ORANGE COUNTY

HYDROLOGY MANUAL

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ORANGE COUNTY ENVIRONMENTAL MANAGEMENT AGENCY
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OCTOBER 1986

PREFACE

The County of Orange Hydrology Manual was prepared under a contract (D85-078) with Williamson and Schmid, Irvine, California, approved by the Orange County Board of Supervisors on June 18, 1985.

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OCTOBER 1986

AUTHOR'S ACKNOWLEDGEMENTS

Special acknowledgements are paid to Richard H. McCuen, Ph.D., for his overall review of the several project products and for the preparation of several contributions to the work effort.

Key contributors to the project effort include Tim J. Durbin, MS, RCE, for his work in the stream gage analysis and adjustment for urbanization; Bob Whitley, Ph.D. for his statistical evaluation of the stream gage data confidence levels and the coupling of modeling uncertainty; and Joe DeVries, Ph.D., RCE for his work on the channel routing studies and certainty of results.

Particular acknowledgements are paid to the Los Angeles District Corps of Engineers (COE); Chief of Hydrologic Engineering Section, Andy Sienkiewich; and Chief, Hydrologic Engineering Unit 1, John Pedersen, for their generous cooperation and time.

Other acknowledgements are paid to the several engineering firms and governmental agencies who have contributed time through the course of four drainage committees over the last 5 years, or through the attendance of the public presentations, or through the review of this manuscript.

Finally, acknowledgements are paid to the several engineers who have contributed to the development of this Hydrology Manual. Special acknowledgements are paid to Alan J. Nestlinger, BSAP, MPA, RCE who has been closely involved with Agency hydrology studies and the subsequent manual since 1967. Several sections of this manual were authored or co-authored by Alan. Special acknowledgments are also paid to the many key Agency staff members including (alphabetically) Joe Natsuhara, BS, RCE, Carl Nelson, MSCE, RCE, Dick Runge, BS, RCE and Jim Williams, MSCE, RCE.

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SECTION A

INTRODUCTION

A.1. PURPOSE

This manual provides the computational techniques and criteria for the estimation of runoff, discharges, and volumes for use in submittals to the Orange County Environmental Management Agency (hereinafter "Agency").

A.2. HYDROLOGIC PROTECTION LEVELS

It is the goal of the Agency to provide 100-year return frequency flood protection for all habitable structures and other non-floodproof structures. Consequently, all drainage plans must demonstrate this 100-year flood protection criteria.

Additionally, it is the design objective of the Agency to afford specific design criteria for the more frequent flood events. That is, flood protection levels for 10- and 25-year floods may be required for major street travelways, catch basin sump design, and other conditions. The design criteria may be obtained from the Agency.

For additional related information see Appendices I.1 and I.2.

A.3. PRESENTATION

Precipitation and loss information used in the Rational Method and the Unit Hydrograph procedure for developing flowrates are contained in Sections B and C, respectively. Specific guidelines for application of the Rational Method are contained in Section D. Section E contains the procedures for developing runoff hydrographs using the Unit Hydrograph method and

Sections F through I contain guidelines for application of various flood routing methods. The development of runoff hydrographs for small areas is discussed in Section J and watershed modeling guidelines are provided in Section K. Peak flowrate curves for areas where use of single area unit hydrographs are appropriate are contained in Section L. The appendices provide additional discussion on various hydrology topics.

SECTION B

PRECIPITATION

B.1. PRECIPITATION DEPTH-DURATION-FREQUENCY

The following definitions of precipitation depth, duration, and frequency are used:

Precipitation depth: the amount of precipitation occurring during a specified duration of storm time. Precipitation depth is usually expressed in units of inches.

Duration: the specified length of storm time under study. Duration may be expressed in any time unit such as seconds, minutes, hours, days or season.

Frequency: the frequency of occurrence of events with the specified precipitation depth and duration. This is expressed in terms of either the return period or exceedance probability, both of which are defined below.

Intensity-duration: dividing precipitation depth by duration, an average intensity for a specified duration is obtained.

Critical duration: the critical duration of a design storm event for a hydraulic structure is usually the "time of concentration," which is the time for water deposited at the most remote part of a watershed to flow to the structure, outlet or spillway.

Exceedance (cumulative) probability: the probability that a precipitation event of a specified depth and duration will be exceeded in one year.

Return period (recurrence interval): the long term average number of years between occurrences of an event of a given depth and duration, either equaled or exceeded.

The exceedance probability (p) and return period (T) are related by

$$p = 1/T$$

A 100-year precipitation event will not necessarily occur exactly once in every 100 years but actually has a finite probability that it will occur in several consecutive years or not at all in a period of 100 years.

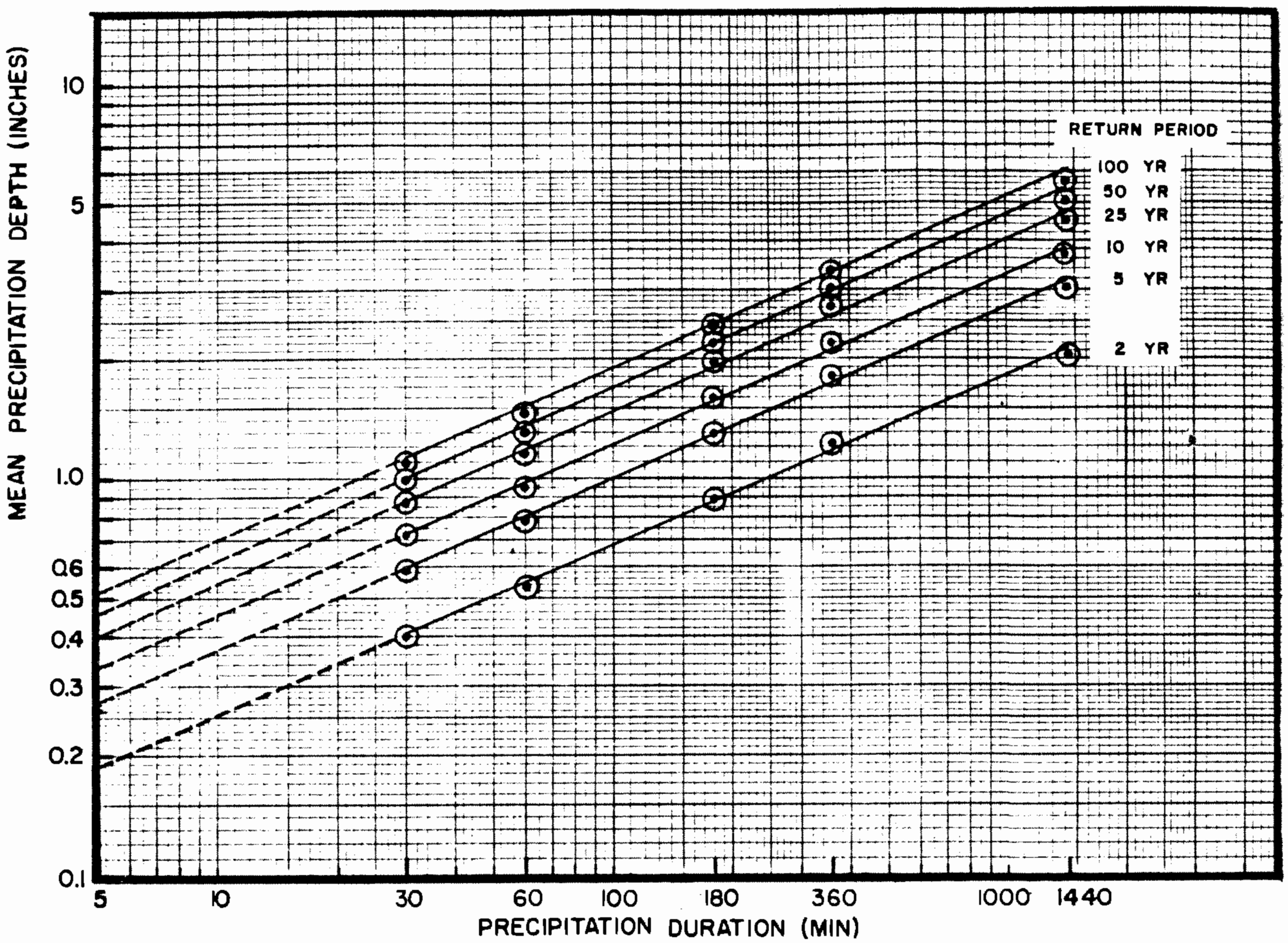
B.2. POINT PRECIPITATION

The depth-duration relationships for the non-mountainous areas of Orange County (i.e., elevations less than about 2,000 feet) are generalized into the logarithmic plots shown in Figure B-1. These design point precipitation plots are appropriate for design hydrology studies. Included in the figure are regression equations which relate mean precipitation depths to precipitation duration. For mountainous areas, the State of California DWR data for rain gage station 156 (Santiago Peak) shall be used (see Figure B-2).

Table B.1. contains the Long Duration-Depth-Frequency values for non-mountainous and mountainous watersheds.

Regression Equations: $D(t) = at^b$
(D= Depth in inches, t= duration in minutes)

Return Frequency (years)	a	b
2	0.095	0.426
5	0.131	0.438
10	0.170	0.427
25	0.200	0.434
50	0.225	0.434
100	0.259	0.427

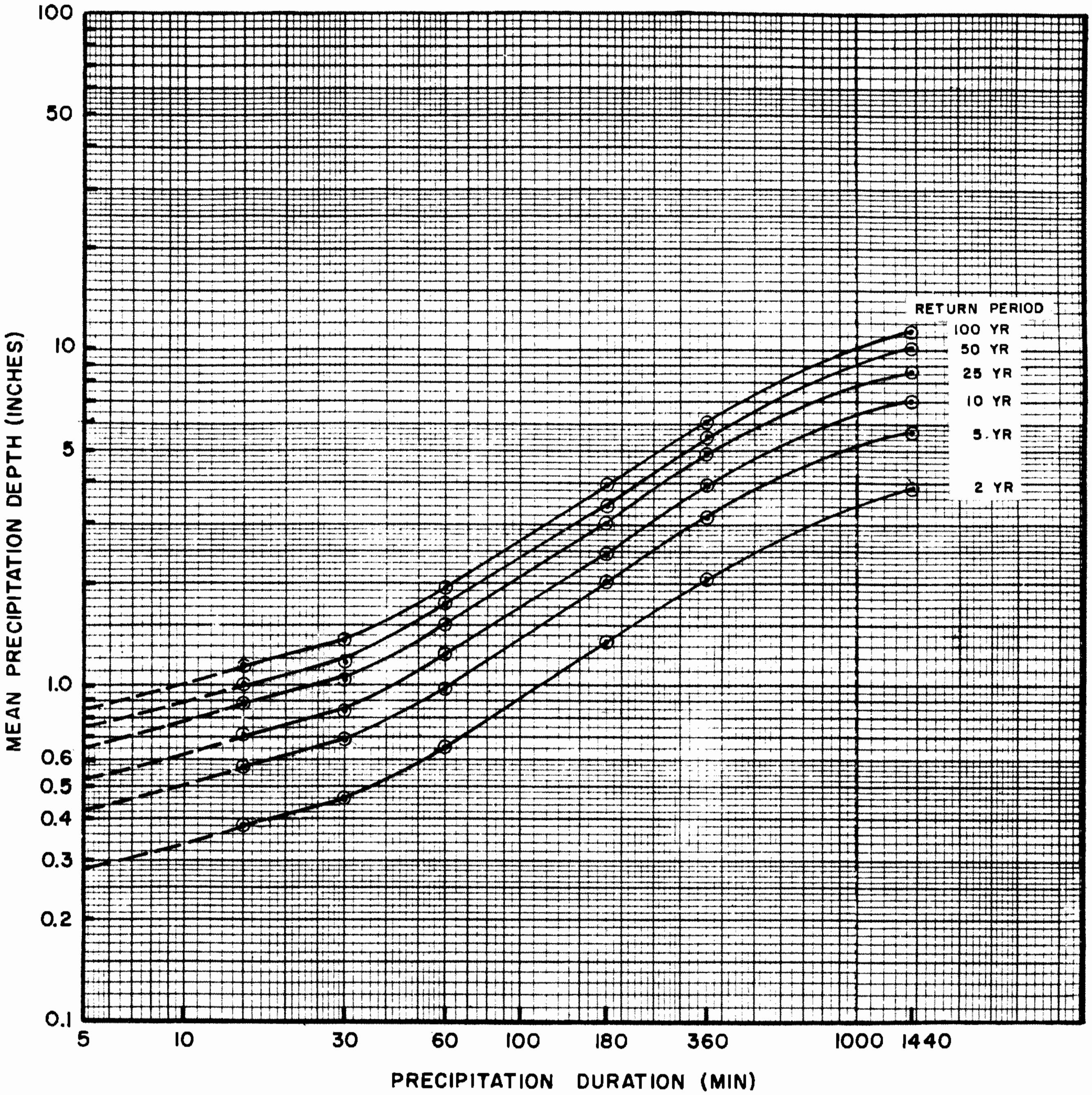


ORANGE COUNTY

HYDROLOGY MANUAL

MEAN PRECIPITATION DEPTHS FOR NONMOUNTAINOUS AREAS

Figure B-1



REF. NOVEMBER 1981 STATE OF CALIF. DWR SANTIAGO PEAK #RC156

ORANGE COUNTY

HYDROLOGY MANUAL

MEAN PRECIPITATION DEPTHS FOR MOUNTAINOUS AREA

Figure B-2

TABLE B.1.

MAXIMUM PRECIPITATION FOR INDICATED DURATION D-DAYS (INCHES)

BELOW 2000' ELEVATION

Return Period In Yrs.	1D	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	365D
2	2.05	2.76	3.08	3.21	3.36	3.61	3.94	4.24	4.73	5.21	6.20	8.44	13.60
5	3.03	4.24	4.79	5.01	5.23	5.59	6.05	6.47	7.20	7.83	9.18	12.69	19.13
10	3.68	5.23	5.92	6.22	6.50	6.94	7.44	7.94	8.79	9.49	11.07	15.48	22.56
20	4.31	6.17	6.99	7.38	7.71	8.22	8.74	9.31	10.26	11.02	12.80	18.08	25.69
25	4.49	6.46	7.33	7.75	8.09	8.63	9.15	9.74	10.72	11.49	13.34	18.90	26.66
40	4.89	7.06	8.03	8.50	8.88	9.47	9.98	10.62	10.95	12.46	14.44	20.58	28.63
50	5.07	7.35	8.35	8.86	9.25	9.86	10.38	11.03	12.11	12.91	14.95	21.37	29.55
100	5.63	8.22	9.35	9.95	10.38	11.07	11.57	12.29	13.45	14.28	16.51	23.77	32.32

ABOVE 2000' ELEVATION

Return Period In Yrs.	1D	2D	3D	4D	5D	6D	8D	10D	15D	20D	30D	60D	365D
2	3.81	5.33	5.89	6.22	6.66	7.17	7.88	8.38	8.97	9.62	11.29	15.91	26.05
5	5.71	8.25	9.23	9.75	10.40	11.12	12.17	12.81	13.72	14.51	16.73	23.74	36.88
10	7.05	10.26	11.58	12.23	12.98	13.80	15.02	15.71	16.83	17.66	20.17	28.69	43.86
20	8.36	12.20	13.85	14.63	15.45	16.35	17.72	18.42	19.74	20.59	23.33	33.25	50.33
25	8.76	12.81	14.58	15.40	16.24	17.16	18.57	19.27	20.65	21.50	24.31	34.66	52.35
40	9.62	14.08	16.08	16.99	17.87	18.82	20.32	21.02	22.53	21.95	26.32	37.56	53.33
50	10.02	14.68	16.79	17.74	18.63	19.61	21.14	21.84	23.41	24.25	27.25	38.91	58.43
100	11.27	16.52	18.98	20.05	20.99	22.01	23.65	24.33	26.09	26.91	30.09	42.99	64.30

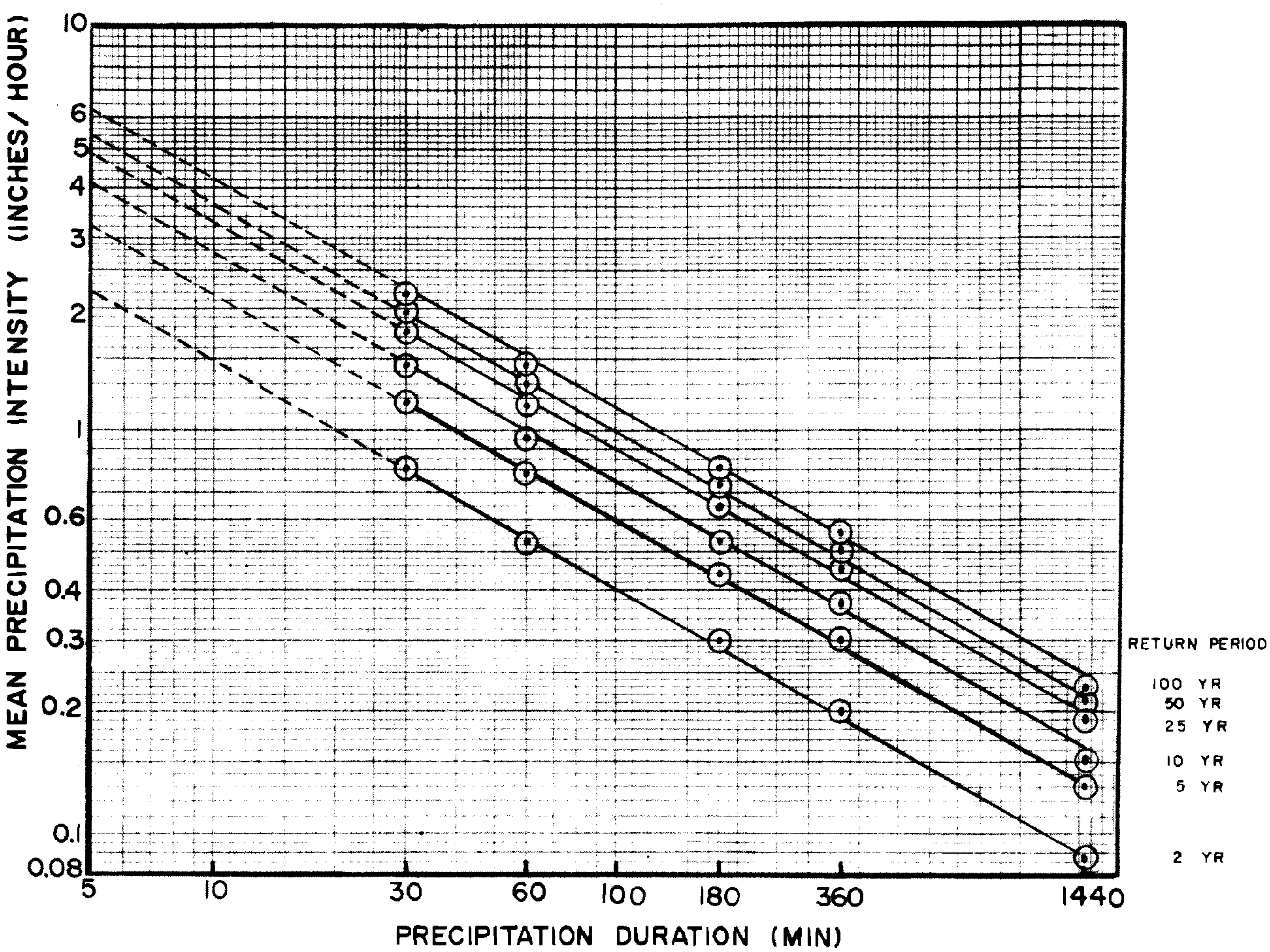
B.3. RATIONAL METHOD PRECIPITATION INTENSITY CURVES

For determining peak discharge by the rational method, which is presented in Section D, precipitation intensity rather than depth is an input value in the calculation. To obtain a plot of intensity versus duration, the curves in Figures B-1 and B-2 are converted by dividing precipitation depth by duration. The non-mountainous area precipitation intensity curves are presented in Figure B-3 and can be used for the rational method analysis of drainage areas below 2000 feet in elevation. For drainage areas above 2000 feet, the mountainous area precipitation intnesity curves in Figure B-4 can be used.

Regression Equations: $I(t) = at^b$

(I= Intensity in inches/hour, t= duration in minutes)

Return Frequency (years)	a	b
2	5.702	-0.574
5	7.870	-0.562
10	10.209	-0.573
25	11.995	-0.566
50	13.521	-0.566
100	15.560	-0.573



ORANGE COUNTY

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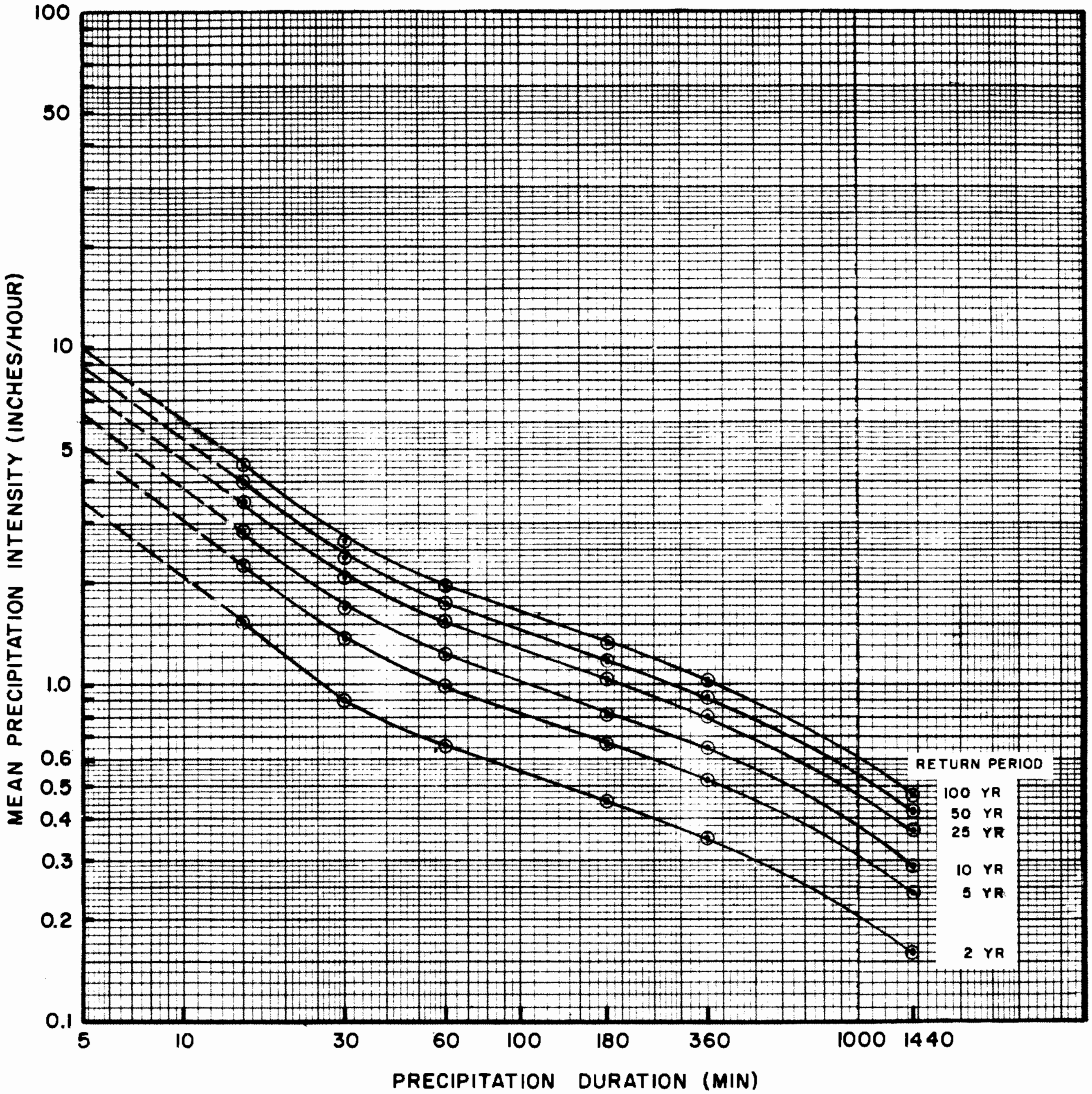
MEAN PRECIPITATION INTENSITIES FOR NONMOUNTAINOUS AREAS



ORANGE COUNTY

HYDROLOGY MANUAL

MEAN PRECIPITATION INTENSITIES FOR MOUNTAINOUS AREA

Figure B-4

B.4. UNIT HYDROGRAPH METHOD DESIGN STORM

The Orange County desgin storm shall be used for all unit hydrograph method calculations (Figures B-5a, b, c).

The point precipitation depths in Table B.2 shall be used for the single-day design storm.

For watersheds with detention basins, a multi-day storm shall be used as shown in Sections B.5 and B.6.

Due to the variations in point precipitation values between mountainous and nonmountainous areas, area averaging of rainfall is required when catchments include areas both above and below the 2,000-foot elevation.

TABLE B.2.
ORANGE COUNTY POINT PRECIPITATION DATA (inches)

	DURATION					
T-YR.	5M	30M	1H	3H	6H	24H
100	0.52(.78)	1.09(1.34)	1.45(1.94)	2.43(3.96)	3.36(6.19)	5.63(11.27)
50	0.45(.71)	0.98(1.19)	1.30(1.73)	2.19(3.52)	3.02(5.51)	5.07(10.02)
25	0.40(.63)	0.87(1.04)	1.15(1.51)	1.94(3.08)	2.71(4.81)	4.49(8.76)
10	0.34(.50)	0.72(.84)	0.95(1.22)	1.59(2.48)	2.20(3.87)	3.68(7.05)
5	0.26(.40)	0.59(.68)	0.78(.99)	1.31(2.01)	1.81(3.14)	3.03(5.71)
2	0.19(.26)	0.40(.45)	0.53(.66)	0.89(1.34)	1.22(2.09)	2.05(3.81)

NOTES:

(1.) Numbers in ( ) are from the Santiago Peak gage station #156, DWR depth-duration-frequency table (1983). Use in areas above 2,000 feet in elevation.

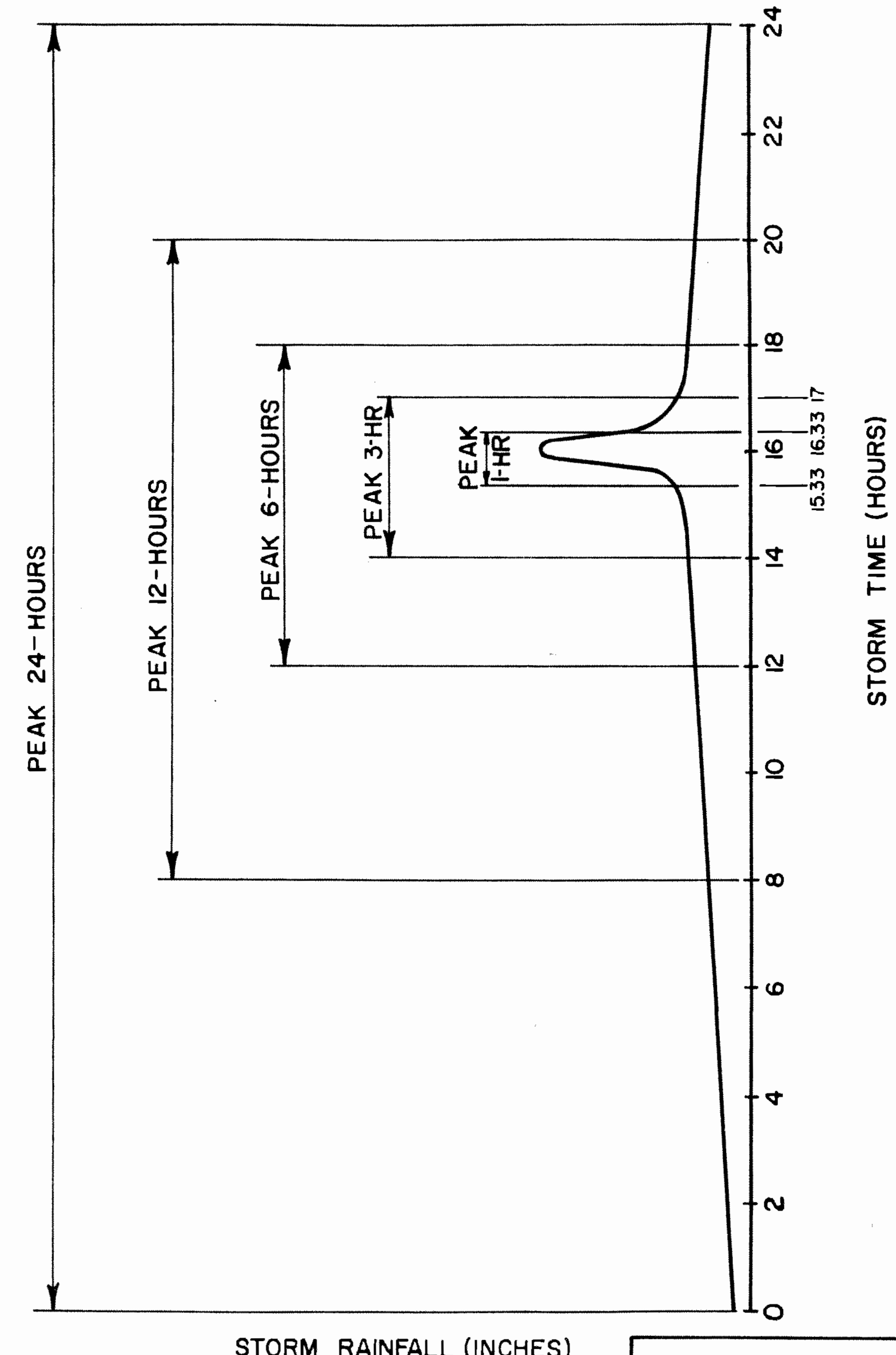
(2.) Precipitation data for nonmountainous areas taken from an average of 25 rain gages (see ref. 7). Use in areas below 2,000 feet in elevation.

(3.) All 5M values are extrapolations (see ref. 7).

(4.) M = minutes; H = hours.

Because the average rainfall intensity for a single storm event tends to decrease with respect to increased area, the point precipitation values in Table B.2 shall be reduced by the factors shown in Figure B-6. The catchment area shall be the total drainage area contributing runoff to the point of concentration where the design discharge is being calculated. For example, at a confluence, in order to provide peak discharges for the two tributaries and the downstream channel, three different sets of reduction factors are required: a separate analysis for each of the two tributaries, and another analysis for the summed area of the two tributaries at the confluence point.

After the point precipitation depths have been reduced based on Figure B-6, precipitation depths for each unit time interval (usually 5 minutes) can be determined graphically using Figure B-7. This procedure is demonstrated in the sample problem of Section E. After calculation of the unit time interval precipitation depths, the depths are arranged into the storm pattern shown in Figures B-5 a, b, c so that the peak unit interval occurs at two-thirds of the storm duration.

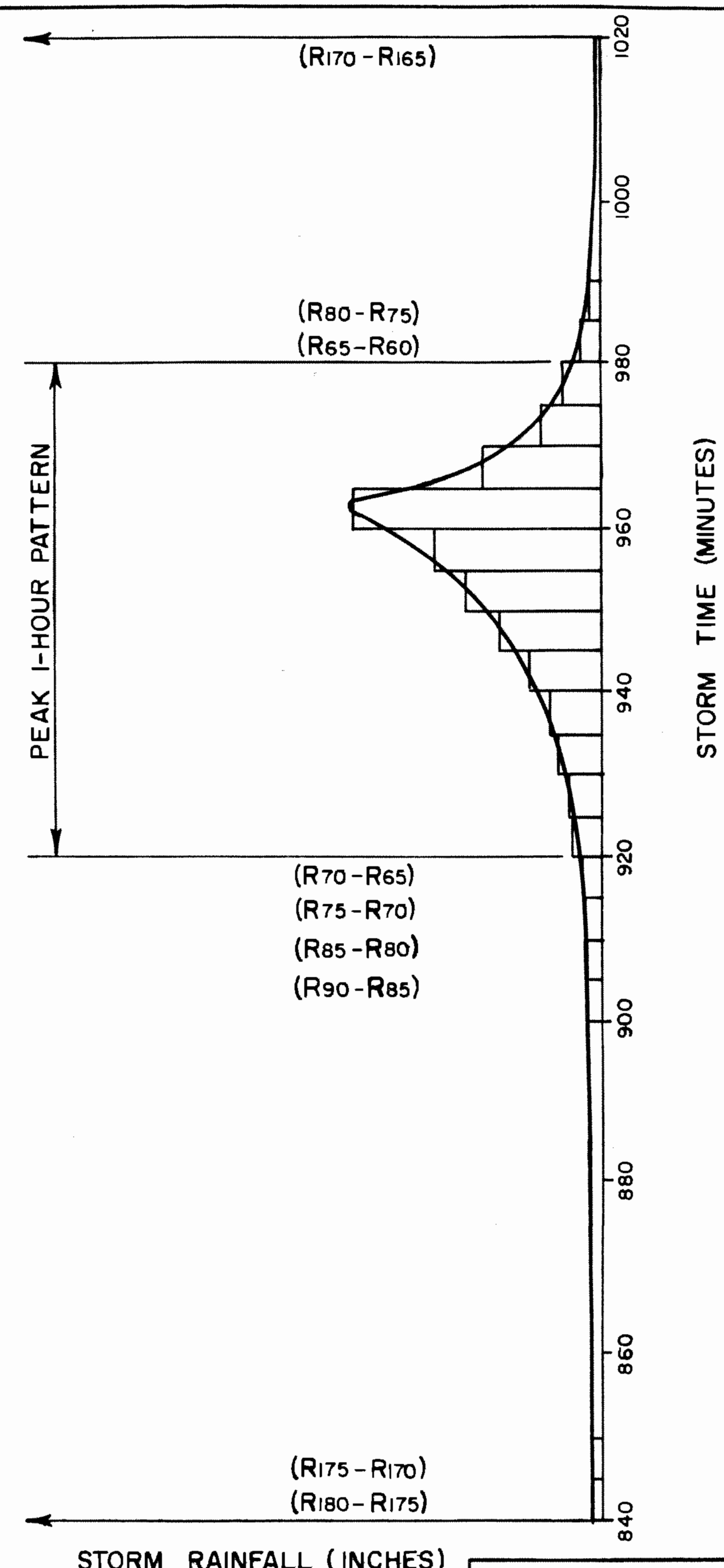


ORANGE COUNTY

HYDROLOGY MANUAL

DESIGN CRITICAL STORM PEAK 24-HOUR PATTERN

Figure B-5a

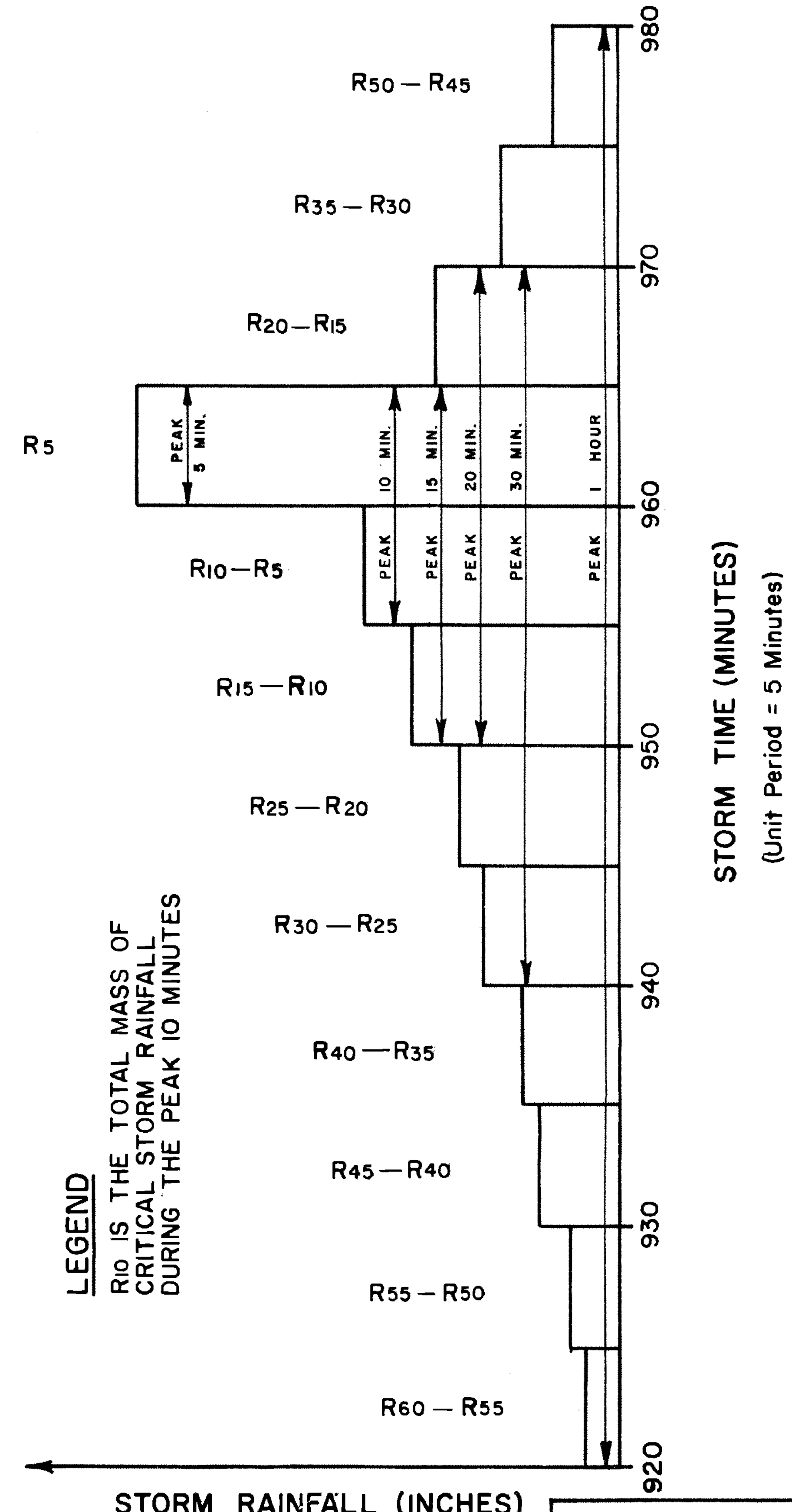


ORANGE COUNTY

HYDROLOGY MANUAL

DESIGN CRITICAL
STORM PEAK
3-HOUR PATTERN

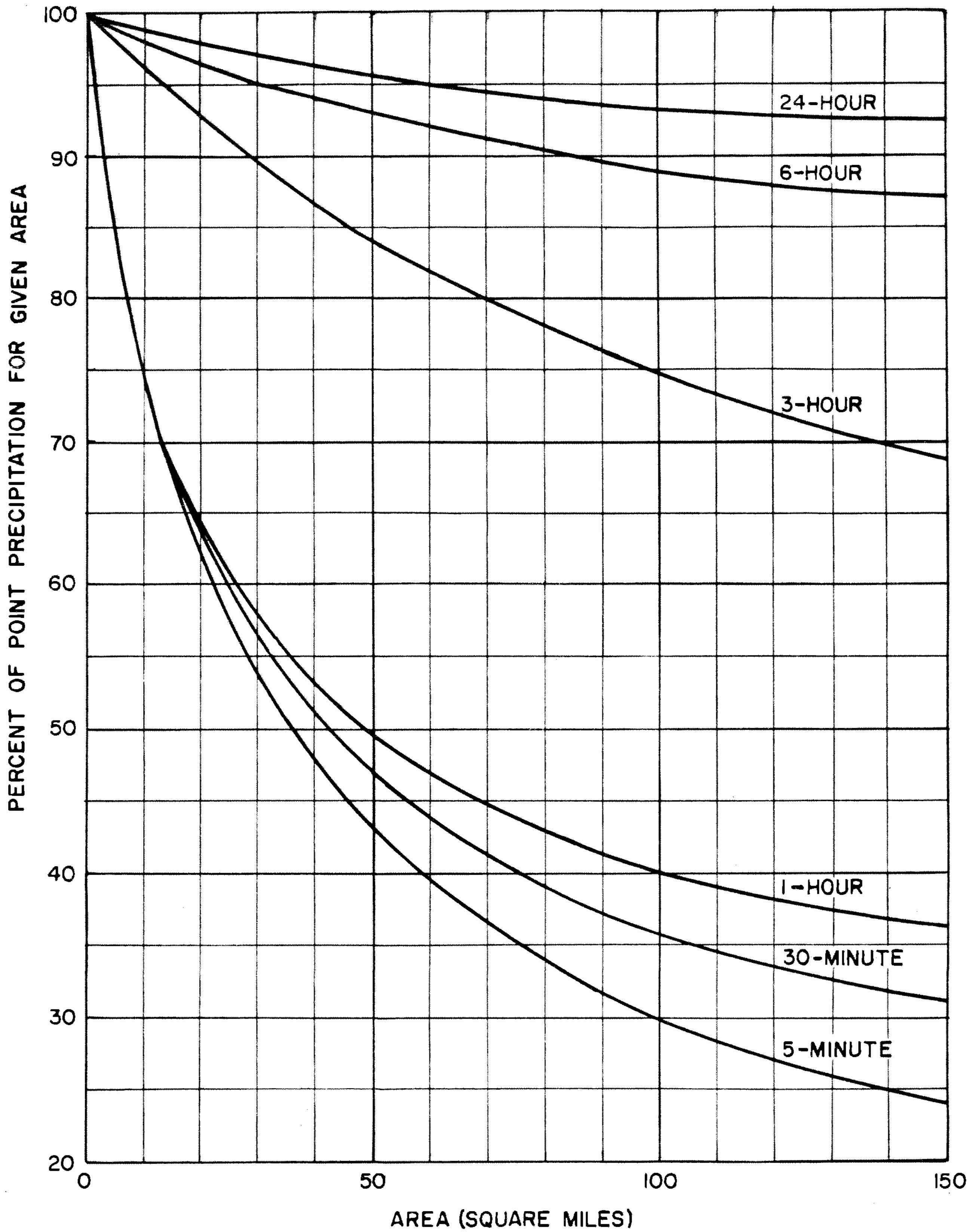
Figure B-5b



ORANGE COUNTY
HYDROLOGY MANUAL

DESIGN CRITICAL STORM PEAK 1-HOUR PATTERN

Figure B-5c



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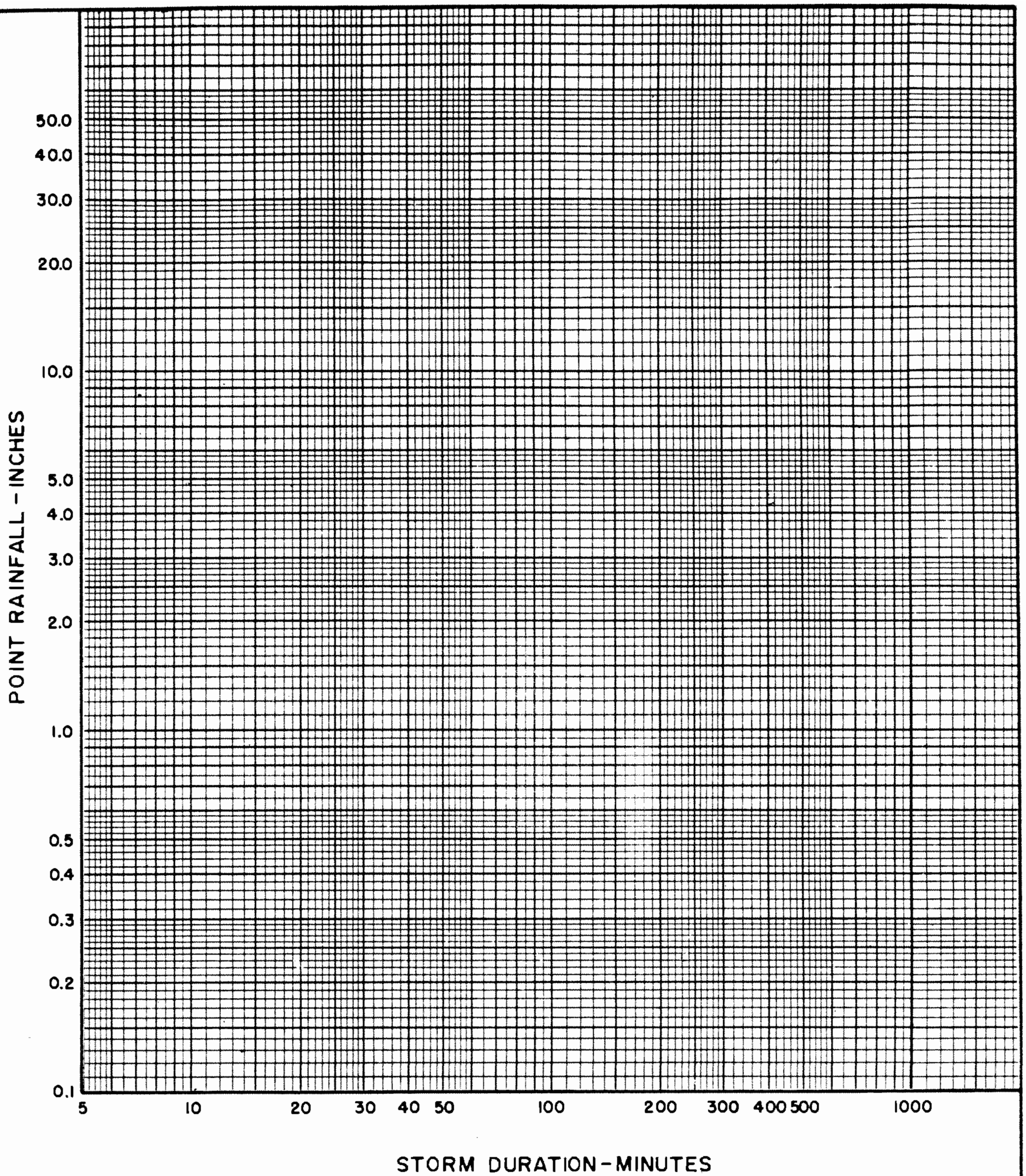
DESIGN STORM
DEPTH AREA
CURVES

Figure B-6



PROJECT LOCATION ______________________________

NOTES ______________________________

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ORANGE COUNTY

HYDROLOGY MANUAL

AREA - AVERAGED
MASS RAINFALL
PLOTTING SHEET

Figure B-7

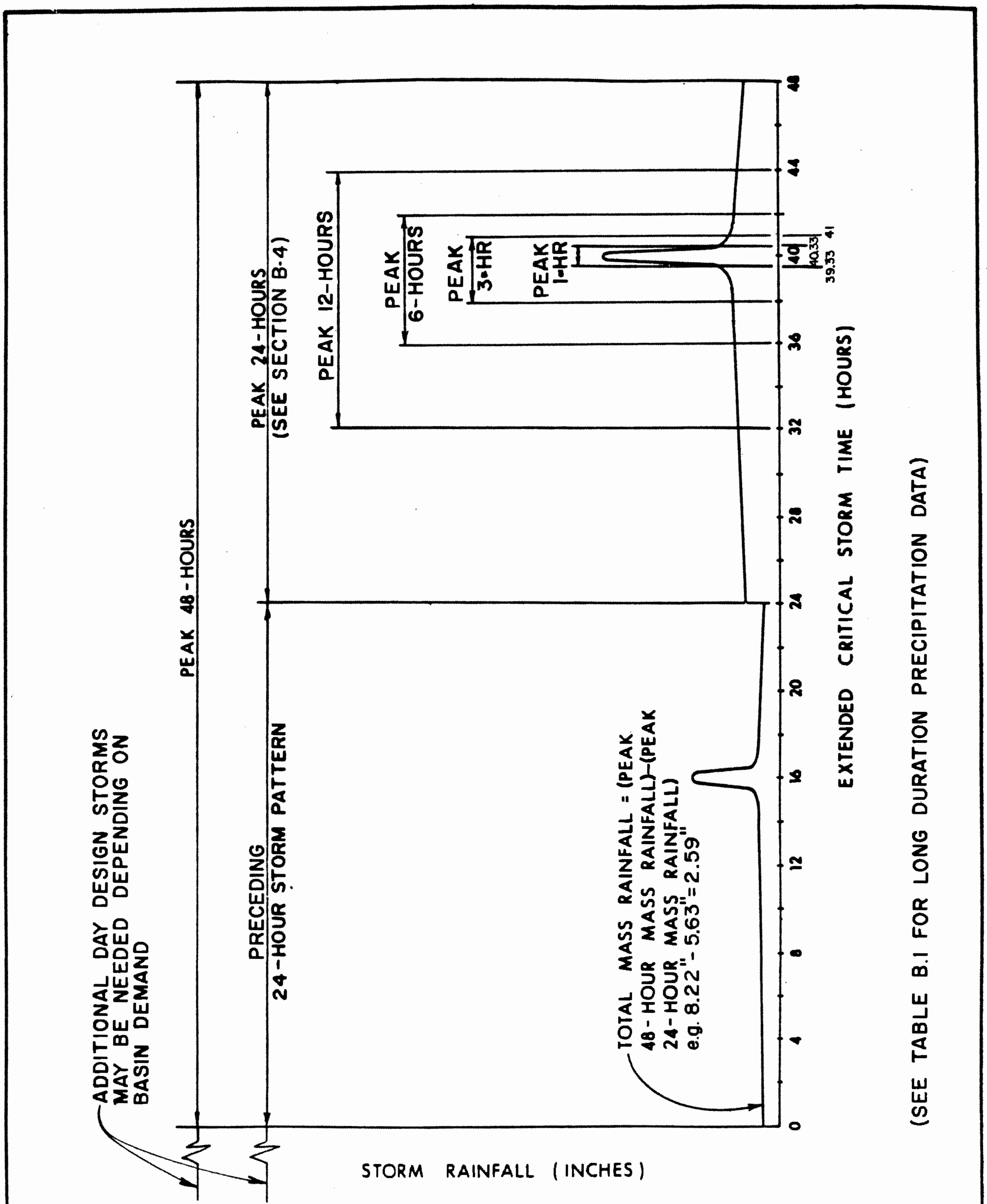
B.5. DESIGN STORM FOR WATERSHEDS WITH FLOW-THROUGH DETENTION BASINS

Due to the interaction of watershed size, Tc, percentage of peak discharge reduction, and basin volume, the critical storm duration is generally not known (in advance) for a watershed flood control system which includes one or several detention basins. Hence, the use of the 24-hour design storm may not be the "critical" storm for flow-through detention basin design purposes, and a longer duration design storm may be needed. Figure B-8 illustrates the extended design storm (multiday) for a two day duration. Longer duration design storms are developed in a similar fashion.

The multiday design storm utilizes the structure of Figure B-8 for all flow-through detention basin systems. Successive day storms are developed and added in the front of the previously developed design storm patterns until the detention basin system demonstrates no increase in the required basin volume due to the further extension of the design storm pattern. By increasing the basin outlet capacity, the critical duration can be reduced.

From Figure B-8 it is seen that the extended design storm is constructed from an arrangement of rainfalls of identical T-year return frequency. That is, even though a two day or longer duration multiday storm is being used to test the detention basin's level of flood protection, the extended design storm still contains no more than T-year rainfall depths for the extended duration. Each of the 24-hour storm patterns are constructed by a simple scaling of the peak 24-hour design pattern according to a ratio of the respective 24-hour precipitation values.

The point precipitation values for durations longer than one day may be taken from Table B.1.



ORANGE COUNTY

HYDROLOGY MANUAL

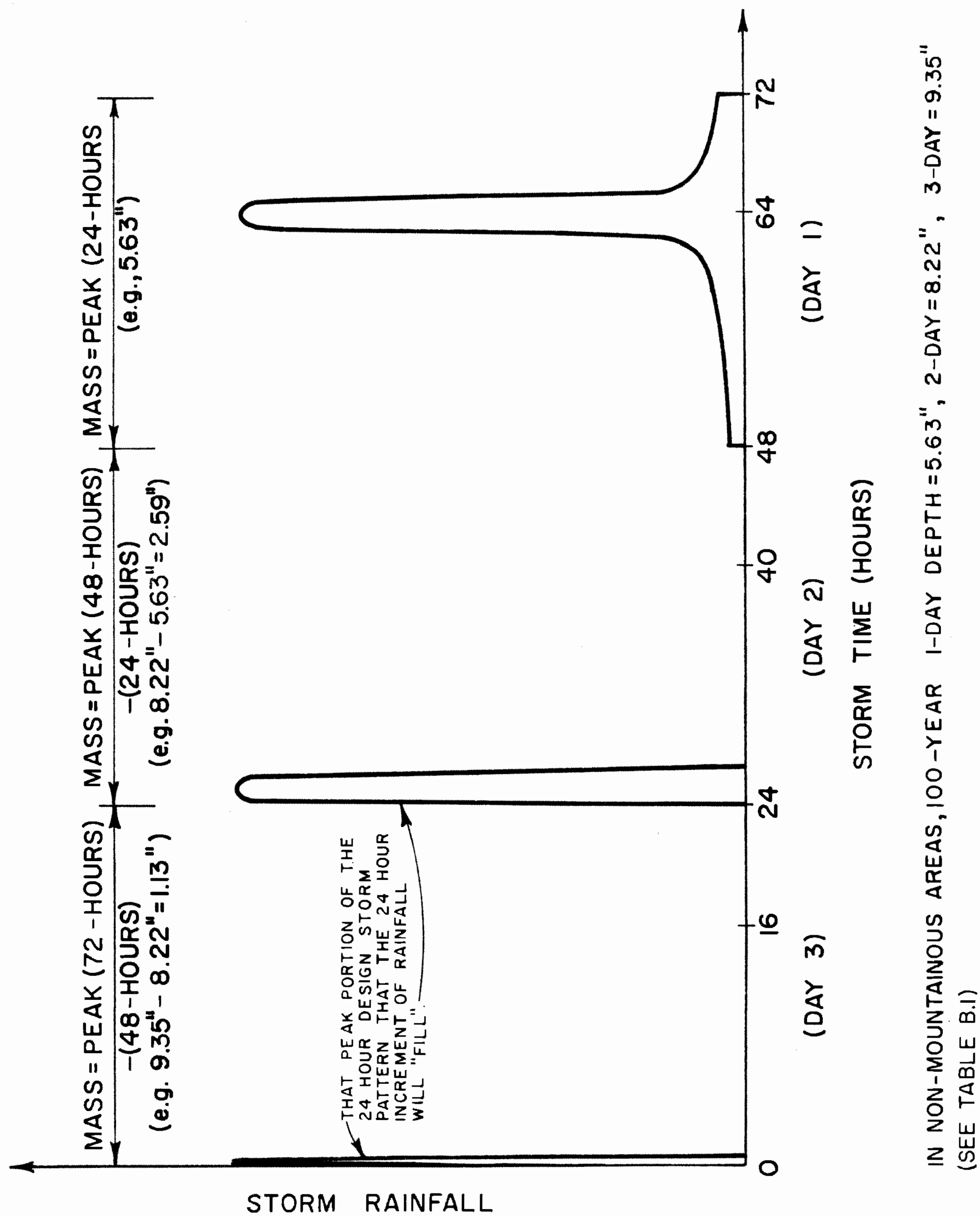
EXTENDED DESIGN STORM:
FLOW-THROUGH BASIN

Figure B-8

The multi-day design storm shall be reduced based on the Depth-Area Reduction curves shown in Figure B-6. As in the case of a watershed with no detention basins, the reduction area shall be the total drainage area contributing runoff to the point of concentration where the design discharge is being calculated. However, if a reduction area of a storm applied to the area downstream of the basin produces a higher discharge than the storm applied to the entire watershed, the Agency will require flood control facility design based on the higher discharge.

