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Hydrologic Design Manual

for Maricopa County, Arizona

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Special Projects Branch
Hydrology Division
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Hydrologic Design Manual for Maricopa County, Arizona

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Executive Summary

The objective of the *Hydrologic Design Manual of Maricopa County* is to provide technical procedures for the estimation of flood hydrology for the purpose of designing stormwater drainage facilities in Maricopa County. Two methodologies are defined for the development of flood hydrology; the Rational Method, and rainfall-runoff modeling using a design storm. For small, urban watersheds, less than 160 acres and fairly uniform land-use, the Rational Method is acceptable. Use of this method will only produce peak discharges and runoff volume and this method should not be used if a complete runoff hydrograph is needed, such as for routing through detention facilities. For larger, more complex watersheds or drainage networks, a rainfall-runoff model should be developed. The *Hydrologic Design Manual of Maricopa County* provides guidance in the development of such a model and the estimation of the necessary input parameters to the model. Although not necessarily required, the use of the U.S. Army Corps of Engineers' HEC-1 Flood Hydrology Program facilitates the use of the procedures that are contained in the *Hydrologic Design Manual of Maricopa County*. (The *Hydrologic Design Manual* was written to supplement the *HEC-1 Users Manual*.)

The *Hydrologic Design Manual* can be used to develop design hydrology magnitudes for storms of frequencies up to and including the 100-year event. The design storm is of 6-hour duration and that storm is to be used for the design of all stormwater drainage facilities except detention and retention basins. According to the *Uniform Drainage Policies and Standards for Maricopa County, Arizona* (February 25, 1987), all development shall make provisions to retain the peak flow and volume of runoff from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development. Accordingly, the criteria to be applied to the 2-hour storm is also provided in the *Hydrologic Design Manual*.

The rainfall-runoff modeling procedure that is contained in the manual is physically based, that is, the procedures are based—to the extent practical—on the physical processes that occur during the generation of storm runoff from rainfall. While the basic procedure is physically based this does not assure that the rigorous application of the procedures will, in fact, reproduce the actual rainfall-runoff phenomenon of any storm that has occurred or may occur in the future. However, the procedure,

when applied with good hydrologic judgement, should yield consistent results for design purposes.

Throughout the development of the *Hydrologic Design Manual* three benchmarks were continually applied in judging the applicability of individual procedures and the overall methodologies; accuracy, practicality, and reproducibility. *Accuracy* is a measure of how well the results of the procedure reproduce the physical process being simulated. Although accuracy is highly desired, it is theoretically impossible to achieve in an earth science such as hydrology, and in a practical sense, accuracy is not feasible to assess except for a few situations where adequate verification data are available. Relative accuracy was assessed throughout the development of the procedures in the manual through testing and verification against recorded data.

Practicality is a users decision regarding the best and most appropriate level of technology to apply considering the information that is available: anticipated user, consequences of error, and desired or required output. Whereas both simpler procedures and more sophisticated procedures are available, the adopted methodologies provide a compromise between these two extremes, and the best practical level of technology is judged to be recommended in the manual considering the state of current hydrologic knowledge of arid and semi-arid lands.

Reproducibility is a characteristic that provides a reasonable assurance that consistent results will be achieved by all qualified users. Reproducibility is highly desirable for a design standard in order to eliminate—to the extent possible—unnecessary conflicts over the interpretation and application of the design method. Reproducibility is achieved through clear and concise manual procedures and user guidance. Every effort has been made toward this end.

A brief discussion of the contents of each chapter of the *Hydrologic Design Manual* follows:

Chapter 1, *Introduction*: The introduction states the purpose, scope and limitations, and general use of the manual.

Chapter 2, *Rainfall*: The characteristics of severe storms in Maricopa County are documented as a setting for defining the design rainfall criteria. Procedures and information are provided for the determination of depth-duration-frequency statistics of storms in Maricopa County. These are derived from NOAA Atlas 2, Arizona, which is the most comprehensive and authoritative source of such information. The limitations and potential inaccuracy of the NOAA Atlas is recognized and until an equivalently accepted source of rainfall statistics is provided, this source must be used. Recent reanalysis of the short duration (less than 1-hour) rainfalls by the National Oceanographic and Atmospheric Administration have been used as a supplement to the NOAA Atlas.

The temporal distribution of rainfall for the majority of design conditions is a 6-hour local storm. The 6-hour storm distribution is based on an analysis by the U.S. Army Corps of Engineers, Los Angeles District, of the August 19, 1954 Queen Creek storm. The Corps' distribution has been modified somewhat to reflect the design rainfall criteria that is desired for use in Maricopa County,

and this modification includes using the hypothetical distribution for drainage areas less than 0.5 square mile. The temporal distribution is a function of drainage area and this is to reflect the spatial variability of rainfall intensities that are known to exist with severe local storms in Maricopa County. A 2-hour distribution is provided for use in the design of detention/retention facilities. The reduction of rainfall depth with storm area for the 6-hour rainfall is accounted for by a depth-area reduction curve based on the 1954 Queen Creek storm.

Chapter 3, *Rational Method*: Use of the Rational Method is to be limited to areas of up to 160 areas, and is generally limited to urbanized conditions. The watershed should be of uniform land use for application of this method. Intensity-duration-frequency (i-d-f) statistics are to be obtained from the information contained in Chapter 2, and an i-d-f curve for general use is contained in the manual. An equation for the estimation of time of concentration is provided which is a partial function of rainfall intensity. Values of the runoff coefficient "C" to be applied to various land uses in Maricopa County are provided.

Chapter 4, *Rainfall Losses*: Several procedures are provided for the estimation equation of rainfall losses. The preferred method is the Green and Ampt infiltration equation with an estimate of surface retention loss. This requires the classification of soil according to soil texture, which is available for most of Maricopa County. Adjustment of the loss rate is available as a function of vegetation cover. Other methods are available to estimate rainfall losses if adequate soils and/or vegetation data are not available. One alternative method is to use the Initial Loss plus Uniform Loss Rate (IL+ULR) method, if only the hydrologic soil groups can be estimated.

Chapter 5, *Unit Hydrograph Procedures*: The use of unit hydrographs to route rainfall excess from the land's surface is recommended and the procedures recommended to do so are either the Clark unit hydrograph or the application of selected S-graphs. The Clark unit hydrograph is recommended for watersheds or subbasins less than five square miles in size with an upper limit of application of ten square miles. Procedures are provided for the estimation of the two numeric parameters: time of concentration and storage coefficient. Two default time-area relations are provided; one for urban watershed and the other for natural watersheds. Two regional S-graphs have been selected for use in flood hydrology studies of major watercourses in Maricopa County. The Phoenix Mountain S-graph is to be used in studies of watersheds that drain predominantly mountainous terrain. The Phoenix Valley S-graph is to be used in studies of watersheds that have little topographic relief or urbanized watersheds. A procedure is provided for the estimation of the S-graph parameter, lag.

Chapter 6, *Channel Routing*: General guidance is provided for the use of Kinematic Wave routing and Muskingum routing. Kinematic Wave routing can be applied to urbanized or artificial channels and closed conduits. Muskingum routing is to be used for natural channels.

Chapter 7, *Application*: General guidelines and some specific aids in the use of the manual are provided in this chapter.

Appendices: Loss rate tables for soils in Maricopa County, Textural Class Diagram, and blank figures and worksheets are provided in the appendices.

Examples: Detailed examples are provided that clearly illustrate the use of the procedures in practical applications.

Acknowledgments

The information, procedures, and recommendations that are presented in this manual are mainly the result of previously published efforts of many diligent and talented engineers and scientists. The authors of this manual have made every effort to cite the original authors and researchers whose contributions to this manual, and to the science of hydrology, are gratefully appreciated.

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1 Introduction

1.1 Purpose

In April 1985 a task force was formed by the Flood Control District of Maricopa County to establish a common basis for drainage management in all jurisdictions within Maricopa County. Among the goals of the task force were provisions for consistent analysis of drainage requirements, reducing costs and staff time when annexing County areas, and supplying equal and common protection from the hazards of stormwater drainage for all County residents. Additionally, developers would be benefitted by having only one set of drainage standards with which to comply when developing land within the incorporated or unincorporated areas of Maricopa County. The task force determined that these efforts would be achieved in three phases:

Phase 1 Research, evaluate, develop, and produce uniform policies and standards for drainage of new development within Maricopa County (Resolution FCD 87-7).

Phase 2 Establish a *Stormwater Drainage Design Manual* for use by all jurisdictional agencies within the County.

Phase 3 Prepare an in-depth evaluation of regional rainfall data and establish precipitation design rainfall guidelines and isohyetal maps for Maricopa County.

As a part of Phase 2, the *Hydrologic Design Manual of Maricopa County* will provide the necessary data for the *Stormwater Drainage Design Manual.*

1.2 Scope and Limitation

When using the procedures detailed in this manual, it is important to keep several things in mind. First, this is a *hydrologic design manual.* The methods, techniques and parameter values described herein are *not necessarily valid* for real-time prediction of flow values, nor for recreating historic events—although some of the methods are physically based and would be amenable for uses other than design hydrology.

Second, the lack of runoff data for urbanizing areas of the County, for the most part, precludes the use of flood frequency analysis for stormwater drainage design. For those watercourses with sufficient record, flood frequency analysis may be acceptable. Similarly, for those watercourses with established regulatory floodplains, the FEMA accepted flood frequency curves may be used for design purposes, unless they are demonstrably inappropriate. The purpose of this manual is to provide a means of assisting in the prediction of runoff which might result from a design storm of a given return interval.

Third, the design storm has no point of reference in terms of a singular historic event. Rather, it is intended to provide the best available information by utilizing historic data as well as other precipitation design concepts. The design storm provides not only the peak intensities which would be expected from a storm of a given duration and return interval, but also the volumes associated with it. The tables describing the temporal distribution of the design storm for use in a hydrologic model, i.e., HEC-1, are approximately equivalent to the graphs used to determine the rainfall intensity to be used in the Rational Method. The net effect is that regardless of the size of the area being investigated or the method of analysis, the same design storm is used as the driving input.

1.3 Using this Manual

The use of the methods presented in this manual, even the rigorous application thereof, in no way ensures that the predicted values are reasonable or correct. Hydrology is a discipline which, in some respects, is much like music—quality requires not only technical competence but also a *feel* for what is right. It often requires the exercise of *hydrologic judgement.* The Flood Control District of Maricopa County does not warrant or guarantee the reliability of the hydrologic methods, techniques, and/or parameter values set forth in this design manual. The user of the *Hydrologic Design Manual* has no right to rely or depend on the methodology, techniques, and/or parameter values described herein. The user of this manual is thus directed to validate the reasonableness of the predicted values by applying alternative methods, such as envelope curves, regression equations, or other checks which have been developed for this area. Failure to do so may result in erroneous values.

Section 7 of this manual is intended to provide some general suggestions for the user attempting to solve a particular problem. A number of examples were designed to aid the user with the development of input variables and parameter estimation.

It is not the intent nor purpose of this manual to inhibit sound innovative design or the use of new techniques.* It is anticipated that, over time, as more data becomes available and/or more appropriate techniques are developed, this manual will be revised. With the exception of minor editorial corrections, such revisions will probably take place every three to five years. If, in the intervening period, gross inadequacies/inaccuracies are found with any of these procedures, they should be brought to the attention of the Flood Control District of Maricopa County, or any other agency that might subscribe to these suggested procedures.

* Therefore, where special conditions or needs exist, other methods and procedures may be used with prior approval.
italics

2 Rainfall

2.1 General

Precipitation in Maricopa County is strongly influenced by variation in climate, changing from a warm and arid desert environment to a cool and moderately humid mountainous area. Mean annual precipitation ranges from about 7 inches in the Phoenix vicinity to more than ~~30~~ 25 inches in the mountain regions of northern Maricopa County. Precipitation is typically divided into two seasons, summer season (June through October) and winter (December through March), and these seasonal rainfall depths are about equal. The storm patterns are generally categorized into three types, though any combination of the storm types is possible. Warm, moist tropical air can move into Arizona at anytime of the year, but most often does so in the summer months.

2.1.1 General Winter Storms

This type of storm normally moves in from the north Pacific Ocean, and produces light to moderate precipitation over relatively large areas. These storms occur between late October and May, producing the heaviest precipitation from December to early March. A storm could last over several days with slight breaks between individual storms. Because of orographic effects, the mountain areas generally receive more precipitation than the lower desert areas. These storms are characterized by low intensity, long duration, and large areal extent, but on occasion, with an additional surge of moisture from the southwest, can contribute to substantial runoff volumes and peak discharge on major river systems.

2.1.2 General Summer Storms

The Pacific Ocean north of the equator and south of Mexico is a breeding ground for tropical storms. On the average, about two dozen tropical storms and hurricanes are generated in this area from June through early October. Most move in a northwesterly direction. The remnants of these storms can be caught up in the large scale circulation around a low pressure center in southern California and therefore can bring a persistent flow of moist tropical air into Arizona. The storm pattern consists of a band of locally heavy rain cells within a larger area of light to moderate

rainfall. Whereas general winter storms can cover much of the state, general summer storms are more localized along a southeast to northwest band of rainfall. They are similar to winter storms in that higher elevations receive greater rainfall because of orographic influences. The period of late September through October may have storm patterns which are similar to both general summer and winter events.

2.1.3 Local Storms

These storms consist of scattered heavy downpours of rain over areas of up to about 500 square miles for a time period of up to 6 hours. Within the storm area, exceptionally heavy rains usually cover up to 20 square miles and often last for less than 60 minutes. They are typically associated with lightning and thunder, and are referred to as thunderstorms or cloudbursts. While they can occur any time during the year, they are more frequent during summer months (July to September) when tropical moisture pushes into the area from the southeast or southwest. These storms turn into longer duration events in late summer and may be associated with general summer storms (see above). Local storms generally produce record peaks for small watersheds. They can result in flash floods, and sometimes loss of life and property damage.

2.2 Depth–Duration–Frequency Analysis

The commonly required precipitation parameters used in hydrologic modeling are depth, intensity, duration, spatial distribution and frequency of rainfall. The selection of a design frequency is often influenced by administrative or economic decisions as well as hydrologic ones. The duration of the design storm is usually a function of the size and topography of the watershed. In general, one should insure that the design storm is of sufficient duration to allow the entire watershed to contribute to the flow at the point of interest.

Design rainfall in Maricopa County is difficult to develop because of the spatial and temporal variation of precipitation and lack of long-term rainfall data. Regardless of whether the desired output is a peak discharge for sizing a conveyance structure, or a volume for sizing a basin, or the overland flow from a natural watershed, the designer needs to know the total depth of the design precipitation event and how it is structured both in time and space. However, selection of the appropriate criteria is constrained by availability and quality of data.

2.2.1 Source of Data

The most comprehensive, available source of data for depth-duration-frequency analysis is the *Precipitation-Frequency Atlas for Arizona,* published by the National Weather Service (NWS), National Oceanic and Atmospheric Administration (NOAA), (Miller, et al., 1973). Until a more up-to-date data base becomes available, the NOAA Atlas is to be used for all drainage design purposes within Maricopa County.

2.3 Depth–Area Relation

The problem of spatial variability of rainfall is quite difficult to handle because of an irregular, limited network of rain gauges. Work in the southwest by the United States Department of Agriculture, Agricultural Research Service, indicates that high intensity storms do not have large areal extent. Most runoff producing thunderstorms south of Tombstone cover less than twenty square miles.

The above argument supports development of areal reduction curves which reflect the nature of the thunderstorms in the southwest. However, drainage facilities (storm drains, channels, and culverts) should be sized to handle the peak discharge resulting from the design storm critically centered above them to create the worst case discharge. Retention/detention facilities serving as an outfall for a small contributing area of up to 10 square miles would not appear to justify areal reduction of the depth. In all other applications, areal reduction seems appropriate for runoff calculations of contributing areas of any size.

2.3.1 Procedure for Depth-Area Adjustments

Use the Depth-Area Reduction Curve developed for the historic storm of 1954 over the Queen Creek area (U.S. Army Corps of Engineers, 1974). This curve was developed for a major peak producing event within Maricopa County and should be representative of local conditions for design purposes.

a. Determine the size of the drainage area, and decide if areal reduction is necessary.

b. Use Section 2.4 to calculate depth for the design frequency.

c. If more than one isoline is shown over the drainage area, calculate average depth.

d. Use Figure 2.1 and Table 2.1 to select the reduction coefficient.

e. Multiply average rainfall by the depth reduction coefficient.

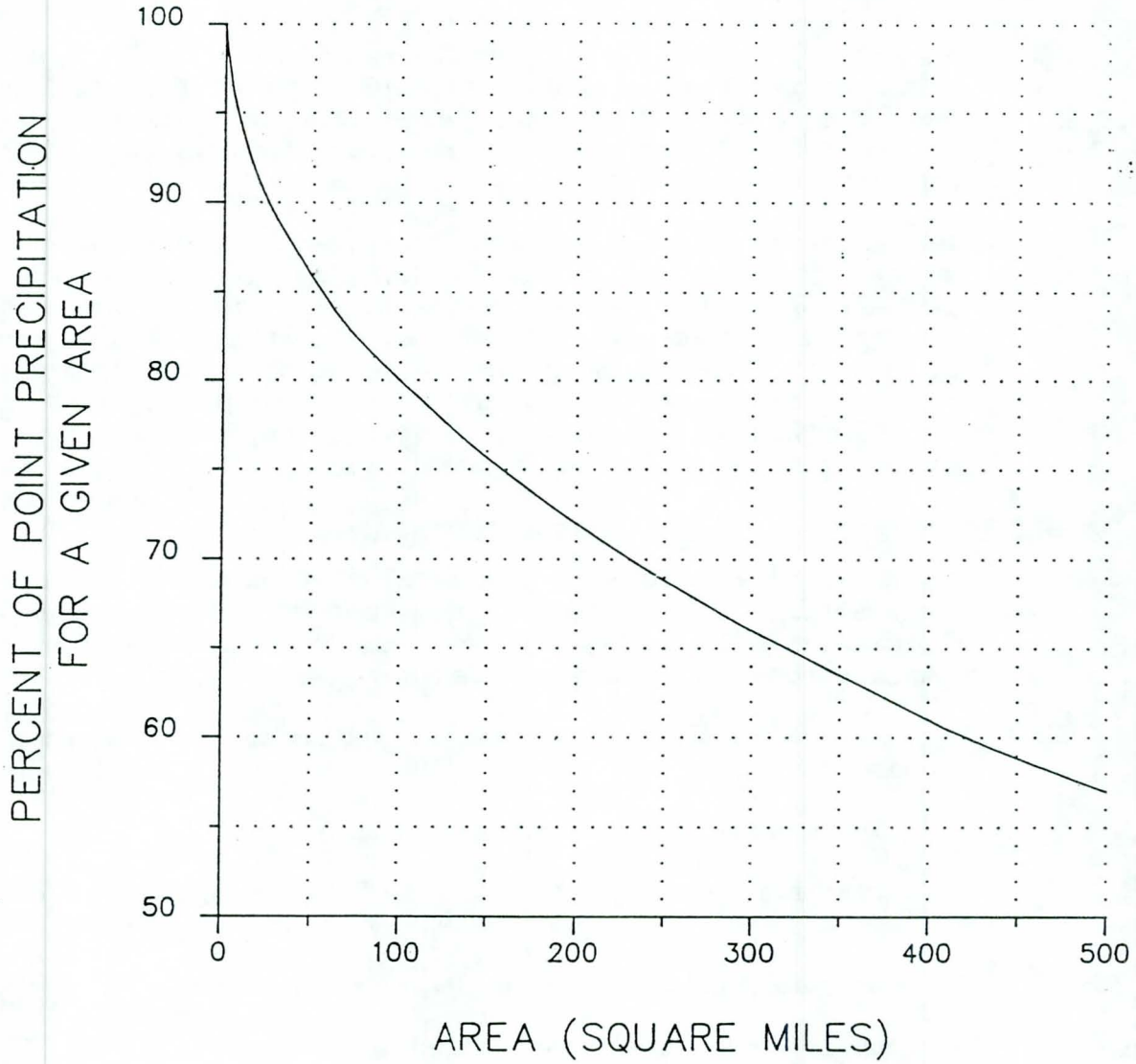


Figure 2.1
Depth-Area Curve for Maricopa County

(To be used only with 6-hour duration rainfall and for all watersheds less than or equal to 100 square miles. Can be used for watersheds greater than 100 square miles, depending on the other site-specific rainfall design criteria that is to be used.)

Table 2.1
Depth-Area Reduction Factors
for 6-Hour Duration Rainfall

Area, Square Miles	Ratio to Point of Rainfall
0	1.0
1	0.987
5	0.96
10	0.94
20	0.91
30	0.89
40	0.87
50	0.86
100	0.80
200	0.72
300	0.66
400	0.61
500	0.57

2.4 Selection of Appropriate Design Storm

The design hydrologist must specify the appropriate rainfall frequency, duration, depth and the corresponding time distribution for any design purposes which require calculation of runoff volume and peak discharge. Application of the Rational ~~Formula~~ Method does not require a time distribution. The Hydrology Manual applies the NOAA procedures which led to the 100-year, 6-hour mass curve for small areas up to 0.5 miles^2 square. This mass curve is also known as Pattern #1, and will be discussed later. If a particular application requires that a specific mass curve other than Pattern #1 should be developed, the following procedures (NOAA) or, alternatively, a program referred to as PREFRE by the ~~National Weather Service~~ U.S. Bureau of Reclamation can be used:

1. Using Figures 2.2 through 2.13, read rainfall depths for 2-, 5-, 10-, 25-, 50, and 100-year return periods for 6- and 24-hour durations, employing linear interpolation between isolines when required. The numbers on the isolines show tenths of inches of rainfall (i.e., 23 = 2.3 inches).

2. Plot the values from 1 for each duration on a separate line on Figure 2.14, look for any deviation from a straight line and make corrections ~~on the line~~ to conform to the straight line values. This process will minimize any error due to transposition of values on the maps. Also, any error due to reading and interpolating values between the isolines will be minimized. Note that these numbers are already in partial-duration series, so there is no need for annual to partial-duration conversion.

3. At this point, the data should include 6-hour and 24-hour durations for all frequencies ~~with the exception of 1-year values.~~ from 2-years to 100-years.

4. A particular design may require a duration different from 6- or 24-hour. For example retention design requires a 100-year frequency, 2-hour duration design storm. In such cases the following procedure (the established method in NOAA, 1973) is used. [Note: The only exception is the use of the values by Arkell and Richards (1986) for durations of less than 1 hour.]

First, the 100-year and the 2-year, 1-hour depths are calculated as follows:

Compute $Y_2 = -0.011 + 0.942(X_1)(X_1/X_2)$

Compute $Y_{100} = 0.494 + 0.755(X_3)(X_3/X_4)$

where:

Y_2 = 2-yr, 1-hr estimated value;

Y_{100} = 100-yr, 1-hour estimated value;

X_1 = 2-yr, 6-hr value from precipitation-frequency maps;

X_2 = 2-yr, 24-hr value from precipitation-frequency maps;

X_3 = 100-yr, 6-hr value from precipitation-frequency maps;

X_4 = 100-yr, 24-hr value from precipitation-frequency maps.

Then the 100-year, 2-hour, and the 2-year, 2-hour depths, as well as depths for other durations are calculated:

Compute 2-hr depth = 0.341 (6-hr) + 0.659 (1-hr)

Compute 3-hr depth = 0.569 (6-hr) +0.431 (1-hr)

Compute 12-hr depth, Figure 2.15, using 6-hr and 24-hr values

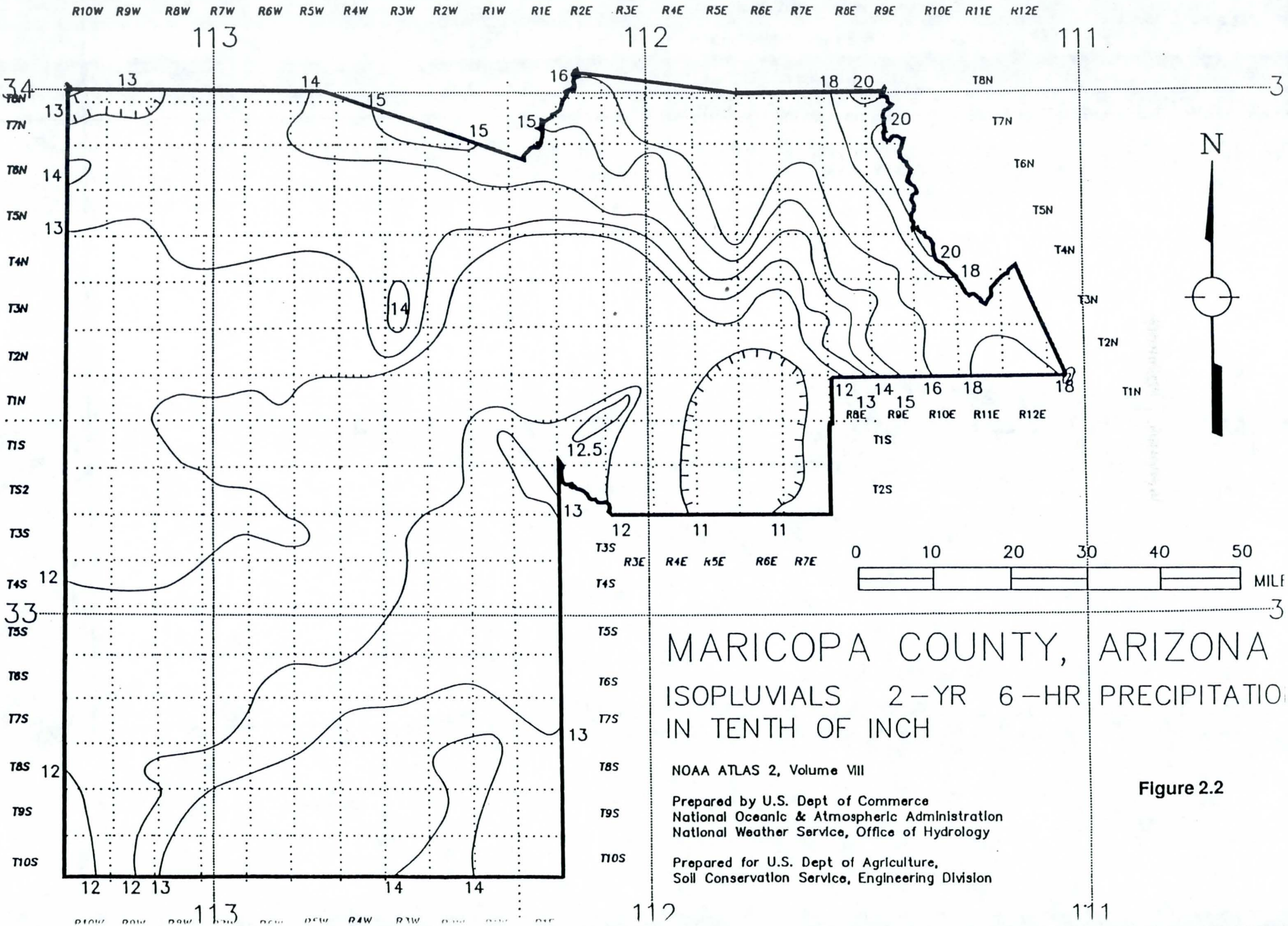
Compute 5-min depth = 0.34(1-hr)

Compute 10-min depth = 0.51(1-hr)

Compute 15-min depth = 0.62(1-hr)

Compute 30-min depth = 0.82(1-hr)

At this point the data includes all depths for the 100-year and the 2-year frequencies, for all durations. Depths for 5-, 10-, 25-, and 50-year frequencies will be estimated by reading the corresponding values from Figure 2.14. A rainfall mass curve can then be constructed by nesting around a desired duration, i.e., 15-min, or 30-min (see example 3).


MARICOPA COUNTY, ARIZONA
ISOPLUVIALS 2-YR 6-HR PRECIPITATIO
IN TENTH OF INCH
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National Oceanic & Atmospheric Administration
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Figure 2.2

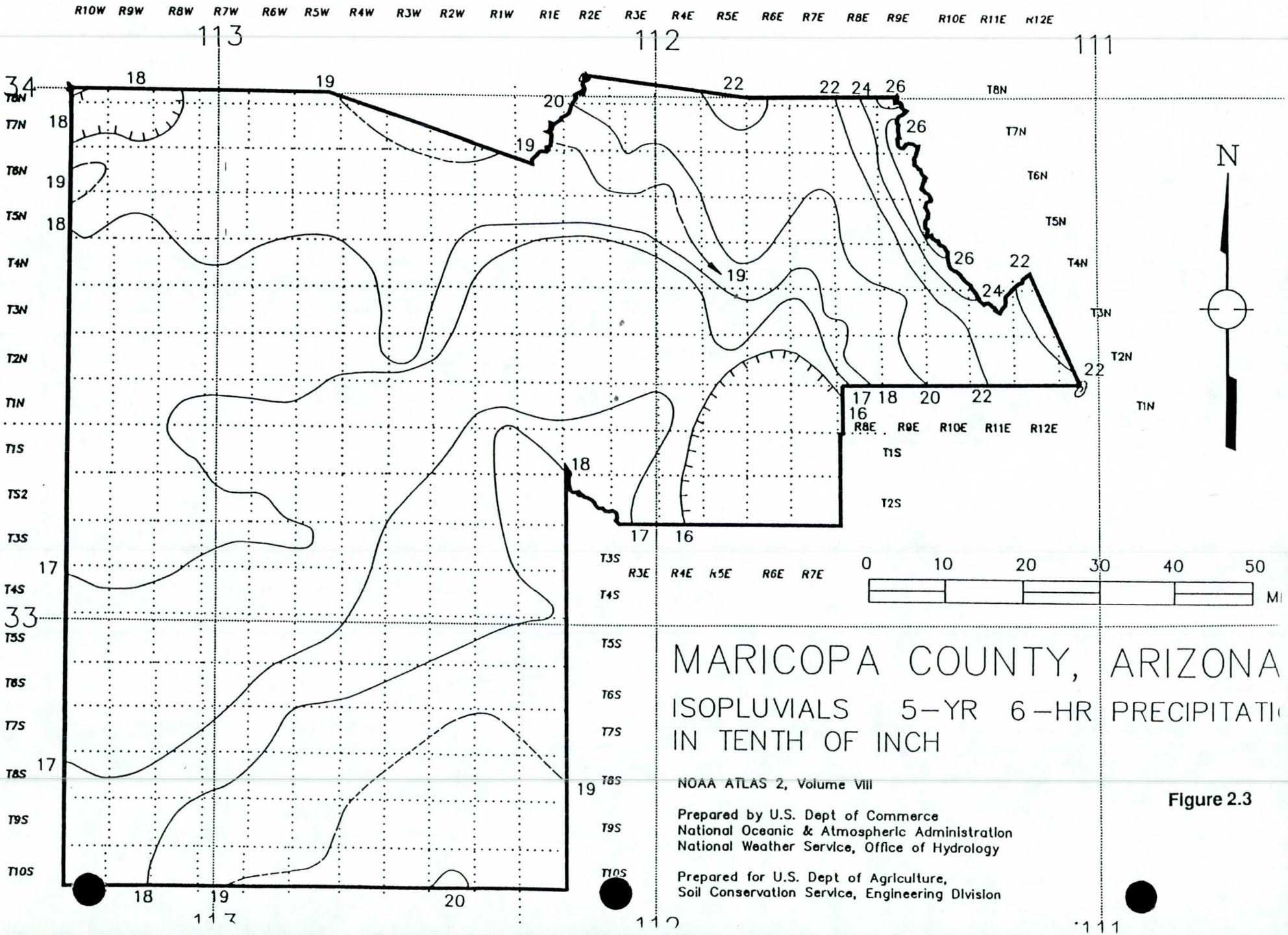
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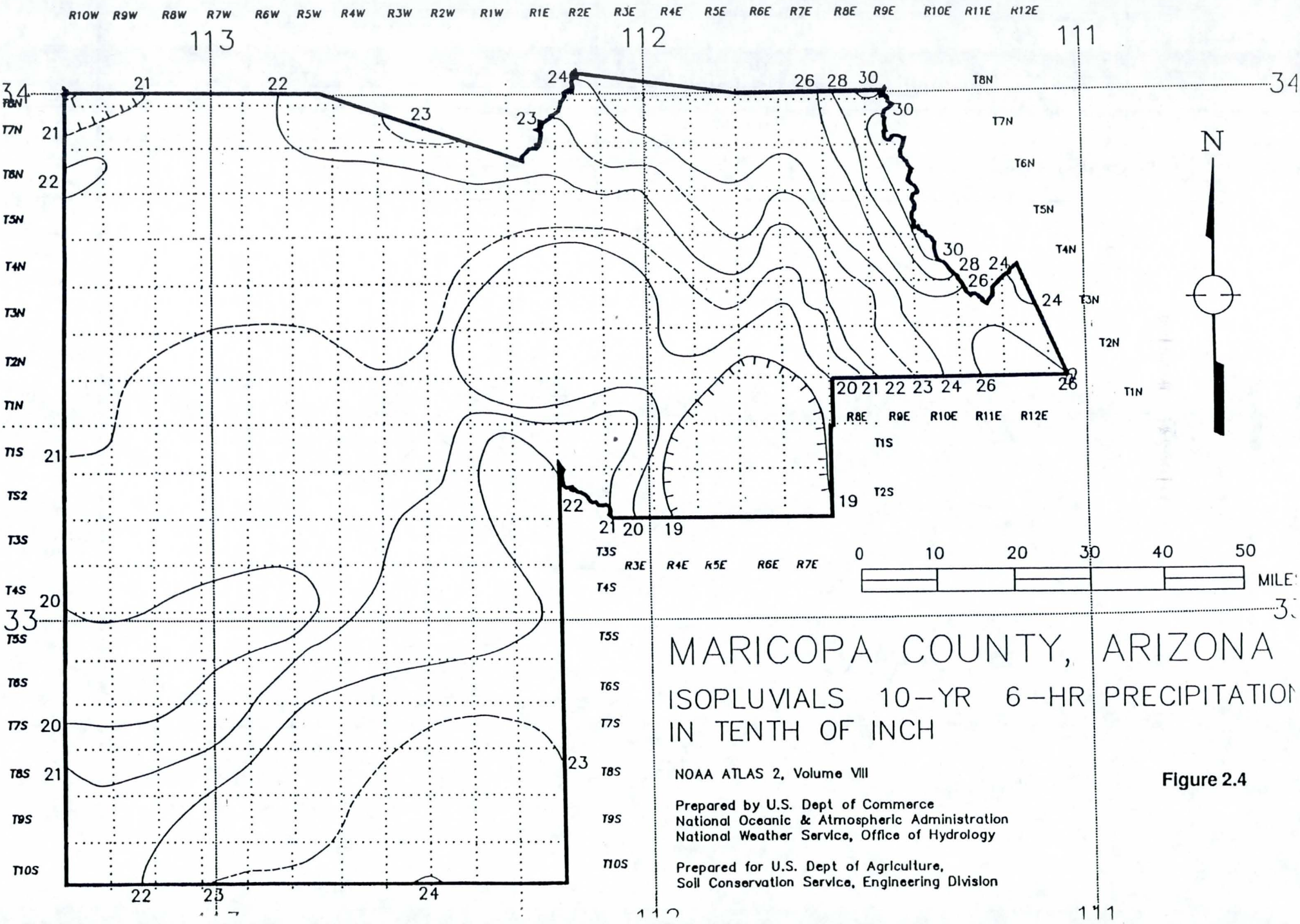
Figure 2.3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ISOPLUVIALS 10-YR 6-HR PRECIPITATION
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Figure 2.4

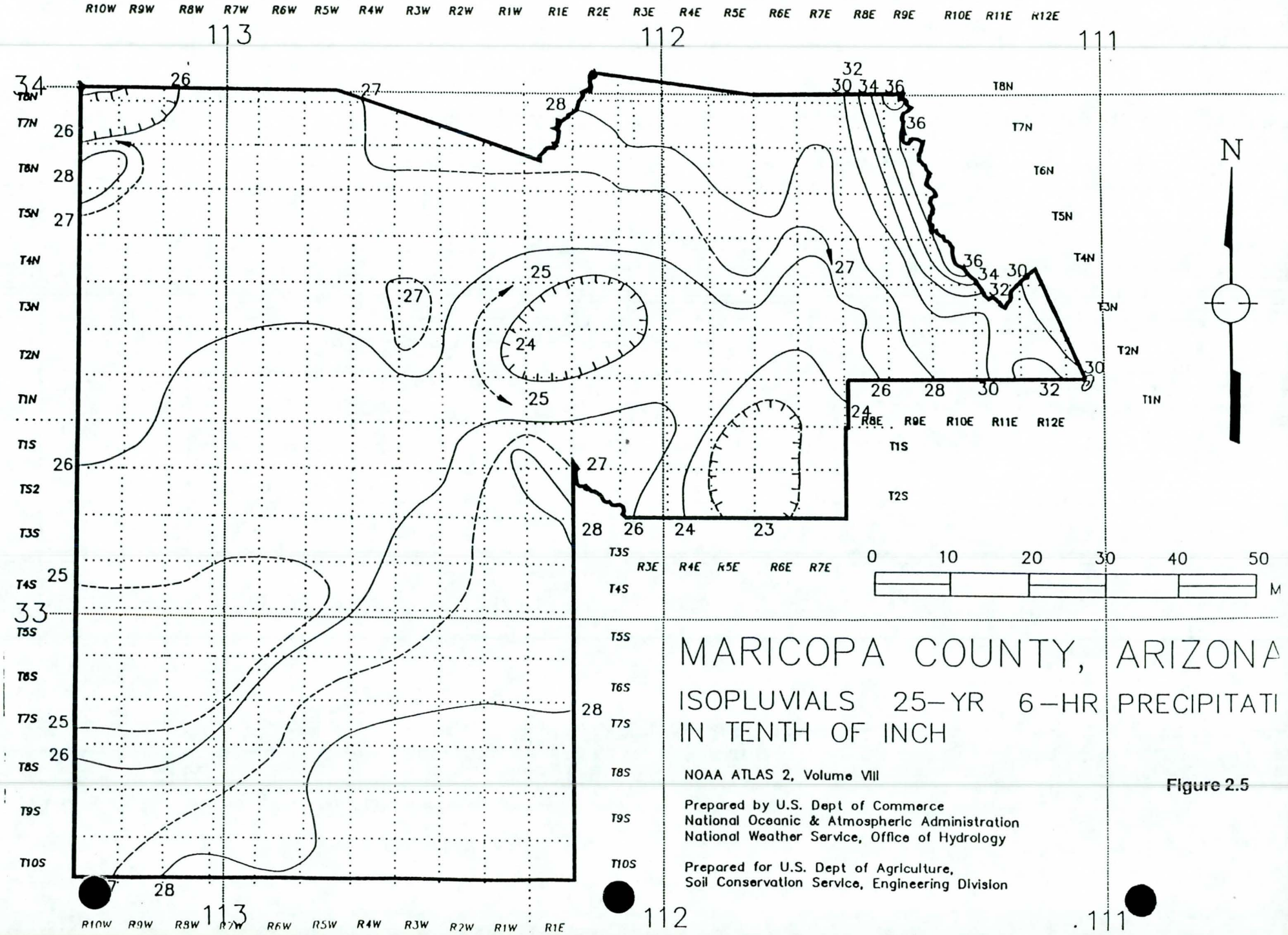
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Figure 2.5

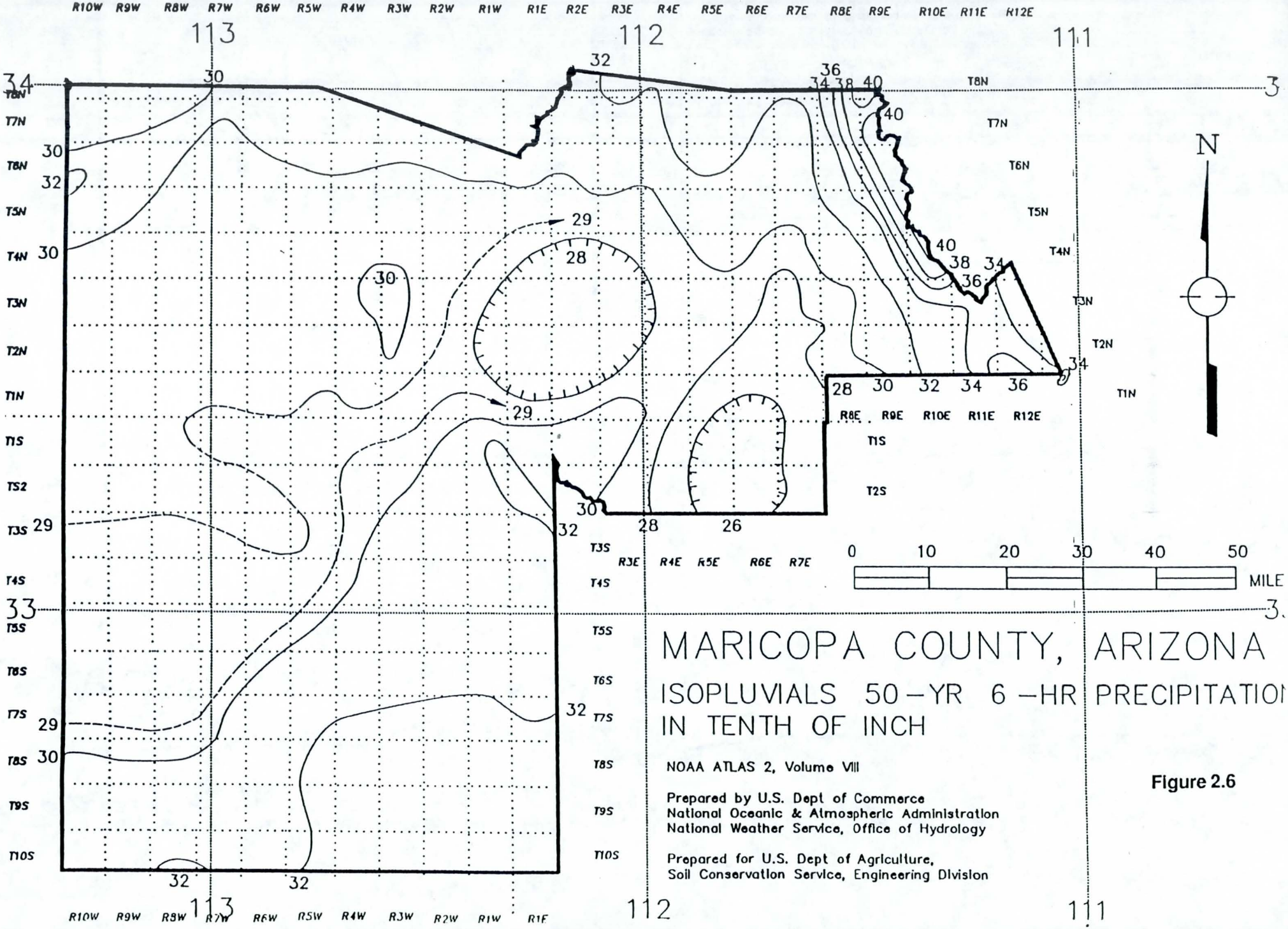
MARICOPA COUNTY, ARIZONA
ISOPLUVIALS 50-YR 6-HR PRECIPITATION
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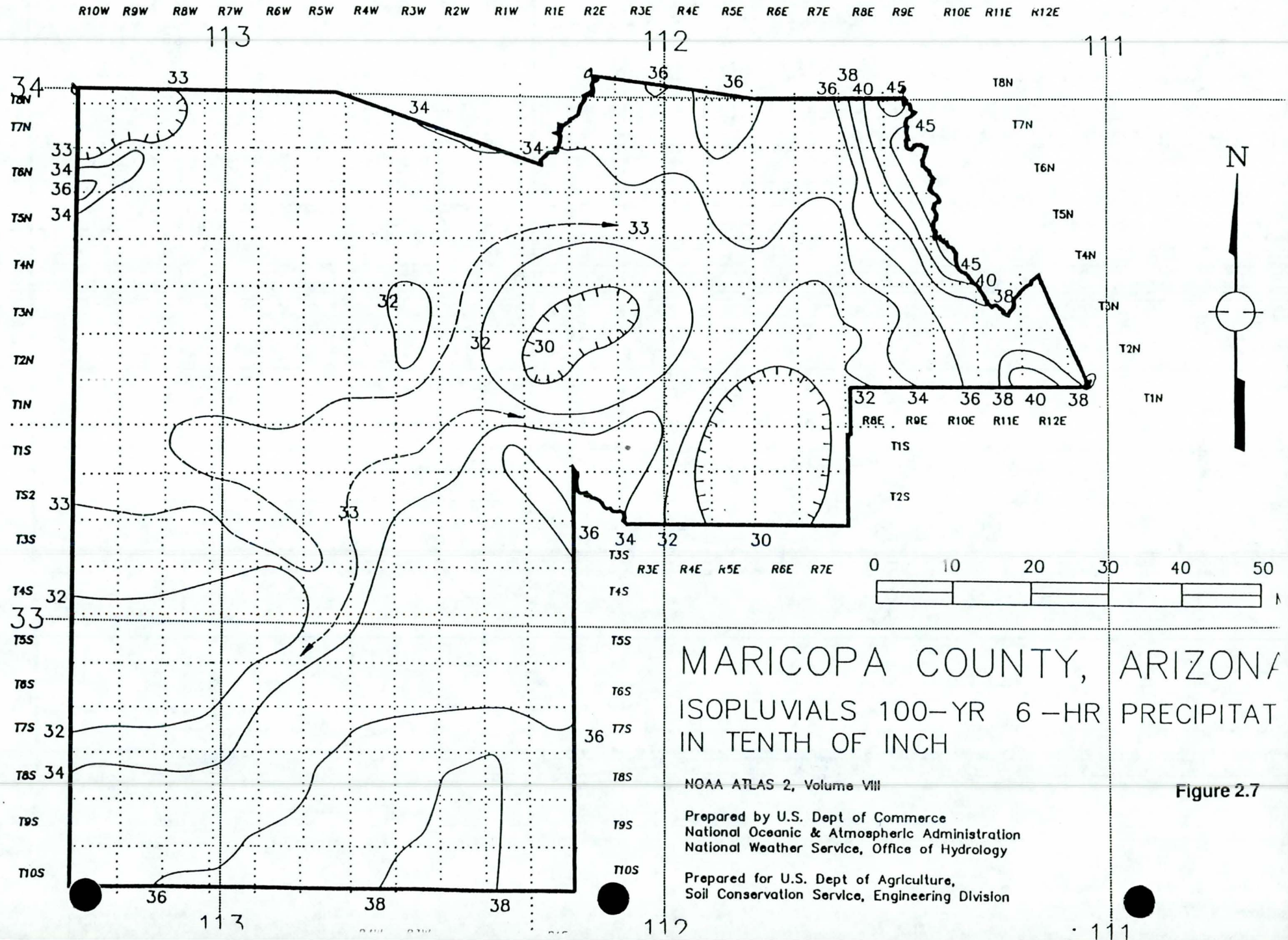
Figure 2.6



MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 100-YR 6-HR PRECIPITAT
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Figure 2.7

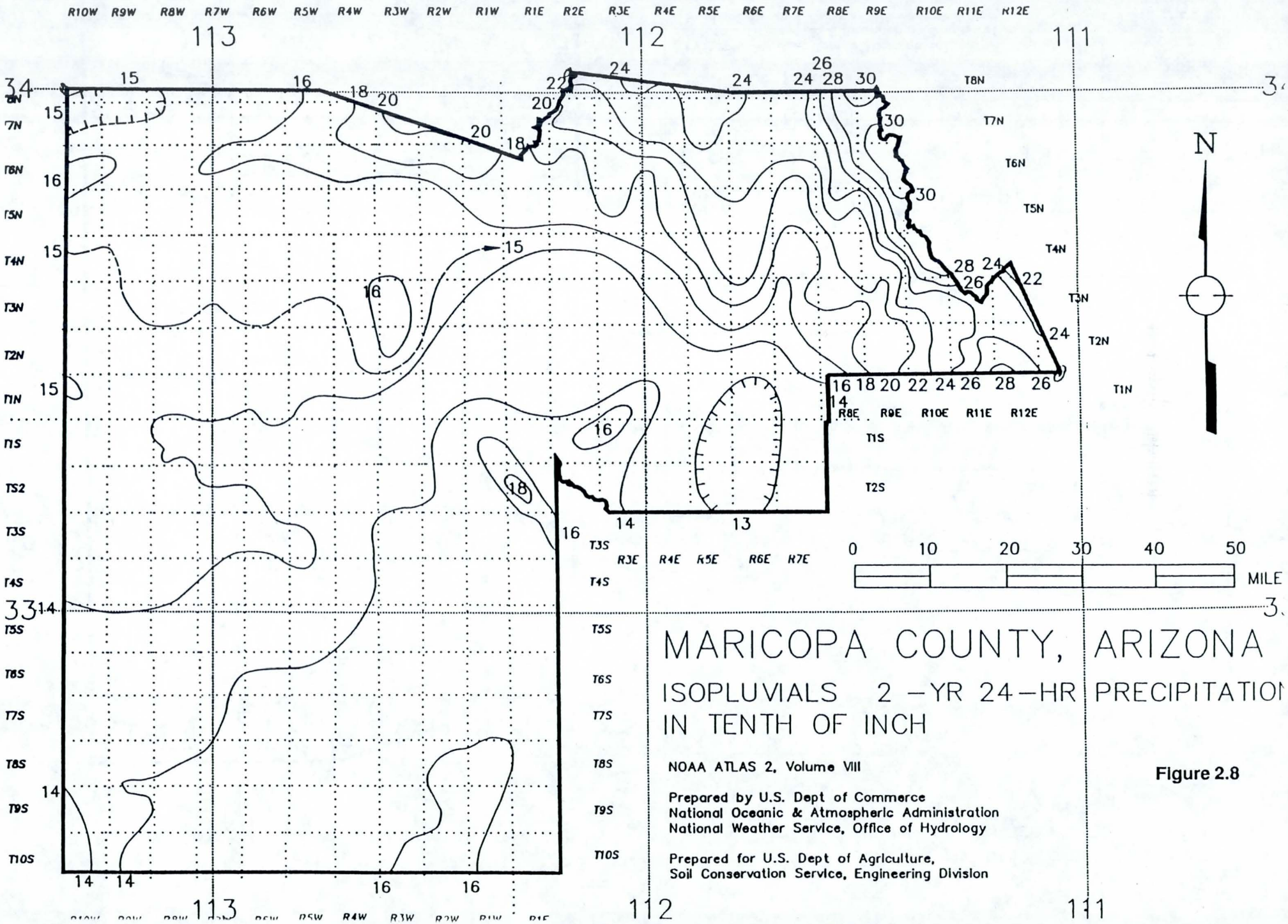
MARICOPA COUNTY, ARIZONA
ISOPLUVIALS 2 -YR 24-HR PRECIPITATION
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Figure 2.8

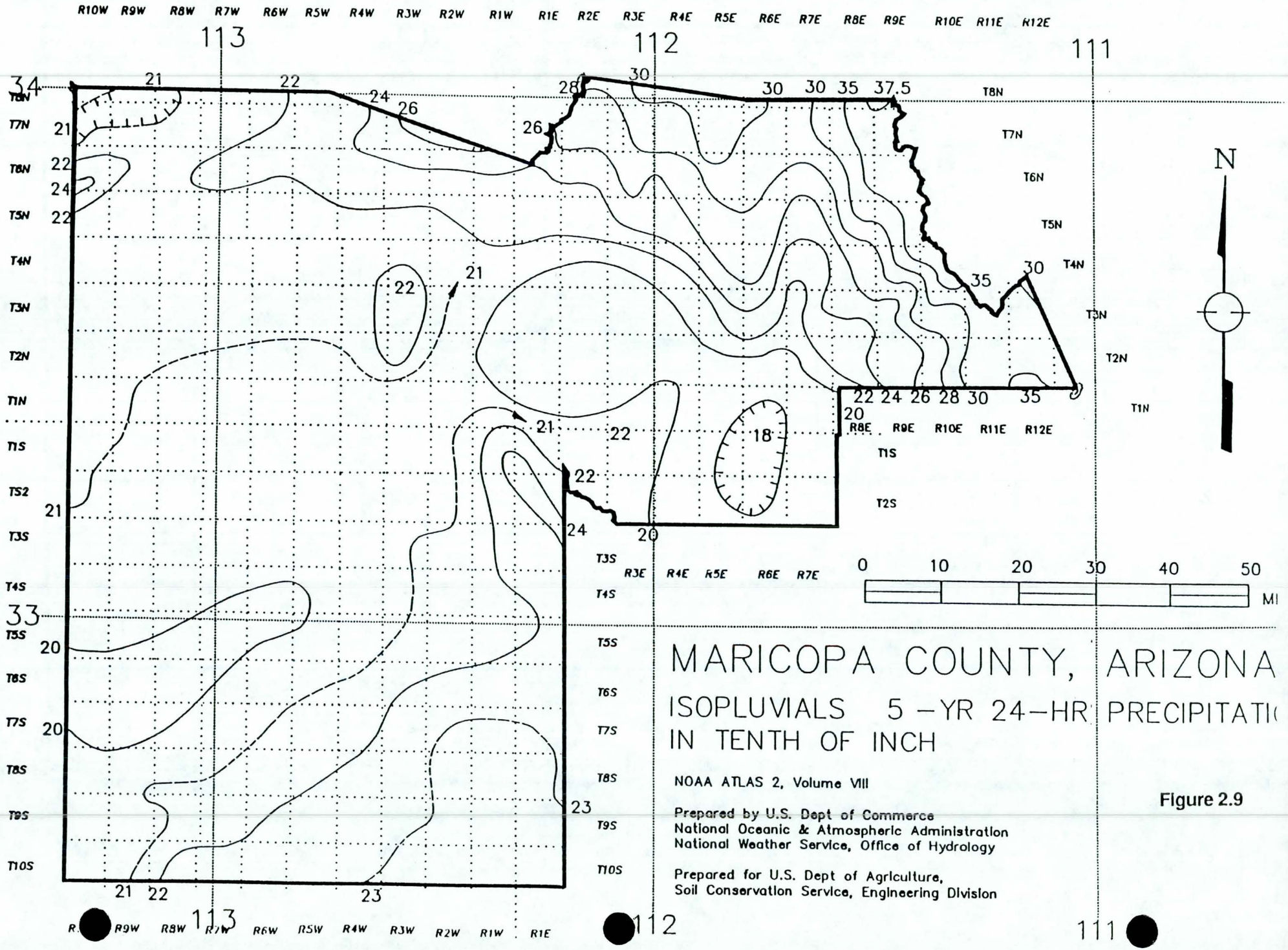
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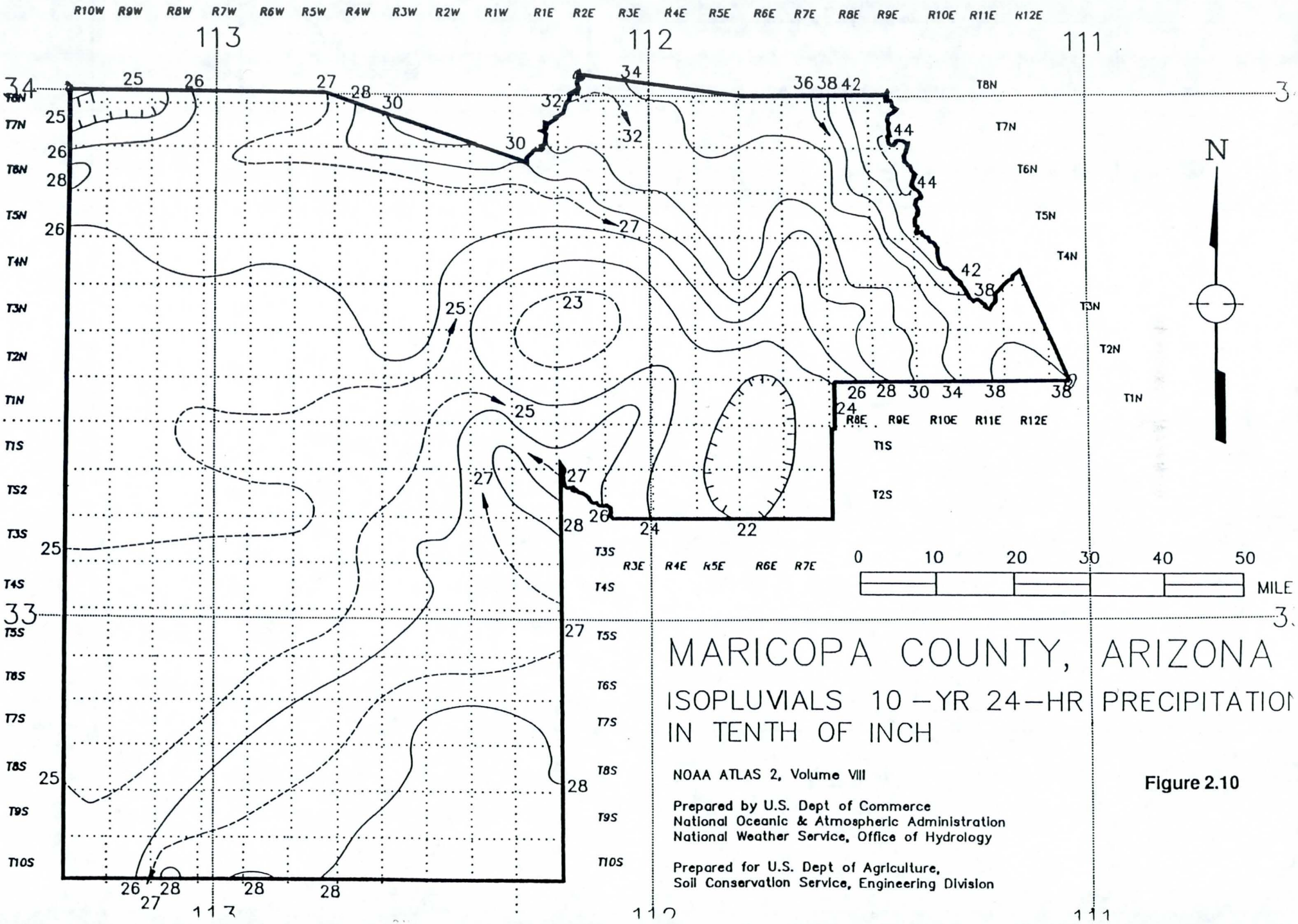
Figure 2.9



MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 10 – YR 24 – HR PRECIPITATION IN TENTH OF INCH

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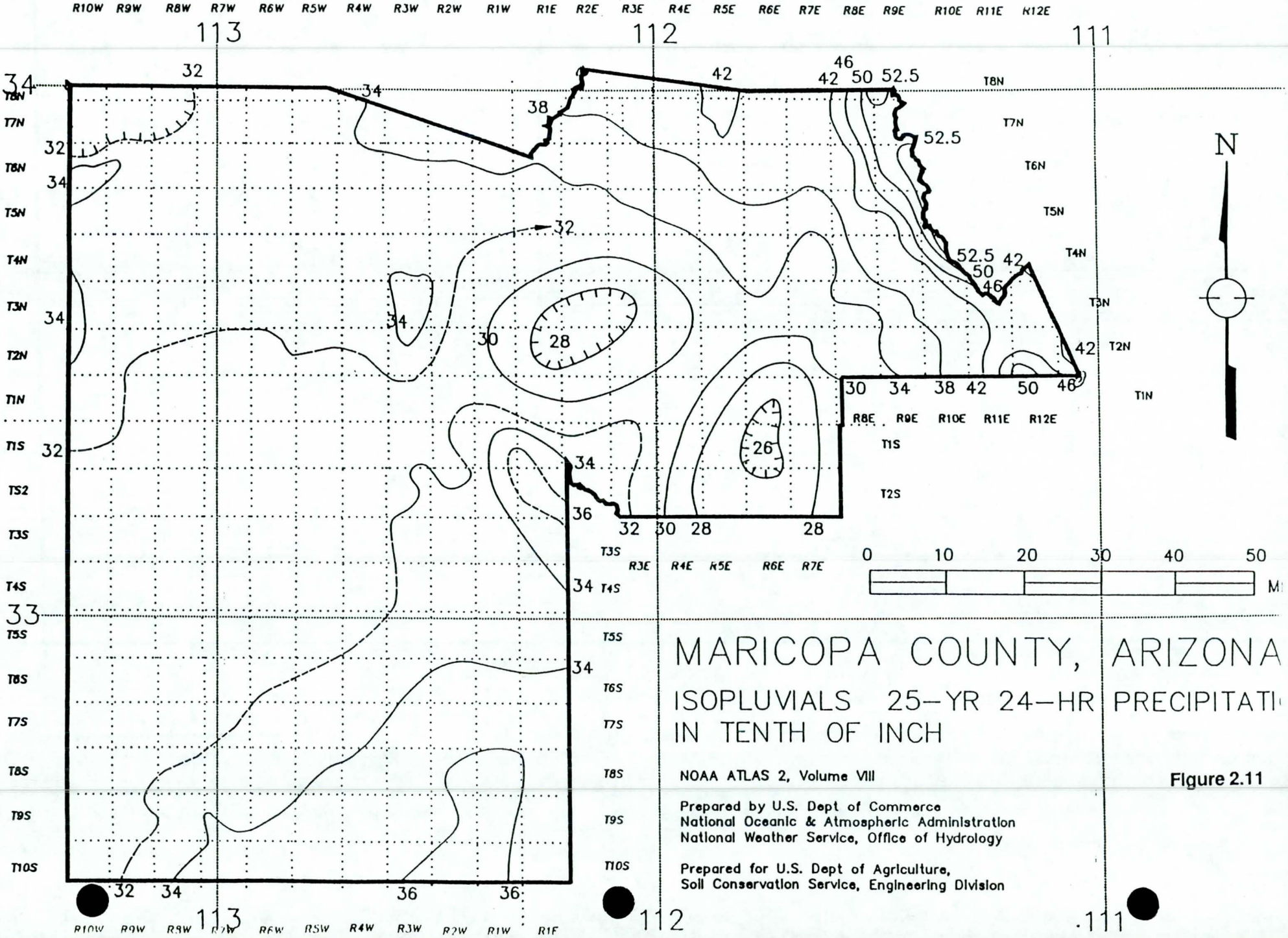
Figure 2.10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ISOPLUVIALS 25-YR 24-HR PRECIPITATI
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Figure 2.11

