drawn such that the area under line is equal to area under the profile diagram.

$$S_2 = \frac{\Delta H}{L} = \frac{\Delta H}{\Sigma L_i}$$

and area under line is:

$$1/2 \ \Delta H(\Sigma L_i)$$

and area under profile is:

$$\Sigma(L_i Z_i)$$

Equating these two areas:

$$1/2 \ \Delta H(\Sigma L_i) = \Sigma(L_i Z_i)$$

Eliminating ΔH using the slope definition

$$\Delta H = S_2 \ \Sigma L_i$$

$$1/2 \ S_2(\Sigma L_i)^2 = \Sigma(L_i Z_i)$$

$$S_2 = \frac{2\Sigma(L_i Z_i)}{(\Sigma L_i)^2}$$

CHARACTERISTICS OF FLOODS FROM URBAN REGIONS

Savini and Kammerer (1961) traced the stages of urban development and classified the effects of this development on the hydrologic regimen. Their classifications were broadly divided into the effects on water quality and water quantity. The changes were related to the hydrologic processes--evaporation, transpiration, infiltration, groundwater and flood flow. The problem of storm runoff was investigated in the urban Rochester, New York area by Kuichling (1889), in the St. Louis area by Horner and Flynt (1936), in the Los Angeles area by Hicks (1944) and in the Chicago region by Tholin and Kiefer (1960). The Procedures developed by these investigators are summarized in Chow (1964).

Tucker (1969a) assembled lists of urban watersheds having rainfall and runoff data. One of the earlier systematic programs for gaging urban watersheds began in 1948 at Johns Hopkins University under the direction of Dr. John Geyer. During this period a number of cities in the United States began programs to gage the runoff from urban catchments in order to develop the data base in order to apply the hydrograph design methods of Hicks (1944) or Tholin and Kiefer. Many of these watersheds are described by Tucker (1969a and 1969b).

Houston, Texas, was one of the metropolitan regions assembling urban runoff data. The Houston metropolitan region is a relatively flat plain which is drained by six streams which discharge toward the east into the San Jacinto River. One of these streams is Brays Bayou which has an 88.4 square mile catchment. This gaging station has been in operation since 1937.

One of the first studies to actually document the effect of urbanization on the unit hydrograph was presented by Van Sickle (1962) in a paper to the Texas Section of the American Society of Civil Engineers. Van Sickle presented six unit hydrographs derived from Brays Bayou during the period from July 1939 to June 1960. In this period of time, the watershed evolved from a rural watershed to an urban watershed and peak of the unit hydrograph increased from about 1500 cfs to about 4800 cfs-- more than a 300 percent increase. The watershed response time measured as the time to peak decreased from 12 hours to 3 hours.

The U. S. Geological Survey in Cooperation with the City of Houston has undertaken a comprehensive effort to obtain data on the flood response of urban watersheds in this region. Johnson and Sayre (1973) have presented an analysis of the data for the Houston region. Dempster (1974) has presented a similar analysis of data obtained in the Dallas, Texas area.

Rattapan (1968) applied the Chicago Hydrograph Method of Tholin and Kiefer in developing a method of computing an urban storm runoff hydrograph procedure for Bangkok, Thailand.

Increase in Flood Peak Discharge. -- Johnson and Sayre found that the peak discharge of a T-year flood could be estimated for the Houston area using the equation:

$$Q_T = aA^bI^c ,$$

where

- Q_T is the peak discharge of a flood having a return period of T years in cfs,
- A is the watershed area in square miles,
- I is the percent of impervious watershed,
- a, b, and c are constants for the region.

The values of the constants are given in the following table.

Johnson and Sayre also discussed the storm drainage patterns in use in the Houston area. They analyzed the network data from 28 watersheds having between 8 floods and 19 floods for each station. A log transformed multiple regression equation was found for these flood events. This is similar to a presentation by Dooge (1973). The regression equation is:

$$Q_p = aP^bD_{85}^cM^d ,$$

where Q_p is the peak discharge for the flood in cfs,
- P is the causative rainfall areally averaged over the watershed in inches,
- D_{85} is the storm duration in hours during which 85 percent of the rainfall, P, occurred,
- M is the soil-moisture index found from the relation:

$$M = (M_o + P_o)k^t ,$$

where M is the soil moisture index in inches,
- M_o is the soil moisture index computed or measured t days preceding M,
- P_o is the precipitation on the day when M_o was determined in inches,
- k is soil moisture depletion factor dependent upon the season,
- t is the number of days between M and M_o.

The values of the constant a and exponents b, c and d are given by Johnson and Sayre. The values have a complex relationship between the watershed area, surface storage, channel network and local topography. Because of the flat topography around Houston, there is watershed piracy between adjacent watersheds during some of the events.

Unlike the Houston metropolitan region, the Dallas region is bisected by a major river-the Trinity River. Dempster had data from 19 storms over six smaller basins which discharge into the Trinity River. Dempster's data base contained 205 flood events on 19 sub-basins. These data were used to calibrate a USGS digital model developed by Dawdy, Lichty and Bergmann (1972). The 57-year rainfall record of climatological data was used to simulate a 57-year runoff record for the urban watersheds. The log-pearson Type III distribution was fitted to the derived record.

A regional flood-frequency equation relating the flood peak for an assumed return period with storm and physical watershed characteristics was developed using a stepwise multiple regression equation. The procedure operates on the input data by successively discarding the independent variables which are the

Table 1

Regional Relations for T-Year Flood
Houston, Texas Region

Recurrence Interval	Constant	Exponents		Standard Error	
T, in years	(a)	(b)	(c)	log units	percent (average)
2	38.8	0.86	0.62	0.111	26
5	62.7	.87	.57	.119	28
10	82.0	.87	.54	.129	30
25	109	.88	.50	.141	33
50	132	.88	.48	.150	35
100	156	.89	.45	.159	37

from Johnson and Sayre (1973)

least significant in explaining the relationship between independent watershed parameters and the resultant flood peak. By selecting a T-year flood for use in the regression analysis, the effect of the storm variations was effectively suppressed. Dempster found that the most important watershed variable was the watershed area, A, followed by the imperviousness parameter, K, and lastly a length-slope parameter, $L/\sqrt{S}$. The length-slope parameter was related to the watershed time of concentration by Kirpich (1940). The limits of the amount of basic data available did not statistically justify the inclusion of other independent watershed parameters. As in the case of the analysis by Johnson and Sayre, Dempster also found that the log transformed data worked best. It is interesting to note that Dempster's analysis found that the length-slope parameter was important in the regional equation:

$$Q_T = aA^b K^c (L/\sqrt{S})^d ,$$

where L is the length of the longest collector in miles,
S is the slope of the longest stream in feet/mile.

All other variables have been previously defined. The results of Dempster's regional analysis are presented in Table 2.

All other variables have been previously defined. The results of Dempster's regional analysis are presented in Table 2.

where Q is discharge in cubic feet per second (cfs)
A is drainage area in square miles (sq mi)
L is length in miles (mi)
S is slope in feet per mile (ft/mi)
K is coefficient of imperviousness is 100 + .015I
I is percent of impervious watershed

The Dallas and the Houston metropolitan regions have some interesting contrasts. The main channels and drainageways in the Dallas region are more deeply incised into the watershed. As a result even after urbanization the Dallas channels tend to have better flood conveyance capacity. The soil cover in the Dallas region is thin and the soils are tight and therefore before urbanization, large parts of the storm rainfall quickly drain into the channel systems which were deep and on steeper gradients. In Dallas as the urbanization progresses, the residential areas

Table 2

Regional Regression Equations for T-year Flood
Dallas, Texas Region

Equation for indicated T-year flood discharge (cfs)	Standard error of estimate (percent)	Estimate of minimum prediction error (percent)
$Q_{1.25} = 195(A)^{0.88}(L/\sqrt{S})^{-0.13}(K)^{1.02}$	11	30
$Q_2 = 369(A)^{0.90}(L/\sqrt{S})^{-0.19}(K)^{0.65}$	11	30
$Q_5 = 621(A)^{0.93}(L/\sqrt{S})^{-0.23}(K)^{0.42}$	10	29
$Q_{10} = 776(A)^{0.95}(L/\sqrt{S})^{-0.25}(K)^{0.35}$	10	29
$Q_{25} = 953(A)^{0.98}(L/\sqrt{S})^{-0.27}(K)^{0.32}$	10	29
$Q_{50} = 1{,}067(A)^{1.00}(L/\sqrt{S})^{-0.28}(K)^{0.32}$	10	29
$Q_{100} = 1{,}172(A)^{1.02}(L/\sqrt{S})^{-0.29}(K)^{0.33}$	10	29

develop a deeper, more permeable soil zone, some structures are built on the channels which stabilize the gradients and retard the flood flows. This explains in part the smaller increase in the peak discharges in the Dallas area compared to Houston.

The watersheds in the Houston region have flatter slopes and the streams have flatter gradients. As urbanization progresses, land around the buildings is filled in and built up and street drainage is developed. The street drainage is discharged into either natural or constructed drainage channels. The larger increase in the flood peaks may be explained on the basis of a local increase in the relief as concerns the overland flow and a subtle increase in the drainage density accompanying the development of streets and street drainage. Thus the slope is increased on a micro scale in addition to the construction of a denser drainage network.

Van Sickle (1962) found that the peak discharge of the unit hydrographs for Brays Bayou at Houston increased three times after urbanization had taken place. Utilizing a much more extensive data base, Johnson and Sayre (1973) found that complete urbanization increased the magnitude of the 2-year flood by nine times and that the 50-year flood was increased by five times. On the other hand, Dempster (1974) found that the flood discharge increased by 1.35 times for the 2-year flood while the peak discharge for the 50-year flood increased by 1.16 times for a similar increase in the imperviousness of the watershed. The results are not entirely comparable because Brays Bayou is a relatively large watershed and even in 1970, only 18 percent of the watershed was impervious. Some of the smaller watersheds in the region have up to 35 percent impervious watersheds (Stoney Brook Street Ditch in Houston, and Turtle, Coombs and Cedar Creek in Dallas). The comparison of the flood runoff in these two regions does demonstrate the importance of the relative conveyance capacity of the drainage network in the increase in the flood peaks for a given recurrence interval; however, imperviousness per se is not the key factor.

Decrease in Response Time. -- Espey and Winslow (1968) analyzed data obtained from the Houston network during the period 1964 to 1967. They analyzed data from 17 Houston watersheds of which 6 were rural watersheds and 11 were urban watersheds. Espey and Winslow presented a definition of a channelization factor which takes into account relative efficiency of the storm drainage network. Espey's channelization factor was proposed as a second coefficient which when multiplied by the coefficient in Carter's equation for a pristine watershed produced the rise time for the urban watershed:

$$T_R = 20.8\phi \left(\frac{L}{\sqrt{S}}\right)^a I^b$$

where T_R is the rise time,

ϕ is Espey's channelization factor,

I is the percent of impervious watershed,

a is an exponent $a = 0.29$,

b is an exponent $b = -0.6$.

Channel and Storm Sewer Network. -- Espey and Winslow (1968) found that in the Houston watersheds, the channelization factor ϕ could have two parts:

$$\phi = \phi_1 + \phi_2$$

The first part, ϕ_1, relates to storm sewer-drainage network. The ϕ_1 coefficient should have a constant value as long as these was no change in the drainage density. The second part of the coefficient, ϕ_2, relates to a part of the channel resistance which may change during the year. Espey and Winslow found that in the Houston region, the growth of vegetation in the drainage channels retarded the flow of water which increased the watershed response time. Typical values of the Espey channelization factors are given in Tables 3 and 4.

During the period 1945 to 1949, the Louisville District, U. S. Corps of Engineers made measurements of rainfall and runoff hydrographs in storm drains in Louisville, Kentucky. Six urban watersheds in which the storm runoff was disposed of through storm sewers provided data from which 5-minute unit hydrographs were derived. Eagleson (1962) used these data in an analysis of the unit hydrograph characteristics of urban watersheds. The Louisville watersheds contained a larger proportion of storm sewers when compared with the Texas urban watersheds. Eagleson found that the discharge hydrographs from the smallest watershed (0.096 sq. mi.) were so sensitive to storm rainfall variations over the watershed, that consistent unit hydrographs could not be realized. Of the remaining five urban watersheds (varying from 0.22 to 7.52 sq. mi. in size), 27 flood events were used in his data base.

Carter (1961) found that the degree of imperviousness of the watershed could be used to define a family of lines which were parallel to similar lines for a natural watershed and he suggested that the process of urbanization could be quantified on the basis of the percent of imperviousness. Carter proposed an urbanization factor, K , which is computed from the percent of imperviousness in the watershed, I . The use of the impervious factor always had a value greater than 1.0. In addition the use of this factor also allowed the use of a variable local coefficient to account for those impervious roof areas which drain onto a grassed area and do not result in immediate surface runoff. Dempster also used a variation of the same imperviousness factor:

$$K = \frac{0.3 + 0.0045\ I}{0.3}, \qquad \text{(Carter)}$$

$$K = 1 + .015\ I . \qquad \text{(Dempster)}$$

Carter introduced the term length-slope parameter used by Dempster in the analysis of the Dallas, Texas data. A similar parameter had been used previously by Snyder (1958) and Kirpich (1940):

$$\frac{L}{\sqrt{S}} \qquad \text{(Carter)}$$

$$L_e = 10n \frac{L}{\sqrt{S}} \qquad \text{(Snyder)}$$

Table 3

Espey Channelization Factor

ϕ_1	Classification
0.6	Extensive channel improvement and storm sewer system, closed conduit channel system.
0.8	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel.
1.0	Natural channel conditions.

Table 4

Espey Seasonal Channelization Factor

ϕ_2	Classification
0.0	No channel vegetation.
0.1	Light channel vegetation.
0.2	Moderate channel vegetation.
0.3	Heavy channel vegetation.

$$\phi = \phi_1 + \phi_2$$

where K is the urbanization factor,
I is the percent of impervious watershed,
L_e is the equivalent length,
L is the length of the longest channel,
n is the Manning friction factor,
s is the weighted slope of the longest channel.

Both Carter (1961) and Snyder (1958) found that watershed response time (lag or time of concentration) could be correlated with a length-slope parameter. Carter (1961) found that some of the effects of urbanization could be quantified through the coefficient in the relationship between lag time and the length-slope parameter. These equations were derived for several streams in the Washington area:

$$T_{LC} = 3.1 \left(\frac{L}{\sqrt{S}}\right)^{0.6} \quad \text{(pristine conditions)},$$

$$T_{LC} = 1.2 \left(\frac{L}{\sqrt{S}}\right)^{0.6}$$

(natural main channels but sewered secondary drainage),

$$T_{LC} = 0.53 \left(\frac{L}{\sqrt{S}}\right)^{0.6} \quad \text{(completely sewered, complete urbanization)}.$$

Carter based his analysis on flood peak discharge having a return period of 2.33 years. The evolution of the watershed to a completely sewered watershed resulted in 1.8 increase in the peak discharge over a pristine watershed. Similar relationships were reported by Espey et al. (1965) for the watersheds near Houston. The coefficients for the Houston watersheds are not given here because the units and the definition of lag time appears to differ from Carter.

In the analysis of the Louisville data, Eagleson (1962) used the more conventional expression of the watershed parameter:

$$\Omega = \frac{L\,L_{ca}}{\sqrt{S}} \qquad \text{(Eagleson)}$$

The disadvantage to all of the watershed parameters used is that they have dimensions. There is considerable advantage if these could be dimensionless numbers. Eagleson used the lag curve developed in the Los Angeles District Office of the Corps of Engineers and shown in Linsley et al. (1958). Eagleson gave these equations for basin lag in terms of the basin parameter, Ω:

$$T_{LB} = 1.2\ \Omega^{0.38} \quad \text{Mountain Drainage}$$

$$T_{LB} = 0.72\ \Omega^{0.38} \quad \text{Foothill Drainage}$$

$$T_{LB} = 0.35\ \Omega^{0.38} \quad \text{Valley Drainage}$$

$$T_{LB} = 0.18\ \Omega^{0.38} \quad \text{Urban Drainage (Louisville)}$$

where T_{LB} is the time from beginning of rainfall excess to the centroid of the runoff hydrograph,

$$\Omega = L\,L_{ca}/\sqrt{S} \ .$$

The coefficients in the lag relationships are analogous to the C_t defined by Snyder (1938) however, they are not equivalent to Snyder's C_t because originally Snyder did not include the slope term in his basin parameter. Linsley, Kohler and Paulhus (1958) and Eagleson (1962) give the exponent on the basin parameter as 0.38. Various flood studies published by the Corps of Engineers and the Bureau of Reclamation show values of this exponent between 0.30 and 0.38.

The lag time, as defined by Eagleson, is one way to define the watershed response time. Wilson (1972) carried out an investigation on watershed response time in an effort to establish interrelationships for the various definitions of response time. The response time of the watershed is defined as the significant length of time required for a watershed to respond to a uniform input of rainfall excess. The various ways to quantify the response time were examined and compared by Espey, Morgan and Masch (1965), Wilson (1972) and others. These are presented later in this report in Table 5. Wilson's (1972) investigation was based entirely on data from pristine watersheds. It remains to be established which is the most effective way to define the response time for an urban watershed.

Regardless of the exact form of the definition of the response time, it is clear that the response time is related to either the basin parameter of Eagleson:

$$\Omega = \frac{L\,L_{ca}}{\sqrt{S}}$$

or Carter's length-slope parameter:

$$\frac{L}{\sqrt{S}} \ .$$

Sarma, Delleur and Rao (1969) conducted a research project on analytical models for simulating the effect of urbanization on runoff. Data from four urban watersheds near Purdue University were supplemented with data from other Indiana and Texas watersheds. Four conceptual models were used in the analysis of the data:

1) Single Linear Reservoir Model,
2) Double Routing Method,
3) Nash Model,
4) Single Linear Reservoir with a Linear Channel model.

