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MARICOPA COUNTY HYDROLOGY MANUAL

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INTRODUCTION

1.1 PURPOSE

In April 1985 a task force was formed to establish a common basis for drainage management in all jurisdictions within Maricopa County. This was deemed to be desirable because it would result in consistent analysis of drainage requirements, less staff time and cost in annexing County areas, and equal and common protection from the hazards of stormwater drainage for the residents. Additionally, developers would have the advantage of having only one set of drainage standards to comply with developing land within the incorporated or unincorporated areas of Maricopa County. The task force determined that the effort should be 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 part of fulfilling Phase 2 the Design Hydrology Manual will provide the necessary inputs for the Stormwater Drainage Design Manual.

1.2 SCOPE AND LIMITATIONS

When using the procedures detailed in this manual, it is important to keep several things in mind. First, this is a DESIGN HYDROLOGY MANUAL. The methods, techniques and parameter values described herein are not necessarily valid for real-time prediction of flow values, or for re-creating 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 parts 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 predicting the runoff which would 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. This 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 USES OF 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 respect, 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 user of this manual is thus encouraged to validate the reasonableness of the predicted values by applying alternative methods, such as envelope curves, regression equations, or other such "checks" which have been developed for this area.

The last Chapter in the Manual, APPLICATION, is intended to provide some general suggestions when solving a particular problem. In addition, a number of examples were designed to aid the user with the development of input variables and parameter estimation.

It is not the intent of this manual to inhibit sound innovative design or the utilization 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-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 another agency which subscribes to these procedures.

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 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 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 pattern could last over several days with slight breaks in between 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 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 usually cover the entire 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 300 square miles for a time period up to about 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.

Spatial and temporal variation of precipitation, and lack of long term data in Maricopa County requires a procedure for rainfall input for design purposes. 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 event 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. This data was published by National Weather Service (NWS), National Oceanic and Atmospheric Administration (NOAA), (Miller, et al., 1973). Until a more up to date data base becomes available, NOAA Atlas is to be used for all 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 raingauges. 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 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 such as storm drains, channels, and culverts should be sized to handle the peak discharge resulting from the design storm critically centered above them so as 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. The Depth-Area Reduction Curves developed by Osborn, et al., (1980) are to be used. These curves were based on data from Arizona and are appropriate for use in Maricopa County. Figures 1 to 4 illustrate the curves, which are for 2-, 10-, and 100-year frequencies. If areal reduction is needed for a 25-, or 50- year frequencies, the values for the 10-year and the 100-year frequencies can be used, respectively.

- 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 Figures 2.1 to 2.4 to select the reduction coefficient. For

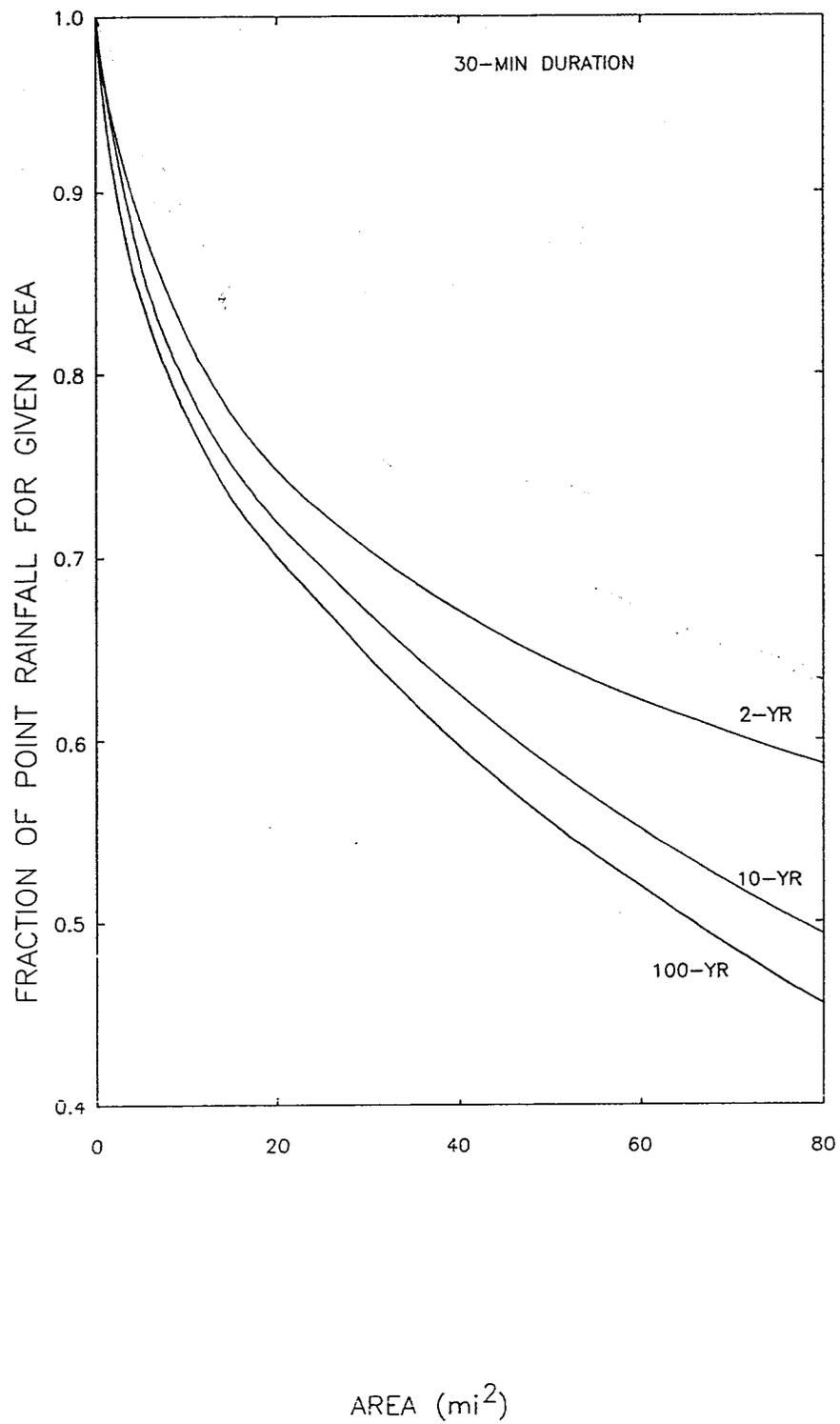


FIGURE 2.1. Point-to-area conversion ratios for 30-min duration rainfalls for selected frequencies.

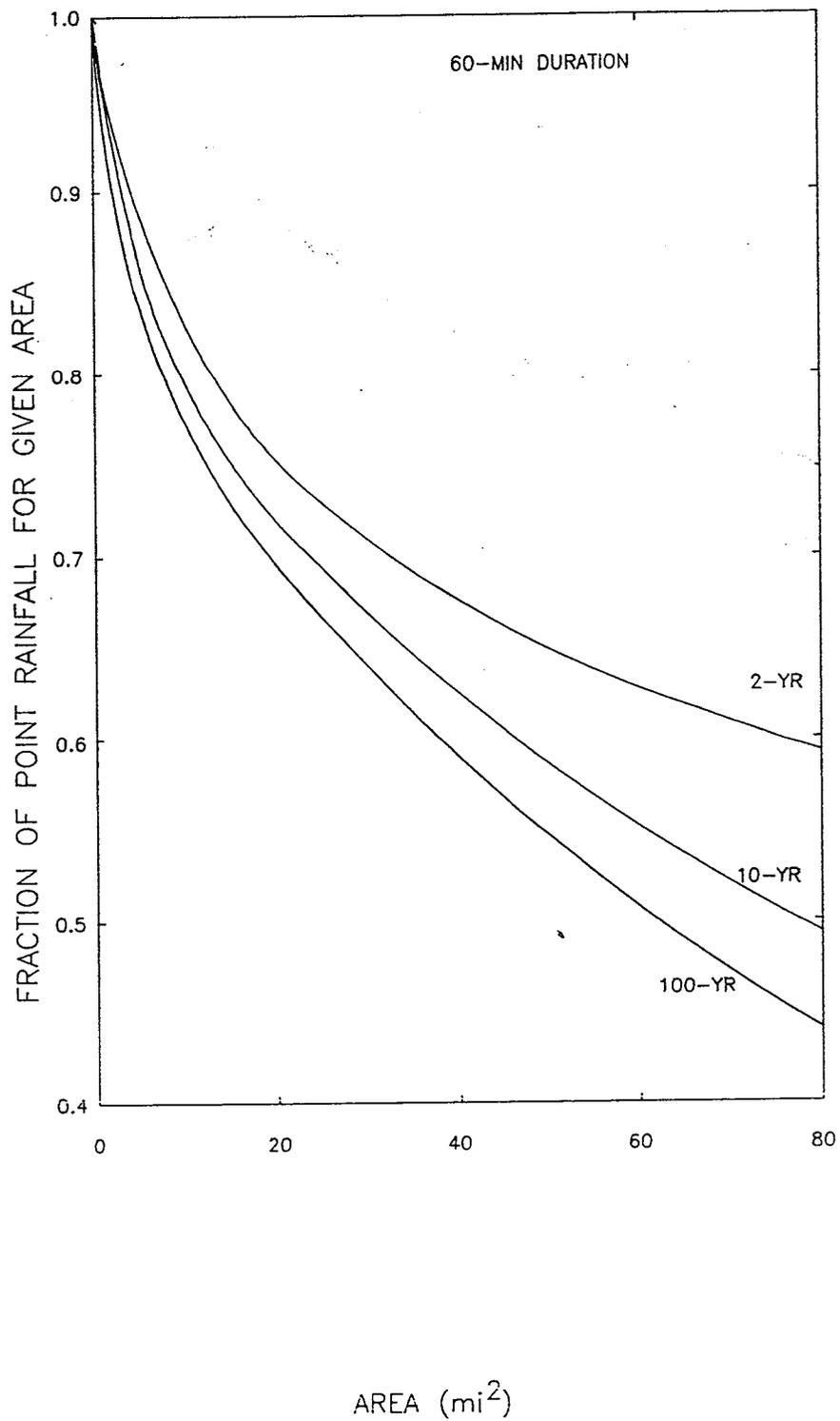


FIGURE 2.2. Point-to-area conversion ratios for 60-min duration rainfalls for selected frequencies.

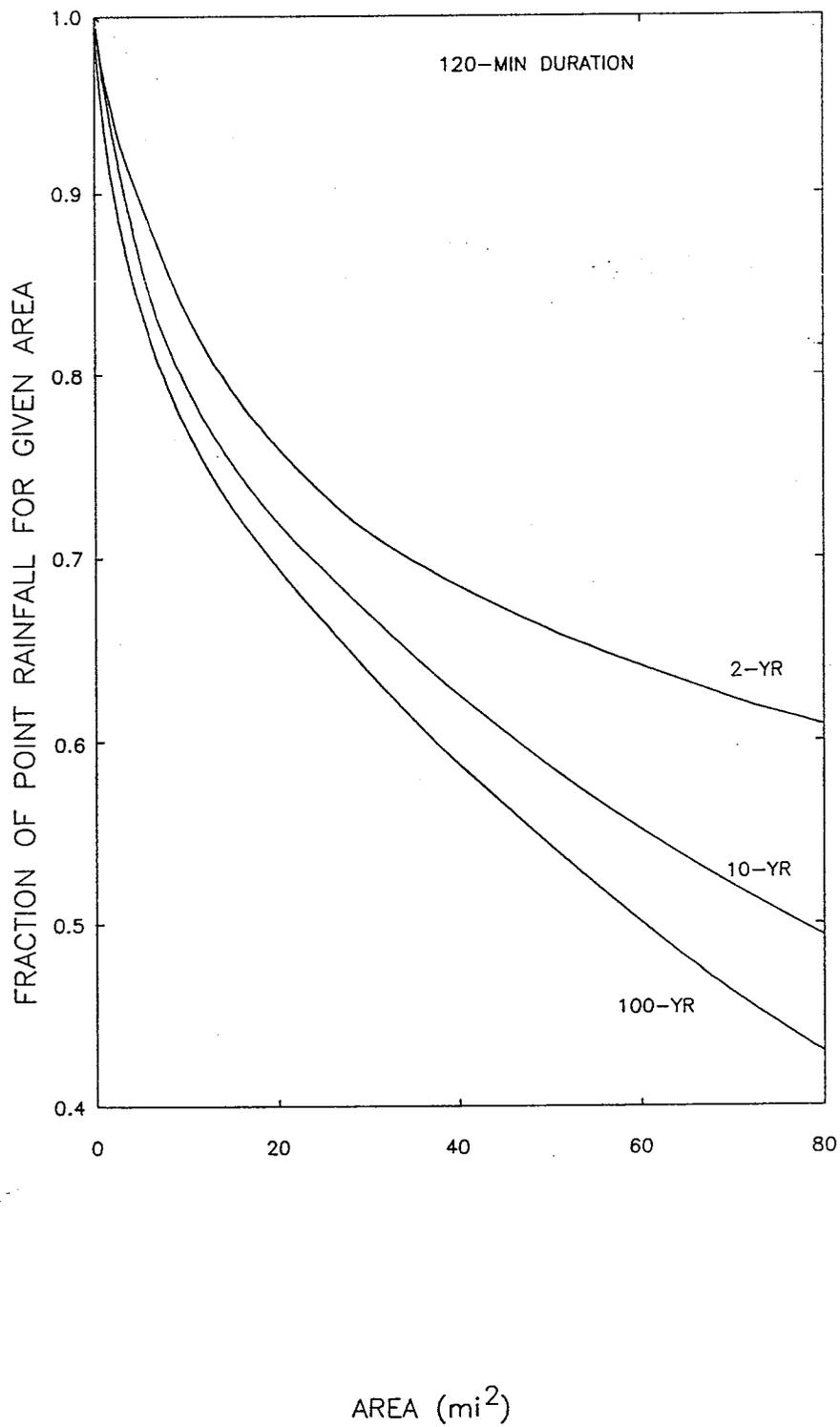


FIGURE 23. Point-to-area conversion ratios for 2-h duration rainfalls for selected frequencies.

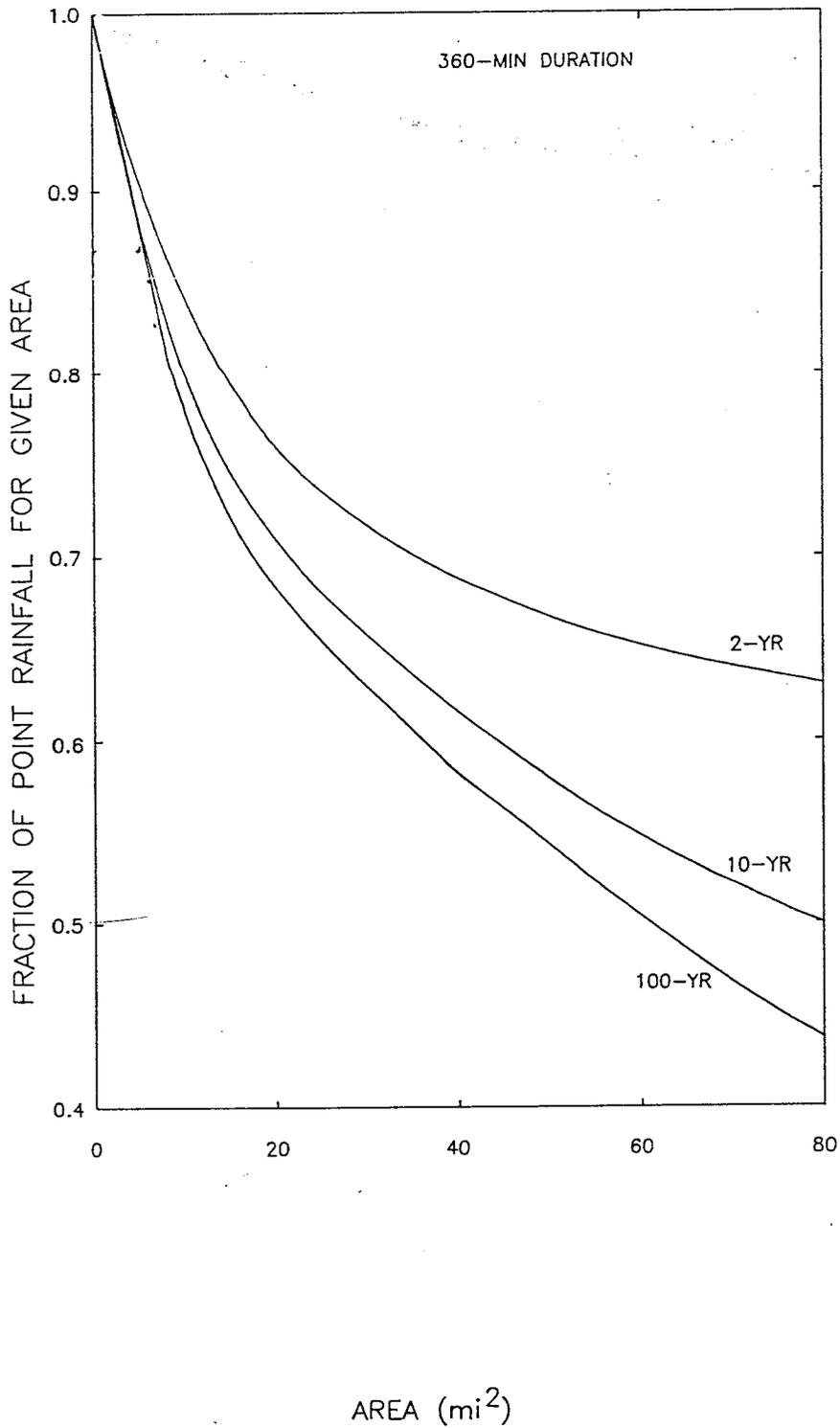


FIGURE 34. Point-to-area conversion ratios for 6-h duration rainfalls for selected frequencies.

large areas (>80 square miles) or durations longer than 6 hours, use Figure 2.4 at the end point values.

e. Multiply reduction coefficient by the average rainfall depth.

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 does not require a time distribution. The Hydrology Manual applied the NOAA procedures which led to the 100-year, 6-hour mass curve for small areas up to 0.5 miles². This mass curve is also known as pattern # 1, and will be discussed later. If a particular application requires that a mass curve should be developed, the following procedures (NOAA) or, alternatively, a program referred to as PREFRE by the National Weather Service can be used:

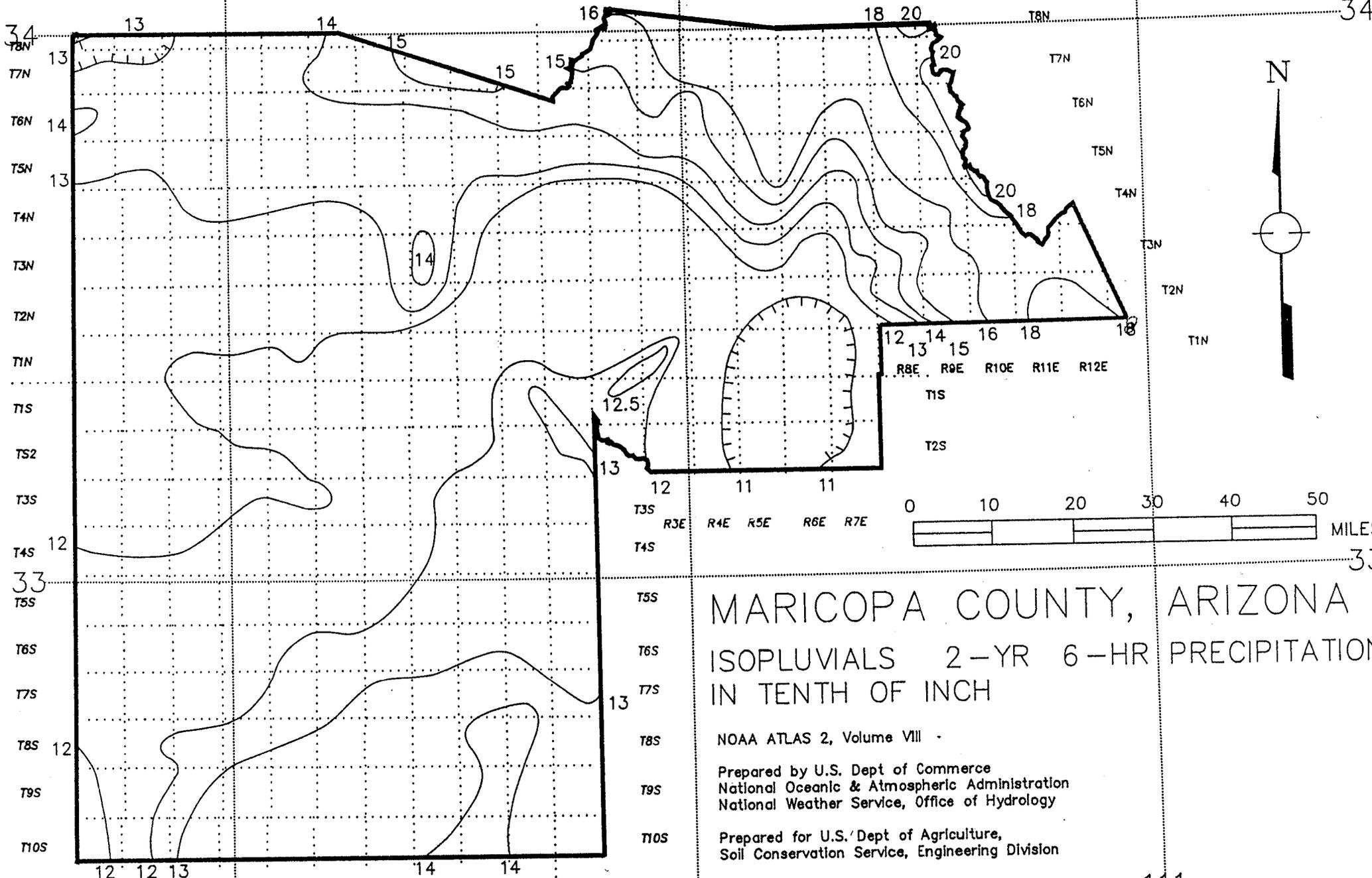
- 1) Using Plates 1-12, read rainfall depths for 2-, 5-, 10-, 25-, 50, and 100-year return periods, for 6- and 24-hour durations, employing linear interpolation between isolines when required. The numbers on the isolines show tenths of inches of rainfall (i.e, 23=2.3 inches).
- 2) Plot the values from 1 for each duration on a separate line on Figure 2.5, look for any deviation from a straight line and make corrections on the line. 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.

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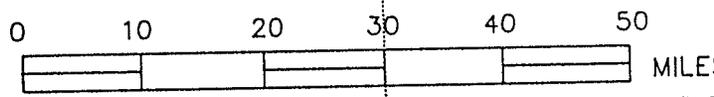
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MARICOPA COUNTY, ARIZONA

ISOPLUVIALS 2-YR 6-HR PRECIPITATION IN TENTH OF INCH

NOAA ATLAS 2, Volume VIII
Prepared by U.S. Dept of Commerce
National Oceanic & Atmospheric Administration
National Weather Service, Office of Hydrology

Prepared for U.S. Dept of Agriculture,
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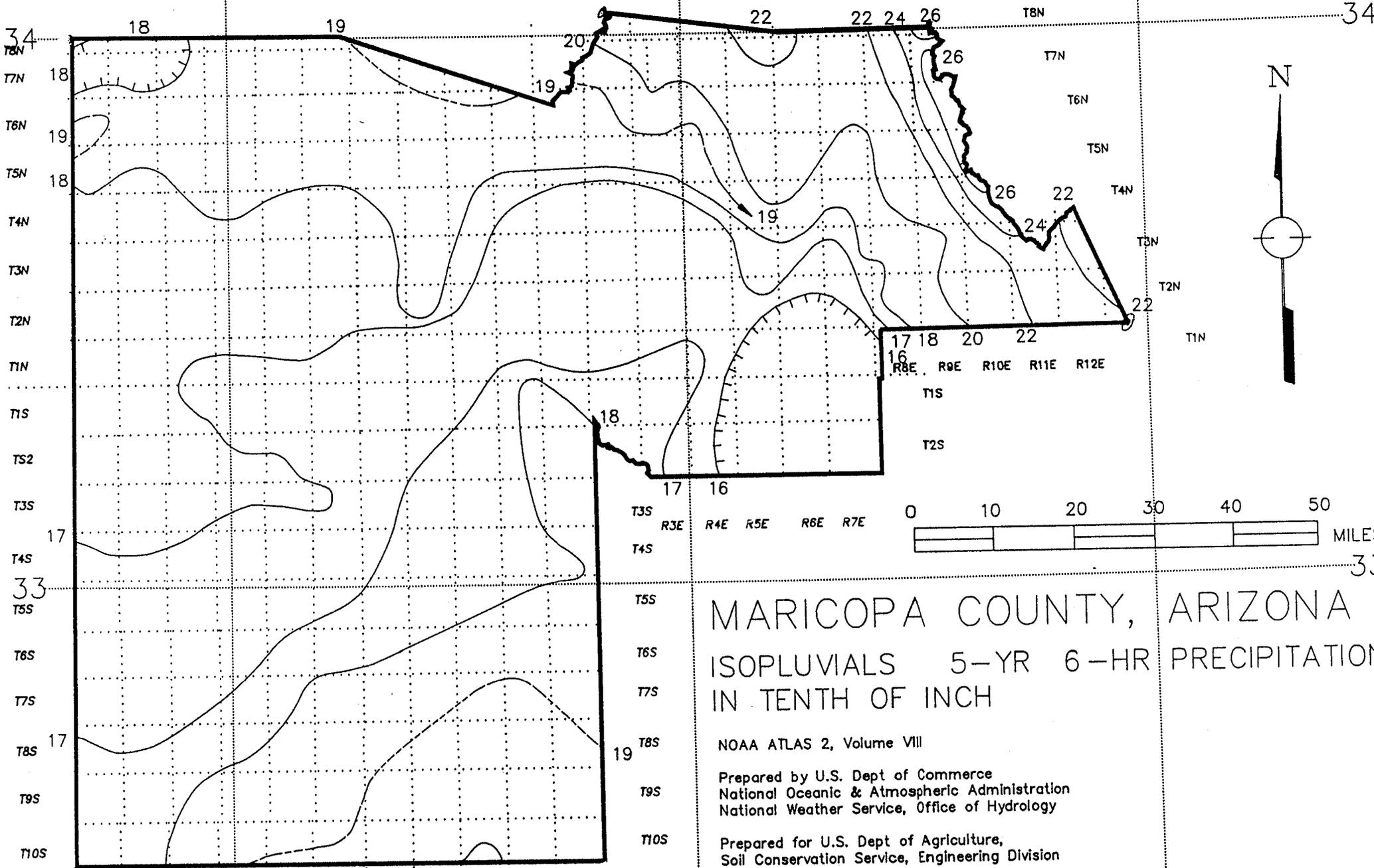
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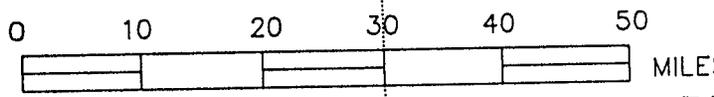


