

STORM WATER MANAGEMENT SEMINAR

PART III

**BY
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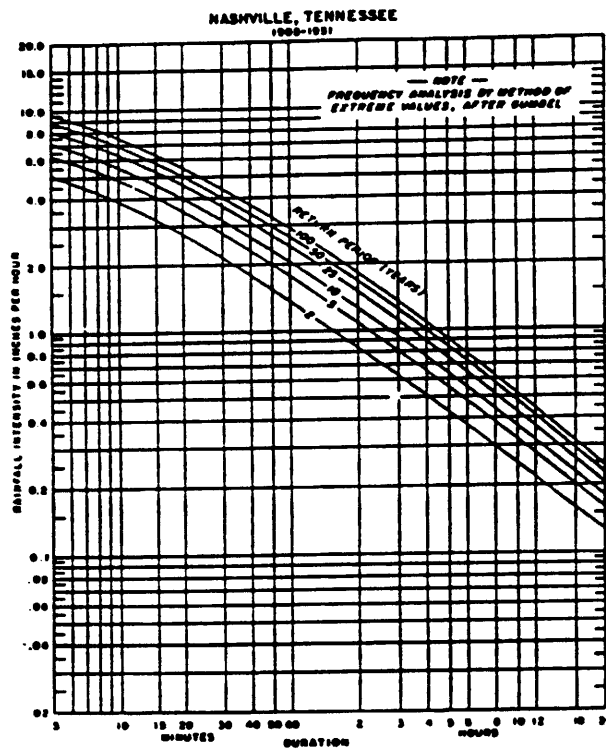
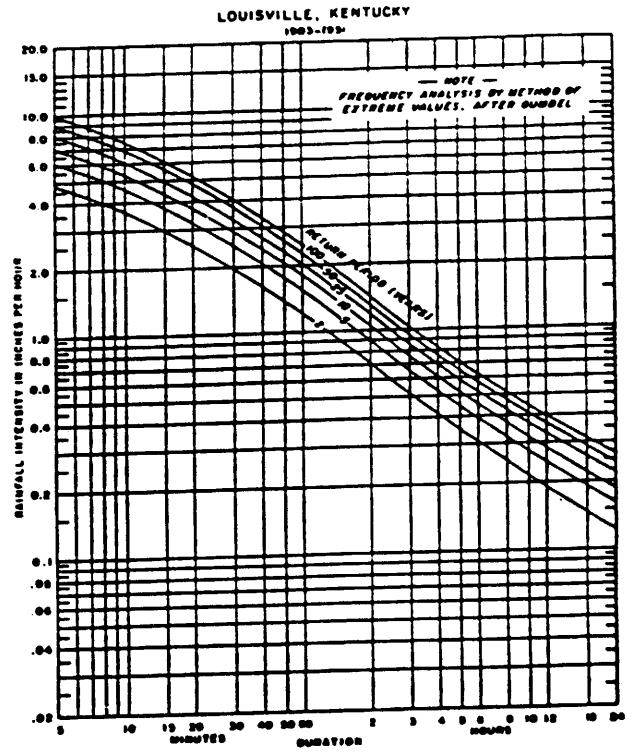
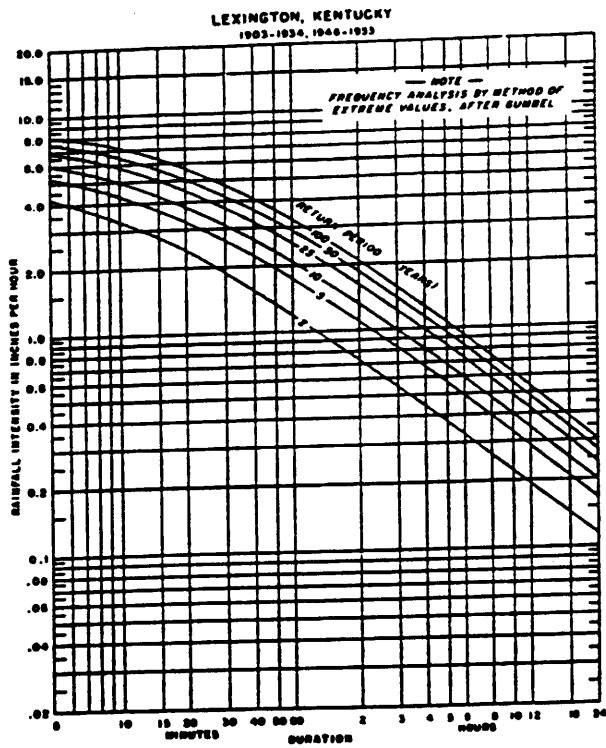
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<u>APPENDIX</u>	<u>CONTENTS</u> <u>TITLE</u>	<u>PLATE</u>
	INTRODUCTION	
"A"	FREQUENCY CURVES.....	A-1 thru A-3
"B"	LAKE AYRE ESTATES PLAN.....	B-1
	CHANGE IN "C" FACTOR.....	B-2
	CHANGE IN RUN-OFF FACTOR.....	B-3
"C"	COLORADO URBAN HYDROGRAPH PROCEDURE COMMENTARY.....	C-1 thru C-16
"D"	CLARK METHOD COMMENTARY	D-1 thru D-9
"E"	HYDRAULIC STUDY-OLDHAM COUNTY SCHOOL:	
	Problem and Solution.....	E-1
	Problem and Solution.....	E-2
	Site Plan.....	E-3
	Clark Method Isochrones.....	E-4
	100-Year Rain-fall.....	E-5
	Clark Method Basic Data.....	E-6
	Clark Method Basic Data.....	E-7
	Clark Method Unitgraph.....	E-8
	Hydrograph, 100-Year Storm.....	E-9
	Hydrograph, 100-Year Storm.....	E-10
	Channel Rating.....	E-11
	Channel Rating.....	E-12
	Swellhead Through Bridge.....	E-13
	Swellhead Through Bridge.....	E-14
	Rational Method-Minor Tributary.....	E-15
	Flow Conditions at Culvert.....	E-16
	Sizing New Culvert.....	E-17
	RETENTION BASIN DESIGN, BRECKINRIDGE ESTATES:	
	Inflow Hydrographs.....	F-1
	Spillway Rating.....	F-2
	Storage Curve Data.....	F-3
	Reservoir Routings.....	F-4
	Stage and Outflow Hydrographs.....	F-5
	INLET CONTROL RATING-CIRCULAR	
	CONDUIT.....	G-1
	STONE DIAMETER VS. VELOCITY.....	G-2
	IMPACT BASIN DESIGN.....	G-3
	IMPACT BASIN DESIGN.....	G-4
	IMPACT BASIN DESIGN.....	G-5

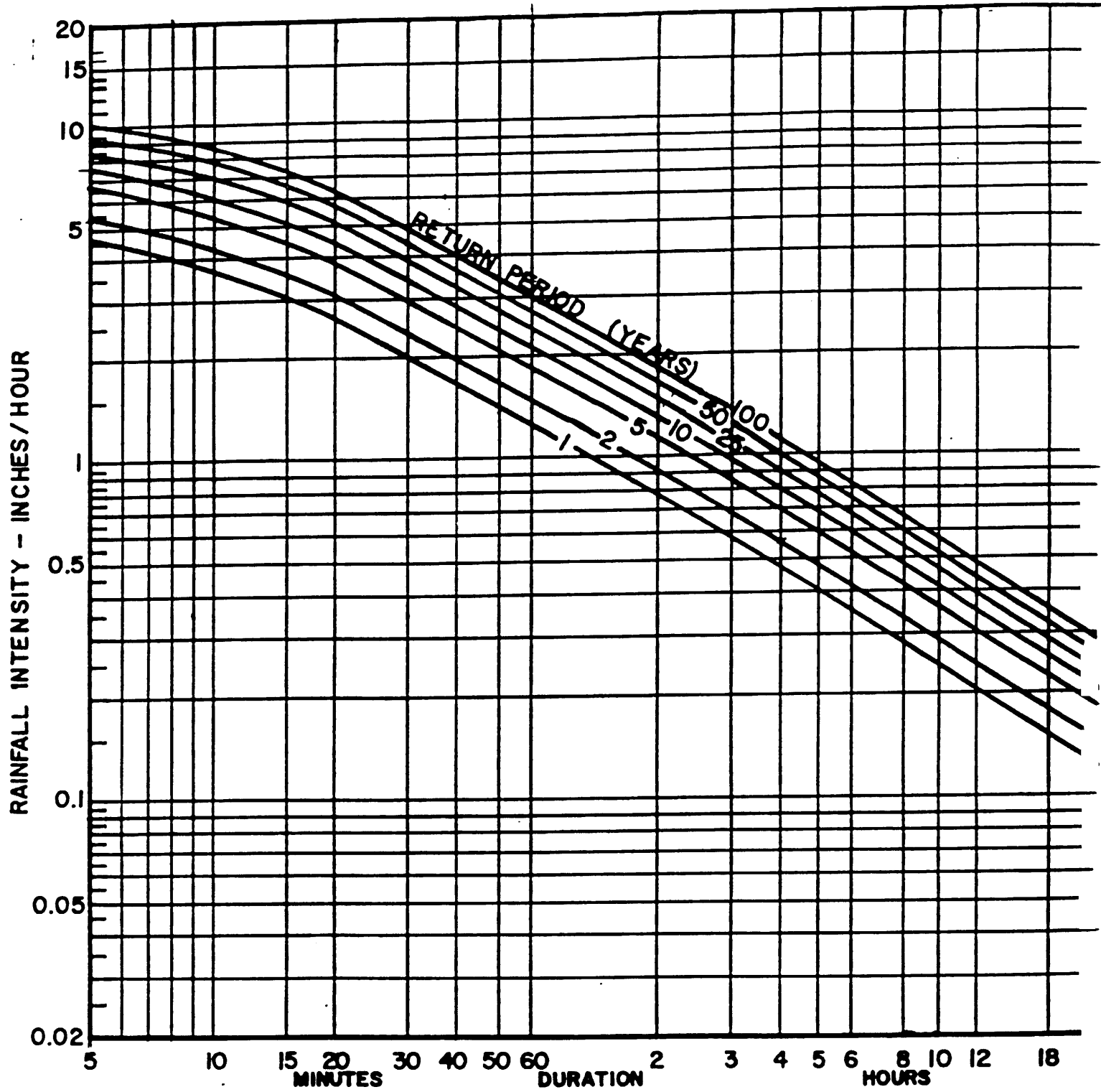
INTRODUCTION

The data herein supplements the written commentary in Part 2 of this series. The computational examples shown are representative of one water management approach to each specific problem and should not be construed as suggesting that other methods are deliberately excluded. As was noted in the introduction to a preceding Part, the author is attempting to portray examples which can be understood by non-specialists.