It was found that for watersheds smaller than five square miles in size, the single linear reservoir provided best reproducibility of the recorded floods. The Nash Model best simulated the rainfall-runoff process on larger watersheds (between 5 and 20 square miles in size).

Sarma, Delleur and Rao (1969) used a multiple regression technique to find relationships between physical watershed parameters, urbanization parameters and unit hydrograph parameters:

1) Lag Time,
2) Time to Peak,
3) Peak Discharge,
4) Frequency of Peak Discharge.

Narayana, Sial, Riley and Israelsen (1970) carried out a similar study utilizing a larger data base. They analyzed the results of 200 events from 50 rural watersheds and 193 events from 20 urban watersheds. Narayana et al., limited their research to developing relationships between watershed, storm and urbanization factors and the peak discharge and the total volume of runoff. No relationships were developed between the watershed response time and the physical watershed and storm parameters. A log transformed model was found to give the best results. The peak discharge was estimated by:

$$Q_p = 0.777\ W_1 S_1 U_1$$

where W_1 is the watershed parameter

$$W_1 = \frac{A^{0.738} S^{0.206}}{L^{0.042}}$$

S_1 is the storm parameter

$$S_1 = \frac{P^{1.016} P_{30}^{0.179}}{D^{0.26}}$$

U_1 is the urbanization parameter

$$U_1 = \frac{1}{\phi^{1.28} c_f^{0.797}}$$

A is watershed area in acres
S is main channel slope in percent
L is main channel length in miles
p is the total storm rainfall in inches
P_{30} is the maximum 30 minute rainfall
D is the storm duration in hours

ϕ is the Espey Channelization factor

c_f is the watershed imperviousness factor

$c_f = 1 - R_i$

R_i is the ratio of impervious watershed to pervious watershed.

The regression equation was tested using an independent set of data measured from Boneyard Creek at Urbana, Illinois.

USE OF STATISTICAL METHODS

The majority of the recent investigations on the effects of urbanization on flood hydrology have used some of the techniques of statistics dealing with correlation and regression. More effective techniques have been under development. Multivariate techniques are better suited to many problems in hydrology. Johnstone and Cross (1949) illustrated the application of correlation and regression in hydrology. Several examples of the test of significance are also given in this book. A more thorough treatment of the application of correlation to problems in hydrology was given by Beard (1962). Beard discussed the matters of multiple correlation, nondetermination and criteria of statistical reliability.

With the advent of the high speed digital computer, several improved procedures for complete multiple regression techniques evolved. These methods along with useful hints regarding their operation are given in a book by Draper and Smith (1966). Davis and Sampson (1973) give and discuss a number of computer programs written in Fortran for applying multiple regression and multivariate analysis. Reich (1962) utilized a stepwise multiple regression technique for selecting most effective regression equations between independent, and dependent hydrologic variables.

Stepwise Multiple Regression. -- In applying the stepwise multiple regression procedure, one independent variable is entered into the regression equation at a time and the coefficient of determination is found. The independent variable which yields the highest coefficient of determination is selected. Following the selection of the initial independent variable, the remaining independent variables are sequentially added to the regression equation and unexplained variance is computed. The independent variable which achieves the greatest reduction in the unexplained variance is the second independent variable added to the regression equation. The selection process is repeated -- each time a selection is made from the remaining independent variables until all of the independent variables have been added.

In the practical case, all of the independent variables are seldom actually used in the operational regression equation. This is because of the problem of limited amounts of hydrologic data and the excessive costs involved in continuing the acquisition of large amounts of data. For this reason, early in the stepwise multiple regression analysis the matter of the "best regression equation" is considered. Draper and Smith (1966) suggest six general procedures in which this selection may be achieved:

1) All possible regressions,
2) Backward elimination,
3) Forward selection,
4) Stepwise regression,
5) Two variations on the four previous methods,
6) Stagewise regression.

In general the fourth method, "Stepwise Regression" is the method used in the investigation reported herein. A computer program for completing these computations is available as a standard software package at the CSU Computer Center (STAT 38R-BDM02R revised).

SIGNIFICANT PARAMETERS OF URBANIZATION

There are many examples of a comparison of two or more photographs taken over intervals of time which graphically depict the evolution of an urban region. These comparisons witness to the fact that urbanization produces a profound change in the Watershed. The extent of the changes caused by urbanization on the hydrology of the watershed vary somewhat with local geology, local customs, local laws, local climate and the intensity of the urban development. There is a need to be able to express the urbanization process quantitatively. Schulz (1971) listed eight measures which could be applied to quantify the urbanization. These were discussed in some detail.

1) *Percent of Impervious Watershed,*
2) *Length of Streets and Roads per Unit of Area,*
3) *Length of Paved Streets per Unit of Area,*
4) *Length of Curbed and Guttered Streets per Unit of Area,*
5) *Length of Storm Sewer Conduit per Unit of Area,*
6) *Effective Channel Roughness of Floodways,*
7) *On-Site Flood Detention Storage,*
8) *Population Density in Watershed.*

Beginning with this list of factors of urbanization, Lopez (1973) carried out a stepwise multiple regression analysis to select the most effective factors of urbanization.

Watershed Imperviousness. -- Referring to the resumé of previous research work, the most obvious measure of urbanization is the proportion of impervious watershed. This measure was listed by Schulz (1971) and used by Lopez (1973). The factor influences the hydrology in two ways:

1) Reduction of Inflitration,
2) Reduction of Response Time.

Waananen (1969) shows two cases in different parts of the United States, where the water yield from a watershed has been increased by urbanization. The explanation is that the paved and roof surfaced replace natural soil surfaces which in their former state have infiltered rainwater and in return lost water by evapotranspiration.

The increasing imperviousness of the watershed has been related to the decrease in the watershed response time by Carter (1961), Espey et al., (1965, 1968, 1969), Van Sickle (1969), Riley and Narayana (1968) Narayana et al., (1970), and Schulz (1971). The decrease in watershed response time results in an increase in the peak discharge of the unit hydrograph.

In this investigation the percent of the watershed which is impervious is used to quantify watershed imperviousness. However in view of the results of Dempster (1974), Carter (1961) and Riley and Narayana (1968), the actual parameter used in the analysis must be modified to obtain values which are always greater than one:

$$U = 1 + I_A$$

where U is the dimensionless Lopez coefficient of imperviousness, I_A is the percent of impervious watershed. The reason for advocating the use of either the Lopez or the Carter equation is that it is never zero and always greater than one. This characteristic is advantageous when using a log transformed multiple regression analysis. In many urban communities, the roof drainage is captured in flower beds or grassed terraces which never really result in any surface runoff. Carter's equation contains a coefficient which can be varied to account for the ultimate disposition of the roof drainage.

Watershed Roads and Streets. -- The proliferation of the ubiquitous roads and streets is the most obvious feature in the evolving urban scene. Hydraulically the highway and the street may perform quite different functions. The road, highway, expressway or freeway evolves from the country road. These arteries of commerce are developed by the placement of a specially designed subgrade on top of a base which sometimes is built on top of fill above the surrounding land. Jones (1971) has pointed out that the roadside ditches or borrow pits which result have a significant effect on the increase of what is normally called depression storage. In the very first stages of urban development, the street is an unpaved roadway, but the density of the soil is increased and the surface slope is developed such that there is little opportunity for rainwater to infiltrate into the soil. As rural development progresses into the suburban stage, the increasing use of the road causes paving the road with asphalt or concrete to solve the dust problem in dry weather and stabilize the surface in wet weather. Whether paved or unpaved, the projected area of the road no longer infiltrates rainwater, but the roadside ditches capture and may store storm runoff.

In the case of the freeway or expressway in an urban environment, a porous gravel surface borders the paved surface. The shoulders of the roadway and the median strip are seeded with a suitable grass to control erosion and to provide a pleasing appearance. At the immediate edge of the roadway, the water supply to the vegetated surface is enhanced by the additional runoff harvested from the impervious roadway surface. In many climates this additional water supply is a benefit to the grassed surface. The benefit may be partly offset by the adverse effect of some of the other constituents of the microclimate of the highway such as lead, nitrous oxides, rubber and asbestos dust, carbon monoxide resulting from the traffic. Hydrologically an unpaved street or road or a paved roadway with a median strip or wide ditch probably has little effect on either runoff yield or the response time of the watershed.

Curbed and Guttered Streets. -- The curbed and guttered streets perform quite a different hydrologic function. Whereas the country road or urban freeway was built at an elevation above the immediate surroundings, the usual neighborhood street is set at an elevation below the immediate surroundings. This often results in the street functioning as a drainage way. Usually there is a crown at the center of the street so that water will drain toward the gutters at either side. The water will not drain from the surface because of the curb. The gutter may also collect runoff from the adjacent sidewalk or adjacent property. The flow in the gutter is relatively deep in relatively straight-smooth-channels. Super critical flow is often observed in gutter flow on moderate slopes. The gutters discharge into storm drains which also are relatively efficient carriers of storm runoff. Each mile of curbed and guttered street adds two miles of drainage channel to the watershed. The flood transit time in the curb-gutter-storm drain system is less than the transit time of the flood wave in the pristine natural channel system.

Watershed Channel System.-- In the urban setting, the natural drainage ways existing in the pristine watershed are altered. The secondary drainage network is obliterated and may be replaced with a curb and gutter system. Larger channels may remain although the hydraulic efficiency may improve. Channels are straightened and often conform to subdivided property boundaries. Many times the banks are shaped to confine the flow. Sometimes steeper banks are stabilized. Higher velocities result from the straight channels and deeper flows. Drop structures are then constructed to stabilize the overall channel gradient.

Gutter flow is discharged into these drainage ways when convenient. Sometimes storm sewers discharge into these drainage ways. The net result on the flood hydrology is to decrease the response time and to increase the peak discharge of the unit hydrograph.

Espey et al. (1965) and Espey and Winslow (1968) used the channelization classification ϕ to quantify the change of the watershed response time for an urban watershed having both storm sewers and improved drainage ways. (See suggested values of ϕ in Tables 3 and 4.)

Van Sickle (1969) proposed a basin factor K for estimating the watershed time to peak and unit hydrograph peak discharge:

$$K = \frac{L_t \bar{L}}{\sqrt{\bar{S}}}$$

where L_t is the total length of all drainage ways and storm sewers larger than 36 inches diameter in miles,
$\bar{L}$ is the mean basin length in miles,
$\bar{S}$ is the mean basin slope in feet/feet.

Van Sickle used a procedure described by Eagleson (1962) for finding $\bar{L}$ and $\bar{S}$ from a hypsometric diagram for the watershed. The Van Sickle basin factor for an urban watershed is analogous to the watershed basin factor Ω as used by the Corps of Engineers for pristine watersheds.

Chapter 3
BASIC DATA — DENVER METROPOLITAN WATERSHEDS

The basic data for this investigation were obtained from a special network of gaging stations established in the Denver metropolitan region as a cooperative project between the U.S. Geological Survey (USGS) and the Urban Drainage and Flood Control District (UDFCD). The USGS provides the technical expertise to collect and process the basic data. Some of the stations have been in operation since June 1968. The USGS has been responsible for design and development of better instrumentation and more effective methods of data reduction. The instrumentation for the Denver network has been developed from the maturity gained from the operation of the Texas networks at Houston, Austin, Bryan, San Antonio, Dallas and Fort Worth. Appendix A contains a detailed list of the gaging stations.

Gaging Small Urban Streams.-- Measuring floods on a small urban stream presents its own unique problems. These streams are ephemeral in nature while the large streams in the metropolitan region are perennial streams. The watersheds are small and the catchments usually do not have a recording rain gage nearby.

Under conditions of zero discharge, the stream gaging stations tend to develop operational problems. The floats stick or do not respond immediately when the flood hydrograph begins. Stilling well inlets tend to become plugged or closed. Since the response time of the small urban watershed is short, any hesitation in the response of the recorder adds very materially to the uncertainty in the determination of the waterwhed response time. The gaging of floods from small urban watersheds requires a high degree of reliability between the rainfall hyetograph and the runoff hydrograph. To achieve precise synchronism between the recording rain gage record and the stream hydrograph, the USGS has developed a Dual Digital Water Stage Recorder-Recording Rain Gage. The stream stage and the level of water in the rainfall measuring cylinder are recorded by two digital punched-tape recorders. The records of stage sensed by both recorders are simultaneously punched at five-minute intervals being activated by the same battery-operated timer.

Water Stage Recorder. -- The water stage recorder is mounted on the top of a five-foot length of standard four-inch galvanized iron pipe which functions as a stilling well. The stilling well intakes are six one-quarter-inch holes drilled radially around the periphery of a standard cap which is screwed to the bottom of the stilling well. The cap is positioned such that one hole is at the upstream face and one hole is at the downstream face. The inlets tend to be self purging. The stilling well is usually set so that the cap is one or two inches above the bottom of the channel. The water stage record does not respond to a small amount of "base flow." The installations have been described by Gonzales and Ducret (1971).

Recording Rain Gage. -- A recording rain gage is installed at each stream gaging station. The measuring tube for the recording rain gage consists of a 5 1/2 -foot length of standard 3-inch galvanized pipe. The pipe is mounted vertically with a sheet steel metal shelter at the top to house the recorder, timer and a 7.5 volt battery. The rain gage receiver is a 5-inch by 10-inch rectangle. The rainfall is concentrated by a funnel into a copper tube which leads to the pipe measuring tube. The recorder senses the water level by means of a float. The vertical pipe has the capacity to collect 7.0 inches of rainfall. When the accumulation exceeds this amount, a siphon is primed and the entire contents of the measuring tube is evacuated in about 90 seconds. A sufficient amount of water is retained in the measuring tube so that the float never rests on the bottom of the pipe. Thus the rain gage will respond immediately to any new rainfall.

Rating Curve.--A number of the stream gaging stations have been installed at the upstream end of a circular culvert pipe. The rating curves have been developed from the head loss relationships of water entering a culvert pipe. Because of the short duration of the runoff, it is usually difficult to obtain field verification of the rating curve. The stream gaging station at Stapleton Airport is installed in a 6-foot diameter storm sewer. The control consists of a Palmer Bowlus flume which has been fabricated of sheet metal and installed in the conduit by bolting to the sewer wall with small bolts anchored in concrete pipe wall. A sheet metal Parshall flume is used at one of the gaging stations. A broad crested weir is used at another of the Boulder watersheds.

Operational Difficulties. -- The location of the gaging stations is shown on Figure 1. Some of the gaging stations are operated during the summer season (the flood season) since June 1968. The original plans called for having 30 stations operational by the end of the summer 1972. The station locations were selected to 1) provide a variety of types of urban environment, 2) have both old and newly developed locations, 3) have simple and stable hydrologic configurations. Among the characteristics of an urban watershed are the dynamic changes taking place. These changes militate against the third attribute listed previously. By its very nature, the urban environment is changing and the hydrologic characteristics also change.

At a number of the stations the culvert configuration has changed or been extended upstream necessitating removal of the gaging station. In a number of instances additional runoff has been diverted into the watershed through changes in the culvert drainage system upstream from the gaging station. In a number of the watersheds the area could change from storm to storm depending upon the direction of gutter flow. A list of the gaging stations is given in Appendix A.

The data processing is accomplished by a computer because the two stage tapes are punched according to a binary code. The rainfall is reported in inches to the nearest 0.01 inch although the rain gage can resolve rainfall to the nearest 0.005 inch. The stream stage is reported to the nearest 0.01 foot. The stage record is converted to discharge using the rating equation and reported to the nearest 0.1 cubic foot per second.

In reducing the data, the stage in the precipitation gage is recorded at midnight of each date. When a storm commences, the precipitation is not recorded until at least 0.015 inch has been recorded in 5 minutes-- then the precipitation amount is recorded at each 5-minute interval until the precipitation ends. The runoff is recorded whenever there are measurable changes in stream stage.

In spite of the fact that these are small watersheds and there is a recording rain gage in each catchment, there are runoff events with no measurable precipitation. There are occasions when the runoff volume exceeded the volume of precipitation.

There are occasions when the 5-minute interval used in recording the data was too long. This time interval was much shorter than the data obtained from the Louisville or Texas watersheds, It seems that the data for storms whose duration is obviously less than five minutes will have to be discarded. Major storms last longer than five minutes. It is questionable if data from very small storms will be useful. According to Minshall (1960), the assumption of linearity is questionable for very small storms.

In spite of the difficulties outlined previously, the time resolution of the events and the reliability of the synchronism between rainfall and runoff is believed superior to the Louisville data used by Eagleson or the Texas data used by Van Sickle, Espey, Sayre and Johnson or Dempster.

Chapter 4
PROCESSING BASIC DATA

CSU SMALL WATERSHED FLOOD DATA FILE

Beginning in 1962 Colorado State University set out to assemble high quality rainfall-runoff data for use in research on floods from small watersheds. Since 1962, the flood data file has evolved into a system for storing and retrieving the pertinent facts from magnetic tape. The data storage system is now entirely computer based.

The basic data for the flood data file are assembled as a series of IBM cards as shown schematically on Fig. 2. Originally the flood data file was intended entirely for flood events from pristine watersheds. The data are prepared for magnetic tape storage as six sets of IBM cards. The arrangement is "open-ended" so that additional floods can be added at any time. Likewise new watersheds with their flood events can also be added at any time. To adapt this storage system for urban flood events, two additional sets of data cards defining the extent of urbanization have been added. Each flood event from an urban watershed will also be accompanied by a set of cards to define the state of urbanization existing for that flood event. A detailed description of these sets of data cards are given in Appendix B.

DERIVING A UNIT HYDROGRAPH

Since the rainfall and runoff event is stored on magnetic tape and since the procedure of deriving a unit hydrograph from a set of rainfall-runoff observations can be tedious hand or desk calculator operation, computer-based methods for deriving have been developed. Six different computer-based methods were used with data stored in the CSU Flood Data File by Jawed (1973).