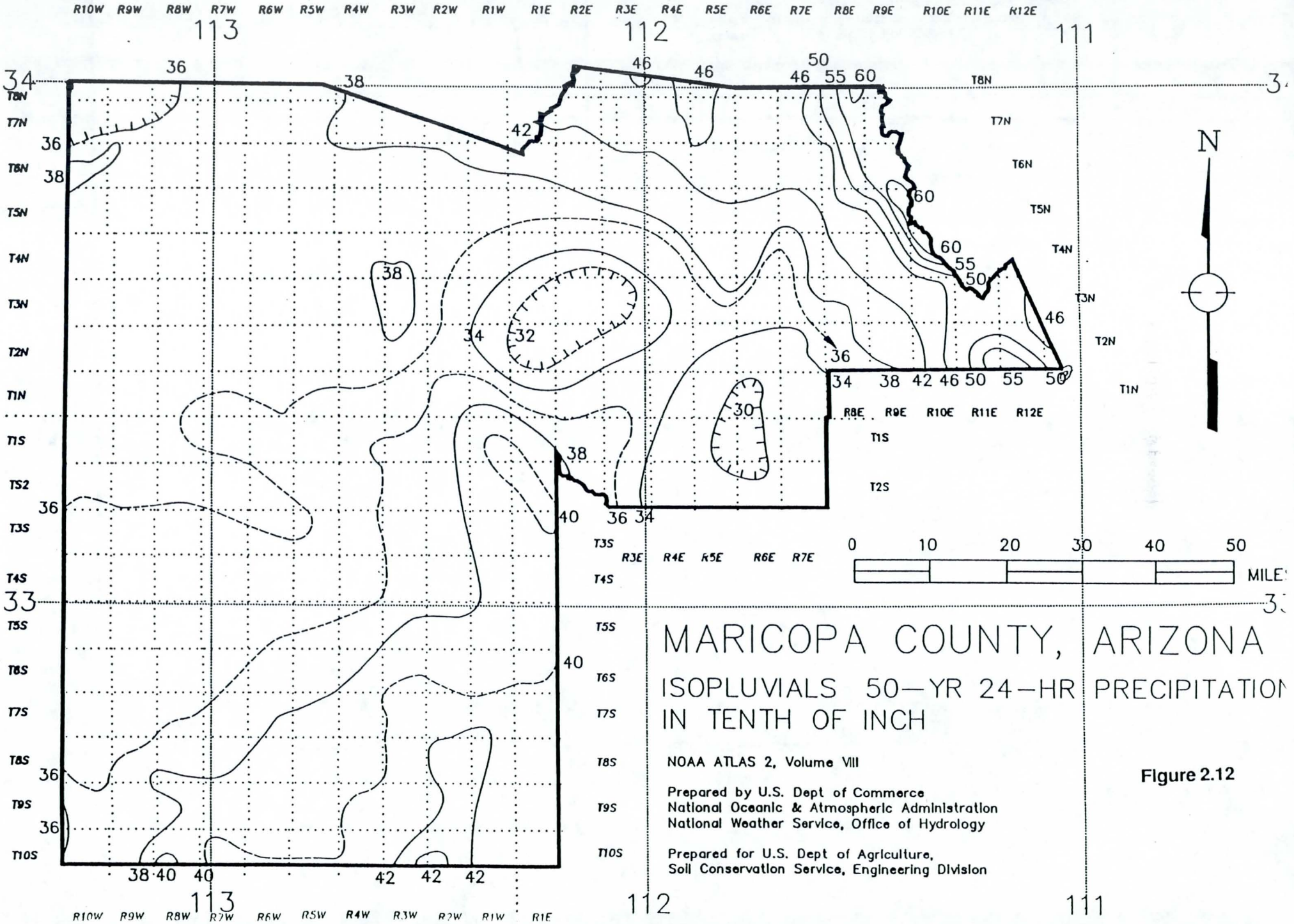
MARICOPA COUNTY, ARIZONA
ISOPLUVIALS 50—YR 24—HR PRECIPITATION
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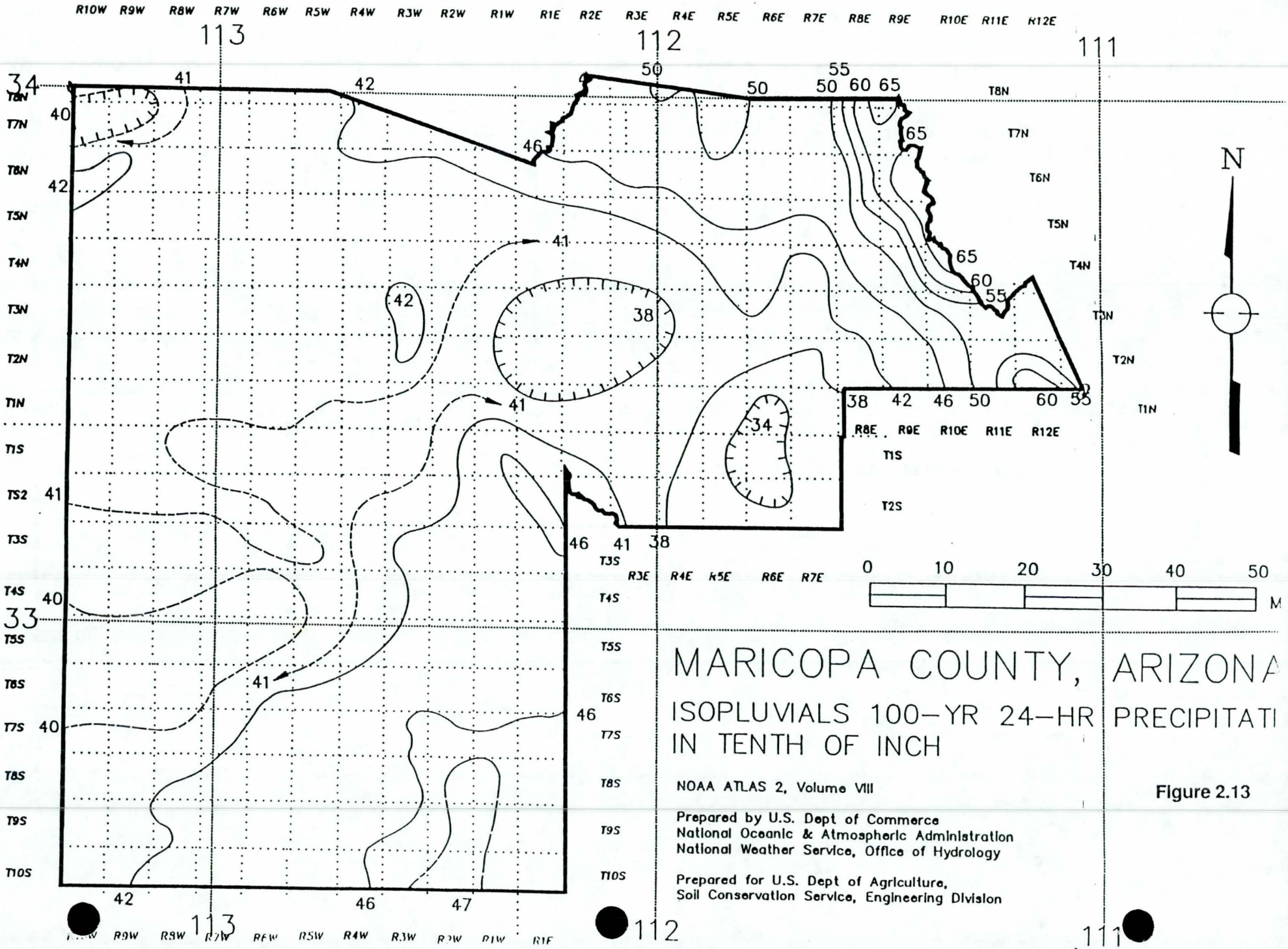
Figure 2.12



MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 100-YR 24-HR PRECIPITATI IN TENTH OF INCH

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Figure 2.13

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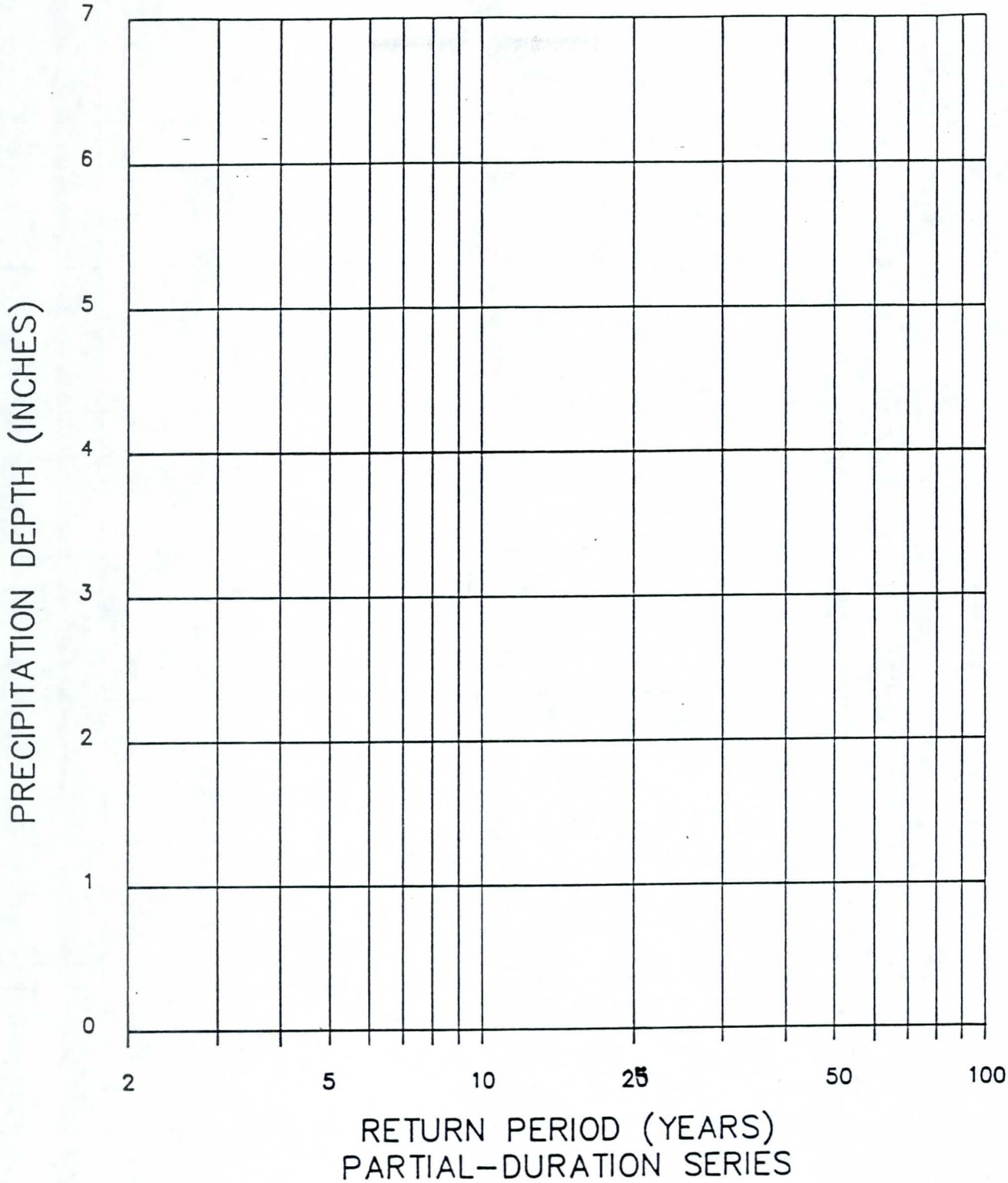

Figure 2.14
~~Per~~cipitation Depth versus Return Period for Partial-Duration Series
Pre

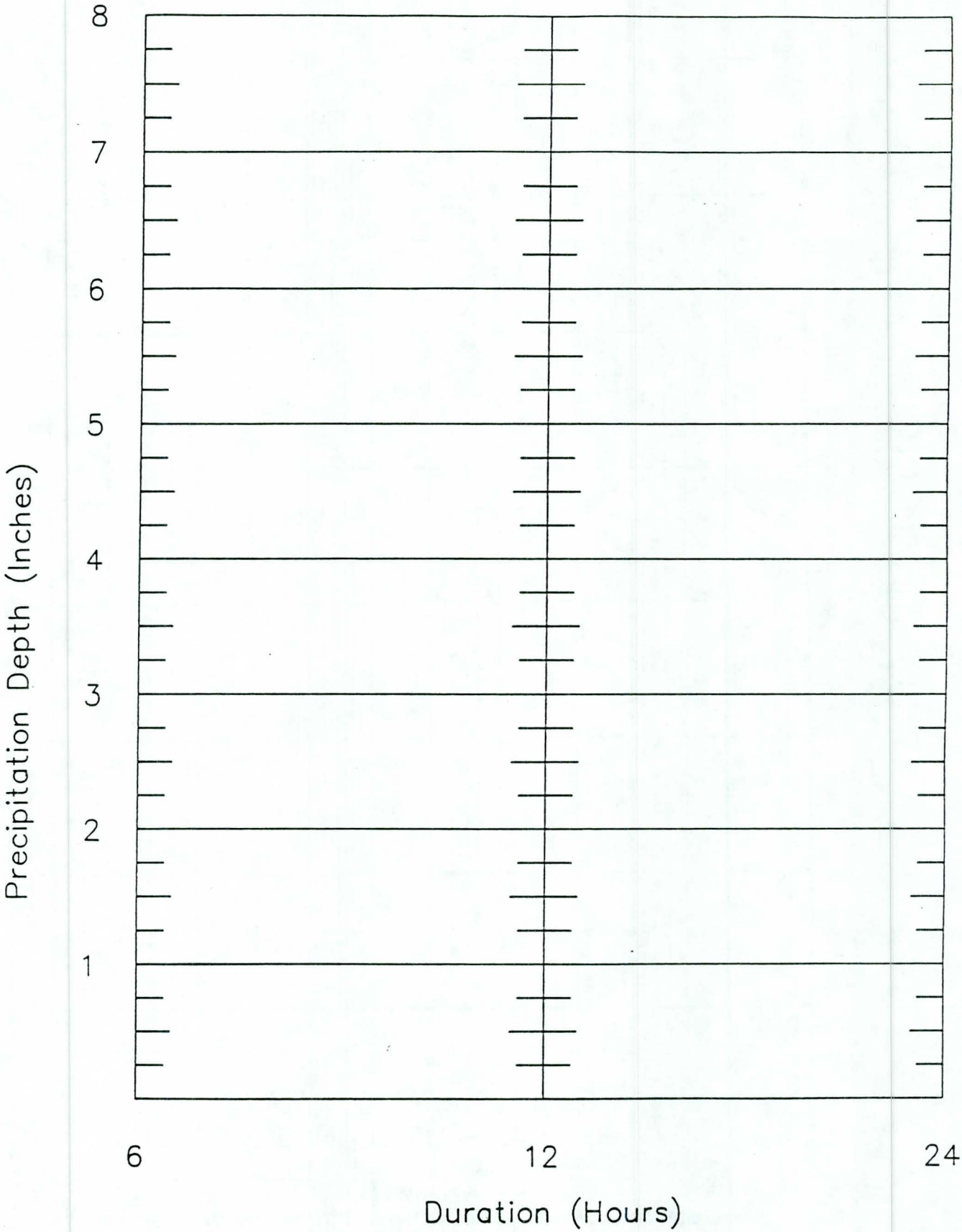

Figure 2.15
Percipitation Depth-Duration Diagram (6 to 24 hour)

2.5 Development of Design Storm Distributions

The design storms for use in Maricopa County will be either a 2-hour or a 6-hour distribution. The 2-hour storm is used for detention and retention design purposes, as mandated by the Uniform Drainage Standards and Policies. The 6-hour storm is for all hydrologic analysis for areas of up to 100 square miles. For watersheds larger than 100 square miles, the selection of design rainfall criteria must be performed on a case-by-case basis, by a qualified hydrologist.

2.5.1 2-hour Storm Distribution

If the Rational Method is used, for retention/detention design there is no need for a time distribution. The selected depth should be used based on the procedures in Chapter 3 of this manual. If a time distribution is required, i.e., rainfall input for HEC-1, the dimensionless 2-hour cumulative rainfall distribution of Table 2.2 should be used. These values are for direct input into HEC-1, assuming ~~either~~ a 5-minute ~~or a 15-minute intensity for~~ rainfall time step. Figure 2.16 illustrates the graphical form of this distribution.

2.5.2 6-hour Storm Distribution

The 6-hour rainfall distribution contains 5 dimensionless rainfall patterns. ~~is a function of drainage area size. For this purpose five dimensionless rainfall patterns have been developed~~. Pattern 1 applies NOAA Atlas procedures to data at the Phoenix Airport. ~~data. Patterns 2 through 5 are intended to provide variability of rainfall intensity as a function of drainage area. A set of rainfall patterns has been developed in *Design Memorandum No. 1, Gila River Basin*, based on the historic event of August 19, 1954 over the Queen Creek area (US Army Corps of Engineers, 1974). This information is modified for a 6-hour duration rainfall.~~ * A rainfall pattern can be selected from Table 2.3 for direct input into HEC-1, once the

Table 2.2
2-Hour Storm Distribution for Retention Design

Time (minutes)	% Rainfall Depth	Time (minutes)	% Rainfall Depth
0	0.0		
5	1.1	65	60.1
10	1.8	70	74.3
15	2.3	75	86.3
20	2.8	80	90.1
25	3.2	85	93.0
30	4.6	90	95.4
35	7.1	95	96.2
40	10.0	100	97.0
45	13.7	105	97.9
50	17.6	110	98.2
55	23.2	115	99.2
60	32.7	120	100.0

97.7

* Patterns 2 through 5 were developed from the information provided by the Corps of Engineers in Design Memorandum No. 1, Gila River Basin, and are based on the historic event of August 19, 1954 over the Queen Creek area. Pattern distributions are intended to provide variability of rainfall intensity as a function of drainage area.

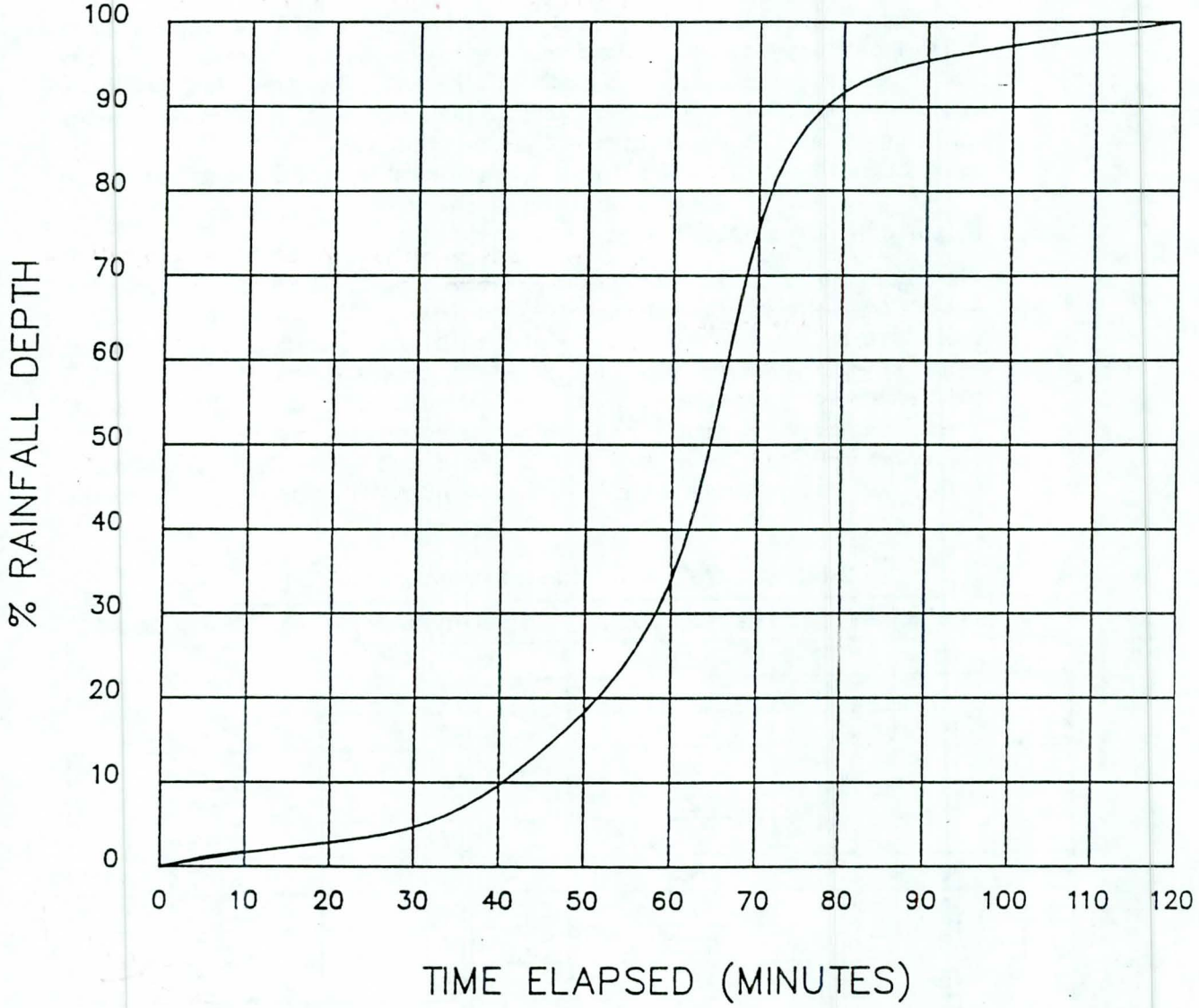

Figure 2.16
2-Hour Mass Curve for Retention Design

size of the drainage area is determined. Figure 2.17 illustrates the dimensionless rainfall patterns. Use Figure 2.18 to select a rainfall pattern between 1 to 5 and interpolate as necessary for watershed areas of up to 100 square miles. Alternatively, programs MCUHP1 and MCUHP2 can be used for this purpose.

As mentioned earlier, any watershed larger than 100 square miles should be analyzed on a case-by-case basis to determine the design rainfall criteria for the critical storm event considering watershed size, location, and other factors including rainfall depth, duration, and temporal distribution.

Table 2.3
6-Hour Distributions*

Time (hrs)	Pattern 1	Pattern 2	Pattern 3	Pattern 4	Pattern 5
0:00	~~.0~~ .0	.0	.0	.0	.0
0:15	~~.5~~ .8	~~.6~~ .9	1.5	2.1	2.4
0:30	~~0.9~~ 1.6	~~1.2~~ 1.6	2.0	3.5	4.3
0:45	~~1.4~~ 2.5	~~2.0~~ 2.5	3.0	5.1	5.9
1:00	~~2.2~~ 3.3	~~3.1~~ 3.4	4.8	7.1	7.8
1:15	~~3.0~~ 4.1	~~3.9~~ 4.2	6.3	8.7	9.8
1:30	~~3.8~~ 5.0	~~4.9~~ 5.1	7.6	10.5	11.9
1:45	~~4.7~~ 5.8	~~5.7~~ 5.9	9.0	12.5	14.1
2:00	~~5.4~~ 6.6	6.7	10.5	14.3	16.2
2:15	~~6.2~~ 7.4	7.6	11.9	16.0	18.6
2:30	~~7.5~~ 8.7	8.7	13.5	17.9	21.2
2:45	~~8.8~~ 9.9	10.0	15.2	20.1	23.9
3:00	~~10.7~~ 11.8	12.0	17.5	23.2	27.1
3:15	~~12.7~~ 13.8	16.3	22.2	28.1	32.1
3:30	~~20.5~~ 21.6	25.2	30.4	36.4	40.8
3:45	~~36.6~~ 37.7	45.1	47.2	50.0	51.5
4:00	~~82.3~~ 83.4	69.4	67.0	65.8	62.7
4:15	~~90.0~~ 91.1	83.7	79.6	77.3	73.5
4:30	~~92.0~~ 93.1	90.0	86.8	84.1	81.4
4:45	~~93.9~~ 95.0	93.8	91.2	88.8	86.4
5:00	~~95.2~~ 96.2	95.0	94.6	92.7	90.7
5:15	~~96.5~~ 97.2	96.3	96.0	94.5	93.0
5:30	~~97.7~~ 98.3	97.5	97.3	96.4	95.4
5:45	~~98.8~~ 99.1	98.8	98.7	98.2	97.7
6:00	100.0	100.0	100.0	100.0	100.0

*Pattern represents percent Rainfall Depth.

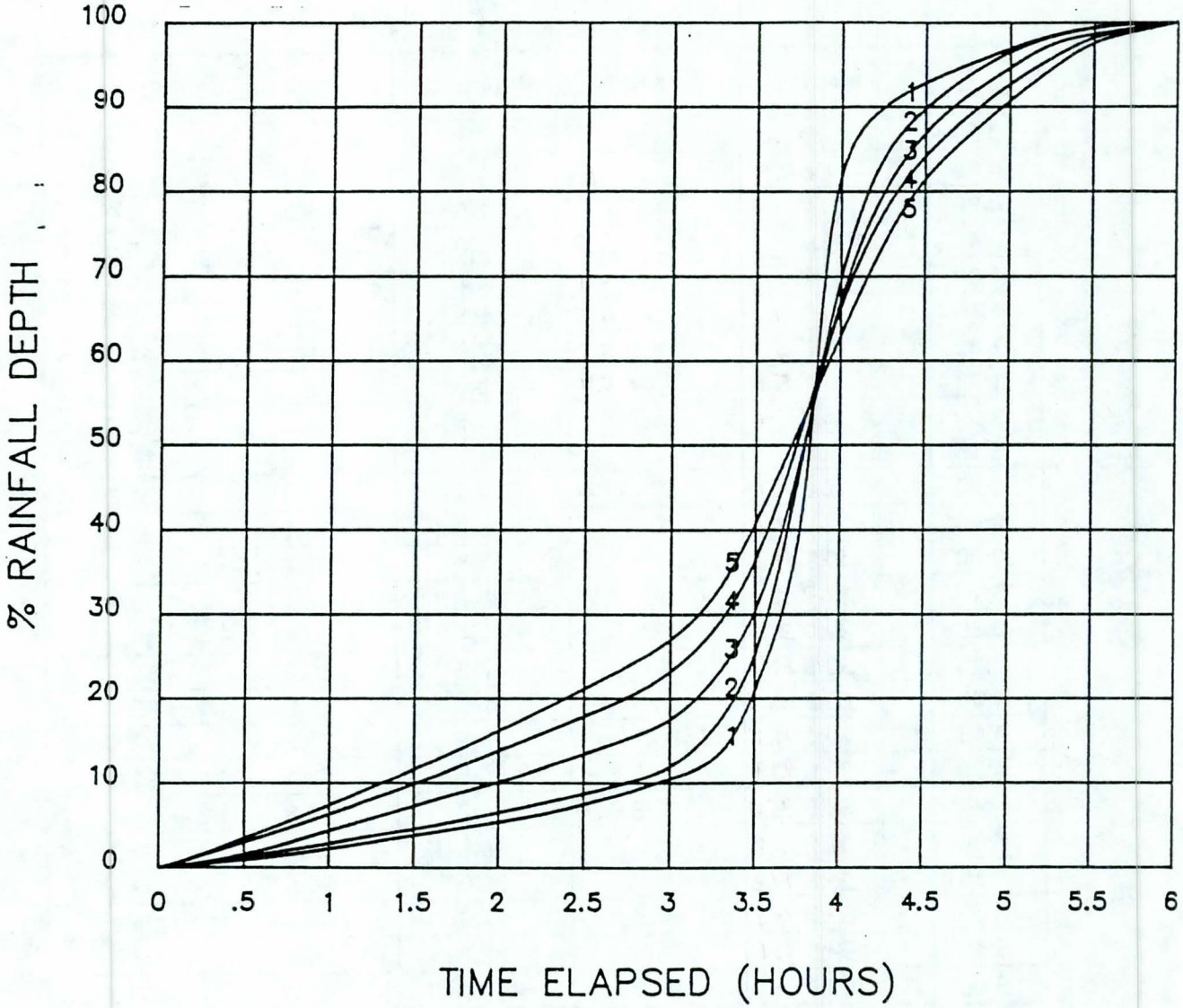

Figure 2.17
6-Hour Mass Curves for Maricopa County

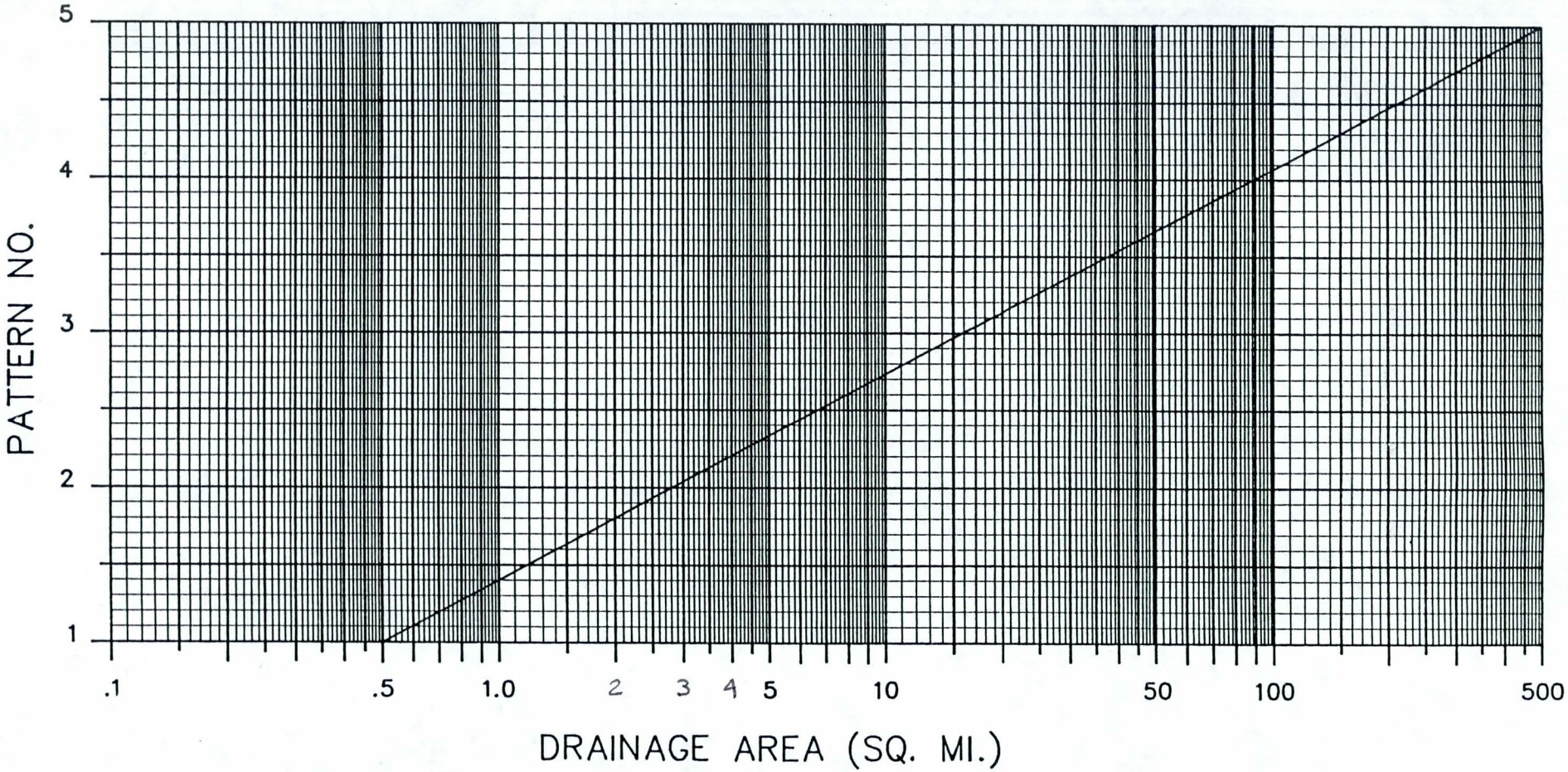

Figure 2.18
Area Versus Pattern Number for Maricopa County

3

Rational Method

3.1 General

The Rational Method was originally developed to estimate runoff from small areas and its use should be generally limited to those conditions. For the purposes of this manual, its use should be limited to areas of up to 160 acres. In such cases the peak discharge and the volume of runoff from rainfall events up to and including the 100-year 2-hour duration storm falling within the boundaries of the proposed development are to be retained. If the development involves channel routing, the procedures given in Chapters 4 through 6 should be used, since the peak generated by the Rational Method cannot be directly routed.

3.2 Rational Equation

The Rational Equation relates rainfall intensity, a runoff coefficient and the watershed size to the generated peak discharge. The following shows this relationship:

$$Q = CiA \tag{1}$$

where

Q	=	the runoff (cfs) from a given area.
C	=	a coefficient relating the runoff to rainfall.
i	=	average rainfall intensity (inches/hour), lasting for a Tc.
Tc	=	the time of concentration (hours).
A	=	drainage area (acres).

The Rational Equation is based on the concept that the application of a steady, uniform rainfall intensity will produce a peak discharge at such a time when all points of the watershed are contributing to the outflow at the point of design. Such

a condition is met when the elapsed time is equal to the time of concentration, Tc, which is defined to be the * time for ~~water to flow~~ from the most remote part of the watershed to the point of design. The time of concentration should be computed by applying the following equation developed by Papadakis and Kazan (1987):

$$Tc = 11.4\, L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38} \qquad (2)$$

where * floodwave travel time

Tc	=	time of concentration in hours
L	=	length of the longest flow path in miles
K_b	=	watershed resistance coefficient (see Figure 3.1, or Table 3.1)
S	=	water course slope in feet/mile
i	=	rainfall intensity in inches/hour*

*It should be noted that i is the "rainfall excess intensity" as originally developed. However, when used in the Rational Equation, rainfall intensity and rainfall excess intensity provide similar values because of the hydrologic characteristics of small, urban watersheds which result in minimal rainfall loss. This is because of the extent of imperviousness associated with urban watersheds and the fact that the time of concentration is usually very short.

Table 3.1
Equation for Estimating K_b in the Tc Equation

$K_b = m \log A + b$ Where A is drainage area, in acres		
Land Classification (1)	**Equation Parameters**	
	m (2)	**b (3)**
Urban	– 0.00625	0.04
Bare or nearly bare ground (alluvial fan, agricultural land, desert rangeland)	– 0.01375	0.08
Rough and/or moderate vegetation (hillslopes)	– 0.025	0.15
Very rough and/or dense vegetation (mountains)	– 0.030	0.20

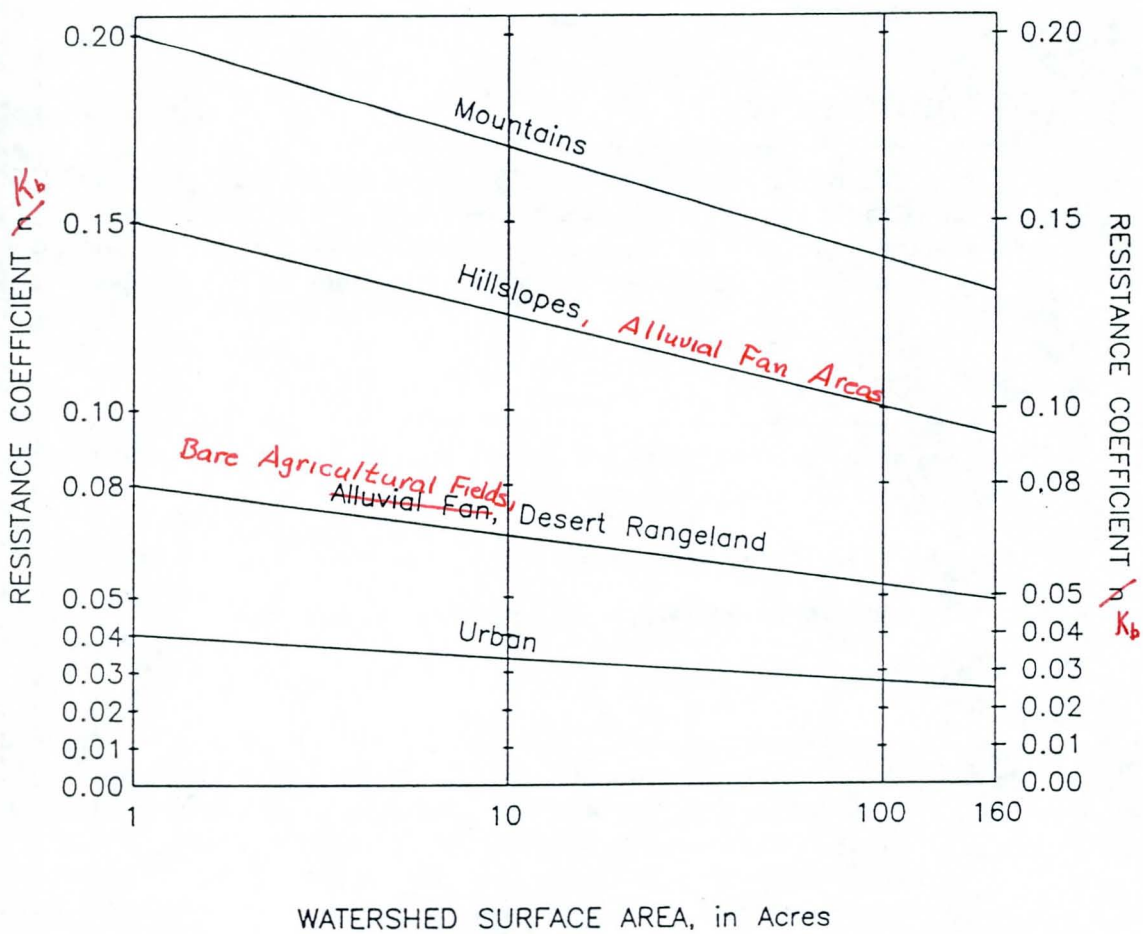

Figure 3.1
Resistance Coefficient "n" as a Function of Watershed Type and Size

3.3 Assumptions

Application of the Rational Equation requires consideration of the following:

1. The peak discharge rate corresponding to a given intensity would occur only if the rainfall duration is at least equal to the time of concentration.

2. The calculated runoff is directly proportional to the rainfall intensity.

3. The frequency of occurrence for the peak discharge is the same as the frequency for the rainfall producing that event.

4. The runoff coefficient would remain the same for all storms for a given watershed.

3.4 Limitations

Application of the Rational Method is appropriate for small watersheds. This is based on the assumption that the rainfall intensity is to be uniformly distributed over the drainage area at a uniform rate lasting for the duration of the storm. Beyond this limitation the rainfall distribution may vary from the indicated point value.

3.5 Application

The Rational Method should be used to calculate the generated peak discharge and runoff volume for small drainage areas.

3.5.1 Peak Discharge Calculation

1. Determine the area within the development boundaries.

2. Select the runoff coefficient, C from Table 3.2

3. Calculate time of concentration (see example 4). This is to be done by an iterative process. Select a duration from the I-D-F curves, Figure 3.2. This value should not be longer than two hours and normally it will be less than an hour. Determine the maximum rainfall intensity indicated on the I-D-F curve for a frequency that includes the 100-year. The intensity value of the corresponding Tc in the above is for the Phoenix ~~Airport data~~. Metro area. Use i_p in the following equation for estimating i for other areas:

$$i = i_p \frac{(P^6_{10})}{2.07} \qquad (3)$$

where

i = the desired intensity for a given duration and frequency.

i_p = the intensity for the Phoenix ~~Airport data.~~ ^Metro area.

P^6_{10} = the 10-year, 6-hour precipitation depth at the point of interest. (Can be read from Figure 2.4.)

4. Use the adjusted intensity in Equation 2 to calculate time of concentration. Repeat this process until the selected and computed Tc values are reasonably close. For more details see example 4.

5. Determine peak discharge (Q) by using the above value of *i* in Equation 1.

Table 3.2
C Coefficients for Use with the Rational Formula

Streets	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Gravel roadways & shoulders	0.40 - 0.60
Industrial Areas	
Flat commercial (about %90 impervious)	0.80
Heavy areas	0.60 - 0.90
Light areas	0.50 - 0.80
Business Areas	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential Areas	
Lawns - flat	0.05 - 0.15
- steep	0.15 - 0.35
Suburban areas	0.25 - 0.40
Single family areas	0.30 - 0.50
Multi - unit areas	0.40 - 0.60
Apartment areas	0.50 - 0.70
Parks, Cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.30
Agricultural Areas	0.10 - 0.20
Bare ground	0.20 - 0.30
Undeveloped Desert	0.30 - 0.40
Mountain Terrain (slopes > 10 percent)	0.60 - 0.80

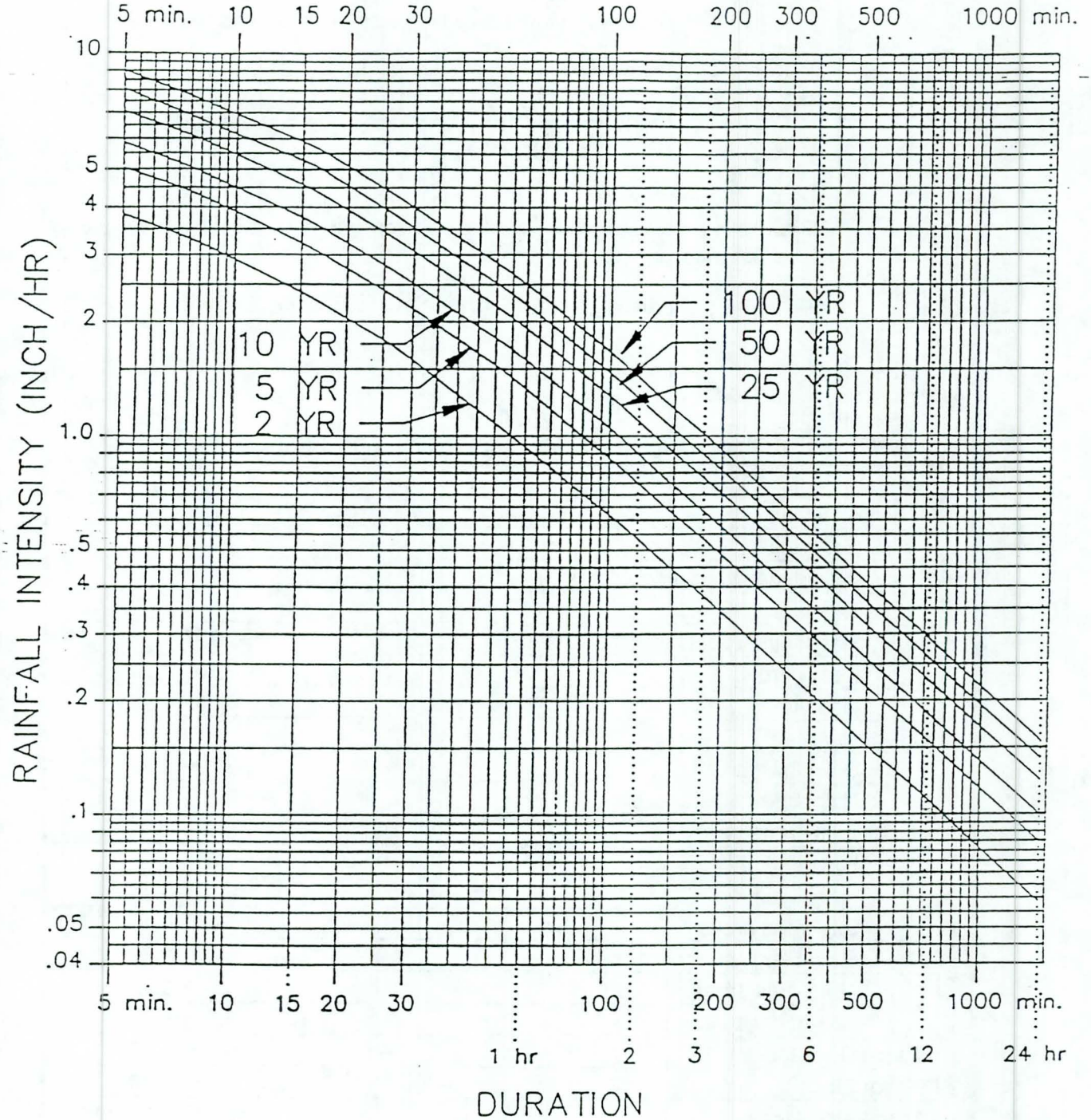

Figure 3.2
Rainfall Intensity-Duration-Frequency Relation
(Phoenix ~~Sky Harbor Airport~~)
Metro Area

3.5.2 Volume Calculations

Volume calculation should be done by applying the following equation:

$$V = C\left(\frac{P}{12}\right)A \tag{4}$$

where

V	=	Calculated volume in acre-feet
C	=	Runoff coefficient from Table 3.2
P	=	~~100-year, 2-hour~~ rainfall depth in inches
A	=	Drainage area in acres

In the case of volume calculations for retention/detention design, P = the 100-year, 2-hour depth in inches from Section 2.4.

4

Rainfall Losses

4.1 General

Rainfall excess is that portion of the total rainfall depth that drains directly from the land surface by overland flow. By a mass balance, rainfall excess plus rainfall loss equals precipitation. When performing a flood analysis using a rainfall-runoff model, the determination of rainfall excess is of utmost importance. Rainfall excess integrated over the entire watershed results in runoff volume, and the temporal distribution of the rainfall excess will, along with the hydraulics of runoff, determine the peak discharge. Therefore, the estimation of the magnitude and time distribution of rainfall losses should be performed with the best practical technology, considering the objective of the analysis, economics of the project, and consequences of inaccurate estimates.

Rainfall losses are generally considered to be the result of evaporation of water from the land surface, interception of rainfall by vegetal cover, depression storage on the land surface (paved or unpaved), and infiltration of water into the soil matrix. A schematic representation of rainfall losses for a uniform intensity rainfall is shown in Figure 4.1. As shown in the figure, evaporation can start at an initially high rate depending on the land surface temperature, but the rate decreases very rapidly and would eventually reach a low, steady-state rate. From a practical standpoint, the magnitude of rainfall loss that can be realized from evaporation during a storm of sufficient magnitude to cause flood runoff is negligible.

Interception, also illustrated in Figure 4.1, varies depending upon the type of vegetation, maturity, and extent of canopy cover. Experimental data on interception have been collected by numerous investigators (Linsley and others, 1982), but little is known of the interception values for most hydrologic problems. Estimates of interception for various vegetation types (Linsley and others, 1982) are:

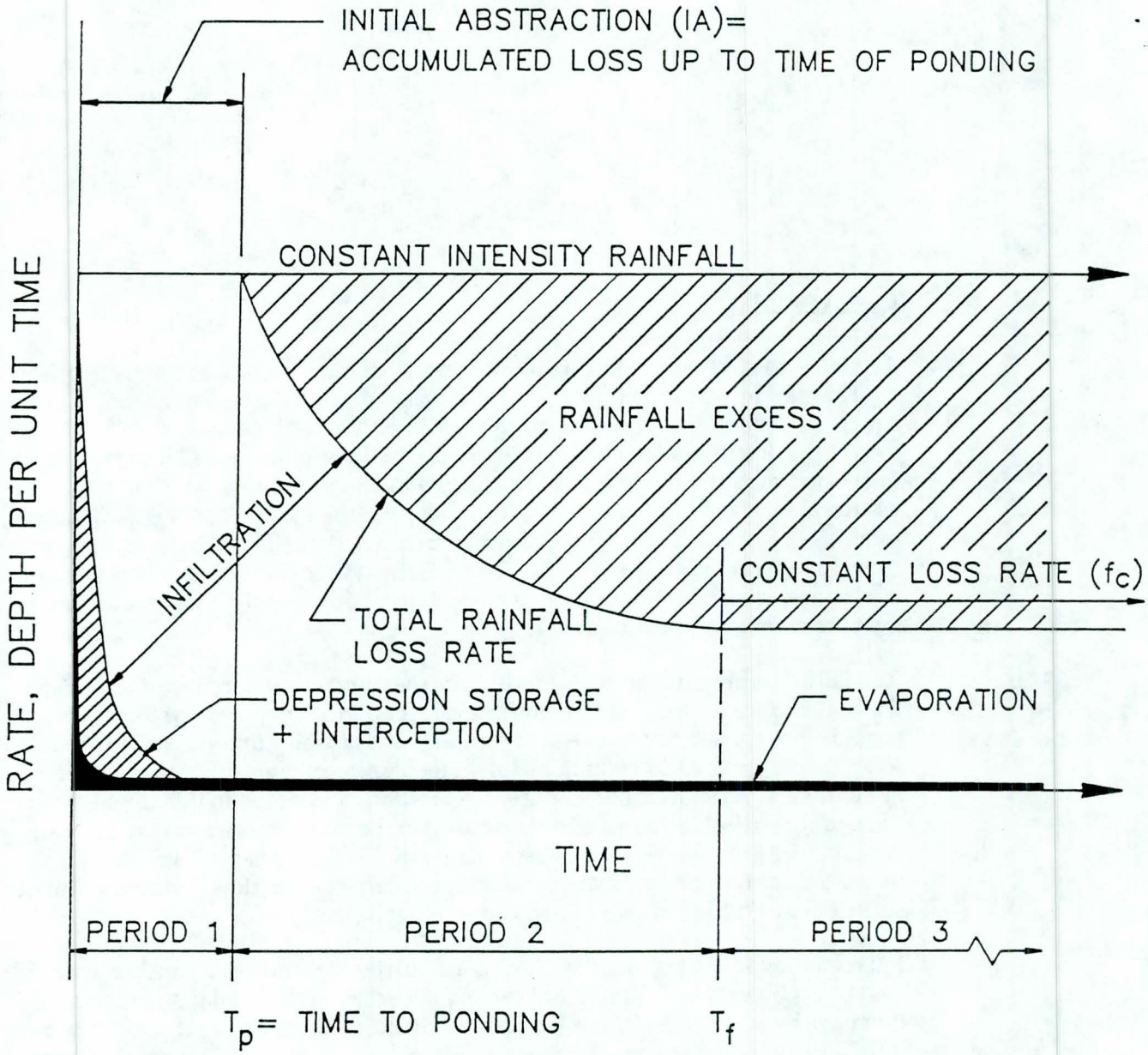

Figure 4.1
Schematic Representation of Rainfall Losses for a Uniform Intensity Rainfall

Vegetation Type	Interception, Inches
hardwood tree	0.09
cotton	0.33
alfalfa	0.11
meadow grass	0.08

No interception estimates are known for natural vegetation that occurs in Maricopa County. For most applications in Maricopa County the magnitude of interception losses is essentially 0.0, and for practical purposes interception is not considered for flood hydrology in Maricopa County.

Depression storage and infiltration losses comprise the majority of the rainfall loss as illustrated in Figure 4.1. The estimates of these two losses will be discussed in more detail in later sections of this manual. Three periods of rainfall losses are illustrated in Figure 4.1, and these must be understood and their implications appreciated before applying the procedures in this manual. First, there is a period of initial loss when no rainfall excess (runoff) is produced. During this initial period, the losses are a function of the depression storage, interception, and evaporation rates plus the initially high infiltration capacity of the soil. The accumulated rainfall loss during this period with no runoff is called the *initial abstraction*. The end of this initial period is noted by the onset of ponded water on the surface, and the time from start of rainfall to this time is the *time of ponding* (Tp). It is important to note that losses during this first period are a summation of losses due to all mechanisms including infiltration.

The second period is marked by a declining infiltration rate and generally very little losses due to other factors.

The third, and final, period occurs for rainfalls of sufficient duration for the infiltration rate to reach the *steady-state, equilibrium rate of the soil* (fc). The only appreciable loss during the final period is due to infiltration.

In Figure 4.2, it is assumed

The actual loss process is quite complex and there is a good deal of interdependence of the loss mechanisms on each other and on the rainfall itself. Therefore, simplifying assumptions are usually made in the modeling of rainfall losses. Figure 4.2 represents a simplified set of assumptions that can be made: that surface retention loss is the summation of all losses other than those due to infiltration, and that this loss occurs from the start of rainfall and ends when the accumulated rainfall equals the magnitude of the capacity of the surface retention loss. It is assumed that infiltration does not occur during this time. After the surface retention is satisfied, infiltration begins. If the infiltration capacity exceeds the rainfall intensity, then no rainfall excess is produced. As the infiltration capacity decreases, it may eventually equal the rainfall intensity. This would occur at the time of ponding (Tp) which signals the beginning of surface runoff. As illustrated in both Figures 4.1 and 4.2, after the time of ponding the infiltration rate decreases exponentially and may reach a steady-state, equilibrium rate (fc). It is these simplified assumptions and processes, as illustrated in Figure 4.2, that are to be modeled by the procedures in this manual.

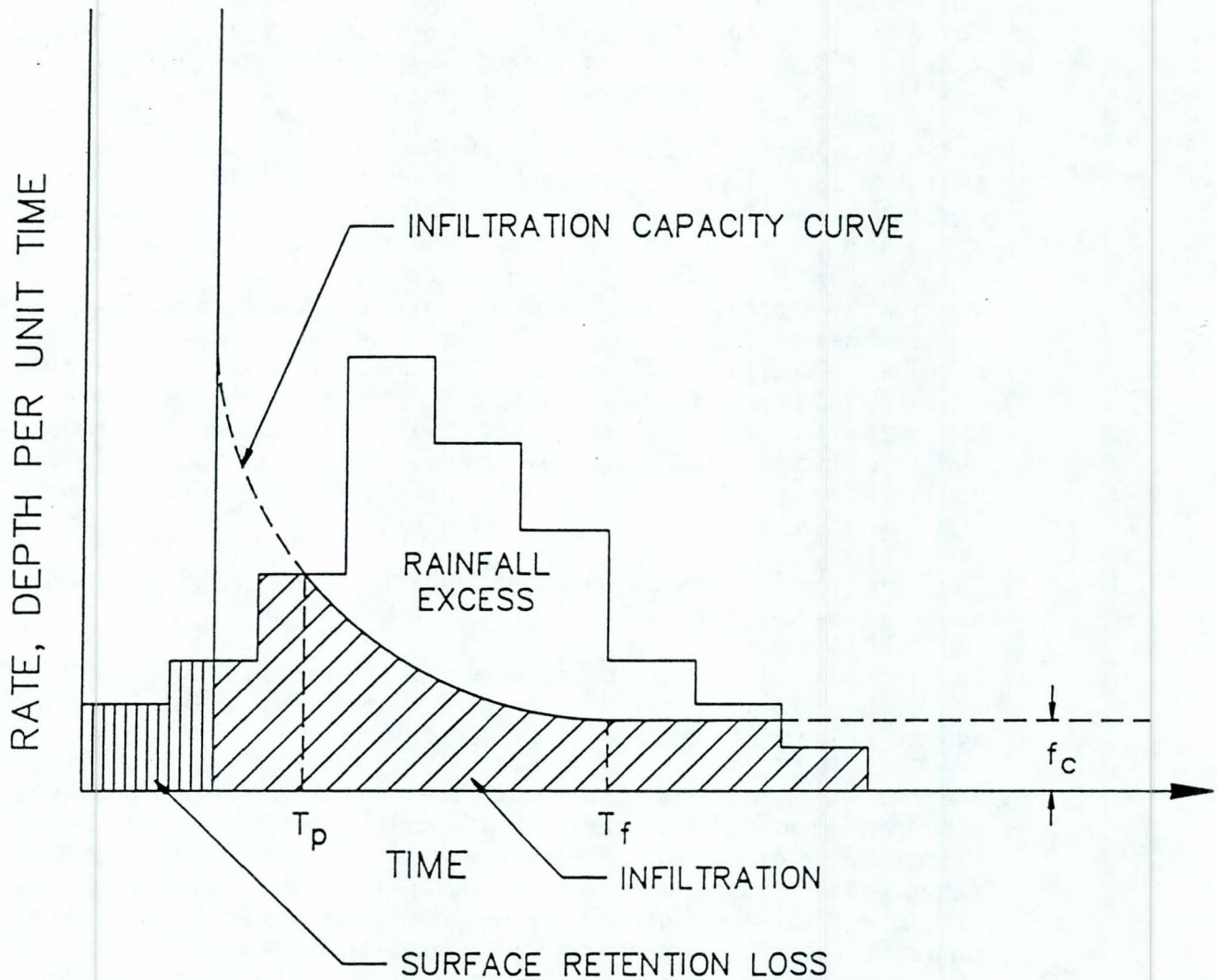

Figure 4.2
Simplified Representation of Rainfall Losses
A Function of Surface Retention Losses Plus Infiltration

4.2 Surface Retention Loss

Surface retention loss, as used herein, is the summation of all rainfall losses other than infiltration. The major component of surface retention loss is depression storage; relatively minor components of surface retention loss are due to interception and evaporation, as previously discussed. Depression storage is considered to occur in two forms. First, in-place depression storage occurs at, and in the near vicinity of, the raindrop impact. The mechanism for this depression storage is the microrelief of the soil and soil cover. The second form of depression storage is the retention of surface runoff that occurs away from the point of raindrop impact in surface depressions such as puddles, roadway gutters and swales, roofs, irrigation bordered fields and lawns, and so forth.

A relatively minor contribution by interception is also considered as a part of the total surface retention loss. Estimates of surface retention loss are difficult to obtain and are a function of the physiography and land-use of the area.

The surface retention loss on impervious surfaces has been estimated to be in the range 0.0625 inch to 0.125 inch by Tholin and Keefer (1960), 0.11 inch for 1 percent slope to 0.06 inch for 2.5 percent slopes by Viessman (1967), and 0.04 inch based on rainfall-runoff data for an urban watershed in Albuquerque by Sabol (1983). Hicks (1944) provides estimates of surface retention losses during intense storms as 0.20 inch for sand, 0.15 inch for loam, and 0.10 inch for clay. Tholin and Keefer (1960) estimated the surface retention loss for turf to be between 0.25 to 0.50 inch. Based on rainfall simulator studies on undeveloped alluvial plains in the Albuquerque area, the surface retention loss was estimated as 0.1 to 0.2 inch (Sabol and others, 1982a). Rainfall simulator studies in New Mexico result in estimates of 0.39 inch for eastern plains rangelands and 0.09 inch for pinon-juniper hillslopes (Sabol and others, 1982b). Surface retention losses for various land-uses and surface cover conditions in Maricopa County have been extrapolated from these reported estimates and these are shown in Table 4.1.

4.3 Infiltration

Infiltration is the movement of water from the land surface into the soil. Gravity and capillary forces drawing water into and through the pore spaces of the soil matrix are the two forces that drive infiltration. Infiltration is controlled by soil properties, by vegetation influences on the soil structure, by surface cover of rock and vegetation, and by tillage practices. The distinction between infiltration and percolation is that percolation is the movement of water through the soil *subsequent to* infiltration.

Table 4.1
Surface Retention Loss for
Various Land Surfaces in Maricopa County

Land-use and/or Surface Cover (1)	Surface Retention Loss IA, inches (2)
Natural	
Desert and rangeland, flat slope	0.35
Hillslopes, Sonoran Desert	0.15
Mountain, with vegetated surface	0.25
Developed (Residential and Commercial)	
Lawn and turf	0.20
Desert landscape	0.10
Pavement	0.05
Agricultural	
Tilled fields and irrigated pasture	0.50

Infiltration can be controlled by percolation if the soil does not have a sustained drainage capacity to provide access for more infiltrated water. However, before percolation can be assumed to restrict infiltration for the design rainfalls being considered in Maricopa County, the extent by which percolation can restrict infiltration of rainfall should be carefully evaluated. SCS soil scientists have defined hydrologic soil group D as:

> "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."

This definition indicates that hydrologic soil groups A, B, or C could be classified as D if a near impervious strata of clay, caliche, or rock is beneath them. When these soils are considered in regard to long-duration rainfalls (the design events for many parts of the United States) this definition may be valid. However, when considered for short-duration and relatively small design rainfall depths in Maricopa County, this definition could result in underestimation of the rainfall losses. This is because even a relatively shallow horizon of soil overlaying an impervious layer still has the ability to store a significant amount of infiltrated rainfall.

For example, consider the situation where only 4 inches of soil covers an impervious layer. If the effective porosity is 0.30, then 1.2 inches (4 inches x 0.30) of water can be infiltrated and stored in the shallow soil horizon. For design rainfalls in Maricopa County, this represents a significant storage volume for infiltrated rainfall and so when ~~using drainage studies for~~ Maricopa County that contain significant areas classified as hydrologic soil group D, the reason for that classification should be determined.

developing loss rate parameters for areas of

Hydrologic soil group D should be retained only for:

» clay soils,

» soils with a permanent high water table, and

» rock outcrop.

Hydrologic soil group D should probably *not* be retained in all situations where the classification is based on shallow soils over nearly impervious layers; site specific studies and sensitivity analyses should be performed to estimate the loss rates to be used for such soils.

4.4 Recommended Methods for Estimating Rainfall Losses

Many methods have been developed for estimating rainfall losses; five are listed as options in the HEC-1 Flood Hydrology Package. They are:

1. Holtan Infiltration Equation
2. Exponential Loss Rate
3. SCS Curve Numbers (CN) Loss Rate
4. Green and Ampt Infiltration Equation
5. Initial Loss Plus Uniform Loss Rate (IL+ULR)

Of these five, however, only two—Green and Ampt and IL+ULR—are recommended for estimating rainfall losses in Maricopa County for the reasons discussed below.

The **Holtan Infiltration Equation** is an exponential decay type of equation for which the rainfall loss rate asymptotically diminishes to the minimum infiltration rate (fc). The Holtan equation is not extensively used and there is no known application of this method in Arizona. Data and procedures to estimate the parameters for use in Maricopa County are not available. Therefore, the Holtan equation is not recommended for general use in Maricopa County.

The **Exponential Loss Rate Method** is a four parameter method that is not extensively used, but it is a method preferred by of the U.S. Army Corps of Engineers. Data and procedures are not available to estimate the parameters for this method for all physiographic regions in Maricopa County, but Exponential loss rate parameters have been developed from the reconstitution of flood events for a flood hydrology study in a portion of Maricopa County (U.S. Army Corps of Engineers, 1982). However, adequate data is not available to estimate the necessary parameters

for all soil types and land uses in Maricopa County, and this method is not recommended for general use in Maricopa County.

The **SCS CN** method is the most extensively used rainfall loss rate method in Maricopa County and Arizona and it has wide acceptance among many agencies, consulting engineering firms, and individuals throughout the community. ~~This method is limited, however, by both theoretical and practical deficiencies, and thus is not recommended for general use in Maricopa County. Deficiencies of the SCS CN method include:~~

However, because of both theoretical concerns and practical limitations, the SCS CN method is not recommended for general use in Maricopa County.

~~This is contrary to the generally accepted infiltration relation as illustrated in Figure 4.2.~~

3. ~~The infiltration rate approaches zero rather than a minimum infiltration rate (fc).~~

4. ~~The initial abstraction is equal to 0.2S~~

   ~~where S = 1000/CN - 10~~

   ~~This equation is not theoretically justified nor is it based on data for hydrologic conditions that are representative of Maricopa County.~~

5. ~~The selection of CN is too subjective and is often based more on traditional acceptance of CN values rather than on scientifically substantiated findings.~~

6. ~~At low rainfalls (less than 4 inches), the estimate of rainfall loss is very sensitive to the selection of CN.~~

¶ As mentioned previously, the two recommended methods for estimating rainfall losses in Maricopa County are the Green and Ampt infiltration equation and the initial loss and uniform loss rate (IL+ULR) method. Both methods, as programmed into HEC-1, can be used to simulate the rainfall loss model as depicted in Figure 4.2. (For a full discussion of these methods, see Sections 4.4.1 and 4.4.2.) The IL+ULR is a simplified model that has been used extensively for flood hydrology and data are ~~is~~ available to estimate the two parameters for this method. The Green and Ampt infiltration equation is a physically based model that has been in existence since 1911, and has recently been incorporated as an option in HEC-1.

Procedures have been developed to estimate the three parameters of the Green and Ampt infiltration equation. The preferred method, and the most theoretically accurate, is the Green and Ampt infiltration equation. The IL+ULR is recommended as an alternative if it is not possible to estimate the Green and Ampt equation

parameters, or for other valid reasons. It should be realized, as explained later, that the use of the Green and Ampt equation and parameters, as defined herein, will probably result in lower peak discharges and runoff volumes than the use of the IL+ULR.

Other methods should be used only if there is technical justification for a variance from this recommendation and if adequate information is available to estimate the necessary parameters. Use of rainfall loss methods other than those recommended should not be undertaken unless previously approved by the Flood Control District and the local regulatory agency.