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MARICOPA COUNTY, ARIZONA
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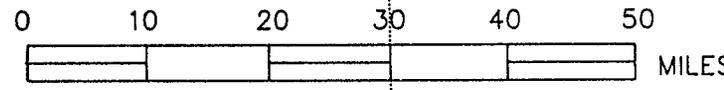
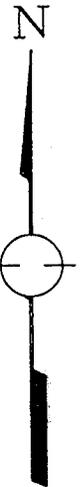
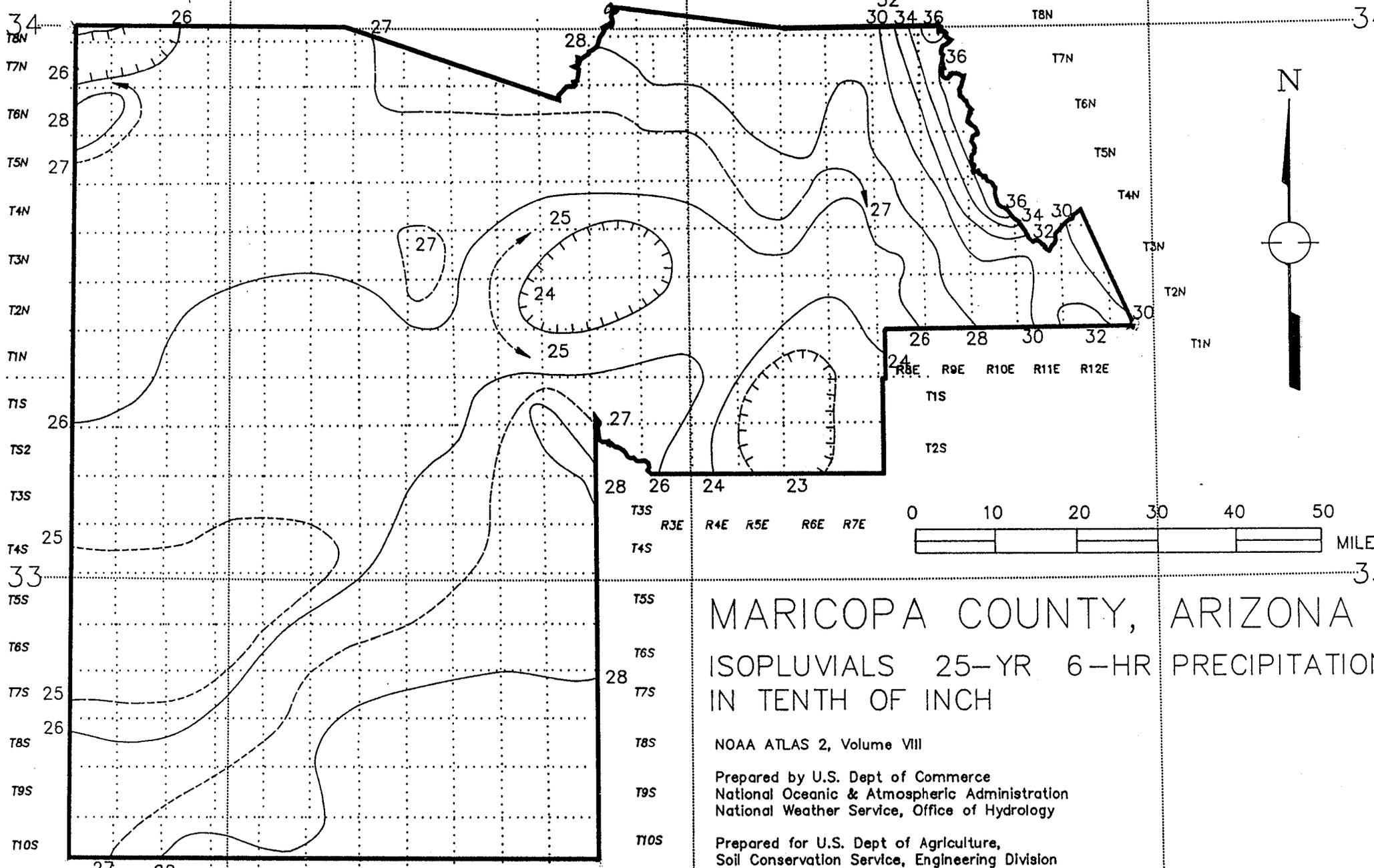
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 ISOPLUVIALS 25-YR 6-HR PRECIPITATION
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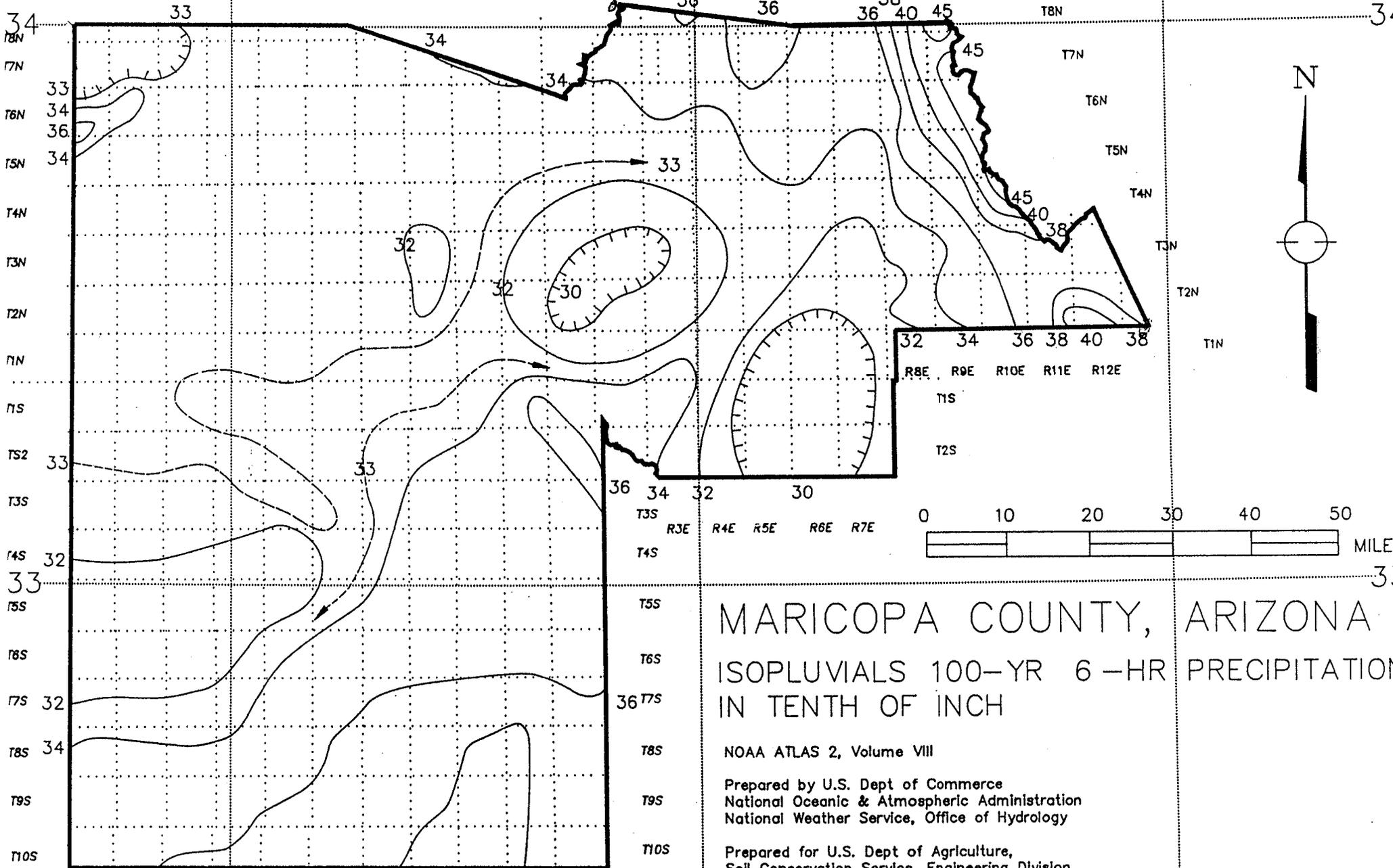
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 ISOPLUVIALS 100-YR 6-HR PRECIPITATION
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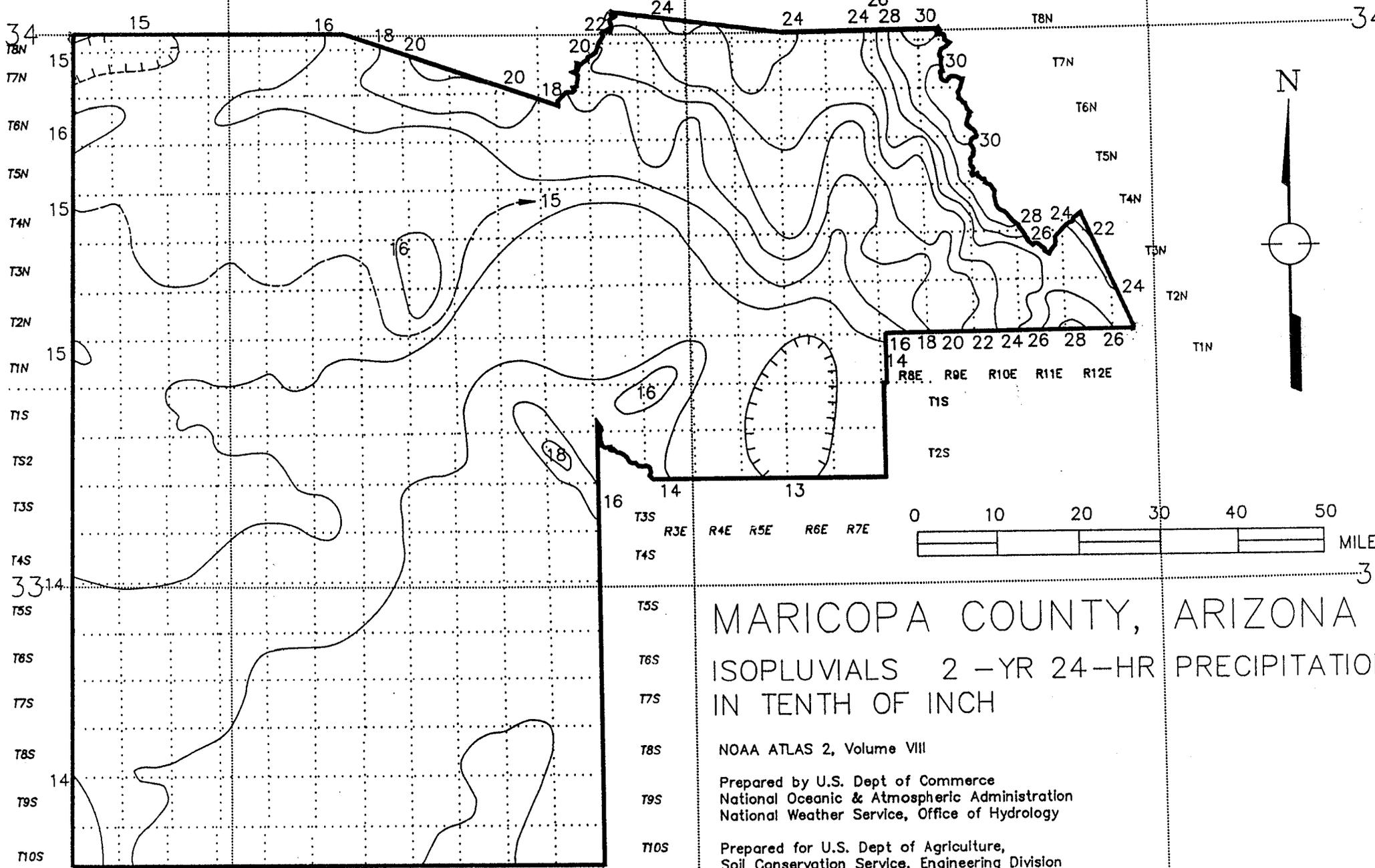
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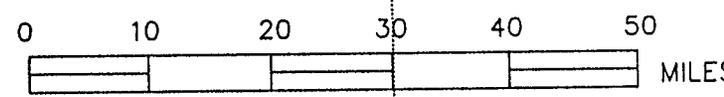
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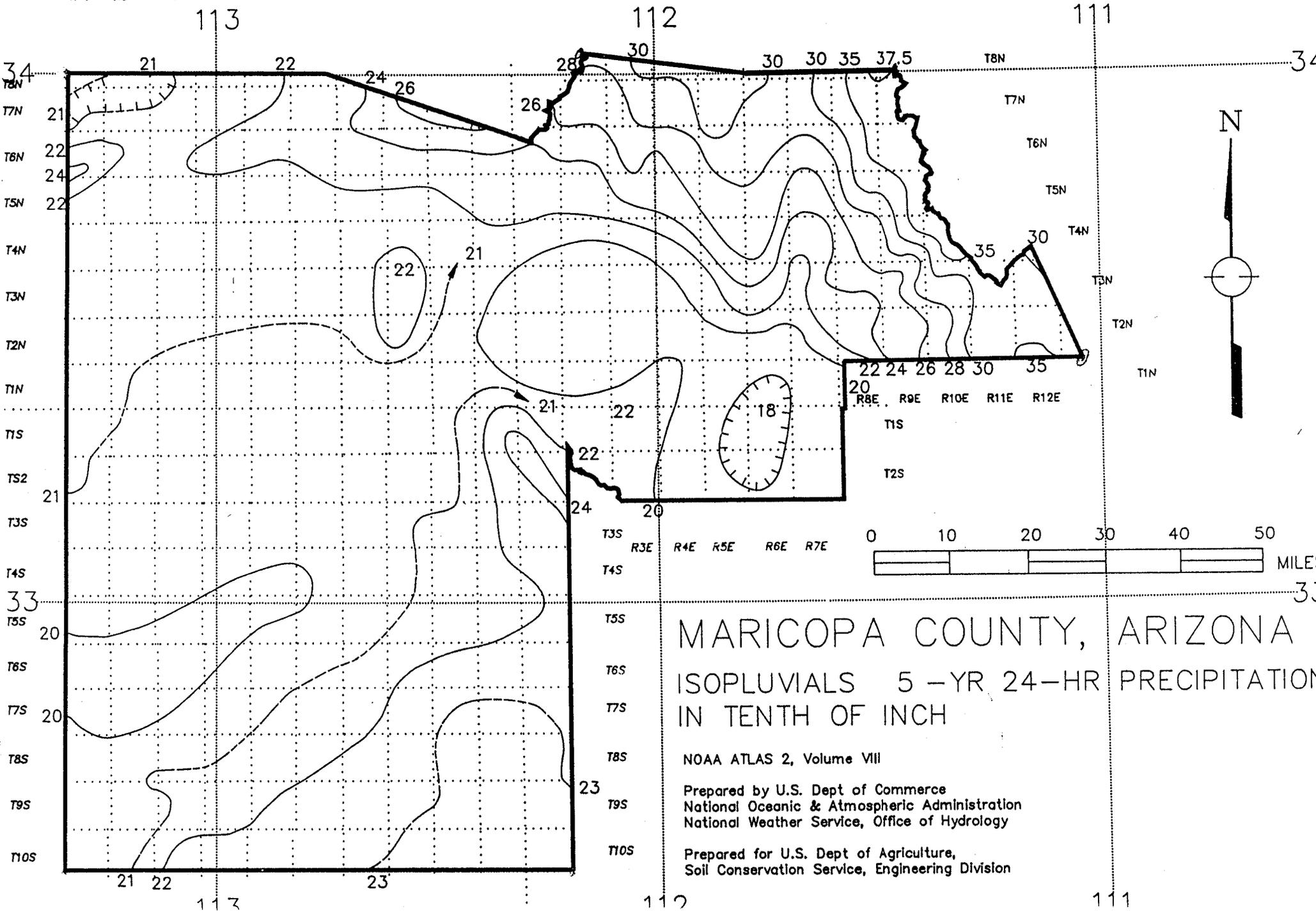
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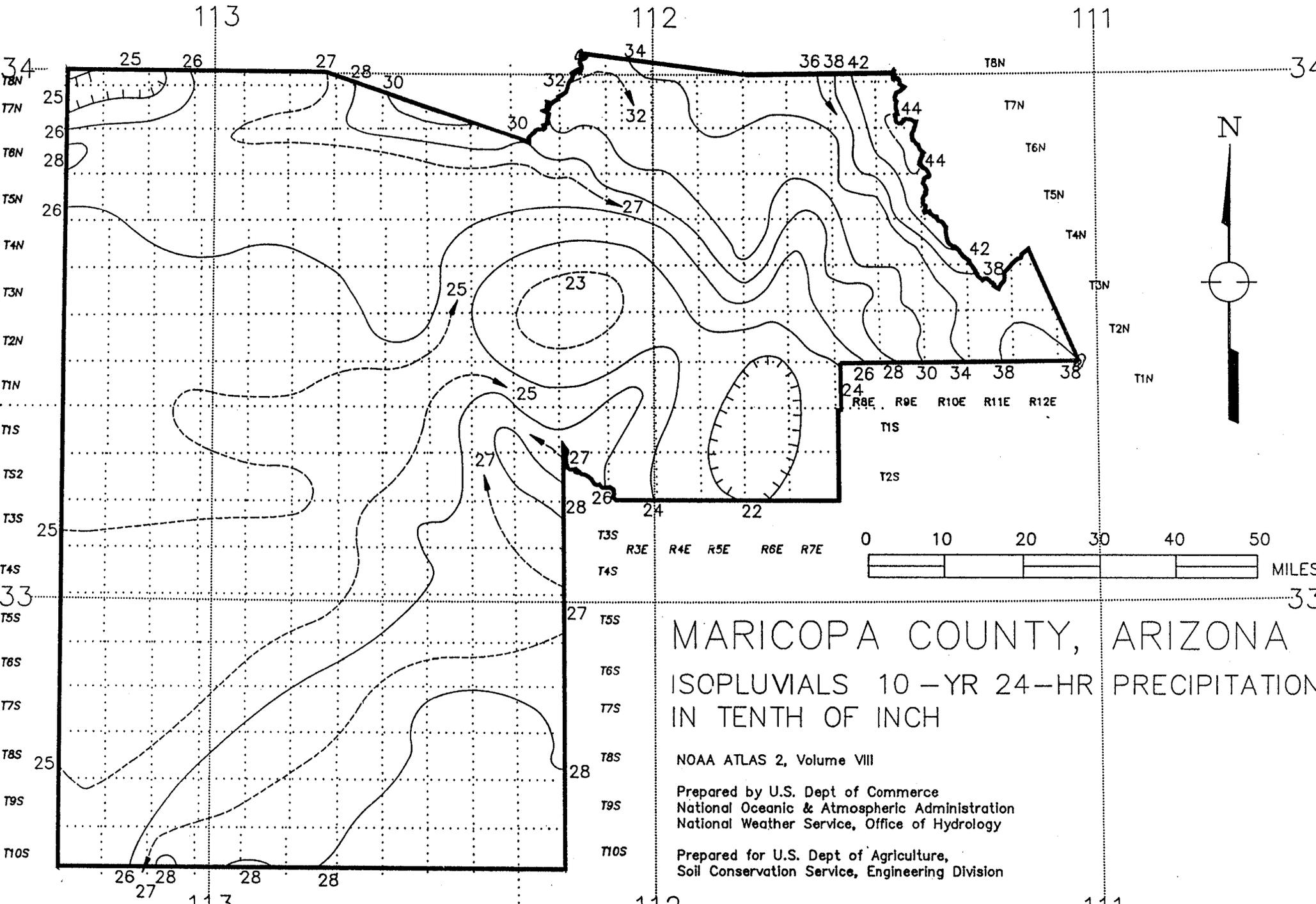
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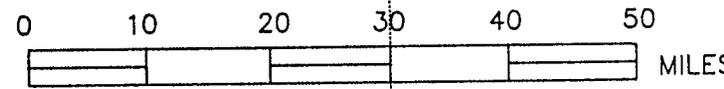
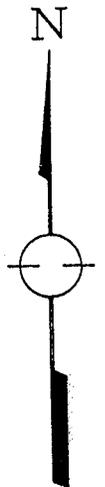
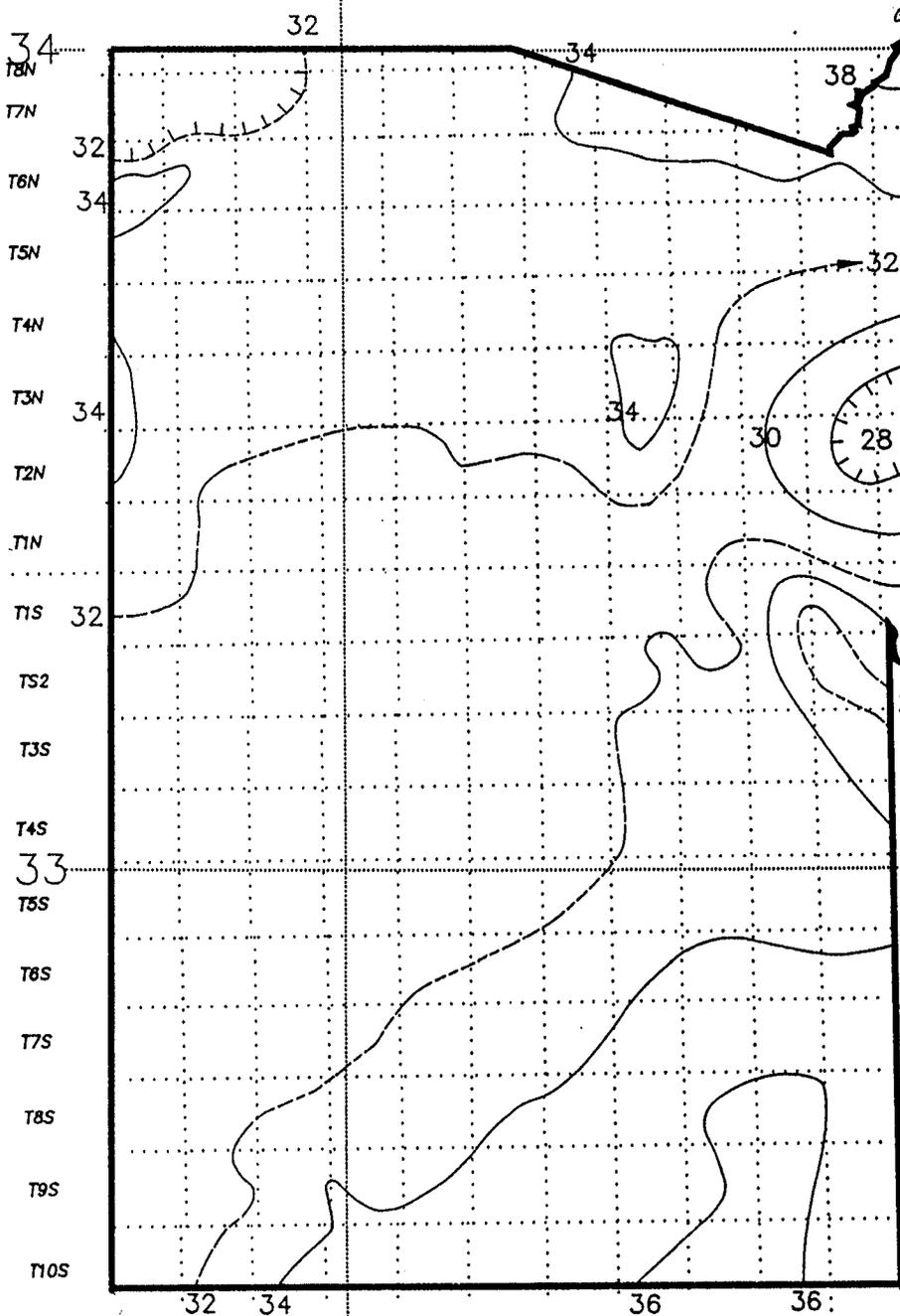
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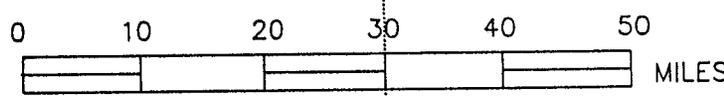
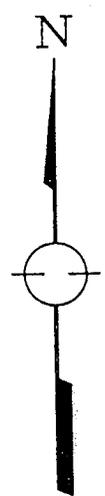
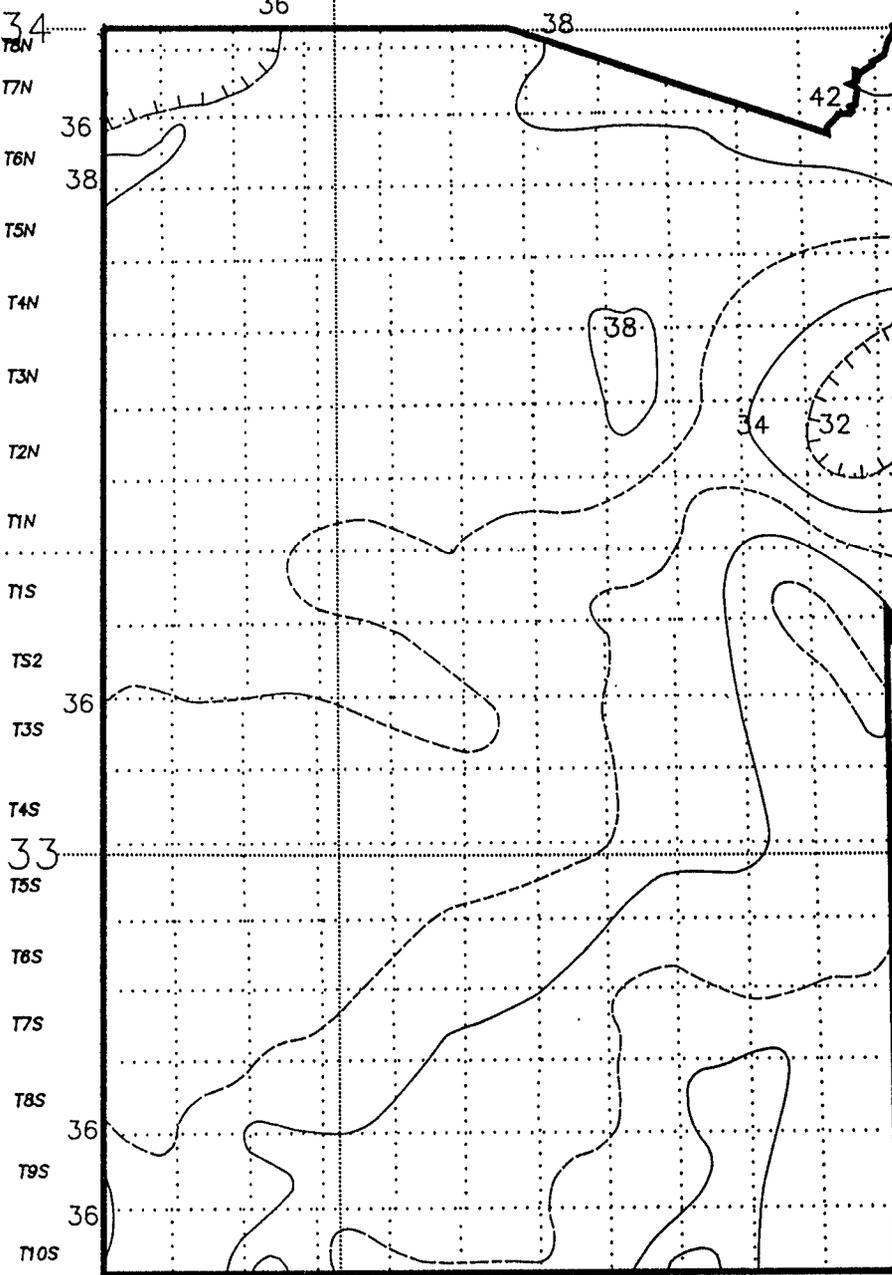
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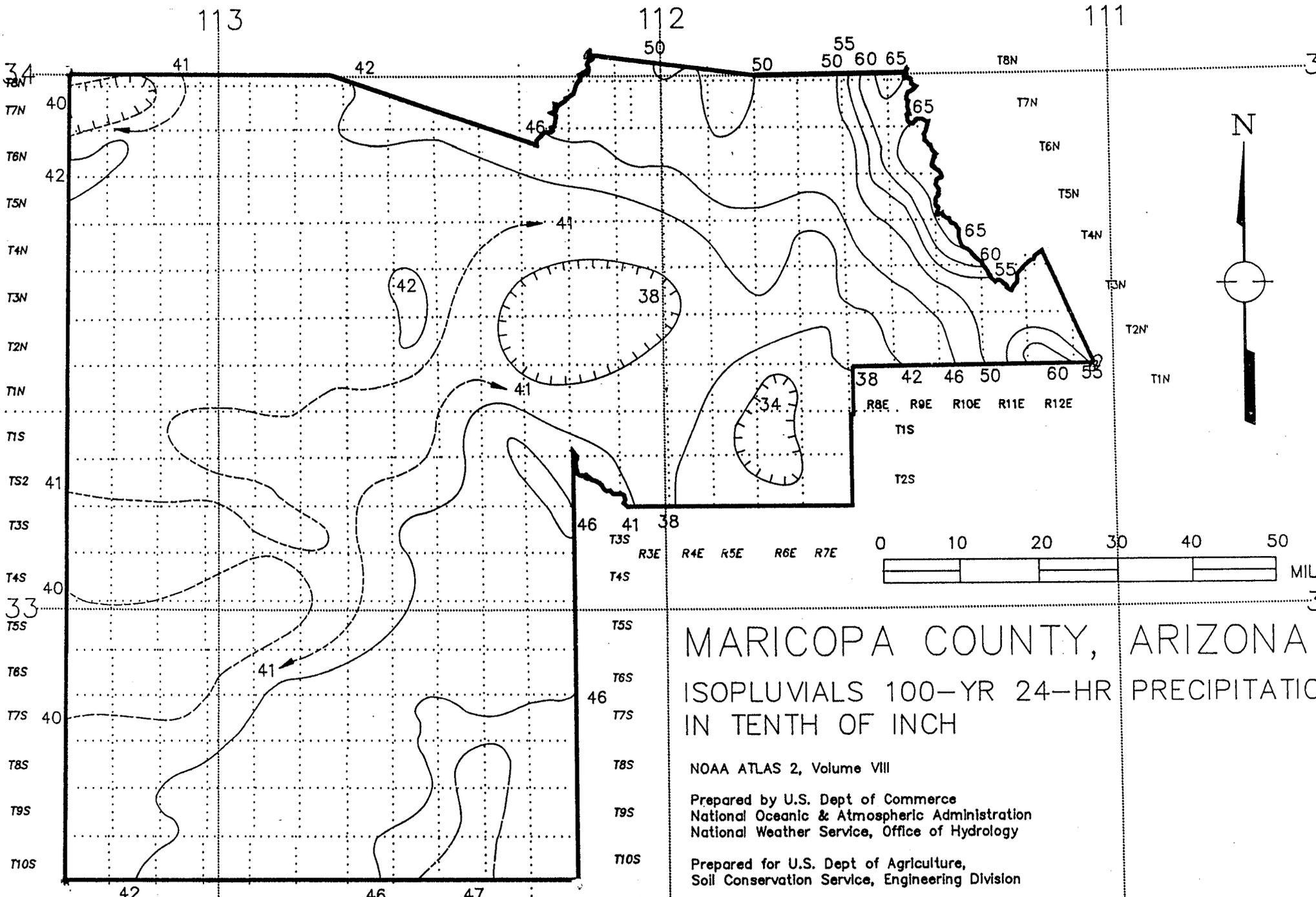
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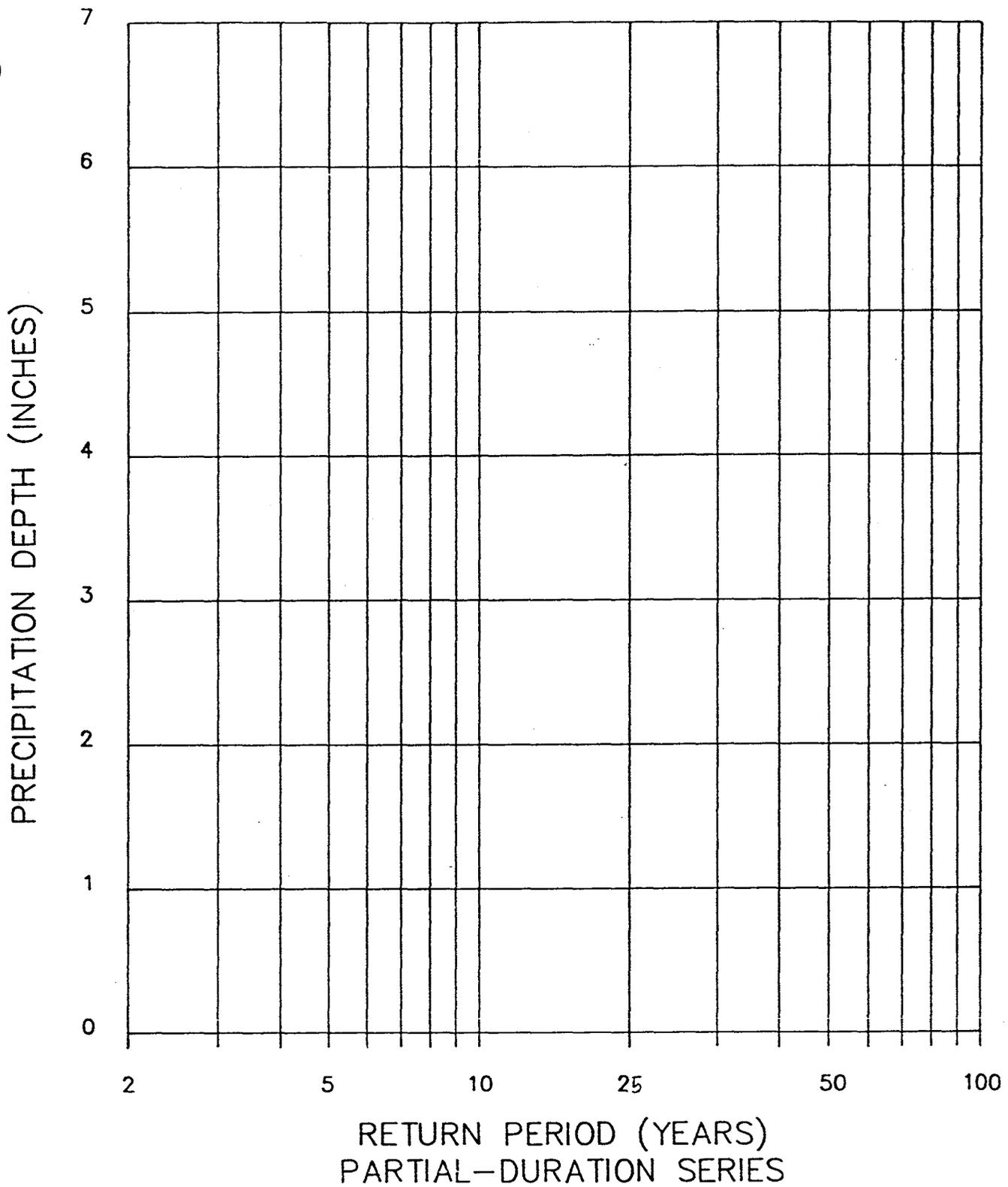


FIGURE-25. PRECIPITATION DEPTH VERSUS RETURN PERIOD FOR PARTIAL-DURATION SERIES

4) A particular design may require a duration different from 24-hour or 6-hour. For example retention design requires a 100-year frequency, 2-hour duration design storm. In such cases the following procedure is used which is the established method in NOAA, 1973. 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, 1-hour and the 2-year, 1-hour depths are calculated as follows:

$$\text{Compute } Y_2 = -0.011 + 0.942(X_1)(X_1/X_2)$$

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

where:

$$Y_2 = 2\text{-yr, 1-hr estimated value;}$$

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

$$X_1 = 2\text{-yr, 6-hr value from precipitation-frequency maps;}$$

$$X_2 = 2\text{-yr, 24-hr value from precipitation-frequency maps;}$$

$$X_3 = 100\text{-yr, 6-hr value from precipitation-frequency maps;}$$

$$X_4 = 100\text{-yr, 24-hr value from precipitation-frequency maps.}$$

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

$$\text{Compute 2-hr} = 0.341 (6\text{-hr}) + 0.659 (1\text{-hr})$$

$$\text{Compute 3-hr} = 0.569 (6\text{-hr}) + 0.431 (1\text{-hr})$$

Compute 12-hr, Figure 2.6, using the 6-hr and the 24-hr values

$$\text{Compute 5-min} = 0.34(1\text{-hr})$$

$$\text{Compute 10-min} = 0.51(1\text{-hr})$$

$$\text{Compute 15-min} = 0.62(1\text{-hr})$$

$$\text{Compute 30-min} = 0.82(1\text{-hr})$$

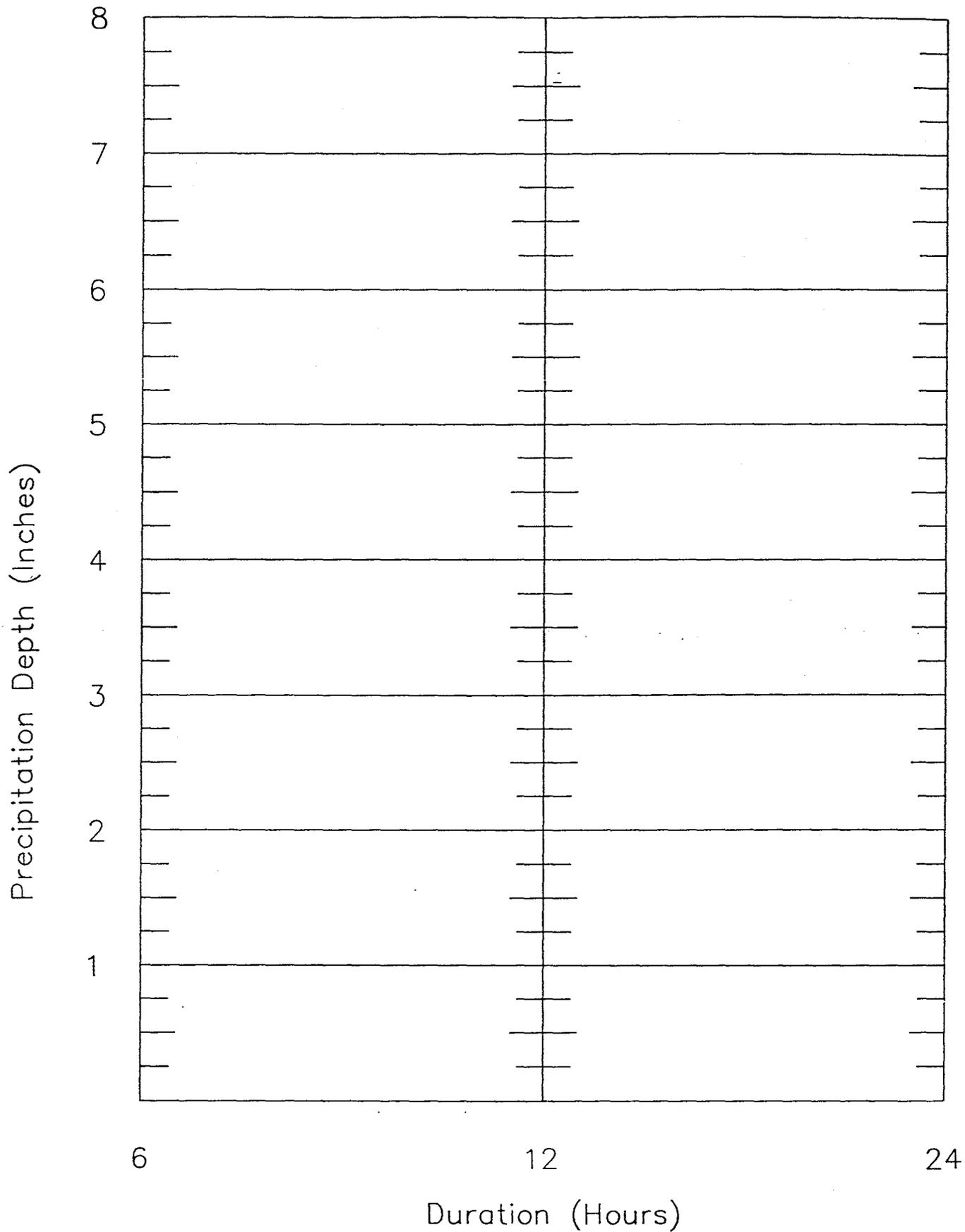


Figure 26. Precipitation depth-duration diagram (6 to 24 hr).

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 2.5. A rainfall mass curve can then be constructed by nesting around a desired duration, i.e., 15-in, or 30-min.

2.5 DEVELOPMENT OF DESIGN STORM DISTRIBUTIONS

The design storms for use in Maricopa County will be a 2-hour, 6-hour, or a 24-hour distribution. The 2-hour storm is used for retention design purposes. The 6-hour storm is for all hydrologic analysis for areas of up to 500 square miles. The 24-hour storm should be used for very large, natural watersheds (> 500 square miles).

2.5.1 2-hour Storm distribution. If the Rational Method is used, 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 2.1 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 2.7 illustrates the graphical form of this distribution.

2.5.2 6-hour Storm Distribution. The 6-hour rainfall distribution is a function of drainage area size. For this purpose five dimensionless rainfall patterns developed. Pattern #1 applied NOAA procedures to Phoenix Airport data. Patterns #2 through 5 are intended to provide variability of rainfall intensity as a function of drainage area. For this purpose the historic event of Aug. 19, 1954 was used. In a study by the US Army Corps of Engineers a set of rainfall patterns were developed, (US Army Corps of Engineers, 1974). This information was modified for a 6-hour duration rainfall. A rainfall pattern can be selected from Table 2.2 for direct input into HEC-1, once the size of

the drainage area is determined. In this case the rainfall time step is 15-minutes. Figure 2.8 illustrates graphical representations of the dimensionless rainfall patterns. The following should be used when selecting a rainfall pattern, which is also shown in Figure 2.9:

- For drainage area of up to 0.5 square miles use pattern #1;
- For drainage area in the range (0.5-2.8) square miles use pattern #2;
- For drainage area in the range (2.8-16.) square miles use pattern #3;
- For drainage area in the range (16.-90.) square miles use pattern #4;
- For drainage area in the range (90.-500) square miles use pattern #5.

2.5.3 24-hour Storm Distribution. In those cases that a 24-hour distribution is found suitable, the SCS TYPE II distribution may be used. Table 2.3 shows its mass curve distribution and Figure 2.10 illustrates its graphical form.

| <u>Time (minutes)</u> | <u>% Rainfall Depth</u> | <u>Time (minutes)</u> | <u>% Rainfall Depth</u> |
|-----------------------|-------------------------|-----------------------|-------------------------|
| 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 |

Table 2.1., 2-hour storm distribution for retention design.

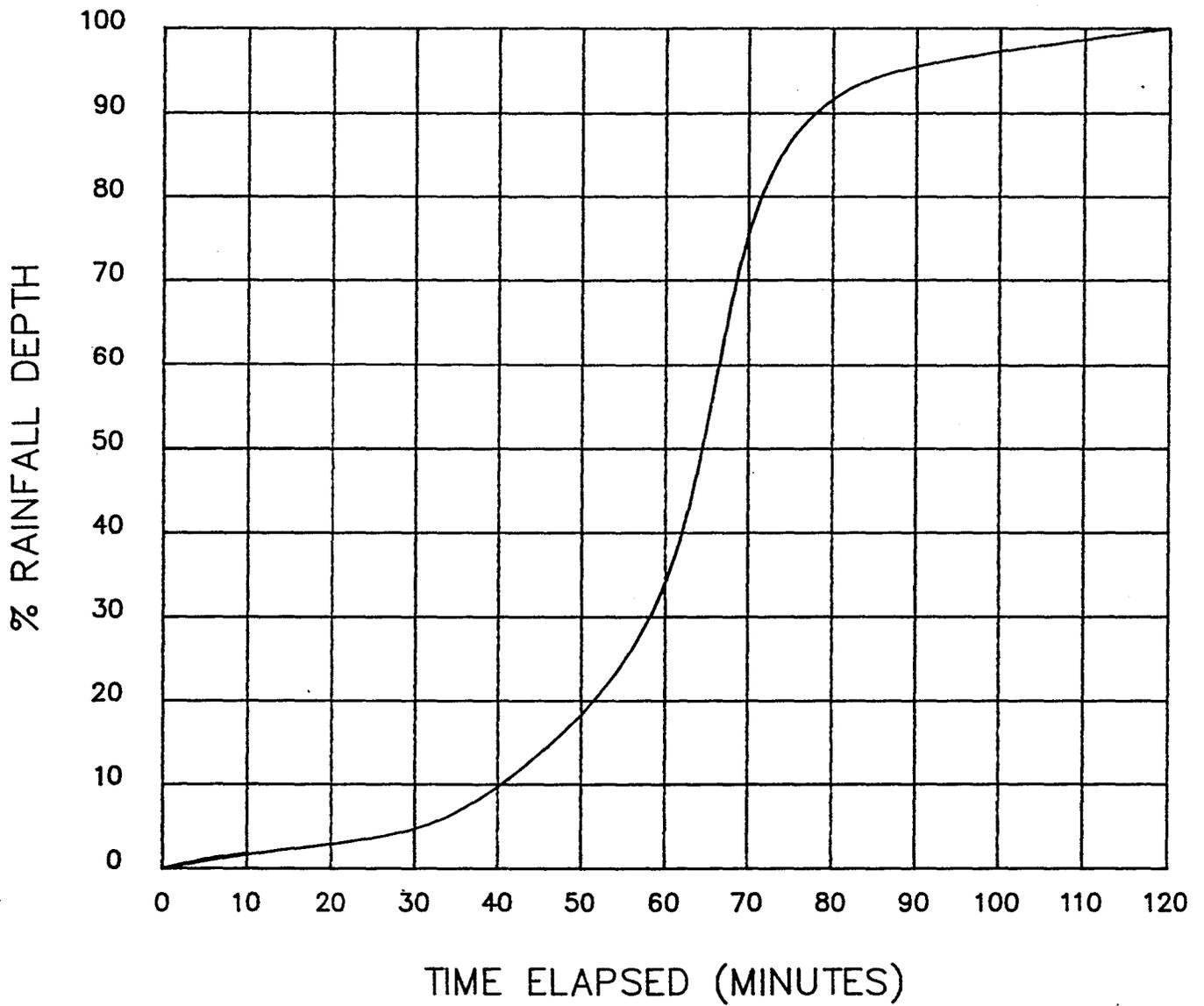


FIGURE-2.7 2-HOUR MASS CURVE FOR RETENTION DESIGN

| Time (hrs) | Pattern #1 | Pattern #2 | Pattern #3 | Pattern #4 | Pattern #5 |
|------------|------------|------------|------------|------------|------------|
| 0:00 | .0 | .0 | .0 | .0 | .0 |
| 0:15 | .5 | .6 | 1.5 | 2.1 | 2.4 |
| 0:30 | 0.9 | 1.2 | 2.0 | 3.5 | 4.3 |
| 0:45 | 1.4 | 2.0 | 3.0 | 5.1 | 5.9 |
| 1:00 | 2.2 | 3.1 | 4.8 | 7.1 | 7.8 |
| 1:15 | 3.0 | 3.9 | 6.3 | 8.7 | 9.8 |
| 1:30 | 3.8 | 4.9 | 7.6 | 10.5 | 11.9 |
| 1:45 | 4.7 | 5.7 | 9.0 | 12.5 | 14.1 |
| 2:00 | 5.4 | 6.7 | 10.5 | 14.3 | 16.2 |
| 2:15 | 6.2 | 7.6 | 11.9 | 16.0 | 18.6 |
| 2:30 | 7.5 | 8.7 | 13.5 | 17.9 | 21.2 |
| 2:45 | 8.8 | 10.0 | 15.2 | 20.1 | 23.9 |
| 3:00 | 10.7 | 12.0 | 17.5 | 23.2 | 27.1 |
| 3:15 | 12.7 | 16.3 | 22.2 | 28.1 | 32.1 |
| 3:30 | 20.5 | 25.2 | 30.4 | 36.4 | 40.8 |
| 3:45 | 36.6 | 45.1 | 47.2 | 50.0 | 51.5 |
| 4:00 | 82.3 | 69.4 | 67.0 | 65.8 | 62.7 |
| 4:15 | 90.0 | 83.7 | 79.6 | 77.3 | 73.5 |
| 4:30 | 92.0 | 90.0 | 86.8 | 84.1 | 81.4 |
| 4:45 | 93.9 | 93.8 | 91.2 | 88.8 | 86.4 |
| 5:00 | 95.2 | 96.7 | 94.6 | 92.7 | 90.7 |
| 5:15 | 96.5 | 98.5 | 97.4 | 95.8 | 94.5 |
| 5:30 | 97.7 | 99.0 | 98.0 | 96.5 | 95.5 |
| 5:45 | 98.8 | 99.0 | 98.7 | 97.6 | 96.9 |
| 6:00 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |

Table 2.2. 6-hour distributions. Pattern # represents %rainfall depth.

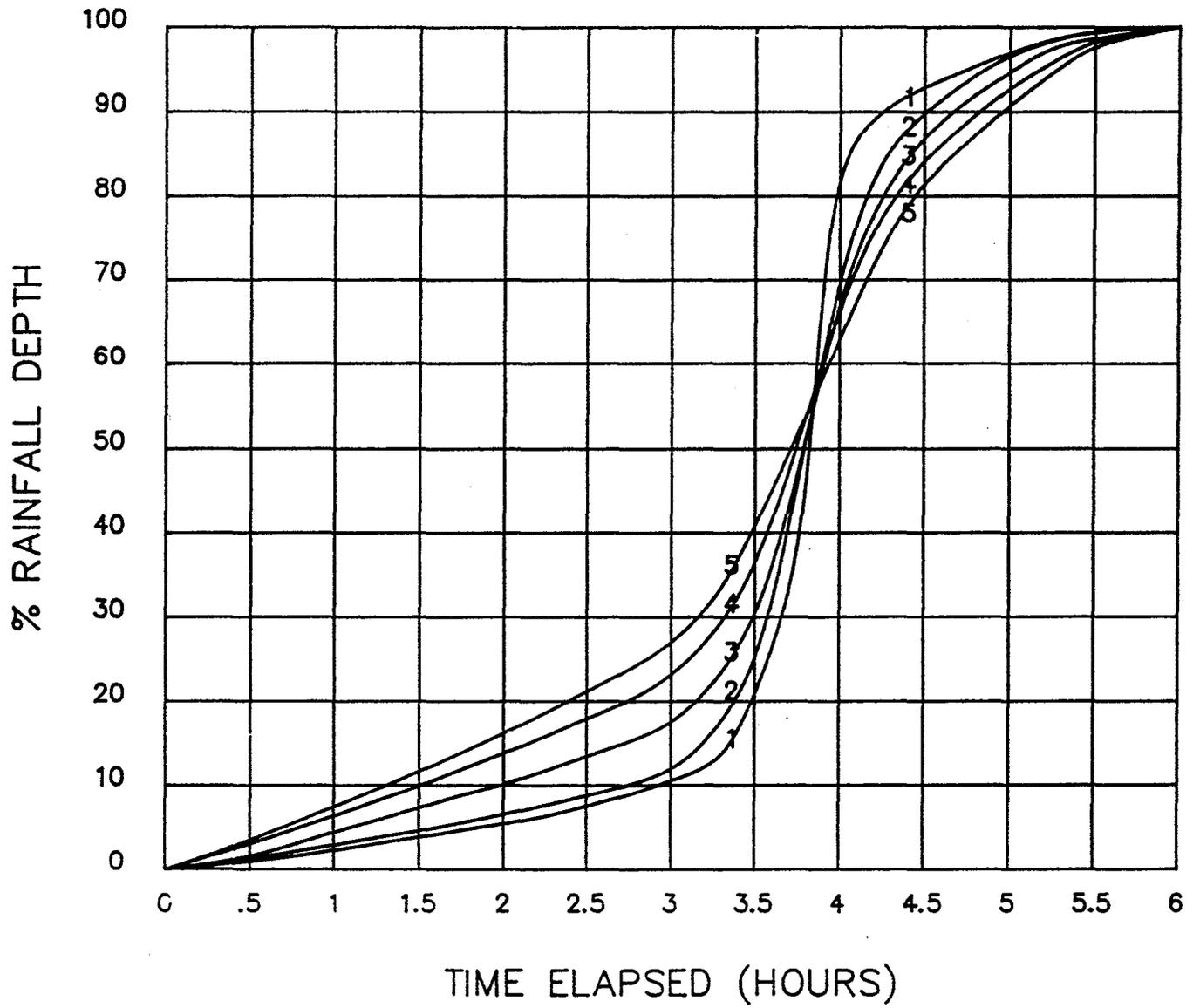


FIGURE-28. 6-HOUR MASS CURVES FOR MARICOPA COUNTY

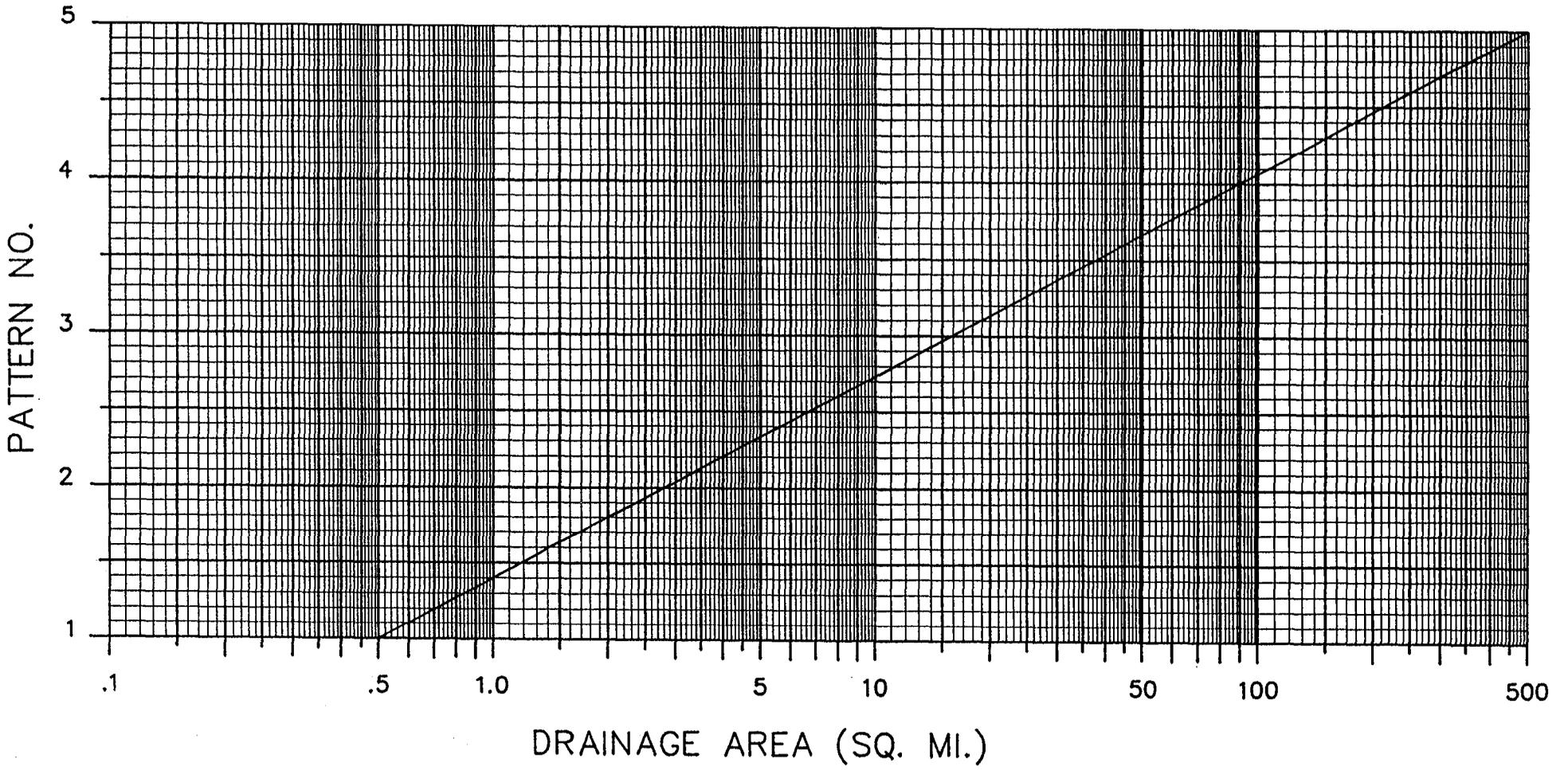


FIGURE-2.9. AREA VERSUS PATTERN NO.
FOR MARICOPA COUNTY

| <u>Time (hours)</u> | <u>%Rainfall Depth</u> | <u>Time (hours)</u> | <u>%Rainfall Depth</u> |
|---------------------|------------------------|---------------------|------------------------|
| 0:00 | 0.0 | 12:30 | 73.5 |
| 0:30 | 0.5 | 13:00 | 77.2 |
| 1:00 | 1.1 | 13:30 | 79.9 |
| 1:30 | 1.6 | 14:00 | 82.0 |
| 2:00 | 2.2 | 14:30 | 83.8 |
| 2:30 | 2.8 | 15:00 | 85.4 |
| 3:00 | 3.5 | 15:30 | 86.8 |
| 3:30 | 4.1 | 16:00 | 88.0 |
| 4:00 | 4.8 | 16:30 | 89.1 |
| 4:30 | 5.6 | 17:00 | 90.2 |
| 5:00 | 6.8 | 17:30 | 91.2 |
| 5:30 | 7.1 | 18:00 | 92.1 |
| 6:00 | 8.0 | 18:30 | 92.9 |
| 6:30 | 8.9 | 19:00 | 93.7 |
| 7:00 | 9.8 | 19:30 | 94.5 |
| 7:30 | 10.9 | 20:00 | 95.2 |
| 8:00 | 12.0 | 20:30 | 95.9 |
| 8:30 | 13.3 | 21:00 | 96.5 |
| 9:00 | 14.7 | 21:30 | 97.2 |
| 9:30 | 16.3 | 22:00 | 97.8 |
| 10:00 | 18.1 | 22:30 | 98.4 |
| 10:30 | 20.4 | 23:00 | 98.9 |
| 11:00 | 23.5 | 23:30 | 99.5 |
| 11:30 | 28.3 | 24:00 | 100.0 |
| 12:00 | 66.3 | | |

Table 2.3, 24-hour SCS TYPE-II distribution

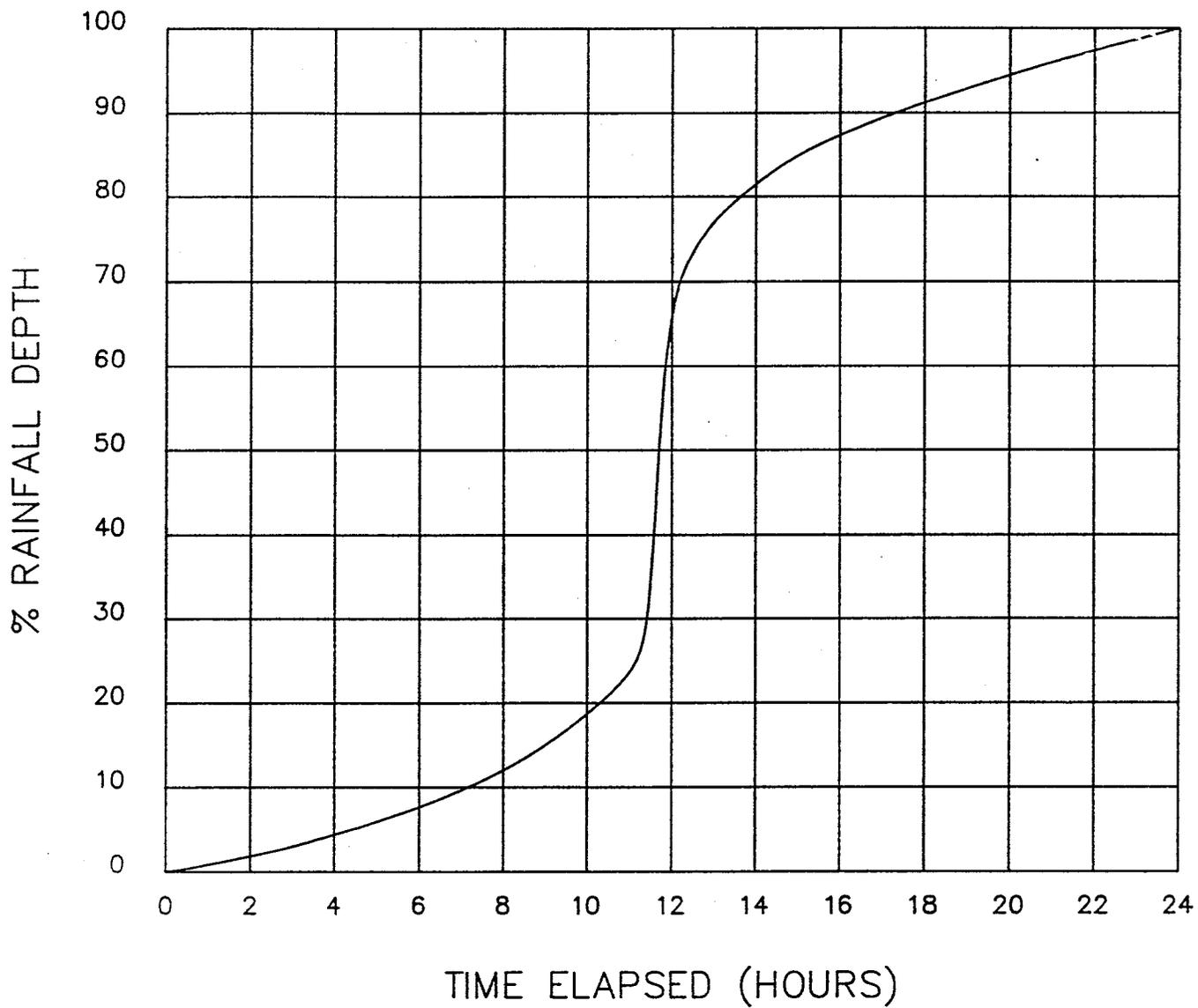


FIGURE-210. 24-HOUR MASS CURVE
(SCS TYPE II)

REFERENCES

Arkell, R.E., and F. Richard, 1986, Short Duration Rainfall Relations for the Western United States, Conference on Climate and Water Management-A Critical Era and Conference on the Human Consequences of 1985's Climate, Aug. 4-7, American Meteorological Society, pp. 136-141.

