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SOME CONSEQUENCES OF AREA WIDE RUNOFF CONTROL STRATEGIES IN URBAN WATERSHEDS

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Water Resources Series
Technical Report No. 48
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SOME CONSEQUENCES OF AREA WIDE RUNOFF CONTROL STRATEGIES IN URBAN WATERSHEDS

by

Richard A. Hardt and Stephen J. Burges

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Project Completion Report: Analysis of Runoff Detention in Urban and Suburban Watersheds

Principal Investigator: Stephen J. Burges, Associate Professor of Civil Engineering, University of Washington

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ABSTRACT

Consequences of "blanket" runoff control management strategies that require all future developments in urban and suburban areas to maintain peak flow rates from their sites at the level that existed prior to urbanization were examined. Hypothetical watersheds were studied to determine some bounds to the overall runoff management issue. Principal findings indicated that:

1. Runoff control strategies must be evaluated at the entire watershed level, not on local jurisdictional boundaries.

2. Without detailed sub-basin discharge hydrographs and main channel characteristics, the optimum drainage management scheme cannot be established.

3. Runoff volume reduction appears to be one means of achieving more effective runoff control but would need to be a significant portion of the incremental volume increase resulting from urbanization.

4. Restricting the outflow from a retention facility to a level less than the undeveloped rate could achieve a composite peak flow rate that would equal the pre-urbanization flow but would run for a much greater duration at that rate. The increased flow duration would have potentially undesirable effects on the channel system.

5. Longer duration storms falling on the sub-basins under consideration produced higher peaked watershed outflow hydrographs than did shorter, more intense rainfalls. This emphasizes the need to know the actual shape of the hydrograph. Runoff volume, as well as peak flow rate, is crucial to determining the most effective storm water management scheme.

6. For storm magnitudes greater than the control system design storm two possibilities exist: a higher peak rate or an equal peak rate to that which would have occurred without control. The shape of the sub-watershed input hydrographs into the control facilities and the sequential linking together of the sub-basins are both critical factors. No general conclusion was evident from this research.

Mixed control strategies were shown to be useful in some circumstances. Generally, each drainage basin must be examined to determine what management options will work. Blanket policies were shown to be unwise in many situations; administrative convenience is not a panacea for managing urban runoff.

Keywords: Urban Runoff Control; Rainfall-Runoff; Flood Mitigation; Hydrologic Design; Streamflow Routing; Precipitation Representation.

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1. INTRODUCTION

1.1 Background

Throughout the United States traditional local drainage requirements and management techniques have been criticized in recent years by an environmentally aware public. The primary concern has been over the traditional approach of installing large storm sewers that dispose of the storm water into the nearest natural drainage network. This method, while effective in removing the storm water from a specific site, results in downstream flows that may cause considerable damage and cost to both private parties and public agencies. As the number of lawsuits for damages resulting form storm water increased, citizens and professionals started to develop alternatives to the customary policy of urban runoff management.

Generally such alternative drainage control strategies (Everhart, 1973) can be articulated as four basic objectives: 1) Existing natural drainage systems will be used as much as possible and left in a natural state, 2) Drainage channels will be grassland swales as opposed to narrow, deep ditches, 3) Flow retarding devices such as retention ponds and recharge berms are used where practical to minimize increases in runoff volume and peak flow rate due to urbanization, and 4) Drainage pipes and other flood control structures are used only where the natural system is inadequate. This approach can be very effective in preserving the amenities of a stream while creating multipurpose reservoirs; the costs for on-site retention have been found to be less than the costs for the traditional piped system (Poertner, 1973).

With these four basic objectives serving as guidelines, states, counties, and cities began revising their drainage policies. Several local jurisdictions have adopted "blanket" regulations (Wiswall & Robbins, 1975; McCuen, 1974) that require all future developments to maintain peak flow rates from their sites at the level that existed prior to urbanization. In most instances, this requirement would dictate on-site retention. They felt this action would decrease the impact of the development but questions are beginning to be raised concerning the effects of such requirements on the downstream composite hydrograph (McCuen, 1974).

Another alternative policy for urban runoff control is to employ a regional (sub-basin) retention pond concept in lieu of individual on-site facilities. This approach would insure that the maximum discharge from each sub-basin does not exceed the pre-urbanization peak flow rate; the composite downstream hydrograph may, however, be substantially changed from the pre-urbanization condition.

The problem with both of these drainage control options is the fact that only the peak flow has been considered and not the total hydrograph. Restricting the peak flow to a pre-developed rate will substantially alter the shape of site or sub-basin hydrographs. The sharp peaked urbanized hydrograph will be converted to a broad based hydrograph having peak flow equal to the natural peak flow condition but with the same runoff volume as the urbanized hydrograph (Figure 1.1). In achieving peak flow reduction there is a significant lengthening of the time base. This extended time base may have a drastic impact on the extended downstream composite hydrograph if the hydrographs from several contributing drainage areas now overlap.

One approach that may alleviate this problem is through implementation of a regional drainage management scheme that would allow varying degrees of control in each of the sub-basins (Wiswall & Robbins, 1975). For example, the middle and upper portions of the watershed might be required to hold peak flow to the pre-developed level. In contrast, the lower section of the basin might have no control. This method would allow the discharge from the downstream sub-basin to peak and recede (under favorable conditions) before the hydrograph resulting from upstream areas arrives.

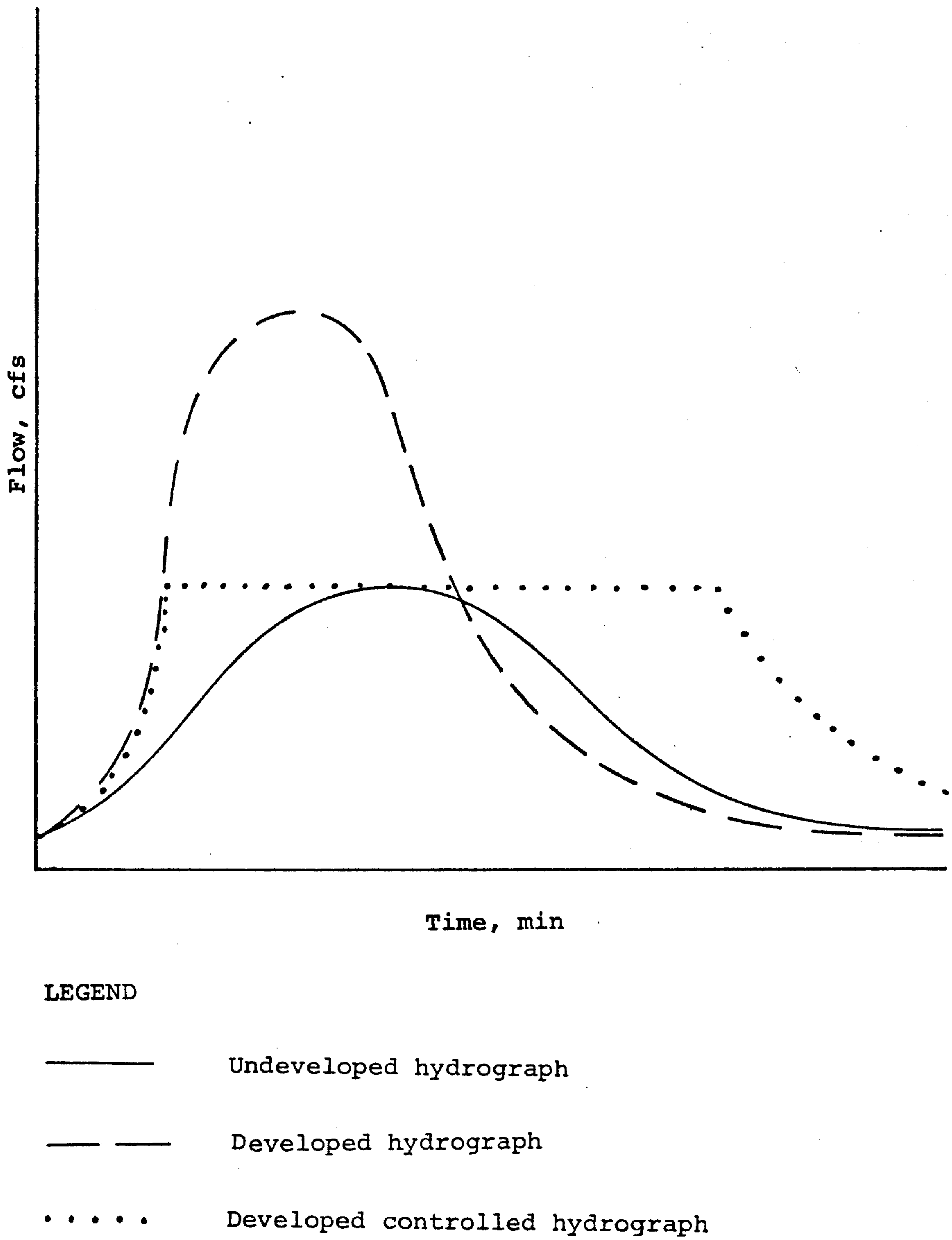


Figure 1.1 Surface Runoff Hydrographs

A regional mixed control approach is examined in this investigation. It allows the greatest variation in control strategies while simplifying the sub-basin hydraulics.

1.2 Research Objectives

The primary objective of this investigation was to analyze the effectiveness of blanket control and other drainage control options for small urban watersheds. The runoff management options were based on an overall control strategy of sub-basin retention instead of individual on-site facilities. This is not an overly restrictive limitation because in limiting cases sub-basins become individual sites. The conclusions developed are for all practical purposes not influenced by the choice of sub-basin over in-situ storage. Had storage been considered for individual sites more complicated hydraulic modeling would have been needed to route the controlled flows from the individual sites to the sub-basin outlet. This routed outflow would then become the input to the routing model (which connected flows from sub-basins) which was used. Where runoff controls were applied they were assumed to be uniformly applied to an entire sub-basin. For a given region mixed levels of control for different sub-basins were examined.

A secondary objective was to examine the importance of runoff volume reduction for accomplishing desired runoff control. This research is an extension of the principal objective with the same management options but with runoff volume reduction.

The effectiveness of the various control schemes for either objective is evaluated by comparison with natural watershed condition peak flows. In addition, the peak flow rates from the different options are also compared with one another. Finally, conclusions for employing mixed runoff control strategies are discussed.

1.3 Research Approach

The research effort could have taken any of several directions. When comparing the effects of combining hydrographs from different sub-basin geometries on composite flow there are many combinations that may result. Does one consider continuously simulated land surface runoff contributions to streamflow to find the integrated effects of spatial and temporal precipitation interacting with watersheds having varying degrees of development when examining control options? Would it be instructive to use "design storms" with linear system hydrograph approaches (see Eagleson, 1970) to examine possible hydrograph combinations? The authors had anticipated using the linear systems approach because of its potential for examining general situations. When actual numerical experiments were planned it was decided to use an unsteady kinematic routing scheme to run the sub-basin hydrographs through the connecting channels. Further details are given in Chapter 3.

An important issue to be faced early in the research effort was: how would the sub-basin hydrographs be generated? A corollary is: how accurately do these hydrographs need to be? The objectives involved finding generalized macro effects of different control strategies so only approximate sub-basin hydrographs were required. Details of how these were obtained are given in Chapter 3. Of considerable interest was the issue of which sub-basins in a watershed had most impact on the watersheds outflow hydrograph when they were subject to peak flow control. A major effort was therefore directed toward examining sub-basin size as a function of spatial distribution and runoff control strategy upon the total outflow hydrograph. Hypothetical watersheds were examined. Three sub-watersheds totaling 2000 acres constituted the hypothetical watersheds. Precipitation patterns representative of the Spokane, Washington area were considered. Other precipitation patterns could be readily examined; the intent of this work was, however, to show representative consequences of runoff control strategies.

Issues of how to effect runoff control in the several sub-basins are not specifically addressed. One possible approach, use of porous pavements throughout sub-basins, is currently being examined at the University of Washington to see what overall impacts it would have on watershed outlet hydrographs. The results of the porous pavement investigation will be reported in the technical literature at a later date.

2. RELATED WORK

2.1 Introduction

The information in this chapter is divided into three sections. The first is a general overview of the impacts of urbanization. The second compiles the state-of-the-art in surface runoff control and references recent research concerning the effectiveness of different control schemes. The final section covers the equations and some solution techniques that are presently being used for flood wave routing in open channels.

2.2 Hydrologic Effects of Urbanization

As man developes a drainage basin striking changes in the hydrologic cycle occur once vegetated slopes and meadows are transformed by roof tops, streets, and compaction into impervious surfaces. Savini and Kammerer (1961) illustrated the hydrologic system of typical large tracts of undeveloped and developed land by Figures 2.1 and 2.2 respectively. Infiltration and evapotranspiration are important variables influencing the flow rate and volume of runoff from a pre-urban hydrologic system.

In the urbanized case (Figure 2.2) direct runoff has become a major flow path while infiltration has been reduced to a minor one. Urban land surfaces, retention ponds, recharge basins, and stormwater collector systems are all major factors which account for the changes shown in Figure 2.2.

Since the effects of urbanization are complex, generalizations are difficult to make and are sometimes misleading. However, the ASCE Task Force on Effect of Urban Development on Flood Discharges (ASCE, 1969) concurred that "the acceleration and concentration of flood waters by runoff from impervious areas, and by the construction of storm sewers, gutters, catch basins, and channel improvements, gave rise to the major contribution of the increased

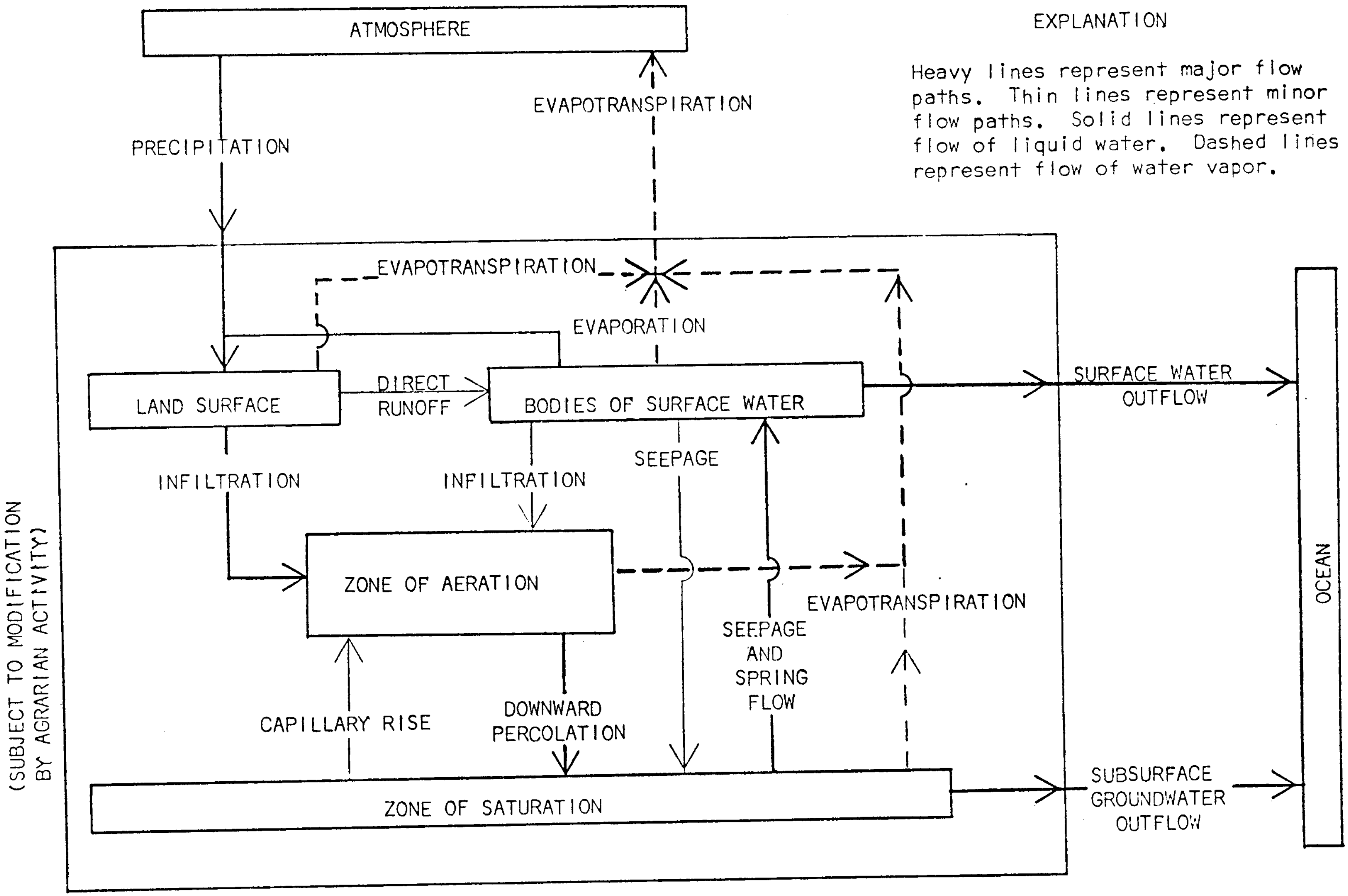
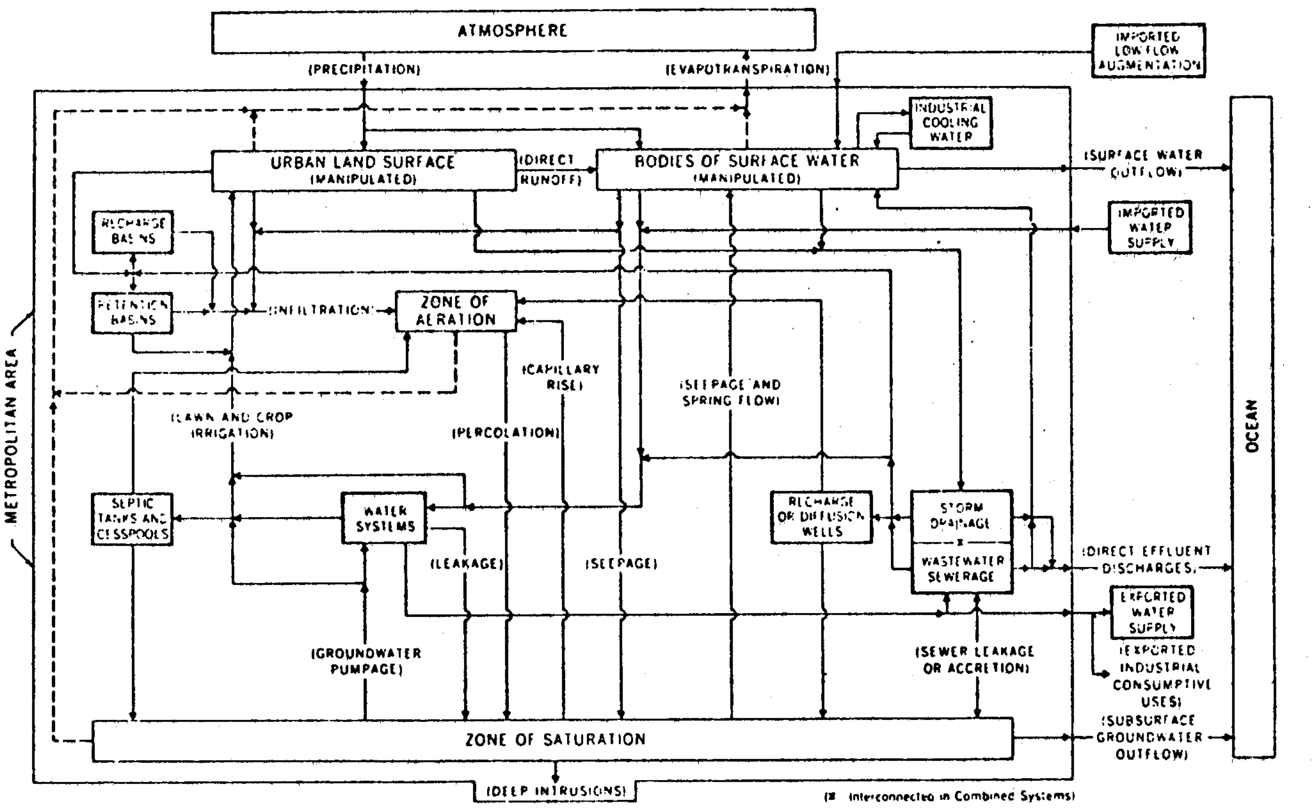


Figure 2.1 Preurban Hydrologic System



(Adapted from "Summary of the Hydrological Situation in Long Island, N.Y. as a Guide to Water Management Alternatives," by O.L. Franke and N.E. McClymonds, U.S. Geological Survey Professional Paper 627-F, 1972)

Figure 2.2 Urban Hydrologic System

flows experienced in urban areas." Waananen (1969) discussed the changes that normally occur in the time of concentration and in peak discharge as a drainage basin becomes more developed. Figure 1.1 schematically shows how the developed watershed peak flow is much larger and how the lag time is shortened for identical precipitation input.

The ASCE Task Committee on the Effects of Urbanization on Low Flow, Total Runoff, Infiltration, and Groundwater Recharge (ASCE, 1975) has compiled a current bibliography and a state-of-the-art report on the effects of urbanization on low flow, total runoff, infiltration, and groundwater recharge. This report concludes that generally the following occur as a result of urbanization: 1) the base flow rate in streams decrease, 2) total water yield increases, and 3) infiltration and groundwater recharge both decrease.

2.3 Runoff Control Strategies in Urban Watersheds

The effects of urbanization on stream regimes are becoming less tolerable to members of the public as, one by one, natural creeks are reduced to storm drainage ditches. In an effort to mitigate these undesirable changes retention ponds or other runoff control devices combined with natural and modified channels have become more common in some recent designs. Poertner (1974) compiled a report that illustrates the various detention methods presently being used.

Crawford (1973), Leclerc and Schaake (1973), and Lumb, et al. (1974) all review and assess the efficiency of runoff control alternatives. Lumb, et al. (1974) made the following observations at the conclusion of their research:

> 1) small detention storage basins are effective in holding runoff from newly developed areas to their former peaks under natural conditions but become progressively less effective when dealing with larger and larger watersheds, 2) using natural instead of paved collector channels has a similar pattern of substantially reducing headwater hydrographs but diminishing effectiveness as one goes downstream, 3) draining roofs and driveways onto dense turf is effective immediately downstream and, because of volume reductions, may cumulatively have a greater effect in larger watersheds, 4) terracing is effective if dense turf is maintained but bare dirt terraces

probably do not justify their costs, and 5) commercial and industrial areas accentuate flood hydrographs most when located near the middle of a watershed.