Lopez (1973) used three of these methods for deriving the unit hydrographs. The method which obtained the largest number of realizable unit hydrographs was the FINVER program which was developed by Kavvas (1972). The unit hydrographs and the recorded floods are given in Appendix C.

Data Evaluation.--A preliminary evaluation of the available data was performed before any unit hydrographs were derived. The events chosen were primarily single peaked events. Complex events having a well defined peak several times larger than a secondary peak were chosen only when the volume under the secondary peak was insignificant compared to the primary peak. Care was also exercised in choosing events with relatively dry antecendent moisture characteristics whenever possible. However, Since rainfall below 0.015 inch per five minute interval was not recorded it is possible that some events may have had wetter antecedent conditions than others. The volumes of rainfall and runoff were calculated and all events having recorded runoff in excess of the rainfall were discarded because this was an indication that the rain gage data did not correctly represent the causal rainfall.

Limitations of Data.-- The initial evaluation revealed several shortcomings of the data:

a) The intervals in recording time are relatively large when the volumes of rainfall and runoff typically found in the data are considered. Smaller intervals would be desirable in the determination of initial abstractions for instance, where the volume of rainfall prior to the beginning of runoff occurs sometime within the five minute interval. The error in this case is magnified by the fact that often a rainfall duration of fifteen minutes is recorded which peaks during the first five minutes. As an example, an event registered at Hillcrest Drain, Northglenn, watershed area--0.28 square miles, on August 20, 1970, had a total abstraction of 0.073 inches and an initial abstraction of 0.09 inch. In this case the high intensity rainfall occurring in the first five minutes seem to not only satisfy the initial abstractions but also produces runoff. A malfunction in the instrumentation and nonuniform areal distribution of the storm could also account for this effect. The average rate of rainfall for the five minute interval also hides the actual time distribution of the event which is often necessary in deriving unit hydrographs of events of relatively small duration.

b) In some events, the volume of runoff exceeded the volume of rainfall as in the event at Westerly Creek Tributary, Aurora, watershed area--0.20 square miles, August 19, 1971. A malfunction of the instrumentation is possible, but a nonuniform areal distribution of the rainfall seems to be the more likely cause of this type of error. Even though the effect of nonuniform areal distribution is minimized in small basins, it should be kept in mind that it is possible.

c) In order to investigate the effect of antecedent conditions on the volume of rainfall excluded from the runoff, information concerning rainfall occurrences prior to the reported event is necessary.

d) In analyzing the effect of the volume of rainfall on the response time of the basin, multiple events occurring continuously could be very useful. When rain falls interruptedly, the initial abstractions are minimized and the infiltration approaches a constant value.

UNIT HYDROGRAPH PARAMETERS

The rainfall runoff data were used to derive unit hydrographs. A five-minute unit hydrograph was obtained from each event. (The unit hydrographs are given in Appendix C.) The choice of the time interval was dictated by the available computer capabilities to invert large matrices. Of the chosen events, only one was too long to obtain a unit hydrograph. Another consideration in choosing the five-minute interval was the fact that the rainfall data were measured at this interval and could be used as given. The interpolation of data often results in the unnecessary introduction of errors since one can only guess the possible time distribution of the rainfall within the recorded interval.

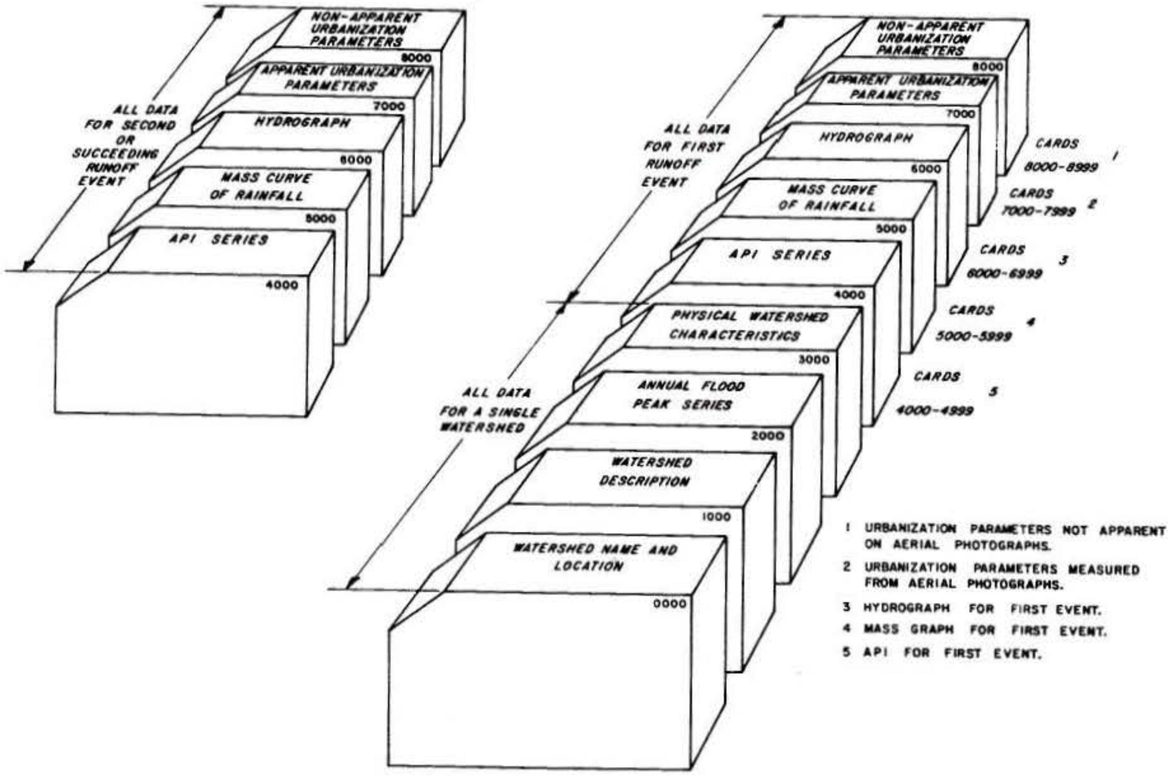

Fig. 2. Schematic Representation of Data Arrangement for a Single Watershed with Two Runoff Events

Unit Hydrograph Peak.-The peak values of the unit hydrograph and the estimated time to peak were then used to obtain regression relationships to determine the reliability of these unit hydrograph parameters. Since no outliers were found in the residuals it was concluded that the data used were adequate for the estimation of the parameters. To avoid the possibility that the tool used in the determination of the unit hydrographs was inadequate, three computer programs were used to derive the unit hydrographs and their results compared as explained later in the text. Graphs of the observed, the computed and the unit hydrographs used are given in Appendix C.

The determination of the time to peak was sensitive to the interval chosen because the peak of the unit hydrograph could occur sometime within the five-minute interval. Two values of the time to peak were obtained and their performance on the regression analysis evaluated. The first value used was the interval between the beginning of the rainfall excess to the beginning of the largest five-minute runoff volume. The second value was obtained by estimating the possible location of the peak when the trends of the ordinates on both sides of the peak were extended to intersect at the peak. The second procedure produced smaller values of the standard error of estimate and was adopted for use in this investigation.

Response Time. -- The importance of a watershed response time has been recognized since the time of Mulvaney in 1851. The concept of response time has acquired many different definitions. A general definition for the Response Time of the watershed is the significant length of time required for a watershed to respond to a uniform input of rainfall excess. The rainfall excess is defined as the rainfall which excess to that which will infiltrate into the soil.

Lopez (1973) determined the watershed response times utilizing eight different ways of defining the response time. The particular definition of the response time which will be selected will be the one which has the highest correlation with the physical watershed and storm characteristics. The definitions and symbols for the Response time is given in Table 5. Lopez used different symbols. His symbols are given in parentheses. These different time intervals are depicted on Figs. 3, 4, and 5. The values of these time variables are given in Table 6.

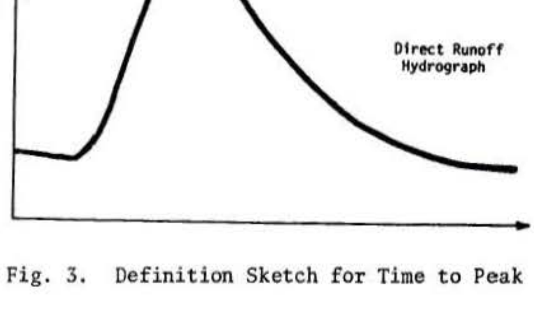

Fig. 3. Definition Sketch for Time to Peak

Table 5

Watershed Response Time

Symbol	Definition	Reference
T_{PC} (T_1)	Time to Peak--Time interval between the centroid of rainfall excess and peak of the direct runoff.	Snyder (1938), Taylor and Schwarz (1952), Eagleson (1962)
T_{PB} (T_2)	Time to Peak--Time interval between the beginning of rainfall excess and the peak of the direct runoff.	Linsley, Kohler and Paulhus (1958)
-- (T_3)	Time to Peak--Time interval between the beginning of RAINFALL and the peak of the runoff.	Lopez (1973)
T_{LC} (T_4)	Lag Time--Time interval between the centroid of rainfall excess and centroid of direct runoff hydrograph.	Horner and Flynt (1936), Mitchell (1948)
T_{LB} (--)	Lag Time--Time interval between the beginning of rainfall excess and the centroid of direct runoff hydrograph.	Wilson (1972)
T_{LC50} (T_5)	Lag Time--Time interval between the centroid of rainfall excess and the time when 50% of the direct runoff has passed the gaging station.	USBR (1965), Wilson (1972)
-- (T_6)	Time to Peak--Time interval between the centroid of rainfall excess and the peak of the unit hydrograph.	Lopez (1973)
-- (T_7)	Time to Peak--Time interval between beginning of rainfall excess and the peak of the unit hydrograph.	Lopez (1973)
T_R (--)	Rise Time--Time interval required for the hydrograph to rise from low flow to the maximum stage (might be equivalent to Lopez T_2).	Ramser (1927) Kirpich (1940) Gray (1961) Wu (1969)
T_C (T_8)	Time of Concentration--Time interval required for a unit volume of water to travel from the most remote point on watershed boundary to the outlet. Also--Time interval between end of rainfall excess and point of inflection on recession of the hydrograph.	Kirpich (1940) USCE (1966)
T_E (T_e)	Equilibrium Time--Time interval required for the runoff rate to become equal to the supply rate.	Izzard (1946), Wei and Larson (1971)

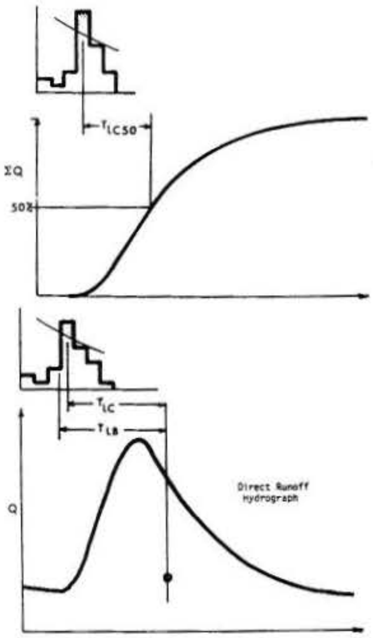

Fig. 4. Definition Sketch for Lag Time

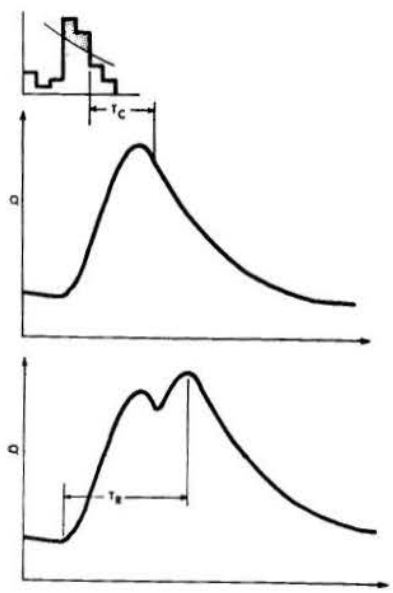

Fig. 5. Definition Sketch for Time of Concentration and Rise Time

Table 6

Watershed Response Times

Catchment	USGS No.	CSU No.	$T_{PC}(T_1)$ (min.)	$T_{PB}(T_2)$ (min.)	T_3 (min.)	$T_{LC}(T_4)$ (min.)	T_{LC50} T_5 (min.)	$T_{PB}(T_7)$	$T_C(T_8)$
Big Dry Cr. Trib. at Littleton, CO	06710200	1060702001	33.5	41.0	41.0	37.5	37.5	.55	35.0
		1060702002	20.0	--	35.0	20.0	20.0	.37	22.0
		1060702003	16.0	25.0	25.0	22.3	21.0	.30	25.0
		1060702004	14.5	27.5	27.5	21.0	21.0	.21	25.0
		1060702005	30.0	43.0	43.0	37.9	35.0	.25	35.0
		1060702006	41.0	45.0	45.0	--	43.0	.79	55.0
Sanderson Gulch Trib. at Lakewood, CO	06711600	1060716001	5.0	7.5	17.5	6.8	6.5	.16	10.0
		1060716002	7.5	10.0	15.0	11.3	10.5	.19	10.0
		1060716003	17.5	20.0	25.0	17.9	17.5	.28	20.0
		1060716004	15.5	23.0	23.0	29.8	22.5	.31	20.0
		1060716005	10.0	23.0	38.0	38.2	27.5	.18	10.0
Schneider Drain at Arvada, CO	06719800	1060798001	10.0	15.0	15.0	14.8	13.0	.19	13.0
		1060798002	5.0	10.0	10.0	12.6	10.0	.16	7.0
Toll Gate Cr. Trib. at Aurora, CO	06714230	1060742302	10.0	20.0	50.0	19.3	17.0	.17	5.0
		1060742305	8.0	15.0	15.0	19.5	15.0	.19	15.0
		1060742307	14.0	38.0	53.0	30.5	23.0	.17	7.0
		1060742306	19.0	35.0	35.0	24.4	24.0	.37	17.0
Westerly Cr. Trib. at Aurora, CO	06714270	1060742708	33.0	45.0	45.0	38.8	36.0	.55	18.0
Concourse D Storm Drain at Stapleton AP	06714300	1060743002	6.0	20.0	20.0	16.3	12.0	.16	5.0
Tuck Drain at Northglenn, CO	06720100	1060701001	9.0	17.0	17.0	13.3	12.0	.21	6.0
		1060701002	9.0	23.0	23.0	16.1	15.0	.16	5.0
		1060701003	8.0	20.0	20.0	11.8	11.0	.21	5.0
		1060701008	7.5	15.0	20.0	11.1	10.5	.16	3.0
		1060701009	9.0	17.5	32.5	--	8.0	.19	3.0
		1060701011	9.5	20.0	20.0	18.1	16.5	.21	3.0
Hillcrest Drain at Northglenn, CO	06720300	1060703001	12.5	15.0	20.0	12.5	12.5	.17	15.0
		1060703002	12.5	15.0	35.0	11.8	13.5	.20	16.0
		1060703003	12.0	15.0	85.0	16.6	16.0	.23	12.0
		1060703009	12.5	20.0	40.0	15.6	15.0	.26	10.0
		1060703010	10.5	13.0	18.0	15.8	14.5	.21	13.0
		1060703015	13.0	20.0	20.0	--	31.0	.29	15.0
		1060703016	12.5	15.0	20.0	14.3	13.5	.20	13.0
Kennedy Drive Drain at Northglenn, CO	06720400	1060704001	4.0	20.0	30.0	4.6	4.0	.11	3.0
		1060704002	4.5	12.0	22.0	7.7	7.5	.12	2.0
		1060704003	6.5	10.0	20.0	10.8	9.5	.11	3.0
		1060704004	2.5	10.0	10.0	5.8	4.5	11	2.0

In calculating time response T_6, a problem was encountered using program FINVER. The location of the unit hydrograph in time is lost and the relative location of the peak is not possible. In calculating T_c (T_8), several rainfall events were found which did not end in a clear and definite manner, but rather continued contributing very small amounts of rainfall past the point of inflection of the runoff hydrograph at a relatively constant rate. The end of rainfall in these cases was interpreted to be at the end of the last volume observed beforc the rainfall rate became constant.

The centroids of the excess rainfall and the runoff were obtained by tracing the graphs with a digitizer which punched the coordinates on cards. A computer program was then used to calculate the coordinates of the centroids.

RAINFALL PARAMETERS

The rainfall producing the runoff influences the hydrograph in a number of ways. The concept of the unit hydrograph and unit duration of rainfall excess tend to remove some of these variations. The rainfall parameters used in this investigation are given in Table 7.

Table 7

Rainfall Parameters
Lopez (1973)

	Definition	Units
V_{RF}	Total volume of rainfall during storm considered average over the watershed. (Size of storm)	inches
E_{RF}	Volume of rainfall excess. Is considered equal to the volume of direct runoff.	inches
RF_{LOSS}	Volume of rainfall loss. That part of the total storm rainfall not appearing as runoff.	inches
T_{10}	Duration of total storm rainfall	minutes

PHYSICAL WATERSHED PARAMETERS

Since the work of Mulvaney in 1851, it was recognized that urbanization produces changes in the physical character of drainage basin. Some of these changes in the watershed change the watershed infiltration characteristics, the watershed response time, and the watershed storage characteristics.

The effect of urbanization on the changes in the hydrologic characteristics are sometimes influenced by local building codes, construction procedures or by requirements established for Federal assistance from VA, FHA and HUD. Various physical variables which other research has shown to be important hydrologic variables have been measured. Some of these variables have been combined into parameters and the effect of these variables or parameters was related to the runoff hydrograph by means of a stepwise multiple regression. The values of these rainfall parameters given in Table 8.