Miller, J.F., R.H. Frederick and R.J. Tracey, 1973, Precipitation-Frequency Atlas of the Western United States: IV: New Mexico and VIII: AZ, NOAA Atlas 2. National Weather Service, NOAA. Silver Springs, MD.

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PREFRE, National Weather Service.

Resolution FCD 87-7, 1987, Uniform Drainage Policies And Standards for Maricopa County, Arizona, Maricopa County Flood Control District, pp. 1-16.

U.S. Army Corps of Engineers, 1974, Gila River Basin, Arizona, New Mexico, and Phoenix City Streams, Design Memorandum No.1, Hydrology Part 1, Los Angeles District.

RATIONAL METHOD

3.1 GENERAL

The Rational Method was originally developed to estimate runoff from small urban areas and its use should be generally limited to those conditions. For the purposes of this manual, its use should be limited to area of up to 160 acres which require retention design. 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 is to be retained. If the development involves channel routing, the procedures given in Chapter 4 and Chapter 6 should be used.

3.2 RATIONAL EQUATION

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

$$Q = CiA \quad (1)$$

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 T_c .

T_c = the time of concentration (hours).

A = drainage area (acres).

The Rational formula 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, T_c , which is defined to be the time for water to flow from the most remote part of the watershed to the point of design. For the purposes of the Hydrology Manual the time of concentration should be computed by applying the following formula developed by Papadakis and Kazan (1987):

$$T_c = 11.4 L^{.5} r^{.52} S^{-.31} i^{-.38} \quad (2)$$

where

T_c = time of concentration (hours).

L = length of flow path.

r = resistance coefficient (Figure 3.1).

S = water course slope (feet/mile).

i = rainfall intensity (inches/hour).*

*It should be noted that i is the "excess rainfall intensity" as originally developed. However, when used in the Rational Equation, rainfall intensity and excess rainfall intensity provide similar values. This is due to the hydrologic characteristics of small, urban watersheds which result in minimal soil loss, as well as a time of concentration which is typically less than thirty minutes.

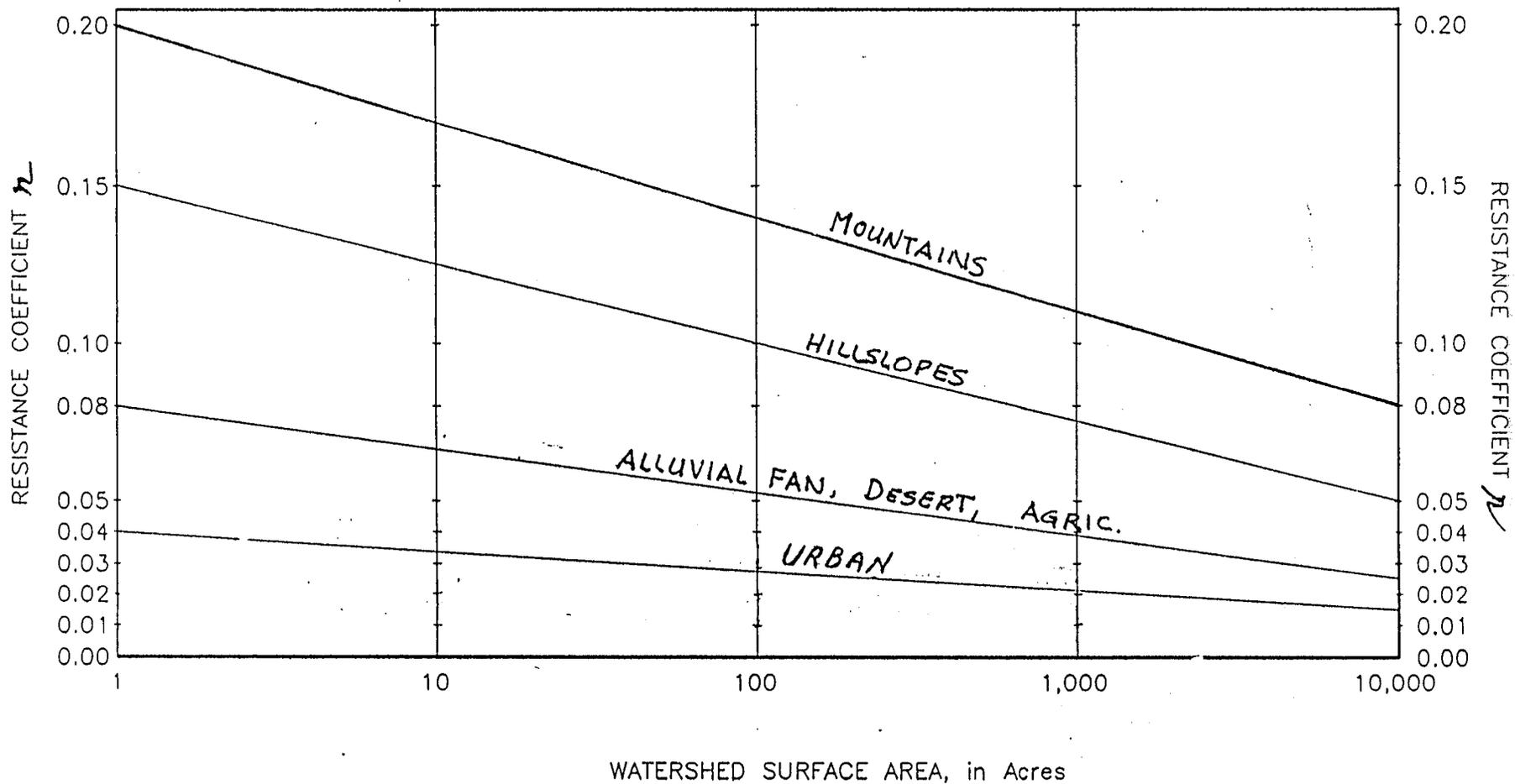


FIGURE 3.4
Resistance coefficient "n" as a function of watershed type and size

3.3 ASSUMPTIONS

Application of Rational Formula requires consideration of the following:

1. The maximum runoff 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 of the peak discharge is the same as the frequency of 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 Formula is appropriate for small urban 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 Formula should be used to calculate the generated peak flow and runoff volume for small urban areas where a retention design is required.

3.5.1 Peak Flow Calculation

1. Determine the area size within the development boundaries.
2. Select the runoff coefficient, C from Table 3.1
3. Calculate time of concentration. 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. Use this intensity in Eq.(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.

4. Calculate rainfall intensity for the location. The intensity value of the corresponding Tc in the above is for the Phoenix area. Use it in the following equation for application in other areas:

$$i = ip(P_{10}^6)/2.07 \quad (3)$$

where

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

ip = the intensity for the Phoenix area.

P_{10}^6 = the 10-year, 6-hour precipitation depth at the point of interest. It can be read from Plate 3.

5. Determine Q, peak flow by using the above value in Eq.(1)

3.5.2 Volume Calculations

Volume calculation should be done by applying the following equation:

$$V = C(P/12)A \quad (4)$$

where

V = calculated volume (acre-feet).

C = runoff coefficient from Table 1.

P = 100-year, 2-hour rainfall depth (inches).

A = drainage area (acres).

Streets

| | |
|-----------------------------|-------------|
| Asphaltic | 0.70 - 0.95 |
| Concrete | 0.80 - 0.95 |
| Gravel roadways & shoulders | 0.40 - 0.60 |

Industrial Areas

| | |
|--|-------------|
| Flat commercial (about 790 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

Table 3.1. C Coefficients for use with the Rational Formula.

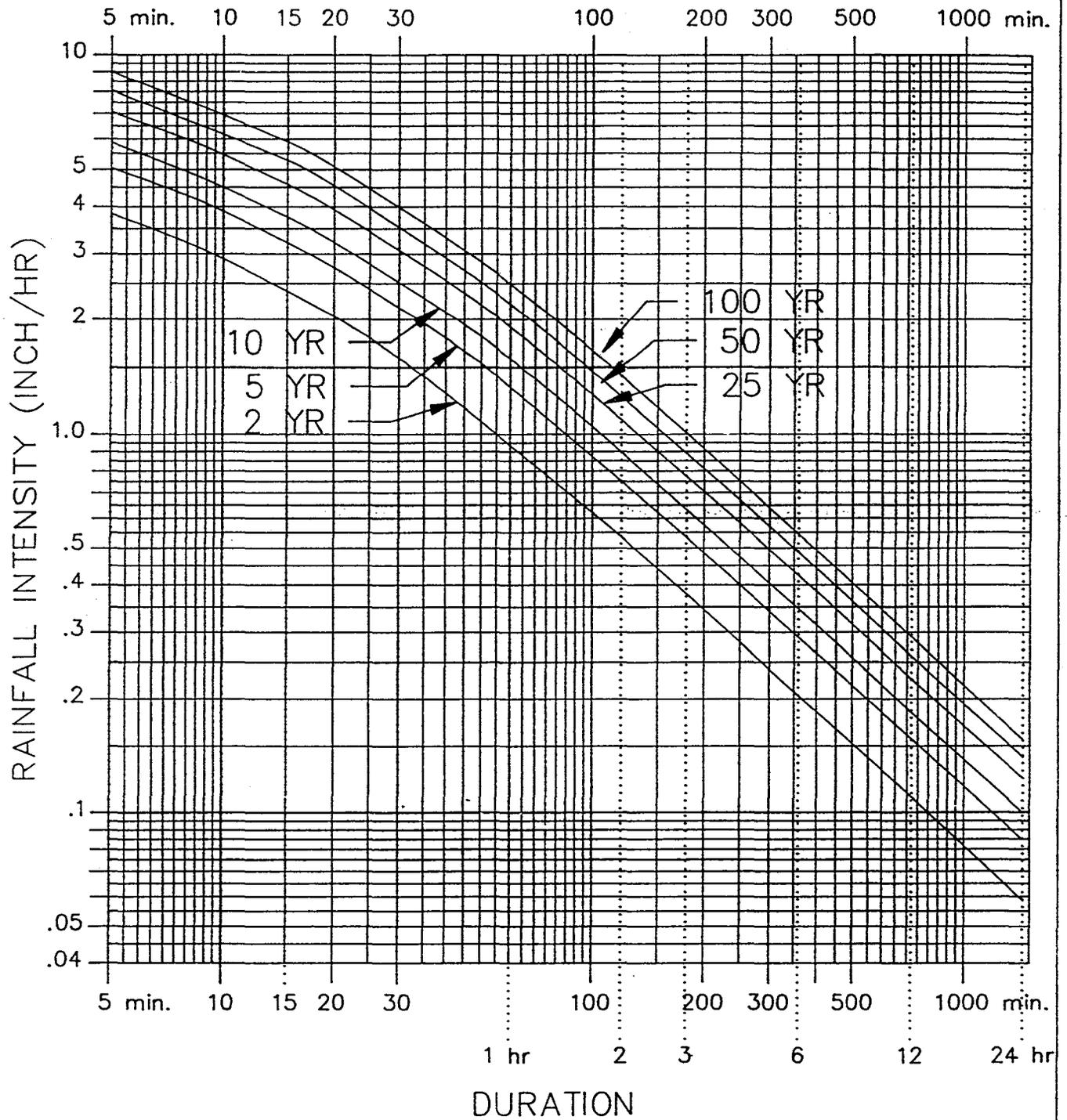


FIGURE 3.2
 RAINFALL INTENSITY-DURATION-FREQUENCY RELATION
 FOR MARICOPA COUNTY, ARIZONA

REFERENCES

Papadakis, C.N., and Kazan, M.N., 1987, Time of Concentration in Small, Rural Watersheds: Proceedings of the Engineering Hydrology Symposium, ASCE, Williamsburg, Virginia, pp. 633-638.

PRELIMINARY

For Review Only

Theory

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 losses 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 interception of rainfall by vegetal cover, depression storage on the land surface (paved or unpaved), evaporation of water from the land surface, and infiltration of water into the soil matrix. For the purposes of this Hydrology Manual rainfall losses are classified as either surface retention loss or infiltration. Losses are also often separated into accumulated losses prior to the initiation of surface runoff (initial abstraction), and losses during surface runoff; however, initial abstraction is not identical to surface retention loss because initial abstraction includes some amount of infiltration losses prior to the start of surface runoff.

Surface Retention Loss

Surface retention loss, as used herein, is the summation of all rainfall losses other than infiltration. The major component of this loss is depression storage consisting of rainfall that is captured in puddles, small surface depressions, road gutters, ditches, roofs, and other non-draining surface features. 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. Estimates of surface retention loss contained in this manual have been obtained from the analysis of rainfall-runoff data, results of rainfall simulator studies conducted on rangelands and turf, and published estimates for such losses.

Infiltration

Infiltration is the movement of water from the land surface into the soil. Infiltration is distinguished from percolation in that percolation is the movement of water through the soil subsequent to infiltration. Infiltration can be controlled by percolation if the soil does not have a sustained drainage capacity to provide access for more infiltrated water. However, the extent by which percolation can restrict infiltration of rainfall should be carefully evaluated before percolation can be assumed to restrict infiltration for the design rainfalls that are being considered in Maricopa County. For example, hydrologic soil group D has been defined by SCS soil scientists 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 soils in hydrologic soil groups A, B, or C could be classified as D if they are underlain by an impervious strata of clay, caliche, or rock. When these soils are considered in regard to long-duration rainfalls that are 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 under estimation of the rainfall losses for these storms. 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 times 0.30) of water can be infiltrated and stored in the shallow soil horizon. For a design rainfall of 6 hours and a rainfall depth of about 4 inches this represents a significant storage volume for infiltrated rainfall. Therefore, for drainage studies in Maricopa County that contain major areas of soil that are classified as hydrologic soil group D, the reason for the soil survey classification as D should be determined. Hydrologic soil group D should be retained 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, and site specific studies and sensitivity analyses should be performed to estimate the loss rates that should be used for such soils.

Infiltration of water into the soil is generally believed to follow an exponential decay function as proposed by Horton (1939):

$$f = f_c + (f_o - f_c)e^{-kt} \quad (1)$$

where f is the infiltration rate at time t ,
 f_c is the minimum (equilibrium) infiltration rate,
 f_o is the initial infiltration rate,
 t is the time from start of rainfall,
 k is a constant that is dependent upon the soil-vegetation complex, and
 e is the base of the Napierian logarithms.

This infiltration function is illustrated in Figure 1. In Figure 1, the infiltration rate is shown to be a continually decreasing function of time and the surface retention loss is shown to be a loss that is independent of and separate from the infiltration of rainfall into the soil. This represents the physical process that is to be simulated in modeling rainfall loss.

The driving force for infiltration is gravity and capillary forces drawing water into and through the pore spaces of the soil matrix.

Infiltration is controlled by soil properties, vegetation influences on the soil structure, surface cover, by rock and vegetation, and by tillage practices.

Methods for Estimating Rainfall Losses

Numerous methods have been developed for estimating rainfall losses. Five methods are available as options in the HEC-1 Flood Hydrology Package:

1. SCS CN loss rate,
2. Initial loss plus uniform loss rate,
3. Exponential loss rate,
4. Holtan infiltration equation, and
5. Green and Ampt infiltration equation.

The Holtan infiltration equation is similar to the Horton equation in that it is an exponential decay type of equation for which the rainfall loss rate asymptotically diminishes to the minimum infiltration rate, f_c . The Holtan equation is not extensively used and data and procedures to estimate the parameters for use in Maricopa County are not available. 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 preferred method of the U.S. Army Corps of Engineers. Data and procedures are not available to estimate the parameters for this loss rate method for all physiographic areas 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 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 it has wide acceptance among many

agencies, consulting engineering firms, and individuals throughout the community. However, the method is limited because of both theoretical and practical deficiencies of the method. Several of the deficiencies of the SCS CN method are:

1. Rainfall losses are independent of the duration of rainfall. That is, for a given depth of rainfall the same rainfall loss results regardless of the duration of rainfall, and the same rainfall excess would be estimated for a given rainfall depth occurring in, for example, either 1 hour or 24 hours.
2. The estimated rainfall loss rate is a function of rainfall intensity. Short periods of high intensity rainfall would often result in large estimates of rainfall losses. This is contrary to the Horton type infiltration relation as illustrated in Figure 1.
3. The infiltration rate approaches zero rather than a minimum infiltration rate, f_c .
4. The initial abstraction of 0.2S is not 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 based more on traditional acceptance of CN values rather than scientifically substantiated findings.
6. At low rainfalls (less than 4 inches), the estimate of rainfall loss is very sensitive to the selection of CN.

For these reasons the SCS CN method is not recommended for general use in Maricopa County.

Two methods are recommended for use in Maricopa County; these are the initial loss plus uniform loss rate (IL+ULR), and the Green and Ampt infiltration equation. 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 parameter values. 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.

Green and Ampt Infiltration Equation

This model, first developed in 1911 by W.H. Green and G.A. Ampt, has since 1973 received increased interest for estimating rainfall infiltration losses. The model has the form:

$$\begin{aligned} f &= K \left(1 + \frac{q}{F} \right) \\ q &= \psi \phi (S_f - S_i) \end{aligned} \quad (2)$$

where

- f = infiltration rate,
- K = hydraulic conductivity in the wetted zone (the ultimate steady state rate),
- ψ = average capillary suction in the wetted zone,
- S_f = final soil saturation (volumetric),
- S_i = initial soil saturation (volumetric);
- ϕ = soil porosity; and
- F = depth of rainfall that has infiltrated into the soil since the beginning of rainfall.

It is important to note that as rain continues, F increases and f approaches K , and therefore, f is inversely related to time. Equation 2 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:

$$\Delta F = -.5(2F - K\Delta t) + .5((2F - K\Delta t)^2 + 8K\Delta t(Q + F))^{1/2} \quad (3)$$

where t is the computation interval and F is accumulated depth of infiltration at the start of t . The average infiltration rate is:

$$\Delta f = \frac{\Delta F}{\Delta t} \quad (4)$$

Use of the Green and Ampt equation as coded in HEC-1 involves the simulation of rainfall loss as a two phase process. The first phase is the simulation of the surface retention loss as previously described, and 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 1. For example, as shown in Table 1 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. The three Green and Ampt equation infiltration parameters as coded in HEC-1 are hydraulic conductivity at natural saturation (XKSAT), wetting front capillary suction (PSIF), and volumetric soil moisture deficit at the start of rainfall (DTHETA). 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 cover, rock 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). Average values of XKSAT and PSIF for each of the soil texture classes are shown in Columns (2) and (3) of Table 2 (Rawls and others, 1983). 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 2 and 3, respectively (Rawls and Brakensiek, 1983). The values of XKSAT and PSIF from Table 2 should be used if general soil texture classification of the drainage area is available. The values of XKSAT

and PSTF from Figures 2 and 3 can be used if more specific soil texture classification is available from a detailed soil survey for which the percentage of sand and clay has been determined by an appropriate field soil survey. The use of the information in Figures 2 and 3 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 and the values from Table 2 should be used.

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. If the soil is 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. Therefore the range of DTHETA is 0.0 to the effective porosity. The porosity of soil as a function of soil texture (percent of sand and percent of clay) is shown in Figure 4 (Brakensiek and others, 1984).

Under natural conditions, soil seldom reaches a state of soil moisture less than the wilting point of vegetation, and a graph of volumetric soil moisture at wilting point as a function of soil texture is shown in Figure 5. Due to the rapid drainage capacity of most soils in Maricopa County the soil would not be expected to be in a state of soil moisture greater than the field capacity at the start of a design storm. A graph of volumetric soil moisture at field capacity as a function of soil texture is shown in Figure 6. 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 had been irrigated. Therefore, for irrigated lands it would be reasonable to assume that soil moisture 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 7. This figure was prepared by subtracting the wilting point soil moisture on Figure 5 from the soil porosity on Figure 4. The value of DTHETA "Normal" as a function of soil texture is shown in Figure 8. This figure was prepared by subtracting the field capacity soil moisture on Figure 6 from the soil porosity on Figure 4. 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 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 is usually 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, it is increased by tillage, and it is 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 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. Procedures have been developed (Rawls and others, 1988) for incorporating the effects of ground cover and canopy cover into the estimation of the parameters for the Green and Ampt equation. Past research has shown that the wetting front capillary suction 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 ground cover and canopy cover.

The set of equations that have been developed to adjust the bare soil hydraulic conductivity for the influence of soil ground cover and canopy cover are:

$$KE = (CAN) (KE_c) + (OP) (KE_o) \quad (5)$$

$$KE_c = (K) (CF) \left[\left(\frac{BC}{CAN} \right) (CRC) + A \left(1 - \frac{BC}{CAN} \right) \right] \quad (6)$$

$$KE_o = (K) \left[\left(\frac{BO}{CAN} \right) (CRC) + A \left(1 - \frac{BO}{CAN} \right) \right] \quad (7)$$

$$CF = 1 + 0.96 CAN \quad (8)$$

$$CRC = \frac{L}{\frac{L - TC}{SC} + \frac{TC}{B}} \quad (9)$$

$$A = e^{[2.82 - 0.099(SA) + 1.94(BD)]} \quad (10)$$

$$B = 0.0099 + 0.0721(TC) + 0.0000068(SA^2) + 0.000021(SA^2)(TC) - 0.000315(SA)(TC) \quad (11)$$

where KE = hydraulic conductivity adjusted for ground cover and canopy cover, and KE is the adjusted value of XKSAT in the Green and Ampt equation,

KE_C = hydraulic conductivity of soil under canopy cover,
 KE_O = hydraulic conductivity of soil outside of canopy cover,
 K = bare ground hydraulic conductivity, equal to $XKSAT$,
 CF = canopy factor,
 CRC = soil crust factor,
 A = macroporosity factor,
 CAN = ratio of area under canopy cover to total area,
 OP = ratio of area outside of canopy cover to total area,
 BC = ratio of bare soil (no vegetation, litter, or rock cover) that is under canopy cover to total area,
 BO = ratio of bare soil that is outside of canopy cover to total area,
 L = wetting front depth (assume 6 cm),
 TC = soil crust thickness (assume 0.5 cm),
 SC = correction factor for partial saturation of the subcrust soil (see Table 3),
 SA = percent sand in the soil,
 BD = soil bulk density (see Figure 9).

These equations have been developed for the SI system of units and they should be solved in this system and the resulting hydraulic conductivity (KE) in cm/hr should be converted to $XKSAT$ in inches/hr.

Equations 5 through 7 can be simplified for use in estimating the hydraulic conductivity of turf by the following equation:

$$KE = (K)(A) \quad (12)$$

where A is defined by Equation 10.

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 three infiltration parameters for design purpose use in Maricopa County. This is because for most flood 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 prediction purposes. However, appropriate adjustments to the infiltration parameters can be made as necessary for special flood studies such as reconstitution of storm events.

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 10. 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. Two parameters are needed to use this method, the initial loss and the uniform loss rate (STRTL and CNSTL), respectively, according to HEC-1 nomenclature).

The initial loss (STRTL) is the sum of all losses prior to the onset of runoff and is made up of surface retention loss and some amount of infiltration. This is also called initial abstraction. Values of the infiltration part of STRTL for bare ground according to soil texture classification are shown in Columns (3) through (5) in Table 4. These values have been derived from the Green and Ampt infiltration equation and parameter values that are shown in Table 2. The value of STRTL "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 STRTL "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 STRTL "Saturated" should be used for soil that is maintained in a state of high soil moisture such as occurs in

irrigated agricultural lands.

Values of STRIL for bare ground that have been classified according to hydrologic soil group are shown in Table 5. These values within each hydrologic soil group have been derived from the data in Table 4 for the various soil texture classifications.

The values of STRIL from Table 4 and 5 represent that portion of the initial loss that is attributable to infiltration and these should be increased to account for the surface retention loss that is a function of land-use and vegetation. These additive STRIL values are shown in Table 1. The value of STRIL that is used is the sum of the portion due to infiltration (Table 4 or 5) plus the portion that is due to surface retention (Table 1).

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 for soils according to soil texture classification are equivalent to the hydraulic conductivity at natural saturation (XKSAT) as determined for the Green and Ampt equation (Table 2). The values of CNSTL for soils classified according to hydrologic soil groups are shown in Table 5. These values within each hydrologic soil group have been selected from inspection of XKSAT values in Table 2 for the various soil texture classifications. These values of CNSTL shown in Table 5 are consistent with general information that is available for estimating CNSTL as shown in Table 6.

Procedure for Estimating Loss Rates

Green and Ampt Method

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 2 if general soil texture classification is available or from Figure 2 if adequate soil texture data is available from an approved sampling program.
3. If desired, adjust the value of XKSAT for the influences of ground cover and canopy cover using Equations 5 through 11 or Equation 12 for turf.
 4. Estimate the wetting front capillary suction parameter (PSIF) from Table 2 if general soil texture classification is available or from Figure 3 if adequate soil texture data is available from an approved sampling program.
 5. Estimate the value of DTHETA from Table 2 if general soil texture classification is available or from either Figure 7 or 8 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 1 to estimate the surface retention loss (IA).
 7. As an alternative to the above procedure, 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 of Maricopa County and the local agency before initiating these procedures.

Initial Loss Plus Uniform Loss Rate Method

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 STRIL from Table 4 if the losses are to be based on soil texture classification.
3. Use values of CNSTL and STRIL from Table 5 if the losses are to be based on hydrologic soil group.
4. Determine the land-use and/or soil cover and use Table 1 to estimate the surface retention loss to be added to STRTL from either Table 4 or 5.

TABLE ~~X3-1~~
4-1

Surface retention loss for various land surfaces in Maricopa County
(addition to STRTL for IL+ULR method and IA for Green and Ampt method)

| Land-Use and/or Surface Cover | Surface Retention Loss inches |
|---|-------------------------------|
| (1) | (2) |
| <u>Natural</u> | |
| Desert and rangeland, flat slope | .35 |
| Hillslopes, Sonoran desert | .15 |
| Mountain, brush | .25 |
| <u>Developed (Residential and Commercial)</u> | |
| Lawn and turf | .20 |
| Desert landscape | .10 |
| Pavement | .05 |
| <u>Agricultural</u> | |
| Tilled | 1.00 |
| Irrigated pasture | .50 |

TABLE 13-2
4-2

Green and Ampt loss rate parameter values for bare ground

| Soil Texture Classification | XKSAT in/hr | PSIF inches | DTHETA ¹ | | |
|--------------------------------|----------------|----------------|---------------------|--------|-----------|
| | | | Dry | Normal | Saturated |
| (1) | (2) | (3) | (4) | (5) | (6) |
| sand | 4.6 | 1.9 | .35 | .30 | 0 |
| loamy sand | 1.2 | 2.4 | .35 | .30 | 0 |
| sandy loam | .40 | 4.3 | .35 | .25 | 0 |
| loam | .15 | 3.5 | .35 | .25 | 0 |
| silty loam | .25 | 6.6 | .40 | .25 | 0 |
| sandy clay loam | .06 | 8.6 | .25 | .15 | 0 |
| clay loam | .04 | 8.2 | .25 | .15 | 0 |
| silty clay loam | .04 | 10.8 | .30 | .15 | 0 |
| sandy clay | .02 | 9.4 | .20 | .10 | 0 |
| silty clay | .02 | 11.5 | .20 | .10 | 0 |
| clay | .01 | 12.4 | .15 | .05 | 0 |

¹ Selection of DTHETA:

- Dry - for nonirrigated lands such as desert and rangeland
- Normal - for irrigated lawn, turf, and permanent pasture
- Saturated - for irrigated agricultural land

TABLE 135
4-3

- Initial Loss plus Uniform Loss Rate parameter values for bare ground according to soil texture classification

| Soil texture Classification | Uniform Loss Rate CNSTL | Initial Loss, In inches STRTL ¹ | | |
|--------------------------------|----------------------------|---|--------|-----------|
| | | Dry | Normal | Saturated |
| (1) | (2) | (3) | (4) | (5) |
| sand | 4.6 | 1.3 | 1.3 | 0 |
| loamy sand | 1.2 | .8 | .8 | 0 |
| sandy loam | .4 | .7 | .6 | 0 |
| loam | .15 | .6 | .5 | 0 |
| silty loam | .25 | .8 | .7 | 0 |
| sandy clay loam | .06 | .6 | .5 | 0 |
| clay loam | .04 | .5 | .4 | 0 |
| silty clay loam | .04 | .6 | .5 | 0 |
| sandy clay | .02 | .4 | .3 | 0 |
| silty clay | .02 | .4 | .3 | 0 |
| clay | .01 | .3 | .2 | 0 |

¹ Selection of STRTL:

- Dry - for nonirrigated lands such as desert and rangeland
- Normal - for irrigated lawn, turf, and permanent pasture
- Saturated - for irrigated agricultural land

TABLE 534
44

Initial Loss plus Uniform Loss Rate parameter values
for bare ground according to hydrologic soil group

| Hydrologic Soil Group | Uniform Loss Rate CNSTL | Initial Loss, in inches STRTL ¹ | | |
|--------------------------|----------------------------|---|--------|-----------|
| | | Dry | Normal | Saturated |
| (1) | (2) | (3) | (4) | (5) |
| A | .40 | .6 | .5 | 0 |
| B | .25 | .5 | .3 | 0 |
| C | .15 | .5 | .3 | 0 |
| D | .05 | .4 | .2 | 0 |

¹ Selection of STRTL:

- Dry - for nonirrigated lands such as desert and rangeland
- Normal - for irrigated lawn, turf, and permanent pasture
- Saturated - for irrigated agricultural land

TABLE ~~4-3~~
4-5

Published values of uniform loss rates

| Hydrologic Soil Group | Uniform Loss Rate, in inches/hour | | |
|--------------------------|-----------------------------------|--------------------------|--------------------------|
| | Musgrave (1955) | USBR (1975) ¹ | USBR (1988) ² |
| (1) | (2) | (3) | (4) |
| A | .30 - .45 | .40 | .30 - .50 |
| B | .15 - .30 | .24 | .15 - .30 |
| C | .05 - .15 | .12 | 0 - .05 |

¹ Design of Small Dams, Second Edition, 1975, Appendix A

² Design of Small Dams, Third Edition, 1988

TABLE 3

Soil crusting reduction factor for the Green and Ampt infiltration equation hydraulic conductivity parameter.

| Soil Texture Classification | Reduction Factor for Subcrust Conductivity SC |
|-----------------------------|---|
| (1) | (2) |
| Sand | 0.91 |
| Loamy sand | 0.89 |
| Sandy loam | 0.86 |
| Loam | 0.82 |
| Silt loam | 0.81 |
| Sandy clay loam | 0.85 |
| Clay loam | 0.82 |
| Silty clay loam | 0.76 |
| Sandy clay | 0.81 |
| Silty clay | 0.73 |
| Clay | 0.75 |

VOID

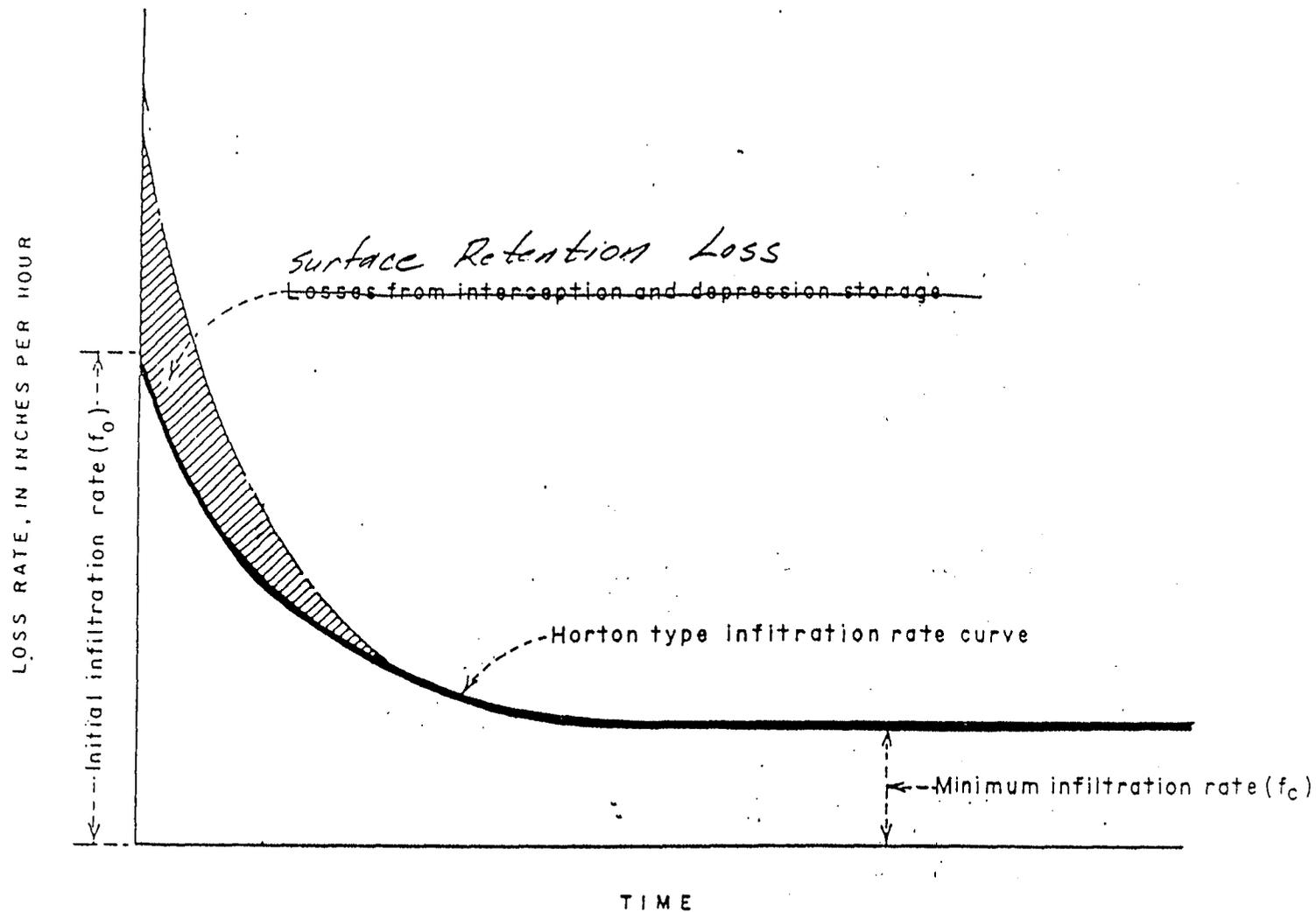


FIGURE 4-1

Representation of rainfall losses by surface retention and infiltration according to an exponential decay (Horton) type of equation

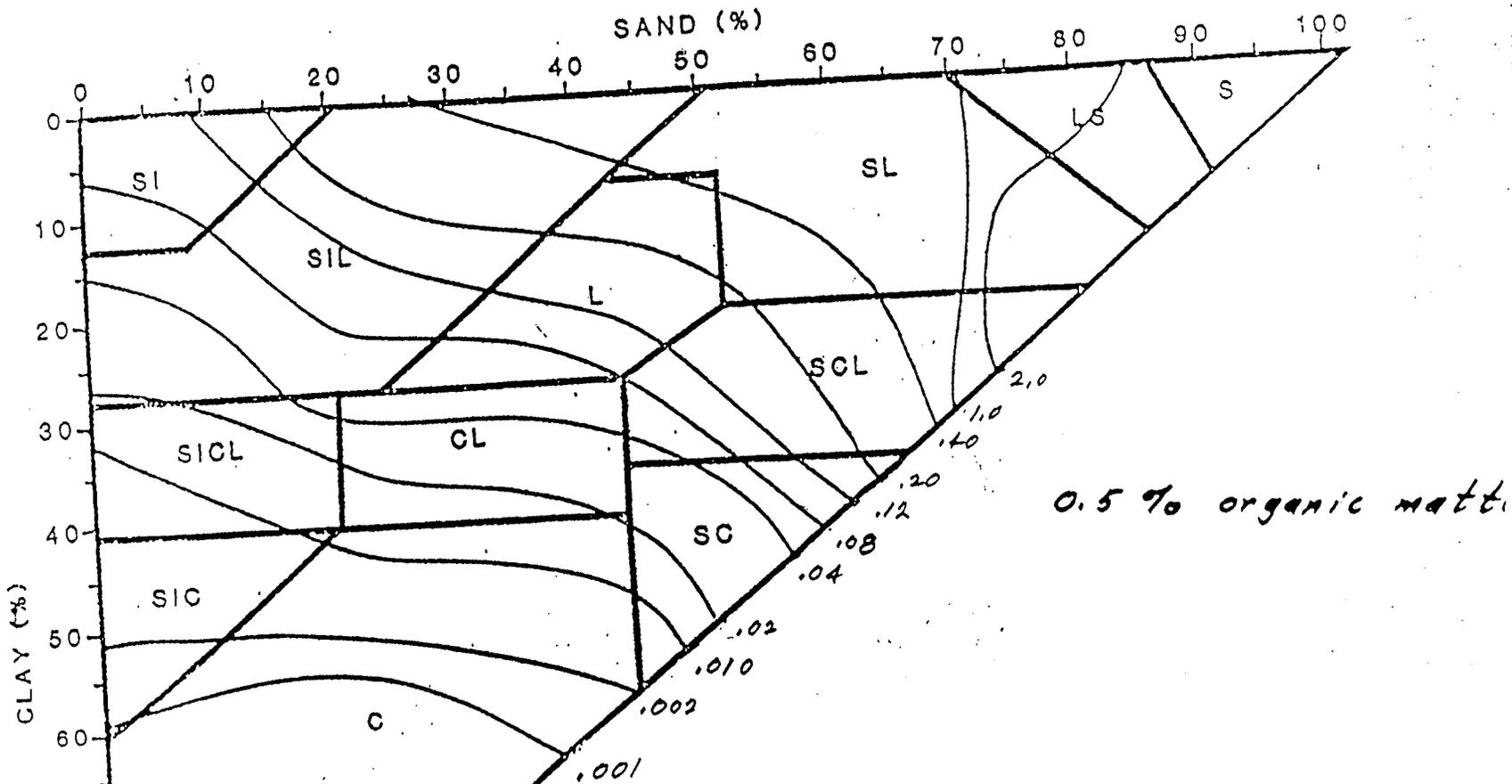


FIGURE 4-2

Green and Ampt Loss Rate
 Hydraulic Conductivity at natural
 saturation in inches/hour (~~cm/hour~~)
 XKSAT