McCuen (1974) and Robbins and Wiswall (1975) both raised the question of the effectiveness of blanket local ordinances that require individual on-site retention measures. They both illustrate cases showing where the individual site approach (i.e. holding peak flow to pre-urbanization levels) can indeed increase the peak flow discharging from a drainage basin. McCuen (1974) suggested that a regional scheme that permits different control criteria in various sections of the basin may be the most effective way for mitigating the effects of urban runoff.

To evaluate the various control options resultant sub-basin hydrographs must be routed down a channel network. The following section reviews relevant routing techniques.

2.4 Flow Equations and Routing Models

Much has been written about the use of the one dimensional Saint-Venant unsteady flow equations for describing the movement of flood waves through open channel networks (see for example Chow, 1959; Henderson, 1966; Sevuk and Yen, 1973; Yevjevich and Mahmood, 1975). The equations for gradually varied unsteady flow in prismatic channels may be written as:

$$D\frac{\partial v}{\partial x} + v\,\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \qquad \text{(Continuity)} \qquad (2.1)$$

$$\frac{\partial y}{\partial x} + \frac{v}{g}\frac{\partial v}{\partial x} + \frac{1}{g}\frac{\partial v}{\partial t} = S_o - S_f \qquad \text{(Momentum)} \qquad (2.2)$$

in which

D = hydraulic depth (flow area/flow surface width), ft

g = gravitational acceleration, ft/sec^2

S_f = friction slope (head loss/unit length), ft/ft

S_o = slope of channel bottom, ft/ft

t = time, sec

v = average flow velocity, ft/sec

x = length along channel, ft

y = flow depth, ft

The above equations apply to cases without lateral inflow and assume the coefficients of velocity and momentum distribution are unity.

The use of both equations is a one dimensional approximation of a dynamic flood wave. A dynamic wave can propogate both upstream and downstream while its shape is attenuating; the energy grade line does not parallel the bottom slope. The dynamic situation generally exists in channels with bottom slopes of less than 10 feet per mile (Linsley, et al., 1975).

For channels with slopes greater than about 10 feet per mile and for typically encountered hydrographs the energy grade line begins to parallel the channel bottom, i.e., $S_o = S_f$, and the inertial terms are usually negligible. Flood waves satisfying these conditions were called kinematic waves by Lighthill and Whitham (1955). For kinematic wave routing the following unsteady flow equations are used:

$$D \frac{\partial v}{\partial x} + v \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \qquad \text{(Continuity)} \qquad (2.3)$$

$$Q - \frac{1.49}{N} S_o^{0.5} * R^{0.6667} * A = 0 \qquad \text{(Momentum)} \qquad (2.4)$$

in which

A = flow area, ft^2

N = Manning's roughness coefficient

Q = discharge, cfs

R = hydraulic radius, ft

Equation 2.4 is a "stage-discharge" relationship. General analytical solutions to the partial differential equations 2.1 and 2.2 do not exist, but they can be conveniently solved by numerical techniques with the aid of a digital computer. Sevuk and Yen (1973) (among others) categorized the computational techniques used to obtain numerical solutions to the St. Venant equations. The three major schemes categorized were: explicit finite difference, implicit finite difference, and the method of characteristics. In addition to Sevuk and Yen's work, Yevjevich and Mahmood (1975), Miller (1971) and others have evaluated each of these techniques. It is generally accepted that the implicit finite difference formulation (Amein, 1968) is superior to the other schemes with respect to convergence and computational speed (Price, 1975). Because of its many advantages, and particularly for its computational speed, an implicit routing scheme was used herein.

Several different implicit finite difference schemes have been used in flood routing applications (Perkins and Gunaratnam, 1970; Brakensiek, 1967). The major difference between these schemes is the finite difference approximation utilized. A non-central four-point finite difference approximation, which has been commonly used because it does not exhibit stability and convergency problems (Fread, 1974), was adopted.

The implicit finite difference approximation of the St. Venant equations generates a system of nonlinear algebraic equations. These equations have two independent variables--time and distance--and two dependent variables--depth and velocity--at each point in a space time node network. The equations may be conveniently solved simultaneously by a functional iterative technique known as the Newton-Raphson Iteration Scheme (Fread, 1973; Froise, 1975).

3. ANALYSIS AND SYSTEM CONFIGURATION DETAILS

3.1 Introduction

To investigate sub-basin-channel-hydrograph interactions, sub-basin hydrographs were routed through a general channel network. A number of different hydrographs were generated from the hypothetical individual sub-basins for various watershed geometries, rainfall intensities and control strategies. To simplify possible watershed configurations while still maintaining sufficient detail to identify important relationships between parameters and variables several assumptions were made.

The assumptions made regarding the channel network, sub-basin hydrographs, and the channel flow routing scheme are given in this chapter. Details of the actual channel network, cross-sectional geometry, sub-basin hydrographs, and routing scheme used are given below.

3.2 Channel Network and Cross-Sectional Geometry

The effectiveness of various runoff control options can be evaluated by studying the hydrologic interactions of several sub-basins. For this study of small urban drainage basins, a 3 sub-basin network was assumed (Fig. 3.1). Although the channel flow routing program used can accommodate a larger number of input hydrographs, the 3 sub-basin scheme is sufficiently detailed to reveal significant impacts of subwatershed runoff management practices. Each sub-basin hydrograph is assumed to discharge to the stream channel at one point (rather than as lateral inflow) as shown in Figure 3.1.

The general sub-basin configuration of Figure 3.1 requires 2 channel reaches to link the sub-basins. The four major channel parameters examined for reaches I and II (Fig. 3.1) are length, slope, hydraulic roughness, and cross-sectional geometry. Channel lengths of 5,000 and 10,000 feet were used to simulate different watershed shapes. A broad drainage basin may have

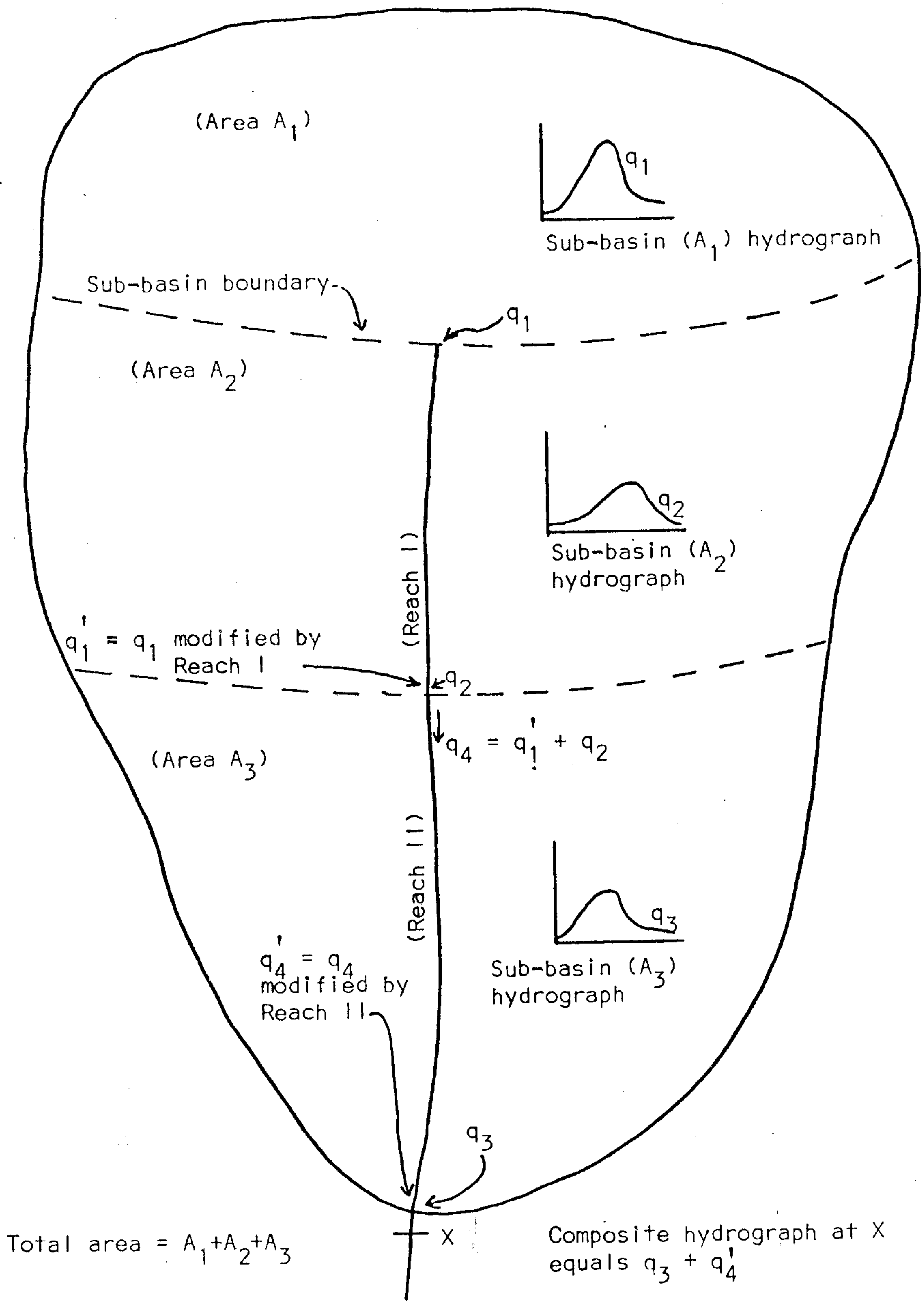


Figure 3.1 Schematic Representation of Watershed Plan Geometry

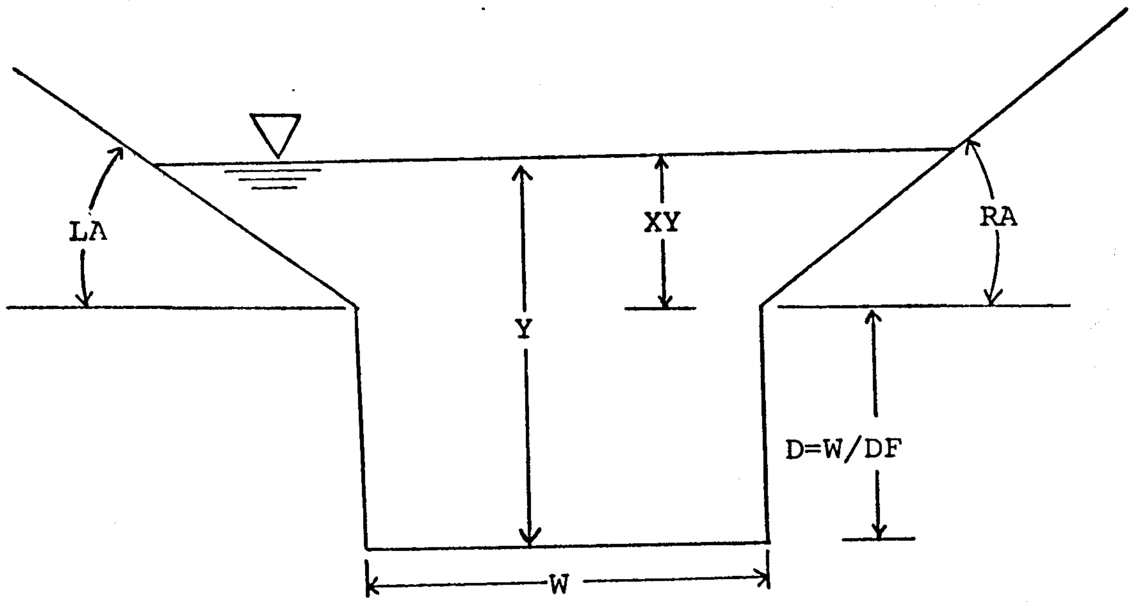
a short channel reach between sub-basins while a narrow, elongated basin could have much greater distances between tributaries. The lengths of reaches I and II directly influence the combined hydrograph at the basin outflow "X" (Fig. 3.1).

The channel slope and roughness were varied to see how they impact the combined watershed hydrograph. A steep channel would have a smaller travel time between sub-basins and a lesser reduction in peak flows than a channel having a flatter gradient. Channel gradients of 10 and 40 feet per mile were investigated. Manning's "n" values of .025 and .045 were examined.

Cross-sectional geometry was the last channel characteristic varied. The majority of the cases analyzed were made with LA = RA = .245 radians (i.e. side slope 1:4), width = W = 5 ft, and DF = 5 (Fig. 3.2). For comparison, and to reflect the impact of channel modifications, LA and RA could be set equal to 1.57 radians, i.e., a rectangular channel. The section shape was described by a number of parameters in the computer program (Appendix A) used for analyzing alternative configurations so that channel geometry could be conveniently altered.

3.3 Sub-Basin Hydrographs

The sub-basin hydrographs of Figure 3.1 were not generated by a sophisticated surface runoff model. Rather, synthetic sinusoidal shaped hydrographs based on sub-basin characteristics were developed. The sub-basin hydrographs are assumed to enter the main channel at the downstream boundary of each sub-basin, instead of uniform lateral inflow along the channel length. The pertinent factors for deriving the representative hydrographs used are soil type, ground cover, basin slope, basin acreage, channel length, amount of impervious area, and rainfall characteristics.



LEGEND

A   = flow area = W*Y+XY* XY*Theta

B   = surface width = W+2*XY*Theta

Beta   = 1./Sin(RA)+1./Sin(LA)

D   = depth of the rectangular portion of the channel, ft

DF   = a constant that relates D to W

LA   = left angle, radians

RA   = right angle, radians

Theta = .5*(1./TAN(RA)+1./TAN(LA))

W   = channel width, ft

WP   = wetted perimeter = W+2*Y-2*XY+XY*Beta

XY   = water depth above rectangular channel section, ft

Y   = water depth above channel bottom, ft

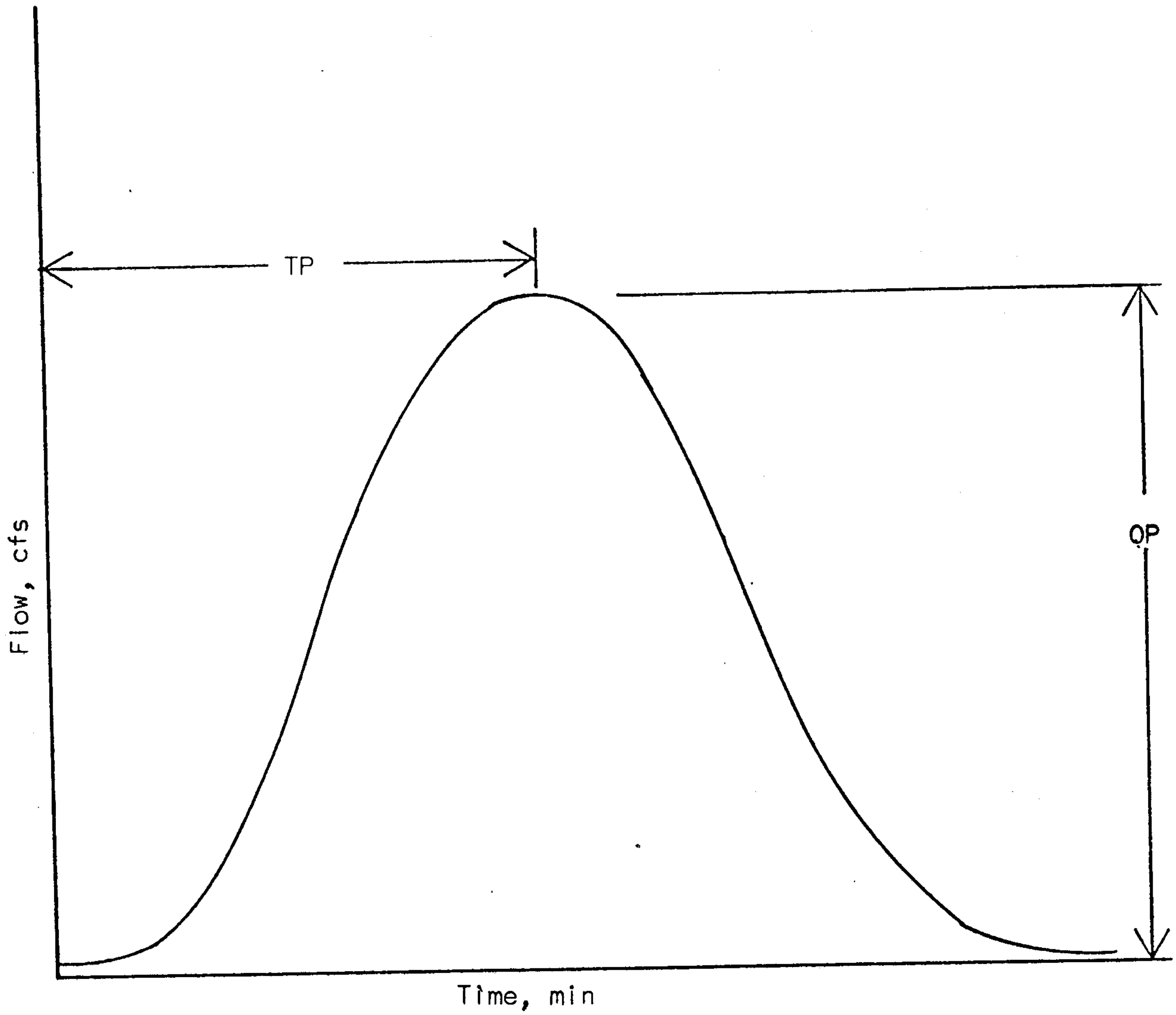
Figure 3.2 Downstream View of Channel Section

Hydrograph Generation - The Soil Conservation Service (SCS) approach was used for convenience to obtain representative approximate land surface runoff hydrographs. They should be clearly recognized as only approximations to actual runoff hydrographs. While the methods employed to generate sub-basin hydrographs were crude they sufficed for the overall purpose of this investigation. Sinusoidal sub-basin hydrographs exhibiting the broad features of the SCS generated hydrographs were used in all channel routing studies reported herein.

The sinusoidal sub-basin hydrograph (Fig. 3.3) can be defined by two parameters: 1) time to peak and 2) peak flow rate in cubic feet per second. (A base flow of 2 cfs was taken for all sub-basins to avoid difficulties created by dry channel initial conditions in the flow routing algorithm.) The time to peak and peak flow rate were calculated (Appendix B) by using the appropriate charts and procedures given by the SCS (Soil Conservation Service, 1975).

It is clear that numerous combinations of hydrographs, channel geometries, etc. are possible so a limited number of combinations, reflecting the bounds of the problem, were tested. To achieve this set the following criteria were used:

1) Total watershed are (A_1+A_2+A_3) is 2,000 acres,
2) The natural land use is grassland with Hydrologic Soil Group B, Curve Number 75 (Soil Conservation Service, 1975),
3) Average land slope is 4%,
4) Developed land is 30% impervious area with Curve Number 80 (Soil Conservation Service, 1975),
5) The travel distance to the outlet of a sub-basin is related to basin area by the equation (length = 209 x area$^{0.6}$) (Soil Conservation Service, 1975),
6) The rainfall (1.8 inches) was a typical 5 year 24 hour storm for the region around Spokane, Washington (U.S. Weather Bureau, 1970),
7) The total watershed is urbanized for the "developed watershed" cases.



LEGEND

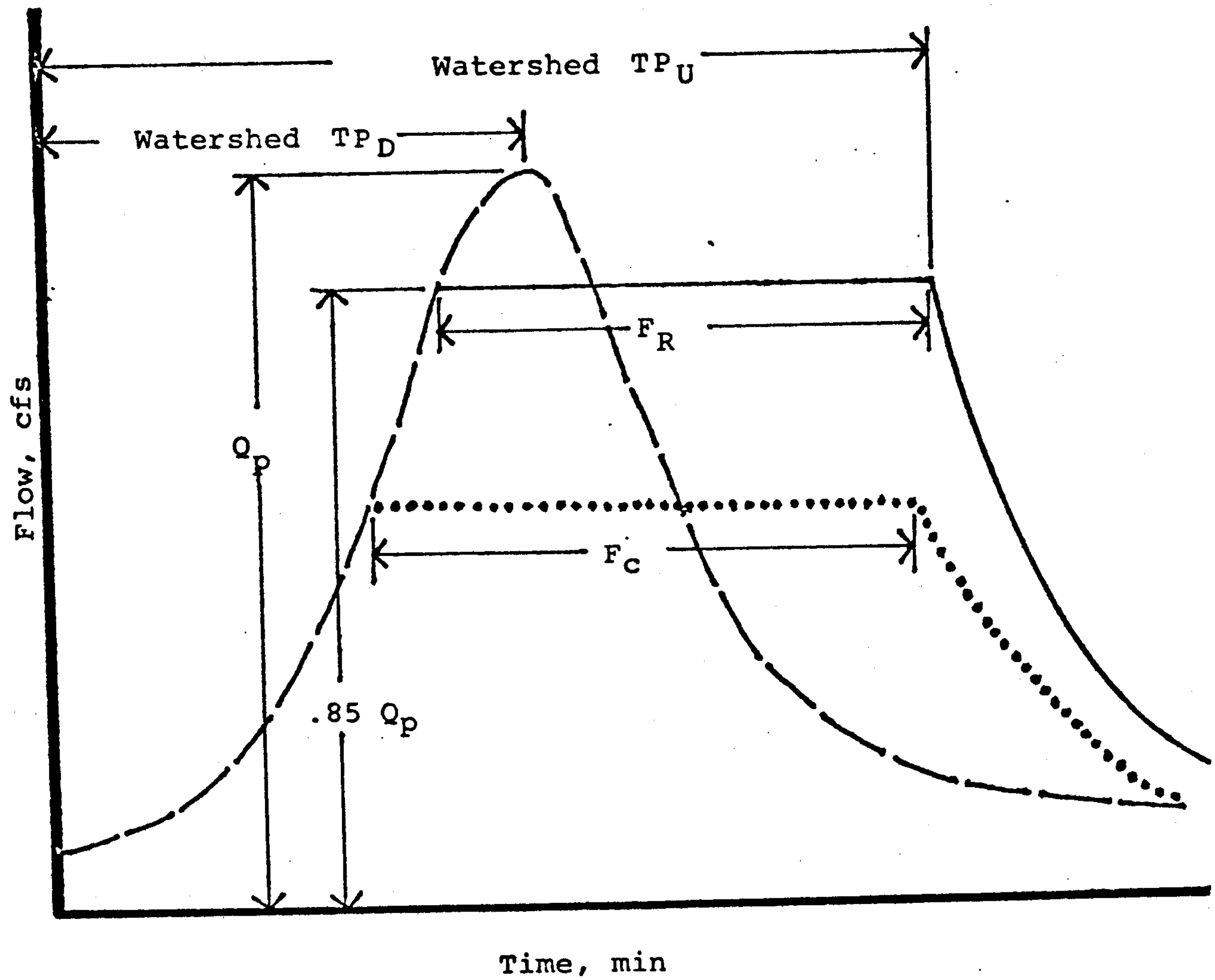
TP = Time to peak, min

QP = Peak runoff rate, cfs

Figure 3.3 Sinusoidal Sub-basin Hydrograph

The acreages of the sub-basins were adjusted to reflect different drainage basin geometries. Table 3.1 summarizes the input hydrograph characteristics used in this investigation.