4.4.1 Green and Ampt Infiltration Equation

This model, first developed in 1911 by W.H. Green and G.A. Ampt, has since the early 1970s, received increased interest for estimating rainfall infiltration losses. The model has the form:

$$f = K_S\,(1 + \frac{\psi\theta}{F}) \qquad \text{for } f < i \qquad (1)$$

$$f = i \qquad \text{for } f \geq i$$

where

f	=	infiltration rate (L/T),
i	=	rainfall intensity (L/T),
K_S	=	hydraulic conductivity, wetted zone, steady-state rate (L/T)
Ψ	=	average capillary suction in the wetted zone (L),
θ	=	soil moisture deficit (dimensionless), equal to effective soil porosity times the difference in final and initial volumetric soil saturations, and
F	=	depth of rainfall that has infiltrated into the soil since the beginning of rainfall (L).

A sound and concise explanation of the Green and Ampt equation is provided by Bedient and Huber (1988).

It is important to note that as rain continues, F increases and f approaches K_S, and therefore, f is inversely related to time. Equation 1 is implicit with respect to f which causes computational difficulties. Eggert (1976) simplified Equation 1 by expanding the equation in a power series and truncating all but the first two terms of the expansion. The simplified solution (Li and others, 1976) is:

$$F = -0.5\,(2F - K_S\,\Delta t) + 0.5\,[(2F - K_S\,\Delta t)^2 + 8K_S\Delta t\,(\theta\psi + F)]^{1/2} \qquad (2)$$

where

Δt = the computation interval

F = accumulated depth of infiltration at the start of Δt.

The average infiltration rate is:

$$f = \frac{\Delta F}{\Delta t} \tag{3}$$

Use of the Green and Ampt equation as coded in HEC-1 involves the simulation of rainfall loss as a two phase process, as illustrated in Figure 4.2. The first phase is the simulation of the surface retention loss as previously described; this loss is called the initial loss (IA) in HEC-1. During this first phase, all rainfall is lost (zero rainfall excess generated) during the period from the start of rainfall up to the time that the accumulated rainfall equals the value of IA. It is assumed, for modeling purposes, that no infiltration of rainfall occurs during this first phase. Initial loss (IA) is primarily a function of land-use and surface cover, and recommended values of IA for use with the Green and Ampt equation are presented in Table 4.1. For example, about 0.35 inches of rainfall will be lost to runoff due to surface retention for desert and rangelands on relatively flat slopes in Maricopa County.

The second phase of the rainfall loss process is the infiltration of rainfall into the soil matrix. For modeling purposes, the infiltration begins immediately after the surface retention loss (IA) is completely satisfied, as illustrated in Figure 4.2. The three Green and Ampt equation infiltration parameters as coded in HEC-1 are:

» hydraulic conductivity at natural saturation (XKSAT) equal to K_S in Equation 1;

» wetting front capillary suction (PSIF) equal to Ψ in Equation 1; and

» volumetric soil moisture deficit at the start of rainfall (DTHETA) equal to θ in Equation 1.

The three infiltration parameters are functions of soil characteristics, ground surface characteristics, and land management practices. The soil characteristics of interest are particle size distribution (soil texture), organic matter, and bulk density. The primary soil surface characteristics are vegetation canopy cover, ground cover, and soil crusting. The land management practices are identified as various tillages as they result in changes to soil porosity.

Values of Green and Ampt equation parameters as a function of soil characteristics alone (bare ground condition) have been obtained from published reports (Rawls and others, 1983; Rawls and Brakensiek, 1983). and Average values of XKSAT and PSIF for each of the soil texture classes ~~from Rawls and Brakensiek (1983)~~ are shown in Columns (2) and (3) of Table 4.2. ~~Values of XKSAT and PSIF (as a function of percent of sand and percent of clay for soil with 0.5 percent organic matter and base value [unaltered] soil porosity) are shown in Figures 4.3 and 4.4, respectively (Rawls and Brakensiek, 1983).~~ The values of XKSAT and PSIF from Table 4.2 should be used if

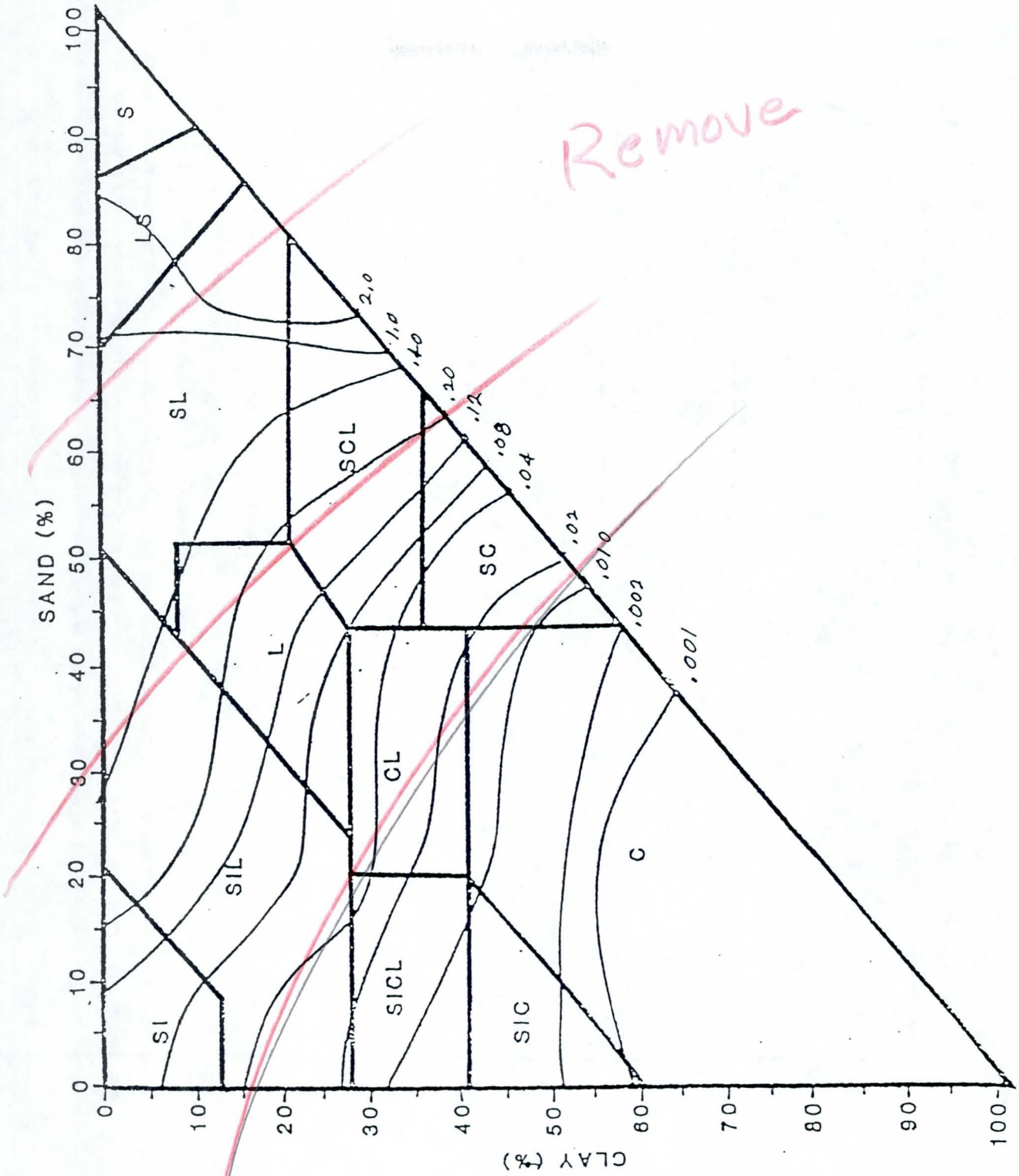

Figure 4.3
Green and Ampt Loss Rate
Hydraulic Conductivity at natural saturation in inches/hour
XKSAT (0.5% organic matter)

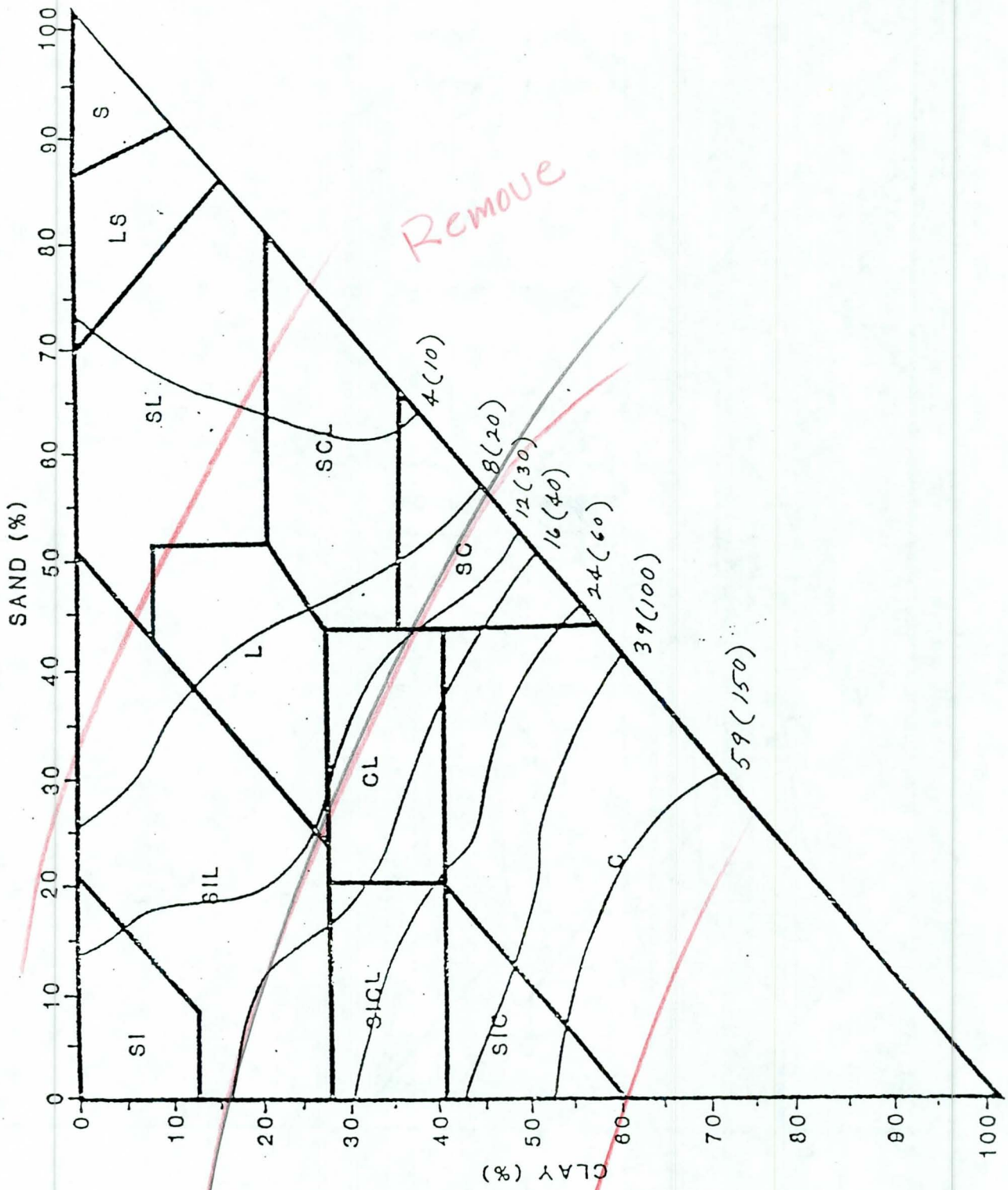

Figure 4.4
Green and Ampt Loss Rate
Wetting Front Suction in inches (cm)
PSIF

general soil texture classification of the drainage area is available. The values of XKSAT and PSIF from Figures 4.3 and 4.4 can be used if more specific soil texture classification is available from a detailed soil survey for which the percentages of sand and clay have been determined by an appropriate field soil survey. The use of the information in Figures 4.3 and 4.4 will require an extensive study of the soil for the drainage area, ~~and~~ For most drainage studies only general soil texture classification will be known so the values from Table 4.2 should be used. References used to create Table 4.2 can be found in the Documentation Manual.

The soil moisture deficit (DTHETA) is a volumetric measure of the soil moisture storage capacity that is available at the start of the rainfall. DTHETA is a function of the effective porosity of the soil. The range of DTHETA is 0.0 to the effective porosity. If the soil is effectively saturated at the start of rainfall then DTHETA equals 0.0; if the soil is devoid of moisture at the start of rainfall the DTHETA equals the effective porosity of the soil. The porosity of soil as a function of soil texture (percent of sand and percent of clay) is shown in Figure 4.5 (Brakensiek and others, 1984).

Under natural conditions, soil seldom reaches a state of soil moisture less than the wilting point of vegetation. Figure 4.6 is a graph of volumetric soil moisture at wilting point as a function of soil texture. Due to the rapid drainage capacity of most soils in Maricopa County, at the start of a design storm the soil would not be expected to be in a state of soil moisture greater than the field capacity. Figure 4.7 is a graph of volumetric soil moisture at field capacity as a function of soil texture.

Table 4.2
Green and Ampt Loss Rate Parameter Values for Bare Ground

Soil Texture Classification (1)	XKSAT inches/hour (2)	PSIF inches (3)	DTHETA[1]		
			Dry (4)	Normal (5)	Saturated (6)
sand	4.6	1.9	0.35	0.30	0
loamy sand	1.2	2.4	0.35	0.30	0
sandy loam	0.40	4.3 ~~3.5~~	0.35	0.25	0
loam	0.25	3.5 ~~4.3~~	0.35	0.25	0
silty loam	0.15	6.6	0.40	0.25	0
silt	0.10	7.5	0.35	0.15	0
sandy clay loam	0.06	8.6	0.25	0.15	0
clay loam	0.04	8.2	0.25	0.15	0
silty clay loam	0.04	10.8	0.30	0.15	0
sandy clay	0.02	9.4	0.20	0.10	0
silty clay	0.02	11.5	0.20	0.10	0
clay	0.01	12.4	0.15	0.05	0

[1] Selection of DTHETA:

Dry = Nonirrigated lands, such as desert and rangeland;
Normal = Irrigated lawn, turf, and permanent pasture;
Saturated = Irrigated agricultural land.

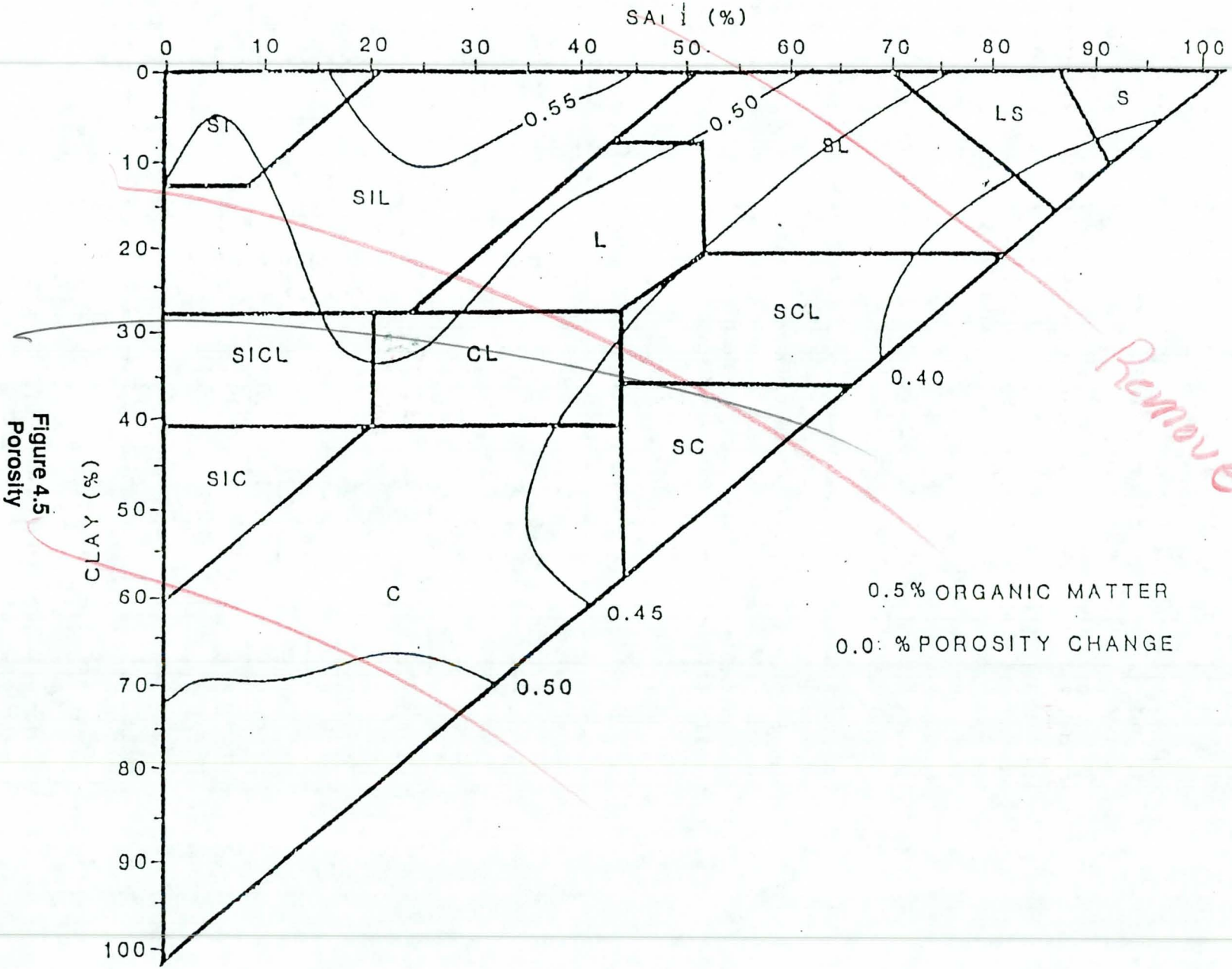

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Figure 4.5
Porosity

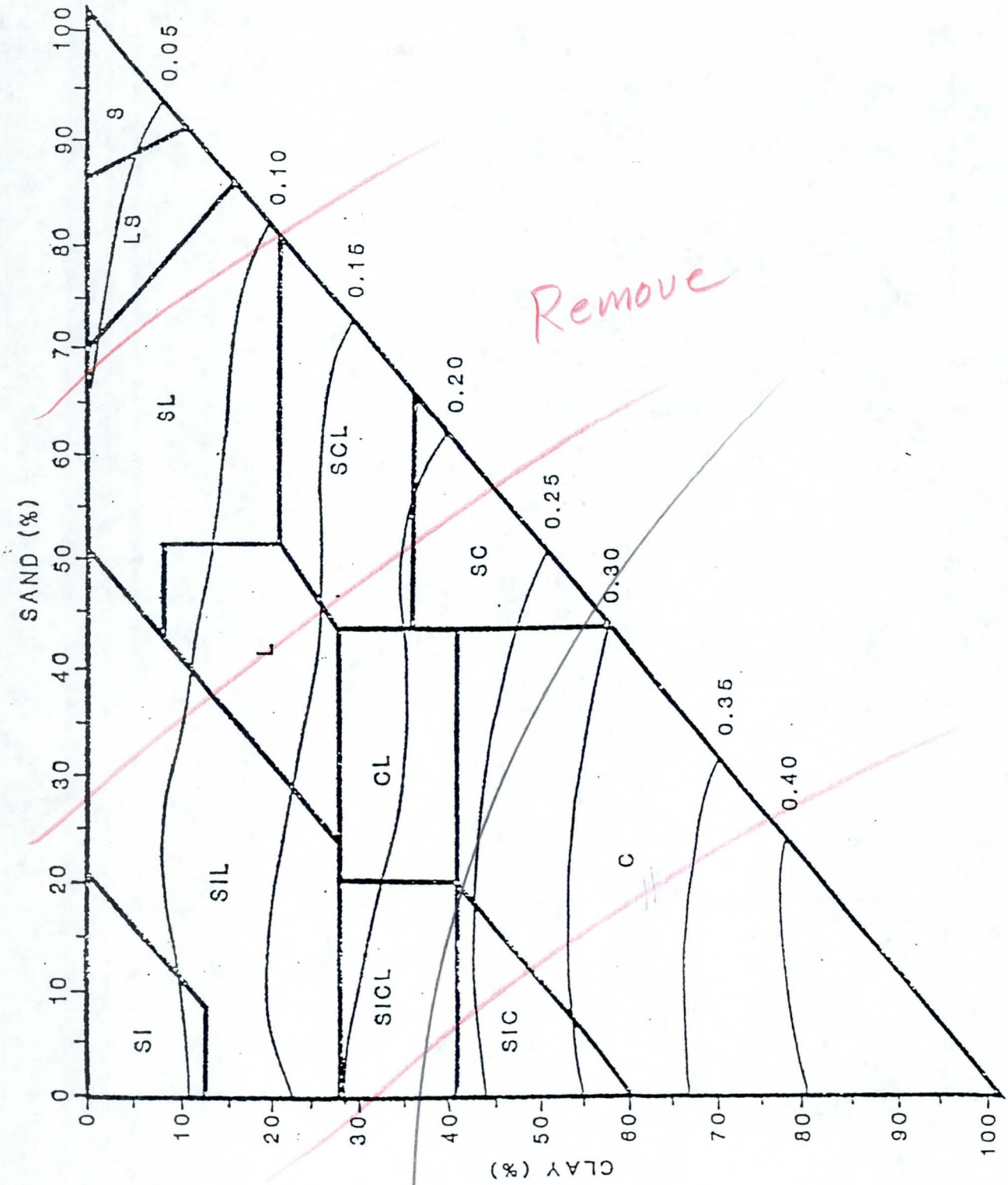


Figure 4.6
Green and Ampt Loss Rate
15-Bar Soil Moisture by Volume (assumed wilting point)

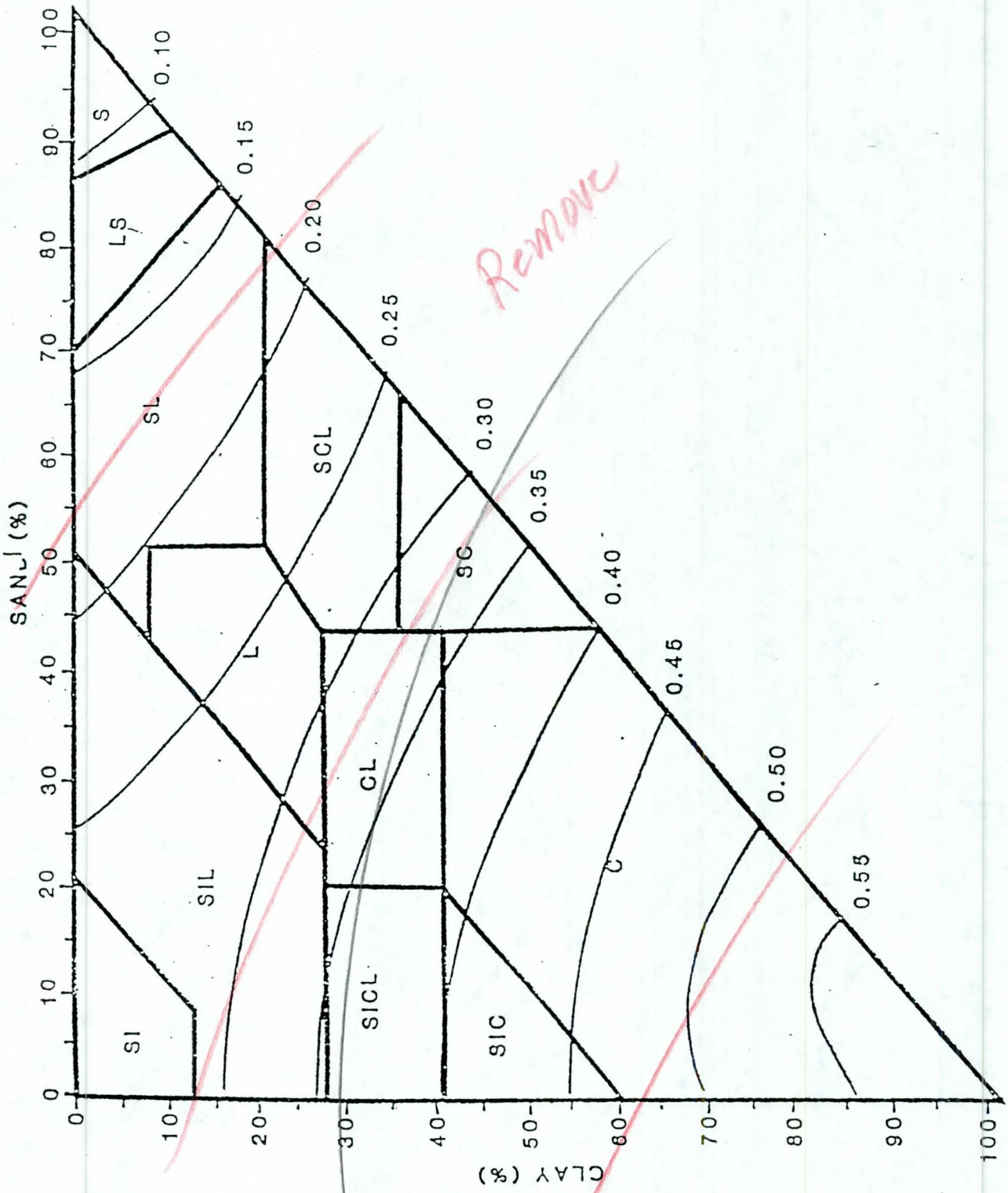


Figure 4.7
Green and Ampt Loss Rate
1/3 Bar Soil Moisture by Volume (assumed field capacity)

However, Maricopa County also has a large segment of its land area under irrigated agriculture, and it is reasonable to assume that the design frequency storm could occur during or shortly after certain lands have been irrigated. Therefore, it would be reasonable to assume that soil moisture for irrigated lands could be at or near effective saturation during the start of the design rainfall.

Three conditions for DTHETA have been defined for use in Maricopa County based on the antecedent soil moisture condition that could be expected to exist at the start of the design rainfall. These three conditions are:

» "Dry" for antecedent soil moisture near the vegetation wilting point;

» "Normal" for antecedent soil moisture condition near field capacity due to previous rainfall or irrigation applications on nonagricultural lands; and

» "Saturated" for antecedent soil moisture near effective saturation due to recent irrigation of agricultural lands.

Values of DTHETA have been estimated by subtracting the initial volumetric soil moisture for each of the three conditions from the soil porosity.

The value of DTHETA "Dry" as a function of soil texture is shown in Figure 4.8. This figure was prepared by subtracting the wilting point soil moisture in Figure 4.6 from the soil porosity in Figure 4.5. The value of DTHETA "Normal" as a function of soil texture is shown in Figure 4.9. This figure was prepared by subtracting the field capacity soil moisture in Figure 4.7 from the soil porosity in Figure 4.5. The value of DTHETA "Saturated" is always equal to 0.0 because for this condition there is no available pore space in the soil matrix at the start of rainfall. Values of DTHETA for the three antecedent soil moisture conditions are shown in Table 4.2. DTHETA "Dry" should be used for soil that is usually in a state of low soil moisture such as would occur in the desert and rangelands of Maricopa County. DTHETA "Normal" should be used for soil that is usually in a state of moderate soil moisture such as would occur in irrigated lawns, golf courses, parks, and irrigated pastures. DTHETA "Saturated" should be used for soil that can be expected to be in a state of high soil moisture such as irrigated agricultural land.

The hydraulic conductivity (XKSAT) can be affected by several factors besides soil texture. For example, hydraulic conductivity is reduced by soil crusting, increased by tillage, and increased by the influence of ground cover and canopy cover. The values of XKSAT that have been presented for bare ground as a function of soil texture alone should be adjusted under certain soil cover conditions.

Ground cover, such as grass, litter, and rock will generally increase the infiltration rate over that of bare ground conditions. Similarly, canopy cover—such as from trees, brush, and tall grasses—can also increase the bare ground infiltration rate. The procedures and data that have been presented are for estimating the Green and Ampt parameters based solely on soil texture and would be applicable for bare ground conditions. Past research has shown that the wetting front capillary suction

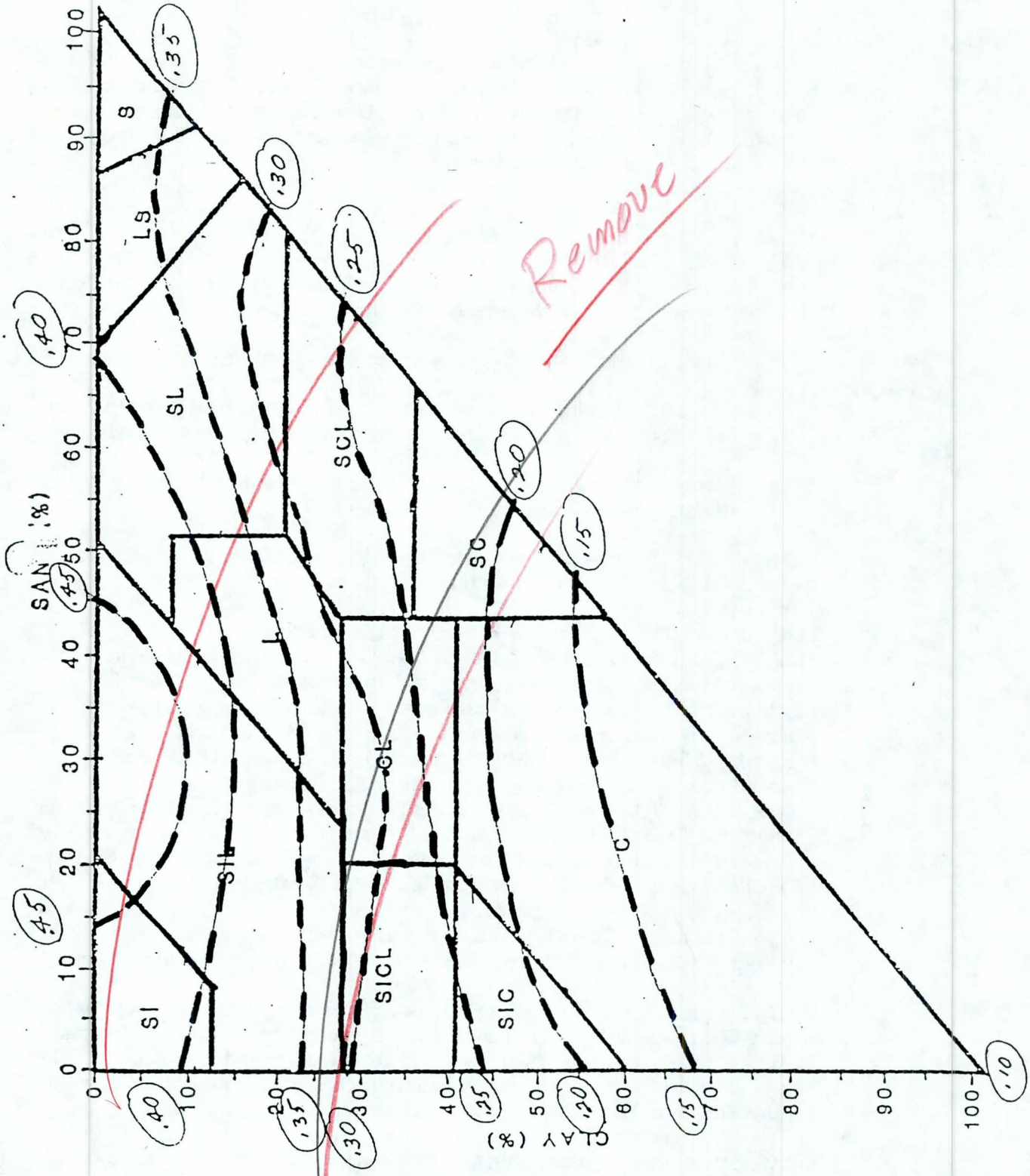


Figure 4.8
DTHETA "Dry"
for Initially Dry Soil Moisture Condition (wilting point)

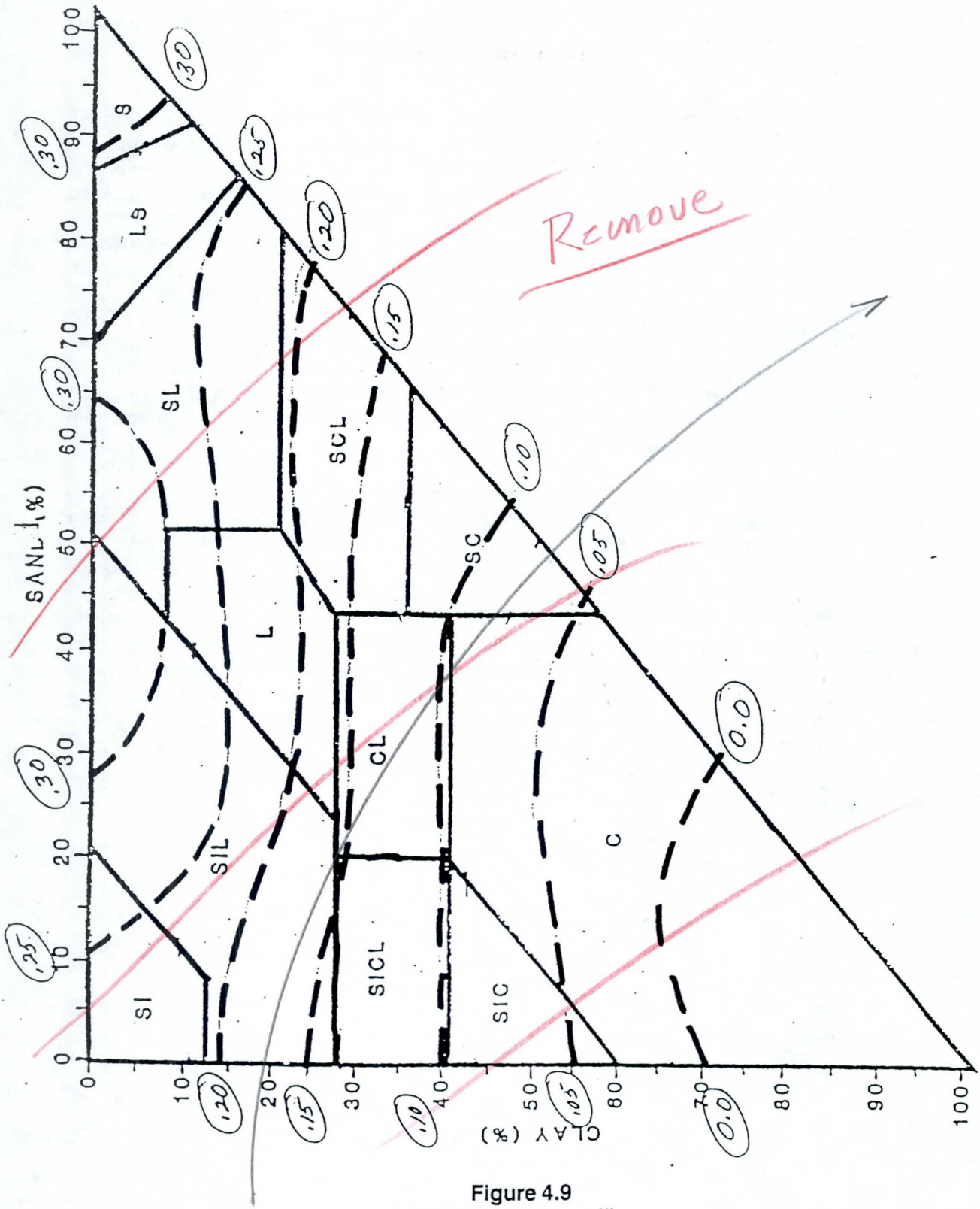


Figure 4.9
DTHETA "Normal"
for Initially Wet Soil Moisture Condition (field capacity)

parameter (PSIF) is relatively insensitive in comparison with the hydraulic conductivity parameter (XKSAT); therefore only the hydraulic conductivity parameter is adjusted for the influences of cover over bare ground.

Procedures have been developed (Rawls and others, 1988) for incorporating the effects of soil crusting, ground cover, and canopy cover into the estimation of hydraulic conductivity for the Green and Ampt equation; however, those procedures are not recommended for use in Maricopa County at this time. A simplified procedure to adjust the bare ground hydraulic conductivity for vegetation cover is shown in Figure 4.~~10~~ 3. This figure is based on the documented increase in hydraulic conductivity due to various soil covers as reported by investigators using rainfall simulators on native western rangelands (Kincaid and others, 1964; Sabol and others, 1982a; Sabol and others, 1982b; Bach, 1984; Ward, 1986; Lane and others, 1987; Ward and Bolin, 1989). This correction factor can be used based on an estimate of vegetation cover as used by the Soil Conservation Service in soil surveys; that is, vegetation cover is evaluated on basal area for grasses and forbs, and is evaluated on canopy cover for trees and shrubs. Note that this correction can be applied only to soils other than sand and ~~sandy loam~~ loamy sand.

F4.3

The influence of tillage results in a change in total porosity and therefore a need to modify the three Green and Ampt equation infiltration parameters. The effect of tillage systems on soil porosity and the corresponding changes to hydraulic conductivity, wetting front capillary suction, and water retention is available (Rawls and Brakensiek, 1983). Although this information is available, it is not presented in this manual, nor is it recommended that these adjustments be made to the infiltration parameters for design purpose use in Maricopa County, because for most flood estimation ~~prediction~~ purposes it cannot be assumed that the soil will be in any particular state of tillage at the time of storm occurrence and therefore the base condition infiltration parameters, as presented, should be used for flood estimation ~~prediction~~ purposes. However, appropriate adjustments to the infiltration parameters can be made, as necessary, for special flood studies such as reconstitution of storm events.

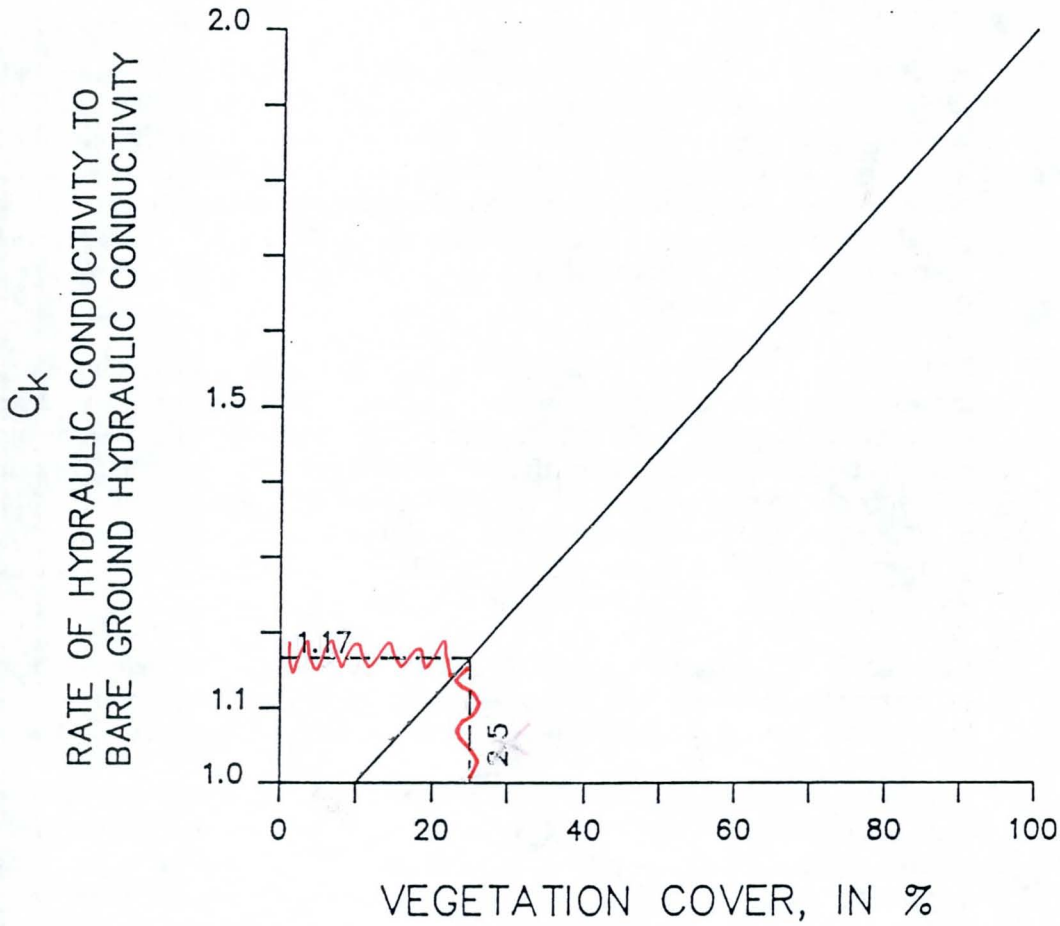


Figure 4.~~10~~ 3
Effect of Vegetation Cover on Hydraulic Conductivity
For Hydraulic Soil Groups B, C, and D, and
For all Soil Textures *other than*
Sand and ~~Sandy Loam~~
Loamy Sand

4.4.2 Initial Loss Plus Uniform Loss Rate (IL+ULR)

This is a simplified rainfall loss method that is often used, and generally accepted, for flood hydrology. In using this simplified method it is assumed that the rainfall loss process can be simulated as a two-step procedure, as illustrated in Figure 4.11. First, all rainfall is lost to runoff until the accumulated rainfall is equal to the initial loss; and second, after the initial loss is satisfied, a portion of all future rainfall is lost at a uniform rate. <F4.4

According to HEC-1 nomenclature, two parameters are needed to use this method; the initial loss (STRTL) and the uniform loss rate (CNSTL). The initial loss (STRTL) is the sum of all losses prior to the onset of runoff and is made up of surface retention loss (IA) and an initial amount of infiltration (IL); therefore, STRTL = IA + IL. Values of the infiltration component (IL) of STRTL for bare ground according to soil texture classification are shown in Columns (3) through (5) in Table 4.3. These values have been derived from the Green and Ampt infiltration equation and parameter values that are shown in Table 4.2. <T4.3

The value of IL "Dry" should be used for soil that is usually in a state of low soil moisture at or near the wilting point for vegetation. This is a reasonable assumption for most nonirrigated lands in Maricopa County because of the infrequency of rainfall and because of the rapid drainage of these soils after rainfall. The value of IL "Normal" should be used for soil that is usually in a state of moderate soil moisture such as occurs for irrigated lawns, turf, and permanent pastures. The value of IL "Saturated" is used for a soil maintained in a state of high soil moisture, such as in irrigated agricultural lands.

Values of IL for bare ground that have been classified according to hydrologic soil group are shown in Table 4.4. These values within each hydrologic soil group have been derived from the data in Table 4.3 for the various soil texture classifications. <T4.4

The uniform loss rate (CNSTL) represents the long-term, equilibrium infiltration capacity of the soil. The values of CNSTL shown in Column (2) of Table 4.3 for soils according to soil texture ~~classification~~ classified are equivalent to the hydraulic conductivity at natural saturation (XKSAT) as determined for the Green and Ampt equation (Table 4.2). The values of CNSTL for soils classified according to hydrologic soil groups are shown in Table 4.~~2~~4. These values within each hydrologic soil group have been selected from inspection of XKSAT values in Table 4.2 for the various soil texture classifications. Values of CNSTL shown in Table 4.4 are consistent with general information available for estimating CNSTL as shown in Table 4.5. <T4.5

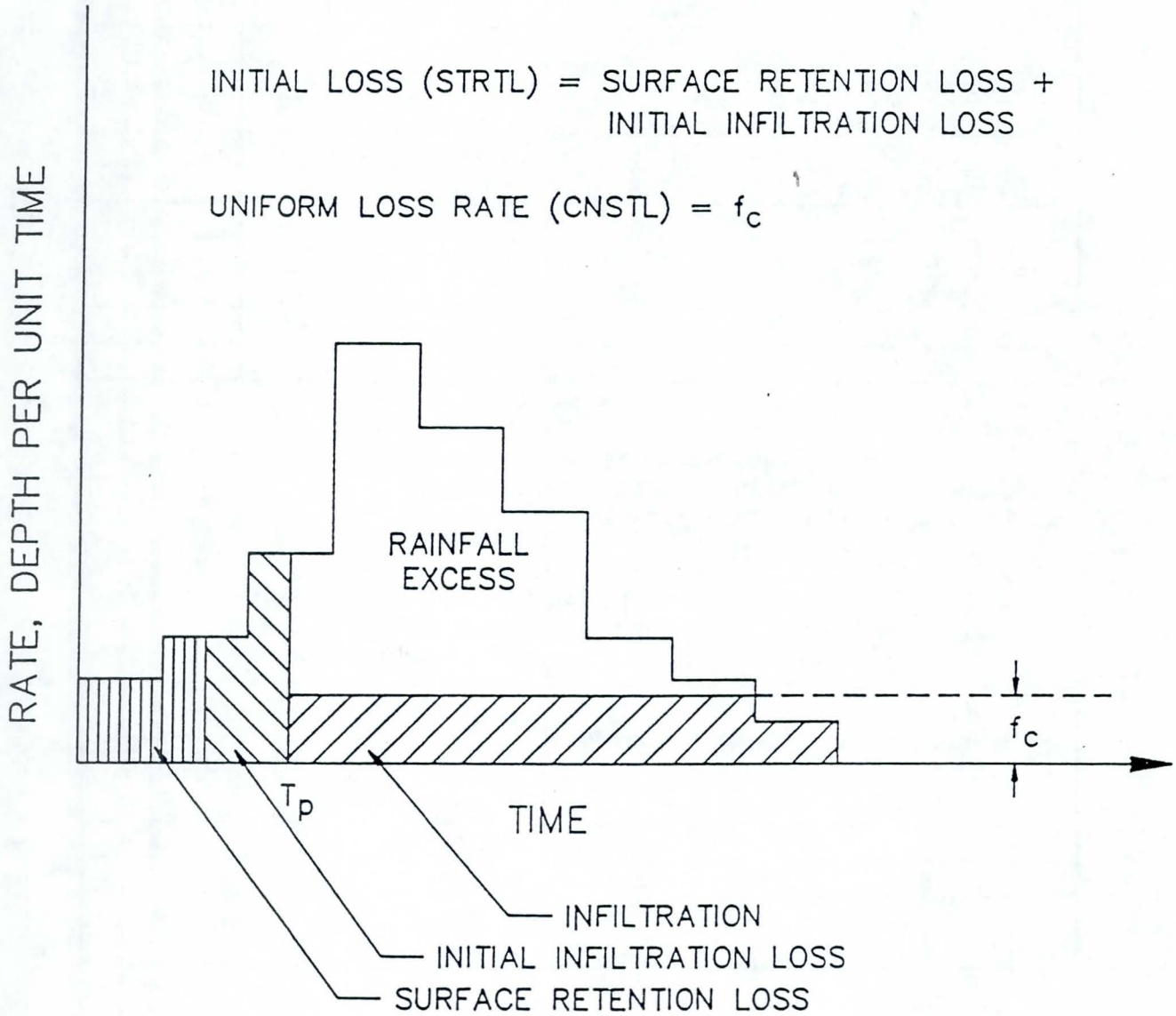


Figure 4.~~11~~4
Representation of Rainfall Loss According to the Initial Loss Plus Uniform Loss Rate (IL+ULR)

Table 4.3
Initial Loss Plus Uniform Loss Rate Parameter Values
for Bare Ground according to Soil Texture Classification

Soil Texture Classification (1)	Uniform Loss Rate CNSTL (2)	Initial Loss, inches IL[1]		
		Dry (4)	Normal (5)	Saturated (6)
sand	4.6	1.3	1.3	0
loamy sand	1.2	0.8	0.8	0
sandy loam	0.40	0.7	0.6	0
loam	0.25	0.8	0.7	0
SILT> silty loam	0.10> 0.15	0.6> 0.6	0.5> 0.5	0> 0
sandy clay loam	0.06	0.6	0.5	0
clay loam	0.04	0.5	0.4	0
silty clay loam	0.04	0.6	0.5	0
sandy clay	0.02	0.4	0.3	0
silty clay	0.02	0.4	0.3	0
clay	0.01	0.3	0.2	0

[1] Selection of IL:
Dry = Nonirrigated lands such as desert and rangeland;
Normal = Irrigated lawn, turf, and permanent pasture;
Saturated = Irrigated agricultural land.

Table 4.4
Initial Loss Plus Uniform Loss Rate Parameter Values
for Bare Ground according to Hydrologic Soil Group

Hydrologic Soil Group (1)	Uniform Loss Rate CNSTL (2)	Initial Loss, inches IL[1]		
		Dry ~~(4)~~ 3	Normal ~~(5)~~ 4	Saturated ~~(6)~~ 5
A	0.4	0.6	0.5	0
B	0.25	0.5	0.3	0
C	0.15	0.5	0.3	0
D	0.05	0.4	0.2	0

[1] Selection of IL:
Dry = Nonirrigated lands such as desert and rangeland;
Normal = Irrigated lawn, turf, and permanent pasture;
Saturated = Irrigated agricultural land.

Table 4.5
Published Values of Uniform Loss Rates

Hydrologic Soil Group (1)	Uniform Loss Rate, Inches/hour		
	Musgrave (1955) (2)	USBR (1975)[1] (3)	USBR (1988)[2] (4)
A	0.30 - 0.45	0.40	0.30 - 0.50
B	0.15 - 0.30	0.24	0.15 - 0.30
C	0.05 - 0.15	0.12	0.05 - 0.15
D	0 - 0.05	0.08	0 - 0.05

[1] *Design of Small Dams*, Second Edition, 1975, Appendix A
[2] *Design of Small Dams*, Third Edition, 1988

4.5 Procedure for Estimating Loss Rates

4.5.1 Green and Ampt Method

A. When soils data are available:

1. Determine the soil texture classification. Soils reports such as those of the Soil Conservation Service can be used if available, or laboratory analysis of appropriate soil samples from the drainage area can be used if adequate documentation on the sampling and laboratory procedure is provided and approved.

2. Estimate the hydraulic conductivity (XKSAT) for bare ground from Table 4.2 if general soil texture classification is available. ~~or from Figure 4.3 if adequate soil texture data is available from an approved sampling program.~~

3. If ~~desired~~ appropriate, adjust the value of XKSAT for the influences of vegetation cover using Figure 4.10.

4. Estimate the wetting front capillary suction parameter (PSIF) from Table 4.2 if general soil texture classification is available. ~~or from Figure 4.4 if adequate soil texture data is available from an approved sampling program.~~ Use Fig. 4.5 if a weighted XKSAT value is calculated. <F4.5

5. Estimate the value of DTHETA from Table 4.2 if general soil texture classification is available. ~~or from either Figure 4.8 or 4.9 if adequate soil texture data is available from an approved sampling program.~~ The value of DTHETA must be selected based on the appropriate antecedent soil moisture condition: "Dry" for nonirrigated lands such as desert and rangeland; "Normal" for soil that would be expected to be near soil moisture field capacity such as irrigated lawn, turf, and permanent pasture; and, "Saturated" for irrigated agricultural land.

6. Determine the land-use and/or soil cover for the drainage area and use Table 4.1 to estimate the surface retention loss (IA).

B. Alternative methods:

As an alternative to the above procedures, Green and Ampt loss rate parameters can be estimated by reconstitution of recorded rainfall-runoff events on the drainage area or hydrologically similar watersheds, or parameters can be estimated by use of rainfall simulators in field experiments. Plans and procedures for estimating Green and Ampt loss rate parameters by either of these procedures should be approved by the Flood Control District and the local agency before initiating these procedures.

4.5.2 Initial Loss Plus Uniform Loss Rate Method

A. When soils data are available:

1. Determine the soil texture classification and/or the hydrologic soil group. Soils reports such as those of the Soil Conservation Service can be used if available, or laboratory analysis of appropriate soil samples from the drainage area can be used to classify the soil if adequate documentation on the sampling and laboratory procedure is provided and approved.

2. Use values of CNSTL and IL from Table 4.3 if the losses are to be based on soil texture classification.

3. Use values of CNSTL and IL from Table 4.4 if the losses are to be based on hydrologic soil group.

4. Determine the land-use and/or soil cover and use Table 4.1 to estimate the surface retention loss (IA).

5. STRTL = IA + IL.

5

Unit Hydrograph Procedures

5.1 General

Rainfall excess can be routed from a watershed to produce a storm discharge hydrograph at a downstream location (concentration point) by one of two methods: 1) hydraulic routing involving the complete or some simplified form of the equations of motion (i.e., the momentum equation plus the continuity equation); or 2) hydrologic routing involving the application of the continuity equation. Kinematic wave routing, as available in HEC-1, is an example of simplified hydraulic routing. Hydrologic routing is usually accomplished by either direct application of the equation of continuity (Equation 1), or a graphical procedure such as the application of the principles of the unit hydrograph.

$$I - O = \frac{dS}{dt} \tag{1}$$

Examples of hydrologic routing by direct application of the equation of continuity are the Clark Unit Hydrograph (Clark, 1945), the Santa Barbara Urban Hydrograph (Stubchaer, 1975), and the Single Linear Reservoir Model (Pedersen and others, 1980). Both the Santa Barbara Urban Hydrograph and the Single Linear Reservoir Model are simplified (one parameter) versions of the Clark Unit Hydrograph (three parameter) procedure (Sabol and Ward, 1985). Examples of unit hydrographs that require a graphical procedure are the SCS Dimensionless Unit Hydrograph, Snyder's Unit Hydrograph, S-graphs, and unit hydrographs that are derived directly from recorded runoff data. Graphical or tabular methods of routing rainfall excess by unit hydrographs are very amenable to hand-calculation methods commonly used before computers became readily available. Direct mathematical solution of the equation of continuity, such as the Clark Unit Hydrograph, is more efficiently conducted with computers and appropriate computer programs.

The recommended procedures for routing rainfall excess in Maricopa County are either the Clark Unit Hydrograph or the application of selected S-graphs. The Clark and these two methods are collectively referred to as the Maricopa County Unit Hydrograph Procedure (MCUHP).

Unit Hydrograph procedure, as described herein, is recommended for watersheds or subbasins less than about 5 square miles in size with an upper limit of application of 10 square miles. The application of S-graphs is recommended for use with major watercourses in Maricopa County.

A unit hydrograph is a graph of the time distribution of runoff from a specific watershed as the result of one inch of rainfall excess that is distributed uniformly over the watershed and that is produced during a specified time period (duration). The duration of rainfall excess is not generally equal to the rainfall duration, because a unit hydrograph is derived from or is to be representative of a specific watershed. A unit hydrograph is a lumped parameter and reflects all of the physical characteristics of the watershed that will affect the time rate at which rainfall excess will drain from the land surface.

The principles of the unit hydrograph were introduced by Sherman (1932) who observed that for a watershed all hydrographs resulting from a rain of the same duration have the same time base, and that ordinates of each storm hydrograph from the watershed are proportional to the volume of runoff if the time and areal distributions of the rainfalls are similar. The principles that are applied when using a unit hydrograph are:

1. For a watershed, hydrograph base lengths are equal for rainfall excesses of equal duration.

2. Hydrograph ordinates are proportional to the amount of rainfall excess.

3. A storm hydrograph can be developed by linear superposition of incremental hydrographs.

Application of these principles requires a linear relation between watershed outflow and storage within the watershed, $S = KO$. However, Mitchell (1962) has shown that nonlinear storage, $S = KO^x$, is a condition that occasionally occurs in natural watersheds. A method has been developed by Shen (1962) to evaluate the linearity of the storage-outflow relation for gaged watersheds. Mitchell (1972) developed the model hydrograph for use in watersheds that have nonlinear storage-outflow characteristics. Presently no method has been devised to evaluate the linearity of an ungaged watershed, and the assumption of linearity is a practical necessity in virtually all cases.

5.2 Clark Unit Hydrograph

Hydrologic routing by the Clark Unit Hydrograph method is analogous to the routing of an inflow hydrograph through a reservoir. This analogy is illustrated in Figure 5.1. The inflow hydrograph, called the translation hydrograph in the Clark method, is determined from the temporal and spatial distribution of rainfall excess over the watershed. The translation hydrograph is then routed by a form of the equation of continuity:

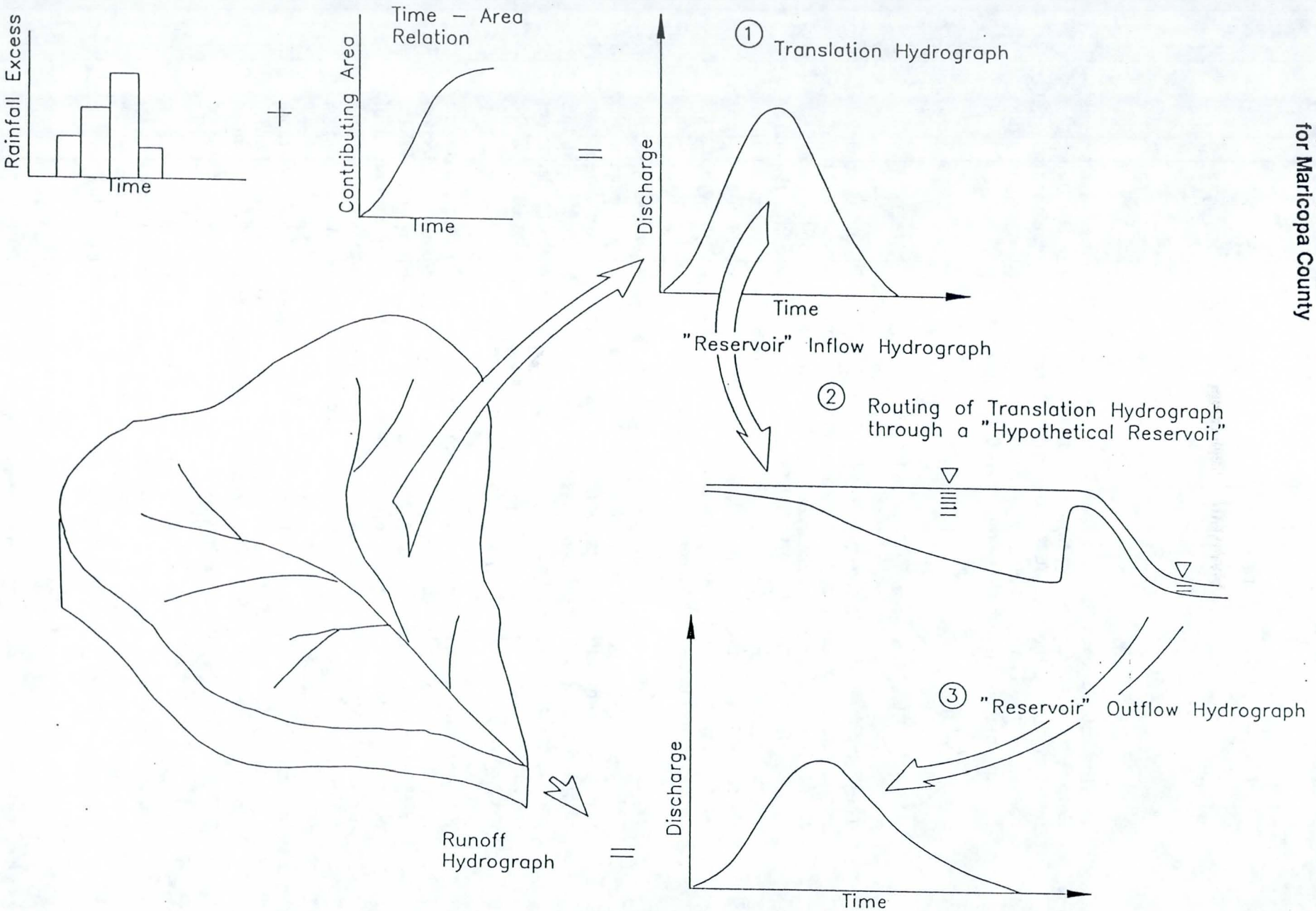


Figure 5.1
Conceptual Analogy of Linear Reservoir Routing to Generation of a Storm Hydrograph by the Clark Unit Hydrograph Method

$$O_i = CI_i + (I + C)O_{i-1} \tag{2}$$

where

$$C = \frac{2\,\Delta t}{2R + \Delta t} \tag{3}$$

O_i is the instantaneous flow at the end of the time period; O_{i-1} is the instantaneous flow at the beginning of the time period; I_i is the ordinate of the translation hydrograph; Δt is the computation time interval; and R is the watershed storage coefficient. The Clark Unit Hydrograph of duration, Δt, is obtained by averaging two instantaneous unit hydrographs spaced Δt units apart:

$$\bar{O}_i = 0.5(O_i + O_{i-1}) \tag{4}$$

where $\bar{O}_i$ = the ordinates of the Clark Unit Hydrograph.

The Clark method uses two numeric parameters, Tc and R, and a graphical parameter, the time-area relation. The first parameter, time of concentration (Tc) is the travel time of water from the hydraulically most distant point in the watershed to the outflow location. Clark (1945) defined this time as the time from the end of effective rainfall over the watershed to the inflection point on the recession limb of the surface runoff hydrograph as shown in Figure 5.2. In practice, for ungaged watersheds this time is usually estimated by empirical equations since runoff hydrographs from the watershed are not often available.

The second parameter is the storage coefficient, R, which has the dimension of time. This parameter is used to account for the effect that temporary storage in the watershed has on the hydrograph. Several methods are available to estimate R from recorded hydrographs for a basin. As originally proposed by Clark (1945), this parameter can be estimated by dividing the discharge at the point of inflection of the surface runoff hydrograph by the rate of change of discharge (slope of the hydrograph) at the inflection point as shown in Figure 5.2.

Another technique for estimating R is to compute the volume remaining under the recession limb of the surface runoff hydrograph following the point of inflection and to divide the volume by the discharge at the point of inflection. Both of these methods require the ability to identify the inflection point on the recession limb of the runoff hydrograph. This is difficult if not impossible for complex hydrographs and flashy hydrographs such as occur from urban basins and natural watersheds in the Southwest. A method to estimate R by a graphical recession analysis of the hydrograph has been proposed (Sabol, 1988) and this method provides much more consistent results than do the previously described methods. The parameter, R, should be estimated by the analysis of several recorded events; however, in most cases recorded discharge hydrographs are not available and R must be estimated by empirical equations.