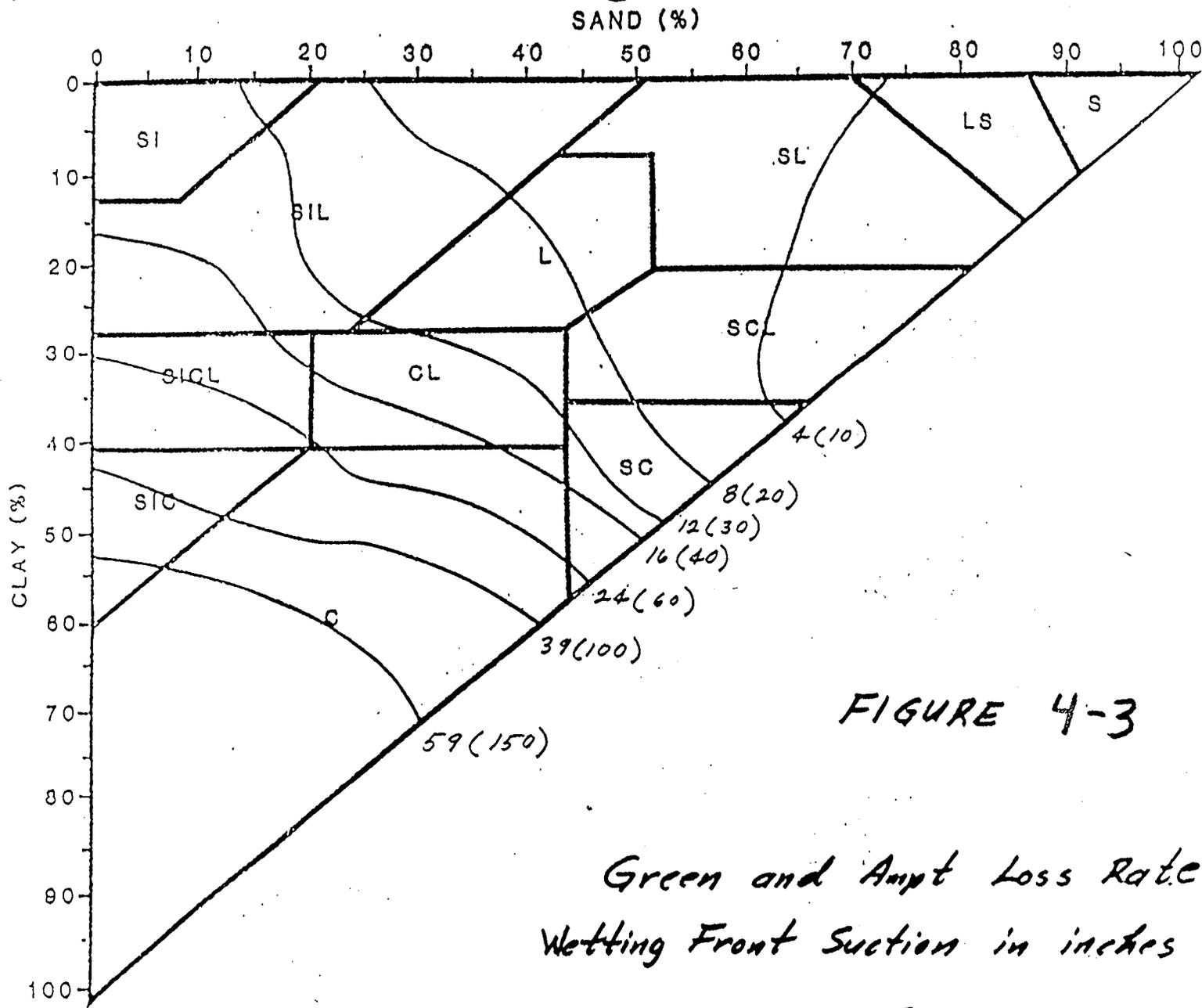


FIGURE 4-3

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

PSIF

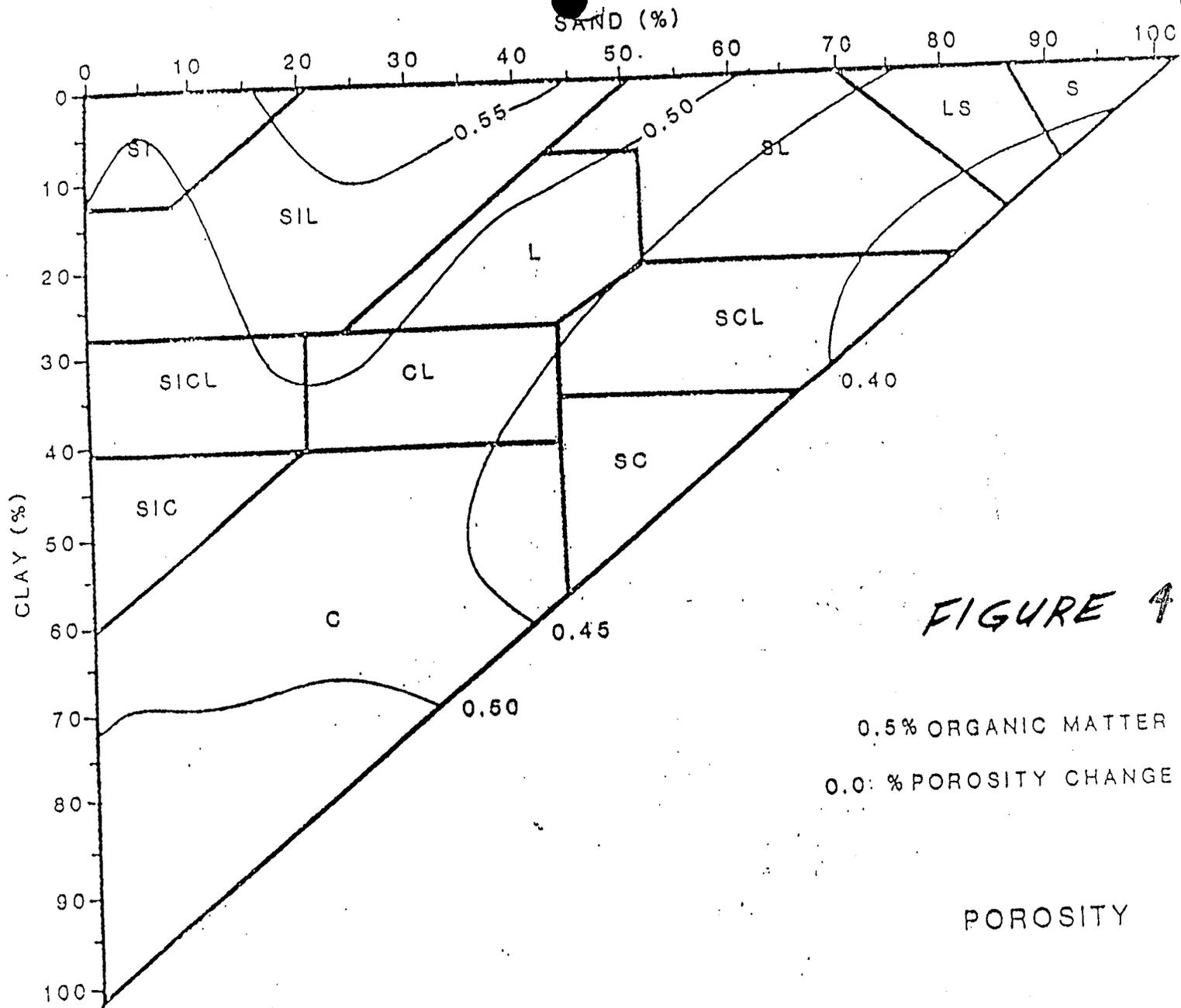


FIGURE 4-4

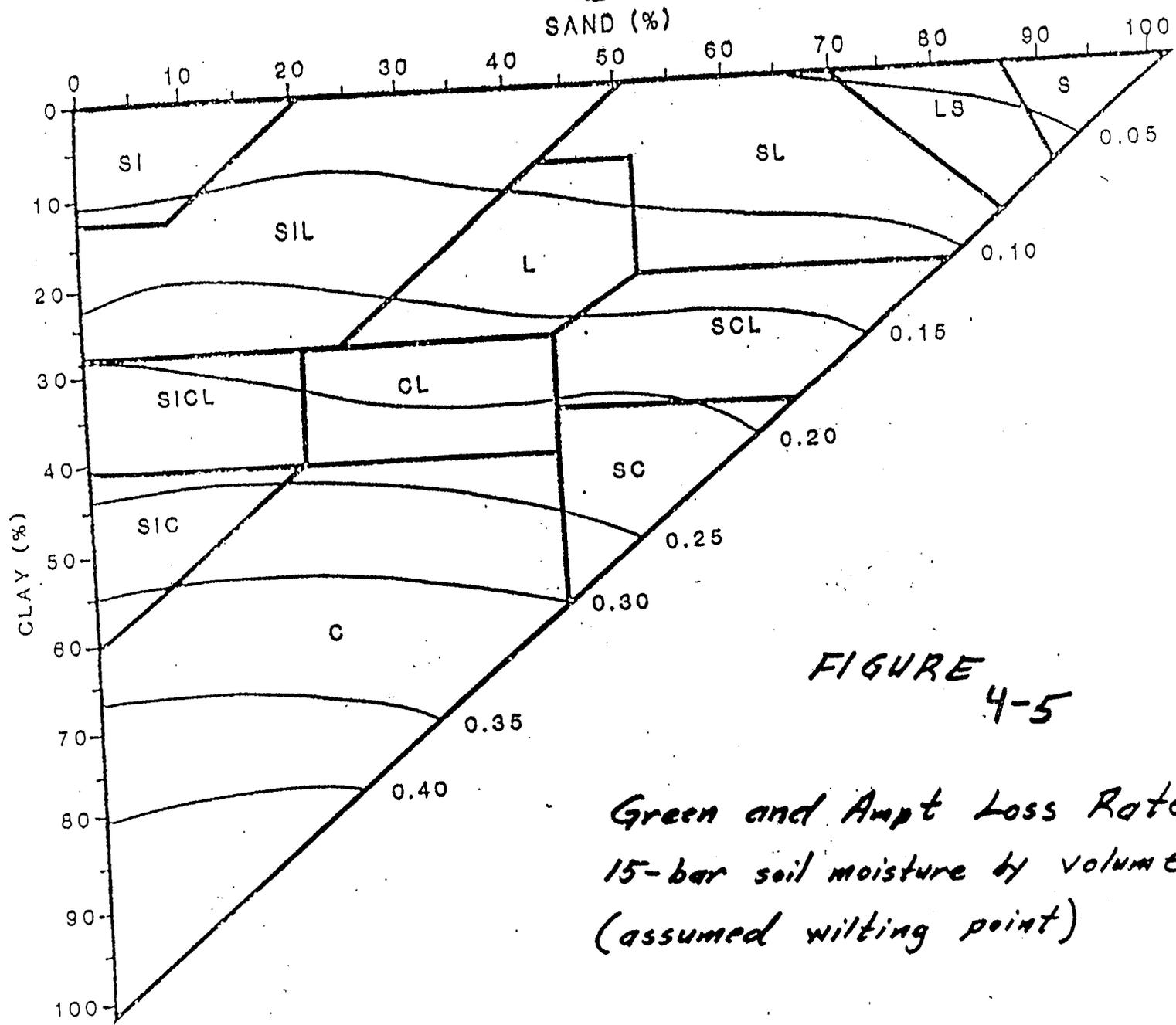


FIGURE 4-5

Green and Awp Loss Rate
 15-bar soil moisture by volume
 (assumed wilting point)

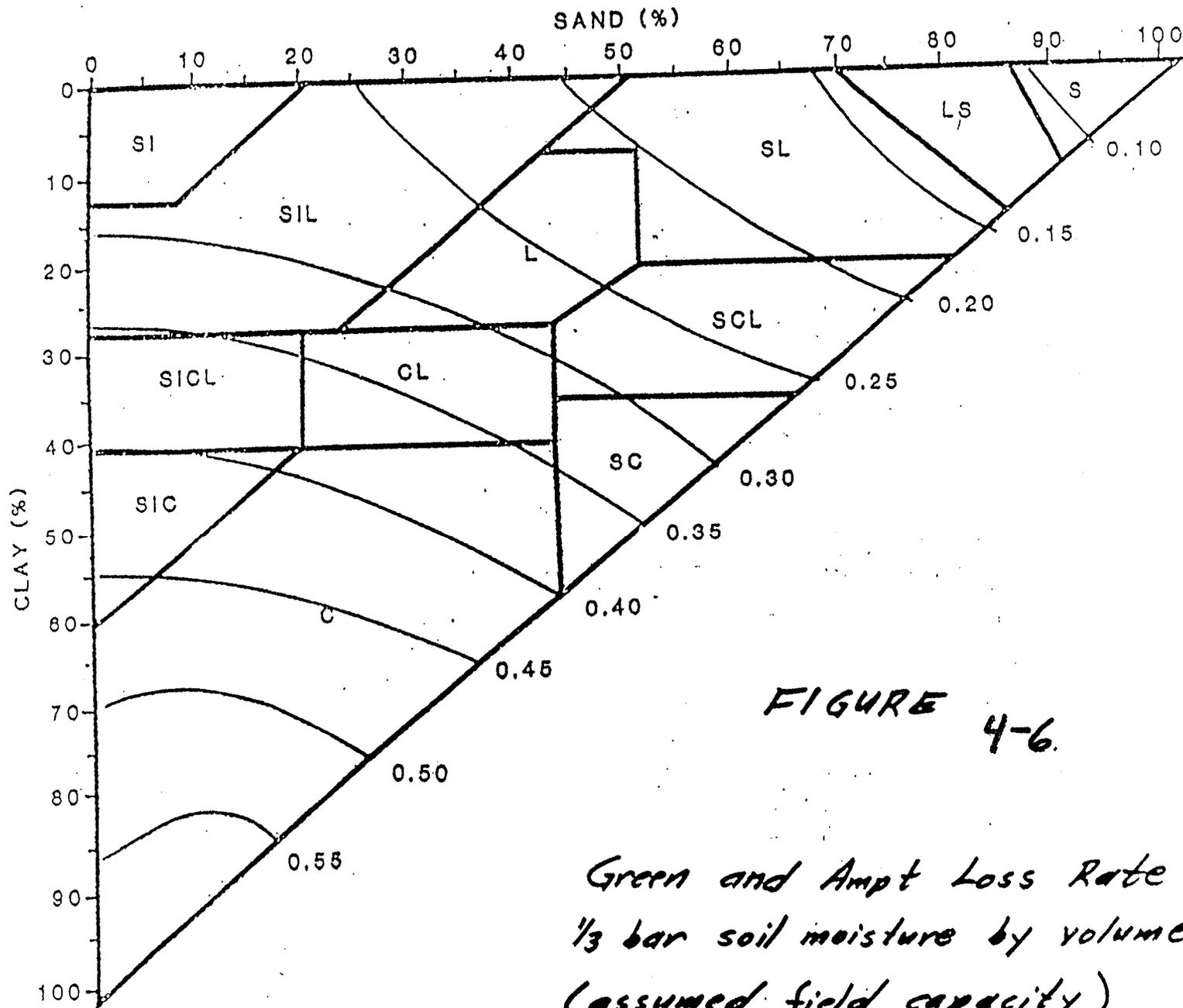


FIGURE 4-6.

Green and Ampt Loss Rate
 1/3 bar soil moisture by volume
 (assumed field capacity)

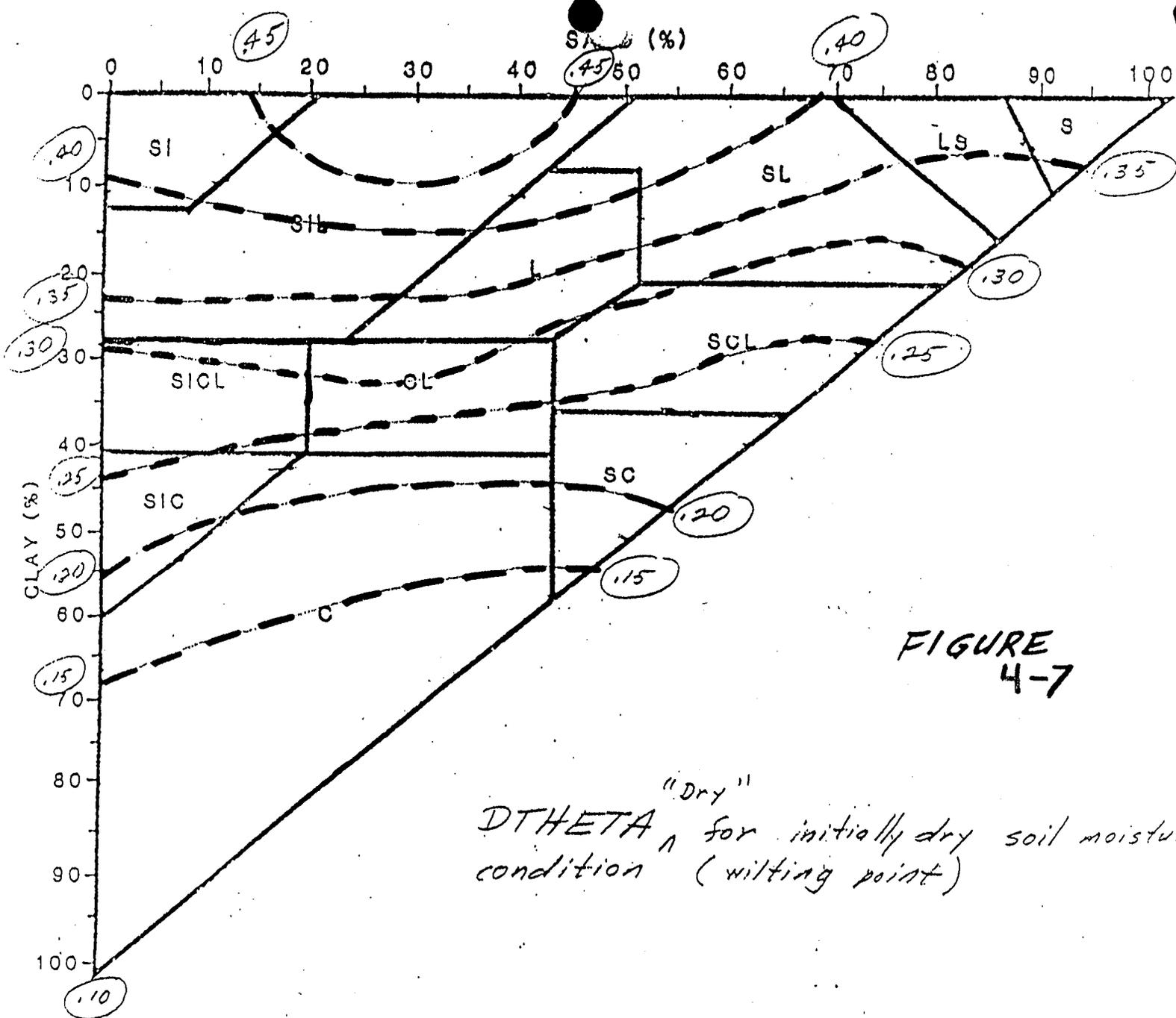


FIGURE
4-7

"Dry"
DTHETA_n for initially dry soil moisture
condition (wilting point)

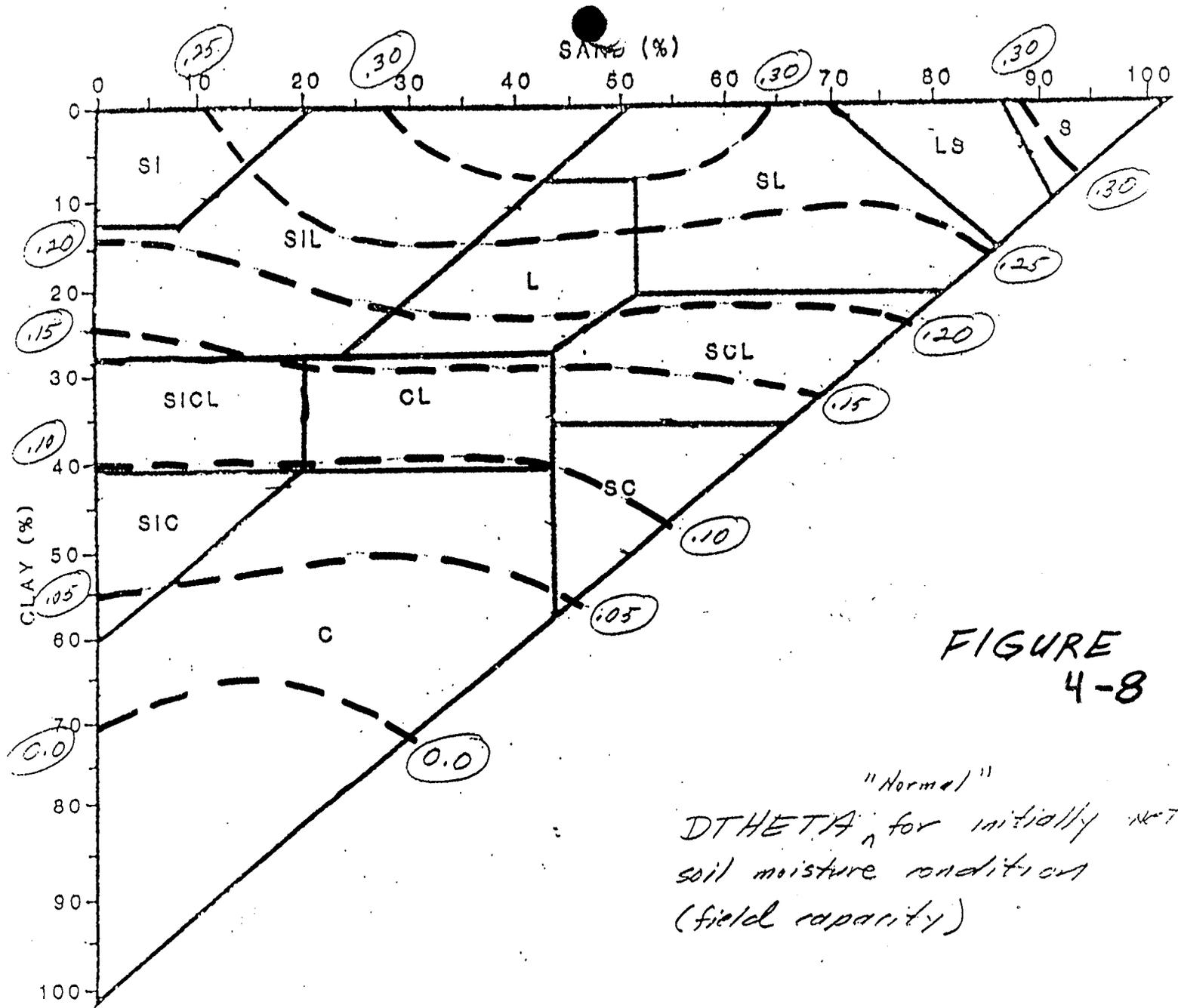


FIGURE
4-8

"Normal"
 $DTHETA_n$ for initially wet
 soil moisture conditions
 (field capacity)

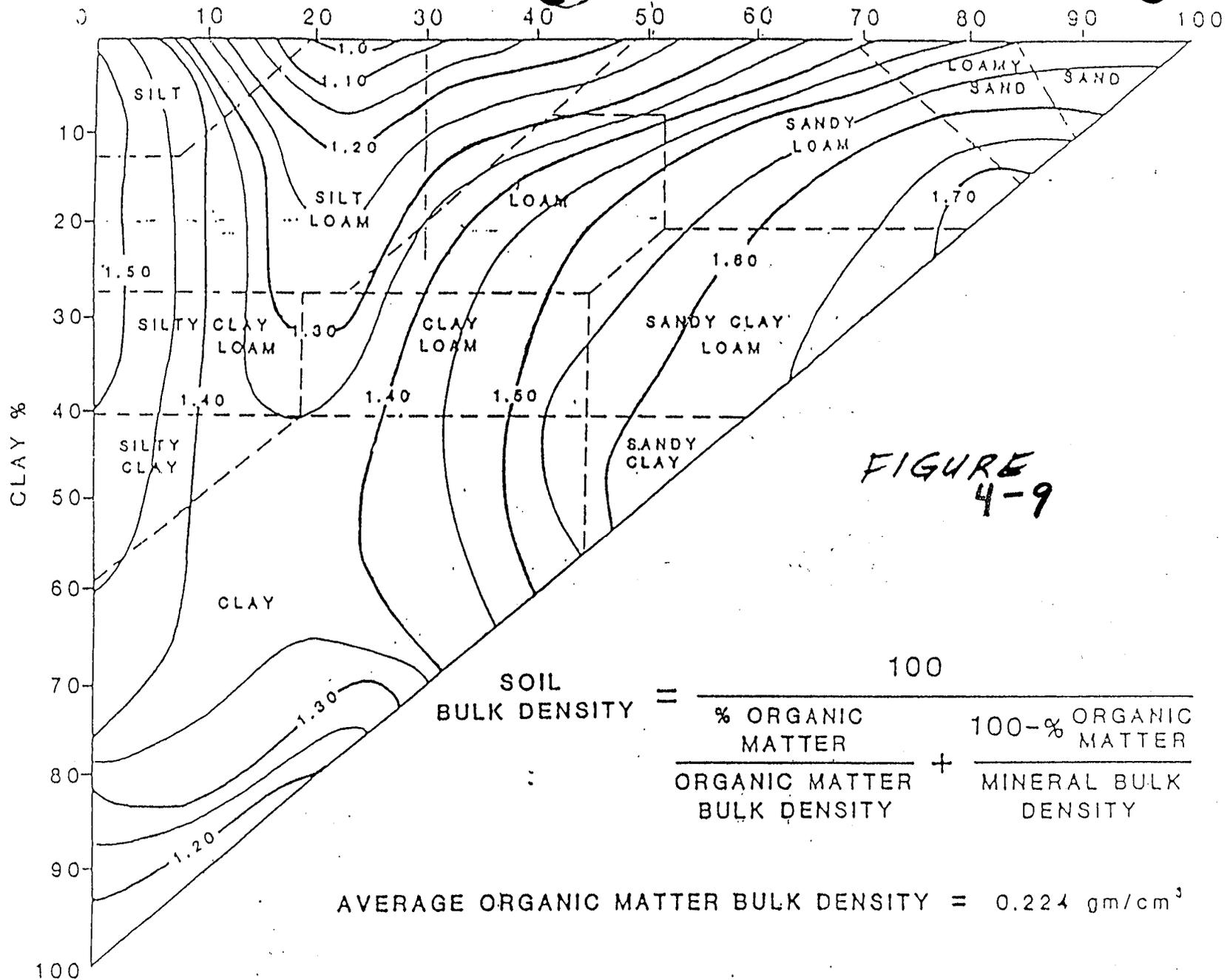


FIGURE 4-9

MINERAL BULK DENSITY (gm/cm³)

~~Fig. 1. Determination of soil bulk density~~

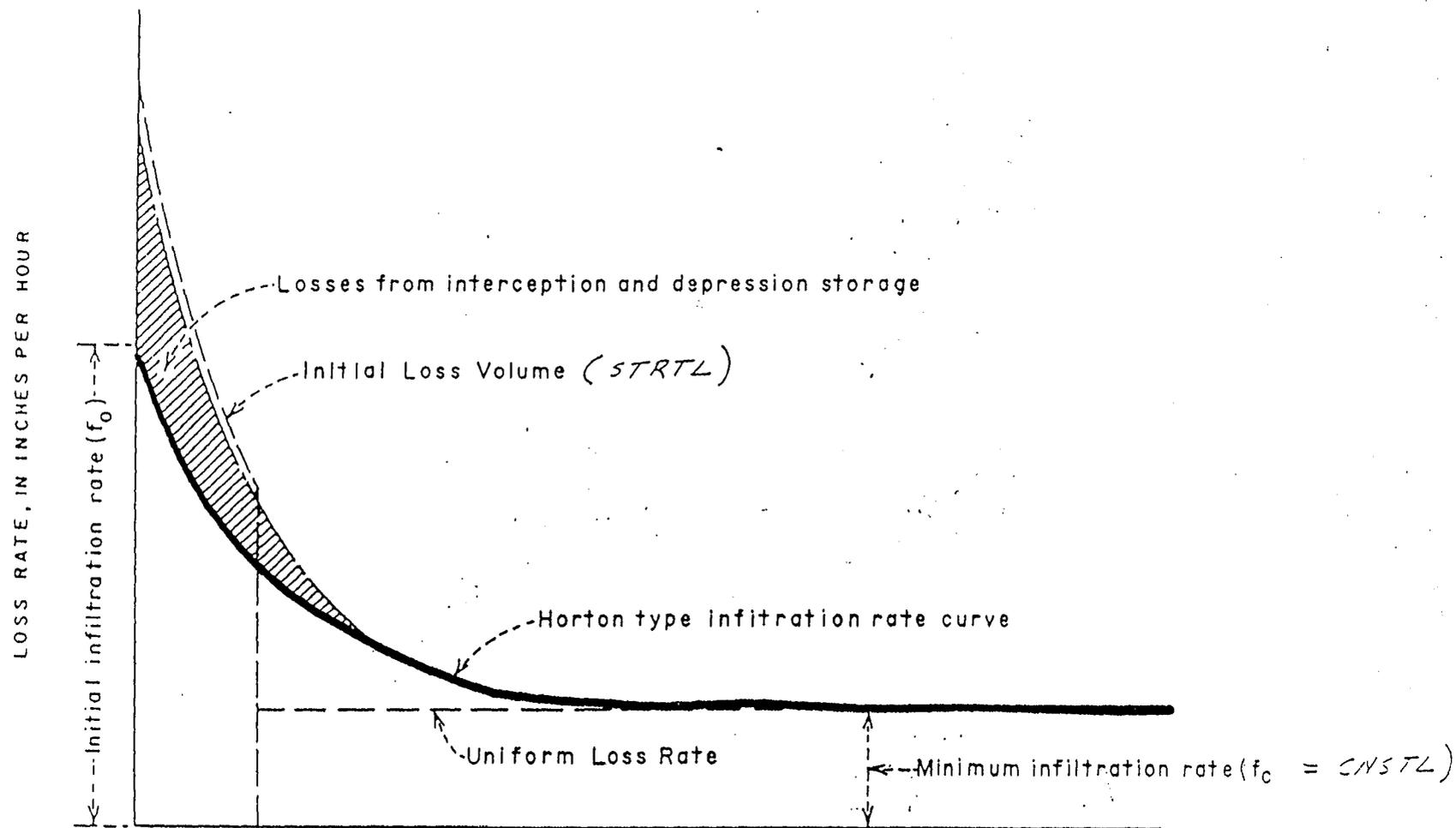


FIGURE 4-10

SIMULATION OF RAINFALL LOSSES BY THE INITIAL LOSS PLUS UNIFORM LOSS RATE METHOD

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General

PRELIMINARY
For Review Only

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, that is, 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 hydraulic routing. Hydrologic routing is usually accomplished by either direct application of the equation of continuity;

$$I - O = dS/dt \quad (1)$$

or, a graphical procedure such as the application of the principles of the unit hydrograph. 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 which were common practice prior to the ready availability of computers. 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 procedure that is recommended for routing rainfall excess in Maricopa County is either the Clark Unit Hydrograph or the application of selected S-graphs. The Clark Unit Hydrograph

procedure, as described herein, is limited to watersheds or sub-basins less than about 5 square miles in size. 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 particular time interval of the rainfall (duration). It is noted that the duration of rainfall excess is not generally equal to the rainfall duration. In that a unit hydrograph is derived from or is to be representative of a specific watershed, it is a lumped parameter and it 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). Sherman 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 an 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, $S = KQ$. However, Mitchell (1962) has shown that nonlinear storage, $S = KQ^x$, 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, however, there is no method that has been devised to evaluate the linearity of an ungaged watershed, and the assumption of linearity is a practical necessity in virtually all cases.

Clark Unit Hydrograph

Theory

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

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

$$\text{where } C = \frac{2}{2R + dt} \quad (3)$$

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, dt is the computation time interval, and R is the watershed storage coefficient. The Clark Unit Hydrograph of duration dt is obtained by averaging two instantaneous unit hydrographs spaced dt units apart

$$Q_i = 0.5(O_i + O_{i-1}) \quad (4)$$

where Q_i are the ordinates of the Clark Unit Hydrograph.

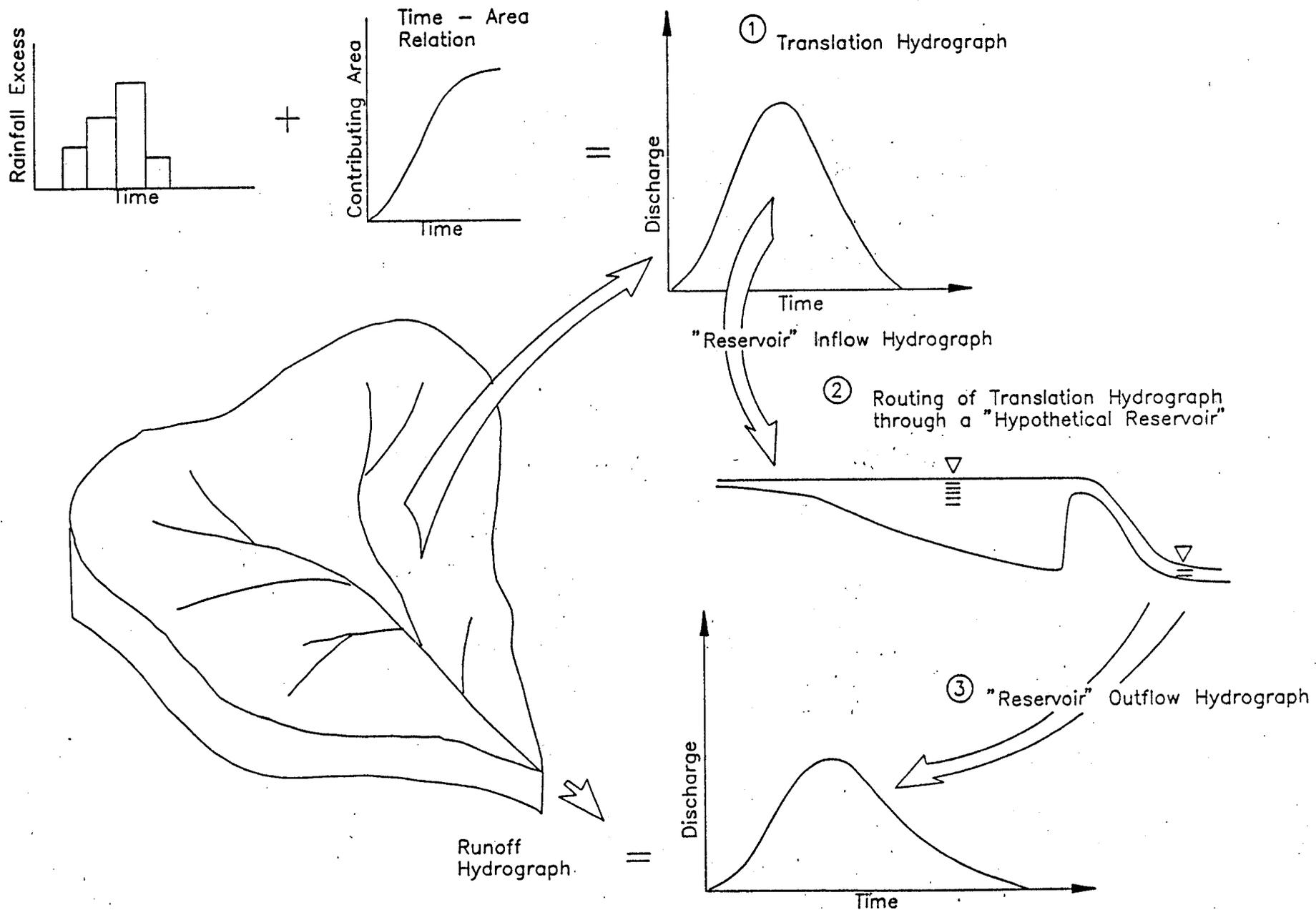


FIGURE 5.1 Conceptual analogy of linear reservoir routing to generation of a storm hydrograph by the Clark Unit-Hydrograph method

The Clark method uses two numeric parameters, T_c and R , and a graphical parameter, the time-area relation. The first parameter, time of concentration (T_c) 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 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 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

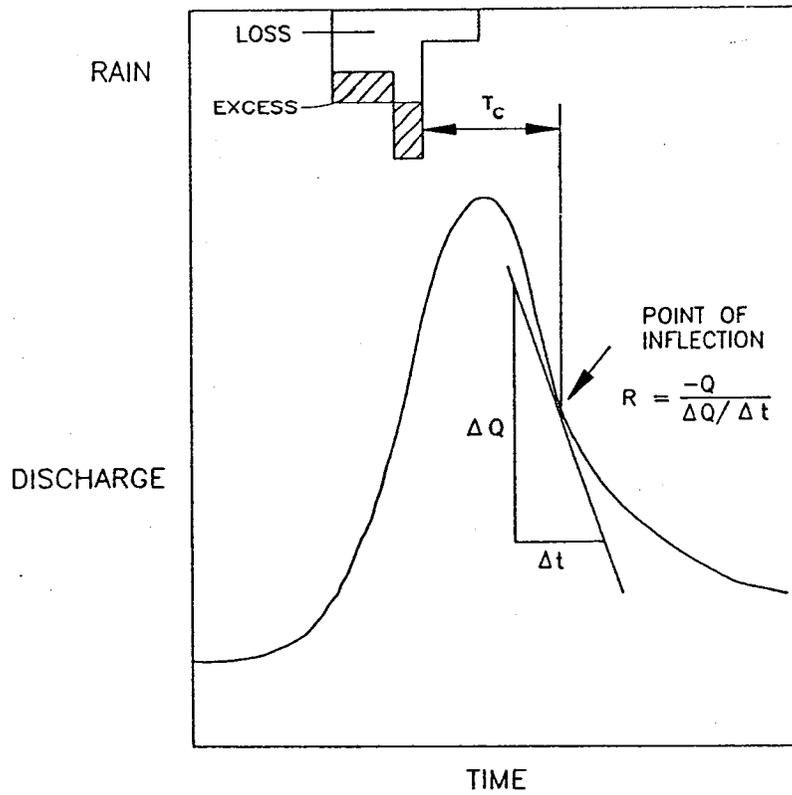


FIGURE 5.2
 Definition sketch of Clark Unit Hydrograph
 parameters from hydrograph analysis.

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 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 as 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 .

$$I_1 = (A_1 R_1) \times c/dt$$

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

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

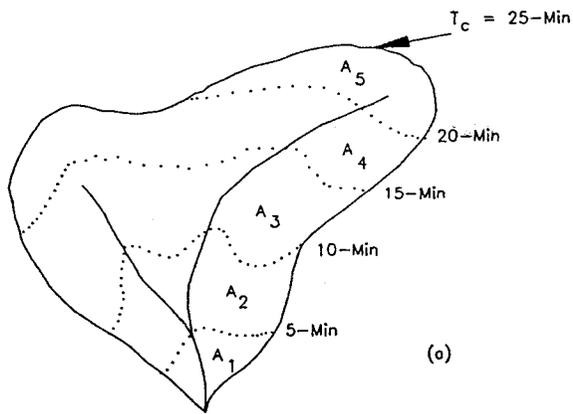
$$I_1 = 9.7 \text{ cfs}$$

At the end of 10 minutes the available runoff is

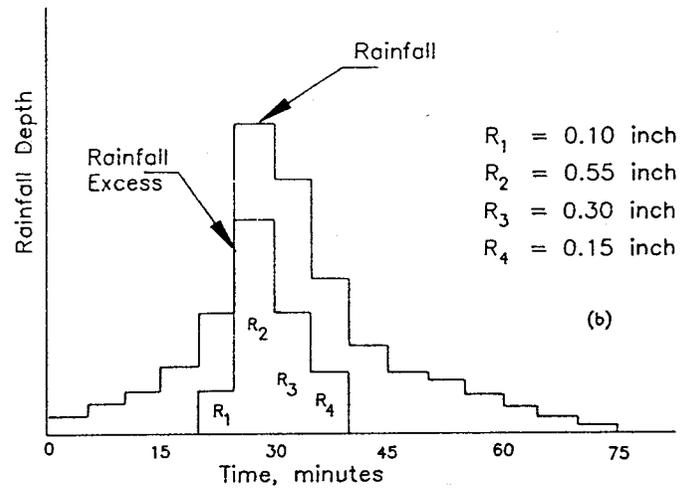
$$I_2 = (A_1 R_2 + A_2 R_1) \times c/dt$$

$$\begin{aligned} I_2 &= (8)(.55) + (24)(.10) \times 60.5/5 \\ &= 82.3 \text{ cfs} \end{aligned}$$

At the end of 15 minutes the available runoff is



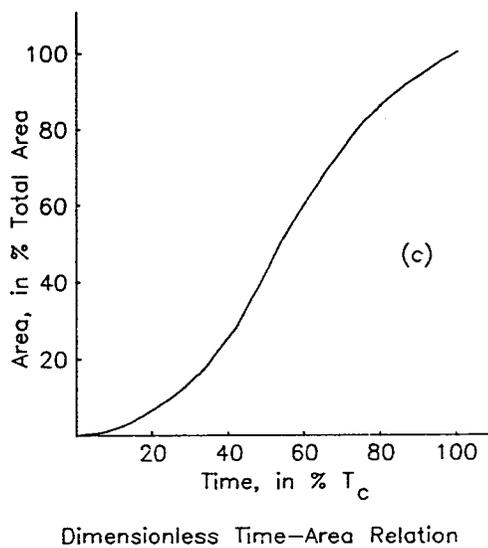
Watershed Map and Isochrones



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

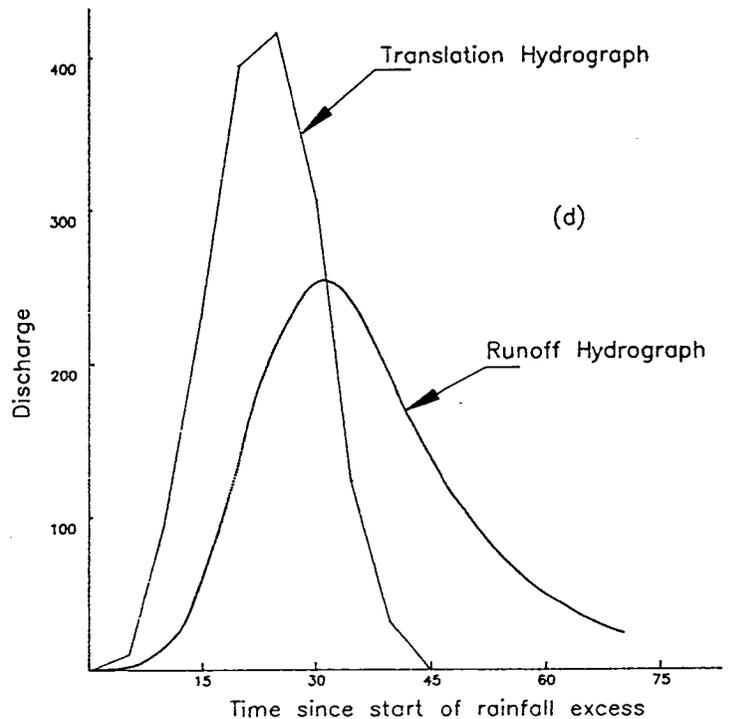


FIGURE 5.3

Example of storm hydrograph generation using the Clark Unit Hydrograph method

$$I_3 = (A_1R_3 + A_2R_2 + A_3R_1) \times c/dt$$

$$\begin{aligned} I_3 &= (8)(.30) + (24)(.55) + (38)(.10) \times 60.5/5 \\ &= 234.7 \text{ cfs} \end{aligned}$$

At the end of 20 minutes the available runoff is

$$I_4 = (A_1R_4 + A_2R_3 + A_3R_2 + A_4R_1) \times c/dt$$

$$\begin{aligned} I_4 &= (8)(.15) + (24)(.30) + (38)(.55) + (32)(.10) \times 60.5/5 \\ &= 393.5 \text{ cfs} \end{aligned}$$

At the end of 25 minutes the available runoff is

$$I_5 = (A_1R_5 + A_2R_4 + A_3R_3 + A_4R_2 + A_5R_1) \times c/dt$$

$$\begin{aligned} I_5 &= (8)(0) + (24)(.15) + (38)(.30) + (32)(.55) + (18)(.10) \times 60.5/5 \\ &= 416.2 \text{ cfs} \end{aligned}$$

Notice that all rainfall increments after $R_4 = 0$.