Flow Retention Simulation - Retention facility behavior was simulated as illustrated in Figures 1.1 and 3.4. An hypothetical retention pond was assumed to have sufficient capacity to control the "developed" hydrograph (i.e. the hydrograph resulting from the urbanized situation) resulting from the five year recurrence interval storm under consideration. It should be noted that for the same initial watershed moisture for the undeveloped state, the developed watershed will come to flow equilibrium, for a given rainfall excess rate more rapidly than will the undeveloped watershed. This poses problems when attempting to use a "design storm" of given recurrence interval for studying pre- and post-development because different storm durations are required to generate corresponding peak flow rates. The peak flow rate for the developed case results from a shorter duration, more intense storm than that causing the peak for the natural state. Here two developed storm conditions are considered. One condition (resulting from the shorter duration, higher intensity storm) gives rise to the dotted controlled outflow hydrograph in Figure 3.4 where the controlled peak corresponds to the peak discharge of the longer duration storm falling on the natural area. This controlled release remains at the controlled peak for a time equal to F_c (Fig. 3.4). The effect of the longer duration storm falling on the developed watershed is approximated by the solid hydrograph in Figure 3.4. The relationship is not strictly correct but approximates watershed behavior sufficiently well for the purpose of this work. Spatial variation in precipitation was not examined nor was storm direction. These variables can be quite significant in particular situations and should be investigated in future.



LEGEND

— — Developed hydrograph

••••••• Developed controlled hydrograph

——— Modified hydrograph to reflect a longer rainfall duration

F_c = Constant flow section of controlled hydrograph, min

F_R = Constant flow section of modified developed hydrograph, min

Q_p = Peak developed flow rate, cfs

TP_D = Time to peak for developed watershed for steady rainfall having m-year recur. interval

TP_U = Time to peak for undeveloped watershed for steady rainfall having m-year recur. interval

Figure 3.4 Schematic Representation of Developed Area Controlled Runoff Hydrographs for Natural and Developed Condition Design Storms

Table 3.1 Sub-Basin Hydrograph Characteristics

Sub-basin Area (Acres)	Sub-basin Travel Distance[c] (Feet)	Curve #75[a]		Curve #80[b]	
		TP (Min)	Peak Flow Q_p (cfs)	TP (Min)	Peak Flow Q_p (cfs)
500	8,700	105	49	66	103
667	10,350	120	61	72	140
750	11,100	126	65	81	150
1000	13,200	150	75	93	184

Notes: $A_1 + A_2 + A_3 = 2000$ Acres

Sub-basin hydrograph $Q_{(t)} = Q_b + 0.5\, Q_p\, (1 - \cos(\pi t/TP))$

$0 \leq t \leq 2TP$

t = time in minutes

Q_b = base flow, cfs (taken as 2 cfs here)

Q_p = peak flow rate above base flow, cfs

$Q_{(t)}$ = sub-basin hydrograph ordinate at time t, cfs

TP = time to reach peak flow, minutes

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[a]Undeveloped watershed; [b]Urbanized watershed (30% impervious area);
[c]Distance = 209 Area$^{0.6}$ (Soil Conservation Service, 1975)

Runoff Volume Reduction - In an attempt to model the effects of runoff volume reduction on the entire watershed outlet hydrograph, the time base of the constant flow portion (F_c in Figure 3.4) of the controlled sub-basin hydrograph was reduced by 25%. This corresponds to a reduction in the incremental volume of runoff due to urbanization of 30 - 50%. Such runoff volume reductions could be achieved by employing infiltration areas and other land treatment measures.

Once the sub-basin hydrographs and channel network were determined, a computer program was needed to route the sub-basin flood waves down the channels. The following section explains the routing techniques used.

3.4 Streamflow Routing Scheme

Section 3.4 is divided into 3 sub-sections. The first, which deals with the equations used to route the hydrographs down the channel reaches, includes a brief description of how the system of equations was derived and of the solution technique utilized. A more detailed discussion of the routing scheme is given by Froise (1975). The second sub-section covers channel junctions while the last deals with numerical verification of the programmed model.

One Dimensional Unsteady Open Channel Flow Equations

Equations 2.1 and 2.2 describe the dynamic movement of a flood wave in an open channel. Both equations must be used for flood wave routing in flat reaches that have a gradient of less than approximately 10 feet per mile (Linsley, et al, 1975). This investigation involves small watersheds with main channel slopes ranging from 10 to 40 feet per mile, therefore, the detailed dynamics of equation 2.2 are not warranted. For this investigation the kinematic wave equations 2.3 and 2.4 were used.

Numerical solutions of the kinematic equation set are obtained by an implicit finite difference formulation (Amein, 1968). A non-central four point finite difference scheme (Fread, 1974) is utilized. Figure 3.5 illustrates the discrete rectangular net of points in the x, t-plane that is used in the development of the numerical integration scheme. In the implicit scheme the values of all net points at time j are used simultaneously to obtain the values for all netpoints at time j + 1 (Figure 3.5).

The continuity and simplified momentum equations (equations 2.3 and 2.4) have independent variables, x and t, and two dependent variables, y and v. Designating the dependent variables by U, the finite difference approximation by the four point non-central scheme can be written as:

$$U = 1/2\ (U_i^{j+1} + U_{i+1}^{j+1}) \tag{3.1}$$

$$\frac{\partial U}{\partial x} = \frac{1}{\Delta x}\ (U_{i+1}^{j+1} - U_i^{j+1}) \tag{3.2}$$

$$\frac{\partial U}{\partial t} = \frac{1}{2\Delta t}\ (U_i^{j+1} + U_{i+1}^{j+1} - U_i^j - U_{i+1}^j) \tag{3.3}$$

in which the subscripts refer to the spatial positions and the superscripts to the temporal position on the point mesh grid (Figure 3.5).

The numerical solution uses the continuity equation (equation 2.3) written in terms of its finite difference approximations. The continuity equation can be written as:

$$\frac{A}{B}\frac{\partial v}{\partial x} + V\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \tag{3.4}$$

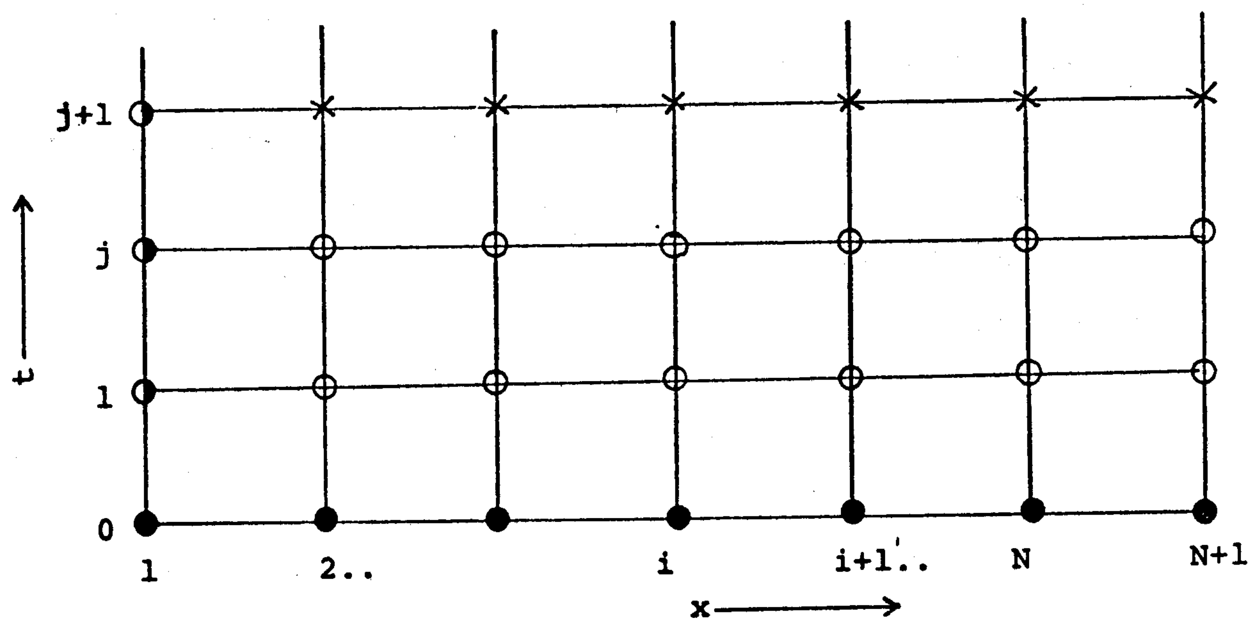
in which

A = flow area, ft^2 (Fig. 3.2)

B = surface flow width, ft

V = average flow velocity, fps

x = length along channel, ft



LEGEND

Initial condition

Upstream boundary condition (from input hydrograph)

Calculated solution

Unknown solution

Figure 3.5 Computational Mesh for the x,t-Plane for a Channel Broken into N Reaches y = wave depth, ft

t = time, sec.

The appropriate finite difference approximations at each node point for the geometry defined in Figure 3.2 are:

$$A = 0.5\ (W(y_{i+1}^{j+1} + y_i^{j+1}) + Theta((XY_{i+1}^{j+1})^2 + (XY_i^{j+1})^2)) \qquad (3.5)$$

$$B = W + Theta(XY_{i+1}^{j+1} + XY_i^{j+1}) \qquad (3.6)$$

$$\frac{\partial v}{\partial x} = \frac{1}{\Delta x}(V_{i+1}^{j+1} - V_i^{j+1}) \qquad (3.7)$$

$$V = 0.5\ (V_{i+1}^{j+1} + V_i^{j+1}) \qquad (3.8)$$

$$\frac{\partial y}{\partial x} = \frac{1}{\Delta x}(y_{i+1}^{j+1} - y_i^{j+1}) \qquad (3.9)$$

$$\frac{\partial y}{\partial t} = \frac{1}{2\Delta t}\ (y_{i+1}^{j+1} + y_i^{j+1} - y_{i+1}^{j} - y_i^{j}) \qquad (3.10)$$

with the unknown velocities in equations 3.5 - 3.10 written in terms of the Manning equation

$$V_{i+1}^{j+1} = \frac{1.49}{N}\ (R_{i+1}^{j+1})^{0.6667}\ S^{0.5} \qquad (3.11)$$

$$V_i^{j+1} = \frac{1.49}{N}\ (R_i^{j+1})^{0.6667}\ S^{0.5} \qquad (3.12)$$

in which

A = flow area, ft^2

B = surface flow width, ft

N = Mannings Roughness Coefficient

R = hydraulic radius, ft

S = slope of channel bed

V = average flow velocity, fps

W = channel bottom width, ft

i = subscript for interior reach (position along channel)

j = superscript for time

t = time

x = length along channel, ft

y = wave depth, ft

XY = water elevation above rectangular portion of channel (Figure 3.2)

Theta = See Figure 3.2

Δt = time step, sec

Δx = distance step, ft

Letting M represent the number of segments a flow reach has been divided into, the kinematic problem contains M+1 unknowns, i.e., y_i^{j+1} for i=1,2,...M, M+1 (Figure 3.5). The finite difference approximation to the continuity equation (equation 3.4) written at each internal grid point and the downstream boundary constitutes M equations. (In this investigation, M=6, Δt = 180 seconds, and Δx = reach length/M.) The remaining required equation is equation 2.4 for the upstream boundary and can be written as:

$$Q_1^{j+1} - \frac{1.49}{N} (R_1^{j+1})^{0.6667} S^{0.5} (W\ y_1^{j+1} + (XY_1^{j+1})^2 \text{ Theta}) = 0 \qquad (3.13)$$

in which

Q_1^{j+1} = discharge to the channel at time j+1, cfs

R_1^{j+1} = hydraulic radius for y_1^{j+1}, ft

S = slope of channel bed

W = channel bottom width, ft

XY_1^{j+1} = normal depth above rectangular portion of channel for discharge Q_1^{j+1}, ft

y_1^{j+1} = normal depth for discharge Q_1^{j+1}, ft

There is no standard method presently available for solving the system of nonlinear algebraic equations defined by equations 3.4 and 3.13. There are, however, a number of well-known schemes for solving a system of linear equations. The Newton-Raphson iteration method (Ralston, 1965; Fread, 1975) was used to reduce the nonlinear equations to linear equations.

This solution technique initially sets the value of the unknown depth at j+1 equal to that which existed at j. In order for the solution scheme to begin the initial conditions at j must be known. This requires that the depth and velocity at all mesh points i = 1, 2, ...M, M+1, are known at a given time. In this investigation a constant base flow of 2 cubic feet per second throughout all reaches at the initial time was assumed. Thus, the initial conditions can be treated as steady flow, where the initial velocities and depth are determined by Manning's equation (equation 3.11).

Channel Junctions

To derive a downstream hydrograph in a tree type channel network (Fig. 3.1) it is necessary to route flood hydrographs through channel junctions. Chow (1959) emphasized the difficulties involved even when steady flow is being considered. For unsteady flow the problem becomes more complicated (Harley, Perkins, and Wood, 1975) when the backwater effects and energy losses due to the junction combined with the nonlinearities of the St. Venant equations (equations 2.1 and 2.2) are considered.

Sevuk and Yen (1974) compared 4 different approaches presently being used to route unsteady flow through a channel junction. Each method makes certain assumptions and has its own drawbacks. For this investigation the sequential type junction has been used. This junction flow model follows the simplest approach where only the continuity equation is used without considering storage within the junction, i.e., $Q_1 + Q_2 = Q_3$, in which Q is the discharge and the subscripts 1 and 2 refer to the upstream channels and 3 to the downstream (outflow) channel. The flow from the upstream channel is computed without considering the backwater effect of the junction (Miller, 1971). The sub-basin hydrograph at the downstream end of each reach is added to the routed flow from upstream reaches to form a composite hydrograph (Fig. 3.1).

Numerical Model Verification

The accuracy of the kinematic wave routing computer program for a straight channel was determined in a comparison with results found in the literature. A single verification test run of the model was based on identical channel and hydrograph features that were used by Gunaratnam and Perkins (1970) in their flood routing investigations. In an effort to reproduce their results, the same time and distance steps (Δt = 3 hours, Δx = 5 miles) were also selected. The comparison between their results and the program used in this investigation can be seen in Figure 3.6. This test indicated that there were no glaring errors in the routing program used herein. There is, however, no guarantee that the model is the best representation of natural flow phenomena.

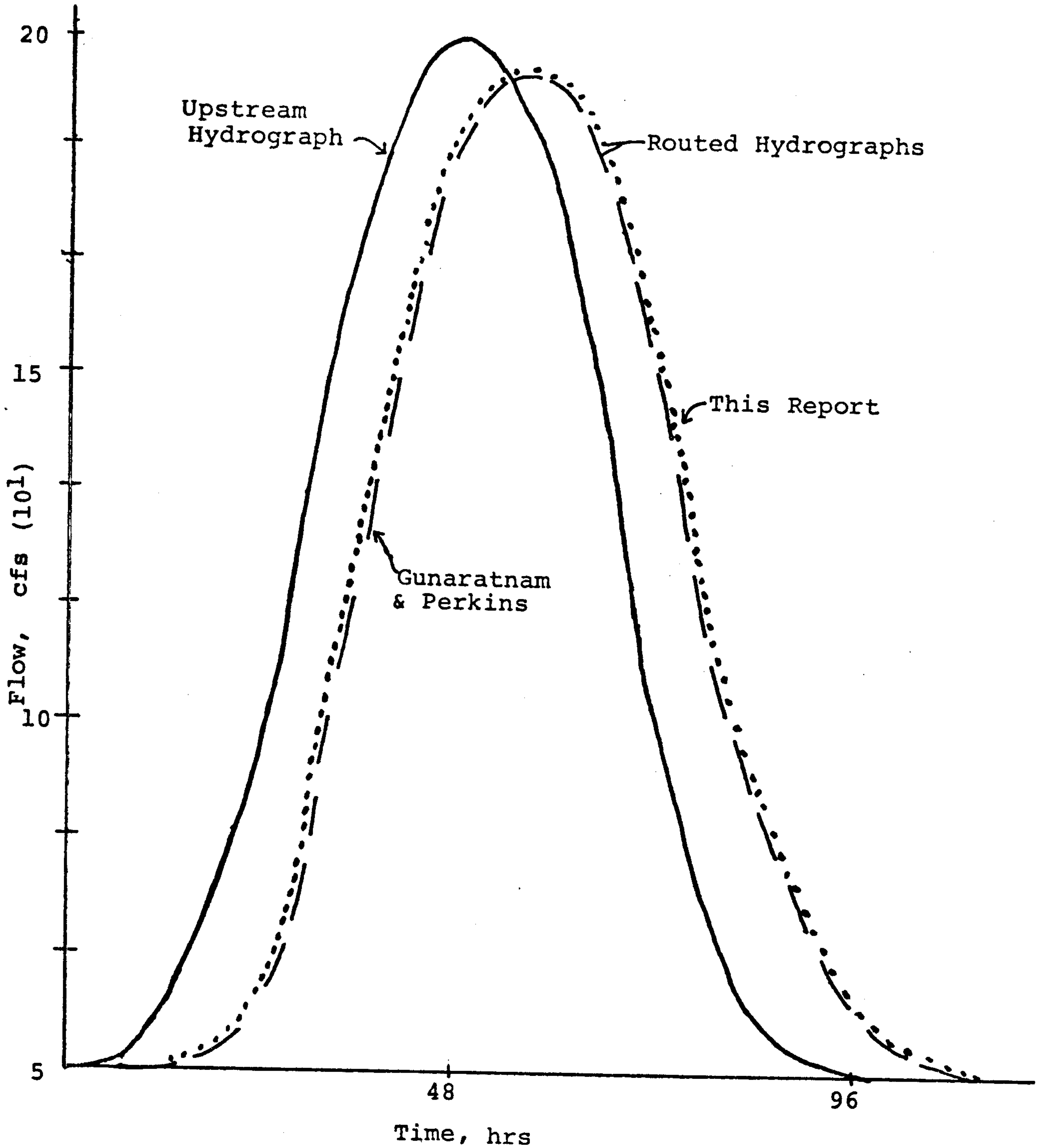


Figure 3.6 Implicit Routing Program Numerical Verification (Comparison with Gunaratnam and Perkins (1970))

4. EXAMINATION OF MIXED CONTROL STRATEGIES

4.1 Introduction

The effectiveness of using various runoff control measures in sub-basins as a means to maintain watershed peak outflow rates to the pre-urbanized peak flow rates was determined by imposing runoff controls to the sub-basins of hypothetical watersheds. In all cases three sub-basins constituted the hypothetical watershed. Mixed control strategies were examined by testing the combined response of the watershed for the eight possible controlled-uncontrolled combinations with all other variables held constant. Areal, channel, and rainfall variations were applied to the watershed to see how each variable influenced the effectiveness of runoff control options. In each test case, two of the variables were held constant while the third was varied.

Sections 4.2 - 4.4 illustrate the results of these test cases. Section 4.5 compares two simulations having equal watershed and rainfall characteristics but with one having a reduction (30 - 50%) in the incremental runoff volume increase due to urbanization. Sections 4.6 - 4.8 discuss the overall effectiveness of blanket and mixed control policies, and include some conclusions on urban runoff control.

4.2 Control Strategy Effectiveness as A Function of Channel Variables

Invariably when tests of the form of those conducted in this investigation are reported the reader is deluged with information. All pertinent test results are given in Figures 4.1 to 4.5, each figure contains considerable information so the reader is advised to take some time examining them. The legend attached to each figure explains all symbols used in the figures. The comparisons all show the ratio of the peak flow at the watershed outlet,

Q_x, to the peak fow that occurs at the outlet in the natural state, Q_{ux}. In all cases the results reflect approximate watershed response to a five year recurrence interval storm (see section 3.2 and Appendix B for details).

A typical set of results is contained in Figure 4.1a. Here the effects of eight strategies are shown. In this case all three sub-basins have equal areas, the upper sub-basin (A_1) discharges into channel 1 where flow travels a length L_1 = 5000 feet before discharge from the next sub-basin (A_2) enters the channel. The combined flow q_4 (Figure 3.1) travels down the second channel segment of length L_2 = 5000 feet. Both channel segments have slopes of 0.002 or approximately ten feet per mile. The channel section was taken to be: W = 5 ft, D = 1 ft and LA = RA = $\pi/8$ radians (i.e. side slope 1:4), where the symbols are defined in Figure 3.2, and Manning's N = 0.025 which is representative of a lower bound for channel roughness in typical urban watersheds.

Each combination of channel variables was evaluated on the same watershed geometry and with the same rainfall duration. Each sub-basin was 667 acres while the rainfall duration was of sufficient length to generate the peak flow rate for each land condition under consideration, i.e., for the undeveloped case (UC in Fig. 4.1), rainfall duration equals 96 minutes (i.e. 80% of the time to peak runoff rate, which takes 120 minutes), while for the developed cases rainfall duration equals 58 minutes (80% of the time to the peak runoff rate which takes 72 minutes) (Table 3.1). The different duration storms were assumed to start at the same time.

Figures 4.1a, 4.1b, 4.1c and 4.1d are representative of a large number of test cases where the values of channel slope, length, and Manning's "N" were varied. Figure 4.1a represents the results for the baseline parameter set ($L_1 = L_2$ = 5,000 ft, $S_1 = S_2$ = .002, "N" = .025). Figure 4.1b changes "N" to .045; Figure 4.1c changes channel slopes to .008; Figure 4.1d varies channel lengths to 10,000 ft.