B.6. DESIGN STORM FOR WATERSHEDS WITH FLOW-BY DETENTION BASINS

For many of the same reasons cited in Section B.5, a single day storm may not be the critical storm for a flow-by basin. A slightly different multi-day storm configuration as shown in Figure B-9 shall be used for watersheds with flow-by basins. Point precipitation values from Tables B.1 and B.2 shall be used. The Depth-Area Reduction Curves (Figure B-6) shall be applied in the same manner as descibed in Section B.5.



ORANGE COUNTY
HYDROLOGY MANUAL

FLOW-BY BASIN CRITICAL STORM PATTERN (3-DAY EXAMPLE)

Figure B-9

SECTION C

LOSSES

C.1. WATERSHED LOSSES

Watershed outflow is a function of precipitation, watershed losses, and routing processes. Watershed routing processes are presented in Sections D and E where the rational and unit hydrograph methods are presented in detail. Precipitation estimation procedures and data are presented in Section B. This section will present watershed loss computation methods and data.

Watershed losses are considered to be depression storage, vegetation interception and transpiration, minor amounts of evaporation, and infiltration. Infiltration is the process of water entering the soil surface and percolating downward into the soil where it is stored during a precipitation event. Subsequently, the stored soil water may be consumptively used by vegetation, percolate further downward to groundwater storage, or exit the soil surface as seeps or springs. Seepage from stream bank storage is the primary source of baseflow which is derived from prior precipitation events. When making estimates of stormwater runoff it is assumed that infiltration is a loss for the storm event under consideration. For purposes of the hydrologic methods used in this manual, infiltration is expressed as a rate in inches per hour (refs. 2, 9, 10).

C.2. HYDROLOGIC SOIL GROUPS

The major factor affecting infiltration is the nature of the soil itself. The soil surface characteristics, its ability to transmit water to subsurface layers and total storage capacity are all major factors in the infiltration capabilities of a particular soil. Soils are classified into four hydrologic soil groups as follows (refs. 2, 3):

GROUP A: Low runoff potential. Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well-drained sands or gravels. These soils have a high rate of water transmission.

GROUP B: Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained sandy-loam soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

GROUP C: Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of silty-loam soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

GROUP D: High runoff potential. Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

C.2.1. Infiltration Rates

Soil infiltration rates have been estimated for each of the soil groups by laboratory studies and measurements. These measurements show that an initially dry soil will have an associated infiltration rate which essentially decreases with time as the soil becomes wetted. As the soil is subjected to continual rainfall, this infiltration rate approaches a minimum infiltration rate which represents the percolation rate of the saturated soil.

C.2.2. Soil Maps

Maps have been prepared which designate the locations of the various soil groups within the County of Orange (see Figure C-1) and are contained in the pocket in the back of this manual.

C.3. SOIL COVER AND HYDROLOGIC CONDITIONS

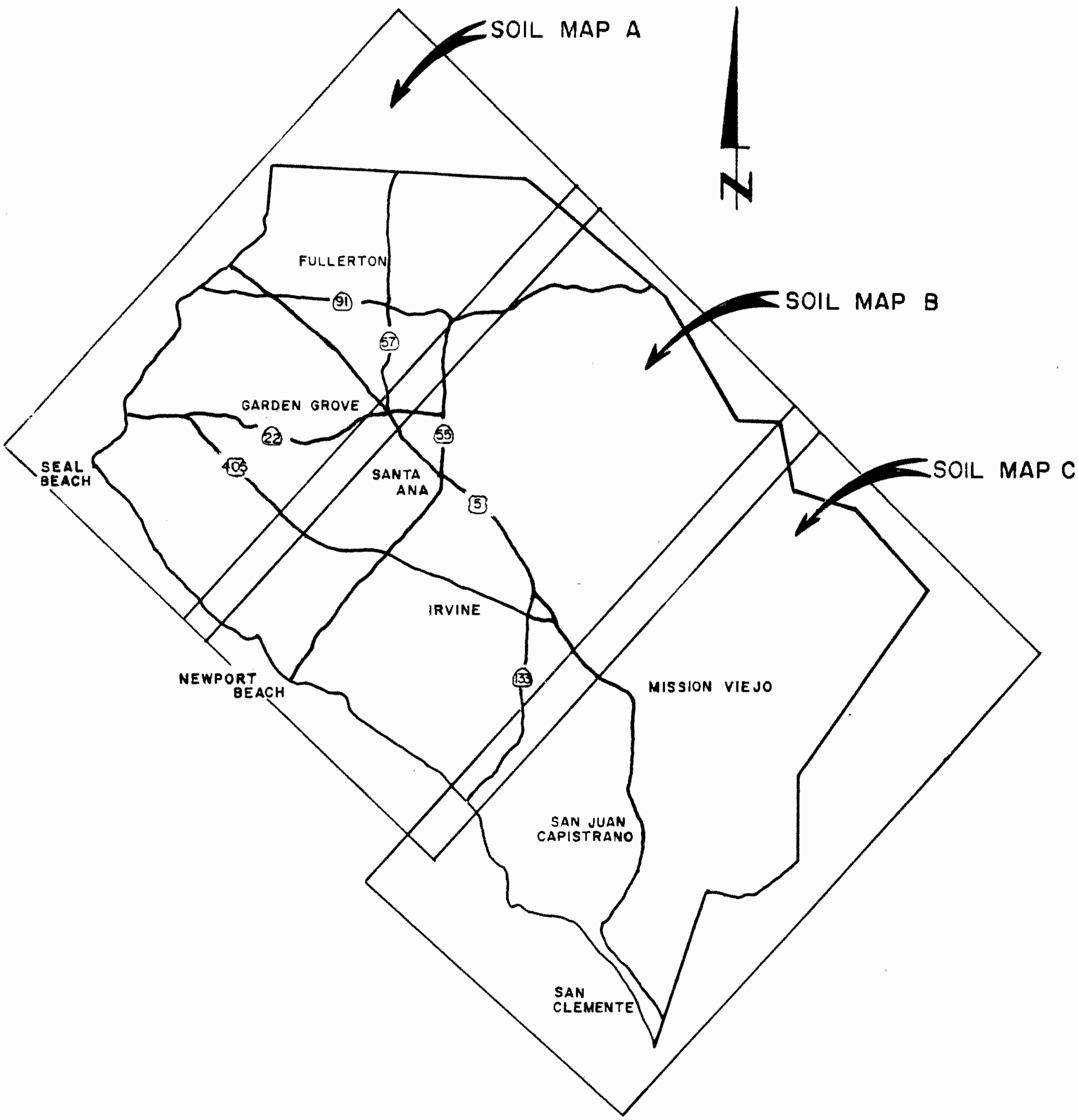
The type of vegetation or ground cover on a watershed, and the quality or density of that cover, have a major impact on the infiltration capacity of a given soil. Definitions of specific cover types are provided in Figure C-2. Further refinement in the cover type descriptions is provided by the definition of cover quality as follows:

POOR: Heavily grazed or regularly burned areas. Less than 50 percent of the ground surface is protected by plant cover or brush and tree canopy.

FAIR: Moderate cover with 50 percent to 75 percent of the ground surface protected by vegetation.

GOOD: Heavy or dense cover with more than 75 percent of the ground surface protected by vegetation.

In most cases, watershed existing conditions cover type and quality can be readily determined by a field review of a watershed. In ultimate planned open spaces, the soil cover condition shall be considered as "good." Curve Number (CN) is one measure of runoff potential for a particular soil group and cover complex conditions. Figure C-3 provides the CN values for various types and quality of ground cover. Impervious areas shall be assigned a CN of 98. It is noted that for ultimately developed conditions, the CN for urban landscaping (turf) is provided in Figure C-3.



(SEE SOIL MAPS IN POCKET)

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SOIL MAP INDEX

Figure C-1

Residential Landscaping (Lawn, Shrubs, etc.) - The pervious portions of commercial establishments, single and multiple family dwellings, trailer parks and schools where the predominant land cover is lawn, shrubbery and trees.

Row Crops - Lettuce, tomatoes, beets, tulips or any field crop planted in rows far enough apart that most of the soil surface is exposed to rainfall impact throughout the growing season. At plowing, planting and harvest times it is equivalent to fallow.

Small Grain - Wheat, oats, barley, flax, etc. planted in rows close enough that the soil surface is not exposed except during planting and shortly thereafter.

Legumes - Alfalfa, sweetclover, timothy, etc. and combinations are either planted in close rows or broadcast.

Fallow - Fallow land is land plowed but not yet seeded or tilled.

Woodland - grass - Areas with an open cover of broadleaf or coniferous trees usually live oak and pines, with the intervening ground space occupied by annual grasses or weeds. The trees may occur singly or in small clumps. Canopy density, the amount of ground surface shaded at high noon, is from 20 to 50 percent.

Woodland - Areas on which coniferous or broadleaf trees predominate. The canopy density is at least 50 percent. Open areas may have a cover of annual or perennial grasses or of brush. Herbaceous plant cover under the trees is usually sparse because of leaf or needle litter accumulation.

Chaparral - Land on which the principal vegetation consists of evergreen shrubs with broad, hard, stiff leaves such as manzonita, ceanothus and scrub oak. The brush cover is usually dense or moderately dense. Diffusely branched evergreen shrubs with fine needle-like leaves, such as chamise and redchank, with dense high growth are also included in this soil cover.

Annual Grass - Land on which the principal vegetation consists of annual grasses and weeds such as annual bromes, wild barley, soft chess, ryegrass and filaree.

Irrigated Pasture - Irrigated land planted to perennial grasses and legumes for production of forage and which is cultivated only to establish or renew the stand of plants. Dry land pasture is considered as annual grass.

Meadow - Land areas with seasonally high water table, locally called cienegas. Principal vegetation consists of sod-forming grasses interspersed with other plants.

Orchard (Deciduous) - Land planted to such deciduous trees as apples, apricots, pears, walnuts, and almonds.

Orchard (Evergreen) - Land planted to evergreen trees which include citrus and avocados and coniferous plantings.

Turf - Golf courses, parks and similar lands where the predominant cover is irrigated mowed close-grown turf grass. Parks in which trees are dense may be classified as woodland.

SOIL COVER
TYPE DEFINITIONS

Figure C-2

Curve Numbers of Hydrologic Soil-Cover Complexes For Pervious Areas-AMC II

Cover Type (3)	Quality of Cover (2)	Soil Group A	B	C	D
NATURAL COVERS -					
Barren (Rockland, eroded and graded land)		78	86	91	93
Chaparral, Broadleaf (Manzonita, ceanothus and scrub oak)	Poor	53	70	80	85
	Fair	40	63	75	81
	Good	31	57	71	78
Chaparral, Narrowleaf (Chamise and redshank)	Poor	71	82	88	91
	Fair	55	72	81	86
Grass, Annual or Perennial	Poor	67	78	86	89
	Fair	50	69	79	84
	Good	38	61	74	80
Meadows or Cienegas (Areas with seasonally high water table, principal vegetation is sod forming grass)	Poor	63	77	85	88
	Fair	51	70	80	84
	Good	30	58	71	78
Open Brush (Soft wood shrubs - buckwheat, sage, etc.)	Poor	62	76	84	88
	Fair	46	66	77	83
	Good	41	63	75	81
Woodland (Coniferous or broadleaf trees predominate. Canopy density is at least 50 percent.)	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	25	55	70	77
Woodland, Grass (Coniferous or broadleaf trees with canopy density from 20 to 50 percent)	Poor	57	73	82	86
	Fair	44	65	77	82
	Good	33	58	72	79
URBAN COVERS -					
Residential or Commercial Landscaping (Lawn, shrubs, etc.)	Good	32	56	69	75
Turf (Irrigated and mowed grass)	Poor	58	74	83	87
	Fair	44	65	77	82
	Good	33	58	72	79
AGRICULTURAL COVERS -					
Fallow (Land plowed but not tilled or seeded)		77	86	91	94

ORANGE COUNTY

HYDROLOGY MANUAL

CURVE NUMBERS FOR PERVIOUS AREAS

Cover Type (3)	Quality of Cover (2)	Soil Group A	B	C	D
AGRICULTURAL COVERS (Continued)					
Legumes, Close Seeded	Poor	66	77	85	89
(Alfalfa, sweetclover, timothy, etc.)	Good	58	72	81	85
Orchards, Evergreen	Poor	57	73	82	86
(Citrus, avocados, etc.)	Fair	44	65	77	82
	Good	33	58	72	79
Pasture, Dryland	Poor	68	79	86	89
(Annual grasses)	Fair	49	69	79	84
	Good	39	61	74	80
Pasture, Irrigated	Poor	58	74	83	87
(Legumes and perennial grass)	Fair	44	65	77	82
	Good	33	58	72	79
Row Crops	Poor	72	81	88	91
(Field crops - tomatoes, sugar beets, etc.)	Good	67	78	85	89
Small grain	Poor	65	76	84	88
(Wheat, oats, barley, etc.)	Good	63	75	83	87

Curve Numbers of Hydrologic Soil-Cover Complexes For Pervious Areas-AMC II

Notes:

1. All curve numbers are for Antecedent Moisture Condition (AMC) II.

2. Quality of cover definitions:

   Poor-Heavily grazed, regularly burned areas, or areas of high burn potential. Less than 50 percent of the ground surface is protected by plant cover or brush and tree canopy.

   Fair-Moderate cover with 50 percent to 75 percent of the ground surface protected.

   Good-Heavy or dense cover with more than 75 percent of the ground surface protected.

3. See figure C-2 for definition of cover types.

4. Impervious areas are assigned curve number 98.

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HYDROLOGY MANUAL

CURVE NUMBERS FOR PERVIOUS AREAS

C.4. WATERSHED DEVELOPMENT CONDITIONS

Ultimate development of the watershed should normally be assumed since watershed urbanization is reasonably likely within the expected life of most hydraulic facilities. Long range master plans for the County and incorporated cities should be reviewed to insure that reasonable land use assumptions are made for the ultimate development of the watershed. A field review shall also be made to confirm existing use and drainage patterns. Particular attention shall be paid to existing and proposed landscape practices, as it is common in some areas to use ornamental gravels underlain by impervious plastic materials in place of lawns and shrubs. Appropriate actual impervious percentages can then be selected from Figure C-4. It should be noted that the recommended values from these figures are for average conditions and, therefore, some adjustment for particular applications may be required.

C.5. ANTECEDENT MOISTURE CONDITION (AMC)

The definitions for the AMC classifications are:

AMC I: Lowest runoff potential. The watershed soils are dry enough to allow satisfactory grading or cultivation to take place.

AMC II: Moderate runoff potential; an average study condition.

AMC III: Highest runoff potential. The watershed is practically saturated from antecedent rains. Heavy rainfall or light rainfall and low temperatures have occurred within the last five days.

In the rainfall based hydrology methods it is normally assumed that a low AMC index (high loss rates) will be used in developing short return period storms (2-5 years), and a moderate to high AMC index (low loss rates) will be used in developing longer return period storms (10-100 year). For the

ACTUAL IMPERVIOUS COVER

Land Use (1)	Range-Percent	Recommended Value For Average Conditions-Percent (2)
Natural or Agriculture	0 - 0	0
Public Park	10 - 25	15
School	30 - 50	40
Single Family Residential: (3)		
2.5 acre lots	5 - 15	10
1 acre lots	10 - 25	20
2 dwellings/acre	20 - 40	30
3-4 dwellings/acre	30 - 50	40
5-7 dwellings/acre	35 - 55	50
8-10 dwellings/acre	50 - 70	60
More than 10 dwellings/acre	65 - 90	80
Multiple Family Residential:		
Condominiums	45 - 70	65
Apartments	65 - 90	80
Mobile Home Park	60 - 85	75
Commercial, Downtown Business or Industrial	80 - 100	90

Notes:

1. Land use should be based on ultimate development of the watershed. Long range master plans for the County and incorporated cities should be reviewed to insure reasonable land use assumptions.

2. Recommended values are based on average conditions which may not apply to a particular study area. The percentage impervious may vary greatly even on comparable sized lots due to differences in dwelling size, improvements, etc. Landscape practices should also be considered as it is common in some areas to use ornamental gravels underlain by impervious plastic materials in place of lawns and shrubs. A field investigation of a study area shall always be made, and a review of aerial photos, where available, may assist in estimating the percentage of impervious cover in developed areas.

3. For typical equestrian subdivisions increase impervious area 5 percent over the values recommended in the table above.

ORANGE COUNTY

HYDROLOGY MANUAL

ACTUAL IMPERVIOUS COVER FOR DEVELOPED AREAS

Figure C-4

purposes of design hydrology, AMC I will be used for the 2- and 5-year storm events. The watershed condition AMC II will be used for the 10-, 25-, and 50-year return frequency storms. For the case of the 100-year return frequency design storm, AMC III will be used.

C.5.1. Adjustment of Curve Numbers (CN) for AMC

The CN values selected for a particular soil cover type and quality also depend upon the AMC condition assumed. The CN values listed in Figure C-3 correspond to AMC II and require adjustment in order to represent either AMC I or AMC III. Table C.1 provides the necessary CN adjustments to account for AMC.

TABLE C.1. CURVE NUMBER RELATIONSHIPS

CN for AMC Condition II	Corresponding CN for AMC Condition	
	I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

C.6. ESTIMATION OF LOSS RATES

In estimating infiltration rates for design hydrology, a watershed curve number (CN) is determined for each soil-cover complex within the watershed using Figure C-3. The CN scale has a range of 0 to 98, where a low CN indicates low runoff potential (high infiltration), and a high CN indicates high runoff potential (low infiltration). Selection of a CN takes into account the major factors affecting infiltration on pervious surfaces including the hydrologic soil group, cover type and quality, and antecedent moisture condition (AMC).

Also included in the CN selection are the effects of "initial abstraction" (Ia) which represents the combined effects of other effective rainfall losses including depression storage, vegetation interception, evaporation, and transpiration, among other factors.

C.6.1. Estimation of Initial Abstraction (Ia)

The initial abstraction (Ia) for an area is a function of land use, treatment, and condition; interception; infiltration; depression storage; and antecedent soil moisture. An estimate for Ia is given by the SCS as

$$Ia = 0.2S \tag{C.1}$$

where S is an estimate of total soil capacity given by

$$S = \frac{1000}{CN} - 10 \tag{C.2}$$

where CN is the area curve number.

C.6.2. Estimation of Storm Runoff Yield

Given the CN for a subarea A_j, the corresponding 24-hour storm runoff yield fraction, Y_j, is estimated by

$$Y_j = \frac{(P_{24} - Ia)^2}{(P_{24} - Ia + S)P_{24}} \tag{C.3}$$

where

Y_j	=	24-hour storm runoff yield fraction for subarea A_j
P_{24}	=	24-hour storm rainfall
Ia	=	initial abstraction from (C.1)
S	=	see (C.2)

It is noted that should Ia be greater than P_{24} in (C.3), then Y_j is defined to be zero. In this manual, the notation Y and Y_j will represent the yield fraction rather than the volume of runoff.

If the area under study contains several (say m) CN designations, then the yield, Y, for the total area must represent the net effect of the several curve numbers. By weighting each of the subarea yield values according to the respective areas,

$$Y = (Y_1 A_1 + \cdots + Y_m A_m)/(A_1 + A_2 + \cdots + A_m) \tag{C.4}$$

where each Y_j follows from (C.3).

C.6.3. Low Loss Rate, F*

In design storm runoff hydrograph studies, the following formula is used to estimate that portion of rainfall to be attributed to watershed losses:

$$\bar{Y} = 1 - Y \tag{C.5}$$

where

$\bar{Y}$	=	catchment low loss fraction
Y	=	catchment 24-hour storm runoff yield fraction computed from (C.4)

Using the low loss fraction, $\bar{Y}$, the corresponding low loss rate, F*, is given by

$$F^* = \bar{Y} \cdot I \tag{C.6}$$

where I is the rainfall intensity and F* has units of inches/hour.

C.6.4. Estimation of Maximum Loss Rates for Pervious Areas, F_p

Table C.2 lists the maximum loss rates (inch/hour), F_p, for pervious area as a function of soil group.

TABLE C.2.

MAXIMUM EFFECTIVE PERVIOUS AREA LOSS RATES (inch/hour), F_p

SOIL GROUP:	**A**	**B**	**C**	**D**
F_p:	0.40	0.30	0.25	0.20

Table C.2 reflects the model calibration assuming an F_p of 0.30 in/hr. for all the considered catchments and storm return frequencies. This mean value of F_p of 0.30 in/hr. was assigned to Hydrologic Soil Group B due to the actual average soil conditions in the reconstitution study areas. The F_p values for Hydrologic Soil Groups A, C, and D, were assigned to account for the different soil types that may be found in Orange County.

C.6.5. Estimation of Catchment Maximum Loss Rates, F_m

The maximum loss rate selected from Table C.2 applies to the pervious area fraction of the watershed. The loss rate assumed for an impervious surface is 0.0 inch/hour. The maximum loss rate, F_m, for a catchment is therefore given by

$$F_m = a_p F_p \tag{C.7}$$

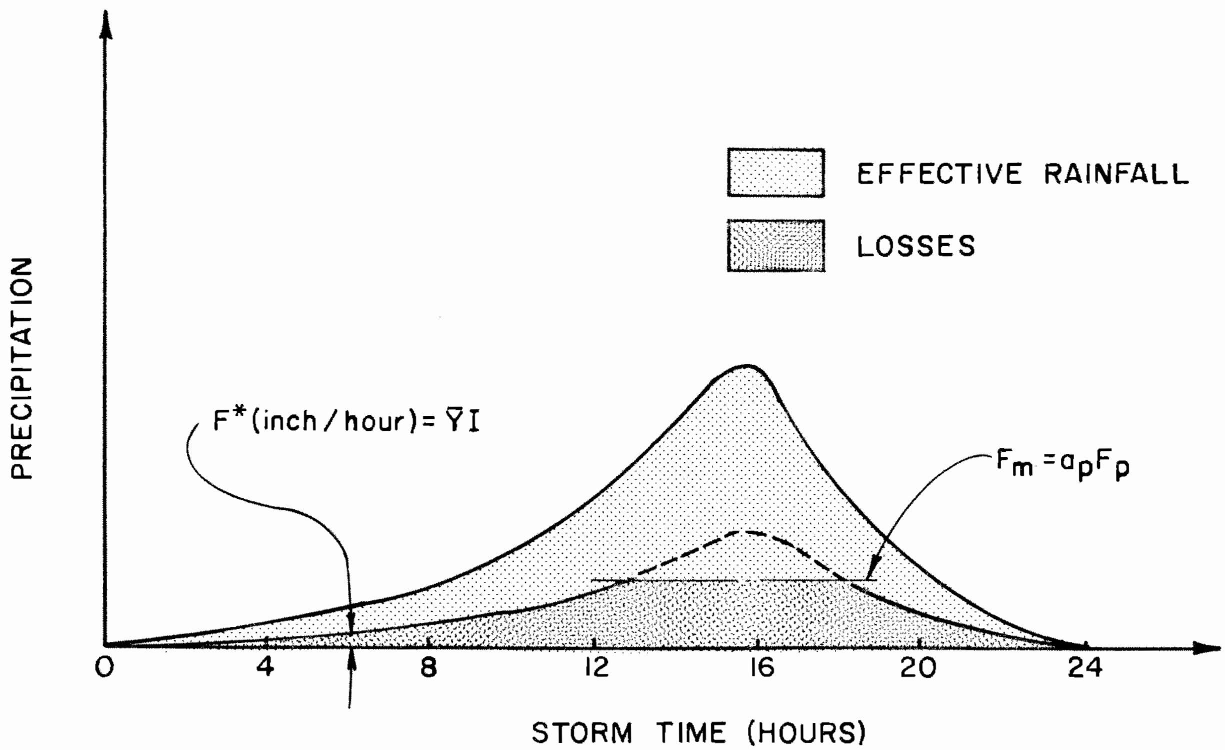
where a_p is the pervious area fraction and F_p is the maximum loss rate for the pervious area (Section C.6.4).

Should a catchment contain several F_m values, the composite F_m value is determined as a simple area average of the several F_m values.

C.6.6. Design Storm Loss Rates

In design storm runoff hydrograph studies, a 24-hour duration storm pattern is used to develop the time distribution of effective rainfall over the watershed. The effective rainfall quantities are determined by subtracting the watershed losses from the design storm rainfall.

The loss rate used for a particular catchment is a combination of the maximum loss rate F_m and the low loss rate F^*. F^* is used as the loss rate unless F^* exceeds F_m, in which case F_m is used as the loss rate. Typically in 100-year storm studies, F^* serves as the loss rate for the entire storm pattern except for the most intense rainfalls where F_m would apply. However for lower frequency storm studies such as the 5-year return event, F^* often applies for the entire 24-hour storm pattern. The example problem of section E provides an illustration in the use of F^* and F_m values. Figure C-5 illustrates the loss rate function used with the design storm.



ORANGE COUNTY
HYDROLOGY MANUAL

DESIGN STORM
LOSS FUNCTION

Figure C-5

SECTION D

RATIONAL METHOD

D.1. RATIONAL METHOD EQUATION

The rational method was originally developed to estimate runoff from small (less then one square mile) urban and developed areas and its use shall be limited to those conditions. Basically, the rational method equation relates rainfall intensity, a runoff coefficient, and drainage area size to the direct peak runoff from the drainage area. This relationship is expressed by the equation:

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

where

Q	=	the runoff in cubic feet per second (cfs) from a given area
C	=	a runoff coefficient representing the ratio of runoff to rainfall
I	=	the time-averaged rainfall intensity in inches per hour corresponding to the time of concentration
A	=	drainage area (acres)

The values of the runoff coefficient (C) and the rainfall intensity (I) are based on a study of drainage area characteristics such as type and condition of the runoff surfaces and the time of concentration. These factors and the limitations of the rational method equation are discussed in the following sections. Drainage area (A) may be determined by planimetering a suitable topographic map of the project area.

Data required for the computation of peak discharge by the rational method are: (i) rainfall intensity (I) for a storm of specified duration and selected design frequency; (ii) drainage area characteristics of size (A), shape, slope; and (iii) a runoff coefficient (C).

D.2. LIMITATIONS OF THE RATIONAL METHOD

The validity of the relationship expressed by the rational method equation holds true only if certain assumptions are reasonably correct and limitations of the method are observed. Two basic assumptions are that (i) the frequency of a storm runoff is the same as the frequency of the rainfall producing this runoff; i.e., a 25-year recurrence interval rainfall will provide a 25-year recurrence interval storm runoff, and (ii) that the peak runoff occurs when all parts of the drainage area are contributing to the runoff. The use of the rational method equation is limited to watersheds of size less than 640 acres.

The rational method equation is only applicable where the rainfall intensity (I) can be assumed to be uniformly distributed over the drainage area at a uniform rate throughout the duration of the storm. This assumption applies fairly well to small areas of less than 640 acres. Beyond this limit, the rainfall distribution may vary considerably from the point values given in rainfall isohyetal maps and the rational method equation should not be used.

The selection of the runoff coefficient (C) is another major limitation for the rational method equation. For small urban and developed areas the runoff coefficient can be reasonably well estimated from field and aerial photo studies. For larger areas where the determination of the runoff coefficient is to be based on vegetation type, cover density, the infiltration capacity of the ground surface, and the slope of the drainage area, an estimate of the runoff coefficient may be subject to a much greater error due to the variability of the drainage area characteristics. Rainfall losses due to evaporation, transpiration, depression and channel storage are inadequately evaluated, and may appreciably affect the estimate of the watershed peak rate of runoff. The effects of depth-area-duration (or depth-area) factors are not accounted for in the simple intensity-duration curve used for rational method studies.

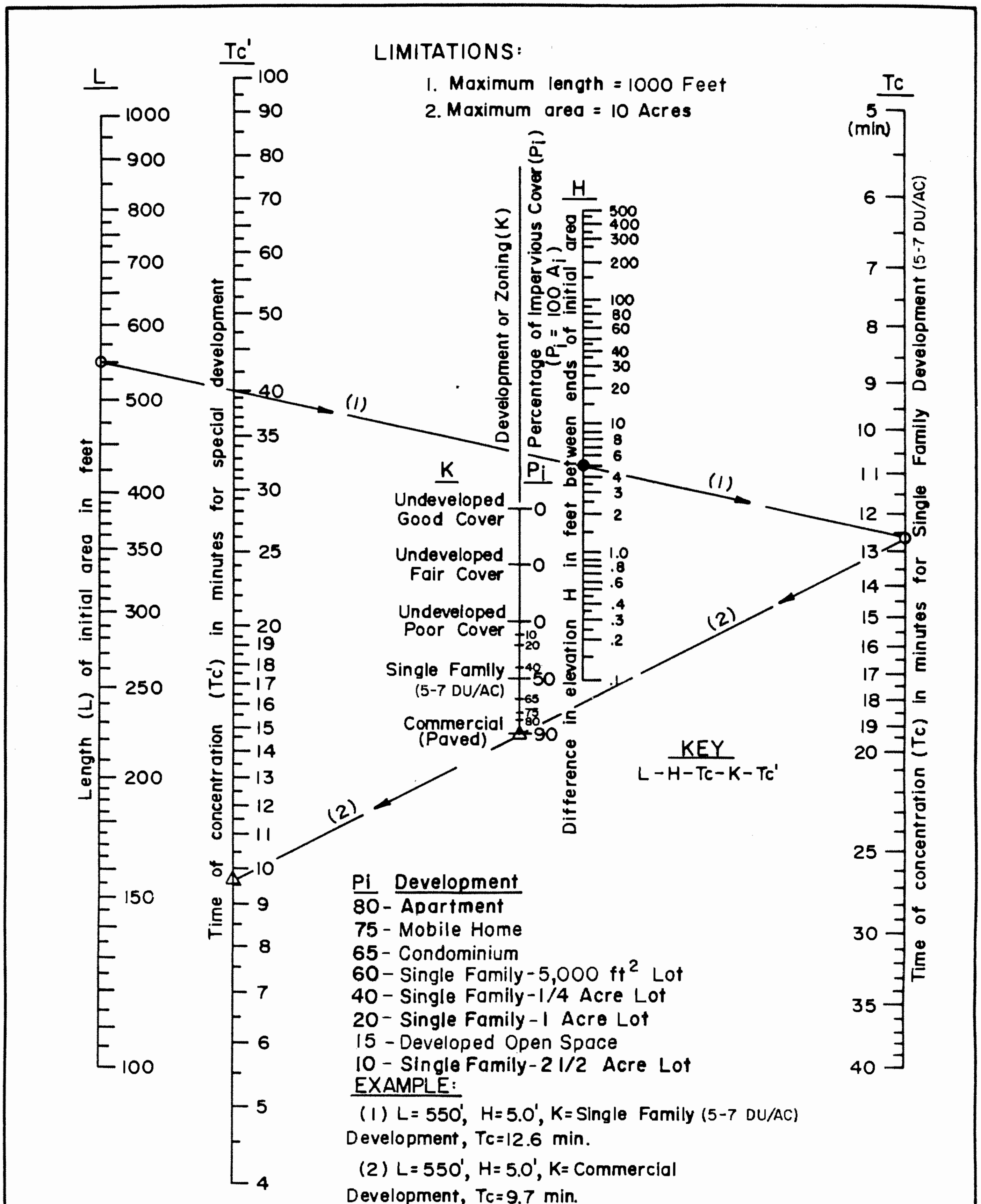
For large drainage areas, the absence of depth-area adjustments can result in significant differences in the estimate of the average depth of catchment point rainfalls.

The above limitations indicate that an estimate of the peak rate of runoff becomes less reliable as the drainage area becomes larger and the rational method equation should, therefore, not be used for drainage areas larger than 640 acres.

D.3. CRITICAL DURATION (TIME OF CONCENTRATION)

The critical duration of the storm rainfall required in the rational method equation is based on the time of concentration of the drainage area.

The time of concentration (Tc) is defined as the interval of time (in minutes) required for the flow at a given point to become a maximum under a uniform rainfall intensity. Generally, this occurs when all parts of the drainage area are contributing to the flow. Generally, the time of concentration is the interval of time from the beginning of rainfall for water from the hydraulically most remote portion of the drainage area to reach the point of concentration; e.g., the inlet of the drainage structure. The time of concentration is a function of many variables including the length of the flow path from the most remote point of an area to the concentration point, the slope and other characteristics of natural and improved channels in the area, the infiltration characteristics of the soil, and the extent and type of development. For rational method studies based on this manual, the time of concentration for an initial subarea may be estimated from the nomograph of Figure D-1. The time of concentration for the next downstream subarea is computed by adding to the initial Tc, the time required for the computed peak flow to travel to the next concentration point. Time of concentration is computed for each subsequent subarea by computing travel time between subareas and adding to the cumulative sum.



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TIME OF CONCENTRATION NOMOGRAPH FOR INITIAL SUBAREA

Figure D-1

When the flow is concentrated in curb and gutters, drainage channels or conduits, the flow velocity may be estimated by the well-known Manning's equation

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \tag{D.2}$$

where

V	=	mean velocity (fps)
n	=	Manning coefficient of roughness (see Design Manual)
R	=	hydraulic radius (feet)
S	=	energy slope which equals the conduit invert slope for uniform flow

The travel time will then be the flow distance divided by the velocity of flow.

Computations of travel time through subareas which continually add to the peak flow (e.g., streetflow) should be based on the average peak flow through the subarea. This average peak flow is generally a simple average of the peak flow rates estimated at the upstream and downstream points of the subarea.

The initial subarea Tc estimation often is the most significant factor leading to the Tc computation of a watershed. Small development studies typically utilize only initial subarea estimations due to the small subarea sizes. Larger study areas generally show high sensitivity to the initial subarea Tc. Consequently, judgment is needed when developing initial subarea Tc estimates. The nomograph of Figure D-1 is based on the Kirpich formula and relates an initial subarea Tc to subarea slope and development type. It is assumed in the nomograph that overland flow effects dominate the travel time hydraulics.

It is noted that the Tc computation procedure is based upon the summation of an initial subarea time of concentration with the several travel times estimated by normal depth flow-velocities through subsequent subareas.

D.4. INTENSITY-DURATION CURVES

The precipitation intensity-duration curves presented in Section B.3 (Figures B-3 and B-4) are appropriate for the rational method.

D.5. RUNOFF COEFFICIENT

The runoff coefficient (C) is the ratio of rate of runoff to the rate of rainfall at an average intensity (I) when the total drainage area is contributing. The selection of the runoff coefficient depends on rainfall intensity, soil infiltration rate (F_p), and impervious and pervious area fractions (a_i and a_p).

Since one acre-inch/hour is equal to 1.008 cfs, the rational formula is generally assumed to estimate a peak flowrate in cfs. Runoff coefficient curves are developed using the relationship:

$$C = \begin{cases} 0.90\left(a_i + \dfrac{(I - F_p)a_p}{I}\right), \text{ for I greater than } F_p; \\ 0.90\, a_i, \text{ for I less than or equal to } F_p \end{cases} \qquad \text{(D.3)}$$

where the proportion factor of 0.90 is a calibration constant determined by an average fit between the rational method and design storm unit hydrograph (see Section E) peak flow rate estimates, and where

C	=	runoff coefficient
I	=	rainfall intensity (inches/hour)
F_p	=	infiltration rate for pervious areas (inches/hour) (see Section C.6.4)
a_i	=	ratio of impervious area to total area (decimal fraction)
a_p	=	ratio of pervious area to total area (decimal fraction), ($a_p = 1 - a_i$)

D.6. PEAK FLOW RATE FORMULA

Combining Equations (D.1) and (D.3), the peak flow estimate for Q is written in simpler terms by

$$Q = .90\ (I - F_m)A \tag{D.4}$$

where $F_m = a_pF_p$ (see section C.6.5), and where in (D.4) it is understood that I is greater than F_p; otherwise $Q = .90\ a_iIA$.

In (D.4), F_m represents the loss rate for the total watershed tributary to the point of concentration. Should the tributary area contain several runoff surfaces, an area-averaged F_m is calculated. Table D.1 illustrates such an area-averaged F_m computation.

TABLE D.1. AREA-AVERAGED F_m COMPUTATION

Subarea Number (1)	a_p (2)	Soil Group (3)	F_p (inch/hour) (4)	Area (acres) (5)	Area Weighting of (4)
1	0.60	A	0.40	8	1.92
2	0.80	B	0.30	12	2.88
3	0.75	C	0.25	11	2.06
4	0.10	D	0.20	15	0.30
5	0.50	C	0.25	16	2.00
				62	9.16

From Table D.1., the area-averaged maximum loss rate, F_m, is given by F_m = (9.16)/(62) = 0.147 inch/hour, say 0.15.

D.7. DRAINAGE AREA

The contributing drainage area may be determined from topographic contour maps, aerial photos, and field surveys. Watershed divides are then drawn on a suitable topographic map and the enclosed drainage area is determined by planimeter or other methods. In areas where lateral and transverse slopes on the watershed are very mild, the nominal watershed area (or drainage subdivision) runoff may "cascade" under severe rainfall. That is, when the divide between one watershed and another is defined by a low relief feature such as the crown of a road, the runoff from such a watershed may "spill over" into the adjacent watershed or watershed subdivision. This may occur, for example, when gutter capacity is exceeded thereby increasing runoff contributions at downstream or adjacent concentration points above those anticipated by analysis of the nominal or "low flow" drainage boundaries. The possibility of such cascading shall be considered and provided for by the hydrologist.

D.8. RATIONAL METHOD CONFLUENCE ANALYSIS

In most studies, the calculation of peak flow rates along a main channel or stream involves only the direct application of (D.4). Such studies typically involve the inclusion of subarea runoff to the stream where the effect on the stream peak flow rate is relatively minor and, consequently, only (D.4) is needed for the analysis.

At the junction of two or more streams, however, the estimation of the peak flow rate involves a confluence analysis of the associated runoff hydrographs (see Appendix III).

For the confluence of two streams, let T_1, I_1, Fm_1, A_1, and Q_1, be the time of concentration, rainfall intensity, area-averaged loss rate, catchment area, and peak flow rate for stream #1 while T_2, I_2, Fm_2, A_2 and Q_2 correspond to stream #2. Also, let Q_1 be less than Q_2. Finally, let T_p, A_p, and Q_p be the resulting confluence estimates for Tc, area, and peak flow rate, respectively. Then two cases are possible:

*Case 1: $T_1 = T_2$. The runoff hydrographs must both peak at $T_p = T_1 = T_2$. And $Q_p = Q_1+Q_2$ for a total contributing area of $A_p = A_1+A_2$.

*Case 2: $T_1 \neq T_2$. In this case, the sum of the two runoff hydrographs must be considered. Except in very unusual conditions, flow rates of the summed runoff hydrograph typically achieve a maximum at either T_1 or T_2, and the peak flow rate estimates are calculated as follows:

Case 2a: T_1 is less than T_2. In this case, the stream with the largest Q has the longest Tc. The flow rate of the summed runoff hydrograph at time T_2 is estimated by

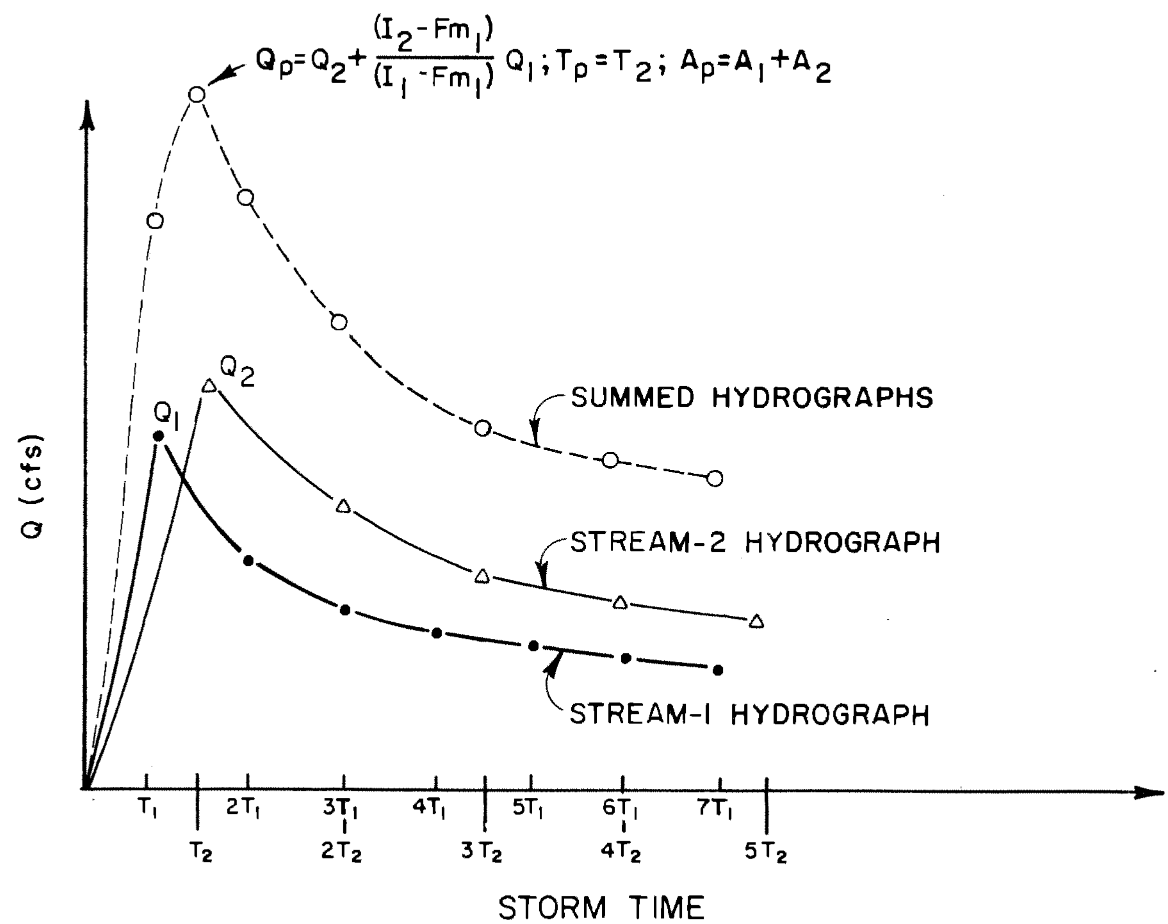
$$Q_p = Q_2 + \frac{(I_2 - Fm_1)}{(I_1 - Fm_1)} Q_1 \tag{D.5}$$

and $T_p = T_2$ (see Figure D-2). It is noted that the confluence peak Q of (D.5) equals the peak flow rate estimated from direct use of (D.4). Additionally, the total contributing area is $A_p = A_1 + A_2$.

Case 2b: T_1 is greater than T_2. In this case, the stream with the largest Q has the shortest Tc. The flow rate of the summed runoff hydrograph at time T_1 is estimated using a ratio of stream 1 effective rainfall intensities and Tc values corresponding to times T_2 and T_1 giving

$$Q_p = Q_2 + \frac{(I_2 - F_{m_1})}{(I_1 - F_{m_1})} \frac{(T_2)}{(T_1)} Q_1 \tag{D.6}$$

and $T_p = T_2$. Equation (D.6) indicates that the peak flow rate at time T_2 is the result of the high peak discharge from stream 2 and the runoff contribution from a fraction of the stream 1 catchment area.



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HYDROLOGY MANUAL

RATIONAL METHOD
CONFLUENCE ANALYSIS
(Summation of Runoff Hydrographs)

Figure D-2

That is, a portion of the catchment tributary to stream 1 is not contributing at time T_2 and, in the general case, only $(T_2/T_1)A_1$ of the stream 1 catchment area is contributing to the peak flow rate (at time T_2). Consequently for downstream study purposes, the "effective" catchment area corresponding to the subject peak flow rate is

$$A_p = A_2+(T_2/T_1)A_1 \tag{D.7}$$

It is noted that in the confluence peak flow rate estimate of (D.6), the critical duration is $T_p = T_2$ which corresponds to the effective catchment area of (D.7) Consequently, the peak flow rate contribution from the effective catchment area of stream 1 must reflect the higher rainfall intensity corresponding to time T_2 rather than time T_1. Use of (D.6) results in a peak flow which equals the governing rational method peak flow rate estimate from (D.4) applied to the effective catchment area computed by (D.7). It is noted that the estimation of the effective catchment area is only an approximation, and shall be verified by the hydrologist.

D.9. RATIONAL METHOD Tc CALCULATIONS FOR UNIT HYDROGRAPH STUDIES

Although the peak flow rate formula should generally not be used for catchments larger than 1 square mile, the rational method can be used to estimate Tc values for larger areas. That is, the rational method estimate for Tc in large areas is adequate for use in the unit hydrograph studies of section E. T-year storm estimates for Tc are determined for areas less than 1 square mile using the T-year intensity-duration curves and the appropriate Fm values to generate cfs/acre estimates. For larger areas, cfs/acre estimates for use in the rational method are obtained from the cfs/acre curves of section L.

D.10. REQUIRED FORMAT

Figure D-3 illustrates the required format for the submittal of rational method hydrology studies. All rational method calculations must be summarized on the form shown in Figure D-4.

D.11. INSTRUCTIONS FOR RATIONAL METHOD HYDROLOGY CALCULATIONS

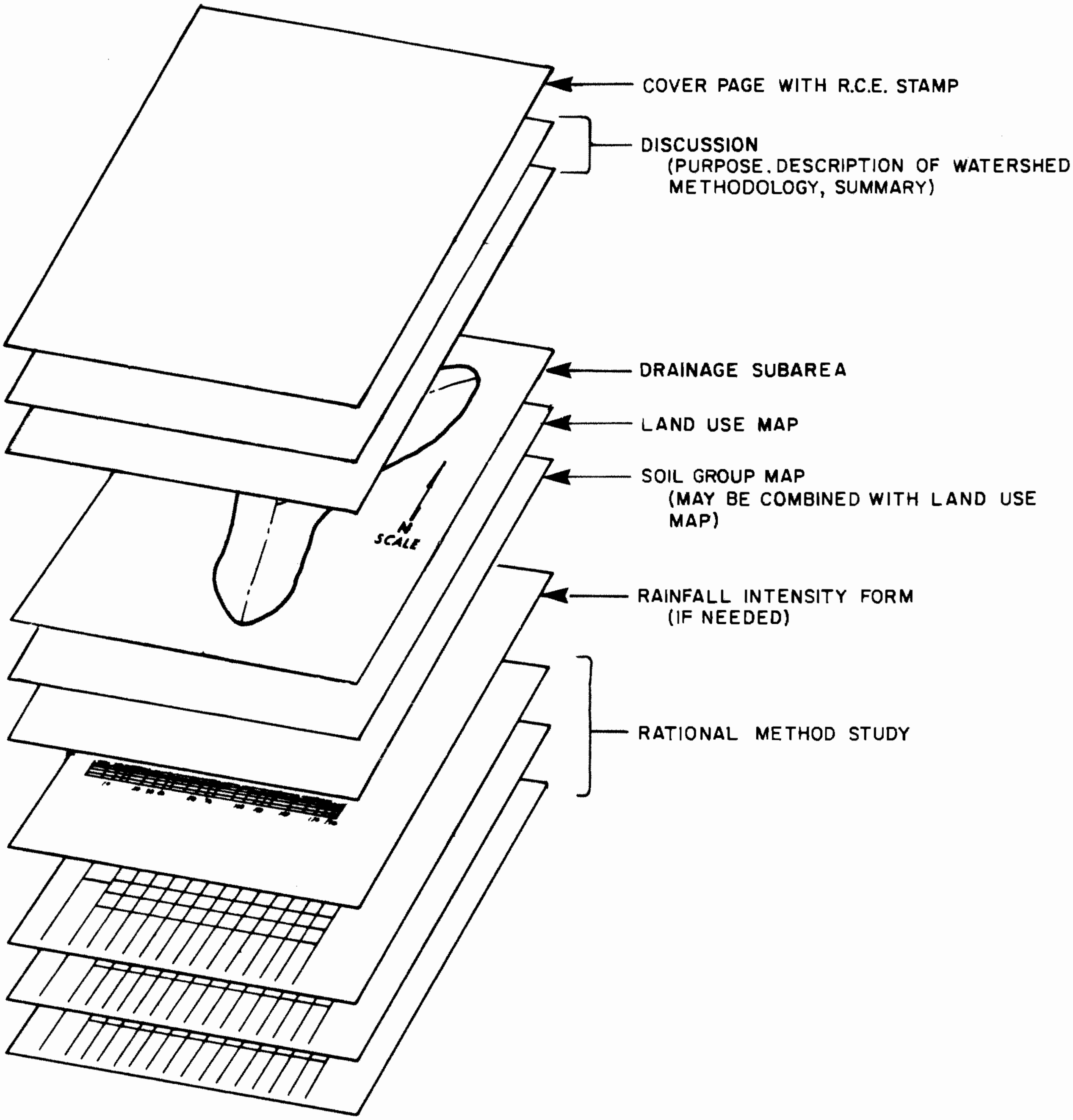
1. On a topographic map of the drainage area, draw the study drainage system and designate subareas tributary to the various points of concentration (see example problem).

2. Determine the initial time of concentration, (Tc), using Figure D-1. The initial subarea should be less than 10 acres, have a flow path of less than 1,000 feet, and generally should be the most upstream subarea of the watershed drainage system.

3. Using the time of concentration, determine (I) (intensity of rainfall in inches per hour) from the appropriate intensity-duration curve for the particular area under study using Figures B-3 and B-4.

4. Calculate the area-averaged maximum loss rate, F_m, which corresponds to the soil group, cover complex , and imperviousness of the drainage subarea. Loss rates for the pervious area, F_p, follow from section C.6.4.

5. Determine the area (A, acres) of the total watershed tributary to the point of concentration. Because the rational method computational results are sensitive to the subarea size definitions (especially in the most upstream reaches of the watershed), limit the size of subareas to allow for a gradual increase in subarea size as the study progresses downstream. The method is sensitive to large differences in successive subarea shapes, and lengths of reaches where travel times are estimated. Points of concentration should be selected downstream of the initial subarea such that subarea travel



ORANGE COUNTY
HYDROLOGY MANUAL

TYPICAL REPORT FORMAT
FOR
RATIONAL METHOD STUDY

Figure D-3

RATIONAL METHOD STUDY FORM

ORANGE COUNTY
HYDROLOGY MANUAL

STUDY NAME:

_____-YEAR STORM RATIONAL METHOD STUDY

Calculated by __________ Date __________
Checked by __________ Date __________
Page _____ of _____

Concentration Point	Area (Acres)		Soil Type	Dev. Type	T_t min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
	Subarea	Total												

Figure D-4

times are less than 3-minutes and 5-minutes for Tc values of 30-minutes and 60-minutes, respectively. After a Tc of 1-hour, subarea travel times should be limited to less than 10-minutes.

6a. Compute $Q = .90\ (I-F_m)A$ for the point of concentration.

6b. Should the computed Q be less than the previous upstream point of concentration Q, use the upstream Q value.

7. Measure the length that the peak runoff must travel to the point of concentration of the next downstream subarea. Determine the average velocity of flow in this reach using the peak Q in the appropriate type of conveyance being considered (natural channel, street, pipe, or open channel) using Manning's formula. Where necessary, the mean flow in the conveyance (e.g., streetflow) should be used to compute mean flow velocity.

Using the reach length and average flow velocity, compute the travel time and add to the time of concentration from the upstream subarea to determine a new time of concentration.

8. Calculate Q for the new point of concentration using steps 3 through 6 and the new time of concentration. Determine the time of concentration for the next downstream subarea using Step 7. Continue the above computation procedure downstream until a junction with a lateral drain is reached.

9. Start at the upstream end of the lateral and compute its Q down to the junction with the main line, using the methods outlined in the previous steps.