At the end of 30 minutes the available runoff is

$$I_6 = (A_3R_4 + A_4R_3 + A_5R_2) \times c/dt$$

$$\begin{aligned} I_6 &= (38)(.15) + (32)(.30) + (18)(.55) \times 60.5/5 \\ &= 304.9 \text{ cfs} \end{aligned}$$

At the end of 35 minutes the available runoff is

$$I_7 = (A_4R_4 + A_5R_3) \times c/dt$$

$$\begin{aligned} I_7 &= (32)(.15) + (18)(.30) \times 60.5/5 \\ &= 123.4 \text{ cfs} \end{aligned}$$

At the end of 40 minutes the available runoff is

$$I_8 = (A_5 R_4) \times c/dt$$

$$\begin{aligned} I_8 &= (18)(.15) \times 60.5/5 \\ &= 32.7 \text{ cfs} \end{aligned}$$

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 (I_i) 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 3(d). Notice that the Clark Unit Hydrograph itself was never really developed but that the three principles of the unit hydrograph were applied directly (mathematically) without having to perform 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.

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 limited to 5 square miles or less. 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.

Development of Parameter Estimators

The procedures for parameter estimation are based on available literature, research results, and analysis of original data. For example, the T_c 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.

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 such procedures.

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, T_C , has been adopted with some procedural modifications from Papadakis and Kazan (1987)

$$T_C = 11.4 L^{.50} n^{.52} s^{-.31} i^{-.38} \quad (5)$$

where T_C is in hours,
 L is length of the flow path for T_C , in miles,
 n is a representative watershed resistance coefficient,
 s is watercourse slope, in feet/mile, and
 i is the average rainfall excess intensity, during the time T_C , in inches/hour.

The selection of a representative watershed resistance coefficient, " n ," 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 for the selection of " n ." This graph shown in Figure 5.4 provides for the selection of " n " based on watershed classification and watershed size. Interpolation can be used for a given watershed size and mixed classification. Equations for estimating " n " are given in Table 1.

TABLE 1

Equation for estimating " n " in the T_C equation.

$$n = m \log A + b$$

where A is drainage area, in acres

| Land Classification (1) | Equation Parameters | |
|--|---------------------------------------|--------------------|
| | m (2) | b (3) |
| Urban | .0025 \therefore .00625 | .02 .04 |
| Bare or nearly bare ground (alluvial fan, agricultural land, desert rangeland) | .00625 \therefore .01375 | .04 .08 |
| Rough and/or moderate vegetation (hillslopes) | .01375 \therefore .025 | .08 .15 |
| Very rough and/or dense vegetation (mountains) | .025 \therefore .030 | .15 .20 |

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 T_C 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 T_C is reasonably close to the value that was assumed for "i" then the solution is finished; if not, then assume a new value of T_C , recalculate "i," and recalculate T_C with Equation 5. The solution for T_C should converge within three trials.

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 T_C and R since they can both be defined by locating the inflection point of a runoff hydrograph (Figure 5.2). The Corps of

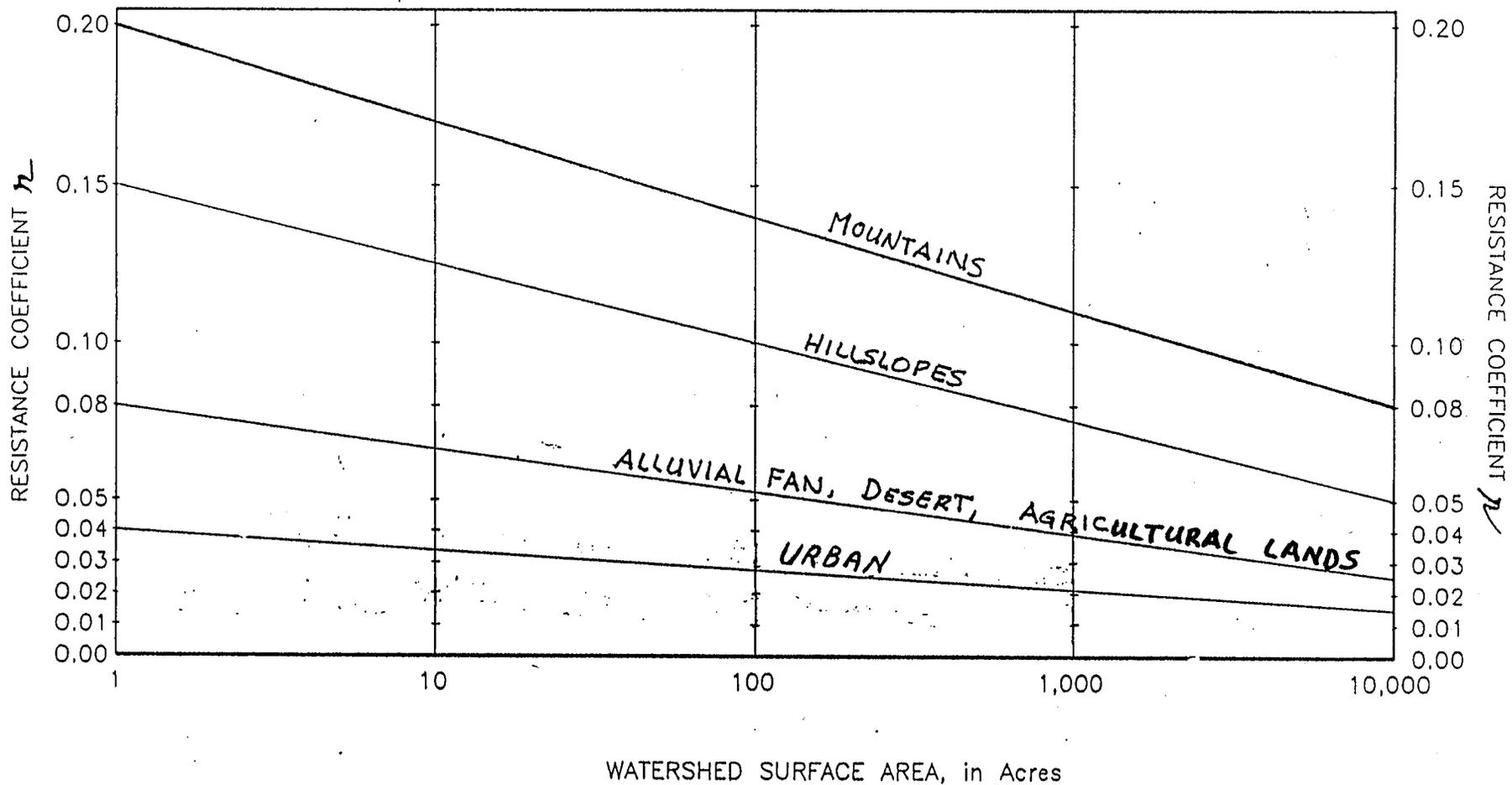


FIGURE B.4
Resistance coefficient " n " as a function of watershed type and size

Engineers has discussed the development of regionalized relations for T_C and R as functions of watershed characteristics in Training Document No. 15 (U.S. Army Corps of Engineers, 1982b). According to Corps procedures, T_C and R are estimated from relations of $T_C + R$ and $R/(T_C + R)$ as functions of watershed characteristics. These forms of empirical equations indicate an interrelation of T_C 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

$$R = 0.37 T_C^{1.11} A^{-0.57} L^{0.80} \quad (6)$$

where R is in hours,

T_C is time of concentration, in hours,

A is drainage area, in square miles, and

L is length of flow path, in miles.

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. First, using a topographic map of the watershed, the distance from the hydraulically most distant point in the watershed is traced along the flow path to the outflow location; this defines L in both Equations 5 and 6. Isochrones are drawn on the map that represent equal travel time 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. Next, the incremental areas are measured and tabulated in an upstream sequence along with the corresponding travel time for each area. A graph is prepared of travel time versus contributing area, or a dimensionless graph can be prepared of time as a percent of T_C

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 T_C .

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

$$\begin{aligned} A^* &= 1.414(T^*)^{1.5} & 0 & T & 0.5 & (7) \\ 1-A^* &= 1.414(1-T^*)^{1.5} & .5 & T & 1.0 \end{aligned}$$

where A is contributing area, in percent of total area, and T^* is time, in percent of T_C .

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 and 6, respectively. Each of these figures show a synthetic time-area relation and a shaded zone where the time-area relation is expected to lie. It is recommended that for an urban watershed that the synthetic time-area relation of Figure 5 be used, and for a natural (undeveloped) watershed that the synthetic time-area relation of Figure 6 be used. 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 or 6. 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 2.

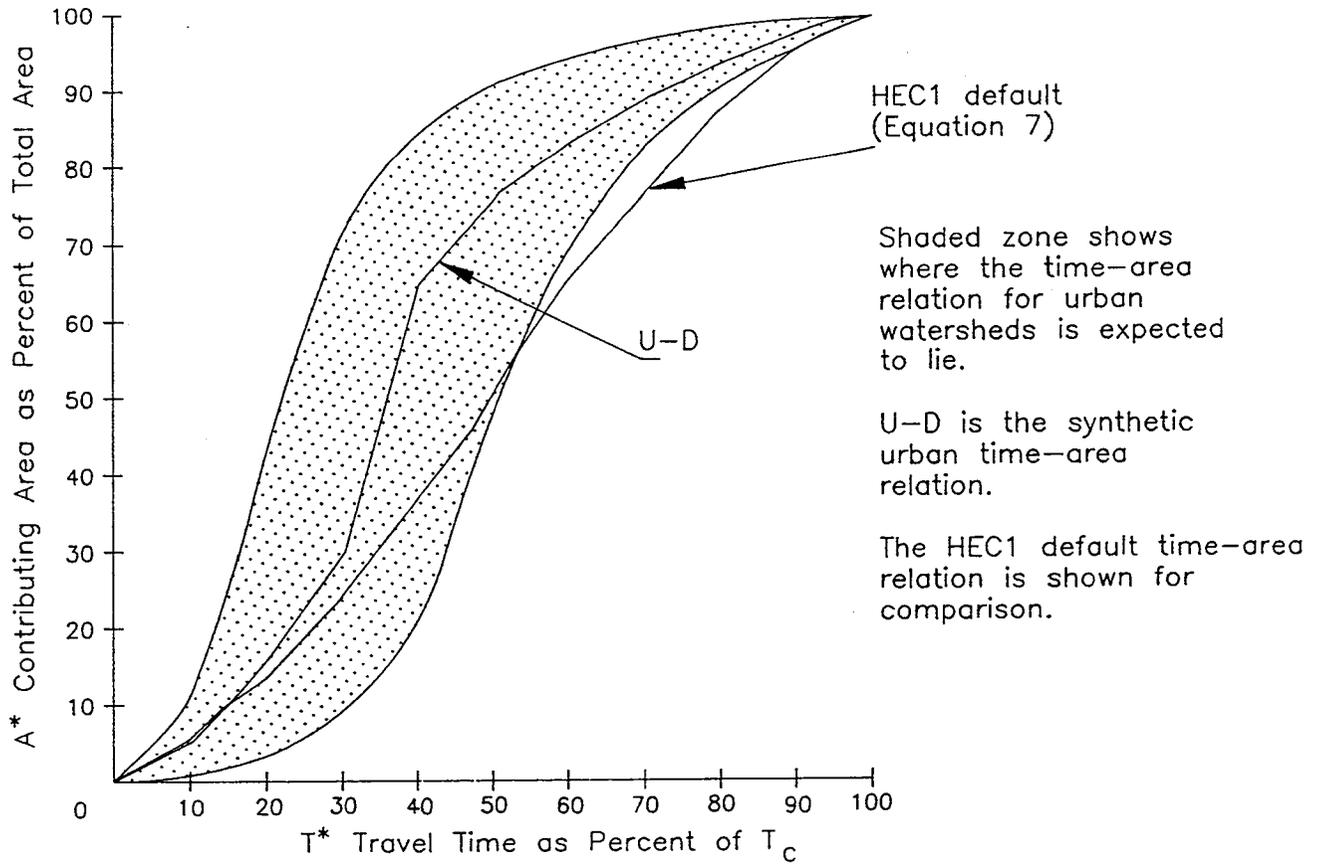
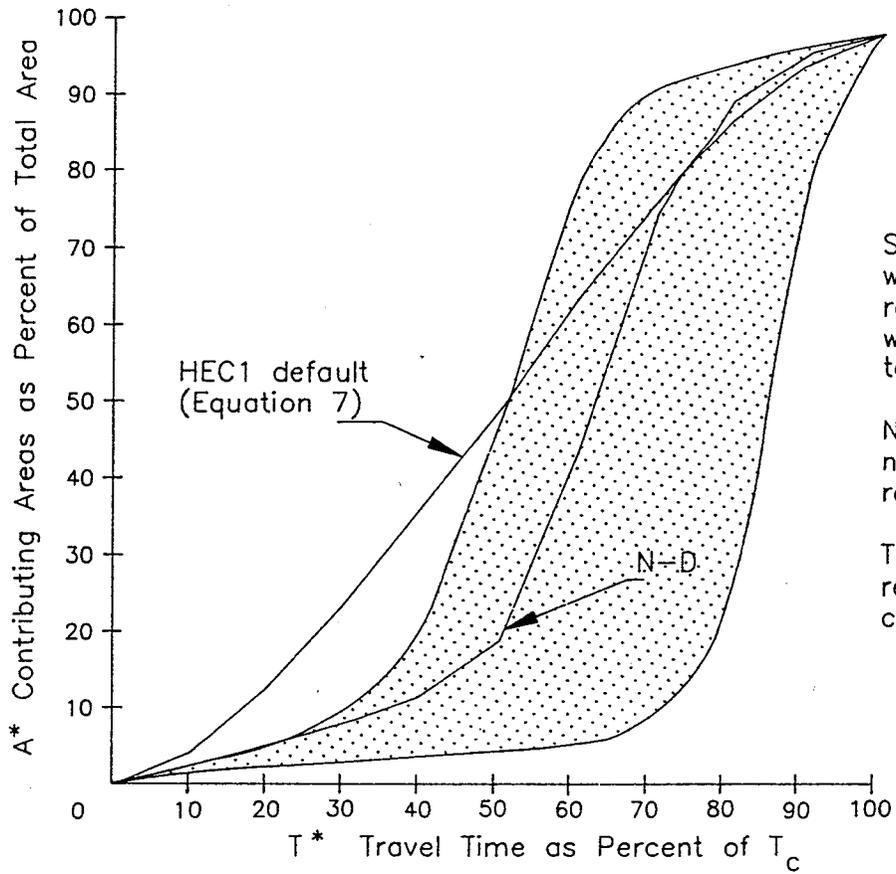


FIGURE 5.5
 Synthetic Time - Area Relation for Urban Watershed



Shaded zone shows where the time-area relation for natural watersheds is expected to lie.

N-D is the synthetic natural time-area relation.

The HEC1 default time-area relation is shown for comparison.

FIGURE 5.6
Synthetic Time - Area Relation for Natural Watersheds

TABLE 2

Values of the synthetic dimensionless time-area relations for the Clark Unit Hydrograph

| Time as a Percent of Time of Concentration | Contributing Area as a Percent of Total Area | | |
|--|---|------------------------------|-------------------------|
| | Urban Watersheds (2) | Natural Watersheds (3) | HEC-1 Default (4) |
| (1) | | | |
| 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 |

S-Graphs

Theory

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, (referred to as the Los Angeles District), and the U.S. Bureau of Reclamation (USBR).

An example of an S-graph is shown in Figure 5.7. The discharge scale is expressed as percent of ultimate discharge (Q_{ult}), 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

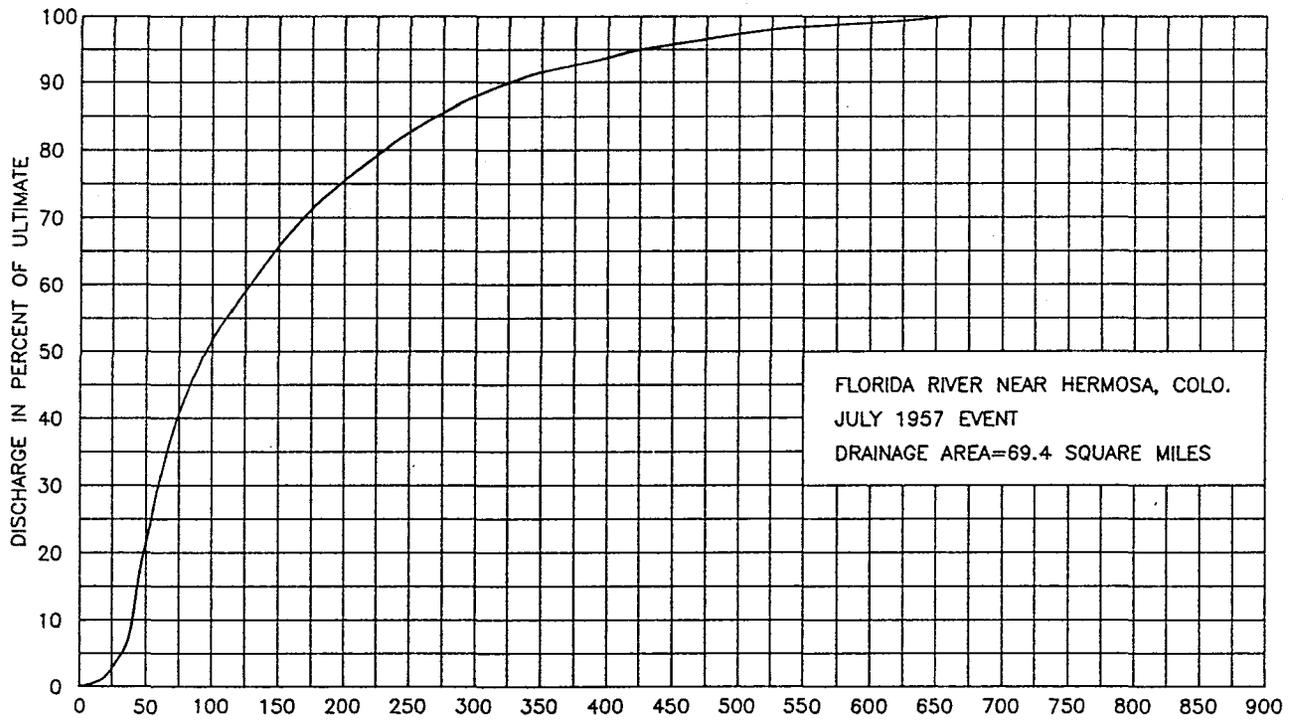


FIGURE 5.7
 Example of S-Graph from Design of Small Dams (USBR, 1987)

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 (D). 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 will be achieved from a particular watershed when subjected to a continuous intensity of rainfall excess of 1 inch per duration (D) uniformly over the basin. Ultimate discharge (Q_{ult}), in cubic feet per second (cfs), can be calculated from Equation 1

$$Q_{ult} = \frac{645.33 A}{D} \quad (8)$$

where A is drainage area, in square miles, and

D is 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. 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 Q_{ult} 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 Q_{ult} . This summation graph is then converted to a dimensionless S-graph by dividing the discharge scale by Q_{ult} 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 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.

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 the Third Edition of Design of Small Dams (USBR, 1987), and it has identified six S-graphs for application in generalized regional and physiographic type of watersheds. The flood hydrology procedure in Design of Small Dams should be reviewed and used as a reference when performing drainage studies for major watersheds in Maricopa County. Recently 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 a 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 and an implementation guide for the microcomputer version of LAPRE1 is contained in Appendix A.

An example of the manual conversion of an S-graph to an unit hydrograph follows.

Example: Conversion of an S-graph to a unit hydrograph

(To be supplied.)

Although an S-graph is completely dimensionless and does not have a rainfall excess duration associated with it, an unit hydrograph does require the specification of a duration. As a general rule, the duration selected for the development of the unit hydrograph should equal about 0.15 times the lag. A duration in the range 0.10 to 0.25 times the lag is usually acceptable.

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 source of S-graphs for that compilation was reports and file data of the 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.

S-Graphs for Use in Maricopa County

Two 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. For example, this S-graph 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. This S-graph 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, Skunk

Creek, Cave Creek, and urbanized watersheds.

These two S-graphs are shown in Figures 5.8 and 5.9, and the coordinates of the graphs listed in Table 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).

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 9

$$\text{Lag} = C \frac{L L_{ca}}{S^p}{}^m \quad (9)$$

where Lag is basin lag, in hours,

L is length of the longest watercourse, in miles,

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

S is watercourse slope, in feet per mile,

C is a coefficient, and

m and p are exponents.

The Los Angeles District often uses $C = 20n$ where "n" is the estimated mean Manning's "n" for all the channels within an area, and $m = 0.38$. The USBR (1987) has recommended that $C = 26n$ and $m = 0.33$. Both sets of values in Equation 9 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 "n" is a measure of the hydraulic efficiency of the watershed and it is not necessarily a constant

TABLE 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 |
| 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 |

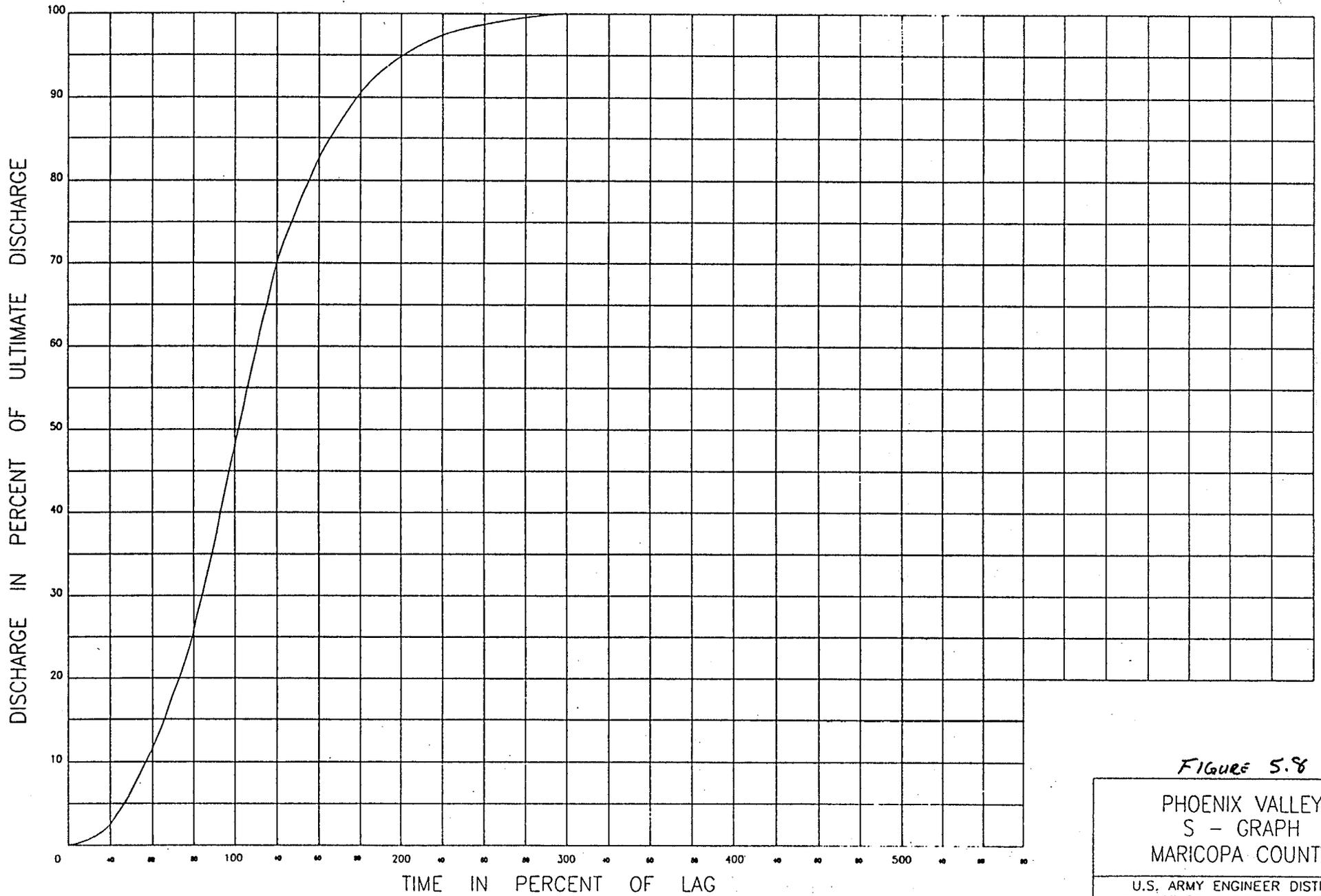


FIGURE 5.8

PHOENIX VALLEY
 S - GRAPH
 MARICOPA COUNTY

U.S. ARMY ENGINEER DISTRICT
 LOS ANGELES, CORPS OF ENGINEERS

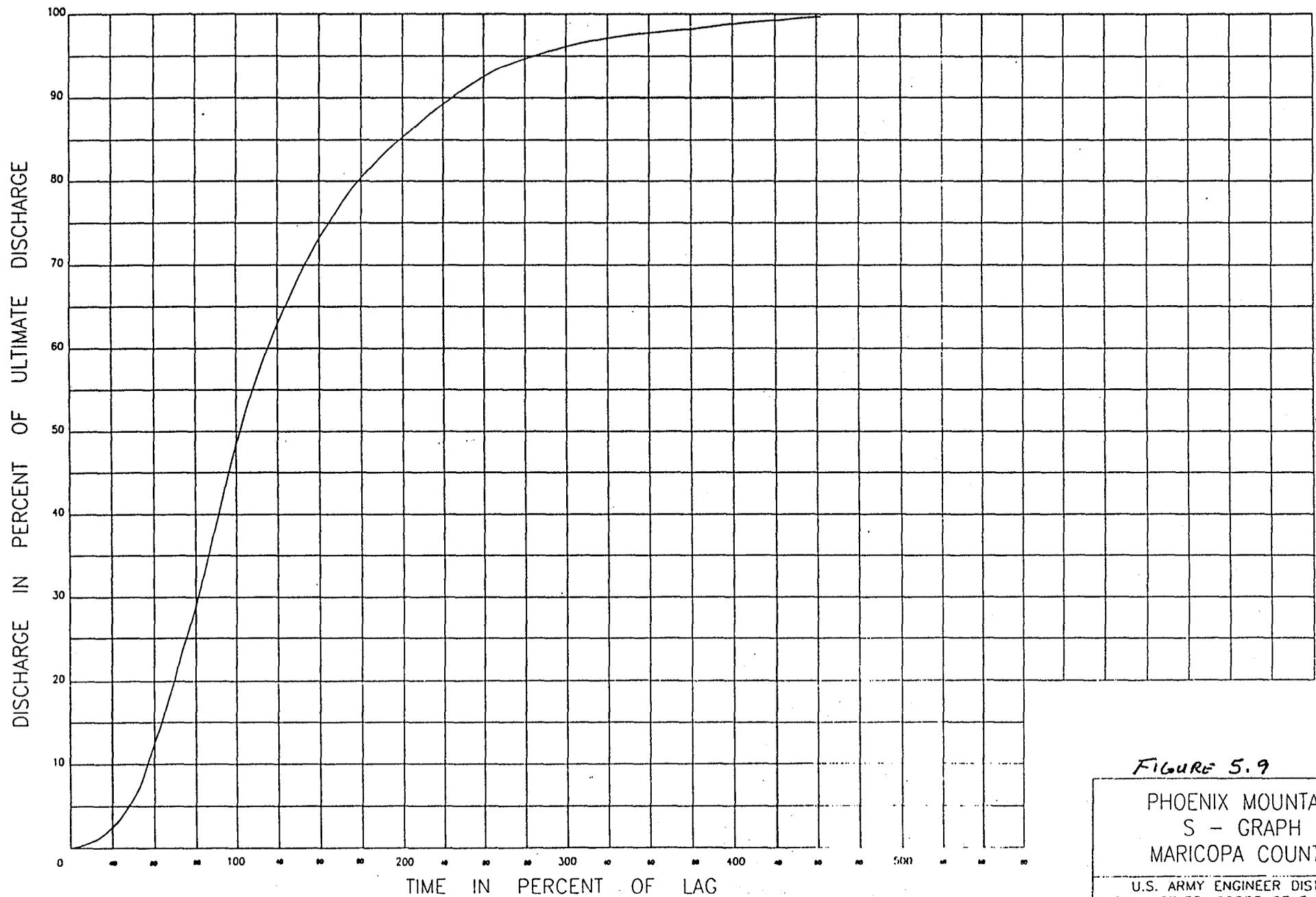


Figure 5.9

PHOENIX MOUNTAIN
S - GRAPH
MARICOPA COUNTY

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

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 "n" decreases. Therefore, some adjustment in "n" should be made for use with rainfalls of different magnitudes (frequencies). Generally, "n" 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 5.10. Several other relations that should be consulted when using S-graphs are contained in Design of Small Dams (USBR, 1987).

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 ($L L_{ca}/S^{0.5}$).
3. Use data in Figure 5.10 and the tables in Design of Small Dams (USBR, 1987) 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 "n" for those watersheds, and the measured lag.
4. Estimate "n" for the watershed by inspection of the tabulation from 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 "n".
6. Compare the calculated lag with the measured lag for similar watersheds from step 3.

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

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

$\bar{n} = 0.200$: DRAINAGE AREA 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, EXCLUDING 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.

| | CONTRIBUTING AREA | L | L _{ca} | S | LAG | ESTIMATED \bar{n} |
|---|-------------------|-------|-----------------|---------|-------|---------------------|
| | SQ. MI. | MILES | MILES | FT./MI. | HOURS | |
| 1. SAN GABRIEL RIVER AT SAN GABRIEL DAM, CA | 182.0 | 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 | 4.4 | 440 | 1.5 | .050 |
| 5. EATON WASH AT EATON WASH DAM, CA | 9.5 | 7.3 | 3.0 | 800 | 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 | 28.0 | 11.3 | 150 | 3.7 | .050 |
| 9. SANTA MARGARITA RIVER NEAR FALLBROOK, CA | 845.0 | 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 ANGELES 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 | 0.8 | .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 |

TERMINOLOGY

- L = LENGTH OF LONGEST WATERCOURSE
- L_{ca} = LENGTH ALONG LONGEST WATERCOURSE, MEASURED UPSTREAM TO POINT OPPOSITE 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:

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

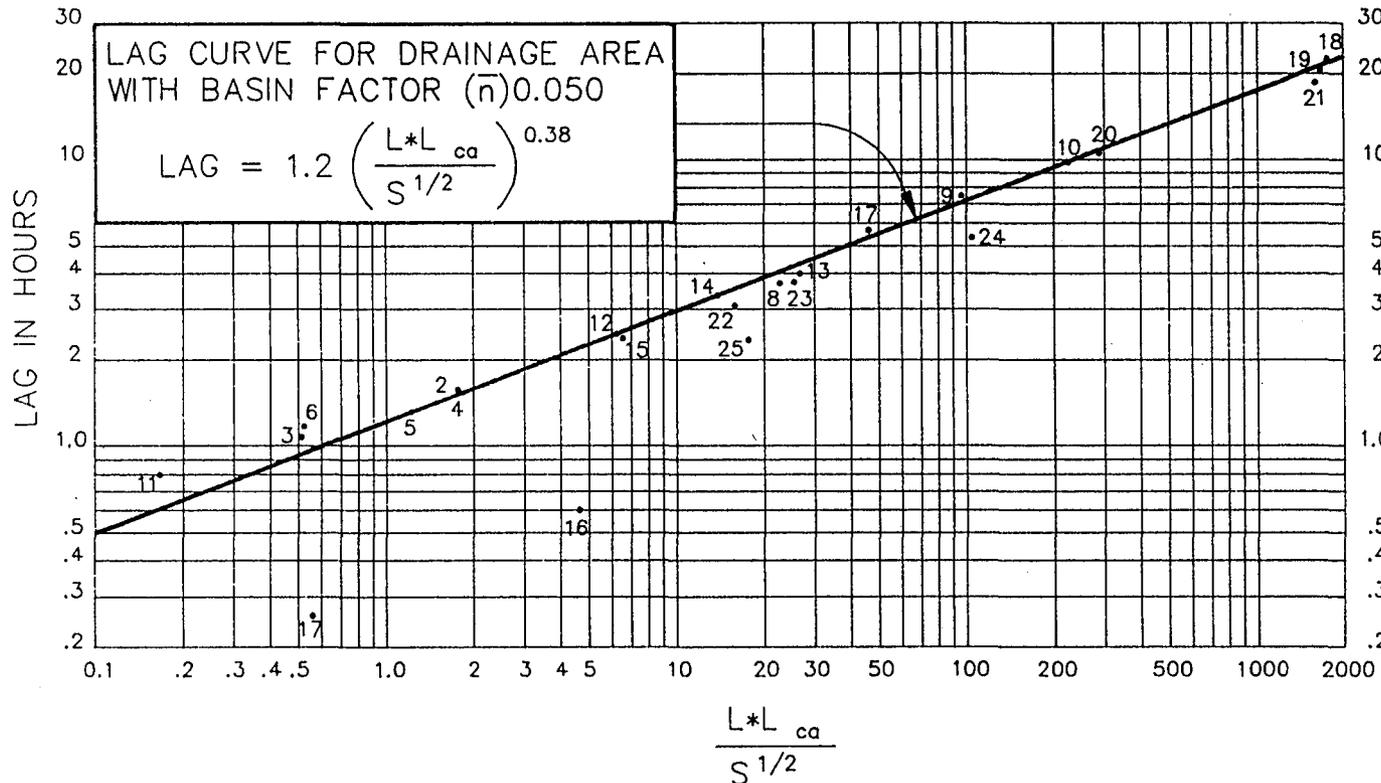


FIGURE 5.10
LAG RELATIONSHIPS
U.S. ARMY CORPS OF
ENGINEERS (1982)

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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 a certain geometrical/structural specifications, or a pipe with the characteristics of an open channel. 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 be through Kinematic Wave Routing procedures, also. Muskingum-Cunge Routing is to be used for natural, undeveloped channels since the method explains outflow peak attenuation resulting from storage loss. Both Muskingum-Cunge and Kinematic Wave Routing methods are options in HEC-1 which is again the principle modeling tool of the Hydrology Manual. The Modified puls method which is typically used for routing through a structure or a detention basin is discussed in details in the Hydraulics 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 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 since 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 avoid this problem, the reach length for a "collector channel" should be limited to one mile in length. For the "main channel" it is difficult to restrict the length. 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 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 was used.

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

APPLICATION

7.1 GENERAL

The methodologies presented in this Manual are for most parts standard procedures and practices commonly used in hydrologic design. 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 so as to facilitate a particular application.

7.2 NOTES ON THE DESIGN OF RAINFALL

Examples #1-3 illustrate the development of Depth-Duration-Frequency (D-D-F) table, Intensity-Duration-Frequency (I-D-F) table, and 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 most of Maricopa County. Chapter 2, Table 2.1, and Pattern #1 of Table 2.2 contains these distributions, which were developed from the Phoenix Airport data. If different distributions are needed, Table 2.1 and Pattern #1 of Table 2.2 should be redeveloped. Patterns #2-5 should be appropriate for all locations without modification.

A particular site might have orographic features, resulting in a 100-year, 6-hour rainfall depth, significantly different from Phoenix Airport. In such a case, the short duration rainfall part of the rainfall such as the 15-minute depth would be different from the one by Pattern #1. This will give a different peak outflow, substantiating the design of new distributions.

As a note to developing D-D-F table, the user can alternatively use FREPRE, a computer program by the National Weather Service. FREPRE 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 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 as that percentage of the soil matrix between 0.5 and 2.0 mm. 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 particles between .074mm and 3.0 inches 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 10-35% clay is listed, choose 22.5%. On rare occasions, the sum of the median values for percent sand and clay will be greater than 100%. In this case, adjust both values equally until they total 100. With a known percent sand and clay, enter Figure *** to determine the textural class for that particular soil. Then choose Green & Ampt loss parameters from Table *** or IL+ULR parameters from Table ***.

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 percentage for all (usually 2 to 4) minor soils. When calculating weighted averages for the minor soils, assume an equal contribution from each. For example, if a minor group makes up 20% of the map unit and consists of 3 soils, then each group member contributes $(20/3)=6.67\%$.

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.

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% B, 25% C, and 55% 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.

6. Textural Classes: Textural class descriptions, as used in this context, contain only adjectives from the three primary textures: sand, silt, and clay. To determine the textural class, calculate the percent sand and clay for the soil, then use Figure 4.11.

7. When using the IL+ULR loss rate methods, remember that the variable STRTL in HEC-1 is composed of two parameters: IL-the initial loss due to infiltration, and IA- the loss due to surface retention. $STRTL=IL+IA$.

8. Examples #5 and #6, and the loss rate parameters in Tables 4.2,4.3, and 4.40 are for bare ground only. In areas where surface cover and/or vegetation influences are significant, the saturated conductivity parameters (XKSAT & CNSTL) should be adjusted using procedures outlined in Section *** of Chapter *** and Example ***.

9. As an option to the methods of loss parameter calculation presented in the examples, Green & Ampt and IL+ULR (by soil texture) loss parameters have been calculated for Maricopa County soils and are presented in Tables ***, *** and ***. 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. A Note on T_c : It should be noted that T_c represents the time for water 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) directs the conveyance of water to the outlet. In the case of a natural watershed, L is the length of the main, well-defined channel, such as shown in blue on USGS maps, or that section of a channel referred to as the "Channel Flow" length on the SCS's "Worksheet 3" in TR-55 (Technical Release-55, 1986). In the case of 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, 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 decreasing order unless they are contiguous. Example:

| <u>Time</u> | <u>Excess(in)</u> | | <u>Rank</u> | | <u>Sorted</u> |
|-------------|-------------------|------|-------------|------|---------------|
| 1415 | .21 | | 6 | | .40 |
| 1420 | .28 | | 5 | | .35 |
| 1425 | .35 | | 2 | | .32 |
| 1430 | .40 | ---> | 1 | ---> | .33 |
| 1435 | .32 | | 3 | | .28 |
| 1440 | .33 | | 4 | | .21 |
| 1445 | .18 | | 7 | | .18 |

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 $\Delta t = 5$ minutes (where Δt is hydrograph time step), then Tc should be less than $(8*5)=40$ minutes. For $\Delta t = 10$ minutes, $T_c < 80$ 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 frequencies are desired for a given duration, it may be acceptable to construct a graph of Tc vs. Frequency, when the peak producing portion of the distribution is maintained. In such a case plot the 2, 10, and 100 year Tc values on semi-log paper, and interpolate intermediate values.

5. In watersheds where variations in slope are significant, the following equation may be used to determine an average slope (Simon, et al., 1986):

$$S = \left[\frac{L}{\sum_{i=1}^n \left[\frac{\Delta L_i^3}{\Delta H_i} \right]^{.5}} \right]^2$$

where

S = average slope in (ft/ft)

L = total reach length (ft)

ΔL_i = subreach length (ft)

ΔH_i = elevation difference corresponding to L_i

n = number of subreaches

To use S in the Tc equation, convert to ft/ft by multiplying by 5280.

7.5 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):

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

where K = the travel time through the entire reach (hrs),

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, 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.

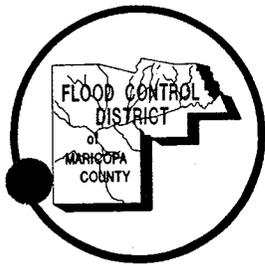
4. X: For wide, shallow channels with low to moderate slopes and significant overbank flow during the design flood being modeled, choose X = .15 to .25. For steep to very steep, narrow, deep channels with little overbank flow, choose X = .25 to .40.

REFERENCES

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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA County Hydrology Manual PAGE 1 OF 7
DETAIL EXAMPLE # 1 COMPUTED _____ DATE _____
DEPTH-DURATION-FREQ. CALC. CHECKED BY _____ DATE _____

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

LOCATION: CAREFREE AIR PORT AT T6N, R4E, SEC. 36

- ① USING PLATES 1 TO 12, FOLLOWING RAINFALL DEPTHS WERE READ.
- ② DATA WAS PLOTTED ON FIGURE 5, AND LINE OF BEST FIT DRAWN.