The evaluation was done by first determining the peak undeveloped flow rate (Q_{ux} in Figure 4.1a-d) for each combination of channel variables. This was done by routing the undeveloped sub-basin hydrographs through the channel network to point X the outlet of the three sub-basin watershed (Figure 3.1). After Q_{ux} was known, the sub-basins were assumed to be totally developed with mixed control options in force. A combined watershed outlet hydrograph for each mixed control option was then generated. The peak runoff rate (Q_x in Figure 4.1a-d) from the developed watershed hydrograph is then non-dimensionalized by dividing it by Q_{ux} for the particular channel characteristics under consideration. These ratios are graphed in Figures 4.1a-d and tabulated in Table 4.1 and can be used to evaluate the effectiveness of the different mixed control options. For example, the base line parameter set (Figure 4.1a) with all three sub-basins controlled (C-C-C) has a ratio of 1.06. But Figure 4.1d ($L_1 = L_2$ = 10,000 ft.) with the same control has a ratio of 1.30. Therefore, an increase in the channel length significantly reduced the effectiveness of the control option for the areal distribution and rainfall duration under consideration. For comparison, an increase in Manning's "N" to .045 (Figure 4.1b) resulted in a (CCC) ratio of 1.16 while an increase in channel slope to .8% (Figure 4.1c) produced a ratio of 1.02.

Varying the channel parameters does significantly alter the effectiveness of the different mixed control options. Figure 4.1d ($L_1 = L_2$ = 10,000 ft.) illustrates a test case where leaving one sub-basin uncontrolled (C-C-D) produces a ratio (1.26) which is less than the ratio (1.30) obtained with control on all three sub-basins (C-C-C). These results suggest that a watershed must be specifically examined for behavior to different control options. A general blanket policy, while administratively relatively easy to handle, can be counter productive.

Figure 4.1a Baseline Parameter Set

$L_1=L_2=$ 5000 ft. $S_1=S_2=0.002$ ft/ft "N"=.025 Qux=172 cfs
Areal Variation = $A_1:A_2:A_3$ = 667:667:667
Rainfall duration equals .8 TP for land condition under consideration.

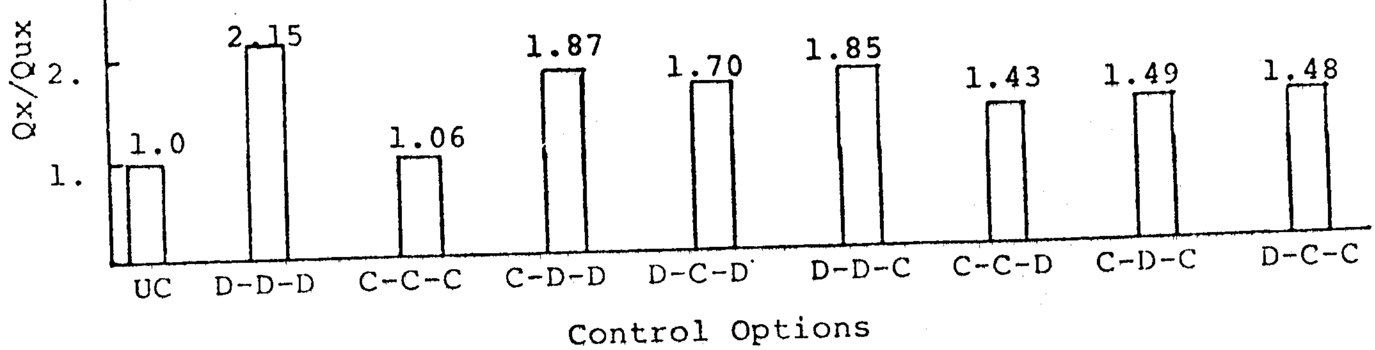
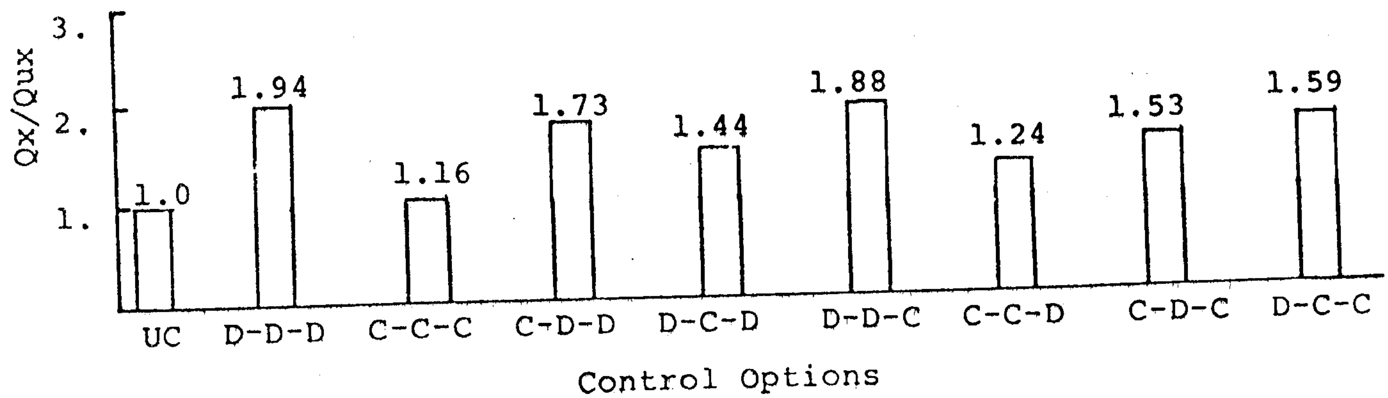


Figure 4.1b "N" = .045

$L_1=L_2=$ 5000 ft. $S_1=S_2=0.002$ ft/ft "N"=.045 Qux=158 cfs
Areal Variation = $A_1:A_2:A_3$ = 667:667:667
Rainfall duration equals .8 TP for land condition under consideration.



LEGEND

A_1 = upstream sub-basin area, acres (Figure 3.1)

A_2 = middle sub-basin area, acres (Figure 3.1)

A_3 = downstream sub-basin area, acres (Figure 3.1)

C = controlled release of developed hydrograph at preurbanization peak flow, cfs

D = uncontrolled developed hydrograph, cfs

C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed

L_1 = channel length for reach I (Figure 3.1), ft

L_2 = channel length for reach II (Figure 3.1), ft

"N" = Manning's roughness coefficient

Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs

Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs

$\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)

S_1 = channel slope for reach I (Figure 3.1), ft/ft

S_2 = channel slope for reach II (Figure 3.1), ft/ft

UC = undeveloped conditions in all sub-basins

V_R = runoff volume reduction

Figure 4.1 Effectiveness as a Function of Channel Variables

Figure 4.1c $S_1 = S_2 = .008$

$L_1 = L_2 =$ 5000 ft. $S_1 = S_2 = 0.008$ ft/ft "N"=.025 Qux=179 cfs
Areal Variation = $A_1:A_2:A_3$ = 667:667:667
Rainfall duration equals .8 TP for land condition under consideration.

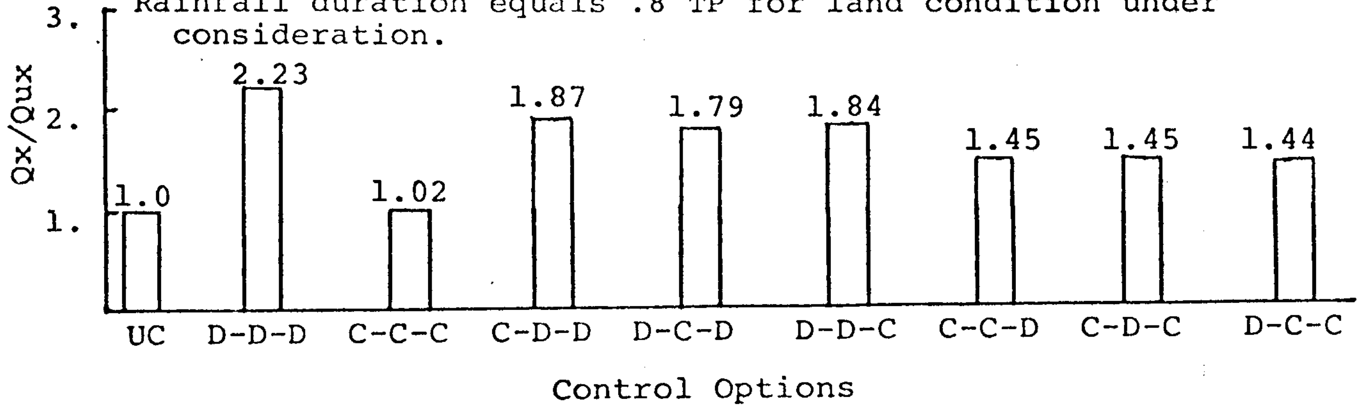
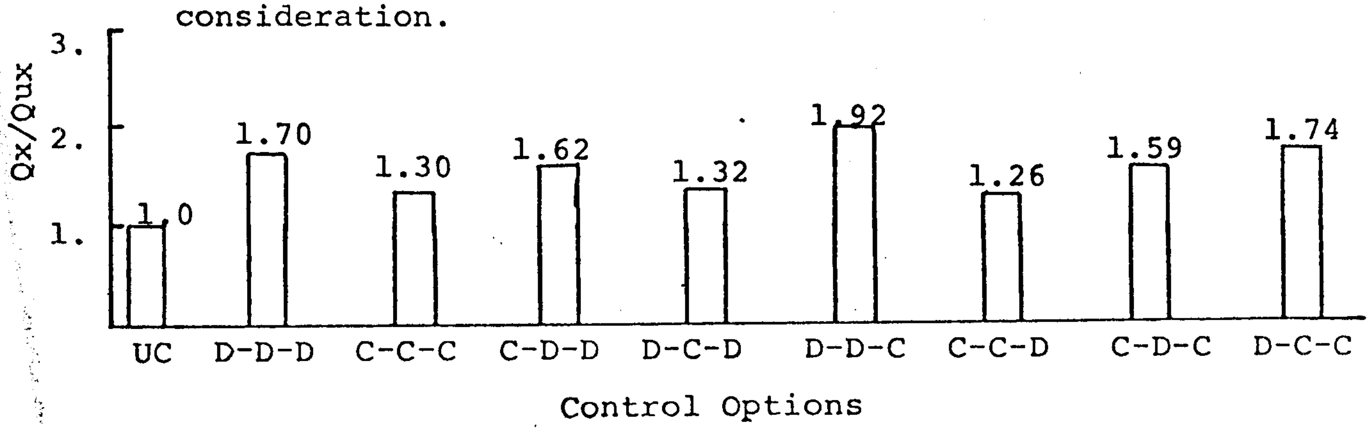


Figure 4.1d $L_1 = L_2$ = 10,000 ft.

$L_1 = L_2$ =10,000 ft. $S_1 = S_2 = 0.002$ "N"=.025 Qux=141 cfs
Areal Variation = $A_1:A_2:A_3$ = 667:667:667
Rainfall duration equals .8 TP for land condition under consideration.



LEGEND

- A_1 - upstream sub-basin area, acres (Figure 3.1)
- A_2 - middle sub-basin area, acres (Figure 3.1)
- A_3 - downstream sub-basin area, acres (Figure 3.1)
- C - controlled release of developed hydrograph at preurbanization peak flow, cfs
- D - uncontrolled developed hydrograph, cfs
- C-D-D - A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 - channel length for reach I (Figure 3.1), ft
- L_2 - channel length for reach II (Figure 3.1), ft
- "N" - Manning's roughness coefficient
- Qux - peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx - routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ - ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 - channel slope for reach I (Figure 3.1), ft/ft
- S_2 - channel slope for reach II (Figure 3.1), ft/ft
- UC - undeveloped conditions in all sub-basins
- $\frac{V}{R}$ - runoff volume reduction

Figure 4.1 Effectiveness as a Function of Channel Variables (con't)

Table 4.1 Values of Q_x/Q_{ux} for the Control Strategies Given in Figure 4.1

Test Cases	Conditions	Control Options								Q_{ux} cfs
		D-D-D[a]	C-C-C[a]	C-D-D[a]	D-C-D[a]	D-D-C[a]	C-C-D[a]	C-D-C[a]	D-C-C[a]	
4.1a	base conditions[b]	2.15	1.06	1.87	1.70	1.85	1.43	1.49	1.48	172
4.1b	N = 0.045	1.94	1.16	1.73	1.44	1.88	1.24	1.53	1.59	158
4.1c	$S_1=S_2=$ 0.008	2.23	1.02	1.87	1.79	1.84	1.45	1.45	1.44	179
4.1d	$L_1=L_2=$ 10000 ft	1.70	1.30	1.62	1.32	1.92	1.26	1.59	1.74	141

[a] See Figure 4.1 for definitions of the control combinations.

[b] $A_1=A_2=A_3$ = 667 acres; $L_1=L_2$=5000 ft; $S_1=S_2$=0.002; N=0.025; input hydrographs, Table 3.1 precipitation duration = 0.8 TP for land condition under consideration.

4.3 Control Strategy Effectiveness As A Function of Rainfall Duration

Figures 4.2a and 4.2b represent the same sub-basin areal distribution and channel parameters but with differing precipitation duration. For Figure 4.2a, the duration equals 80% of the time to peak (TP) for the land condition under consideration. Figure 4.2b shows the effects of a rainfall duration of 96 minutes for all cases. (It must be reemphasized, however, that rainfall duration is assumed to be a function of time of concentration (Appendix B) and is used only to generate approximate sub-basin hydrographs.)

In this investigation, when the rainfall duration exceeded 80% of the time to peak for the developed case, the developed condition runoff hydrographs had to be modified since rainfall duration is directly related to rainfall intensity (Figure 4.3). The peak flow resulting from use of the SCS method for the developed area was therefore arbitrarily reduced by 15% to reflect the reduced rainfall intensity that is associated with the actual longer duration rain. The flow was held constant at 0.85 Q_p (where Q_p corresponds to the shorter duration storm) for the period TP'_D to TP_U corresponding to the time when 0.85 Q_p occurred and the time to peak for the undeveloped cases respectively. This effect is illustrated by the solid curve in Figure 3.4. The modified peak flow was maintained for a period of F_R minutes.

Figures 4.2c, 4.2d, and 4.2e illustrate three different rainfall durations falling on a watershed where A_3 (Figure 3.1) is a large portion of the total watershed area. Figure 4.2c illustrates the case where the rainfall duration equals the 80% of TP for the land condition under consideration. Here the rainfall duration is just sufficient to generate the peak discharge from each sub-basin.

Figure 4.2d shows the effects of rainfall duration equal to 80% of the largest sub-basin's undeveloped TP falling on each sub-basin. Again, the developed area hydrographs were modified to reflect decreased intensity with

Figure 4.2a Rainfall Duration Case I-A

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=141 cfs
Areal Variation = $A_1:A_2:A_3$ = 667:667:667
Rainfall duration equals .8 TP for land condition under consideration.

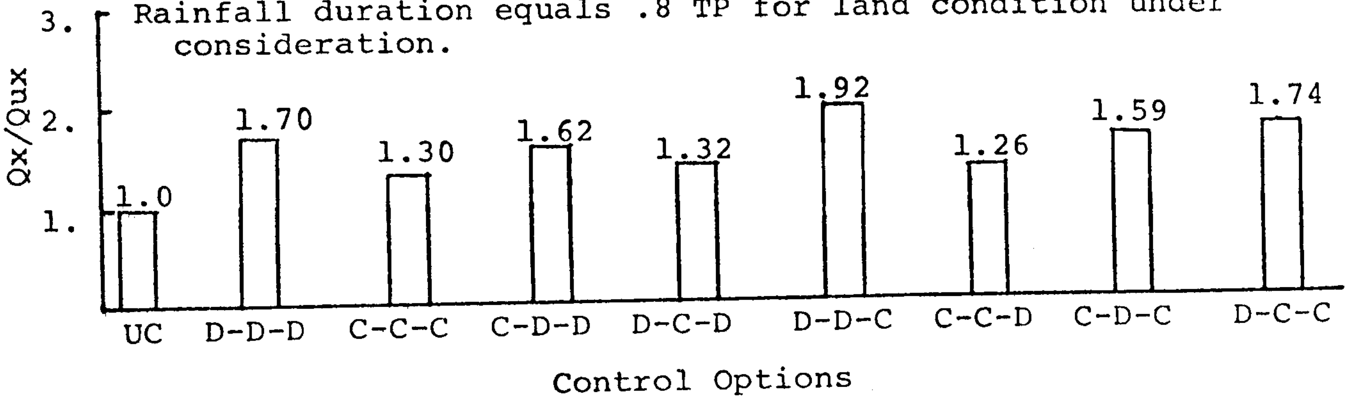
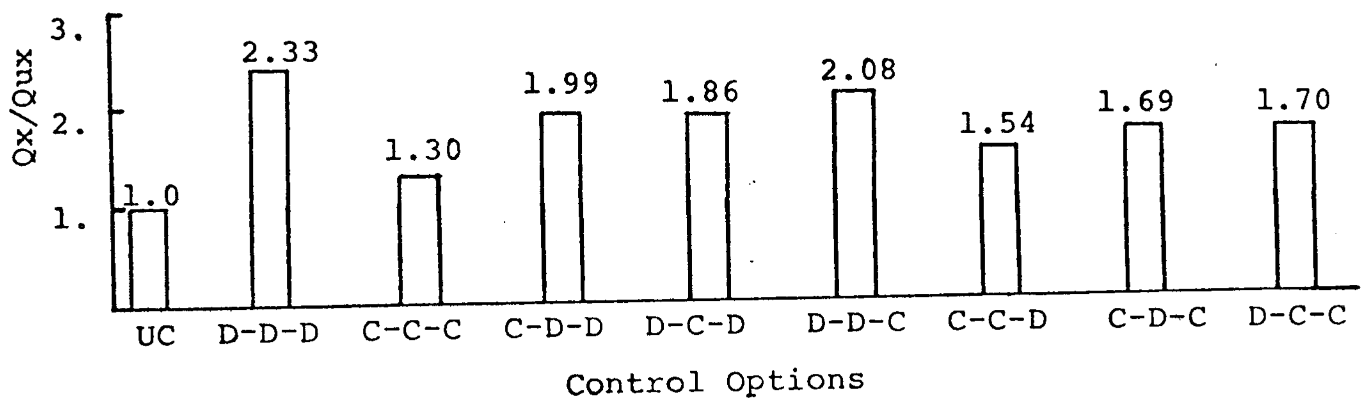


Figure 4.2b Rainfall Duration Case I-B

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=141 cfs
Areal Variation = $A_1:A_2:A_3$ = 667:667:667
Rainfall duration equals .8 of undeveloped TP = 96 minutes



LEGEND

- A_1 = upstream sub-basin area, acres (Figure 3.1)
- A_2 = middle sub-basin area, acres (Figure 3.1)
- A_3 = downstream sub-basin area, acres (Figure 3.1)
- C = controlled release of developed hydrograph at preurbanization peak flow, cfs
- D = uncontrolled developed hydrograph, cfs
- C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 = channel length for reach I (Figure 3.1), ft
- L_2 = channel length for reach II (Figure 3.1), ft
- "N" = Manning's roughness coefficient
- Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 = channel slope for reach I (Figure 3.1), ft/ft
- S_2 = channel slope for reach II (Figure 3.1), ft/ft
- UC = undeveloped conditions in all sub-basins
- V_R = runoff volume reduction

Figure 4.2 Effectiveness as a Function of Rainfall Duration

Figure 4.2c Rainfall Duration Case II-A

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=152 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:500:1000
Rainfall duration equals .8 TP for land condition under consideration for each sub-basin.