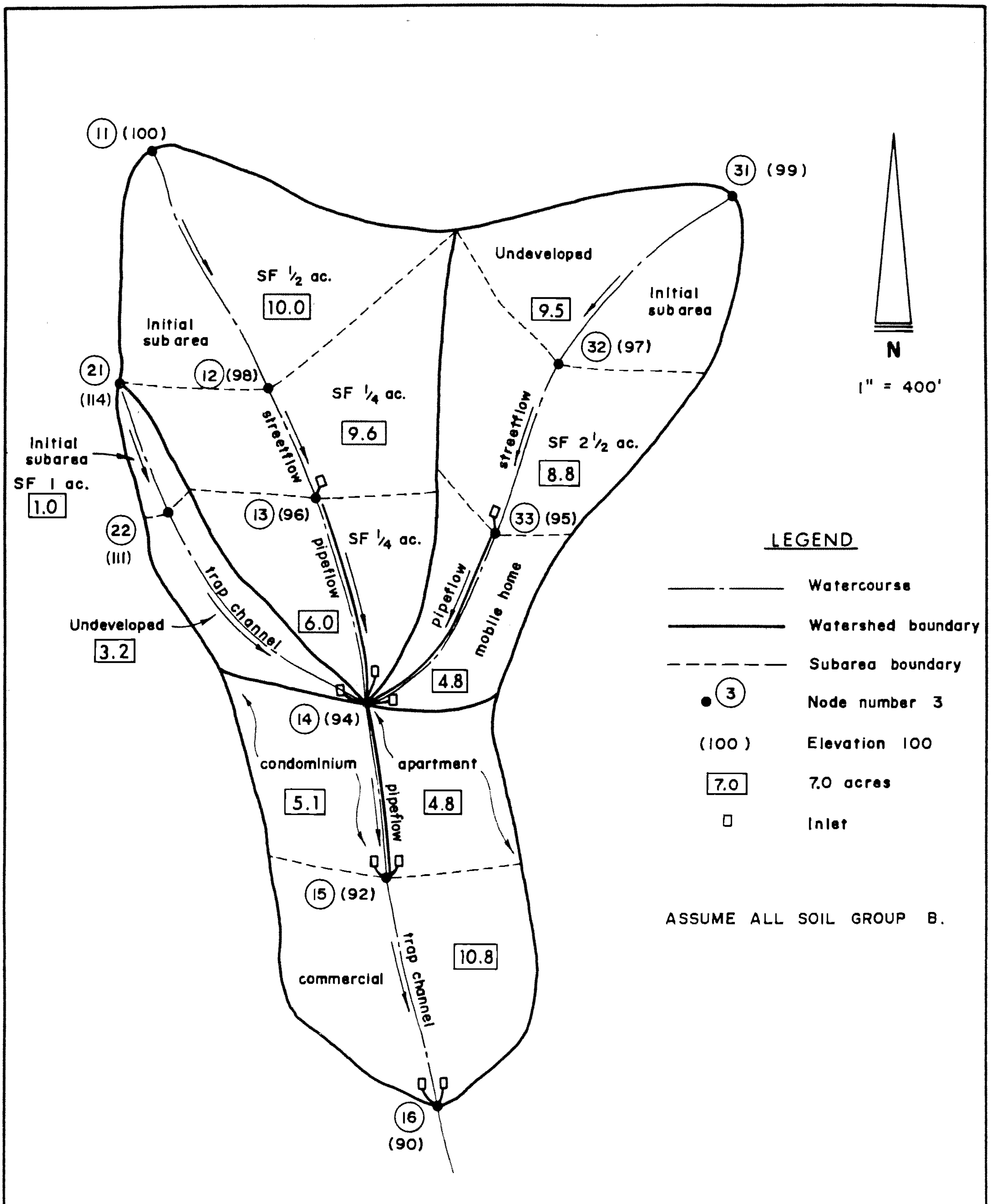
10. Compute the peak Q at the junction (confluence analysis--see Section D.8) and evaluate the sensitivity of the computed results to using the other Q and Tc values determined. That is, the downstream estimated peak Q values may be higher had a lower Q and lower Tc value been used at an upstream confluence point. The largest Q is, therefore, estimated along the entire watershed main channel.

D.12. EXAMPLE PROBLEM

The following example problem illustrates the format required for rational method hydrology studies. In the following, an example watershed is analyzed using the rational method approach. Additional and expanded examples are contained in the Hydrology Manual Workbook which can be obtained separately from the Agency. The example problem presentation contains the following information:

<u>Description</u>

- o Example Problem Drainage System Map
- o Example Problem Rational Method Calculation Sheets



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

DRAINAGE SYSTEM

Figure D-5

RATIONAL METHOD STUDY FORM

ORANGE COUNTY
HYDROLOGY MANUAL

STUDY NAME: *EXAMPLE PROBLEM*

10 -YEAR STORM RATIONAL METHOD STUDY

Calculated by *TRW* Date *5-12-86*
Checked by *MHS* Date *5-12-86*
Page *1* of *2*

Concentration Point	Area (Acres) Subarea	Total	Soil Type	Dev. Type	T_t min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
12.00	10.0	10.0	B	SF(1/2)	—	21.0	1.78	0.21	0.21	14.2	800	0.0025		INITIAL SUBAREA
13.00	9.6	19.6	B	SF(1/4)	2.3	23.3	1.68	0.18	0.20	26.1	350	0.0057	2.5	44 ft. STREET Q_{AVG} = 20.4 Dn = 0.55 ft
14.00	6.0	25.6	B	SF(1/4)	1.9	25.2	1.61	0.18	0.19	32.7	650	0.0031	5.6	33" RCP n = 0.013 Dn = 2.01
14.00		25.6				25.2				32.7				STREAM SUMMARY
22.00	1.0	1.0	B	SF(1)	—	13.7	2.28	0.24	0.24	1.8	400	0.0075	—	INITIAL SUBAREA
14.00	3.2	4.2	B	UNDEV.	3.0	16.7	2.03	0.30	0.29	6.6	850	0.020	4.7	B = 0.5' Z = 2.0 n = 0.015 Dn = 0.33
14.00		4.2				16.7				6.6				STREAM SUMMARY
32.00	9.5	9.5	B	UNDEV.		42.0	1.20	0.30	0.30	7.7	750	0.0027	—	INITIAL SUBAREA
33.00	8.8	18.3	B	SF(2 1/2)	5.1	47.1	1.12	0.27	0.29	14.0	550	0.0036	1.8	44 ft STREET Q_{AVG} = 10.9 Dn = 0.49 ft
14.00	4.8	23.1	B	MH	3.3	50.4	1.08	0.08	0.24	17.5	700	0.0014	3.5	30" RCP n = 0.013 Dn = 1.88
14.00		23.1				50.4				17.5				STREAM SUMMARY
$\bar{Q}_1$	= 32.7	+ (6.6)	$\left(\frac{1.60}{2.03}\right.$	$\left.\frac{-0.29}{-0.29}\right)$	+	(17.5)	$\left(\frac{25.2}{50.4}\right)$	$\left(\frac{1.61}{1.08}\right.$	$\left.\frac{-0.24}{-0.24}\right)$	= 52.0				CONFLUENCE ANALYSIS FOR PT. # 14.00
$\bar{Q}_2$	= (32.7)	$\left(\frac{16.7}{25.2}\right)\left(\frac{2.03}{1.60}\right.$	$\left.\frac{-0.19}{-0.19}\right)$	+ 6.6	+	(17.5)	$\left(\frac{16.7}{50.4}\right)$	$\left(\frac{2.03}{1.08}\right.$	$\left.\frac{-0.24}{-0.24}\right)$	= 47.0				
$\bar{Q}_3$	= (32.7)	$\left(\frac{1.08 -}{1.61 -}\right.$	$\left.\frac{0.19}{0.19}\right)$		+	(6.6)	$\left(\frac{1.08}{2.03}\right.$	$\left.\frac{-0.29}{-0.29}\right)$	+ 17.5 =	41.0				CONFLUENCE RESULTS
$\bar{A}$ = 25.6 + 4.2 +	$\frac{25.5}{50.4}$ (23.1)	= 41.4				25.2				52.0				

RATIONAL METHOD STUDY FORM

ORANGE COUNTY HYDROLOGY MANUAL	STUDY NAME: EXAMPLE PROBLEM 10 -YEAR STORM RATIONAL METHOD STUDY	Calculated by IRW Date 5-12-86 Checked by MHS Date 5-12-86 Page 2 of 2

Concentration Point	Area (Acres) Subarea	Area (Acres) Total	Soil Type	Dev. Type	T_t min.	T_c min.	I in/hr	Fm in/hr	Fm avg.	Q Total	Flow Path Length ft.	Slope ft./ft.	V ft./sec.	Hydraulics and Notes
15.00	5.1	46.5	B	CONDO	1.3	26.5	1.56	0.11	—	—	550	0.0036	7.1	42" RCP Dn=2.5 n=0.013
					—						—	—	—	ADDITION OF SUBAREA
15.00	4.8	51.3	B	APT		26.5	1.56	0.06	0.19	63.3				
					2.1						700	0.0029	5.6	n=0.015 Dn=1.9 Z=2.0 B=2.0
16.00	10.8	62.1	B	COM		28.6	1.49	0.03	0.16	74.3				
														STREAM SUMMARY
16.00		62.1				28.6				74.3				

SECTION E

THE UNIT HYDROGRAPH METHOD FOR CATCHMENT RUNOFF HYDROGRAPHS

E.1. BACKGROUND

The unit hydrograph method assumes that watershed discharge is related to the total volume of runoff, and that the time factors which affect the unit hydrograph shape are invariant, and that watershed storm rainfall-runoff relationships are characterized by watershed area, slope, and shape factors. The UH method is used to estimate the time distribution of watershed runoff in drainage basins where stream gage information is either unavailable or inadequate to justify statistical interpretation (refs. 4-10). The unit hydrograph method for determining the time distribution of runoff shall be used for hydrology studies of all Orange County watersheds in excess of 640 acres.

For a catchment of one (1) square mile (640 acres) or larger, where only the peak discharge is required, and where the Valley:Developed S-Graph applies, the Peak Flowrate Curves in Section L may be used.

E.2. TERMINOLOGY

The following definitions are used in the discussion of unit hydrograph and runoff hydrograph synthesis:

Effective rainfall is that part of rainfall that runs off in a relatively brief time period. (Here, the brief time period is selected sufficiently small such that the significant hydrologic effects are adequately represented by the time-period's average values.) Effective rainfall is the total rainfall less infiltration, evaporation, transpiration, absorption, and detention.

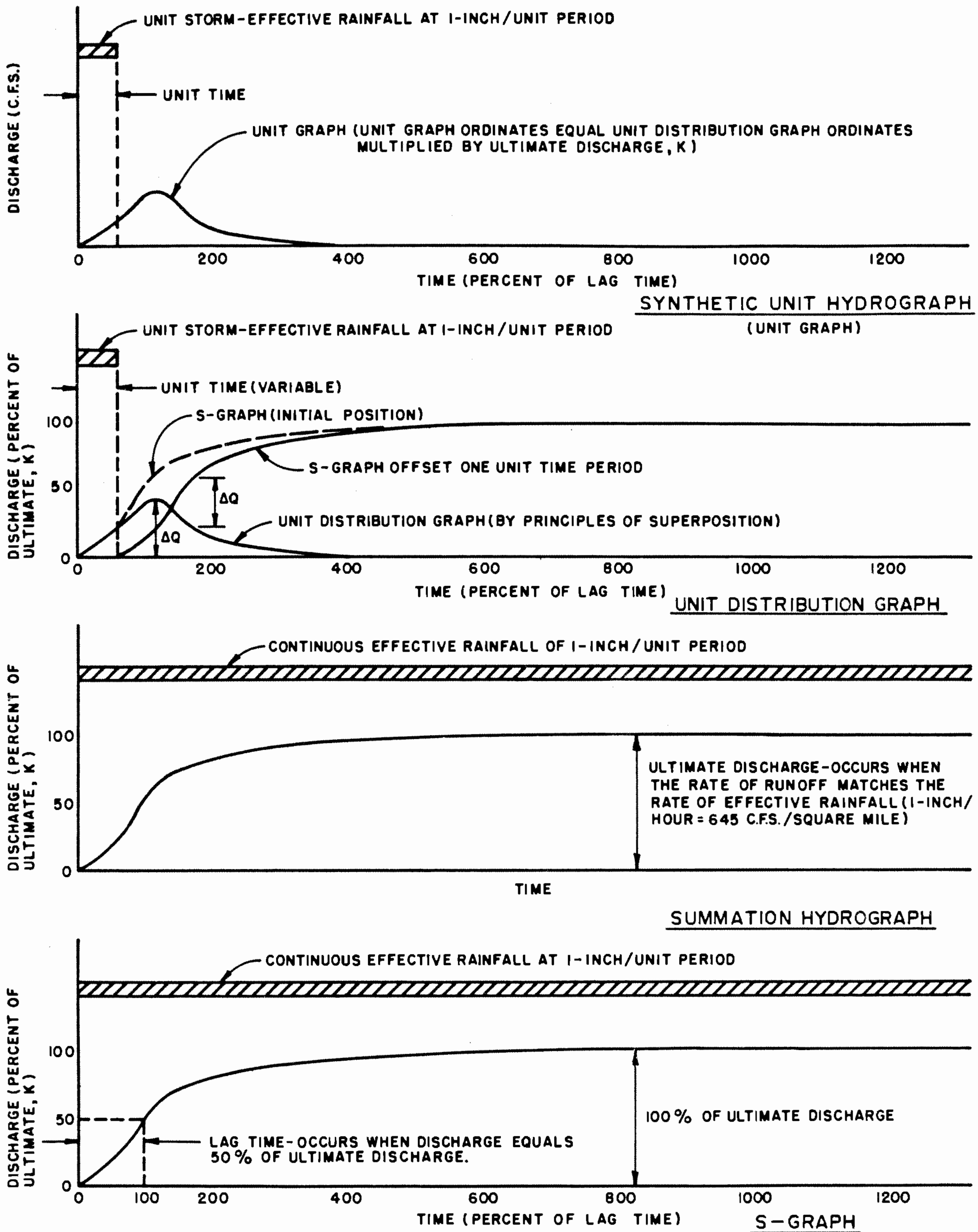
Unit hydrograph (or unit graph) for a point of concentration on a watershed (catchment) stream is a curve (hydrograph) showing the time distribution of rates of runoff which results from one inch of effective rainfall during a unit period of time over the tributary watershed upstream of the point of concentration. The unit effective rainfall is generally assumed to occur as an equivalent constant rainfall intensity during a specified unit period of time (such as 5, 10, 15 or 30-minutes). Figure E-1 illustrates the general formulation of the unit hydrograph.

Distribution graph is a unit hydrograph whose ordinates are expressed in terms of percent of ultimate discharge. A distribution graph is generally developed as a block graph with each block representing its associated percent of unit runoff which occurs during the specified unit time. The unit time used in the distribution graph is identical to the unit time specified for the unit hydrograph.

Summation hydrograph for a point of concentration on a given stream is a curve (hydrograph) showing the time distribution of the rates of runoff that would result from a continuous series of unit period effective rainfalls over the tributary watershed upstream of the subject point of concentration. The ordinates of the summation hydrograph are expressed in percent of the ultimate discharge.

Lag for a watershed is the time (hours) from the beginning of a continuous series of unit period effective rainfalls over the watershed area (tributary to a point of concentration) to the instant when the rate of resulting tributary watershed runoff (at the point of concentration) equals 50 percent of the ultimate rate of the resulting runoff.

Ultimate discharge is the maximum rate of watershed runoff which can result from a specified effective rainfall intensity. Ultimate discharge from a watershed occurs when the rate of runoff on the summation hydrograph is equivalent to the rate of effective rainfall. For an effective rainfall rate of one inch occurring in a unit period of one



ORANGE COUNTY

HYDROLOGY MANUAL

DERIVATION OF A SYNTHETIC UNIT HYDROGRAPH

Figure E-1

hour, the ultimate discharge is 645 cfs for every square mile of watershed. Ultimate discharge for different unit periods is given by dividing 645 by the unit period in hours, and multiplying by the watershed area in square miles.

S-Graph is a summation hydrograph developed by plotting watershed discharge expressed in percent of ultimate discharge as a function of time expressed in percent of lag.

E.3. DETERMINATION OF SYNTHETIC DISTRIBUTION GRAPHS

Adequate storm rainfall and watershed runoff information are available for the determination of distribution graphs for several streams in Southern California. The distribution graphs for each of the gaged streams can be determined by trial-and-error attempts to duplicate the runoff hydrographs produced by major storm events (i.e., reconstitution studies). The derived distribution graphs are then verified by using them to reproduce runoff hydrographs from other major storm events.

The method of determining synthetic distribution graphs is used to estimate the time distribution of watershed runoff in drainage basins where stream gage data is inadequate. The procedure develops a time distribution of runoff based on the properties of distribution graphs from several gaged watersheds (refs. 4-10).

It is assumed that the drainage areas within a given region are physiographically and hydrologically similar. Because no two drainage areas have identical hydrologic characteristics, the runoff patterns from these areas are generally dissimilar and the distribution graphs of these areas may differ considerably. Therefore, direct transposition of distribution graphs from one watershed to another is usually precluded. However, most distribution graphs exhibit certain similarities which the introduction of a factor called "lag" will bring the arrangement of ordinates along the bases of distribution graphs into a generally consistent relationship. Lag, which was first defined as the time difference in phase between salient features of the rainfall and runoff rate curves, is an empirical expression of the hydrologic characteristics of a watershed in terms of time. Details of the determination of lag for watersheds where the time distributions of runoff are known and of the use of lag in developing synthetic distribution graphs are discussed in the following:

1. Summation Hydrograph - The first step in determining lag for a watershed is the determination of the summation hydrograph, which is the hydrograph of runoff that would result from the continuous generation of unit effective rainfall over the watershed. The ordinates of summation hydrographs are expressed in percent of ultimate discharge and a summation hydrograph for a point of concentration is determined by adding a continuous series of identical distribution graphs out of phase one unit period. On such a hydrograph, the time required to reach maximum (ultimate) discharge is equal to the length of the base of one distribution graph less one unit period.

2. Lag - Lag for a watershed can be defined as the elapsed time (in hours) from the beginning of unit effective rainfall to the instant that the summation hydrograph for the point of concentration reaches 50 percent of ultimate discharge. When the lags determined from summation hydrographs for several gaged watersheds are correlated to the hydrologic characteristics of the watersheds, an empirical relationship is usually apparent. This relationship can then be used to determine the lags for comparable drainage areas for which the hydrologic characteristics can be determined, but for which the distribution graphs for concentration points cannot be determined because of inadequate hydrologic data. By comparing lag values (obtained from the analysis of rainfall-runoff data) to catchment time of concentration Tc values estimated from either a detailed rational method analysis (Section D) or use of the peak flowrate curves of section L, a relationship is readily determined,

$$\text{lag} = 0.8Tc \tag{E.1}$$

It is noted that the rational method time of concentration, used for the estimation of basin lag time, is a critical parameter in the unit hydrograph method. Extreme care must be taken in the evaluation of the catchment Tc in order to reduce uncerainty, and enable "reproducibility" of this parameter. Section D provides the procedure for estimating Tc using the rational method for small areas. For larger areas, the Tc estimation procedure follows the methods of section D except that cfs/acre values are estimated using the cfs/acre curves of section L.

Lag = 0.8Tc (E.1) is used in all unit hydrograph studies where sufficient topographic information is available to compute the time of concentration, Tc. It is noted that due to Tc being the sum of the rational method's initial subarea Tc and the subsequent downstream reach hydraulic traveltimes, Tc values will vary depending on the return frequency of rainfall used in the analysis. That is, a 2-year storm estimated Tc value typically is longer in duration than a 100-year storm estimated Tc value. Consequently, when computing the lag corresponding to a T-year design storm event, the Tc is estimated using the T-year intensity-duration rainfalls in the rational method, or by using the T-year peak flowrate curves. For certain large scale natural condition catchment studies (e.g., Carbon Canyon, Santiago Creek, Trabuco Creek, San Juan Creek) the Agency may consider the use of the lag relationship given by the empirical formula:

$$\text{lag (hours)} = C_t \left((L \cdot L_{ca}) / S^{0.5} \right)^m \qquad \text{(E.2)}$$

where

C_t = a constant (determined by regional flood reconstitution studies)

L = length of longest watercourse (miles)

L_{ca} = length along longest watercourse, measured upstream to a point opposite center of area (miles)

S = overall slope of drainage area between the headwaters and the collection point (feet per mile)

m = a constant determined by regional flood reconstitution studies

It is then assumed that there exists a relationship between watershed lag and the quotient $((L \cdot L_{ca})/(S^{0.5}))^m$. This relationship is given by the above empirical formula for lag when

C_t	=	$24\ \bar{n}$; ($\bar{n}$ is the visually estimated basin factor of all collection streams and watershed channels, see Figure E-2)
m	=	0.38

3. <u>S-graph</u> - After lag factors are determined for several gaged watersheds the next step in determining synthetic distribution graphs is the development of S-graphs, which are summation hydrographs modified so that the percent of ultimate discharge is related to time expressed in percent of lag. The derivation of an S-graph is identical to the derivation of a summation hydrograph, except that the factor of lag has been introduced. Time in percent of lag has been used to determine S-graphs for four major groupings of watersheds.

Four S-graphs are used for unit hydrograph development in Orange County. These S-graphs are entitled Valley:Developed, Valley:Undeveloped, Foothill, and Mountain (Figures E-3a, b, c, d). In conformity with the definition of lag, each S-graph reaches 50 percent of ultimate discharge at 100 percent of lag. The average of the several S-graphs determined for mountain watersheds is assumed to be applicable to the mountain drainage basins with unknown runoff characteristics. Similarly the average of the S-graphs determined for valley watersheds is assumed to be applicable to the valley drainage basins, and so forth. Use of the Foothill S-graph is only for watersheds characterized by natural channels that are sharply incised in canyon bottoms, i.e., overbank flows are confined near the defined channel. Use of the Mountain S-graph is only for watersheds characterized by natural channels with numerous plunging flow reaches and lodged boulders/debris. Use of the Valley:Undeveloped S-graph is for natural watersheds whose channels are not sharply incised, i.e., where overbank flows may spread widely from the defined channel. Use of the Valley:Developed S-graph is for all watershed conditions where prismatic channels exist or are to be provided for conveyance of T-year peak flows.

$\bar{n}$ = **0.015**

1. Drainage area has fairly uniform, gentle slopes
2. Most watercourses either improved or along paved streets
3. Groundcover consists of some grasses - large % of area impervious
4. Main water course improved channel or conduit

$\bar{n}$ = **0.020**

1. Drainage area has some graded and non-uniform, gentle slopes
2. Over half of the area watercourses are improved or paved streets
3. Groundcover consists of equal amount of grasses and impervious area
4. Main watercourse is partly improved channel or conduit and partly greenbelt (see n = 0.025)

$\bar{n}$ = **0.025**

1. Drainage area is generally rolling with gentle side slopes
2. Some drainage improvements in the area - streets and canals
3. Groundcover consists mostly of scattered brush and grass and small % impervious
4. Main watercourse is straight channels which are turfed or with stony beds and weeds on earth bank (greenbelt type)

$\bar{n}$ = **0.030**

1. Drainage area is generally rolling with rounded ridges and moderate side slopes
2. No drainage improvements exist in the area
3. Groundcover includes scattered brush and grasses
4. Watercourses meander in fairly straight, unimproved channels with some boulders and lodged debris

$\bar{n}$ = **0.040**

1. Drainage area is composed of steep upper canyons with moderate slopes in lower canyons
2. No drainage improvements exist in the area
3. Groundcover is mixed brush and trees with grasses in lower canyons
4. Watercourses have moderate bends and are moderately impeded by boulders and debris with meandering courses

$\bar{n}$ = **0.050**

1. Drainage area is quite rugged with sharp ridges and steep canyons
2. No drainage improvements exist in the area
3. Groundcover, excluding small areas of rock outcrops, includes many trees and considerable underbrush
4. Watercourses meander around sharp bends, over large boulders and considerable debris obstruction

$\bar{n}$ = **0.200**

1. Drainage area has comparatively uniform slopes
2. No drainage improvements exist in the area
3. Groundcover consists of cultivated crops or substantial growths of grass and fairly dense small shrubs, cacti, or similar vegetation
4. Surface characteristics are such that channelization does not occur

BASIN FACTOR DESCRIPTIONS

Figure E-2

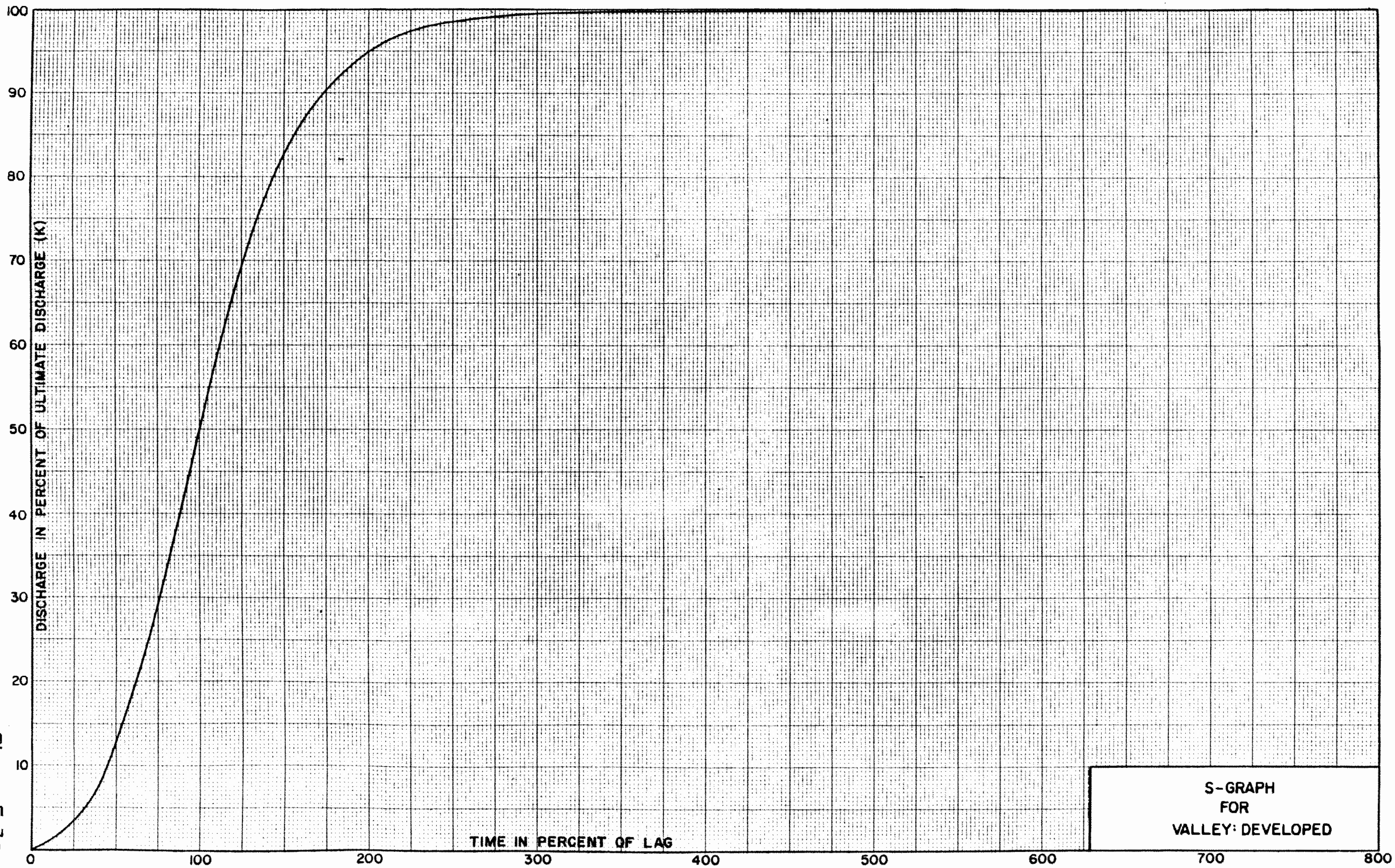

DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE (K)
TIME IN PERCENT OF LAG
100
90
80
70
60
50
40
30
20
10
0
100
200
300
400
500
600
700
800
S-GRAPH
FOR
VALLEY: DEVELOPED

Figure E-3a

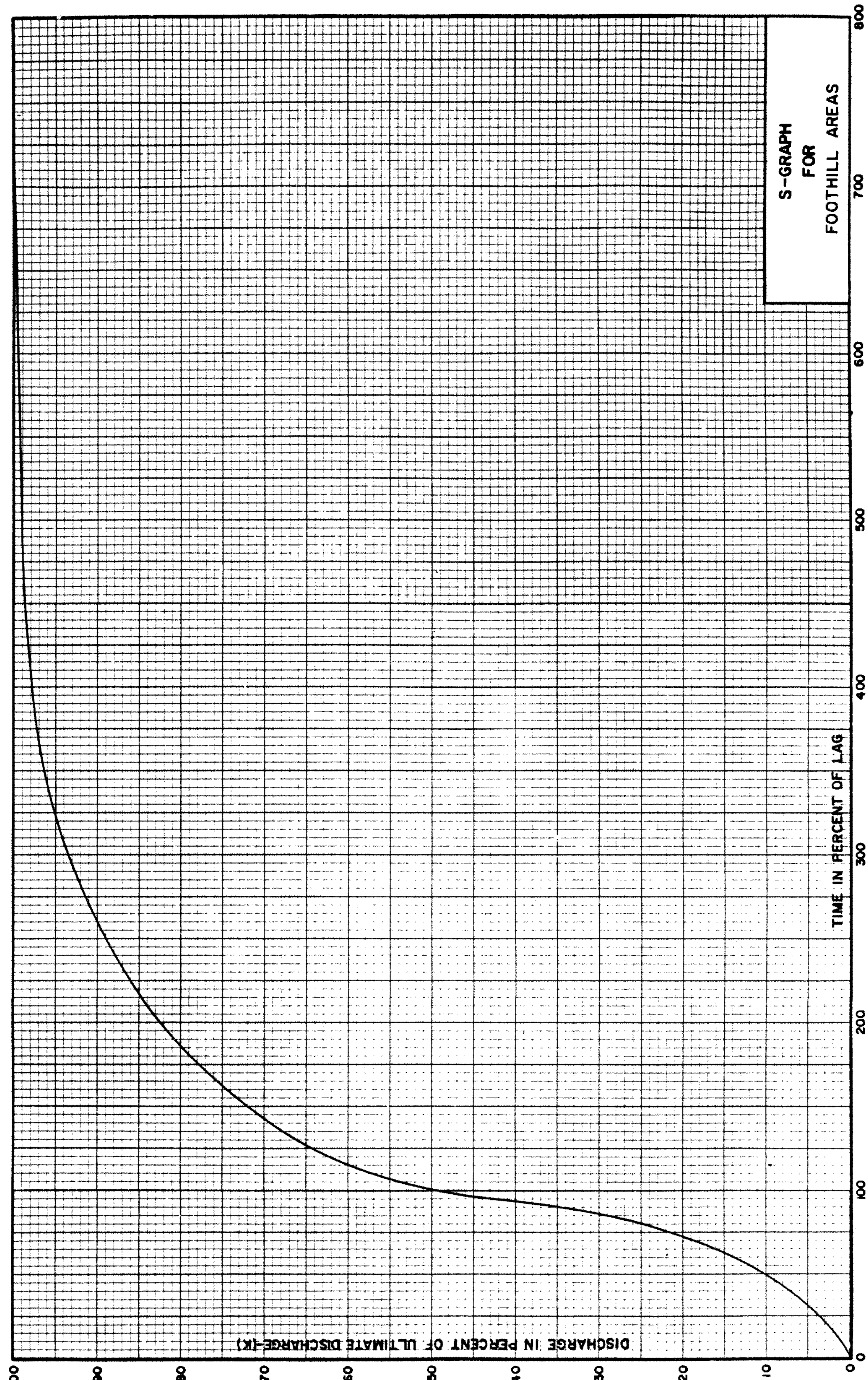

S-GRAPH
FOR
FOOTHILL AREAS
TIME IN PERCENT OF LAG
DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE-(K)
0
100
200
300
400
500
600
700
800
0
10
20
30
40
50
60
70
80
90
100

Figure E-3b

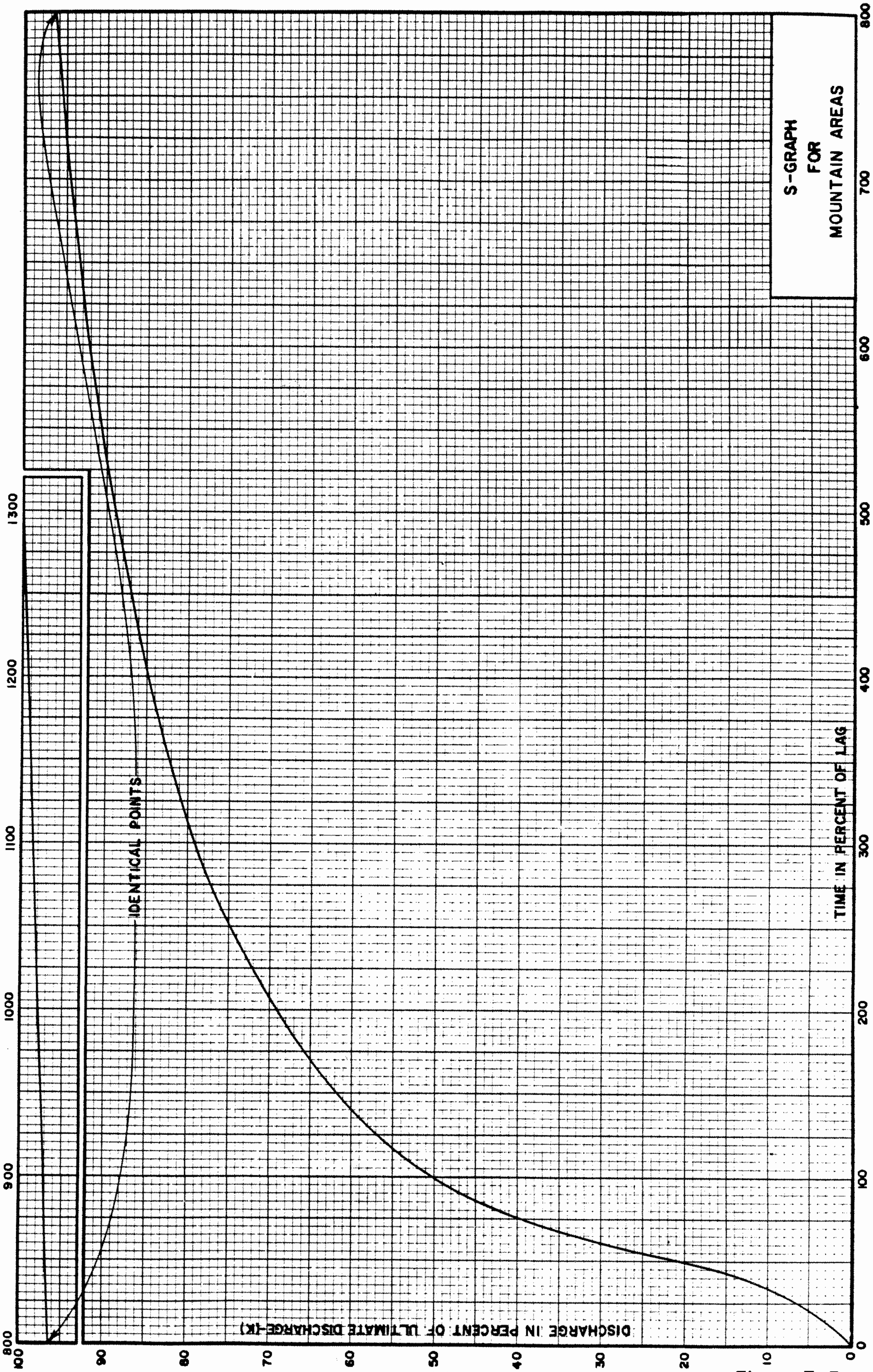

S-GRAPH
FOR
MOUNTAIN AREAS
TIME IN PERCENT OF LAG
DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE-(K)
IDENTICAL POINTS

Figure E-3c

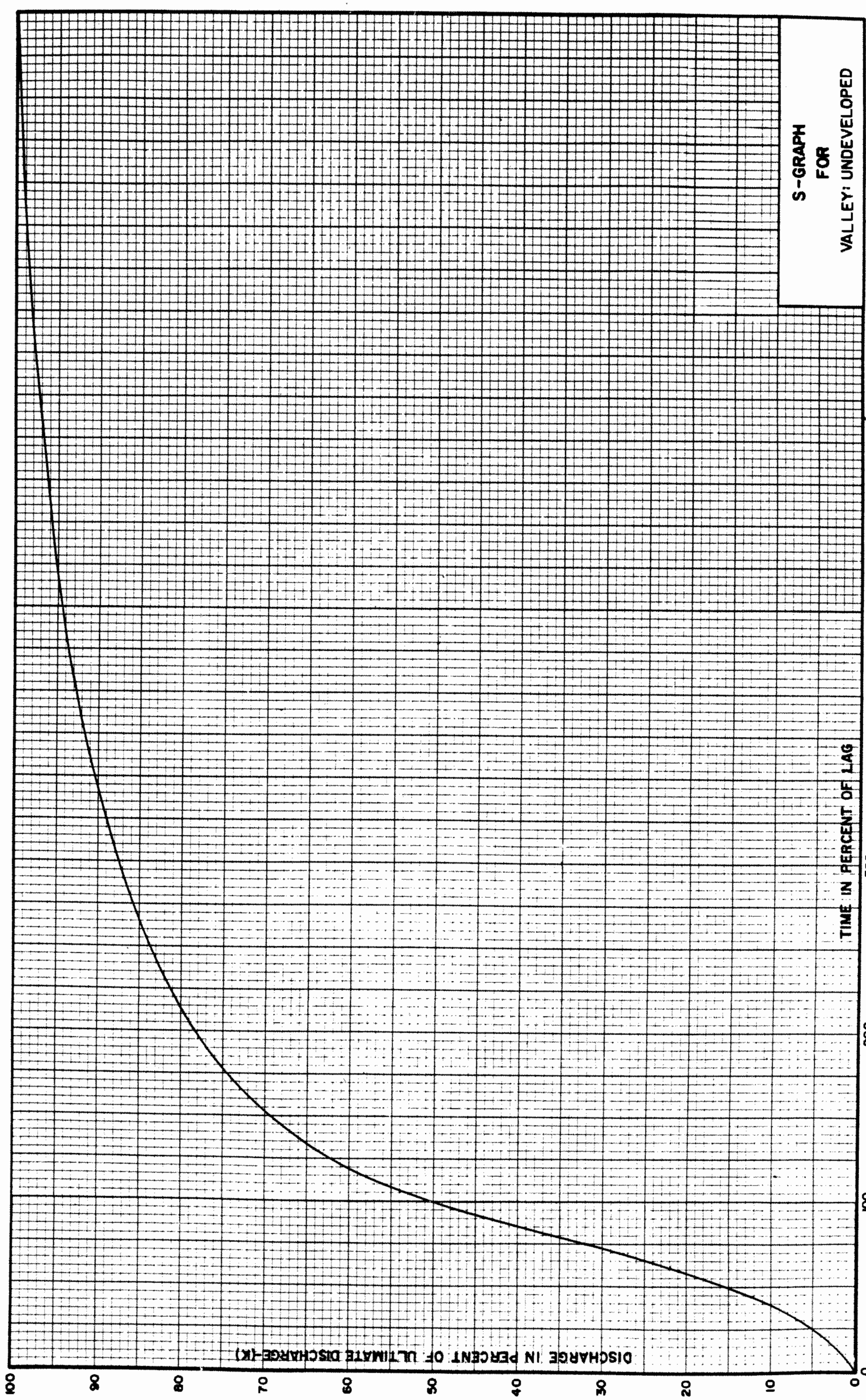

S-GRAPH
FOR
VALLEY: UNDEVELOPED
TIME IN PERCENT OF LAG
DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE-(K)
0
100
200
300
400
500
600
700
800
10
20
30
40
50
60
70
80
90
100

Figure E-3d

E.3.1. Application of Lag and S-graphs

Using the rational method, the watershed time of concentration (Section D) is computed and lag is determined using (E.1). A unit time is selected (generally 15 to 25 percent of the lag) and accumulated unit time periods are expressed as accumulated percentages of the watershed lag. These percentages of lag are used in superimposing a "block" graph on the appropriate average S-graph for the watershed and the resulting pattern is used in determining the accumulated mean percentage of ultimate discharge for each accumulated unit time (see example problem). Because these accumulated mean percentages represent the accumulated mean percentages for the synthetic distribution graph for the watershed, the mean percentage for successive unit periods are determined by a series of subtractions.

E.4. DEVELOPMENT OF THE SYNTHETIC UNIT HYDROGRAPH

For watersheds where stream gage data is inadequate, it may generally be assumed that a synthetic unit hydrograph adequately approximates the time distribution of runoff at the subject watershed point of concentration. From the above discussion, a method to develop a synthetic unit hydrograph is described in the following steps:

1. Estimate the watershed lag using topographic information and a rational method Tc calculation based on the appropriate T-year rainfall.

2. Select a unit period to be used for the hydrograph analysis. This unit period will be used for development of design critical storm unit rainfalls and the runoff hydrograph. The unit period is generally chosen to be within 15 and 25 percent of the watershed lag in order to provide sufficient definition of the unit hydrograph.

3. An S-graph is chosen which is appropriate for the catchment being studied.

4. The appropriate watershed S-graph can be approximated by a block graph where the base of each block is the selected unit period percentage of lag (Step 2) and the ordinate of each block is the time-averaged percentage of ultimate discharge (from the S-graph) for that unit period. The area of each block equals the area under the S-graph for each resulting pattern is used in determining the accumulated mean percentage of respective unit period. Consequently, at the end of each unit period the total area under the S-graph equals the sum of the areas of the equivalent unit period blocks.

5. The unit distribution block graph is developed by computing the difference between the ordinates (percentage of ultimate discharge) assigned to the unit period blocks used to approximate the S-graph of Step 4. This is equivalent to computing the difference between the ordinates of two S-graphs which have been offset by one unit period.

6. The final step to develop the synthetic unit hydrograph (or unit graph) is to multiply the ordinates of the distribution block graph (Step 5) by the factor K, the ultimate discharge. The ultimate discharge is defined by

$$K \text{ (cfs)} = 645\ A/T \qquad \text{(E.3)}$$

A	=	drainage area (square miles)
T	=	unit time period (hours)

E.5. DESIGN STORM PRECIPITATION DATA

The Agency's prescribed level of flood protection is obtained by using T-year rainfalls for the development of the T-year runoff hydrograph. Section B.4 provides the necessary information for developing the design storm pattern.

E.6. DESIGN STORM PATTERN

The design storm pattern is based upon a single synthetic 24-hour critical storm pattern which includes the peak rainfall intensities estimated for the 5-minute, 30-minute, 1-hour, 3-hour, 6-hour, and 24-hour durations. The storm pattern is developed from the watershed area-averaged point precipitation values, and modified incrementally according to the depth-area curves of Figure B-6. The assignment of peak rainfall values within the synthetic critical storm pattern is shown in Figure B-5(a,b,c) (refs. 4, 10).

For large watersheds (e.g., 5 square miles or larger) or for detention basin studies, the entire 24-hour synthetic storm pattern may be required for hydrologic study purposes. For small watersheds (less than 5 square miles) where only peak runoff rates are required, the peak 3-hours of the 24-hour synthetic storm pattern generally can be used for study purposes, ignoring the remaining 21-hours of lower intensity rainfall.

A detailed discussion of the design storm appropriate for the unit hydrograph method is presented in Section B-4.

E.7. DESIGN STORM LOSS RATES

Where sufficient stream gage information is of adequate quantity and quality as determined by the Agency, loss rates for unit hydrograph hydrology may be estimated from a study of rainfall-runoff relationships of major storms. Where such data is not available, loss rates for pervious areas shall be estimated using the methods of Section C.6.

E.7.1. Maximum Loss Rate, F_m

The maximum loss rate, F_m, for a catchment is computed by

$$F_m = a_p F_p \tag{E.4}$$

where

F_m = maximum loss rate (inches/hour)

a_p = pervious area fraction (see Figure C-4)

F_p = maximum loss rate for pervious areas (inches/hour); (see Section C.6.4)

Maximum loss rates, F_m, for runoff hydrograph studies are usually within the range of 0.05 to 0.25 inches per hour in urbanized areas. The range of values for the maximum loss rate for pervious areas, Fp is 0.20 to 0.40 in/hr. (see Table C.2).

E.7.2. Low Loss Rate, F*

During the peak rainfall intensities of the synthetic design storm pattern, the loss rate used to estimate effective rainfall is typically the maximum loss rate, F_m. At lower rainfall intensities, however, a low loss rate, F*, is used for the estimation of effective rainfall. The low loss rate F* is based upon the low loss fraction, $\bar{Y}$, defined by (see Section C.6.3)

$$\bar{Y} = 1 - Y \qquad \text{(E.5)}$$

where

$\bar{Y}$ = catchment low loss fraction

Y = catchment 24-hour storm runoff yield fraction computed from (C.4)

The corresponding low loss rate based on the $\bar{Y}$ value is

$$F^* = \bar{Y} \cdot I \qquad \text{(E.6)}$$

where

F^* = low loss rate (inches/hour)

$\bar{Y}$ = low loss fraction

I = rainfall intensity (inches/hour)

The low loss fraction should be used to estimate effective rainfall whenever the maximum loss rate, F_m, exceeds F^*. In all cases, however, the maximum loss rate is the constant value, F_m. Use of these two loss rate concepts enables the 24-hour design storm runoff hydrograph model to develop peak runoff rates based upon a maximum watershed loss rate (phi index), and also develop 24-hour storm runoff yields which approximate the values obtained from the curve number approach.

E.8. BASEFLOWS

Baseflow is usually a minor factor in developing flood hydrographs for relatively rare flood events in Orange County. Generally, 10 cfs per watershed square mile is adequate for unlined channels that intercept mountainous regions where many geologic strata are crossed by the stream bed. Baseflow can be included in the watershed runoff by adding to the ordinates of the computed runoff hydrograph. In fully urbanized areas, baseflow can be entirely neglected.

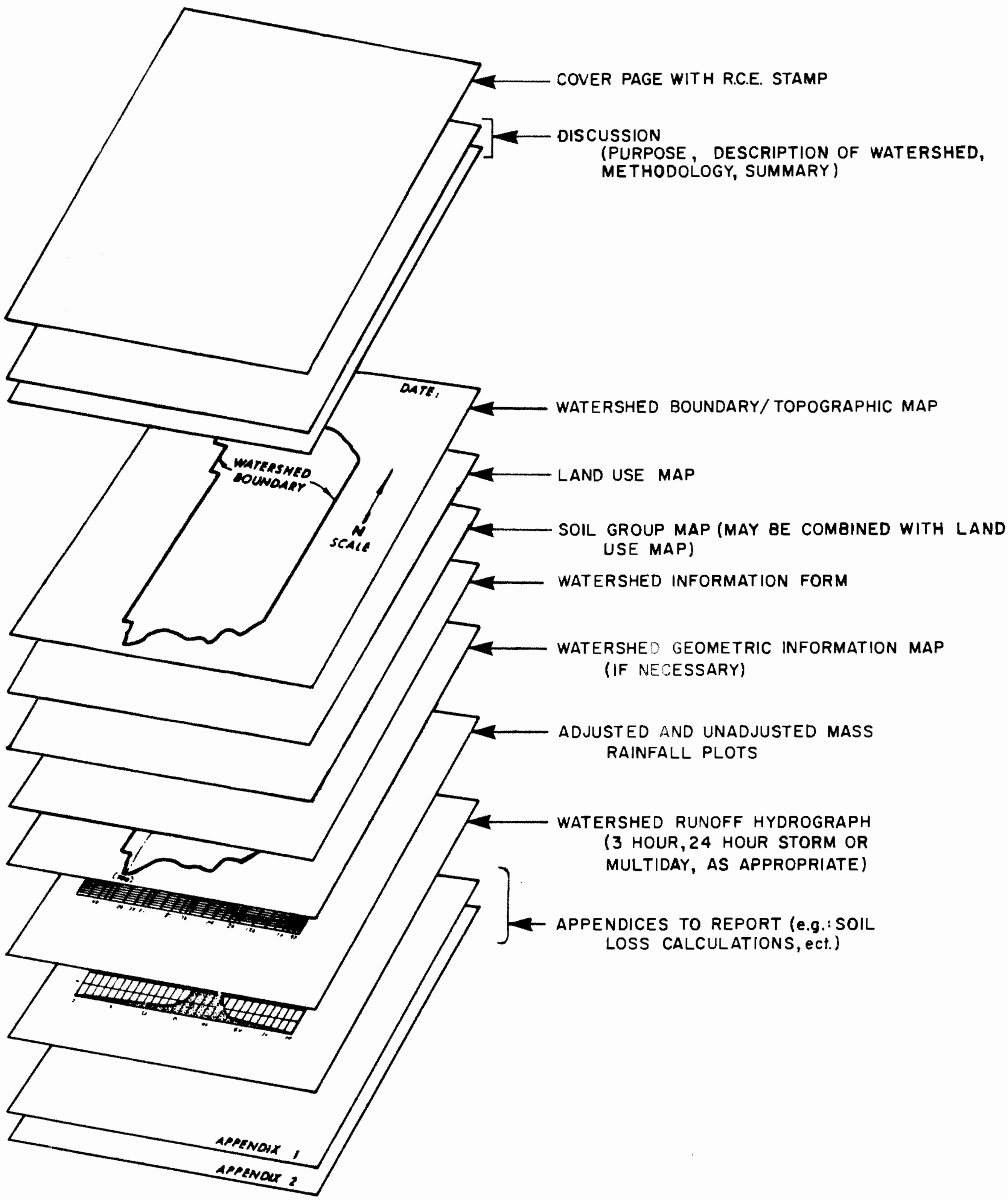
E.9. ALTERNATIVE RUNOFF HYDROGRAPH METHODS

The Orange County Hydrology Manual has been calibrated to local watershed conditions (without the use of channel routing parameters) in order that the unit hydrograph hydrologic methods achieve the desired level of protection in estimating the return frequency of floodflows. The introduction of additional parameters, in particular the routing of subarea flows, may alter the calibration, resulting in a failure to achieve the flood control protection level objectives. Because model sensitivity analysis and model calibration is essentially precluded for studies involving ungaged watersheds, any hydrologic study not prepared in accordance with this hydrology manual may be rejected (see Section K).

E.10. REQUIRED FORMAT

Figure E-4 illustrates the required format for submitting unit hydrograph study results for review.

Figure E-5 is to be used to supply the necessary hydrology information to determine the runoff hydrograph. Figure E-6 is used to plot both the unadjusted and adjusted mass rainfall curves. The Flood Computation Form is contained in the pocket in the back of this manual.



ORANGE COUNTY

HYDROLOGY MANUAL

TYPICAL REPORT FORMAT
FOR
UNIT HYDROGRAPH STUDY

Figure E-4

PROJECT: ______________________ DATE:

ENGINEER: ______________________

1. Enter the design storm return frequency **(years)** ______

2. Enter catchment lag **(hours)** ______

3. Enter the catchment area **(acres)** ______

4. Enter baseflow **(cfs/square mile)** ______

5. Enter S-Graph proportions **(decimal)**

Valley: Developed	______
Foothill	______
Mountain	______
Valley: Undeveloped	______

6. Enter maximum loss rate, F_m **(inch/hour)** ______

7. Enter low loss fraction, $\bar{Y}$ **(decimal)** ______

8. Enter watershed area-averaged 5-minute point rainfall **(inches)*** ______

   Enter watershed area-averaged 30-minute point rainfall **(inches)*** ______

   Enter watershed area-averaged 1-hour point rainfall **(inches)*** ______

   Enter watershed area-averaged 3-hour point rainfall **(inches)*** ______

   Enter watershed area-averaged 6-hour point rainfall **(inches)*** ______

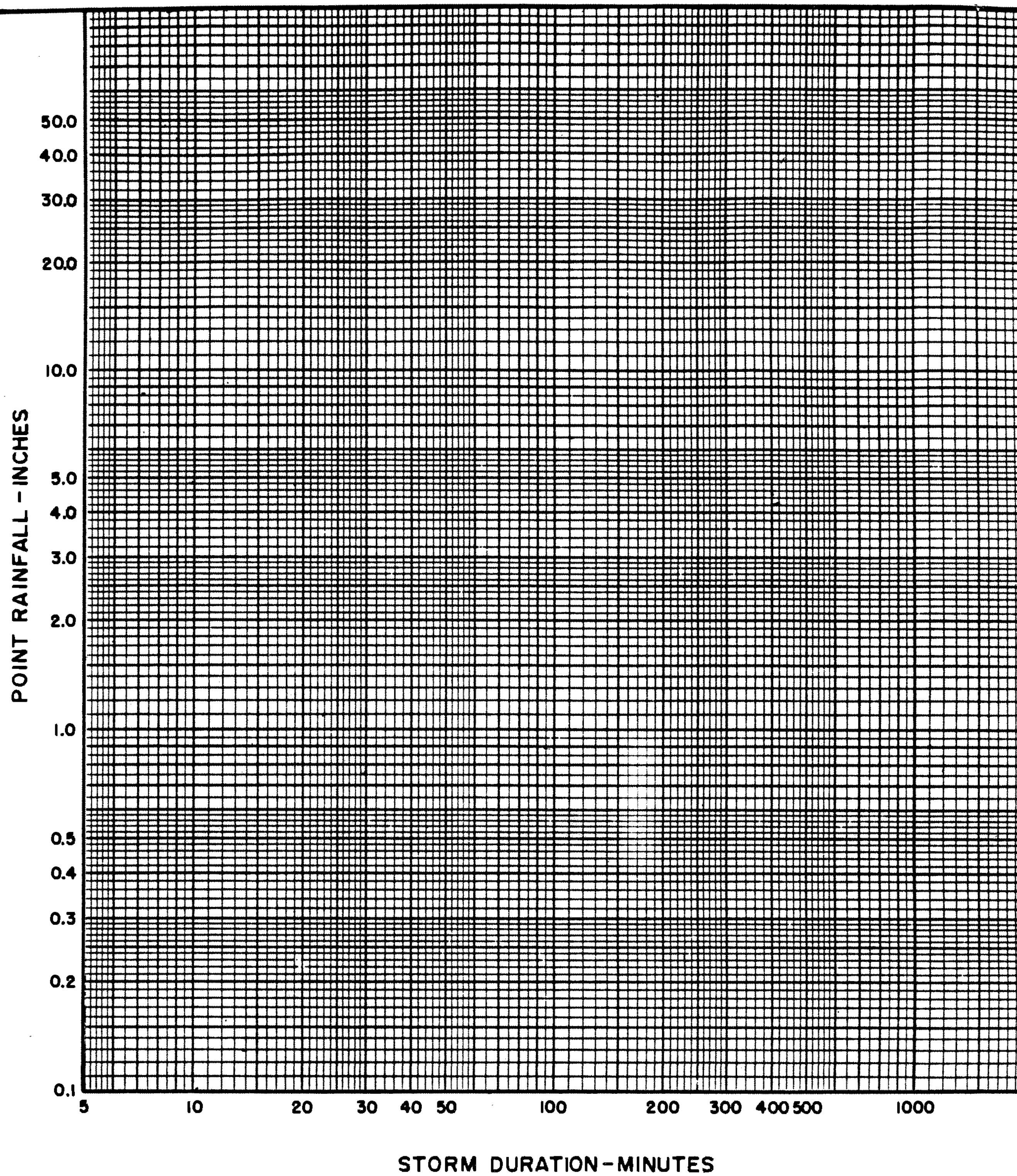
   Enter watershed area-averaged 24-hour point rainfall **(inches)*** ______

9. Enter 24-hour storm unit interval **(minutes)** ______

*Note: enter values unadjusted by depth-area factors

ORANGE COUNTY
HYDROLOGY MANUAL

WATERSHED INFORMATION FORM



PROJECT LOCATION ____________________

NOTES ____________________

____

ORANGE COUNTY

HYDROLOGY MANUAL

AREA - AVERAGED MASS RAINFALL PLOTTING SHEET

Figure E-6

E.11. INSTRUCTIONS FOR SYNTHETIC UNIT HYDROGRAPH METHOD HYDROLOGY CALCULATIONS

I. Synthetic Unit Hydrograph Development

A. On a USGS topographic quadrangle sheet or other topographic map of suitable scale, outline the watershed boundary.

B. Calculate the catchment time of concentration (Tc) by using either a rational method analysis for the T-year storm, or by using the peak flowrate curves of section L.