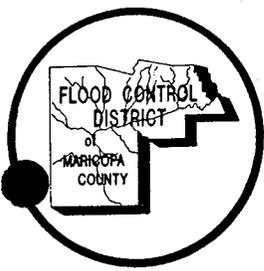
6-HOUR { 2-YEAR, 6-HOUR DEPTH = 1.55
5-YEAR, 6-HOUR DEPTH = 2.00
10-YEAR, 6-HOUR DEPTH = 2.30
25-YEAR, 6-HOUR DEPTH = ~~2.78~~ 2.60*
50-YEAR, 6-HOUR DEPTH = 3.10
100-YEAR, 6-HOUR DEPTH = 3.40

24-HOUR { 2-YEAR, 24-HOUR DEPTH = 2.00
5-YEAR, 24-HOUR DEPTH = ~~2.70~~ 2.60
10-YEAR, 24-HOUR DEPTH = ~~3.05~~ 3.10
25-YEAR, 24-HOUR DEPTH = ~~3.70~~ 3.55
50-YEAR, 24-HOUR DEPTH = 4.20
100-YEAR, 24-HOUR DEPTH = 4.70

* CORRECTED VALUE FROM FIGURE 5.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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 DETAIL EXAMPLE # 1 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____



③ THEN, OTHER DEPTHS FOR VARIOUS FREQUENCIES & DURATIONS ARE CALCULATED

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

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

WHERE: $Y_2 = 2\text{-}YR, 1\text{-}HR$

$$Y_{100} = 100\text{-}YR, 1\text{-}HR$$

$$\begin{cases} X_1 = 2\text{-}YR, 6\text{-}HR, \\ X_2 = 2\text{-}YR, 24\text{-}HR, \end{cases} \begin{cases} X_3 = 100\text{-}YR, 6\text{-}HR \\ X_4 = 100\text{-}YR, 24\text{-}HR \end{cases}$$

1-HR DEPTHS $\begin{cases} Y_2 = -.011 + .942 (1.55)(1.55/2.00) = 1.12 \\ Y_{100} = .494 + .755 (3.40)(3.40/4.70) = 2.35 \end{cases}$

④ Y_2 AND Y_{100} WILL BE PLOTTED ON FIGURE 5 AND DEPTHS FOR OTHER FREQUENCIES ARE CALCULATED:

$$\begin{cases} 5\text{-}YR, 1\text{-}HR = 1.35 \\ 10\text{-}YR, 1\text{-}HR = 1.58 \\ 25\text{-}YR, 1\text{-}HR = 1.82 \\ 50\text{-}YR, 1\text{-}HR = 2.12 \end{cases}$$



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #1 COMPUTED _____ DATE _____
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⑤ THEN 2-HR DEPTHS ARE CALCULATED:

$$100\text{-YR, 2-HR} = .341(100\text{-YR, 6-HR}) + .659(100\text{-YR, 1-HR})$$

$$\Rightarrow 100\text{-YR, 2-HR} = .341(3.40) + .659(2.35) = 2.71$$

$$2\text{-YR, 2-HR} = .341(2\text{-YR, 6-HR}) + .659(2\text{-YR, 1-HR})$$

$$\Rightarrow 2\text{-YR, 2-HR} = .341(1.55) + .659(1.12) = 1.27$$

ABOVE 2-YR AND 100-YR DEPTHS ARE PLOTTED ON FIGURE 5

AND DEPTHS FOR OTHER FREQUENCIES ARE ESTIMATED FROM GRAPH:

$$\left\{ \begin{array}{l} 5\text{-YR, 2-HR} = 1.62 \\ 10\text{-YR, 2-HR} = 1.87 \\ 25\text{-YR, 2-HR} = 2.12 \\ 50\text{-YR, 2-HR} = 2.46 \end{array} \right.$$

⑥ NEXT 3-HR DEPTHS ARE CALCULATED:

$$100\text{-YR, 3-HR} = .569(100\text{-YR, 6-HR}) + .431(100\text{-YR, 1-HR})$$

$$\Rightarrow 100\text{-YR, 3-HR} = .569(3.40) + .431(2.35) = 2.95$$

$$2\text{-YR, 3-HR} = .569(2\text{-YR, 6-HR}) + .431(2\text{-YR, 1-HR})$$

$$\Rightarrow 2\text{-YR, 3-HR} = .569(1.55) + .431(1.12) = 1.36$$



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DETAIL EXAMPLE # 1 COMPUTED _____ DATE _____
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VALUES FOR 2-YR AND 100-YR DEPTHS ARE PLOTTED ON
FIGURE 5 AND DEPTHS FOR OTHER FREQUENCIES ARE ESTIMATED
FROM GRAPH:

$$\left\{ \begin{array}{l} 5\text{-YR, } 3\text{-HR} = 1.74 \\ 10\text{-YR, } 3\text{-HR} = 2.05 \\ 25\text{-YR, } 3\text{-HR} = 2.30 \\ 50\text{-YR, } 3\text{-HR} = 2.65 \end{array} \right.$$

(7) USING FIGURE 6, DEPTHS FOR THE 12-HR DURATION ARE FOUND.
THIS IS DONE BY DRAWING A STRAIGHT LINE BETWEEN 6-HR AND
24-HR DEPTHS FOR ALL FREQUENCIES:

$$2\text{-YR, } 12\text{-HR} = 1.80$$

$$5\text{-YR, } 12\text{-HR} = 2.30$$

$$10\text{-YR, } 12\text{-HR} = 2.70$$

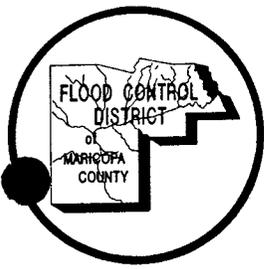
$$25\text{-YR, } 12\text{-HR} = 3.10$$

$$50\text{-YR, } 12\text{-HR} = 3.65$$

$$100\text{-YR, } 12\text{-HR} = 4.10$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE # 1 COMPUTED _____ DATE _____
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⑧ 30-MIN DEPTHS ARE ESTIMATED FROM:

$$(2\text{-YR}, 30\text{-MIN}) = .82(2\text{-YR}, 1\text{-HR}) = .82(1.12) = .92$$

$$(100\text{-YR}, 30\text{-MIN}) = .82(100\text{-YR}, 1\text{-HR}) = .82(2.35) = 1.93$$

OTHER DEPTHS ARE ESTIMATED BY PLOTTING ABOVE POINTS ON
FIGURE 5 AND READING THE VALUES FOR CORRESPONDING FREQUENCIES

$$\left\{ \begin{array}{l} 5\text{-YR}, 30\text{-MIN} = 1.15 \\ 10\text{-YR}, 30\text{-MIN} = 1.35 \\ 25\text{-YR}, 30\text{-MIN} = 1.50 \\ 50\text{-YR}, 30\text{-MIN} = 1.75 \end{array} \right.$$

⑨ 15-MIN DEPTHS ARE ESTIMATED FROM:

$$(2\text{-YR}, 15\text{-MIN}) = .62(2\text{-YR}, 1\text{-HR}) = .62(1.12) = .69$$

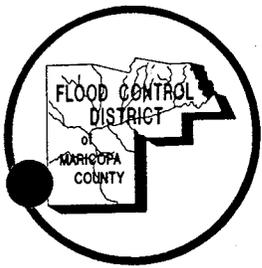
$$(100\text{-YR}, 15\text{-MIN}) = .62(100\text{-YR}, 1\text{-HR}) = .62(2.35) = 1.46$$

OTHER VALUES ARE ESTIMATED FROM FIGURE 5 AS BEFORE:

$$\left\{ \begin{array}{l} 5\text{-YR}, 15\text{-MIN} = .87 \\ 10\text{-YR}, 15\text{-MIN} = 1.00 \\ 25\text{-YR}, 15\text{-MIN} = 1.15 \\ 50\text{-YR}, 15\text{-MIN} = 1.28 \end{array} \right.$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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⑩ 10-MIN VALUES

$$(2\text{-YR}, 10\text{-MIN}) = .51 (2\text{-YR}, 1\text{-HR}) = .51 (1.12) = .57$$

$$(100\text{-YR}, 10\text{-MIN}) = .51 (100\text{-YR}, 1\text{-HR}) = .51 (2.35) = 1.20$$

OTHER VALUES FROM FIGURE 5:

$$\left\{ \begin{array}{l} 5\text{-YR}, 10\text{-MIN} = .70 \\ 10\text{-YR}, 10\text{-MIN} = .83 \\ 25\text{-YR}, 10\text{-MIN} = .92 \\ 50\text{-YR}, 10\text{-MIN} = 1.10 \end{array} \right.$$

⑪ 5-MIN VALUES:

$$(2\text{-YR}, 5\text{-MIN}) = .34 (2\text{-YR}, 5\text{-MIN}) = .34 (1.12) = .38$$

$$(100\text{-YR}, 5\text{-MIN}) = .34 (100\text{-YR}, 5\text{-MIN}) = .34 (2.35) = .80$$

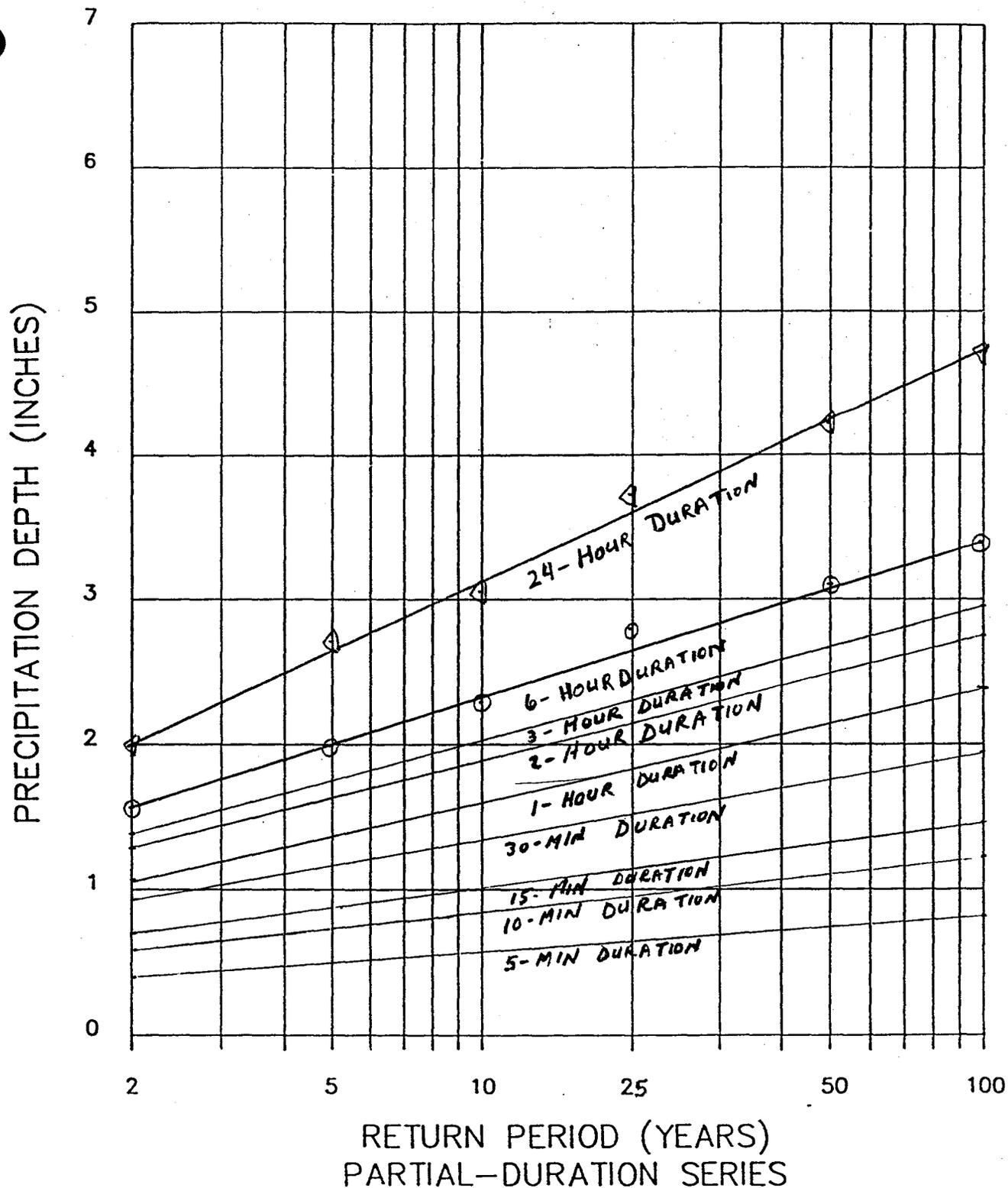
OTHER VALUES FROM FIGURE 5:

$$5\text{-YR}, 5\text{-MIN} = .50$$

$$10\text{-YR}, 5\text{-MIN} = .58$$

$$25\text{-YR}, 5\text{-MIN} = .62$$

$$50\text{-YR}, 5\text{-MIN} = .73$$



FIGURE— 5 PRECIPITATION DEPTH VERSUS RETURN PERIOD FOR PARTIAL-DURATION SERIES

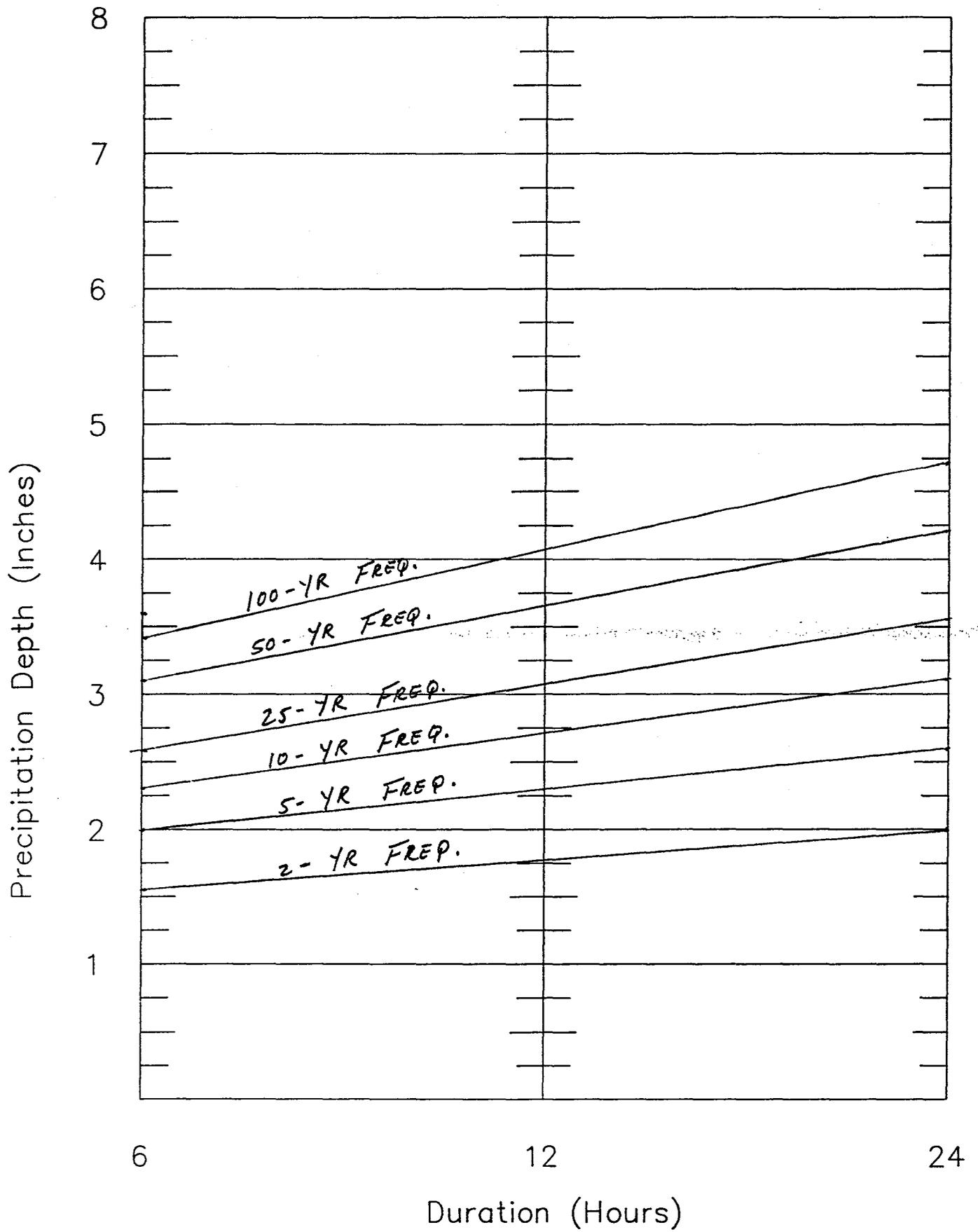
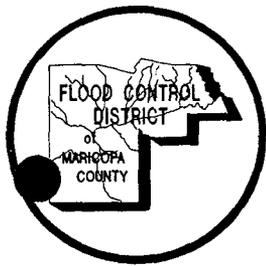


Figure 6 Precipitation depth-duration diagram (6 to 24 hr).



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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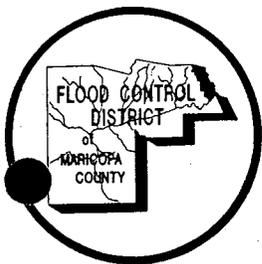
DEPTH-DURATION-FREQUENCY IS NOW PUT TOGETHER:

CARE FREE AIR PORT, T6W, R4E, SEC. 36

FREQUENCY

| | 2-YR | 5-YR | 10-YR | 25-YR | 50-YR | 100-YR |
|---------|------|------|-------|-------|-------|--------|
| 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 | 1.40 |
| 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 |

DURATION



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE # 2 COMPUTED _____ DATE _____
INTENSITY-DURATION-FREQ. CALC. CHECKED BY _____ DATE _____

EXAMPLE # 2, DEVELOPMENT OF INTENSITY-DURATION-FREQUENCY TABLE AND GRAPH

LOCATION: CARE FREE AIR PORT AT TGN, R4E, SEC. 36

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

THE CALCULATED DEPTHS WILL BE CONVERTED TO INTENSITY (IN/HR):

5-MIN DURATION DEPTHS:

$$2\text{-YR} \rightarrow \frac{.38 \text{ (INCHES)}}{5\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = 4.56 \text{ IN/HR}$$

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

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

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

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

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

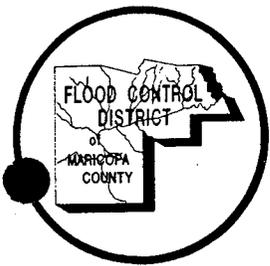
10-MIN DURATION DEPTHS:

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

$$5\text{-YR} \rightarrow \text{SAME AS ABOVE} = 4.20 \text{ IN/HR}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #2 COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____



OTHER VALUES ARE CALCULATED SIMILARLY.

15-MIN DURATION DEPTHS:

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

OTHER VALUES FOLLOW THE SAME PROCEDURE.

30-MIN DURATION DEPTHS:

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

OTHER VALUES FOLLOW THE SAME PROCEDURE.

1-Hour DURATION DEPTHS:

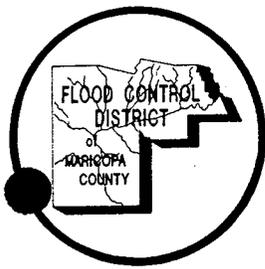
$$2\text{-YR} \rightarrow \frac{1.12(\text{IN})}{60\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = 1.12 \text{ IN/HR}$$

OTHER VALUES FOLLOW THE SAME PROCEDURE.

2-Hour DURATION DEPTHS:

$$2\text{-YR} \rightarrow \frac{1.27(\text{IN})}{120\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = .64 \text{ IN/HR}$$

OTHER VALUES FOLLOW THE SAME PROCEDURE.



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #2 COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____

3-Hour DURATION DEPTHS:

$$2\text{-YR} \rightarrow \frac{1.36 \text{ (IN)}}{180\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = .45 \text{ IN/HR}$$

6-Hour DURATION DEPTHS:

$$2\text{-YR} \rightarrow \frac{1.55 \text{ (IN)}}{360\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = .26 \text{ IN/HR}$$

12-Hour DURATION DEPTHS:

$$2\text{-YR} \rightarrow \frac{1.80 \text{ (IN)}}{720\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = .15 \text{ IN/HR}$$

24-Hour DURATION DEPTHS:

$$2\text{-YR} \rightarrow \frac{2.00 \text{ (IN)}}{1440\text{-MIN}} \times \frac{60\text{-MIN}}{\text{HR}} = .08 \text{ IN/HR}$$

VALUES FOR OTHER
FREQUENCIES ARE
FOUND SIMILARLY.



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

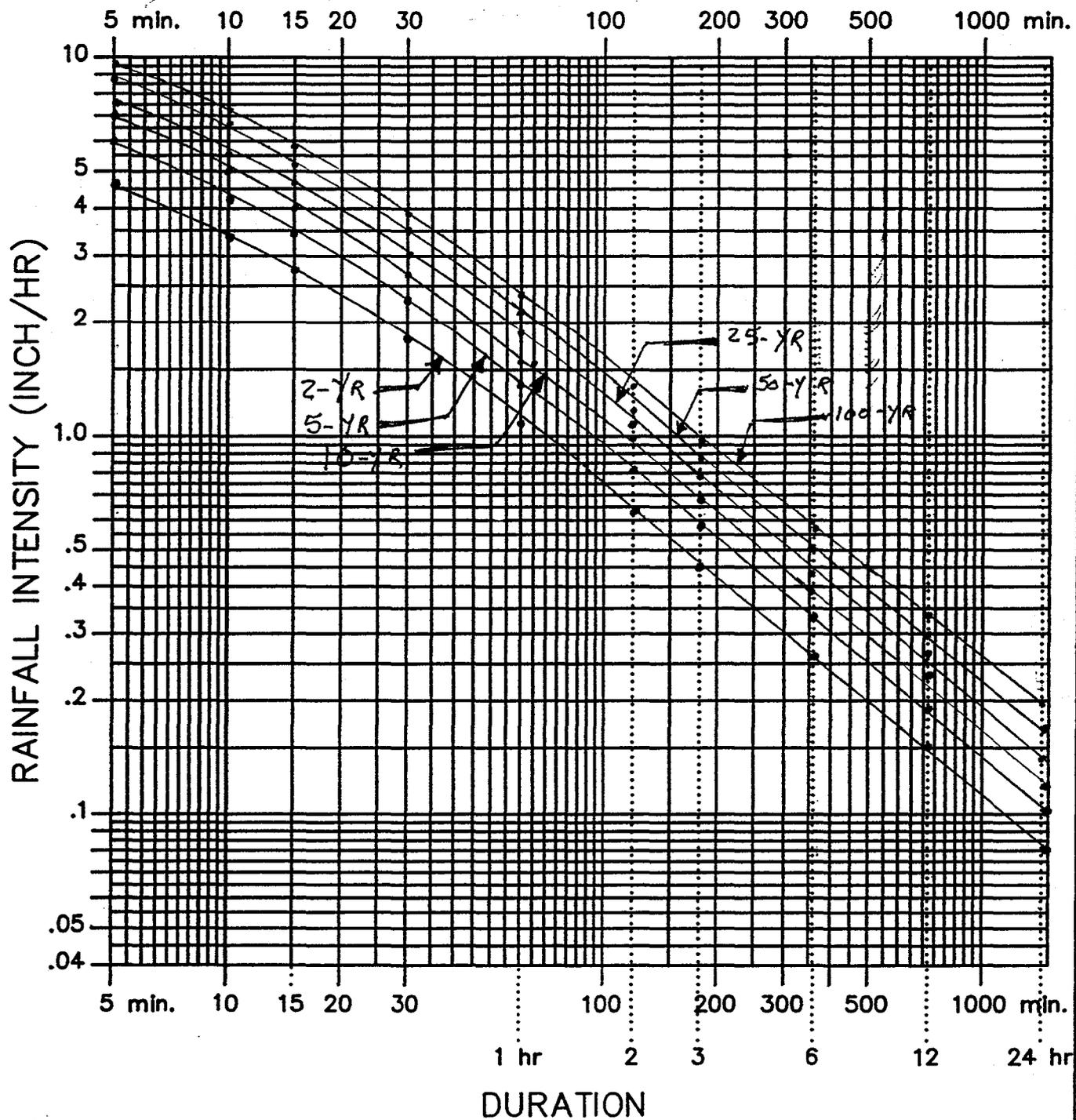
PROJECT Maricopa County Hydrology Manual PAGE 4 OF 4
 DETAIL EXAMPLE #1 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____

THE INTENSITY-DURATION-FREQUENCY TABLE IS NOW PUT TOGETHER.

FREQUENCY

| | 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 |

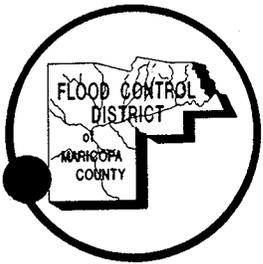
DURATION



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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY Hydrology Manual PAGE 1 OF 5
DETAIL EXAMPLE # 3 COMPUTED _____ DATE _____
DESIGN OF RAINFALL INPUT CHECKED BY _____ DATE _____

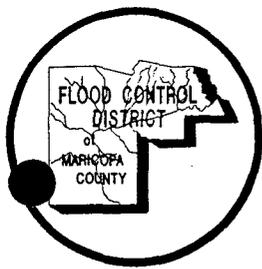


EXAMPLE # 3-A: NEED TO DESIGN 100-YR, 6-HR RAINFALL FOR A 0.2 mi^2 BASIN IN THE CAREFREE AIRPORT AREA.

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 SHOULD BE SIMILAR TO PATTERN #1 MASS CURVE OF THE MANUAL. SO THE SAME PROCEDURES (NOAA) AS PATTERN #1 WILL BE FOLLOWED. THE DESIRED INTENSITY IS 15-MIN.

EXAMPLE # 1 ILLUSTRATED DEPTH-DURATION-FREQUENCY DATA FOR THE ABOVE LOCATION. SAME DATA FOR THE 100-YEAR FREQUENCY, FOR THE DURATION OF 15-MIN TO 6-HOUR SHOULD BE USED.

- ① DESIGN A TABLE FOR A 6-HOUR DURATION, WITH 15-MIN TIME INTERVALS (SEE NEXT PAGE)
- ② PLACE THE 15-MIN DEPTH AT 4:00 (THIS ASSUMES THAT THE PEAK PRODUCING PORTION OF THE STORM IS SHIFTED BY 45-MIN AS EVIDENCED BY PATTERN #1 IN THE HYDROLOGY MANUAL. AS AN ALTERNATIVE, IF JUSTIFIED ONE MIGHT PLACE THE CENTER OF STORM AT 3:15.



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 DETAIL EXAMPLE # 3 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____

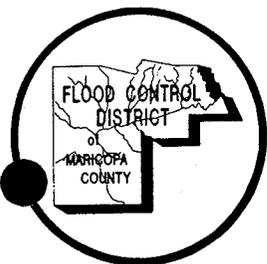
DESIGN RAINFALL DISTRIBUTION

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 _____



③ THE (30-MIN) - (15-MIN) DEPTH OF $(1.93 - 1.46) = 0.47$ IS
PLACED AT 3:45

THE (1-HOUR) - (30-MIN) DEPTH OF $(2.35 - 1.93) = 0.42/2 = .21$
PLACED AT 3:30 AND 4:15.

THE (2-HOUR) - (1-HOUR) DEPTH OF $(2.71 - 2.35) = .36/4 = .09$
IS PLACED AT 3:00, 3:15, 4:30, AND 4:45.

THE (3-HOUR) - (2-HOUR) DEPTH OF $(2.95 - 2.71) = .24/4 = .06$
IS PLACED AT 2:30, 2:45, 5:00, AND 5:15.

THE (6-HOUR) - (3-HOUR) DEPTH OF $(3.40 - 2.95) = .45/12 = .0375$
IS PLACED FROM 1:00 TO 2:15 AND 5:30 TO 6:00

PLEASE NOTE THAT DUE TO SHIFTING, THE LAST 3 VALUES
WOULD GO BEYOND 6:00. THIS IS EASILY HANDLED BY
PLACING THEM FROM 0:15 TO 0:45.

④ THE CUMULATIVE RAINFALL VALUES CAN BE USED FOR INPUT
INTO HEC-1.



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE # 3 COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____

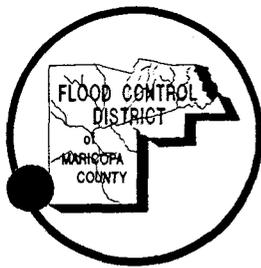
EXAMPLE # 3-B: NEED TO DESIGN 100-YR, 6-HR RAINFALL FOR
A 5.0 Mi^2 BASIN IN THE CAREFREE AIRPORT AREA.

AREAL REDUCTION IS REQUIRED SINCE THIS IS LARGER THAN 0.5 Mi^2 .
USING FIGURE 2.4, A REDUCTION FACTOR OF .865 IS USED. THIS
WILL GIVE $.865(3.40) = 2.94 \text{ IN}$, WHERE 3.40 IS THE 100-YR, 6-HR
DEPTH, COMPUTED IN EXAMPLE # 1.

FROM FIGURE 2.9, THE PATTERN FOR THIS DRAINAGE AREA WOULD
BE SOMEWHERE BETWEEN PATTERN # 2 AND PATTERN # 3. THE
CLOSEST NUMBER WOULD BE 2.34, REFERED 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.
FOR MORE DETAILS SEE NEXT PAGE.

(*) ONCE PATTERN # 2.34 IS CONSTRUCTED, ITS ELEMENTS ARE
MULTIPLIED BY THE RAINFALL DEPTH (2.94 INCHES), DIVIDED BY
100, AND PLACED IN THE LAST COLUMN, UNDER DESIGN
RAINFALL. THESE ARE FOR INPUT INTO HEC-1.



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #3 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____

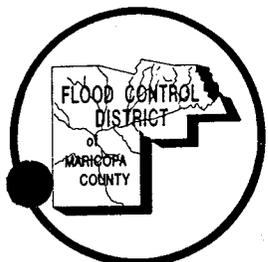
DESIGN RAINFALL WITH AREAL REDUCTION

| TIME | PATTERN #2 | PATTERN #3 | .34 (#3-#2) | PATTERN #2.34 | DESIGN RAINFALL (*) |
|------|------------|------------|-------------|---------------|---------------------|
| 0:00 | 0.0 | 0.0 | 0.0 | 0.0 | 0 |
| 0:15 | 0.6 | 1.5 | 0.3 | 0.9 | .026 |
| 0:30 | 1.2 | 2.0 | 0.3 | 1.5 | .044 |
| 0:45 | 2.0 | 3.0 | 0.3 | 2.3 | .068 |
| 0:60 | 3.1 | 4.8 | 0.6 | 3.7 | .109 |
| 1:15 | 3.9 | 6.3 | 0.8 | 4.7 | .138 |
| 1:30 | 4.9 | 7.6 | 0.9 | 5.8 | .171 |
| 1:45 | 5.7 | 9.0 | 1.1 | 6.8 | .200 |
| 2:00 | 6.7 | 10.5 | 1.3 | 8.0 | .235 |
| 2:15 | 7.6 | 11.9 | 1.5 | 9.1 | .268 |
| 2:30 | 8.7 | 13.5 | 1.6 | 10.3 | .303 |
| 2:45 | 10.0 | 15.2 | 1.8 | 11.8 | .347 |
| 3:00 | 12.0 | 17.5 | 1.9 | 13.9 | .409 |
| 3:15 | 16.3 | 22.2 | 2.0 | 18.3 | .538 |
| 3:30 | 25.2 | 30.4 | 1.7 | 26.9 | .791 |
| 3:45 | 45.1 | 47.2 | 0.7 | 45.8 | 1.346 |
| 4:00 | 69.1 | 67.0 | -0.7 | 68.4 | 2.01 |
| 4:15 | 83.7 | 79.6 | -1.4 | 82.3 | 2.42 |
| 4:30 | 90.0 | 86.8 | -1.1 | 88.9 | 2.614 |
| 4:45 | 93.8 | 91.2 | -0.9 | 92.9 | 2.731 |
| 5:00 | 96.7 | 94.6 | -0.7 | 96.0 | 2.82 |
| 5:15 | 98.5 | 97.4 | -0.4 | 98.1 | 2.884 |
| 5:30 | 99.0 | 98.0 | -0.3 | 98.7 | 2.90 |
| 5:45 | 99.0 | 98.7 | -0.1 | 98.9 | 2.908 |
| 6:00 | 100.0 | 100.0 | 0.0 | 100.0 | 2.94 |

(*) PLEASE SEE PREVIOUS PAGE.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 3
 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 AND THE VOLUME OF FLOW TO BE RETAINED

STEP 1: DETERMINE THE RUNOFF COEFFICIENT 'C' (TABLE 3-1)

- 70% SINGLE FAMILY AREAS → .40
- 30% LIGHT INDUSTRIAL → .65
- $(.70)(.40) + (.30)(.65) = \underline{\underline{0.475}}$

STEP 2: CALCULATE T_c

$$T_c = 11.4 L^{.50} r^{.52} S^{-.31} L_{100}^{-.38}$$

$L = 1.236 \text{ mi}$

$r = .027$ (FIGURE 3.4 OR THE EQUATIONS ON P.10 OF CHAPTER 5)

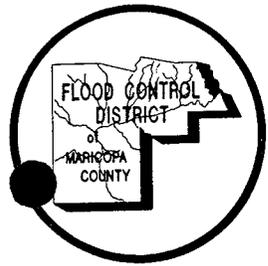
$S = 33 \text{ ft/mi}$

$$T_c = 11.4 (1.236)^{.50} (.027)^{.52} (33)^{-.31} (L_{100})^{-.38}$$

$$T_c = 0.655 (L_{100})^{-.38}$$

CHOOSE A STARTING VALUE FOR T_c , SAY 30 min.
 AT 30 min., THE 100-YEAR RAINFALL INTENSITY IS 4.00 in/hr.
 (I-D-F CURVES, FIGURE 3-2)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY



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$$T_c = 0.655 (4.00)^{-0.38}$$

$$T_c = 0.387 \text{ hr} = 23.2 \text{ min.}$$

TRY $T_c = 20 \text{ min} \rightarrow L_{100} = 5.10 \text{ in/hr}$

$$T_c = 0.655 (5.10)^{-0.38}$$

$$T_c = 0.353 \text{ hr} = 21.2 \text{ min.}$$

TRY $T_c = 22 \text{ min.} \rightarrow L_{100} = 4.80 \text{ in/hr}$

$$T_c = 0.655 (4.80)^{-0.38}$$

$$T_c = 0.361 \text{ hr} = 21.7 \text{ min.}$$

OK, USE $T_c = 22 \text{ min.}$, $L_{100} = 4.80 \text{ in/hr}$

STEP 3: CALCULATE PEAK DISCHARGE

$$Q_p = C L_{100} A$$

$$Q_p = (.475)(4.80)(140) = \underline{\underline{319 \text{ cfs}}}$$

STEP 4: DETERMINE RETENTION VOLUME (V)

$$V = C \left(\frac{P_{100}^2}{12} \right) A$$

WHERE $P_{100}^2 = 2\text{-HOUR, 100-YEAR POINT RAINFALL DEPTH IN INCHES}$

SINCE THE WATERSHED IS OUTSIDE THE PHOENIX AREA, P_{100}^2 MUST BE CALCULATED FROM THE DURATION CONVERSION EQUATIONS IN CHAPTER 2.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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Y_{100} = 1-hour, 100-year point rainfall

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

where: X_3 = 6-hr, 100-year point rainfall = 3.4"

X_4 = 24-hr, 100-year point rainfall = 4.7"

$$Y_{100} = 0.494 + 0.755 (3.4)(3.4/4.7)$$

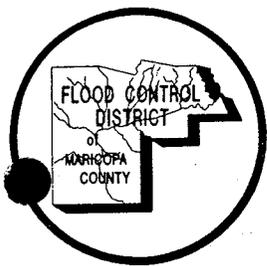
$$Y_{100} = 2.351 \text{ Ln}$$

$$X_2 \equiv P_{100}^2 = 0.341 (X_3) + 0.659 (Y_{100})$$

$$P_{100}^2 = 0.341 (3.4) + 0.659 (2.351) = 2.71 \text{ Ln}$$

$$V = 0.475 \left(\frac{2.71}{12} \right) 140$$

$$\underline{\underline{V = 15.02 \text{ ac-ft}}}$$



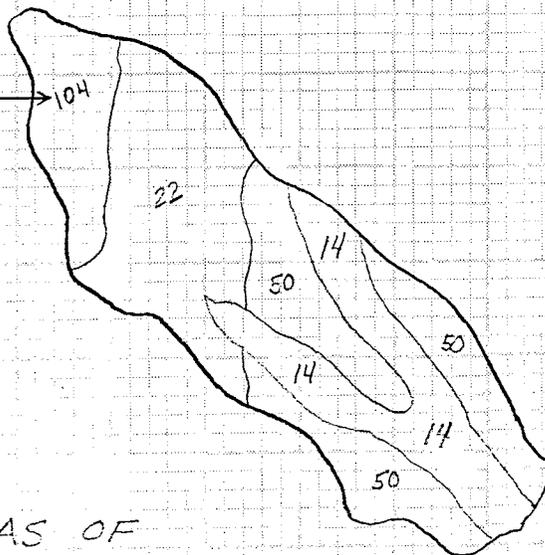
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 3
 DETAIL EXAMPLE #5 COMPUTED _____ DATE _____
 _____ CHECKED BY _____ DATE _____

GREEN & AMPT LOSS METHOD

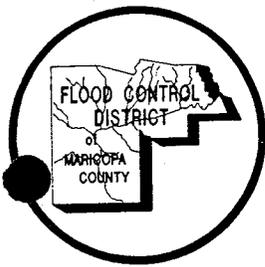
SCENARIO: CALCULATE THE GREEN AND AMPT LOSS RATE PARAMETERS FOR SUBBASIN #4 OF THE EXAMPLE WATERSHED. ASSUME THAT THE WATERSHED IS LOCATED WITHIN THE BOUNDARIES OF THE "SOIL SURVEY OF AGUILA - CAREFREE AREA, PARTS OF MARICOPA AND PINAL COUNTIES, ARIZONA". ASSUME THE DESIGN STORM TO BE A 6-HOUR, 100-YEAR EVENT OF 3.5 TO 4.0 INCHES.

NUMBERS INDICATE THE SOIL MAP UNIT, AND APPEAR ON THE SOIL SURVEY MAPS



STEP 1: PLANIMETER THE AREAS OF EACH MAP UNIT WITHIN THE SUBBASIN. ASSUME FOR THIS CASE:

| <u>MAP UNIT</u> | <u>PERCENT TOTAL AREA</u> |
|-----------------|---------------------------|
| 14 | 15 |
| 50 | 30 |
| 22 | 40 |
| 104 | 15 |
| | <hr/> |
| | 100 |



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NOTE: REFER TO "NOTES ON CALCULATING LOSS RATE PARAMETERS" WITHIN THE "APPLICATIONS" CHAPTER FOR ADVICE ON CONSTRUCTION OF THE FOLLOWING TABLE.

STEP 2: CONSTRUCT A TABLE SIMILAR TO THE FOLLOWING:

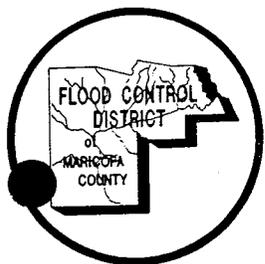
| MAP UNIT # | % MAP UNIT | SOIL NAME | MEDIAN % SAND & GRAVEL | MEDIAN % CLAY | GENERAL SOIL TEXTURE | XKSAT (Ln/hr) | PSIF (Ln) | DTHETA (ASSUME DRY) |
|------------|------------|--------------|------------------------|---------------|----------------------|---------------|-----------|---------------------|
| 14 | 80 | CARRIZO | 92.5 | 2.5 | SAND | 4.60 | 1.9 | .35 |
| | 20 | ANTHO | 65 | 10 | SANDY LOAM | .40 | 4.3 | .35 |
| | | BRIOS | 87.5 | 7.5 | LOAMY SAND | 1.20 | 2.4 | .35 |
| | | MARIPO | 65 | 10 | SANDY LOAM | .40 | 4.3 | .35 |
| 50 | 80 | ESTRELLA | 40 | 17.5 | LOAM | .15 | 3.5 | .35 |
| | 20 | GILMAN | 35 | 23.5 | LOAM | .15 | 3.5 | .35 |
| | | VALENCIA | 67.5 | 12.5 | SANDY LOAM | .40 | 4.3 | .35 |
| | | MOHALL | 25 | 27.5 | SILTY LOAM | .25 | 6.6 | .40 |
| 22 | 80 | CONTINE | 22.5 | 64 | CLAY | .01 | 12.4 | .15 |
| | 20 | CAREFREE | 17.5 | 52.5 | CLAY | .01 | 12.4 | .15 |
| | | EBON | 72.5 | 27.5 | SANDY CLAY LOAM | .06 | 8.6 | .25 |
| | | MOHALL | 25 | 27.5 | SILTY LOAM | .25 | 6.6 | .40 |
| 104 | 60 | ROCK OUTCROP | — | — | — | — | — | — |
| | 20 | LEHMANS | 70 | 30 | SANDY CLAY LOAM | .06 | 8.6 | .25 |
| | 20 | ARIZO | 70 | 12.5 | SANDY LOAM | .40 | 4.3 | .35 |
| | | EBa | 75 | 12.5 | SANDY LOAM | .40 | 4.3 | .35 |
| | | PINALENO | 60 | 35 | SANDY CLAY | .02 | 7.4 | .20 |

STEP 3: CALCULATE WEIGHTED PARAMETERS FOR EACH MAP UNIT

UNIT 14:
 $XKSAT = .80(4.6) + .067(.40) + .067(1.2) + .067(.40) = 3.81 \text{ Ln/hr}$
 $PSIF = .80(1.9) + .067(4.3) + .067(2.4) + .067(4.3) = 2.26 \text{ Ln}$
 $DTHETA = .80(.35) + .067(.35) + .067(.35) + .067(.35) = .35$

UNIT 50:
 $XKSAT = .80(.15) + .067(.15) + .067(.40) + .067(.25) = .174 \text{ Ln/hr}$
 $PSIF = .80(3.5) + .067(3.5) + .067(4.3) + .067(6.6) = 3.77 \text{ Ln}$
 $DTHETA = .80(.35) + .067(.35) + .067(.35) + .067(.40) = .35$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY



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UNIT 22

$$\begin{aligned} XKSAT &= .80(.01) + .067(.01) + .067(.06) + .067(.25) = .029 \text{ in/hr} \\ PSIF &= .80(12.4) + .067(12.4) + .067(8.6) + .067(6.6) = 11.77 \text{ in} \\ DTHETA &= .80(.15) + .067(.15) + .067(.25) + .067(.40) = .17 \end{aligned}$$

UNIT 104

$$\begin{aligned} XKSAT &= .50(.06) + .167(.40) + .167(.40) + .167(.02) = .167 \text{ in/hr} \\ PSIF &= .50(8.6) + .167(4.3) + .167(4.3) + .167(9.4) = 7.31 \text{ in} \\ DTHETA &= .50(.25) + .167(.35) + .167(.35) + .167(.20) = .28 \\ RTIMP &= 60\% \end{aligned}$$

STEP 4

CALCULATE WEIGHTED PARAMETERS FOR THE SUBBASIN BY PERCENTAGE OF SOIL MAP UNITS

$$\begin{aligned} XKSAT &= .15(3.81) + .30(.174) + .40(.029) + .15(.167) = \underline{0.66 \text{ in/hr}} \\ PSIF &= .15(2.26) + .30(3.77) + .40(11.77) + .15(7.31) = \underline{7.3 \text{ in}} \\ DTHETA &= .15(.35) + .30(.25) + .40(.17) + .15(.28) = \underline{.27} \\ RTIMP &= \underline{\hspace{10em}} \rightarrow .15(.60) = \underline{9\%} \end{aligned}$$

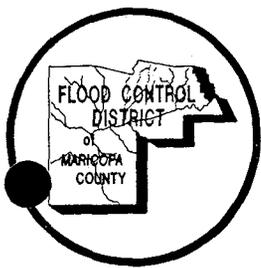
STEP 5

SELECT A SURFACE RETENTION LOSS (IA) FOR THE SUBBASIN FROM TABLE 3-1.

FOR THIS EXAMPLE, ASSUME:

85% HILLSLOPES \rightarrow .15 in
 15% MOUNTAIN \rightarrow .25 in

$$IA = .85(.15) + .15(.25) = \underline{0.17 \text{ in}}$$



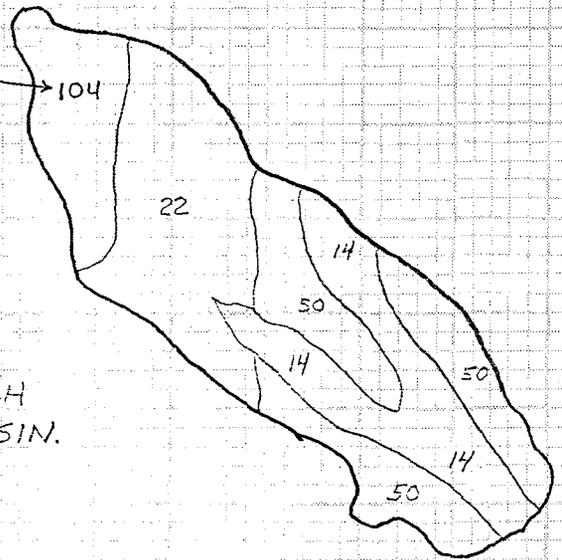
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 3
 DETAIL EXAMPLE # 6 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____

INITIAL AND UNIFORM LOSS RATE METHOD BY SOIL TEXTURE AND HYDROLOGIC SOIL GROUP (HSG)

SCENARIO: CALCULATE THE INITIAL AND UNIFORM LOSS RATE PARAMETERS BY SOIL TEXTURE AND BY HYDROLOGIC SOIL GROUP 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-HOUR, 100-YEAR EVENT OF 3.5 TO 4.0 INCHES.

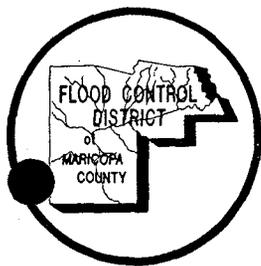
NUMBERS INDICATE THE SOIL MAP UNIT, AND APPEAR ON THE SOIL SURVEY MAPS



STEP 1:

PLANIMETER THE AREAS OF EACH MAP UNIT WITHIN THE SUBBASIN. ASSUME FOR THIS CASE:

| <u>MAP UNIT</u> | <u>PERCENT TOTAL AREA</u> |
|-----------------|---------------------------|
| 14 | 15 |
| 50 | 30 |
| 22 | 40 |
| 104 | 15 |
| | <u>100</u> |



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NOTE: REFER TO THE "NOTES ON CALCULATING LOSS RATE PARAMETERS" SECTION WITHIN THE "APPLICATIONS" CHAPTER FOR ADVISE ON CONSTRUCTION OF THE FOLLOWING TABLE.