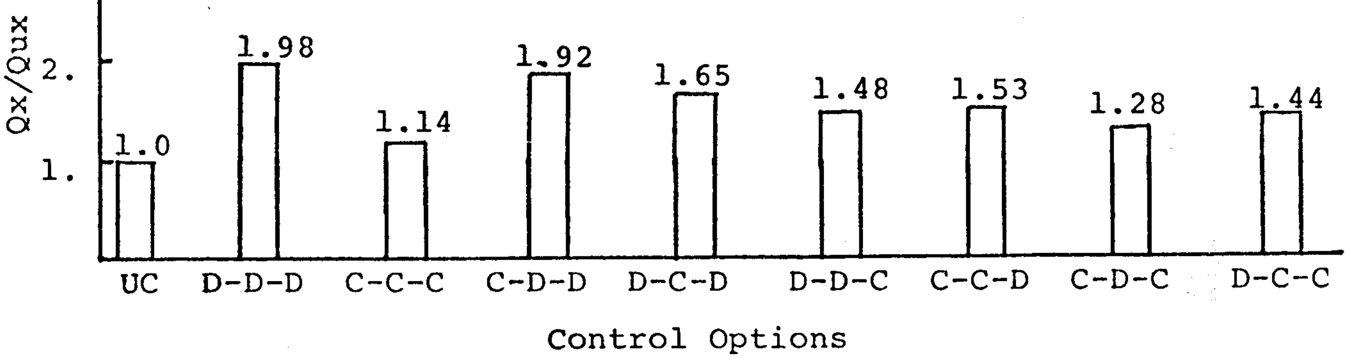
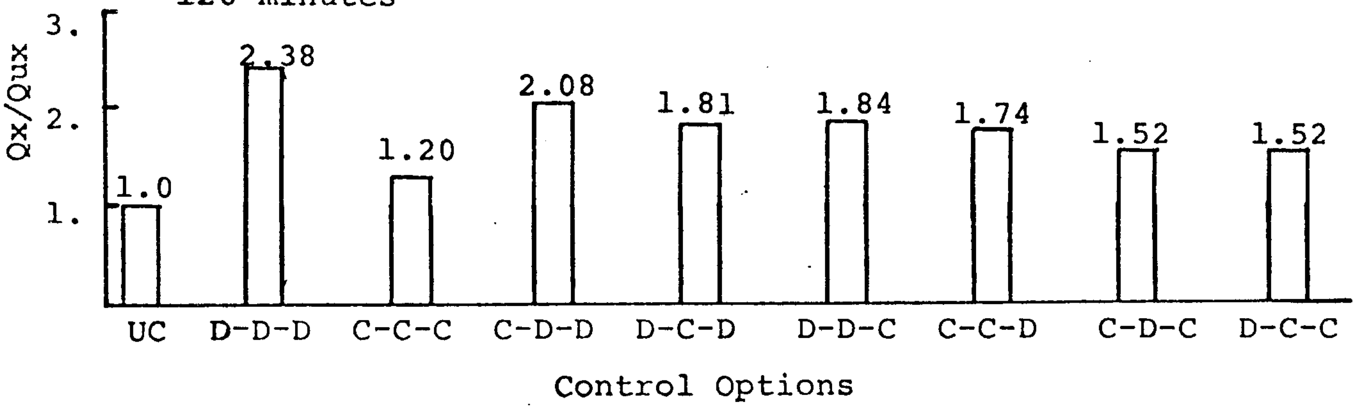


Figure 4.2d Rainfall Duration Case II-B

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=144 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:500:1000
Rainfall duration equals .8 of undeveloped 1000 acre TP = 120 minutes



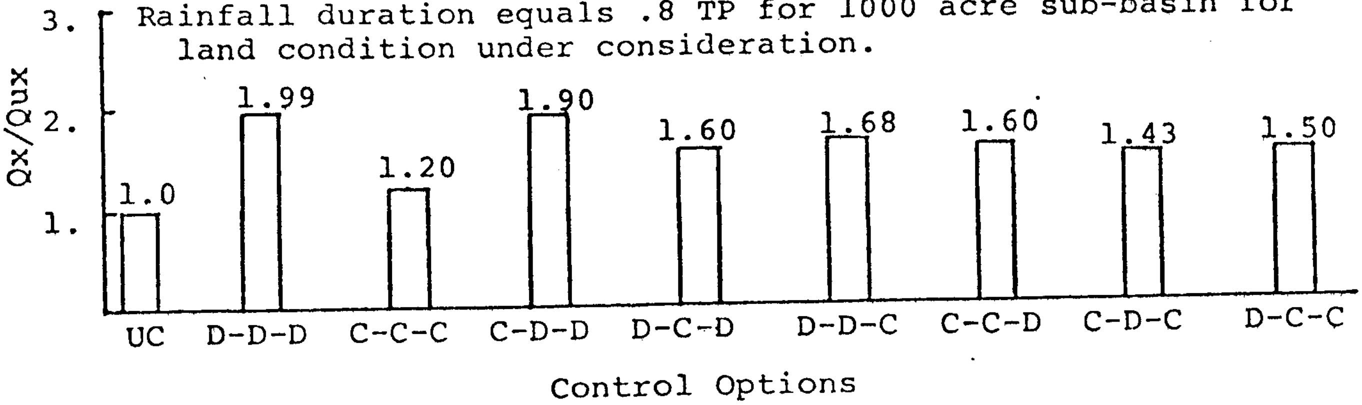
LEGEND

- A_1 = upstream sub-basin area, acres (Figure 3.1)
- A_2 = middle sub-basin area, acres (Figure 3.1)
- A_3 = downstream sub-basin area, acres (Figure 3.1)
- C = controlled release of developed hydrograph at preurbanization peak flow, cfs
- D = uncontrolled developed hydrograph, cfs
- C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 = channel length for reach I (Figure 3.1), ft
- L_2 = channel length for reach II (Figure 3.1), ft
- "N" = Manning's roughness coefficient
- Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 = channel slope for reach I (Figure 3.1), ft/ft
- S_2 = channel slope for reach II (Figure 3.1), ft/ft
- UC = undeveloped conditions in all sub-basins
- V_R = runoff volume reduction

Figure 4.2 Effectiveness as a Function of Rainfall Duration (con't)

Figure 4.2e Rainfall Duration Case II-C

$L_1=L_2=10,000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=144 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:500:1000
Rainfall duration equals .8 TP for 1000 acre sub-basin for land condition under consideration.



LEGEND

A_1 = upstream sub-basin area, acres (Figure 3.1)

A_2 = middle sub-basin area, acres (Figure 3.1)

A_3 = downstream sub-basin area, acres (Figure 3.1)

C = controlled release of developed hydrograph at preurbanization peak flow, cfs

D = uncontrolled developed hydrograph, cfs

C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed

L_1 = channel length for reach I (Figure 3.1), ft

L_2 = channel length for reach II (Figure 3.1), ft

"N" = Manning's roughness coefficient

Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs

Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs

$\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)

S_1 = channel slope for reach I (Figure 3.1), ft/ft

S_2 = channel slope for reach II (Figure 3.1), ft/ft

UC = undeveloped conditions in all sub-basins

V_R = runoff volume reduction

Figure 4.2 Effectiveness as a Function of Rainfall Duration (con't) the increased rainfall duration.

Figure 4.2e illustrates the effects of rainfall durations equal to 80% of the largest sub-basin TP for the land condition under consideration. This required that the hydrographs from sub-basins with areas less than the largest sub-basin be modified. These hydrographs were modified by the same procedure used for the developed cases in Figures 4.2b and 4.2d.

These test cases show that rainfall duration is of considerable importance in determining the effectiveness of runoff control options. For example, consider the control option D-C-D (i.e. A_1 and A_3 are developed and not controlled, A_2 is developed and controlled) in Figures 4.2a and 4.2b. A less intense longer duration storm (Figure 4.2b) yielded a Q_x/Q_{ux} ratio of 1.86 while a more intense storm (Figure 4.2a) had a ratio of only 1.32. Generally for the cases tested, the less intense longer duration storms produced larger ratios than did the more intense storms for all control options employed (Table 4.2).

4.4 Control Strategy Effectiveness As A Function Of Areal Distribution of Sub-Basins

For this activity two different areal schemes were investigated. The first scheme contained sub-basins with areas of 500, 500 and 1,000 acres. The second was comprised of 500, 750, and 750 acre sub-basins. In both cases the different areal combinations were utilized, i.e., $A_1:A_2:A_3$ = 500:500:1,000, 500:1,000:500, 1,000:500:500; and 500:750:750, 750:500:750, and 750:750:500. Figures 4.4a, 4.4b, and 4.4c constitute the 500-500-1,000 acre possibilities while 4.4d, 4.4e, and 4.4f comprise the 500-750-750 acre combinations.

These figures clearly illustrate how the effectiveness of the various control strategies depends heavily on areal distribution. For example, take the control option of D-D-C in Figures 4.4a, 4.4b, and 4.4c. Controlling the lower sub-basin (A_3) when it comprises 50% of the watershed area (Figure 4.4a) yields a ratio of 1.68. But when upper sub-basin (A_1) comprises 50% of the

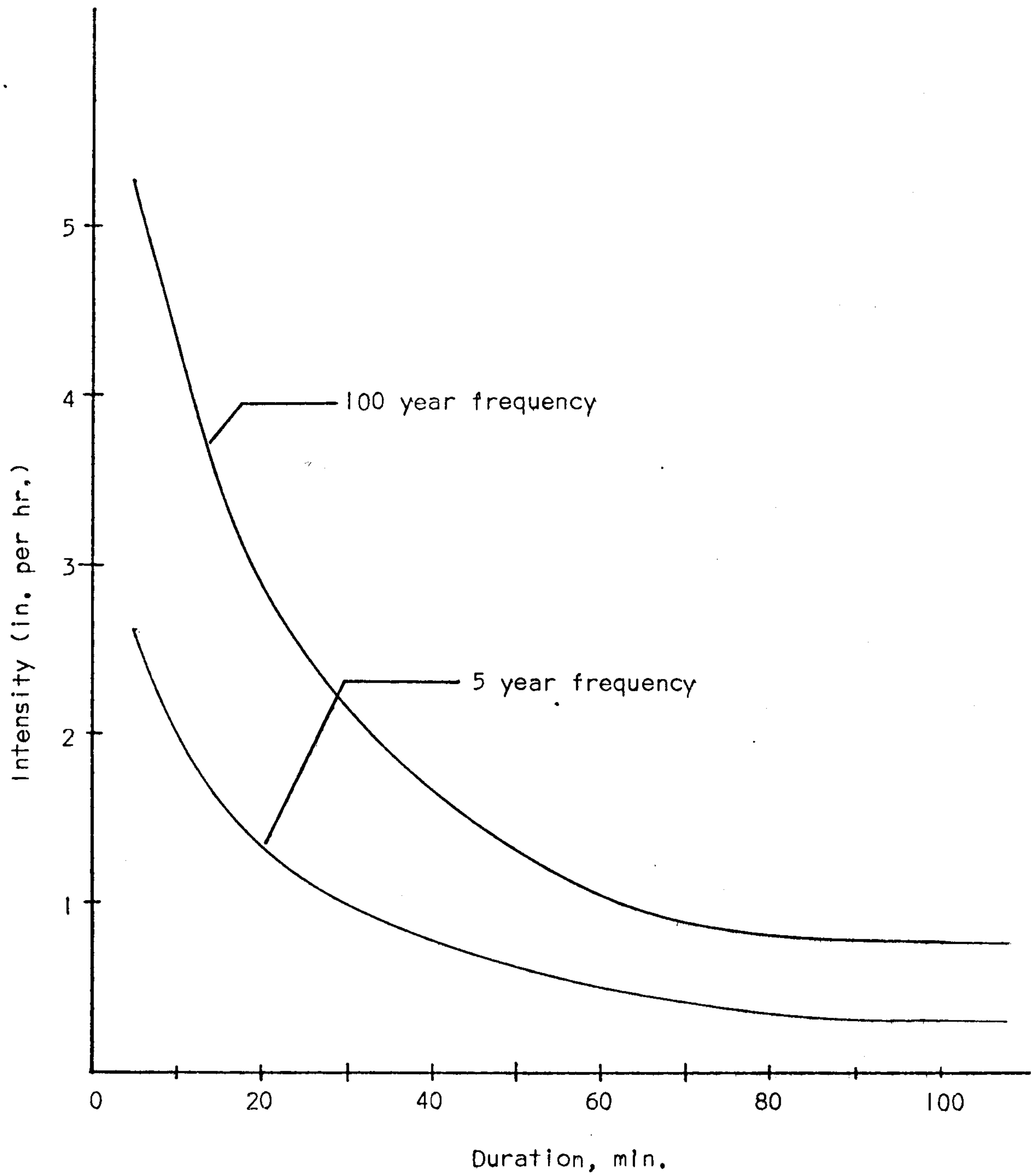


Figure 4.3 Rainfall Intensity-Duration-Frequency for Spokane, WA

Table 4.2 Values of Q_x/Q_{ux} for the Control Strategies Given in Figure 4.2

Test Cases	Conditions	Control Options								Q_{ux} cfs
		D-D-D[a]	C-C-C[a]	C-D-D[a]	D-C-D[a]	D-D-C[a]	C-C-D[a]	C-D-C[a]	D-C-C[a]	
4.2a	Modified Base[b,c] Conditions (L_1 and L_2 changed)	1.70	1.30	1.62	1.32	1.92	1.26	1.59	1.74	141
4.2b	Uniform[b,d] Storm Duration on all Sub-basins	2.33	1.30	1.99	1.86	2.08	1.54	1.69	1.70	141
4.2c	Modified Base[b,c] Conditions A_1=A_2=500, A_3=1000	1.98	1.14	1.92	1.65	1.48	1.53	1.28	1.44	152
4.2d	Uniform Storm[b,d] duration on all sub-basins A_1=A_2=500, A_3=1000	2.38	1.20	2.08	1.81	1.84	1.74	1.52	1.52	144
4.2e	Storm Duration[b,e] Vary A_1=A_2=500, A_3=1000	1.99	1.20	1.90	1.60	1.68	1.60	1.43	1.50	144

[a] See Figure 4.2 for definitions of the control options.

[b] A_1=A_2=A_3=667 acres; L_1=L_2=10000 ft; S_1=S_2=0.002; N=0.025; Input hydrographs, Table 3.1.

[c] Precipitation duration = 0.8 TP for land condition under consideration.

[d] Precipitation duration = 0.8 TP for underdeveloped land condition of A_3

[e] Precipitation duration = 0.8 TP for land condition of A_3

Figure 4.4a $A_1:A_2:A_3$ = 500:500:1000

$L_1=L_2$=10,000 ft. $S_1=S_2$=0.002 "N"=.025 Qux=144 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:500:1000
Rainfall duration equals .8 TP for 1000 acre sub-basin for land condition under consideration.

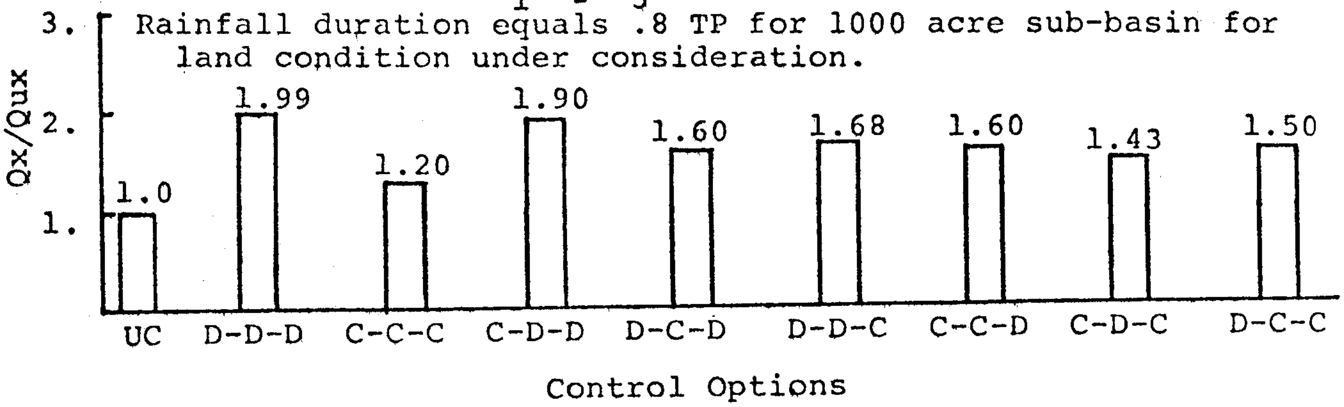
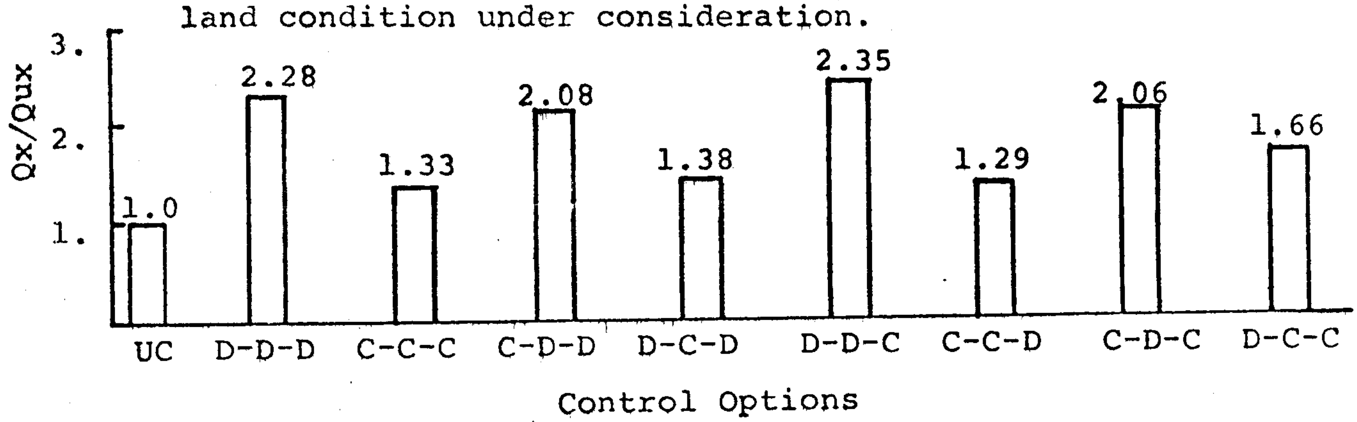


Figure 4.4b $A_1:A_2:A_3$ = 500:1000:500

$L_1=L_2$=10,000 ft. $S_1=S_2$=0.002 "N"=.025 Qux=130 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:1000:500
Rainfall duration equals .8 TP for 1000 acre sub-basin for land condition under consideration.



LEGEND

A_1 = upstream sub-basin area, acres (Figure 3.1)

A_2 = middle sub-basin area, acres (Figure 3.1)

A_3 = downstream sub-basin area, acres (Figure 3.1)

C = controlled release of developed hydrograph at preurbanization peak flow, cfs

D = uncontrolled developed hydrograph, cfs

C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed

L_1 = channel length for reach I (Figure 3.1), ft

L_2 = channel length for reach II (Figure 3.1), ft

"N" = Manning's roughness coefficient

Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs

Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs

$\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)

S_1 = channel slope for reach I (Figure 3.1), ft/ft

S_2 = channel slope for reach II (Figure 3.1), ft/ft

UC = undeveloped conditions in all sub-basins

V_R = runoff volume reduction

Figure 4.4 Effectiveness as a Function of Areal Distribution

Figure 4.4c $A_1:A_2:A_3 = 1000:500:500$

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=100 cfs
Areal Variation = $A_1:A_2:A_3$ = 1000:500:500
Rainfall duration equals .8 TP for 1000 acre sub-basin for condition under consideration.

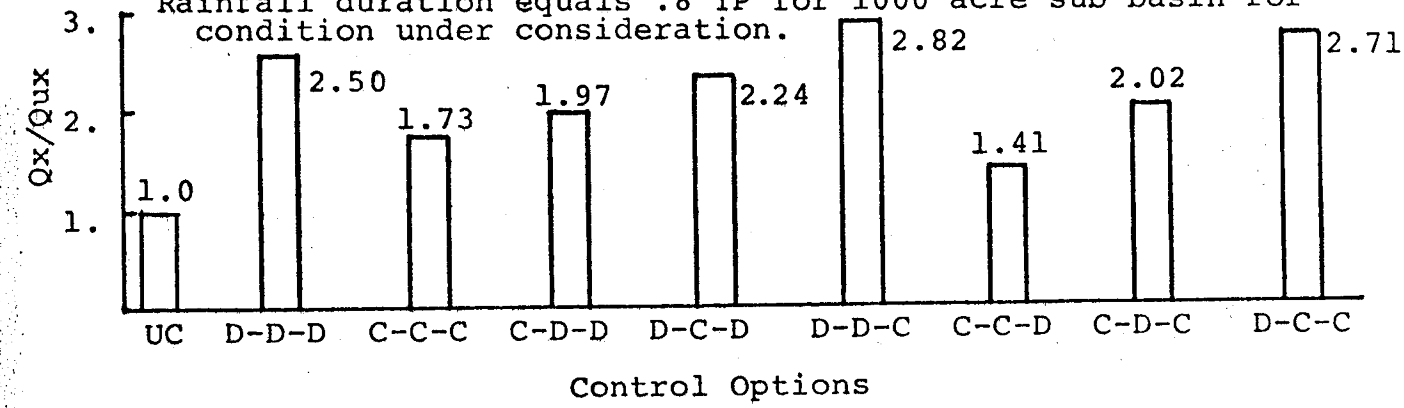
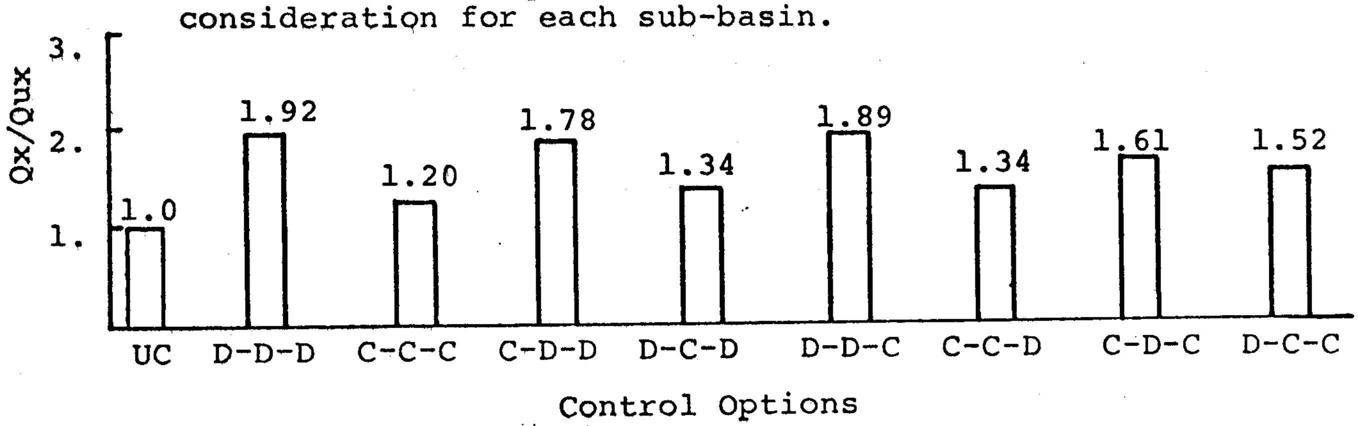


Figure 4.4d $A_1:A_2:A_3 = 500:750:750$

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=149 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:750:750
Rainfall duration equals .8 TP for land condition under consideration for each sub-basin.



LEGEND

- A_1 = upstream sub-basin area, acres (Figure 3.1)
- A_2 = middle sub-basin area, acres (Figure 3.1)
- A_3 = downstream sub-basin area, acres (Figure 3.1)
- C = controlled release of developed hydrograph at preurbanization peak flow, cfs
- D = uncontrolled developed hydrograph, cfs
- C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 = channel length for reach I (Figure 3.1), ft
- L_2 = channel length for reach II (Figure 3.1), ft
- "N" = Manning's roughness coefficient
- Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 = channel slope for reach I (Figure 3.1), ft/ft
- S_2 = channel slope for reach II (Figure 3.1), ft/ft
- UC = undeveloped conditions in all sub-basins
- V_R = runoff volume reduction

Figure 4.4 Effectiveness as a Function of Areal Distribution (con't)

Figure 4.4e $A_1:A_2:A_3$ = 750:500:750

$L_1=L_2$=10,000 ft. $S_1=S_2$=0.002 "N"=.025 Qux=133 cfs
Areal Variation = $A_1:A_2:A_3$ = 750:500:750
Rainfall duration equals .8 TP for land condition under consideration for each sub-basin.