1. Catchment lag is computed by

$$\text{lag} = 0.8Tc$$

2. For certain large scale natural condition catchment studies, the Agency may use the lag relationship given by

$$\text{lag (hours)} = 24\bar{n}(L \cdot L_{ca}/S^{0.50})^{0.38}$$

where

A	=	drainage area (square miles)
L	=	length of longest watercourse (miles)
L_{ca}	=	length along the longest watercourse, measured upstream to a point opposite the centroid of the area (miles)
H	=	difference in elevation between the concentration point and the most remote point of the basin (feet)
S	=	overall slope of longest watercourse between headwaters and concentration point (S = H/L, feet per mile)
$\bar{n}$	=	visually estimated average basin factor from Figure E-2.

C. Select a unit time period. To adequately define the unit hydrograph the unit time period should be about 20 percent of lag time, and never more than 25 percent of lag time. If possible, use the unit time of the synthetic critical storm pattern of 5-minutes.

D. Select the S-graph applicable to the drainage basin (Figures E-3a,b,c,d). Determine the average percentage of the ultimate discharge for each unit period. In reading the percentage of ultimate discharge from the S-graph, the average ordinate over the time increment should be determined rather than the mean of the ordinates at the beginning and end of the time increment (see example problem).

E. Compute the unit distribution graph by subtracting from the percentage of ultimate discharge for each unit time period, the percentage of ultimate discharge for the previous time period.

F. Compute the ordinates of the synthetic unit hydrograph (unit graph) by multiplying the distribution graph values by the ultimate discharge K, using:

$$K \text{ (cfs)} = 645A/T$$

where

A = drainage area (square miles)
T = unit time period (hours)

II. T-Year Design Storm Pattern Development

A. Using the appropriate T-year point precipitation values from Table B.2, compute the area-averaged precipitation values for the 5-minute, 30-minute, 1-hour, 3-hour, 6-hour, and 24-hour durations.

B. Adjust all point precipitation values for areal effect by using Figure B-6.

C. Develop a synthetic critical storm peak rainfall mass plot using Figure B-7 (see example problem for demonstration).

D. Using the unit interval duration for the unit hydrograph development, calculate the synthetic storm unit interval rainfall quantities by successive subtraction of mass peak rainfall values, each offset in time by one unit period.

E. Arrange the unit rainfall quantities determined in step D into the critical storm pattern shown in Figures B-5a,b,c. For most hydrology studies, only the peak 3-hours of the synthetic critical storm may need consideration.

III. Runoff Hydrograph Development

A. Find the pervious area loss rates for subareas within the drainage area using Figures C-3 and C-4. Adjust these rates to account for impervious area using the relationship below, and then compute an area-averaged maximum loss rate for the catchment.

$$F_m = a_p F_p$$

where

F_m = maximum loss rate (inches/hour)

a_p = pervious area fraction (decimal percent of total area). See Figure C-4.

F_p = maximum loss rate for pervious areas fraction. See Section C.6.4.

B. Compute the low loss fraction, $\bar{Y}$. Use F^* in each unit time period where the maximum loss rate F_m exceeds the low loss rate F^*, ($F^* = \bar{Y} \cdot I$, see Section C.6.3).

C. Compute the unit effective rainfall for each unit time period by subtracting the unit loss from the unit rainfall.

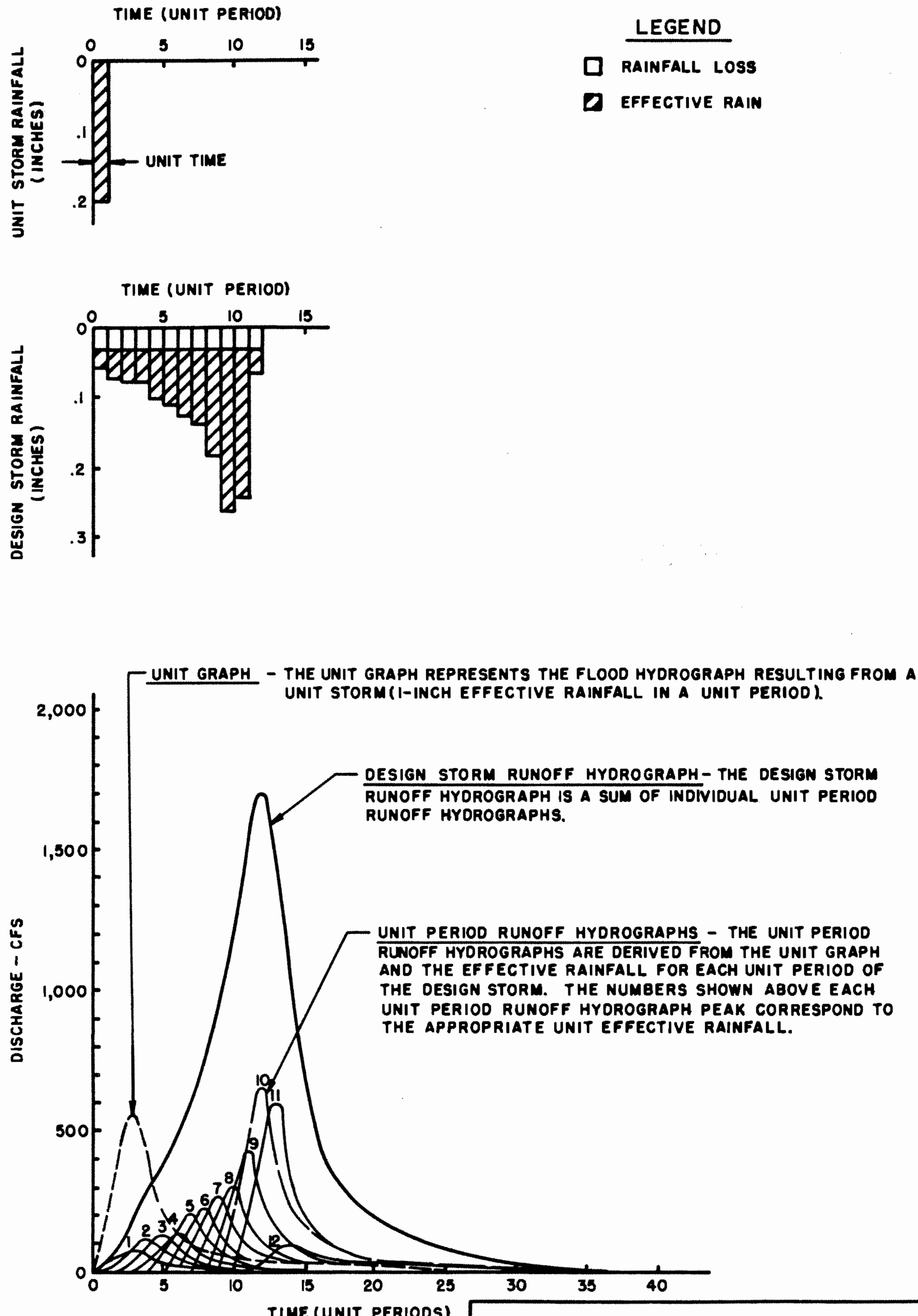
D. Compute the flood hydrograph.

1. Multiply the effective unit rainfall for the first unit time period by each synthetic unit hydrograph value to determine the flood hydrograph which would result from that rainfall increment.

2. Repeat the above process for each succeeding effective rainfall value, advancing the resultant flood hydrographs one unit time period for each computation cycle. See Figure E-7.

3. Sum the flow ordinates found in the steps above to determine the average flow ordinate per unit time period for the design storm flood hydrograph.

E. Add the appropriate base flow to the flood hydrograph ordinates determined in Step D.



ORANGE COUNTY

HYDROLOGY MANUAL

DERIVATION OF A RUNOFF HYDROGRAPH

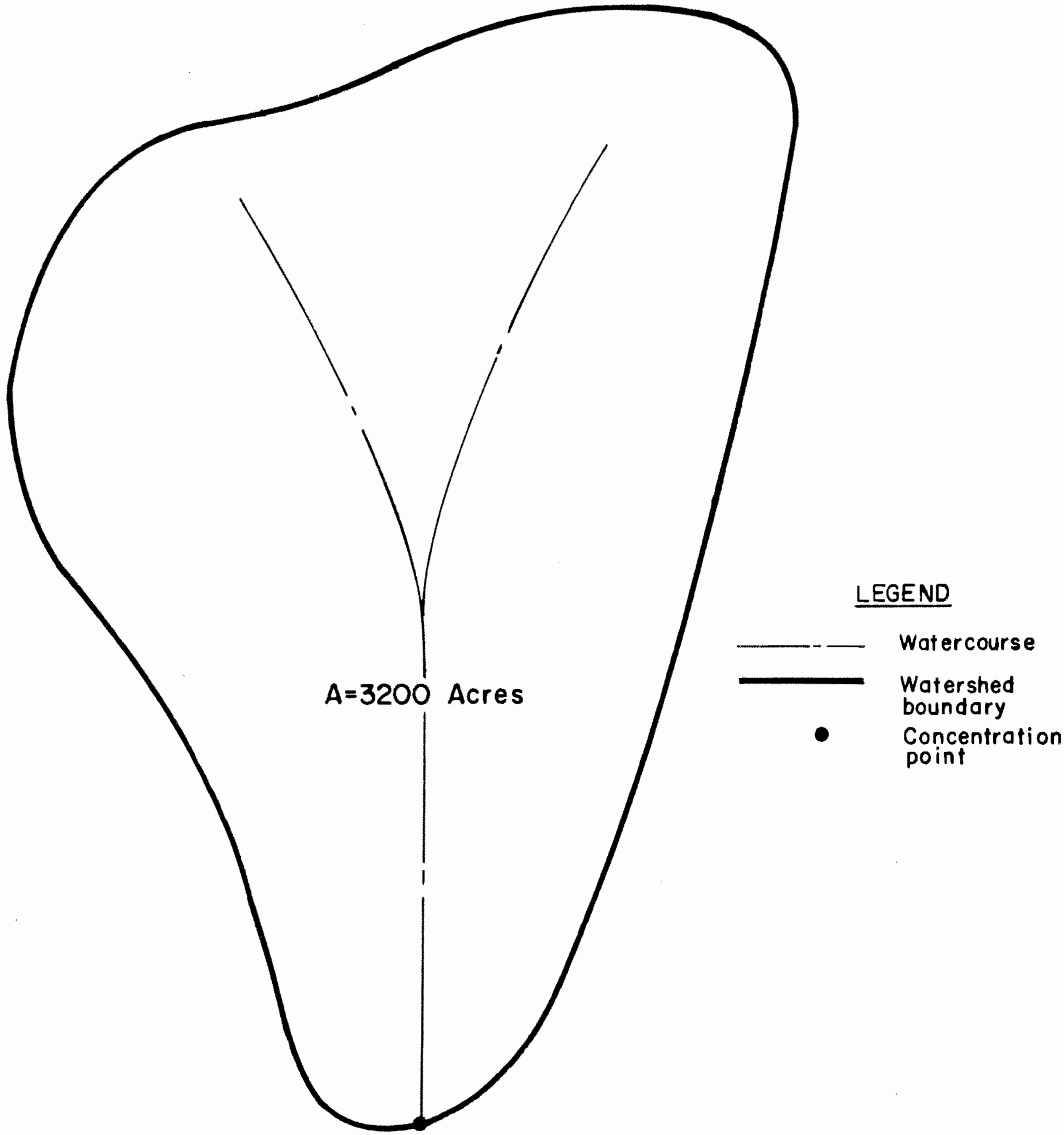
Figure E-7

E.12. EXAMPLE PROBLEM

The following example problem illustrates the format suggested for synthetic unit hydrograph hydrology studies to be submitted for review. Additional and expanded examples are contained in the Hydrology Manual Workbook which can be obtained separately from the Agency. In the following, an example watershed is analyzed using the Agency unit hydrograph approach. The example problem presentation contains the following information:

<u>Description</u>

- o Watershed Map including Boundary and Geometric Information
- o Watershed Information Form
- o Adjusted and Unadjusted Mass Rainfall Plots (Depth-Area Effects)
- o 24-Hour Storm Unit Rainfall Determination (30-Minute Unit Interval)
- o Watershed-Loss Information Map
- o Area-Averaged Maximum Loss Rate (F_m) Determination
- o Area-Averaged Low Loss Fraction ($\overline{Y}$) Determination
- o Effective Rainfall Determination
- o 24-Hour Critical Storm
- o S-Graph Approximation
- o Unit Hydrograph Determination
- o Runoff Hydrograph Determination
- o Runoff Hydrograph



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

WATERSHED BOUNDARY

PROJECT: *EXAMPLE PROBLEM* DATE:

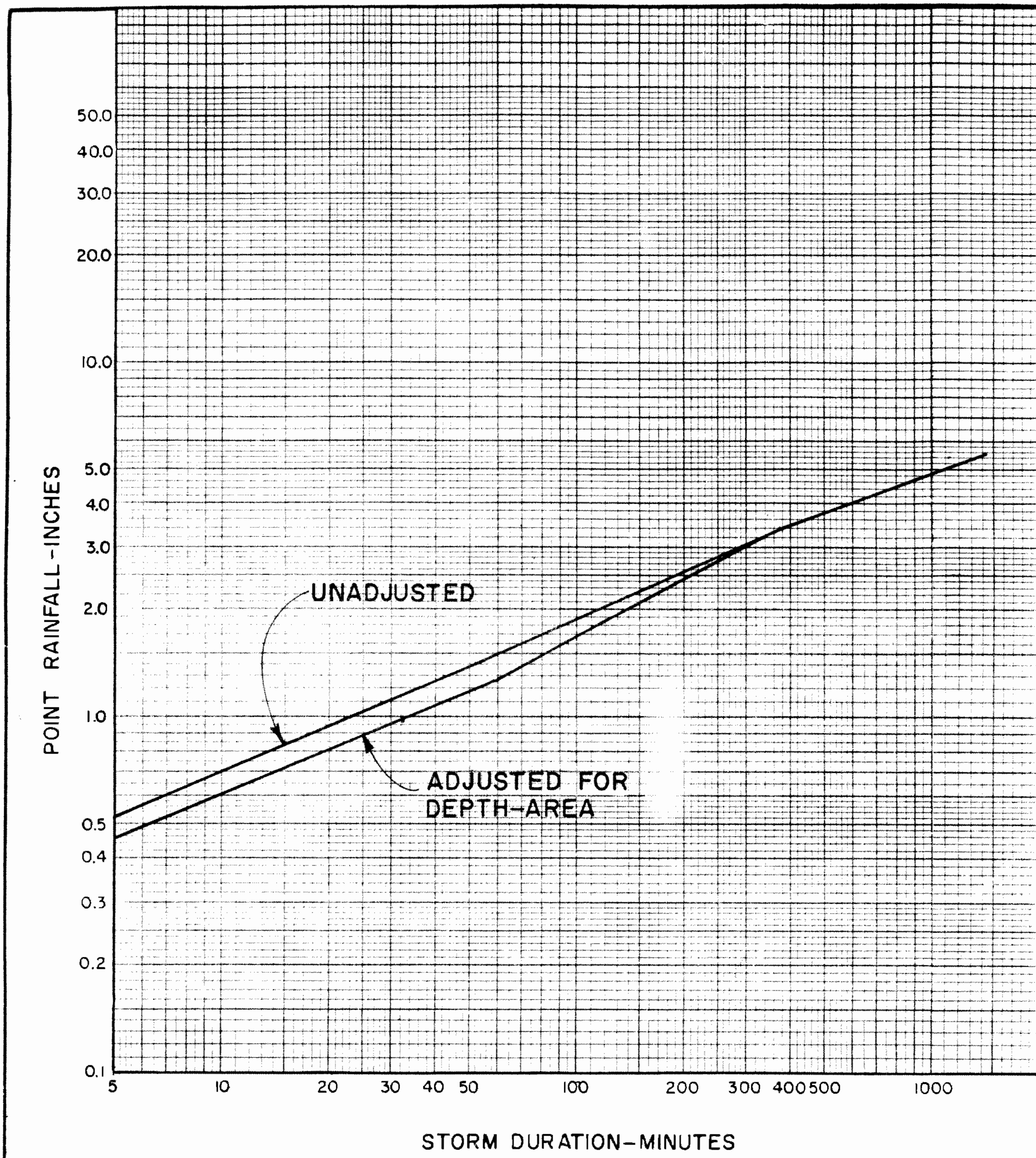
ENGINEER:

1.	Enter the design storm return frequency **(years)**	100
2.	Enter catchment lag **(hours)**	0.75
3.	Enter the catchment area **(acres)**	3200
4.	Enter baseflow **(cfs/square mile)**	10
5.	Enter S-Graph proportions **(decimal)**	
	Valley: Developed	1.0
	Foothill	0.0
	Mountain	0.0
	Valley: Undeveloped	0.0
6.	Enter maximum loss rate, F_m **(inch/hour)**	0.19
7.	Enter low loss fraction, $\bar{Y}$ **(decimal)**	0.337
8.	Enter watershed area-averaged 5-minute point rainfall **(inches)***	0.52
	Enter watershed area-averaged 30-minute point rainfall **(inches)***	1.09
	Enter watershed area-averaged 1-hour point rainfall **(inches)***	1.45
	Enter watershed area-averaged 3-hour point rainfall **(inches)***	2.43
	Enter watershed area-averaged 6-hour point rainfall **(inches)***	3.36
	Enter watershed area-averaged 24-hour point rainfall **(inches)***	5.63
9.	Enter 24-hour storm unit interval **(minutes)**	5.0

*Note: enter values unadjusted by depth-area factors

ORANGE COUNTY
HYDROLOGY MANUAL

WATERSHED INFORMATION FORM



PROJECT LOCATION *EXAMPLE PROBLEM*

NOTES *100-YEAR STORM*

ORANGE COUNTY

HYDROLOGY MANUAL

AREA – AVERAGED MASS RAINFALL PLOTTING SHEET

UNIT HYDROGRAPH STUDY:
EXAMPLE PROBLEM UNIT RAINFALL DETERMINATION

(Example Unit Period = 5 minutes)

Peak Rainfall Unit Number	Adjusted Mass Rainfall (inches)	Unit Rainfall (inches)
1	0.45	0.45
2	0.60	0.15
3	0.71	0.11
4	0.80	0.09
5	0.88	0.08
6	0.95	0.07
7	1.02	0.07
8	1.08	0.06
9	1.13	0.05
10	1.19	0.06
11	1.24	0.05
12	1.28	0.04
13	1.33	0.05
14	1.39	0.05
15	1.45	0.06
16	1.50	0.05
17	1.55	0.05
18	1.60	0.05
19	1.65	0.05
20	1.70	0.05
21	1.74	0.04
22	1.79	0.05
23	1.84	0.05
24	1.89	0.05
25	1.93	0.04
26	1.97	0.04
27	2.01	0.04
28	2.05	0.04
29	2.09	0.04
30	2.13	0.04
31	2.17	0.04
32	2.21	0.04
33	2.25	0.04
34	2.29	0.04
35	2.33	0.04
36	2.38	0.04

TIME = 3 HOURS TOTAL = 2.38 INCHES

UNIT HYDROGRAPH STUDY:
EXAMPLE PROBLEM WATERSHED LOSS DETERMINATIONS

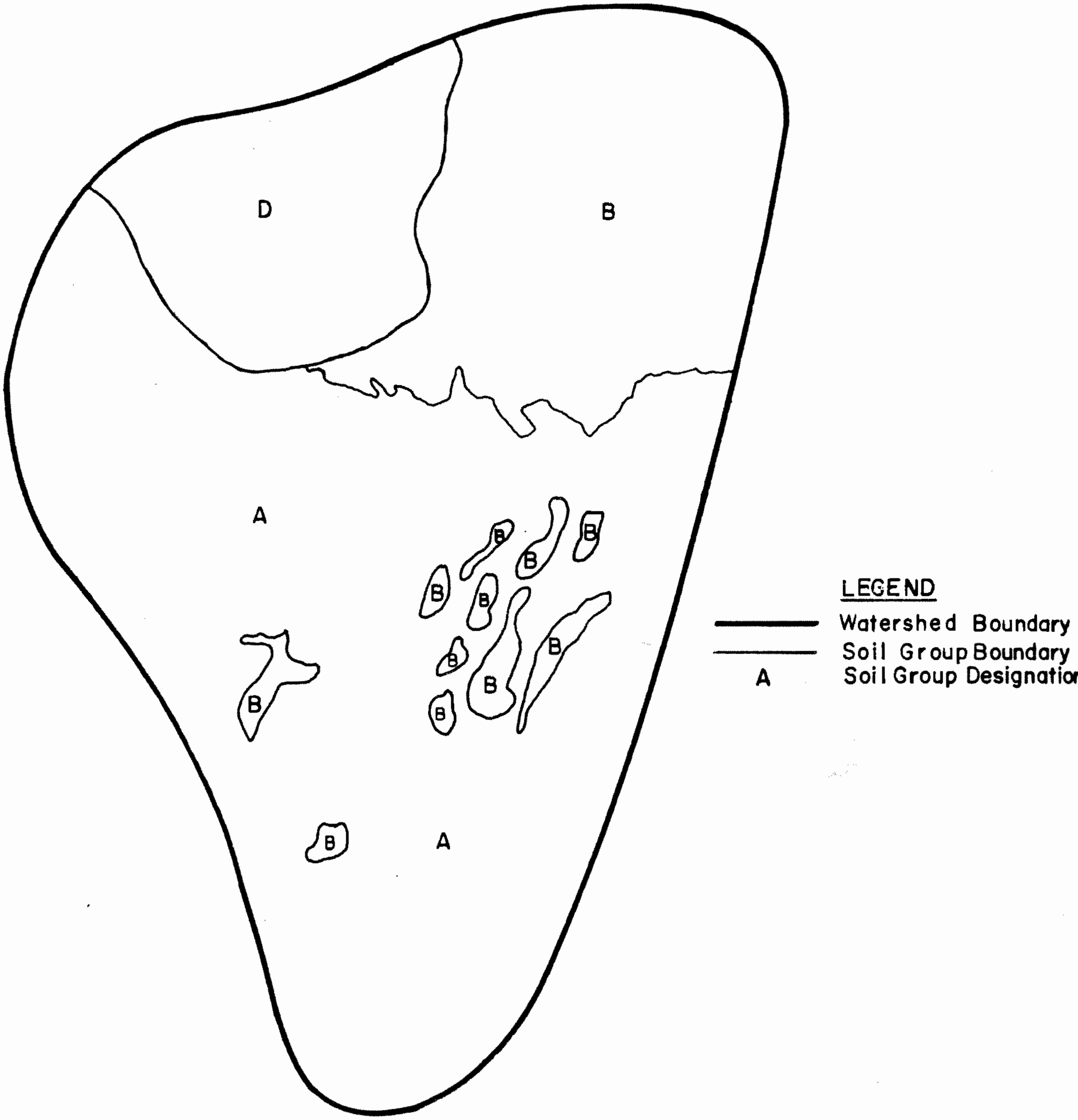
Area-Averaged Maximum Loss Rate, F_m

1. Using the watershed soil and development characteristics, estimate the area-averaged maximum loss rate:

Land Use and Condition	Area Fraction	Soil Group	F_p (inch/hour) (Table C.2.)	a_p (Fig. C-4)	F_m (inch/hour)
Woodland; good cover (100% pervious)	.15	B	0.30	1.0	0.30
Woodland; good cover (100% pervious)	.15	D	0.20	1.0	0.20
Residential:S.F. (1/2 acre) Lots (60% pervious*)	.42	A	0.40	0.60	0.24
Residential:S.F. (1/2 acre) Lots (60% pervious*)	.03	B	0.30	0.60	0.18
Commercial: (10% pervious)	.23	A	0.40	0.10	0.04
Commercial: (10% pervious)	.02	B	0.30	0.10	0.03

Area-Averaged Adjusted Loss Rate (inch/hour) = 0.19

* Field conditions indicate use of the lower end of the suggested percent pervious range.



ORANGE COUNTY
HYDROLOGY MANUAL

EXAMPLE PROBLEM
WATERSHED-LOSS DETERMINATION
INFORMATION MAP

UNIT HYDROGRAPH STUDY:
EXAMPLE PROBLEM WATERSHED LOSS DETERMINATIONS

Area-Averaged Low Loss Rate Fraction, $\bar{Y}$

1. Referring to watershed soil group maps, estimate area-averaged composite curve numbers (see Section C):

Land Use and Condition	Area Fraction	Soil Group	Curve Number $CN^{(1)}$ (Fig. C-3)	$S^{(2)}$	Pervious Area Yield Fraction $Y^{(3)}$
Woodland; good cover (100% pervious)	.15	B	55 (75)	3.33	0.53
Woodland; good cover (100% pervious)	.15	D	77 (93)	0.75	0.86
Residential: S.F. (1/2 acre) Lots (60% pervious)$^{(5)}$	.25	A	32 (52)	9.23	0.20
	.17	A	98	0.20	0.96
Residential: S.F. (1/2 acre) Lots (60% pervious)$^{(5)}$	.018	B	56 (76)	3.16	0.54
	.012	B	98	0.20	0.96
Commercial: (10% pervious)	.023	A	32 (52)	9.23	0.20
	.207	A	98	0.20	0.96
Commercial: (10% pervious)	.002	B	56 (76)	3.16	0.54
	.018	B	98	0.20	0.96

Area-Averaged Catchment Yield Fraction (Y) = 0.663

Area-Averaged Low Loss Fraction $(\bar{Y})^{(4)}$ = 0.337

NOTES:

(1): (75) indicates AMC III CN (Table C.1)

(2): $S = (1000/CN)-10$

(3): $Y = (P24-0.2S)^2/((P24+0.8S)P24)$

(4): $\bar{Y} = 1-Y$

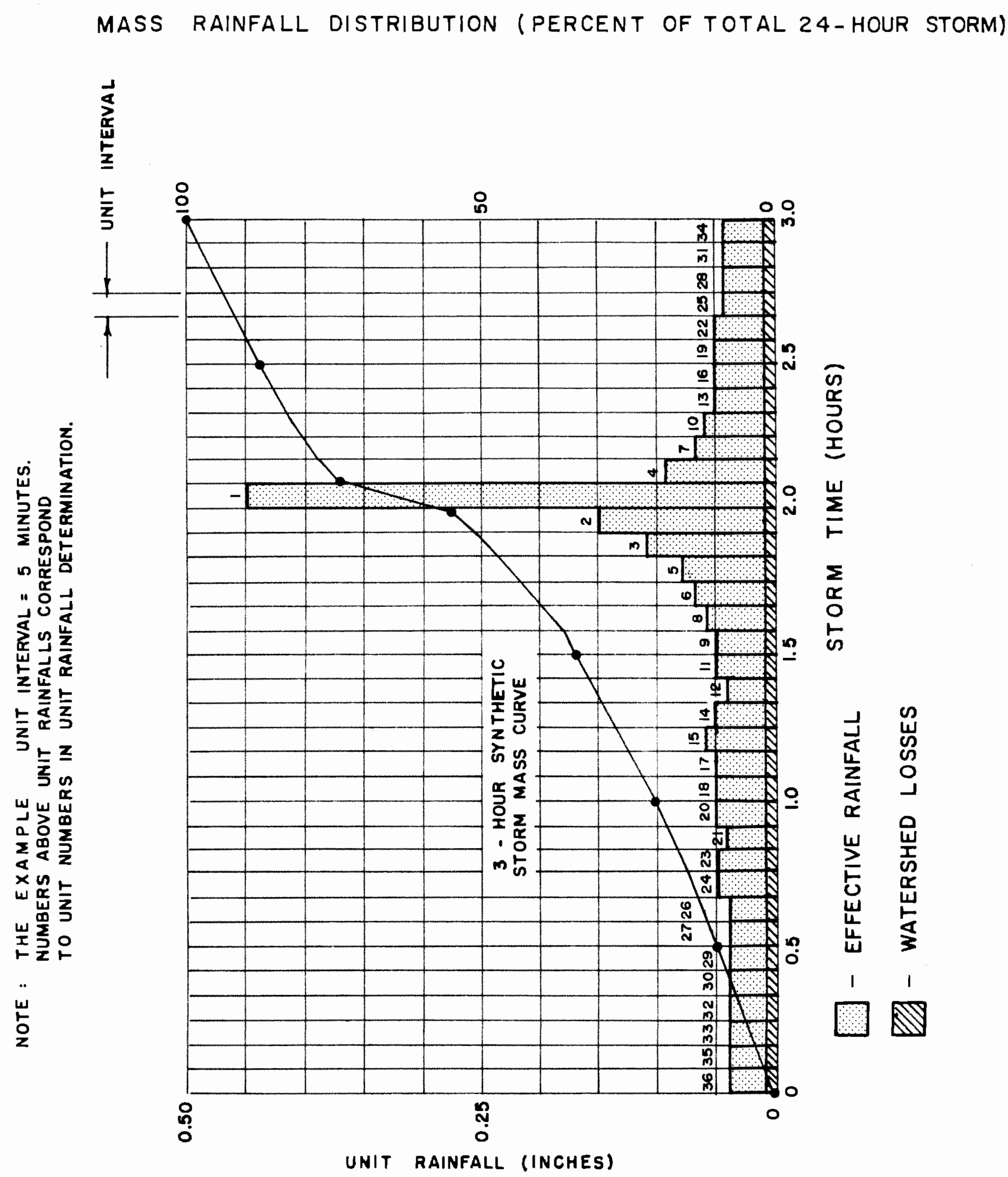
(5): Field conditions indicate use of the lower end of the suggested pervious range

UNIT HYDROGRAPH STUDY:
EXAMPLE PROBLEM 3-HOUR STORM
EFFECTIVE RAINFALL DETERMINATION

(Example Unit Period = 5 minutes)

Unit Period Number	Unit Rainfall (inches)	Unit Loss (inches)	Effective Rainfall (inches)
1	.04	.013	.025
2	.04	.013	.026
3	.04	.013	.026
4	.04	.014	.027
5	.04	.014	.027
6	.04	.014	.028
7	.04	.015	.029
8	.04	.015	.029
9	.05	.015	.030
10	.05	.016 *	.031
11	.04	.015	.029
12	.05	.016 *	.034
13	.05	.016 *	.036
14	.05	.016 *	.037
15	.06	.016 *	.040
16	.05	.016 *	.042
17	.04	.015	.029
18	.05	.016 *	.031
19	.05	.016 *	.037
20	.06	.016 *	.042
21	.07	.016 *	.053
22	.08	.016 *	.062
23	.11	.016 *	.094
24	.15	.016 *	.135
25	.45	.016 *	.438
26	.09	.016 *	.074
27	.07	.016 *	.054
28	.06	.016 *	.034
29	.05	.016 *	.044
30	.05	.016 *	.039
31	.05	.016 *	.035
32	.05	.016 *	.032
33	.04	.015	.030
34	.04	.014	.028
35	.04	.014	.027
36	.04	.013	.026
TOTAL =	2.38	0.55	1.83

*Unit low loss exceeds unit adjusted loss

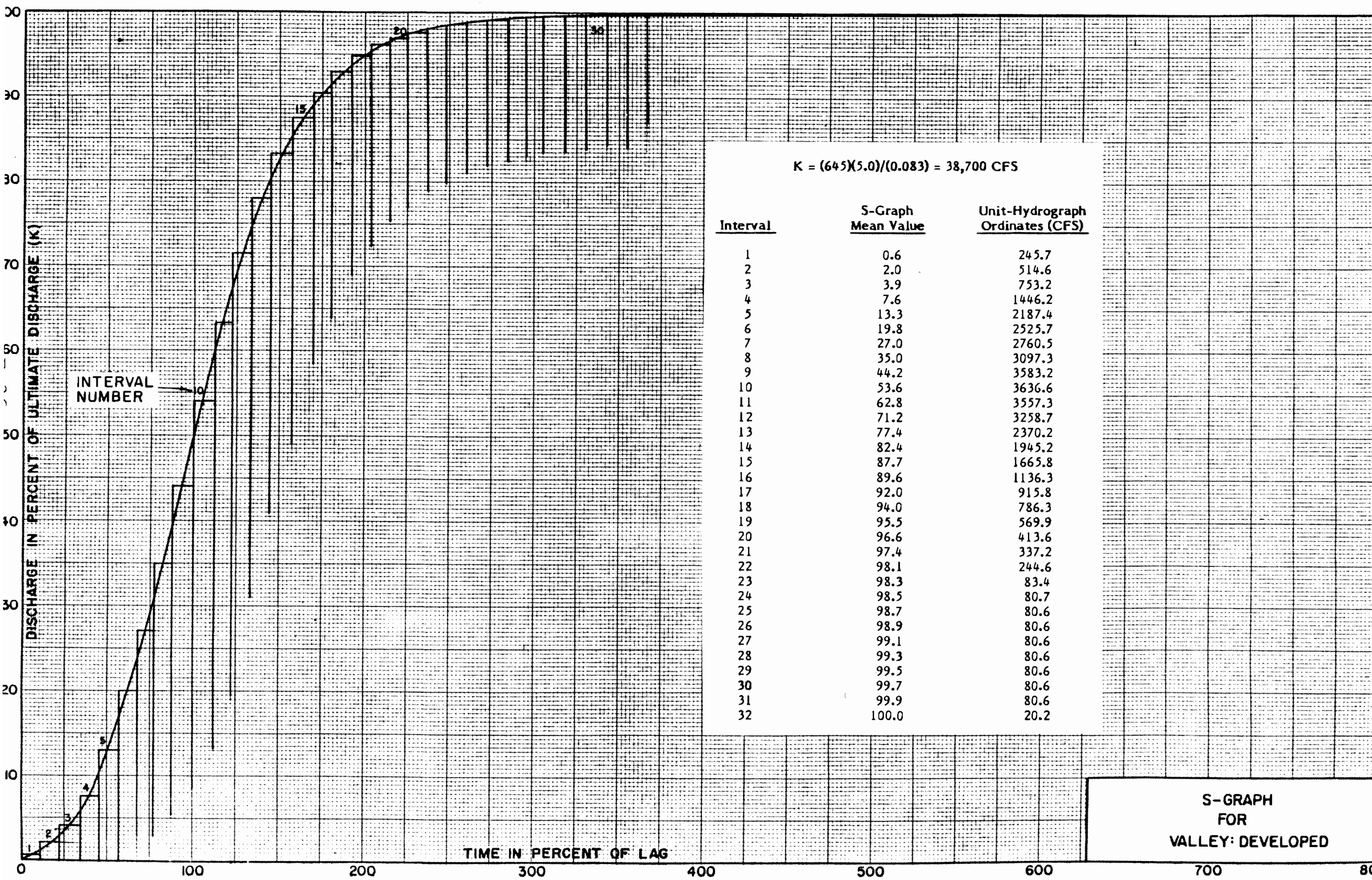


ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE

SYNTHETIC 3-HOUR CRITICAL STORM

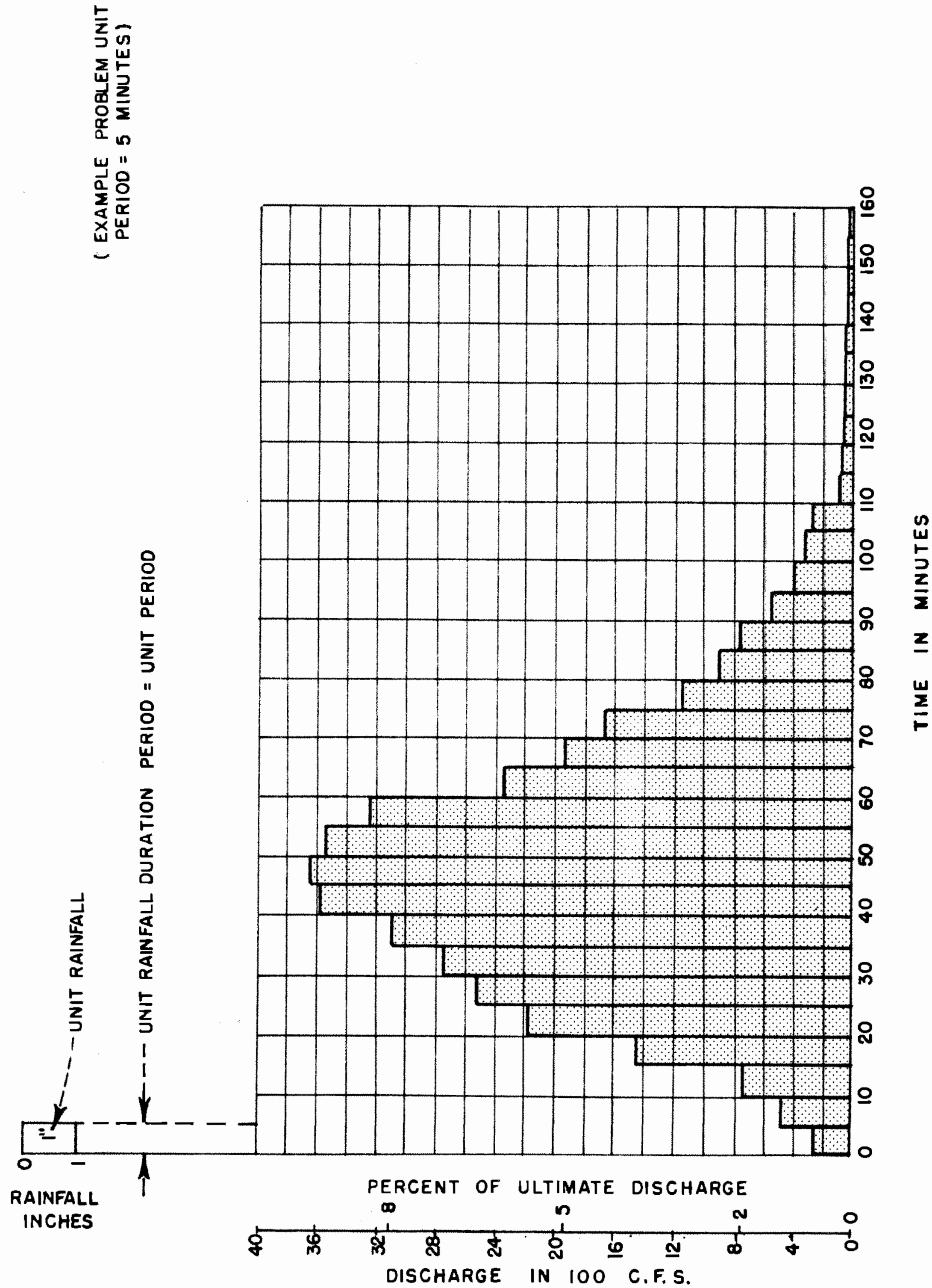


K = (645)(5.0)/(0.083) = 38,700 CFS

Interval	S-Graph Mean Value	Unit-Hydrograph Ordinates (CFS)
1	0.6	245.7
2	2.0	514.6
3	3.9	753.2
4	7.6	1446.2
5	13.3	2187.4
6	19.8	2525.7
7	27.0	2760.5
8	35.0	3097.3
9	44.2	3583.2
10	53.6	3636.6
11	62.8	3557.3
12	71.2	3258.7
13	77.4	2370.2
14	82.4	1945.2
15	87.7	1665.8
16	89.6	1136.3
17	92.0	915.8
18	94.0	786.3
19	95.5	569.9
20	96.6	413.6
21	97.4	337.2
22	98.1	244.6
23	98.3	83.4
24	98.5	80.7
25	98.7	80.6
26	98.9	80.6
27	99.1	80.6
28	99.3	80.6
29	99.5	80.6
30	99.7	80.6
31	99.9	80.6
32	100.0	20.2

S-GRAPH
FOR
VALLEY: DEVELOPED

EXAMPLE PROBLEM



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

UNIT DISTRIBUTION GRAPH

ORANGE COUNTY

HYDROLOGY MANUAL

SYNTHETIC UNIT HYDROGRAPH METHOD

Flood Hydrograph Calculation Form

Project EXAMPLE PROBLEM

By TRW Date 5-15-86

Checked MHS Date 5-15-86

Sheet 1 / 2

UNIT GRAPH (cfs)	UNIT PER.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	FLOOD HYDROGRAPH (cfs)	BASE-FLOW (cfs)	DESIGN FLOOD HYDROGRAPH (cfs)
	EFFEC RAIN (in)	.025	.026	.026	.027	.027	.028	.029	.029	.030	.031	.029	.034	.036	.037	.040	.042	.029	.031	.037	.042	.053	.062	.094	.135			
246	1	6																								6	50	56
515	2	14	6																							20	50	70
753	3	19	15	6																						40	50	90
1446	4	36	20	15	7																					78	50	128
2187	5	55	38	20	16	7																				136	50	186
2526	6	63	57	38	20	16	7																			201	50	251
2761	7	69	66	57	39	20	16	7																		274	50	324
3097	8	77	72	66	59	39	21	17	7																	358	50	408
3583	9	90	81	72	68	59	40	22	17	7																456	50	506
3637	10	91	93	81	75	68	61	42	22	17	8															558	50	608
3557	11	89	95	93	84	75	71	63	42	23	18	7														660	50	710
3259	12	81	92	95	97	84	77	73	63	43	23	17	8													753	50	803
2370	13	59	85	92	98	97	87	80	73	66	45	22	20	9												833	50	883
1945	14	49	62	85	96	98	100	90	80	76	68	42	26	21	9											902	50	952
1666	15	42	51	62	88	96	102	104	90	83	78	63	49	27	21	10										966	50	1016
1136	16	28	43	51	64	88	100	105	164	93	86	73	74	52	28	23	10									1022	50	1072
916	17	23	30	43	53	64	91	103	105	107	96	80	86	79	54	30	24	7								1075	50	1125
786	18	20	24	30	45	53	66	95	103	109	111	90	94	91	81	58	32	17	8							1127	50	1177
570	19	14	26	24	31	45	54	69	95	107	113	104	105	99	93	87	61	22	18	9						1170	50	1220
414	20	10	15	20	25	31	47	56	69	98	110	105	122	111	102	101	92	42	23	21	10					1210	50	1260
337	21	8	11	15	21	25	32	48	56	71	101	103	124	129	115	110	106	63	45	28	24	13				1248	50	1298
245	22	6	9	11	15	21	26	33	48	58	73	95	121	131	133	124	116	73	68	54	32	30	15			1292	50	1342
83	23	2	6	9	11	15	22	27	33	50	60	69	111	128	135	143	130	80	78	81	61	40	36	23		1350	50	1400
81	24	2	2	6	9	11	16	23	27	34	52	56	81	117	132	145	150	90	86	93	92	77	47	54	33	1435	50	1485
81	25	2	2	2	7	9	12	17	23	27	35	48	66	85	121	142	153	104	96	102	106	116	90	71	78			
81	26	2	2	2	2	7	9	12	17	24	28	33	57	70	88	130	149	105	111	115	116	134	136	136	102			
81	27	2	2	2	2	2	7	10	12	17	24	27	39	60	72	95	137	103	113	133	130	146	157	206	195			
81	28	2	2	2	2	2	2	7	10	12	18	23	31	41	62	78	100	95	110	135	150	164	171	237	295			
81	29	2	2	2	2	2	2	2	7	10	13	17	27	33	42	67	82	69	101	132	153	190	192	260	341			
81	30	2	2	2	2	2	2	2	2	7	10	12	19	28	34	45	70	56	73	121	149	193	222	291	373			
81	31	2	2	2	2	2	2	2	2	2	7	10	14	21	29	37	48	48	60	88	137	189	225	337	418			
20	32	1	2	2	2	2	2	2	2	2	3	7	11	15	21	31	38	33	52	72	100	173	221	342	484			
			1	2	2	2	2	2	2	2	3	2	8	12	15	23	33	27	35	62	82	126	202	334	491			
				1	2	2	2	2	2	2	3	2	3	9	12	17	24	23	28	42	70	103	147	306	480			
					1	2	2	2	2	2	3	2	3	3	9	13	17	17	24	34	48	88	121	223	440			
						1	2	2	2	2	3	2	3	3	3	10	14	12	18	29	38	60	103	183	320			
							1	2	2	2	3	2	3	3	3	3	10	10	13	21	33	49	70	157	263			
								1	2	2	3	2	3	3	3	3	3	7	10	15	24	42	57	107	225			
									1	2	3	2	3	3	3	3	3	2	7	12	17	30	49	86	153			
										1	3	2	3	3	3	3	3	2	3	9	14	22	35	74	124			
											1	2	3	3	3	3	3	2	3	3	10	18	26	54	106			
												1	3	3	3	3	3	2	3	3	3	13	21	39	77			
													1	3	3	3	3	2	3	3	3	4	15	32	56			
														1	3	3	3	2	3	3	3	4	5	23	45			
															1	3	3	2	3	3	3	4	5	8	33			
																1	3	2	3	3	3	4	5	8	11			
																	1	2	3	3	3	4	5	8	11			
																		1	3	3	3	4	5	8	11			
																			1	3	3	4	5	8	11			
																				1	3	4	5	8	11			
																					1	4	5	8	11			
																						1	5	8	11			
																							1	8	11			
																								2	11			
																									3			

ORANGE COUNTY

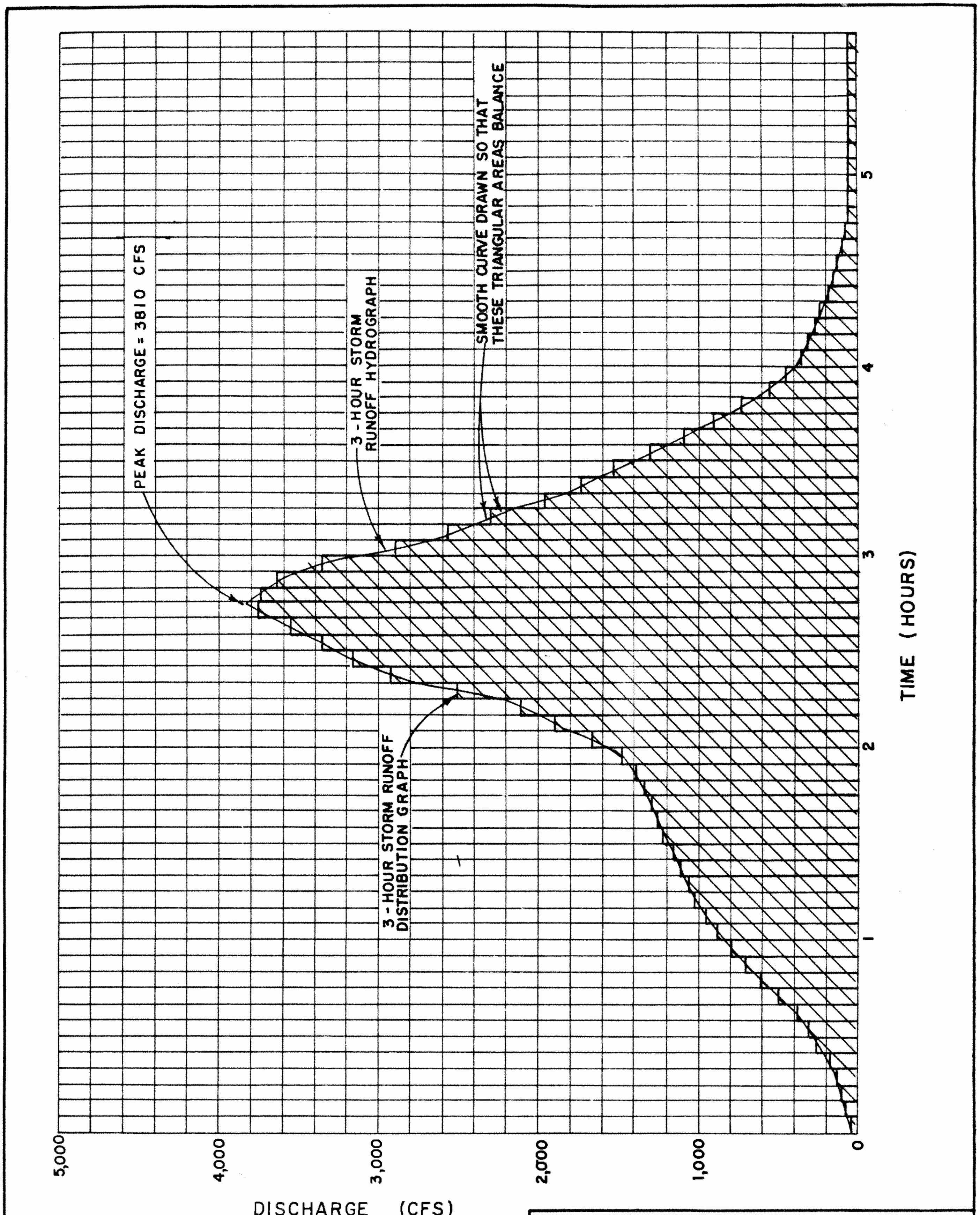
HYDROLOGY MANUAL

FLOOD HYDROGRAPH CALCULATION FORM

ORANGE COUNTY HYDROLOGY MANUAL	SYNTHETIC UNIT HYDROGRAPH METHOD Flood Hydrograph Calculation Form	Project EXAMPLE PROBLEM By TRW Date 5-15-86 Checked MHS Date 5-15-86	Sheet 2 / 2

	EFFEC RAIN (in)	.438	.074	.054	.034	.044	.039	.035	.032	.030	.028	.027	.026	FLOOD HYDRO-GRAPH (cfs)	BASE-FLOW (cfs)	DESIGN FLOOD HYDRO-GRAPH (cfs)
UNIT GRAPH (cfs)	UNIT PER.	25	26	27	28	29	30	31	32	33	34	35	36			
246	1	108												1622	50	1672
515	2	252	18											1857	50	1907
753	3	330	43	13										2079	50	2129
1446	4	633	56	31	8									2479	50	2529
2187	5	958	107	41	20	11								2887	50	2937
2526	6	1106	162	78	26	25	10							3126	50	3176
2761	7	1209	187	118	49	33	22	9						3313	50	3363
3097	8	1356	204	136	74	64	29	20	8					3511	50	3561
3583	9	1569	229	149	86	96	56	26	18	7				3706	50	3756
3637	10	1593	265	167	94	111	85	51	34	17	7			3696	50	3746
3557	11	1558	268	193	105	121	99	77	46	23	16	7		3570	50	3620
3259	12	1427	263	196	122	136	108	88	70	43	21	16	6	3306	50	3356
2370	13	1038	241	192	124	158	121	97	81	66	40	20	15	2893	50	2893
1945	14	852	175	176	121	160	140	108	88	76	61	39	20	2528	50	2578
1666	15	730	144	128	111	157	142	125	99	83	71	59	38	2266	50	2316
1136	16	498	123	105	81	143	139	127	115	93	77	68	57	1930	50	1980
916	17	401	84	90	66	104	127	124	116	107	87	75	66	1687	50	1737
786	18	344	68	61	57	86	92	114	114	109	100	84	72	1478	50	1528
570	19	250	58	49	39	73	76	83	104	107	102	97	81	1250	50	1300
414	20	181	42	42	31	50	65	68	76	98	100	98	93	1042	50	1092
337	21	148	31	31	27	40	44	58	62	71	91	96	95	862	50	912
245	22	107	25	22	19	35	36	40	53	58	66	88	92	684	50	734
83	23	36	18	18	14	25	31	32	36	50	54	64	85	503	50	553
81	24	35	6	13	11	18	22	28	29	34	47	53	62	396	50	446
81	25	35	6	4	8	15	16	20	25	27	32	45	51	319	50	369
81	26	35	6	4	3	11	13	14	18	24	26	31	43	260	50	310
81	27	35	6	4	3	4	10	12	13	17	22	25	30	210	50	260
81	28	35	6	4	3	4	3	9	11	12	16	21	24	173	50	223
81	29	35	6	4	3	4	3	3	8	10	12	15	20	143	50	193
81	30	35	6	4	3	4	3	3	3	7	9	11	15	116	50	166
81	31	35	6	4	3	4	3	3	3	2	7	9	11	93	50	143
20	32	9	6	4	3	4	3	3	3	2	2	7	9	55	50	105
			1	4	3	4	3	3	3	2	2	2	6	33	50	83
				1	3	4	3	3	3	2	2	2	2	25	50	75
					1	4	3	3	3	2	2	2	2	22	50	72
						1	3	3	3	2	2	2	2	18	50	68
							1	3	3	2	2	2	2	15	50	65
								1	3	2	2	2	2	12	50	62
									1	2	2	2	2	9	50	59
										1	2	2	2	7	50	57
											1	2	2	5	50	55
												1	2	3	50	53
													1	1	50	51

ORANGE COUNTY
HYDROLOGY MANUAL
FLOOD HYDROGRAPH
CALCULATION FORM



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM
3-HOUR STORM
RUNOFF HYDROGRAPH

SECTION F

FLOW-THROUGH BASIN ANALYSIS

F.1. INTRODUCTION

There are two types of routing that are important in watershed planning; reservoir routing and streamflow routing. Both processes operate upon an inflow hydrograph to produce an outflow hydrograph. In this section, reservoir routing will be presented based on the modified Pul's method. Section H will present the convex method for streamflow routing. It should be noted that the Agency has hydraulic design criteria which must be considered in addition to the hydrologic criteria established in this manual.

Section E of this hydrology manual includes a detailed discussion of the unit hydrograph approach to be used for hydrologic studies of watersheds. Section B.5 presents a discussion on the appropriate design storm for watershed with flow-through detention basins.

For a discussion on the use of detention basins see Appendix IV.

F.2. DETENTION BASIN ANALYSIS

F.2.1. Detention Basin Routing Procedure

The modified Pul's (refs. 2, 3, 5) method may be used for detention basin routing studies. The basin routing relationships are based upon the formula

$$I - O = \frac{\Delta S}{\Delta t} \qquad \text{(F.1)}$$

where

I	=	basin inflow rate (cfs)
O	=	basin outflow rate (cfs)
ΔS	=	change in basin storage during the time step (cubic feet)
Δt	=	time step (sec)

Equation (F.1) is approximated by replacing the variables I and O by an average value during the timestep using

$$I = \frac{I_1 + I_2}{2} \tag{F.2}$$

$$O = \frac{O_1 + O_2}{2} \tag{F.3}$$

where the subscript 1 indicates the beginning of a time period and subscript 2 indicates the end of the subject time period. Substituting (F.2) and (F.3) into the basin routing equation of (F.1) and rearranging terms gives

$$(S_2 + O_2 \Delta t/2) = (S_1 - O_1 \Delta t/2) + (I_1 + I_2) \Delta t/2 \tag{F.4}$$

In (F.4), the right side is known from the previously computed values of storage, S_1, outflow, O_1, and the average basin inflow $(I_1+I_2)/2$ for time step Δt.