STEP 2: CONSTRUCT A TABLE SIMILAR TO THE FOLLOWING:

| MAP UNIT # | % MAP UNIT | SOIL NAME | MEDIAN % SAND & GRAVEL | MEDIAN % CLAY | GENERAL SOIL TEXTURE | H S G | SOIL TEXTURE | | HSG | |
|------------|------------|--------------|------------------------|---------------|----------------------|-------|--------------|---------------|---------|---------------|
| | | | | | | | IL (in) | CNSTL (in/hr) | IL (in) | CNSTL (in/hr) |
| 14 | 80 | CARRIZO | 92.5 | 2.5 | SAND | A | 1.3 | 4.6 | .6 | .40 |
| | 20 | ANTHO | 65 | 10 | SANDY LOAM | | .7 | .40 | | |
| | | BRIOS | 87.5 | 7.5 | LOAMY SAND | | .8 | 1.2 | | |
| | | MARIPO | 65 | 10 | SANDY LOAM | | .7 | .40 | | |
| 50 | 80 | ESTRELLA | 40 | 17.5 | LOAM | B | .6 | .15 | .5 | .25 |
| | 20 | GILMAN | 35 | 23.5 | LOAM | | .6 | .15 | | |
| | | VALENCIA | 67.5 | 12.5 | SANDY LOAM | | .7 | .40 | | |
| | | MOHALL | 25 | 27.5 | SILTY LOAM | | .8 | .25 | | |
| 22 | 80 | CONTINE | 22.5 | 64 | CLAY | C | .3 | .01 | .5 | .15 |
| | 20 | CAREFREE | 17.5 | 52.5 | CLAY | | .3 | .01 | | |
| | | EBON | 72.5 | 27.5 | SANDY CLAY LOAM | | .6 | .06 | | |
| | | MOHALL | 25 | 27.5 | SILTY LOAM | | .8 | .25 | | |
| 104 | 60 | ROCK OUTCROP | — | — | — | D | — | — | .4 | .05 |
| | 20 | LEHMANS | 70 | 30 | SANDY CLAY LOAM | | .6 | .06 | | |
| | | ARIZO | 70 | 12.5 | SANDY LOAM | | .7 | .40 | | |
| | | EBA | 75 | 12.5 | SANDY LOAM | | .7 | .40 | | |
| 20 | PINALENO | 60 | 35 | SANDY CLAY | .4 | .02 | | | | |

STEP 4: CALCULATE WEIGHTED PARAMETERS FOR EACH MAP UNIT
NOTE: SKIP THIS STEP IF USING THE LOSS PARAMETERS FOR HYDROLOGIC SOIL GROUPS

UNIT 14:

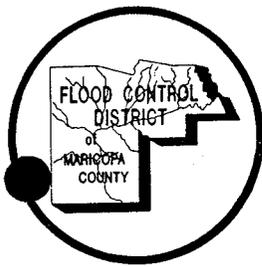
$$IL = .80(1.3) + .067(.7) + .067(.8) + .067(.7) = 1.19 \text{ in}$$

$$CNSTL = .80(4.6) + .067(.40) + .067(1.2) + .067(.40) = 3.81 \text{ in/hr}$$

UNIT 50:

$$IL = .80(.6) + .067(.6) + .067(.7) + .067(.8) = .62 \text{ in}$$

$$CNSTL = .80(.15) + .067(.15) + .067(.40) + .067(.25) = .17 \text{ in/hr}$$



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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UNIT 22: $IL = .80(.3) + .067(.3) + .067(.6) + .067(.8) = .35 \text{ in}$
 $CNSTL = .80(.01) + .067(.01) + .067(.06) + .067(.25) = .03 \text{ in/hr}$

UNIT 104: $IL = .50(.6) + .167(.7) + .167(.7) + .167(.4) = .60 \text{ in}$
 $CNSTL = .50(.06) + .167(.40) + .167(.40) + .167(.02) = .17 \text{ in/hr}$

STEP 5: CALCULATE WEIGHTED PARAMETERS FOR THE SUBBASIN BY PERCENTAGE OF SOIL MAP UNITS

FOR SOIL TEXTURE:

$$IL = .15(1.19) + .30(.62) + .40(.35) + .15(.60) = .59 \text{ in}$$
$$CNSTL = .15(3.81) + .30(.17) + .40(.03) + .15(.17) = \underline{.66 \text{ in/hr}}$$
$$RTIMP = \text{_____} \rightarrow .15(.60) = \underline{9\%}$$

FOR HYDROLOGIC SOIL GROUPS:

$$IL = .15(.6) + .30(.5) + .40(.5) + .15(.4) = .50 \text{ in}$$
$$CNSTL = .15(.40) + .30(.25) + .40(.15) + .15(.05) = \underline{.20 \text{ in/hr}}$$

STEP 6: SELECT A SURFACE RETENTION LOSS (IA) FOR THE SUBBASIN FROM TABLE 3-1 AND CALCULATE STRTL.

FOR THIS EXAMPLE, ASSUME: 85% HILLSLOPES \rightarrow .15 in
15% MOUNTAIN \rightarrow .25 in

$$IA = .85(.15) + .15(.25) = \underline{0.17 \text{ in}}$$

FOR SOIL TEXTURE:

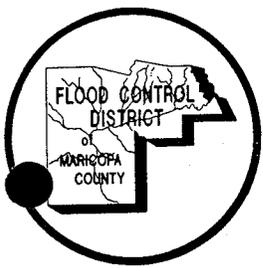
$$STRTL = 0.17 + 0.59 = \underline{0.76 \text{ in}}$$

FOR HYDROLOGIC SOIL GROUPS:

$$STRTL = 0.17 + .50 = \underline{0.67 \text{ in}}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE # 7 COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____



CLARK UNIT HYDROGRAPH FOR AN URBAN BASIN

SCENARIO:

DEVELOP THE CLARK UNIT HYDROGRAPH PARAMETERS FOR SUBBASIN # 2 OF THE EXAMPLE WATERSHED.

STEP 1:

PHYSICAL CHARACTERISTICS:

AREA: $2.17 \text{ mi}^2 = 1389 \text{ acres}$

FLOW PATH (L) = 1.85 miles

SLOPE (S) = 30.5 ft/mi

% IMPERVIOUSNESS = 21

STEP 2:

CALCULATE "r" USING THE EQUATIONS ON P. 10, OR THE "CALCULATION OF T_c & R" WORKSHEET (APPENDIX)

$$r = m (\log A) + b$$

SINCE THIS IS AN URBAN BASIN, $m = -.00625$ and $b = .04$

$$r = -.00625 (\log 1389) + .04$$

$$r = .020$$

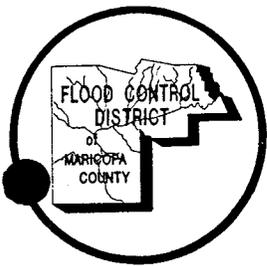
STEP 3:

(REFER TO THE WORKSHEET DURING THE REMAINING STEPS)

CALCULATE T_c AS A FUNCTION OF "L":

$$T_c = [11.4 L^{.50} r^{.52} S^{-.31}] L^{-.38}$$

$$T_c = 0.703 (L)^{-.38}$$



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STEP 4:

ENTER RAINFALL, LOSS, AND CLARK PARAMETER DATA INTO AN HEC-1 INPUT DECK, BUT SET $T_c \neq R$ EQUAL TO ZERO. RUN TO GENERATE A RAINFALL/LOSS/EXCESS TABLE.

STEP 5:

COMPUTE THE AVERAGE EXCESS INTENSITIES FOR A TIME PERIOD GREATER THAN T_c . (SEE WORKSHEET AND SAMPLE HEC-1 RUN)

STEP 6:

CREATE THE GRAPH OF AVERAGE EXCESS INTENSITY VS. TIME.

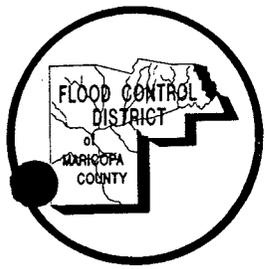
STEP 7:

CALCULATE T_c BY ITERATION. "L" VALUES ARE READ FROM THE GRAPH. CALCULATE R.

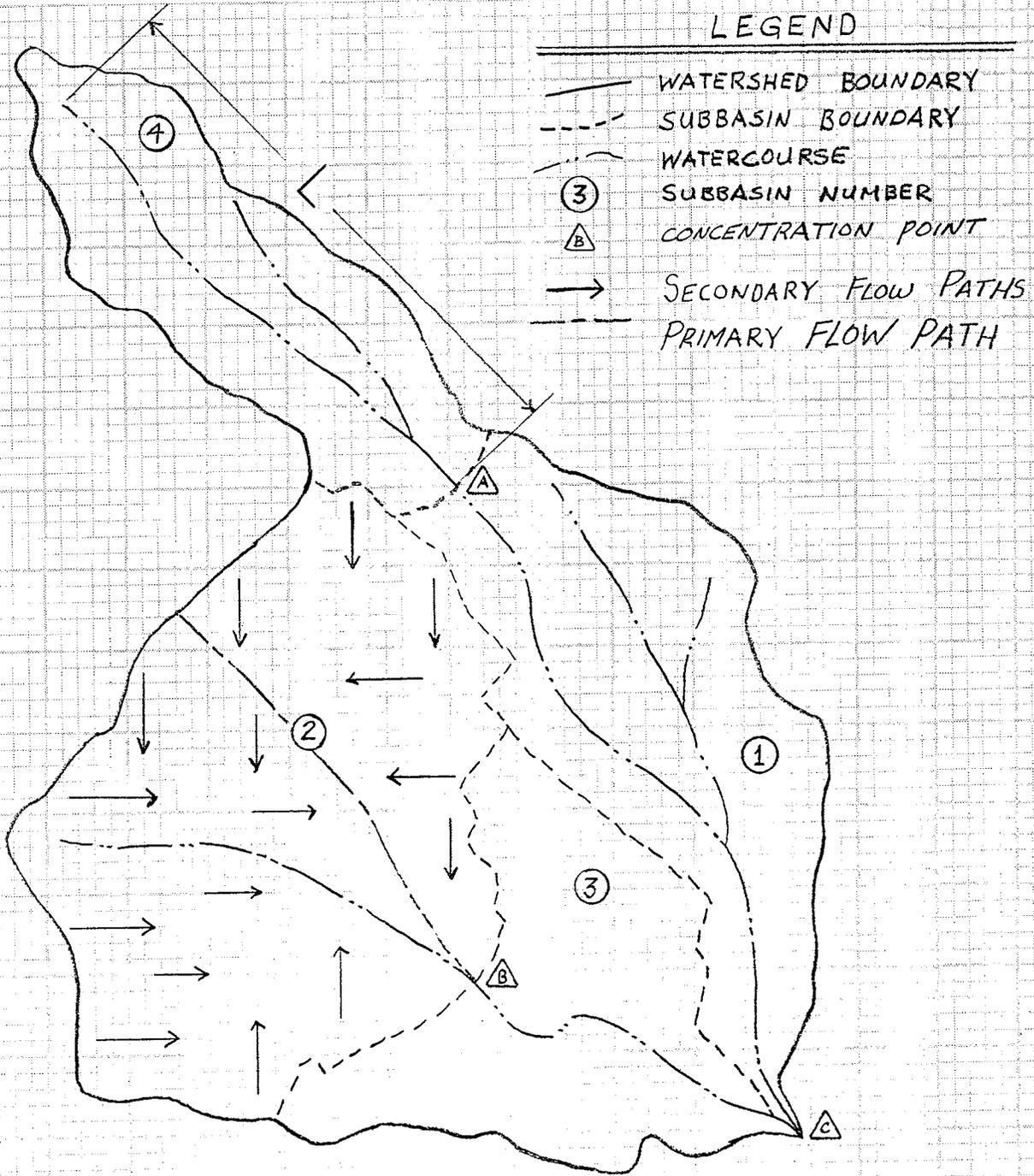
STEP 8:

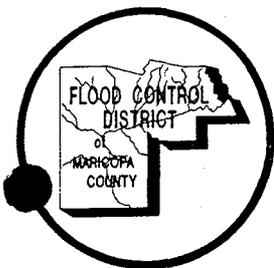
ENTER $T_c \neq R$ VALUES INTO HEC-1 AND RUN AGAIN. (SEE SAMPLE HEC-1 RUN)

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DETAIL EXAMPLE WATERSHED COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____





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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 MOUNTAIN FRONT |

WATERCOURSES

| SUBBASIN # | DESCRIPTION | GEOMETRY | AVE. TOP 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. | 20 | 4 | 3:1 | .015 |
| 4 | NATURAL DESERT STREAM | TRAP | 15 | 2 | 2:1 | .040 |

CALCULATION OF Tc & R

Calculated by: _____ Date: _____

Checked by: _____ Project: _____

Watershed: EXAMPLE WATERSHED - SUBBASIN # 2

Rainfall Frequency: 100 - yr Duration: 6 - hr. Pattern #: 1.5

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 Increment. | Increment. 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. | Increment. 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.

$r = m [\log(A * 640)] + b$
 $r = (-.00625) \log (2.17 * 640) + (.04)$
 $r = .020$

$Tc = 11.4 L^r S^i$
 .50 .52 -.31 -.38

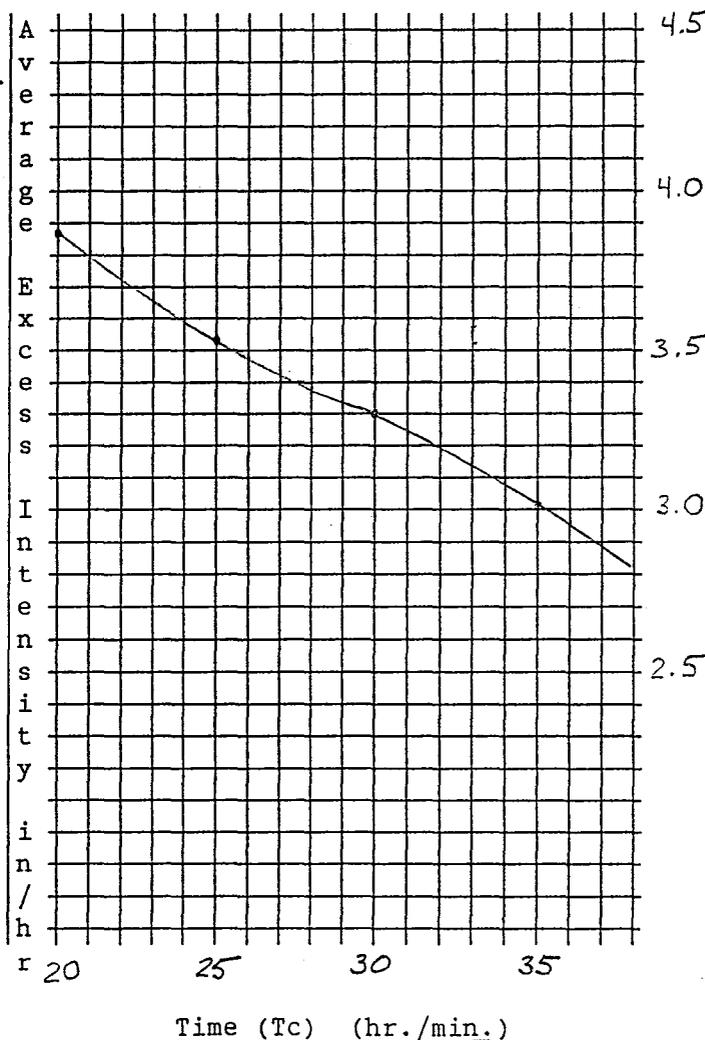
$Tc = (.0703)^i$
 -.38

| Trial Tc | i | Calc. Tc |
|----------|------|----------|
| .417 | 3.53 | .435 |
| .450 | 3.42 | .441 |
| .430 | 3.48 | .438 |
| .440 | 3.45 | .439 |

Tc = .440 hr.

$R = .37 Tc^A L^B$
 1.11 -.57 .80

R = .156 hr.



```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
* FEBRUARY 1981 *
* REVISED 31 JAN 85 *
*
* RUN DATE 9/ 5/1989 TIME15: 4: 0 *
*
*****

```

```

*****
*
* 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 XXXX X
X X X X X XX
X X X X X
XXXXXX XXXX X XXXX X
X X X X X
X X X X X
X X XXXXXXX XXXX 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

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 #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
    
```

 *
 * FLOOD HYDROGRAPH PACKAGE (HEC-1) *
 * FEBRUARY 1981 *
 * REVISED 31 JAN 85 *
 *
 * RUN DATE 9/ 5/1989 TIME15: 4: 3 *
 *

 *
 * U.S. ARMY CORPS OF ENGINEERS *
 * THE HYDROLOGIC ENGINEERING CENTER *
 * 609 SECOND STREET *
 * DAVIS, CALIFORNIA 95616 *
 * (916) 440-3285 OR (FTS) 448-3285 *
 *

SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
 MARICOPA COUNTY HYDROLOGY MANUAL
 EXAMPLE #7 - CLARK UNIT HYDROGRAPH
 RAINFALL: 6-HR, 100-YEAR POINT RAINFALL DEPTH OF 3.25 INCHES
 HYDROGRAPH: CLARK - TC & R SET EQUAL TO ZERO
 URBAN TIME-AREA CURVE
 LOSSES: IL+ULR BY SOIL TEXTURE
 BASIN AREA: 1.10 SQUARE MILES, PATTERN #1.5

10 IO OUTPUT CONTROL VARIABLES

IPRNT 0 PRINT CONTROL
 IPLOT 0 PLOT CONTROL
 QSCAL 0. HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA

NMIN 5 MINUTES IN COMPUTATION INTERVAL
 IDATE 5SEP89 STARTING DATE
 ITIME 0000 STARTING TIME
 NQ 85 NUMBER OF HYDROGRAPH ORDINATES
 NDDATE 5SEP89 ENDING DATE
 NDTIME 0700 ENDING TIME

COMPUTATION INTERVAL .08 HOURS
 TOTAL TIME BASE 7.00 HOURS

ENGLISH UNITS

DRAINAGE AREA SQUARE MILES
 PRECIPITATION DEPTH INCHES
 LENGTH, ELEVATION FEET
 FLOW CUBIC FEET PER SECOND
 STORAGE VOLUME ACRE-FEET
 SURFACE AREA ACRES
 TEMPERATURE DEGREES FAHRENHEIT

*** **

* *
11 KK * BASIN2 *
* *

COMPUTE DISCHARGE AT THE OUTLET OF SUBBASIN #2

13 IN TIME DATA FOR INPUT TIME SERIES
JXMIN 15 TIME INTERVAL IN MINUTES
JXDATE 5SEP89 STARTING DATE
JXTIME 0 STARTING TIME

SUBBASIN RUNOFF DATA

18 BA SUBBASIN CHARACTERISTICS
TAREA 2.17 SUBBASIN AREA

PRECIPITATION DATA

14 PB STORM 3.25 BASIN TOTAL PRECIPITATION

15 PI INCREMENTAL PRECIPITATION PATTERN

| | | | | | | | | | |
|------|------|------|------|------|-------|-------|-------|------|------|
| .18 | .18 | .18 | .17 | .17 | .17 | .22 | .22 | .22 | .32 |
| .32 | .32 | .27 | .27 | .27 | .30 | .30 | .30 | .28 | .28 |
| .28 | .28 | .28 | .28 | .28 | .28 | .28 | .40 | .40 | .40 |
| .43 | .43 | .43 | .65 | .65 | .65 | 1.05 | 1.05 | 1.05 | 2.78 |
| 2.78 | 2.78 | 6.00 | 6.00 | 6.00 | 11.67 | 11.67 | 11.67 | 3.67 | 3.67 |
| 3.67 | 1.38 | 1.38 | 1.38 | .95 | .95 | .95 | .70 | .70 | .70 |
| .52 | .52 | .52 | .28 | .28 | .28 | .18 | .18 | .18 | .37 |
| .37 | .37 | | | | | | | | |

19 LU UNIFORM LOSS RATE
STRTL .65 INITIAL LOSS
CNSL .20 UNIFORM LOSS RATE
RTIMP 21.00 PERCENT IMPERVIOUS AREA

22 UC CLARK UNITGRAPH
TC .44 TIME OF CONCENTRATION
R .16 STORAGE COEFFICIENT

18 UA ACCUMULATED-AREA VS. TIME, 11 ORDINATES
.0 5.0 16.0 30.0 65.0 77.0 84.0 90.0 94.0 97.0
100.0

UNIT HYDROGRAPH PARAMETERS

CLARK TC- .44 HR, R- .16 HR
SNYDER TP- .23 HR, CP- .64

UNIT HYDROGRAPH

13 END-OF-PERIOD ORDINATES

525. 2343. 3727. 3386. 2548. 1746. 1066. 617. 357. 206.
119. 69. 40.

TOTAL RAINFALL - 3.25, TOTAL LOSS - .87, TOTAL EXCESS - 2.38

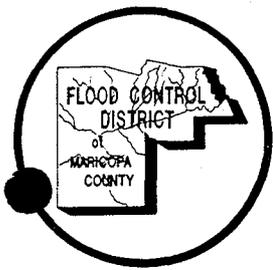
| | | | | | | |
|-----------|------|----------|----------------------|-------|-------|---------|
| PEAK FLOW | TIME | | MAXIMUM AVERAGE FLOW | | | |
| (CFS) | (HR) | | 6-HR | 24-HR | 72-HR | 7.00-HR |
| 4552. | 4.08 | (CFS) | 554. | 475. | 475. | 475. |
| | | (INCHES) | 2.372 | 2.375 | 2.375 | 2.375 |
| | | (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 | | | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---------------|---------|-----------|--------------|---------------------------------|---------|---------|------------|---------------|-------------------|
| | | | | 6-HOUR | 24-HOUR | 72-HOUR | | | |
| HYDROGRAPH AT | BASIN2 | 4552. | 4.08 | 554. | 475. | 475. | 2.17 | | |

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



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 2
DETAIL EXAMPLE # 8 COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____

CLARK UNIT HYDROGRAPH FOR AN UNDEVELOPED BASIN

SCENARIO: DEVELOP THE CLARK UNIT HYDROGRAPH PARAMETERS FOR SUBBASIN # 4 OF THE EXAMPLE WATERSHED.

STEP 1: PHYSICAL CHARACTERISTICS:

$$L = 1.49 \text{ mi}$$

$$\text{AREA} = 0.86 \text{ mi}^2 = 550.4 \text{ ac}$$

$$S = 537 \text{ ft/mi}$$

% IMPERVIOUSNESS: OMIT SINCE LOSSES WILL BE CALCULATED FROM HYDROLOGIC SOIL GROUP DATA. SEE EXAMPLE # 6.

STEP 2: CALCULATE "r" USING THE EQUATIONS ON PAGE 10, OR THE "CALCULATION OF $T_c \in R$ " WORKSHEET (APPENDIX). ALTHOUGH THIS BASIN IS QUITE STEEP, THE TERRAIN IS DESCRIBED AS "ROUGH" AND THE VEGETATION "MODERATE", SO INTERPOLATE BETWEEN "HILLSLOPES" AND "MOUNTAINS".

$$r = m (\log A) + b$$

$$m = (-.025 + -.030) / 2 = -.0275$$

$$b = (.15 + .20) / 2 = .175$$

$$r = -.0275 (\log 550.4) + .175$$

$$r = .100$$

STEP 3: (REFER TO THE WORKSHEET DURING THE REMAINING STEPS)

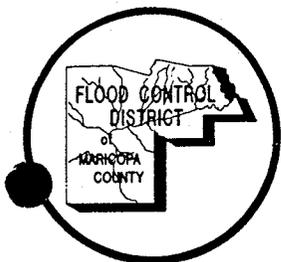
CALCULATE T_c AS A FUNCTION OF L :

$$T_c = [11.4 L^{.50} r^{.52} S^{-.31}] L^{-.38}$$

$$T_c = [5.99] L^{-.38}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____ PAGE 2 OF 2
DETAIL _____ COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____



STEP 4:

ENTER RAINFALL, LOSS, AND CLARK PARAMETER DATA INTO AN HEC-1 INPUT DECK, BUT SET $T_c \dot{=} R$ EQUAL TO ZERO. RUN TO GENERATE A RAINFALL/LOSS/EXCESS TABLE.

STEP 5:

COMPUTE THE AVERAGE EXCESS INTENSITIES FOR A TIME PERIOD GREATER THAN T_c . (SEE WORKSHEET AND SAMPLE HEC-1 RUN)

STEP 6:

CREATE THE GRAPH OF AVERAGE EXCESS INTENSITY VS. TIME.

STEP 7:

CALCULATE T_c BY ITERATION. "L" VALUES ARE READ FROM THE GRAPH. CALCULATE R.

STEP 8:

ENTER $T_c \dot{=} R$ VALUES INTO HEC-1 AND RUN AGAIN. (SEE SAMPLE HEC-1 RUN)

CALCULATION OF Tc & R

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

Watershed: EXAMPLE WATERSHED # 4
 Rainfall Frequency: 100 - yr Duration: 2 - hr. Pattern #: NA

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 Increment. | Increment. Excess in. |
|--------------------------------|-----------------------|
| 0100 | .20 |
| 0105 | .72 |
| 0110 | .37 |
| 0115 | .31 |
| 0120 | .09 |
| 0125 | .06 |
| 0130 | .05 |

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

| Accum. Time hr./min. | Increment. 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 |

A = 0.86 sq.mi.
 L = 1.49 mi.
 S = 537. ft/mi.

$r = m [\log(A * 640)] + b$
 $r = (-.0275) \log (.86 * 640) + (.175)$
 $r = \underline{0.100}$

$T_c = 11.4 L^{.50} r^{.52} S^{-.31} i^{-.38}$

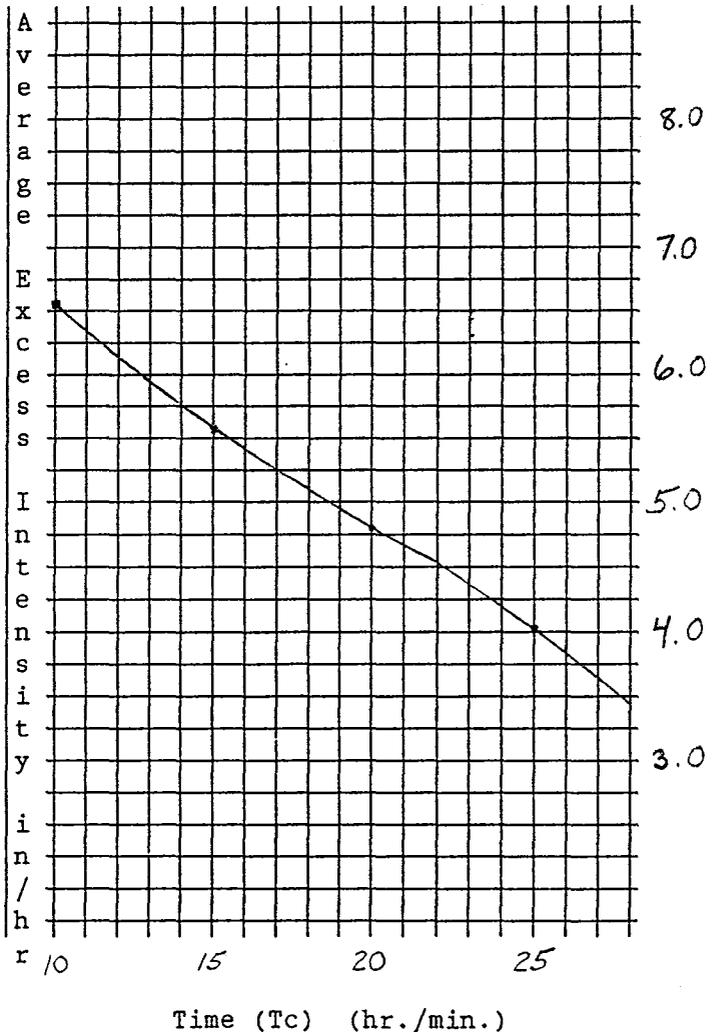
$T_c = (\underline{0.599}) i^{-.38}$

| Trial Tc | i | Calc. Tc |
|----------|------|----------|
| .333 | 4.80 | .330 |
| .325 | 4.87 | .328 |

Tc = .325 hr.

$R = .37 T_c^{1.11} A^{-.57} L^{.80}$

R = .159 hr.



```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
*   FEBRUARY 1981                 *
*   REVISED 31 JAN 85            *
*
*   RUN DATE 9/ 6/1989   TIME10: 9:40 *
*
*****

```

```

*****
*
*   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 XXXXXXXX XXXXX      X
X   X X      X   X      XX
X   X X      X           X
XXXXXXXX XXXX   X       XXXXX X
X   X X      X           X
X   X X      X   X      X
X   X XXXXXXXX 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 -AMSK- 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 # 8

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 #8 - CLARK UNIT HYDROGRAPH (UNDEVELOPED)
* *****
4 ID RAINFALL: 2-HR, 100-YEAR POINT RAINFALL DEPTH OF 2.70 INCHES
5 ID HYDROGRAPH: CLARK - TC & R FROM WORKSHEET
6 ID NATURAL TIME-AREA CURVE
7 ID LOSSES: IL+ULR BY HYDROLOGIC SOIL GROUPS
8 ID BASIN AREA: 0.86 SQUARE MILES
* *****
9 IT 5 05SEP89 0000 37
10 IO 0
* *****

```

```

11 KK BASIN4
12 KM COMPUTE DISCHARGE AT THE OUTLET OF SUBBASIN #4
13 IN 5 05SEP89 0000
14 PB 2.70
15 PC 0. 1.1 1.8 2.3 2.8 3.2 4.6 7.1 10. 13.7
16 PC 17.6 23.2 32.7 60.1 74.3 86.3 90.1 93. 95.4 96.2
17 PC 97. 97.9 98.2 99.2 100.
18 BA .86
19 LU .67 .20
20 UA 0 3 5 8 12 20 43 75 90 96
21 UA 100
22 UC .325 .159
23 ZZ

```

 *
 * FLOOD HYDROGRAPH PACKAGE (HEC-1) *
 * FEBRUARY 1981 *
 * REVISED 31 JAN 85 *
 *
 * RUN DATE 9/ 6/1989 TIME10: 9:44 *
 *

 *
 * U.S. ARMY CORPS OF ENGINEERS
 * THE HYDROLOGIC ENGINEERING CENTER
 * 609 SECOND STREET
 * DAVIS, CALIFORNIA 95616
 * (916) 440-3285 OR (FTS) 448-3285
 *

SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
 MARICOPA COUNTY HYDROLOGY MANUAL
 EXAMPLE #8 - CLARK UNIT HYDROGRAPH (UNDEVELOPED)
 RAINFALL: 2-HR, 100-YEAR POINT RAINFALL DEPTH OF 2.70 INCHES
 HYDROGRAPH: CLARK - TC & R FROM WORKSHEET
 NATURAL TIME-AREA CURVE
 LOSSES: IL+ULR BY HYDROLOGIC SOIL GROUPS
 BASIN AREA: 0.86 SQUARE MILES

10 IO OUTPUT CONTROL VARIABLES
 IPRNT 0 PRINT CONTROL
 IPLOT 0 PLOT CONTROL
 QSCAL 0. HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA
 NMIN 5 MINUTES IN COMPUTATION INTERVAL
 IDATE 5SEP89 STARTING DATE
 ITIME 0000 STARTING TIME
 NQ 37 NUMBER OF HYDROGRAPH ORDINATES
 NDDATE 5SEP89 ENDING DATE
 NDTIME 0300 ENDING TIME
 COMPUTATION INTERVAL .08 HOURS
 TOTAL TIME BASE 3.00 HOURS

ENGLISH UNITS
 DRAINAGE AREA SQUARE MILES
 PRECIPITATION DEPTH INCHES
 LENGTH, ELEVATION FEET
 FLOW CUBIC FEET PER SECOND
 STORAGE VOLUME ACRE-FEET
 SURFACE AREA ACRES
 TEMPERATURE DEGREES FAHRENHEIT

 * *
 11 KK * BASIN4 *
 * *

COMPUTE DISCHARGE AT THE OUTLET OF SUBBASIN #4

13 IN TIME DATA FOR INPUT TIME SERIES
 JXMIN 5 TIME INTERVAL IN MINUTES
 JXDATE 5SEP89 STARTING DATE
 JXTIME 0 STARTING TIME

SUBBASIN RUNOFF DATA

18 BA SUBBASIN CHARACTERISTICS
 TAREA .86 SUBBASIN AREA

PRECIPITATION DATA

14 PB STORM 2.70 BASIN TOTAL PRECIPITATION

15 PI INCREMENTAL PRECIPITATION PATTERN

| | | | | | | | | | |
|------|------|-------|-------|-------|------|------|------|------|------|
| 1.10 | .70 | .50 | .50 | .40 | 1.40 | 2.50 | 2.90 | 3.70 | 3.90 |
| 5.60 | 9.50 | 27.40 | 14.20 | 12.00 | 3.80 | 2.90 | 2.40 | .80 | .80 |
| .90 | .30 | 1.00 | .80 | | | | | | |

19 LU UNIFORM LOSS RATE
 STRTL .67 INITIAL LOSS
 CNSTL .20 UNIFORM LOSS RATE
 RTIMP .00 PERCENT IMPERVIOUS AREA

22 UC CLARK UNITGRAPH
 TC .32 TIME OF CONCENTRATION
 R .16 STORAGE COEFFICIENT

18 UA ACCUMULATED-AREA VS. TIME, 11 ORDINATES

| | | | | | | | | | |
|-------|-----|-----|-----|------|------|------|------|------|------|
| .0 | 3.0 | 5.0 | 8.0 | 12.0 | 20.0 | 43.0 | 75.0 | 90.0 | 96.0 |
| 100.0 | | | | | | | | | |

UNIT HYDROGRAPH PARAMETERS

CLARK TC= .32 HR, R= .16 HR
 SNYDER TP= .28 HR, CP= .95

UNIT HYDROGRAPH

13 END-OF-PERIOD ORDINATES

| | | | | | | | | | |
|-----|------|-------|-------|-------|------|------|------|------|-----|
| 93. | 371. | 1305. | 1829. | 1271. | 743. | 435. | 254. | 149. | 87. |
| 51. | 30. | 17. | | | | | | | |

HYDROGRAPH AT STATION BASIN4

| 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. | 0. | * | 5 SEP | 0135 | 20 | .02 | .02 | .00 | 8 | 1284. |
| 5 SEP | 0005 | 2 | .03 | .03 | .00 | 0. | 0. | * | 5 SEP | 0140 | 21 | .02 | .02 | .00 | 9 | 888. |
| 5 SEP | 0010 | 3 | .02 | .02 | .00 | 0. | 0. | * | 5 SEP | 0145 | 22 | .02 | .02 | .01 | 10 | 591. |
| 5 SEP | 0015 | 4 | .01 | .01 | .00 | 0. | 0. | * | 5 SEP | 0150 | 23 | .01 | .01 | .00 | | 368. |
| 5 SEP | 0020 | 5 | .01 | .01 | .00 | 0. | 0. | * | 5 SEP | 0155 | 24 | .03 | .02 | .01 | | 231. |
| 5 SEP | 0025 | 6 | .01 | .01 | .00 | 0. | 0. | * | 5 SEP | 0200 | 25 | .02 | .02 | .00 | | 148. |
| 5 SEP | 0030 | 7 | .04 | .04 | .00 | 0. | 0. | * | 5 SEP | 0205 | 26 | .00 | .00 | .00 | | 99. |
| 5 SEP | 0035 | 8 | .07 | .07 | .00 | 0. | 0. | * | 5 SEP | 0210 | 27 | .00 | .00 | .00 | | 67. |
| 5 SEP | 0040 | 9 | .08 | .08 | .00 | 0. | 0. | * | 5 SEP | 0215 | 28 | .00 | .00 | .00 | | 43. |
| 5 SEP | 0045 | 10 | .10 | .10 | .00 | 0. | 0. | * | 5 SEP | 0220 | 29 | .00 | .00 | .00 | | 23. |
| 5 SEP | 0050 | 11 | .11 | .11 | .00 | 0. | 0. | * | 5 SEP | 0225 | 30 | .00 | .00 | .00 | | 12. |
| 5 SEP | 0055 | 12 | .15 | .15 | .00 | 0. | 0. | * | 5 SEP | 0230 | 31 | .00 | .00 | .00 | | 7. |
| 5 SEP | 0100 | 13 | .26 | .06 | .20 | 4 | 18. | * | 5 SEP | 0235 | 32 | .00 | .00 | .00 | | 3. |
| 5 SEP | 0105 | 14 | .74 | .02 | .72 | 1 | 141. | * | 5 SEP | 0240 | 33 | .00 | .00 | .00 | | 2. |
| 5 SEP | 0110 | 15 | .38 | .02 | .37 | 2 | 562. | * | 5 SEP | 0245 | 34 | .00 | .00 | .00 | | 1. |
| 5 SEP | 0115 | 16 | .32 | .02 | .31 | 3 | 1473. | * | 5 SEP | 0250 | 35 | .00 | .00 | .00 | | 1. |
| 5 SEP | 0120 | 17 | .10 | .02 | .09 | 5 | 2176. | * | 5 SEP | 0255 | 36 | .00 | .00 | .00 | | 0. |
| 5 SEP | 0125 | 18 | .08 | .02 | .06 | 6 | 2177. | * | 5 SEP | 0300 | 37 | .00 | .00 | .00 | | 0. |
| 5 SEP | 0130 | 19 | .06 | .02 | .05 | 7 | 1792. | * | | | | | | | | |

PEAK EXCESSES

TOTAL RAINFALL = 2.70, TOTAL LOSS = .88, TOTAL EXCESS = 1.82

PEAK FLOW TIME
 (CFS) (HR)
 2177. 1.42

MAXIMUM AVERAGE FLOW

| | | | | |
|----------|-------|-------|-------|---------|
| | 6-HR | 24-HR | 72-HR | 3.00-HR |
| (CFS) | 336. | 336. | 336. | 336. |
| (INCHES) | 1.818 | 1.818 | 1.818 | 1.818 |
| (AC-FT) | 83. | 83. | 83. | 83. |

CUMULATIVE AREA = .86 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 | | | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---------------|---------|-----------|--------------|---------------------------------|---------|---------|------------|---------------|-------------------|
| | | | | 6-HOUR | 24-HOUR | 72-HOUR | | | |
| HYDROGRAPH AT | BASIN4 | 2177. | 1.42 | 336. | 336. | 336. | .86 | | |

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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 9

DETAIL EXAMPLE # 9 COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

APPLICATION OF S-GRAPHS

SCENARIO: DEVELOP THE APPROPRIATE UNIT-GRAPH FOR THE ENCLOSED EXPERIMENTAL WATERSHED.

STEP 1: LIST PHYSICAL CHARACTERISTICS:

$$\text{AREA (A)} = 5.19 \text{ mi}^2$$

$$\text{LENGTH OF WATER COURSE (L)} = 5.2 \text{ mi}$$

$$\text{LENGTH OF WATER COURSE TO A POINT OPPOSITE TO CENTROID (LCA)} = 1.7 \text{ mi}$$

$$\text{SLOPE (S): } S = \left(L / \sum_{i=1}^n \left[\frac{\Delta L_i^3}{\Delta H_i} \right]^{1/2} \right)^2$$

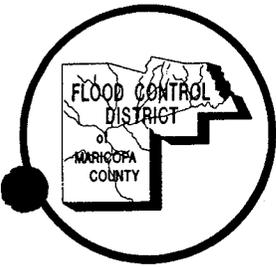
$$\Delta L_1 = 7392 \text{ ft} \quad \Delta H_1 = 600 \text{ ft}, \quad \Delta L_2 = 8448 \text{ ft}, \quad \Delta H_2 = 300, \quad \Delta L_3 = 11616 \text{ ft} \\ \Delta H_3 = 300 \text{ ft}$$

$$\sum_{i=1}^3 \left[\frac{\Delta L_i^3}{\Delta H_i} \right]^{1/2} = \left[\frac{7392^3}{600} \right]^{1/2} + \left[\frac{8448^3}{300} \right]^{1/2} + \left[\frac{11616^3}{300} \right]^{1/2} \\ = 143056.9$$

FOR DETAILS OF ΔH_i AND ΔL_i SEE THE ENCLOSED MAP.

$$S = \left(5.2 (5280) / 143056.9 \right)^2 = .0368 \text{ ft/ft}$$

$$S = .0368 (5280) = 194.5 \text{ ft/mi}$$

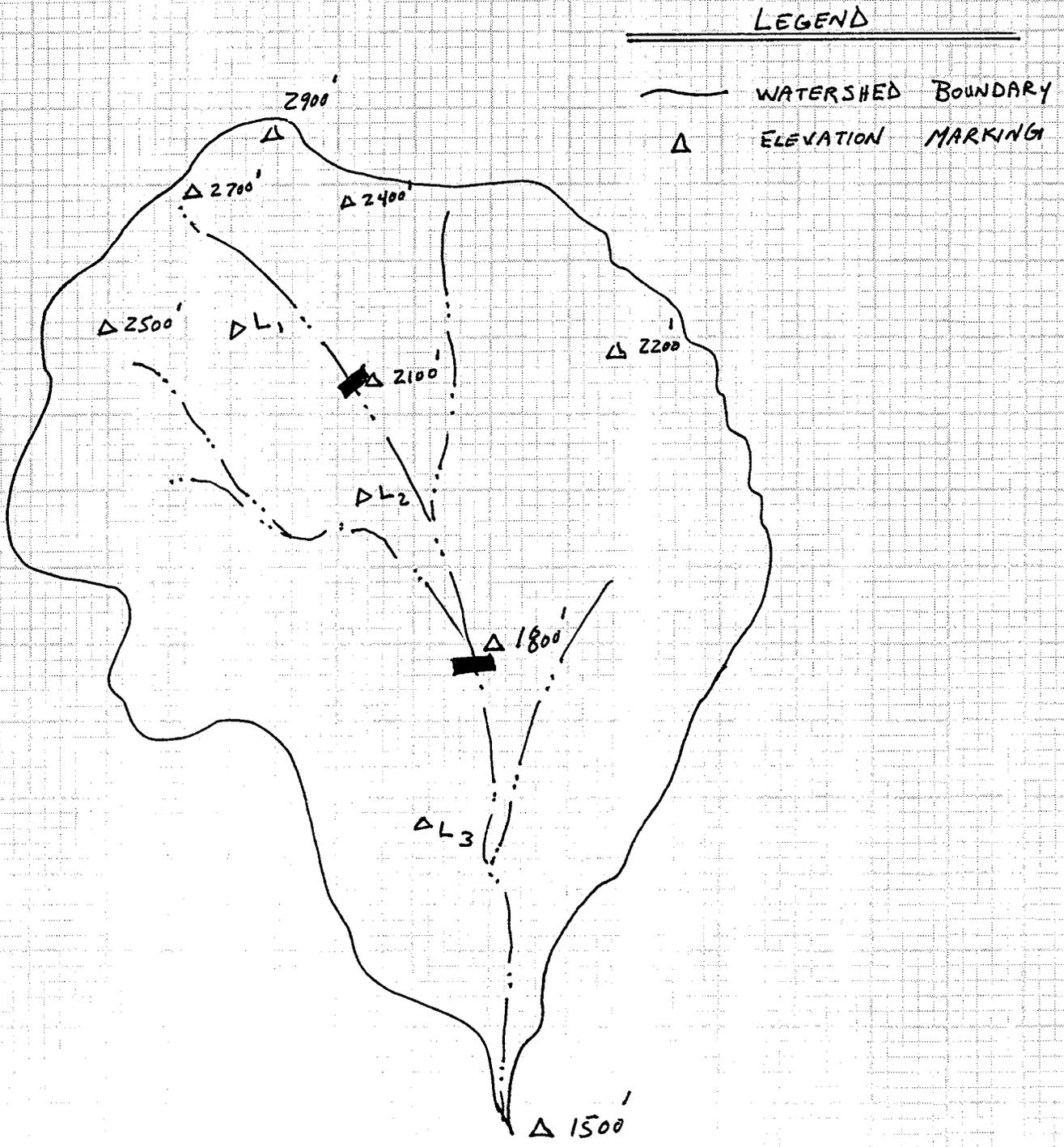


FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA County Hydrology Manual PAGE 3 OF 9

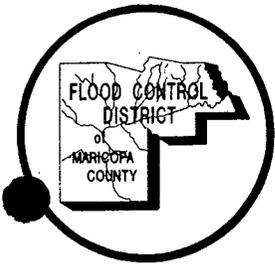
DETAIL EXAMPLE # 9 COMPUTED _____ DATE _____

EXPERIMENTAL WATERSHED CHECKED BY _____ DATE _____



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY Hydrology MANUAL PAGE 3 OF 9
 DETAIL EXAMPLE # 9 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____



STEP 2: CALCULATE BASIN FACTOR $(L L_{ca} / S^{0.5})$

$$(L L_{ca} / S^{0.5}) = (5.2)(1.7) / (194.5)^{0.5} = .63$$

COMPARE BASIN FACTOR WITH THOSE OF FIGURE 5.10 TO IDENTIFY SIMILAR DRAINAGE AREAS AND BASIN FACTORS, TO SELECT \bar{n} .