RAINFALL INTENSITY-DURATION-FREQUENCY CURVES



BOWLING GREEN, KENTUCKY

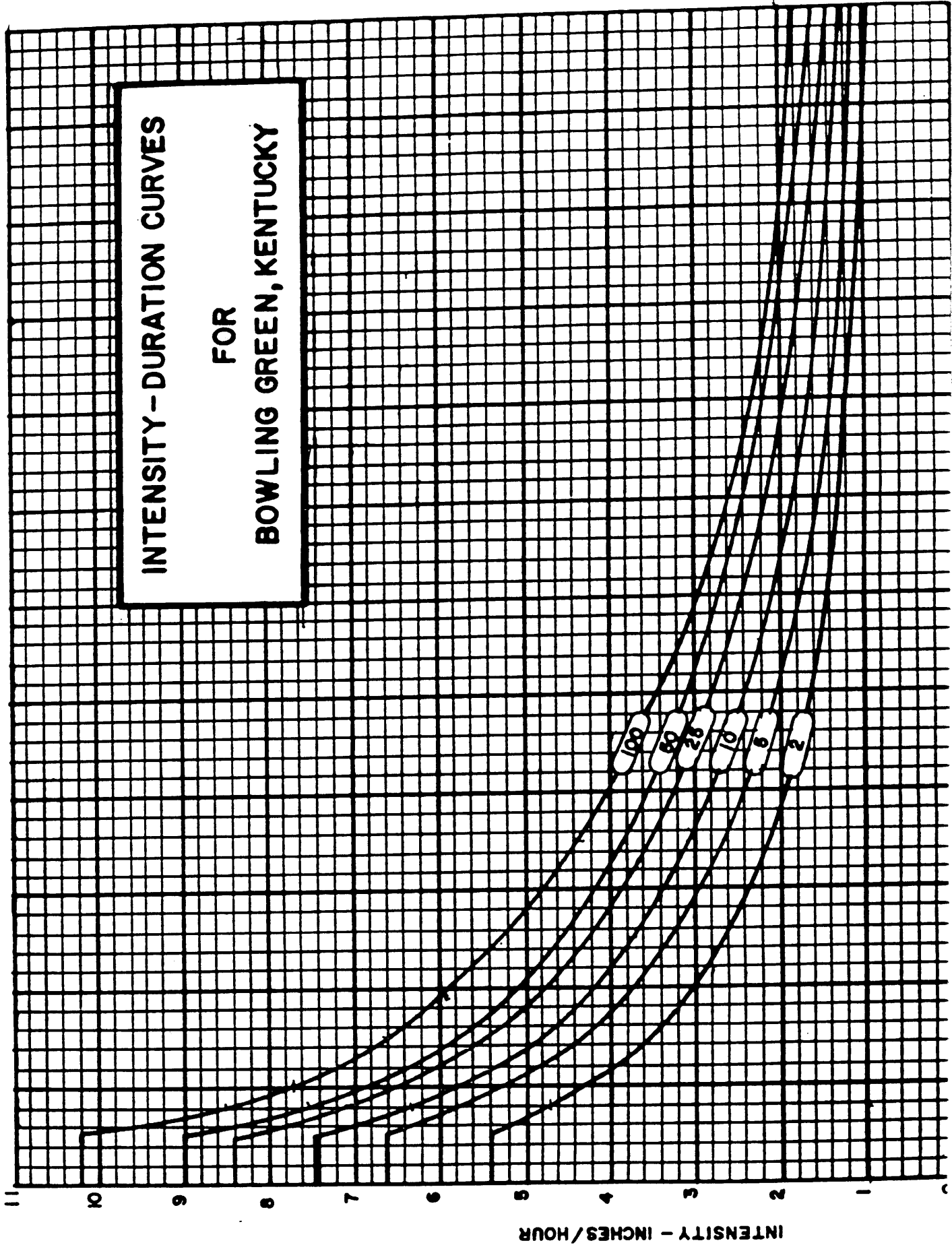


RAINFALL INTENSITY — DURATION

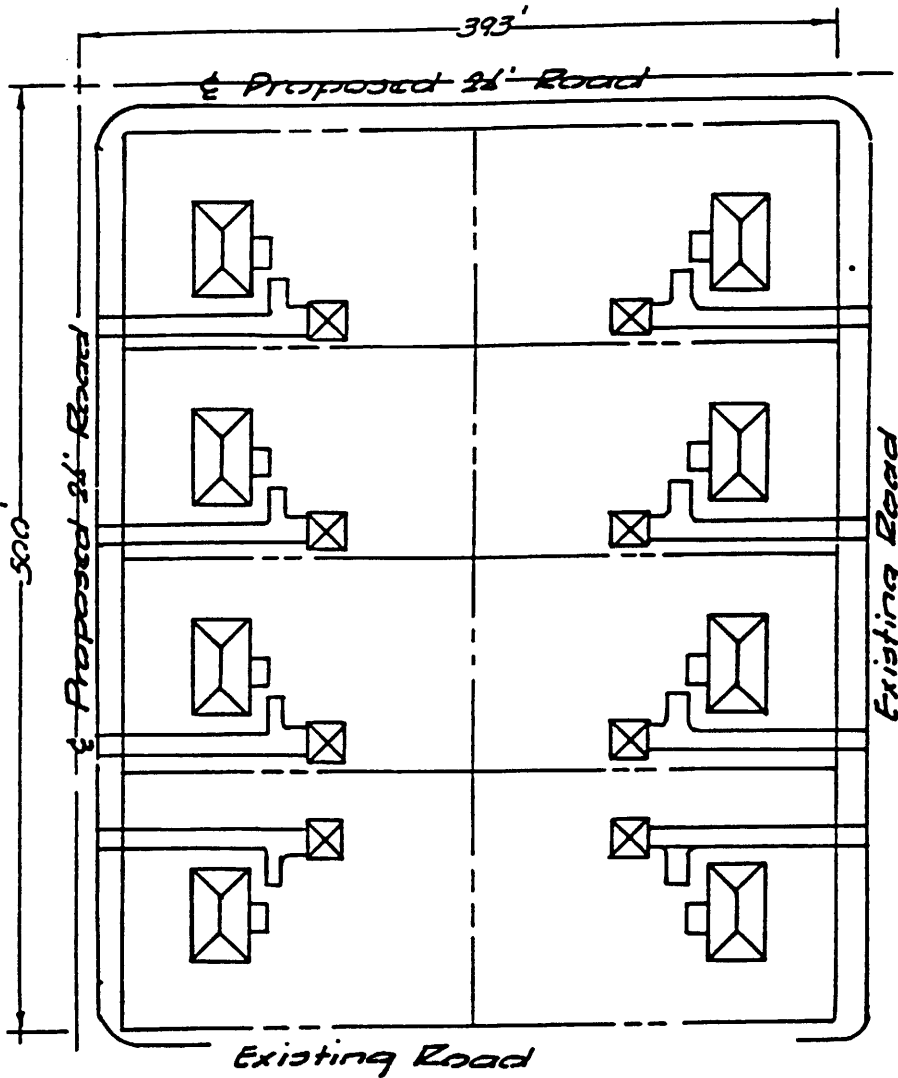
FREQUENCY CURVES

BASED ON DATA IN U.S.W.B. TECHNICAL PAPER NO. 40, DATED 1961.

**INTENSITY - DURATION CURVES
FOR
BOWLING GREEN, KENTUCKY**



CHANGE IN 'C' FACTOR



EXISTING CONDITIONS

Area = 393 x 500
= 196,500 sf
 $C_N = 0.25$

PROPOSED CONDITIONS

Area =

Impervious Areas

Houses (8)

(8) x 1500 = 12,000 sf

Patios (8)

(8) x 150 = 1,200 sf

Garages (8)

(8) x 440 = 3,520 sf

Drives (8)

(8) x 110 x 10 = 8,800 sf

(8) x 20 x 20 = 3,200 sf

12,000

Total Impervious -

28,720 sf

Roadways (prop.)

(500 + 393 - 12) x 12'

= 10,572 sf

Total Lot + Road

Impervious Areas

39,292 sf

$C_1 = 0.95$

TYPICAL PARTIAL DEVELOPMENT PLAN

Computation of C_M :

$$\frac{(C_N \times A_P)}{A} + \frac{(C_1 \times A_I)}{A} + k \left(\frac{A_P}{A} \right) = C_M$$

where: $C_N = 0.25$; $C_1 = 0.95$

$A = 196,500$

$A_I = 39,292$

$A_P = 157,208$

$k = .20$

$$\frac{(.25 \times 157,208)}{196,500} + \frac{(.95 \times 39,292)}{196,500} + 0.2 \left(\frac{157,208}{196,500} \right) = C_M$$

$$0.39 + 0.16 = \underline{\underline{0.55}} = C_M$$

Note: "k" factor of 0.20 shown represents drainage enhancement from re-grade of pervious areas, and reflects the engineer's judgement.