Table 8

Rainfall-Runoff Parameters for Denver Watersheds

Catchment	CSU No.	$Q_{P/A}$ (in./hr.)	V_{RF} (in.)	E_{RF} (in.)	RF_{LOSS} (in.)	T_{10} (min.)
Big Dry Cr. Trib. at Littleton, CO	1060702001	1.580	.17	.035	.135	25
	1060702002	1.069	.26	.025	.235	55
	1060702003	1.588	.22	.072	.148	15
	1060702004	1.596	.35	.072	.278	30
	1060702005	.641	.75	.245	.505	35
Sanderson Gulch Trib. at Lakewood, CO	1060716001	3.400	.22	.025	.195	15
	1060716002	4.000	.26	.012	.248	20
	1060716003	3.307	.49	.060	.430	15
	1060716004	2.194	.41	.110	.300	25
	1060716005	1.357	.32	.116	.204	70
Schneider Drain at Arvada, CO	1060798001	4.310	.05	.023	.027	10
	1060798002	3.437	.34	.048	.292	15
Toll Gate Cr. Trib. at Aurora, CO	1060742302	2.753	.71	.240	.470	70
	1060742305	2.949	1.09	.245	.845	30
	1060742307	2.360	.65	.260	.390	100
	1060742306	2.484	.44	.145	.295	35
Westerly Cr. Trib. at Aurora, CO	1060742708	1.294	.64	.330	.310	35
Concourse D Storm Drain at Stapleton Airport	1060743002	3.240	.37	.148	.222	30
Tuck Drain at Northglenn, CO	1060701001	4.272	.14	.068	.072	15
	1060701002	3.143	.38	.252	.128	40
	1060701003	3.653	.24	.176	.064	35
	1060701008	5.800	.27	.108	.162	25
	1060701011	3.099	.30	.224	.076	25
Hillcrest Drain at Northglenn, CO	1060703001	5.246	.58	.040	.540	40
	1060703002	4.660	.22	.010	.210	35
	1060703003	3.188	.31	.028	.282	45
	1060703009	3.586	.16	.032	.410	30
	1060703010	3.514	.30	.055	.245	30
	1060703015	--	.19	.108	.082	50
	1060703016	4.660	.24	.025	.215	20
Kennedy Drive Drain at Northglenn, CO	1060704001	6.632	.41	.108	.302	70
	1060704002	5.423	.23	.074	.156	40
	1060704003	6.632	.38	.145	.235	25
	1060704004	6.632	.24	.128	.112	20

The definitions of the physical watershed variables of the basin and the methods used in obtaining them are given as follows:

a) Length of Paved Streets and Roads, L_{PSR}--The length of all paved streets and roads, in miles, was obtained from aerial photographs and records kept by the city engineer's offices.

b) Length of Curbed and Guttered Streets, L_{CG}--The length of paved, curbed and guttered streets, in miles, was obtained as in (a) above.

c) Length of Unpaved Streets and Roads, L_{USR}--The length of unpaved streets and roads, in miles, whether curbed and guttered or not was obtained as in (a).

d) Length of Streets and Roads, L_{SR}--The sum total of all lengths of streets and roads, in miles, whether paved, unpaved, with and without curb and gutter.

e) Length of Storm Sewer, L_{SS}--The length of storm sewers in miles, was obtained from "as built" drawings generally kept at the jurisdictional city engineer's offices. Design drawings and field measurements were also used to obtain this variable.

f) Average Width of Curbed and Guttered Streets, W_{CGS}--This variable, in feet, was calculated with the following equation:

$$W_{CGS} = \frac{\Sigma\ l_m W_m}{\Sigma\ l_m}$$

where l_m is the length of a reach of street of constant width W_m. The widths of the streets were measured from the back of the paved walks on each side. As-built and design drawings, aerial photos and field measurements were the source of this data.

g) Slope of Curbed and Guttered Streets, S_{CGS}--This variable in feet per foot, was calculated with the following equation:

$$S_{CGS} = \frac{2\Sigma\ l_i Z_i}{(\Sigma\ l_i)^2} = \frac{2\Sigma\ l_i Z_i}{L^2_{CG}}$$

where l_i is the distance measured between successive contours along the paved curbed and guttered streets, and z_i is the average elevation above the outlet for each l_i which means that the elevation of the outlet must be subtracted from each average elevation of l_i. Large scale contour maps were often obtained from the jurisdictional city engineer's offices, which made the necessary measurements not only easier but more accurate.

h) Catchment area, A, and Perimeter, P. -- The values of the areas, in square miles, used in this work were reported by Ducret and Hodges (1972) and checked by Lopez (1973) using 7.5 minute USGS quadrangle sheets. The area and the perimeter were obtained using a digitizer and a computer program, but could be obtained with a planimeter and a paper strip quite easily.

i) Density of Paved Streets and Roads, D_{PSR}--This parameter, expressed in miles per square mile, was calculated using the equation:

$$D_{PSR} = \frac{L_{PSR}}{A} \ ,$$

where L_{PSR} is the length of paved streets and roads, and A the area of the basin.

j) Density of Paved Curbed and Guttered Streets, D_{CGS}--This parameter, expressed in miles per square mile, was calculated using the equation:

$$D_{CGS} = \frac{L_{CG}}{A} \ ,$$

where L_{CG} is the length of paved curbed and guttered streets, and A is the area of the catchment.

k) Density of Unpaved Streets and Roads, D_{USR}--This parameter, expressed in miles per square mile, was calculated using the equation:

$$D_{USR} = \frac{L_{USR}}{A} \ ,$$

where L_{USR} is the length of unpaved streets and roads, and A is the area of the catchment.

l) Total Street and Road Density, D_{SR}--This parameter, expressed in miles per square mile, was calculated using the equation:

$$D_{SR} = D_{PSR} + D_{USR} \ ,$$

where D_{PSR} and D_{USR} are the densities of paved and unpaved streets and roads respectively.

m) Average Hydraulic Capacity of Curbed and Guttered Streets, Q_{CGS}--This value, in inches per hour, was calculated using Manning's formula with the value n = 0.013, the slope of the curbed and guttered paved streets, S_{CGS}, and the average cross sectional area obtained for the average width of curbed and guttered streets, W_{CGS}.

n) Average Storm Sewer Diameter, D_{SS}--This variable, expressed in inches, was calculated using the equation:

$$D_{SS} = \frac{\Sigma l_j d_j}{\Sigma\ l_j} \ ,$$

where l_j is the length of a reach of storm sewer of constant diameter d_j.

o) Average Slope of Storm Sewer, S_{SS}--This variable, expressed in feet per foot, was calculated using the equation:

$$S_{SS} = \frac{\Sigma\ l_k S_k}{\Sigma\ l_k} \ ,$$

where l_k is the length of a reach of storm sewer of constant slope S_k.

p) Average Capacity of Storm Sewer System, Q_{SSS}--This parameter, was calculated using Manning's formula with a value of n = 0.013, the average storm sewer diameter D_{SS}, and the average slope of the storm sewer S_{SS}.

q) Average Hydraulic Capacity of Urban Area, C_Q--This parameter, expressed in inches per hour, was calculated with the equation:

$$C_Q = Q_{CGS} + Q_{SSS} \ ,$$

where Q_{CGS} and Q_{SSS} are the average capacities of the curbed and guttered streets and of the storm sewer system respectively.

Table 9

Physical Watershed Variables

Catchment	USGS No.	SWID No.	L_{PSR} mi.	L_{CG} mi.	L_{USR} mi.	L_{SR} mi.	L_{SS} mi.	L_{UCH} mi.	W_{CGS} ft.	D_{SS} in.	S_{CGS} %	S_{SS} %	Q_{CGS} in./hr.	Q_{SSS} in./hr.	C_Q in./hr.
Sanderson Gulch Tributary at Lakewood, CO	06711600	1060716001	7.70	7.70	0.120	7.82	0.050	0.440	48.00	12	2.34	1.200	0.27	0.118	0.39
		1060716002	7.70	7.70	0.120	7.82	0.050	0.440	48.00	12	2.34	1.200	0.27	0.118	0.39
		1060716003	7.82	7.82	0	7.82	0.050	0.440	48.00	12	2.34	1.200	0.27	0.118	0.39
		1060716004	7.82	7.82	0	7.82	0.050	0.440	48.00	12	2.34	1.200	0.27	0.118	0.39
		1060716005	7.82	7.82	0	7.82	0.050	0.440	48.00	12	2.34	1.200	0.27	0.118	0.39
Big Dry Creek Trib. at Littleton, CO	06710200	1060702001	13.40	11.30	0.434	13.83	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
		1060702002	13.40	11.30	0.434	13.83	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
		1060702003	13.40	11.30	0.434	13.83	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
		1060702004	13.40	11.30	0.434	13.83	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
		1060702005	13.40	11.30	0.434	13.83	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
		1060702006	13.40	11.30	0.434	13.83	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
		1060702007	13.60	11.4	0.434	14.03	0.900	0.542	48.36	66	2.29	2.000	0.13	0.760	0.89
Tuck Drain at Northglenn, CO	06720100	1060701001	1.17	1.17	0	1.17	0.142	0	49.00	30	2.82	0.476	23.47	0.620	24.09
		1060701002	1.17	1.17	0	1.17	0.142	0	49.00	30	2.82	0.476	23.47	0.620	24.09
		1060701003	1.17	1.17	0	1.17	0.142	0	49.00	30	2.82	0.476	23.47	0.620	24.09
		1060701008	1.17	1.17	0	1.17	0.142	0	49.00	30	2.82	0.476	23.47	0.620	24.09
		1060701009	1.17	1.17	0	1.17	0.142	0	49.00	30	2.82	0.476	23.47	0.620	24.09
		1060701011	1.17	1.17	0	1.17	0.142	0	49.00	30	2.82	0.476	23.47	0.620	24.09
Schneider Drain at Arvada, CO	06719800	1060798001	5.00	5.00	1.130	6.13	1.140	0	48.00	27	2.92	1.190	0.45	0.200	0.65
		1060798002	5.00	5.00	1.130	6.13	1.140	0	48.00	27	2.92	1.190	0.45	0.200	0.65
Concourse D Storm Drain at Stapleton Airport, Denver, CO	06714300	1060743002	0	0	0	0	0.950	0	0	49	0	0.853	0	1.970	1.97
		1060743003	0	0	0	0	0.950	0	0	49	0	0.853	0	1.970	1.97
Toll Gate Creek Trib. at Aurora, CO	06714230	1060742302	2.73	2.17	0.130	2.86	0.262	0.313	48.00	54	1.99	0.338	0.41	0.620	1.03
		1060742305	2.80	2.30	0.150	2.95	0.262	0.313	48.00	54	1.99	0.338	0.41	0.620	1.03
		1060742307	2.80	2.30	0.150	2.95	0.262	0.313	48.00	54	1.99	0.338	0.41	0.620	1.03
Westerly Creek Trib. at Aurora, CO	06714270	1060742708	2.86	2.86	0.520	3.38	0	0	48.00	0	0.83	0	0.39	0	0.39
Hillcrest Drain at Northglenn, CO	06720300	1060703001	5.63	5.63	0.013	5.64	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
		1060703002	5.63	5.63	0.013	5.64	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
		1060703003	5.63	5.63	0.013	5.64	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
		1060703009	5.63	5.63	0.013	5.64	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
		1060703010	5.63	5.63	0.013	5.64	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
		1060703015	5.63	5.63	0.013	5.64	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
		1060703016	5.63	5.63	0.022	5.65	0.192	0.208	48.00	26	2.44	2.720	0.49	0.26	0.75
Kenneth Drive Drain at Northglenn, CO	06720400	1060704001	1.78	1.78	0	1.78	0.729	0	48.60	37	2.03	1.000	1.27	1.12	2.39
		1060704002	1.78	1.78	0	1.78	0.729	0	48.60	37	2.03	1.000	1.27	1.12	2.39
		1060704003	1.78	1.78	0	1.78	0.729	0	48.60	37	2.03	1.000	1.27	1.12	2.39
		1060704004	1.78	1.78	0	1.78	0.729	0	48.60	37	2.03	1.000	1.27	1.12	2.39

Table 10

Physical Watershed Variables

Catchment	USGS No.	SWID No.	A sq.mi.	D_{CGS} mi./sq.mi.	D_{PSR} mi./sq.mi.	D_{USR} mi./sq.mi.	D_{SR} mi./sq.mi.	P mi.	L_L mi.	H_R mi.	F_L	A_{PSR} %	I_A %	U
Sanderson Gulch Tributary at Lakewood, CO	06711600	1060716001	0.50	15.40	15.40	0.240	15.64	2.569	1.00	0.188	0.500	14.00	49	1.49
		1060716002	0.50	15.40	15.40	0.240	15.64	2.569	1.00	0.188	0.500	14.00		
		1060716003	0.50	15.64	15.64	0	15.64	2.569	1.00	0.188	0.500	14.22		
		1060716004	0.50	15.64	15.64	0	15.54	2.569	1.00	0.188	0.500	14.22		
		1060716005	0.50	15.64	15.64	0	15.54	2.569	1.00	0.188	0.500	14.22		
Big Dry Creek Trib. at Littleton, CO	06710200	1060702001	1.0	11.30	13.40	0.434	13.93	4.869	2.10	0.205	0.227	12.27	28	1.28
		1060702002	1.0	11.30	13.40	0.434	13.93	4.869	2.10	0.205	0.227	12.27		
		1060702003	1.0	11.30	13.40	0.434	13.93	4.869	2.10	0.205	0.227	12.27		
		1060702004	1.0	11.30	13.40	0.434	13.93	4.869	2.10	0.205	0.227	12.27		
		1060702005	1.0	11.30	13.40	0.434	13.93	4.869	2.10	0.205	0.227	12.27		
		1060702006	1.0	11.30	13.40	0.434	13.93	4.869	2.10	0.205	0.227	12.27		
		1060702007	1.0	11.40	13.60	0.434	14.03	4.869	2.10	0.205	0.227	12.46		
Tuck Drain at Northglenn, CO	06720100	1060701001	0.07	16.71	16.71	0	16.71	1.226	0.45	0.057	0.346	15.51	50	1.50
		1060701002	0.07	16.71	16.71	0	16.71	1.226	0.45	0.057	0.346	15.51		
		1060701003	0.07	16.71	16.71	0	16.71	1.226	0.45	0.057	0.346	15.51		
		1060701008	0.07	16.71	16.71	0	16.71	1.226	0.45	0.057	0.346	15.51		
		1060701009	0.07	16.71	16.71	0	16.71	1.226	0.45	0.057	0.346	15.51		
		1060701011	0.07	16.71	16.71	0	16.71	1.226	0.45	0.057	0.346	15.51		
Schneider Drain at Arvada, CO	06719800	1060798001	0.33	15.15	15.15	3.424	18.58	2.548	0.90	0.130	0.407	13.77	25	1.25
		1060798002	0.33	15.15	15.15	3.424	18.58	2.548	0.90	0.130	0.407	13.77		
Concourse D Storm Drain at Stapleton Airport, Denver, CO	06714300	1060743002	0.11	0	0	0	0	1.594	0.55	0.069	0.364	100.00	100	2.00
		1060743003	0.11	0	0	0	0	1.594	0.55	0.069	0.364	100.00		
Toll Gate Creek Trib. at Aurora, CO	06714230	1060742302	0.30	7.23	9.10	0.430	9.53	2.240	0.70	0.134	0.612	8.27	37	1.37
		1060742305	0.30	7.66	9.33	0.500	9.83	2.240	0.70	0.134	0.612	8.48		
		1060742307	0.30	7.66	9.33	0.500	9.83	2.240	0.70	0.134	0.612	8.48		
Westerly Creek Trib. at Aurora, CO	06714270	1060742708	0.20	14.30	14.30	2.600	16.90	1.665	0.60	0.120	0.556	13.00	39	1.39
Hillcrest Drain at Northglenn, CO	06720300	1060703001	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28	46	1.46
		1060703002	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28		
		1060703003	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28		
		1060703009	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28		
		1060703010	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28		
		1060703015	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28		
		1060703016	0.28	20.11	20.11	0.050	20.14	2.067	0.70	0.135	0.571	18.28		
Kennedy Drive Drain at Northglenn, CO	06720400	1060704001	0.10	17.80	17.80	0	17.80	1.717	0.60	0.058	0.278	16.38	55	1.55
		1060704002	0.10	17.80	17.80	0	17.80	1.717	0.60	0.058	0.278	16.38		
		1060704003	0.10	17.80	17.80	0	17.80	1.717	0.60	0.058	0.278	16.38		
		1060704004	0.10	17.80	17.80	0	17.80	1.717	0.60	0.058	0.278	16.38		

r) Longest Dimension of the Basin, L_L--This variable, expressed in miles, was obtained by measuring the longest straight line distance between two points on the perimeter of the basin.

s) Form Factor, F_L--This dimensionless parameter was obtained with the equation:

$$F_L = \frac{A}{L^2_L} \quad ,$$

where A is the area of the catchment, and L_L is the longest dimension of the basin.

t) Hydrologic Radius, H_R--This parameter, expressed in square miles per mile, was calculated using the equation:

$$H_R = \frac{A}{P} \quad ,$$

where A and P are the area and the perimeter of the basin respectively.

u) Percent of Area in Paved Streets and Roads, A_{PSR}--This parameter was calculated with the equation:

$$A_{PSR} = \frac{100 L_{PSR} \, W_{CGS}}{A}$$

where L_{PSR} is the length of paved streets and roads, W_{CGS} is the average width of curbed and guttered streets, and A the area of the catchment. In this case the average width of curbed and guttered streets is used instead of the average width of paved streets and roads.

v) Percent of Impervious Area, I_A--This parameter is defined by the equation:

$$I_A = \frac{A_i}{A} \times 100 \quad ,$$

where A_i is the impervious area within the basin and A the basin area. Since a logarithmic transformation is to be performed on each descriptor the following definition is used:

$$U = 1 + \frac{A_i}{A}$$

which prevents zero values from occurring.