The solution of the basin routing problem requires the following information:

1. Known initial conditions for basin storage and outflow
2. A routing timestep, Δt
3. The basin inflow hydrograph
4. Basin volume vs. depth and outflow vs. depth relationships

To solve (F.4), a storage indication curve should be developed. Such a curve may be derived from the known storage-discharge relationship by selecting various values of depth and computing $(S+O\,\Delta t/2)$ from the associated values for storage and outflow. This quantity is plotted versus outflow such as shown in Figure F-4.

The solution procedure then proceeds with the following steps:

1. Determine the average inflow volume from the inflow hydrograph during the timestep Δt; i.e., calculate $(I_1 + I_2)\,\Delta t/2$.

2. Compute $S_1 - O_1\,\Delta t/2$ from the assumed initial condition of the basin flowdepth or the last computed values of S and O.

3. Use (F.4) to compute $(S_2 + O_2\,\Delta t/2)$.

4. Use the estimate from step 3 and the storage indication curve (see Figure F-4) to compute O_2.

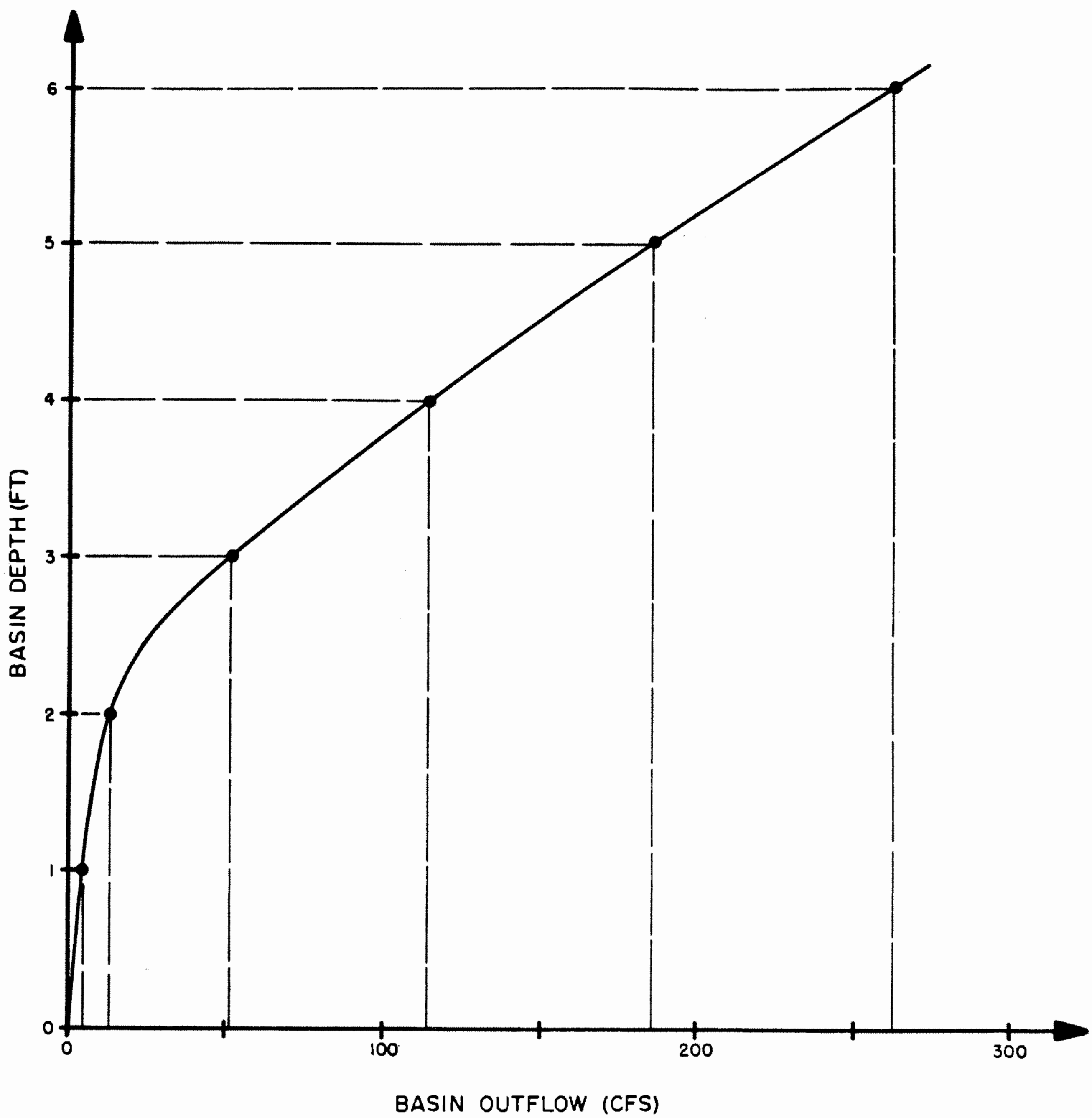
5. Use O_2 and the known storage vs. depth and outflow vs. depth relationships to compute S_2.

These five steps are repeated for the next timestep using I_2, O_2, and S_2 as the new values of I_1, O_1, and S_1, respectively. This procedure is repeated until the basin inflow hydrograph has been completely analyzed and basin outflow becomes negligible.

The example problem illustrates the basin routing procedure.

F.2.2. Example Problem: Detention Basin Hydrograph Routing

The assumed detention basin depth vs. outflow and depth vs. storage relationships are shown in Figures F-1 and F-2, respectively. The detention basin information sheet (Figure F-8) is completed in Figure F-3. Using a timestep of 60 minutes (3600 seconds), the associated storage-indication curve is developed in the following table and plotted in Figure F-4.

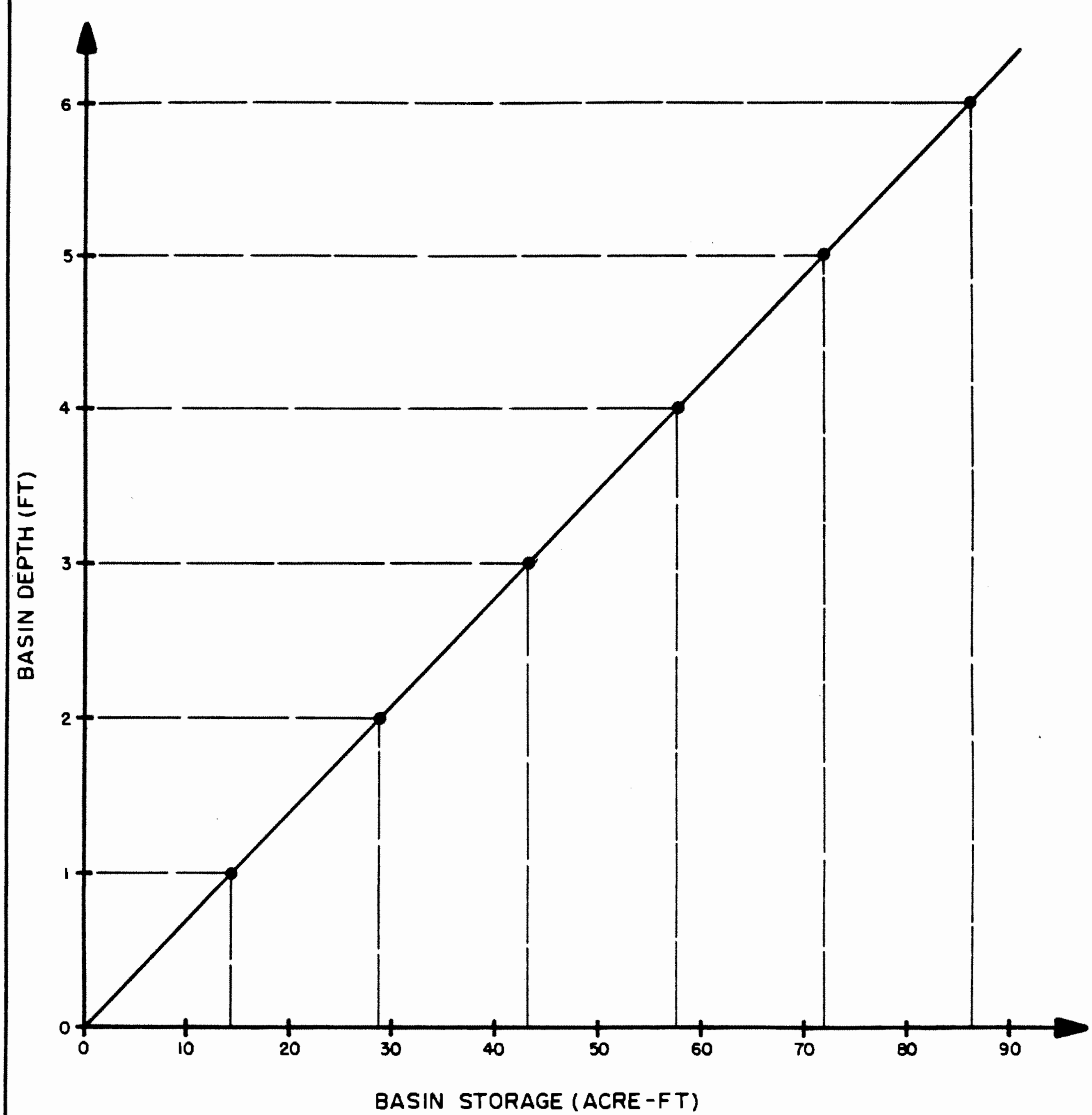


ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM
DETENTION BASIN OUTFLOW (CFS)
AS A FUNCTION OF DEPTH (FT)

Figure F-1



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM
DETENTION BASIN STORAGE (AF)
AS A FUNCTION OF DEPTH (FT)

Figure F-2

PROJECT: EXAMPLE PROBLEM DATE:

ENGINEER:

1. Enter the hydrograph unit interval duration **(minutes)** 60

2. Enter total number of basin depth-versus-outflow values **(maximum of 20)** 7

3. Enter basin outflow (cfs) and storage volume (AF) for each basin depth value in the following table. Enter values in order of increasing outflow basin depth:

Entry No.	Water Surface Elevation (FT)	Basin Depth (FT)	Basin Storage (AF)	Basin Outflow (CFS)
1	100	0.0 (defined)	0	0.0 (defined)
2	101	1	14.4	4.2
3	102	2	28.8	12.0
4	103	3	43.2	51.7
5	104	4	57.6	114.7
6	105	5	72.0	186.8
7	106	6	86.4	263.2
8				
9				
10				
11				
12				
13				
14				
15				
16				
17				
18				
19				
20				

4. Enter assumed initial depth (feet) of water in detention basin 0

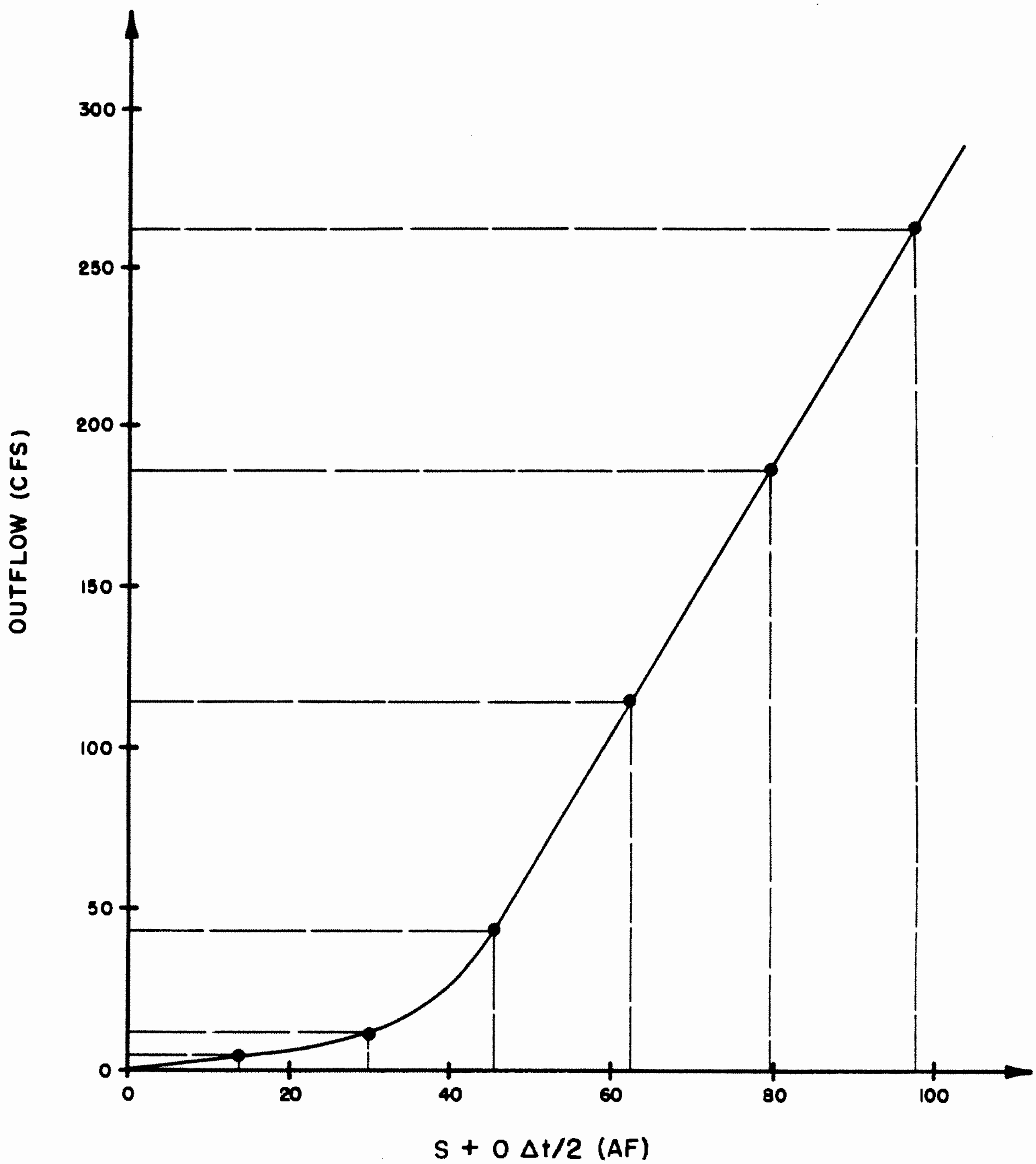
ORANGE COUNTY

HYDROLOGY MANUAL

DETENTION BASIN

INFORMATION FORM

Figure F-3



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM

STORAGE-INDICATION CURVE

Figure F-4

TABLE F.1.

EXAMPLE PROBLEM STORAGE-INDICATION CURVE DEVELOPMENT

Depth (ft)	O (cfs)	S (AF)	S-O $\Delta t/2$ (AF)	S+O $\Delta t/2$ (AF)
0	0	0	0	0.0
1	4.2	14.4	14.22	14.57
2	12.0	28.8	28.30	29.30
3	51.7	43.2	41.06	45.34
4	114.7	57.6	52.86	62.34
5	186.8	72.0	64.28	79.72
6	263.2	86.4	75.52	97.28

Assuming an initial condition of zero basin outflow and storage, an example basin inflow hydrograph (unit period of 60 minutes) is routed using the modified Pul's method in the tabulation of Table F.2. The 60-minute timestep is used for demonstration purposes only. Typically, a 5-minute timestep is needed in order to adequately describe the runoff hydrograph peak flow rates. Important features of a routed detention basin hydrograph are shown in Figure F-5.

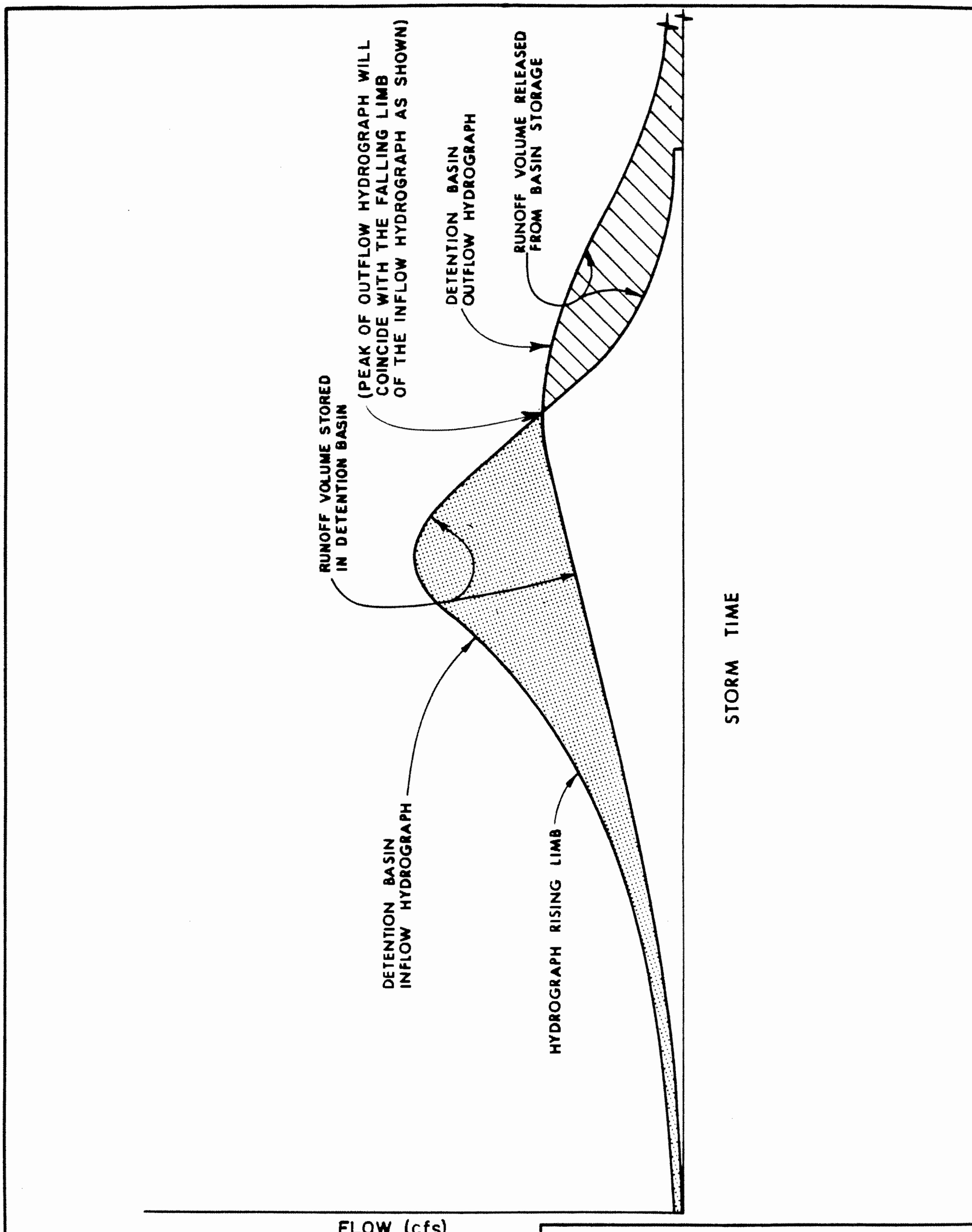
F.3. REQUIRED FORMAT

Figure F-6 illustrates the required format for submitting detention basin study results for review.

Figure F-7 is to be used to supply the necessary detention basin information to determine the routing results.

TABLE F.2.
EXAMPLE PROBLEM BASIN ROUTING TABULATION

Time (min.)	Inflow (cfs)	Average Inflow (cfs)	$(I_1+I_2)\Delta t/2$ (AF)	$S_1-O_1\Delta t/2$ (AF)	$S_2+O_2\Delta t/2$ (AF)	Outflow (cfs)	Storage (AF)
0	0					0	0
		30	2.48	0	2.48		
60	60					.7	2.45
		90	7.44	2.42	9.86		
120	120					2.8	9.74
		200	16.53	9.62	26.16		
180	280					10.3	25.73
		265	21.90	25.31	47.21		
240	250					58.6	44.79
		235	19.42	42.37	61.79		
300	220					112.7	57.14
		170	14.05	52.48	66.53		
360	120					132.1	61.07
		110	9.09	55.61	64.70		
420	100					124.5	59.56
		80	6.61	54.41	61.02		
480	60					109.8	56.48
		30	2.48	51.94	54.42		
540	0					85.4	50.89
		0	0	47.36	47.36		
600	0					59.20	44.91
		0	0	42.46	42.46		

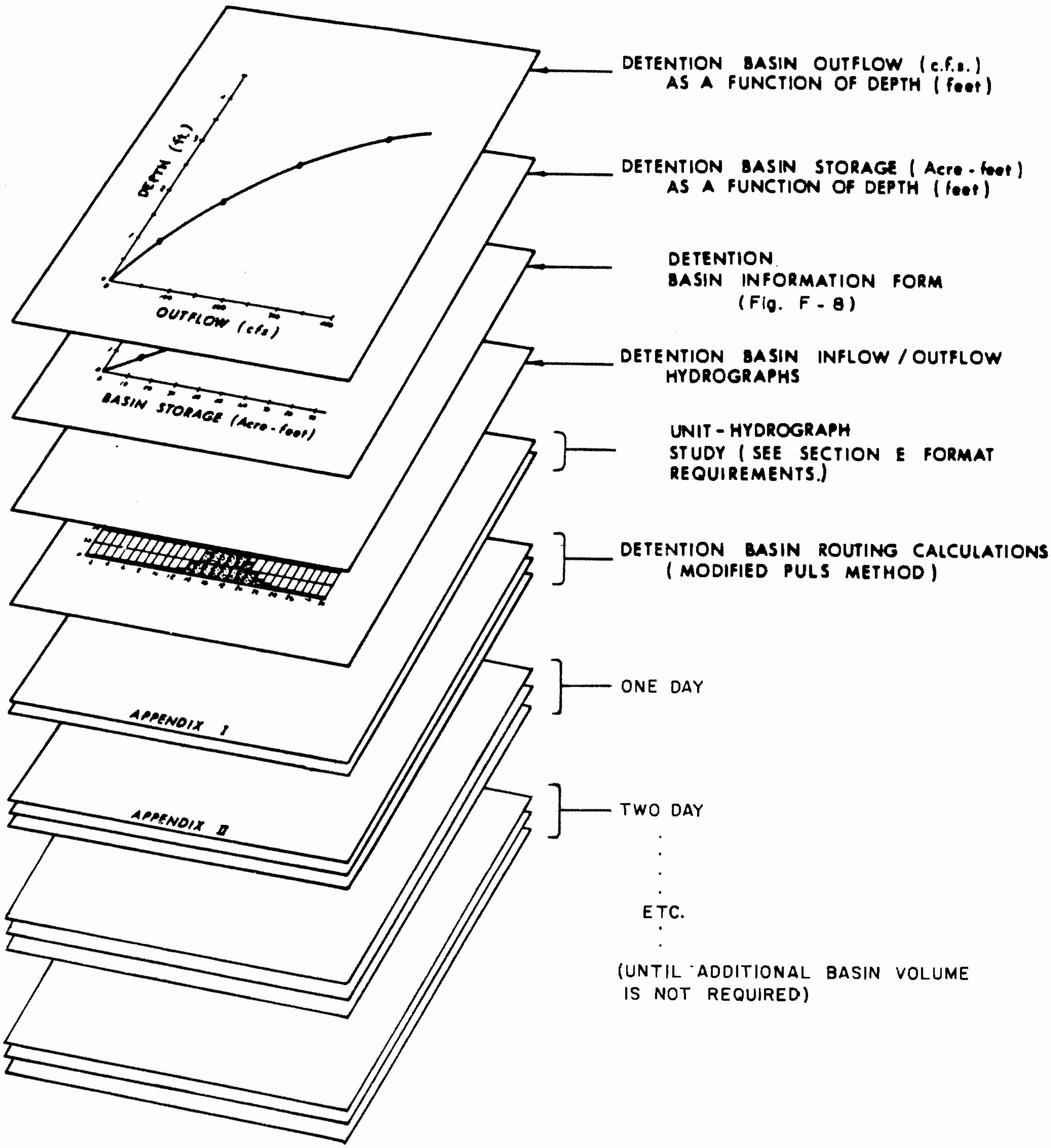


ORANGE COUNTY

HYDROLOGY MANUAL

DETENTION BASIN INFLOW AND OUTFLOW HYDROGRAPHS

Figure F-5



ORANGE COUNTY

HYDROLOGY MANUAL

REQUIRED REPORT FORMAT FOR DETENTION BASIN STUDY

Figure F-6

PROJECT: ______________________ DATE:

____

ENGINEER: ______________________

1. Enter the hydrograph unit interval duration **(minutes)** ________

2. Enter total number of basin depth-versus-outflow values **(maximum of 20)** ________

3. Enter basin outflow (cfs) and storage volume (AF) for each basin depth value in the following table. Enter values in order of increasing outflow basin depth:

Entry No.	Water Surface Elevation (FT)	Basin Depth (FT)	Basin Storage (AF)	Basin Outflow (CFS)
1	______	0.0 (defined)	______	0.0 (defined)
2	______	______	______	______
3	______	______	______	______
4	______	______	______	______
5	______	______	______	______
6	______	______	______	______
7	______	______	______	______
8	______	______	______	______
9	______	______	______	______
10	______	______	______	______
11	______	______	______	______
12	______	______	______	______
13	______	______	______	______
14	______	______	______	______
15	______	______	______	______
16	______	______	______	______
17	______	______	______	______
18	______	______	______	______
19	______	______	______	______
20	______	______	______	______

4. Enter assumed initial depth (feet) of water in detention basin ________

ORANGE COUNTY

HYDROLOGY MANUAL

DETENTION BASIN INFORMATION FORM

SECTION G

FLOW-BY BASIN ANALYSIS (HYDROGRAPH SEPARATION)

G.1. INTRODUCTION

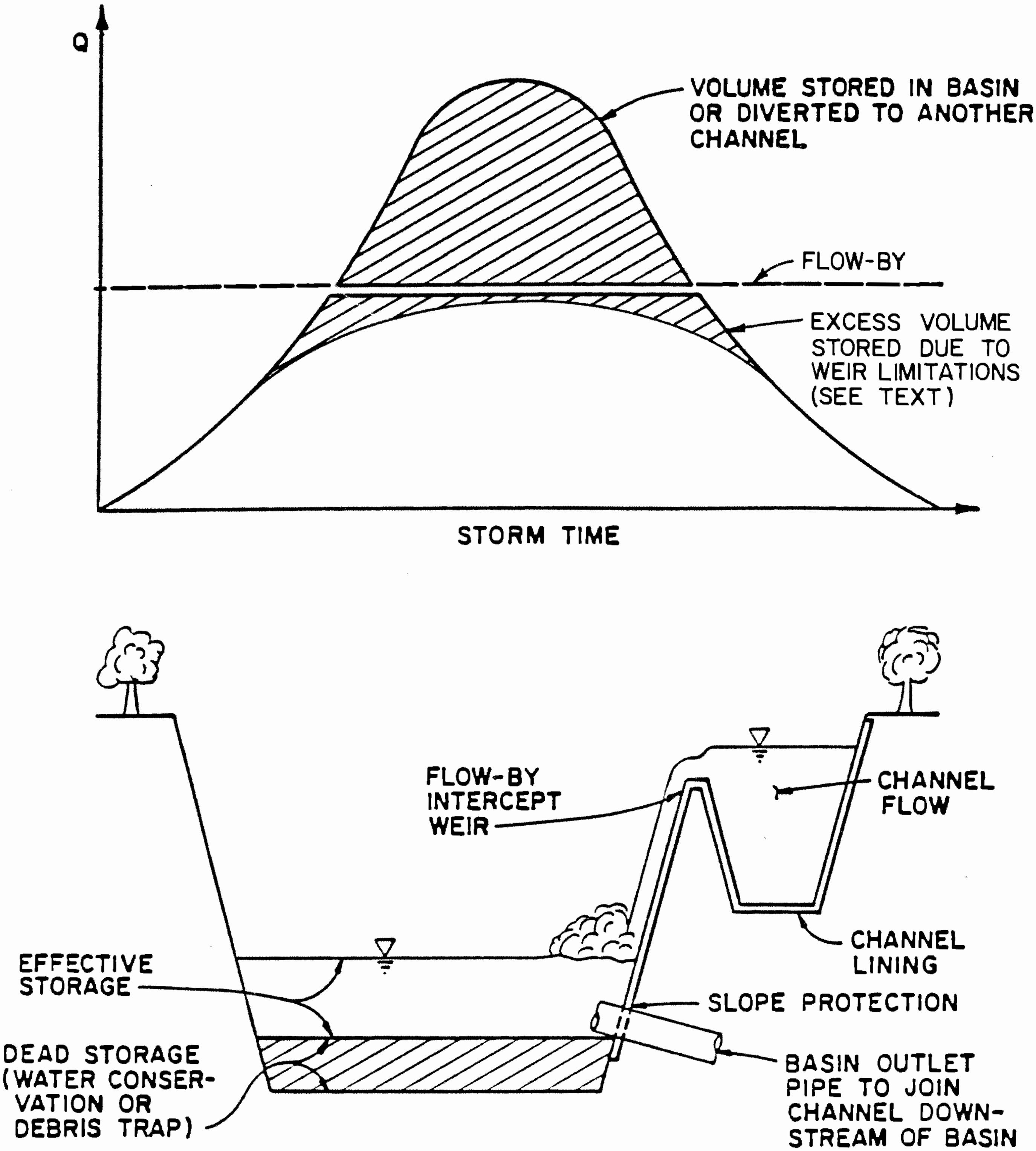
This process models the effect of a channel flowing by a detention basin or structure which intercepts and diverts away from the channel all runoff flows in excess of some specified flowrate. Although simple in concept, use of a flow-by basin in a flood control system can provide a useful reduction in the runoff hydrograph peak flowrate. Figure G-1 shows the main elements of a typical flow-by basin.

G.2. EXTENDED DESIGN STORM CRITERIA

A multiple day storm may be required to guarantee that the basin has an adequate storage capacity remaining when a peak 24-hour storm event occurs. For a multiple day storm condition involving a flow-by basin system, the peak rainfall intensities of the selected T-year return frequency should be incorporated within each 24 hour duration (up to the appropriate mass rainfall volume with the desired return frequency). A detailed discussion of the design for watersheds with flow-by basins is presented in Section B.6.

G.3. FLOW-BY BASIN VOLUME ANALYSIS: WEIR STRUCTURE EFFICIENCY

Figure G-1 illustrates a typical weir structure flow-by basin design. Due to the finite weir length, the actual flow-by discharge (i.e., no overflow into the basin) is less than the desired flow-by discharge and, consequently, the actual basin storage requirements are higher than idealized by a simple separation of the hydrograph. Generally, this volume excess ranges between 20 and 50 percent depending on the weir length used for the overflow.



ORANGE COUNTY

HYDROLOGY MANUAL

FLOW-BY BASIN

CONCEPT AND ELEMENTS

Figure G-1

SECTION H

STREAMFLOW ROUTING

H.1. INTRODUCTION

Although a calibrated peak discharge is obtained when using the unit hydrograph method applied to a single area, there are instances where a runoff hydrograph must be routed through a stream or channel. There are three common situations where streamflow routing is required: (1) downstream of a detention basin, (2) when the watershed has a "finger" or other non-elliptical shape such as an hour-glass shape, and (3) significant inhomogeneity of ultimate land use or soil type within the watershed.

The convex routing technique (refs. 2, 3) shall be used whenever streamflow routing is necessary; however, the Agency may accept some other routing techniques such as Muskingum Routing only if the results are comparable to those obtained by convex routing.

H.2. CONVEX ROUTING METHOD FOR UNSTEADY OPEN CHANNEL FLOW

The governing relationship used in the convex routing approach is:

$$O_{T+dT} = (1-C)O_T + CI_T \qquad (H.1)$$

where

I_T = hydrograph inflow at time T

O_T = channel outflow at time T

O_{T+dT} = channel outflow at time T + dT

C = a routing coefficient (where C is between 0 and 1)

Rearranging (H.1) gives the explicit statement

$$O_{T+dT} = O_T + C(I_T - O_T) \tag{H.2}$$

and

$$C = (O_{T+dT} - O_T)/(I_T - O_T) \tag{H.3}$$

The routing coefficient may be estimated by the empirical relationship (ref. 3),

$$C = V/(V + 1.7) \tag{H.4}$$

where V is a mean flow velocity assumed for the inflow hydrograph. One method of computing V is to calculate the normal depth corresponding to the average flowrate of all unit flows greater than 50 percent of the inflow hydrograph peak flowrate. From this normal depth calculation, V is defined to be the corresponding flow velocity. Thus V represents an average velocity which is used to translate the inflow hydrograph along the total length of the channel. Obviously, other values for V can result depending on the choice of the average flowrate value from the inflow hydrograph, and care must be taken to avoid offsetting the hydrograph peak flowrates at channel confluences by the selection of the channel V parameter.

The routing timestep, dT, is given by

$$dT = \frac{CL}{3600\ V} \tag{H.5}$$

where C is given by (H.4), L is the channel length in feet, and dT is in units of hours.

Because the unit hydrograph analysis base unit period dT^* is usually different than the dT time period of (H.1) a modification of C is required. For a unit period of 5 minutes, $dT^* = 0.0833$ hours and the modified routing coefficient is

$$C^* = 1 - (1-C)^E \tag{H.6}$$

where $$E = (dT^* + 0.5dT)/(1.5dT)$$

To determine the dT which corresponds to (H.1), it is assumed that

$$(O_{T+dT} - O_T)/(I_T - O_T) = dT/K \tag{H.7}$$

where K is the channel reach travel time as estimated from the selection of the inflow hydrograph mean V value and the channel length, L. Figure (H-1) illustrates the geometric interpretation of the relationship given by (H.7). Thus,

$$dT = CK \tag{H.8}$$

A 5-minute unit period is used for all convex routing applications.

An examination of the convex routing method reveals that the entire routing approach is a function of the routing coefficient, C. Consequently, a watershed link-node model composed of m such channel links necessarily includes m channel routing parameters, each with an associated unknown uncertainty function. Additionally, the uncertainty involved in combining the m channel links is further aggravated by the fact that each channel-routed hydrograph is also a function of the number and channel reach lengths used for each channel link. That is, the routed hydrograph through a channel with a length of 20,000 feet will differ from the results of routing a hydrograph through two successive reaches with a length of 10,000 feet, and so forth. Channel routing processes usually involve relatively short reaches of improved channel where storage effects are minor, or where confluences from other channels (or pipes) enter the main channel and a summation of hydrographs occurs. Finally, the routing coefficient is a function of the calculation timestep. The example problem demonstrates the variation in C due to the calculation timestep used.

H.2.1. Example Problem: Convex Channel Routing

The example problem channel is a rectangular concrete section with a base of 10 feet, a Manning friction factor of 0.015, length of 3000 feet, and a mean slope of 0.005 foot/foot. The problem inflow hydrograph is tabulated in Table H.1. From the table, the average flowrate in excess of the 50-percent peak flowrate value is 767.4 cfs.

Using Manning's equation, the normal depth flow velocity is calculated as

$$V = (1.486 R_h^{0.67} S_0^{0.50})/n \qquad \text{(H.9)}$$

where R_h is the hydraulic radius, and S_0 is the channel slope. For the example problem, V is 13.5 fps (feet per second) and the default routing coefficient from (H.4) is C = 0.89. From (H.8),

$$K = (3000\text{ ft})/(13.5\text{ fps})(3600\text{ sec/hr}) = 0.062\text{ hour}$$

$$dT = (0.89)(0.062) = 0.055\text{ hour}$$

From (H.5), C* is estimated for a timestep of dT* equal to 5 minutes by

$$C^* = 1 - (1-0.89)^E = 0.948$$

where $E = (0.0833 + (0.5)(0.055))/((1.5)(0.055)) = 1.343$. Thus the appropriate convex method routing approximation statement is

$$O_{T+dT} = (1-C^*)O_{T+dT-dT^*} + C^*I_T \tag{H.10}$$

where for the example problem

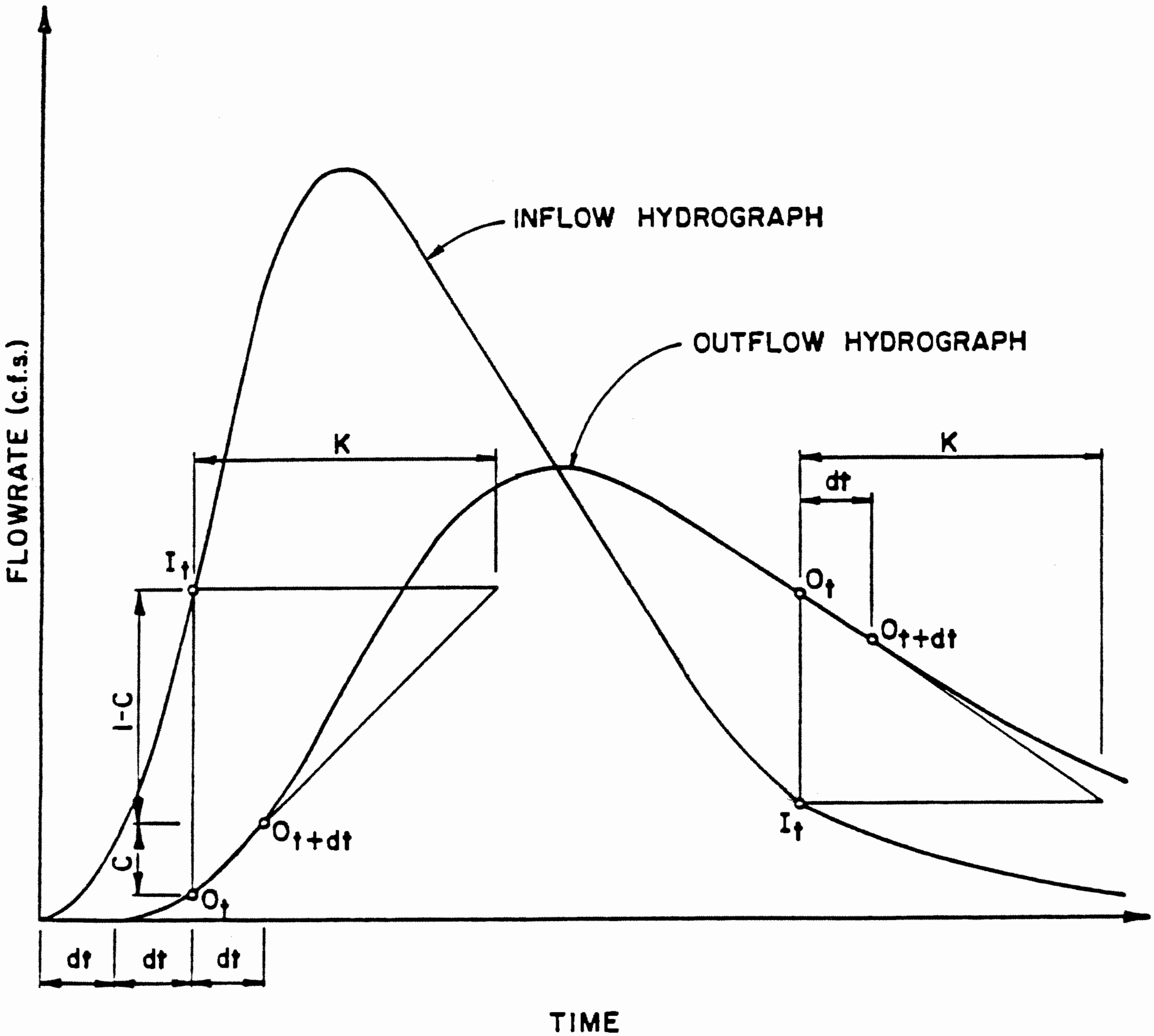
$$O_{T+dT} = (0.052)O_{T+dT-dT^*} + (0.948)I_T$$

TABLE H.1.

CONVEX ROUTING EXAMPLE PROBLEM SOLUTION

Storm Time (minutes)	Inflow (cfs)	Outflow[1] (cfs)
0	0.0	0.0
5	0.8	0.3
10	0.9	0.8
15	40.5	7.7
20	202.7	91.1
25	445.1	274.6
30	602.9	486.8
35	653.7	613.1
40	600.9	634.7
45	608.0	605.0
50	917.1	706.9
55	1186.7	992.5
60	1001.2	1117.1
65	763.6	931.1
70	714.9	756.7

Note 1: outflow is to be offset by 3.7 minutes of travel time due to a computed mean flow velocity of 13.5 fps.



ORANGE COUNTY

HYDROLOGY MANUAL

ASSUMED ROUTING COEFFICIENT GEOMETRIC RELATIONSHIPS

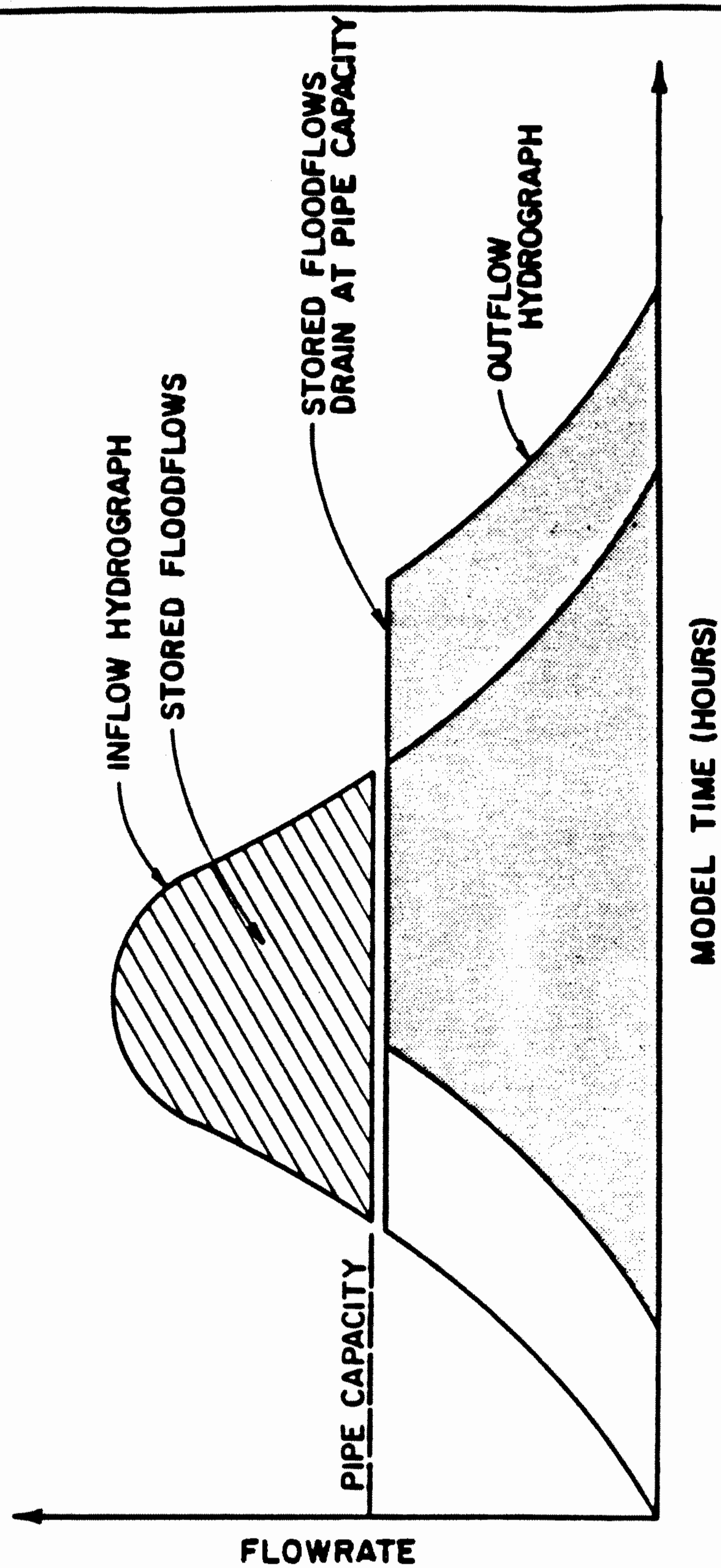
Figure H-1

SECTION I

A PIPEFLOW ROUTING MODEL

Similar to the convex routing approximation procedure, the pipeflow routing model develops an outflow hydrograph from a reach of pipe given an inflow hydrograph and appropriate pipe section data. In the considered pipeflow model, however, a limiting value of outflow is assumed whereby all inflows greater than this pipe capacity are temporarily stored at the upstream endpoint of the pipe. The stored floodwaters subsequently drain into the pipe at a rate equal to the pipe capacity. Where this assumption is not valid, an alternative approach should be used. This modeling approach approximates the ponding of floodwaters where a significant volume of storage is available with a small change in flooding depths. Similar to the convex channel routing method, backwater effects are not included in the modeling approach.

The pipeflow routing process is modeled by calculating a normal depth flow velocity for each unit period (e.g., 5-minute) runoff value from the inflow hydrograph, and translating the unit runoff forward in storm time by the appropriate time increment. Generally, flowdepths in excess of 0.82 of the pipe diameter are assumed to be sealed and the unit interval flow velocities are computed based on a full flow condition. Figure I-1 shows the salient features of the pipeflow modeling approach.



ORANGE COUNTY

HYDROLOGY MANUAL

PIPEFLOW MODELING ELEMENTS

Figure I-1

SECTION J

SMALL AREA RUNOFF HYDROGRAPH DEVELOPMENT

J.1. INTRODUCTION

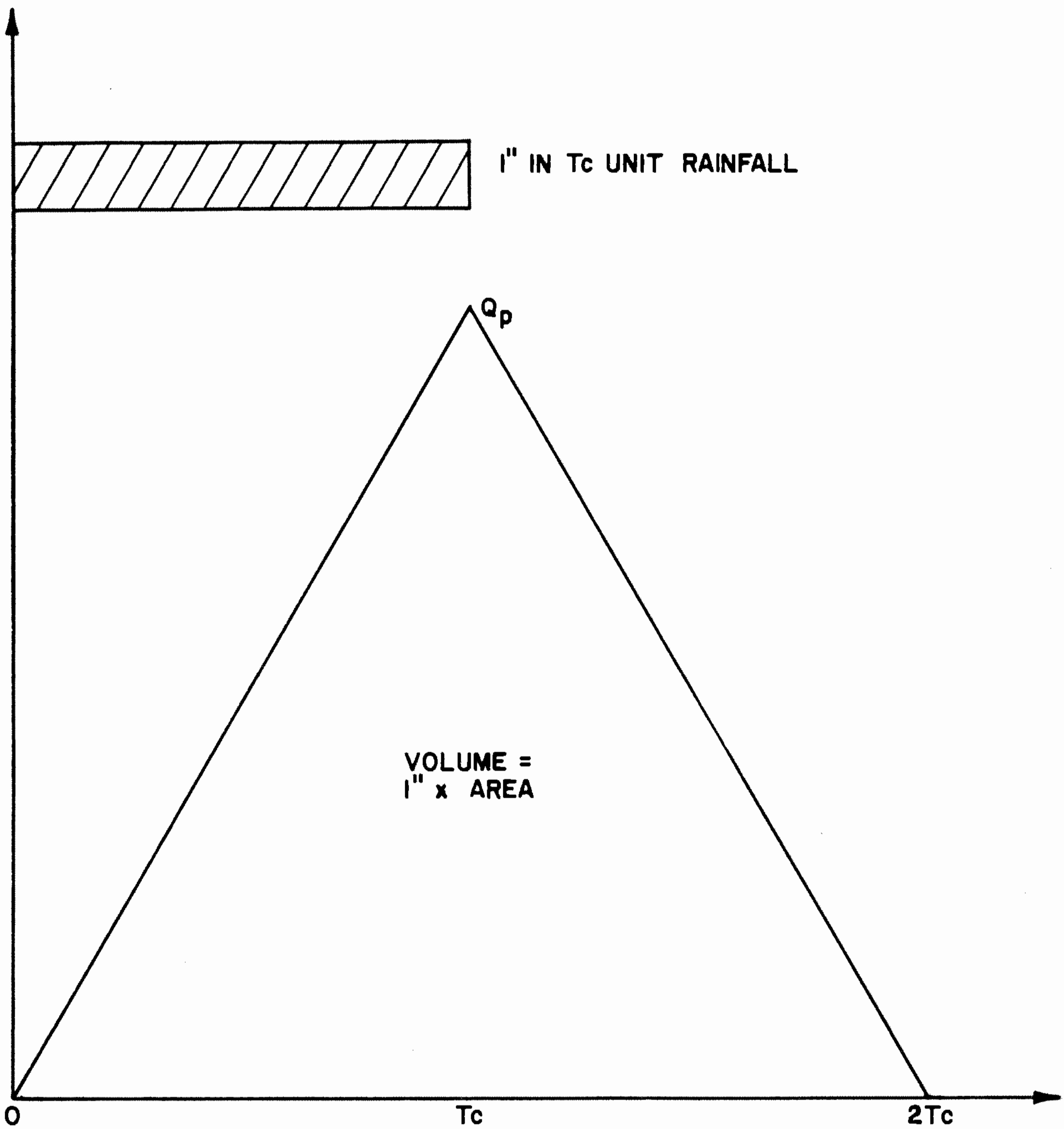
For watersheds whose time of concentration (see Section D) is less than 25 minutes, a simpler procedure can be used to develop the design storm runoff hydrograph. Additionally, the 25-minute limitation corresponds to a 25-percent unit interval (of watershed lag) when using 5-minute unit rainfalls in the unit hydrograph technique. Consequently, in order to avoid the unit hydrograph being too coarse an approximation, a small-area unit hydrograph method is needed. This technique is analogous to the design storm approach of Sections B and E but has the following simplifications:

i. Depth-area Adjustment - Generally, watersheds whose time of concentration is less than 30 minutes have a drainage area small enough that depth-area adjustment is not required; i.e., the regionalized point rainfall depths are used without depth-area adjustment.

ii. Design Storm Pattern Development - Using a unit interval equal to the time of concentration (Tc), unit rainfalls are determined by successive subtractions along the mass rainfall plot (see Example).

iii. Loss Rates - Conforms to Section E.

iv. Unit Hydrograph - The unit hydrograph is defined to be a triangle with a base of 2Tc, and a peak at time of Tc (see Figure J-1). The volume of the unit hydrograph is (1-inch)(area). The example problem illustrates the triangle unit hydrograph. Note that in this case, lag is defined to equal the Tc estimate; (i.e., 50-percent volume occurs at time of Tc). Also note that lag is not computed, but the rational method Tc is used directly.



ORANGE COUNTY
HYDROLOGY MANUAL

SMALL AREA
UNIT HYDROGRAPH

Figure J-1

v. Convolution - The convolution of the unit hydrograph with the unit effective rainfalls (storm rainfall less losses) is simply the addition of peak runoff values at each of the Tc unit intervals (see Example), where peak flow estimates follow from the rational method of Section D.

It is noted that in the small area runoff hydrograph method, the total catchment area shall be used in the calculations; i.e., although the effective area may be used for rational method estimates for peak flow rates, the total catchment area is needed for runoff hydrograph volume study purposes. Any deviation from the use of the total catchment area must be approved by the Agency.