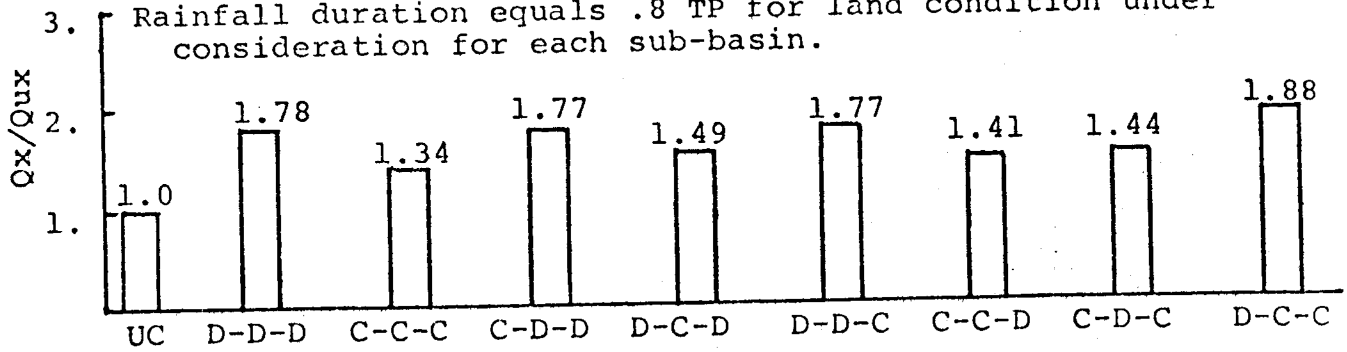
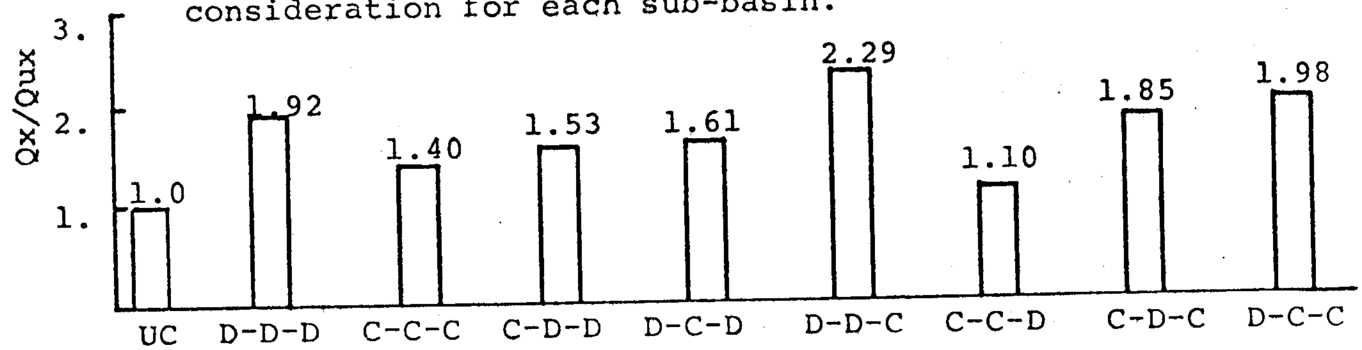


Figure 4.4f $A_1:A_2:A_3$ = 750:750:500

$L_1=L_2$=10,000 ft. $S_1=S_2$=0.002 "N"=.025 Qux=127 cfs
Areal Variation = $A_1:A_2:A_3$ = 750:750:500
Rainfall duration equals .8 TP for land condition under consideration for each sub-basin.



LEGEND

- A_1 = upstream sub-basin area, acres (Figure 3.1)
- A_2 = middle sub-basin area, acres (Figure 3.1)
- A_3 = downstream sub-basin area, acres (Figure 3.1)
- C = controlled release of developed hydrograph at preurbanization peak flow, cfs
- D = uncontrolled developed hydrograph, cfs
- C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 = channel length for reach I (Figure 3.1), ft
- L_2 = channel length for reach II (Figure 3.1), ft
- "N" = Manning's roughness coefficient
- Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 = channel slope for reach I (Figure 3.1), ft/ft
- S_2 = channel slope for reach II (Figure 3.1), ft/ft
- UC = undeveloped conditions in all sub-basins
- V_R = runoff volume reduction

Figure 4.4 Effectiveness as a Function of Areal Distribution (con't) watershed area, the ratio for controlling A_3 jumps to 2.82. This ratio (2.82) is even higher than the ratio (2.50) for no sub-basin control (D-D-D). The information contained in the figures is summarized in Table 4.3.

4.5 Control Strategy Effectiveness As A Function Of Runoff Volume Reduction

This portion of the inquiry examined the impact of reducing the time for which controlled release was at its maximum rate. This duration (F_c in Fig. 3.4) was cut by 25%. For example, if the controlled release ran at 61 cfs for 60 minutes, this release time would be reduced from 60 to 45 minutes. This reduction corresponds to a reduction in the incremental volume of runoff due to urbanization of about 30 to 50%. Figures 4.5a, 4.5b, and 4.5c list the results in comparison with the results from no reduction in the volume increase due to urbanization.

Reducing the runoff volume increase due to urbanization by 30 - 50% had very little impact on the effectiveness of the control options. Figure 4.5c is the only test case that exhibited any significant improvement with this runoff volume reduction. *In situ* measures which would hold the land response to precipitation to be the same as the natural state would, however, be beneficial provided stream channels were left in the natural state. Such measures, however, are costly and difficult to implement.

4.6 Effectiveness of Blanket Control Policies

Blanket control (C-C-C) results in flow rates that exceed the maximum undeveloped flow rate from 2 to 74% for 5 year recurrence interval storms falling on the test watershed configurations used here. The majority fall in the 15-35% range. If the drainage management goal was to maintain the peak discharge rate from the watershed at the rate which existed prior to urbanization, then most of the cases are not very effective. Not only were the undeveloped peak flows surpassed but the time during which the stream

Table 4.3 Values of Q_x/Q_{ux} for the Control Strategies Given in Figure 4.4

Test Cases	Conditions	Control Options								Q_{ux} cfs
		D–D–D[a]	C–C–C[a]	C–D–D[a]	D–C–D[a]	D–D–C[a]	C–C–D[a]	C–D–C[a]	D–C–C[a]	
4.4a	Modified base[b,c] $A_1:A_2:A_3$=500:500:1000	1.99	1.20	1.90	1.60	1.68	1.60	1.43	1.50	144
4.4b	$A_1:A_2:A_3$= 500:1000:500[c]	2.28	1.33	2.08	1.38	2.35	1.29	2.06	1.66	130
4.4c	$A_1:A_2:A_3$= 100:500:500[c]	2.50	1.73	1.97	2.24	2.82	1.41	2.02	2.71	100
4.4d	$A_1:A_2:A_3$ = 500:750:750[d]	1.92	1.20	1.78	1.34	1.89	1.34	1.61	1.52	149
4.4e	$A_1:A_2:A_3$ = 750:500:750[d]	1.78	1.34	1.77	1.49	1.77	1.41	1.44	1.88	133
4.4f	$A_1:A_2:A_3$= 750:750:500[d]	1.92	1.40	1.53	1.61	2.29	1.10	1.85	1.98	127

[a]See Figure 4.4 for definition of the control options.

[b]L_1=L_2=10000 ft; S_1=S_2=0.002; N=0.025; Input hydrographs, Table 3.1.

[c]Precipitation duration = 0.8 TP for 1000 Acres for land under condition considered.

[d]Precipitation duration = 0.8 TP for land condition under consideration for each sub-basin.

Figure 4.5a Volume Reduction Case I

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=152 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:500:1000
Rainfall duration equals .8 TP for land condition under consideration for each sub-basin.

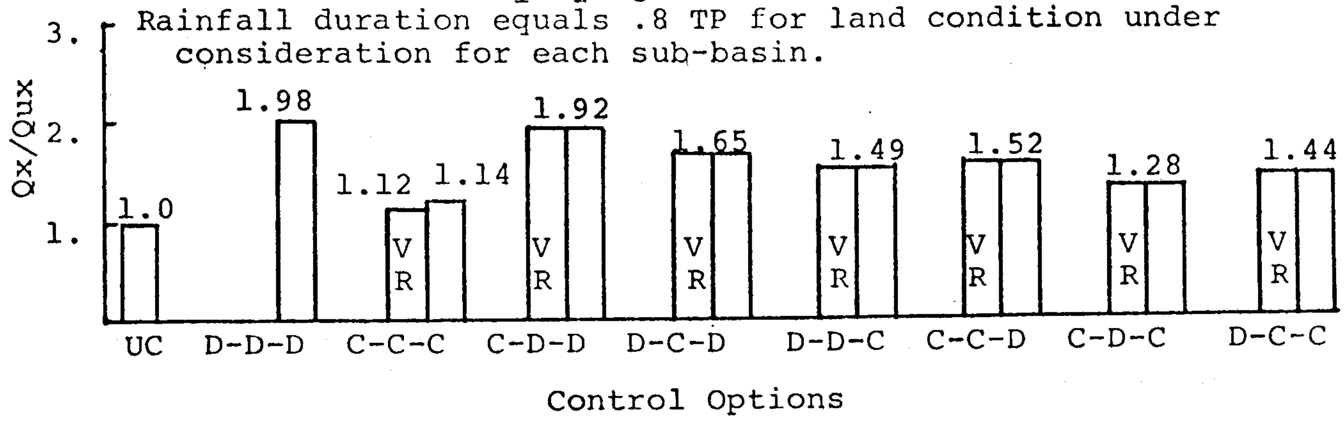
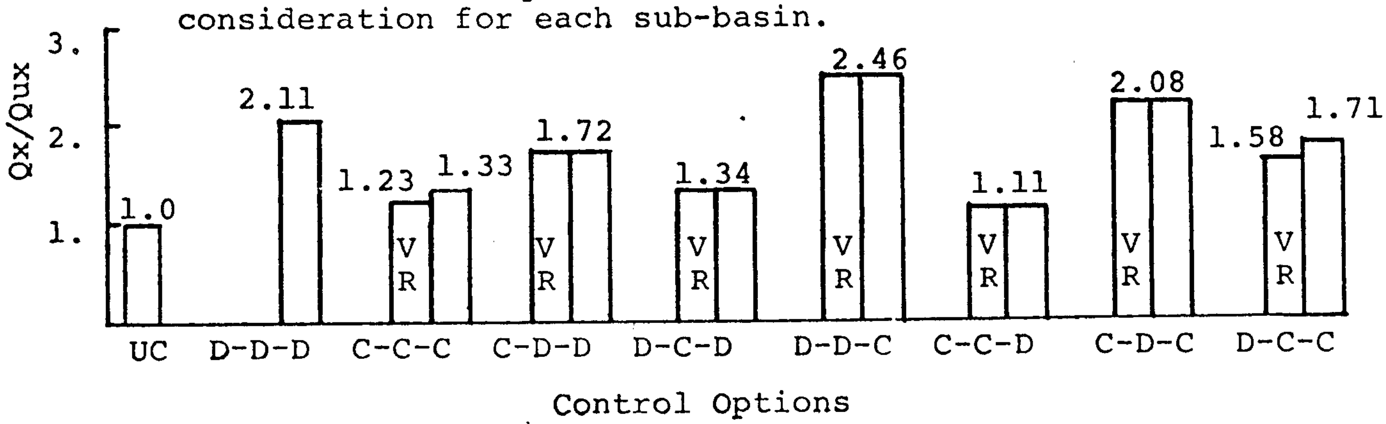


Figure 4.5b Volume Reduction Case II

$L_1=L_2=10{,}000$ ft. $S_1=S_2=0.002$ "N"=.025 Qux=129 cfs
Areal Variation = $A_1:A_2:A_3$ = 500:1000:500
Rainfall duration equals .8 TP for land condition under consideration for each sub-basin.

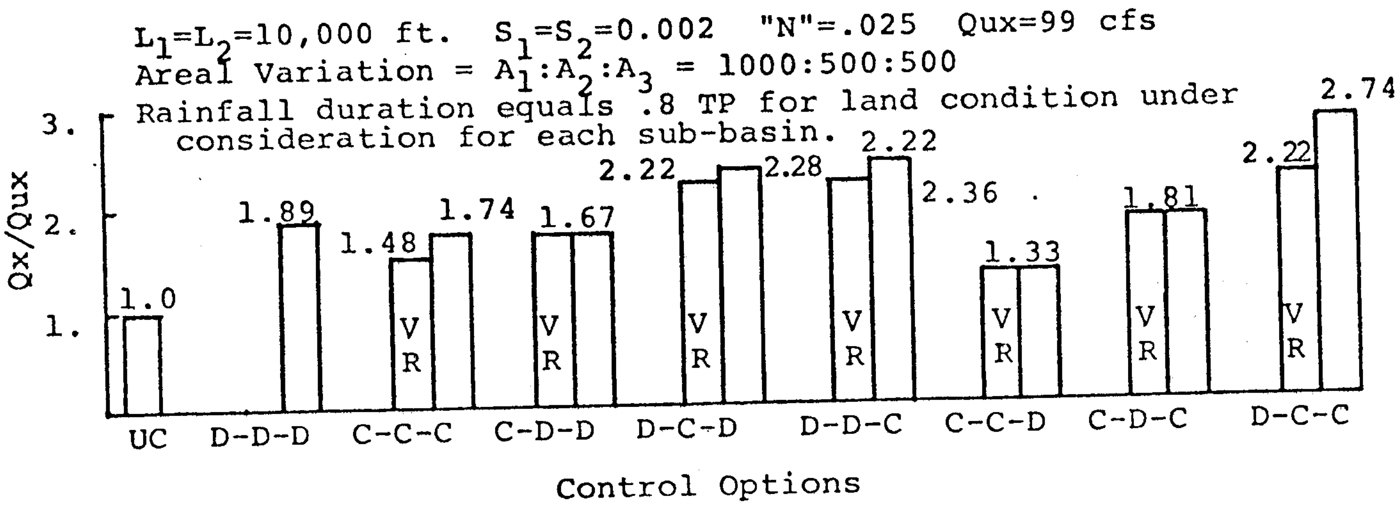


LEGEND

- A_1 = upstream sub-basin area, acres (Figure 3.1)
- A_2 = middle sub-basin area, acres (Figure 3.1)
- A_3 = downstream sub-basin area, acres (Figure 3.1)
- C = controlled release of developed hydrograph at preurbanization peak flow, cfs
- D = uncontrolled developed hydrograph, cfs
- C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 = channel length for reach I (Figure 3.1), ft
- L_2 = channel length for reach II (Figure 3.1), ft
- "N" = Manning's roughness coefficient
- Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 = channel slope for reach I (Figure 3.1), ft/ft
- S_2 = channel slope for reach II (Figure 3.1), ft/ft
- UC = undeveloped conditions in all sub-basins
- V R = runoff volume reduction

Figure 4.5 Effectiveness as a Function of Runoff Volume Reduction

Figure 4.5c. Volume Reduction Case III



LEGEND

- A_1 = upstream sub-basin area, acres (Figure 3.1)
- A_2 = middle sub-basin area, acres (Figure 3.1)
- A_3 = downstream sub-basin area, acres (Figure 3.1)
- C = controlled release of developed hydrograph at preurbanization peak flow, cfs
- D = uncontrolled developed hydrograph, cfs
- C-D-D = A_1 controlled developed; A_2 uncontrolled developed; A_3 uncontrolled developed
- L_1 = channel length for reach I (Figure 3.1), ft
- L_2 = channel length for reach II (Figure 3.1), ft
- "N" = Manning's roughness coefficient
- Qux = peak flow at X (Figure 3.1) under undeveloped conditions (Control case), cfs
- Qx = routed peak flow at X (Figure 3.1) under various land conditions, cfs
- $\frac{Qx}{Qux}$ = ratio of peak watershed outflow to control case (undeveloped conditions)
- S_1 = channel slope for reach I (Figure 3.1), ft/ft
- S_2 = channel slope for reach II (Figure 3.1), ft/ft
- UC = undeveloped conditions in all sub-basins
- V R = runoff volume reduction

Figure 4.5 Effectiveness as a Function of Runoff Volume Reduction (con't) discharges at or above the natural level was significant, usually longer than the rainfall duration.

The area wide (blanket) control of flow from all sub-basins strategy was usually more effective than mixed contol. In contrast, Figures 4.1d, 4.4b, 4.4c, 4.4f, 4.5b, and 4.5c illustrate cases of mixed control that yield lower peak flow rates than does blanket control with an acompanying reduction in control costs. In all test cases where mixed control was more effective than blanket control, the control option was C-C-D, i.e. the lower subwatershed was uncontrolled.

Another instance that might dictate mixed control in lieu of a blanket policy is the case of a partially developed watershed without existing controls. If existing developed sub-basins cannot be feasibly controlled, a mixed control strategy would be required.

4.7 Effectiveness of Mixed Control Policies

Mixed control as indicated in Sec. 4.6 can be more efficient and effective than blanket control in some cases when considering the transition from undeveloped to urbanized watersheds. In watersheds already containing developed unregulated sub-basins that cannot be feasibly controlled, mixed control can be very effective. However, this does not mean adopting a blanket control policy for the remaining undeveloped sub-basins.

For example, consider Figure 4.4b and for discussion purposes assume the upper sub-basin is developed without any feasible means of control. It is immediately apparent that controlling the middle sub-basin (A_2) while leaving the lower one (A_3) uncontrolled yields the best results (1.38) when the remainder of the watershed (A_1) is developed. A blanket policy requiring both lower watersheds to provide runoff control would produce a peak runoff rate of 1.66, an increase of 20% above the mixed strategy. These results agree qualitatively with the findings of Lumb, et al. (1974).

Furthermore, consider Figure 4.4c and assume that the upper two sub-basins (A_1 and A_2) are developed without control placed on runoff from them. Requiring control on the lower sub-basin (A_3) would result in an increase of 13% in the peak flow rate which would have existed if A_3 had been left uncontrolled. Figure 4.4a however, which reflects a different areal distribution, would yield a decrease of 16% in the peak flow rate if subjected to the same conditions as Figure 4.4c.

4.8 Conclusions

1. Urban runoff control strategies must be evaluated at the entire watershed level, not on local jurisdictional boundaries. Blanket control may be appropriate in one particular drainage basin while a mixed control is best in another. Watersheds that span two different local jurisdictions and strategies may experience a compounding of their storm drainage problems.

2. Without detailed sub-basin discharge hydrographs and main channel characteristics, the optimum drainage management scheme cannot be established.

3. Retention of flow volumes in sub-basins alone will not maintain the main channel in its pre-urbanization state due to the increase in peak flow rates and to substantially longer durations at these elevated levels.

4. Runoff volume reduction appears to be one means of achieving more effective runoff control but would need to be a significant portion of the incremental volume increase resulting from urbanization. Several useful ways for reducing runoff volumes are infiltration beds and grass-lined ditches where practicable in lieu of the traditional curb and gutter storm drainage system. Other approaches are discussed briefly in Chapter 2 and in more detail in Lumb, et al., (1974).

5. Restricting the outflow from a retention facility to a level less than the undeveloped rate could achieve a composite peak flow rate that would equal the pre-urbanization flow but would run for a much greater duration at that rate. The increased flow duration would have potentially undesirable effects on the channel system.

6. Longer duration storms falling on the sub-basins under consideraiton produced higher peaked watershed outflow hydrographs than did shorter, more intense rainfalls. This emphasizes the need to know the actual shape of the hydrograph. Runoff volume, as well as peak flow rate, is crucial to determining the most effective storm water management scheme.

7. For storm magnitudes greater than the control system design storm two possibilities exist: a higher peak rate or an equal peak rate to that which would have occurred without control. The shape of the subwatershed input hydrographs into the control facilities and the sequential linking together of the sub-basins are both critical factors. No general conclusion was evident from this research.

8. If area A_3 (Figure 3.1) is less than 33% of the total watershed and runoff from the remainder of the watershed is either totally controlled or uncontrolled, A_3 in most instances should not be controlled. Volume reduction or channel modifications to alleviate flooding within the sub-basin channel network would be appropriate if the time base of the sub-basin discharge hydrograph is not extended.

9. While the effects of control strategies have been examined no information concerning the economics of controls is given. The whole concept of sub-basin runoff control can be potentially dangerous. For example, an ordinance that controls runoff from five year recurrence interval rainfall events might cause flood peaks for say ten year (and longer) storms to exceed the effect that would be experienced in the absence of controls. The design

storm approach, while useful for examining event combinations here, is not necessarily a satisfactory way to approach the problem of mitigating the flood producing effects of urbanization. Some design considerations are given in Chapter 5.

5. URBAN RUNOFF CONTROL--DESIGN CONSIDERATIONS

5.1 Introduction

The previous chapter discussed the effectiveness of different runoff control strategies for hypothetical watersheds. This chapter outlines the procedures and information required to determine the most effective storm water management plan for a specific drainage basin.

5.2 Management Options Available

Economic, physical, and political factors will all restrict the storm water management options open to a given community or its representative control agency. For discussion purposes, assume that the control agency for a drainage basin has the authority and capability of implementing a sub-basin control scheme for the total watershed.

5.3 Survey of Drainage Basin

The drainage basin in question must have its boundary delineated and its area divided into major sub-basins. Each of these sub-basins could potentially accommodate one control facility if one was required. This is done by using a topographic map of the watershed and delineating the drainage divides. Also on this map, existing developments, control facilities, and channel sections experiencing flooding problems can and should be located. An economical alternative would be to make direct use of aerial photographs and computer recordings of land features (Ragan, 1975).

5.4 Surface Runoff Hydrographs

The most difficult part of any urban runoff management design work involves determining the "land surface runoff" hydrographs. We take the term "land surface runoff" to mean the contribution of flow to the tributary streams from the land drained by the streams. The term does not necessarily imply Hortonian runoff. Generation of land surface runoff is important because changes to the land surface hydrologic response can be directly considered. Methods that generate hydrographs from watersheds (including land surface and channel effects) are entirely unsuitable for developing hydrographs for studying the effects of urbanization and various runoff control strategies.

Continuous Simulation Approach

The required analysis dictates use of some relatively sophisticated precipitation--runoff model that models land and channel effects separately. The most difficult design detail that remains is to determine what precipitation event (or events) to use with the model. Different approaches might be followed depending on the design situation. For example, if an area is entirely developed and flooding impacts in the area are to be mitigated perhaps the most satisfactory approach would be to use a long (40-60 years) precipitation base to generate hydrographs via continuous simulation, e.g. Crawford and Linsley (1966), James and Lumb (1975). The land surface runoff files generated for any desired number of sub-basins by this approach would then be routed through a desired "control strategy algorithm" to yield sub-basin input hydrographs to the channel system. These continuous hydrographs (or preferably hydrographs above a given flow level) which cover the entire precipitation record would be routed through the channel network to yield hydrograph information at any desired number of locations in the channel system which would permit flood frequency (or other physical analyses) and economic analysis to be performed. In this way the most satisfactory physical and economic strategies could be determined. The approach has some severe limitations, however. Recorded streamflows at one or more locations in the channel system are needed to calibrate the models. Additionally the precipitation recording station must very closely represent actual precipitation in the watershed of interest.