The determination of the impervious areas used in this investigation was carried out by Root and Miller (1971). The authors identified the impervious areas using remote multispectral sensing and analyzed thirteen of the small experimental catchments described by Ducret and Hodges (1972). Airphotos for each of the catchments were taken during the months between April and August and used in the analysis. Root and Miller claim that "*changes in impervious cover with time due to urban development, can be detected from this data to within five percent.*" Since year-to-year interpolation of the percent of imperviousness data was necessary, errors could have been introduces in the process. The dates of the photographs used for a given basin sometimes span five or more years and in rapidly urbanizing basins the error of interpolation could be significant.

The data for the physical watershed variables and parameters are given in Tables 9 and 10. Two tables were used because of the large list of variables evaluated. The Concourse D Storm Drain at Stapleton Airport is entirely paved. Many of the variables defined for a more "normal" urban watershed were not evaluated.

Chapter 5
CORRELATION STUDIES

The variables can be divided into three basic groups, 1) Unit Hydrograph Parameters, 2) Storm parameters and 3) Physical Watershed Variables (some of which are combined into Parameters). The objective is to find the simplest and most accurate regression equations such that the unit hydrograph parameter may be predicted for a future urbanizing region given that we know the storm rainfall characteristics and knowledge about the physical watershed characteristics which the new urbanizing region will have. These regression equations will be developed from the actual observations of flood hydrographs from measured rainfalls on a group of watersheds having various physical watershed properties.

Draper and Smith (1966) had concluded that a Stepwise Multiple Regression was a recommended method for obtaining these regression equations. A correlation matrix was prepared from all of the data from the 37 observed floods utilizing the measurements of the 27 variables and parameters. This correlation matrix is given in three-part Table 11.

STEPWISE MULTIPLE REGRESSION ANALYSIS

The multiple regression analysis in this work was done using the program STAT 38R (BDMO2R revised), provided as a standard software package by the Statistics Department through the CSU Computer Center. This program computes a sequence of multiple linear regression equations in a stepwise manner (or log transformed linear regression equations). At each step one variable is added to the regression equation. The variable added is the one which makes the greatest reduction in the sum of the squares of the deviations. Equivalently it is the variable which has the highest partial correlation with the response variable partially correlated with the variables which have already been added; and equivalently it is the variable which, if it were added, would yield the highest F value. In addition, variables are automatically removed when their F values become too low. Logarithmic transformations of the variables are performed using the transgeneration capabilities of the program and linear models of nonlinear variables can be obtained. Several regression equations may be formulated in a single run by creating subproblems with different dependent and independent variables.

Draper and Smith (1966) are of the opinion that the stepwise regression procedure is the "*best of the variable selection procedures*" discussed in their textbook. Careful selection of variables and critical evaluation of the models through the examination of residuals is essential in regression analysis.

The problem of identification of the best multiple linear regression model is discussed by Kisiel (1972). Given the general multiple linear regression model:

$$Y = \beta_0 + \sum_{j=1}^{J} \beta_j X_j + \xi \qquad (5)$$

the magnitude of J, the appropriate X_j and the values of β_j which give the "best" regression model are desired. The following assumptions are made as given by Kisiel (1972):

1) *Correctness of overall model form.*
2) *Independence or orthogonality of the X_j's in relation to each other.*
3) *Values of each of the independent (predictor) variables are known without error.*
4) *The observations on each observation of X_j and Y are independent of each other (zero serial correlation).*
5) *The entire data set is representative not only of values around the mean value of Y but also of behavior near the extremes. Future values Y of the estimated equation depend on whether the data has "captured" the sum total of system behavior.*
6) *The error (residual or random component) for the observed system response is normally distributed with zero mean and constant variance Var ξ. This assumption is necessary only to obtain confidence limits and to conduct tests of significance but is not essential to least squares estimation of model parameters.*
7) *The X_j's include any functions such as squares, cubes, cross products (for example, X_1X_2 or $X_1X_2X_3$). These nonlinear terms assume the form of the X_j's and are handled in the straight multiple linear regression (MLR) model. The model is linear because the terms are additive.*

As noted by Draper and Smith (1966, p. 163), choice of the best regression model requires the balancing of two opposing criteria:

1) *A useful and reliable predictive equation should include as many X_j's as possible. Reliability would be measured in terms of the standard error of estimate (fitted on past data) and standard error of prediction (a check of prediction against actual "future" data).*
2) *Cost considerations in collecting data on all X_j's of potential interest force us to include as few X_j's as possible in Eq. (5).*

In the analysis undertaken here all the conditions necessary to obtain the best linear regression model as outlined by Kisiel (1972) are not present. The choice of the linear regression model may not be the best choice; however, Rao, Delleur and Sarma (1972) have shown that the linear model may be a statisfactory model. Even if the correctness of the overall model form is accepted, the remaining assumptions cannot be made without loss of rigor and cannot be honestly made in some cases. Assumptions 2, 3 and 4 are violated because orthogonality of the independent variables does not exist in all cases; error-free independent variables are nonexistent and serial correlation is known to exist in rainfall-runoff data as well as in parameters of urban growth even though they may be stochastic in nature.

Since the choice of the linear regression model is rarely the "best" in actual practice, one must evaluate how badly the above assumptions deviate from the ideal. If the stepwise regression procedure is used, the effect of dependency between assumed independent variables results in a reduction of the partial F value of these variables thereby minimizing their probability of inclusion into the regression equation for a given confidence limit. The effect of errors in the independent variables can be minimized if a confidence limit is chosen where the residuals are normally distributed or nearly so. The Central Limit Theorem can be used to argue that the residuals should become normally distributed as the number of errors making it up increase, regardless of the original distribution of the individual errors. In this analysis assumptions 5 and 7 are reasonable in the case of the physiographic parameters but higher rainfall-runoff values would have improved the range of applicability of the equations. Criteria 1 and 2 are observed by giving equations which contain the maximum information by inclusion of variables that explain the largest percentage of the variance as measured by the coefficient of determination, and equations which have the least possible number of variables for a partial F value corresponding to the 95 percent point of the F distribution. The effect of serial dependence in rainfall compared to that of runoff may be considered negligible as a rule. Serial dependency in runoff is a well known phenomena in hydrology, but its evaluation in the Denver area has not yet been possible due to the lack of appropriate data. The degree of serial dependency for values of daily flows would give a minimum time interval necessary for independence of rainfall-runoff events. Autoregressive linear models can easily be used to analyze serial dependency (Yevjevich, 1972a).

RESULTS OF CORRELATION STUDIES

Stepwise regression was used to select the best predictors for the descriptors of response time, peak value of the unit hydrograph, time to rise of the unit hydrograph and lumped rainfall losses. All the variables used in the analysis were transformed into logarithmic form in order to normalize the descriptors as much as possible, and to formulate a multiplicative model for the desired regression equations. The transformation is also desirable to increase the probability of obtaining normally distributed residuals of the transformed values which makes the residuals lognormally distributed.

Unit Hydrograph Parameters.--The peak flow of the unit hydrograph was obtained by using three different programs to derive the unit hydrograph. The results were compared to make sure that reasonable values were used in this study. A unit time interval of five minutes was used for the computation of unit hydrographs with these programs.

The first program (HEC) used was furnished by the Hydrologic Engineering Center, U.S. Army, Corps of Engineers, USCE (1966). The program determines the unit hydrograph, loss coefficients and reproduces the runoff hydrograph from the rainfall event. Best reproduction is measured by the least squares of the difference between the computed and the observed flows. The unit hydrograph is computed from the Clark coefficients, time of concentration and routing coefficient, and a time-area histogram. The program also computes Snyder's C_t and C_p for the unit hydrograph.

A trial run was made with HEC and 14 out of 37 events gave unit hydrographs which reproduced the corresponding runoff hydrographs quite well. Several trial runs were then made with the remaining 23 events following suggested instructions to improve the fit of the computed runoff hydrographs. Improved fits were obtained for an additional 17 events.

The second program (PWCB) used was written by Cheng (1970) for use by the Taiwan Provincial Water Conservancy Bureau. This program derives the unit hydrograph from rainfall and discharge records. By separating the base flow from the observed hydrograph the direct runoff volume and the intensities of the effective rainfall are obtained. A set of normal equations of simultaneous equations are derived for a given rainfall-runoff event by the method of least squares and the ordinates of the unit hydrograph are obtained from the solution of the equations by matrix inversion. The ordinates of the direct runoff hydrograph can then be expressed in terms of the ordinates of the unit hydrograph and the intensities of the effective rainfall.

Cheng's program produced 20 unit hydrographs out of 37 events. The peak values from these unit hydrographs were in close agreement with those obtained with HEC. In order to have a basis for comparison, the absolute value of the differences were averaged for the peaks obtained with the two programs, and an average difference of 0.304 in./hr. obtained. The number of times that the results obtained by one program exceeded the other's were almost equal. A great deal of oscillation was observed in the remaining 17 unit hydrographs giving unreasonable values for the peak discharge and the time to peak.

The third program used (FINVER), was written by Kavvas (1972). The theoretical basis for FINVER is similar to that of Mr. Cheng's program. The difference between the two lies in the methods of solution with FINVER making more extensive use of computer software available from the University Computer Center.

Using matrix algebra the ordinates of the unit hydrograph can be expressed by the equation:

$$[U] = [I^T \; I]^{-1} \, [I]^T \, [Q] \tag{6}$$

where I represents the effective rainfall intensities and Q the direct runoff. Using FINVER, 34 unit hydrographs were obtained from the 37 events available, and the peak values were in close agreement with those derived with the other programs. The average difference between FINVER and HEC values was 0.23 in./hr. with HEC's values being generally higher than FINVER's. Comparing the values obtained by the PWCB program, an average difference of 0.30 in./hr. was obtained with FINVER's values being generally lower. The maximum difference found between the peak values of the unit hydrographs derived with the three programs was 0.40 in./hr.

In summary the largest number of unit hydrographs were first obtained with program FINVER. The computed runoff hydrographs and their respective descriptors were then compared to the observed ones rejecting those that were obviously in error. Errors in the computed volume as compared to the observed volume of runoff were overlooked if the peak value and the time to rise were accurately reproduced. Since program FINVER has yielded the largest number of unit hydrographs, the time to rise and the peak values of these hydrographs were used in the regression analyses by Lopez (1973) and in this report.

Peak Discharge Q_p.--An initial run was made with the stepwise regression program STAT 38R, version of November 1972, originally BMD02R, of the Colorado State University Statistical Laboratory. In this run the transgenerated (transformed) values of the dependent variable, the peak flow of the unit hydrograph, was allowed to select the best predictors from a list of 24 descriptors (also transformed) which were: the percent of impervious area (U); six methods of defining the response time (T_1, T_2, T_3, T_4, T_5 and T_8) ; the duration of the rainfall event (T_{10}); three variables of the length of streets (L_{PSR}, L_{CG}, L_{SR}); a parameter of the average hydraulic capacity of the urban development (C_Q); the area of the basin (A); three parameters of the densities of the lengths of streets (D_{USR}, D_{CGS} and D_{SR}); a variable of the slope of curbed and guttered streets (S_{CGS}); the excess (or effective) rainfall (E_{RF}); the basin shape parameters (H_R and F_L); the area of paved streets and roads (A_{PSR}); the volume of rainfall (V_{RF}); and the lumped rainfall losses (RF_{LOSS}).

From the correlation matrix furnished by the program, the highest values obtained were the correlation coefficients of the response times T_2, T_3, T_4 and T_7 ranging from 0.8 to 0.9. These storm and watershed variables were correlated with the unit hydrograph peak discharge, T_{10}, D_{CGS}, D_{PSR}, D_{SR}, A_{PSR} and V_{RF} with correlation coefficients ranging from 0.5 to 0.6. In this run the best predictors chosen at the 95 percent confidence level were Lag Time ($T_{LC}(T_4)$), and the volume of rainfall V_{RF}, with an F ratio of 127.99 and a coefficient of determination of 0.84. The standardized values of the residuals were tested for log-normality (Yevjevich, 1972b) using the Smirnov-Kolgomorov statistic $\Delta = 0.087 < \Delta_o = 0.23$ for α = 0.05 and a sample size of 33, which shows an acceptable fit. From this the inference may be made that the assumed confidence level of 95 percent is really the tolerance limit at which the partial F value of 4.15 for inclusion and rejection will accept regression coefficients, and for which the hypothesis $H_o:\beta_i = 0$ is rejected running a risk of less than five percent of being wrong. If the variables are independent and the residuals are independent and $N(0, \sigma^2)$, the estimated coefficients would be the maximum likelihood estimates of the population values; but if the residuals are neither, the coefficients are the least squares estimates. In the equation mentioned above orthogonality cannot be argued because the response time is dependent on the volume of rainfall to some degree, but serial independence is more probably the result of selection of the events to minimize it.

Based on the results obtained in the initial run, six subproblems were formulated using respectively the five response time varialbes separately in each subproblem.

The regression equations obtained for an F value of 4.15 (95 percent level) and their respective coefficients of determination are:

$$Q_p = 6.321\, T_{PC}^{-0.082}\, T_{10}^{-0.249}\, L_{SR}^{-0.347}\, E_{RF}^{-0.287}$$

$$R^2 = 0.8375 \tag{7}$$

$$Q_p = 1.725\, T_{PB}^{-0.466}\, A_{PSR}^{0.681}$$

$$R^2 = 0.7363 \tag{8}$$

$$Q_p = 5.528\, T_3^{-0.571}\, D_{CGS}^{-.411}$$

$$R^2 = 0.8233 \tag{9}$$

$$Q_p = 32.223\, T_{LC}^{-0.867}\, V_{RF}^{-0.036} \tag{10}$$

(selected regression equation)

$$R^2 = 0.6874$$

$$Q_p = 0.606\, L_{SR}^{-0.330}\, D_{CGS}^{-.473}\, E_{RF}^{-0.305}$$

$$R^2 = 0.7320 \tag{11}$$

$$Q_p = 2.803\, T_{PB}^{-0.483}\, D_{CGS}^{0.506}$$

$$R^2 = 0.7186 \tag{12}$$

Note: The units of Q_p are in./hr. and therefore the effect of watershed size is at least partially removed.

It is interesting to note that the peak of the unit hydrograph given by Eq. (12) is, approximately, directly proportional to the square root of the density of curbed and guttered streets and inversely proportional to the square root of the time to rise of the observed hydrograph explaining 72 percent of the variance. Also it may be noted that the same percentage of the variance is explained by Eq. (10), as by Eq. (7), with the latter having two variables more, which gives a good indication of the capability of T_4 in explaining the effect of the physical characteristics of the basin since the volume of rainfall is independent of the basin characteristics. Response time T_5 was not included in the regression equation (11).

Wetz gives as a criterion for judging a satisfactory predictor that the F ratio for the equation be four times larger than the selected partial F_p value (Draper and Smith, 1966). All the equations shown above satisfy this criteria.

In the above equations, it is worth noticing that the unit peak flows are inversely proportional to the response time. This is in agreement with Snyder (1938), Komsatra (1969) and others. Figure 6 graphically illustrates the inverse proportionality of the peak and the response time, where $t_2 > t_1$ for $Q_{P1} > Q_{P2}$. The inverse proportionality of the length of streets and roads is probably associated with the choice of inches per hour as the units of runoff contain an inverse proportionality of the basin area. The inverse proportionality of the rainfall duration is important in that it describes an inherent characteristic of the rainfall events typical of the area

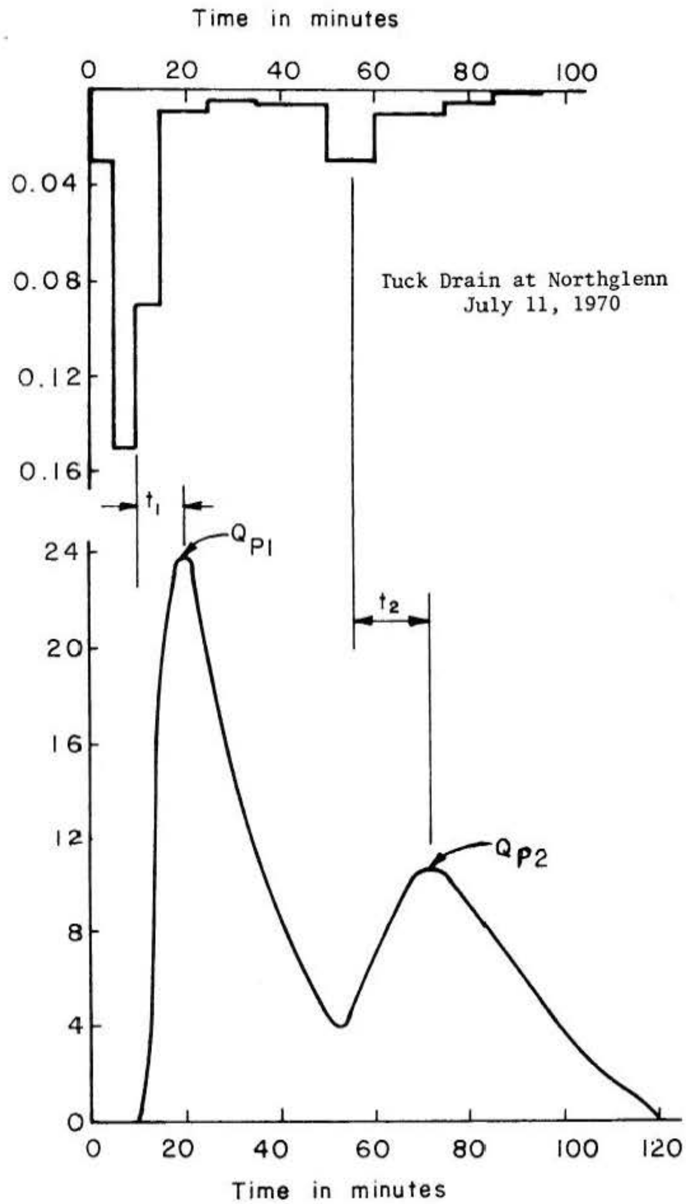

Fig. 6 Comparison of Q_p and Time to Peak

e.g., rainfall events in general are known to have a characteristic high intensity and short duration. The inverse proportionality of the volume of rainfall and the effective rainfall confirms the above statement. It should be noted that events with very wet antecedent conditions are not considered in this work purposefully. Events producing extreme floods are also not found in the data base to date. The characteristics of flood producing storms could conceivable be different to those here described. The effect of the characteristics of flood producing storms on the unit peak can be investigated using an approach similar to the one just presented.