RUN-OFF FOR PURPOSE OF RETENTION

As an example of run-off to be retained, the foregoing "TYPICAL PARTIAL DEVELOPMENT PLAN" portrayed a method of computing the modified run-off factor, C_M , and this value of $C_M = 0.55$ will be applied to the entire 39.5 acre project.

$$C_M = 0.55 \quad \Delta C = C_M - C_N = 0.55 - 0.25 = 0.30$$
$$A = 39.5 \text{ acres}$$
$$R = 2.95/12 \quad (\text{for one hour of 100-yr. storm})$$

For change in run-off in one hour and a gravity-surface project outlet:

$$\text{Retention} = \Delta C R A$$
$$= 0.30 \times \frac{2.95}{12} \times 39.5 \text{ acres}$$
$$= \underline{2.91 \text{ acre-ft.}}$$

For total run-off in three hours and a sinkhole out.

$$\text{Retention} = C_M R A$$
$$= 0.55 \times \frac{4.0}{12} \times 39.5 \text{ acres}$$
$$= \underline{7.17 \text{ acre-ft.}}$$

4. COLORADO URBAN HYDROGRAPH PROCEDURE

For basins that are larger than about 200 acres and for some complex basins that are less than 200 acres, it is recommended that the design storm runoff be analyzed by deriving synthetic unit hydrographs. The unit hydrograph principle was originally developed by Sherman in 1932 (12). The synthetic unit hydrograph, which is used for analysis when there is no rainfall-runoff data for the basin under study, as is often the case in the Denver region, was developed by Snyder in 1938 (13). The presentation given in this chapter is termed the Colorado Urban Hydrograph Procedure (CUHP) because coefficients are based upon data collection and studies financed by the City of Denver, the Denver Regional Council of Governments and the Urban Drainage and Flood Control District.

4.1 Definition

A unit hydrograph is defined as the hydrograph of one inch of direct runoff from the tributary area resulting from a unit storm. A unit storm is a rainfall of such duration that the period of surface runoff is not appreciable less for any rain of shorter duration. The unit hydrograph thus represents the integrated effects of factors such as tributary area, shape, street pattern, channel capacities, and street and land slopes (14, 15, 16, 17, 18, 19).

To apply the unit hydrograph the effective precipitation depth for the "unit storm" periods are multiplied by the ordinates of the unit hydrograph and added to obtain a design storm runoff.

the basic premise of the unit hydrograph is that individual hydrographs resulting from the successive increments of rainfall excess that occur throughout a storm period will be proportional in discharge throughout their length, and that when properly arranged with respect to time the ordinates of the individual unitgraphs can be added to give ordinates representing the total storm discharge. The hydrograph of total storm discharge is obtained by summing the ordinates of the individual hydrographs.

4.2 Basic Assumptions

The derivation and application of the unit hydrograph are based on the following assumptions:

1. The rainfall intensity is constant during the storm that produces the unit hydrograph.
2. The rainfall is uniformly distributed throughout the whole area of the drainage basin.
3. The base or time duration of the design runoff due to an effective rainfall of unit duration is constant.

4. The ordinates of the design runoff with a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.
5. The effects of all physical characteristics of a given drainage basin, including shape, slope, detention, infiltration, drainage pattern, channel storage, etc., are reflected in the shape of the unit hydrograph for that basin.

4.3 Equations

There are two basic equations used in defining the limits of the synthetic unit hydrograph. The first equation defines the lag time of the basin in terms of time to peak, t_p , which, for the CUHP Method, is defined as the time from the center of the unit storm duration to the peak of the unit hydrograph as shown in Figure 4-6. For most urban studies the unit storm duration should range between 5 to 50 minutes.

$$t_p = C_1 (L L_{ca})^3 \quad (4-1)$$

Where t_p = time to peak of hydrograph from midpoint of unit rainfall in hours.

L = length along stream from study point to upstream limits of the basin in miles.

L_{ca} = distance from study point along stream to the centroid of the basin in miles.

C_1 = a coefficient reflecting time to peak.

The second equation defines the unit peak of the unit hydrograph.

$$q_p = \frac{640 C_p}{t_p} \quad (4-2)$$

Where q_p = peak rate of runoff in cfs per square mile

C_p = a coefficient related to peak rate of runoff.

For discussions of C_p and C_1 values refer to paragraph 4.4.

4.4 C_p and C_t Data from Denver Watersheds

The C_p and C_t values in equations 4-1 and 4-2 are determined from the following equations:

$$C_t = \frac{7.81}{(I_a)^{0.78}} \quad (4-3)$$

$$r^2 = 0.95 \text{ (coefficient of determination)}$$

where I_a = percent of watershed which is impervious.

$$C_p = 0.89 (C_t)^{0.46} \quad (4-4)$$

$$r^2 = 0.21 \text{ (coefficient of determination)}$$

Equations 4-3 and 4-4 were developed from a statistical analysis of ninety-six 5-minute unit hydrographs derived from flood events measured on nineteen different urban watersheds in the Denver-Boulder metropolitan region during the period from 1967 to 1973. The 5-minute unit hydrographs were derived from the measured floods using the HEC1 computer program (35). The Snyder Time and Peak Coefficients, C_t and C_p , were obtained from these derived unit hydrographs. The percent of impervious watershed existing at the time of the flood event was determined from aerial photographs. The time to peak, t_p , of the unit hydrographs is shown as a function of the watershed parameter LL_{ca} on Figure 4-1. It was assumed that the equation of the line through the data would follow the general form of Equation 4-1 with percent of impervious watershed, I_a , as the third parameter. A line was first drawn through the data for a $I_a = 50\%$ because there were more data available over a larger range of the watershed parameter, LL_{ca} . Lines for $I_a = 8\%$, 30% , 40% , and 100% were subsequently drawn parallel to the 50 percent line on the lag curve.

The scatter of the data on Figure 4-1 is attributed to the fact that the floods observed during the 1976 to 1973 period were mainly small floods. Based on unit hydrograph research in this field (17, 32, 33, 34), there is a tendency for non-linearity and scatter to exist amounts the unit hydrograph parameters when the unit hydrographs were derived from small amounts of rainfall excess.

(This section will be revised as additional data becomes available.)

The values of C_t and C_p can be estimated either from equations 4-3 and 4-4, or estimated graphically from Figure 4-2 and 4-3. Some additional data from unit hydrograph studies elsewhere in the United States are shown on Figure 4-2 to assist in defining the curve.

The percent of the impervious watershed, I_a , for an urban watershed in the first stages of planning may be estimated using the values suggested in Table 2-1. Alternatively the percent of impervious watershed could be estimated from aerial photographs of an existing urban watershed having a similar plan of development adjacent to the planned watershed.

For estimating C_1 : Add 10% for sparsely sewered areas. Subtract 10% for fully sewered areas.

Add 10% for very flat basins. Subtract 10% for steep basins.

For estimating C_p : Subtract 10% for sparsely sewered areas; add 10% for fully sewered areas.

Subtract 10% for very flat basins; add 10% for steep basins.

For estimating: See Table 2-1 for percent impervious data.

4.5 Unit Hydrograph Shape

The shape of the unit hydrograph is a function of the physical characteristics of the watershed. The shape is developed from empirical relationships.

The peak rate of discharge, q_p , is determined from equation 4-2. The value of q_p can be checked by consulting Figure 4-4. The regression line shown on Figure 4-4 is:

$$q_p = \frac{Q_p}{A} = 1387 (A)^{-0.348} \tag{4-5}$$

If the basin is excessively long or flat, the value should be somewhat below the regression line. If the watershed is excessively steep or has high velocity in its channels, the value of q_p should be above the regression line.

Both Figures 4-4 and 4-5 were prepared from the characteristics of the 5-minute unit hydrographs derived from the floods measured on 19 Denver metropolitan region urban watersheds. Equations 4-6 and 4-7 may be used to estimate the width of the unit hydrograph at 50 percent and 75 percent of the peak discharge:

$$W_{@ 50\% Q_p} = \frac{500}{\frac{Q_p}{A}} \tag{4-6}$$

$$W_{@ 75\% Q_p} = \frac{260}{\frac{Q_p}{A}} \tag{4-7}$$

These values could also be obtained from Figure 4-5.