The time-area relation, a graphical parameter, is necessary to compute the translation hydrograph. The time-area relation specifies the accumulated area of the watershed that is contributing runoff to the outlet of the watershed at any point in

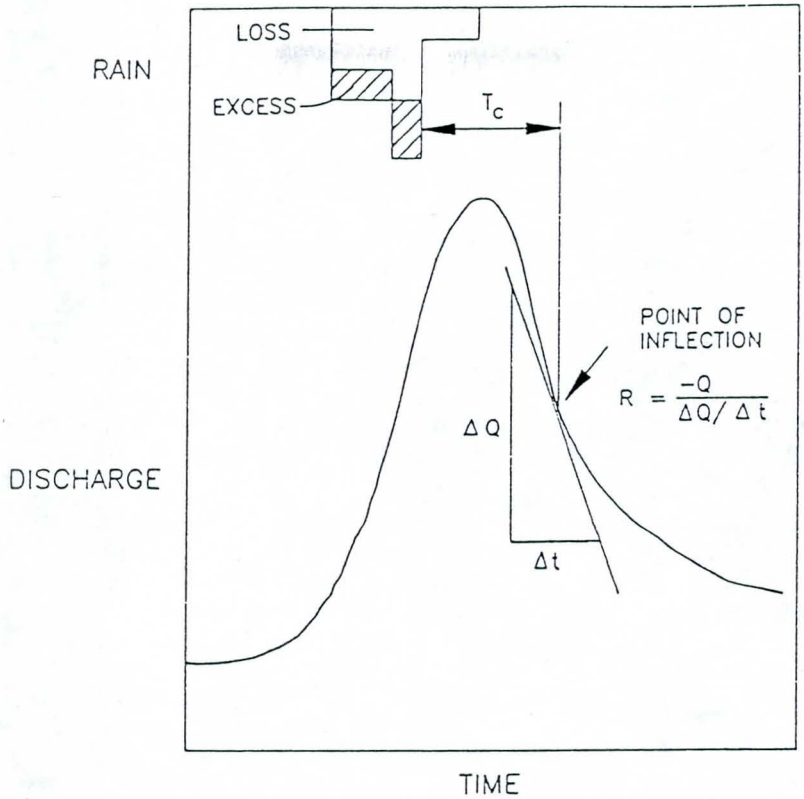


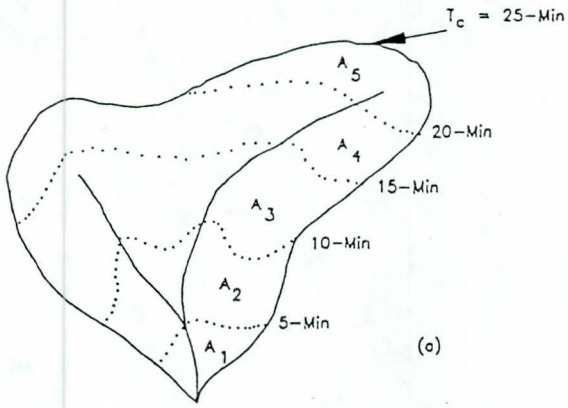
Figure 5.2
Definition Sketch of Clark Unit Hydrograph Parameters from hydrograph analysis

time. Procedures to develop a time-area relation for a watershed are discussed in a later section of this manual.

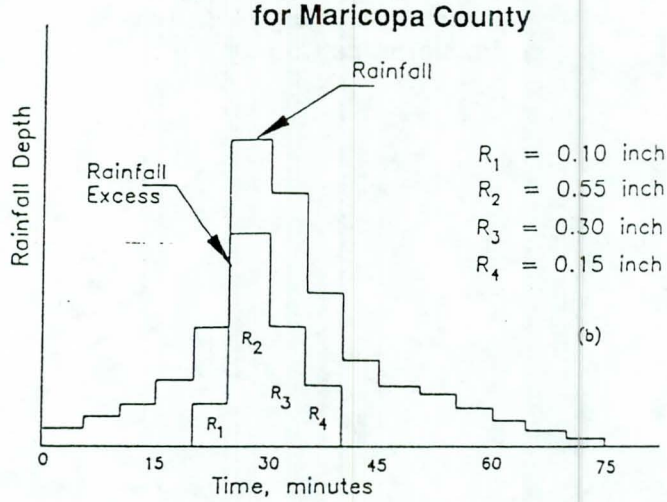
The application of the Clark Unit Hydrograph method is best described with a simple example. A watershed is shown in Figure 5.3(a), and a rainfall hyetograph and rainfall excess distribution are shown in Figure 5.3(b). For the example watershed and given intensity of rainfall excess the time of concentration is estimated at 25 minutes. An isochrone interval of 5 minutes is selected and the watershed is divided into five zones by isochrones as shown in Figure 5.3(a). The areas within each isochrone zone are measured and the dimensionless time-area relation is developed as shown in the table and depicted in Figure 5.3(c). The translation hydrograph of the time rate of runoff is developed by considering each incremental unit of runoff production that would be available as inflow to a watershed routing model. For example, at the end of the first 5 minutes of rainfall excess the runoff that is available at the outlet of the watershed is the product of incremental area A_1, and the rainfall excess R_1.

$$I_1 = (A_1 R_1) \times \frac{c}{\Delta t}$$

where c = 60.5 cfs/acre-inch/minute, and Δt = 5 minutes.



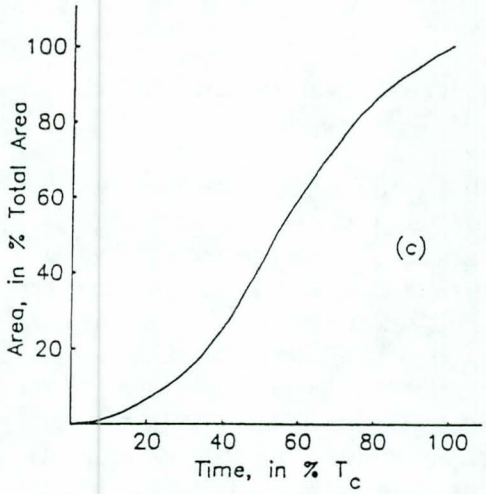
Watershed Map and Isochrones



Rainfall hydrograph and rainfall excess distribution

Table showing development of dimensionless time–area relation

Isochrone Zone (1)	Area Acres (2)	Accumulated Area (3)	Accumulated Area as % of Total Area (4)	Travel Time as % of T_c (5)
A_1	8	8	6.7	20
A_2	24	32	26.7	40
A_3	38	70	58.3	60
A_4	32	102	85.0	80
A_5	18	120	100.0	100



Dimensionless Time–Area Relation

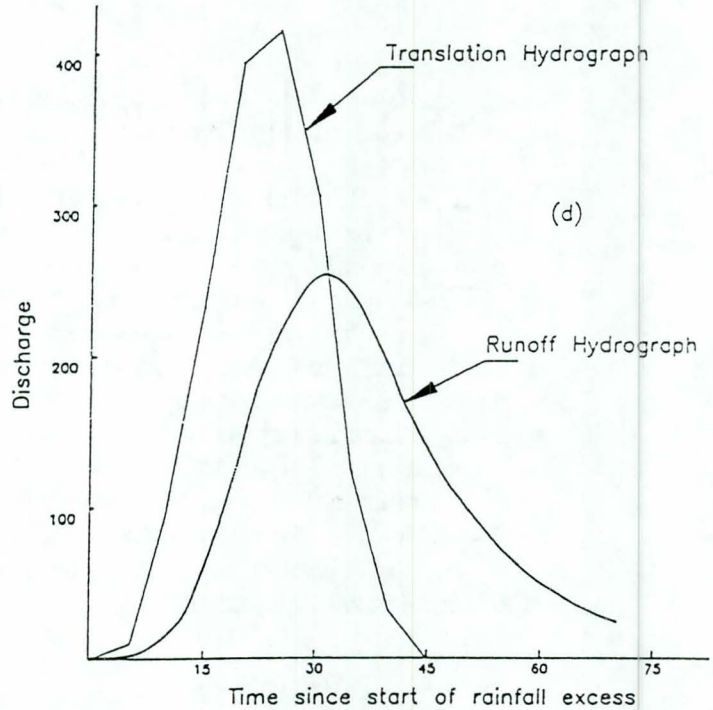


Figure 5.3
Example of Storm Hydrograph Generation using the Clark Unit Hydrograph Method

$$I_1 = (8 \text{ acres})(.10 \text{ inch})(60.5 \text{ cfs/acre-inch/minute})/(5 \text{ minutes})$$

$$= 9.7 \text{ cfs}$$

At the end of 10 minutes the available runoff is

$$I_2 = (A_1R_2 + A_2R_1) \times \frac{c}{\Delta t}$$

$$= [(8)(.55) + (24)(.10)] \times \frac{60.5}{5}$$

$$= 82.3 \; cfs$$

At the end of 15 minutes the available runoff is

$$I_3 = (A_1R_3 + A_2R_2 + A_3R_1) \times \frac{c}{\Delta t}$$

$$= [(8)(.30) + (24)(.55) + (38)(.10)] \times \frac{60.5}{5}$$

$$= 234.7 \; cfs$$

At the end of 20 minutes the available runoff is

$$I_4 = (A_1R_4 + A_2R_3 + A_3R_2 + A_4R_1) \times \frac{c}{\Delta t}$$

$$= [(8)(.15) + (24)(.30) + (38)(.55) + (32)(.10)] \times \frac{60.5}{5}$$

$$= 393.5 \; cfs$$

At the end of 25 minutes the available runoff is

$$I_5 = (A_1R_5 + A_2R_4 + A_3R_3 + A_4R_2 + A_5R_1) \times \frac{c}{\Delta t}$$

$$= [(8)(0) + (24)(.15) + (38)(.30) + (32)(.55) + (18)(.10)] \times \frac{60.5}{5}$$

$$= 416.2 \; cfs$$

Notice that, for this example, all incremental rainfalls equal 0.0 from R5 onward. At the end of 30 minutes the available runoff is

$$I_6 = (A_3R_4 + A_4R_3 + A_5R_2) \times \frac{c}{\Delta t}$$

$$= [(38)(.15) + (32)(.30) + (18)(.55)] \times \frac{60.5}{5}$$

$$= 304.9 \; cfs$$

At the end of 35 minutes the available runoff is

$$I_7 = A_4R_4 + A_5R_3) \times \frac{c}{\Delta t}$$
$$= [(32)(.15) + (18)(.30)] \times \frac{60.5}{5}$$
$$= 123.4\ cfs$$

At the end of 40 minutes the available runoff is

$$I_8 = (A_5R_4) \times \frac{c}{\Delta t}$$
$$= [(18)(.15)] \times \frac{60.5}{5}$$
$$= 32.7\ cfs$$

After 45 minutes (rainfall excess of 20 minutes plus travel time of 25 minutes) the available runoff is

$$I_9 = 0\ cfs$$

The translation hydrograph (Ii) is shown in Figure 5.3(d). This theoretical hydrograph has the correct volume of runoff from the watershed, however it does not reflect the effects of routing through the watershed. The translation hydrograph is then routed and averaged using Equations 2 through 4 resulting in the final runoff hydrograph. For this example, assume that R = 15 minutes, and the runoff hydrograph is shown in Figure 5.3(d). Notice that the Clark Unit Hydrograph itself was never developed per se but that the three principles of the unit hydrograph were applied directly (mathematically) to the rainfall excess without performing graphical superposition of ratios of a unit hydrograph. Computationally, this process can be completed very quickly and conveniently with a computer program such as is done with HEC-1.

5.3 Limitations and Applications

There are no theoretical limitations governing the application of the Clark Unit Hydrograph; however, there are some practical limitations that should be observed. The method that is used to estimate the parameters may dictate limitations in regard to the type or size of watershed that is being considered. If the parameters are estimated through an analysis or reconstitution of a recorded rainfall-runoff event, the parameters would be considered to be appropriate for that particular watershed, regardless of type or size. This is the preferred method of parameter estimation, but there will be limited opportunity for this approach because of the scarcity of instrumented watersheds in Maricopa County. The parameters could be estimated by indirect methods, such as a regional analysis of recorded data. In this case, application of the parameter estimation procedures should be applied only to those ungaged watersheds that are representative of the watersheds in the data base. Most often, the parameters are estimated by generalized relations that may have been

developed from a relatively large and diverse data base. The parameter estimation procedures that are recommended herein are of this last category.

The Clark Unit Hydrograph parameter estimation procedures that are presented in this manual have been adopted, modified, or developed from an analysis of a large data base of instrumented watersheds, controlled experimental watersheds, and laboratory studies; therefore, the application of these procedures is considered to be appropriate for most conditions that occur in Maricopa County. The types of watersheds for which the procedures can be applied include urban, rangeland, developed and natural alluvial fans, agricultural, hillslopes, and mountains.

Watershed size should be 5 square miles or less, with an upper limit of application to a single basin of 10 square miles. Watersheds larger than 5 square miles should be divided into smaller sub-basins for modeling purposes. Many watersheds smaller than 5 square miles should also be divided into sub- basins depending on the drainage network and degree of homogeneity of the watershed. The subdivision of the watershed into near homogeneous units should result in improved accuracy. Subdivision may also be desirable or required to determine discharges at concentration points within the watershed.

5.4 Development of Parameter Estimators

The procedures for parameter estimation are based on available literature, research results, and analysis of original data. For example, the Tc equation is based on the recent research of Papadakis and Kazan (1987). A large data base of recorded rainfall-runoff data was compiled and analyzed in developing and testing the procedures. These data are for instrumented watersheds in Arizona, New Mexico, Colorado, and Wyoming. A discussion of the development and testing of these procedures is contained in the Documentation Manual that is a companion to this ~~Hydrology Manual.~~ Hydrologic Design Manual.

5.5 Estimation of Parameters

The following procedures are recommended for the calculation of the Clark Unit Hydrograph parameters for use in Maricopa County. Other general procedures, as previously discussed, can be used, however, these should be approved by the jurisdictional agency prior to ~~adopting~~ undertaking such procedures.

5.5.1 Time of Concentration

~~Time of concentration is defined as the travel time, during the corresponding period of most intense rainfall excess, for water to travel from the hydraulically most distant point in the watershed to the point of interest (concentration point).~~ An empirical equation for time of concentration, Tc, has been adopted with some procedural modifications from Papadakis and Kazan (1987).

Time of concentration is defined as the travel time, during the corresponding period of most intense rainfall excess, for a floodwave to travel from the hydraulically most distant point in the watershed to the point of interest (concentration point). Note especially that Tc is not the travel time taken for a particle of water to move down the catchment, as is often cited in engineering texts. The catchment is in equilibrium when Tc is reached because the outlet then "feels" the inflow from every portion of the catchment (Bedient and Huber). Since a wave moves faster than a particle of water, the time of concentration (and catchment equilibrium) occurs sooner than if based on overland flow or channel water velocities. Replace text in brackets below

where Tc = time of concentration in hours

L = length of the flow path for Tc in miles

K_b = representative watershed resistance coefficient

S = watercourse slope in feet/mile and

i = the average rainfall excess intensity, during the time Tc, in inches/hour.

Watercourse slope, S, is the average slope of the flow path ~~which is~~ for the same watercourse that is used to define L. The magnitude of S can be calculated as the difference in elevation between the two points used to define L divided by the length, L. Watersheds in mountains can result in large values for S—which may result in an underestimation of Tc. This is because as slope increases in natural watersheds the runoff velocity does not usually increase in a corresponding manner. The slope of steep natural watercourses is often adjusted to reduce the slope, and the reduced slope is used in calculating runoff travel times. The slope of steep natural watercourses should be adjusted by using Figure 5.4. <F5.4

The selection of a representative watershed resistance coefficient, K_b, similar in concept to *Manning's n* in open-channel flow, is very subjective and therefore a high degree of uncertainty is associated with its use. To diminish this uncertainty and to increase the reproducibility of the procedure, a graph is provided in Figure 5.5 for the selection of K_b based on watershed classification and watershed size. Interpolation can be used for a given watershed size and mixed classification. Equations for estimating K_b are given in Table 5.1. <F5.5 <T5.1

The value of *i* in Equation 5 requires the knowledge of both the distribution of rainfall excess intensity and the time of concentration, which is, of course, unknown. Therefore, Equation 5 must be solved in a trial-and-error procedure. First, the time distribution of rainfall excess must be estimated for the design rainfall distribution and a graph of average rainfall excess intensity versus time prepared. Then a value of Tc is assumed and the corresponding value of i is read from the graph. Equation 5 is solved with that value of i. If the calculated value of Tc is reasonably close to the value that was assumed for i then the solution is finished; if not, then assume a new value of Tc, recalculate i, and recalculate Tc with Equation 5. The solution for Tc should converge within three trials.

A worksheet has been prepared that facilitates the calculation of Tc. Appendix E is a copy of this worksheet and the Examples section of this manual shows how it is used. Alternatively, program "MCUHP1" can be used which will also provide the necessary HEC-1 input file.

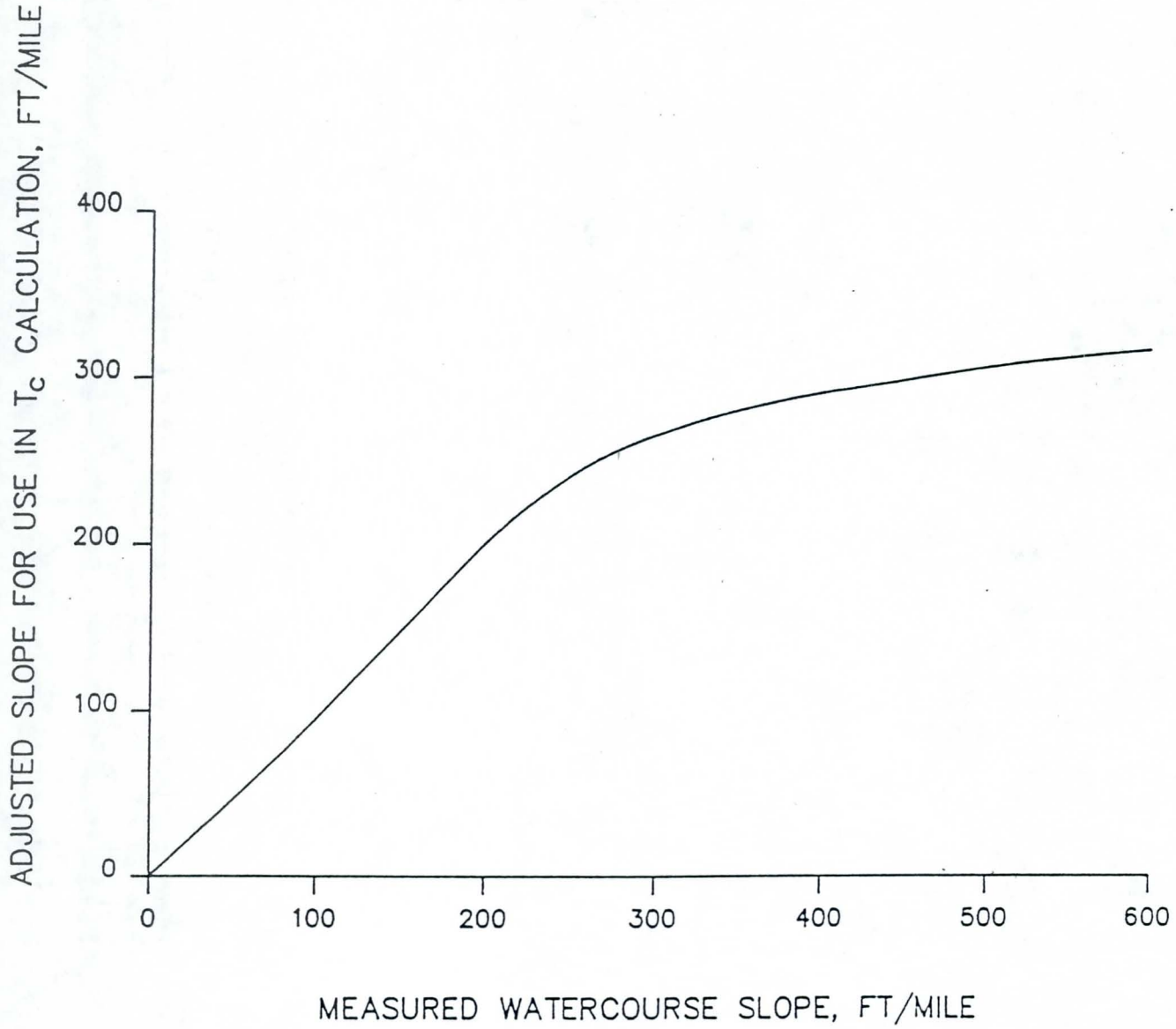


Figure 5.4
Slope Adjustment for Steep Watercourses in Natural Watersheds
(Source: *Drainage Criteria Manual*, Urban Drainage and Flood Control District, Colorado, May 1984.)

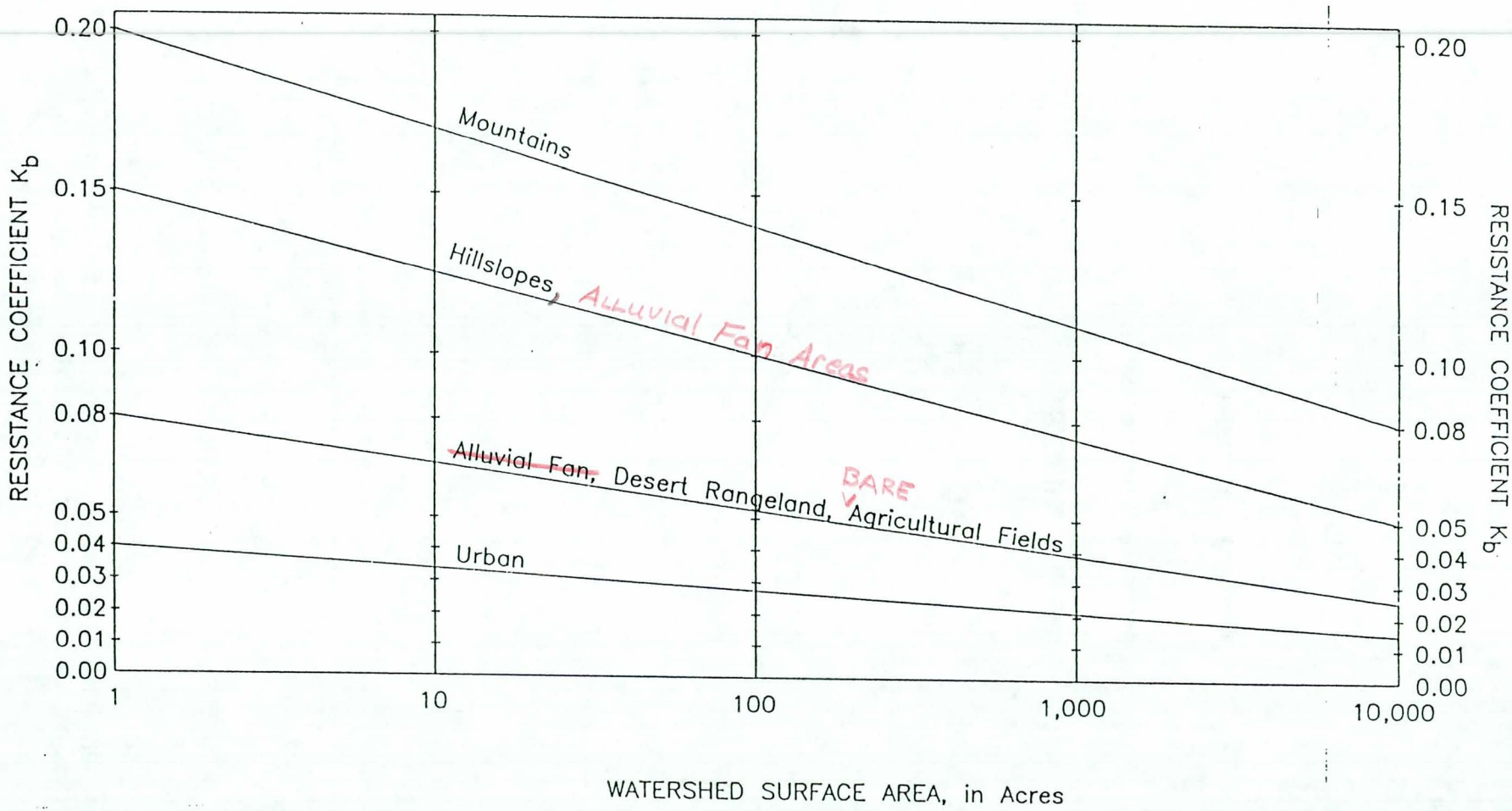


Figure 5.5
Resistance Coefficient "K_b" as a Function of Watershed Type and Size

Table 5.1
Equation for Estimating K_b in the Tc Equation

$K_b = m \log A + b$ Where A is drainage area, in acres		
Land Classification (1)	**Equation Parameters**	
	m (2)	**b (3)**
Urban	−0.00625	0.04
Bare or nearly bare ground (~~alluvial fan,~~ bare agricultural land, desert rangeland)	−0.01375	0.08
Rough and/or moderate vegetation (hillslopes, alluvial fan)	−0.025	0.15
Very rough and/or dense vegetation (mountains)	−0.030	0.20

5.5.2 Storage Coefficient

Very little literature exists on the estimation of the storage coefficient (R) for the Clark Unit Hydrograph. Clark (1945) had originally proposed a relation between Tc and R since they can both be defined by locating the inflection point of a runoff hydrograph (refer to Figure 5.2). The Corps of Engineers has discussed the development of regionalized relations for Tc and R as functions of watershed characteristics in *Training Document No. 15* (U.S. Army Corps of Engineers, 1982b). According to Corps procedures, Tc and R are estimated from relations of Tc + R and R/(Tc + R) as functions of watershed characteristics. These forms of empirical equations indicate an interrelation of Tc and R, and such dependence was observed in the data base, as discussed in the Documentation Manual. The equation for estimating R for Maricopa County is:

$$R = 0.37 Tc^{1.11} A^{-0.57} L^{0.80} \tag{6}$$

where R = storage coefficient in hours

Tc = time of concentration in hours

A = drainage area in square miles, and

L = length of flow path in miles.

5.5.3 Time-Area Relation

Either a synthetic time-area relation must be adopted or the time-area relation for the watershed must be developed. If a synthetic time-area relation is not used, the time-area relation is developed by dividing the watershed into incremental runoff producing areas that have equal incremental travel times to the outflow location.

This is a difficult task and well defined and reliable procedures for this are not available. The following general procedure is often used:

1. Use a topographic map of the watershed to trace along the flow path the distance from the hydraulically most distant point in the watershed to the outflow location; this defines L in both Equations 5 and 6.

2. Draw isochrones on the map to represent equal travel times to the outflow location. These isochrones can be established by considering the land surface slope and resistance to flow, and also whether the runoff would be sheet flow or would be concentrated in watercourses. A good deal of judgement and interpretation is required for this.

3. Measure and tabulate the incremental areas (in an upstream sequence) as well as the corresponding travel time for each area.

4. Prepare a graph of travel time versus contributing area (or a dimensionless graph of time as a percent of Tc versus contributing area as a percent of total area). The dimensionless graph is preferred because this facilitates the rapid development of new time-area relations should there be a need to revise the estimate of Tc.

Synthetic time-area relations can be used such as the default relation in the HEC-1 program:

$$A^* = 1.414(T^*)^{1.5} \qquad 0 \le T^* \le 0.5 \tag{7}$$

$$1 - A^* = 1.414(1 - T^*)^{1.5} \qquad 0.5 \le T^* \le 1.0$$

where A* = contributing area in percent of total area and

T* = time in percent of Tc.

Equation 7 is a symmetric relation and is not recommended for most watersheds in Maricopa County.

Two other dimensionless time-area relations have been developed during the reconstitution of recorded rainfall-runoff events as described in the Documentation Manual. These dimensionless relations for urban and natural watersheds are shown in Figures 5.6 and 5.7. Each of these figures show a synthetic time-area relation and a shaded zone where the time-area relation is expected to lie. For an urban watershed, the synthetic time-area relation of Figure 5.6 is recommended, and for a natural (undeveloped) watershed the synthetic time-area relation of Figure 5.7 is recommended. If a time-area relation is developed from the watershed map, which is generally recommended for unusually shaped watersheds, then the resulting relation should lie within the shaded zones in either Figures 5.6 or 5.7. The HEC-1 default time-area relation is shown for comparison in each figure. Tabulated values of the dimensionless time-area relations are shown in Table 5.2.

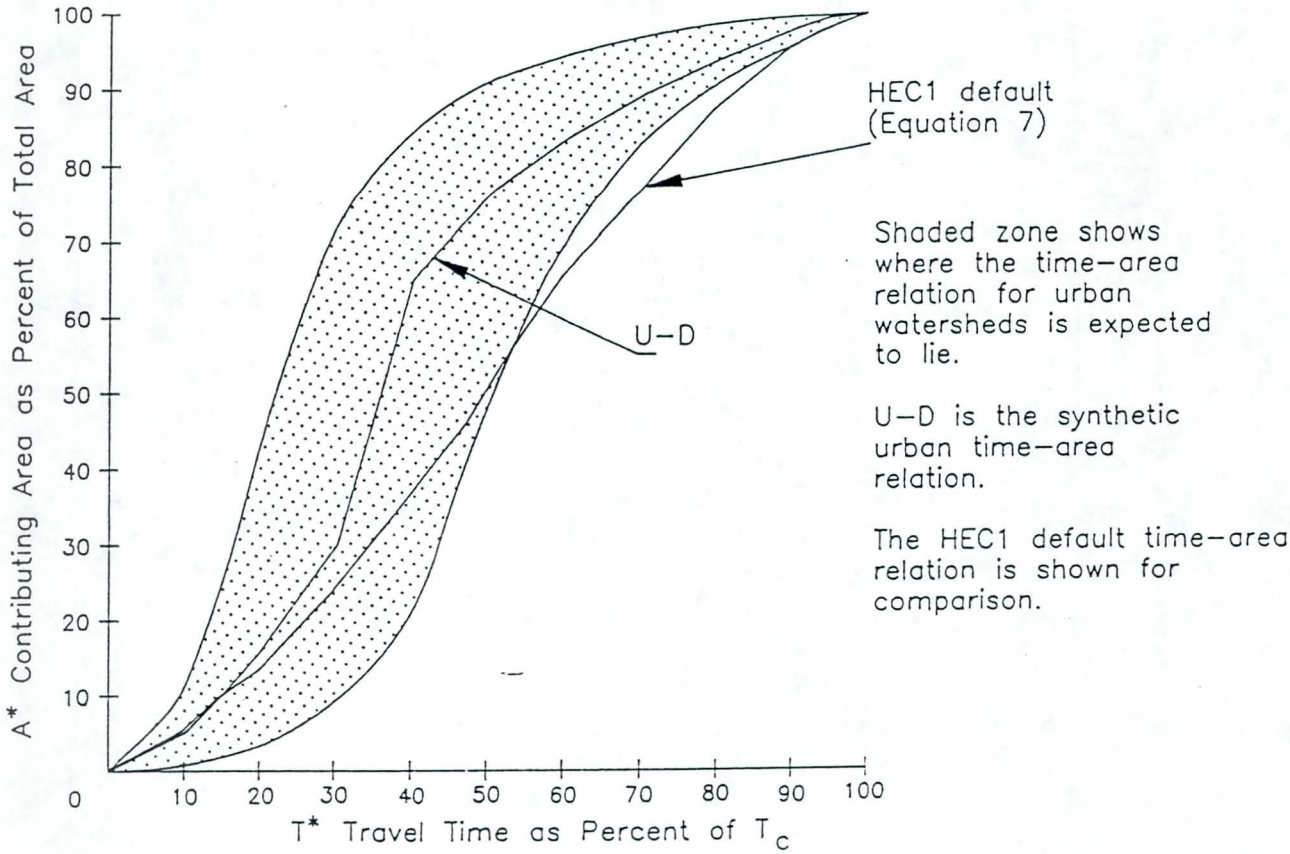


Figure 5.6
Synthetic Time-Area Relation for Urban Watershed

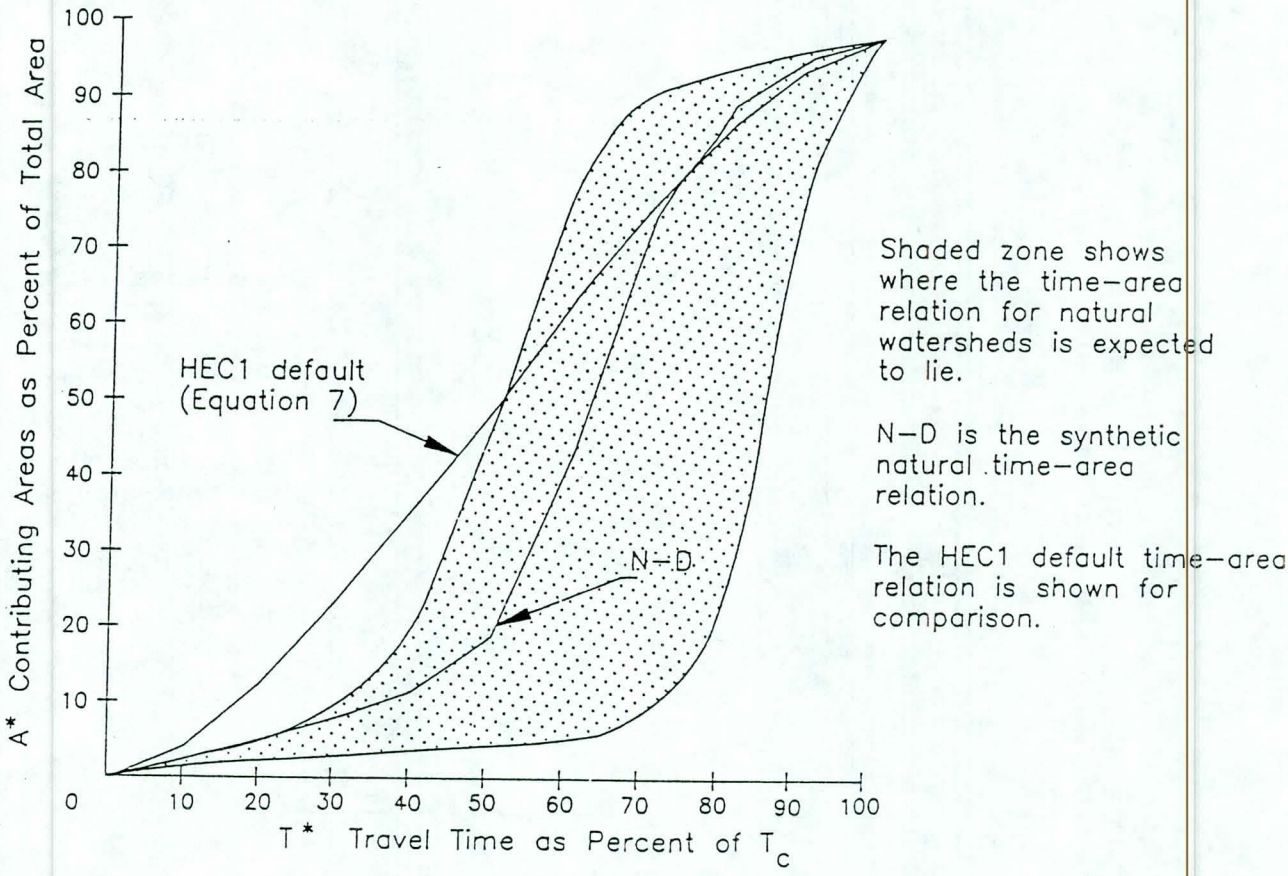


Figure 5.7
Synthetic Time-Area Relation for Natural Watersheds

Table 5.2
Values of the Synthetic Dimensionless Time-Area Relations for the Clark Unit Hydrograph

Time, as a percent of Time of Concentration (1)	Contributing Area, as a Percent of Total Area		
	Urban Watersheds (2)	Natural Watersheds (3)	HEC-1 Default (4)
0	0	0	0.0
10	5	3	4.5
20	16	5	12.6
30	30	8	23.2
40	65	12	35.8
50	77	20	50.0
60	84	43	64.2
70	90	75	76.8
80	94	90	87.4
90	97	96	95.5
100	100	100	100.0

The computation interval (NMIN) on the IT record of HEC-1 must be selected to correspond to the time of concentration for the unit hydrograph. This requirement is necessary to adequately define the shape of the unit hydrograph. From Snyder's unit hydrograph theory, the unit rainfall duration for a unit hydrograph (computation interval) is equal to lag time divided by 5.5. For the SCS Dimensionless Unit Hydrograph, the unit rainfall duration is to equal 0.133Tc, and although small variation in the selection of computation interval is allowed, the SCS recommends that the duration not exceed 0.25 Tc. Although there is not a rigid theoretical limitation to how small the computation interval can be, from a practical standpoint, too small of a NMIN could result in excessive computer output. Therefore, as a general rule the computation interval should meet the following:

$$NMIN = 0.15Tc \tag{8}$$

Equation 8 is preferred, however, as a general requirement NMIN should fall in the range indicated in Equation 9.

$$0.10Tc \leq NMIN \leq 0.25Tc \tag{9}$$

NMIN is normally selected as a multiple of five minutes. This may require that watersheds with significantly different sub-basin sizes be modeled with some sub-basins run separately and the outflow hydrographs from these separate runs read directly into the multi-basin model.

5.6 S-Graphs

An S-graph is a dimensionless form of a unit hydrograph and it can be used in the place of a unit hydrograph in performing flood hydrology studies. The concept of the S-graph dates back to the development of the unit hydrograph itself, although the application of S-graphs has not been as widely practiced as that of the unit hydrograph. The use of S-graphs has been practiced mainly by the U.S. Army Corps of Engineers, Los Angeles District, and the U.S. Bureau of Reclamation (USBR).

An example of an S-graph from Design of Small Dams (USBR, 1987) is shown in Figure 5.8. The discharge scale is expressed as percent of ultimate discharge (Qult), and the time scale is expressed as percent lag. Lag is defined as the elapsed time, usually in hours, from the beginning of an assumed continuous series of unit rainfall excess increments over the entire watershed to the instant when the rate of resulting runoff equals 50 percent of the ultimate discharge. The intensity of rainfall excess is 1 inch per duration of computation interval (Δt). An equivalent definition of lag is the time for 50 percent of the total volume of runoff of a unit hydrograph to occur. It is to be noted that there are numerous definitions for lag in hydrology and the S-graph lag should not be calculated by methods that are not consistent with this definition.

Ultimate discharge is the maximum discharge that would be achieved from a particular watershed when subjected to a continuous intensity of rainfall excess of 1 inch per duration (Δt) uniformly over the basin. Ultimate discharge (Qult), in cubic feet per second (cfs), can be calculated from Equation 10:

$$Q_{ult} = \frac{645.33A}{\Delta t} \tag{10}$$

where A = drainage area in square miles, and

Δt = duration of the 1 inch of rainfall excess in hours.

S-graphs are developed by summing a continuous series of unit hydrographs, each lagged behind the previous unit hydrograph by a time interval that is equal to the duration of rainfall excess for the unit hydrograph (Δt). The resulting summation is a graphical distribution that resembles an S-graph except that the discharge scale is accumulated discharge and the time scale is in units of measured time. This graph is terminated when the accumulated discharge equals Qult which occurs at a time equal to the base time of the unit hydrograph less one duration interval. The basin lag can be determined from this graph at the time at which the accumulated discharge equals 50 percent of Qult. This summation graph is then converted to a dimensionless S-graph by dividing the discharge scale by Qult and the time scale by lag.

In practice, S-graphs have generally been developed by reconstituting observed floods to define a representative unit hydrograph and then converting this to an S-graph. Prior to the advent of computerized models, such as HEC-1, flood

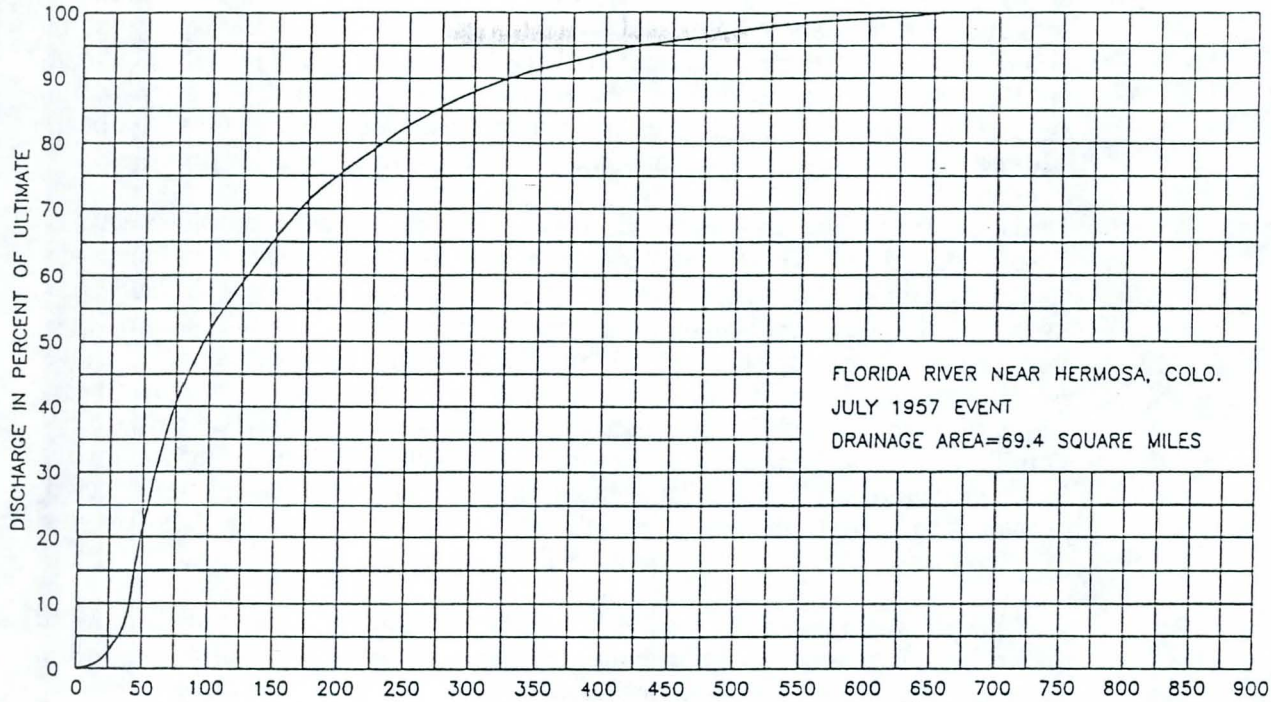


Figure 5.8
Example of an S-Graph from *Design of Small Dams* (USBR, 1987)

reconstitution was a laborious task of rainfall and hydrograph separation along with numerous hand-cranked simulations to define the representative unit hydrograph. Modern S-graph development generally relies on use of optimization techniques, such as coded into HEC-1, to identify unit hydrograph parameters that best reproduce the observed flood.

Although an S-graph is completely dimensionless and does not have a duration of rainfall excess associated with it as does a unit hydrograph, its general shape and the magnitude of lag is influenced by the distribution of rainfall over the watershed and the time distribution of the rainfall. Therefore, the transposition of an S-graph from a gaged watershed to application in another watershed must be done with consideration of both the physiographic characteristics of the watersheds and the hydrologic characteristics of the rainfalls for the two watersheds.

5.6.1 Limitations and Applications

S-graphs are empirical, lumped parameters that represent runoff characteristics for the watershed for which the S-graph was developed. S-graphs that are developed from recorded runoff data from one watershed can be applied to another watershed only if the two watersheds are hydrologically and physiographically similar. In addition, a recent study for the Flood Control District of Maricopa County (Sabol, 1987) has demonstrated that the shape of S-graphs is significantly affected by storm characteristics, particularly the maximum intensity of the rainfall. Therefore, it may

not be advisable to adopt S-graphs that have been developed from one hydrologic zone and to apply these to watersheds in other hydrologic zones because of possible differences in rainfall characteristics in the two zones that may affect the shape of the S-graph. Application of S-graphs requires the selection of an appropriate S-graph and the estimation of the one parameter, basin lag. Two S-graphs have been selected for use in Maricopa County and a method to estimate lag is provided.

The USBR has revised the Flood Hydrology Studies chapter of *Design of Small Dams* (USBR, 3rd Edition, 1987), and it has identified S-graphs for application in six generalized regional and physiographic type of watersheds. Recently, the USBR has issued a *Flood Hydrology Manual* (Cudworth, 1989) that contains extensive discussion of flood hydrology in general, and S-graphs in particular. Both of these references should be consulted before using S-graphs. The S-graph has been adopted as the unit hydrograph procedure by Orange County and San Bernardino County, California, and selected S-graphs are presented in the hydrology manuals for those counties. The S-graphs in those hydrology manuals have been selected primarily from S-graphs that previously had been defined by the Los Angeles District from a rather long and extensive history of analyses of floods in California.

An S-graph can, in theory, be used in any application for which an unit hydrograph can be used. In practice an S-graph must be first converted to an unit hydrograph, and this can be done by one of two methods. First, The S-graph can be converted to an unit-hydrograph manually; or second, the S-graph can be converted to an unit hydrograph by use of the LAPRE1 program. The LAPRE1 program is a HEC-1 preprocessor program that converts a psuedo- HEC-1 input file containing input for an S-graph to a valid HEC-1 input file. The LAPRE1 program outputs the HEC-1 input file with the S-graph converted to an unit hydrograph, and the unit hydrograph is written to the HEC-1 input file using the UI (Given Unit Graph) record. The use of LAPRE1 greatly facilitates the use of S-graphs.

Although the S-graph is completely dimensionless and does not have a rainfall excess duration associated with it, the unit hydrograph does require the specification of a duration. In general, the same rules and recommendations apply to the S-graph as were made for the Clark Unit Hydrograph; that is, the duration (computation interval, NMIN) selected for the development of the unit hydrograph from a S-graph should equal about 0.15 times the lag. A duration (NMIN) in the range 0.10 to 0.25 times the lag is usually acceptable.

5.6.2 Sources of S-Graphs

S-graphs for Maricopa County have been selected from a compilation of S-graphs for the Southwestern United States that was recently completed (Sabol, 1987). The sources of S-graphs for that compilation were reports and file data of the U.S. Army Corps of Engineers, Los Angeles District, and the USBR. That compilation included 55 individual S-graphs and 18 regional S-graphs. An individual S-graph is one that can be identified with the watershed from which data was used to develop the S-graph. Regional S-graphs are those that are graphical averages or modifications of individual S-graphs to produce an S-graph that is representative of a specific physiographic type of watershed.

5.6.3 S-Graphs for Use in Maricopa County

Two regional S-graphs have been selected for use in flood hydrology studies of major watercourses in Maricopa County. These two are referred to as the Phoenix Mountain and the Phoenix Valley S-graphs. The Phoenix Mountain S-graph is to be used in flood hydrology studies of watersheds that drain predominantly mountainous terrain. It should be used for the Agua Fria River above Rock Springs, New River above the Town of New River, the Verde River, Tonto Creek, and the Salt River above Phoenix. Although the Corps of Engineers developed a separate S-graph for Indian Bend Wash, it is nearly identical to the Phoenix Mountain S-graph which is also appropriate for Indian Bend Wash.

The Phoenix Valley S-graph is to be used in flood hydrology studies of watersheds that have little topographic relief. For example, this S-graph should be used for the Agua Fria River below Rock Springs, New River below the Town of New River, lower Skunk Creek, lower Cave Creek, and urbanized watersheds.

These two S-graphs are shown in Figures 5.9 and 5.10, and the coordinates of the graphs listed in Table 5.3. These same two S-graphs have been selected for similar use in Maricopa County by the U.S. Army Corps of Engineers (1974 and 1982). The justification for the selection of these two S-graphs is provided in the Documentation Manual, and a more comprehensive presentation of S-graphs for Maricopa County is provided in the *S-Graph Study* report for the Flood Control District of Maricopa County (Sabol, 1987). It is possible that S-graphs other than the two that have been recommended for general use in Maricopa County be selected. The selection of S-graph should be made based on a comparison of the watershed of interest to the watershed(s) used to develop the various S-graphs.

<F5.9
<F5.10
T5.3

Therefore, either one of the two recommended S-graphs should be selected or the selection of other S-graphs, such as from *Design of Small Dams* (USBR, 1987), should be approved by the jurisdictional agency before proceeding.

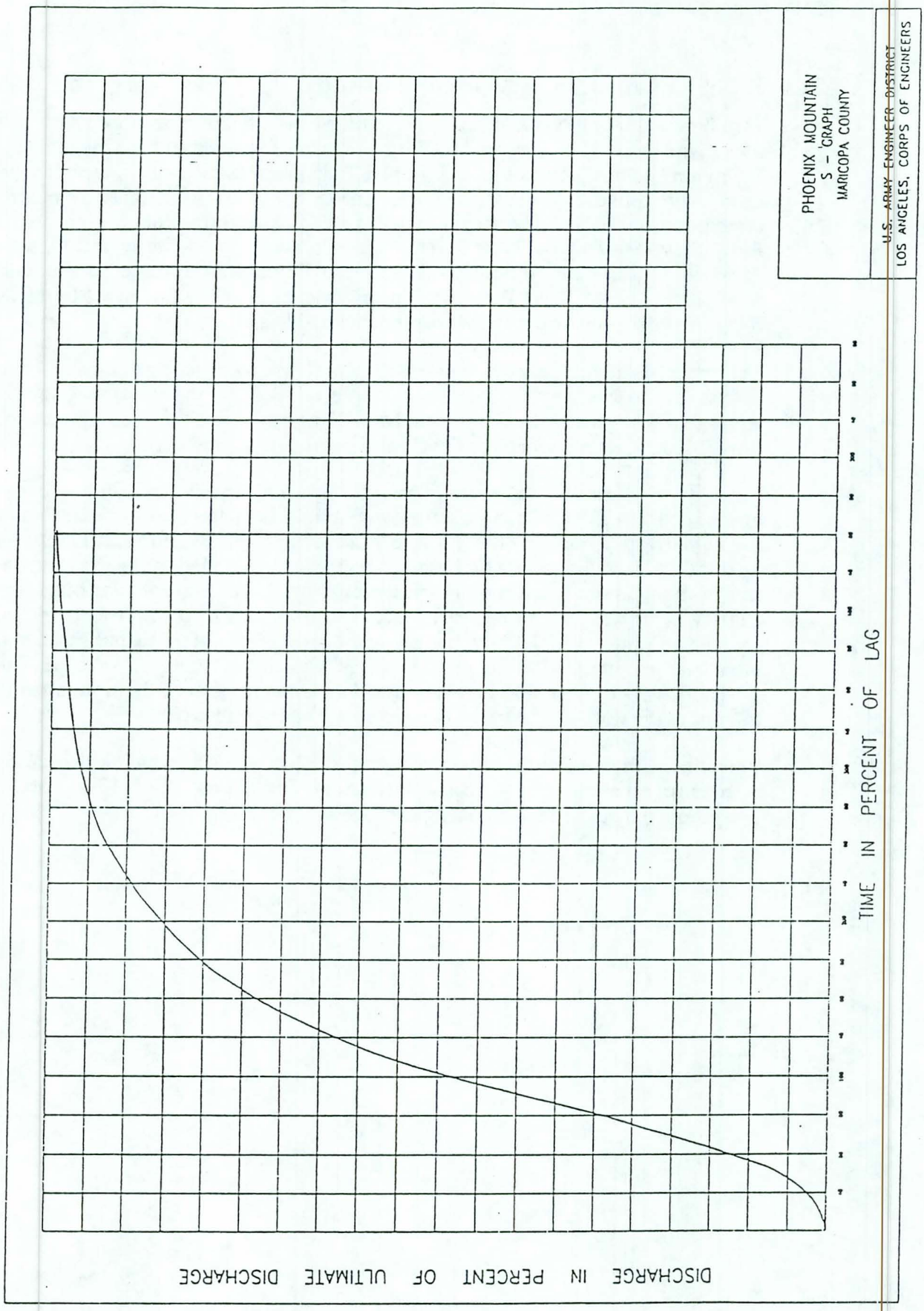

PHOENIX MOUNTAIN
S - GRAPH
MARICOPA COUNTY
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TIME IN PERCENT OF LAG
DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE

Figure 5.9

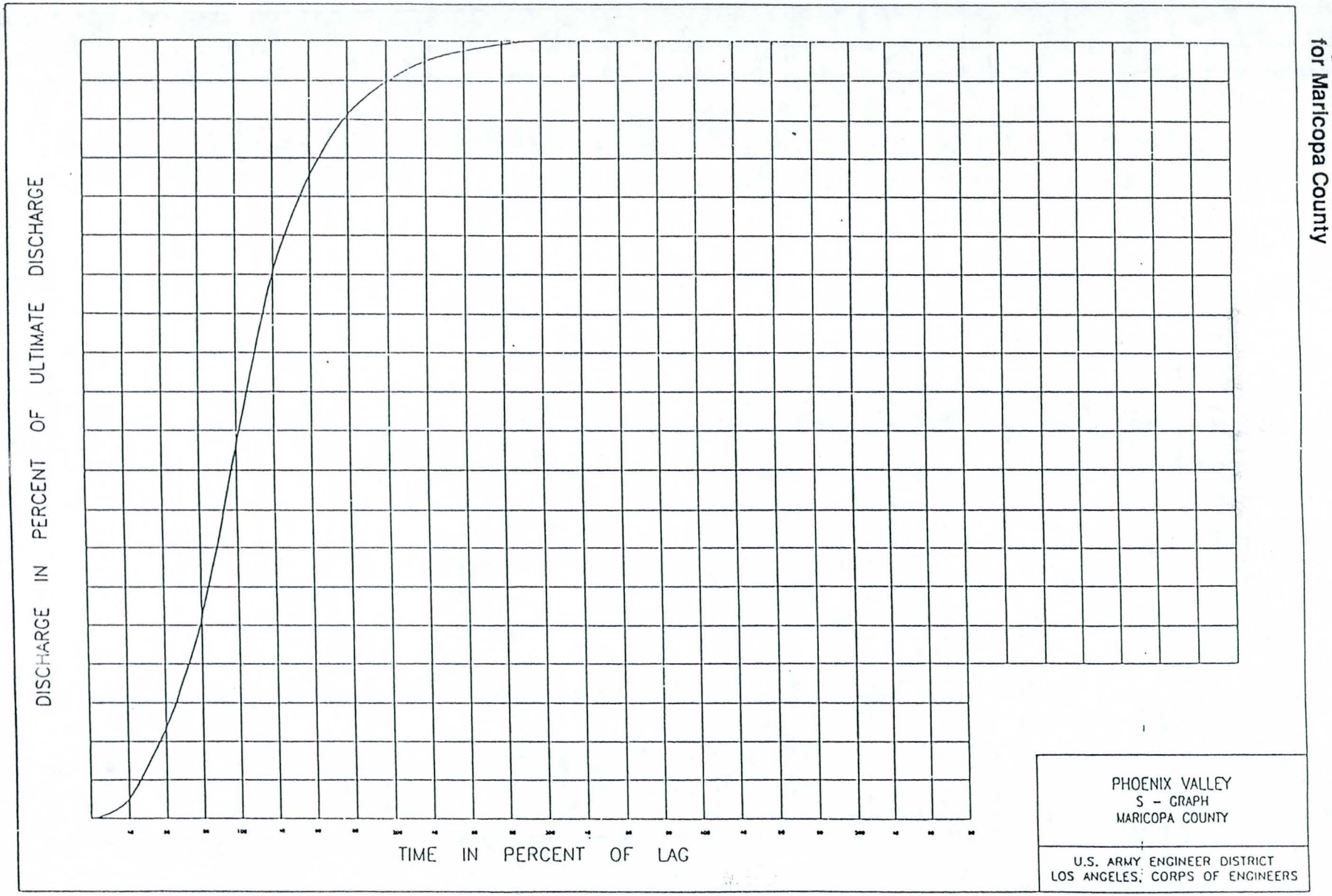

DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE
TIME IN PERCENT OF LAG
PHOENIX VALLEY
S - GRAPH
MARICOPA COUNTY
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

Figure 5.10

Table 5.3
Tabulation of Coordinates for the
Phoenix Valley and the Phoenix Mountain S-Graphs

Percent Ultimate Discharge (1)	Time in Percent Lag	
	Phoenix Valley (2)	Phoenix Mountain (3)
0	0.0	0.0
2	23.0	23.0
4	30.0	31.0
6	36.0	37.0
8	41.0	42.0
10	45.7	46.0
12	50.0	49.8
14	54.1	53.4
16	58.0	56.8
18	61.7	60.0
20	65.2	63.1
22	68.5	66.1
24	71.6	69.0
26	74.6	71.8
28	77.5	74.4
30	80.2	76.8
32	82.7	79.1
34	85.0	81.2
36	87.2	83.2
38	89.0	85.1
40	91.1	86.8
42	92.9	88.8
44	94.6	91.0
46	96.3	93.8
48	98.1	96.8
50	100.0	100.0
52	102.0	103.4
54	104.1	107.0
56	106.3	110.8
58	108.6	114.7
60	111.0	118.7
62	113.5	122.9
64	116.1	127.3
66	118.8	131.9
68	121.6	136.7
70	124.5	141.7
72	127.5	147.1
74	130.7	152.8
76	134.1	158.8
78	137.7	165.5
80	141.5	172.9

Table 5.3, continued

Percent Ultimate Discharge (1)	Time in Percent Lag	
	Phoenix Valley (2)	Phoenix Mountain (3)
82	145.5	181.6
84	149.9	191.0
86	154.6	201.0
88	159.6	212.0
90	165.6	226.0
92	173.6	244.0
94	186.6	265.0
96	200.6	295.0
98	223.6	342.0
100	298.6	462.0

5.6.4 Estimation of Lag

The application of an S-graph requires the estimation of the parameter, basin lag. A general relationship for basin lag as a function of watershed characteristics is given by Equation 11:

$$Lag = C\left(\frac{LL_{ca}}{S^p}\right)^m \tag{11}$$

where Lag = basin lag in hours

L = length of the longest watercourse in miles

L_{ca} = length along the watercourse to a point opposite the centroid in miles

S = watercourse slope in feet per mile

C = coefficient and

m & p = exponents.

The Corps of Engineers often uses C = 20Kn where Kn is the estimated mean *Manning's n* for all the channels within an area, and m = 0.38. The USBR (1987) has recommended that C = 26Kn and m = 0.33. Both sets of values in Equation 11 will often result in similar estimates for Lag. Traditionally the exponent, p, on the slope is equal to 0.5.

It should be noted that Kn is a measure of the hydraulic efficiency of the watershed and it is not necessarily a constant for a given watershed for all rainfall depths and rainfall intensities. As rainfall depth and/or rainfall intensity increases the efficiency of runoff increases and Kn decreases. Therefore, some adjustment in Kn should

be made for use with rainfalls of different magnitudes (frequencies). Generally, Kn is the smallest for extreme floods such as PMFs and increases as the frequency of event increases.

Several graphical relations are available for estimating basin lag. One such relation (U.S. Army Corps of Engineers, 1982) is shown in Figure 5.11. Several other relations that should be consulted when using S-graphs are contained in *Design of Small Dams* and the USBR *Flood Hydrology Manual* (Cudworth, 1989).

When estimating basin lag the following steps should be used:

1. From an appropriate map of the watershed, measure drainage area (A), and the values of L, L_{ca}, and S.

2. Calculate the basin factor $LL_{ca}/(S^{0.5})$.

3. Use data in Figure 5.11 *or* the tables in *Design of Small Dams* or the *Flood Hydrology Manual* to attempt to identify watersheds of the same physiographic type and similar drainage area and basin factor. Make a list of watersheds with similar drainage areas and basin factors, and tabulate the estimated value of Kn for those watersheds, and the measured lag.

4. Estimate Kn for the watershed by inspection of the tabulation, step 3.

5. Estimate lag by Equation 9. Use values of C and m corresponding to the source (U.S. Army Corps of Engineers or USBR) that was used to estimate Kn.

6. Compare the calculated lag with the measured lag for similar watersheds (step 3).

The use of measured values of Kn from hydrograph reconstitutions of similar watersheds will provide the most reliable estimates of Kn and basin lag.

		CONTRIBUTING AREA (SQ. MI.)	L (MILES)	L_{ca} (MILES)	S (FT./MI.)	LAG (HOURS)	ESTIMATED $\bar{n}$
1.	SAN GABRIEL RIVER AT SAN GABRIEL DAM, CA	162.0	23.2	11.6	350	3.3	0.050
2.	WEST FORT SAN GABRIEL RIVER AT COGSWELL DAM, CA	40.4	9.3	4.3	450	1.6	.050
3.	SAN ANITA CREEK AT SANTA ANITA DAM, CA	10.8	5.8	2.5	690	1.1	.050
4.	SAN DIMAS CREEK AT SAN DIMAS DAM, CA	16.2	8.6	4.8	440	1.5	.050
5.	EATON WASH AT EATON WASH DAM, CA	9.5	7.3	4.4	600	1.3	.050
6.	SAN ANTONIO CREEK NEAR CLAREMONT, CA	16.9	5.9	3.0	1017	1.2	.055
7.	SANTA CLARA RIVER NEAR SAUGAS, CA	355.0	36.0	15.8	140	5.6	.050
8.	TEMECULA CREEK AT PAUBA CANYON, CA	168.0	26.0	11.3	150	3.7	.050
9.	SANTA MARGARITA RIVER NEAR FALLBROOK, CA	645.0	46.0	22.0	105	7.3	.055
10.	SANTA MARGARITA RIVER AT YSIDORA, CA	740.0	61.2	34.3	85	9.5	.055
11.	LIVE OAK CREEK AT LIVE OAK DAM, CA	2.3	2.9	1.5	700	0.8	.070
12.	TUJUNGA CREEK AT BIG TUJUNGA DAM, CA	81.4	15.1	7.3	290	2.5	.050
13.	MURRIETA CREEK AT TEMECULA, CA	220.0	27.2	10.3	95	4.0	.050
14.	LOS ANGLES RIVER AT SEPULVEDA DAM, CA	152.0	19.0	9.0	145	3.5	.050
15.	PACOIMA WASH AT PACOIMA DAM, CA	27.8	15.0	8.0	315	2.4	.050
16.	ALHAMBRA WASH ABOVE SHORT STREET, CA	14.0	9.5	4.6	85	0.6	.015
17.	BROADWAY DRAIN ABOVE RAYMOND DIKE, CA	2.5	3.4	1.7	100	0.28	.015
18.	GILA RIVER AT CONNOR NO. 4 DAM SITE, AZ	2840.0	131.0	71.0	29	21.5	.050
19.	SAN FRANCISCO RIVER AT JUNCTION WITH BLUE RIVER, AZ	2000.0	30.0	74.0	32	20.6	.050
20.	BLUE RIVER NEAR CLIFTON, AZ	790.0	77.0	37.0	65	10.3	.050
21.	SALT RIVER NEAR ROOSEVELT, AZ	4310.0	160.0	66.0	45	18.6	.050
22.	NEW RIVER AT ROCK SPRINGS, AZ	67.3	20.2	9.7	141	3.1	.045
23.	NEW RIVER AT NEW RIVER, AZ	85.7	23.2	13.6	145	3.7	.045
24.	NEW RIVER AT BELL ROAD, AZ	187.0	47.6	20.7	83	5.3	.037
25.	SKUNK CREEK NEAR PHOENIX, AZ	64.6	17.6	10.0	89	2.4	.033

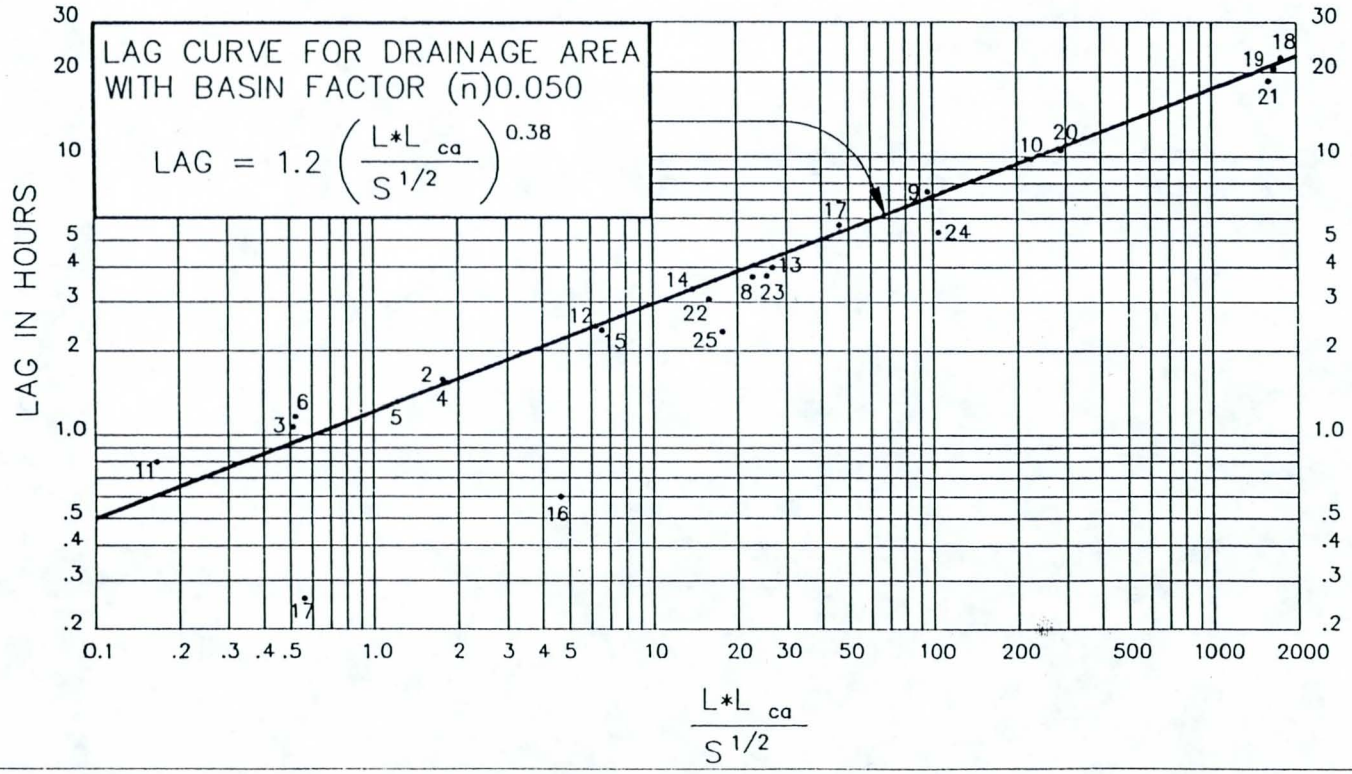
GUIDE FOR ESTIMATING BASIN FACTOR ($\bar{n}$)

$\bar{n}$ = 0.200; DRAINAGE AREAS HAS COMPARATIVELY UNIFORM SLOPES AND SURFACE CHARACTERISTICS SUCH THAT CHANNELIZATION DOES NOT OCCUR. GROUND COVER CONSISTS OF CULTIVATED CROPS OR SUBSTANTIAL GROWTHS OF GRASS AND FAIRLY DENSE SMALL SHRUBS, CACTI, OR SIMILAR VEGETATION. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$ = 0.050; DRAINAGE AREA IS QUITE RUGGED, WITH SHARP RIDGES AND NARROW, STEEP CANYONS THROUGH WHICH WATERCOURSES MEANDER AROUND SHARP BENDS, OVER LARGE BOULDERS, AND CONSIDERABLE DEBRIS OBSTRUCTION. THE GROUND COVER, EXLUDING SMALL AREAS OF ROCK OUTCROPS, INCLUDES MANY TREES AND CONSIDERABLE UNDERBRUSH, NO DRAINAGE IMPROVEMENTS EXIST IN THIS AREA.