In order to increase the explained variance of the regression equations derived, the inclusion of more predictors having an R^2 value of at least 0.01 was allowed and the following equations were obtained:

$$Q_P = 2.757\ T_{PB}^{-0.31611}\ A^{-0.272}\ E_{RF}^{-0.216} \qquad R^2 = 0.8424 \qquad (13)$$

$$Q_P = 7.388\ T_3^{-0.593}\ D_{CGS}^{0.269}\ RF_{LOSS}^{-0.094} \qquad R^2 = 0.860 \qquad (14)$$

$$Q_P = 43.939\ T_{LC}^{-0.822}\ C_Q^{0.109}\ E_{RF}^{-0.037}\ F_L^{0.578}$$

$R^2 = 0.8149$ (*Selected regression equation*) (15)

$$Q_P = 0.414\ T_{10}^{-0.187}\ C_Q^{-0.123}\ E_{RF}^{-0.276}\ H_R^{-0.911} \qquad R^2 = 0.7661 \qquad (16)$$

$$Q_P = 0.169\ C_Q^{-0.138}\ E_{RF}^{-0.315}\ H_R^{-0.981} \qquad R^2 = 0.7313 \qquad (17)$$

$$Q_P = 1.261\ U^{1.827}\ T_{PB}^{-0.339}\ E_{RF}^{-0.192}\ H_R^{-0.262} \qquad R^2 = 0.8521 \qquad (18)$$

$$Q_P = 9.118\ T_{LC}^{-0.538}\ F_L^{0.178}\ V_{RF}^{-0.121}\ S_{CGS}^{0.274} \qquad R^2 = 0.8714 \qquad (19)$$

Suspecting that Eq. (16) could be improved by forcing the basin shape descriptor F_L, the following equation was derived:

$$Q_p = 1.929\ T_{10}^{-.197}\ L_{SR}^{-.330}\ D_{CGS}^{.362}\ E_{RF}^{-.278}\ F_L^{.112} \qquad R^2 = 0.7750 \qquad (20)$$

The correlation coefficient between the length of streets and roads, L_{SR}, and the density of curbed and guttered streets D_{CGS} is 0.254 which shows low dependency between the variables. The equation including response time $T_{LC}(T_4)$, has again the highest coefficient of determination, but this time substituting the volume of rainfall by the rainfall excess and the physical parameters of the hydraulic capacity of the urban development and the shape of the basin. In order to avoid the introduction of excessive noise by including variables of low F values only variables contributing at least one percent of the explained variance were included. The residuals are lognormally distributed as determined by the Smirnov-Kolmogorov test which is the test used throughout this work for goodness of fit.

In conclusion $T_{LC}(T_4)$, the time interval between centroid of rainfall excess and the centroid of the direct runoff hydrograph, is the best descriptor of the response time for the determination of peaks of the unit hydrograph as evidenced by a correlation coefficient of 0.897 and the consistently higher coefficients of determination are obtained when the regression equation contains T_{LC}. This agrees with Wilson's (1972) results. The response time definition $T_{LC50}(T_5)$ had the poorest predictive capabilities.

Time to Rise T_R (T_7). -- The procedure followed in the analysis of the time to rise, or time to peak, of the unit hydrograph is similar to that followed in the previous section. The correlation matrix gave coefficient values from 0.75 to 0.85 for the response times (T_2, T_3 and T_4) and the peak of the unit hydrograph (Q_p), and values from 3.3 to 4.3 for all the basin variables. The best predictor for a partial F value of 4.15 (95 percent level) is the lag time $T_{LC}(T_4)$, giving the following equation:

$$T_R = 3.108\ T_{LC}^{0.554},\ R^2 = 0.7086 \tag{21}$$

$$T_R = 2.361\ T_{LC}^{0.594}\ E_{RF}^{-0.064} \qquad R^2 = 0.7338 \tag{22}$$

$$T_R = 3.533\ T_{LC}^{0.567}\ E_{RF}^{-0.078}\ S_{CGS}^{-0.442} \qquad R^2 = 0.7813 \tag{23}$$

$$T_R = 1.461\ T_{LC}^{0.680}\ E_{RF}^{-0.150}\ H_R^{-0.217}\ S_{CGS}^{-0.518} \qquad R^2 = 0.8019 \tag{24}$$

$$T_R = 4.624\ T_{LC}^{0.606}\ T_{10}^{-0.153} \qquad R^2 = 0.7313 \tag{25}$$

$$T_R = 3.379\ T_{LC}^{-0.625}\ T_{10}^{-0.115}\ E_{RF}^{-0.502} \qquad R^2 = 0.7465 \tag{26}$$

$$T_R = 1.855\ T_{LC}^{0.708}\ T_{10}^{-0.129}\ E_{RF}^{-0.096}\ H_R^{-0.146} \qquad R^2 = 0.7565 \tag{27}$$

$$T_R = 2.051\ U^{0.855}\ T_{LC}^{0.675}\ T_{10}^{-0.114}\ E_{RF}^{-0.076} \qquad R^2 = 0.7575 \tag{28}$$

The following equation were obtained by including $T_{PB}(T_2)$.

$$T_R = 6.525\ T_{PB}^{0.713}\ T_{10}^{-0.350} \qquad R^2 = 0.7557 \tag{29}$$

$$T_R = 8.006\ T_{PB}^{0.715}\ T_{10}^{-0.365}\ RF_{LOSS}^{0.095} \qquad R^2 = 0.9022 \tag{30}$$

$$T_R = 5.864\ T_{PB}^{0.739}\ T_{10}^{-0.334}\ E_{RF}^{-0.049}\ RF_{LOSS}^{0.088} \qquad R^2 = 0.8164 \tag{31}$$

$$T_R = 6.153\ T_{PB}^{0.709}\ T_{10}^{-0.322}\ C_Q^{-0.069} \qquad R^2 = 0.807 \tag{32}$$

$$T_R = 7.068\ T_{PB}^{0.712}\ T_{10}^{-0.340}\ C_Q^{-0.046} \qquad R^2 = 0.8175 \tag{33}$$

$$T_R = 9.326\ T_{PB}^{0.681}\ T_{10}^{-0.327}\ H_R^{0.116} \qquad R^2 = 0.841 \tag{34}$$

Lumped Rainfall Losses RF_{LOSS}.--In the determination of peak flows, the amount of rainfall not included in the measured discharge is an important variable. The volume of effective rainfall is widely used in the derivation of unit hydrographs, and can be obtained from the expressions given for an F value of 4.15 (95 percent level):

$$RF_{LOSS} = 0.448\ L_{CG}^{0.348}\ V_{RF}^{1.088} \qquad R^2 = 0.8867 \tag{35}$$

$$RF_{LOSS} = 0.439\ L_{PSR}^{0.334}\ V_{RF}^{1.0699} \qquad R^2 = 0.8855 \tag{36}$$

$$RF_{LOSS} = A^{0.303}\ V_{RF}^{1.024} \qquad R^2 = 0.8742 \tag{37}$$

The above expressions give the highest values of the coefficient of determination, their residuals are lognormally distributed and independent which makes the regression coefficients the maximum likelihood estimators of the population values since the predictors are indeed independent variables. The test for independence was based on the theory of runs and the unit normal deviate form of the residuals was used (Wallis and Matalas, 1971). The test was performed for a 98 percent confidence limit for which the hypothesis for independence was accepted if the number of runs was between nine and 22. The equation for the number of runs for a given confidence is given by C.L. $(n) - [(N-1) \pm Z_\alpha (N-1)^{\frac{1}{2}}]^{\frac{1}{2}}$, where n is the number of runs, Z_α the normal deviate for a probability α, and N the sample size.

A significant result in this analysis is the rejection of the percent of impervious area in favor of variables based on the length of streets, with the highest entry value observed belonging to the length of curbed and guttered streets (L_{CG}), and the second highest to the length of paved streets and roads. This result is intuitively appealing since the experimental basins used in this work are predominantly suburban, and typically, rooftop areas drain mostly into the lawns with one or two drains running into the driveways. The accuracy of this interpretation is enhanced by the fact that the descriptors of urbanization are independent of the volume of rainfall. A relationship including C_Q is given as follows:

$$RF_{LOSS} = 0.659\ C_Q^{-0.338}\ V_{RF}^{0.973} \qquad R^2 = 0.6753 \tag{38}$$

In examining the relationships derived one may also notice that:

a) the losses are directly proportional to the length of curbed and guttered streets, the length of paved streets and roads and the volume of rainfall;

b) The losses are very well predicted by the area of the basin and the volume of rainfall.

In (a) it is evident that a zero length of street will give zero losses which is incorrect, and that $RF_{LOSS} = f(L_S, V_{RF})$ loses its reliability as it approaches zero. The length of curbed and guttered streets does not include the street width. A better predictor should be the actual area of curbed and guttered streets. The correlation matrix shows that the percentage of the average area of paved streets is inversely proportional to the losses, but in the calculation of the paved area a weighted average of the width was multiplied by the length which may be the reason for the lower correlation value. The variable impervious area is inversely proportional to the losses as might be expected. To be safe the equations discussed should be used for basins predominantly suburban.

In (b) the fact that the losses are directly proportional to the volume of rainfall and to the cube root of the area of the basin does not seem to have a simple physical interpretation. Also defying explanation is the fact that the area of the basin is almost as good a predictor as the lengths of streets and much better than the percent of impervious area.

Response Time. -- The response of a basin to a given intensity duration and frequency of rainfall is of interest in hydrology. In this section several definitions of response time used in hydrology are used in order to assess their predictiveness.

Time to Peak $T_{PC}(T_1)$.-- None of the equations derived for this definition of response time passed the Wetz a test for satisfactory predictors. The highest F ratio obtained was 4.58 which was almost as small as the partial F_P value used for inclusion and rejection. The following equations were obtained by including all variables contributing at least one percent of the explained variance, and the best descriptors of urban development.

$$T_{PC} = 1.423\ U^{-6.405}\ T_{PB}^{2.440}\ T_{10}^{-1.449}\ E_{RF}^{-0.314}\ H_R^{-0.592} \qquad R^2 = 0.3922 \tag{39}$$

$$T_{PC} = 19.313\ T_{PB}^{2.136}\ T_{10}^{-1.348}\ C_Q^{0.613}\ E_{RF}^{-0.247}\ H_R^{1.491} \qquad R^2 = 0.4212 \tag{40}$$

$$T_{PC} = 69.738\ T_{PB}^{2.406}\ T_{10}^{-1.640}\ D_{CGS}^{-2.455}\ E_{RF}^{-0.541}\ H_R^{-0.884} \qquad R^2 = 0.4525 \tag{41}$$

$$T_{PC} = 678.828\ T_{PB}^{2.406}\ T_{10}^{-1.577}\ D_{PSR}^{-3.340}\ E_{RF}^{-0.580}\ H_R^{-0.86} \qquad R^2 = 0.4588 \tag{42}$$

$$T_{PC} = 1686.437\ T_{PB}^{2.379}\ T_{10}^{-1.642}\ D_{SR}^{-3.415}\ E_{RF}^{-0.548}\ H_R^{-0.757} \qquad R^2 = 0.4521 \tag{43}$$

Equations excluding T_{PB} were derived and the highest F ratio and R^2 values were obtained for the following equation:

$$T_{PC} = 29904.049\ T_{10}^{-0.338}\ C_Q^{0.917}\ E_{RF}^{0.288}\ H_R^{3.294} \qquad R^2 = 0.2211 \tag{44}$$

The F ratio for this equation was 1.99, and it should be emphasized that none of the equations presented in this section are considered reliable predictors of T_{PC}.

Time to Peak $T_{PB}(T_2)$.-- The duration of rainfall T_{10} was found to be the best predictor explaining 37 percent of the variance. The following equations include only variables which contribute at least one percent of the variance:

$$T_{PB} = 11.881\ T_{10}^{0.580}\ E_{RF}^{0.242}\ H_R^{0.550} \qquad R^2 = 0.5432 \tag{45}$$

$$T_{PB} = 51.749\ T_{10}^{0.512}\ C_Q^{0.265}\ E_{RF}^{0.250}\ H_R^{1.169} \qquad R^2 = 0.65014 \tag{46}$$

$$T_{PB} = 2.341\ T_{10}^{0.731}\ H_R^{0.241}\ V_{RF}^{1.791} \qquad R^2 = 0.4463 \tag{47}$$

$$T_{PB} = 9.558\ T_{10}^{0.666}\ C_Q^{0.260}\ H_R^{0.838}\ V_{RF}^{0.191} \qquad R^2 = 0.5492 \tag{48}$$

The equations given do not satisfy the Wetz criteria. Equation (46) has an F ratio of 13.01 which comes the closest to being a satisfactory predictor equation.

Time to Peak T_3.--The time to peak of the runoff hydrograph T_{PB} is an excellent predictor of T_3 explaining 76 percent of the variance with an F ratio of 98.22:

$$T_3 = 2.078\ T_{PB}^{0.849},\quad R^2 = 0.7601 \tag{49}$$

The equation can be improved without losing its predictive value. The following equations improve the total explained variance as shown:

$$T_3 = 9.453\ U^{-0.990}\ T_{PB}^{0.567}\ T_{10}^{0.161}\ E_{RF}^{0.177}\ H_R^{0.234}\qquad R^2 = 0.8458 \tag{50}$$

$$T_3 = 17.243\ T_{PB}^{0.504}\ T_{10}^{0.177}\ C_Q^{0.125}\ E_{RF}^{0.193}\ H_R^{0.636}\qquad R^2 = 0.8642 \tag{51}$$

$$T_3 = 7.320\ T_{PB}^{0.570}\ T_{10}^{0.176}\ E_{RF}^{0.173}\ H_R^{0.309}\qquad R^2 = 0.8379 \tag{52}$$

$$T_3 = 2.003\ U^{-0.862}\ T_{PB}^{0.730}\ T_{10}^{0.195}\ V_{RF}^{0.021}\qquad R^2 = 0.7870 \tag{53}$$

$$T_3 = 2.670\ T_{PB}^{0.700}\ T_{10}^{0.215}\ C_Q^{0.092}\ H_R^{0.0300}\qquad R^2 = 0.7961 \tag{54}$$

$$T_3 = 2.376\ T_{PB}^{0.731}\ T_{10}^{0.179}\ D_{CGS}^{-0.163}\qquad R^2 = 0.7825 \tag{55}$$

$$T_3 = 1.371\ T_{PB}^{0.756}\ T_{10}^{0.191}\qquad R^2 = 0.7771 \tag{56}$$

$$T_3 = 10.739\ T_{PB}^{0.650}\ E_{RF}^{0.181}\ H_R^{0.293}\qquad R^2 = 0.8243 \tag{57}$$

$$T_3 = 14.811\ T_{PB}^{0.645}\ E_{RF}^{0.271}\ H_R^{0.441}\ V_{RF}^{-0.221}\qquad R^2 = 0.8512 \tag{58}$$

$$T_3 = 35.293\ T_{PB}^{0.580}\ E_Q^{0.125}\ E_{RF}^{0.291}\ H_R^{0.768}\ V_{RF}^{-0.221}\qquad R^2 = 0.8772 \tag{59}$$

$$T_3 = 23.706\ T_{PB}^{0.492}\ T_{10}^{0.190}\ C_Q^{0.125}\ E_{RF}^{0.285}\ H_R^{0.793}\ V_{RF}^{-0.230}\quad R^2 = 8930 \tag{60}$$

Excluding T_{PB} from the regression analysis, the following equations were obtained:

$$T_3 = 98.808\ T_{10}^{0.415}\ C_Q^{0.197}\ E_{RF}^{0.319}\ H_R^{1.062}\qquad R^2 = 0.7497 \tag{61}$$

$$T_3 = 33.108\ T_{10}^{0.466}\ E_{RF}^{0.313}\ H_R^{0.602}\qquad R^2 = 0.6795 \tag{62}$$

$$T_3 = 17.782\ T_{10}^{.435}\ C_Q^{.090}\ A^{.555}\ E_{RF}^{.357}\ RF_{LOSS}^{.150}\qquad R^2 = 0.7641 \tag{63}$$

These equations are satisfactory predictors according to the Wetz criteria. The lowest F ratio of 17.49 was obtained for Eq. (63).