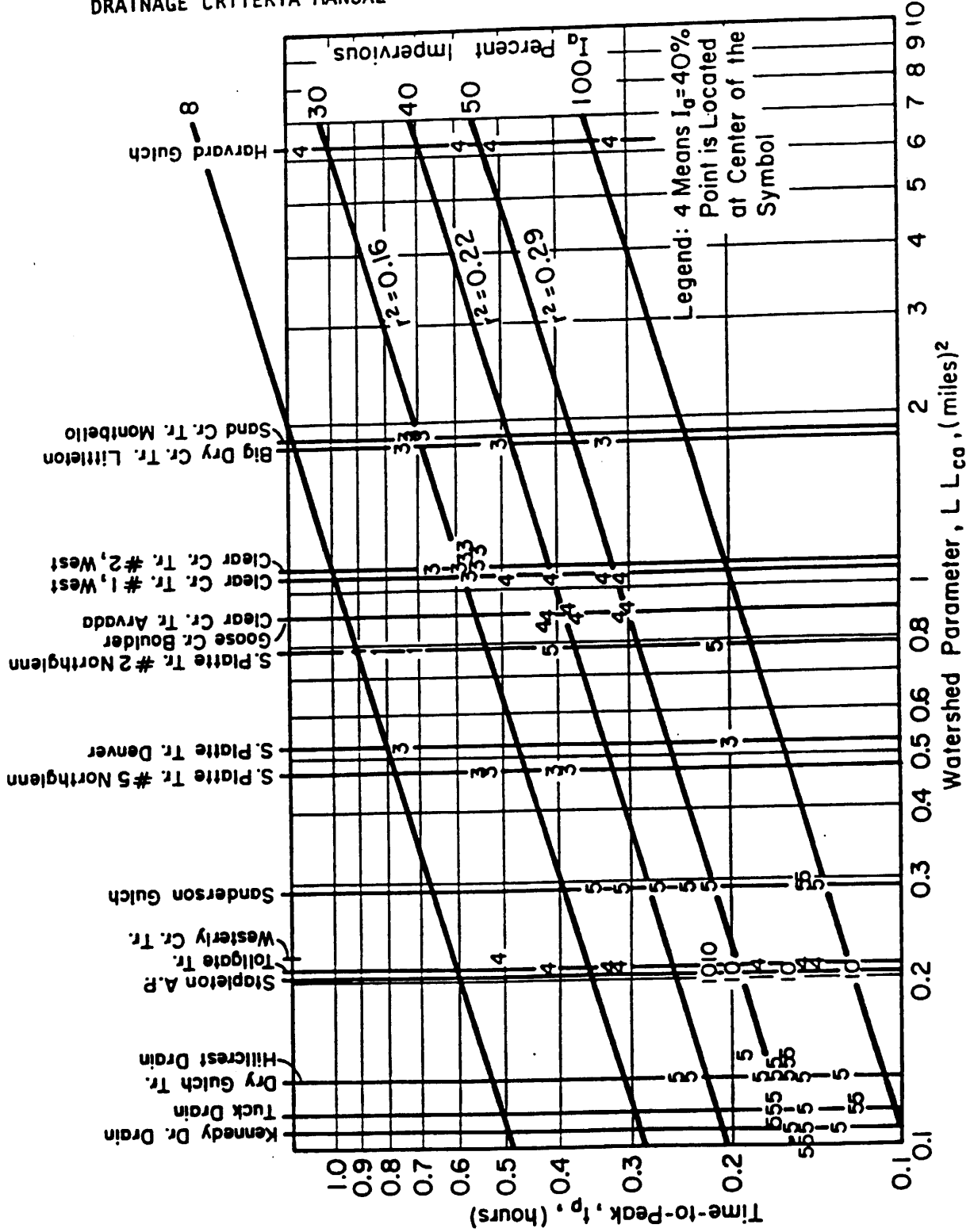


Figure 4-1 LAG CURVE FOR DENVER URBAN WATERSHEDS

1-15-69

Revised 5-15-75 follows paragraph 4.4 and Equation 4-4

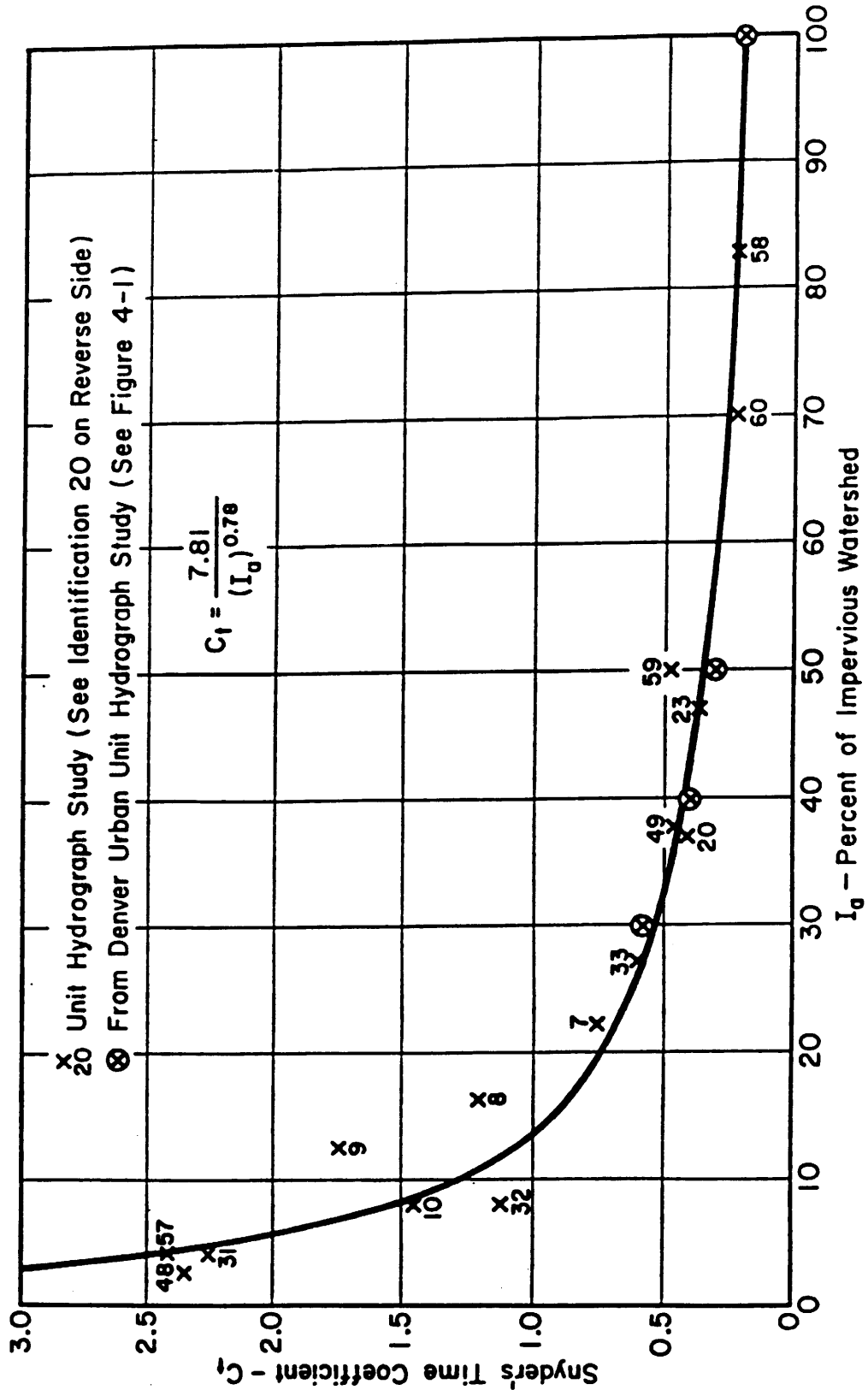
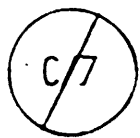


Figure 4-2 RELATIONSHIP BETWEEN C_t AND IMPERVIOUSNESS

1-15-69
 Revised 5-15-75 follows Figure 4-1



RUNOFF

Identification Number	Stream	Drainage Area (sq.mi.)	I _a (%)	C _t
7	Wooden Bridge Run, Philadelphia, PA	3.35	22.1	0.76
9	Poquessing Cr. at Trevoese Rd., Philadelphia, PA	5.1	12.5	1.74
8	Wissahichon Cr. at Bells Mill Rd., Philadelphia, PA	53.6	16.3	1.22
10	Pennypack Cr. at Pine Rd., Philadelphia, PA	37.9	9.1	1.45
20	Brushy Cr. at Hiway 311, Winston-Salem, NC	0.55	37	0.41
23	Turtle Cr. Dallas, TX	7.98	47	0.37
31	Cole Cr. at Guhn Rd., Houston, TX	7.05	4	2.25
32	Brickhouse Gully at Costa Rica St., Houston, TX	10.5	8	1.13
33	Waller Cr. at 38th St., Austin, TX	2.31	27	0.51
48	Anacostia Cr., IL	72.4	2.7	2.36
49	Boneyard Cr. at Urbana, IL	4.45	37.4	0.45
57	Salt Fork, West Br. IL	71.4	4	2.42
58	Louisville at 17th st. KY	0.22	83	0.22
59	Louisville, North Trunk Sewer	1.9	50	0.26
60	Louisville, West Outfall, KY	2.77	70	0.21

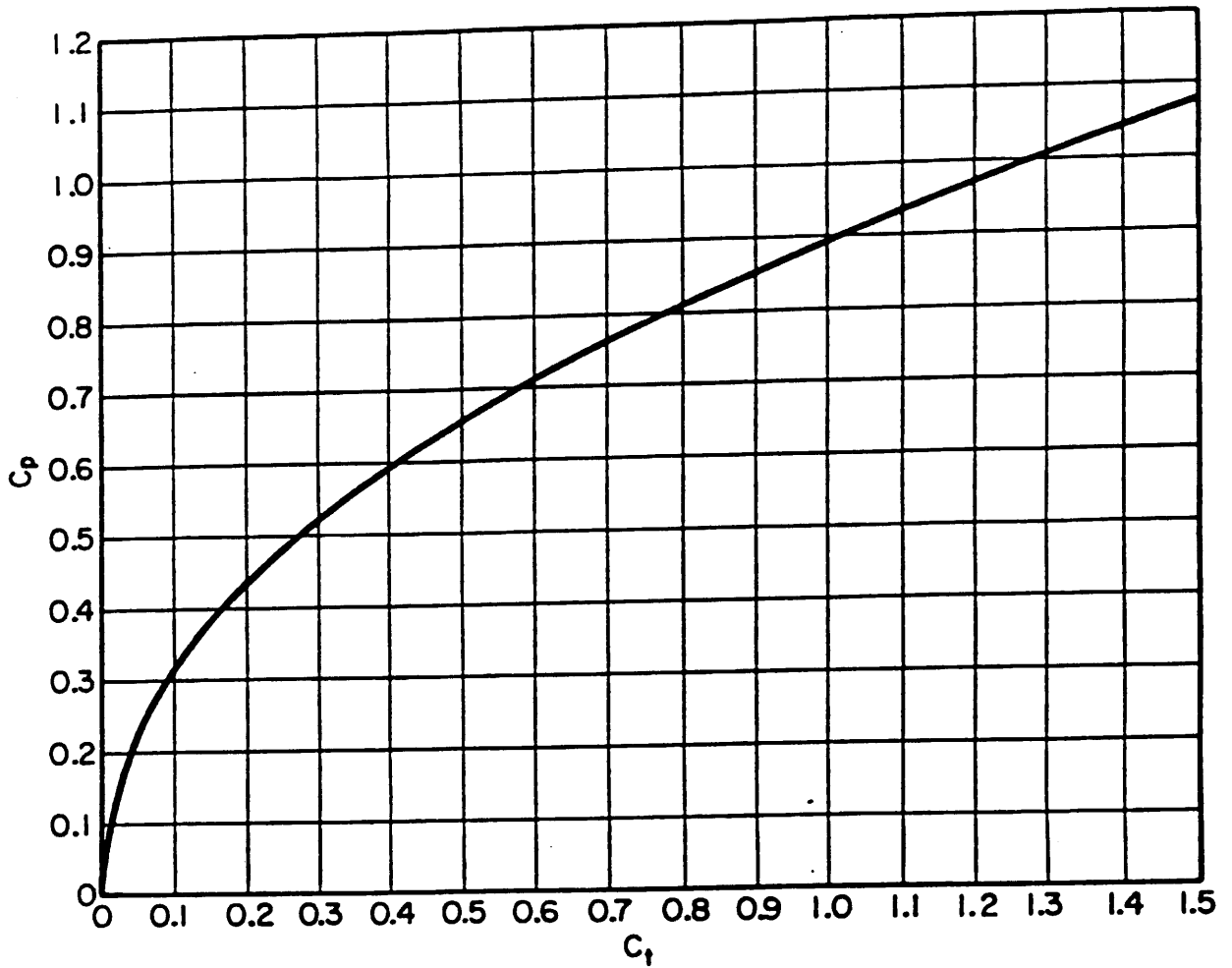


Figure 4-3 RELATIONSHIP BETWEEN C_p AND C_t

1-15-69
Revised 5-15-75 follows Figure 4-2

4.6 Drawing the Unit Hydrograph

Once q_p is determined from Equation 4-2, Q_p , the maximum unit hydrograph peak for the basin, can be computed by:

$$Q_p = q_p A \tag{4-8}$$

Where A is the area of the basin in square miles.