$\bar{n}$ = 0.030; DRAINAGE AREA IS GENERALLY ROLLING, WITH ROUNDED RIDGES AND MODERATE SIDE SLOPES, WATERCOURSES MEANDER IN FAIRLY STRAIGHT, UNIMPROVED CHANNELS WITH SOME BOULDERS AND LODGED DEBRIS. GROUND COVER INCLUDES SCATTERED BRUSH AND GRASSES. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$ = 0.015; DRAINAGE AREA HAS FAIRLY UNIFORM GENTLE SLOPES WITH MOST WATERCOURSES EITHER IMPROVED OR ALONG PAVED STREETS. GROUND COVER CONSISTS OF SOME GRASSES WITH APPRECIABLE AREAS DEVELOPED TO THE EXTENT THAT A LARGE PERCENTAGE OF THE AREA IS IMPERVIOUS.



TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE

L_{ca} = LENGTH ALONG LONGEST WATERCOURSE, MEASURED UPSTREAM TO POINT OPPOSITE CENTER CENTER OF AREA.

S = OVER-ALL SLOPE OF LONGEST WATERCOURSE BETWEEN HEADWATER AND COLLECTION POINT.

LAG = ELAPSED TIME FROM BEGINNING OF UNIT PRECIPITATION TO INSTANT THAT SUMMATION HYDROGRAPH REACHES 50% OF ULTIMATE DISCHARGE.

$\bar{n}$ = VISUALLY ESTIMATED MEAN OF THE n (MANNING'S FORMULA) VALUES OF ALL THE CHANNELS WITHIN AN AREA.

NOTE:
TO OBTAIN THE LAG (IN HOURS) FOR ANY AREA, MULTIPLY THE LAG OBTAINED FROM THE CURVE BY:

$\frac{\bar{n}}{0.050}$ OR 20n

Figure 5.11

LAG RELATIONSHIPS

U.S. ARMY CORPS OF ENGINEERS (1982)

6

Channel Routing

6.1 General

Channel routing involves generation of an outflow hydrograph for a reach where an inflow hydrograph is specified. A reach is either an open channel with certain geometrical/structural specifications, or a pipe with open channel flow. This type of application assumes that the flow is not confined, and that surface configuration, flow pattern and pressure distribution within the flow depend on gravity. It also assumes that there is no movement of the bed or banks. In addition no backwater effects are considered.

A routing technique is normally required for a multi-basin design where flow is to be moved through time and space from one flow concentration point to the next. For the purposes of this manual two types of open channels, natural and urbanized, are considered. Kinematic Wave Routing is to be applied for urbanized channels since the routing process involves minimal attenuation. Non-pressurized pipe flow will also be through Kinematic Wave Routing procedures. Muskingum Routing is to be used for natural, undeveloped channels since the method simulates outflow peak attenuation resulting from storage in the system. Both Muskingum and Kinematic Wave Routing methods are options in HEC-1 which is again the principle modeling tool of the *Hydrologic Design Manual*. The Modified Puls method which is typically used for routing through a structure or a detention basin is discussed in detail in the *Hydraulic Design Manual*.

6.2 Kinematic Wave Routing

The Kinematic Wave Routing as described in HEC-1 can be applied for routing of overland flow, collector channels and the main channel. However, for the purposes of this manual the overland flow option of the Kinematic Wave will not be used. The overland flow analysis will be performed using the Maricopa County Unit Hydrograph Procedure (MCUHP), described in Chapter 5 of this manual. Once a hydrograph is generated through the MCUHP, it can be used as the inflow hydrograph for an urbanized open channel or a pipe where an outflow hydrograph

is required. These reaches can be treated as collector channels or the main channel as the case may be.

6.2.1 Collector Channel

Modeling of flow at a point where it becomes channel flow to a point where it enters the main channel is done as a collector channel element. It is assumed that the flow along the path of the channel is uniformly distributed. This is a proper assumption for a case when overland flow runs directly into a gutter. It is also a reasonable approximation of the flow as it passes through a storm drain system from a catch basin and the collector pipes along the collector channels.

6.2.2 Main Channel

The main channel element can be used to route inflow from an upstream subbasin or a combination of inflows from collector channels along a subbasin. The flow is assumed to be uniformly distributed, which appears to be a reasonable assumption when the flow is received from collector channels at several locations.

6.2.3 Parameter Selection

The data requirement for channel routing include surface drainage area, channel length and slope, channel shape and geometry, Manning's n, and the inflow hydrograph. The designer is referred to the HEC-1 manual for the proper selection of these parameters.

When working with the Kinematic Wave Method, it is important to be familiar with the computational procedures inherent in the model. In order to solve the governing equations which theoretically describe the Kinematic Wave Method, proper selection of time step and reach length are required. The designer will specify a channel reach length and a computational time step for the inflow hydrograph. This time step could very well be different from the one selected by the computer for computational purposes. Further more, the computer will use this information to select distance intervals based on the given reach length.

The computational process could unrealistically attenuate the outflow peak. It appears that a longer reach length would cause more attenuation. To overcome this problem, the new version of HEC-1 will calculate the outflow peak by applying both the time step selected by the designer as well as the one selected by the computer. If the resulting peaks are not reasonably close, the designer can modify the selected time step or the reach length to improve the calculations. It should be noted that the computer will compare peak flow values for the main channel and not the collector channels.

6.3 Muskingum Routing

Flow routing through natural channels can be accomplished by applying the Muskingum Routing technique. The main characteristic of natural channels with respect to routing is that the outflow peak can be drastically attenuated through storage loss, a process which is simulated by Muskingum routing.

6.3.1 Parameter Selection

Application of Muskingum Routing requires input values for parameters X and K. Parameter X has a range of values 0.0 to 0.5, where 0.0 represents routing through a linear reservoir and 0.5 indicates pure translation. Parameter K indicates ~~travel time~~ the travel time of a flood wave through the entire routed reach. There are several methods which can be used to estimate K such as average flow velocity,* the time difference between peak inflow and peak outflow, or by using stage-discharge relationships. For more details the reader is referred to the HEC-1 manual.* Once again, since the computational method within HEC-1 may result in an unstable solution, parameters K, X, and NSTPS (number of steps) must be checked to insure that an adequate number of subreaches is used.

In those rare situations that observed inflow *and* outflow hydrographs are available, K, X, and NSTPS can be calibrated by trial and error to enable reproduction of outflow hydrographs. ☒

* adjusted by a celerity factor,

* and the Application chapter of this manual.

☐ Chapter 5 of the U.S. Bureau of Reclamation's "Flood Hydrology Manual" (1989) is an excellent source of Muskingum routing information.

7 Application

7.1 General

The methodologies presented in this Manual are, for the most part, standard procedures and practices commonly used in hydrologic ~~design.~~ analysis However, the user of the manual may not always be familiar with these techniques because of a different previous experience or interest. A number of examples were developed to familiarize the user with the presented methods as well as the details of parameter estimation. In addition, this Chapter should provide some general suggestions to facilitate particular applications.

7.2 Notes on Design Rainfall

Examples 1 through 3 illustrate the development of a Depth-Duration-Frequency (D-D-F) table, an Intensity-Duration-Frequency (I-D-F) table, and a rainfall distribution for a particular site. The user does not necessarily have to redesign the rainfall distributions since those presented in the manual are adequate for all of Maricopa County. Chapter 2, Table 2.1, and Pattern 1 of Table 2.2 contain those distributions, which were developed from data at Phoenix Airport. If different distributions are needed, Table 2.1 and Pattern 1 of Table 2.2 can be redeveloped. However, Patterns 2 through 5 are appropriate for all locations without modification.

A particular site might have orographic features, resulting in a 100-year, 6-hour rainfall depth, which differ significantly from the ~~Airport~~ Phoenix Metro area value. In that case, the short duration part of the rainfall such as the 15-minute depth may be different from the one by Pattern 1. This will give a different peak outflow, justifying the design of a new distribution.

As a note to developing D-D-F table, the user can alternatively use PREFRE, a computer program by the National Weather Service. PREFRE will produce the D-D-F Table ~~by performing the computations internally.~~

7.3 Notes on Calculating Loss Parameters

1. Since many of the soil groups contain horizons of different textures, the top horizon may or may not control the total volume and rate of infiltration. The decision of which soil layer controls the infiltration rate is based on soil texture, horizon thickness, and the accumulated depth of water during the initial low-intensity period of a design storm. As a general rule, sandy and loamy sand soils less than 2 inches thick will not act as the controlling horizon during a 100-year design storm.

2. Percent Sand & Gravel: Sand is defined by the SCS as that percentage of the soil matrix between ~~0.5~~ 0.05 and 2.0 mm. in diameter. The SCS Soil Survey books list a percentage of each soil type passing sieve #200, which has openings of 0.074 mm. It can therefore be assumed as an estimate that the percentage of particles retained by this sieve are sand size and larger. It will also be assumed that soils with particle size between 2.0 mm and 3.0 inches (gravel) have infiltration rates greater than or equal to sand. This is necessary because Green & Ampt and IL+ULR loss parameters have not been developed for cobbly, gravelly, channery, etc. soils.

   When choosing the value for percent sand and clay, choose the median value from the range listed in the "Engineering Index Properties" and "Physical and Chemical Properties" tables. For example, if a range of 10 to 35 percent clay is listed, choose 22.5 percent. On rare occasions, the sum of the median values for percent sand and clay will be greater than 100 percent. In this case, adjust both values equally until they total 100. With a known percent sand and clay, enter ~~Figure 4 in the appendix~~ Appendix D to determine the textural class for that particular soil. Then choose Green & Ampt loss parameters from Table 4.2 or IL+ULR parameters from Tables 4.3 and 4.4.

3. Most soil map units consist of major and minor soil areas, as listed in the "General Soil Map Units" sections of the Soil Survey books. The descriptions will list the percentage of each of the major soils, and one or more percentage~~s~~ for ~~all~~ the (usually 2 to 4) minor soils. When calculating weighted averages for the minor soils, when only one percentage is listed, assume an equal contribution from each. For example, if a minor group makes up 20 percent of the map unit and consists of 3 soils, then each group member contributes (20/3)=6.67 percent. *

4. Hydrologic Soil Groups: It is often necessary to check the hydrologic soil group classifications against the textural infiltration rates and the controlling horizon. In some cases, "C" and "D" soils may be so designated because of an underlying hardpan, but it may be at an unreasonable depth given a two or six-hour design storm. In many cases, "D" soils are so designated because of a large percentage of exposed, impervious rock outcrop. When using the IL+ULR loss rate method in HEC-1 with hydrologic soil groups in this situation, do not use the "D" soil loss rate parameters with the impervious cover value (RTIMP), or severe underestimation of losses will occur.

* A weighted value of bare ground XKSAT can be used to calculated a corresponding value of PSIF from Figure 4.5. This Figure eliminates the need to weight PSIF values.

6. ~~5.~~ Hydrologic soil groups can be weighted in the following manner:

A=1 B=2 C=3 D=4

Say a particular soil group is 20 percent B, 25 percent C, and 55 percent D. Then the weighted value is:

$$(.20)(2) + (.25)(3) + (.55)(4) = 3.35$$

Since 3.35 is less than 3.5, round down to 3.0, and choose "C" group loss parameters for this soil group.

7. ~~6.~~ Textural Classes: Textural class descriptions, as used in this context, contain only adjectives from the three primary textures: sand, silt, and clay. To deter-

ADDITIONS

7.3

#5. Use caution when applying impervious cover percentages using the RTIMP variable. Remember that RTIMP will directly convert the assigned percentage of areal excess rainfall to runoff. If the SCS soil description lists a soil group as having 25% rock outcrop, 25% of the area will contribute direct runoff to the outlet only if the rock outcrop areas are hydraulically connected, which is rarely the case. This situation also exists in urban areas, where the impervious areas are streets and driveways rather than rock outcrop. Good judgement should be used to assess flowpaths and the infiltration characteristics of soils adjacent to impervious areas when using the RTIMP variable.

examples, Green & Ampt and IL+ULR (by soil texture) loss parameters have been calculated for Maricopa County soils and are presented in Appendices A, B, and C. Choose the parameters for each soil type within a Map Unit, then calculate a weighted average as in Step 3 of Example 5.

10. There are currently three Soil Survey volumes available for Maricopa County and adjoining areas, generally in the central, eastern, and northern regions. Copies of the Soil Surveys can be obtained from the Soil Conservation Service Field Offices.

7.4 Notes on Calculating Parameters for use in the Clark Unit Hydrograph

1. Tc represents the time for ~~water~~ a floodwave to travel from the hydraulically most distant point in the watershed to the outlet during the most intense period of rainfall excess. The flow path length (L) represents the hydraulic length corresponding to Tc. For a natural channel, L is the length of watercourse from the outlet to a point defining the hydraulically most distant point. For an urban basin where flow is mainly in streets and no primary channels exist, an average flow path should be selected, such as a line parallel to grade from the outlet to the upper watershed boundary.

2. Excess Rainfall Values: When developing the peak period of rainfall excess on the "Calculation of Tc & R" worksheet (Appendix E), start at the highest depth for the Δt used, then choose the largest value above or below the peak, then the value above or below those two, and so on so that a contiguous grouping results. Do not list the depth values in a strictly descending order unless they are contiguous. Example:

Time	Excess(in)		Rank		Sorted
1415	0.21		6		0.40
1420	0.28		5		0.35
1425	0.35		2		0.32
1430	0.40	—>	1	—>	0.33
1435	0.32		3		0.28
1440	0.33		4		0.21
1445	0.18		7		0.18

Alternatively, program "MCUHP1" can be used to calculate Tc and R. This program will also construct the basin HEC-1 input file for immediate application.

3. Worksheet: The worksheet allows a maximum of eight excess rainfall values to be entered, and this is sufficient in most cases. As a result, if Δt = 5 minutes (where Δt is hydrograph time step), then Tc should be less than (8*5)=40 minutes. For Δt = 10 minutes, Tc < 80 minutes, and so on. Remember that in no case should Tc be less than Δt for computational stability.

4. Remember that Tc is a function of excess rainfall intensity and must be recalculated when the duration or frequency of a design storm is changed. If multiple [illegible]ncies are desired for a given duration, it may be acceptable to construct [illegible] vs. Frequency, when the peak producing portion of the distribu- [illegible] In such a case plot the 2, 10, and 100 year Tc values on [illegible] intermediate values.

[illegible]ith overall slopes greater than 200

7.4

#6. In cases where more than one basin roughness exists in a watershed, the basin roughness factor (Kb) should be weighted in the following manner:

Say a 3.75 sq.mi. watershed is 35% "nearly bare ground" and 65% "rough". Calculate Kb separately for each roughness category, then weigh according to percentages, i.e.:

Nearly bare ---> -0.01375 (log 3.75 * 640) + 0.08 = 0.034
Rough ---------> -0.025 (log 3.75 * 640) + 0.15 = 0.065

Kb = (.35)(.034) + (.65)(.065) = 0.054

The [illegible]
realistica[illegible]
cause more a[illegible]
HEC-1 will calcula[illegible]

the designer as well as the one selected by the computer. If the resulting peaks are not reasonably close, the designer can modify the selected time step or the reach length to improve the calculations. It should be noted that the computer will compare peak flow values for the main channel and not the collector channels.

When working with Kinematic Wave Routing, channel capacity must be checked to assure proper conveyance of flow prior to the HEC-1 run. Otherwise, if the channel is undersized, the model will automatically extend channel boundaries to contain the flow.

7.6 Notes on Developing Muskingum Parameters

1. The following parameter estimation procedures apply primarily to natural stream channels which convey a significant amount of flow in the overbank areas during design-frequency events.

2. NSTPS: The choice of a number of subreaches for a particular stream reach can be checked for computational stability using the following equation from the HEC-1 Manual, (Hydrologic Engineering Center, HEC-1, ~~1985~~): 1987

$$\frac{1}{2\,(1-X)} \leq \frac{K}{NSTPS\Delta t} \leq \frac{1}{2\,(X)}$$

where K = the travel time through the entire reach in hours

X = Muskingum 'X'

Δt = the computational time step (hrs),

NSTPS = the integer number of subreaches.

3. K: K is the travel time of the floodwave peak through the entire reach. Calculation using Manning's equation is usually an appropriate method for estimating the ~~floodwave velocity, *V*,~~ Vm, with the following provisions:

   A. Use an average channel area and wetted perimeter for the reach—assume bankfull conditions.

   B. Choose an 'n' value representative of the main channel only—do not include the overbank roughness in a weighted average.

   C. Calculate an average flow velocity for the reach (*V*).

D. Use the following ratios by the U.S. Bureau of Reclamation, 1989, to estimate *Vm*, ~~wave celerity~~: the velocity of the floodwave:

Channel Geometry	Vm/V
Wide rectangular	1.67
Wide parabolic	1.44
Triangular	1.33

The value of K is then estimated by dividing the reach length by *Vm*.

4. X: For wide, shallow channels with low to moderate slopes and significant overbank flow during the design flood being modeled, choose X = 0.15 to 0.25. For steep to very steep, narrow, deep channels with little overbank flow, choose X = 0.25 to 0.40.

7.7 Notes on the Application of S-graphs

The recommended S-graphs for Maricopa County, i.e., Phoenix Mountain and Phoenix Valley S-graphs, should only be applied to large, natural watersheds.* This is in part due to the fact that the original data base in Arizona applied the methodology to large watersheds. As a lower limit of application a watershed area of 5 square miles can be considered, although that should be used as the absolute minimum size.

The manual discusses two slightly different methods of LAG computation, one by the US Corps of Engineers and one by the US Bureau of Reclamation. The recommended method would be the one by the US Corps of Engineers.

Program "MCUHP2" can be used to convert an S-graph into a unit-graph. This program, while similar to LAPRE1 (Section 5.6.1), shall provide the necessary basin HEC-1 file with the appropriate rainfall pattern distribution.

* The Phoenix Valley S-graph can also be applied to large, urban basins.

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Appendix A

Aguila–Carefree Loss Rate Parameters

Agulla–Carefree Loss Rate Parameters

Map Unit #	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (In/hr)	PSIF (In)	DTHETA (Dry)	DTHETA (Normal)	IL (In) (Dry) (Top Horizon)	IL (In) (Normal) (Top Horizon)	H.S.G.
1	Antho Calc. & Non.	Sandy Loam (0-3)	65	10	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
2	Antho Calc. & Non.	Gravelly Sandy Loam (0-3)	75	10	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
3, 4	Antho	Sandy Loam (0-3)	65	10	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Carrizo	Very Gravelly Sand (0-28) Sandy	92.5	5	Sand	4.60	1.9	0.35	0.30	1.3	1.3	A
	Maripo	Loam (0-18)	65	10	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
5	Anthony	Very Gravelly Sandy Loam (2-40)	80	7.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B
6, 7	Anthony	Very Gravelly Sandy Loam (2-40)	80	7.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B
	Arizo	Very Gravelly Sandy Loam (1-8)	85	12.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	A
8	Arizo	Very Cobby Sandy Loam (1-8)	70	12.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	A
9	Beeline	Sandy Loam (1-9)	57.5	17.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
	Cipriano	Very Gravelly Loam (0-6)	72.5	20	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
10, 11	Brios	Gravelly Coarse Sand (2-60) ~~Gravelly~~	87.5	8.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	A
	Carrizo	Coarse Sand (2-60)	92.5	2.5	Sand	4.60	1.9	0.35	0.30	1.3	1.3	A
12	Carefree	Cobbly Clay Loam (0-1)	35	35	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	D*

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13	Carefree	Cobbly Clay (0-1)	25	47.5	Clay	0.01	12.4	0.15	0.05	0.3	0.2	D
	Beardsley	Cobbly Clay Loam (0-2)	37.5	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	C
14	Carrizo	Gravelly Coarse Sand (1-60)	92.5	2.5	Sand	4.60	1.9	0.35	0.30	1.3	1.3	A
15	Carrizo	Gravelly Sandy Loam (0-5)	75	10	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	A
	Gunsight	Very Gravelly Loam (1-60)	80	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
16, 17	Cellar	Very Gravelly Sandy Loam (0-3)	80	17.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
18	Cherioni	Very Gravelly Sandy Loam (1-10)	87.5	12.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	D*
19, 20	Chuckawalla	Extremely Gravelly Loam (0-2)	77.5	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Gunsight	Very Gravelly Loam (0-3)	77.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
21	Cipriano	Very Gravelly Loam (0-6)	72.5	20	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
22, 23	Contine	Clay Loam (0-2)	22.5	36	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
24	Continental	Clay Loam (0-1)	37.5	27.5	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
25	Continental	Clay (0-1)	22.5	45	Clay	0.01	12.4	0.15	0.05	0.3	0.2	C*
26	Continental	Cobbly Clay Loam (0-2)	40	31	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
27	Continental	Clay Loam (0-1)	37.5	27.5	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
	Mohave	Sandy Loam (0-2)	65	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B

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28	Continental	Cobbly Clay Loam (0-2)	40	31	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
	Ohaco	Gravelly Loam (0-2)	52.5	23.5	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
29, 30	Denure	Fine Sandy Loam (0-2)	65	10								B
	Momoli	Gravelly Sandy Loam (0-10)	75	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Carrizo	Gravelly Sandy Loam (0-10)	75	10								A
31, 32	Dixaleta	Extremely Channery Sandy Loam (1-8)	83.75	16.25	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
33, 34, 35	Eba	Very Gravelly Loam (0-3)	75	13	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	C*
36	Eba	Very Gravelly Loam (0-3)	75	13	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	C*
	Continental	Clay Loam (0-1)	37.5	27.5	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
37, 38	Eba	Very Gravelly Loam (0-3)	75	13	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	C*
	Continental	Clay Loam (0-1)	37.5	27.5	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
	Cave	Loam (1-14)	50	12.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	D*
39	Eba	Very Gravelly Loam (0-3)	75	13	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	C*
	Nickel	Gravelly Loam (1-10)	65	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Cave	Loam (1-14)	50	12.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	D*

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40, 41,	Eba	Very Gravelly Loam (0-3)	75	13	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	C*
42, 43	Pinaleno	Very Gravelly Clay Loam (0-1)	65	35	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
44, 45	Ebon	Very Gravelly Sandy Clay (1-43)	65	35	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
46	Ebon	Very Gravelly Sandy Clay (1-43)	65	35	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
	Contine	Clay Loam (0-2)	22.5	36	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
47	Ebon	Very Gravelly Sandy Clay (1-43)	65	35	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
	Gunsight	Very Gravelly Sand Loam (0-3)	85	12.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B
	Cipriano	Very Gravelly Loam (0-8)	72.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
48, 49	Ebon	Very Gravelly Sandy Clay (1-43)	65	35	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
	Pinampt	Extremely Gravelly Sandy Loam (0-3)	88.75	11.25	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B
50	Estrella	Loam (0-21)	40	17.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
51, 52	Gachado	Very Gravelly Sandy Clay Loam (2-8)	77.5	22.5	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
	Lomitas	Very Gravelly Sand Loam (2-17)	80	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
53	Gadsden	Clay (0-3)	15	52.5	Clay	0.01	12.4	0.15	0.05	0.3	0.2	D
54	Gila	Fine Sandy Loam (0-2)	42.5	15	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
55, 56	Gilman	Loam (0-5)	35	23.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
57	Gilman	Clay Loam (0-11)	25	32.5	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*

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58, 59	Gilman	Loam (0-2)	35	23.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
	Momoli	Gravelly Sandy Loam (0-22)	75	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Denure	Gravelly Sandy Loam (0-9)	75	12.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
60	Glenbar	Loam (0-6)	25	23.5	Silty Loam	0.15	6.6	0.40	0.25	0.8	0.7	B
61, 62,	Gran	Very Gravelly Sandy Clay (1-12)	66.25	33.75	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
63, 64	Wickenburg	Very Gravelly Sandy Loam (1-12)	77.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
65	Grey Eagle	Very Gravelly Loam (0-5)	67.5	14	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
	Continental	Loam (0-2)	37.5	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	C
	Nickel	Very Gravelly Loam (0-5)	70	17.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
66	Grey Eagle	Very Gravelly Loam (0-5)	67.5	14	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
	Sun City Variant	Clay Loam (2-9)	50	30	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
67	Guest	Clay (0-2)	15	45	Clay	0.01	12.4	0.15	0.05	0.3	0.2	D
68, 69	Gunsight	Very Gravelly Loam (1-60)	81.25	18.75	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Cipriano	Very Gravelly Loam (0-6)	72.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
70, 71	Gunsight	Very Gravelly Loam (0-11)	77.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Rillito	Gravelly Loam (0-12)	60	14	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B

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72, 73	Lehmans	Very Gravelly Clay Loam (0-2)	70	30	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
74	Luke	Very Gravelly Sandy Clay (1-28)	63.75	36.25	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
	Cipriano	Very Gravelly Loam (0-6)	72.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
75, 76,	Mohall	Loam (0-7)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
77, 78,		Loam (0-7)	32.5	22.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
79,		Clay Loam (0-2)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
		Clay Loam (0-6)	32.5	35	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
		Clay (0-12)	20	45	Clay	0.01	12.4	0.15	0.05	0.3	0.2	B*
80, 81	Mohall	Loam (0-2)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
	Tremant	Clay Loam (1-5)	45	32.5	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
82, 83,	Mohave	Clay Loam (2-20)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
84, 85,		Loam (0-2)	30	23.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
86		Loam (0-2)	35	17.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
		Clay Loam (0-6)	25	35	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
		Clay Loam (0-2)	27.5	33.5	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
87	Mohave (Noncalc)	Clay Loam (2-20)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
	Mohave (Calc)	Loam (2-20)	35	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B

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88	Mohave	Clay Loam (2-20)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
	Guest	Clay Loam (0-2)	20	35	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	D
89	Mohave	Clay Loam (2-20)	25	27.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
	Tres Hermanos	Gravelly Loam (0-2)	40	17.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
90	Momoli	Gravelly Sandy Loam (0-3)	75	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
91, 92	Momoli	Very Gravelly Loam (1-60)	77.5	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Carrizo	Very Gravelly Sandy Loam (0-11)	85	10	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	A
93, 94	Nickel	Gravelly Loam (1-10)	65	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Cave	Loam (1-14)	50	12.5	Loam	0.25	4.3	0.35	0.25	0.6	0.5	D*
95	Ohaco	Clay (2-27)	27.5	40	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
96, 97	Pinaleno	Gravelly Clay Loam (1-12)	60	35	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
	Tres Hermanos	Very Gravelly Clay Loam (2-22)	47.5	25	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
98, 99	Pinamt	Very Gravelly Sandy Clay Loam (1-28)	76.25	23.75	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.5	0.4	B*
	Tremant	Gravelly Loam (0-5)	65	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
100	Quilotosa	Extremely Gravelly Loam (2-14)	88.75	11.25	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	D*
	Vaiva	Very Gravelly Loam (0-3)	67.5	12.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*

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101, 102	Rillito	Loam (0-24)	57.5	14	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Rillito	Gravelly Loam (0-14)	60	14	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
103	Gachado	Very Gravelly Sandy Clay Loam (1-7)	73.75	26.25	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
104, 105	Lehmans	Very Gravelly Clay Loam (0-2)	70	30	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
106, 107	Sal	Very Gravelly Clay Loam (2-20)	71.75	27.75	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
	Cipriano	Very Gravelly Sandy Loam (0-20)	72.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
108, 109	Schenco	Very Channery Loam (2-11)	70	21	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
110	Sun City	Clay Loam (1-9)	45	30	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
	Cipriano	Very Gravelly Loam (0-6)	72.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
Add #111 112, 113, 114	Tremant	Gravelly Sandy Loam (0-9)	65	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
115	Tremant	Gravelly Sandy Loam (0-9)	65	15	Sandy Loam	0.40	4.3	0.35	0.25	0.7	0.6	B
	Antho	Sandy Loam (0-3)	65	10	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
116, 117	Tremant	Clay Loam (2-26)	45	32.5	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
	Gunsight	Very Gravelly Sandy Loam (0-10)	85	12.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B*
	Rillito	Gravelly Loam (2-60)	55	20.	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B

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118	Tremant	Gravelly Sandy Loam (0-9)	65	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Rillito	Gravelly Loam (0-12)	60	14	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
119	Tremant	Gravelly Loam (0-9)	65	15	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
	Sun City	Clay Loam (2-12)	45	30	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
120	Tres Hermanos	Very Gravelly Loam (2-22)	50	32.5	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
121	Tres Hermanos	Very Gravelly Clay Loam (2-22)	50	32.5	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
	Anthony	Very Gravelly Sandy Loam (2-40)	80	7.5	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B*
122	Vado	Gravelly Sandy Loam (0-60)	80	13	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
123	Vaiva	Very Gravelly Loam (0-3)	67.5	20	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
124	Valencia	Sandy Loam (0-20)	67.5	12.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
125	Vint	Loamy Fine Sand (0-60)	70	7.5	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B

* WARNING: Hydrologic soil group does not accurately represent soil texture characteristics. Check soil description for rock outcrop, cemented hardpan, soil group associations, percent course fraction, etc.

Appendix B

Eastern Maricopa/ Northern Pinal Counties Loss Rate Parameters

Eastern Maricopa/Northern Pinal Counties Loss Rate Parameters

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
Af, Ag	Agault	Loam (0-27)	45	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
Am	Alluvial Land	Gravelly Sand (0-60)	55 to100	–	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	A
AnA, AnB, AoB	Antho	Sandy Loam, Gravelly Sandy Loam (0-46)	72.5	9	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Av	Avondale	Clay Loam (0-13)	25	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
Ca, Cb	Carrizo	Fine Sandy Loam, (0-15) & Gravelly Loamy Sand	57.5	–	Sandy Loam	0.40	3.5	0.35	0.30	0.7	0.6	A*
Cc	Cashion	Clay (0-28)	10	–	Clay	0.01	12.4	0.15	0.05	0.3	0.2	C*
CeC	Cavelt	Gravelly Loam (0-10)	50	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	D*
Co	Contine	Clay Loam (0-12)	30	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
Es	Estrella	Loam (0-26)	35	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
Gf, Gm	Gilman	Loam (0-13)	27.5	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
Gn	Glenbar	Clay Loam (0-14)	15	–	Silty Loam	0.15	6.6	0.40	0.25	0.8	0.7	B
Gr	Gravelly Alluvial Land	Very Gravelly Sandy Loam & Loamy Sand (0-60)	90	–	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	A
LaA, LaB, LeA	Laveen	Loam (0-14)	30	–	Silty Loam	0.15	6.6	0.40	0.25	0.8	0.7	B

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
Mo, Mv	Mohall	Loam (0-15)	40	21	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
Pm	Pimer	Clay Loam (0-15)	15	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
PnA, PnC	Pinal	Gravelly Loam (0-5)	50	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	D*
Po	Pinal Variant	Loam (0-9)	45	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	C
PvA, PvC	Pinamt	Gravelly Loam (0-13)	85	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
R1A, R1b	Rillito	Gravelly Loam (0-13)	62.5	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Ro, Ru	Rock Land	50 - 70% Rock Outcrop, Shallow Areas of Clay Loam, Sandy Loam, and Gravelly Loam.	–	–	Use Sandy Loam For Pervious Areas	0.40	3.5	0.35	0.25	0.7	0.6	B
TrB	Tremant	Gravelly Sandy Clay Loam (1-15)	60	–	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
Tx	Trix	Clay Loam (0-14)	15	–	Silty Clay Loam	0.04	10.8	0.30	0.15	0.6	0.5	B*
Va	Valencia	Sandy Loam (0-13)	65	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Ve	Vecont	Clay (0-14)	15	48	Clay	0.01	12.4	0.15	0.05	0.3	0.2	C*
Vf	Vint	Loamy Fine Sand (0-12)	80	–	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B*

WARNING: Hydrologic soil group does not accurately represent soil texture characteristics. Check soil description for rock outcrop, cemented hardpan, soil group associations, percent course fraction, etc.

Appendix C

Maricopa County–Central Part Loss Rate Parameters

Maricopa County–Central Part Loss Rate Parameters

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
Aa	Agualt	Loam (0-27)	40	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
AbA, AbB, AdA, AdB, Ae, AfA, AfB, AGB, AHC, AkB, AL, AM, Ac	Antho	Sandy Loam, Gravelly Sandy Loam (0-60)	77.5	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
An	Avonda	Clay Loam (0-13)	25	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
Ao, Ap	Avondale	Clay Loam (0-12)	25	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
BE	Beardsley	Clay Loam (0-10)	25	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
Br, Bs, Bt	Brios	Sandy Loam (0-14)	70	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	A*
CA2	Calciorthids	Loamy Sand To Clay Loam	–	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Cb, CeD, CF	Carrizo	Gravelly Sandy Loam (0-5)	75	–	~~Loamy Sand~~ Sandy Loam	~~1.20~~ .40	~~2.4~~ 4.3	0.35	0.30	0.8	0.8	A
Cg, Ch, Ck, Cm	Casa Grande	Clay Loam (1-22)	49	15	Loam	0.25	~~4.3~~ 3.5	0.35	0.25	0.6	0.5	C
Cn	Cashion	Clay (0-27)	12.5	–	Clay	0.01	12.4	0.15	0.05	0.3	0.2	D

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
CO	Cherioni	Very Gravelly Loam (1-6)	77.5	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	D*
Cp, CrB, Cs, CV	Coolidge	Sandy Loam (0-13)	70	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Dn	Dune Land	—	–	–	Sand	4.60	1.9	0.35	0.30	1.3	1.3	A
EbD, EPD	Ebon	Very Cobbly Clay Loam (2-13)	62.5	–	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	C
Es, Et	Estrella	Loam (0-24)	30	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
GA	Gachado	Very Gravelly Clay Loam (0-1)	67.5	–	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	D*
Gb, Gc, Gd	Gadsden	Clay (0-10)	15	–	Clay	0.01	12.4	0.15	0.05	0.3	0.2	D
Ge, GgA, GgB, GM, GN, Gf, Gh, GL, Go3	Gilman	Loam (0-36)	52.6	10.6	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
Gp	Gilman Variant	Loam (0-3)	30	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	C
Gr, Gt, Gv, Gs, Gu	Glenbar	Clay Loam (0-15)	30	–	Silty Loam	0.15	6.6	0.40	0.25	0.8	0.7	B

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
GWD, GxA, GxB, GYD	Gunsight	Gravelly Loam (0-60)	82.5	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
HAB, HAC, HLC, HM, HrB	Harqua	Gravelly Clay Loam (0-14)	37	37	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C
La	La Palma	Very Fine Sandy Loam (0-7)	41	6	Silty Loam	0.15	6.6	0.40	0.25	0.8	0.7	C
Lb, LcA, Le, Ld, Lf	Laveen	Loam (0-6)	52	13	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B
Ma	Maripo	Sandy Loam (0-13)	65	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Mo, Mp, Mr, Ms, MTB, MV	Mohall	Clay Loam (0-12)	25	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
Pa, PeA, Pb, PeB, PRB	Perryville	Gravelly Loam (0-9)	63	18	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
PsA, PsB, PT, PvB, PWB	Pinal	Loam (0-8)	50	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	D*
PYD	Pinamt	Very Gravelly Sand Loam (2-6)	77.5	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
RaA, RaB, RbA, RbB, RhB, RpE	Rillito	Gravelly Loam (0-60)	65	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
—	Sun City	Clay Loam (0-13)	35	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	D*
Ta	Toltec	Loam (0-28)	32.5	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	C
Tc, TB, TD	Torri-	Cobby Sandy Loam	–	–	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	A
Te, TfA, TfB, Tg, Th, TPB, TrA, TrB, TSC	Tremant	Clay Loam (1-8)	57.5	–	Sandy Clay Loam	0.06	8.6	0.25	0.15	0.6	0.5	B*
Tt	Trix	Clay Loam (0-10)	22.5	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	B*
Tu, Tw	Tucson	Loam (0-14)	32.5	–	Loam	0.25	4.3	0.35	0.25	0.6	0.5	B

Map Symbol	Soil Name	U.S.D.A. Soil Texture (Control Horizon)	Fragments >0.0074 mm, %	Clay, %	Textural Class (Appendix)	XKSAT/ CNSTL (in/hr)	PSIF (in)	DTHETA (Dry)	DTHETA (Normal)	IL (in) (Dry) (Top Horizon)	IL (in) (Normal) (Top Horizon)	H.S.G.
Va, Vb, Vc	Valencia	Sandy Loam (0-10)	70	–	Sandy Loam	0.40	3.5	0.35	0.25	0.7	0.6	B
Ve, Vf	Vecont	Clay (0-15)	17.5	–	Clay	0.01	12.4	0.15	0.05	0.3	0.2	D
Vg, Vh, Vk, Vn, Vr	Vint	Loamy Fine Sand (0-27)	80	–	Loamy Sand	1.20	2.4	0.35	0.30	0.8	0.8	B*
Wg	Wintersburg	Clay Loam (0-12)	25	–	Clay Loam	0.04	8.2	0.25	0.15	0.5	0.4	C

* WARNING: Hydrologic soil group does not accurately represent soil texture characteristics. Check soil description for rock outcrop, cemented hardpan, soil group associations, percent course fraction, etc.

Appendix D

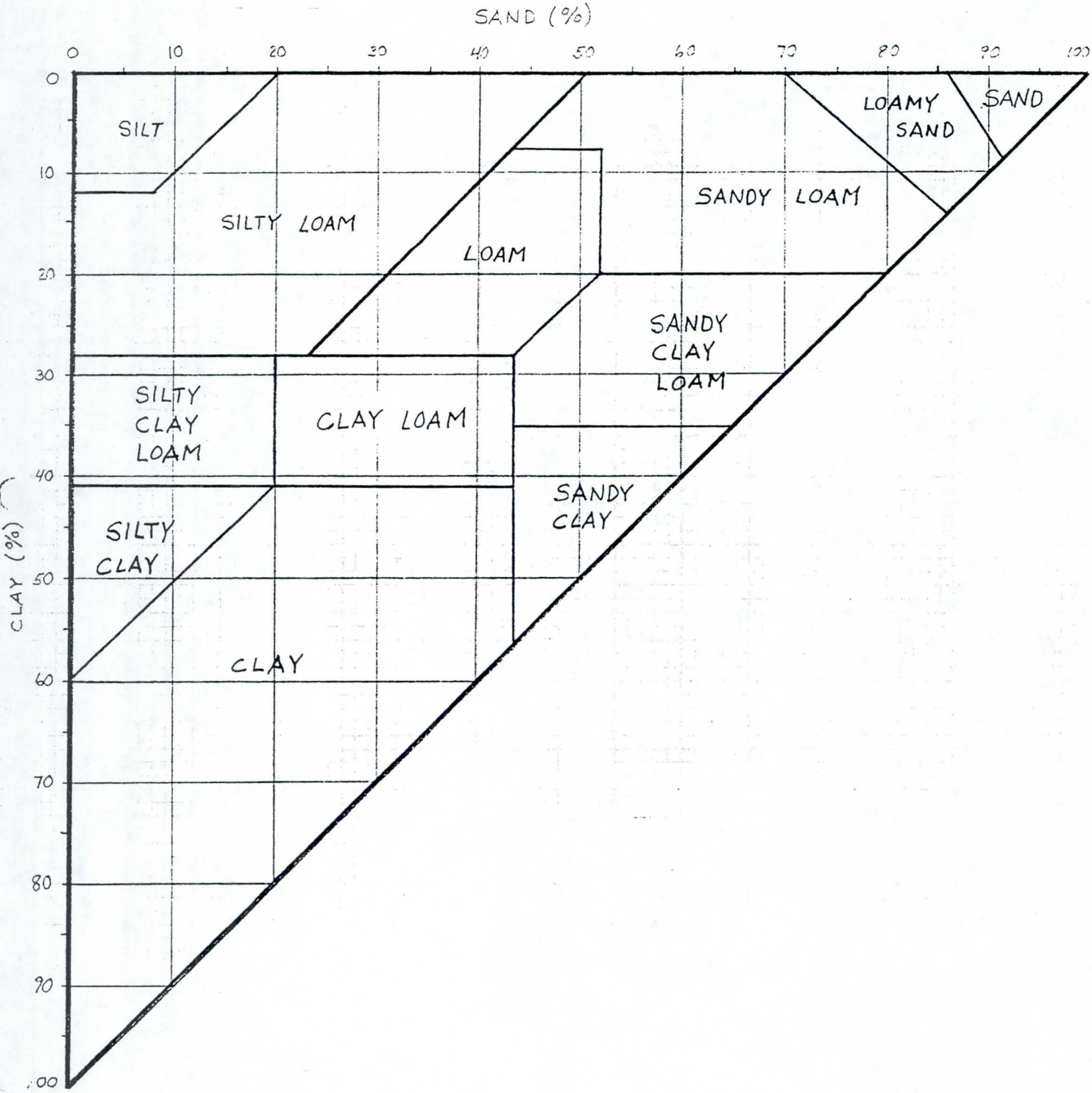
Textural Classes

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MCFCD HYDROLOGY MANUAL PAGE ___ OF ___

DETAIL ___ COMPUTED ___ DATE ___

___ CHECKED BY ___ DATE ___



Appendix E

Tc and R Worksheet

CALCULATION OF Tc & R

Calculated by: _____ Date: _____
Checked by: _____ Project: _____

Watershed: _____
Rainfall Frequency: _____ - yr Duration: _____ - hr. Pattern #: _____

Rainfall Loss Method:
[ ] Green & Ampt Method
[ ] IL + ULR by soil texture
[ ] IL + ULR by hydrologic soil group

Tabulate Period of Peak Rainfall Excess

Clock Time @ end of Increm.	Increm. Excess in.

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

Accum. Time hr./min.	Increm. Excess in.	Accum. Excess in.	Avg. Excess Intensity in./hr.

A = _____ sq.mi.
L = _____ mi.
S = _____ ft/mi.

$Kb = m\,[\log(A * 640)] + b$

Kb = ( ) log (*640) + ( )

Kb = _____

$Tc = 11.4\,L^{.50}\,Kb^{.52}\,S^{-.31}\,i^{-.38}$

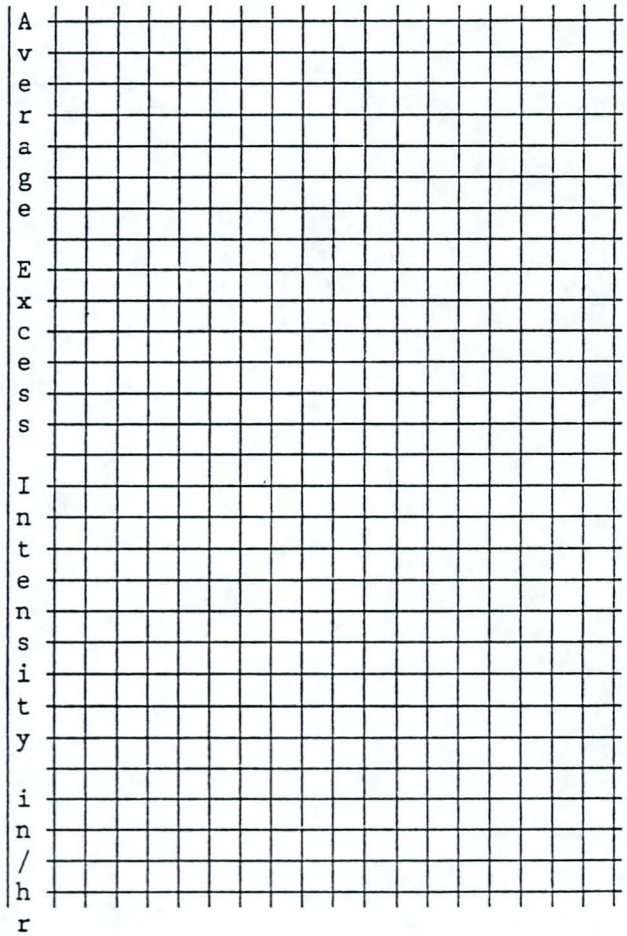
Tc = (_____) $i^{-.38}$

Trial Tc	i	Calc. Tc

Tc = hr.

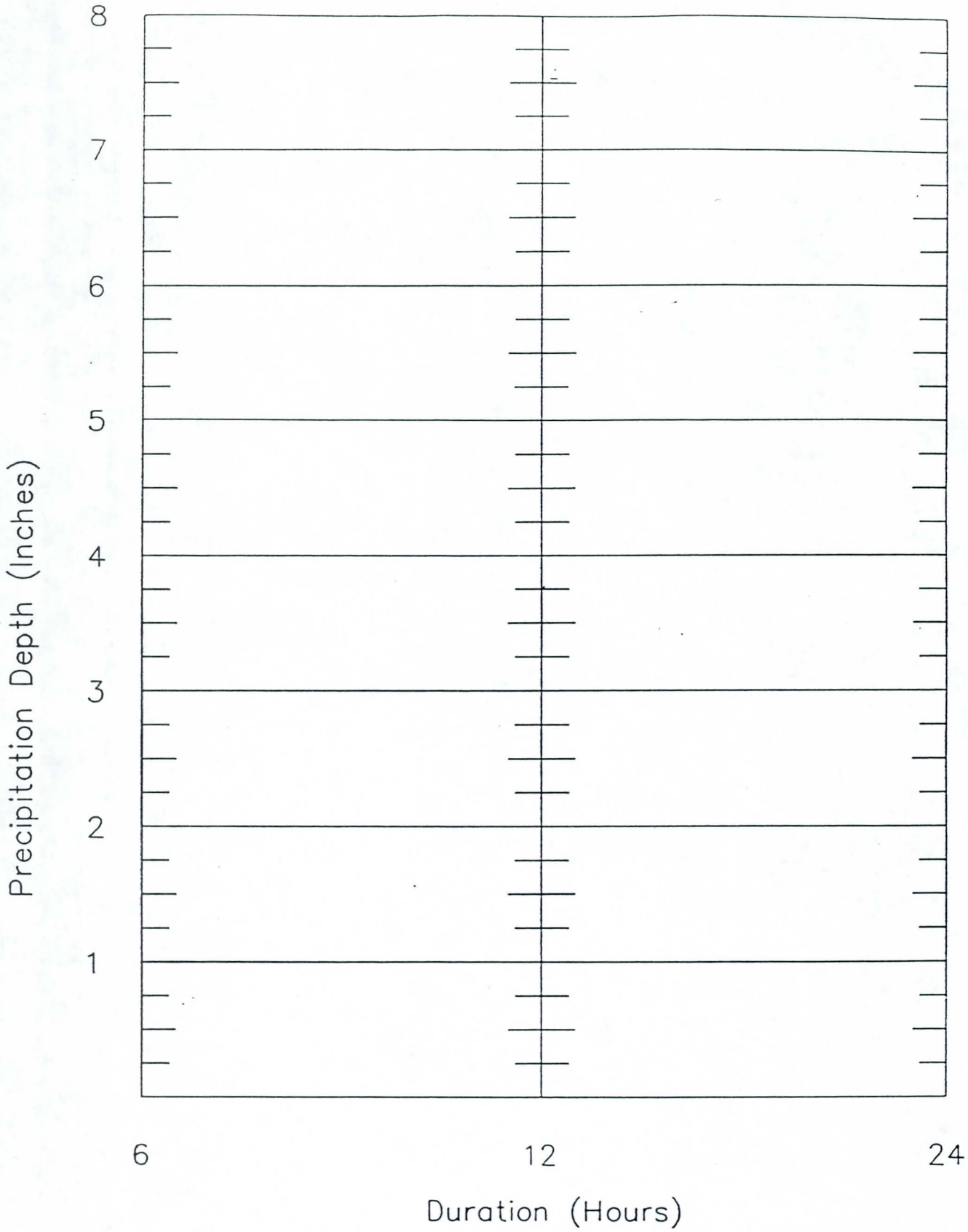
$R = .37\,Tc^{1.11}\,A^{-.57}\,L^{.80}$

R = hr.



Appendix F

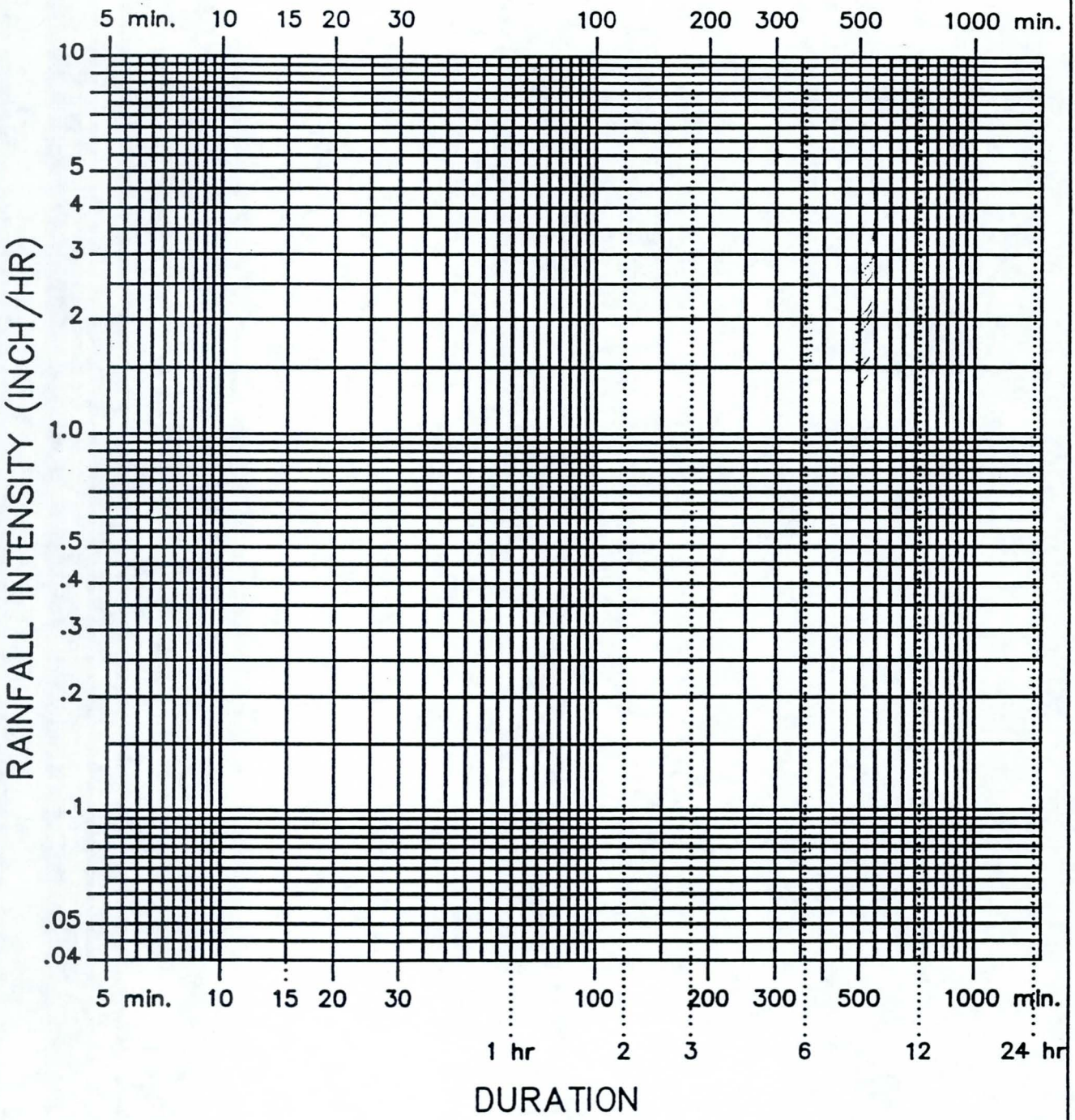
Precipitation Depth-Duration Diagram (6-24 hour)



Precipitation depth–duration diagram (6 to 24 hr).

Appendix G

Rainfall I-D-F Relation (blank)



RAINFALL INTENSITY–DURATION–FREQUENCY RELATION
FOR MARICOPA COUNTY, ARIZONA

Use updated originals for printing.

Appendix H

Method Comparison

The Flood Control District staff has conducted a comparison of different hydrologic methods for tutorial purposes. The results from these comparisons support a major objective for developing the *Drainage Design Manual of Maricopa County:* the standardization of drainage analyses. This alleviate problems that occur after a developed parcel is annexed. The comparisons are summarized below.

Three separate applications were considered for making a comparison of hydrologic analyses. The first looks at a small urbanized watershed using several different methodologies, but primarily the Rational Method, and is summarized in Table 1.

Table 1
Peak Discharge from a Small Urban Watershed

Tr* years	1 Maricopa Co. Rational, cfs	2 Phoenix Rational, cfs	3 Maricopa Co. U.H.P.**, cfs	4 Phoenix Computer, cfs	5 Phoenix SCS***, cfs
2	29	29	11 ~~13~~	20	7
5	48	41	21 ~~27~~	44	17
10	60	48	32 ~~41~~	61	26
25	78	57	48 ~~59~~	86	42
50	93	67	79 ~~88~~	105	53
100	110	74	103 ~~112~~	126	68

*Tr = Return Frequency.
**U.H.P. = Unit Hydrograph Procedure.
***SCS = Soil Conservation Service.

For virtually all return periods the Maricopa rational method generates higher peak discharges than that being used by the City of Phoenix. However, in most instances, these figures are not overly conservative when compared to recorded data. The significance of this difference depends on which return frequency is used and for what purpose.

The second application compares retention requirements for various cities with those outlined in the *Hydrologic Design Manual*, and is summarized in Table 2.

Table 2
Comparison of Retention Requirements

City	City Method		Maricopa County Method	
	Q100 *, cfs	V**, ac-ft	Q100, cfs	V, ac-ft
Chandler	188	13.19	144	7.38
Glendale	105	7.74	150	6.91
Mesa	144***	11.34	147	7.29
Phoenix	138	7.74	154	7.88
Scottsdale	208	~~2.44~~ 10.23	189	7.68
Tempe	138	15.80	147	7.10

*One-hundred Year Peak Discharge.
**Volume.
***Fifty Year Peak Discharge.

There is a 50 percent difference in discharge values between municipalities, as compared to a 24 percent difference using the *Hydrologic Design Manual*; and there is an ~~85~~ 51 percent difference in volumes between cities, as compared to a 12 percent difference using the *Hydrologic Design Manual*. The significance of the difference becomes important when the runoff from one jurisdiction impacts another.

The third comparison looks at the application of hydrologic methods on a large watershed for floodplain delineation purposes, and is summarized in Table 3. This table indicates that current methodologies may be too conservative, thereby significantly reducing the amount of potentially developable property when delineating floodplains.

Table 3
Comparison of Flow Frequencies for a Large Watershed

Method of Analysis	10 Year Discharge, cfs	50 Year Discharge, cfs	100 Year Discharge, cfs
Hydrologic Design Manual	18,455	26,801	29,790
SCS Methods	20,601	32,975	36,841
Flow Frequency	9,000	22,000	28,000

If a further understanding of the results is needed before a decision is made on whether or not to accept the impacts from these differences, please contact us. The Flood Control District will make every attempt to present the Drainage Design Manual in a comprehendable format.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 1 OF 7

DETAIL RETENTION REQUIREMENTS COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

FOLLOWING IS A COMPARISON OF THE METHODOLOGIES USED BY SIX DIFFERENT CITIES IN MARICOPA COUNTY TO CALCULATE RETENTION VOLUMES, AND THE MARICOPA COUNTY METHOD AS APPLIED TO SPECIFIC LOCATIONS IN EACH CITY. A BASIN INLET CULVERT WILL ALSO BE SIZED USING THE 100-YEAR FREQUENCY STORM FOR EACH JURISDICTION WHERE POSSIBLE.

FOR EASE OF COMPARISON, THE YOUNGTOWN WATERSHED WILL BE USED IN SLIGHTLY MODIFIED FORM:

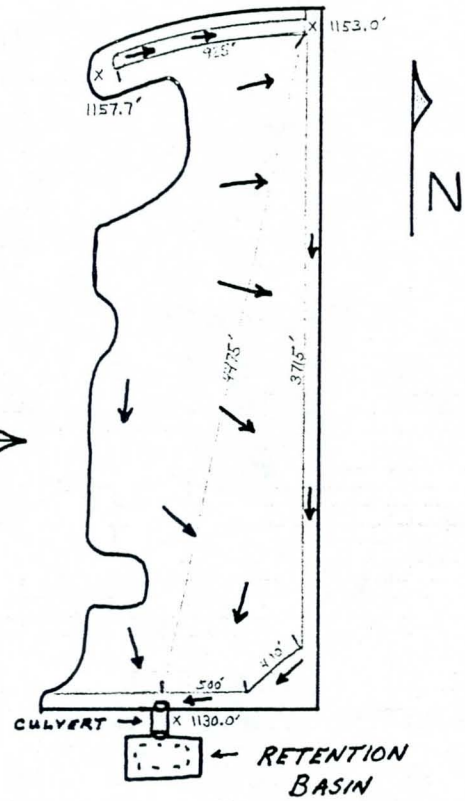
$A = 0.13\ mi^2 = 83.2\ ac$

ZONING: R1-6 (PHX), R1-7 (Scottsdale)

LAND USE: ROOFS - 10%
PAVEMENT - 10%
DESERT LANDSCAPE - 40%
LAWNS - 30%
BARE GROUND - 10%

SOIL TYPE: VARIES WITH LOCATION

AVERAGE OVERLAND FLOW LENGTH: 130'



① CITY OF CHANDLER

SOURCE: CITY OF CHANDLER TECHNICAL DESIGN MANUAL #3 - STORM DRAINAGE SYSTEM DESIGN 1987

CITY METHOD: RATIONAL EQUATION

BASIN OUTLET LOCATION: WARNER RD. AT ARIZONA AVE

$C = 0.65$ (DETACHED SINGLE FAMILY)

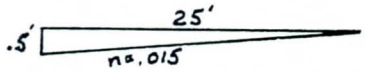
RAINFALL SOURCE: USWB MAPS IN ADOT DRAINAGE MANUAL

$P_{100}^{6} = 3.00\ in$

$P_{100}^{24} = 3.40\ in \rightarrow P_{100}^{1} = .494 + .755 \frac{(P_{100}^{6})^2}{P_{100}^{24}} = 2.49\ in$

$T_c = t_i + t_t \qquad t_i = \frac{.04593\,(130)^{.77}}{(.5)^{.385}} = 2.55\ min$

t_t: Use Mannings' EQ.



$A = .5(25)\tfrac{1}{2} = 6.25\ ft^2$

$P = .5 + \sqrt{.5^2 + 25^2} = 25.505\ ft$

$R = A/P = .245$

$V = \frac{1.49}{.015}(.245)^{.67}(.005)^{.5} = 2.75\ ft/s$

$t_t = L/60V = 5540/60(2.75) = 33.58\ min$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT *METHOD COMPARISON* PAGE *1* OF *7*

DETAIL *RETENTION REQUIREMENTS* COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

Reverse the order of "Retention Requirements" and "Youngtown Watershed" to match the order of the tables in the Method Comparison text.

FOLLOWING IS A COMPARISON OF T
METHODOLOGIES USED BY SIX DIFFER
CITIES IN MARICOPA COUNTY TO CALC
RETENTION VOLUMES, AND THE MARIC
COUNTY METHOD AS APPLIED TO SPECIF
LOCATIONS IN EACH CITY. A BASIN IN
CULVERT WILL ALSO BE SIZED USING
100-YEAR FREQUENCY STORM FOR EAC
JURISDICTION WHERE POSSIBLE.

FOR EASE OF COMPARISON, THE YOUNGT
WATERSHED WILL BE USED IN SLIGHTLY
MODIFIED FORM:

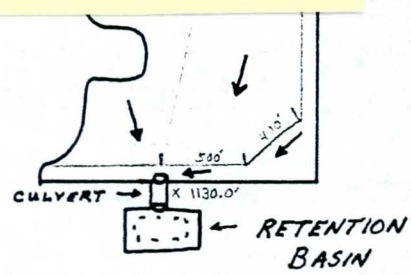
$A = 0.13\ mi^2 = 83.2\ ac$

ZONING: R1-6 (PHX), R1-7 (Scottsdale)

LAND USE: ROOFS - 10%
PAVEMENT - 10%
DESERT LANDSCAPE - 40%
LAWNS - 30%
BARE GROUND - 10%

SOIL TYPE: VARIES WITH LOCATION

AVERAGE OVERLAND FLOW LENGTH: 130'



① CITY OF CHANDLER

SOURCE: CITY OF CHANDLER TECHNICAL DESIGN MANUAL #3 - STORM DRAINAGE SYSTEM DESIGN 1987

CITY METHOD: RATIONAL EQUATION

BASIN OUTLET LOCATION: WARNER RD. AT ARIZONA AVE

$C = 0.65$ (DETACHED SINGLE FAMILY)

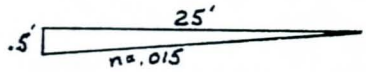
RAINFALL SOURCE: USWB MAPS IN ADOT DRAINAGE MANUAL

$P_{100}^{6} = 3.00\ in$

$P_{100}^{24} = 3.40\ in \rightarrow P_{100}^{1} = .494 + .755 \frac{(P_{100}^{6})^2}{P_{100}^{24}} = 2.49\ in$

$T_c = t_i + t_t \qquad t_i = \frac{.04593\,(130)^{.77}}{(.5)^{.385}} = 2.55\ min$

t_t: Use Mannings' EQ.