Equations including the total volume of rainfall, but excluding the time to peak of the runoff, T_{PB}, do not satisfy the Wetz criteria or explain as much of the variance as those including the rainfall excess. The following equation has an F ratio of eight and explains the largest percentage of the variance of the equations containing the volume of rainfall:

$$T_3 = 10.090\ T_{10}^{0.634}\ C_Q^{0.189}\ H_R^{0.649}\ V_{RF}^{0.186}\qquad R^2 = 0.5308 \tag{64}$$

Lag Time $T_{LC}(T_4)$.--The time to peak of the runoff hydrograph (T_{PB}) is a good predictor of T_{LC}, but explains a smaller percentage of the variance than in the cases of T_{PB} or T_3. The equation:

$$T_{LC} = 1.375\ T_2^{0.848}\quad R^2 = 0.6041 \tag{65}$$

has an F ratio of 47.30 which proves the worth of this variable as a predictor. An improvement of the coefficient of determination is obtained with the following equations:

$$T_{LC} = 34.058\ U^{-1.614}\ T_{PB}^{0.541}\ E_{RF}^{0.227}\ H_R^{0.571}\qquad R^2 = 0.7855 \tag{66}$$

$$T_{LC} = 116.230\ T_{PB}^{0.437}\ C_Q^{0.228}\ E_{RF}^{0.256}\ H_R^{0.286}\qquad R^2 = 0.8383 \tag{67}$$

$$T_{LC} = 23.638\ T_{PB}^{0.557}\ E_{RF}^{0.220}\ H_R^{0.691}$$

$$R^2 = 0.7686 \qquad (68)$$

$$T_{LC} = 4.840\ U^{-1.402}\ T_2^{0.782}\ H_R^{0.278}$$

$$R^2 = 0.7077 \qquad (69)$$

$$T_{LC} = 13.346\ T_{PB}^{0.680}\ C_Q^{0.191}\ H_R^{0.833}\ V_{RF}^{0.105}$$

$$R^2 = 0.7485 \qquad (70)$$

$$T_{LC} = 8.576\ T_{PB}^{0.739}\ D_{CGS}^{-0.309}\ H_R^{0.332}$$

$$R^2 = 0.7084 \qquad (71)$$

$$T_{LC} = 13.237\ T_{PB}^{0.734}\ D_{PSR}^{-0.453}\ H_R^{0.336}$$

$$R^2 = 0.7117 \qquad (72)$$

$$T_{LC} = 10.743\ T_{PB}^{0.742}\ D_{SR}^{-0.367}\ H_R^{0.355}$$

$$R^2 = 0.7047 \qquad (73)$$

The greater inclusion of descriptors of urban development in the regression equations shows that T_{LC} is more sensitive than T_{PC}, T_{PB}, and T_3 to this phenomena. Comparing the above equations, a higher percentage of the variance is explained by the average hydraulic capacity as estimated by C_Q, the density of curbed and guttered street (D_{CGS}) and the density of paved streets and roads (D_{PSR}) than by the imperviousness factor, U . Care should be exercised in comparing results like this as will be shown below.

Excluding the time to peak of the runoff hydrograph, T_{PB}, from regression analysis the following equations were obtained.

$$T_{LC} = 210.233\ T_{10}^{0.031}\ E_{RF}^{0.323}\ H_R^{0.80}\ U^{-0.342}$$

$$R^2 = 0.613 \qquad (74)$$

$$T_{LC} = 338.361\ T_{10}^{0.048}\ E_{RF}^{0.304}\ H_R^{1.154}\ C_Q^{0.145}$$

$$R^2 = 0.656 \qquad (75)$$

$$T_{LC} = 125.817\ T_{10}^{0.292}\ D_{CGS}^{-0.145}\ E_{RF}^{0.339}\ H_R^{0.925}$$

$$R^2 = 0.6581 \qquad (76)$$

$$T_{LC} = 147.547\ T_{10}^{0.295}\ D_{PSR}^{-0.208}\ E_{RF}^{0.336}\ H_R^{0.924}$$

$$R^2 = 0.6586 \qquad (77)$$

$$T_{LC} = 131.996\ T_{10}^{0.296}\ D_{SR}^{-0.147}\ E_{RF}^{0.342}\ H_R^{0.942}$$

$$R^2 = 0.6568 \qquad (78)$$

These equations pass the Wetz criteria and should adequately predict T_{LC} when T_{PB} is not used. Comparing the last five equations, where only the descriptors of urbanization have been changed, one may see that the descriptor of the hydraulic capacity still explains a higher percentage of the variance followed now by the percent of impervious area. This may be thought to contradict the results mentioned previously, but in reality, it only points to the differing effects of the dependency between the variables entered into the regression equation. Many of the variables are not independent of each other. When adding a second variable related to the same general phenomenon adds to the coefficient of determination, which is readily apparent in comparing Eq. (68) to (73) with (74) to (78).

The equations obtained by substituting the total volume of rainfall V_{RF}, do not pass the Wetz criteria, with the best equation having an F ratio of 9.19 given here:

$$T_{LC} = 37.851\ T_{10}^{0.467}\ C_Q^{0.280}\ H_R^{1.172}\ V_{RF}^{0.252}$$

$$R^2 = 0.5677 \qquad (79)$$

Lag Time T_{LC50} (T_5).--This definition of lag time is poorly correlated with all the descriptors formulated in this analysis. The highest correlation coefficient obtained was 0.360 with the length of curbed and guttered streets. The best regression equation only had an F ratio of 3.11 and is given as follows:

$$T_{LC50} = 9514.807\ T_{PB}^{.570}\ T_{10}^{1.33}\ C_Q^{.711}\ E_{RF}^{.054}\ H_R^{.161}$$

$$R^2 = 0.3652 \qquad (80)$$

As can be seen this equation does not pass the Wetz criteria for a satisfactory predictor.

Time of Concentration T_C (T_8).--Values for this variable were obtained as described previously. Since poor correlations were obtained, the values computed with program HEC for this variable were used. Very poor correlations were obtained from these results also, with the highest coefficient being 0.456. The highest F ratio obtained for a relationship containing a descriptor of urbanization was 4.4 for Eq. (81):

$$T_C = 15.116\ D_{CGS}^{1.018}\ E_{RF}^{0.571}\ H_R^{0.783}$$

$$R^2 = 0.3147 \qquad (81)$$

$$T_C = 218.122\ E_{RF}^{0.756}\ H_R^{0.926}\ V_{RF}^{-0.751}$$

$$R^2 = 0.3664 \qquad (82)$$

The F value of 4.49 for Eq. (82) was the highest of the equations derived for T_C.

The correlation matrix including the twenty-seven variables and parameters in this investigation is presented in Table 11. In this table, rise time T_{71} is the value of the definition of response time T_7 used in the final regression analysis.

Table 11

Correlation Matrix

Variable Name	U	T_1	T_2	T_3	T_4	T_5	T_{10}	Q_P	L_{PSR}
U	1.000	-.291	-.200	-.226	-.390	-.190	-.058	.399	-.625
T_1		1.000	.439	.265	.418	.238	.022	-.465	.154
T_2			1.000	.929	.861	.162	.608	-.800	.102
T_3				1.000	.910	.088	.622	-.874	.302
T_4					1.000	.215	.450	-.897	.302
T_5						1.000	-.290	-.108	.097
T_{10}								-.502	-.135
Q_P								1.000	-.238
L_{PSR}									1.000

Variable Name	L_{CG}	L_{SR}	C_Q	A	D_{CGS}	D_{PSR}	D_{SR}	E_{RF}	F_R	F_L
U	-.576	-.652	.339	-.708	.606	.531	.413	.317	-.617	.305
T_1	.126	.160	.016	.239	-.402	-.407	-.398	-.005	.346	-.121
T_2	.069	.100	.063	.183	-.390	-.380	-.407	.410	.193	-.0"9
T_3	.054	.093	.030	.183	-.432	-.433	-.438	.528	.189	-.057
T_4	.267	.310	-.139	.400	-.512	-.506	-.489	.348	.431	.026
T_5	.306	.335	-.011	.283	.135	.168	.152	-.208	.273	-.243
T_{10}	-.074	-.142	.126	-.056	-.327	-.306	-.360	.406	-.056	.073
Q_P	-.193	-.250	.137	-.359	.597	.588	.562	-.466	-.348	.090
L_{PSR}	.936	.998	-.795	.974	-.239	-.158	-.133	-.604	.951	-.093
L_{CG}	1.000	.994	-.803	.952	-.156	-.081	-.053	-.625	939	-.068
L_{SR}		1.000	-.806	.976	-.254	-.178	-.136	-.599	.955	-.056
C_A			1.000	-.783	.183	.165	.091	.433	-.831	-.318
A				1.000	-.451	-.379	-.347	-.520	.971	-.096
D_{CGS}					1.000	.987	.967	-.145	-.395	.113
D_{PSR}						1.000	.966	-.199	-.343	.041
D_{SR}							1.000	-.219	-.308	.065
E_{RF}								1.000	-.501	.096
F_R									1.000	.129
F_L										1.000

Variable Name	A_{PSR}	V_{RF}	RF_{LOSS}	T_7	T_{71}	L_L	S_{CGS}	T_8
U	.538	-.026	-.247	.034	-.324	-.746	-.078	.164
T_1	-.402	.297	.213	.077	.465	.258	-.105	.072
T_2	-.374	.377	.032	.212	.809	.193	-.253	.257
T_3	-.428	.393	.054	.332	.748	.185	-.244	.306
T_4	-.507	.419	.212	.203	.841	.351	-.238	.288
T_5	.171	-.048	-.027	-.097	.267	.342	.262	.083
T_{10}	-.302	.306	.068	.360	.239	-.076	-.146	.088
Q_P	.587	-.540	-.321	-.407	-.751	-.356	.376	-.319
L_{PSR}	-.182	.060	.482	-.201	.354	.910	-.057	-.116
L_{CG}	-.106	.022	.458	-.211	.334	.882	-.042	-.108
L_{SR}	-.202	.053	.479	-.198	.362	.911	-.073	-.114
C_A	.199	-.138	-.573	.178	-.210	-.592	.396	.049
A	-.401	.160	.533	-.171	.411	.935	-.118	-.110
D_{CGS}	.986	-.451	-.385	-.065	-.352	-.447	.257	.039
D_{PSR}	.999	-.449	-.353	-.076	-.343	-.356	.276	.005
D_{SR}	.962	-.501	-.367	-.079	-.312	-.336	.222	.007
E_{RF}	-.185	.495	-.089	.490	.138	-.502	-.227	.456
F_R	-.369	.177	.536	-.160	.429	.829	-.136	-.056
F_L	.019	.114	.118	.052	-.062	-.442	-.134	.155
A_{PSR}	1.000	-.451	-.370	-.071	-.345	-.368	.286	.006
V_{RF}		1.000	.663	.245	.349	.103	-.411	.078
RF_{LOSS}			1.000	.045	.221	.439	-.393	-.056
T_7				1.000	.001	-.172	-.005	.411
T_{71}					1.000	.394	-.384	.195
L_L						1.000	-.059	-.154
S_{CGS}							1.000	-.131
T_8								1.000

Many cities are considering enacting on-site storage ordinances. Calculating the urban flood hydrograph will have to take into account the existence and storage capacity of any on-site storage.

The results presented here show an insensitivity to channel slope and therefore are not in agreement with the results of Eagleson (1962), Carter (1961), Dempster (1974), or the Bureau of Reclamation, USBR (1965). The multiple regression methods used in this research on the available data did not find that the channel slope was a significant variable because for the data available the variation in the channel slope was not large. Under these conditions, it is quite possible that the channel slope would not be selected as an important variable. There are two watersheds in the Boulder area (Skunk Creek and Two-mile Canyon) which are very steep. Lopez did not have any flood events measured on these two watersheds.

Retardation Ponds.--The difference between the equations for computing the peak flood from Houston urban watersheds and the Dallas urban watersheds is in part attributed to the construction of flood retarding structures and detention ponds at convenient points in the Dallas area. A practice which is becoming more popular is to incorporate parks, golf courses and bridle paths into floodways and temporary storage for flood waters. There may be some difficulty in maintaining grass in some of these places if they are subjected to frequent inundation.

The existence of on-site storage or detention ponds in the urban drainage network can be taken into account by actually routing the hydrograph through the detention facility. If the storage volume is minor in comparison to the volume of the total hydrograph, the peak attenuation will be minor and the effect of this type of storm could be accounted for in the adjustment of Lopez's Hydraulic Capacity or Espey's Channelization Factor, ϕ.

NEEDS FOR ADDITIONAL RESEARCH

As in the case with many research projects, more questions have been raised.

Imperviousness in the Urban Watershed.--A detailed survey should be made of the fate of the runoff from the impervious parts of the urban watershed. In the residential zones in the Denver Metropolitan Area for the relatively small storms observed, it appears that only a minor part of the roof drainage appears as surface runoff because the roof downspouts discharge onto grassed areas. The other impervious areas may contribute both flood runoff varying concentrations of pollutants and sediment. This research was mainly concerned about the flood runoff, but the day is coming when plans must be made to treat this flood runoff. The fate of the runoff from the impervious parts of the watersheds must be known. Two of the watersheds in the Denver Area-Concourse D at Stapleton Airport and Kennedy Drive at Northglenn are well suited for this research work.

On-site Storage of Flood Runoff. -- Routing the flood hydrograph through each element of on-site storage will be a tedious task. There is need for a simple empirical technique for predicting the influence of this type of runoff given the storm characteristics. A procedure must be developed to relate the status of any on-site storage to modification of the imperviousness factor and the hydraulic capacity factor.

Detention Ponds.--The performance of the flood detention ponds on urban runoff needs to be investigated. Their function is similar to the on-site storage except that the scale is larger. Since there are less opportunities for building detention basins than on-site storage, simply routing the flood through the detention basin may be a practical way of obtaining the hydrograph at a downstream point. Routing of the flood through the detention basin will yield stage hydrograph in the basin. This stage hydrograph will give information about depth of inundation and duration of inundation. These are important when considering the joint use of the detention basin for parks or recreational purposes. One watershed in the Denver Metropolitan Area has a Detention Pond incorporated in a small park. This watershed (Harvard Gulch) is not being gaged in the Denver Urban Network.

Channel Slope.--The role of the channel slope on the watershed response needs to be investigated. As more data become available from the Denver Urban Network, these results will become available.

(1974) are given in terms of the peak discharge estimated to recur on the average of once in 25 years.

Converting the peak discharge data to a particular recurrence interval ia a useful step in developing a regionalized relationship; however, sufficient data must be available to enable extrapolation to enable moderate value say 25 years.

Response Time.--Lopez (1973) devoted a considerable effort to study the particular definition of the response time. He concluded that the Lag Time, T_{LC}, was the most desirable way to express the response time for an urban watershed. This generally confirms a similar conclusion made earlier by Wilson (1972). It would be more practical if the definition were the Time to Peak, T_{PC}. These two definitions produce quite different results if the hydrograph shape is unusual. The unusual shape of the hydrograph in turn seems to be related to the watershed basin characteristics as defined by:

$$\Omega = \frac{L\, L_{ca}}{\sqrt{s}} \quad \text{or}$$

$$\frac{L}{\sqrt{s}} \; .$$

Conversion to metric units would be greatly enhanced if a meaningful dimensionless parameter describing the general watershed characteristics could be derived. Lopez (1973) results have terms including the hydraulic capacity, C_Q, and the hydrologic radius, H_R. The hydrologic radius is Lopez's term to define the watershed shape which replaces the L_{ca} term in Snyder's definition and the L_t term in Van Sickle's definition.

It is interesting to note that Lopez found that the response time was sensitive to two variables involving storm magnitude and duration, E_{RF} and T_{10}. Minshall (1960) and others have found evidence of nonlinearity in the unit hydrograph related to rainfall intensity and to storm magnitude. Lopez shows that these influence the lag time.

Lopez (1973) found that the effect of urbanization on the response time was predominantly related to hydraulic characteristics of the drainage system rather than to imperviousness of the watershed. This conclusion seems to confirm the findings of Espey et al. (1969) who have shown that the Time to Peak in an urban watershed is related to both the channelization parameter, ϕ, and the percent of imperviousness, I.

Imperviousness Factor.--It seems clear that the future research will also find that the logarithmic transformation will be needed to cope with the inherent nonlinearity of the relationship; therefore, it is recommended that Carter's (1961) imperviousness factor be used in future research:

$$K = 1.00 + .015\, I \; . \tag{96}$$

The coefficient, .015, may be altered to suit the fate of the roof drainage. As more mandatory on-site storage of roof drainage is required, it may be that need for an imperviousness factor will disappear entirely under these conditions.

One possible explanation for relative insensitivity of Lopez's results to the imperviousness is the fact that the roof drainage is mainly onto flower beds and grassed areas in the Denver region. This in addition to the fact that Lopez had no really large storms in his data base. During a very large storm the roof drainage could be more directly linked to the street drainage.

Channel Efficiency.--Carlston (1963) found that in a pristine watershed, drainage density had a bearing on peak flood discharge. The real meaning of this observation is that, the peak flood is directly correlated with the efficiency with which the runoff is conveyed from the surface. A complimentary corollary is that as the drainage efficiency improves, the response time goes down. The drainage efficiency is related to how quickly the surface detention storage is converted from a thin laminar sheet to a relatively deeper turbulent flow in a small channel.

Urbanization accomplishes a higher drainage efficiency in a number of subtle ways. The imperviousness of the watershed has two hydrologic functions--1) The infiltration into the watershed is reduced resulting in a larger proportion of surface funoff; 2) the surface runoff occurs over a smoother surface than the natural watershed and hence the velocity is higher. The imperviousness caused more of the total storm rainfall to appear as surface runoff and the runoff occurs faster. Hence we should expect to see floods more often (because even the minor rainfalls are no longer infiltrating) and the response time is reduced which in turn increases the peak discharge for a unit volume of runoff.

Urbanization that results in curbed and guttered streets causes reduced response time. The drainage density is greatly increased. Each mile of street has approximately two miles of channels in the gutters plus the sewer pipe into which the gutters discharge. The transit time of the flood hydrograph in a curb-gutter-sewer hydraulic system is faster than the transit time in the pristine watershed. The street and sidewalk are part of the impervious watershed which is quantified under imperviousness. The curbed gutter must also appear in the evaluation of urbanization. Lopez (1973) does this by means of the Hydraulic Capacity term. Espey et al. (1969) does this in a more subjective manner through the use of the ϕ term. Van Sickle (1969) does this by means of the length of channels plus storm drain in his Basin Factor term.