If temporal and spatial variability of precipitation within the watershed is significant use of a single gauge would be insufficient for modeling purposes. Practical limitations on data availability may restrict the utility of this approach.

A promising approach is via the use of flow files which correspond to various types of land surface. Lumb (1975) and James and Lumb (1975) developed an approach where the Stanford Watershed Model (Crawford and Linsley, 1966) was used with long precipitation records to estimate continuous land surface runoff per unit area from several different land surfaces. When DeKalb County (Georgia) Commissioners want to determine impacts of certain types of land use in varying amounts and at different locations they need only scale the relevant runoff files, route the subarea flows through "management strategy algorithms" to obtain input sub-basin hydrographs to be routed through the main channel system. Again this approach provides flow frequency information for analysis purposes. The approach is probably conservative because it does not take into account spatial variations in precipitation. If it were modified to provide phased runoff to reflect storm pattern movement it would provide powerful design information. Representative precipitation data are of course needed. Similarly stream gauge records are needed to calibrate the basic precipitation--runoff model. Lumb selected the Stanford Model after evaluating five relatively sophisticated models (Lumb, 1976).

The use of flow files corresponding to various land features is most useful when examining areas where land use changes might occur or where substantial development has occurred and potential development needs to be investigated.

Design Storm(s) Approach

While the above two approaches best accommodate the total urban runoff design situation there are many situations where insufficient data exist for the approach to be workable. In such cases preliminary management strategies could be based on analysis that use design storm combinations.

Each sub-basin will generate a particular hydrograph when subjected to a given storm. A five to ten year recurrence interval design storm may be appropriate. On a regional basis, a benefit/cost analysis would be required to determine the most efficient regional design storm(s). The design storm(s) duration should be sufficient to generate the peak discharge from the watershed for that recurrence interval storm. This storm would probably not produce the worst case (highest peak flow) for the sub-basin discharge hydrographs. However, this rainfall would usually generate as much if not more runoff volume than a more intense storm that would produce higher sub-basin peak flows. (This runoff volume is a most important factor in designing retention facilities.)

Once the design storm has been selected, a surface runoff model can generate the sub-basin discharge hydrographs for any existing or predicted land use configuration. Choice of the model is important. A sophisticated precipitation runoff model having perhaps regional (local average) calibration parameters should be used. Refinements to accommodate dominant storm movement can be made by varying the time at which the design storm is applied to particular sub-basins. (In the work reported in earlier chapters we assumed that storms started at the same time over all sub-basins.)

The composite hydrograph at any channel location can then be determined by routing the upstream sub-basin hydrographs to this point. The model that generates the discharge sub-basin hydrographs must be capable of reflecting actual sub-basin slopes, soil types, ground cover, existing or proposed storm water collection systems, antecedent moisture conditions, and impervious areas. The time increments for the rainfall and generated discharge hydrograph should be in the order of 5 minutes to produce a sufficiently detailed hydrograph. In addition, a model that could simulate on-site or sub-basin control facilities and/or runoff volume reduction measures would be necessary.

The sub-basin discharge hydrographs can be simulated or adjusted manually to reflect various control options. These hydrographs would be stored for input to different control configurations. Different combinations of these adjusted hydrographs can then be routed through the channel system to determine which is the most effective combination for the land condition under consideration.

This approach allows the control agency to decouple a problem sub-basin while attempting to solve the entire drainage basin's problems. The agency still has the capability of evaluating the impacts of any control strategy utilized in the problem sub-basin on the total watershed.

5.5 Summary Observations

The importance of the quality of computer programs that generate the surface runoff hydrographs and route the hydrographs through the channel network cannot be over emphasized. Sizing of the control facilities as well as the overall management scheme requires detialed information on the surface runoff hydrographs. The surface runoff model must be sophisticated enough to adequately simulate the complex interactions within an urban or urbanizing watershed. The actual shapes of the discharge hydrograph must be known. This is not sufficient, however, unless the individual sub-basin hydrographs can be linked together into the total watershed drainage network. This requires a routing program that can closely reproduce the actual travel times experienced in the watershed channel network. Actual average channel slope, length, Manning's roughness coefficient, and cross-sectional shape all must be simulated. In most instances, the unsteady one-dimensional kinematic wave routing equations would be adequate. The actual channel reaches that the system is divided into would be dependent upon the channel characteristics of that stream. Some stream flow information will be needed to calibrate the routing model.

These types of computer simulation could probably be justified on sub-basins of 1 square mile or more. Most urban watersheds could be divided into sub-basins with a minimum area of approximately 1 square mile. James and Lumb (1975) have found it necessary and computationally feasible to use smaller areas than one square mile, however.

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APPENDIX A: COMPUTER MODEL FOR ROUTING PROGRAM

```
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C      *****        DEFINITION OF MAJOR VARIABLES               *****   .
C      BETA          = CONSTANT BASED ON CHANNEL SHAPE
C      C2            = COEFFICIENT IN NUMERICAL INTEGRATION             .
C      C3            = COEFFICIENT IN NUMERICAL INTEGRATION             .
C      DMAX          = DEVELOPED MAX. FLOW RATE, CFS                    .
C      DT            = TIME INCREMENT IN WAVE ROUTING PROBLEM           .
C      DTC           = DEVELOPED TIME OF CONCENTRATION, MIN             .
C      DX            = DISTANCE INCREMENT IN WAVE ROUTING PROBLEM,FT    .
C      E             = MANNINGS ROUGHNESS COEFFICIENT                   .
C      FU(22)        = SYSTEM EQUATIONS IN WAVE ROUTING PROBLEM         .
C      G             = ACCELERATION OF GRAVITY,FT/SEC**2                .
C      IT            = NUMBER OF TIME INTERVALS IN ROUTING SCHEME       .
C      N             = NUMBER OF INTERNAL CHANNEL REACHES               .
C      NJ            = DIMENSION OF KINEMATIC ROUTING SCHEME            .
C      NJUNCT        = NUMBER OF CHANNEL JUNCTIONS IN CHANNEL NETWORK   .
C      O             = INFLOW DISCHARGE,CFS                             .
C      OB            = BASE FLOW                                        .
C      OC(100)       = CONTROLLED DEVELOPED HYDROGRAPH, CFS             .
C      OD(100)       = DEVELOPED HYDROGRAPH,CFS                         .
C      OI(100)       = INPUT HYDROGRAPH AT UPSTREAM END OF REACH, CFS   .
C      OO(100)       = INPUT HYDROGRAPH AT CHANNEL JUNCTION, CFS
C      OR(100)       = ROUTED HYDROGRAPH AT DOWNSTREAM END OF REACH, CFS.
C      OU(100)       = UNDEVELOPED HYDROGRAPH, CFS                      .
C      SO            = CHANNEL BED SLOPE,FT/FT                          .
C      THETA         = CONSTANT BASED ON CHANNEL SHAPE                  .
C      TOLV          = ITERATION TOLERANCE WITH RESP. TO WAVE VEL,FPS   .
C      TOLY          = ITERATION TOLERANCE WITH RESP. TO WAVE DEPTH,FT  .
C      UMAX          = UNDEVELOPED MAX. FLOW RATE, CFS                  .
C      UTC           = UNDEVELOPED TIME OF CONCENTRATION, MIN           .
C      V(11,2)       = KINEMATIC WAVE VELOCITY,FT/SEC                   .
C      Y(11,2)       = KINEMATIC WAVE DEPTH,FT                          .
C      YI(11)        = ITERATED VALUES OF KINEMATIC WAVE DEPTH,FT       .
C      W             = WIDTH OF CHANNEL BOTTOM,FT                       .
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C      *****        SUMMARY OF THE PROGRAM STRUCTURE      *****         .
C                         *MAIN                                         .
C                         *    CALLS                                    .
C                         *          *DREAD                             .
C                         *          *JUNCT                             .
C                         *          *GRAPH                             .
C                         *          *KINEMA                            .
C                         *          *    CALLS                         .
C                         *          *          *DNORM                  .
C                         *          *          *IKINEM                 .
C                         *          *          *    CALLS              .
C                         *          *          *          *DNORM       .
C                         *          *          *          *INVR        .
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
```

```
      PROGRAM MAIN (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     MAIN PROGRAM WHICH CONTROLS THE NUMBER OF CHANNEL JUNCTIONS   .
C     AND CALLS THE APPROPRIATE SUBROUTINES                         .
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C                                                                   .
      COMMON/WAVE/QI(100),QR(100),QO(100),QB,Q,QC(100),QD(100),QU(100),
     1FU(22), THETA,YI(11),V(11,2),G,SO,W,VS,E,DX,DT,N,NJ, Y(11,2),
     2IT,C2,C3,TOLV,TOLY,BETA,DF
      REAL LA
      G=32.17
      READ(5,103) NJUNCT
      CALL DREAD(W,SO,E,DELX,DELTA,N,IT,IH,QD,QU,QC,RA,LA,DF)
      TOLY=.05
      TOLV=.10
      C3=.5/DT
      C2=1./DX
      DX=DELX/N
      DT=DELTA
      NJ=N+1
      BETA=1./SIN(RA)+1./SIN(LA)
      THETA=.5*(1./TAN(RA)+1./TAN(LA))
      IF(IH.EQ.1) GO TO 17
      IF(IH.EQ.4) GO TO 20
      DO 16 I=1,IT
   16 QI(I)=QD(I)
      GO TO 19
   17 DO 18 I=1,IT
   18 QI(I)=QC(I)
      GO TO 19
   20 DO 21 I=1,IT
   21 QI(I)=QU(I)
   19 DO 59 III = 1,NJUNCT
      QB=QI(1)
      CALL KINEMA
      CALL DREAD(W,SO,E,DELX,DELTA,N,IT,IH,QD,QU,QC,RA,LA,DF)
      TOLY=.05
      TOLV=.10
      C3=.5/DT
      C2=1./DX
```

```
      DX=DELX/N
      DT=DELTA
      NJ=N+1
      BETA=1./SIN(RA)+1./SIN(LA)
      THETA=.5*(1./TAN(RA)+1./TAN(LA))
      IF(IH.EQ.1) GO TO 4
      IF(IH.EQ.4) GO TO 22
      DO 2  I=1,IT
    2 QO(I)=QD(I)
      GO TO 6
    4 DO 5 I=1,IT
    5 QO(I)=QC(I)
      GO TO 6
   22 DO 23 I=1,IT
   23 QO(I)=QU(I)
    6 CALL JUNCT(IT,QR,QO,QI)
   59 CONTINUE
      WRITE(6,102)
      DO 3 I=1,IT
      J=DT/60*I
      WRITE(6,101) J,QO(I),QR(I)
    3 CONTINUE
  101 FORMAT(8X,I4,8X,F9.1,18X,F9.1)
      CALL GRAPH(DELX)
  102 FORMAT(*1*,15X,*HYDROGRAPH SUMMARY*,//,
     1          18X,*INFLOW HYDROGRAPH*,1CX,*ROUTED HYDROGRAPH*,/,
     28X,*TIME*,/,
     38X,*MIN*,  14X,*CFS*,24X,*CFS*,/)
  103 FORMAT(I5)
      STOP
      END
```

```
      SUBROUTINE DREAD(W,SO,E,DELX,DELTA,N,IT,IH,QD,QU,QC,RA,LA,DF)
C
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     CALCULATES THE SUB-BASIN INPUT HYDROGRAPHS BASED ON THE SOIL
C     CONSERVATION SERVICE METHOD
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C
      DIMENSION QD(100),QU(100),QC(100)
      REAL LA
    1 FORMAT(5F12.4,2I4)
    8 FORMAT(I4,5F10.0,2F8.4)
      READ(5,1) W,SO,E,DELX,DELTA,N,IT
      READ(5,8) IH,UMAX,UTC,DMAX,DTC,DF,RA,LA
      OUTPUT,W,DELX,DELTA
      OUTPUT,IH,UMAX,UTC,DMAX,DTC
      OUTPUT,N,SO,E
      IF(IH.EQ.1) GO TO 16
      IF(IH.EQ.2) GO TO 16
      DO 4 I=1,IT
      IF(I.GT. 2) GO TO 2
    3 QU(I)=2.
      GO TO 4
    2 IF(I.GT.(2.*UTC*60./DELTA+2.)) GO TO 3
      T=(I-2)*DELTA/60.
      QU(I)=UMAX/2.+1. - (UMAX/2.-1.)*COS(3.14159/UTC*T)
    4 CONTINUE
      GO TO 17
   16 DO 7 I=1,IT
      IF(I.GT.2)GO TO 5
    6 QD(I)=2.0
      GO TO 7
    5 IF(I.GT.(2.*DTC*60./DELTA+2.)) GO TO 6
      T=(I-2)*DELTA/60.
      QD(I)=DMAX/2.+1. - (DMAX/2.-1.)*COS(3.14159/DTC*T)
    7 CONTINUE
      IF(IH.EQ.2) GO TO 17
      DO 10 I=1,IT
      IF(QD(I).GT. UMAX) GO TO 11
   10 QC(I)=QD(I)
   11 NUM=I
      STOR = 0.0
      DO 12 I=NUM,IT
      IF(QD(I).LT. UMAX) GO TO 13
      STOR=STOR+(QD(I)+QD(I-1)-2.*UMAX)/2.0
   12 CONTINUE
   13 MUM=I
      IUMAX=STOR/UMAX
      ITOTAL=MUM-NUM+IUMAX
      DO 14 I=NUM,ITOTAL
   14 QC(I)=UMAX
      J=ITOTAL + 1
      K=0
      DO 15 I=J,IT
      K=K+1
   15 QC(I)=QD(MUM-1+K)
   17 RETURN  $  END
```

```
      SUBROUTINE KINEMA
C                                                                       .
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     CALCULATES INTEGPATION CONSTANTS AND CONTROLS WAVE ROUTING BY     .
C     KINEMATIC EQUATION SET                                            .
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C                                                                       .
      COMMON/WAVE/QI(100),QR(100),QO(100),QB,Q,QC(100),QD(100),QU(100),
     1FU(22), THETA,YI(11),V(11,2),G,SO,W,VS,E,DX,DT,N,NJ, Y(11,2),
     2IT,C2,C3,TOLV,TOLY,BETA,DF
      DIMENSION A(11,11)
      NG=NJ
C-----INITIAL CONDITIONS
      CALL DNORM(QB,SO,G,BETA,THETA,DF,E,W,YI,VI)
      DO 1 J=1,2
      DO 1 I=1,NJ
      V(I,J)=VI
      Y(I,J)=YI
    1 CONTINUE
      QR(1)=QB
C-----LOOP ON INLET HYDROGRAPH
      IC=1
C-----UPSTPEAM BOUNDARY CONDITIONS
    2 Q=QI(IC)
      IF(IC.GT.IT) GO TO 5
      IF(Q.GT.QB) GO TO 3
      Q=QB $ QI(IC)=QB
C-----ITERATE ONCONTINUITY EQUATION
    3 CALL IKINEM(A,NG,ITER)
      XY=Y(NJ,2)-W/DF
      IF(XY.LT.0.) XY=0.
      QR(IC)=V(NJ,2)*(Y(NJ,2)*W+XY**2*THETA)
      IF(ITER .EQ. 20) GO TO 5
      DO 4 I=1,NJ
      V(I,1)=V(I,2)
      Y(I,1)=Y(I,2)
    4 CONTINUE
      IF(IC.GT.40) GO TO 6
    8 IC=IC+1  $  GO TO 2
    6 IF(ABS(QR(IC)-QR(1)).LT..1) GO TO 7
      GO TO 8
    7 DO 9 I=IC,IT
    9 QR(I)=QR(1)
    5 OUTPUT,QR(1)
      RETURN  $  END
```

```
      SUBROUTINE IKINEM(A,NG,ITER)
C                                                                      .
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     NUMEPICAL INTEGRATION OF KINEMATIC WAVE EQUATIONS WHERE IMPLICIT .
C     SOLUTION IS FOUND BY N-DIMENSIONAL NEWTON-RAPHSON ITERATION      .
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C                                                                      .
      COMMON/WAVE/OI(100),OR(100),OO(100),OB,O,OC(100),OD(100),QU(100),
     1FU(22), THETA,YI(11),V(11,2),G,SO,W,VS,E,DX,DT,N,NJ, Y(11,2),
     2IT,C2,C3,TOLV,TOLY,BETA,DF
      DIMENSION A(NG,NG)
      IMAX=20
      ITER=1
      CALL DNORM(Q,SO,G,BETA,THETA,DF,E,W,YN,V(1,2))
    1 DO 2 J=1,NJ
      FU(J)=0.
      DO 2 K=1,NJ
    2 A(J,K)=0.
C-----SYSTEM EQUATIONS
      IF(Y(1,2).LT.0.) Y(1,2)=YN
      XY=Y(1,2)-W/DF
      IF(XY.LT.0.) XY=0.
      FU(1)=O-V(1,2)*(Y(1,2)*W+XY**2*THETA)
      DO 3 I=2,NJ
      J=I-1
      XY1=Y(J,2)-W/DF
      IF(XY1.LT.0.) XY1=0.
      XY2=Y(I,2)-W/DF
      IF(XY2.LT.0.) XY2=0.
      AREA=Y(I,2)*W+XY2**2*THETA
      WP=2.*Y(I,2)+W-2.*XY2+XY2*BETA
      R=AREA/WP
      V(I,2)=1.49*R**.67*SO**.5/E
      D=.5*(W*(Y(J,2)+Y(I,2))+THETA*(XY1**2+XY2**2))
      B=W+THETA*(XY1+XY2)
      DVDX=C2*(V(I,2)-V(J,2))
      VEL=.5*(V(I,2)+V(J,2))
      DYDX=C2*(Y(I,2)-Y(J,2))
      DYDT=C3*(Y(I,2)+Y(J,2)-Y(I,1)-Y(J,1))
      FU(I)=D/B*DVDX+VEL*DYDX+DYDT
    3 CONTINUE
C-----JACOBIAN MATRIX ENTRIES
      A(1,1)=-V(1,2)*(W+2.*XY*THETA)
      DO 4 K=2,NJ
      J=K-1
      XY1=Y(J,2)-W/DF
      IF(XY1.LT.0.) XY1=0.
      XY2=Y(K,2)-W/DF
      IF(XY2.LT.0.) XY2=0.
      D=.5*(W*(Y(J,2)+Y(K,2))+THETA*(XY1**2+XY2**2))
      B=W+THETA*(XY1+XY2)
      DVDX=C2*(V(K,2)-V(J,2))
      DYDX=C2*(Y(K,2)-Y(J,2))
      DYDT=C3*(Y(K,2)+Y(J,2)-Y(K,1)-Y(J,1))
      VEL=.5*(V(K,2)+V(J,2))
```

```
      AREA=Y(J,2)*W+XY1**2*THETA
      WP=2.*Y(J,2)+W-2.*XY1+XY1*BETA
      R=AREA/WP
      DWDY=2.0
      IF(XY1.GT.0.) DWDY=BETA
      DVDY=1.00/E*SO**.5/R**.33*((W+2.*XY1*THETA)*WP-DWDY*AREA)/WP**2
      DBDYY=0.0
      IF(XY1.GT.0.) DBDYY=THETA
      A(K,K-1)=DVDX*((.5*W+XY1*THETA)*B-DBDYY*D)/B**2-D/B*C2*DVDY-VEL*
     1C2+.5*DYDX*DVDY+C3
      AREA=Y(K,2)*W+XY2**2*THETA
      WP=2.*Y(K,2)+W-2.*XY2+XY2*BETA
      R=AREA/WP
      DWDY=2.0
      IF(XY2.GT.0.) DWDY=BETA
      DVDY=1.00/E*SO**.5/R**.33*((W+2.*XY2*THETA)*WP-DWDY*AREA)/WP**2
      DBDYY=0.0
      IF(XY2.GT.0.) DBDYY=THETA
      A(K,K  )=DVDX*((.5*W+XY2*THETA)*B-DBDYY*D)/B**2+D/B*C2*DVDY+VEL*
     1C2+.5*DYDX*DVDY+C3
    4 CONTINUE
      CALL INVR (A,NJ,0.,0.,DETERM,NJ,NJ)
C-----ITERATED VALUES
      DO 6 I=1,NJ
      DUMMY=0.
      DO 5 K=1,NJ
      DUMMY=DUMMY+A(I,K)*FU(K)
    5 CONTINUE
      YI(I)=Y(I,2)-DUMMY
    6 CONTINUE
C-----CONVERGENCE CHECK
      DO 7 I=1,NJ
      IF(ABS(YI(I)-Y(I,2)).GT.TOLY) GO TO 9
    7 CONTINUE
      DO 8 I=1,NJ
    8 Y(I,2)=YI(I)
      GO TO 11
    9 IF(ITER.EQ.IMAX) GO TO 11
      DO 10 I=1,NJ
      Y(I,2)=YI(I)
   10 CONTINUE
      ITER=ITER+1
      GO TO 1
   11 RETURN $ END
```

```
      SUBROUTINE INVR(A,N,B,M,DETERM,ISIZE,JSIZE)
C
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C      MATRIX INVERSION BY JORDANS ELIMINATION METHOD
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C
      DIMENSION IPIVOT(100),A(ISIZE,JSIZE),B(ISIZE,M),INDEX(100,2),
     1PIVOT(100)
      EQUIVALENCE (IROW,JROW),(ICOLUM,JCOLUM),(AMAX,T,SWAP)
C
   10 DETERM=1.0
   15 DO 20 J=1,N
   20 IPIVOT(J)=0
   30 DO 550 I=1,N
C
C      SEARCH FOR PIVOT ELEMENT
C
   40 AMAX=0.0
      ICOLUM=0
   45 DO 105 J=1,N
   50 IF (IPIVOT(J)-1) 60, 105, 60
   60 DO 100 K=1,N
   70 IF (IPIVOT(K)-1) 80, 100, 740
   80 IF(ABS(AMAX)-ABS(A(J,K)))85,100,100
   85 IROW=J
   90 ICOLUM=K
   95 AMAX=A(J,K)
  100 CONTINUE
  105 CONTINUE
  110 IPIVOT(ICOLUM)=IPIVOT(ICOLUM)+1
C
C      INTERCHANGE ROWS TO PUT PIVOT ELEMENT ON DIAGONAL
C
  130 IF (IROW-ICOLUM) 140, 260, 140
  140 DETERM=-DETERM
  150 DO 200 L=1,N
  160 SWAP=A(IROW,L)
  170 A(IROW,L)=A(ICOLUM,L)
  200 A(ICOLUM,L)=SWAP
  205 IF(M) 260, 260, 210
  210 DO 250 L=1, M
  220 SWAP=B(IROW,L)
  230 B(IROW,L)=B(ICOLUM,L)
  250 B(ICOLUM,L)=SWAP
  260 INDEX(I,1)=IROW
  270 INDEX(I,2)=ICOLUM
  310 PIVOT(I)=A(ICOLUM,ICOLUM)
  320 DETERM=DETERM*PIVOT(I)
C
C      DIVIDE PIVOT ROW BY PIVOT ELEMENT
C
  330 A(ICOLUM,ICOLUM)=1.0
  340 DO 350 L=1,N
  350 A(ICOLUM,L)=A(ICOLUM,L)/PIVOT(I)
  355 IF(M) 380, 380, 360
```

```
  360 DO 370 L=1,M
      WRITE(6,987) B(ICOLUM,L),PIVOT(I)
  987 FORMAT(*1*,2F15.5)
  370 B(ICOLUM,L)=B(ICOLUM,L)/PIVOT(I)
C
C     REDUCE NON-PIVOT ROWS
C
  380 DO 550 L1=1,N
  390 IF(L1-ICOLUM) 400, 550, 400
  400 T=A(L1,ICOLUM)
  420 A(L1,ICOLUM)=0.0
  430 DO 450 L=1,N
      A(L1,L)=A(L1,L)-A(ICOLUM,L)*T
  450 CONTINUE
  455 IF(M) 550, 550, 460
  460 DO 500 L=1,M
  500 B(L1,L)=B(L1,L)-B(ICOLUM,L)*T
  550 CONTINUE
C
C     INTERCHANGE COLUMNS
C
  600 DO 710 I=1,N
  610 L=N+1-I
  620 IF (INDEX(L,1)-INDEX(L,2)) 630, 710, 630
  630 JROW=INDEX(L,1)
  640 JCOLUM=INDEX(L,2)
  650 DO 705 K=1,N
  660 SWAP=A(K,JROW)
  670 A(K,JROW)=A(K,JCOLUM)
  700 A(K,JCOLUM)=SWAP
  705 CONTINUE
  710 CONTINUE
  740 RETURN
      END
```

```
      SUBROUTINE DNORM(Q,S,G,BETA,THETA,DF,E,W,YN,VN)
C
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     NORMAL DEPTH AND VELOCITY FOUND BY NEWTON-RAPHSON ITERATION ON
C     MANNINGS EQUATION
C. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C
      XY=0.
      TOL=.01
      IMAX=25
      Y1=.5
      I=1
      QT=W**2.67*S**.5/E*1.49/(2.+DF)**.67/DF
    1 IF(Q.GT.QT) GO TO 15
      R=(W*Y1+XY**2*THETA)/(2.*Y1+W-2.*XY+XY*BETA)
      V=1.49*R**.67*S**.5/E
      Y2=Q/(V*W)
      GO TO 10
   15 A=THETA
      B=W*(1.-.4*THETA)
      D=.04*W**2*THETA
      XY=Y1-W/DF
      IF(XY.LT.0.) XY=0.
      R=(W*Y1+XY**2*THETA)/(2.*Y1+W-2.*XY+XY*BETA)
      V=1.49*R**.67*S**.5/E
      C=D-Q/V
      Y2=(-B+SQRT((ABS(B))**2-4.*A*C))/(2.*A)
   10 IF(ABS(Y1-Y2)-TOL)6,6,5
    5 IF(I.GT.IMAX) GO TO 7
      I=I+1 $ Y1=Y2 $ GO TO 1
    6 YN=Y2
      GO TO 8
    7 OUTPUT,CONV
    8 VN=Q/(YN*W+XY**2*THETA)
      RETURN $ END
```

```
      SUBROUTINE JUNCT(IT,QR,QO,QI)
C
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     CALCULATES COMBINED FLOW AT THE JUNCTION
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C
      DIMENSION QR(100),QO(100),QI(100)
      DO 1 I=1,IT
      QI(I)=QR(I)+QO(I)
    1 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE GRAPH(DIST)
C
C . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
C     PLOTS ROUTED COMPOSITE WATERSHED HYDROGRAPH AND ONE INPUT          .
C     SUB-BASIN HYDROGRAPH                                               .
C . . . . . . . . . . . . . . . . . . . . . .   . . . . . . . . . . . . .
C
      COMMON/WAVE/QI(100),QR(100),QO(100),QB, , QC(100),QD(100),QU(100),
     1FU(22), THETA,YI(11),V(11,2),G,SO,W,VS,E,DX,DT,N,NJ, Y(11,2),
     2IT,C2,C3,TOLV,TOLY,BETA,DF
      DIMENSION NSCALE(5),X(100),IMAGE(1500)
  104 FORMAT(*1*)
      WRITE(6,104)
      MAXY=0.
      DO 99 I=1,IT
      IF(QI(I).GT.MAXY) MAXY=QI(I)
   99 CONTINUE
      KOUNT=(MAXY-QB)/4
      NHL=(MAXY-QB)/KOUNT+1
      MAXY=(MAXY/10+1)*10
      YMAX=MAXY
      T=DT
      MAXX=IT*DT/60
      NOUNT=MAXX/10
      XMAX=IT*DT/60 $ NVL=MAXX/NOUNT
      DATA(NSCALE(I),I=1,5)/1,0,0,0,0/
      XMIN=0.
      MINY=QB/10 $ YMIN=MINY*10.
      DO 102 I=1,IT
  102 X    (I)=I*DT/60.
      MAXX=MAXY/(DT/60.)
      CALL PLOT1(NSCALE,NHL,10,NVL,8)
      CALL PLOT2(IMAGE,XMAX,XMIN,YMAX,YMIN)
      CALL PLOT3(1H*,X    ,QO,MAXX)
      CALL PLOT3(1H+,X    ,QR,MAXX)
      CALL PLOT4(15,15HDISCHARGE/CFS     )
      WRITE(6,103)  SO,T,E,DX,DIST,W,N
  103 FORMAT(45X,*TIME,MINUTES*,//
     1,5X,2H*-,*INFLOW HYDROGRAPH*,7X,*CHANNEL DATA*,10X,*ROUTING DATA*,
     2/,5X,2H+-,*ROUTED HYDROGRAPH*,7X,*SLOPE            =*,F5.4,2X,*TIME S
     3TEP   =*,F5.0,/,31X,*WALL ROUGHNESS=*
     4,F5.4, 1X,* LENGTH STEP=*,F5.0,/,31X,*CHANNEL LENGTH=*,F5./,
     531X,*CHANNEL WIDTH    =*,F3.0,2X,*N           =*,I5)
      RETURN $ END
```