The time from the beginning of rainfall to the peak of the unit hydrograph is determined by:

$$T_p = 60 t_p + 0.5 t_u$$

Where t_u = time of unit rainfall duration in minutes, and

T_p = time from beginning of unit rainfall to peak of hydrograph in minutes.

Once Q_p is located, the unit hydrograph can be sketched with the aid of the approximate widths $Q_{50\%}$ and $Q_{75\%}$. After the hydrograph is sketched, the area under the hydrograph should be planimetered to determine the volume of runoff in acre feet or other suitable units.

This volume should equal the volume of 1 inch of runoff from the entire basin, or $Vol. = Area \text{ in acres} \times 1/12$. If the two volumes are within 5 percent, then the sketched unit hydrograph is acceptable. If the volume from the drawn hydrograph should be adjusted to within 5 percent of one inch of runoff. the final step is to define the unit hydrograph in tabular form showing time vs. rate of flow in cfs. If Q_p does not fall on a chosen time interval so that the tabulation does not represent the graph, then the graph may be shifted so that the table will more truly represent the graph.

4.7 Design Storm Runoff

Now that the unit hydrograph has been calculated (4.6) and the effective precipitation from the design storm determined (2.4), the design storm hydrograph can be calculated. The time units of the unit hydrograph abscissa should be the same as the time units of the excess precipitation which for convenience should all be equal to the unit storm duration, and can generally be taken as 10 minutes for an urban area less 5 square miles. Unit times of 5 minutes can be used for small basins up to 0.5 square miles, and unit times of 15 minutes and more for larger basins.

Set up a table such as Table 4-1, putting time intervals in the first column and unit hydrograph ordinates in the second column. Place the design excess precipitation values as determined in Column 13 of Table 2-3 across the top, and then multiply the first excess precipitation value (.02 in example) times all the unit hydrograph ordinates in Column 2 and put answers in the third column. Next multiply the second excess precipitation value (.05) times the unit hydrograph

ordinates lagged one time unit as shown in Column 4. Multiply each succeeding precipitation value times the unit hydrograph value and lag them appropriately in the table. Finally, add up all the multiplied values horizontally to obtain the design storm runoff hydrograph.

4.8 Example

Given: A basin when fully developed is expected to have the following characteristics:

Area = 0.85 square miles = 544 acres.

- L = 1.21 miles
- L_{ca} = 0.85 miles
- 60% = pervious area
- 40% = impervious area

Use a unit duration of 10 minutes.

Determine a 10 year design runoff from the basin, using the CUHP method.

Step 1. Determine C_i given the percent of impervious cover using Equation 4-3. Alternatively C_i may be estimated from Figure 4-2.

$$C_i = \frac{7.81}{(40)0.78} = 0.44$$

Step 2. Determine t_p using Equation 4-1.

$$t_p = C_i(L - L_{ca})^3 = .44 (1.21 - .85)^3$$

$$= .44 (1.008) = 0.44 \text{ hour} = 27 \text{ minutes}$$

Step 3. Determine C_p using Equation 4-4 and value of C_i found in Step 1.

$$C_p = 0.89 C_i^{0.46} = .89 (.44)^{0.46} = 0.61$$

Step 4. Determine q_p using Equation 4-2.

$$q_p = \frac{640 C_p}{t_p} = \frac{640 (.61)}{.44} = 887 \text{ cfs/sq.mi.}$$

Step 5. Determine Q_p = q_pA = 887 (.85) = 754 cfs,
say 750 cfs.

DRAINAGE CRITERIA MANUAL

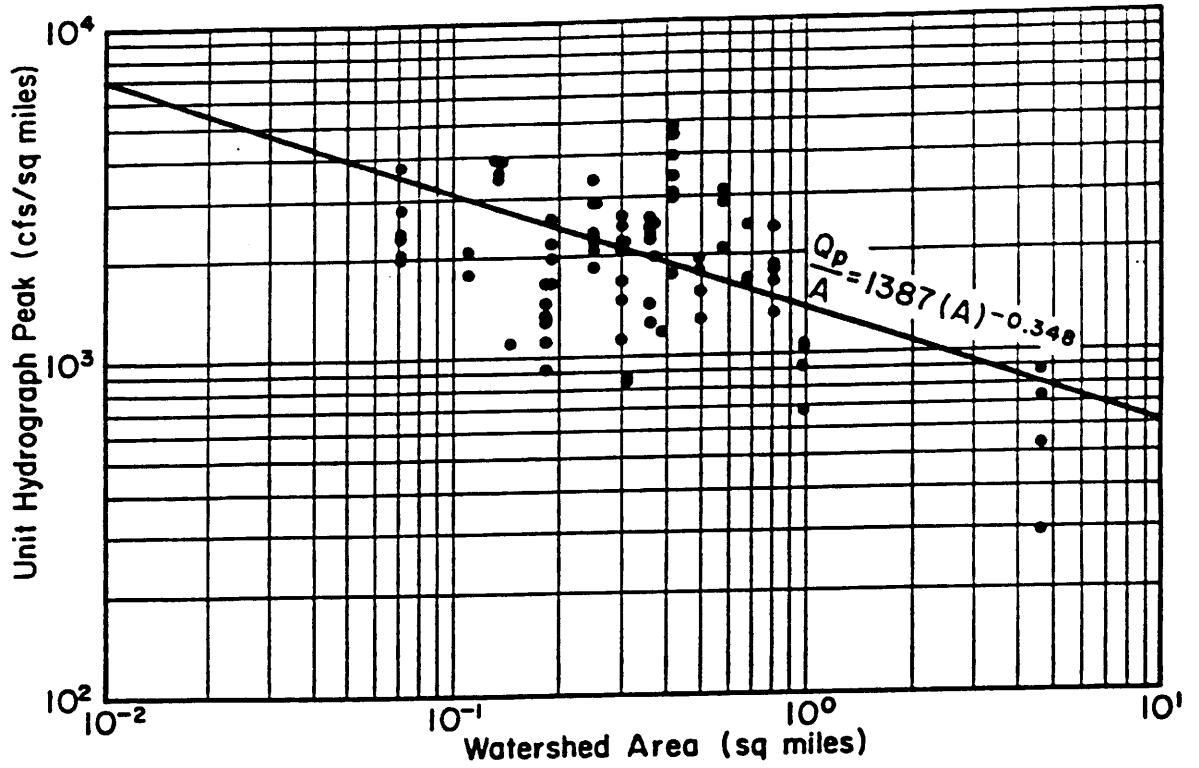


Figure 4-4 Check for Unit Hydrograph Peak

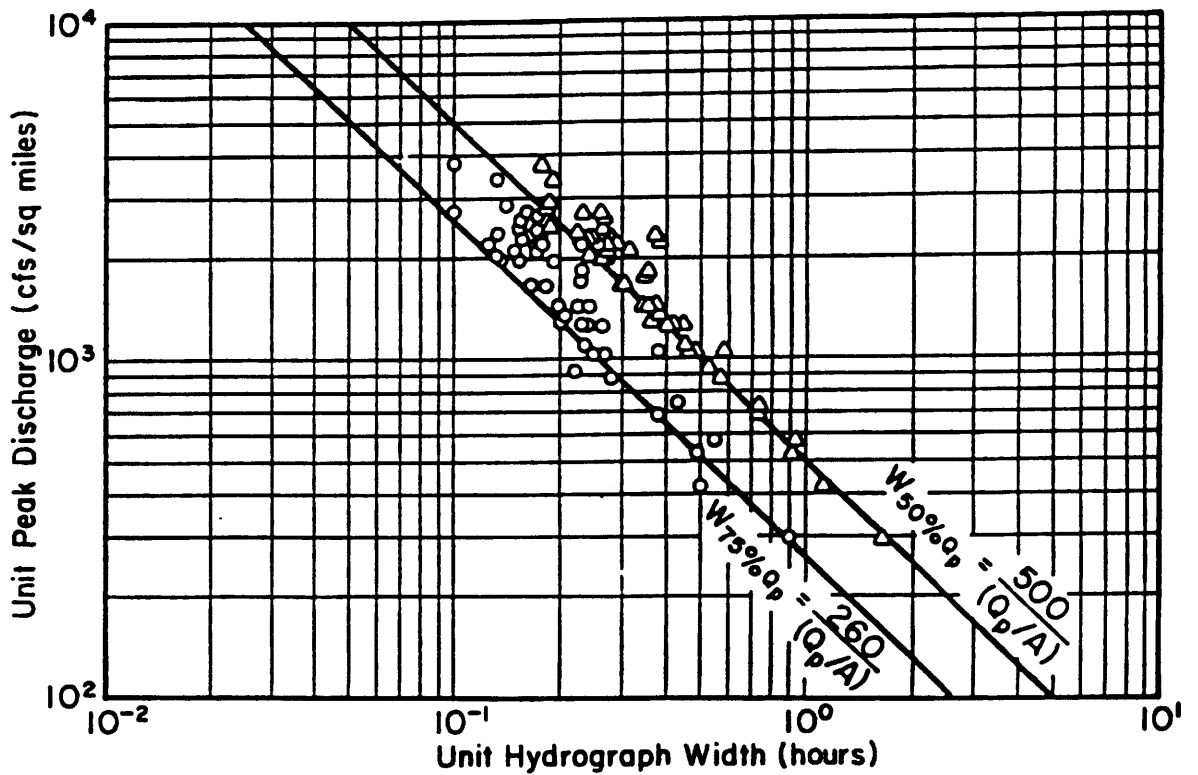


Figure 4-5 UNIT HYDROGRAPH WIDTHS

1-15-69

Revised 5-15-75 follows paragraph 4.5 and Equation 4-7

TABLE 4-1
Determination of Storm Hydrograph (Example)