$A = .5(25)\tfrac{1}{2} = 6.25\ ft^2$

$P = .5 + \sqrt{.5^2 + 25^2} = 25.505\ ft$

$R = A/P = .245$

$V = \frac{1.49}{.015}(.245)^{.67}(.005)^{.5} = 2.75\ ft/s$

$t_t = L/60V = 5540/60(2.75) = 33.58\ min$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 2 OF 7

DETAIL RETENTION REQUIREMENTS COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

CITY OF CHANDLER CONTINUED

$T_c = t_i + t_t = 36.13$ min

AT $T_c = 36.13$ min. and

$P_{100}^1 = 2.49$ in, $i = 3.48$ in/hr

$Q_{100} = (.65)(3.48)(83.2) = \underline{188 \text{ CFS}}$

VOLUME : $V = 1.1\left[CA\,\frac{P_{100}^2}{12}\right]$

$P_{100}^2 = .341\,(P_{100}^6) + .659\,(P_{100}^1) = 2.66$ in

$V = 1.1\left[(.65)(83.2)\left(\frac{2.66}{12}\right)\right] = \underline{13.19 \text{ ac-ft}}$

MARICOPA METHOD FOR CHANDLER

$C = 0.40$ (Single Family Residential)

$T_c = 11.4\,L^{.5} K_b^{.52} S^{-.31} i^{-.38}$

$T_c = .646\, i^{-.38}$

$L = 1.023$ mi

$K_b = -.00625\,(\log 83.2) + .04 = 0.028$

$S = 27.08$ ft/mi

$P_{10}^6 = 1.87$ in

TRY $T_c = 20$ min, $i_p = 5.1$ in/hr, $i_{100} = (5.1)(1.87)/2.07 = 4.61$ in/hr

$T_c = .646\,(4.61)^{-.38} = 21.69$ min. → NO GOOD

TRY $T_c = 23$ min, $i_p = 4.7$ in/hr, $i_{100} = (4.7)(1.87)/2.07 = 4.25$ in/hr

$T_c = .646\,(4.25)^{-.38} = 22.4$ min

TRY $T_c = 22$ min, $i_p = 4.8$ in/hr, $i_{100} = (4.8)(1.87)/2.07 = 4.34$ in/hr

$T_c = .646\,(4.34)^{-.38} = 22.2$ min. OK

$Q_{100} = (.40)(4.34)(83.2) = \underline{144 \text{ CFS}}$

VOLUME : $V = CA\left(\frac{P_{100}^2}{12}\right)$ $P_{100}^2 = .341\,(P_{100}^6) + .659\,(P_{100}^1) = 2.66$ in

$V = (.40)(83.2)\left(\frac{2.66}{12}\right) = \underline{7.38 \text{ ac-ft}}$

---

② CITY OF GLENDALE

Source: CITY OF GLENDALE DESIGN GUIDELINES FOR SITE DEVELOPMENT AND IFRASTRUCTURE CONSTRUCTION-1990

BASIN OUTLET LOCATION: NORTHERN AT 67TH AVE.

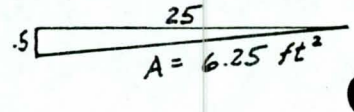
CITY METHOD: RATIONAL EQUATION

$C = 0.45$ (from City of Phx. Manual)

$T_c = t_i + t_t + 10$ min

$t_i = \frac{.04593\,(130)^{.77}}{(.5)^{.385}} = 2.55$ min

t_t : $Q = .56\left(\frac{50}{.015}\right)(.005)^{1/2}(.5)^{8/3} = 20.78$ CFS



$V = Q/A = 3.33$ CFS

$t_t = L/60V = 5540/60(3.33) = 27.73$ min

$T_c = 2.55 + 27.73 + 10 = 40.3$ min

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 3 OF 7

DETAIL RETENTION REQUIREMENTS COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

CITY OF GLENDALE CONTINUED

AT $T_c = 40.3$ min, $i_{100} = 2.8$ in/hr (USWB - City of Phx Manual)

$Q_{100} = (.45)(2.8)(83.2) = \underline{105 \text{ cfs}}$

VOLUME: $V_r = 7200\ A_a\ C\ I$

$C = .45$

$A_a = 83.2$ ac $= 3{,}624{,}192$ ft^3

$I = 1.25$ in/hr

$V_r = \dfrac{7200(83.2)(.45)(1.25)}{43{,}560 \text{ ft}^2/\text{ac}}$

$V_r = \underline{7.74 \text{ ac-ft}}$

MARICOPA COUNTY METHOD FOR CITY OF GLENDALE

$T_c = 11.4\ (1.023)^{.5}(.028)^{.52}(27.08)^{-.31}\ i^{-.38}$

$P_{10}^{6} = 1.95$ in

$T_c = .646\ i^{-.38}$

TRY $T_c = 22$ min, $i_p = 4.8$ in/hr, $i_{100} = (4.8)(1.95)/2.07 = 4.52$ in/hr

$T_c = .646\ (4.52)^{-.38} = 21.85$ min OK

$Q_{100} = (.40)(4.52)(83.2) = \underline{150 \text{ cfs}}$

VOLUME: $P_{100}^{1} = .494 + .755\ (2.95)\left(\frac{2.95}{3.75}\right) = 2.25$ in

$P_{100}^{6} = 2.95$ in

$P_{100}^{24} = 3.75$ in

$P_{100}^{2} = .341\ (2.95) + .659(2.25) = 2.49$ in

$V = CA\left(\frac{P_{100}^{2}}{12}\right) = (.40)(83.2)\left(\frac{2.49}{12}\right) = \underline{6.91 \text{ ac-ft}}$

---

③ CITY OF MESA

SOURCE: MESA ENGINEERING PROCEDURE MANUAL JUNE 1983, AUG. 1989

BASIN OUTLET LOCATION: McDOWELL AND RECKER Rd.

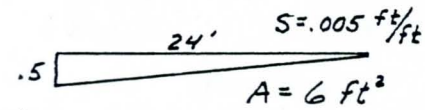
CITY METHOD: RATIONAL EQUATION

"C": 10% ROOFS (.85), 10% PAVEMENT (.85), 40% DESERT LANDSCAPE (.70), 10% BARE GROUND (.50?), 30% GRASS LANDSCAPE (.15)

$C = .20(.85) + .40(.70) + .10(.50) + .30(.15) = 0.545$

T_c: $t_i = \dfrac{.04593\ (130)^{1.77}}{(.5)^{.385}} = 2.55$ min

$t_t = 17.5$ cfs/side (nomograph)



$V = Q/A = 2.92$ ft/s, $t_t = 5540/60\ (2.92) = 31.62$ min

$T_c = 2.55 + 31.62 = 34.2$ min

@ $T_c = 34.2$ min, $i_{50} = 3.17$ in/hr (no 100-yr curve available)

$Q_{50} = (.545)(3.17)(83.2) = \underline{144 \text{ cfs}}$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 4 OF 7

DETAIL RETENTION REQUIREMENTS COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

CITY OF MESA CONTINUED

VOLUME = (.25)CA = (.25)(.545)(83.2) = 11.34 ac-ft

MARICOPA COUNTY METHOD for CITY OF MESA

$P_{10}^{6} = 1.90$ in
$P_{100}^{6} = 3.00$ in
$P_{100}^{24} = 3.50$ in

$T_c = 11.4\,(1.023)^{.5}(.028)^{.52}(27.08)^{-.31} i^{-.38}$

$T_c = .646\, i^{-.38}$

TRY $T_c = 22$ min, $i_p = 4.8$ in/hr, $i_{100} = (4.8)(1.90)/2.07 = 4.41$ in/hr

$T_c = .646\,(4.41)^{-.38} = 22.05$ min OK

$Q_{100} = (.40)(4.41)(83.2) = 147$ CFS

VOLUME:

$P_{100}^{1} = .494 + .755\,(3.00)\left(\frac{3.00}{3.50}\right) = 2.44$ in

$P_{100}^{2} = .341(3.00) + .659(2.44) = 2.63$ in

$V = CA\left(\frac{P_{100}^{2}}{12}\right) = (.40)(83.2)\left(\frac{2.63}{12}\right) = 7.29$ ac-ft

---

④ CITY OF PHOENIX

Source: City of Phx. STORM DRAIN DESIGN MANUAL, JULY 1988
BASIN OUTLET LOCATION: RAY RD. @ 40TH ST.
CITY METHOD: SCS

C = .45 (R1-6 ZONING)
S = .005 ft/ft, HYDROLOGIC Soil Group B, CN 84
W = A/L = 43560 × 82.3 / 5540 = 654', Wf = 1.10

$T_c = t_i + t_t$ $\qquad t_i = \frac{.04593\,(130)^{.77}}{(.5)^{.385}} = 2.55$ min

t_t: V = 3.25 ft/s (from p.33), $t_t = L/60V = 5540/60(3.25) = 28.41$ min

$T_c = 2.55 + 28.41 = 31$ min, $T_p = T_c \times Wf = 34.1$ min = .568 hrs.

$P_{100}^{1} = 2.66$ in, Q = 1.25 in

$Q_p = \frac{484\,(.13)(1.25)}{.568} = 138$ CFS

VOLUME: V = 7200 CIA, C = .45
I = 1.25 in/hr
A = 83.2 ac

$V = \frac{7200\,(.45)(1.25)(83.2)}{43{,}560} = 7.74$ ac-ft

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT *METHOD COMPARISON* PAGE *5* OF *7*

DETAIL *RETENTION REQUIREMENTS* COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

CITY OF PHOENIX CONTINUED:

MARICOPA COUNTY METHOD FOR CITY OF PHOENIX

$P_{10}^{6} = 2.00$ in
$P_{100}^{6} = 3.30$ in
$P_{100}^{24} = 3.90$ in

$$T_c = 11.4\,(1.023)^{.5}(.028)^{.52}(27.08)^{-.31}\,i^{-.38} = .646\,i^{-.38}$$

TRY $T_c = 22$ min, $i_p = 4.8$ in/hr, $i_{100} = (2.00)(4.8)/2.07 = 4.64$ in/hr

$$T_c = .646\,(4.64)^{-.38} = 21.63 \text{ min} \quad \underline{\text{OK}}$$

$$Q_{100} = (.40)(4.64)(83.2) = \underline{\underline{154 \text{ CFS}}}$$

VOLUME: $V = CA\left(\frac{P_{100}^{2}}{12}\right)$

$P_{100}^{1} = .494 + .755\,(3.3)\,(3.3/3.9) = 2.60$ in
$P_{100}^{2} = .341\,(3.3) + .659\,(2.60) = 2.84$ in

$$V = (.40)(83.2)(2.84/12) = \underline{\underline{7.88 \text{ ac-ft}}}$$

---

⑤ CITY OF SCOTTSDALE

SOURCE: CITY OF SCOTTSDALE DESIGN PROCEDURES AND CRITERIA, SECTION 2, JULY 1985
BASIN OUTLET LOCATION: JOMAX AT 136TH ST.
CITY METHOD: RATIONAL EQUATION

ZONING: R1-7, HYDROLOGIC SOIL GROUP B, CN = 92, $C_D = 0.65$

$T_c = t_i + t_t$: $t_i = \frac{.04593\,(130)^{.77}}{(.5)^{.385}} = 2.55$ min ; $V = 54.5^{1/2}$ (6" CURB) $= 3.82$ ft/s,

$t_t = 24.17$ min, $T_c = 2.55 + 24.17 = 26.72$ min

AT $T_c = 26.72$ min AND $P_{100}^{1} = 2.27$ in, $i_{100} = 3.85$ in/hr

$$Q_{100} = (.65)(3.85)(83.2) = \underline{\underline{208 \text{ CFS}}}$$

$$V = C_D A\left(\frac{P_{100}^{1}}{12}\right) = (.65)(83.2)(2.27/12) = \underline{\underline{10.23 \text{ ac-ft}}}$$

OPTIONAL SCOTTSDALE METHOD: TECHNIQUES USED IN THE "GENERAL DRAINAGE PLAN FOR NORTH SCOTTSDALE, ARIZ"

RUNOFF: KINEMATIC WAVE
LOSSES: SCS CURVE NUMBERS
RAINFALL: SCS 24-HOUR TYPE IIA
THE HEC-1 MODEL GENERATED A PEAK DISCHARGE OF 225 cfs AND A RUNOFF VOLUME OF 18.4 ac-ft USING THE FOLLOWING INPUT:

```
IT     5                  200
IO     3
KK
PB     4.5
PC     ...  TYPE IIA
LS            74                  95
UK     130   .005   .200    65
UK     50    .01    .075    35
RK     620.  .005   .020   .0108  TRAP    40.
RK     4615. .005   .025    .13   TRAP    50.
```

CITY OF SCOTTSDALE CONTINUED

MARICOPA COUNTY METHOD FOR CITY OF SCOTTSDALE

$T_c = 11.4\,(1.023)^{.5}\,(.028)^{.52}\,(27.08)^{-.31}\,i^{-.38}$

$T_c = .646\,i^{-.38}$

$P_{10}^{6} = 2.30$ in

$P_{100}^{6} = 3.42$ in

$P_{100}^{24} = 4.55$ in

TRY $T_c = 22$ min, $i_p = 4.8$ in/hr,

$i_{100} = \frac{(4.8)(2.3)}{2.07} = 5.33$ in/hr

$T_c = .646\,(5.33)^{-.38} = 20.52$ min

TRY $T_c = 20$ min, $i_p = 5.1$ in/hr, $i_{100} = 5.67$ in/hr

$T_c = .646\,(5.67)^{-.38} = 20.05$ min OK

$Q_{100} = (.40)(5.67)(83.2) =$ 189 CFS

VOLUME: $P_{100}^{1} = .494 + .755\,(3.42)\,(3.42/4.55) = 2.435$ in

$P_{100}^{2} = .341\,(3.42) + .659\,(2.435) = 2.77$ in

$V = (.40)(83.2)(2.77/12) =$ 7.68 ac-ft

---

(6) CITY OF TEMPE

Source: PUBLIC WORKS DEPT. - PRIVATE DEVELOPMENT DESIGN CRITERIA, JUNE 1987

BASIN OUTLET LOCATION: PRICE @ SOUTHERN

CITY METHOD: RATIONAL EQUATION

"C": 10% ROOFS (.95), 10% PAVEMENT (.95), 40% DESERT LANDSCAPE (.70), 10% BARE GROUND (.25), 30% Ave. Sloped LAWNS (.20)

$C = .20(.95) + .40(.70) + .10(.25) + .30(.20) = 0.56$

$T_c = t_i + t_t$: $t_i = \frac{KL^{.37}}{S^{.20}} = \frac{1.57\,(130)^{.37}}{(.5)^{.20}} = 10.9$ min

$t_t =$ $V = 2.74$ ft/s, $t_t = L/60V = 5540/60(2.74) = 33.70$ min

$T_c = 10.9 + 33.7 = 44.6$ min

AT $T_c = 44.6$ min, $i_{100} = 2.97$ in/hr, $Q_{100} = (.56)(2.97)(83.2) =$ 138 CFS

VOLUME: $V = \frac{D}{12}AC = \frac{2.40}{12}(83.2)(.95) =$ 15.8 ac-ft

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 7 OF 7

DETAIL RETENTION REQUIREMENTS COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

CITY OF TEMPE CONTINUED

MARICOPA COUNTY METHOD FOR CITY OF TEMPE

$T_c = 11.4\,(1.023)^{.5}\,(.028)^{.52}\,(27.08)^{.31}\,i^{-.38}$

$T_c = .646\,i^{-.38}$

$P_{10}^{6} = 1.90$ in

$P_{100}^{6} = 3.0$ in

$P_{100}^{24} = 3.7$ in

TRY $T_c = 22$ min, $i_p = 4.8$ in/hr, $i_{100} = \frac{(4.8)(1.9)}{2.07} = 4.41$ in/hr

$T_c = .646\,(4.41)^{-.38} = 22.05$ min OK

$Q_{100} = (.40)(4.41)(83.2) = \underline{\underline{147 \text{ cfs}}}$

VOLUME: $P_{100}^{1} = .494 + .755\,(3.0)^2/3.7 = 2.33$ in

$P_{100}^{2} = .341(3.0) + .659(2.33) = 2.56$ in

$V = CA\left(\frac{P_{100}^{2}}{12}\right) = (.40)(83.2)\left(\frac{2.56}{12}\right) = \underline{\underline{7.10 \text{ ac-ft}}}$

SUMMARY TABLE

	CITY METHOD		MARICOPA COUNTY METHOD	
	Q_{100} (cfs)	(V ac-ft)	Q_{100} (cfs)	V (ac-ft)
1. CHANDLER	188	13.19	144	7.38
2. GLENDALE	105	7.74	150	6.91
3. MESA	144 (Q_{50})	11.34	147	7.29
4. PHOENIX	138	7.74	154	7.88
5. SCOTTSDALE	208	10.23	189	7.68
6. TEMPE	138	15.80	147	7.10

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 1 OF 5

DETAIL YOUNGTOWN WATERSHED COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

AN URBAN WATERSHED NEAR 111TH AVE. AND PEORIA HAS BEEN DELINIATED ON FIGURE 1. FIVE METHODS WILL BE USED TO CALCULATE MULTI-FREQUENCY PEAK DISCHARGES AT THE OUTLET:

1. THE MARICOPA CO. RATIONAL METHOD
2. THE CITY OF PHOENIX RATIONAL METHOD
3. THE MARICOPA CO. UNIT HYDROGRAPH PROCEDURE (CLARK UNIT HYDROGRAPH)
4. THE CITY OF PHOENIX SCS METHOD
5. THE CITY OF PHOENIX COMPUTER GENERATED DRAINAGE ANALYSIS PROCEDURE

(1) MARICOPA COUNTY RATIONAL METHOD

APPLICABLE BASIN PARAMETERS:

AREA: $0.13\ mi^2 = 83.2\ ac$

LAND USE: SINGLE FAMILY AREA $\rightarrow C = 0.40$

$L = 1.023\ mi$

$S = 5.8\ ft/mi$

$K_b = -.00625(\log 83.2) + .04 = .028$

2-YEAR PEAK DISCHARGE

$$T_c = 11.4\, L^{.5} K_b^{.52} S^{-.31} i_2^{-.38}$$

$$T_{c_2} = (11.4)(1.023)^{.5}(.028)^{.52}(5.8)^{-.31}(i_2^{-.38})$$

$$T_{c_2} = 1.042\, i_2^{-.38}$$

TRY $T_{c_2} = 1.0$ HOURS: $i_2 = 0.93\ in/hr$

$T_{c_2} = 1.042(.93)^{-.38} = 1.071\ hrs \longrightarrow$ NO GOOD

TRY $T_{c_2} = 1.10$ HOURS: $i_2 = 0.88\ in/hr$

$T_{c_2} = 1.042(.88)^{-.38} = 1.094\ hr \longrightarrow$ OK

THEN $Q_2 = CiA = (.40)(.88)(83.2) = \underline{29\ cfs}$

5-YEAR PEAK DISCHARGE

TRY $T_c = .917$ HOURS: $i_5 = 1.45\ in/hr$

$T_{c_5} = 1.042(1.45)^{-.38} = .905\ hrs \longrightarrow$ OK

THEN $Q_5 = (.40)(1.45)(83.2) = \underline{48\ cfs}$

10-YEAR PEAK DISCHARGE

TRY $T_c = .75\ hrs$, $i_{10} = 1.95\ in/hr$

$T_{c_{10}} = 1.042(1.95)^{-.38} = 0.808\ hr \rightarrow$ NO GOOD

TRY $T_c = .817\ hrs$, $i_{10} = 1.80\ in/hr$

$T_{c_{10}} = 1.042(1.80)^{-.38} = .833 \longrightarrow$ OK

THEN $Q_{10} = (.40)(1.80)(83.2) = \underline{60\ cfs}$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 2 OF 5

DETAIL YOUNGTOWN WATERSHED COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

25-YEAR PEAK DISCHARGE

TRY $T_c = .717$ hrs : $i_{25} = 2.40$ in/hr

$T_{c25} = 1.042\,(2.40)^{-.38} = 0.747$ hr → NO GOOD

TRY $T_c = .75$ hrs : $i_{25} = 2.35$ in/hr

$T_{c25} = 1.042\,(2.35)^{-.38} = 0.753$ hr → OK

THEN $Q_{25} = (.40)(2.35)(83.2) =$ 78 CFS

50-YEAR PEAK DISCHARGE

TRY $T_c = .650$ hrs : $i_{50} = 2.95$ in/hr

$T_{c50} = 1.042\,(2.95)^{-.38} = 0.691$ hr → NO GOOD

TRY $T_c = .700$ hrs : $i_{50} = 2.80$ in/hr

$T_{c50} = 1.042\,(2.80)^{-.38} = 0.705$ hr → OK

THEN $Q_{50} = (.40)(2.80)(83.2) =$ 93 CFS

100-YEAR PEAK DISCHARGE

TRY $T_c = .633$ hrs : $i_{100} = 3.40$ in/hr

$T_{c100} = 1.042\,(3.40)^{-.38} = 0.654$ hrs → NO GOOD

TRY $T_c = .667$ hrs : $i_{100} = 3.30$ in/hr

$T_{c100} = 1.042\,(3.30)^{-.38} = 0.662$ hrs

THEN $Q_{100} = (.40)(3.30)(83.2) =$ 110 CFS

---

② CITY OF PHOENIX RATIONAL METHOD

FROM: CITY OF PHOENIX "STORM DRAIN DESIGN MANUAL", JULY 1988

APPLICABLE BASIN PARAMETERS:

AREA = 83.2 ac

OVERLAND FLOW LENGTH (AVE. ALLEY TO STREET) = 130'

MAX. GUTTER FLOW LENGTH: 5540'

S = .001 ft/ft

C = 0.45 (RESIDENTIAL AREA, AVERAGE ZONING)

CALCULATE T_c → Sum of Overland & Gutter Flow

$$t_i = \frac{.04593\,(130)^{.77}}{(.1)^{.385}} = 4.7 \text{ min}$$

$$t_t = \frac{L}{60V}, \quad V = 1.5 \text{ ft/sec @ } S = .001 \text{ and } y = 0.5'$$

$$t_t = 5540/60(1.5) = 61.6 \text{ min}$$

THEN $T_c = 4.7 + 61.6 =$ 66.3 min

Q = CiA →

T_R	i (in/hr)	Q_{PK} (cfs)
2	.78	29
5	1.09	41
10	1.29	48
25	1.52	57
50	1.78	67
100	1.98	74

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 3 OF 5

DETAIL YOUNGTOWN WATERSHED COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

③ MARICOPA COUNTY UNIT HYDROGRAPH PROCEDURE

APPLICABLE BASIN PARAMETERS:

AREA = .13 mi²
S = 5.8 ft/mi
L = 1.023 mi
Kb = .028
Time AREA Curve: URBAN

RUNOFF MODEL: HEC-1

LOSSES BY GREEN & AMPT METHOD:

SOILS: 50% LcA, 35% PeA, & 15% Vf

DTHETA = .50(.35) + .35(.35) + .15(.15) = .32 in
PSIF = .50(4.3) + .35(3.5) + .15(12.4) = 5.24 in
XKSAT = .50(.25) + .35(.40) + .15(.01) = 0.27 in/hr
IA = .50(.20) + .50(.10) = 0.15 in (50% Desert Landscaping & 50% LAWNS)
RTIMP = 25% (connected Imperviousness)

6-HR RAINFALL DEPTHS (adjusted) & Tc, R (from MCUHP.EXE)

TR	2	5	10	25	50	100
D(in)	1.11	1.60	1.95	2.31	2.80	3.15
Tc (hr)	1.217	1.133	.983	.883	.783	.717
R (hr)	1.499	1.385	1.183	1.050	.919	.833

PEAK DISCHARGE (CFS) → 13 27 41 59 88 112

④ CITY OF PHOENIX SCS METHOD

APPLICABLE BASIN PARAMETERS: →

A = .13 mi² = 83.2 ac
L = 5,670 ft
S = .01 ft/ft (1% slope is minimum on A vs. Tc graph)
W = (43,560 × 83.2)/5670 = 639 ft, Wf = 1.10
Tc = 1.04 hr, Tp = Tc Wf = 1.144 hrs
Soil Group B, CN = 84 (R1-6 ZONING)

$$Q_p = \frac{484\,A\,Q}{T_p}$$

Where: Qp = Peak Discharge (cfs)
A = Drainage Area (mi²)
Q = Storm Runoff (in)
Tp = time to peak (hr)

Tr (years)	2	5	10	25	50	100
$P_1^{T_r}$ (in)	.88	1.30	1.61	2.02	2.35	2.66
$Q_1^{T_r}$ (in)	.12	.31	.48	.77	.97	1.23
$Q_{pk}^{T_r}$ (cfs)	7	17	26	42	53	68

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 4 OF 5

DETAIL YOUNGTOWN WATERSHED COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

⑤ CITY OF PHOENIX "COMPUTER GENERATED ANALYSIS PROCEDURE"

CURVE NUMBER: 85% B soil, 15% D soil $.85(84)+.15(90) = \underline{85}$

LAG TIME: $T_L = 0.6(T_c)$

$T_c = 66.3$ min. from C.O.P. RATIONAL METHOD CALCS.

$T_L = 0.6(66.3) = 39.8$ min $\equiv$ 0.663 hours

RAINFALL: 24-hour DEPTHS & DISTRIBUTION FROM p. 16 of the manual

RESULTS:

RUNOFF MODEL: HEC-1

TR (years)	DEPTH (in)	PEAK DISCHARGE (cfs)
2	1.44	20
5	2.10	44
10	2.53	61
25	3.12	86
50	3.57	105
100	4.04	126

SUMMARY TABLE: PEAK DISCHARGES in cfs

TR (years)	1 MARICOPA Co. RATIONAL METHOD	2 CITY OF PHX. RATIONAL METHOD	3 MARICOPA Co. U.H.P.	4 CITY OF PHX. SCS	5 CITY OF PHX COMPUTER
2	29	29	~~13~~ 11	7	20
5	48	41	~~27~~ 21	17	44
10	60	48	~~41~~ 32	26	61
25	78	57	~~59~~ 48	42	86
50	93	67	~~88~~ 79	53	105
100	110	74	~~112~~ 103	68	126

Figure 2 illustrates the Summary TABLE in graphical form

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE 5 OF 5

DETAIL YOUNGTOWN WATERSHED COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

RECORDED DATA:

The U.S.G.S. operated a stream/precipitation gauge* at the outlet of the Youngtown Watershed from March 1961 through September 1968. A summary of this data follows:

RANK (m)	ANNUAL MAXIMUM SERIES (cfs)	PARTIAL-DURATION SERIES (cfs)	GRINGORTEN T_R (yr)	GRINGORTEN P (%)
1	73	73	13.7	7.3
2	21	21	5.1	19.5
3	17	20	3.2	31.3
4	16	17	2.3	43.9
5	9	16	1.8	56.2
6	8	15	1.5	68.5
7	7	14	1.2	80.6
8	6	13	1.1	92.6

The Partial DURATION Series plotted in Discharge-Frequency format (Fig. 3) suggests a 2-yr. return period discharge of 17 cfs and a 5-yr. of 25 cfs. Extrapolation beyond $T_R = 5$ years will likely result in invalid discharge estimates because the 73 cfs value appears to be an outlier (i.e. it has a greater return period than is indicated by it's plotting position).

* GAUGE # 9-5137: AGUA FRIA TRIBUTARY AT YOUNGTOWN, "SURFACE WATER SUPPLY OF THE U.S.", water years 61-65 & 66-70, PART 9, VOL. 3, USGS

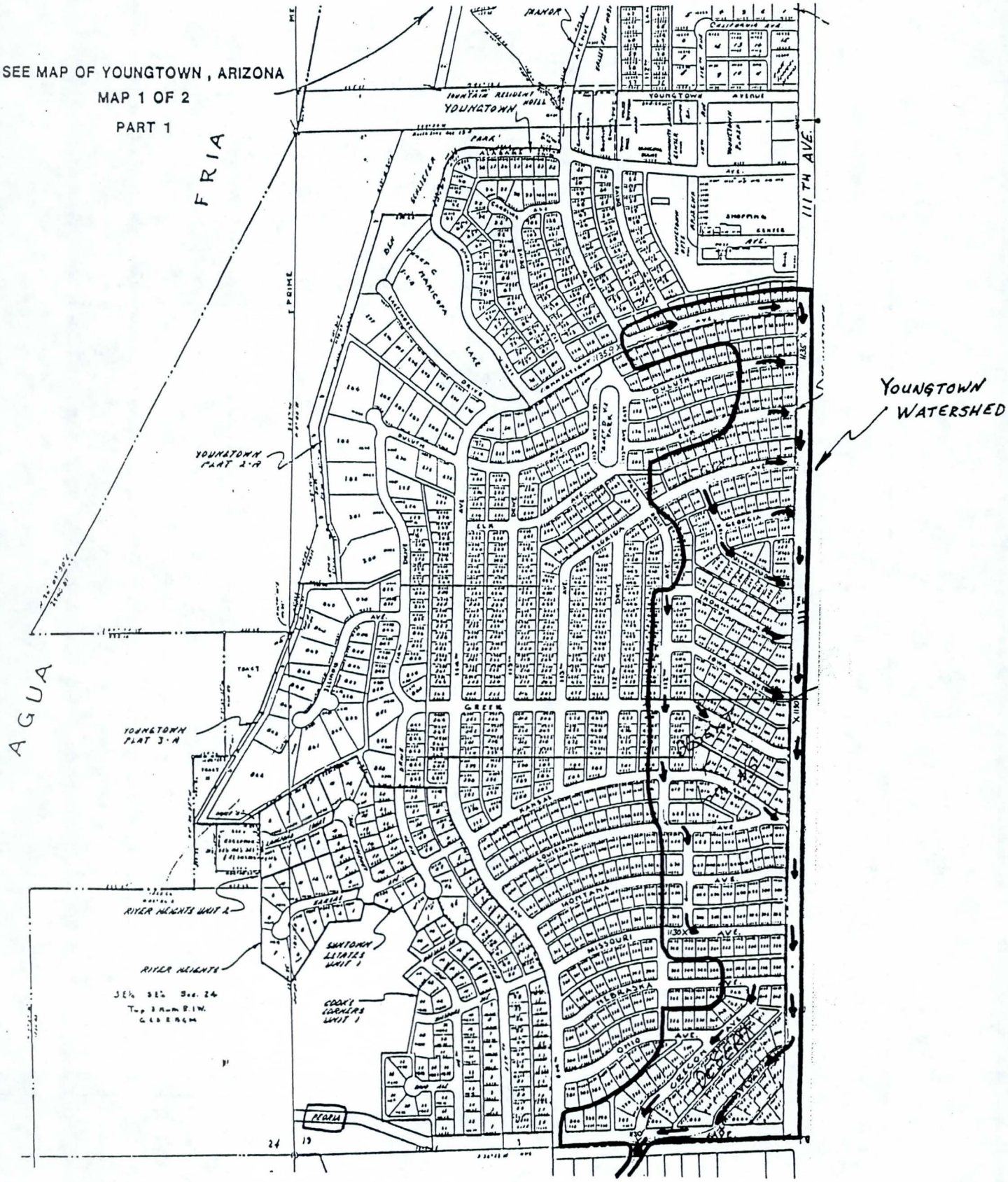
FIGURE 1

MAP OF YOUNGTOWN , ARIZONA
MAP 1 OF 2

PART 2

SCALE
1" ≈ 680'

N



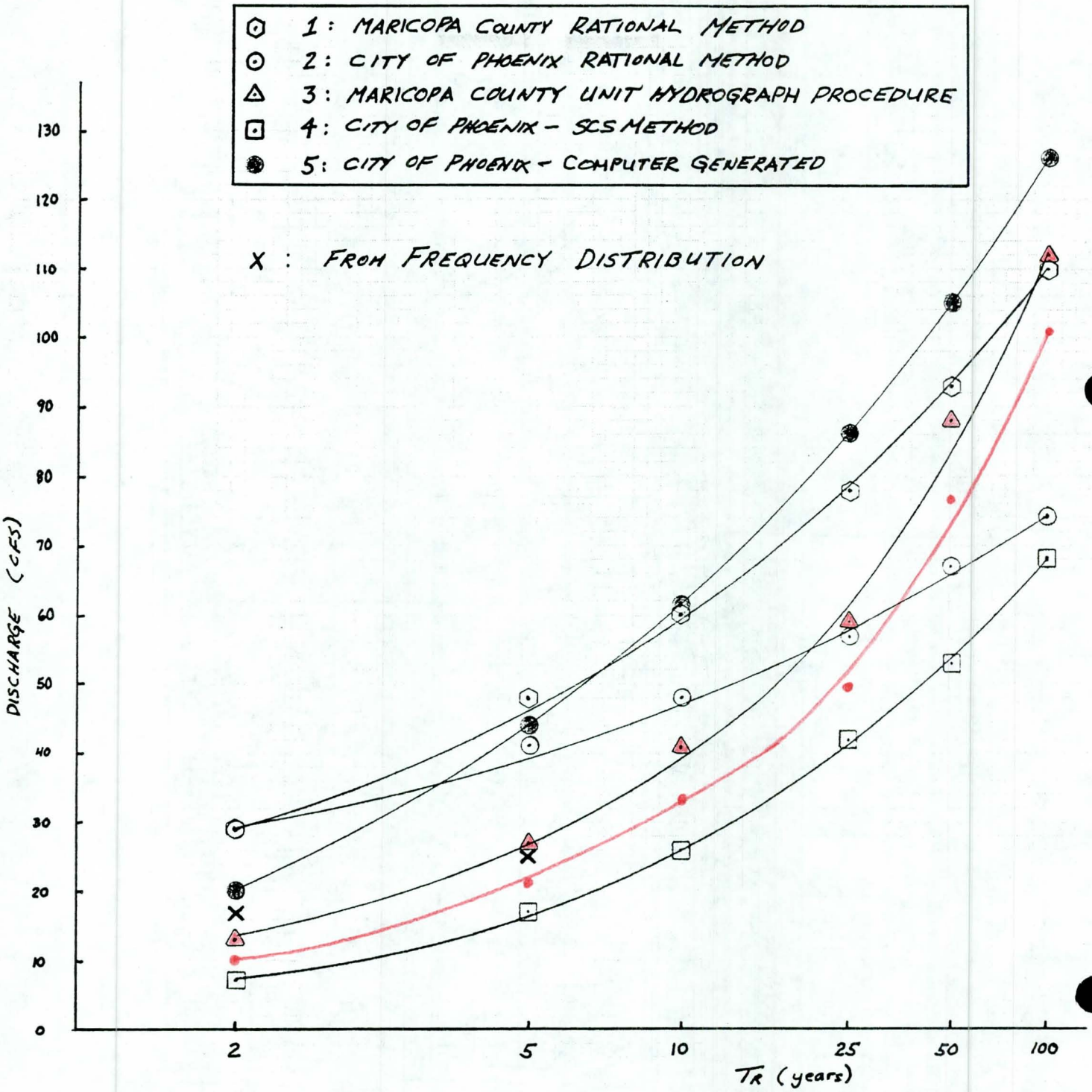
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT METHOD COMPARISON PAGE ___ OF ___

DETAIL YOUNGTOWN WATERSHED COMPUTED ___ DATE ___

FIGURE 2 CHECKED BY ___ DATE ___

DISCHARGE - FREQUENCY CURVES
METHODS 1-5



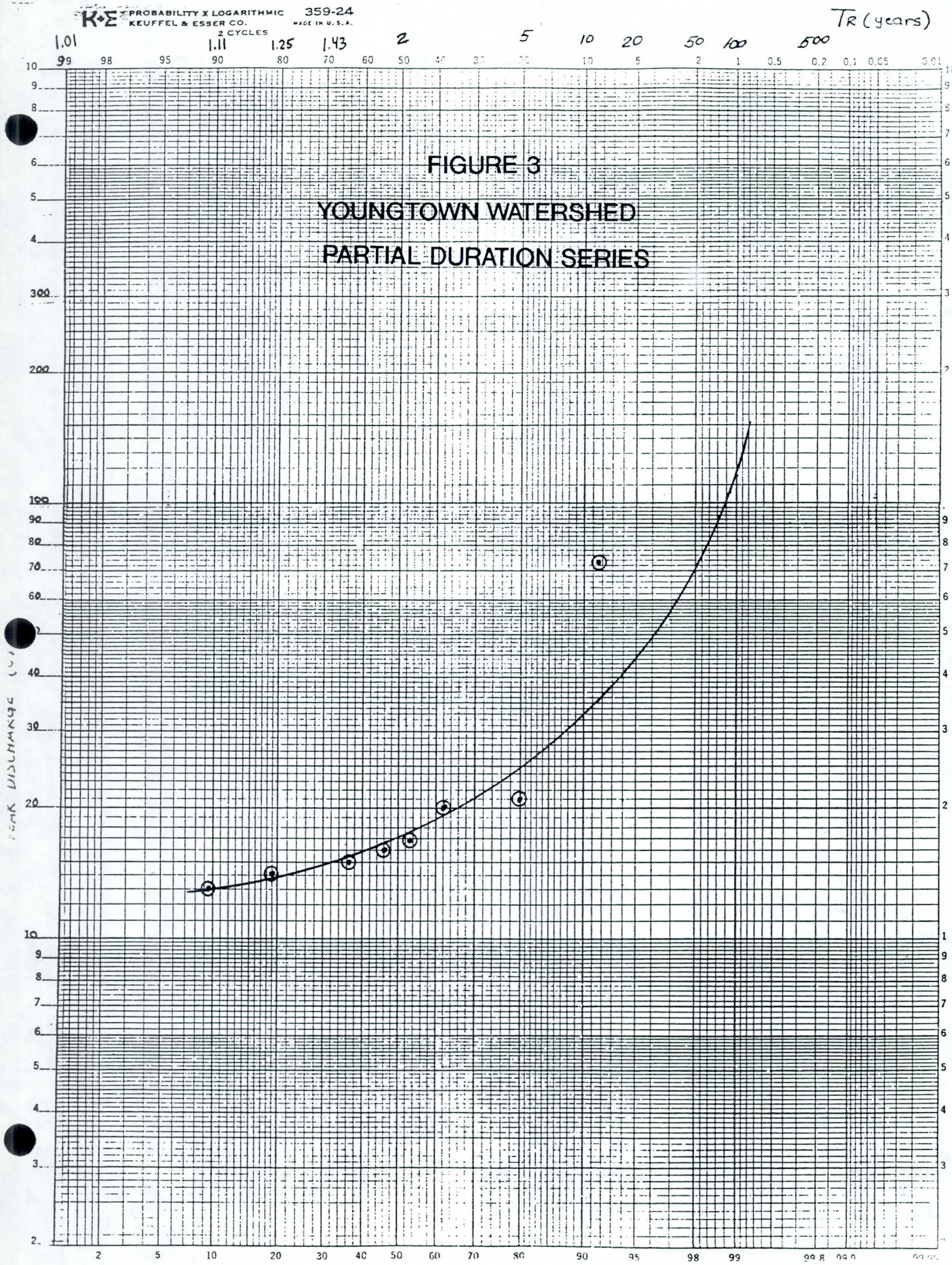


FIGURE 3

YOUNGTOWN WATERSHED

PARTIAL DURATION SERIES

```
ID  MARICOPA COUNTY UNIT HYDROGRAPH PROCEDURE
ID  YOUNGTOWN WATERSHED:  6HR - 100 YR RAINFALL, CLARK UNIT HYDROGRAPH,
ID  GREEN & AMPT LOSSES, URBAN TIME/AREA CURVE
IT     5                    100
IO     0
KK  SUB1
BA   .13
IN    15
PC 0.000   0.016   0.028   0.044   0.069   0.094   0.120   0.148   0.170   0.195
PC 0.236   0.277   0.337   0.400   0.646   1.153   2.592   2.835   2.898   2.958
PC 2.999   3.040   3.078   3.112   3.150
LG   .15     .32    5.24     .27     25.
UA     0       5      16      30      65      77      84      90      94      97
UA   100
UC 0.717   0.833
ZZ
```

```
ID MULTI-FREQUENCY RUN FOR YOUNGTOWN WATERSHED
ID USING CRITERIA ESTABLISHED IN THE CITY OF PHOENIX
ID STORM DRAIN DESIGN MANUAL
IT    10                    250
IO     3
JP     6
KK YOUNG
BA   .13
IN    30
PB  1.44
PC     0    .004    .008    .013    .018    .022    .026    .031    .035    .040
PC  .044    .048    .053    .057    .062    .066    .071    .075    .080    .093
PC  .107    .120    .140    .170     .50    .830    .860    .880    .893    .907
PC  .920    .924    .928    .933    .937    .942    .947    .951    .956    .960
PC  .964    .969    .973    .978    .982    .987    .991    .995    1.00    1.00
LS           85
UD  .663
KP     2
PB   2.1
KP     3
PB  2.53
KP     4
PB  3.12
KP     5
PB  3.57
KP     6
PB  4.04
ZZ
```

DELETE

FIGURE 4

MARICOPA COUNTY AND CITY OF PHOENIX

HEC-1 SAMPLE PROGRAMS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 6

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

METHOD COMPARISON: APPLICATION TO LARGE, NATURAL WATERSHEDS

SCENARIO: THIS EXAMPLE COMPARES THE METHODS OUTLINED IN THE HYDROLOGIC DESIGN MANUAL WITH THE COMMONLY USED TECHNIQUES, i.e., THE SCS METHODS, FOR APPLICATION TO LARGE, NATURAL WATERSHEDS. THEN, THE RESULTS ARE COMPARED WITH DISCHARGE FREQUENCY ANALYSIS.

WATERSHED DISCRIPTION:

LOCATION: UPPER CAVE CREEK WATERSHED

DRAINAGE AREA: 126 Mi^2

INDIVIDUAL BASINS: 80.28 Mi^2; 34.86 Mi^2; 6.38 Mi^2; 4.81 Mi^2 & .59 Mi^2

THE CAVE CREEK WATERSHED IS CHARACTERIZED BY GENERALLY STEEP SLOPING AREAS WITH RATHER DENSE VEGETATIVE COVER AND IRREGUAR CHANNELS.

STUDY DISTRIPTION:

THIS STUDY WILL COMPARE THE GENERATED PEAK DISCHARGE VALUES OF 10-YEAR, 50-YEAR, AND 100-YEAR FREQUENCIES AMOUNG THE THREE METHODS DESCRIBED EARLIER. THE GENERATED BASIN HYDROGRAPHS ARE ADDED AND WHEN NECESSARY, ROUTED THROUGH THE CAVE CREEK WASH. THE FINAL PEAK DISCHARGE VALUE IS COMPUTED AT THE EXTREME SOUTH WEST LOCATION OF THE WATERSHED WHERE A COMPARISON OF VALUES IS MADE (SEE ENCLOSED MAP).

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 2 OF 6

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

HYDROLOGIC INPUT CRITERIA					
METHOD OF COMPUTATION	RAINFALL CRITERIA	METHOD OF LOSS COMPUTATION	HYDROGRAPH GENERATION	ROUTING METHOD	COMPUTER PROGRAM
HYDROLOGIC DESIGN MANUAL	100-YEAR, 6-HOUR PATTERN DISTRIBUTION	INITIAL & UNIFORM LOSS METHOD	CORPS OF ENG. S-GRAPHS	MUSKINGUM	HEC-1
SCS METHODS	100-YEAR, 24-HOUR SCS TYPE II	CURVE NUMBERS	SCS UNIT-GRAPH	MUSKINGUM	TR-20

1. HYROLOGIC DESIGN MANUAL

A - CALCULATION OF RAINFALL DEPTHS:

BASIN SIZE	RED. FACTOR	10-YEAR RAIN FALL	50-YEAR RAIN FALL	100-YEAR RAIN FALL
80.28	.815	.815(2.5) = 2.04	.815 (3.3) = 2.60	.815 (3.65) = 2.97
34.86	.88	.88 (2.5) = 2.20	.88 (3.2) = 2.82	.88 (3.60) = 3.12
6.38	.96	.96 (2.5) = 2.40	.96 (3.2) = 3.07	.96 (3.60) = 3.46
4.81	.97	.97 (2.5) = 2.43	.97 (3.2) = 3.10	.97 (3.60) = 3.49
.59	1.0	1.0 (2.5) = 2.50	1.0 (3.2) = 3.20	1.0 (3.60) = 3.60

B - SELECTION OF LOSS PARAMETERS:

AREA (Mi^2)	80.28	34.86	6.38	4.81	0.59
INITIAL LOSS	0.73	0.75	0.75	0.75	0.75
UNIFORM LOSS	0.12	0.18	0.16	0.20	0.20

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 3 OF 6

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

C- CALCULATION OF BASIN LAGS:

(*) AREA = 80.28 mi^2, L = 116160 ft, Lca = 79200 ft, $\bar{n}$ = .05

$$SLOPE = \frac{4400-2100}{116160/5280} = 105 \text{ ft/mi}; \quad BASIN\ FACTOR = \frac{(116160 \times 79200)/5280^2}{(105)^{1/2}} = 32.3$$

$$\Rightarrow LAG = 20\,(.05)(32.3)^{.38} = 3.70 \text{ HOURS}$$

(*) AREA = 34.86 mi^2, L = 49280 ft, Lca = 29920 ft, $\bar{n}$ = .05

$$SLOPE = \frac{3460-2040}{49280/5280} = 152 \text{ ft/mi}; \quad BASIN\ FACTOR = \frac{(49280 \times 29920)/5280^2}{(152)^{1/2}} = 4.29$$

$$\Rightarrow LAG = 20\,(.05)(4.29)^{.38} = 1.74 \text{ HOURS}$$

(*) AREA = 6.38 mi^2, L = 33440 ft, Lca = 18000 ft, $\bar{n}$ = 0.05

$$SLOPE = \frac{2500-1900}{33440/5280} = 94.7 \text{ ft/mi}; \quad BASIN\ FACTOR = \frac{(33440 \times 18000)/5280^2}{(94.7)^{1/2}} = 2.27$$

$$\Rightarrow LAG = 20\,(.05)(0.29)^{.38} = 1.35 \text{ HOURS}$$

(*) AREA = 4.81 mi^2, L = 44000 ft, Lca = 24640 ft, $\bar{n}$ = .045

$$SLOPE = \frac{3100-2100}{44000/5280} = 120 \text{ ft/mi}; \quad BASIN\ FACTOR = \frac{(44000 \times 24640)/5280^2}{(120)^{1/2}} = 3.55$$

$$\Rightarrow LAG = 20\,(.045)(3.55)^{.38} = 1.46 \text{ HOURS}$$

(*) AREA = .59 mi^2, L = 15000 ft, Lca = 5000 ft, $\bar{n}$ = .045

$$SLOPE = \frac{2120-1881}{15000/5280} = 83 \text{ ft/mi}; \quad BASIN\ FACTOR = \frac{(15000 \times 5000)/5280^2}{(83)^{1/2}} = .29$$

$$\Rightarrow LAG = 20\,(.045)(.29)^{.38} = .56 \text{ HOURS}.$$

2. SCS METHODS

SINCE A COMPLETE DOCUMENTATION OF THE 1977 STUDY OF THE CAVE CREEK WATERSHED WAS NOT AVAILABLE, ONLY THE PARAMETER DESCRIPTION AND THE RESULTS BY THE TR-20 PROGRAM ARE INCLUDED. THIS INFORMATION INCLUDES SELECTION OF CURVE NUMBERS FOR LOSS COMPUTATIONS, LENGTH OF ROUTING REACHES, AND THE RESULTING DISCHARGE VALUES.

3. DISCHARGE-FREQUENCY ANALYSIS:

30 YEARS OF DATA ON ANNUAL DISCHARGE WAS AVAILABLE FOR THE PERIOD OF 1958-1987. THIS INFORMATION WAS USED TO ESTIMATE THE 100-YEAR DISCHARGE BY THE WEIBULL POSITIONING METHOD (NEXT PAGE SHOWS THE INDIVIDUAL AND THE RANKED VALUES. THEN DATA WAS ARRANGED BASED ON WEIBULL POSITION AND PLOTTED ON FREQUENCY PAPER FOR AN ESTIMATION OF A DESIRED FREQUENCY.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 5 OF 6

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

DATE OF FLOW	DISCHARGE (CFS)	RANK	WEIBULL POSITION (*)	WATER YEAR
12-19-67	12400	1	31	68
10-29-59	8570	2	15.5	60
3-2-78	7500	3	10.3	78
12-18-78	6900	4	7.75	79
12-22-65	6000	5	6.2	66
9-12-58	5680	6	5.2	58
9-17-72	3950	7	4.4	72
10-19-72	3950	8	3.9	73
8-5-59	3590	9	3.4	59
8-2-64	3120	10	3.1	64
9-5-70	2700	11	2.8	70
9-6-67	1800	12	2.6	67
8-6-63	1510	13	2.4	63
3-3-83	1420	14	2.2	83
8-5-74	1390	15	2.1	74
7-22-86	1350	16	1.9	86
2-9-76	1260	17	1.8	76
10-2-81	1200	18	1.7	82
12-27-84	910	19	1.6	85
11-2-74	856	20	1.55	75
9-17-61	696	21	1.5	61
7-16-65	610	22	1.4	65
2-4-71	364	23	1.3	71
12-16-61	280	24	1.29	62
8-9-84	140	25	1.24	84
1969	0	26	1.2	69
1977	0	27	1.1	77
1981	0	28	1.1	81
1987	0	29	1.07	87
1980	0	30	1.03	80

(*) WEIBULL POSITION $T = \frac{n+1}{m}$, WHERE m = RANK OF DATA, n IS TOTAL NUMBER OF DATA POINTS.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 6 OF 6

DETAIL _____ COMPUTED _____ DATE _____

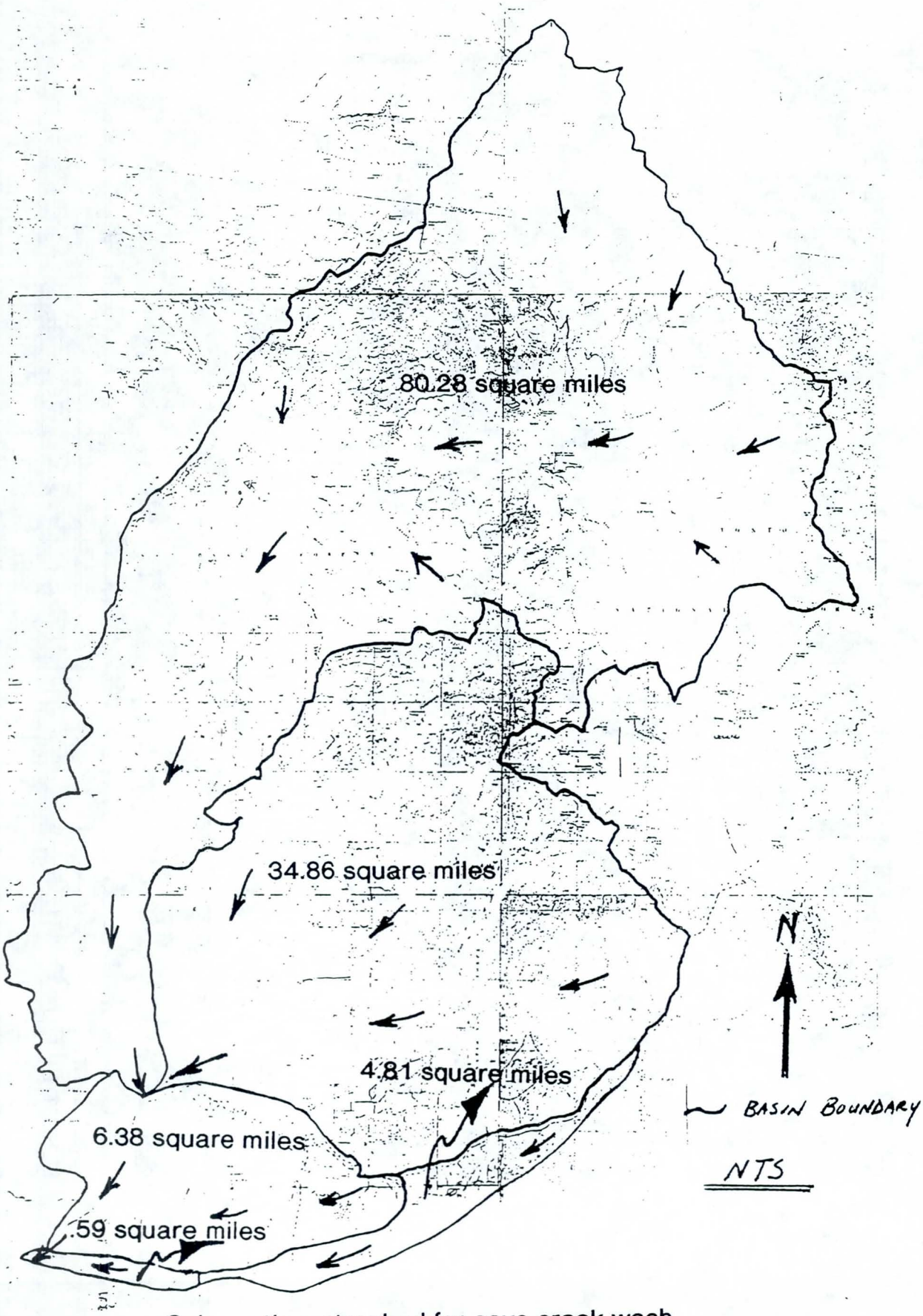
_____ CHECKED BY _____ DATE _____

COMPARISON OF DISCHARGE VALUES:

COMPARISON OF 100-YEAR DISCHARGE AS A FUNCTION OF AREA		
DRAINAGE AREA (Mi^2)	DISCHARGE BY HDM (*) (CFS)	DISCHARGE BY SCS (CFS)
.59	986	780
4.81	3361	3989
6.38	4443	4871
34.86	16759	18864
80.28	19971	23604
126.92	29790	36841

(*) HYDROLOGIC DESIGN MANUAL

COMPARISON OF DISCHARGE VALUES WITH FLOW FREQUENCY			
	10-YR DISCHARGE (CFS)	50-YR DISCHARGE (CFS)	100-YR DISCHARGE (CFS)
HYDROLOGIC DESIGN MANUAL	18455	26081	29790
SCS METHODS	20610	32975	36841
FLOW FREQUENCY	9000	22000	28000



Schematic watershed for cave creek wash.

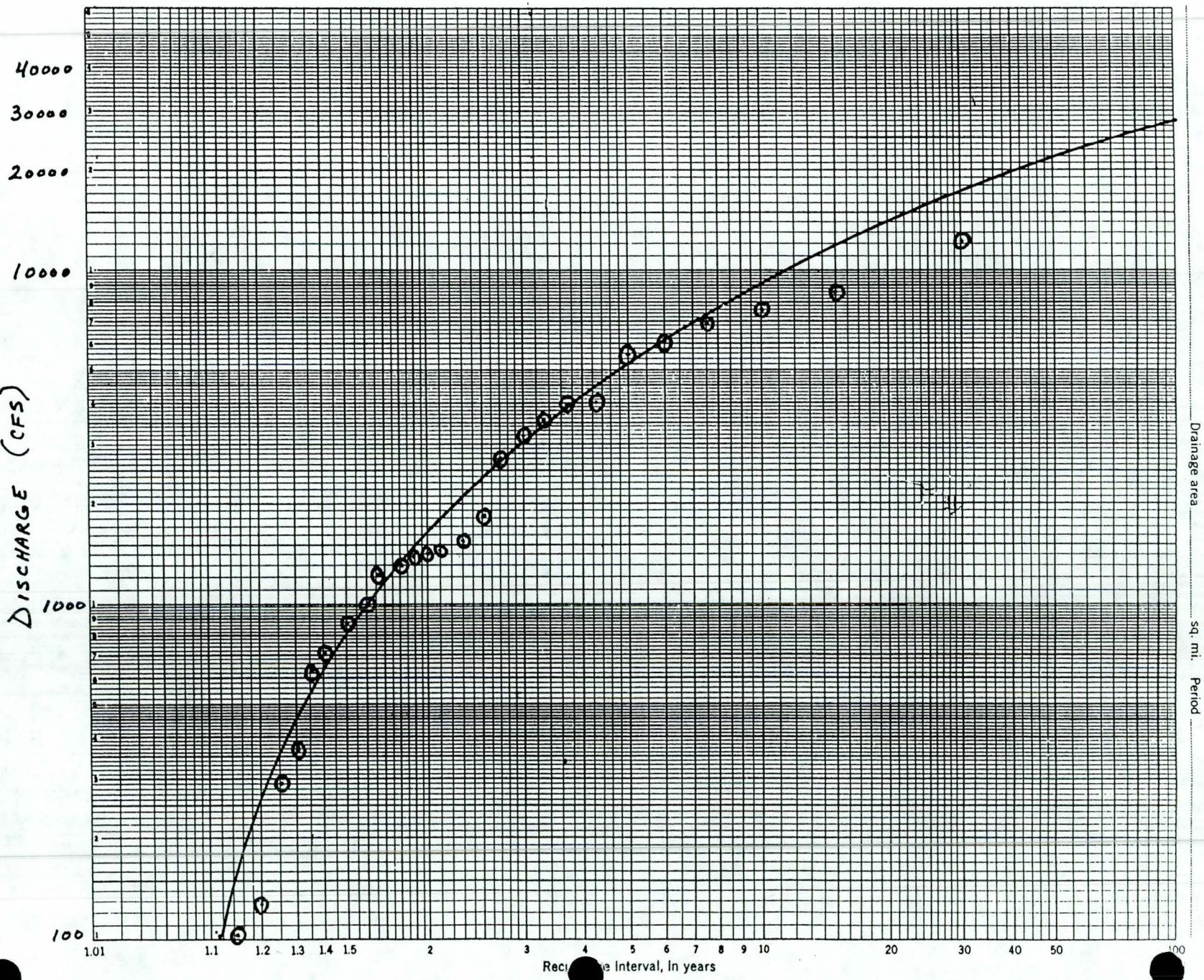
9-179b
Extreme log data plot
April 1955

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

File

Magnitude and frequency of CAVE CREEK WASH NEAR CAVE CREEK on USGS GAGE # 09512300

Drainage area _____ Sq. mi. Period _____



Sheet No. _____ of _____ Sheets Prepared by _____ Date _____ Checked by _____ Date _____

GPO 863751

STANDARD CONTROL INSTRUCTIONS

```
6 RUNOFF    1     1          6          80.2800    82.0000    4.45000 0 0 0 0 1
6 REACH     3    28     6    5        6200.0000      .8800    0.00000 0 0 0 0 1
6 RUNOFF    1    28          6          34.8600    83.5000    2.21000 0 0 0 0 1
6 ADDHYD    4    28     5    6    7                                  0 0 0 0 0 1
6 REACH     3    29     7         5   14100.0000      .8800    0.00000 0 0 0 0 1
6 RUNOFF    1    29               6       6.3800    80.0000    1.10000 0 0 0 0 1
6 ADDHYD    4    29     5    6    7                                  0 0 0 0 0 1
6 RUNOFF    1    30               6       4.8100    79.0000     .90000 0 0 0 0 1
6 REACH     3    31     6         1   15000.0000      .8800    0.00000 0 0 0 0 1
6 RUNOFF    1    31               6        .5900    79.0000     .35000 0 0 0 0 1
6 ADDHYD    4    31     6    1    5                                  0 0 0 0 0 1
6 ADDHYD    4    32     7    5    6                                  0 0 0 0 0 1
  ENDATA
```

END OF LISTING

TR-20 DATA INPUT FOR:
CAVE CREEK WATERSHED

SUMMARY TABLE 1

ALT	STORM	ID	DA SQ-MI.	RAIN TBLE	AMC	DELTA-T HRS.	TZERO HRS.	PRECIP IN.	PRECIP DURATION	PEAK-Q CFS	PEAK-TIME	PEAK-ELEV	RUNOFF IN.	CSM
1	1	1	80.28	1	2	.20	0.00	3.29	24.00	13212.43	14.69	0.00	1.61	164.58
1	1	28	80.28	1	2	.20	0.00	3.29	24.00	13187.85	14.90	0.00	1.61	164.27
1	1	28	34.86	1	2	.20	0.00	3.33	24.00	10746.21	13.27	0.00	1.75	308.27
1	1	28	115.14	1	2	.20	0.00	3.33	24.00	19643.48	13.90	0.00	1.65	170.61
1	1	29	115.14	1	2	.20	0.00	3.33	24.00	19577.37	14.22	0.00	1.65	170.03
1	1	29	6.38	1	2	.20	0.00	3.10	24.00	2377.49	12.57	0.00	1.32	372.65
1	1	29	121.52	1	2	.20	0.00	3.10	24.00	20129.47	14.18	0.00	1.63	165.65
1	1	30	4.81	1	2	.20	0.00	3.10	24.00	1928.89	12.43	0.00	1.26	401.02
1	1	31	4.81	1	2	.20	0.00	3.10	24.00	1828.84	12.77	0.00	1.26	380.22
1	1	31	.59	1	2	.20	0.00	3.10	24.00	376.51	12.06	0.00	1.27	638.16
1	1	31	5.40	1	2	.20	0.00	3.10	24.00	1914.15	12.75	0.00	1.26	354.47
1	1	32	126.92	1	2	.20	0.00	3.10	24.00	20610.58	14.13	0.00	1.62	162.39
1	2	1	80.28	1	2	.20	0.00	4.42	24.00	21480.38	14.66	0.00	2.56	267.57
1	2	28	80.28	1	2	.20	0.00	4.42	24.00	21414.28	14.85	0.00	2.56	266.74
1	2	28	34.86	1	2	.20	0.00	4.41	24.00	16668.87	13.25	0.00	2.68	478.17
1	2	28	115.14	1	2	.20	0.00	4.41	24.00	31428.40	13.88	0.00	2.60	272.96
1	2	29	115.14	1	2	.20	0.00	4.41	24.00	31301.63	14.20	0.00	2.60	271.86
1	2	29	6.38	1	2	.20	0.00	4.20	24.00	4065.01	12.54	0.00	2.20	637.15
1	2	29	121.52	1	2	.20	0.00	4.20	24.00	32182.89	14.15	0.00	2.58	264.84
1	2	30	4.81	1	2	.20	0.00	4.20	24.00	3347.66	12.41	0.00	2.12	695.98
1	2	31	4.81	1	2	.20	0.00	4.20	24.00	3172.77	12.74	0.00	2.12	659.62
1	2	31	.59	1	2	.20	0.00	4.20	24.00	649.57	12.05	0.00	2.13	1100.97
1	2	31	5.40	1	2	.20	0.00	4.20	24.00	3315.77	12.72	0.00	2.13	614.03
1	2	32	126.92	1	2	.20	0.00	4.20	24.00	32975.39	14.10	0.00	2.56	259.81
1	3	1	80.28	1	2	.20	0.00	4.70	24.00	23603.94	14.65	0.00	2.81	294.02
1	3	28	80.28	1	2	.20	0.00	4.70	24.00	23527.20	14.84	0.00	2.81	293.06
1	3	28	34.86	1	2	.20	0.00	4.80	24.00	18864.46	13.24	0.00	3.03	541.15
1	3	28	115.14	1	2	.20	0.00	4.80	24.00	35000.54	13.86	0.00	2.87	303.98
1	3	29	115.14	1	2	.20	0.00	4.80	24.00	34854.32	14.18	0.00	2.88	302.71
1	3	29	6.38	1	2	.20	0.00	4.70	24.00	4871.41	12.53	0.00	2.63	763.54
1	3	29	121.52	1	2	.20	0.00	4.70	24.00	35906.32	14.12	0.00	2.86	295.48
1	3	30	4.81	1	2	.20	0.00	4.70	24.00	4028.78	12.40	0.00	2.54	837.58
1	3	31	4.81	1	2	.20	0.00	4.70	24.00	3818.32	12.73	0.00	2.54	793.83
1	3	31	.59	1	2	.20	0.00	4.70	24.00	780.38	12.04	0.00	2.55	1322.68
1	3	31	5.40	1	2	.20	0.00	4.70	24.00	3988.80	12.71	0.00	2.54	738.67
1	3	32	126.92	1	2	.20	0.00	4.70	24.00	36860.94	14.07	0.00	2.85	290.43

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*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*                                      *
*          FEBRUARY 1981               *
*                                      *
*         REVISED 05 DEC 88            *
*                                      *
*                                      *
*                                      *
*  RUN DATE  01/10/1990  TIME  16:01:06 *
*                                      *
*                                      *
*                                      *
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U.S. ARMY CORPS OF ENGINEERS

THE HYDROLOGIC ENGINEERING CENTER

609 SECOND STREET

DAVIS, CALIFORNIA 95616

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```
X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X              X
XXXXXXX  XXXX     X        XXXXX X
X     X  X        X              X
X     X  X        X     X        X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID    TEST OF COMPARISON: HYDROLOGY MANUAL METHODS VERSUS MOSTLY USED METHODS
  2           ID    APPLICATION TO LARGE WATERSHEDS
  3           ID    UPPER CAVE CREEK WATERSHED
  4           ID    ORIGINAL ANAYLSIS IN 1977, THIS ANAYSIS BY DDK, DEC. 1989.
  5           IT      20                    120
  6           IO

  7           KK
  8           KM   UPPER CAVE CREEK BASIN
  9           PB    2.97
 10           IN      15
 11           PC   0.000   0.021   0.035   0.051   0.071   0.087   0.105   0.125   0.143   0.160
 12           PC   0.179   0.201   0.232   0.281   0.364   0.500   0.658   0.773   0.841   0.888
 13           PC   0.927   0.958   0.965   0.976   1.000
 14           BA   80.29
 15           LU    .73     .12
 16           UI    695.   1410.   2571.   4072.   6090.   8011.  10016.  10120.  11202.  11202.
 17           UI  11202.   8727.   7811.   7156.   5831.   5454.   5325.   3846.   3638.   3451.
 18           UI   2677.   2543.   2430.   2327.   2232.   2119.   1470.   1416.   1366.   1299.
 19           UI    718.    695.    674.    654.    687.    687.    383.    372.    362.    353.
 20           UI    344.    335.    327.    320.    284.    169.    166.    162.    159.    1552
 21           UI    152.    149.     93.      0.      0.