Implication of On-site Storage.--As the effects of urbanization have become recognized, proposals for delaying the watershed response time have been developed. Instead of allowing all of the surface detention to drain away immediately, selected parts of the watershed are designated for temporary storage. The runoff from these selected parts is retarded and the runoff stored for a matter of a few minutes to a few hours. The peak runoff from the retarded parts is thus desynchronized from the peak runoff from the remainder of the urban watershed. Roof tops, parking lots and runoff from large grassed areas can be managed in this way. Usually some type of weir or orifice is fitted at the outlet from these parts of the watershed. This causes water to back up behind the orifice and be stored. The rate of release being a function of the characteristics of the orifice.

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Chapter 7
SUMMARY AND CONCLUSIONS

SUMMARY

Rainfall-runoff data from a network of small urban drainage basins in the Denver Metropolitan Region were assembled. A listing of the 30 watersheds is given in Appendix A. A location map is also given in Appendix A. A total of 37 hydrographs from nine of these urban watersheds were used in this analysis.

Since the rainfall and runoff events are given at five minute intervals (see Chapter 3 for a description of the equipment), it is convenient to derive 5-minute unit hydrographs from the recorded events. Three different methods for deriving the unit hydrograph were used. The unit hydrographs derived by the FINVER method are given in Appendix C.

Measurements have been made of various physical watershed variables. Some of these are combined into parameters. All of the data are stored on magnetic tape for easy retrieval and use. A description of the CSU Flood Data File is given in Appendix B.

A stepwise multiple regression procedure was used to find regression equations between the watershed characteristics and the unit hydrograph parameters. Regression equations were also developed between the storm characteristics and the unit hydrograph parameters. A separate investigation was made to select the most effective definition of the watershed response time.

CONCLUSIONS

It was concluded that the best definition of the watershed response time was the lag time, T_{LC}, the time interval between the centroid of rainfall excess and the centroid of the runoff.

The predictive capabilities of the regression relationships would be improved through acquisition of additional data - particularly data from large floods. Some of the coefficients and exponents will change as more data become available.

The regression equations for the unit hydrograph peak discharge are:

$$\frac{Q_p}{A} = \frac{32.223}{T_{LC}^{0.867} \, V_{RF}^{0.036}} \tag{92}$$

or

$$\frac{Q_p}{A} = \frac{43.939 \, C_Q^{.109} \, F_L^{.578}}{T_{LC}^{0.822} \, E_{RF}^{0.037}} \tag{93}$$

The units and notation are defined in Chapter 4.

The regression equations for the lag time, T_{LC}, are:

$$T_{LC} = 338.361 \, T_{10}^{.048} \, E_{RF}^{.304} \, H_R^{1.154} \, C_Q^{.15} \tag{94}$$

or

$$T_{LC} = 210.233 \, \frac{T_{10}^{.031} \, E_{RF}^{.323} \, H_R^{.80}}{U^{.342}} \tag{95}$$

Technical Appendix A
Denver Metropolitan Urban Network

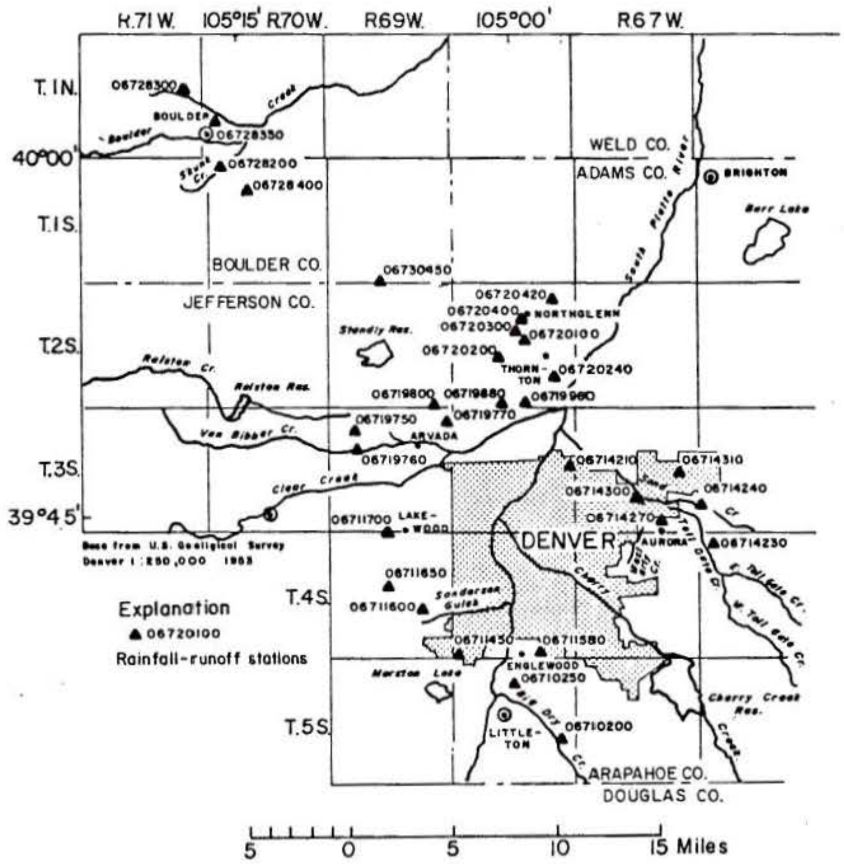

Fig. A-1. Location Map of the Denver Metropolitan USGS Urban Rainfall-Runoff Gaging Stations.

TABLE 1

LIST OF GAGING STATIONS IN THE DENVER METROPOLITAN AREA

USGS NO.	Name and Location	Period of Record	Approximate Drainage Area/sq.mi.
6.7102	Big Dry Creek tributary at Littleton, 39°55'46", 104°56'06"	1969-P	1*
6.7102.5	South Platte River tributary at Englewood, 39°88'08", 104°59'48"	1971-P	1
6.7114.5	Bear Creek tributary at Denver, 39°39'14", 105°02'46"	1971-P	0.2
6.7115.8	Harvard Gulch tributary at Englewood, 39°39'34", 104°58'23"	1971-P	1
6.7116	Sanderson Gulch tributary at Lakewood, 39°41'09", 105°04'54"	1969-P	0.5 *
6.7116.5	Lakewood Gulch tributary at Lakewood, 39°42'17", 105°06'33"	1971-P	0.3
6.7117	Dry Gulch at Lakewood, 39°44'29", 105°06'43"	1971-P	0.2
6.7142.1	South Platte River tributary at Denver, 39°47'18", 104°56'32"	1971-P	0.5
6.7142.3	Toll Gate Creek tributary at Aurora, 39°44'10", 104°48'39"	1970-P	0.3 *
6.7142.4	Sand Creek tributary at Aurora, 39°45'41", 104°49'36"	1971-P	0.3
6.7142.7	Westerly Creek tributary at Aurora, 39°45'13", 104°51'51"	1970-P	0.2 *
6.7143	Concourse D Drain at Stapleton Airport, 39°46'08", 104°53'12"	1970-P	0.11*
6.7143.1	Sand Creek tributary at Denver, 39°47'07", 104°50'31"	1971-P	0.6
6.7197.5	Ralston Creek tributary at Arvada, 39°48'53", 105°08'15"	1970-P	0.6
6.7197.6	Van Bibber Creek at Arvada, 39°37'54", 105°08'15"	1970-P	0.9
6.7197.7	Clear Creek tributary at Arvada, 39°49'20", 105°03'11"	1970-P	0.7
6.7198	Schneider Drain at Arvada, 39°50'12", 105°04'14"	1968-72	0.33*
6.7198.8	Clear Creek tributary No. 1 at Westminster, 39°49'54", 105°00'24"	1971-P	1
6.7199.6	Clear Creek tributary No. 2 at Westminster, 39°49'50", 104°58'59"	1971-P	0.7
6.7201	Tuck Drain at Northglenn 39°52'35", 104°59'16"	1968-P	0.07*
6.7202	South Platte tributary No. 2 Northglenn, 39°51'57", 105°0'27"	1968-P	0.5
6.7202.4	South Platte River tributary at Thornton, 39°51'10", 104°51'18"	1971-72	1.1
6.7203	Hillcrest Drain at Northglenn, 39°52'57", 104°59'36"	1968-P	0.28*
6.7204	Kennedy Drive Drain at Northglenn, 39°53'26", 104°59'14"	1968-72	0.1 *
6.7204.2	South Platte River tributary No. 5 at Northglenn, 39°54'23", 105°57'34"	1971-P	0.5
	Boulder Watersheds		
6.7283	Skunk Creek at Boulder, 39°59'47", 105°15'51"	1970-P	0.8
6.7283	Twomile Canyon at Boulder, 40°02'59", 105°18'16"	1970-P	0.8
6.7283.5	Goose Creek at Boulder, 40°01'35", 105°16'19"	1971-P	0.6
6.7284	Boulder Creek tributary at Boulder, 39°58'48", 105°15'41"	1970-P	0.2
6.7304.5	Rock Creek tributary at Broomfield, 39°54'52", 105°06'51"	1971-P	0.2

*Used in the Analysis herein.

Technical Appendix B
Colorado State University Small Watershed Flood Data File

CSU SMALL WATERSHED DATA FILE

Flood, causal rainfall and physiographic watershed data are systematically assembled for observed floods from small watersheds. The data file is organized so that new watersheds having flood data can be added any time. It is also organized so that additional new hydrographs can be added at any time. The data file was modified so that flood events measured on urban watersheds may be also fully documented and added to the data file. Each urban flood event also has the physical watershed information pertinent to that flood recorded. In the original concept it was assumed that the physiographic features of the watershed were stable and that the watershed was pristine - undisturbed by man. The urban watershed is being altered - the exact antithesis of the pristine watershed. The purpose of developing the urban flood data file is to document and preserve observed flood data for use in future research work on the impact of urbanization on watershed hydrology.

The information in the original flood data was divided into six groups. The urban flood information will add two additional groups of information. The first gives general information about the watershed:

Watershed Information

Set 1. Watershed name, location and identification number.

Set 2. Flood series if available. This provides a frame of reference for the peak discharge for any new flood being considered for inclusion in the data file.

Set 3. Physical watershed characteristics.

Flood Event Information

Set 4. Antecedent Rainfall. Daily rainfall data prior to the storm included in Set 5.

Set 5. Mass curve of rainfall of the storm causing flood event in Set 6.

Set 6. Discharge hydrograph.

Urbanization Information (New addition)

Set 7. Physical urbanization characteristics which could be obtained from topographic maps, aerial photography or aerial observation. These include:

1. Percent of impervious area,
2. Length of paved streets and roads,
3. Length of curbed and guttered streets and roads,
4. Length of unpaved surface drainage channels.

Set 8. Physical urbanization characteristics which cannot be obtained from aerial observation. These include:

1. Length of underground storm sewers.
2. Average capacity of underground storm sewers.
3. Average C & G street gradients.
4. Roughness of surface drainage channels.
5. Population density.

For a given watershed, there can be only one set 1, set 2, and set 3 data. There may be any number of rainfall events, each represented by some combination of set 4, set 5, and set 6 data. If it is an urban watershed and has set 4, set 5, and set 6 data, there will also be set 7 and set 8 data. In principle it is assumed that new set 7 and set 8 data will be obtained for each new flood event; although if the development has not changed these data may be transferred from the previous event. The logic of the data file is shown schematically in the next diagram. The set 7 and set 8 data will simply be added after the hydrograph.

The following generalization may be made about the data:

If a watershed is represented, at least 1 will be present.

If sets 2 and/or 3 are present, they will follow set 1 in numeric order.

Sets pertaining to rainfall events will always follow whatever of sets 1, 2, and 3 are present.

A set 5 will always be followed by a set 6.

If any of sets 4 and 5, 6 are present for a given event they will be in numeric order.

If it is an urban watershed, sets 7 and 8 will follow each set 6 data.

PROGRAMMING INFORMATION

Watersheds 1-600 on Tape A165, 201-1289 on A512.

Tape format: BCD, 80 characters/record, 556 BPI
Both tapes close with an end of file.

On the BCD tape, the information of each set is preceded by a record identifying the kind of set to follow. This identification record contains the set number (1 to 6), the 10 digital serial number (itself a concatenation of 5 codes), and the name of the watershed. In the identification records for sets 1, on the BCD tape, a sequence number of the watershed on this tape (1-1289) has been placed in the last 4 columns. On the basis of the set number one may branch to the appropriate read statements for the record, or records, to follow. This branching is provided for in our program by several brief subroutines, which in turn call the appropriate entry points in the six subroutines for the various sets:

Subroutine TAPERED	Provides for reading a binary tape, except for the identification record, which normally will be read in the main program.

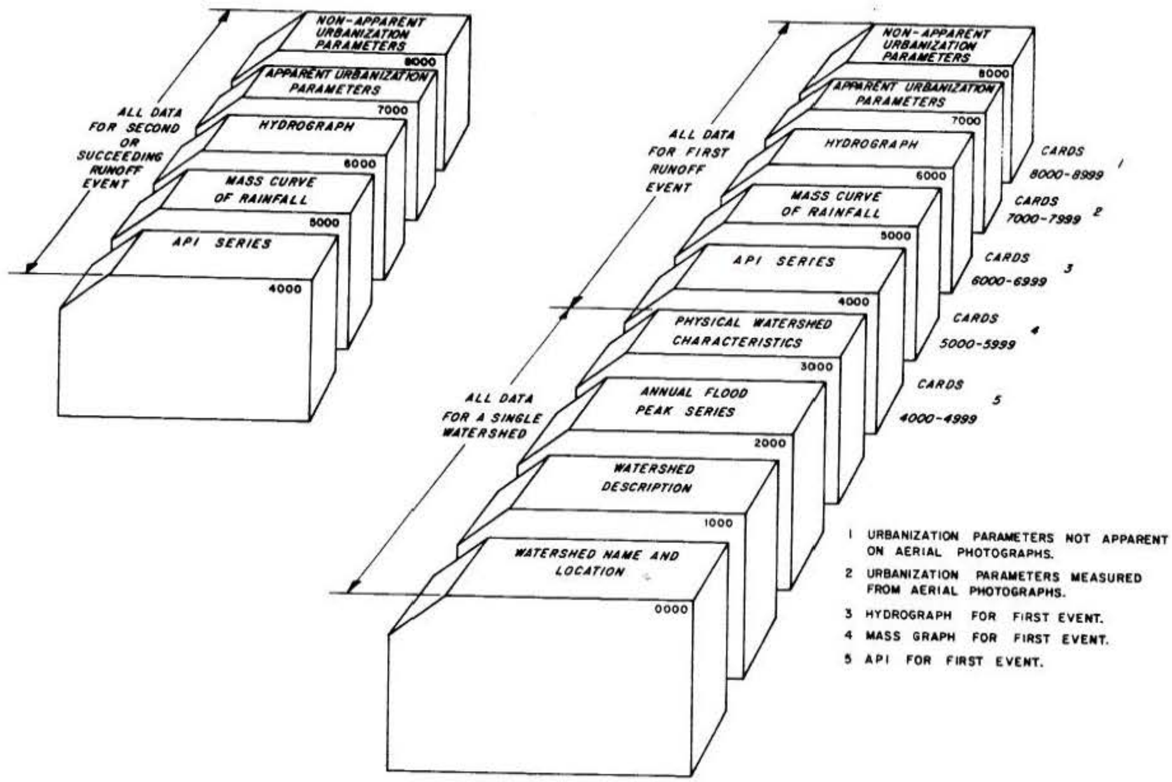

Fig. B-1. Schematic Representation of Data Arrangement for a Single Watershed with Two Runoff Events

Subroutine TPWRT	Provides for writing a binary tape, including the information record for each set.
Subroutine TPLIST	Outputs the data in full print lines in an easily readable listing with headings.
Subroutine PU80	Outputs the data without headings in an 80 column format.
Subroutine RD80	Provides for reading the 80 column format, except for the identification record, which will normally be read in the main program.
Subroutine TCØMP	Calls for computation of certain output data from the minimal set of input items. (Since the results of the computations are now on the BCD tape, you will not be repeating these operations on this data).

Each subroutine for a given set, then contains entry points corresponding to these functions.

The CSU master binary tape is organized in similar fashion, with an identification record preceding each record of set information. The sequence number of each watershed (i.e., 1-1288) is not on the binary tape, but may be supplied by counting each set 1 as it is read, or written.

The KOMENT deck, found with the program deck, is not on the data tape.

In many cases the event codes for a given watershed (last two digits in the 10-digit serial number, or JSER (5), are not consecutive, although they are in ascending order. They should probably be renumbered in consecutive series within each watershed.

The accompanying programs were written at Colorado State University in a version of Fortran IV for the CDC 6400 computer, a machine with a core memory of 65K 60-bit words. The alphameric fields in the format statements were written with this equipment in mind, but may, of course, be segmented in any way to be compatible with another word size.

If there is no provision in your Fortran for multiple entry points into a subroutine, appropriate branching may be easily achieved by adding a variable to the COMMON list, and using it in a multiple branch GO TO in the various set subroutines.

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KEY WORDS: Unit hydrographs, Urban watersheds, Response time, Lag time, Parameters of urbanization.

ABSTRACT: A brief review of previous work is presented. Different methods for quantifying urbanization are discussed. A step wise multiple regression technique was used to select the best parameter of urbanization. The rainfall and flood events from nine urban watersheds in the Denver Metropolitan region were analyzed. Unit hydrographs were derived from the measured floods on these watersheds. The unit hydrograph parameters were correlated with storm and physical watershed parameters. It was found that the changes in the unit hydrograph in the urban region were related to the decrease in the watershed response time. The best way of defining the response time was the lag time. The lag time was found to be sensitive to the increase in the hydraulic capacity to the decrease in the ratio of pervious watershed and the shape of the watershed.

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