J.1.1. Example Problem: Small Area Runoff Hydrograph Development

1. Assume given from a rational method study (see Section D):
   watershed area = 8 acres
   time of concentration = 10 minutes = unit interval
   maximum loss rate (F_m) = 0.12 inches/hour
   low loss fraction ($\overline{Y}$) = 0.35

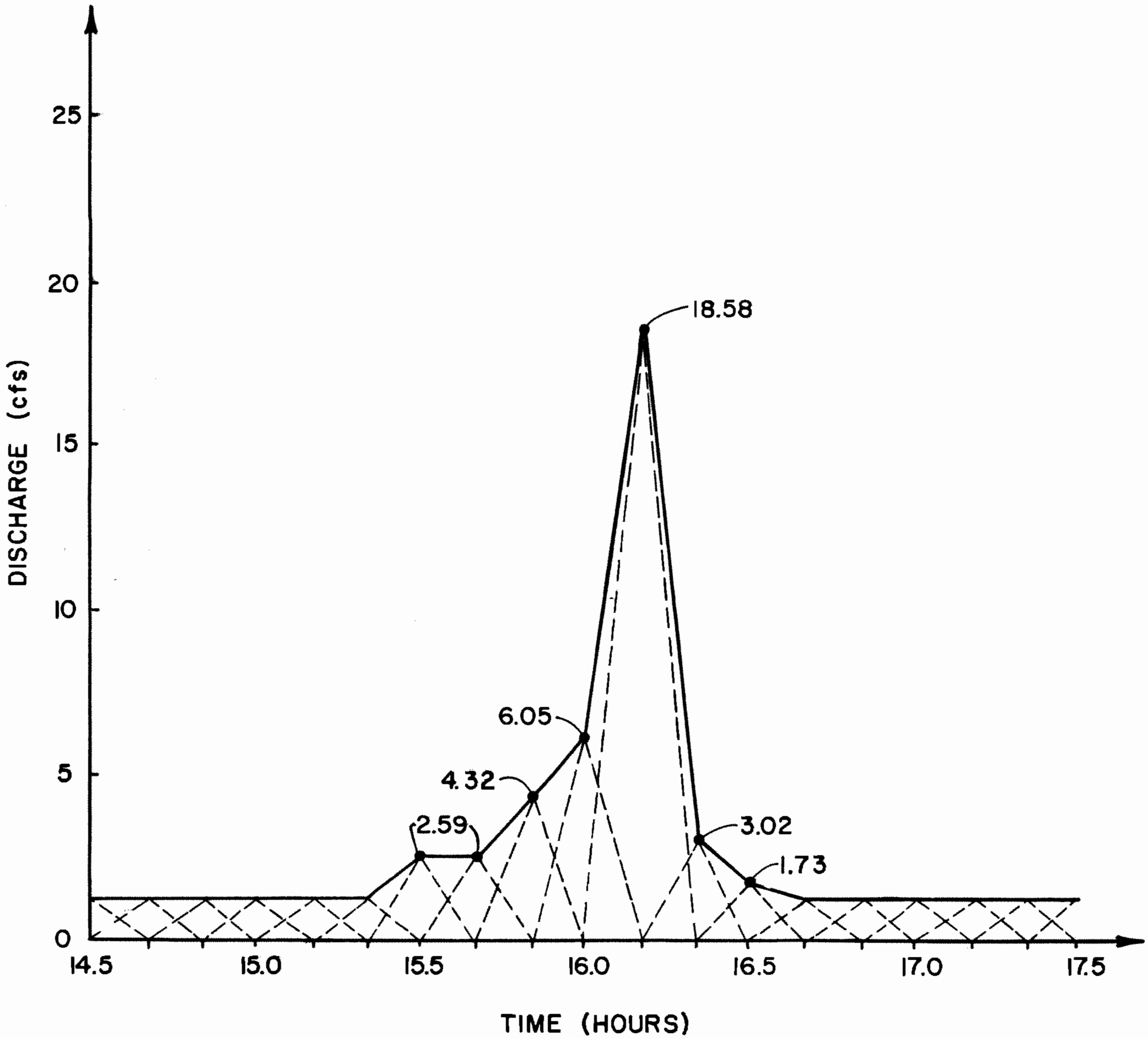
2. $D_{10} = 0.17T^{0.427}$ (see Figure B-1)
   where
   D_{10} = 10-year frequency depth (inches)
   T = duration (minutes)

3. $Q = 0.9\ (I-F_m)A$

TABLE J.1. EXAMPLE PROBLEM RESULTS

Peak Rainfall Unit Number	Mass Rainfall (Inches)	Unit Rainfall (Inches)	Unit Loss (Inches)	Net Rainfall (Inches)	Effective Rainfall (Inch/Hr.)	Discharge (Q) (cfs)
1	0.45	0.45	.02*	0.43	2.58	18.58
2	0.61	0.16	.02*	0.14	.84	6.05
3	0.73	0.12	.02*	0.10	.60	4.32
4	0.82	0.09	.02*	0.07	.42	3.02
5	0.90	0.08	.02*	0.06	.36	2.59
6	0.98	0.08	.02*	0.06	.36	2.59
7	1.04	0.06	.02*	0.04	.24	1.73
8	1.10	0.06	.02*	0.04	.24	1.73
9	1.16	0.06	.02*	0.04	.24	1.73
10	1.21	0.05	.02*	0.03	.18	1.30
11	1.27	0.06	.02*	0.04	.24	1.73
12	1.31	0.04	.01	0.03	.18	1.30
13	1.36	0.05	.02*	0.03	.18	1.30
14	1.40	0.04	.01	0.03	.18	1.30
15	1.44	0.04	.01	0.03	.18	1.30
16	1.48	0.04	.01	0.03	.18	1.30
17	1.52	0.04	.01	0.03	.18	1.30
18	1.56	0.04	.01	0.03	.18	1.30
19	1.60	0.04	.01	0.03	.18	1.30
20	1.63	0.03	.01	0.02	.12	0.86

*Unit low loss exceeds unit adjusted loss
Discharge to 24 hours is calculated by the above method



ORANGE COUNTY

HYDROLOGY MANUAL

EXAMPLE PROBLEM
SMALL AREA
RUNOFF HYDROGRAPH

Figure J-2

SECTION K

WATERSHED MODELING GUIDELINES

K.1. INTRODUCTION

The previous sections provide the several elements used in developing a link-node watershed model for hydrologic planning purposes. In this section, guidelines are presented for development of complex hydrologic models for the analysis of the design storm condition. The combination of the several submodels described in Sections E-J provides the hydrologist with the modeling capability to analyze complex watershed conditions including variations in runoff production caused by flood control measures and alternative watershed development plans. It may be required that the difference in runoff production between existing and the ultimate development conditions be mitigated. For example, in large watersheds, the location of greenbelt channels or detention basins can significantly effect the total watershed peak flowrate estimate. Similarly, the planned location of high density development may mitigate the effects of watershed urbanization.

K.2. SINGLE AREA RUNOFF HYDROGRAPH DEVELOPMENT

In many cases, watershed studies involve a free flowing drainage system where storm runoff is collected by major storm drains or street systems and is carried from the watershed by means of a major flood control channel. These watersheds typically have minor storage or detention effects due to detention basins, channel constrictions, or channel capacity (i.e., overbank flow) problems. Additionally, these watersheds have a time of concentration which approximately equals the watershed critical duration and are comparable to the watersheds from which the S-graphs were derived.

Generally, a single basin unit hydrograph study such as illustrated in the example problem of Section E will be appropriate for the development of a design storm runoff hydrograph and peak flow rate (see Sections E and J).

K.3. COMPLEX WATERSHED RUNOFF HYDROGRAPH DEVELOPMENT

For complex watershed modeling conditions, the watershed is divided into subareas which are "linked" together by routing processes. Such watersheds are characterized by significant detention or storage effects and large areas of different development or soil loss conditions.

The procedures to be used for the various routing processes are given in the preceding sections. Subarea unit hydrograph and subsequent runoff hydrograph development follows directly from Section E.

i. Watershed Division into Subareas - All watershed modeling results differ based on the number and selection of subareas linked together to represent the total watershed. A guideline for the watershed division is to limit subareas such that the largest subarea is no greater than four times the area of the smallest subarea. Generally, subareas are defined which are tributary to detention basins or major channels whose storage routing effects are considered significant. Additionally, subareas should be determined such that the corresponding lag values are between 20 minutes and 2.5 hours; preferably, between 25 minutes and 1.5 hours (the range of lag values used in the calibration effort). Arbitrary subdivision of the watershed into subareas should generally be avoided. It must be remembered that an increase in the watershed subdivision does not necessarily increase the modeling "accuracy" but rather transfers the model's reliability from the calibrated unit hydrograph and lag relationships to the unknown reliability of the several flow routing submodels used to link together the several subareas.

ii. Subarea Design Storm Analysis - Each subarea is subject to the design storm condition. Therefore, all flood control facilities shall be analyzed based on the design storm impacting each subarea independently (see Section E).

iii. Depth-area Adjustment - As the watershed area increases, depth-area adjustment is needed based upon the entire tributary area. For example, should a point of concentration have three tributary subareas with a combined area of 6 square miles, then each of the subareas must be reanalyzed for the design storm condition with the depth-area factors based upon the total area of 6 square miles. All routing procedures are also reevaluated based upon the new subarea runoff hydrographs. In this fashion, each point of concentration has the appropriate depth-area adjustment applied to the design storm.

K.4. USE OF WATERSHED MODEL COMPUTER PROGRAMS

Several single event unit hydrograph computer models are currently available. For example, the unit hydrograph option of the HEC-1 and TR-20 programs have been used for both small and large watershed master planning. As discussed in Section E, unmodified use of these models are precluded. In the following, guidelines are presented which provide the parameter and design storm restrictions needed to conform the various available watershed models to the design storm conditions described in this manual:

i. Effective Rainfall Computation - All watershed loss rates are to conform to the specifications of Section C (i.e., watershed 24-hour storm runoff yields, maximum loss rates (Fm), and low loss fraction ($\overline{Y}$)).

ii. Single Event Design Storm Pattern - The watershed model is to be based upon the design storm patterns shown in Section B. Depth-area adjustment and rainfall depths are to conform to the requirements of Section B.

iii. Routing Processes - Basin and channel routing modeling techniques are to be based upon the modified Pul's and convex methods described in the previous sections. (Full details of these techniques are contained in refs. 2, 3, 5.)

iv. Complex Watershed Modeling - The division of the watershed into subareas and the application of depth-area adjustment to tributary area must conform to the guidelines of this section of the manual.

v. Unit Hydrograph Development - The development of unit hydrographs must conform to the S-graphs and lag computation procedures described in Section E. Calculation of watershed time of concentration must conform to the rational method procedures of Section D.

vi. Preproject Meeting - All complex watershed modeling proposals are to be discussed with the Agency prior to study submittals for review. This preproject meeting will aid in familiarizing the project with the Agency, and also aid in checking whether the modeling approach conforms to the hydrology manual.

K.5. SINGLE AREA RUNOFF HYDROGRAPH COMPARISON CRITERIA

When a complex watershed model (e.g., a "link-node" schematic involving subareas linked by channel routing) is to be used, a single area runoff hydrograph model is also to be developed for comparison purposes. Should detention basins be planned, the complex model without the basins (i.e., "free-draining") is to be compared to a single subarea model.

Should the peak Q from the free-draining complex model be greater than the single area runoff hydrograph model, then the complex model peak Q is to be used as the design Q. The use of a higher Q for design purposes aids in accommodating for the increased uncertainty in the complex model.

Should the peak Q from the free-draining complex model be less than the single area runoff hydrograph model, then the design storm for the complex model is to be modified by uniformly increasing the rainfall used in the design storm until the peak Q values match between the two models.

SECTION L

PEAK FLOWRATE CURVES

L.1. INTRODUCTION

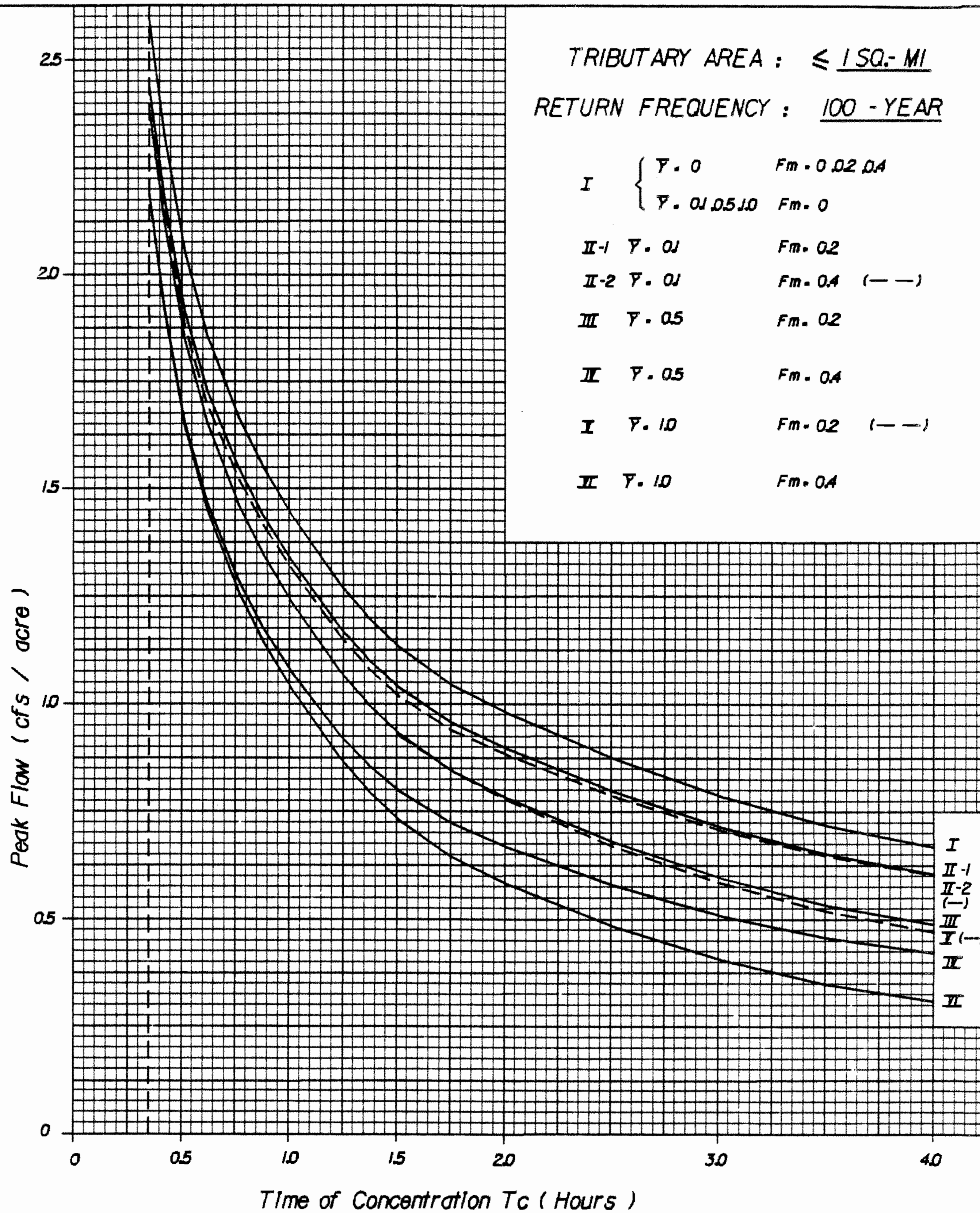
For a catchment where a single area unit hydrograph is appropriate to model the watershed response, the design storm peak flowrate can be readily determined as a function of watershed area, time of concentration (Tc), and the loss parameters of F_m and $\bar{Y}$.

Plots are provided in this section which give cfs/acre for catchment area sizes of 1, 5, 10, 25, and 50 square miles, F_m values between 0.0 and 0.4 inches/hour, $\bar{Y}$ values between 0.0 and 1.0, and Tc values between 20 minutes and 4 hours. Plots are provided for the 2-, 5-, 10-, 25-, 50-, and 100-year design storm conditions for the nonmountainous areas of Orange County where the Valley:Developed S-graph applies.

The peak flowrate curves are used by simply averaging the cfs/acre values between the appropriate watershed area plots of the assumed loss rates.

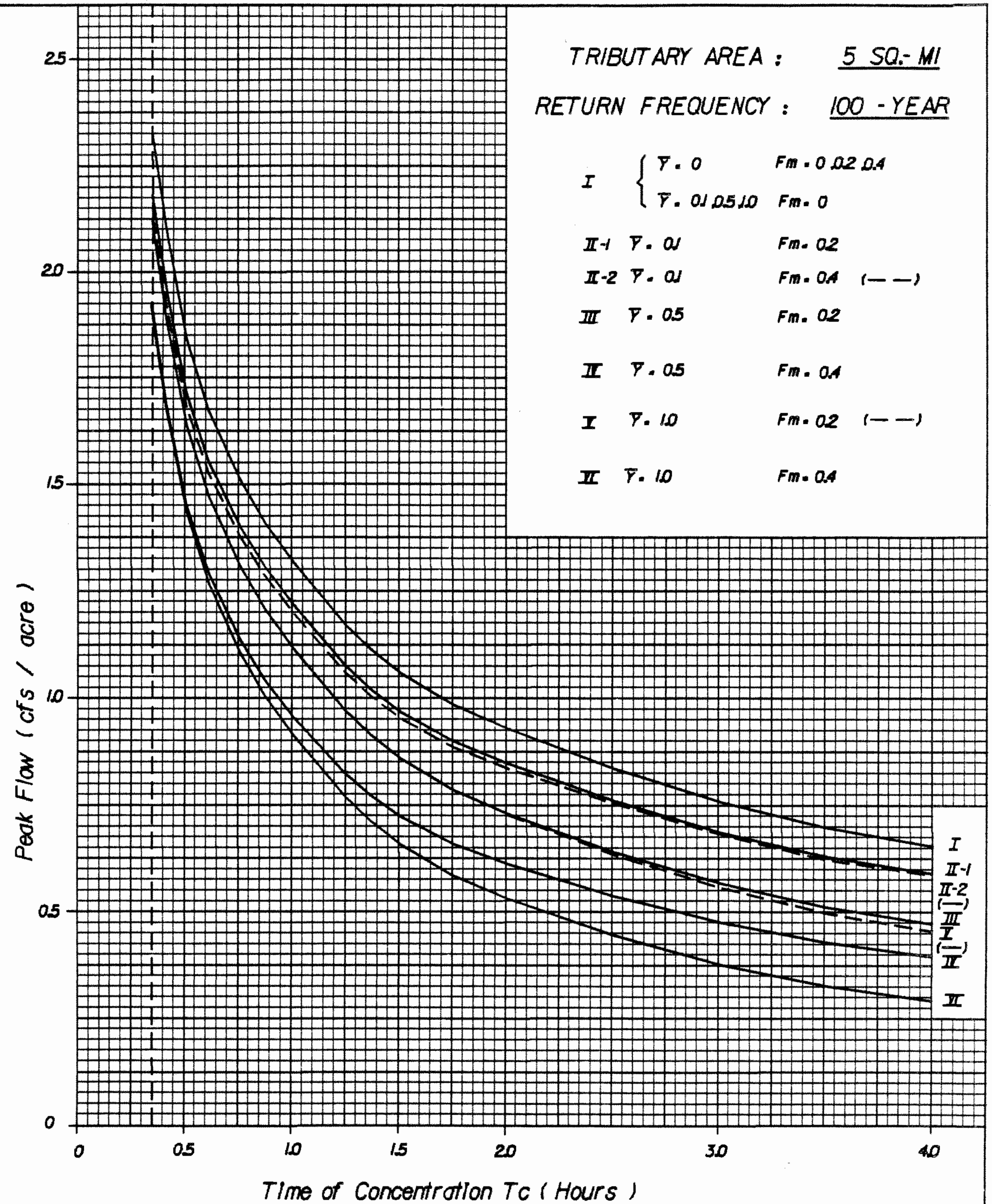
L.2. TIME OF CONCENTRATION ESTIMATION

For estimating the time of concentration, Tc, for a catchment, the rational method (Section D) can be used for catchment areas less than 1 square mile. For larger areas, the peak flowrate curves are used to develop the intermediate Q estimates used in a rational method analysis. That is, rather than using a rational method estimate of CI to develop cfs/acre, the peak flowrate equations are used. Otherwise, the methods used for estimating subarea traveltimes are applied to the entire catchment drainage system just as used in the rational method of Section D.



ORANGE COUNTY
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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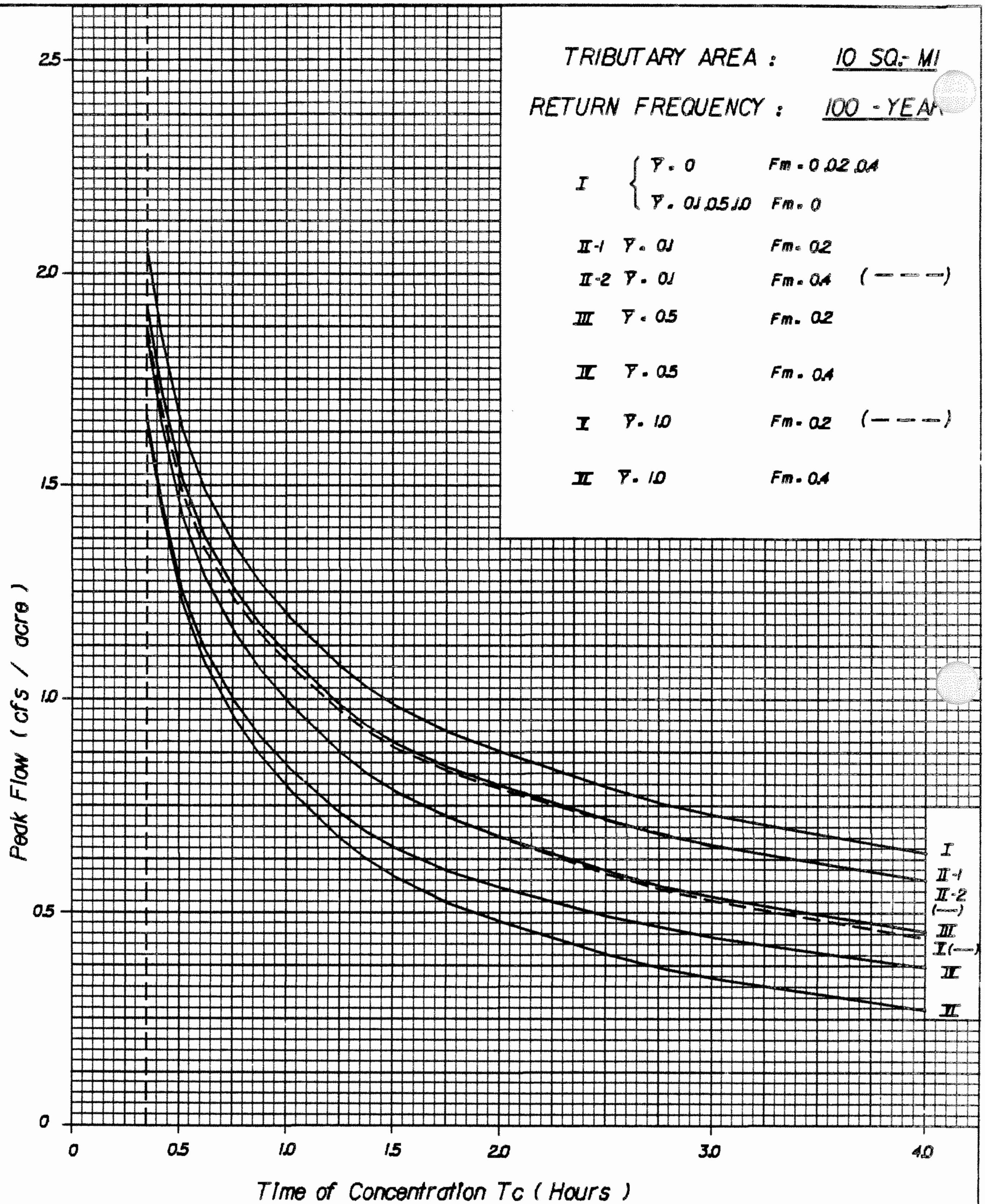
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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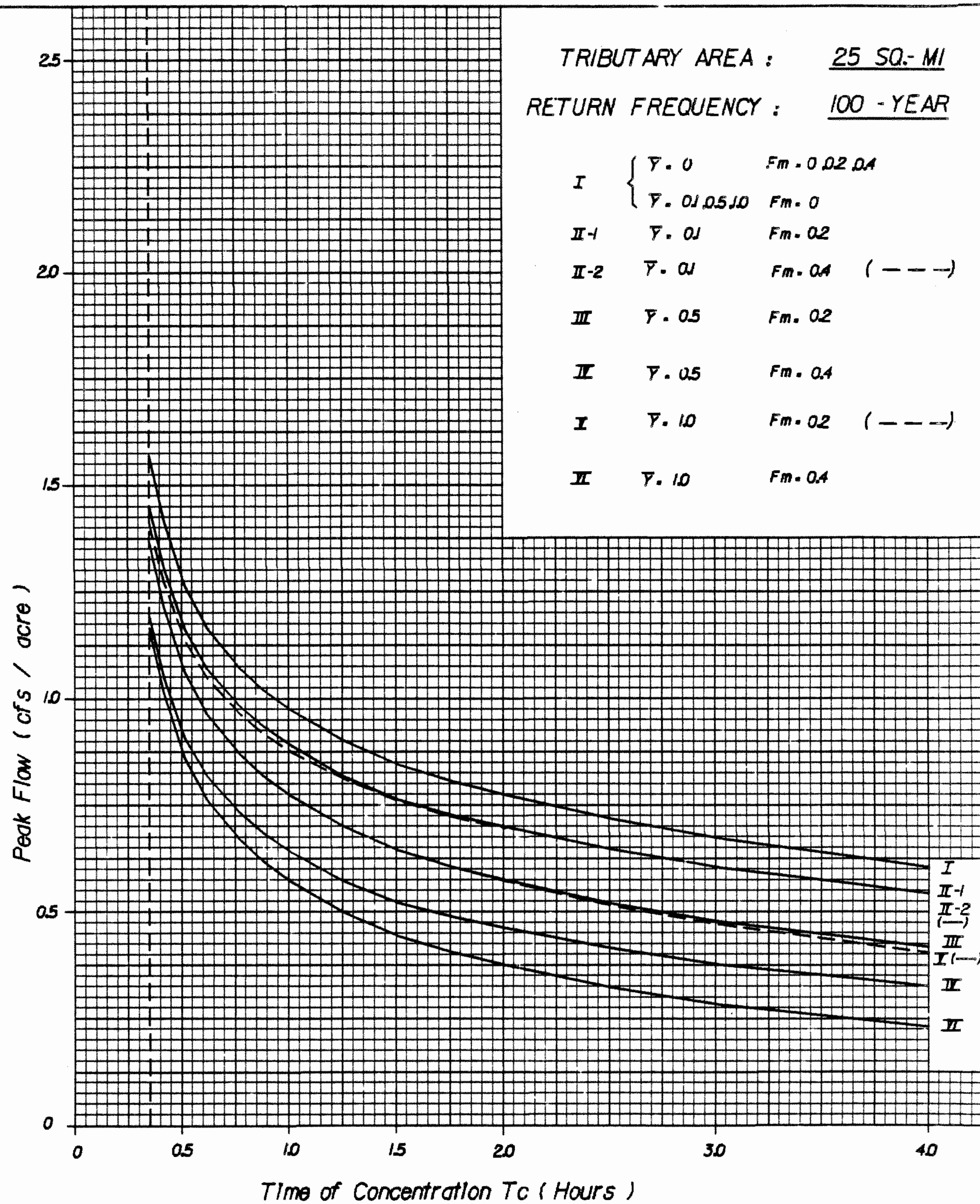
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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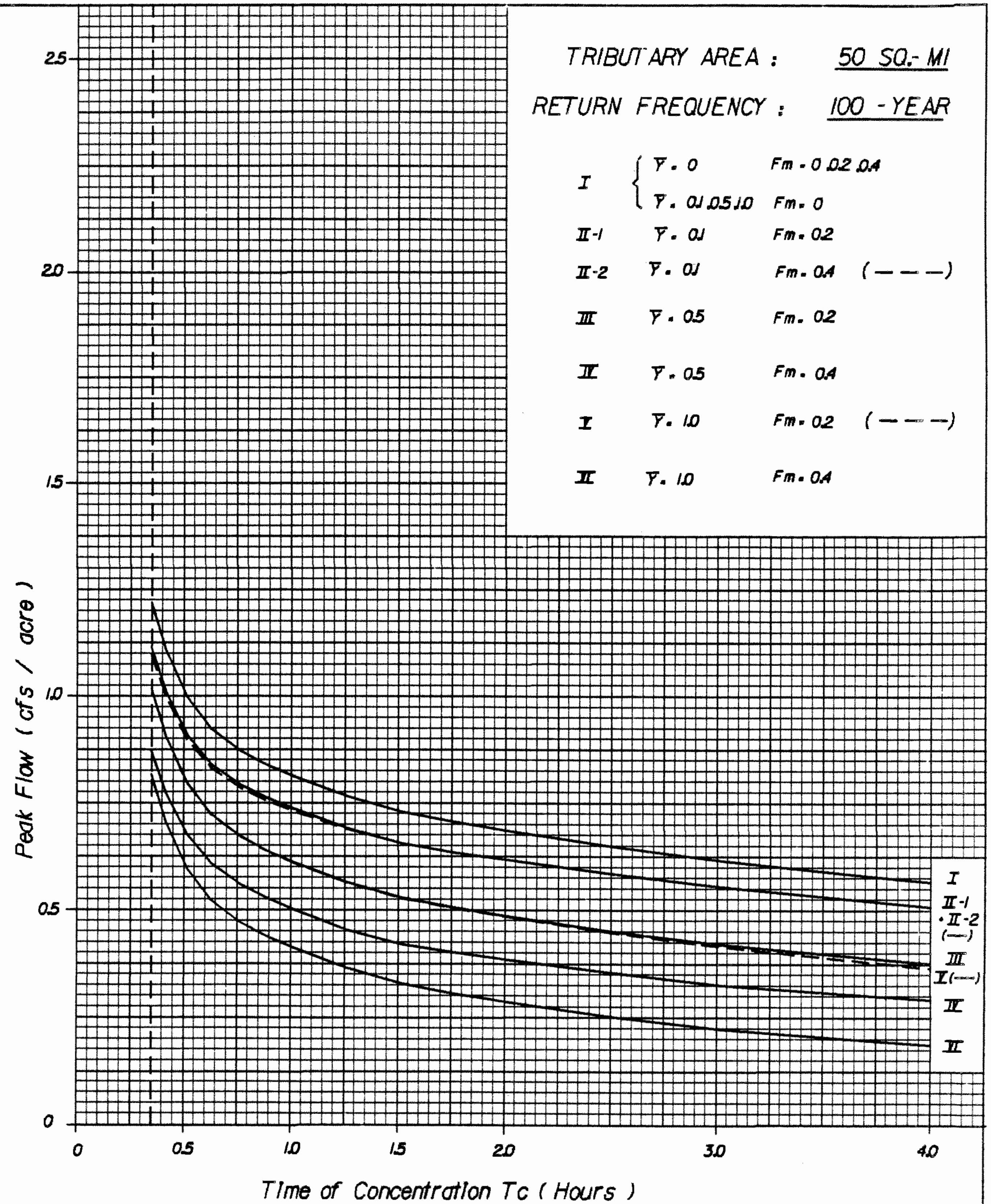
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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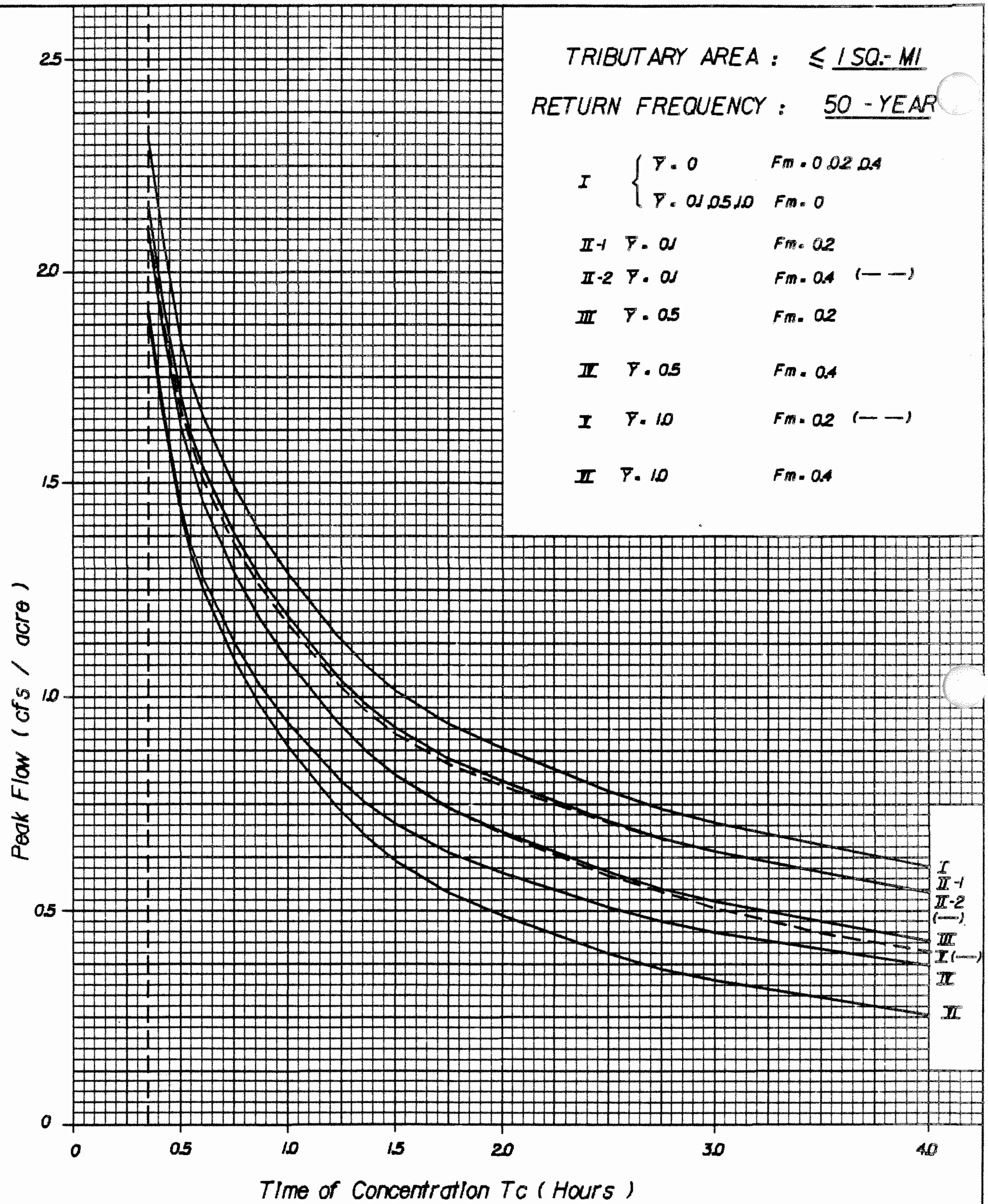
PEAK FLOWRATE CURVES



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HYDROLOGY MANUAL

PEAK FLOWRATE CURVES

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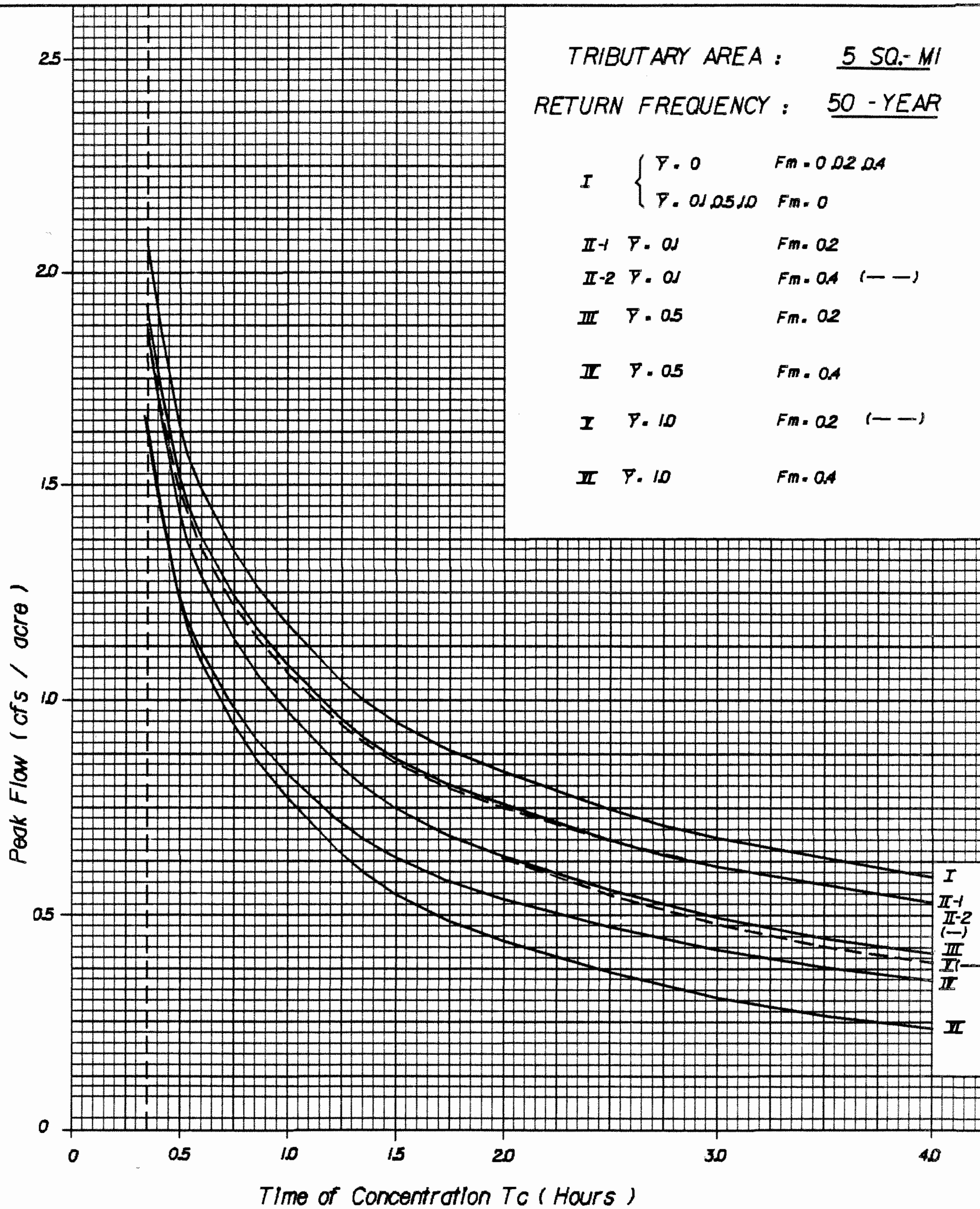

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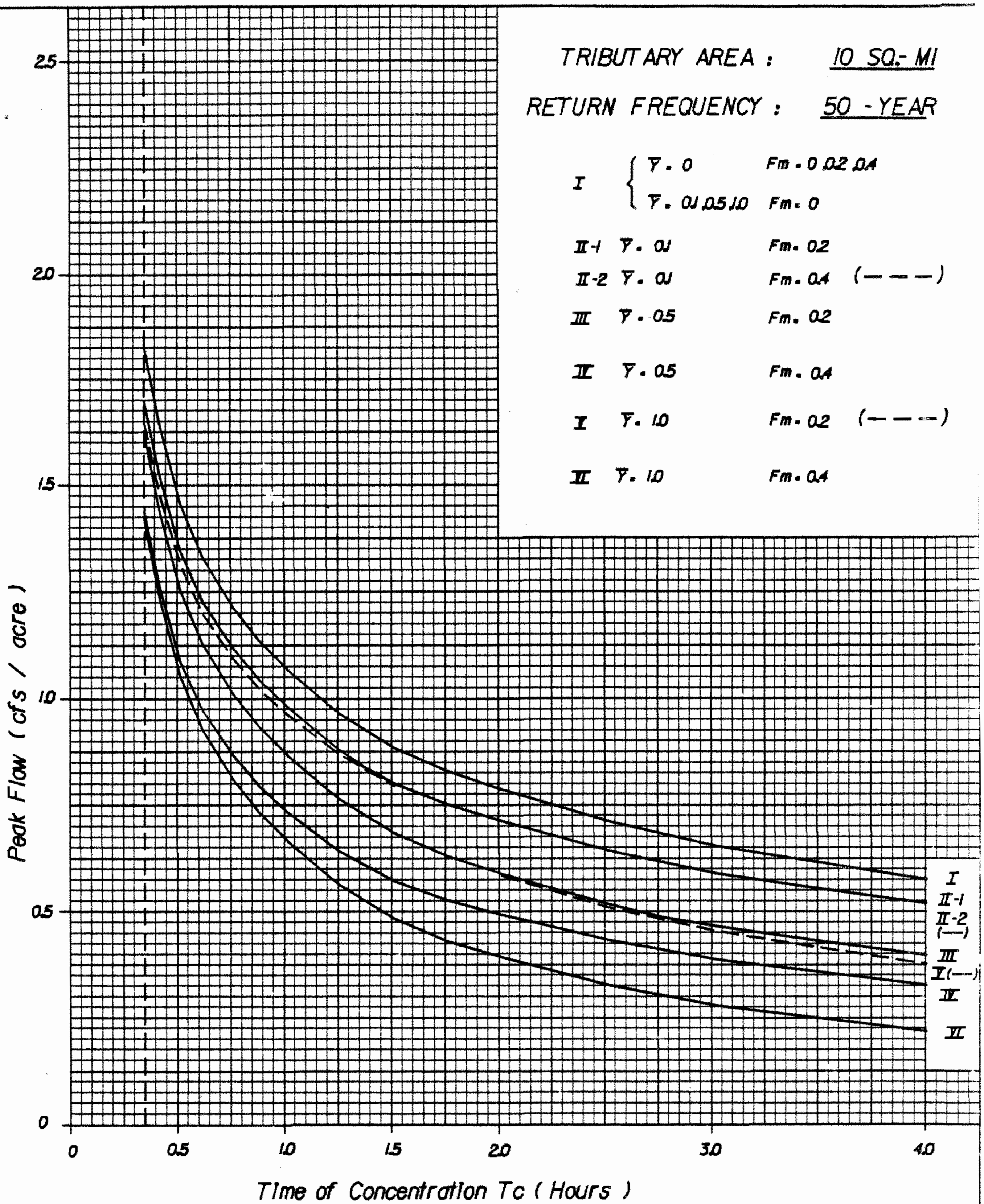
PEAK FLOWRATE CURVES



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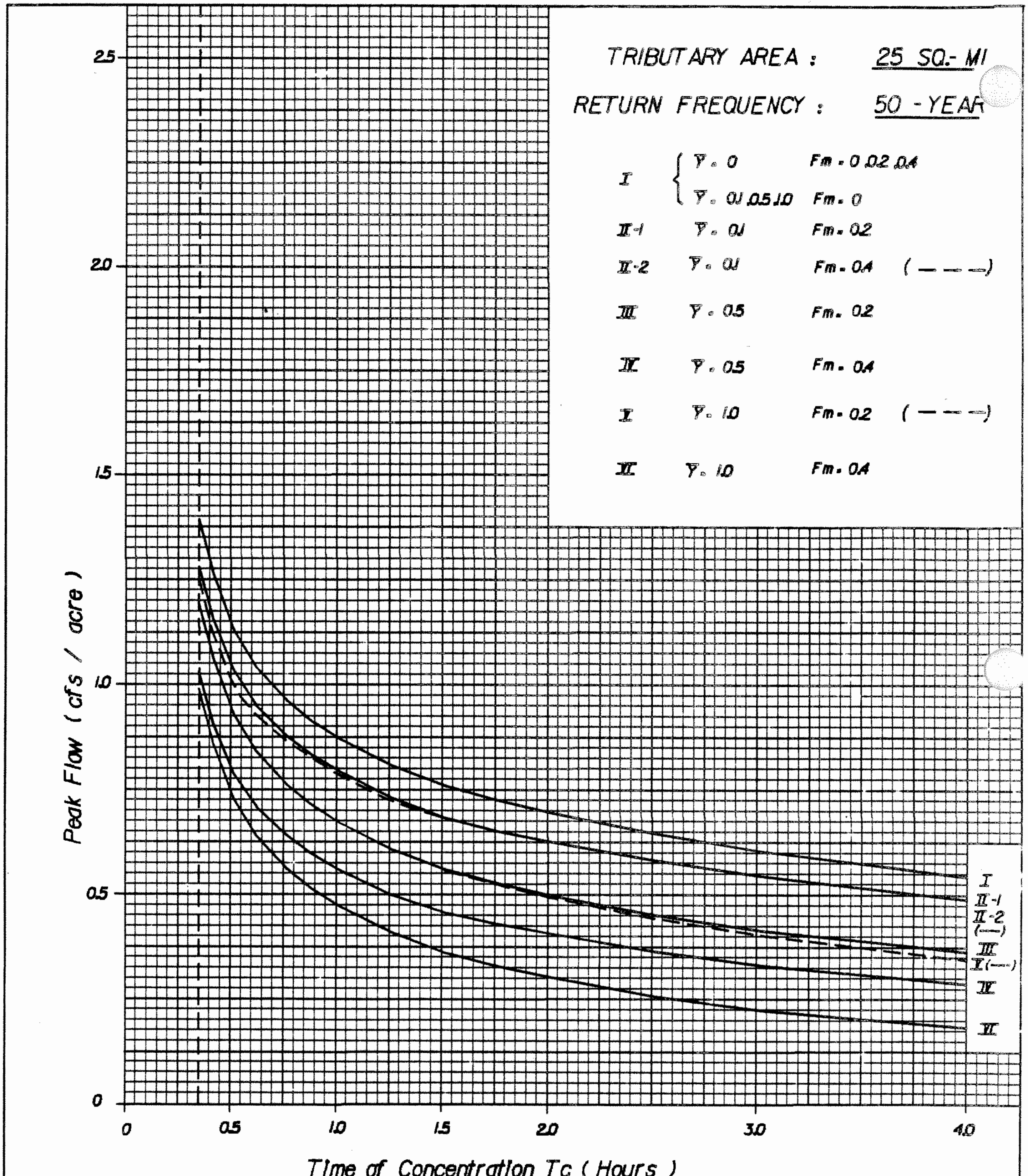
PEAK FLOWRATE CURVES



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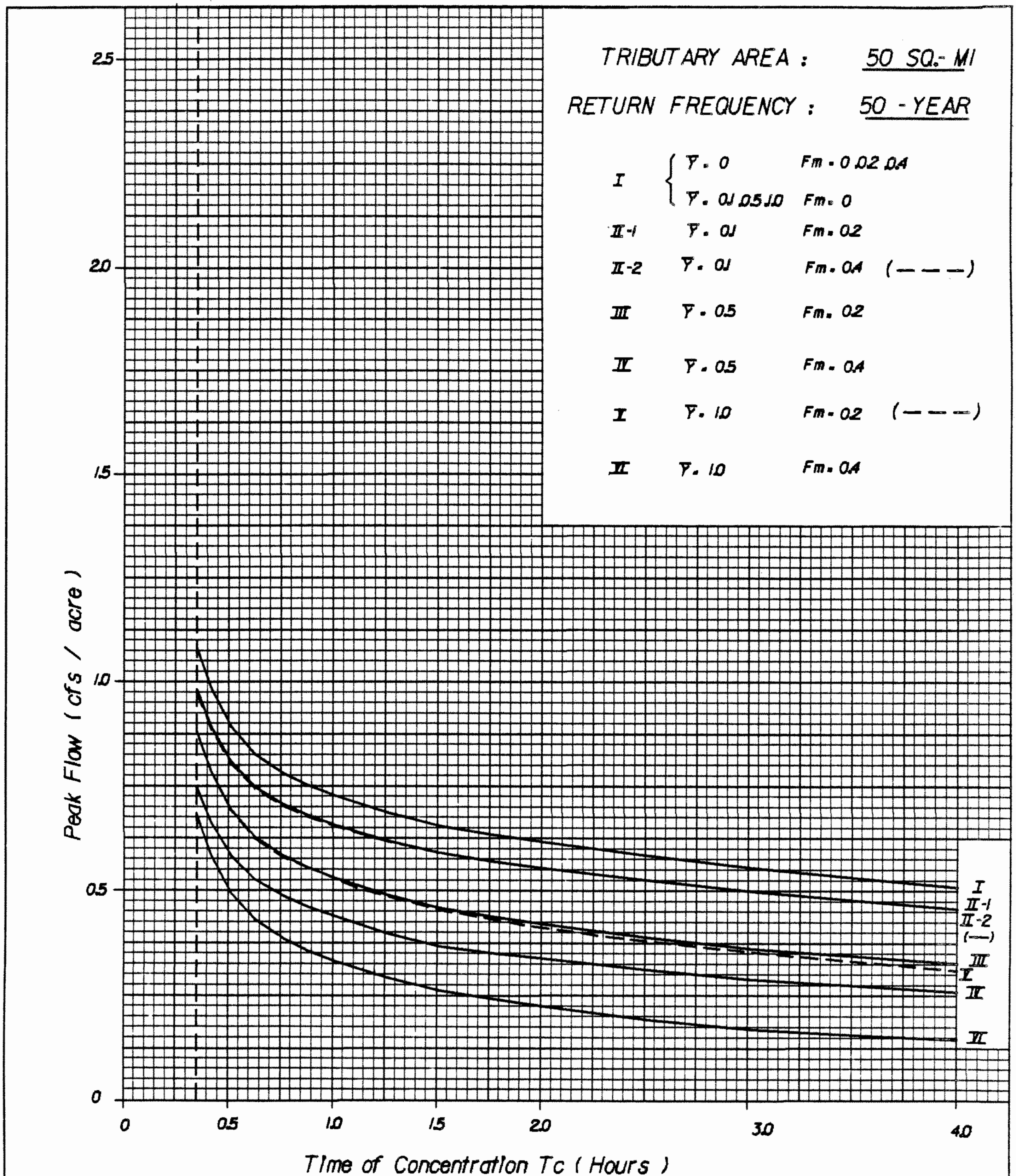
PEAK FLOWRATE CURVES



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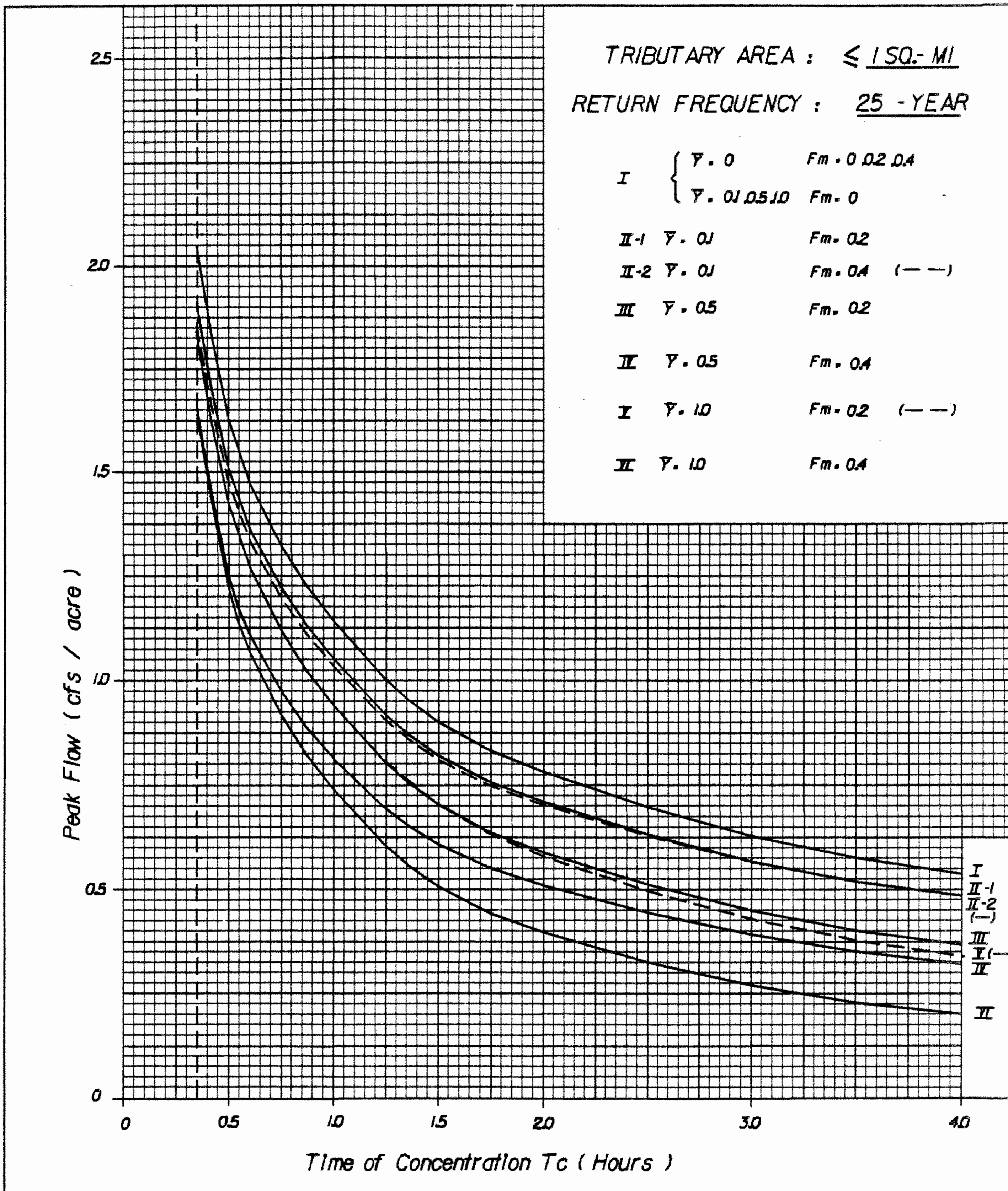
PEAK FLOWRATE CURVES



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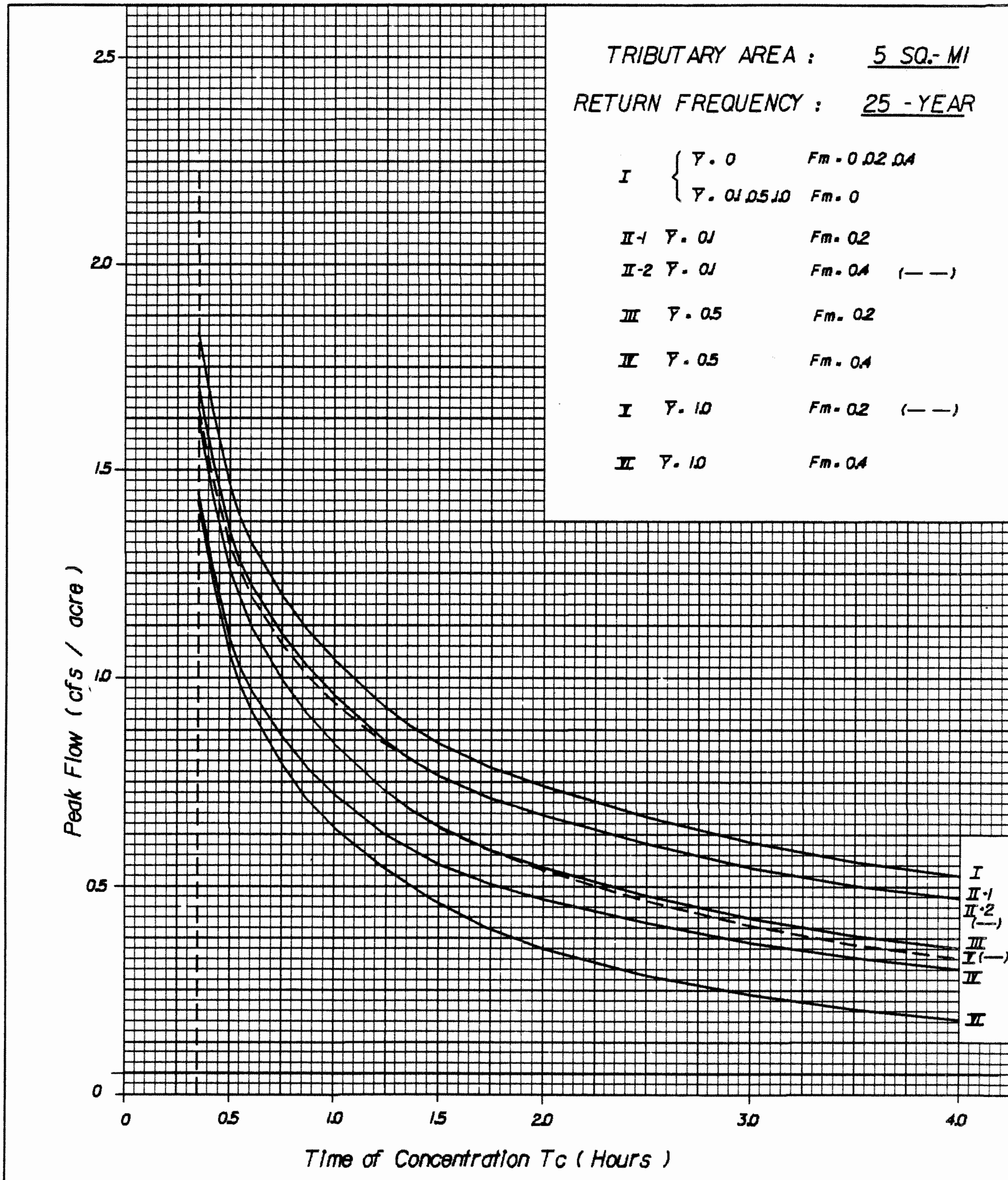
PEAK FLOWRATE CURVES