Time (min) (1)	Unit Hydrograph (cfs) (2)	Excess Precipitation in inches												Storm Hydrograph (cfs) (17)			
		0.02 (3)	0.05 (4)	0.69 (5)	0.24 (6)	0.16 (7)	0.06 (8)	0.03 (9)	0.03 (10)	0.02 (11)	0.02 (12)	0.02 (13)	(14)		(15)	(16)	
0	0	0															0
10	160	3	0														3
20	460	9	8	0													17
30	750	15	23	110	0												148
40	570	11	38	317	38	0											404
50	390	8	29	518	110	26	0										691
60	265	5	20	393	180	74	10	0									682
70	185	4	13	269	137	120	28	5	0								576
80	135	3	9	183	94	91	45	14	6	0							444
90	100	2	7	128	64	62	34	23	14	3	0						337
100	75	2	5	93	44	42	23	17	23	9	3	0					261
110	50	1	4	69	32	30	16	12	17	15	9	3					208
120	40	1	3	52	24	22	11	18	12	11	15	9					168
130	30	1	2	35	18	16	8	6	8	8	11	15					128
140	20	0	2	28	12	12	6	4	6	5	8	11					94
150	10		1	21	10	8	5	3	4	4	5	8					69
160	0		1	24	7	6	3	2	3	3	4	5					48
170			0	7	5	5	2	2	2	2	3	4					32
180				0	2	3	2	1	2	2	2	3					17
190					0	2	1	1	1	1	2	2					10
200						0	1	1	1	1	1	2					7
210							0	0	1	1	1	1					4
220									0	0	1	1					2
											0	1					1
												0					0

- Step 6. Determine the width of the unit hydrograph at 50% and 75% of the using Q_p Figure 4-5 and $q_p = 890$ cfs/sq.mi.

$$\begin{aligned} w_{50\%q_p} &= 0.56 \text{ hours} \\ &= 34 \text{ minutes} \end{aligned}$$

$$\begin{aligned} w_{75\%q_p} &= 0.29 \text{ hours} \\ &= 17 \text{ minutes} \end{aligned}$$

- Step 7. Determine the time to peak from the beginning of rainfall using Equation 4-9.

$$T_p = 60 t_p + \frac{t_w}{2} = 27 + \frac{10}{2} = 32 \text{ minutes}$$

- Step 8. Using the results of Steps 5, 6, and 7, sketch a unit hydrograph. See Figure 4-6.

- Step 9. The volume of the unit hydrograph should be:

$$544 \text{ acres} \times 1 \text{ inch}/12 = 45.3 \text{ ac. ft.}$$

Planimeter the area under the hydrograph and determine the actual volume.

The volume for the first trial was 50.3 ac.ft. which was about 10% too high. The unit hydrograph was revised as shown in Figure 4-6. The volume was 44.6 ac.ft. which is a trifle too small but easily falls within the 5% criterion given in paragraph 4.6. The revised hydrograph as shown is thus accepted.

- Step 10. Repeat Steps 8 and 9 until the runoff volume under the hydrograph is equal to $45 \pm$ acre feet. then present the unit hydrograph in tabular form as shown on Figure 4-6.
- Step 11. Obtain the design excess precipitation values in 10-minute (unit Duration) increments. This is done in Table 2-3.
- Step 12. Set up Table 4-2.
- Step 13. Multiply the precipitation value at the top of Column 3 by each of the unit hydrograph ordinates and put in Column 3 for the corresponding time. Next multiply the precipitation value in Column 4 by each of the unit hydrograph ordinates and place in Column 4 lagged one time from the corresponding unit hydrograph time. Proceed to multiply each of the precipitation values times the unit hydrograph ordinates, each time lagging the new hydrograph by one more time unit.
- Step 14. Column 17 is the design runoff hydrograph obtained by summing horizontally the individual hydrographs in Column 3 through 12. Note that in this example time zero is the beginning of excess rainfall and not the beginning of rainfall. This is important

when lagging and routing several hydrographs from different basins together.

4.9 Acquisition of additional data

In 1969 the basic Snyder synthetic unit hydrograph method was modified for use in the Denver Metropolitan region. The Denver Regional Council of Governments and later the Urban Drainage and Flood Control District in cooperation with the U.S. Geological Survey began a systematic data acquisition program.

In 1975 the significant flood events measured in the Denver Metropolitan network were analyzed. The 5-minute unit hydrographs derived from 96 flood events measured on 19 different watersheds have provided a valuable insight into the effect of urbanization on the unit hydrograph parameters. As the span of records and the magnitude of the floods increases, the equations and graphs will be modified reflecting the more complete knowledge about the formation of floods in the urban environment.

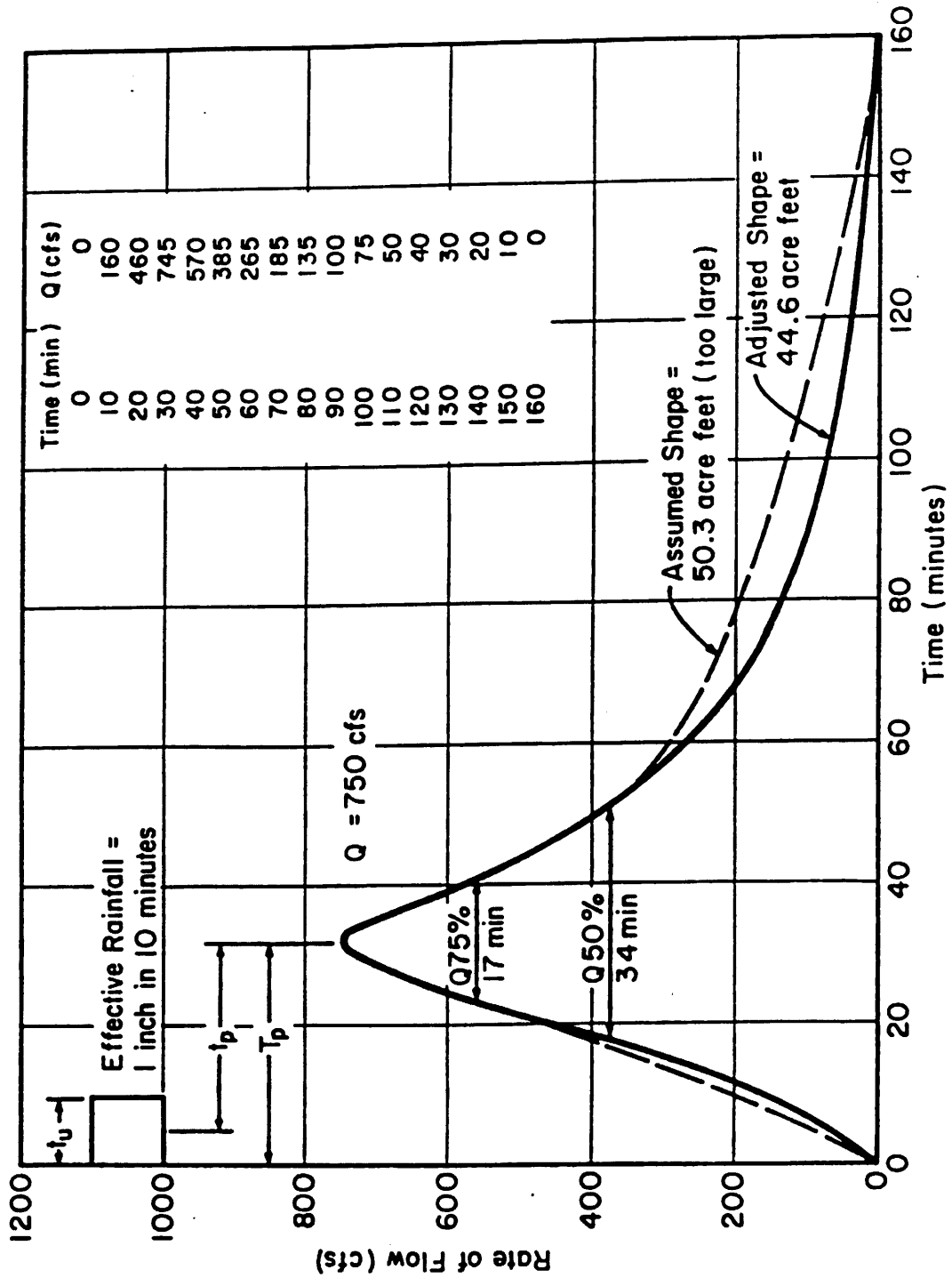


Figure 4-6 UNIT HYDROGRAPH EXAMPLE

1-15-69

Revised 5-15-75 follows paragraph 4.8

C/16

TABLE 2-3
DETERMINATION OF EFFECTIVE RAINFALL
(Example)

Location: Sec. 91, TEN., R. 70W.
Design Storm: 10-Year Recurrence Interval

Time (hr.) (1)	P E R V I O U S A R E A 60 %				I M P E R V I O U S A R E A 40 %				Total Average Effective Precipitation (in.) (13)	
	Maximum Infiltration (in.) (5)	Detention & Depression Storage (in.) (6)	Effective Precipitation (in.) (7)	Effective Precipitation (in.) (8)	Detention & Depression Storage (in.) (9)	Loss (in.) (10)	Effective Precipitation (in.) (11)	Effective Precipitation (in.) (12)		
0	0	0	0	0	0	0	0	0	0	0
10	.89	.05	0	0	.05	0	0	0	0	0
20	1.18	.10	0	0	.05	0	.05	.02	.02	.02
30	1.40	.15	0	0	.05	0	.12	.05	.05	.05
40	1.55	.19	0	.55	.25	0	.55	.54	.54	.69
50	1.64	.21	0	.15	0	0	.25	.11	.11	.24
60	1.74	.22	0	.08	0	0	.21	.08	.08	.16
70	1.82	.11	0	.05	0	0	.10	.04	.04	.06
80	1.89	.08	0	0	0	0	.08	.05	.05	.05
90	1.95	.07	0	0	0	0	.07	.05	.05	.05
100	2.00	.06	0	0	0	0	.06	.02	.02	.02
110	2.04	.04	0	0	0	0	.04	.02	.02	.02
120	2.06	.04	0	0	0	0	.04	.02	.02	.02
Totals	2.08	2.08	2.08	.58	.50	.96	.190	.76	.76	1.34

DERIVATION OF UNIT HYDROGRAPHS BY THE CLARK METHOD

There are many different unit hydrographs for the same basin because the shapes of the hydrographs vary with different unit storm durations. To define a generalized unit hydrograph for a basin, C.O. Clark developed a technique (reference 1) which uses the concept of the instantaneous unit hydrograph (IUH). This is theoretically the hydrograph that would result from one unit of excess occurring over the basin in a specified areal pattern and zero time. The IUH can then be used to compute a unit hydrograph for any unit duration equal to or greater than the time interval used in the computations.