APPENDIX B: SCS METHOD FOR DETERMINING PEAK DISCHARGE RATES AND TIME TO PEAK

The Soil Conservation Service (SCS,1975) determines the time to peak by the following equation:

$$TP = D/2 + L$$

where TP = time from beginning of rainfall to peak discharge, min

D = duration of uniform rainfall, min

L = lag time from centroid of rainfall to peak discharge, min

Figure B.1 illustrates TP, D, and L.

In addition the SCS has a figure (Figure B.2) that relates time of concentration (TC) to peak runoff rate. In order to use Figure B.2 TC must be known. Also TC and TP must be related in some manner so that the sinusoidal sub-basin hydrograph can be defined. The sinusoidal hydrograph is defined by TP and peak discharge rate. TP was set equal to TC for simplicity as follows:

$$TC = 1.67\ L$$

$$\therefore TP = D/2 + .6TC$$

$$\text{Let } TP = TC$$

$$\therefore TC = D/2 + .6TC$$

$$\therefore D = .8TC = .8TP$$

Therefore to generate the peak runoff rate from each sub-basin the rainfall duration was assumed to be .8TC = .8TP.

The time of concentration was calculated by using Figure B.3 (SCS,1975). This figure determines a basin lag time (L) by taking into consideration flow length, basin slope, and curve number. L can be adjusted (Figure B.4) to reflect impervious area in the sub-basin. Once L is known TC = 1.67 L. The next step is to use Figure B.2 to determine the peak discharge rate.

Example of SCS Method:

Criteria: Drainage area = 500 acres
Undeveloped (UND) CN = 75
Developed (D) CN = 80
Basin slope = 4%
Flow length = 8700 ft (based on length=209*area$^{0.6}$)
Impervious area = 30%
Rainfall = 1.8 inch. in 24 hr

A) Calculate TC (UND) and TC (D):

From Figure B.3 L(UND) = 1.05 hrs

TC(UND) = (1.05)*(1.67) = 1.75 hr = 105 min

From Figure B.3 L(D) = .80 hrs

Adjusted L(D) due to impervious area (Figure B.4) equals (.80)*(.82) = .66 hrs

TC(D) = (.66)*(1.67) = 1.1 hr = 66 min

B) Calculate Q_{peak}(UND) and Q_{peak}(D)

From Table B.1 the amount of runoff for 1.8 inches of rainfall is 0.29 (CN=75) and 0.44 (CN=80).

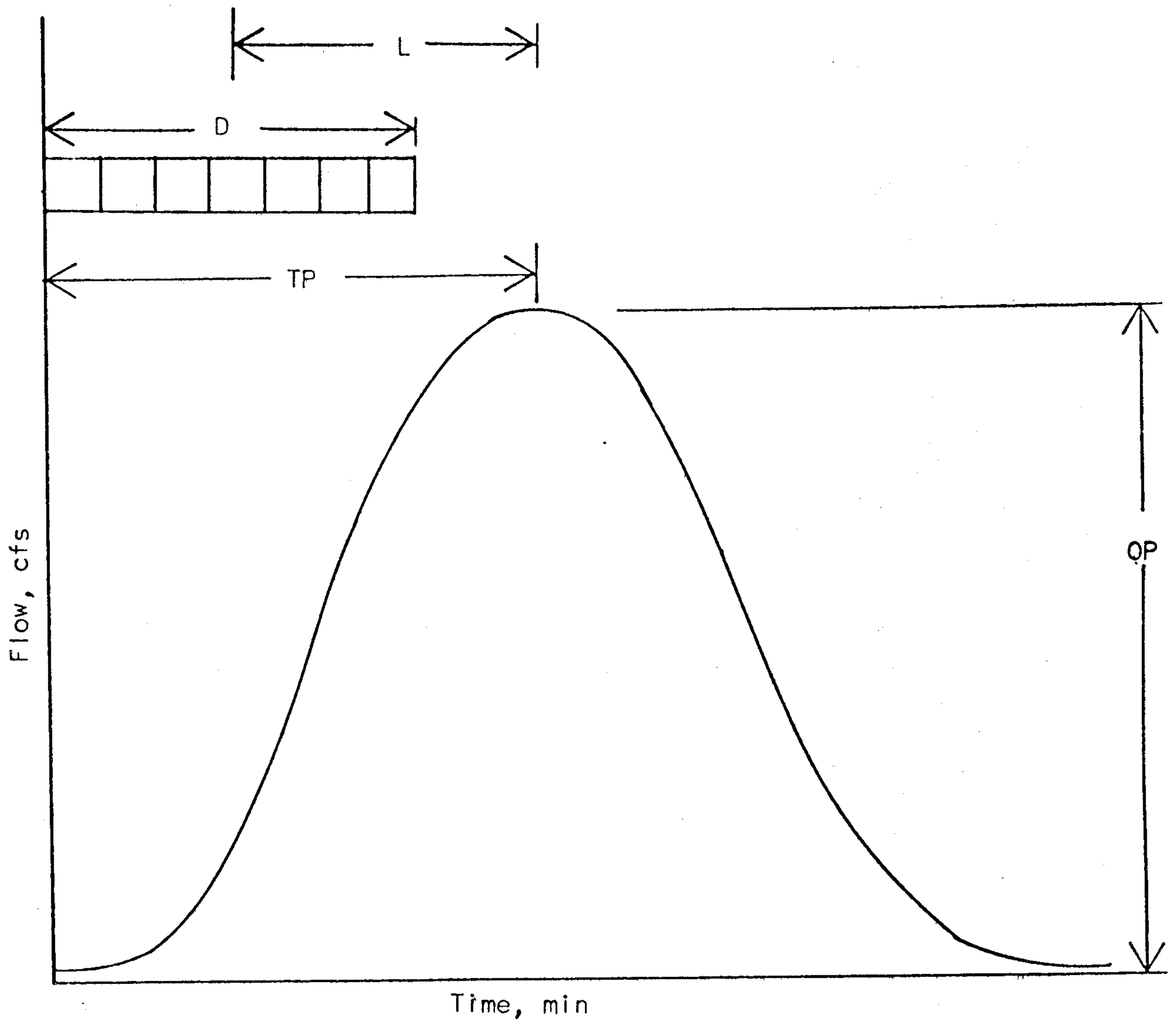
Q_{peak}(UND) = (peak discharge from Figure B.2 for TC=1.75 hr)* (area/640)*(runoff)

= (215)*(500/640)*(.29) = 49 cfs

Q_{peak}(D) = (peak discharge from Figure B.2 for TC=1.1 hr)* (area/640)*(runoff)

= (300)*(500/640)*(.44) = 103 cfs

These times and flow rates are tabulated in Table 3.1.



LEGEND

TP = Time to peak, min

QP = Peak runoff rate, cfs

D = Duration of uniform rainfall, min

L = Lag time from centroid of rainfall to peak discharge, min

Figure B.1 Sinusoidal Sub-basin Hydrograph

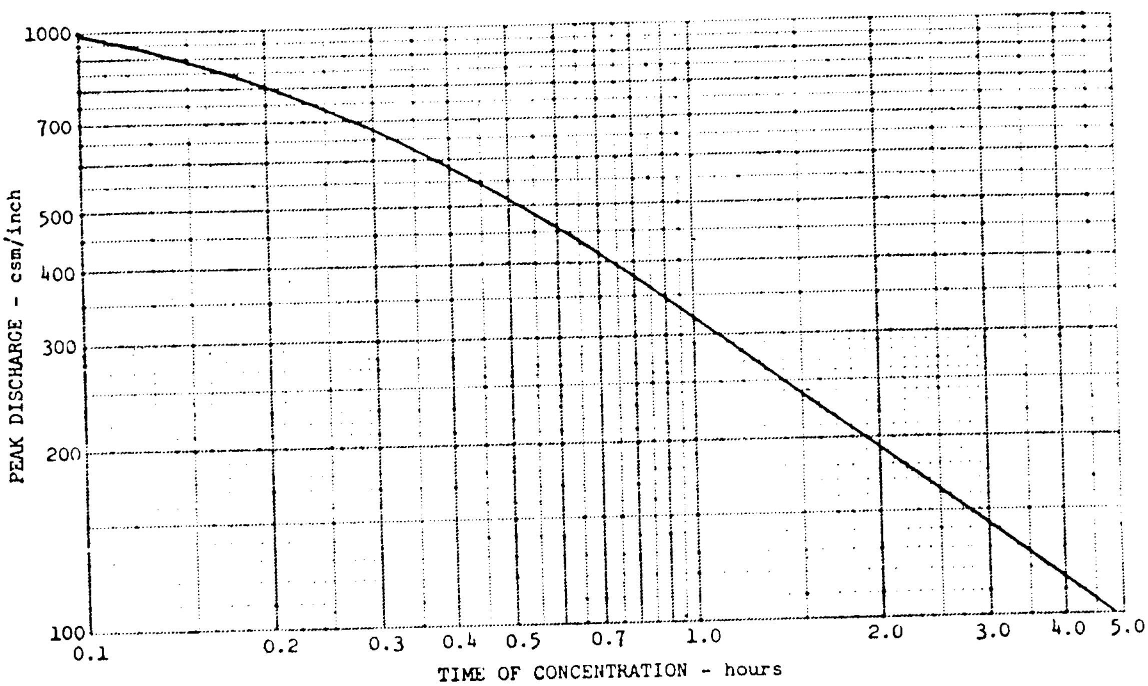


Figure B.2 Peak Discharge in cfs/sq. mile per Inch of Runoff versus TC

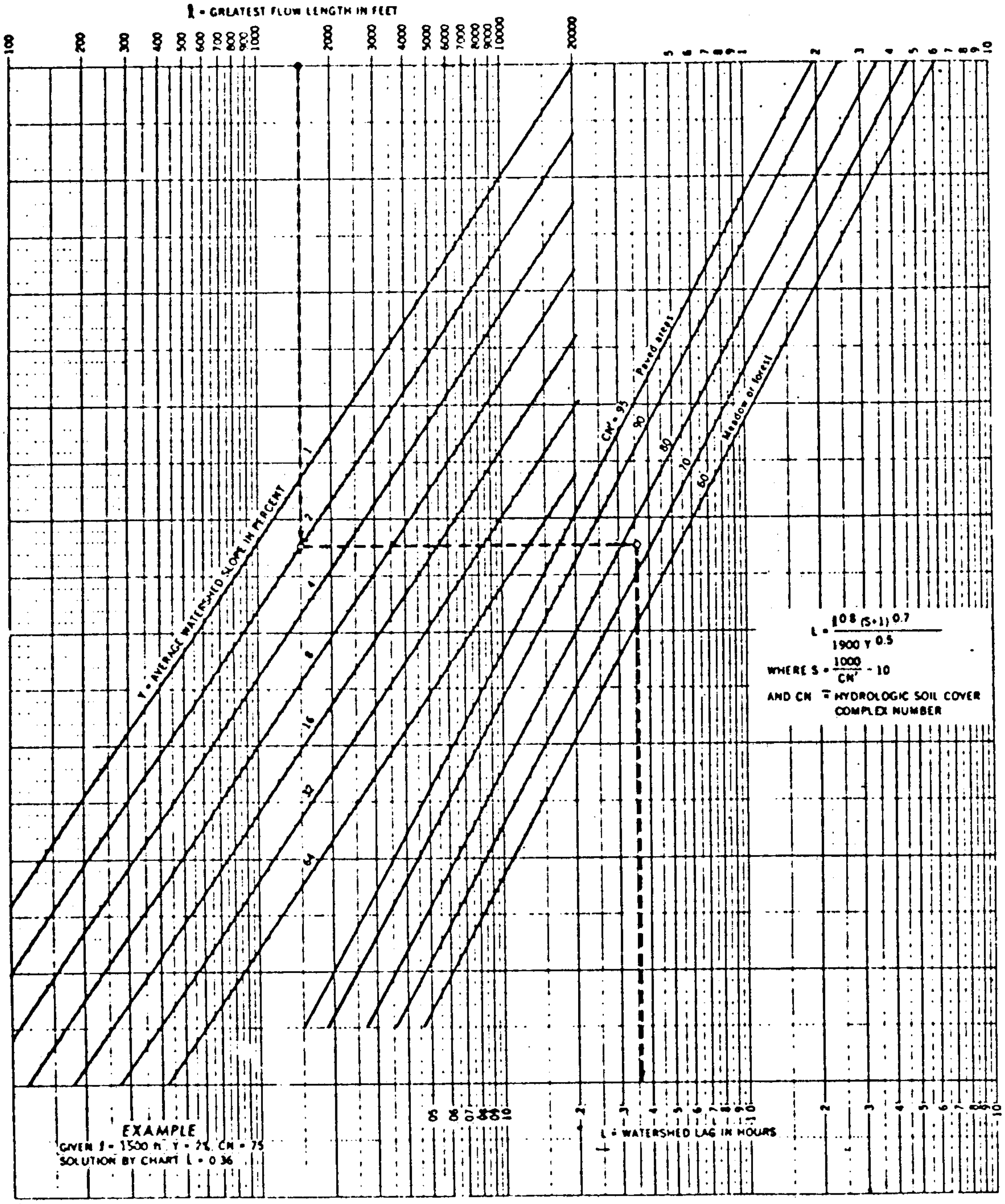


Figure B.3 --Curve number method for estimating lag (L) for homogeneous watersheds under natural conditions up to 2,000 acres.

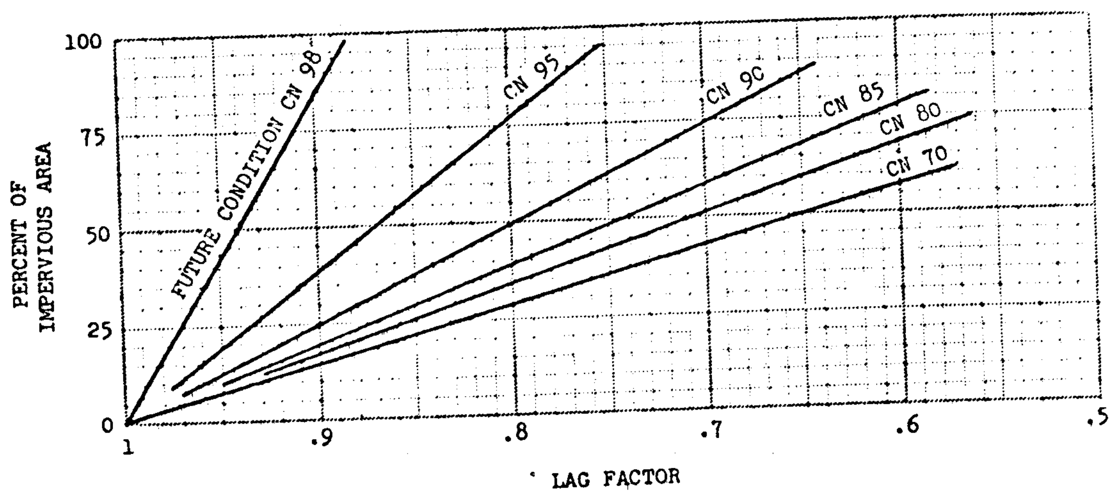


Figure B.4 Factors For Adjusting Lag When Impervious Areas Occur In Basin

Table B.1 Runoff Depth In Inches For Selected CN's And Rainfall Amounts

Rainfall (inches)	Curve Number (CN)[1]								
	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	.56	.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	.74	.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

[1] To obtain runoff depths for CN's and other rainfall amounts not shown in this table, use an arithmetic interpolation.