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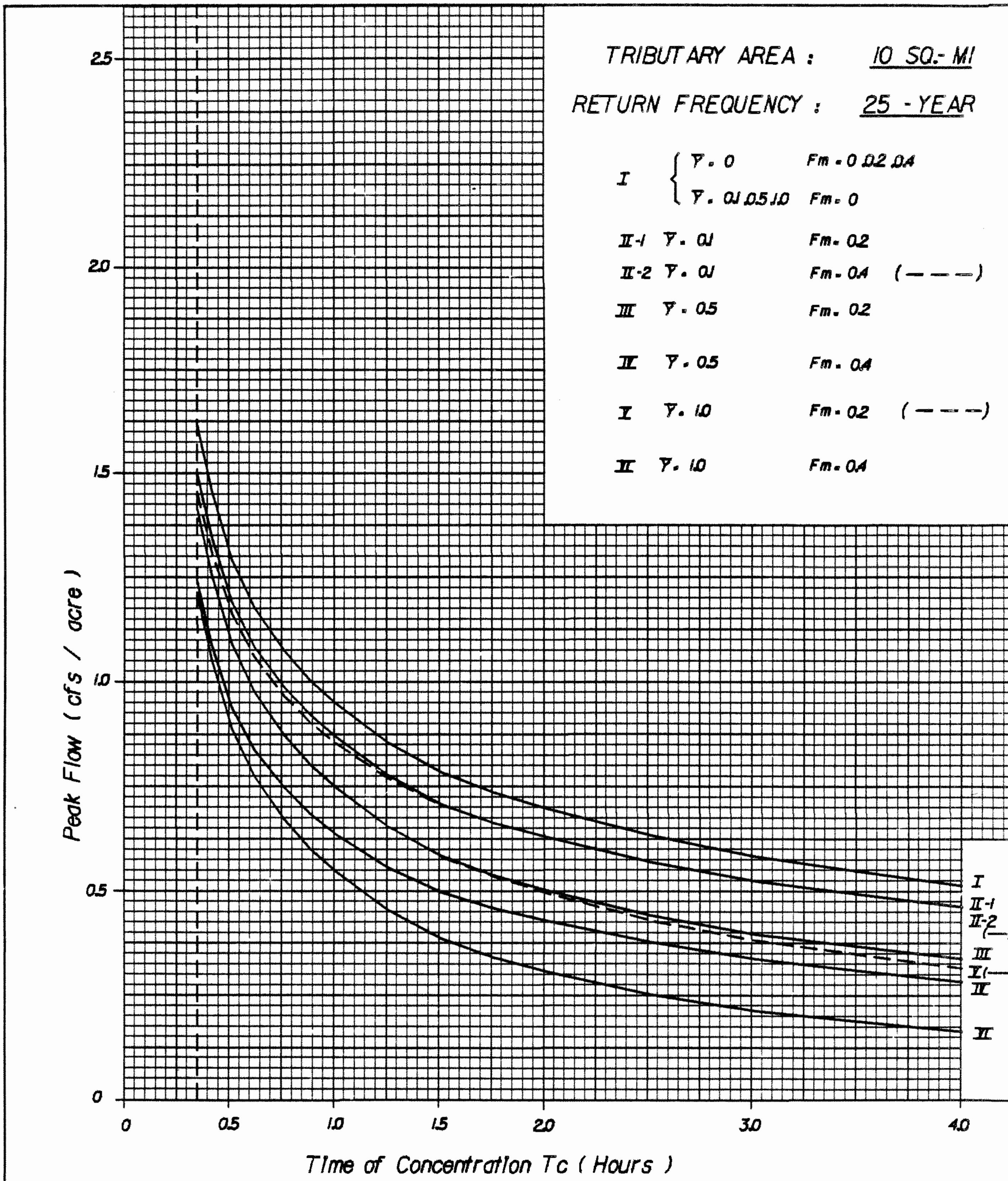
PEAK FLOWRATE CURVES



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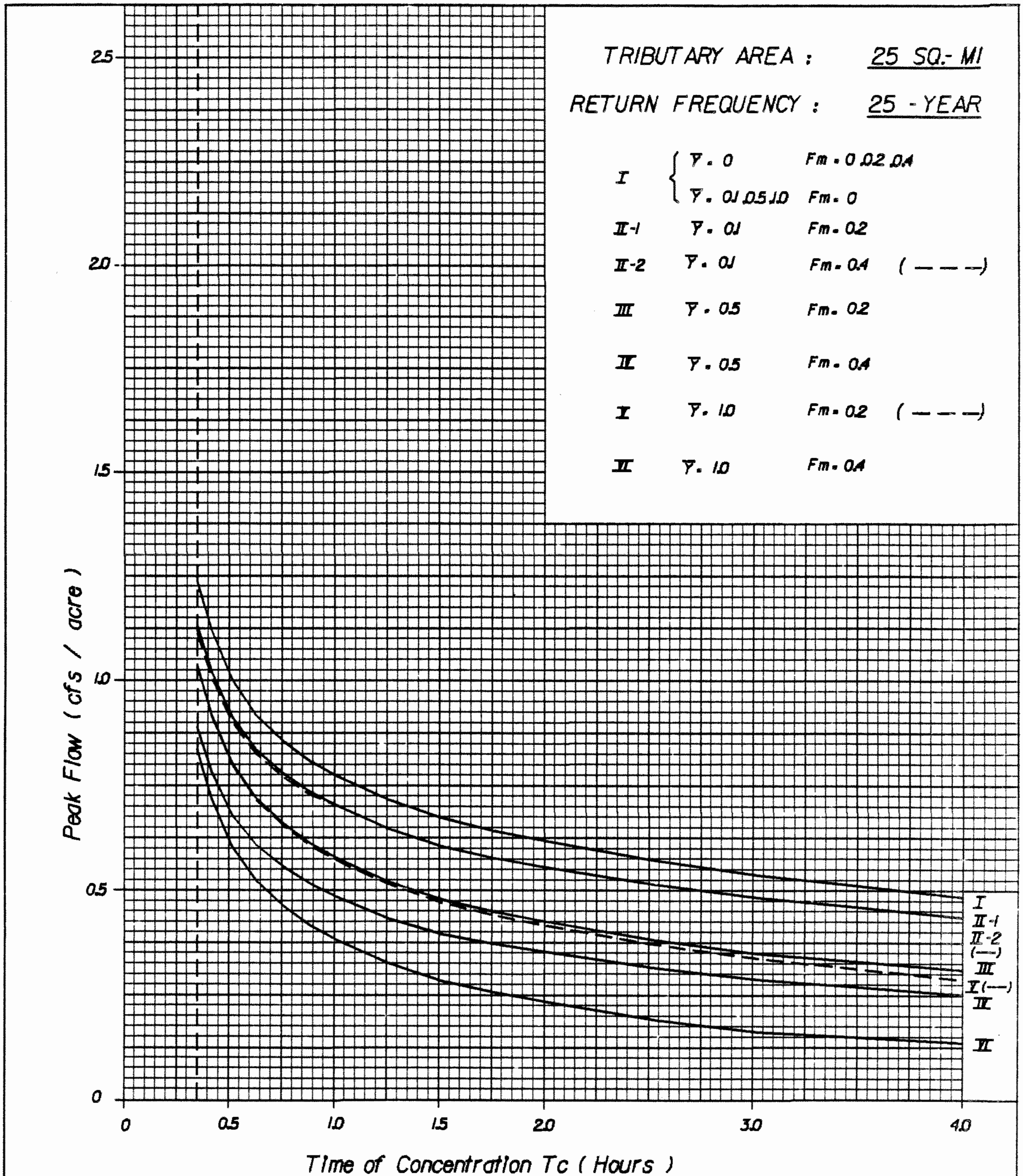
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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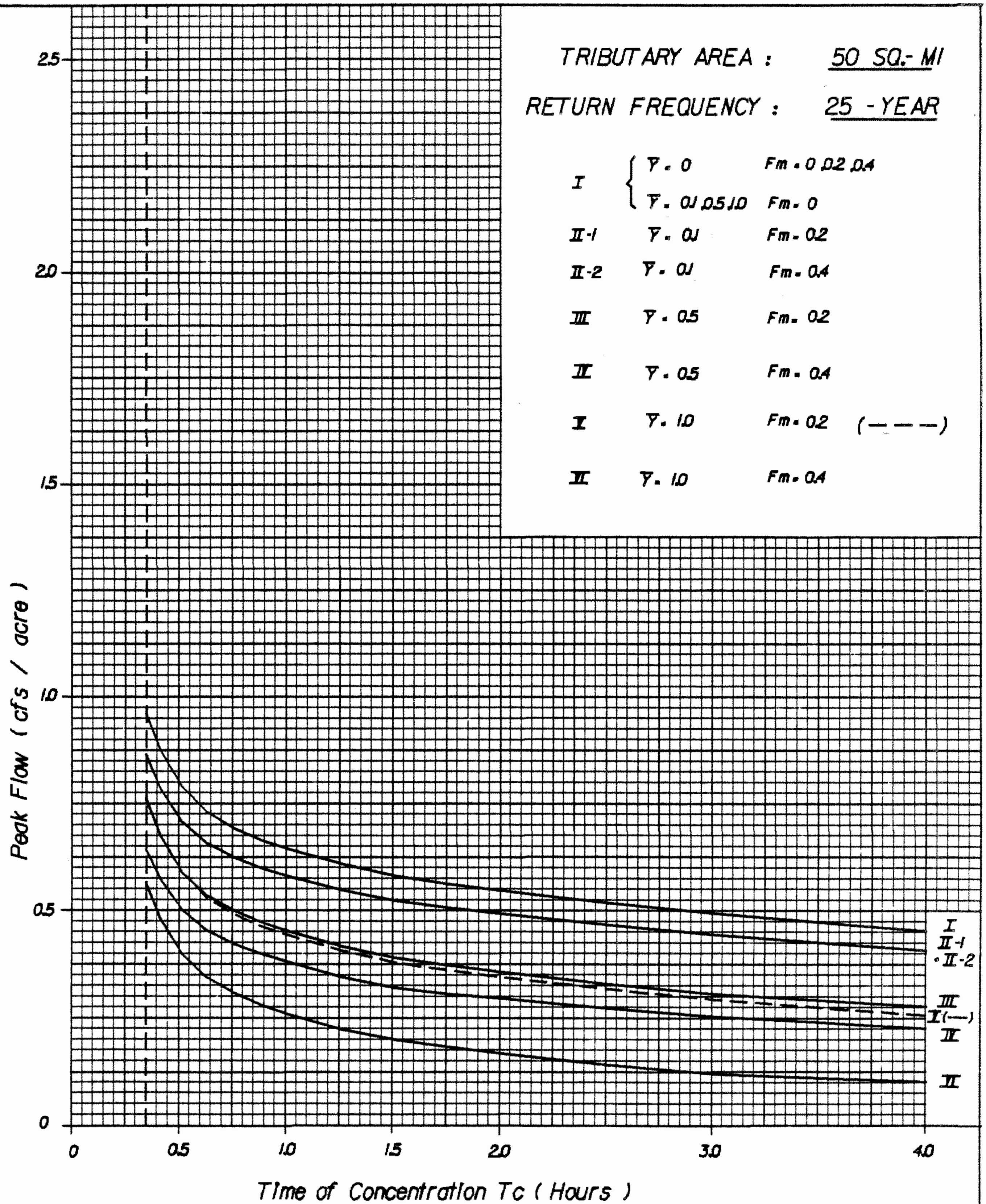
PEAK FLOWRATE CURVES



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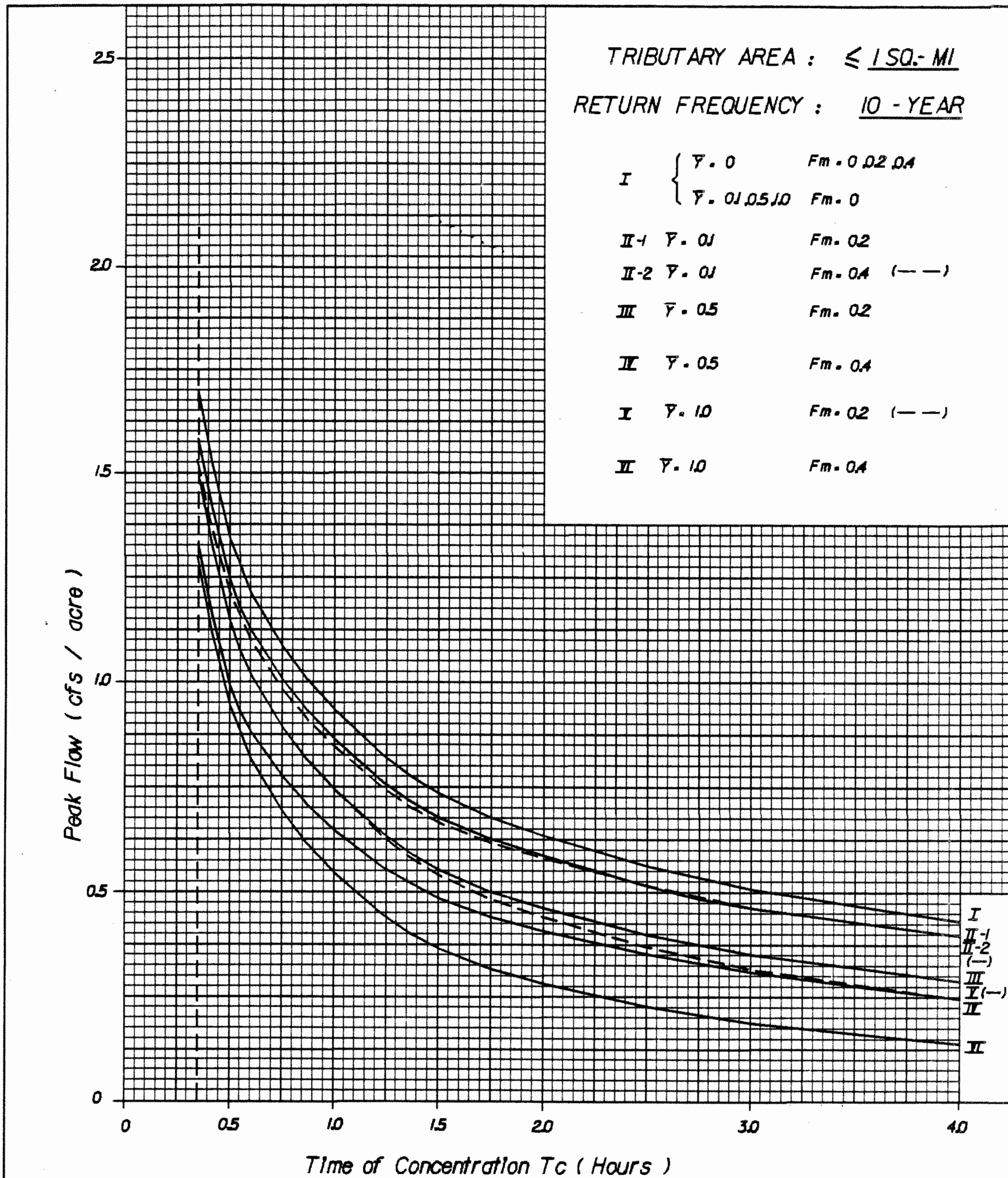
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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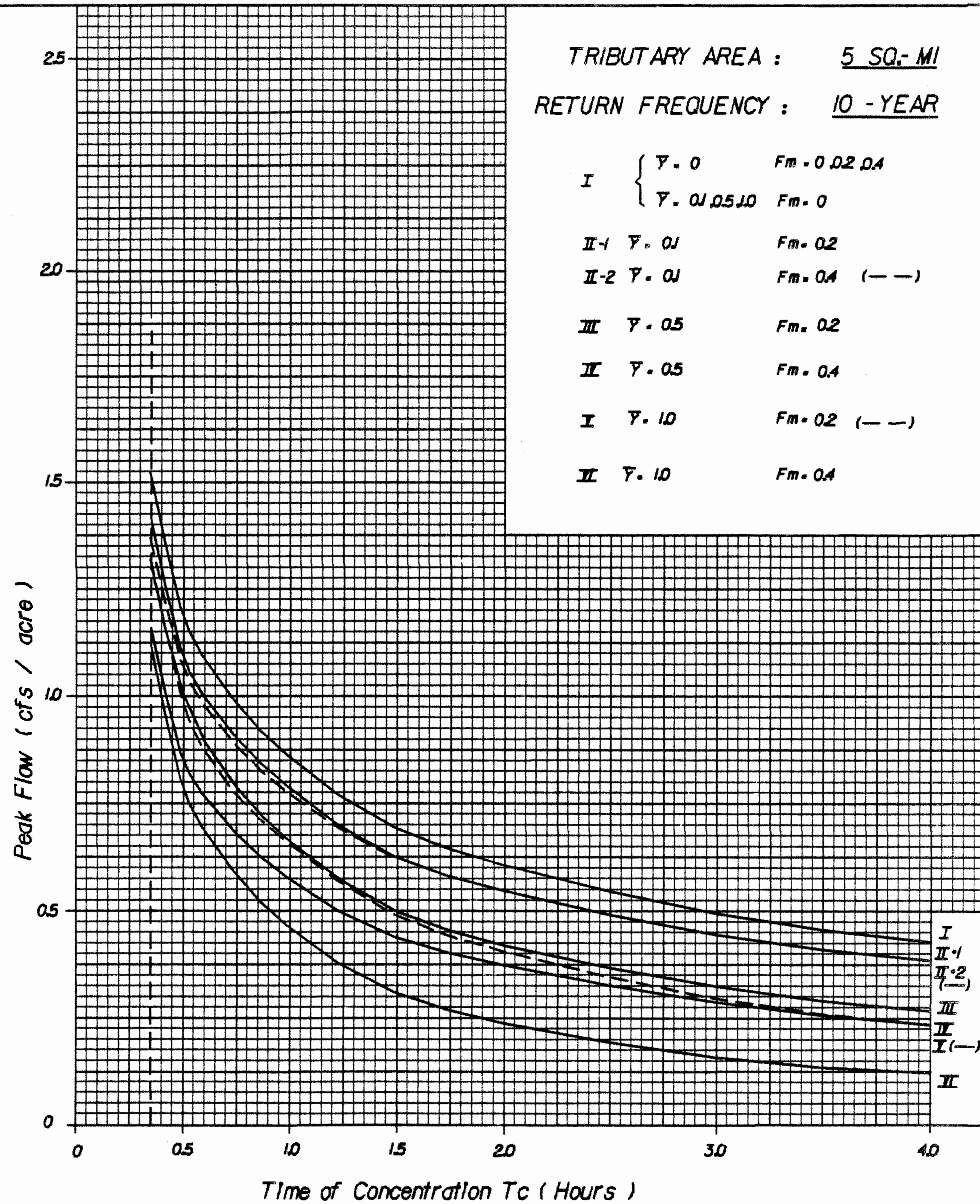
PEAK FLOWRATE CURVES



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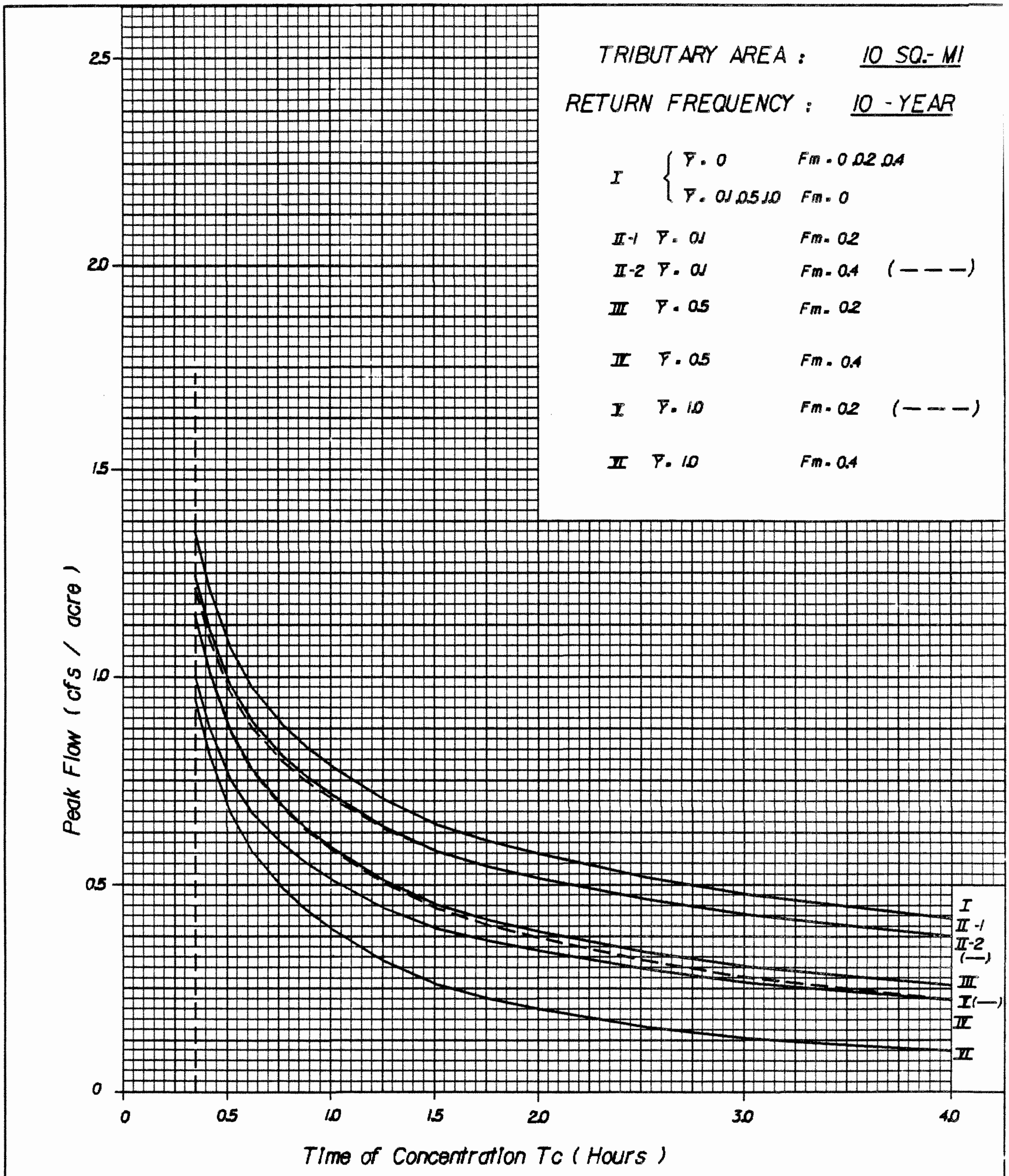
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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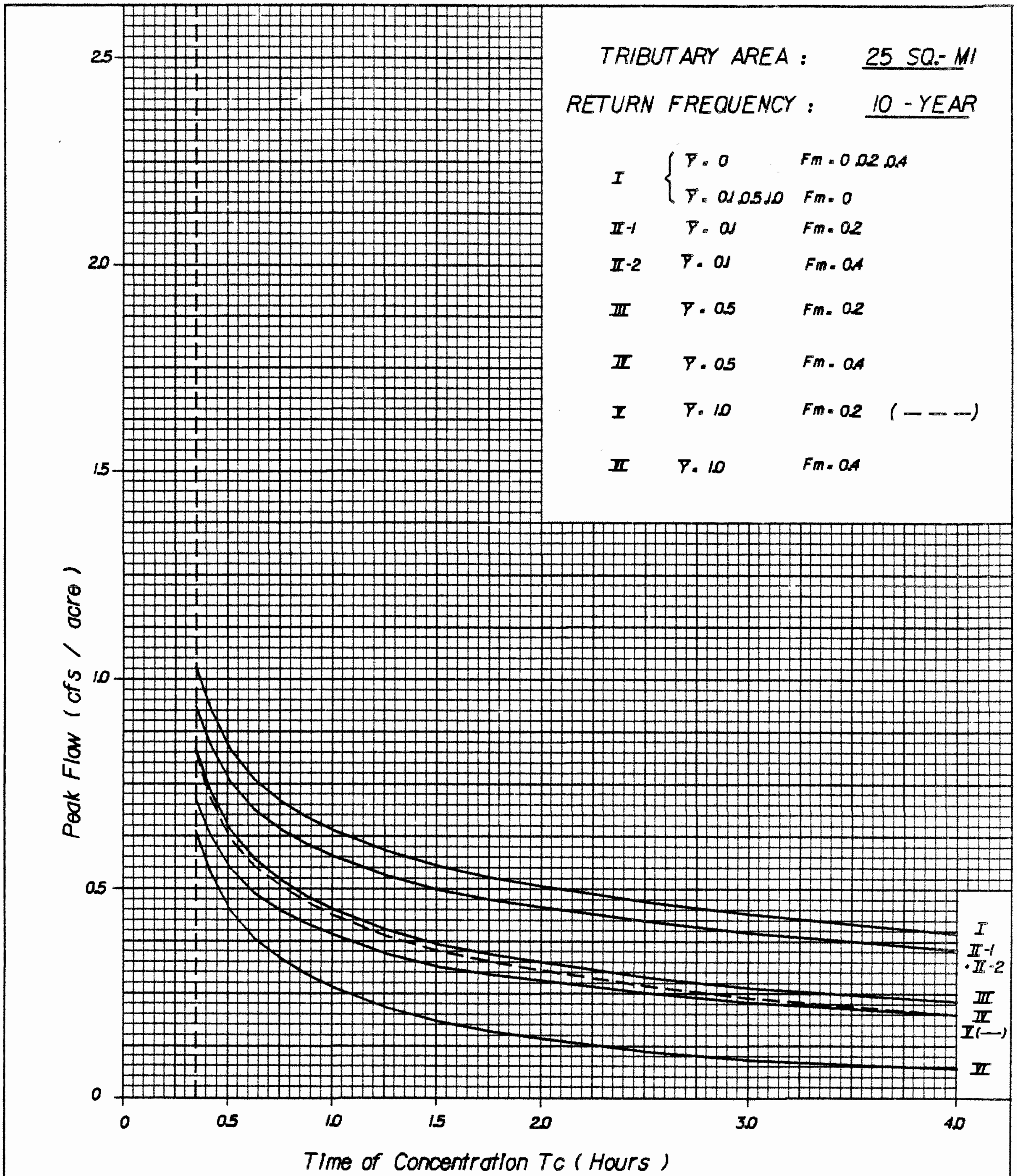
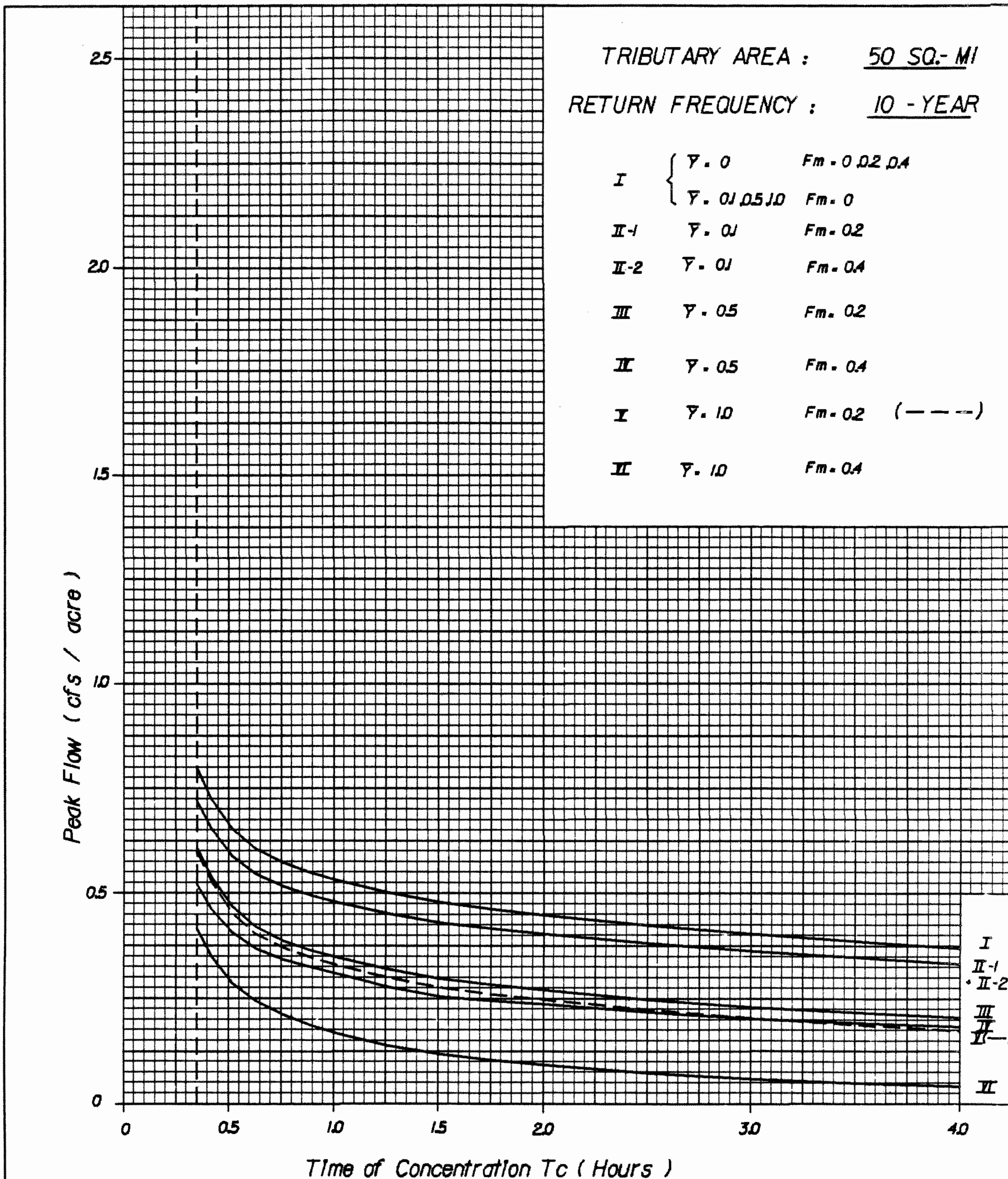
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



ORANGE COUNTY
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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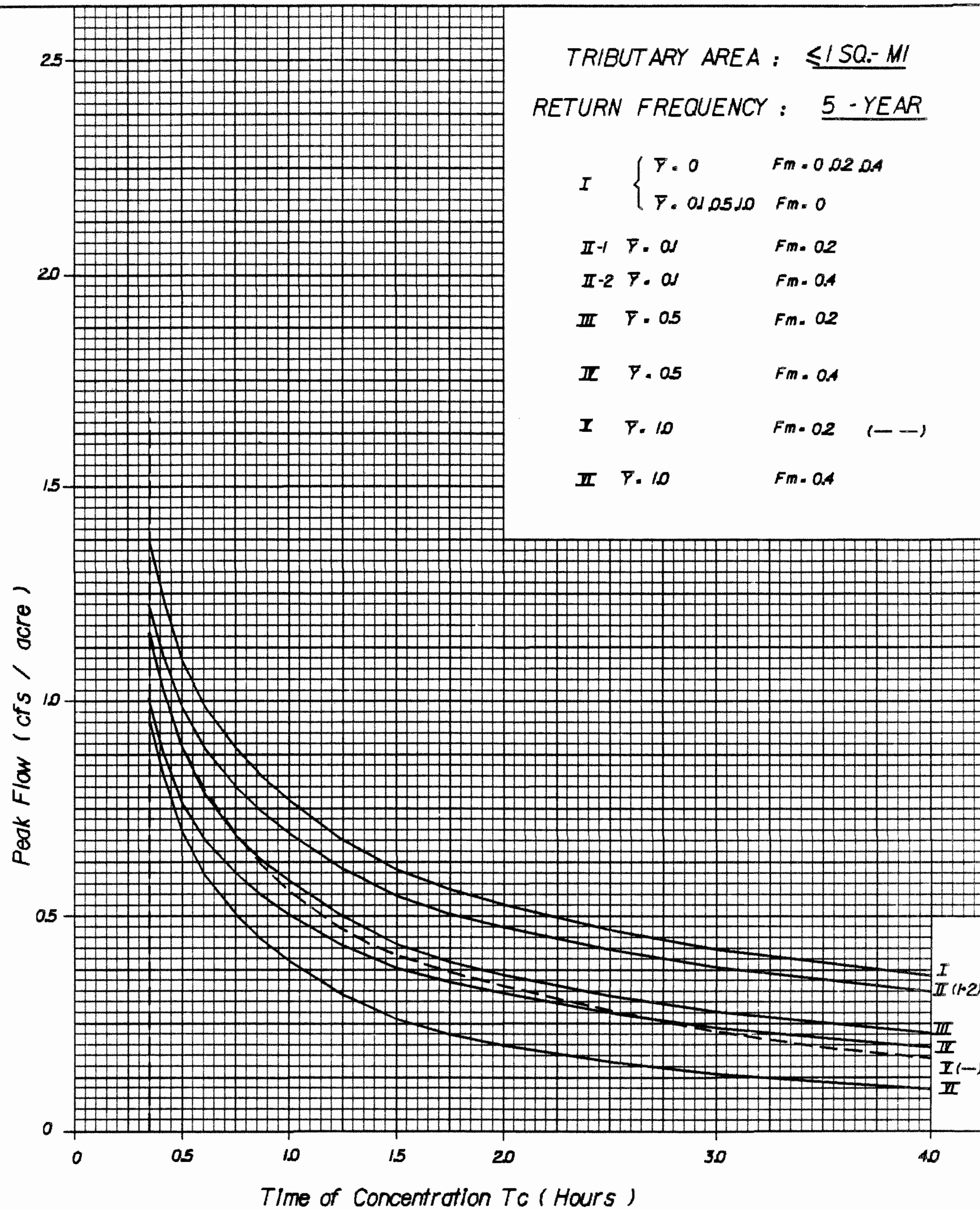
PEAK FLOWRATE CURVES



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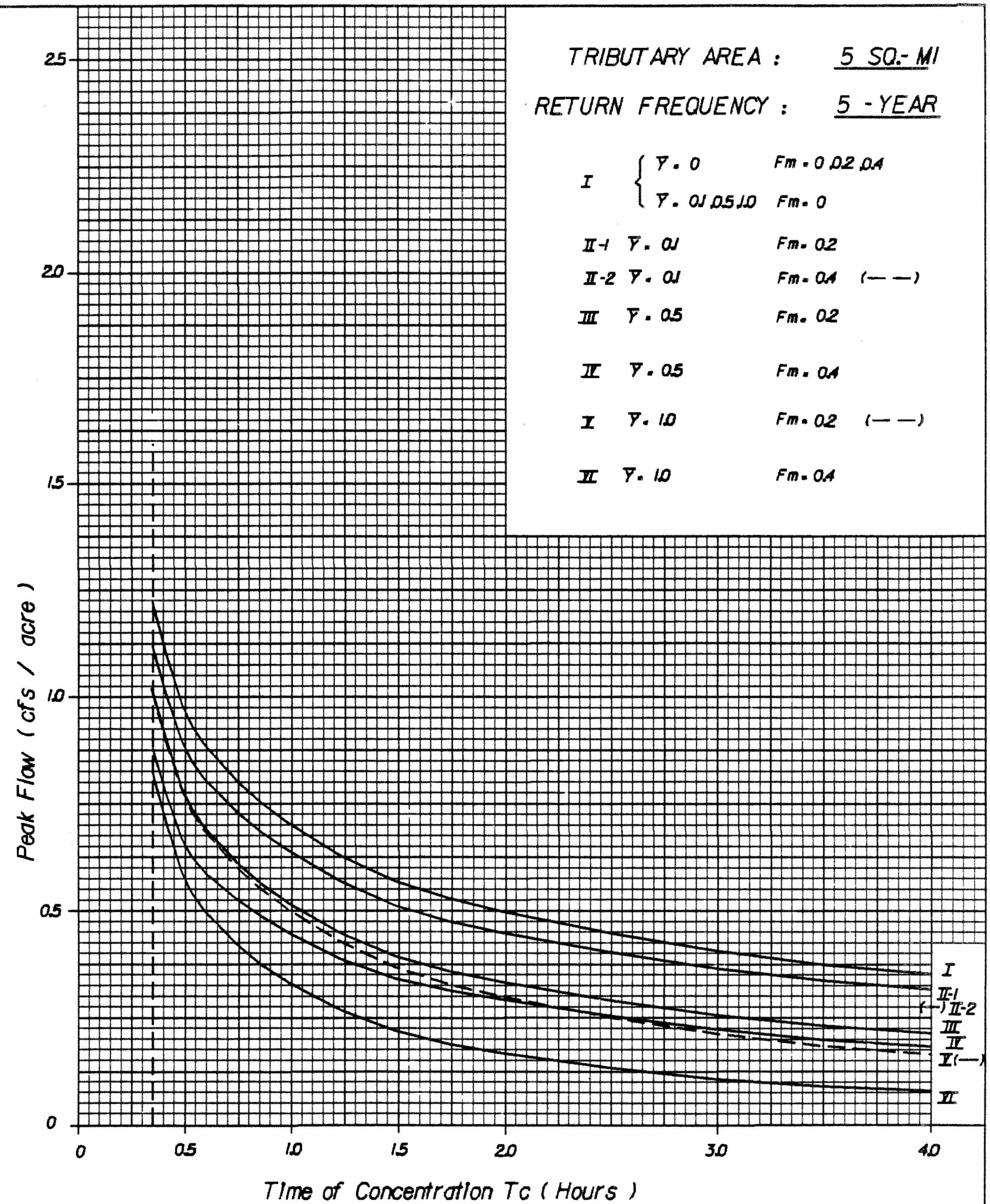
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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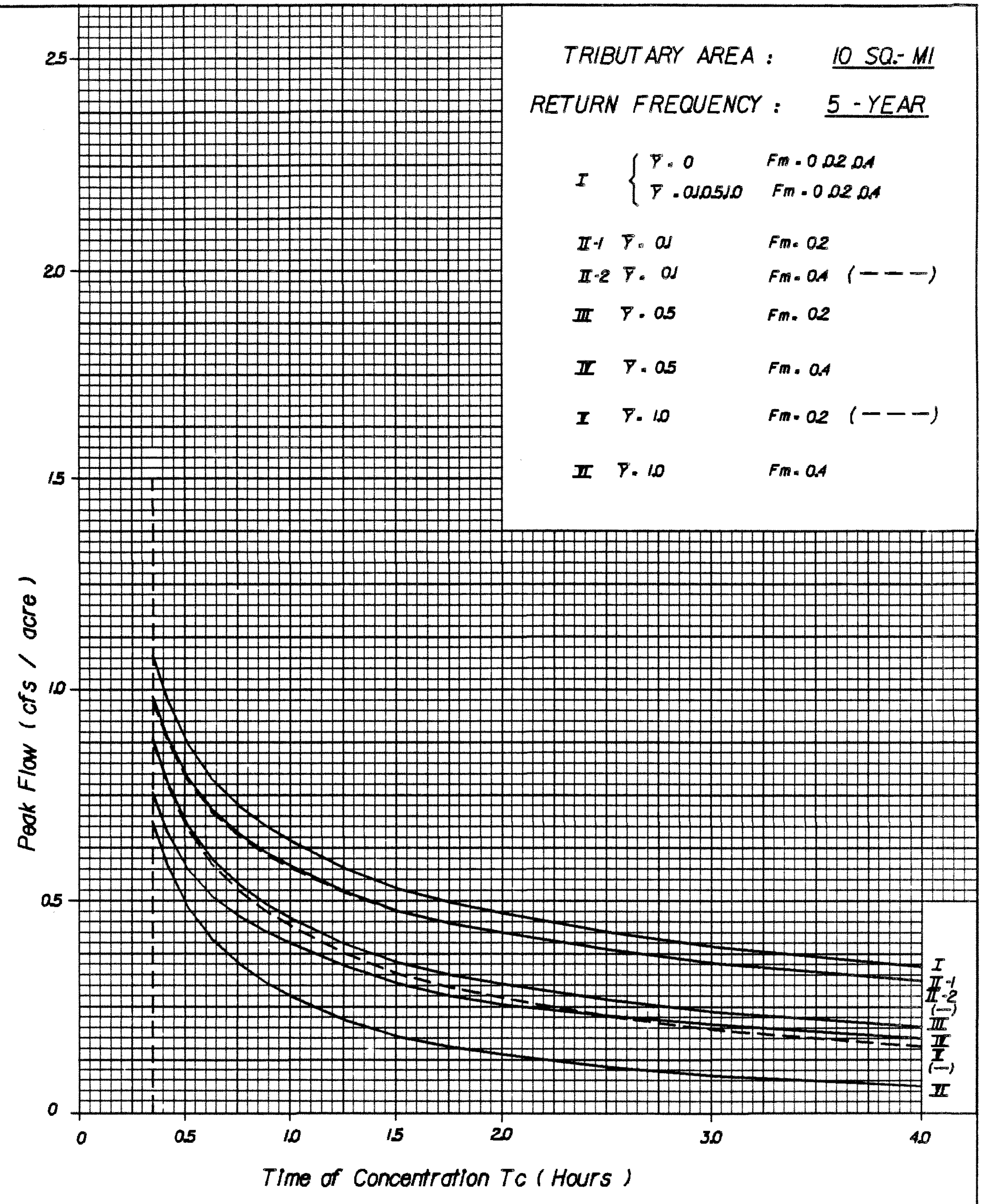
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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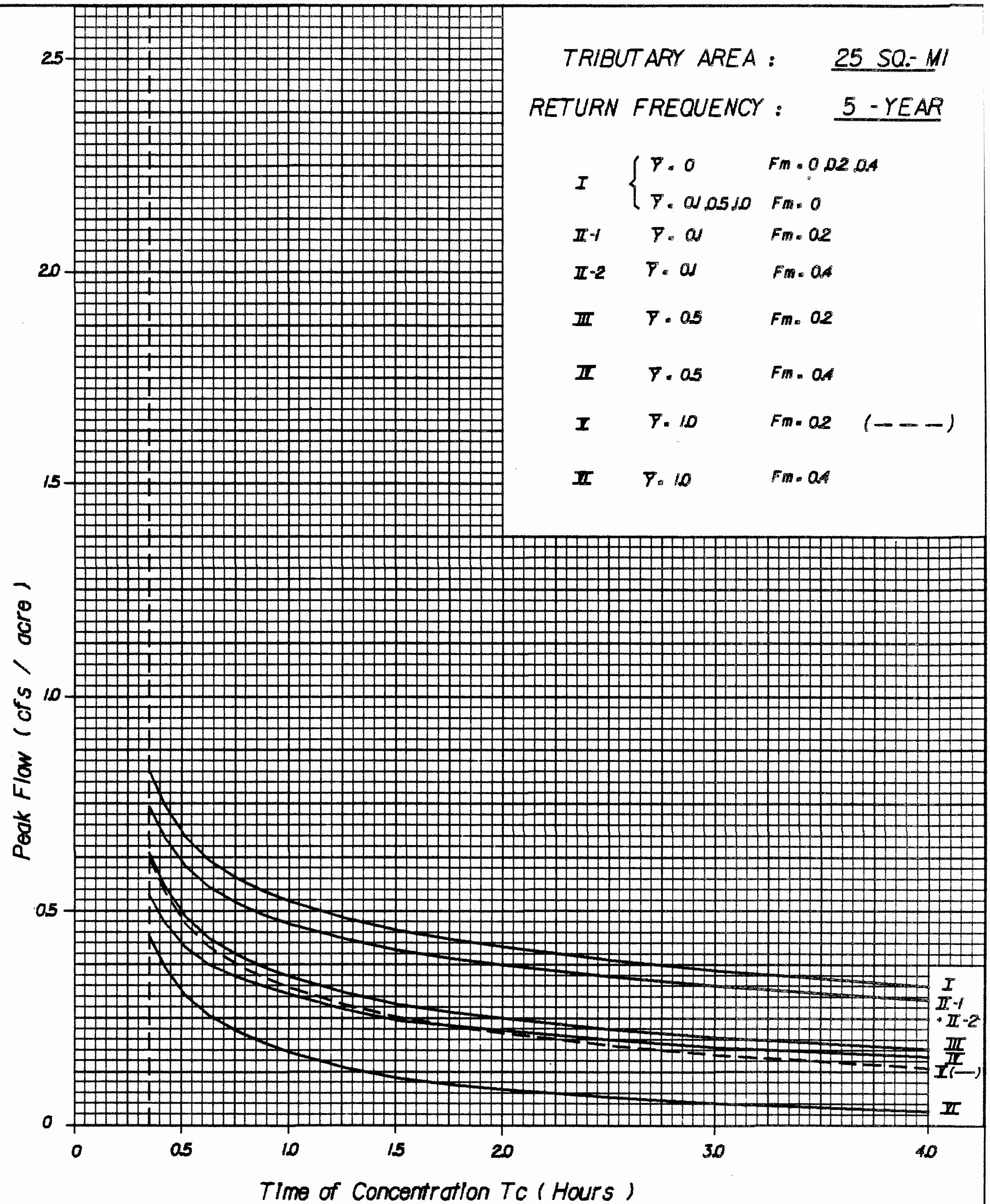
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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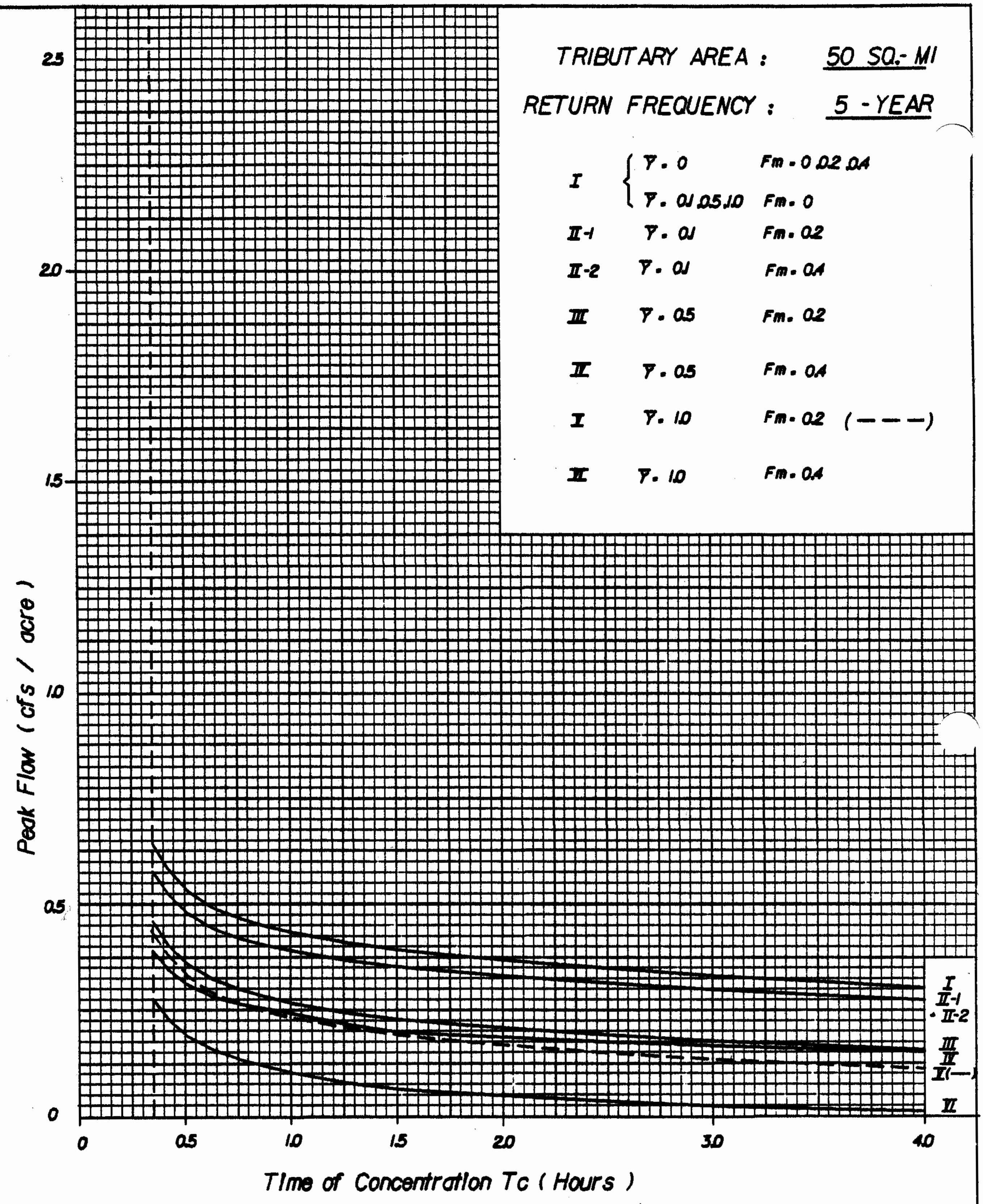
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



ORANGE COUNTY
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES

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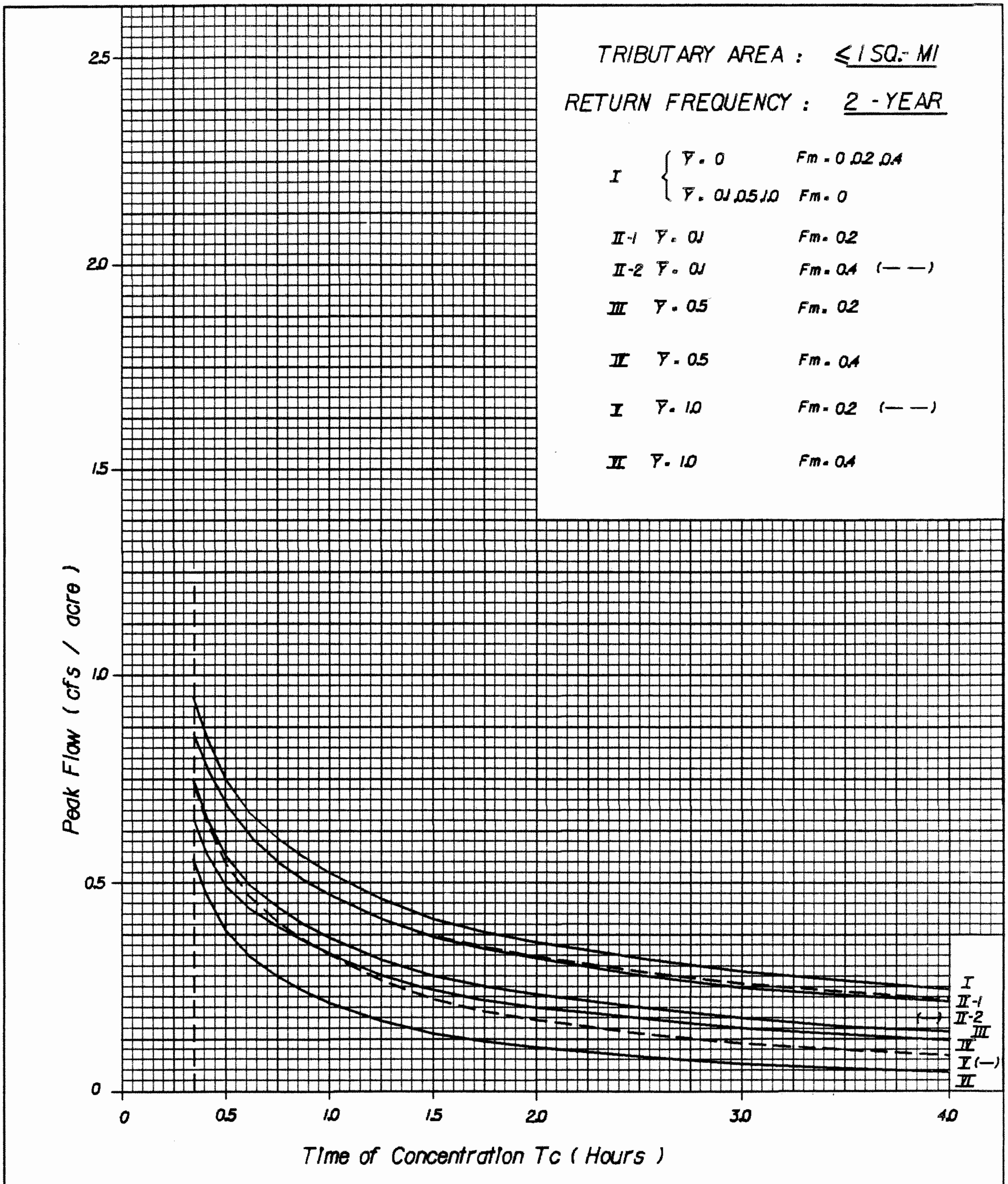
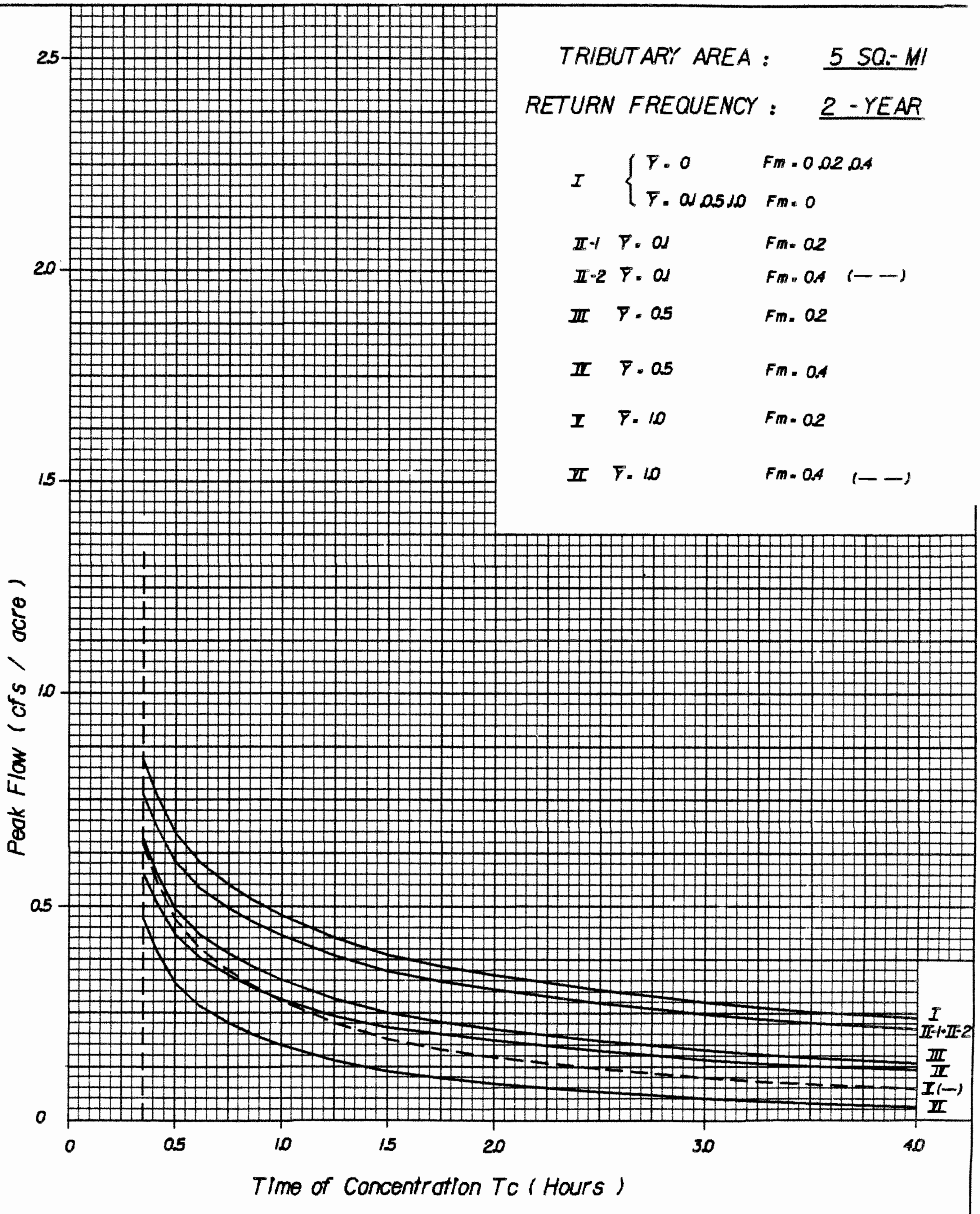