The Clark method translates incremental runoff from subareas within a basin to the basin outflow location according to travel times and then routes this runoff through a linear reservoir in order to account for the storage effects of the basin and channels. The time of concentration (t_c) is defined as the travel time of water particles from the most upstream point (timewise) in the basin to the outflow location. This lag time may be estimated by measuring the time between the end of effective rainfall and snowmelt over the basin and the inflection point on the recession limb of the surface runoff hydrograph, as illustrated in figure 1. When the time of concentration has been determined, the basin is divided into incremental runoff-producing areas that have equal travel times to the outflow location. The distance from the most upstream point in the basin is measured along the principal watercourse to the outflow location. Dividing this distance by t_c gives the rate of travel or the distance traveled in unit time. Isochrones representing equal travel time to the outflow location are laid out using the distance traveled per unit time to establish the location of the lines. The increment of time used to subdivide the basin need only be small enough to adequately define the areal distribution of runoff. The areas between the isochrones are then measured and tabulated with the corresponding travel time (from 0 to t_c) for each incremental area.

The time period selected as the computation interval should be approximately equal to the unit duration of excess. A plot of percent of length/versus accumulative area is useful in determining time-area relationships. Such a curve facilitates rapid development of unit hydrographs for various computation intervals and unit durations of excess. This is especially helpful when making flood predictions for basins where t_c is not firmly established, as unit hydrographs may be easily modified to reflect subsequent changes in t_c . Also, it is possible to refine the curve by considering the variation of velocity from stream reach to stream reach and specified contributions of excess (as ratios of basin-mean contribution) in different portions of the basin. Another advantage is that the unit duration can be changed without deriving a new time-area relationship.

The runoff from the contributing areas (between the isochrones) which has been translated to the outflow location is in units of volume (in-mi³) or mm-km²) and these must be converted to the proper units of discharge. This conversion is shown below.

$$I_i = K a_i / \Delta t \quad (1)$$

where:

I_i = ordinate in proper units of discharge (cfs or m³ /s) of the time-area runoff volumes at the end of period i.

a_i = ordinate in units of depth-area of excess (inch-mile² or mm/km²) of the time-area runoff at the end of period i.

K = conversion factor to convert inch-mile² /hour to cfs ($K = 645$) or mm-km² /hour to m³ /s ($K=.278$).

Δt = time period of computation interval in hours.

The routing of the translated runoff through storage at the outflow location is accomplished as follows:

$$O_i = C I_i + (1 - C) O_{i-1} \quad (2)$$

where:

O_i = outflow from the basin at end of period i in cfs (m³ /s).

I_i = inflow or runoff from each area at end of period i in cfs (m³ /s).

C = dimensionless routing constant.

The above routing equation results from setting the Muskingum "X" equal to zero in the coefficient method of routing (reference 2). The routing constant is:

$$C = \frac{2\Delta t}{2R + \Delta t} \quad (3)$$

where:

Δt = time period of computation interval.

R = attenuation constant having the dimension of time.

It can be shown that when inflow into the principal storage reach has ceased (Muskingum "X" = 0),

$$R = - \frac{Q}{dQ/dt} \quad (4)$$

The magnitude of R can be approximately evaluated at the point of inflection of the recession limb of the observed surface runoff hydrograph. The above ratio decreases to a minimum at the point of inflection and, in theory, remains constant thereafter. Therefore, R may be estimated by dividing the ordinate of the surface runoff hydrograph at the point as shown in figure 1. Another technique is to compute the volume of runoff remaining under the recession limb of the surface runoff hydrograph following the point of inflection and divide by the discharge at the same point. In either case, R should be an average value determined and verified with several hydrographs.

The hydrograph that results from routing these flows from the incremental areas is the instantaneous unit hydrograph. The instantaneous unit hydrograph can be converted to a unit hydrograph of a unit duration Δt by simply averaging two instantaneous unit hydrographs spaced at interval Δt apart as follows:

$$O_i = O_i \quad (5)$$

$$O_i = 0.5 (O_i + O_{i-1}) \text{ for } i \geq 2$$

The instantaneous unit hydrograph can be converted to a unit graph of some unit duration other than Δt , provided that it is an exact multiple of Δt , by taking n successive averages of the instantaneous unit hydrograph ordinates where n is the multiple of Δt for the desired unit graph duration. The first average is taken of the instantaneous unit hydrograph ordinates, as in equation (5) above and the second average is taken of the just computed O_i 's, the third from the results of the second, etc., repeating the procedure n times in total. The ordinate at any time, i , for a unit graph of duration D and tabulation interval of Δt is:

$$Q_i = 1/n (.5O_{i-n} + O_{i-n+1} + \dots + O_{i-1} + .5O_i) \quad (6)$$

where:

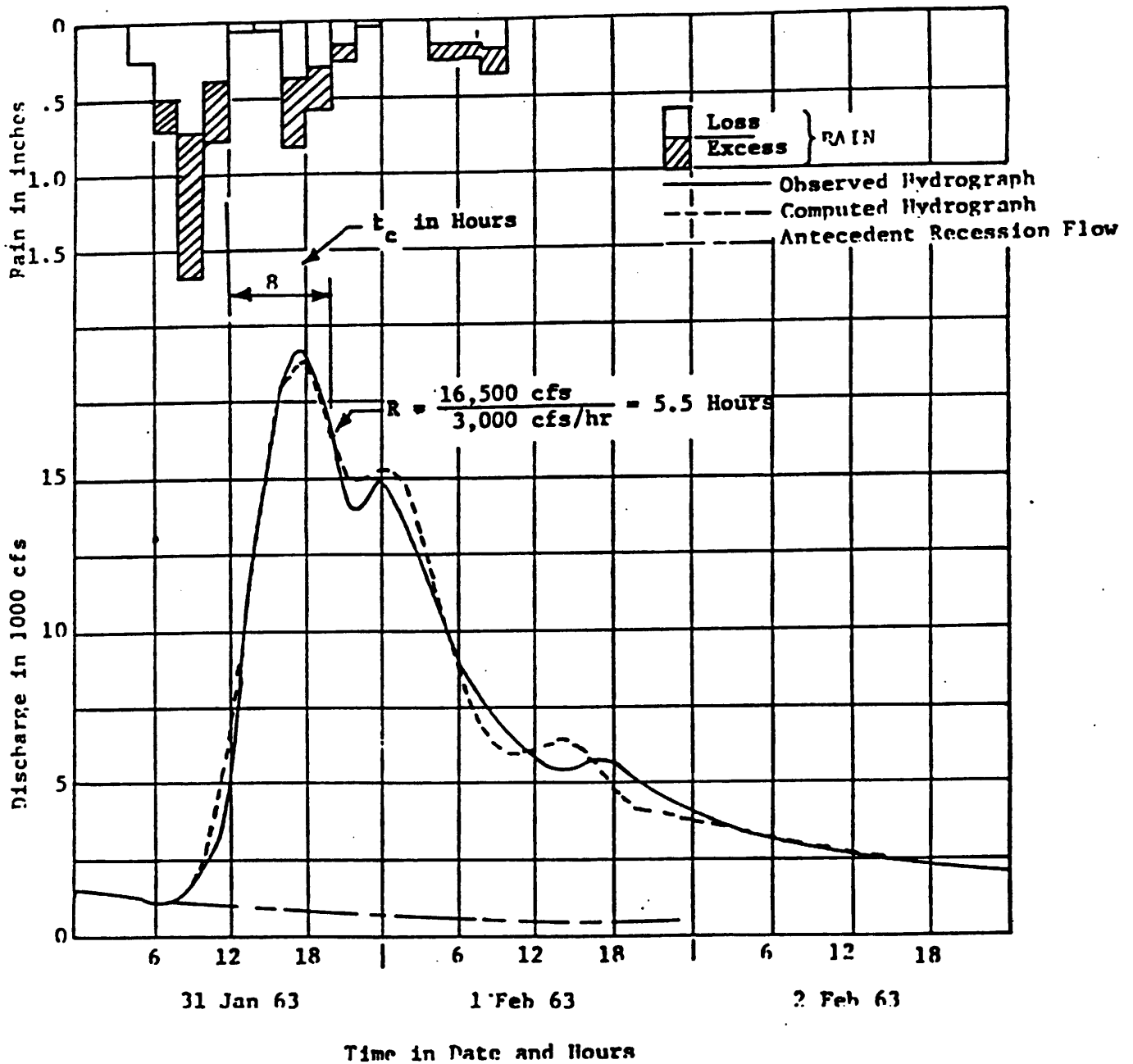
Q_i = ordinate at time i of unit graph of duration D and tabulation interval Δt

$$n = \frac{D}{\Delta t}$$

D = unit graph duration

Δt = tabulation interval

Figure 1. Determination of Clark Coefficients and Flood Reconstitution



Drainage Area: 190 sq. mi.

D 5

To illustrate the complete Clark procedure, a step-by-step example is worked out for the 31 January-4 February 1963 flood that occurred on Thames Creek, Paskenta, California, U.S.A.

Step 1

Draw lines (isochrones) which subdivide the basin into a chosen number of parts as illustrated in figure 2. These isochrones are constructed so that the travel time along a water course is the same from one isochrone to another. For simplicity, they are usually drawn equal distances apart from the outflow location to the uppermost head of the basin. The number of isochrones used is ordinarily chosen so that a convenient scale may be used and a reasonable good definition of the time vs. area relation obtained.

Step 2

Measure the areas between each pair of isochrones (figure 2). If a nonuniform pattern of excess is assumed, multiply each area by the average excess within that subdivision.

Step 3

Plot the curve of time vs. area (or excess) as shown on figure 3. Tabulate increments between points one computation interval apart.

Step 4

Convert the interval volume inflows to flow rates (columns 2 and 3 of table 1) using equation (1) so that the total volume equals the unit hydrograph volume corresponding to one unit of runoff.