 22           KK      1
 23           KM   ROUTE TO ANDORA HILLS CONFLUENCE
 24           RM      1     .25     .20

 25           KK
 26           KM   MIDDLE BASIN
 27           PB   3.168
 28           IN      15
 29           PC   0.000   0.018   0.027   0.040   0.058   0.074   0.089   0.106   0.122   0.137
 30           PC   0.155   0.174   0.201   0.249   0.331   0.485   0.665   0.786   0.856   0.901
 31           PC   0.937   0.967   0.978   0.988   1.000
 32           BA   34.86
 33           LU    .75     .18
 34           UI   1028.   3486.   7373.   9720.  10463.   8065.   6085.   4896.   3441.   2683.
 35           UI   2683.   2263.   2104.   1353.   1215.    656.    614.    440.    337.    319.
 36           UI    349.    160.    152.    145.    139.      1.      0.

 37           KK
 38           KM   ADD HYDROGRAPHS
 39           HC      2

 40           KK
 41           KM   ROUTE TO LOWER WATERSHED
 42           RM      2     .70     .25
```

```
KK
KM   LOWER BASIN
PB   3.456
IN     15
PC   0.000   0.010   0.016   0.025   0.039   0.051   0.062   0.073   0.085   0.097
PC   0.110   0.125   0.146   0.191   0.277   0.461   0.682   0.817   0.885   0.926
PC   0.957   0.980   0.985   0.988   1.000
BA   6.38
LU    .75     .20
UI   304.   1136.   2119.   2422.   1771.   1248.    896.    630.    512.    399.
UI   399.    297.    151.    137.     80.     74.     73.     36.     34.     32.
UI     0.      0.

KK
KM   ADD HYDROGRAPH
HC     2

KK
KM   BASIN ALONG CAREFREE HIGHWAY
PB   3.49
IN     15
PC   0.000   0.009   0.014   0.023   0.036   0.046   0.057   0.067   0.078   0.089
PC   0.101   0.116   0.137   0.181   0.268   0.457   0.687   0.825   0.890   0.930
PC   0.961   0.982   0.987   0.989   1.000
BA   4.81
LU    .75     .16
UI   204.    745.   1431.   1724.   1406.    986.    734.    518.    386.    386.
UI   355.    222.    174.    105.     87.     56.     53.     50.     26.     24.
UI    23.      2.      .0

KK
KM   ROUTE
RM     2     .58     .25

KK
KM   SMALL LOWER BASIN
PB   3.60
IN     15
PC   0.000   0.005   0.009   0.014   0.022   0.030   0.038   0.047   0.054   0.062
PC   0.075   0.088   0.107   0.127   0.205   0.366   0.823   0.900   0.920   0.939
PC   0.952   0.965   0.977   0.988   1.000
BA   0.59
LU    .75     .20
UI   212.    484.    239.    114.     52.     22.     13.     13.      6.      0.

KK
KM   ADD HYDROGRAPH
HC     3
ZZ
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT		19971.	7.00	12977.	4138.	2504.	80.29		
+	ROUTED TO	1	19712.	7.33	12959.	4138.	2504.	80.29		
+	HYDROGRAPH AT		16759.	5.33	7284.	1884.	1140.	34.86		
+	2 COMBINED AT		27089.	6.00	19609.	6022.	3644.	115.15		
+	ROUTED TO		26753.	7.00	19426.	6022.	3644.	115.15		
+	HYDROGRAPH AT		4443.	5.00	1568.	397.	240.	6.38		
+	2 COMBINED AT		27886.	6.67	20586.	6419.	3884.	121.53		
+	HYDROGRAPH AT		3361.	5.00	1255.	318.	193.	4.81		
+	ROUTED TO		3125.	5.67	1250.	318.	193.	4.81		
+	HYDROGRAPH AT		986.	4.33	152.	38.	23.	.59		
+	3 COMBINED AT		29790.	6.33	21845.	6775.	4099.	126.93		

*** NORMAL END OF HEC-1 ***

Examples

Use Original Examples in Printing.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 5

DETAIL EXAMPLE # 1 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

EXAMPLE # 1: DEVELOPMENT OF DEPTH-DURATION-FREQUENCY TABLE.

LOCATION: CAREFREE AIRPORT AT T6N, R4E, SEC 36.

① USING FIGURES 2.2 TO 2.13, READ LOCATION RAINFALL DEPTHS;

② PLOT DATA ON FIGURE 2.14 AND DRAW LINE OF BEST FIT;

RETURN FREQUENCY	6-HOUR DURATION DEPTH (INCHES)	24-HOUR DURATION DEPTH (INCHES)
2-YEAR	1.55	2.00
5-YEAR	2.00	~~2.70~~ 2.60 ⊛
10-YEAR	2.30	~~3.05~~ 3.10 ⊛
25-YEAR	~~2.78~~ 2.60 ⊛	~~3.70~~ 3.50 ⊛
50-YEAR	3.10	4.20
100-YEAR	3.40	4.70

⊛ CORRECTED VALUE FROM FIGURE 2.14.

③ THEN, DEPTHS FROM OTHER FREQUENCIES AND DURATIONS ARE CALCULATED:

$$Y_2 = -.011 + .942\,(X_1)(X_1/X_2)$$

$$Y_{100} = .494 + .755\,(X_3)(X_3/X_4)$$

WHERE: Y_2 = 2-YEAR, 1-HOUR DEPTH;

Y_{100} = 100-YEAR, 1-HOUR DEPTH;

X_1 = 2-YEAR, 6-HOUR DEPTH;

X_2 = 2-YEAR, 24-HOUR DEPTH;

X_3 = 100-YEAR, 6-HOUR DEPTH;

X_4 = 100-YEAR, 24-HOUR DEPTH.

$$\Rightarrow Y_2 = -.011 + .942(1.55)(1.55/2.00) = 1.12$$

$$Y_{100} = .494 + .755(3.40)(3.40/4.70) = 2.35$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 2 OF 5

DETAIL EXAMPLE #1 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

(4) PLOT Y_2 AND Y_{100} ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQUENCIES:

5-YEAR, 1-HOUR DEPTH = 1.35

10-YEAR, 1-HOUR DEPTH = 1.58

25-YEAR, 1-HOUR DEPTH = 1.82

50-YEAR, 1-HOUR DEPTH = 2.12

(5) NEXT THE 2-HOUR DEPTHS ARE CALCULATED:

2-YEAR, 2-HOUR DEPTH = .341(2-YEAR, 6-HOUR) + .659(2-YEAR, 1-HOUR)

= .341(1.55) + .659(1.12) = 1.27

100-YEAR, 2-HOUR DEPTH = .341(100-YEAR, 6-HOUR) + .659(100-YEAR, 1-HOUR)

= .341(3.40) + .659(2.35) = 2.71

PLOT ABOVE DEPTHS ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQUENCIES:

5-YEAR, 2-HOUR DEPTH = 1.62

10-YEAR, 2-HOUR DEPTH = 1.87

25-YEAR, 2-HOUR DEPTH = 2.12

50-YEAR, 2-HOUR DEPTH = 2.46

(6) NEXT THE 3-HOUR DEPTHS ARE CALCULATED:

2-YEAR, 3-HOUR DEPTH = .569(2-YEAR, 6-HOUR) + .431(2-YEAR, 1-HOUR)

= .569(1.55) + .431(1.12) = 1.36

100-YEAR, 3-HOUR DEPTH = .569(100-YEAR, 6-HOUR) + .431(100-YEAR, 1-HOUR)

= .569(3.40) + .431(2.35) = 2.95

PLOT ABOVE DEPTHS ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQ.:

5-YEAR, 3-HOUR DEPTH = 1.74

10-YEAR, 3-HOUR DEPTH = 2.05

25-YEAR, 3-HOUR DEPTH = 2.30

50-YEAR, 3-HOUR DEPTH = 2.65

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 3 OF 5

DETAIL EXAMPLE #1 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

(7) USING FIGURE 2.15 FIND THE 12-HOUR FREQUENCY DEPTHS. THIS IS DONE BY DRAWING A STRAIGHT LINE BETWEEN THE 6-HOUR AND THE 12-HOUR DEPTHS FOR ALL FREQUENCIES:

2-YEAR, 12-HOUR DEPTH = 1.80
5-YEAR, 12-HOUR DEPTH = 2.30
10-YEAR, 12-HOUR DEPTH = 2.70
25-YEAR, 12-HOUR DEPTH = 3.10
50-YEAR, 12-HOUR DEPTH = 3.65
100-YEAR, 12-HOUR DEPTH = 4.10

(8) 30-MIN DEPTHS ARE FOUND FROM THE FOLLOWING:

(2-YEAR, 30-MIN DEPTH) = .82 (2-YEAR, 1-HOUR) = .82 (1.12) = .92
(100-YEAR, 30-MIN DEPTH) = .82 (100-YEAR, 1-HOUR) = .82 (2.35) = 1.93

PLOT THE ABOVE VALUES ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQUENCIES:

5-YEAR, 30-MIN DEPTH = 1.15
10-YEAR, 30-MIN DEPTH = 1.35
25-YEAR, 30-MIN DEPTH = 1.50
50-YEAR, 30-MIN DEPTH = 1.75

(9) 15-MIN DEPTHS ARE ESTIMATED FROM THE FOLLOWING:

(2-YEAR, 15-MIN DEPTH) = .62 (2-YEAR, 1-HOUR) = .69
(100-YEAR, 15-MIN DEPTH) = .62 (100-YEAR, 1-HOUR) = 1.46

PLOT ABOVE DEPTHS ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQ.:

5-YEAR, 15-MIN DEPTH = .87
10-YEAR, 15-MIN DEPTH = 1.00
25-YEAR, 15-MIN DEPTH = 1.15
50-YEAR, 15-MIN DEPTH = 1.28

(10) 10-MIN DEPTHS ARE FOUND FROM THE FOLLOWING:

(2-YEAR, 10-MIN DEPTH) = .51 (2-YEAR, 1-HOUR) = .51 (1.12) = .57

(100-YEAR, 10-MIN DEPTH) = .51 (100-YEAR, 1-HOUR) = .51(2.35) = 1.20

→ PLOT ABOVE DEPTHS ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQUENCIES:

5-YEAR, 10-MIN DEPTH = .70

10-YEAR, 10-MIN DEPTH = .83

25-YEAR, 10-MIN DEPTH = .92

50-YEAR, 10-MIN DEPTH = 1.10

(11) 5-MIN DEPTHS ARE FOUND FROM THE FOLLOWING:

(2-YEAR, 5-MIN DEPTH) = .34 (2-YEAR, 1-HOUR) = .34 (1.12) = .38

(100-YEAR, 5-MIN DEPTH) = .34 (100-YEAR, 1-HOUR) = .34(2.35) = .80

→ PLOT ABOVE DEPTHS ON FIGURE 2.14 AND FIND DEPTHS FOR OTHER FREQUENCIES:

5-YEAR, 5-MIN DEPTH = .50

10-YEAR, 5-MIN DEPTH = .58

25-YEAR, 5-MIN DEPTH = .62

50-YEAR, 5-MIN DEPTH = .73

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 5 OF 5

DETAIL EXAMPLE #1 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

DEPTH-DURATION-FREQUENCY TABLE FOR:

CARE FREE AIRPORT

FREQUENCY

DURATION	2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
5-MIN	.38	.50	.58	.62	.73	.80
10-MIN	.57	.70	.83	.92	1.10	1.20
15-MIN	.69	.87	1.00	1.15	1.28	1.46
30-MIN	.92	1.15	1.35	1.50	1.75	1.93
1-HOUR	1.12	1.35	1.58	1.82	2.12	2.35
2-HOUR	1.27	1.62	1.87	2.12	2.46	2.71
3-HOUR	1.36	1.74	2.05	2.30	2.65	2.95
6-HOUR	1.55	2.00	2.30	2.60	3.10	3.40
12-HOUR	1.80	2.30	2.70	3.10	3.65	4.10
24-HOUR	2.00	2.60	3.10	3.55	4.20	4.70

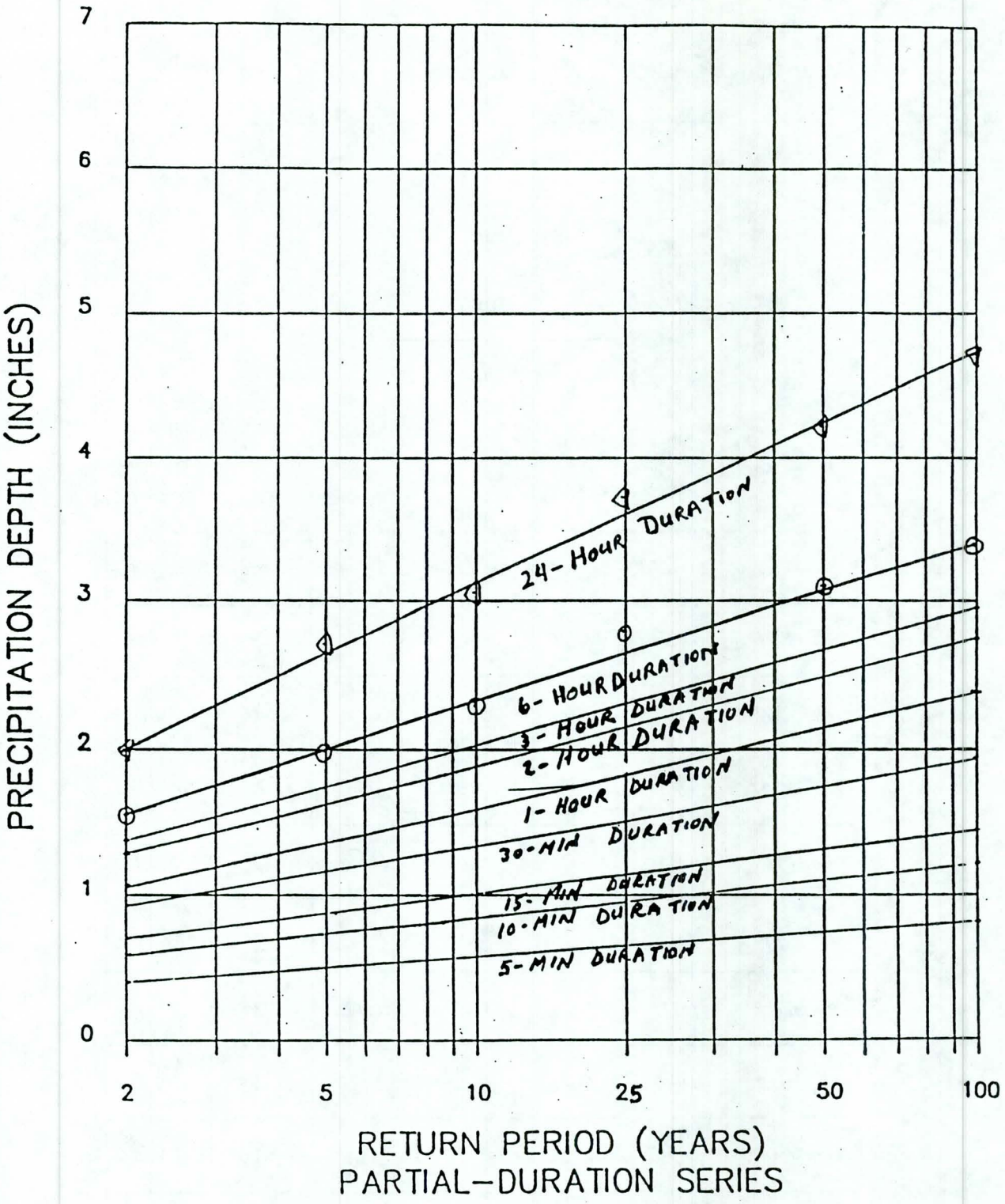


Figure 2.14
Percipitation Depth versus Return Period for Partial-Duration Series

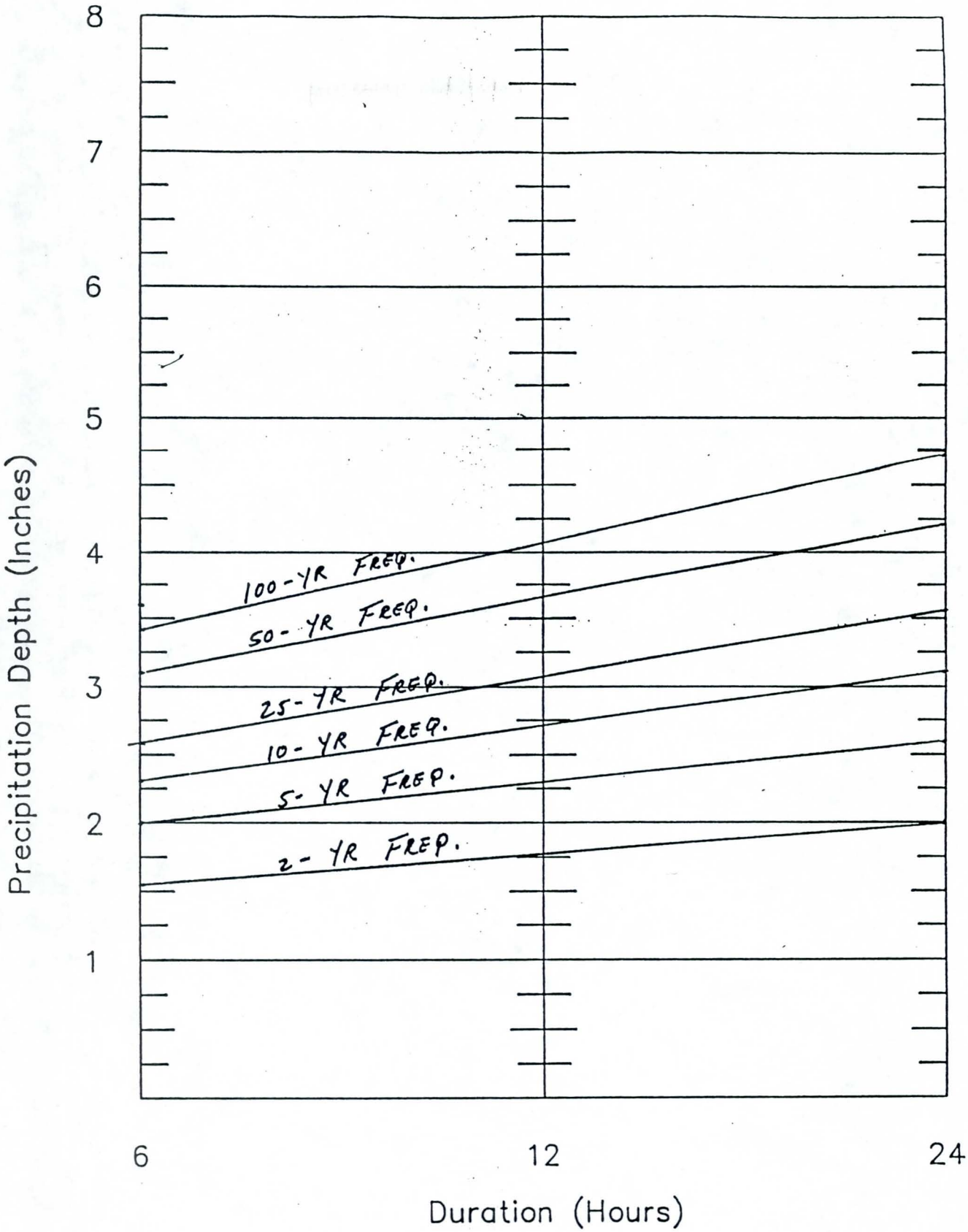


Figure 2.15
Percipitation Depth-Duration Diagram (6 to 24 hour)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 3

DETAIL EXAMPLE #2 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

EXAMPLE # 2: DEVELOPMENT OF INTENSITY-DURATION-FREQUENCY TABLE & GRAPH

LOCATION: CARE FREE AIRPORT AT T6N, R4E, SEC 36.

THE DEPTH-DURATION-FREQUENCY TABLE OF EXAMPLE #1 WILL BE USED.

THE TABLE OF EXAMPLE #1 WILL BE CONVERTED TO INTENSITY VALUES (IN/HR):

5-MIN DURATION DEPTHS:

2-YEAR FREQUENCY: $\frac{.38\ (IN)}{5\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 4.56\ IN/HR$

5-YEAR FREQUENCY: $\frac{.50\ (IN)}{5\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 6.00\ IN/HR$

10-YEAR FREQUENCY: $\frac{.58\ (IN)}{5\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 6.98\ IN/HR$

25-YEAR FREQUENCY: $\frac{.62\ (IN)}{5\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 7.44\ IN/HR$

50-YEAR FREQUENCY: $\frac{.73\ (IN)}{5\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 8.76\ IN/HR$

100-YEAR FREQUENCY: $\frac{.80\ (IN)}{5\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 9.60\ IN/HR$

10-MIN DURATION DEPTHS: ⊗

2-YEAR FREQUENCY: $\frac{.57\ (IN)}{10\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 3.42\ IN/HR$

15-MIN DURATION DEPTHS: ⊗

2-YEAR FREQUENCY: $\frac{.69\ (IN)}{15\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 2.76\ IN/HR$

30-MIN DURATION DEPTH: ⊗

2-YEAR FREQUENCY: $\frac{.92\ (IN)}{30\text{-}MIN} \times \frac{60\ (MIN)}{HR} = 1.84\ IN/HR$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____ PAGE 2 OF 3

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

1-HOUR DURATION DEPTHS: ⊛

2-YEAR FREQUENCY: $\frac{1.12\,(IN)}{60\,MIN} \times \frac{60\,(MIN)}{HR} = 1.12\ IN/HR$

2-HOUR DURATION DEPTHS: ⊛

2-YEAR FREQUENCY: $\frac{1.27\,(IN)}{120\,MIN} \times \frac{60\,(MIN)}{HR} = .64\ IN/HR$

3-HOUR DURATION DEPTHS: ⊛

2-YEAR FREQUENCY: $\frac{1.36\,(IN)}{180\,MIN} \times \frac{60\,(MIN)}{HR} = .45\ IN/HR$

6-HOUR DURATION DEPTHS: ⊛

2-YEAR FREQUENCY: $\frac{1.55\,(IN)}{360\,MIN} \times \frac{60\,(MIN)}{HR} = .26\ IN/HR$

12-HOUR DURATION DEPTHS: ⊛

2-YEAR FREQUENCY: $\frac{1.80\,(IN)}{720\,MIN} \times \frac{60\,(MIN)}{HR} = .15\ IN/HR$

24-HOUR DURATION DEPTHS: ⊛

2-YEAR FREQUENCY: $\frac{2.00\,(IN)}{1440\,MIN} \times \frac{60\,(MIN)}{HR} = .08\ IN/HR$

⊛ VALUES FOR OTHER FREQUENCIES (5-YEAR, 10-YEAR, ETC) CAN BE CALCULATED IN A SIMILAR WAY AS THE 2-YEAR FREQUENCY.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 3 OF 3

DETAIL EXAMPLE # 2 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

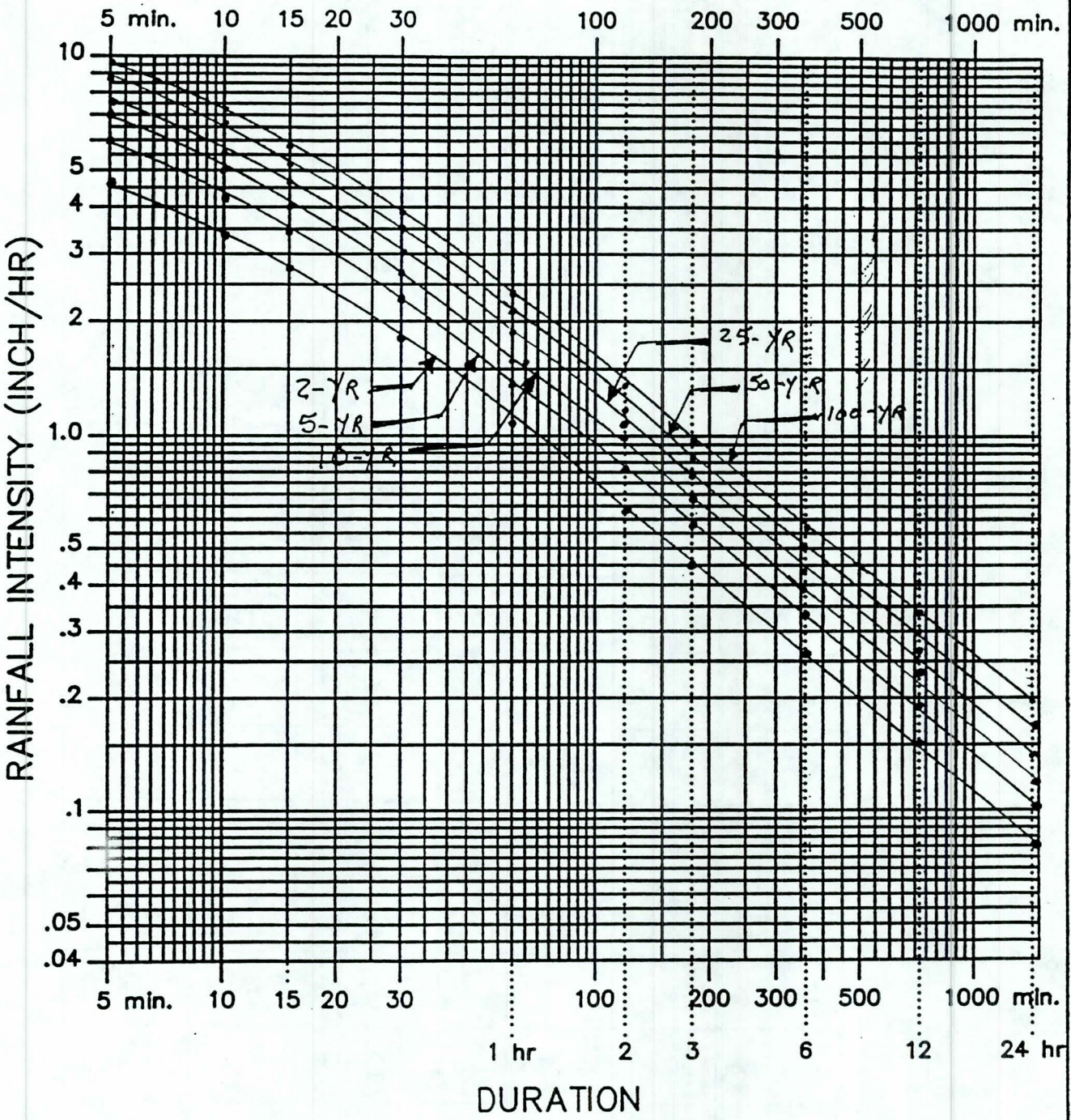
INTENSITY-DURATION-FREQUENCY TABLE FOR:

CARE FREE AIRPORT

FREQUENCY

DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
5-MIN	4.56	6.00	6.98	7.44	8.76	9.60
10-MIN	3.42	4.20	4.98	5.52	6.60	7.20
15-MIN	2.76	3.48	4.00	4.60	5.12	5.84
30-MIN	1.84	2.30	2.70	3.00	3.50	3.86
1-HOUR	1.12	1.35	1.58	1.82	2.12	2.35
2-HOUR	.64	.81	.94	1.06	1.23	1.36
3-HOUR	.45	.58	.68	.77	.88	.98
6-HOUR	.26	.33	.38	.43	.52	.57
12-HOUR	.15	.19	.23	.26	.30	.34
24-HOUR	.08	.11	.12	.14	.17	.19

NEXT PAGE SHOWS THE CORRESPONDING GRAPH.



RAINFALL INTENSITY–DURATION–FREQUENCY RELATION

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL

DETAIL EXAMPLE # 3 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

EXAMPLE # 3-A: DESIGN A 100-YEAR, 6-HOUR RAINFALL DISTRIBUTION FOR A 0.2 Mi^2 BASIN IN THE CAREFREE AIRPORT AREA.

PROCEDURE: SINCE THIS IS A SMALL BASIN, NO AREAL REDUCTION IS NEEDED. ALSO, SINCE THIS BASIN IS LESS THAN 0.5 Mi^2, ITS MASS CURVE WOULD BE SIMILAR TO THE PATTERN #1 MASS CURVE OF THE MANUAL. SO THE SAME PROCEDURES (NOAA) WILL BE USED.

(1) DESIGN A TABLE OF 15-MIN TIME INTERVALS FOR A 6-HOUR LONG DURATION;

(2) CALCULATE THE HIGHEST 15-MIN RAINFALL DEPTH. THIS IS DONE BY USING THE INFORMATION FROM EXAMPLE #1:

15-MIN DEPTH = ((30 MIN) - (15 MIN)) DEPTH = 1.93 - 1.46 = 0.47.

THIS VALUE IS CONSIDERED TO BE THE MOST INTENSE PORTION OF THE RAINFALL DISTRIBUTION AND THUS MUST BE PLACED AT THE CRITICAL LOCATION OF THE MASS CURVE. GENERALLY IT IS PLACED AT THE CENTER OF THE MASS CURVE. FOR THE PURPOSES OF THIS EXAMPLE, WE WILL PLACE IT TO THE RIGHT. THIS WILL BE IN ACCORDANCE WITH THE RAINFALL PATTERNS IN THE MANUAL, ALL OF WHICH ARE SHIFTED TO THE RIGHT BY 45 MINUTES. THEN, PLACE THE 15-MIN PEAK PRODUCING VALUE AT TIME 4:00 OF THE DESIGN TABLE.

CALCULATE THE HIGHEST 30-MIN VALUE AND CONTINUE ON IN THAT FASION:

30-MIN DEPTH = ((1 HOUR) - (30 MIN)) DEPTH = (2.35 - 1.93)/2 = .21

THIS VALUE WILL BE PLACED AT TIMES 3:30 AND 4:15.

1-HOUR DEPTH = ((2 HOUR) - (1 HOUR)) DEPTH = (2.71 - 2.35)/4 = .09

THIS VALUE IS PLACED AT TIMES 3:00, 3:15, 4:30, AND 4:45.

1-HOUR DEPTH = ((3 HOUR) - (2 HOUR)) DEPTH = (2.95 - 2.71)/4 = .06

THIS VALUE IS PLACED AT TIMES 2:30, 2:45, 5:00, AND 5:15.

3-HOUR DEPTH = ((6 HOUR) - (3 HOUR)) DEPTH = (3.40 - 2.95)/12 = .0375

THIS VALUE IS PLACED AT TIMES 1:00 TO 2:15 AND 5:30 TO 6:00.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____ PAGE 2 OF 5

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

SINCE THE 15-MIN PORTION OF THE DISTRIBUTION WAS SHIFTED BY 45 MINUTES, THE LAST 3 NUMBERS WILL GO BEYOND 6:00. THIS IS TAKEN CARE OF BY PLACING THESE NUMBERS FOR TIMES 0:15 TO 0:45 WHICH CURRENTLY HAVE NO NUMBERS IN THEM.

(4) THE GENERATED NUMBERS CAN NOW BE REPRESENTED IN THE CUMULATIVE FORM WHICH CAN BE DIRECTLY USED FOR HEC-1 INPUT.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 3 OF 5

DETAIL EXAMPLE # 3-A COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

DESIGN RAINFALL DISTRIBUTION TABLE

TIME (HOURS)	RAINFALL (INCHES)	CUM. RAINFALL (INCHES)
0:00	0.00	0.00
0:15	.0375	.0375
0:30	.0375	.075
0:45	.0375	.1125
1:00	.0375	.15
1:15	.0375	.1875
1:30	.0375	.225
1:45	.0375	.2625
2:00	.0375	.30
2:15	.0375	.3375
2:30	0.06	.3975
2:45	0.06	.4575
3:00	0.09	.5475
3:15	0.09	.6375
3:30	0.21	.8475
3:45	0.47	1.3175
4:00	1.46	2.7775
4:15	0.21	2.9875
4:30	0.09	3.0775
4:45	0.09	3.1675
5:00	0.06	3.2275
5:15	0.06	3.2875
5:30	.0375	3.325
5:45	.0375	3.3625
6:00	.0375	3.40

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #3 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

EXAMPLE #3-B: DESIGN A 100-YEAR, 6-HOUR RAINFALL DISTRIBUTION FOR A 5.0 MI^2 WATERSHED IN THE CAREFREE AIRPORT AREA.

PROCEDURE: SINCE THIS WATERSHED IS LARGER THAN 0.5 MI^2, IT REQUIRES AREAL REDUCTION. USING FIGURE 2.1 FROM THE MANUAL, A REDUCTION FACTOR OF 0.965 IS USED. THIS WILL RESULT IN $.965(3.40) = 3.28$ IN, WHERE 3.40 IS THE 100-YEAR, 6-HOUR POINT RAINFALL VALUE, COMPUTED IN EXAMPLE #1. FROM FIGURE 2.18, THE PATTERN DISTRIBUTION FOR THIS AREA IS BETWEEN PATTERN #2 AND PATTERN #3. THE CLOSEST VALUE WOULD BE 2.34, WHICH WE WILL REFER TO AS PATTERN # 2.34.

TO GET THE CORRESPONDING MASS CURVE, VALUES FROM PATTERN #2 ARE ADDED TO 34% OF THE DIFFERENCE BETWEEN THE VALUES OF PATTERN #2 AND PATTERN #3, WHICH ARE READ FROM TABLE 2.2.

ONCE PATTERN # 2.34 IS CONSTRUCTED, ITS ELEMENTS ARE MULTIPLIED BY THE RAINFALL (3.28 IN), AND THEN THEY CAN BE USED AS DIRECT INPUT INTO HEC-1 BY DIVIDING THE ELEMENTS BY 100.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 5 OF 5

DETAIL EXAMPLE # 3 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

DESIGN RAINFALL WITH AREAL REDUCTION

TIME	PATTERN #2	PATTERN #3	.34(#3-#2)	PATTERN # 2.34	DESIGN RAINFALL
0:00	0.0	0.0	0.0	0.0	0.0
0:15	0.6	1.5	0.3	0.9	.030
0:30	1.2	2.0	0.3	1.5	.049
0:45	2.0	3.0	0.3	2.3	.075
0:60	3.1	4.8	0.6	3.7	.121
1:15	3.9	6.3	0.8	4.7	.154
1:30	4.9	7.6	0.9	5.8	.190
1:45	5.7	9.0	1.1	6.8	.223
2:00	6.7	10.5	1.3	8.0	.262
2:15	7.6	11.9	1.5	9.1	.294
2:30	8.7	13.5	1.6	10.3	.338
2:45	10.0	15.2	1.8	11.8	.387
3:00	12.0	17.5	1.9	13.9	.456
3:15	16.3	22.2	2.0	18.3	.600
3:30	25.2	30.4	1.7	26.9	.882
3:45	45.1	47.2	0.7	45.8	1.502
4:00	69.1	67.0	-0.7	68.4	2.24
4:15	83.7	79.6	-1.4	82.3	2.70
4:30	90.0	86.8	-1.1	88.9	2.92
4:45	93.8	91.2	-0.9	92.9	3.05
5:00	96.7	94.6	-0.7	96.0	3.15
5:15	98.5	97.4	-0.4	98.1	3.22
5:30	99.0	98.0	-0.3	98.7	3.24
5:45	99.0	98.7	-0.1	98.9	3.244
6:00	100.0	100.0	0.0	100.0	3.28

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 2

DETAIL EXAMPLE #4 COMPUTED _____ DATE _____

RATIONAL METHOD CHECKED BY _____ DATE _____

SCENARIO: A RETENTION BASIN IS TO BE DESIGNED FOR AN URBAN WATERSHED WHICH HAS THE FOLLOWING PHYSICAL CHARACTERISTICS:

LOCATION → CAREFREE, T6N - R4E

AREA → 140 acres

FLOW PATH LENGTH → 1.236 mi.

AVERAGE SLOPE → 33 ft/mi

LAND USE → 70% SINGLE FAMILY RESIDENTIAL
30% LIGHT INDUSTRIAL

ESTIMATE THE PEAK DISCHARGE AT THE BASIN INLET AND THE VOLUME OF RUNOFF TO BE RETAINED.

STEP 1: DETERMINE THE RUNOFF COEFFICIENT "C" (TABLE 3.2)

70% RESIDENTIAL → 0.40

30% LIGHT INDUSTRIAL → 0.65

$$(.70)(.40) + (.30)(.65) = \underline{\underline{0.475}}$$

STEP 2: CALCULATE T_c

$$\underline{T_c = 11.4\, L^{.50}\, K_b^{.52}\, S^{-.31}\, i^{-.38}}$$

where $L = 1.236$ mi.

$K_b = 0.027$ (TABLE 3.1 OR FIGURE 3.1)

$S = 33$ ft/mi

PLUG IN THE KNOWN VARIABLES:

$$T_c = 11.4\,(1.236)^{.50}(.027)^{.52}(33.)^{-.31}(i)^{-.38}$$

$$T_c = 0.655\, i^{-.38}$$

CHOOSE A STARTING VALUE FOR T_c, SAY 30 min. AT T_c = 30 min., THE 100-YEAR RAINFALL INTENSITY IS 4.00 in/hr. (I-D-F CURVES, FIG. 3.2). SINCE THE WATERSHED IS OUTSIDE THE PHOENIX AREA, THE INTENSITY VALUES MUST BE ADJUSTED USING THE EQUATION IN SECTION 3.5.1.3:

$$i = i_p \frac{P_{10}^6}{2.07}$$

WHERE i_p IS THE INTENSITY VALUE FROM FIG. 3.2 AND P_{10}^6 IS THE 10-YR, 6-HR POINT RAINFALL DEPTH FROM FIG. 2.4.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____ PAGE 2 OF 2

DETAIL EXAMPLE #4 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

IN THIS CASE : $i_p = 4.00$ in/hr , $P_{10}^{6} = 2.30$ in

$$i_{100} = 4.00\left(\frac{2.30}{2.07}\right) = 4.44 \text{ in/hr}$$

AT $i_{100} = 4.44$ in/hr : $T_c = 0.655\,(4.44)^{-.38} = 22.3$ min No Good

TRY $T_c = 20$ min : $i_p = 5.10$ in/hr

$$i_{100} = 5.10\left(\frac{2.30}{2.07}\right) = 5.67 \text{ in/hr}$$

AT $i_{100} = 5.67$ in/hr : $T_c = 0.655\,(5.67)^{-.38} = 20.3$ min OK

So $T_{c_{100}} = 20$ min, $i_{100} = 5.67$ in/hr

STEP 3: CALCULATE PEAK DISCHARGE

$$Q_{PK} = C\,i_{100}\,A = (.475)(5.67)(140) = 377 \text{ cfs}$$

STEP 4: CALCULATE RETENTION VOLUME (V)

$$V = C\left(\frac{P_{100}^{2}}{12}\right)A$$

WHERE P_{100}^{2} IS THE 2-HOUR, 100-YEAR POINT RAINFALL DEPTH (in). AGAIN, SINCE THE WATERSHED IS OUTSIDE THE PHOENIX AREA, P_{100}^{2} MUST BE CALCULATED FROM THE DURATION CONVERSION EQUATIONS IN SECTION 2.4.4:

Y_{100} = 1-HOUR, 100-YEAR POINT RAINFALL (in)

$Y_{100} = 0.494 + 0.755\,(X_3)\,(X_3/X_4)$

WHERE X_3 = 6-HR, 100 YEAR POINT RAINFALL = 3.40 in (FIG 2.7)

X_4 = 24-HR, 100 YEAR POINT RAINFALL = 4.70 in (FIG 2.13)

SO: $Y_{100} = 0.494 + 0.755\,(3.4)(3.4/4.7) = 2.35$ in

$$X_2 \equiv P_{100}^{2} = 0.341\,(X_3) + 0.659\,(Y_{100})$$
$$= 0.341\,(3.4) + 0.659\,(2.35) = 2.71 \text{ in}$$

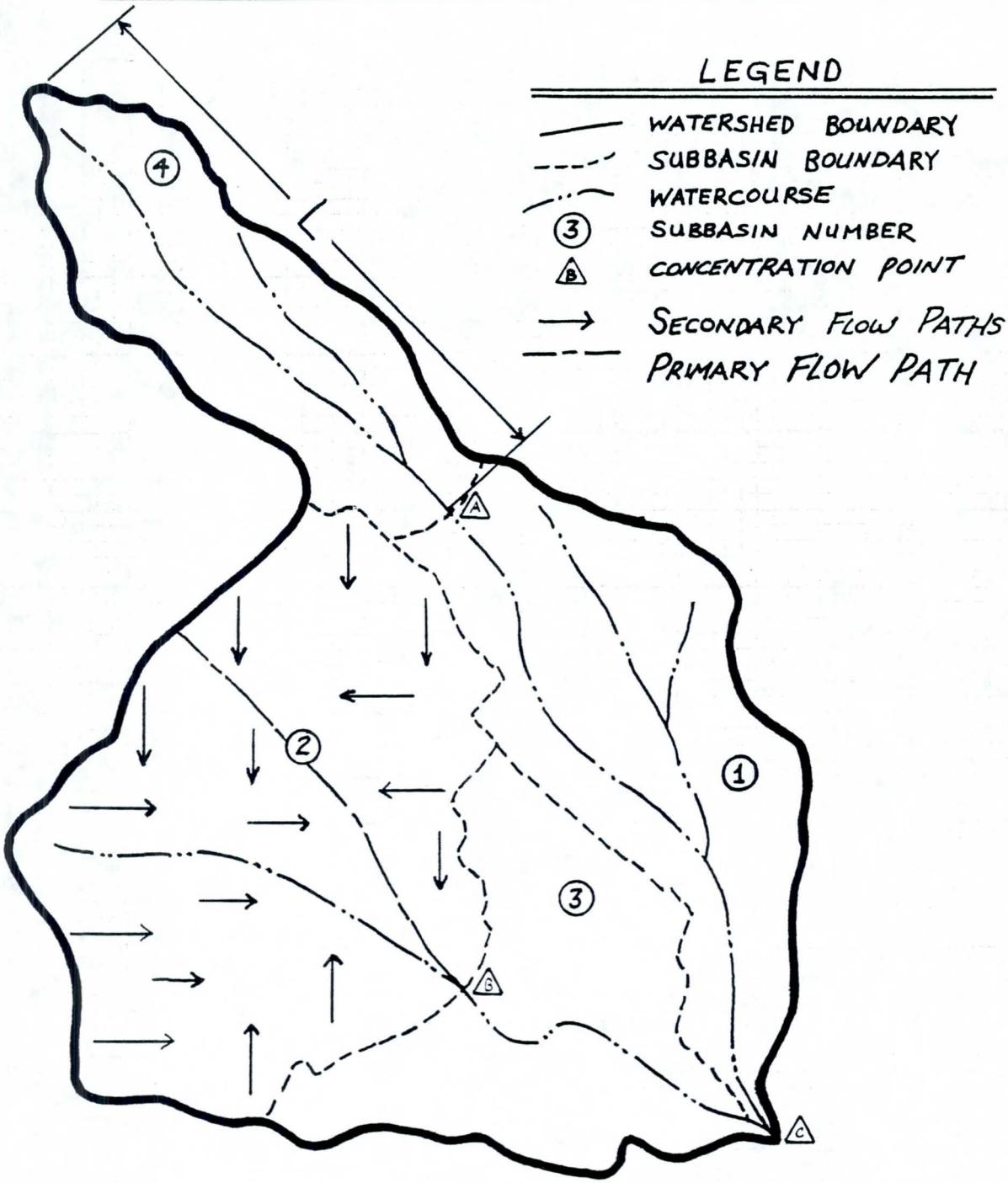
FINALLY: $V = 0.475\left(\frac{2.71}{12}\right)140 \text{ ac} = 15.02$ ac-ft

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 2

DETAIL EXAMPLE WATERSHED COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 2 OF 2

DETAIL EXAMPLE WATERSHED COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

SUBBASIN CHARACTERISTICS

SUBBASIN #	AREA (mi^2)	IMPERVIOUSNESS (%)	FLOW PATH LENGTH (mi)	SLOPE (ft/mi)	LAND USE
1	1.52	33	2.65	170.	40% MULTI-UNIT AREAS 60% APARTMENT AREAS
2	2.17	21	1.85	30.5	100% SINGLE FAMILY RESIDENTIAL
3	0.96	42	1.13	104.	50% LIGHT INDUSTRIAL 50% DOWNTOWN AREAS
4	0.86	9	1.49	537.	100% UNDEVELOPED Desert MOUNTAI

WATERCOURSES

SUBBASIN #	DESCRIPTION	GEOMETRY	AVE. BOTTOM WIDTH (ft)	AVE. DEPTH (ft)	SIDE SLOPE	MANNINGS 'n'
1	SOIL CEMENT LINED	TRAP.	25	5	2:1	.018
2	DREDGED EARTH	Rect.	15	4	—	.022
3	CONCRETE LINED	TRAP.	35	4	3:1	.015
4	NATURAL DESERT STREAM	TRAP.	15	2	2:1	.040

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

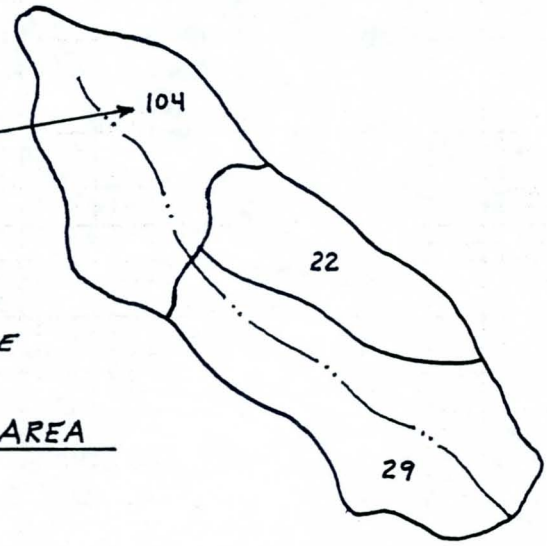
PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 2

DETAIL EXAMPLE #5 COMPUTED _____ DATE _____

GREEN & AMPT LOSS METHOD CHECKED BY _____ DATE _____

SCENARIO: CALCULATE THE GREEN AND AMPT LOSS PARAMETERS FOR SUBBASIN #4 OF THE EXAMPLE WATERSHED. ASSUME THAT THE WATERSHED IS LOCATED WITHIN THE BOUNDARIES OF THE "SOIL SURVEY OF AGUILA - CAREFREE AREA, PARTS OF MARICOPA AND PINAL COUNTIES, ARIZONA". ASSUME THE DESIGN STORM TO BE A 6-HR, 100-YR EVENT OF 3.5 to 4.0 INCHES.

NUMBERS ARE FOUND ON THE SOIL SURVEY MAPS AND INDICATE SOIL MAP UNITS.



STEP 1: PLANIMETER MAP UNIT AREAS WITHIN THE SUBBASIN. ASSUME FOR THIS CASE:

MAP UNIT	PERCENT TOTAL AREA
22	25%
29	35%
104	40%
	100%

NOTE: REFER TO "NOTES ON CALCULATING LOSS PARAMETERS" IN CHAPTER 7 AND APPEDICES A-D FOR ADVICE ON CONSTRUCTION OF THE FOLLOWING TABLE.

STEP 2: CONSTRUCT A TABLE SIMILAR TO THE FOLLOWING:

MAP UNIT #	% MAP UNIT	SOIL NAME	TEXTURAL CLASS	DTHETA (in)	PSIF (in)	XKSAT (in/hr)
22	80	CONTINE	CLAY	.15	12.4	.01
	20	CAREFREE	CLAY	.15	12.4	.01
		EBON	SANDY CLAY LOAM	.25	8.6	.06
		MOHALL	SILTY LOAM	.40	6.6	.15
29	40	DENURE	FINE SANDY LOAM	.35	3.5	.40
	30	MOMOLI	GRAVELLY SANDY LOAM			
	20	CARRIZO	GRAVELLY SANDY LOAM			
	10	GILMAN	LOAM	.35	4.3	.25
		MARIPO	SANDY LOAM	.35	3.5	.40
		CARRIZO	SAND	.35	1.9	4.60
104	20	LEHMANS	SANDY CLAY LOAM	.25	8.6	.06
	20	ARIZO	SANDY LOAM	.35	3.5	.40
		EBA	SANDY LOAM	.35	3.5	.40
		PINALENO	SANDY CLAY	.20	9.4	.02
	60	ROCK OUTCROP	———	—	—	—

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____ PAGE 2 OF 2

DETAIL EXAMPLE 5 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

STEP 3 : CALCULATE WEIGHTED PARAMETERS FOR EACH MAP UNIT

UNIT 22 : DTHETA = .80 (.15) + .067(.15) + .067(.25) + .067(.40) = .17 in
PSIF = .80(12.4) + .067(12.4) + .067(8.6) + .067(6.6) = 11.8 in
XKSAT = .80 (.01) + .067 (.01) + .067(.06) + .067(.15) = .02 in/hr

UNIT 29 : DTHETA = .35 in
PSIF = .90(3.5) + .033 (4.3) + .033 (3.5) + .033 (1.9) = 3.5 in
XKSAT = .90 (.40) + .033(.25) + .033(.40) + .033(4.6) = .53 in/hr

UNIT 104: DTHETA = .50 (.25) + .167 (.35) + .167(.35) + .167 (.20) = .28 in
PSIF = .50(8.6) + .167 (3.5) + .167 (3.5) + .167 (9.4) = 7.0 in
XKSAT = .50 (.06) + .167 (.40) + .167 (.40) + .167 (.02) = .17 in/hr
RTIMP = 60%

STEP 4: CALCULATE WEIGHTED PARAMETERS FOR THE SUBBASIN BY PERCENTAGE OF SOIL MAP UNITS

DTHETA = .25 (.17) + .35 (.35) + .40 (.28) = .28 in
PSIF = .25 (11.8) + .35 (3.5) + .40 (7.0) = 7.0 in
XKSAT = .25 (.02) + .35 (.53) + .40 (.17) = .26 in/hr
RTIMP = .40 (60) = 24%

STEP 5 : ADJUST XKSAT FOR PERCENT VEGETATION COVER USING FIGURE 4.10

ASSUME 25% AVERAGE VEGETATION COVER OVER THE BASIN, THEN

XKSAT = .26 (1.17) = 0.30 in/hr

STEP 6 : SELECT A SURFACE RETENTION LOSS (IA) FOR THE SUBBASIN FROM TABLE 4.1.

FOR THIS EXAMPLE, ASSUME

85% HILLSLOPES → .15 in
15% MOUNTAIN → .25 in

IA = .85 (.15) + .15 (.25) = 0.17 in

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 2

DETAIL EXAMPLE #6 COMPUTED _____ DATE _____

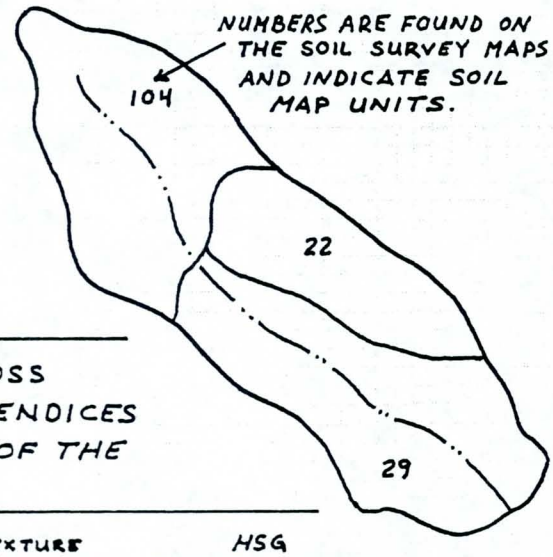
IL + ULR CHECKED BY _____ DATE _____

INITIAL LOSS PLUS UNIFORM LOSS RATE BY SOIL TEXTURE AND HYDROLOGIC SOIL GROUP (HSG)

SCENARIO: CALCULATE THE INITIAL AND UNIFORM LOSS RATE PARAMETERS FOR SUBBASIN #4 OF THE EXAMPLE WATERSHED. ASSUME THAT THE WATERSHED IS LOCATED WITHIN THE BOUNDARIES OF THE "SOIL SURVEY OF AGUILA - CAREFREE AREA, PARTS OF MARICOPA AND PINAL COUNTIES, ARIZONA". ASSUME THE DESIGN STORM TO BE A 6-HR, 100-YEAR EVENT OF 3.5 to 4.0 INCHES.

STEP 1: PLANIMETER MAP UNIT AREAS WITHIN THE SUBBASIN. ASSUME FOR THIS CASE:

MAP UNIT	PERCENT TOTAL AREA
22	25%
29	35%
104	40%
	100%



NOTE: REFER TO "NOTES ON CALCULATING LOSS PARAMETERS" IN CHAPTER 7 AND APPENDICES A-D FOR ADVICE ON CONSTRUCTION OF THE FOLLOWING TABLE.

STEP 2: CONSTRUCT A TABLE:

MAP UNIT #	% MAP UNIT	SOIL NAME	TEXTURAL CLASS	HSG	SOIL TEXTURE IL (in)	SOIL TEXTURE CNSTL (in/hr)	HSG IL (in)	HSG CNSTL (in/hr)
22	80	CONTINE	CLAY	C	.3	.01	.5	.15
	20	CAREFREE	CLAY		.3	.01		
		EBON	SANDY CLAY LOAM		.6	.06		
		MOHALL	SILTY LOAM		.6	.15		
29	40	DENURE	FINE SANDY LOAM	B	.7	.40	.5	.25
	30	MOMOLI	GRAVELLY SANDY LOAM					
	20	CARRIZO	GRAVELLY SANDY LOAM					
	10	GILMAN	LOAM		.6	.25		
		MARIPO	SANDY LOAM		.7	.40		
		CARRIZO	SAND		1.3	4.60		
104	20	LEHMANS	SANDY CLAY LOAM	D	.6	.06	.4	.05
	10	ARIZO	SANDY LOAM		.7	.40		
		EBA	SANDY LOAM		.7	.40		
		PINALENO	SANDY CLAY		.4	.02		
	60	ROCK OUTCROP	—		—	—	—	—

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____ PAGE 2 OF 2

DETAIL EXAMPLE #6 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

STEP 4: CALCULATE WEIGHTED PARAMETERS FOR EACH MAP UNIT

NOTE: SKIP THIS STEP IF USING THE LOSS PARAMETERS FOR HYDROLOGIC SOIL GROUPS

UNIT 22: $I_L = .80(.3) + .067(.3) + .067(.6) + .067(.6) = .34$ in

CNSTL $= .80(.01) + .067(.01) + .067(.06) + .067(.15) = .02$ in/hr

UNIT 29: $I_L = .90(.7) + .033(.6) + .033(.7) + .033(1.3) = .72$ in

CNSTL $= .90(.40) + .033(.25) + .033(.40) + .033(4.60) = .53$ in/hr

UNIT 104: $I_L = .50(.6) + .167(.7) + .167(.7) + .167(.4) = .60$ in

CNSTL $= .50(.06) + .167(.40) + .167(.40) + .167(.02) = .17$ in/hr

RTIMP = 60%

STEP 5: CALCULATE WEIGTED PARAMETERS FOR THE SUBBASIN BY PERCENTAGE OF SOIL MAP UNITS

FOR SOIL TEXTURE:

$I_L = .25(.34) + .35(.72) + .40(.60) = .58$ in

CNSTL $= .25(.02) + .35(.53) + .40(.17) = .26$ in/hr

RTIMP $= .40(60) = 24\%$

FOR HYDROLOGIC SOIL GROUPS:

$I_L = .25(.5) + .35(.5) + .40(.4) = .46$ in

CNSTL $= .25(.15) + .35(.25) + .40(.05) = .15$ in/hr

NOTE: DO NOT INCLUDE A VALUE FOR RTIMP WHEN USING HSG'S.

STEP 6: SELECT A SURFACE RETENTION LOSS (IA) FROM TABLE 4.1 AND CALCULATE STRTL:

FOR THIS EXAMPLE:

70% HILLSLOPES ⟶ .15 in

30% MOUNTAIN ⟶ .25 in

$IA = .7(.15) + .3(.25) = .18$ in

FOR SOIL TEXTURE: STRTL $= .58$ in $+ .18$ in $= .76$ in

FOR HYDROLOGIC SOIL GROUPS: STRTL $= .46$ in $+ .18$ in $= .64$ in

---

STEP 7: SELECT A VEGETATION COVER ADJUSTMENT FACTOR FOR THE BASIN FROM FIGURE 4.10.

ASSUME 25% AVERAGE VEGETATION COVER FOR THE ENTIRE BASIN, THEN

FOR SOIL TEXTURE: CNSTL $= .26(1.17) = .30$ in/hr

FOR HYDROLOGIC SOIL GROUPS: CNSTL $= .15(1.17) = .18$ in/hr

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 1

DETAIL EXAMPLE #7 COMPUTED _____ DATE _____

CLARK UNIT HYDROGRAPH (URBAN) CHECKED BY _____ DATE _____

SCENARIO: DEVELOP THE CLARK UNIT HYDROGRAPH INPUT PARAMETERS FOR SUBBASIN #2 OF THE EXAMPLE WATERSHED.

STEP 1: ASSEMBLE PHYSICAL BASIN CHARACTERISTICS:

AREA = 2.17 mi^2 = 1389 ac SLOPE (S) = 30.5 ft/mi

FLOW PATH (L) = 1.85 mi % IMPERVIOUSNESS = 21

STEP 2: CALCULATE THE BASIN RESISTANCE COEFFICIENT K_b USING FIG. 5.5, TABLE 5.1, OR THE "T_c & R WORKSHEET" (APPENDIX E).

SINCE THIS IS AN URBAN BASIN, m = -.00625 AND b = .04

$$K_b = m(\log A) + b = -.00625(\log 1389) + .04 = 0.020$$

STEP 3: CALCULATE T_c AS A FUNCTION OF i:

NOTE: REFER TO THE WORKSHEET DURING THE REMAINING STEPS, OR USE THE FORTRAN PROGRAM MCUHP1.EXE.

$$T_c = 11.4\, L^{.50} K_b^{.52} S^{-.31} i^{-.38}, \quad T_c = 11.4(1.85)^{.50}(.020)^{.52}(30.5)^{-.31} i^{-.38}, \quad \underline{T_c = .703\, i^{-.38}}$$

STEP 4: ENTER RAINFALL, LOSS, AND CLARK PARAMETER DATA INTO AN HEC-1 INPUT DECK, WITH T_c & R SET EQUAL TO ZERO. RUN TO GENERATE A RAINFALL-LOSS-EXCESS TABLE.

STEP 5: USING THE WORKSHEET AND THE RESULTS OF STEP 4, COMPUTE THE AVERAGE EXCESS INTENSITIES FOR A TIME PERIOD GREATER THAN T_c.

STEP 6: CREATE THE GRAPH OF AVERAGE EXCESS INTENSITY VS. TIME.

STEP 7: CALCULATE T_c BY ITERATION. "i" VALUES ARE READ FROM THE GRAPH. CALCULATE R.

STEP 8: ENTER THE T_c & R VALUES INTO THE HEC-1 INPUT DECK. SELECT A TIME-AREA RELATION, IN THIS CASE COLUMN 2 OF TABLE 5.2. RUN THE HEC-1 PROGRAM. (See the following sample HEC-1 input and output.)

(UPDATED)

CALCULATION OF Tc & R

Calculated by: _____ Date: _____
Checked by: _____ Project: _____

Watershed: EXAMPLE WATERSHED - SUBBASIN #2, EXAMPLE #7
Rainfall Frequency: _____ - yr Duration: _____ - hr. Pattern #: _____

Rainfall Loss Method:
[ ] Green & Ampt Method
[X] IL + ULR by soil texture
[ ] IL + ULR by hydrologic soil group

Tabulate Period of Peak Rainfall Excess

Clock Time @ end of Increm.	Increm. Excess in.
0335	.18
0340	.18
0345	.18
0350	.37
0355	.37
0400	.37
0405	.11
0410	.11

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

Accum. Time hr./min.	Increm. Excess in.	Accum. Excess in.	Avg. Excess Intensity in./hr.
5	.37	.37	4.44
10	.37	.74	4.44
15	.37	1.11	4.44
20	.18	1.29	3.87
25	.18	1.47	3.53
30	.18	1.65	3.30
35	.11	1.76	3.02
40	.11	1.87	2.81

A = 2.17 sq.mi.
L = 1.85 mi.
S = 30.5 ft/mi.