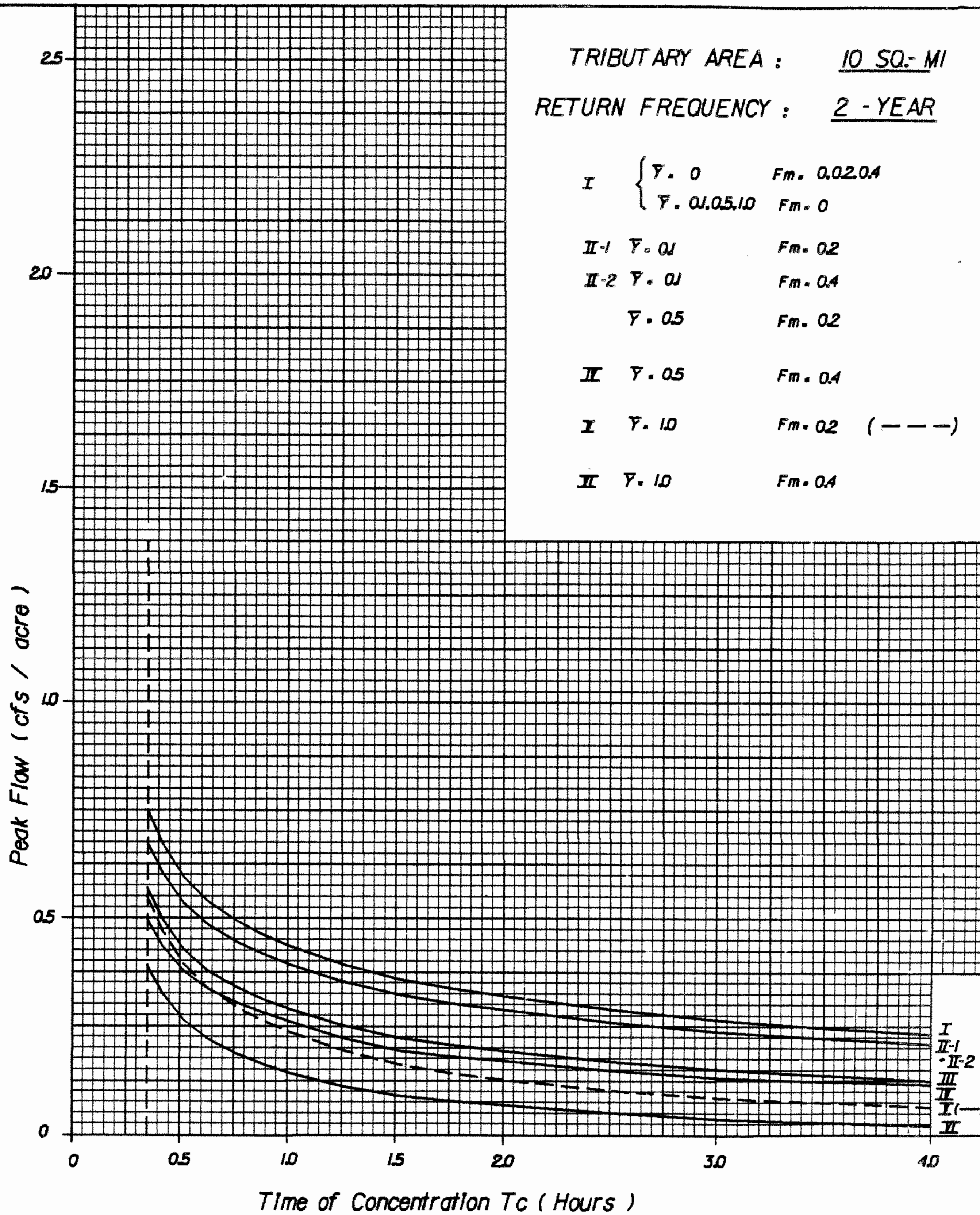
ORANGE COUNTY
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



ORANGE COUNTY
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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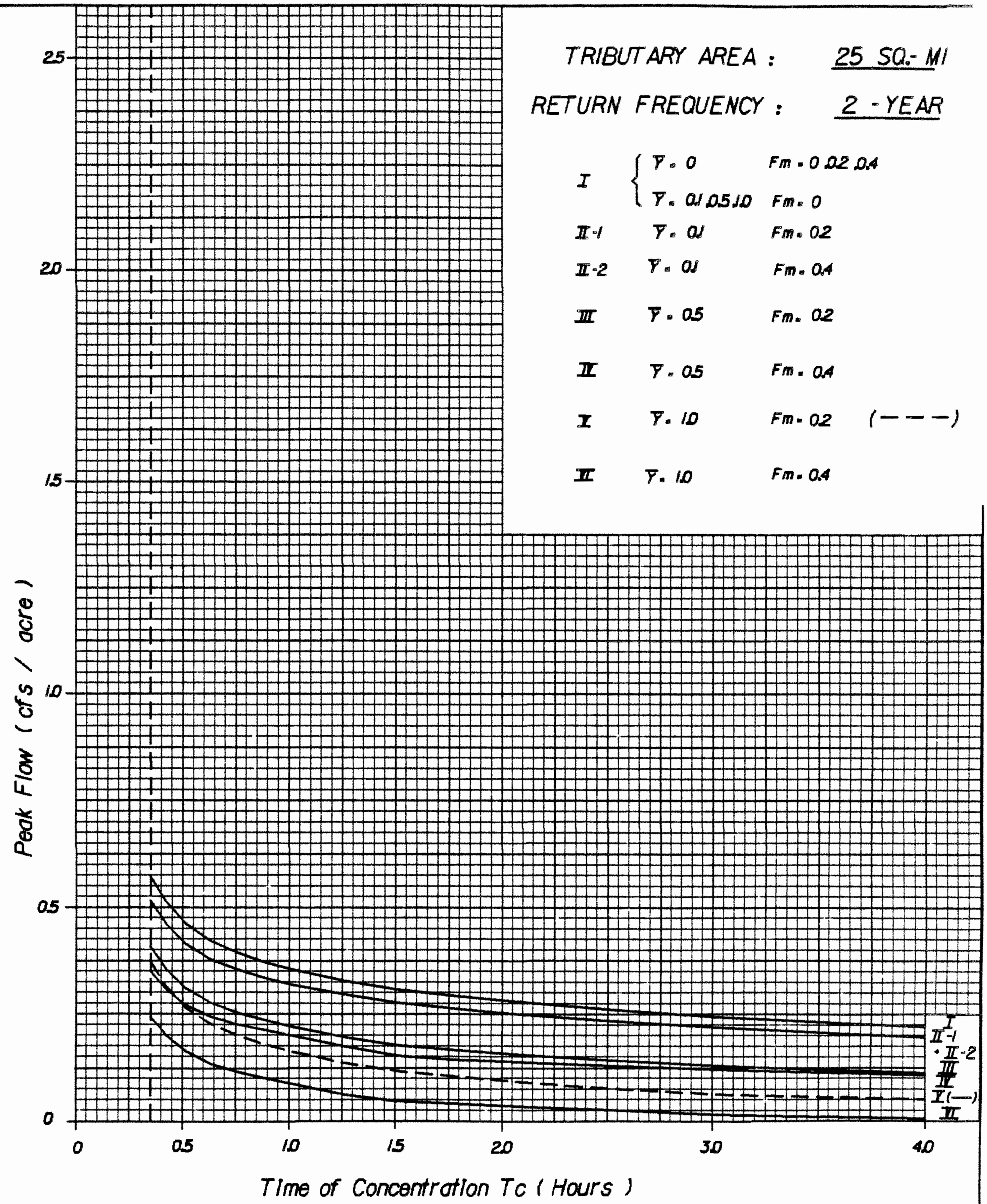
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



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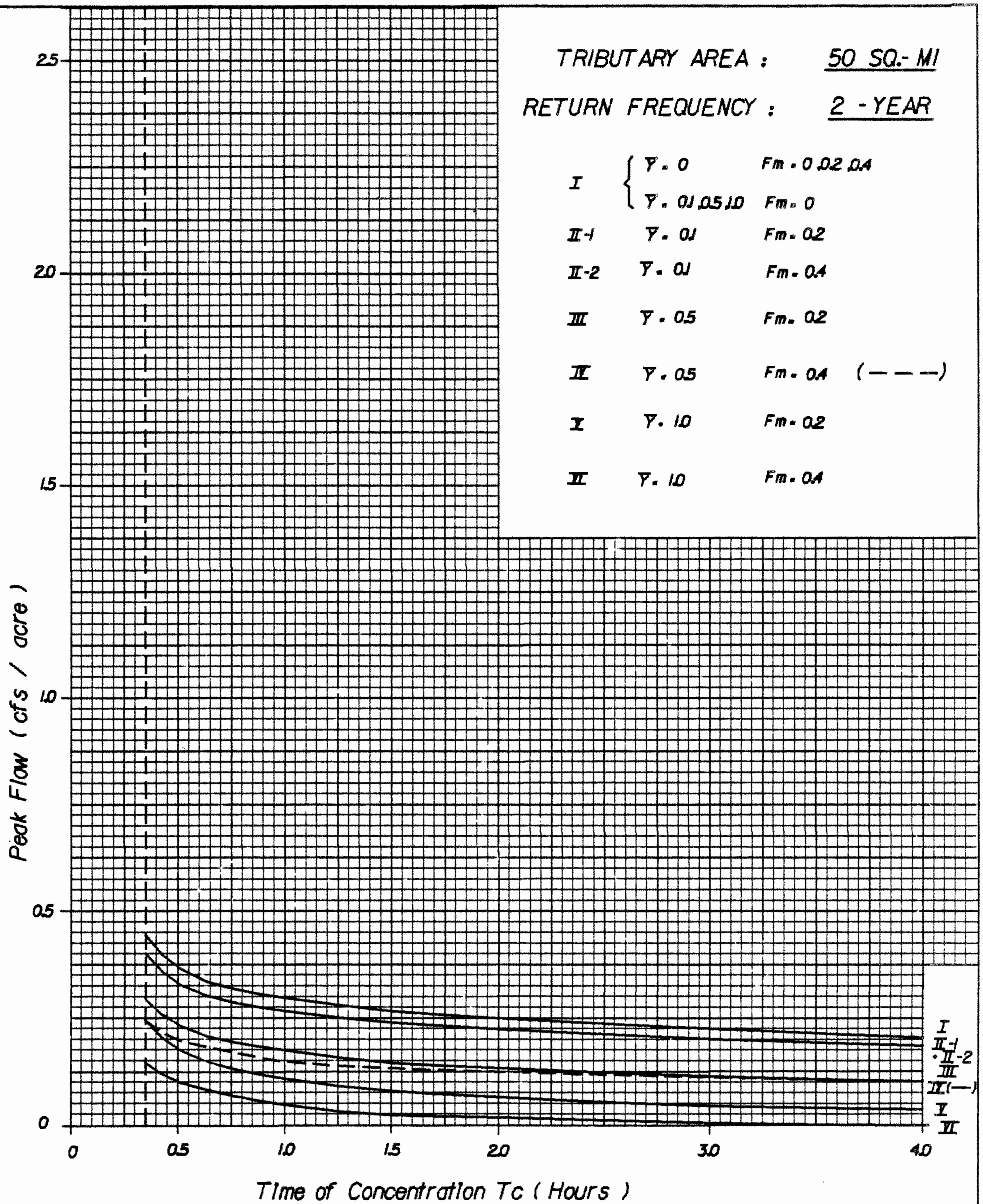
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES



ORANGE COUNTY
HYDROLOGY MANUAL

PEAK FLOWRATE CURVES

APPENDIX I

I.1. HYDROMETEOROLOGICAL CHARACTERISTICS

Precipitation in the County results from three distinct mechanisms. The most important is the convergence mechanism associated with general winter storms. These storms originate as low pressure cells in southern Alaska, arcing across the Pacific Ocean and picking up moisture as they move south and east. On occasion, these storm tracks move far enough south so that precipitation is widespread across southern California. The second major precipitation mechanism occurs through orographic lifting, and is also associated with general winter storms. As storm tracks move into the southern California coastal areas, winds usually travel easterly. Mountain masses deflect moisture laden air masses upward, increasing condensation and precipitation. The Santa Ana Mountains present a natural barrier to such air masses and increase precipitation amounts by orographic lifting in the eastern most portion of Orange County.

The third class of mechanisms causing intense precipitation are the convective mechanisms. Such mechanisms produce thunderstorms which usually produce very intense rainfall and hail for relatively short durations. Such storms are usually of small areal extent. One of the most intense convective rainfall events of record in southern California occurred at Campo, California, near the Mexican border, producing over 11 inches of rainfall in about 80 minutes.

Occasionally, unstable tropical air masses invade southern California and produce rainfall. These air masses are generally associated with convergence mechanisms; however, because of instability, pockets of convective activity may occur and produce intense thunderstorms (e.g., Arnold Court, NWS WR-158, Oct. 1980, "Tropical Cyclone Effects on Southern California.")

The major floods in Orange County have been primarily the result of orographic storm precipitation. However, it is known that convergence precipitation can contribute a significant portion of the total rainfall in a predominantly orographic storm. The cyclonic circulation inherent in all large orographic storms, for example, involves horizontal convergence and assures widespread convergence precipitation in nearby non-orographic areas. Convergence bursts are observed during periods of heavy orographic rain. Thus, the occurrence of large amounts of convergence and orographic precipitation in the same storm is an established fact. These general observations are supplemented by the severe coastal storms of 1974 and 1983, both of which produced more rainfall near the coast than in the eastern mountains.

Major storms in Orange County occur in the cooler months from October to April. Storms typically originate with cyclonic disturbances along the polar front in the vicinity of the Aleutian Islands. As a storm center moves southward, meteorological conditions usually force the storm inland before the southern California latitude is reached, precipitating the greatest quantity of the storm's moisture on the northern Pacific areas. The usual result is relatively gentle rainfall in the southern California areas continuing sometimes for many days. Occasionally, with the right combination of conditions, storm producing air masses move directly southward over the Pacific Ocean picking up warmth and moisture at low levels and remaining cold and humid at higher levels. Such storms may sweep in on the southern California Coast. As moisture laden air encounters the Santa Ana, San Bernardino, San Gabriel and San Jacinto mountains, it is deflected upward where cooling and pressure reduction induce precipitation. A typical storm of this type was that of February 27 to March 4, 1938 which resulted in one of the most disastrous floods of record in southern California.

The approach direction of the storms which reach Orange County may vary from northwest through west to southwest. For example, satellite tracking records indicate that storm cells may approach Orange County from the southwest, originating from tropical hurricanes located westerly of central Mexico.

It is common for successive storms of varying duration and intensities to compound their effects, with the heavy rainfall of the second or third storm creating the more severe flood conditions. It is known that the rainfall once lasted for approximately thirty days with relatively few breaks, covering the period from December 24, 1861 through January 24, 1862. It is probable that this deluge included two or more individual storms.

I.2. FLOOD HISTORY

The history of floods in Southern California pivots on the year 1825. Prior to 1825 the surviving documents, principally from the Spanish missions and a few personal diaries, are very few and quantified data and maps of inundation are entirely absent.

The following chronology is abstracted from many sources and is focused on the great floods which have inundated southern California (excluding descriptions of storms such as Campo (1891), Indio (1939), San Bernardino (1983) and others that struck desert areas):

1825 - The Los Angeles River changed its outlet from Santa Monica Bay to its present location in San Pedro Bay. The Santa Ana River changed its outlet from Anaheim Bay (Seal Beach) to Newport Bay.

1862 - The greatest flood in the history of California. Water flowed four feet deep through central Anaheim as the Santa Ana River sought to find its historic outlet in Seal Beach. Documented by paleohydrologic methods in 1967 as delivering 315,000 cfs near the present Route 60 bridge, a discharge three times larger than any subsequently measured flow at any point on the Santa Ana River. The recently acquired knowledge about the magnitude of this event has influenced the design of the proposed Santa Ana River improvements by the Corps of Engineers.

1891 - Thirty inches of rain measured at Big Bear in 36 hours. Overbank flows produced great damage in San Bernardino County.

1916 - Thirty inches of rain over 15 days measured in San Diego where Otay Dam failed disastrously, Sweetwater Dam was severely damaged and all road and rail traffic into the city of San Diego was blocked by damaged bridges for a month. Supplies were delivered by boat and barge until the bridges were restored. Detailed maps show that western Orange County was inundated by the Santa Ana River (75,000 cfs) again seeking its historic outlet in Seal Beach. Santiago Creek overflow along Prospect Avenue reached Newport Bay and created a large lake near the present blimp hangers at the Marine Corps Air Station.

1938 - The largest flood in Southern California since 1862. The Santa Ana River inundated western Orange County again in a pattern similar to 1916 but the higher flows (100,000 cfs) and denser development led to 45 fatalities.

1969 - Two long duration, moderate intensity but high volume storms in January and February led to uncontrolled spillway flow at Santiago Dam and Villa Park Dam. The January storm was the most severe in Orange County since 1938 and the February storm was still larger. The sustained discharges over many weeks produced severe erosion damage to unlined flood control channels throughout the county. Water flowed over the Santa Ana Freeway in the Irvine area. The spillways at both dams on Santiago Creek were damaged. Two bridges failed and many were damaged.

1974 - The first large scale high intensity storm to strike Orange County after the Orange County Flood Control District installed its network of automatic recording rain gauges. Three hour, 100-year rain depths covered approximately 100 square miles near the coast. Record breaking rainfall did not extend inland past the Santa Ana Freeway contradicting the conventional prediction that rainfall from large storms would be greater in the mountainous inland areas than at the coast. Widescale local flooding occurred in coastal areas.

1978-1980 - Both years had extremely high annual total rainfall (second and seventh greatest since 1909). The duration of the 1980 flows in the Santa Ana River combined with levees saturated by weeks of rainfall led to severe invert erosion (greater than 20 vertical feet at the Fifth Street bridge) and lining failures over the entire length of the channelized river in Orange County.

1983 - At approximately 7:00 a.m. on March 1, 1983 a winter storm of record breaking intensity struck the westerly portions of Orange County. By 2:00 p.m., when the rainfall slackened, most of the short duration rainfall intensity records for the heavily urbanized central Orange County area had been broken. Arterial highway traffic was heavily congested by flooded intersections, local streets were overflowing, the Santa Ana Freeway was closed to all traffic for a period of 20 hours in Irvine, excessive storm runoff had overtopped the banks of numerous flood control channels and several leveed channels had been breached by overflowing storm water. Fortunately, the devastation was limited by the short duration (approximately 6 hours) of the storm and limited areal coverage (100 square miles) of the County. Even so, 1,100 homes were flooded with damages over $160,000,000 including public property losses.

Recording rain gages maintained by the county showed record breaking rainfall intensities for durations between 15 minutes and 6 hours at several locations. Watershed areas tributary to most Orange County flood control facilities (other than the Santa Ana River) have times of concentration ranging from 30 minutes to 3 hours for the water to drain from the uppermost watershed boundary to a point of discharge into a larger facility or the ocean. Therefore, the March 1st storm was a particularly severe test of these local facilities. In contrast, the much larger watershed of the Santa Ana River is most sensitive to heavy rainfall sustained over a period of several days. Accordingly, the Santa Ana River was not severely tested in this storm.

The recording rain gage for which the county has the longest record of short duration rainfall amounts has been in operation for 51 years in the city of Santa Ana. A shorter period of record is available at the Costa Mesa recording rain gage. Table I.1 illustrates the record breaking intensity of the March 1st rainfall by comparison with previous records.

TABLE I.1. MARCH 1, 1983 STORM RAINFALL

	Santa Ana		Costa Mesa	
Duration (Min.)	51-Year Record	March 1 Rain (Inch)	28-Year Record	March 1 Rain (Inch)
30	1.06 (1941)	1.12	1.11 (1978)	1.47
60	1.45 (1941)	1.72	1.28 (1978)	1.93
120	1.76 (1974)	2.25	1.58 (1978)	2.26
180	2.07 (1974)	2.65	1.65 (1978)	2.98
360	2.93 (1941)	4.00	1.86 (1979)	3.82

Figures I-1 and I-2 illustrate the area of most significant rainfall, which in turn, indicate the areas where flood channels were most severely affected. Flood channels which overflowed were largely those which were constructed of earthen levees. It is important to note that

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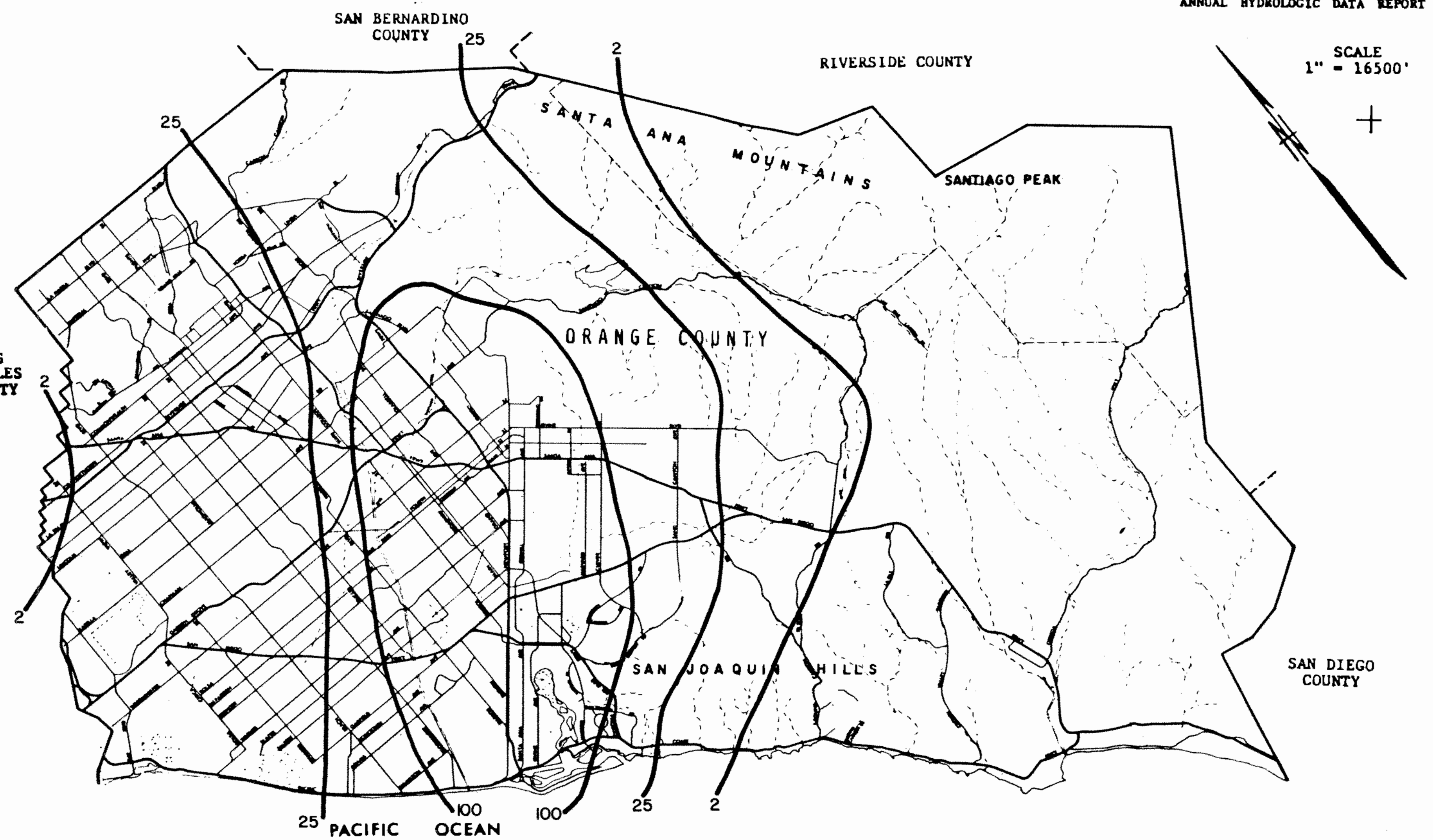


ORANGE COUNTY

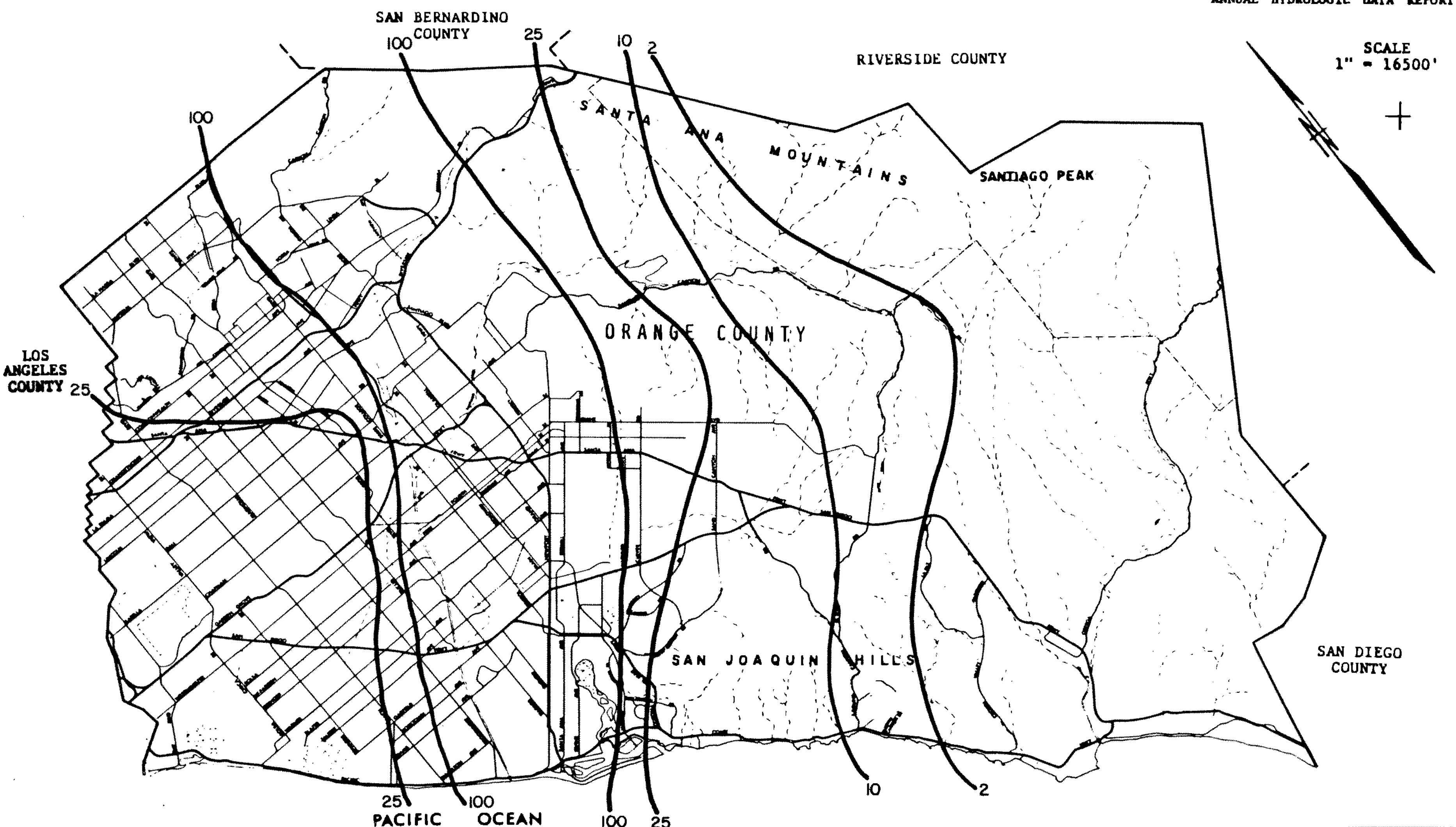
HYDROLOGY MANUAL

ISOFREQUENCY MAP
MAXIMUM 1-HOUR PRECIPITATION
MARCH 1, 1983

Figure I-1

ORANGE COUNTY ENVIRONMENTAL MANAGEMENT AGENCY

ANNUAL HYDROLOGIC DATA REPORT



ORANGE COUNTY

HYDROLOGY MANUAL

ISOFREQUENCY MAP
MAXIMUM 3-HOUR PRECIPITATION
MARCH 1, 1983

Figure I-2

the rain depths in Santa Ana and Costa Mesa for durations of 30 minutes through 6 hours are all approximately one hundred year values according to the State Department of Water Resources depth-duration frequency tables for Orange County. Figures I-1 and I-2 show that large areas were blanketed by one hundred year recurrence interval rain depths. In this regard the March 1 event is similar to the storm of December 4, 1974, in Orange County where comparably large areas were blanketed by 100-year rain depths (Fig. I-3) and rain depths were also greater near the coast than in the eastern mountains. Flooding resulting from the 1974 event was lessened by the mild antecedent moisture conditions. The 1974 event was the first severe storm of the season. The March 1, 1983, event followed five days of light to moderate rainfall. Such antecedent rainfall fills depression storage in the watershed and reduces soil infiltration to the saturated rate.

The March 1, 1983 storm is important in County of Orange Storm Water Management (SWM) policy due to the magnitude of rainfall intensities during the peak six hours being comparable with the design storm pattern used in this hydrology manual, and also due to the eight days of antecedent rainfall.

From the above list it is evident that seven storms in the last 161 years (every 23 years on the average) have produced widespread flooding in Orange County and only two of the storms are adequately represented in the rainfall data used in this manual (1974 and 1983).

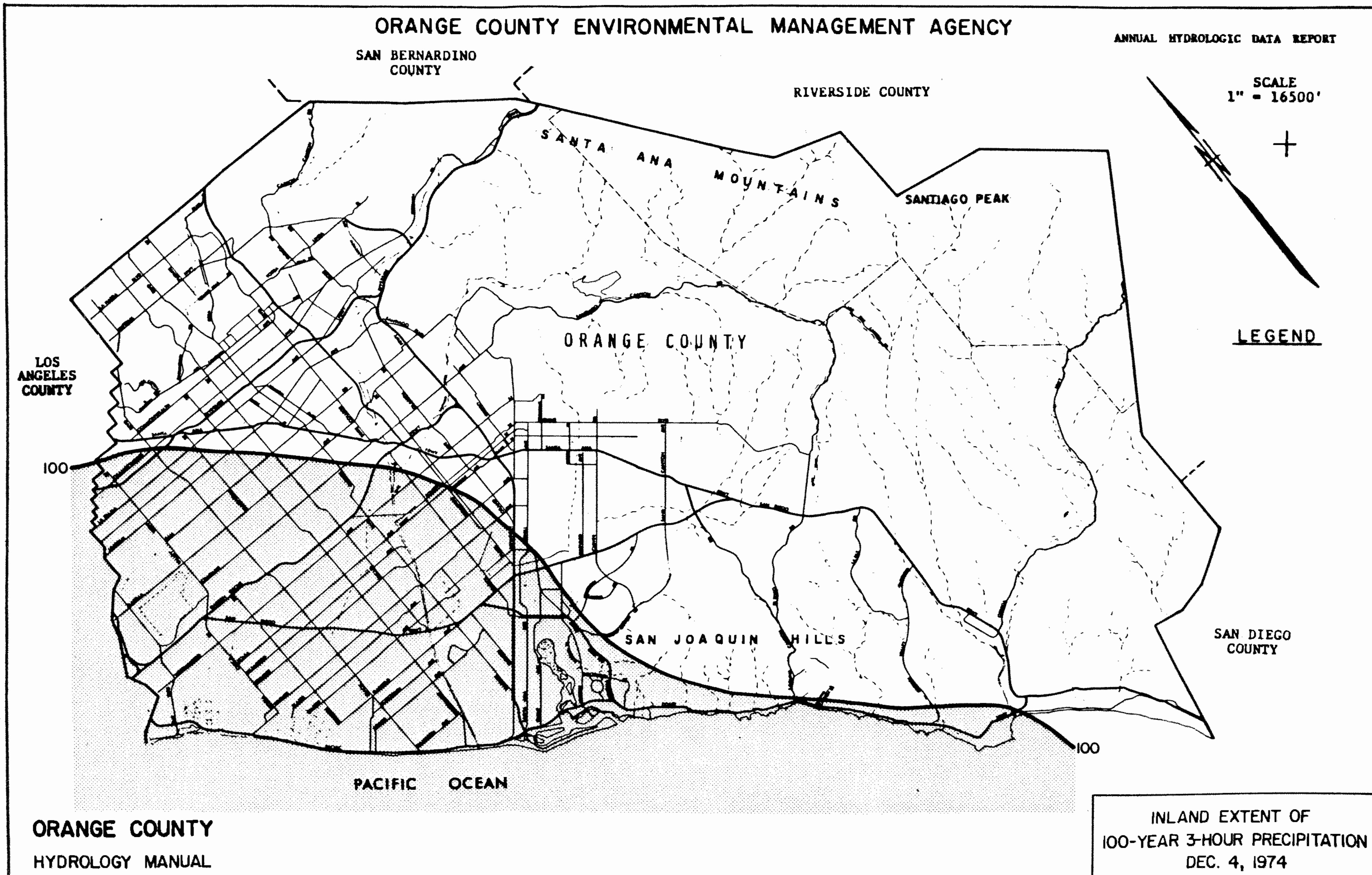

ORANGE COUNTY ENVIRONMENTAL MANAGEMENT AGENCY
ANNUAL HYDROLOGIC DATA REPORT
SAN BERNARDINO COUNTY
RIVERSIDE COUNTY
SCALE 1" = 16500'
LOS ANGELES COUNTY
SANTA ANA MOUNTAINS
SANTIAGO PEAK
ORANGE COUNTY
LEGEND
SAN JOAQUIN HILLS
SAN DIEGO COUNTY
100
100
PACIFIC OCEAN
ORANGE COUNTY
HYDROLOGY MANUAL
INLAND EXTENT OF
100-YEAR 3-HOUR PRECIPITATION
DEC. 4, 1974

Figure I-3

APPENDIX II

II.1. MEASUREMENT AND SYNTHESIS OF PRECIPITATION DATA

Of interest for hydrologic studies is the maximum intensities of precipitation possible throughout a watershed. Given a long history of such maximum rainfall intensities for various durations of time, a reasonable statistical interpretation can be made of the data to determine estimates of maximum rainfall intensities or depths as a function of storm duration and of return frequency. The Orange County Flood Control District maintains and operates both automatic recording and standard (manual) rain-gages throughout the county and summarizes the data in its annual Hydrologic Data Report. Other sources of precipitation data include the U. S. Weather Bureau, U. S. Army Corps of Engineers, U. S. Geological Survey and other private and governmental cooperative weather observers.

For each automatic recording rainfall gage, the precipitation records are analyzed to determine the annual maximum rainfall depth for several durations of interest (e.g., 5-minutes, 10-minutes, 15-minutes, etc.). This data can then be arranged in an increasing order of magnitude for each storm duration for the history of the rain-gage, and plotted on normal probability paper. From this accumulation of rainfall depth-duration data, various statistical models can be applied to assign a return frequency (or period) to the known data values and to estimate maximum rainfall depth-duration values for typically unmeasured higher return frequencies (e.g., the 100-year return frequency). The resulting data for each rain gage is generally termed "point precipitation" values to distinguish them from average values for large areas.

Because storm events seldom locate their peak intensities over rain-gages, and because the rain-gage network is widely distributed (allowing small intense rainfall events to miss the gage network), and because of mechanical defects of the gage devices and wind effects, the rainfall data can generally be assumed to underestimate the true maximum point rainfall intensities.

II.2. SYNTHETIC 24-HOUR CRITICAL STORM PATTERN

The United States Department of Agriculture Soil Conservation Service (SCS) developed dimensionless critical storm patterns using the U.S. National Weather Service's (NWS) rainfall frequency atlases (ref. 9). The rainfall frequency data for areas less than 400 square miles, for durations to 24 hours, and for frequencies from 1 to 100 years were used.

These critical storm patterns are based on the generalized precipitation depth-duration-frequency relationships shown in technical publications of the NWS and precipitation depths for durations from 1 minute to 24 hours were used to derive the storm patterns. Using increments of 30-minutes, incremental precipitation depths were determined. For example, the 30-minute depth was subtracted from the 1-hour depth and the 1-hour depth was subtracted from the 1.5-hour depth. The storm patterns were formed by arranging these 30-minute incremental depths such that the maximum 30-minute depth is contained within the maximum 1-hour depth, and the maximum 1-hour depth is contained within the maximum 1.5-hour depth and so forth. Because all of the critical precipitation depths are contained within the storm pattern, the critical storm patterns may be assumed appropriate for designs on both small and large watersheds (ref. 9).

The Agency's design storm pattern is based upon a modification of the SCS 24-hour storm pattern. The design storm pattern provides a representation of local precipitation depth-duration-frequency tendencies by constructing the several nested intervals to fit local recorded rainfall data. Additionally, the SCS storm pattern is further modified to include the necessary adjustments (reduction in shorter duration point precipitation values) due to watershed areal effects. The procedures used to construct the 24-hour storm pattern and determine the associated rainfall depths (adjusted for depth-area) follow the U.S. Corps. of Engineers methods as published in the HEC Training Document No. 15 (ref. 3). Details of the 24-hour storm pattern and the necessary adjustments for depth-area effects (indexing) are contained in Section E.

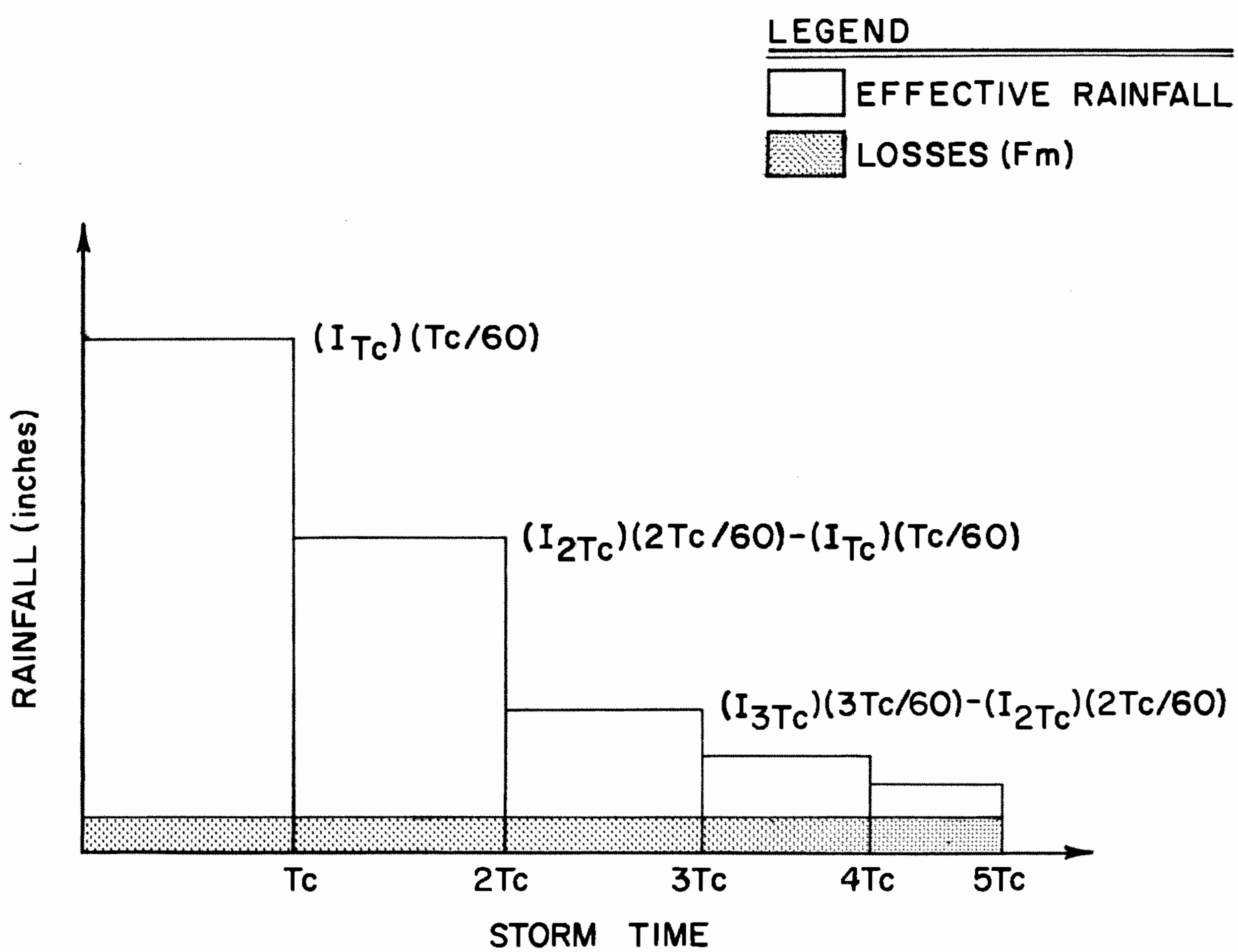
APPENDIX III

THE RATIONAL METHOD AS A DESIGN STORM UNIT HYDROGRAPH METHOD

The rational method can be interpreted as a design storm unit hydrograph method. The design storm pattern is developed by using a selected return frequency rainfall intensity - duration curve. At a point of concentration with time of concentration, Tc, the rational method design storm pattern is constructed from an intensity duration curve by first determining the total amount of rainfall (i.e., unit rainfalls) which falls in several successive unit periods, each of duration Tc. The next step is to arrange these several unit rainfalls into the rational method design storm pattern (see Figure III-1) by placing the largest unit rainfall as the first unit, followed by the second largest unit rainfall, and so forth until a sufficiently long design storm pattern is developed (usually about 1-hour in total length, but may be longer depending on the various stream confluence Tc values).

Using the area-averaged loss rate F_m (e.g., see Table D.1), the design storm unit effective rainfalls are calculated by subtracting the appropriate proportion of F_m from each unit rainfall. It is noted that the design storm unit rainfalls are given in units of inches of precipitation whereas F_m is given as a rate (inch/hour).

The unit hydrograph corresponding to the rational method is a triangle with base 2Tc, and a peak occurring at time Tc (see Figure III-2). For a unit period of duration equal to Tc and a unit effective rainfall of 1 inch, the associated unit period runoff hydrograph must have a peak flow rate of (60/Tc) cfs per acre where Tc is given in minutes. Similarly, a unit period effective rainfall of only 1/2-inch must have an associated unit period runoff hydrograph with a base of 2Tc and a peak flow rate of (1/2)(60/Tc) cfs per acre. The runoff hydrographs associated to each unit effective rainfall are determined similarly, and then arranged as shown in Figure III-3 so that the resulting unit period runoff hydrographs correspond in timing to the proper unit period effective rainfalls. The runoff hydrograph is developed by adding the flow contributions from the several unit period runoff hydrographs.



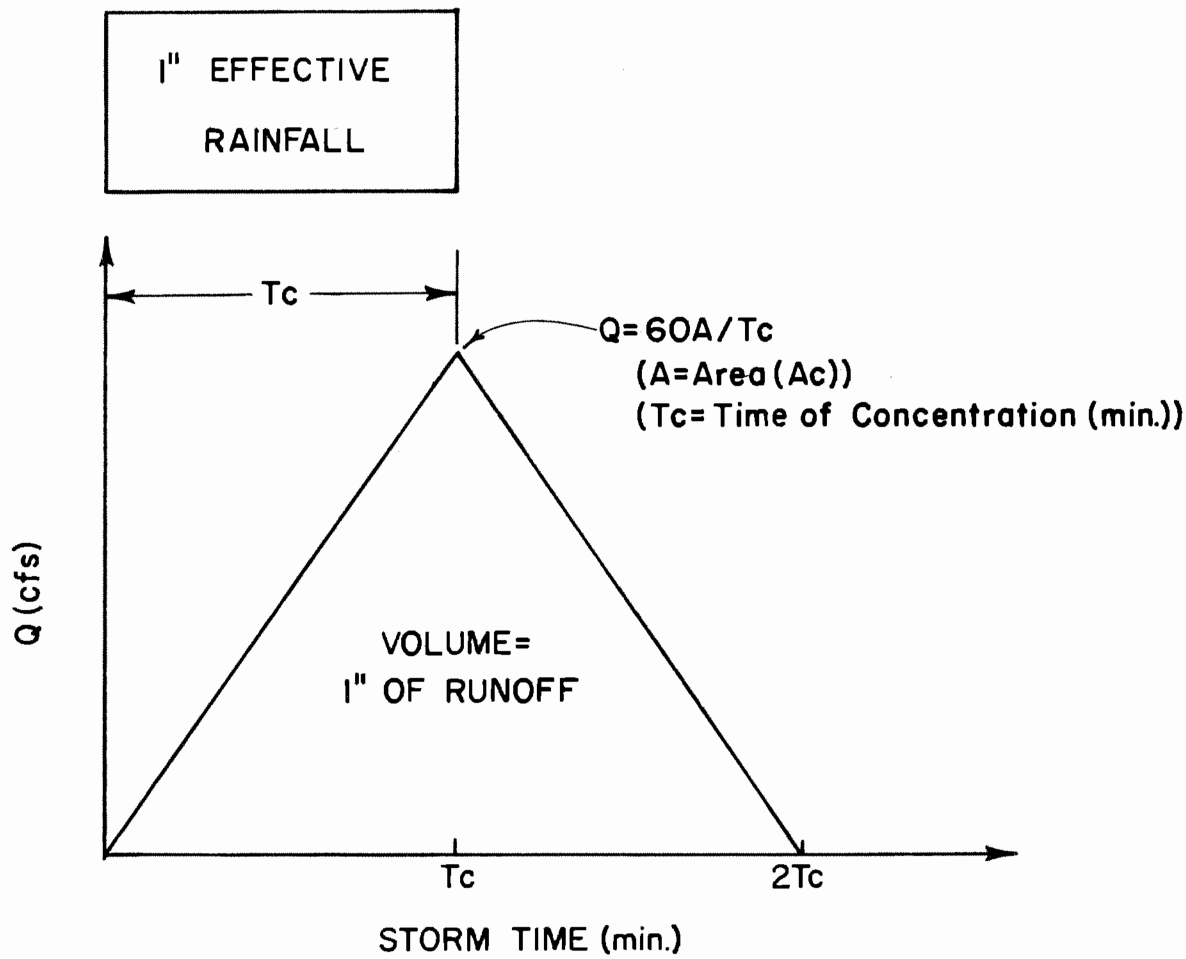
ORANGE COUNTY

HYDROLOGY MANUAL

RATIONAL METHOD

DESIGN STORM PATTERN

Figure III-1



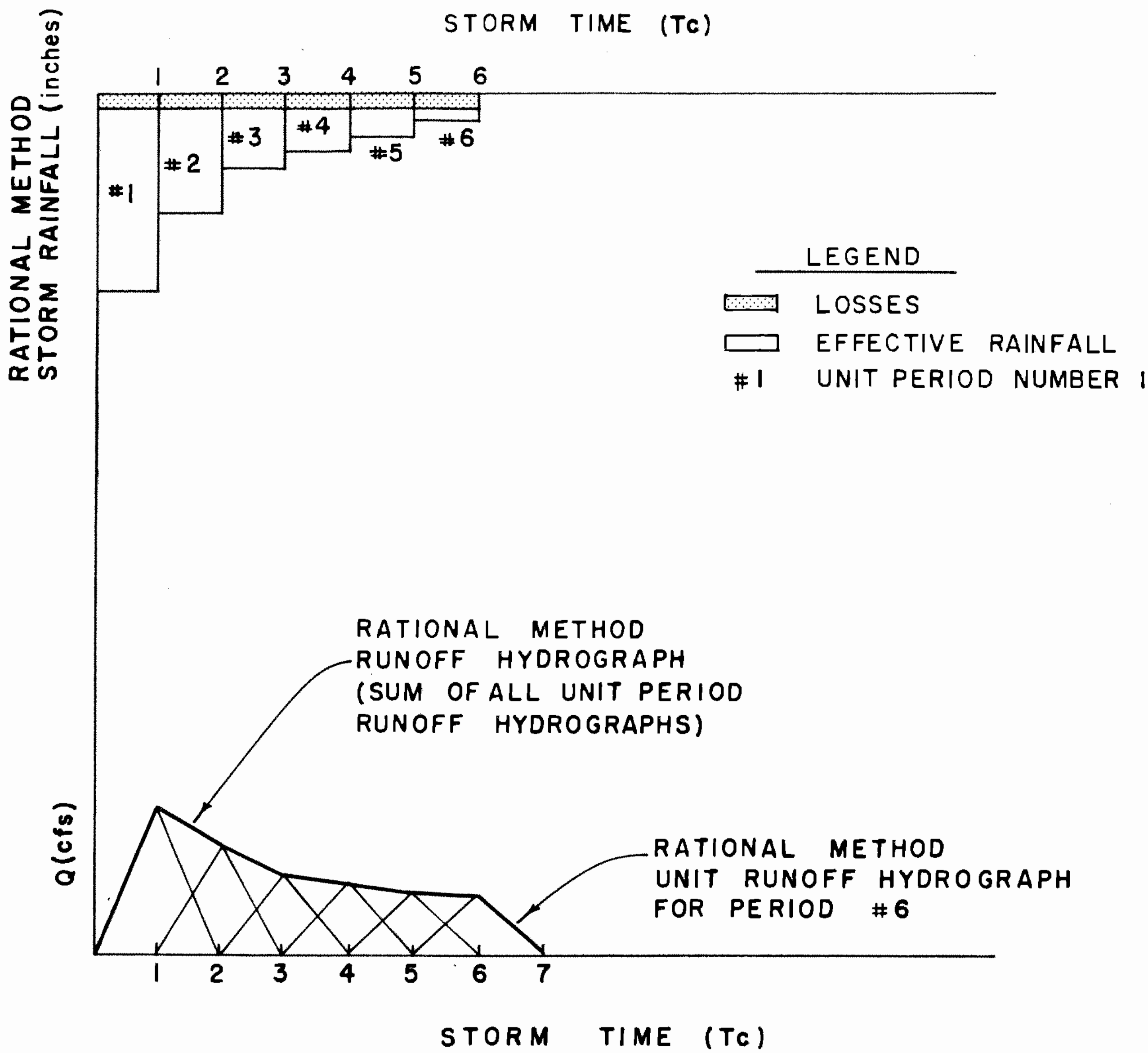
ORANGE COUNTY

HYDROLOGY MANUAL

RATIONAL METHOD

UNIT HYDROGRAPH

Figure III-2



ORANGE COUNTY

HYDROLOGY MANUAL

RATIONAL METHOD

RUNOFF HYDROGRAPH DEVELOPMENT

Figure III-3

APPENDIX IV

DETENTION BASIN CONSIDERATIONS

Generally, the main purpose for inclusion of a stormwater detention basin in a flood control system is to reduce peak rates of runoff generated from an upstream watershed and to control peak flows into downstream areas. Some of the advantages and disadvantages of use of detention basins are listed in the following:

BENEFITS

- Reduce peak rates of runoff to downstream areas.
- Basin reduces transport of sediments carried in floodwaters.
- Reduces size of downstream flood control facilities.
- Provides location for groundwater recharge if aquifer contact exists.
- Provides location to concentrate floodwaters for contaminant treatments.

POTENTIAL CONCERNS

- Detention basins do not reduce total storm runoff volume (unless the groundwater recharge potential is large).
- Maintenance of storage capacity, inflow and outflow facilities is critical.
- Basins increase the duration of flows which may increase erosion effects downstream from the basin. Downstream erosion may be further increased due to sediment extraction in the basin.
- Improperly sized and placed basins may aggravate rather than reduce downstream flooding potential (especially in large complex systems).
- Accumulated debris from runoff decreases flood control storage volume in a detention basin.
- Cost of debris removal.
- Detention basins in urban areas may become unsightly and/or vermin infested without intensive maintenance.

The consideration of a detention basin system needs to address the various hydrologic, hydraulic, environmental and flood control concerns listed above, as well as any other concern which may arise during the course of the project study, and determine the necessary mitigative measures which are acceptable to the Agency. Of special concern is the interplay between the several components of the total system network. Unplanned placement of detention basins without consideration of other watershed detention basins and tributary watersheds can increase the downstream peak flow rate above the anticipated runoff peak flow rate attained without any detention basins in the watershed.

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