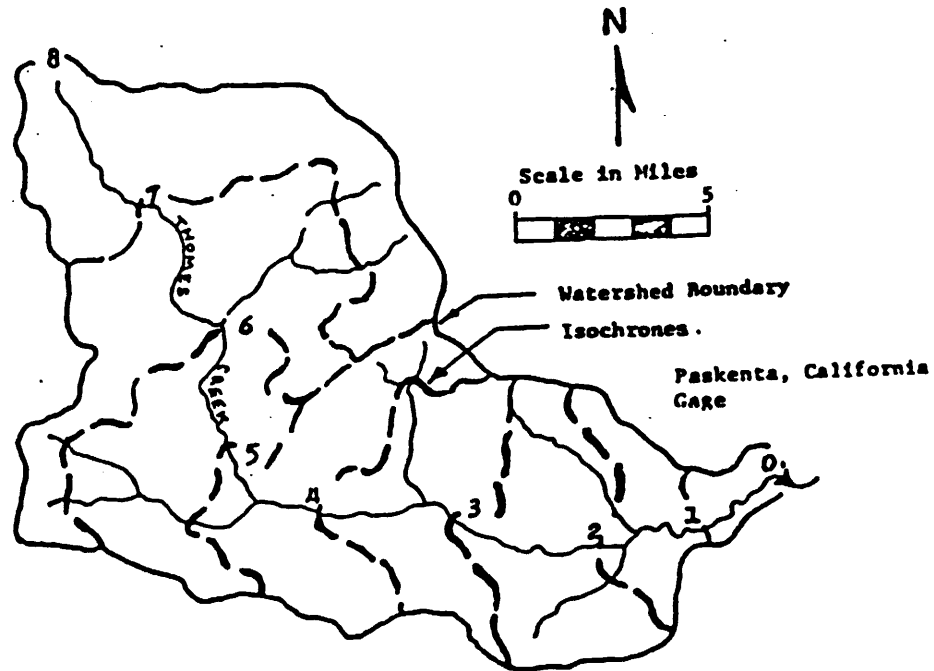
Step 5

Route the inflows (column 3 of table 1) from step 4 through storage at the outflow location (column 4 of table 1) using equations (2) and (3). This procedure results in the instantaneous unit hydrograph.

Step 6

Average the ordinates of the instantaneous unit hydrograph with those of the same instantaneous unit hydrograph one computation interval, Δt , earlier, equation (5). The resulting hydrograph is the unit hydrograph of duration equal to the computation interval, Δt (2-hour). The 4-hour unit graph is computed by averaging the ordinates of the 2-hour unit graph.

Figure 2. Computation of the Time-Area Relation



Travel Time from "8" to Gage is 8.0 Hours for the 32 Miles

Map Area Number	Planimeter Values from Map		Accumulated area (sq.mi.) (Col 3) · (58.8)	Travel Time in Percent [(1/8) · (100)]
	Incremental units (2)	Accumulated units (3)		
(1)				
1	0.08	0.08	5	12.5
2	0.15	0.23	14	25.0
3	0.40	0.63	37	37.5
4	0.36	0.99	58	50.0
5	0.45	1.44	85	62.5
6	0.45	1.89	111	75.0
7	0.66	2.55	150	87.5
8	0.68	3.23	190	100.0
Total	3.23			

Sq.mi./Planimeter unit = 190/3.23 = 58.8
 Drainage Area = 190 square miles

Figure 3. Watershed Time-Area Relation

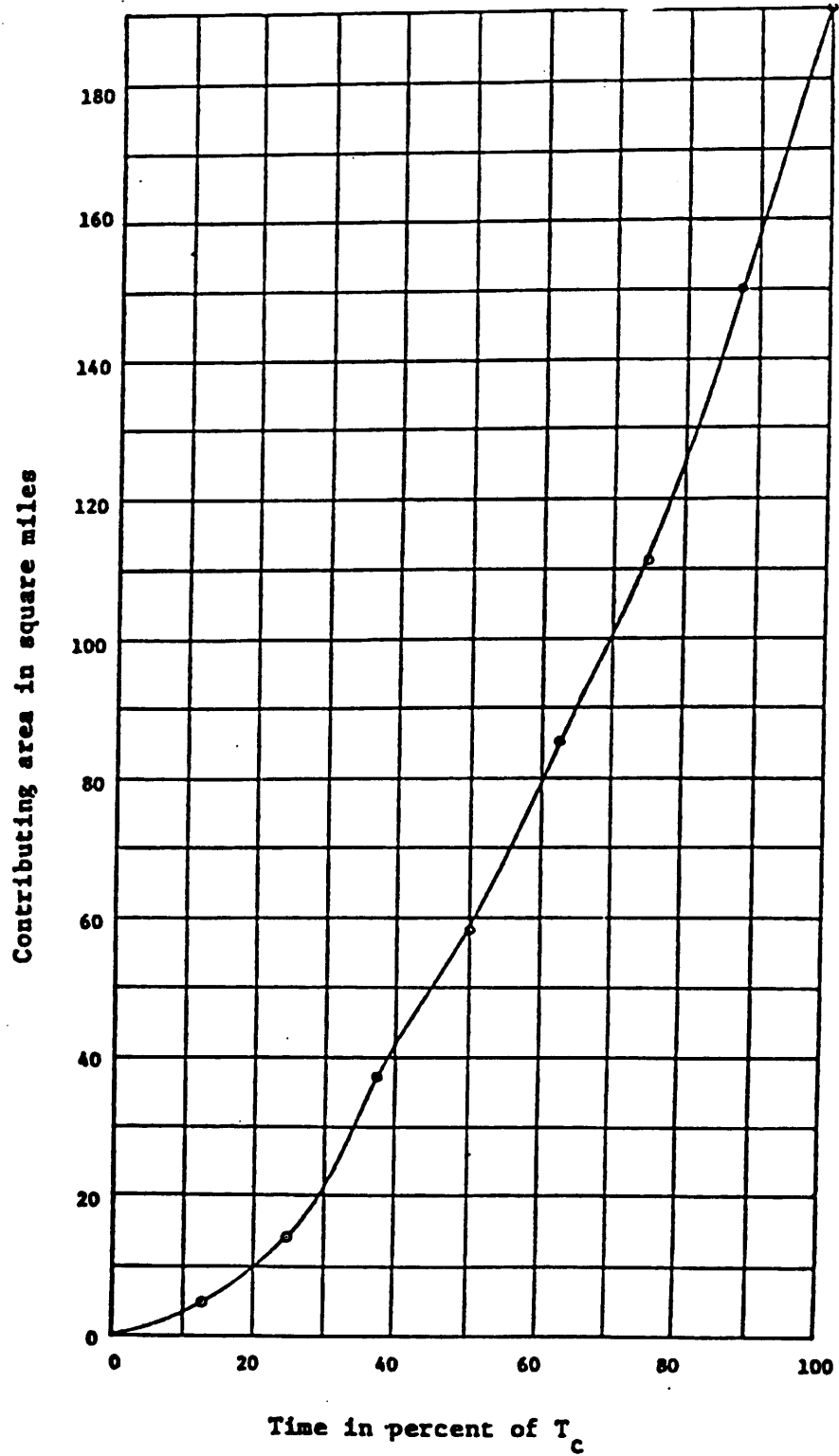


Table 1. Unit Graph Computation Clark Method
(Thomes Creek at Paskenta, California)

DRAINAGE AREA = 190 SQUARE MILES
 TIME OF CONCENTRATION (T_c) = 8.0 HOURS (See Figure 1)
 ATTENUATION VALUE (R) = 5.5 HOURS (See Figure 1)
 TIME INTERVAL (Δt) = 2.0 HOURS

EQUATIONS (Subscript 1 refers to current period)

$$I_1 = a_1 645 / \Delta t$$

$$C = \Delta t / (R + .5\Delta t) = 0.308$$

$$O_1 = CI_1 + (1-C)O_{1-1}$$

$$O_1 = .5(O_{1-1} + O_1)$$

TIME	INFLOW (Fig. 2)	INSTANTANEOUS UNIT GRAPH	2-HOUR UNIT GRAPH
hr (1)	a ₁ sq.mi.-in. (2)	I ₁ cfs (3)	O ₁ cfs (4)
			(5)
0	0	0	0
2	14	4,515	700
4	44	14,190	3,360
6	53	17,093	7,150
8	79	25,478	11,500
10	0	0	11,880
12			8,220
14			5,690
16			3,940
18			2,720
20			1,890
22			1,300
24			900
26			630
28			430
30			300
32			200
34			140
36			100
38			70
40			50
42			30
44			20
46			20

The Clark method has two advantages that make it particularly attractive. First, the procedure described herein provides a means of direct computation of unit hydrographs for electronic computer applications. Most other procedures require trial-and-error adjustments of the computed unit hydrograph. Second, the fact that a time-area curve is used provides a means of adjusting objectively for changes in drainage patterns resulting from urbanization or construction of reservoirs, channels, or diversions without requiring that the basin be subdivided into many subareas. This is accomplished simply by constructing a time-area curve (with modified t_c and R) that corresponds to new travel times through reaches and reservoirs.

The Clark unit hydrograph coefficients, t_c and R , are given physical significance in the previous discussion, but in practice, uncertainties of the concepts and of recorded data usually preclude their reliable determination in a simple fashion. It is known that t_c and R are not rigid, and by analyzing several different storms on the same basin, different values will probably be obtained for different storms. For instance, t_c for a storm centered over the head of the basin will probably be larger than one centered over the foot of the basin.

If discharge and rainfall records and snowmelt data are available, t_c and R can be estimated from observed events. As illustrated in figure 1, t_c can be estimated as the time from the end of heavy excess to the inflection point on the recession limb of the flood hydrograph. Likewise, R can be estimated by dividing the discharge at the inflection point by the rate of change of flow at that point on the hydrograph. However, the shapes of hydrographs reflect many irregularities of rainfall, snowmelt and stream patterns, and estimates obtained in this manner are usually satisfactory only for first approximation.

REFERENCES

1. Clark, C.O.. "Storage and the Unit Hydrograph," Trans. American Society of Civil Engineers, Vol. 110, pp. 1419-1488, 1945.
2. U. S. Army Corps of Engineers. Engineering and Design. "Routing of Floods through River Channels," EM 1110-2-1408, 1 March 1960.