$Kb = m\,[\log(A * 640)] + b$

$Kb = (.00625)\log(2.17 * 640) + (.04)$

$Kb = .020$

$Tc = 11.4\,L^{.50}\,Kb^{.52}\,S^{-.31}\,i^{-.38}$

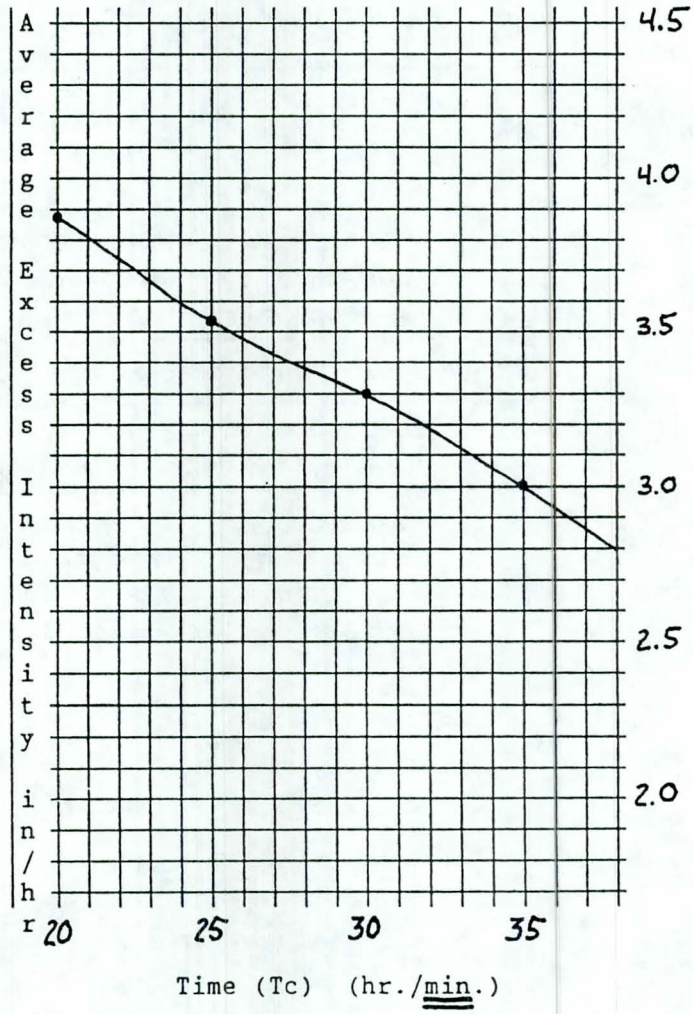
$Tc = (0.703)\,i^{-.38}$

Trial Tc	i	Calc. Tc
.417	3.53	.435
.450	3.42	.441
.430	3.48	.438
.440	3.45	.439

Tc = .440 hr.

$R = .37\,Tc^{1.11}\,A^{-.57}\,L^{.80}$

R = .156 hr.



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*             FEBRUARY 1981             *
*           REVISED 31 JAN 85           *
*                                       *
*     RUN DATE 9/ 5/1989   TIME15: 4: 0 *
*                                       *
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*************************************
*                                   *
*    U.S. ARMY CORPS OF ENGINEERS   *
*  THE HYDROLOGIC ENGINEERING CENTER *
*          609 SECOND STREET        *
*       DAVIS, CALIFORNIA 95616     *
*  (916) 440-3285 OR (FTS) 448-3285 *
*                                   *
*************************************
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X     X  XXXXXXX   XXXXX          X
X     X  X       X     X        XX
X     X  X       X               X
XXXXXXX  XXXX    X       XXXXX   X
X     X  X       X               X
X     X  X       X     X         X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE. THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85 CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT DESCRIPTION FOR NEW DEFINITIONS.

SAMPLE RUN

EXAMPLE #7

(UPDATED)

(UPDATED)


```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

*** FREE ***
  1           ID     SAMPLE EEC-1 RUN USING TECHNIQUES OUTLINED IN THE
  2           ID     MARICOPA COUNTY HYDROLOGY MANUAL
              * ******************************************************************************
  3           ID     EXAMPLE #7 - CLARK UNIT HYDROGRAPH
              * ******************************************************************************
  4           ID     RAINFALL: 6-HR, 100-YEAR POINT RAINFALL DEPTH OF 3.25 INCHES
  5           ID     HYDROGRAPH: CLARK - TC & R FROM WORKSHEET
  6           ID                 URBAN TIME-AREA CURVE
  7           ID     LOSSES: IL+ULR BY SOIL TEXTURE
  8           ID     BASIN AREA: 2.17  SQUARE MILES, PATTERN #1.5
              * ******************************************************************************

  9           IT       5 05SEP89    0000      85
 10           IO       0
              * ******************************************************************************
 11           KK  BASIN2
 12           KM   COMPUTE DISCHARGE AT THE OUTLET OF SUBBASIN #2
 13           IN      15 05SEP89    0000
 14           PB    3.25
 15           PC      0.      .55    1.05     1.7    2.65    3.45    4.35     5.2    6.05     6.9
 16           PC     8.1      9.4   11.35    14.5   22.85   40.85   75.85   86.85     91.   93.85
 17           PC   95.95     97.5   98.35    98.9    100.
 18           BA    2.17
 19           LU     .65      .20     21.
 20           UA       0        5      16      30      65      77      84      90      94      97
 21           UA     100
 22           UC    .440     .156
 23           ZZ
```

(UPDATED)

HYDROGRAPH AT STATION BASIN2

DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
5 SEP 0000	1	.00	.00	.00	0.	5 SEP 0335	44	.19	.01	.18	461.
5 SEP 0005	2	.01	.00	.00	1.	5 SEP 0340	45	.19	.01	.18	955.
5 SEP 0010	3	.01	.00	.00	4.	5 SEP 0345	46	.19	.01	.18	1563.
5 SEP 0015	4	.01	.00	.00	8.	5 SEP 0350	47	.38	.01	.37	2174.
5 SEP 0020	5	.01	.00	.00	12.	5 SEP 0355	48	.38	.01	.37	2980.
5 SEP 0025	6	.01	.00	.00	15.	5 SEP 0400	49	.38	.01	.37	3915.
5 SEP 0030	7	.01	.00	.00	17.	5 SEP 0405	50	.12	.01	.11	4552.
5 SEP 0035	8	.01	.01	.00	18.	5 SEP 0410	51	.12	.01	.11	4498.
5 SEP 0040	9	.01	.01	.00	20.	5 SEP 0415	52	.12	.01	.11	3901.
5 SEP 0045	10	.01	.01	.00	21.	5 SEP 0420	53	.04	.01	.03	3207.
5 SEP 0050	11	.01	.01	.00	23.	5 SEP 0425	54	.04	.01	.03	2501.
5 SEP 0055	12	.01	.01	.00	25.	5 SEP 0430	55	.04	.01	.03	1845.
5 SEP 0100	13	.01	.01	.00	28.	5 SEP 0435	56	.03	.01	.02	1351.
5 SEP 0105	14	.01	.01	.00	31.	5 SEP 0440	57	.03	.01	.02	991.
5 SEP 0110	15	.01	.01	.00	32.	5 SEP 0445	58	.03	.01	.02	729.
5 SEP 0115	16	.01	.01	.00	32.	5 SEP 0450	59	.02	.01	.01	552.
5 SEP 0120	17	.01	.01	.00	32.	5 SEP 0455	60	.02	.01	.01	420.
5 SEP 0125	18	.01	.01	.00	32.	5 SEP 0500	61	.02	.01	.01	321.
5 SEP 0130	19	.01	.01	.00	33.	5 SEP 0505	62	.02	.01	.00	249.
5 SEP 0135	20	.01	.01	.00	33.	5 SEP 0510	63	.02	.01	.00	197.
5 SEP 0140	21	.01	.01	.00	33.	5 SEP 0515	64	.02	.01	.00	150.
5 SEP 0145	22	.01	.01	.00	33.	5 SEP 0520	65	.01	.01	.00	115.
5 SEP 0150	23	.01	.01	.00	33.	5 SEP 0525	66	.01	.01	.00	89.
5 SEP 0155	24	.01	.01	.00	33.	5 SEP 0530	67	.01	.01	.00	68.
5 SEP 0200	25	.01	.01	.00	33.	5 SEP 0535	68	.01	.00	.00	54.
5 SEP 0205	26	.01	.01	.00	32.	5 SEP 0540	69	.01	.00	.00	43.
5 SEP 0210	27	.01	.01	.00	32.	5 SEP 0545	70	.01	.00	.00	35.
5 SEP 0215	28	.01	.01	.00	32.	5 SEP 0550	71	.01	.01	.00	30.
5 SEP 0220	29	.01	.01	.00	33.	5 SEP 0555	72	.01	.01	.00	29.
5 SEP 0225	30	.01	.01	.00	35.	5 SEP 0600	73	.01	.01	.00	32.
5 SEP 0230	31	.01	.01	.00	38.	5 SEP 0605	74	.00	.00	.00	33.
5 SEP 0235	32	.01	.01	.00	40.	5 SEP 0610	75	.00	.00	.00	30.
5 SEP 0240	33	.01	.01	.00	43.	5 SEP 0615	76	.00	.00	.00	23.
5 SEP 0245	34	.01	.01	.00	45.	5 SEP 0620	77	.00	.00	.00	15.
5 SEP 0250	35	.02	.02	.00	48.	5 SEP 0625	78	.00	.00	.00	10.
5 SEP 0255	36	.02	.02	.00	52.	5 SEP 0630	79	.00	.00	.00	6.
5 SEP 0300	37	.02	.02	.00	58.	5 SEP 0635	80	.00	.00	.00	3.
5 SEP 0305	38	.03	.03	.01	65.	5 SEP 0640	81	.00	.00	.00	2.
5 SEP 0310	39	.03	.03	.01	76.	5 SEP 0645	82	.00	.00	.00	1.
5 SEP 0315	40	.03	.03	.01	89.	5 SEP 0650	83	.00	.00	.00	1.
5 SEP 0320	41	.09	.07	.02	106.	5 SEP 0655	84	.00	.00	.00	0.
5 SEP 0325	42	.09	.07	.02	142.	5 SEP 0700	85	.00	.00	.00	0.
5 SEP 0330	43	.09	.01	.08	225.						

(UPDATED)

****

TOTAL RAINFALL - 3.25, TOTAL LOSS - .87, TOTAL EXCESS - 2.38

PEAK FLOW (CFS)	TIME (HR)		MAXIMUM AVERAGE FLOW 6-HR	24-HR	72-HR	7.00-HR
4552.	4.08	(CFS)	554.	475.	475.	475.
		(INCHES)	2.372	2.375	2.375	2.375
		(AC-FT)	274.	275.	275.	275.

CUMULATIVE AREA = 2.17 SQ MI

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	BASIN2	4552.	4.08	554.	475.	475.	2.17		

*** NORMAL END OF HEC-1 ***

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 1

DETAIL EXAMPLE #8 COMPUTED _____ DATE _____

CLARK UNIT HYDROGRAPH (NATURAL) CHECKED BY _____ DATE _____

SCENARIO: DEVELOP THE CLARK UNIT HYDROGRAPH INPUT PARAMETERS FOR SUBBASIN #4 OF THE EXAMPLE WATERSHED.

STEP 1: ASSEMBLE PHYSICAL BASIN CHARACTERISTICS:

AREA = 0.86 mi^2 = 550.4 ac

FLOW PATH LENGTH (L) = 1.49 mi

SLOPE (S) = 310.0 ft/mi (ADJUSTED USING FIG. 5.4)

% IMPERVIOUSNESS: ASSUME HYDROLOGIC SOIL GROUPS WERE USED TO CALCULATE LOSS PARAMETERS, SO OMIT

STEP 2: CALCULATE THE BASIN RESISTANCE COEFFICIENT K_b USING FIG. 5.5, TABLE 5.1, OR THE "Tc & R WORKSHEET (APPENDIX E). ASSUME THAT THE BASIN IS 50% "HILLSLOPES" AND 50% "MOUNTAIN".

$m = (-.025 + -.030)/2 = -.0275$, $b = (.15 + .20)/2 = .175$

$$K_b = m(\log A) + b = -.0275(\log 550.4) + .175 = 0.100$$

STEP 3: CALCULATE Tc AS A FUNCTION OF i:

NOTE: REFER TO THE WORKSHEET DURING THE REMAINING STEPS, OR USE THE FORTRAN PROGRAM MCUHP1.EXE.

$$T_c = 11.4\, L^{.50} K_b^{.52} S^{-.31} i^{-.38} = 11.4(1.49)^{.50}(.1)^{.52}(310)^{-.31} i^{-.38}, \quad T_c = \cancel{0.599}\ 0.710\ i^{-.38}$$

STEP 4: ENTER RAINFALL, LOSS, AND CLARK PARAMETER DATA INTO AN HEC-1 INPUT DECK WITH Tc & R SET EQUAL TO ZERO. RUN TO GENERATE A RAINFALL - LOSS - EXCESS TABLE.

STEP 5: USING THE WORKSHEET AND THE RESULTS OF STEP 4, COMPUTE THE AVERAGE EXCESS INTESITIES FOR A TIME PERIOD GREATER THAN Tc.

STEP 6: CREATE THE GRAPH OF EXCESS INTENSITY VS. TIME

STEP 7: CALCULATE Tc BY ITERATION. "i" VALUES ARE READ FROM THE GRAPH. CALCULATE R.

STEP 8: ENTER THE Tc & R VALUES INTO THE HEC-1 INPUT DECK. SELECT A TIME-AREA RELATION, IN THIS CASE COLUMN 3 OF TABLE 5.2. RUN THE HEC-1 PROGRAM AGAIN WITH THE NEW CLARK PARAMETERS. (see the following sample HEC-1 input and output).

CALCULATION OF Tc & R

Calculated by: _____ Date: _____
Checked by: _____ Project: _____

Watershed: EXAMPLE #8, SUBBASIN #4
Rainfall Frequency: 100 - yr Duration: 2 - hr. Pattern #: N/A

Rainfall Loss Method:
[ ] Green & Ampt Method
[ ] IL + ULR by soil texture
[X] IL + ULR by hydrologic soil group

Tabulate Period of Peak Rainfall Excess

Clock Time @ end of Increm.	Increm. Excess in.
0100	.20
0105	.72
0110	.37
0115	.31
0120	.09
0125	.06
0130	.05
0135	0

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

Accum. Time hr./min.	Increm. Excess in.	Accum. Excess in.	Avg. Excess Intensity in./hr.
5	.72	.72	8.64
10	.37	1.09	6.54
15	.31	1.40	5.60
20	.20	1.60	4.80
25	.09	1.69	4.06
30	.06	1.75	3.50
35	.05	1.80	3.09
40	0	1.80	2.70

A = 0.86 sq.mi.
L = 1.49 mi.
S = 310. ft/mi.

Kb = m [log(A * 640)]+ b
Kb = (-0.275) log (.86 *640) + (.175)
Kb = .100

$Tc = 11.4\ L^{.50}\ Kb^{.52}\ S^{-.31}\ i^{-.38}$

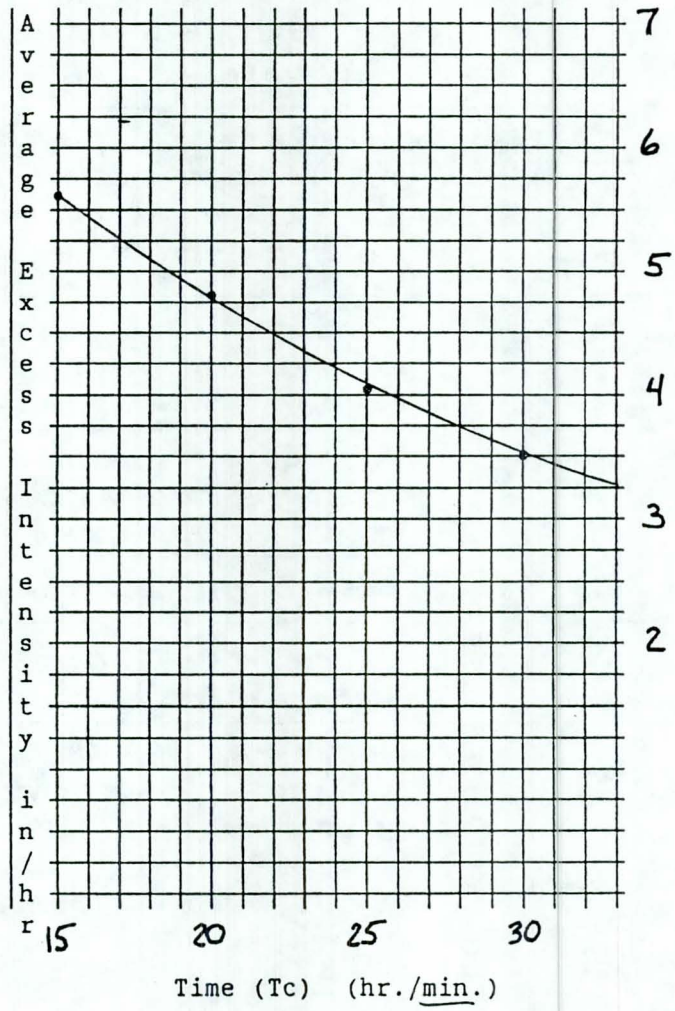
$Tc = (\ .710\ )\ i^{-.38}$

Trial Tc	i	Calc. Tc
30	3.50	26.5
24	4.22	24.6
25	4.06	25.0

Tc = .417 hr.

$R = .37\ Tc^{1.11}\ A^{-.57}\ L^{.80}$

R = .210 hr.



```
1*****************************************
*                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
*            FEBRUARY 1981              *
*          REVISED 05 DEC 88            *
*                                       *
*  RUN DATE  12/07/1989  TIME  11:31:11 *
*                                       *
*****************************************
```

```
****************************
*
*    U.S. ARMY CORPS OF ENGINEE
*  THE HYDROLOGIC ENGINEERING C
*          609 SECOND STREET
*       DAVIS, CALIFORNIA 95616
*          (916) 551-1748
*
****************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X       X     X        XX
X     X  X       X               X
XXXXXXX  XXXX    X       XXXXX   X
X     X  X       X               X
x     x  x       x     x         x
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTUR
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

SAMPLE RUN

EXAMPLE #8

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID       SAMPLE HEC-1 RUN USING TECHNIQUES PRESENTED IN THE
  2           ID       MARICOPA COUNTY HYDROLOGY MANUAL
              * ******************************************************************************
  3           ID       EXAMPLE #8 - CLARK UNIT HYDROGRAPH, UNDEVELOPED SUBBASIN
              * ******************************************************************************
  4           ID       RAINFALL: 2-HR, 100-YEAR. POINT RAINFALL DEPTH OF 2.70 INCHES
  5           ID       HYDROGRAPH: CLARK - Tc & R FROM WORKSHEET, NATURAL TIME-AREA CURVE
  6           ID       LOSSES: IL+ULR BY HYDROLOGIC SOIL GROUPS
  7           ID       SUBBASIN AREA = 0.86 SQUARE MILES
              * ******************************************************************************
  8           IT       5 05SEP89    0000      40
  9           IO       0
              * ******************************

 10           KK  BASIN4
 11           KM       COMPUTE DISCHARGE AT OUTLET OF SUBBASIN 4
 12           IN       5 05SEP89    0000
 13           PB    2.70
 14           PC      0.      1.1      1.8      2.3      2.8      3.2      4.6      7.1      10.     13.7
 15           PC    17.6     23.2     32.7     60.1     74.3     86.3     90.1     93.      95.4     96.2
 16           PC     97.     97.9     98.2     99.2     100.
 17           BA     .86
 18           LU     .67      .20
 19           UA       0        3        5        8       12       20       43       75       90       96
 20           UA     100
 21           UC    .417     .210
 22           ZZ
```

HYDROGRAPH AT STATION BASIN4

DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP
5	SEP	0000	1	.00	.00	.00	0.	5	SEP	0140	21	.02	.02	.00	1232
5	SEP	0005	2	.03	.03	.00	0.	5	SEP	0145	22	.02	.02	.01	917
5	SEP	0010	3	.02	.02	.00	0.	5	SEP	0150	23	.01	.01	.00	658
5	SEP	0015	4	.01	.01	.00	0.	5	SEP	0155	24	.03	.02	.01	457
5	SEP	0020	5	.01	.01	.00	0.	5	SEP	0200	25	.02	.02	.00	317
5	SEP	0025	6	.01	.01	.00	0.	5	SEP	0205	26	.00	.00	.00	223
5	SEP	0030	7	.04	.04	.00	0.	5	SEP	0210	27	.00	.00	.00	161
5	SEP	0035	8	.07	.07	.00	0.	5	SEP	0215	28	.00	.00	.00	118
5	SEP	0040	9	.08	.08	.00	0.	5	SEP	0220	29	.00	.00	.00	83
5	SEP	0045	10	.10	.10	.00	0.	5	SEP	0225	30	.00	.00	.00	55
5	SEP	0050	11	.11	.11	.00	0.	5	SEP	0230	31	.00	.00	.00	30
5	SEP	0055	12	.15	.15	.00	0.	5	SEP	0235	32	.00	.00	.00	17
5	SEP	0100	13	.26	.06	.20	11.	5	SEP	0240	33	.00	.00	.00	9
5	SEP	0105	14	.74	.02	.72	73.	5	SEP	0245	34	.00	.00	.00	5
5	SEP	0110	15	.38	.02	.37	248.	5	SEP	0250	35	.00	.00	.00	3
5	SEP	0115	16	.32	.02	.31	705.	5	SEP	0255	36	.00	.00	.00	2
5	SEP	0120	17	.10	.02	.09	1416.	5	SEP	0300	37	.00	.00	.00	1
5	SEP	0125	18	.08	.02	.06	1883.	5	SEP	0305	38	.00	.00	.00	1
5	SEP	0130	19	.06	.02	.05	1882.	5	SEP	0310	39	.00	.00	.00	0
5	SEP	0135	20	.02	.02	.00	1599.	5	SEP	0315	40	.00	.00	.00	0

TOTAL RAINFALL = 2.70, TOTAL LOSS = .88, TOTAL EXCESS = 1.82

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	3.25-HR
+ (CFS)	(HR)					
		(CFS)				
+ 1883.	1.42		310.	310.	310.	310.
		(INCHES)	1.818	1.818	1.818	1.818
		(AC-FT)	83.	83.	83.	83.

CUMULATIVE AREA = .86 SQ MI

*** NORMAL END OF HEC-1 ***

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 7

DETAIL EXAMPLE #9 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

APPLICATION OF S-GRAPHS

SCENARIO: DEVELOP THE APPROPRIATE UNIT-GRAPH FOR THE ENCLOSED BASIN.

STEP 1: LIST PHYSICAL CHARACTERISTICS:

AREA (A) = 5.19 mi^2

LENGTH OF WATER COURSE (L) = 5.2 mi
(FROM OUTLET TO DRAINAGE BOUNDARY)

LENGTH OF WATER COURSE TO A POINT OPPOSITE TO CENTROID (L_{ca}) = 3.0 mi^2

$$\text{SLOPE } (S) = \frac{\Delta H}{L} = \frac{2900 - 1500}{5.2} = 269 \text{ ft/mi}$$

$$\text{BASIN FACTOR} = \frac{L L_{ca}}{S^{1/2}} = \frac{(5.2)(3.0)}{269^{1/2}} = .95$$

STEP 2: SELECT K_n BY COMPARISON WITH SIMILAR WATERSHEDS. LIST THE DATA FROM SIMILAR WATERSHEDS. FROM LAG RELATIONSHIP IN THE HYDROLOGIC MANUAL.

No.	NAME	A	L	L_{ca}	S	LAG	K_n
3	SANTA ANITA CR.	10.8	5.8	2.5	690	1.1	.05
5	EATON WASH	9.5	7.3	4.4	600	1.3	.05
11	LIVE OAK CR.	2.3	2.9	1.5	700	0.8	.070

NEXT LOOK AT K_n FOR HYDROLOGICALLY SIMILAR WATERSHEDS. AGAIN FROM LAG RELATIONSHIPS:

No.	NAME	K_n
22	NEW RIVER AT ROCK SPRING	.045
23	NEW RIVER AT NEW RIVER	.045
24	NEW RIVER AT BELL ROAD	.037
25	SKUNK CREEK	.033

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 2 OF 7

DETAIL EXAMPLE #9 COMPUTED _____ DATE _____

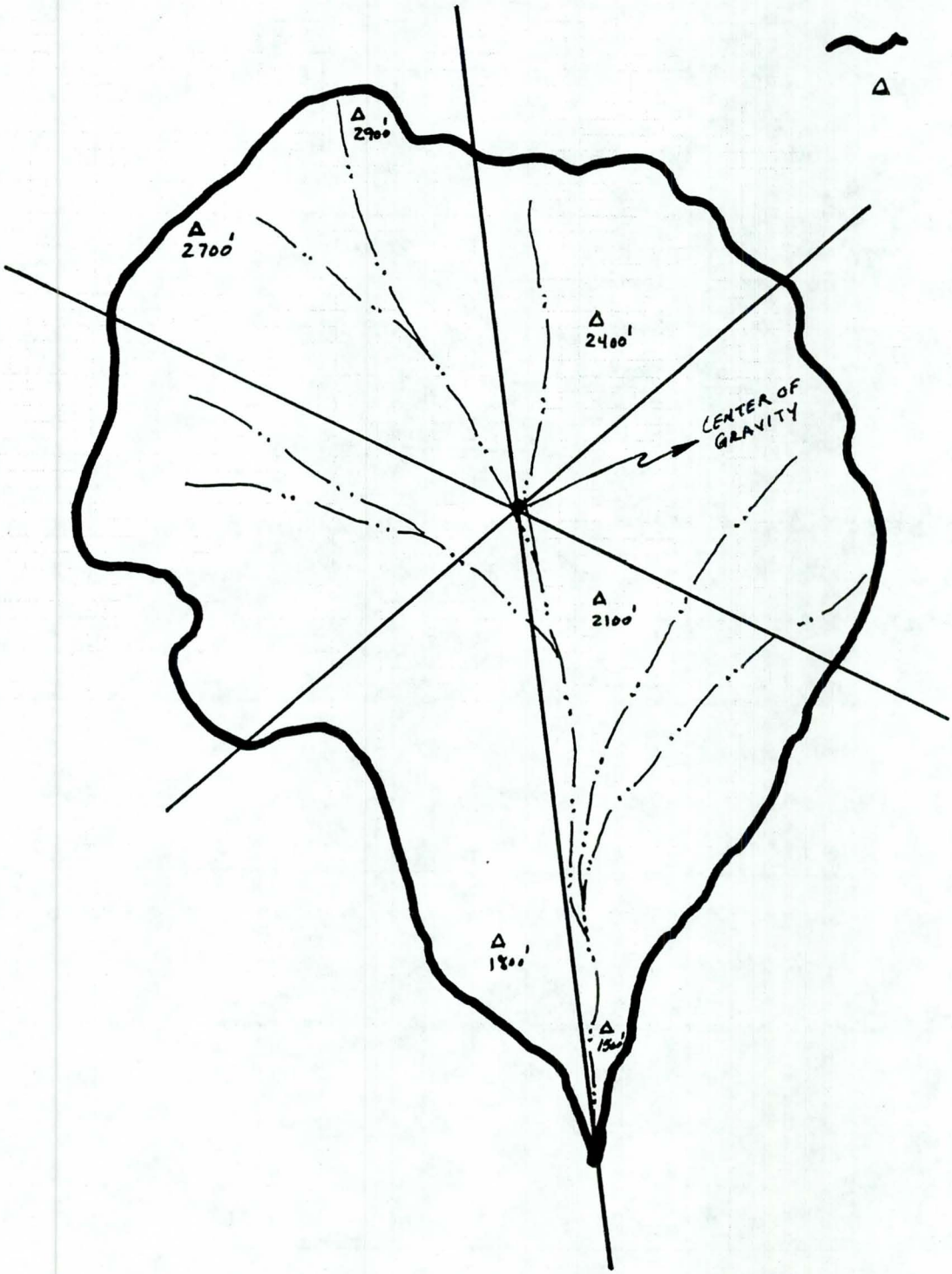
CHECKED BY _____ DATE _____

LEGEND

WATERSHED BOUNDARY

ELEVATION MARK

NTS



THEN, ASSUMING THAT THIS IS A NATURAL WATERSHED IN THE MOUNTAINS AND FOOTHILLS OF MARICOPA COUNTY, Kn IS SELECTED TO BE BETWEEN .040 AND .050.

EVENTHOUGH THE WATERSHED IS SMALL WHICH WILL NOT CONTRIBUTE TO GREAT HYDRAULIC EFFICIENCY, i.e., POORLY DEFINED CHANNELS, STILL THE SLOPE IS QUITE STEEP. AS A RESULT Kn = .040 IS SELECTED.

STEP 3: CALCULATE THE LAG. THE FOLLOWING LAG RELATION BY THE CORPS OF ENGINEERS IS USED:

$$LAG = 20\,Kn\left(L\,L_{ca}/S^{1/2}\right)^{.38} = 20(.04)(.95)^{.38} = 0.78 \text{ HOURS}$$

STEP 4: CALCULATE THE ULTIMATE DISCHARGE, Q_{ULT}:

$$Q_{ULT} = \frac{645.33\,A}{D} = \frac{645.33(5.19)}{(10/60)} = 20096 \text{ CFS}$$

WHERE:

Q_{ULT} = ULTIMATE DISCHARGE (CFS)

A = DRAINAGE AREA (mi^2)

D = DURATION OF EXCESS RAINFALL (HOURS).

D WAS SELECTED TO BE 10 MINUTES. THIS VALUE IS WITHIN THE RANGE OF (.10 → .25) X (LAG TIME) AS SUGGESTED IN THE MANUAL. NOTE THAT D IS ALSO THE HYDROGRAPH TIME STEP, ΔT, USED IN HEC-1 AS THE "IT" RECORD.

STEP 5: AT THIS POINT ALL OF THE NECESSARY PARAMETERS ARE FOUND. A UNIT-GRAPH CAN BE DEVELOPED BY USING THE "MCUHP2" PROGRAM.

ALTERNATIVELY, A UNIT-GRAPH CAN BE BUILT MANUALLY, WHICH IS EXPLAINED NEXT.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL

DETAIL EXAMPLE #9 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

MANUAL CONSTRUCTION OF A 10-MINUTE UNIT HYDROGRAPH FROM THE PHOENIX MOUNTAIN DIMENSIONLSS S-GRAPH

CONSIDER A WATERSHED WITH THE FOLLOWING CHARACTERISTICS:

$A = 5.19 \text{ mi}^2$

$LAG = 0.78$ HOURS

$Q_{ULT} = [645.33(A)]/D = [645.33(5.19)]/.167 = 20,096$ CFS

① CONSTRACT A TABLE SIMILAR TO THE FOLLOWING:
(THE DISCHARGE AND TIME COLUMN FROM THE "DIMENSIONLESS" S-GRAPH)

ORDINATE	% Q_{ULT}	DISCHARGE (CFS)	% LAG	TIME (HOUR)
0	0	0	0.0	.0
1	2	402	23.0	.179
2	4	804	31.0	.242
3	6	1206	37.0	.289
4	8	1608	42.0	.328
5	10	2010	46.0	.359
6	12	2411	49.8	.388
7	14	2813	53.4	.416
8	16	3215	56.8	.443
9	18	3617	60.0	.468
10	20	4019	63.1	.492
11	22	4421	66.1	.516
12	24	4823	69.0	.538
13	26	5225	71.8	.560
14	28	5627	74.4	.580
15	30	6029	76.8	.599
16	32	6431	79.1	.617
17	34	6832	81.2	.634
18	36	7234	83.2	.649
19	38	7636	85.1	.664
20	40	8038	86.8	.677
21	42	8440	88.8	.693
22	44	8842	91.0	.710
23	46	9244	93.8	.732
24	48	9646	96.8	.755
25	50	10048	100.0	.780
26	52	10450	103.4	.806
27	54	10852	107.0	.836
28	56	11254	110.8	.864
29	58	11655	114.7	.895
30	60	12057	118.7	.926
31	62	12459	122.9	.958
32	64	12861	127.3	.993
33	66	13263	131.9	1.029
34	68	13655	136.7	1.067
35	70	14067	141.7	1.106
36	72	14469	147.1	1.148
37	74	14871	152.8	1.192
38	76	15273	158.8	1.239
39	78	15675	165.8	1.294
40	80	16076	172.9	1.348
41	82	16478	181.6	1.417
42	84	16880	191.0	1.490
43	86	17282	201.0	1.568
44	88	17684	212.0	1.654
45	90	18086	226.0	1.763
46	92	18488	244.0	1.903
47	94	18890	265.0	2.067
48	96	19292	295.0	2.301
49	98	19694	342.0	2.668
50	100	20096	462.0	3.604

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 5 OF 7

DETAIL EXAMPLE #9 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

(2) TRANSFORM THE S-GRAPH TO A 10-MINUTE UNIT HYDROGRAPH. USE LINEAR INTERPOLATION IN 10-MIN. INCREMENTS FOR TIME AND DISCHARGE VALUE.

ORDINATE	TIME (HRS)	Q_{S1} (CFS)	Q_{S2} (CFS) ⊗	Q_{UG} (CFS) ▼
1	0	0	0	0
2	.167	375	0	375
3	.333	1673	375	1298
4	.500	4153	1673	2480
5	.667	7729	4153	3576
6	.833	10824	7729	3095
7	1.000	12939	10824	2115
8	1.167	14643	12939	1704
9	1.333	15942	14643	1299
10	1.500	16931	15942	989
11	1.667	17732	16931	801
12	1.833	18287	17732	555
13	2.000	18725	18287	438
14	2.167	19061	18725	336
15	2.333	19327	19061	266
16	2.500	19510	19327	183
17	2.667	19694	19510	184
18	2.833	19765	19694	71
19	3.000	19837	19765	72
20	3.167	19908	19837	71
21	3.333	19971	19908	63
		0	19971	0

⊗ 10-MIN LAG

▼ $Q_{UG} = Q_{S1} - Q_{S2}$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 6 OF 7

DETAIL EXAMPLE #9 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

(2) CONT.... NOTICE THE BEHAVIOR OF THE UNIT GRAPH VALUES FROM TIME = 2.5 HR ON. THIS IS CAUSED BY THE LONGER TIME INCREMENTS AT THE END OF THE S-GRAPH. TO CORRECT THIS, CONSTRUCT A GRAPH OF THE "TAIL" REGION OF THE S-GRAPH, LAG IT BY THE APPROPRIATE DURATION, AND SUBTRACT THE ORDINATES (SEE GRAPH A, NEXT PAGE)

FINAL 10-MIN. UNIT-GRAPH

TIME (HRS)	DISCHARGE (CFS)
0	0
.167	375
.333	1298
.500	2480
.667	3576
.883	3095
1.000	2115
1.167	1704
1.333	1299
1.500	981
1.667	801
1.833	555
2.000	438
2.167	336
2.333	266
2.500	183
2.667	150
2.833	125
3.000	95
3.167	75
3.333	60
3.500	45
3.667	0

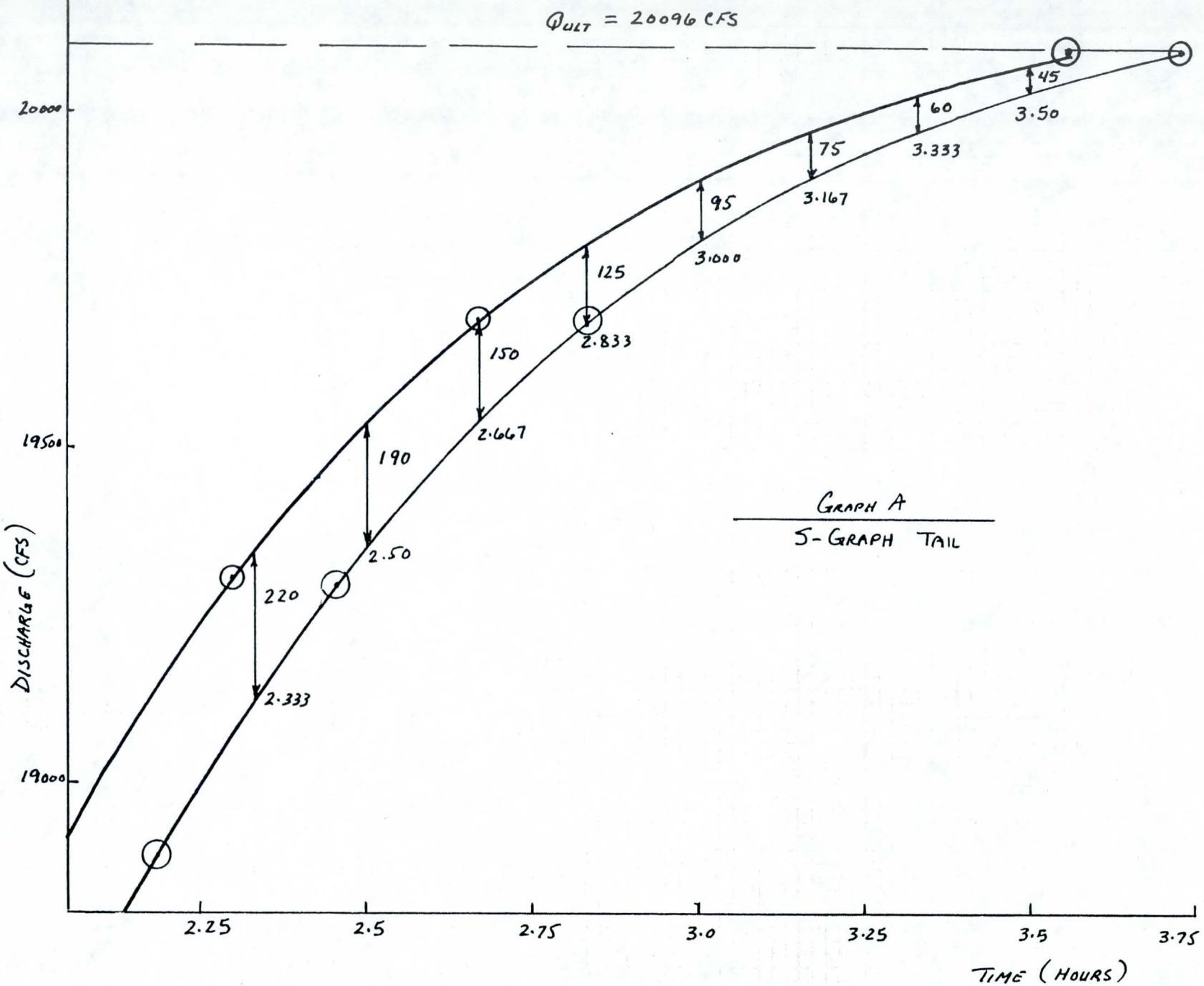
THE USER CAN EITHER RUN PROGRAM "MCUHP2" TO GENERATE THE UNIT-GRAPH, OR ALTERNATIVELY PRODUCE IT MANUALLY, AS WAS SHOWN. IN EITHER CASE THE VALUES CAN BE USED AS THE "UI" RECORD FOR HEC-1 APPLICATION.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 7 OF 7

DETAIL EXAMPLE #9 COMPUTED DATE

CHECKED BY DATE



```
****************************************
*                                      *                                        *
*                                      *                                        *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                        *  U.S. ARMY CORPS OF ENGINEERS
*                                      *                                        *
*          FEBRUARY 1981               *                                        * THE HYDROLOGIC ENGINEERING CENTER
*                                      *                                        *
*          REVISED 05 DEC 88           *                                        *        609 SECOND STREET
*                                      *                                        *
*                                      *                                        *      DAVIS, CALIFORNIA 95616
*                                      *                                        *
*  RUN DATE  01/10/1990  TIME  16:01:06 *                                       *         (916) 551-1748
*                                      *                                        *
*                                      *                                        *
****************************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X               X
XXXXXXX  XXXX     X        XXXXX  X
X     X  X        X               X
X     X  X        X     X         X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID      SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
  2           ID      MARICOPA COUNTY HYDROLOGY MANUAL
              * ******************************************************************************
  3           ID      EXAMPLE # 9 - S-GRAPH APPLICATIONS
              * ******************************************************************************
  4           ID      RAINFALL: 6-HR, 100-YEAR POINT RAINFALL DEPTH OF 3.40 INCHES
  5           ID      UNIT-GRAPH: DERIVED FROM PHOENIX MOUNTAIN S-GRAPH
  6           ID      LOSSES: IL + ULR BY SOIL TEXTURE                    OK
  7           ID      BASIN AREA: 5.19 SQUARE MILES, PATTERN #2.35, AREAL REDUCTION 0.965
              * ******************************************************************************
  8           IT      10 05SEP89    0000      50
  9           IO

 10           KK   BASIN
 11           KM   COMPUTE DISCHARGE AT BASIN OUTLET
 12           IN      15
 13           PB    3.28
 14           PC   0.000   0.009   0.015   0.024   0.037   0.047   0.058   0.069   0.080   0.091
 15           PC   0.104   0.118   0.139   0.184   0.270   0.458   0.686   0.823   0.889   0.929
 16           PC   0.960   0.981   0.986   0.989   1.000
 17           BA    5.19
 18           LU    0.75    0.25    3.00
 19           UI    373.   1308.   2472.   3570.   3110.   2107.   1703.   1327.    963.    799.
 20           UI    556.    438.    336.    266.    183.    183.     72.     72.     72.     72.
 21           UI     72.      0.
 22           ZZ
```

HYDROGRAPH AT STATION BASIN

DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
5 SEP 0000	1	.00	.00	.00	0.	5 SEP 0410	26	.30	.04	.26	3399.
5 SEP 0010	2	.02	.02	.00	0.	5 SEP 0420	27	.22	.04	.18	4414.
5 SEP 0020	3	.02	.02	.00	1.	5 SEP 0430	28	.14	.04	.10	4818.
5 SEP 0030	4	.01	.01	.00	2.	5 SEP 0440	29	.09	.04	.05	4609.
5 SEP 0040	5	.02	.02	.00	4.	5 SEP 0450	30	.08	.04	.04	4060.
5 SEP 0050	6	.02	.02	.00	6.	5 SEP 0500	31	.07	.04	.03	3435.
5 SEP 0100	7	.03	.03	.00	7.	5 SEP 0510	32	.05	.04	.01	2812.
5 SEP 0110	8	.02	.02	.00	9.	5 SEP 0520	33	.03	.03	.00	2255.
5 SEP 0120	9	.02	.02	.00	10.	5 SEP 0530	34	.01	.01	.00	1792.
5 SEP 0130	10	.02	.02	.00	11.	5 SEP 0540	35	.01	.01	.00	1377.
5 SEP 0140	11	.02	.02	.00	12.	5 SEP 0550	36	.02	.01	.00	1054.
5 SEP 0150	12	.02	.02	.00	12.	5 SEP 0600	37	.02	.02	.00	808.
5 SEP 0200	13	.02	.02	.00	13.	5 SEP 0610	38	.00	.00	.00	623.
5 SEP 0210	14	.02	.02	.00	13.	5 SEP 0620	39	.00	.00	.00	466.
5 SEP 0220	15	.03	.03	.00	14.	5 SEP 0630	40	.00	.00	.00	354.
5 SEP 0230	16	.03	.03	.00	14.	5 SEP 0640	41	.00	.00	.00	262.
5 SEP 0240	17	.03	.03	.00	14.	5 SEP 0650	42	.00	.00	.00	212.
5 SEP 0250	18	.04	.04	.00	15.	5 SEP 0700	43	.00	.00	.00	173.
5 SEP 0300	19	.05	.04	.00	16.	5 SEP 0710	44	.00	.00	.00	129.
5 SEP 0310	20	.10	.10	.00	18.	5 SEP 0720	45	.00	.00	.00	90.
5 SEP 0320	21	.14	.14	.00	23.	5 SEP 0730	46	.00	.00	.00	53.
5 SEP 0330	22	.19	.08	.11	69.	5 SEP 0740	47	.00	.00	.00	31.
5 SEP 0340	23	.41	.04	.37	313.	5 SEP 0750	48	.00	.00	.00	17.
5 SEP 0350	24	.45	.04	.41	940.	5 SEP 0800	49	.00	.00	.00	9.
5 SEP 0400	25	.50	.04	.46	2042.	5 SEP 0810	50	.00	.00	.00	5.

TOTAL RAINFALL = 3.28, TOTAL LOSS = 1.24, TOTAL EXCESS = 2.04

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	8.17-HR
+ (CFS)	(HR)					
		(CFS)				
+ 4818.	4.50		1132.	833.	833.	833.
		(INCHES)	2.028	2.032	2.032	2.032
		(AC-FT)	561.	562.	562.	562.

CUMULATIVE AREA = 5.19 SQ MI

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	BASIN	4818.	4.50	1132.	833.	833.	5.19		

*** NORMAL END OF HEC-1 ***

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

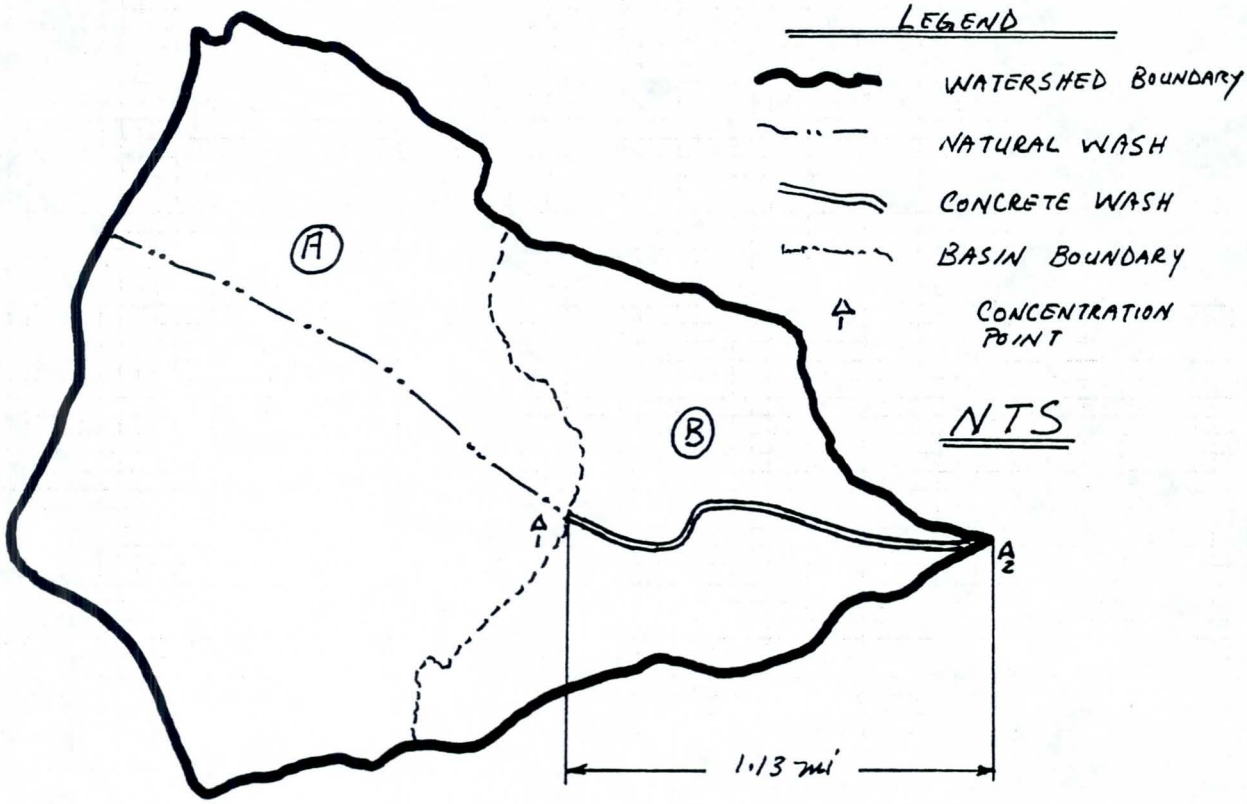
PROJECT HYDROLOGIC DESIGN MANUAL PAGE 1 OF 2

DETAIL EXAMPLE # 10 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

Updated 7/12/90

KINEMATIC WAVE ROUTING



SCENARIO: PREVIOUSLY IN EXAMPLE #7, PEAK FLOW AT CONCENTRATION POINT (Δ_1) WAS CALCULATED FOR SUB-BASIN (A). THIS EXAMPLE WILL USE KINEMATIC WAVE ROUTING THE 1.13 MILE LONG REACH TO CALCULATE THE PEAK AT CONCENTRATION POINT (Δ_2).

STEP 1: COLLECT NECESSARY DATA FOR THE CHANNEL:

TYPE: CONCRETE LINE, TRAPOZOIDAL

LENGTH: 1.13 MILE

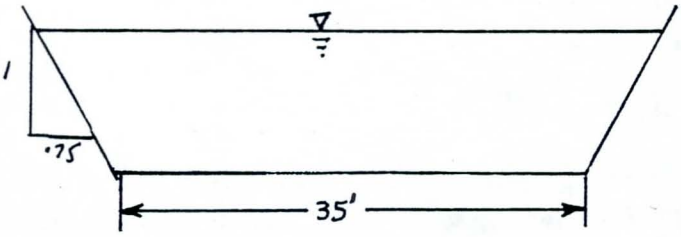
AVE. DEPTH: 4 FEET

SIDE SLOPE: 0.75 : 1.00

MANNING'S n: .015

BOTTOM WIDTH: 35 FEET

CHANNEL SLOPE: .018 ft/ft



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HYDROLOGIC DESIGN MANUAL PAGE 2 OF 2

DETAIL EXAMPLE # 10 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

(*) SINCE KINEMATIC WAVE ROUTING DOES NOT CHECK FOR CHANNEL CAPACITY, DESIGN PARAMETERS MUST BE CHECKED TO ASSURE PROPER CONVEYANCE THROUGH THE REACH. OTHERWISE, THE MODEL WILL AUTOMATICALLY EXTEND THE CHANNEL BOUNDARIES TO CONTAIN THE FLOW.

(*) THE MORE RECENT VERSIONS OF HEC-1 (1988 AND BEYOND) ACCOUNT FOR THE PROPER SELECTION OF COMPUTATIONAL TIME STEP TO ASSURE THAT THE ROUTED PEAK IS NOT UNREALISTICALLY ATTENUATED.

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*             FEBRUARY 1981            *
            REVISED 31 JAN 85          *
*                                      *
*  RUN DATE 9/ 8/1989   TIME11:18: 8 *
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*  THE HYDROLOGIC ENGINEERING CENTER   *
*          609 SECOND STREET           *
*        DAVIS, CALIFORNIA 95616       *
*  (916) 440-3285 OR (FTS) 448-3285    *
*                                      *
****************************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X       X     X        XX
X     X  X       X              X
XXXXXXX  XXXX    X       XXXXX  X
X     X  X       X              X
X     X  X       X     X        X
X     X  XXXXXXX   XXXXX        XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE. THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85 CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT DESCRIPTION FOR NEW DEFINITIONS.

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

*** FREE ***
  1           ID      SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
  2           ID      MARICOPA COUNTY HYDROLOGY MANUAL
              * ***************************************************************
  3           ID      EXAMPLE # 10 - KINEMATIC WAVE ROUTING
              * ***************************************************************
  4           IT       5 05SEP89    0000      75
  5           IO       3
              * ***************************************************************

  6           KK  BASIN2
  7           KM   COMPUTE DISCHARGE AT THE OUTLET OF BASIN #2
  8           IN      15
  9           PB    3.25
 10           PC     0.0    .550    1.05    1.70    2.65    3.45    4.35    5.20    6.05    6.90
 11           PC     8.1    9.40   11.35    14.5   22.85   40.85   75.85   86.85    91.0   93.85
 12           PC   95.95    97.5   98.35    98.9    100.
 13           BA    2.17
 14           LU     .65     .20     21.
 15           UA       0       5      16      30      65      77      84      90      94      97
 16           UA     100
 17           UC    .440    .156
              * ***************************************************************

 18           KK   ROUTE
 19           KM   ROUTE THROUGH DOWNSTREAM BASIN USING KINEMATIC ROUTING
 20           KO       1       2
 21           RK  5966.4    .018   0.015            TRAP     35.    0.75
 22           ZZ
```

```
**************
*            *
18 KK    *     ROUTE  *
*            *
**************
                ROUTE THROUGH THE DOWNSTREAM BASIN USING KINEMATIC WAVE ROUTING

20 KO        OUTPUT CONTROL VARIABLES
                 IPRNT          1  PRINT CONTROL
                 IPLOT          2  PLOT CONTROL
                 QSCAL         0.  HYDROGRAPH PLOT SCALE

           HYDROGRAPH ROUTING DATA

21 RK        KINEMATIC WAVE STREAM ROUTING
                   L      5966.  CHANNEL LENGTH
                   S      .0180  SLOPE
                   N       .015  CHANNEL ROUGHNESS COEFFICIENT
                  CA        .00  CONTRIBUTING AREA
               SHAPE       TRAP  CHANNEL SHAPE
                  WD      35.00  BOTTOM WIDTH OR DIAMETER
                   Z        .75  SIDE SLOPE
               DXMIN          2  MINIMUM NUMBER OF DX INTERVALS

                                                  ***
                          COMPUTED KINEMATIC PARAMETERS
                               VARIABLE TIME STEP
                            (DT SHOWN IS A MINIMUM)
      ELEMENT    ALPHA       M        DT        DX       PEAK    TIME TO     VOLUME    MAXIMUM
                                                                    PEAK               CELERITY
                                      (MIN)      (FT)     (CFS)    (MIN)       (IN)      (FPS)

          3       1.51      1.59       .35    994.40   4548.78   247.02       2.36      47.18

CONTINUITY SUMMARY (AC-FT) - INFLOW=  274.296  EXCESS=     .000 OUTFLOW=  273.483 BASIN STORAGE=     .762 PERCENT ERROR=     .019

                          INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

          3       1.51      1.59      5.00            4516.93   250.00       2.36
```

HYDROGRAPH AT STATION ROUTE

DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW
5 SEP 0000	1	0.	5 SEP 0135	20	32.	5 SEP 0310	39	60.	5 SEP 0445	58	898.
5 SEP 0005	2	0.	5 SEP 0140	21	32.	5 SEP 0315	40	69.	5 SEP 0450	59	679.
5 SEP 0010	3	0.	5 SEP 0145	22	33.	5 SEP 0320	41	81.	5 SEP 0455	60	524.
5 SEP 0015	4	0.	5 SEP 0150	23	33.	5 SEP 0325	42	97.	5 SEP 0500	61	407.
5 SEP 0020	5	0.	5 SEP 0155	24	33.	5 SEP 0330	43	129.	5 SEP 0505	62	318.
5 SEP 0025	6	0.	5 SEP 0200	25	33.	5 SEP 0335	44	221.	5 SEP 0510	63	253.
5 SEP 0030	7	0.	5 SEP 0205	26	33.	5 SEP 0340	45	569.	5 SEP 0515	64	204.
5 SEP 0035	8	2.	5 SEP 0210	27	33.	5 SEP 0345	46	1208.	5 SEP 0520	65	161.
5 SEP 0040	9	12.	5 SEP 0215	28	33.	5 SEP 0350	47	1873.	5 SEP 0525	66	127.
5 SEP 0045	10	17.	5 SEP 0220	29	32.	5 SEP 0355	48	2631.	5 SEP 0530	67	101.
5 SEP 0050	11	19.	5 SEP 0225	30	32.	5 SEP 0400	49	3553.	5 SEP 0535	68	81.
5 SEP 0055	12	20.	5 SEP 0230	31	33.	5 SEP 0405	50	4323.	5 SEP 0540	69	65.
5 SEP 0100	13	22.	5 SEP 0235	32	34.	5 SEP 0410	51	4517.	5 SEP 0545	70	53.
5 SEP 0105	14	24.	5 SEP 0240	33	37.	5 SEP 0415	52	4119.	5 SEP 0550	71	43.
5 SEP 0110	15	27.	5 SEP 0245	34	40.	5 SEP 0420	53	3477.	5 SEP 0555	72	36.
5 SEP 0115	16	30.	5 SEP 0250	35	43.	5 SEP 0425	54	2799.	5 SEP 0600	73	31.
5 SEP 0120	17	32.	5 SEP 0255	36	45.	5 SEP 0430	55	2151.	5 SEP 0605	74	30.
5 SEP 0125	18	32.	5 SEP 0300	37	48.	5 SEP 0435	56	1608.	5 SEP 0610	75	31.
5 SEP 0130	19	32.	5 SEP 0305	38	53.	5 SEP 0440	57	1200.			

PEAK FLOW (CFS)	TIME (HR)		MAXIMUM AVERAGE FLOW 6-HR	24-HR	72-HR	6.17-HR
		(CFS)				
+ 4517.	4.17		551.	536.	536.	536.
		(INCHES)	2.361	2.361	2.361	2.361
		(AC-FT)	273.	273.	273.	273.

CUMULATIVE AREA = 2.17 SQ MI

1 STATION ROUTE

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+									
HYDROGRAPH AT									
+	B-2	4552.	4.08	553.	538.	538.	2.17		
ROUTED TO									
+	ROUTE	4517.	4.17	551.	536.	536.	2.17		
1									

SUMMARY OF KINEMATIC WAVE ROUTING
(FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)

ISTAQ	ELEMENT	DT	PEAK	TIME TO PEAK	VOLUME	INTERPOLATED TO COMPUTATION INTERVAL DT	PEAK	TIME TO PEAK	VOLUME
		(MIN)	(CFS)	(MIN)	(IN)	(MIN)	(CFS)	(MIN)	(IN)
ROUTE	3	.35	4548.78	247.02	2.36	5.00	4516.93	250.00	2.36

CONTINUITY SUMMARY (AC-FT) - INFLOW= 274.296 EXCESS= .000 OUTFLOW= 273.483 BASIN STORAGE= .762 PERCENT ERROR= .019

*** NORMAL END OF HEC-1 ***

STATION ROUTE

(I) INFLOW, (O) OUTFLOW

DAHRMN	PER	0.	1000.	2000.	3000.	4000.	5000.	0.	0.	0.	0.	0.	0.	0.
)00	1I													
.005	2I													
50010	3I													
50015	4I													
50020	5I													
50025	6I													
50030	7I													
50035	8I													
50040	9I													
50045	10I													
50050	11I													
50055	12I													
50100	13I													
50105	14I													
50110	15I													
50115	16I													
50120	17I													
50125	18I													
50130	19I													
50135	20I													
50140	21I													
50145	22I													
50150	23I													
50155	24I													
50200	25I													
50205	26I													
50210	27I													
50215	28I													
50220	29I													
50225	30I													
50230	31I													
50235	32I													
50240	33I													
50245	34I													
)250	35I													
,0255	360I													
50300	370I													
50305	38.I													
50310	39.I													
50315	40.I													
50320	41.I													
50325	42.I													
50330	43.OI													
50335	44. O I													
50340	45. O I													
50345	46. O I													
50350	47. O. I													
50355	48. O I													
50400	49. O I.													
50405	50. O I													
50410	51. .I.													
50415	52. IO													
50420	53. I O													
50425	54. I O													
50430	55. I .O													
50435	56. IO													
50440	57. I O													
50445	58. I O.													
50450	59. IO													
50455	60. IO													
50500	61. .IO													
50505	62. IO													
50510	63. I													
50515	64. I													
50520	65.IO													
50525	66.I													
50530	67.I													
50535	68.I													
50540	69IO													
,0545	70IO													
50550	71I													
50555	72I													
50600	73I													
50605	74I													
50610	75I													

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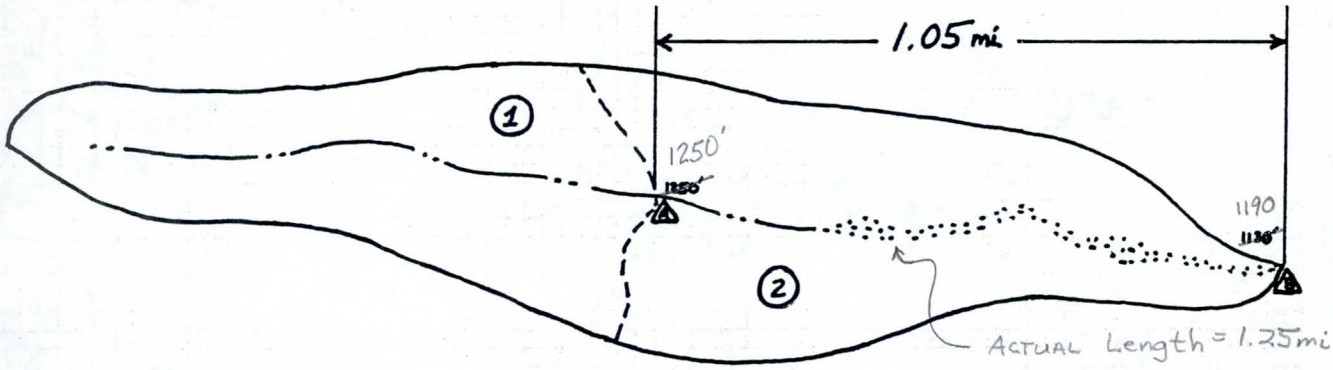
DETAIL EXAMPLE # 11 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

REVISED ~~5/9/90~~ 7/12/90

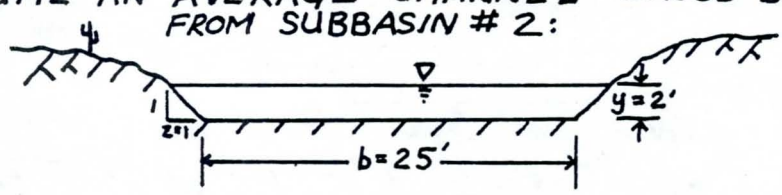
MUSKINGUM ROUTING



SCENARIO: DEVELOP MUSKINGUM PARAMETERS FOR SUBBASIN #2, ROUTE A FLOOD HYDROGRAPH FROM SUBBASIN #1 THROUGH SUBBASIN #2 FROM CONCENTRATION POINT A TO B.

STEP 1: DEVELOP MUSKINGUM PARAMETERS:

ASSUME AN AVERAGE CHANNEL CROSS-SECTION FROM SUBBASIN # 2:



A. CALCULATE VELOCITY:

$A = (b + zy)\,y = (25 + (1)(2))\,2 = 54\ ft^2$

$P = b + 2y(1 + z^2)^{1/2} = 25 + (2)(2)(1 + (1)^2)^{1/2} = 30.66\ ft$

$R = A/P = 54\ ft^2 / 30.66\ ft = 1.761\ ft$

$S = \cancel{.0170}\ .0091\ ft/ft$

$n = .040$

$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{.040}(1.761)^{2/3}(\cancel{.0170}\ .0091)^{1/2} = \cancel{7.08}\ 5.18\ ft/s$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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B. ESTIMATE WAVE CELERITY (VM) $\longrightarrow \frac{V_m}{V} = 1.67$; $V_m = 1.67(\cancel{7.08}\ 5.18) = \cancel{11.82}\ 8.65$ ft/s

C. CALCULATE K :

$$K = 1.25\,mi \times 5280\,\frac{ft}{mi} \times \frac{1}{\cancel{7.08}\ 8.65}\,\frac{s}{ft} \times \frac{1}{3600}\,\frac{s}{hr} = \cancel{0.259}\ .212\ hr$$

D. ESTIMATE X :

SINCE THE CHANNEL IS SHALLOW WITH MODERATE TO LOW SLOPE, CHOOSE X = 0.20

E. CHECK NSTPS :

NSTPS MUST BE WITHIN THE FOLLOWING LIMITS : $\frac{1}{2(1-X)} \le \frac{(AMSKK \times 60)}{(NMIN \times NSTPS)} \le \frac{1}{2X}$

TRY NSTPS = 1 : $\frac{1}{2(1-.2)} \le \frac{\cancel{.259}\ .212(60)}{(5)(1)} \le \frac{1}{2(.2)}$

$.625 \le \cancel{3.11}\ 2.54 \le 2.5 \longrightarrow$ No Good !

TRY NSTPS = 2 : $\frac{\cancel{.259}\ .212(60)}{5(2)} = \cancel{1.55}\ 1.27$ OK NSTPS = 2

STEP 2 : ENTER MUSKINGUM PARAMETERS INTO AN HEC-1 INPUT FILE ON THE RM CARD. HAND CALCULATION TECHNIQUES CAN BE FOUND IN MOST HYDROLOGY TEXTBOOKS.