DRAINAGE AREA # 17: $A = 2.5 \text{ mi}^2$, $L = 3.4 \text{ mi}$, $L_{ca} = 1.7 \text{ mi}$, $S = 100 \text{ ft/mi}$, $\bar{n} = .015$

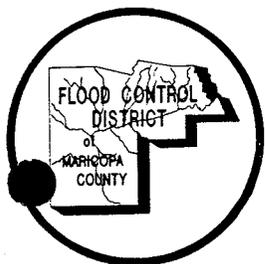
DRAINAGE AREA # 3: $A = 10.8 \text{ mi}^2$, $L = 5.8 \text{ mi}$, $L_{ca} = 2.5 \text{ mi}$, $S = 690 \text{ ft/mi}$, $\bar{n} = .050$

SELECTION OF BASIN COEFFICIENT \bar{n} CAN BE BASED UPON COMPARISON OF ABOVE DRAINAGE AREAS WITH THE BASIN IN QUESTION, OR IT CAN BE SELECTED FROM ESTABLISHED TABLES OF \bar{n} VALUES. IN EITHER CASE THE UNIQUE CONDITIONS OF A PARTICULAR BASIN SHOULD ALSO HELP WITH THE SELECTION. AS FOR OUR EXAMPLE, $\bar{n} = .040$ WAS SELECTED AFTER A COMPARISON WAS MADE WITH DRAINAGE AREAS # 3 AND # 17.

STEP # 3: CALCULATE BASIN LAG:

$$\text{LAG} = 20 \bar{n} \left(\frac{L L_{ca}}{S^{0.5}} \right)^{.38} = 20 (.04) \left(\frac{(1.72)(5.2)}{(194.5)^{0.5}} \right)^{.38} = .673 \text{ HRS}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY



PROJECT MARICOPA COUNTY Hydrology MANUAL PAGE 4 OF 9
DETAIL EXAMPLE # 9 COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____

STEP 4: CALCULATE ULTIMATE DISCHARGE, Q_{ULT} :

$$Q_{ULT} = \frac{645.33 A}{D}$$

WHERE:

Q_{ULT} = ULTIMATE DISCHARGE (CFS)

A = DRAINAGE AREA (mi^2)

D = DURATION OF EXCESS RAINFALL (HOURS)

$$Q_{ULT} = \frac{645.33 (5.19)}{(10/60)^{*}} =$$

* TIME INTERVAL (DURATION) OF 10 min. WAS SELECTED.

THIS IS WITHIN THE RANGE OF $(0.10 - 0.25) \times (\text{LAG TIME})$

AS SUGGESTED IN THE HYDROLOGY MANUAL. PLEASE NOTE

THAT THIS IS ALSO THE HYDROGRAPH TIME STEPS, ΔT .

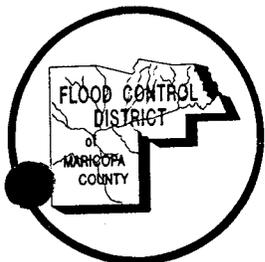
STEP 5: SELECT THE APPROPRIATE S-GRAPH. IN THIS CASE DUE TO

THE MOUNTAINOUS NATURE OF THIS WATERSHED, THE PHOENIX-MOUNTAIN S-GRAPH SHOULD BE APPROPRIATE.

STEP 6: AT THIS POINT ALL OF THE NECESSARY PARAMETERS ARE FOUND.

A UNIT-GRAPH CAN BE DEVELOPED EITHER MANUALLY OR BY USING THE "LAPREI" PROGRAM. ALTHOUGH MANUAL CALCULATION WILL DEVIATE FROM LAPREI RESULTS DUE TO COMPUTATIONAL ERRORS.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY



PROJECT MARICOPA COUNTY Hydrology MANUAL PAGE 5 OF 9
 DETAIL EXAMPLE #9 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____

THE FOLLOWING ILLUSTRATES THE GENERATED HYDROGRAPH

| <u>ΔT (MIN)</u> | <u>DISCHARGE (CFS) *</u> <u>MANUAL CALC.</u> | <u>DISCHARGE (CFS)</u> <u>(COMPUTER OUTPUT)</u> |
|-----------------|---|--|
| 0 | 0 | 0 |
| 10 | 491 | 506 |
| 20 | 1888 | 1896 |
| 30 | 3226 | 3513 |
| 40 | 4333 | 3981 |
| 50 | 2601 | 2847 |
| 60 | 2036 | 2014 |
| 70 | 1522 | 1434 |
| 80 | 1067 | 1008 |
| 90 | 832 | 833 |
| 100 | 563 | 628 |
| 110 | 431 | 460 |
| 120 | 321 | 246 |
| 130 | 212 | 211 |
| 140 | 186 | 130 |
| 150 | 84 | 121 |
| 160 | 82 | 110 |
| 170 | 83 | 59 |
| 180 | 83 | 56 |
| 190 | 53 | 42 |
| 200 | 0 | 0 |
| 210 | 0 | 0 |

* THE HAND CALCULATED UNIT-GRAPH IS PRESENTED FOR COMPARISON, THOUGH NOT ALWAYS RECOMMENDED DUE TO POSSIBLE COMPUTATIONAL ERROR. NEXT FEW PAGES ILLUSTRATE TRANSFORMATION OF S-GRAPH TO UNIT-GRAPH.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 6 OF 9

DETAIL EXAMPLE #9 COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____



CONSTRUCTION OF A 10-minute UNIT HYDROGRAPH FROM THE "PHOENIX MOUNTAIN" DIMENSIONLESS S-CURVE

CONSIDER A WATERSHED WITH THE FOLLOWING CHARACTERISTICS:

$$A = 5.19 \text{ mi}^2$$

$$\text{LAG} = 0.673 \text{ hr}$$

$$Q_{ULT} = [645.33(A)] / D = [645(5.19)] / .167 = 20,096 \text{ cfs}$$

① CONSTRUCT A TABLE SIMILAR TO THE FOLLOWING:

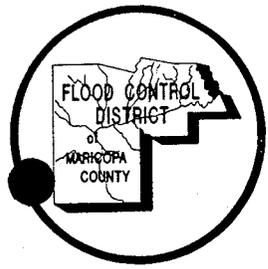
| ORDINATE | % Q _{ULT} | DISCHARGE (cfs) | % LAG | TIME (hr) |
|----------|--------------------|-----------------|-------|-----------|
| 1 | 0 | 0 | 0.0 | 0 |
| 2 | 2 | 402 | 23.0 | .155 |
| 3 | 4 | 804 | 31.0 | .209 |
| 4 | 6 | 1206 | 37.0 | .249 |
| 5 | 8 | 1608 | 42.0 | .283 |
| 6 | 10 | 2010 | 46.0 | .310 |
| 7 | 12 | 2412 | 49.9 | .335 |
| 8 | 14 | 2814 | 53.4 | .365 |
| 9 | 16 | 3216 | 56.8 | .382 |
| 10 | 18 | 3618 | 60.0 | .404 |
| 11 | 20 | 4020 | 63.1 | .425 |
| 12 | 22 | 4422 | 66.1 | .445 |
| 13 | 24 | 4824 | 69.0 | .464 |
| 14 | 26 | 5226 | 71.8 | .483 |
| 15 | 28 | 5628 | 74.4 | .501 |
| 16 | 30 | 6030 | 76.8 | .517 |
| 17 | 32 | 6432 | 79.1 | .532 |
| 18 | 34 | 6834 | 81.2 | .546 |
| 19 | 36 | 7236 | 83.2 | .560 |
| 20 | 38 | 7638 | 85.1 | .573 |
| 21 | 40 | 8040 | 86.8 | .584 |
| 22 | 42 | 8442 | 88.8 | .598 |
| 23 | 44 | 8844 | 91.0 | .612 |
| 24 | 46 | 9246 | 93.3 | .621 |
| 25 | 48 | 9648 | 96.8 | .651 |
| 26 | 50 | 10050 | 100.0 | .673 |
| 27 | 52 | 10452 | 103.4 | .696 |
| 28 | 54 | 10854 | 107.0 | .720 |
| 29 | 56 | 11256 | 110.8 | .746 |
| 30 | 58 | 11658 | 114.7 | .772 |
| 31 | 60 | 12060 | 118.7 | .799 |
| 32 | 62 | 12462 | 122.9 | .827 |
| 33 | 64 | 12864 | 127.3 | .855 |
| 34 | 66 | 13266 | 131.9 | .888 |
| 35 | 68 | 13668 | 136.7 | .920 |
| 36 | 70 | 14070 | 141.7 | .954 |
| 37 | 72 | 14472 | 147.1 | .990 |
| 38 | 74 | 14874 | 152.8 | 1.022 |
| 39 | 76 | 15276 | 158.8 | 1.060 |
| 40 | 78 | 15678 | 165.8 | 1.114 |
| 41 | 80 | 16080 | 172.9 | 1.164 |
| 42 | 82 | 16482 | 181.0 | 1.222 |
| 43 | 84 | 16884 | 191.0 | 1.299 |
| 44 | 86 | 17286 | 201.0 | 1.395 |
| 45 | 88 | 17688 | 212.0 | 1.427 |
| 46 | 90 | 18090 | 226.0 | 1.521 |
| 47 | 92 | 18492 | 244.0 | 1.642 |
| 48 | 94 | 18894 | 279.0 | 1.995 |
| 49 | 96 | 19296 | 342.0 | 2.302 |
| 50 | 98 | 19698 | 462.0 | 3.109 |
| 51 | 100 | 20100 | | |

→ (.38)(20096) =

→ (.851)(.673) =

THE 'DISCHARGE' VS. 'TIME' COLUMNS ARE THE "DIMENSIONED" S-CURVE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

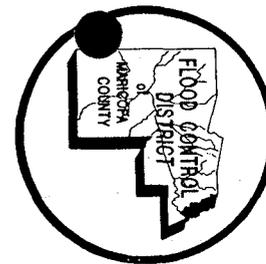


PROJECT _____ PAGE 7 OF 9
 DETAIL EXAMPLE #9 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____

② CONVERT THE S-CURVE TO A 10-MIN. UNIT HYDROGRAPH. USE LINEAR INTERPOLATION IN 10-MIN. INCREMENTS FOR TIME AND DISCHARGE VALUES

| ORDINATE | TIME (HR) | Q_{s_1} (cfs) | 10-MIN LAG Q_{s_2} (cfs) | Q_{UGRAPH} (cfs) |
|----------|-----------|--------------------|----------------------------------|-----------------------|
| 1 | 0 | 0 | 0 | 0 |
| 2 | .167 | 491 | 0 | 491 |
| 3 | .333 | 2379 | 491 | 1888 |
| 4 | .500 | 5605 | 2379 | 3226 |
| 5 | .667 | 9938 | 5605 | 4333 |
| 6 | .833 | 12539 | 9938 | 2601 |
| 7 | 1.000 | 14575 | 12539 | 2036 |
| 8 | 1.167 | 16097 | 14575 | 1522 |
| 9 | 1.333 | 17164 | 16097 | 1067 |
| 10 | 1.500 | 17996 | 17164 | 832 |
| 11 | 1.667 | 18559 | 17996 | 563 |
| 12 | 1.833 | 18990 | 18559 | 431 |
| 13 | 2.000 | 19311 | 18990 | 321 |
| 14 | 2.167 | 19523 | 19311 | 212 |
| 15 | 2.333 | 19709 | 19523 | 186 |
| 16 | 2.500 | 19793 | 19709 | 84 |
| 17 | 2.667 | 19876 | 19793 | 82 |
| 18 | 2.833 | 19959 | 19876 | 83 |
| 19 | 3.000 | 20042 | 19959 | 83 |
| 20 | 3.167 | 20096 | 20042 | 54 |
| 21 | 3.333 | | 20096 | 0 |

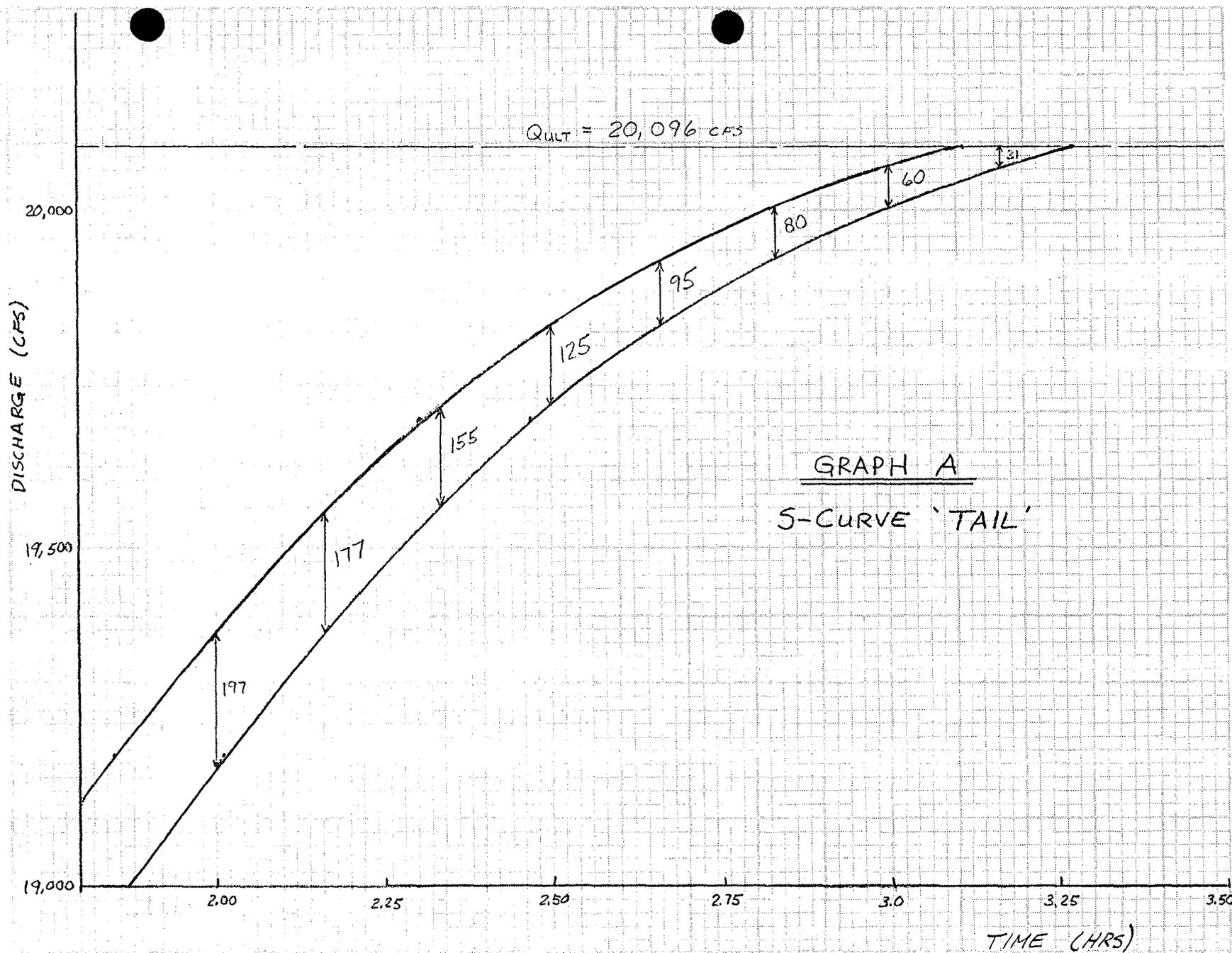
NOTICE HOW THE UNIT-GRAPH VALUES BECOME LINEAR FROM TIME 2.5 - 3.167 HRS. THIS IS CAUSED BY THE LONG TIME INCREMENTS AT END OF THE S-CURVE. TO CORRECT THIS, CONSTRUCT A GRAPH OF THE 'TAIL' REGION OF THE S-CURVE, LAG IT BY THE APPROPRIATE DURATION, AND SUBTRACT THE ORDINATES. (SEE GRAPH A)



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT _____
 DETAIL EXAMPLE #9 COMPUTED _____
 PAGE 8 OF 9

CHECKED BY _____ DATE _____





FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT EXAMPLE #9 PAGE 9 OF 9

DETAIL _____ COMPUTED _____ DATE _____

_____ CHECKED BY _____ DATE _____

FINAL 10-MIN. UNIT GRAPH

| <u>TIME (HR)</u> | <u>DISCHARGE (CFS)</u> |
|------------------|------------------------|
| 0 | 0 |
| .167 | 491 |
| .333 | 1898 |
| .500 | 3226 |
| .667 | 4333 |
| .833 | 2601 |
| 1.000 | 2036 |
| 1.167 | 1522 |
| 1.333 | 1067 |
| 1.500 | 832 |
| 1.667 | 563 |
| 1.833 | 431 |
| 2.000 | 321 |
| 2.167 | 212 |
| 2.333 | 186 |
| 2.500 | 125 |
| 2.667 | 95 |
| 2.833 | 80 |
| 3.000 | 60 |
| 3.167 | 31 |
| 3.333 | 0 |

```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1)
* FEBRUARY 1981
* REVISED 31 JAN 85
*
* RUN DATE 9/12/1989 TIME 9:10:49
*
*****

```

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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
*
*****

```

```

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 -AMSK- 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 # 9 - S-GRAPH APPLICATIONS
* *****
4 ID RAINFALL: 6-HR, 100-YEAR POINT RAINFALL DEPTH OF 3.40 INCHES
5 ID HYDROGRAPH: SCS UNIT-GRAPH, LAG TIME
6 ID LOSSES IL+ULR BY SOIL TEXTURE
7 ID BASIN AREA: 5.19 SQUARE MILES, PATTERN #2.35, AREAL REDUCTION .85
* *****
8 IT 10 05SEP89 0000 50
9 IO 0

10 KK BASIN
11 KM COMPUTE DISCHARGE AT THE OUTLET OF BASIN
12 IN 15
13 PB 2.89
14 PC .000 .009 .015 .024 .037 .047 .058 .069 .082 .091
15 PC .104 .118 .139 .184 .400 .458 .686 .823 .889 .929
16 PC .960 .981 .987 .989 1.00
17 BA 5.19
18 LU .75 .25 3
19 UI 0 506 1986 3513 3981 2847 2014 1434 1008 833
20 UI 628 460 246 211 130 121 110 59 56 42
21 UI 0 0
22 ZZ
    
```

 *
 * FLOOD HYDROGRAPH PACKAGE (HEC-1) *
 * FEBRUARY 1981 *
 * REVISED 31 JAN 85 *
 *
 * RUN DATE 9/12/1989 TIME 9:10:52 *
 *

 *
 * U.S. ARMY CORPS OF ENGINEERS *
 * THE HYDROLOGIC ENGINEERING CENTER *
 * 609 SECOND STREET *
 * DAVIS, CALIFORNIA 95616 *
 * (916) 440-3285 OR (FTS) 448-3285 *
 *

SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
 MARICOPA COUNTY HYDROLOGY MANUAL
 EXAMPLE # 9 - S-CRAPH APPLICATIONS
 RAINFALL: 6-HR, 100-YEAR POINT RAINFALL DEPTH OF 3.40 INCHES
 HYDROGRAPH: SCS UNIT-GRAPH, LAG TIME
 LOSSES IL+ULR BY SOIL TEXTURE
 BASIN AREA: 5.19 SQUARE MILES, PATTERN #2.35, AREAL REDUCTION .85

9 IO OUTPUT CONTROL VARIABLES
 IFRNT 0 PRINT CONTROL
 IPLCT 0 PLOT CONTROL
 QSCAL 0. HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA
 NMIN 10 MINUTES IN COMPUTATION INTERVAL
 IDATE 5SEP89 STARTING DATE
 ITIME 0000 STARTING TIME
 NQ 50 NUMBER OF HYDROGRAPH ORDINATES
 NDDATE 5SEP89 ENDING DATE
 NDTIME 0810 ENDING TIME

 COMPUTATION INTERVAL .17 HOURS
 TOTAL TIME BASE 8.17 HOURS

ENGLISH UNITS
 DRAINAGE AREA SQUARE MILES
 PRECIPITATION DEPTH INCHES
 LENGTH, ELEVATION FEET
 FLOW CUBIC FEET PER SECOND
 STORAGE VOLUME ACRE-FEET
 SURFACE AREA ACRES

 * *
 * BASIN *
 * *

10 KK

COMPUTE DISCHARGE AT THE OUTLET OF BASIN

12 IN

TIME DATA FOR INPUT TIME SERIES
 JXMIN 15 TIME INTERVAL IN MINUTES
 JXDATE 5SEP89 STARTING DATE
 JXTIME 0 STARTING TIME

SUBBASIN RUNOFF DATA

17 BA

SUBBASIN CHARACTERISTICS
 TAREA 5.19 SUBBASIN AREA

PRECIPITATION DATA

13 PB

STORM 2.89 BASIN TOTAL PRECIPITATION

14 PI

INCREMENTAL PRECIPITATION PATTERN

| | | | | | | | | | | |
|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| .01 | .01 | .00 | .01 | .01 | .01 | .01 | .01 | .01 | .01 | .01 |
| .01 | .01 | .01 | .01 | .01 | .01 | .01 | .01 | .01 | .03 | .09 |
| .14 | .04 | .10 | .15 | .09 | .07 | .04 | .03 | .02 | .02 | |
| .01 | .01 | .00 | .00 | .00 | .01 | | | | | |

18 LU

UNIFORM LOSS RATE
 STRTL .75 INITIAL LOSS
 CNSTL .25 UNIFORM LOSS RATE
 RTIMP 3.00 PERCENT IMPERVIOUS AREA

17 UI

INPUT UNITGRAPH, 20 ORDINATES, VOLUME = 1.00

| | | | | | | | | | |
|-------|-------|--------|--------|--------|--------|--------|--------|--------|-------|
| .0 | 506.0 | 1986.0 | 3513.0 | 3981.0 | 2847.0 | 2014.0 | 1434.0 | 1008.0 | 833.0 |
| 628.0 | 460.0 | 246.0 | 211.0 | 130.0 | 121.0 | 110.0 | 59.0 | 56.0 | 42.0 |

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 | .26 | .04 | .22 | 2411. |
| 5 | SEP | 0010 | 2 | .02 | .02 | .00 | 0. | * | 5 | SEP | 0420 | 27 | .20 | .04 | .16 | 3082. |
| 5 | SEP | 0020 | 3 | .01 | .01 | .00 | 0. | * | 5 | SEP | 0430 | 28 | .13 | .04 | .09 | 3818. |
| 5 | SEP | 0030 | 4 | .01 | .01 | .00 | 1. | * | 5 | SEP | 0440 | 29 | .08 | .04 | .04 | 4076. |
| 5 | SEP | 0040 | 5 | .02 | .02 | .00 | 3. | * | 5 | SEP | 0450 | 30 | .07 | .04 | .03 | 3716. |
| 5 | SEP | 0050 | 6 | .02 | .02 | .00 | 5. | * | 5 | SEP | 0500 | 31 | .06 | .04 | .02 | 3170. |
| 5 | SEP | 0100 | 7 | .03 | .02 | .00 | 6. | * | 5 | SEP | 0510 | 32 | .04 | .04 | .00 | 2535. |
| 5 | SEP | 0110 | 8 | .02 | .02 | .00 | 7. | * | 5 | SEP | 0520 | 33 | .03 | .03 | .00 | 1977. |
| 5 | SEP | 0120 | 9 | .02 | .02 | .00 | 9. | * | 5 | SEP | 0530 | 34 | .01 | .01 | .00 | 1514. |
| 5 | SEP | 0130 | 10 | .02 | .02 | .00 | 10. | * | 5 | SEP | 0540 | 35 | .00 | .00 | .00 | 1158. |
| 5 | SEP | 0140 | 11 | .02 | .02 | .00 | 11. | * | 5 | SEP | 0550 | 36 | .01 | .01 | .00 | 835. |
| 5 | SEP | 0150 | 12 | .02 | .02 | .00 | 11. | * | 5 | SEP | 0600 | 37 | .02 | .02 | .00 | 599. |
| 5 | SEP | 0200 | 13 | .03 | .02 | .00 | 12. | * | 5 | SEP | 0610 | 38 | .00 | .00 | .00 | 439. |
| 5 | SEP | 0210 | 14 | .02 | .02 | .00 | 12. | * | 5 | SEP | 0620 | 39 | .00 | .00 | .00 | 309. |
| 5 | SEP | 0220 | 15 | .02 | .02 | .00 | 13. | * | 5 | SEP | 0630 | 40 | .00 | .00 | .00 | 240. |
| 5 | SEP | 0230 | 16 | .03 | .02 | .00 | 13. | * | 5 | SEP | 0640 | 41 | .00 | .00 | .00 | 183. |
| 5 | SEP | 0240 | 17 | .03 | .03 | .00 | 13. | * | 5 | SEP | 0650 | 42 | .00 | .00 | .00 | 122. |
| 5 | SEP | 0250 | 18 | .03 | .03 | .00 | 13. | * | 5 | SEP | 0700 | 43 | .00 | .00 | .00 | 92. |
| 5 | SEP | 0300 | 19 | .04 | .04 | .00 | 14. | * | 5 | SEP | 0710 | 44 | .00 | .00 | .00 | 62. |
| 5 | SEP | 0310 | 20 | .09 | .08 | .00 | 15. | * | 5 | SEP | 0720 | 45 | .00 | .00 | .00 | 35. |
| 5 | SEP | 0320 | 21 | .25 | .24 | .01 | 17. | * | 5 | SEP | 0730 | 46 | .00 | .00 | .00 | 20. |
| 5 | SEP | 0330 | 22 | .42 | .05 | .37 | 24. | * | 5 | SEP | 0740 | 47 | .00 | .00 | .00 | 10. |
| 5 | SEP | 0340 | 23 | .11 | .04 | .07 | 223. | * | 5 | SEP | 0750 | 48 | .00 | .00 | .00 | 5. |
| 5 | SEP | 0350 | 24 | .28 | .04 | .24 | 811. | * | 5 | SEP | 0800 | 49 | .00 | .00 | .00 | 3. |
| 5 | SEP | 0400 | 25 | .44 | .04 | .40 | 1594. | * | 5 | SEP | 0810 | 50 | .00 | .00 | .00 | 1. |

TOTAL RAINFALL = 2.89, TOTAL LOSS = 1.24, TOTAL EXCESS = 1.65

| PEAK FLOW (CFS) | TIME (HRS) | MAXIMUM AVERAGE FLOW | | | | (CFS) | (HR) |
|--------------------|---------------|----------------------|-------|-------|---------|-------|------|
| | | 6-HR | 24-HR | 72-HR | 8.17-HR | | |
| 4076. | 4.67 | 922. | 679. | 679. | 679. | | |
| | (INCHES) | 1.651 | 1.654 | 1.654 | 1.654 | | |
| | (AC-FT) | 457. | 458. | 458. | 458. | | |

CUMULATIVE AREA = 5.19 SQ MI

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

| 6-HOUR | OPERATION | STATION | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD | | | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|--------|---------------|---------|-----------|--------------|---------------------------------|------|------|------------|---------------|-------------------|
| | 24-HOUR | 72-HOUR | | | | | | | | |
| | HYDROGRAPH AT | BASIN | 4076. | 4.67 | 922. | 679. | 679. | 5.19 | | |

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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

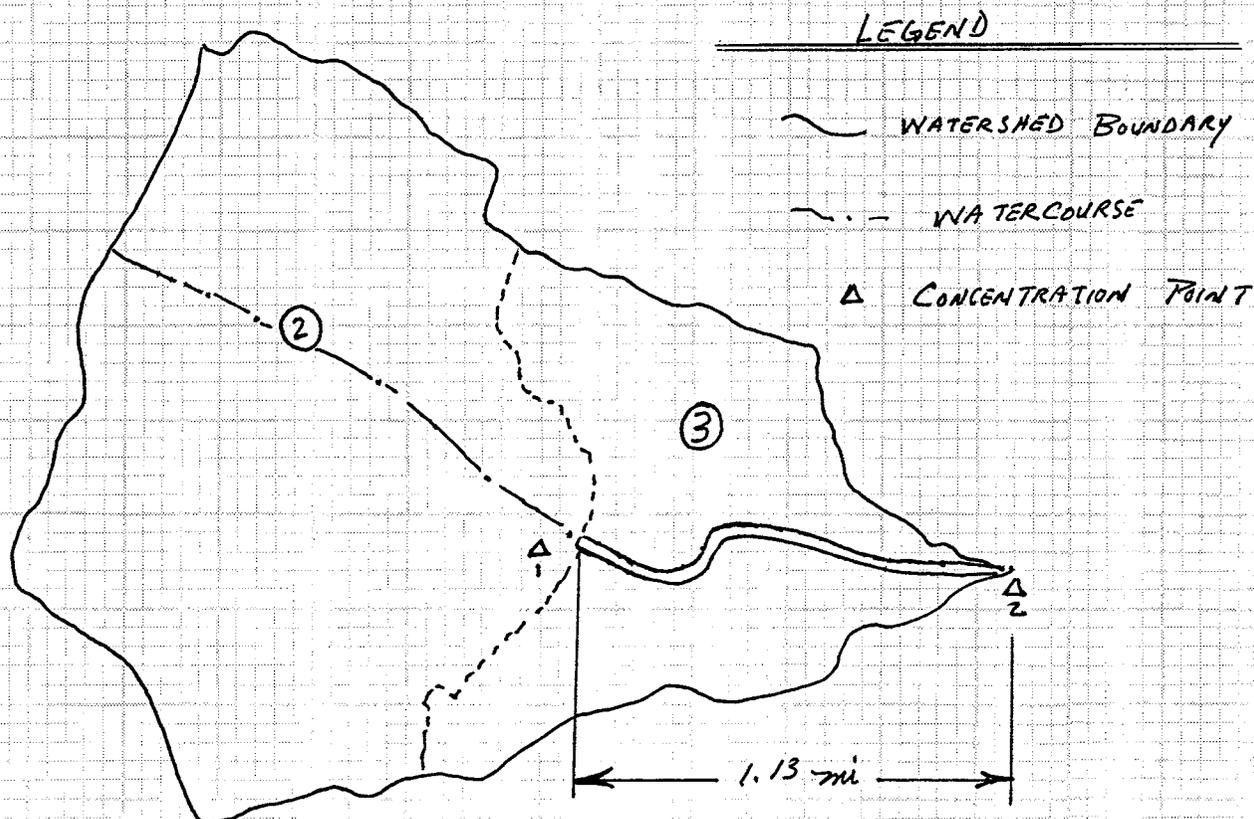
PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 2

DETAIL EXAMPLE # 10 COMPUTED _____ DATE _____

CHECKED BY _____ DATE _____



KINEMATIC WAVE ROUTING

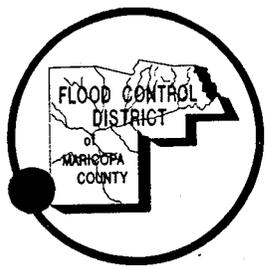


SCENARIO: PREVIOUSLY, IN EXAMPLE # 7, PEAK FLOW AT CONCENTRATION POINT 1 (Δ_1) WAS CALCULATED FOR SUB-BASIN (2). THIS EXAMPLE WILL USE KINEMATIC WAVE ROUTING THROUGH THE 1.13 MI LONG REACH TO CALCULATE THE PEAK AT CONCENTRATION POINT 2 (Δ_2).

STEP 1: COLLECT NECESSARY DATA FOR THE CHANNEL.

TYPE: CONCRETE LINED, TRAPEZOIDAL

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY



PROJECT MARICOPA COUNTY Hydrology Manual PAGE 2 OF 2
DETAIL _____ COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____

LENGTH : 1.13 mi

AVG. TOP WIDTH : 20 ft

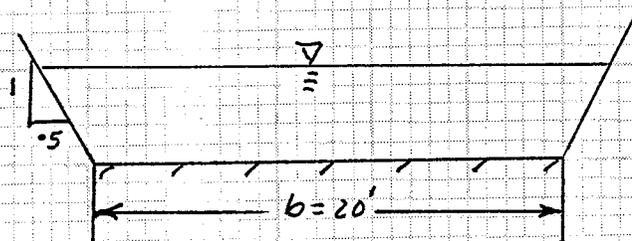
AVG. DEPTH : 4 ft

SIDE SLOPE : : 0.5

MANNING'S n : .015

BOTTOM WIDTH : 20 ft

CHANNEL SLOPE : .018 ft/ft



RUN THE HEC-1.

```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
* FEBRUARY 1981 *
* REVISED 31 JAN 85 *
*
* RUN DATE 9/ 8/1989 TIME11:18: 8 *
*
*****

```

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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 *
*
*****

```

```

X X XXXXXXX XXXXX X
X X X X X XX
X X X X X
XXXXXXXX XXXX X XXXXX 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 -AMSK- 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 20. 0.5
22 ZZ
    
```

 *
 * FLOOD HYDROGRAPH PACKAGE (HEC-1) *
 * FEBRUARY 1981 *
 * REVISED 31 JAN 85 *
 *
 * RUN DATE 9/ 8/1989 TIME11:18:13 *
 *

 *
 * U.S. ARMY CORPS OF ENGINEERS *
 * THE HYDROLOGIC ENGINEERING CENTER *
 * 609 SECOND STREET *
 * DAVIS, CALIFORNIA 95616 *
 * (916) 440-3285 OR (FTS) 448-3285 *
 *

SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
 MARICOPA COUNTY HYDROLOGY MANUAL
 EXAMPLE # 10 - KINEMATIC WAVE ROUTING

5 IO OUTPUT CONTROL VARIABLES

IPRNT 3 PRINT CONTROL
 IPLOT 0 PLOT CONTROL
 QSCAL 0. HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA

NMIN 5 MINUTES IN COMPUTATION INTERVAL
 IDATE 5SEP89 STARTING DATE
 ITIME 0000 STARTING TIME
 NQ 75 NUMBER OF HYDROGRAPH ORDINATES
 NDDATE 5SEP89 ENDING DATE
 NDTIME 0610 ENDING TIME

COMPUTATION INTERVAL .08 HOURS
 TOTAL TIME BASE 6.17 HOURS

ENGLISH UNITS

DRAINAGE AREA SQUARE MILES
 PRECIPITATION DEPTH INCHES
 LENGTH, ELEVATION FEET
 FLOW CUBIC FEET PER SECOND
 STORAGE VOLUME ACRE-FEET
 SURFACE AREA ACRES
 TEMPERATURE DEGREES FAHRENHEIT

*** **

*
 *
 6 KK * BASIN2 *
 *

COMPUTE DISCHARGE AT THE OUTLET OF BASIN #2

8 IN TIME DATA FOR INPUT TIME SERIES

JXMIN 15 TIME INTERVAL IN MINUTES
 JXDATE 5SEP89 STARTING DATE
 JXTIME 0 STARTING TIME

SUBBASIN RUNOFF DATA

13 BA SUBBASIN CHARACTERISTICS
 TAREA 2.17 SUBBASIN AREA

PRECIPITATION DATA

9 PB STORM 3.25 BASIN TOTAL PRECIPITATION

10 PI INCREMENTAL PRECIPITATION PATTERN

| | | | | | | | | | |
|------|------|------|------|------|-------|-------|-------|------|------|
| .18 | .18 | .18 | .17 | .17 | .17 | .22 | .22 | .22 | .32 |
| .32 | .32 | .27 | .27 | .27 | .30 | .30 | .30 | .28 | .28 |
| .28 | .28 | .28 | .28 | .28 | .28 | .28 | .40 | .40 | .40 |
| .43 | .43 | .43 | .65 | .65 | .65 | 1.05 | 1.05 | 1.05 | 2.78 |
| 2.78 | 2.78 | 6.00 | 6.00 | 6.00 | 11.67 | 11.67 | 11.67 | 3.67 | 3.67 |
| 3.67 | 1.38 | 1.38 | 1.38 | .95 | .95 | .95 | .70 | .70 | .70 |
| .52 | .52 | .52 | .28 | .28 | .28 | .18 | .18 | .18 | .37 |
| .37 | .37 | | | | | | | | |

14 LU UNIFORM LOSS RATE
 STRTL .65 INITIAL LOSS
 CNSTL .20 UNIFORM LOSS RATE
 RTIMP 21.00 PERCENT IMPERVIOUS AREA

17 UC CLARK UNITGRAPH
 TC .44 TIME OF CONCENTRATION
 R .16 STORAGE COEFFICIENT

13 UA ACCUMULATED-AREA VS. TIME, 11 ORDINATES
 .0 5.0 16.0 30.0 65.0 77.0 84.0 90.0 94.0 97.0
 100.0

UNIT HYDROGRAPH PARAMETERS

CLARK TC= .44 HR, R= .16 HR
 SNYDER TP= .23 HR, CP= .64

UNIT HYDROGRAPH

13 END-OF-PERIOD ORDINATES

| | | | | | | | | | |
|------|-------|-------|-------|-------|-------|-------|------|------|------|
| 525. | 2343. | 3727. | 3386. | 2548. | 1746. | 1066. | 617. | 357. | 206. |
| 119. | 69. | 40. | | | | | | | |

*** **

HYDROGRAPH AT STATION BASIN2

TOTAL RAINFALL - 3.25, TOTAL LOSS - .87, TOTAL EXCESS - 2.38

| PEAK FLOW (CFS) | TIME | MAXIMUM AVERAGE FLOW | | | | (CFS) | (HR) |
|--------------------|----------|----------------------|-------|-------|---------|-------|------|
| | | 6-HR | 24-HR | 72-HR | 6.17-HR | | |
| 4552. | 4.08 | 553. | 538. | 538. | 538. | | |
| | (INCHES) | 2.370 | 2.370 | 2.370 | 2.370 | | |
| | (AC-FT) | 274. | 274. | 274. | 274. | | |

CUMULATIVE AREA - 2.17 SQ MI

* *
* ROUTE *
* *

18 KK

ROUTE THROUGH DOWNSTREAM BASIN USING KINEMATIC 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 20.00 BOTTOM WIDTH OR DIAMETER
Z .50 SIDE SLOPE

KINEMATIC STREAM ROUTING USED FOR THIS REACH

COMPUTED KINEMATIC PARAMETERS

| ALPHA | M | DT (MIN) | DX (FT) |
|--------|-------|----------|---------|
| 2.2392 | 1.559 | .71 | 2983.20 |

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 | 62. | * | 5 | SEP | 0445 | 58 | 866. | * |
| 5 | SEP | 0005 | 2 | 0. | * | 5 | SEP | 0140 | 21 | 33. | * | 5 | SEP | 0315 | 40 | 71. | * | 5 | SEP | 0450 | 59 | 658. | * |
| 5 | SEP | 0010 | 3 | 0. | * | 5 | SEP | 0145 | 22 | 33. | * | 5 | SEP | 0320 | 41 | 84. | * | 5 | SEP | 0455 | 60 | 508. | * |
| 5 | SEP | 0015 | 4 | 0. | * | 5 | SEP | 0150 | 23 | 33. | * | 5 | SEP | 0325 | 42 | 104. | * | 5 | SEP | 0500 | 61 | 395. | * |
| 5 | SEP | 0020 | 5 | 0. | * | 5 | SEP | 0155 | 24 | 33. | * | 5 | SEP | 0330 | 43 | 147. | * | 5 | SEP | 0505 | 62 | 310. | * |
| 5 | SEP | 0025 | 6 | 2. | * | 5 | SEP | 0200 | 25 | 33. | * | 5 | SEP | 0335 | 44 | 275. | * | 5 | SEP | 0510 | 63 | 247. | * |
| 5 | SEP | 0030 | 7 | 5. | * | 5 | SEP | 0205 | 26 | 33. | * | 5 | SEP | 0340 | 45 | 648. | * | 5 | SEP | 0515 | 64 | 197. | * |
| 5 | SEP | 0035 | 8 | 9. | * | 5 | SEP | 0210 | 27 | 33. | * | 5 | SEP | 0345 | 46 | 1278. | * | 5 | SEP | 0520 | 65 | 156. | * |
| 5 | SEP | 0040 | 9 | 12. | * | 5 | SEP | 0215 | 28 | 33. | * | 5 | SEP | 0350 | 47 | 1946. | * | 5 | SEP | 0525 | 66 | 123. | * |
| 5 | SEP | 0045 | 10 | 15. | * | 5 | SEP | 0220 | 29 | 33. | * | 5 | SEP | 0355 | 48 | 2730. | * | 5 | SEP | 0530 | 67 | 98. | * |
| 5 | SEP | 0050 | 11 | 18. | * | 5 | SEP | 0225 | 30 | 33. | * | 5 | SEP | 0400 | 49 | 3669. | * | 5 | SEP | 0535 | 68 | 78. | * |
| 5 | SEP | 0055 | 12 | 20. | * | 5 | SEP | 0230 | 31 | 34. | * | 5 | SEP | 0405 | 50 | 4402. | * | 5 | SEP | 0540 | 69 | 63. | * |
| 5 | SEP | 0100 | 13 | 22. | * | 5 | SEP | 0235 | 32 | 36. | * | 5 | SEP | 0410 | 51 | 4510. | * | 5 | SEP | 0545 | 70 | 52. | * |
| 5 | SEP | 0105 | 14 | 25. | * | 5 | SEP | 0240 | 33 | 38. | * | 5 | SEP | 0415 | 52 | 4045. | * | 5 | SEP | 0550 | 71 | 43. | * |
| 5 | SEP | 0110 | 15 | 28. | * | 5 | SEP | 0245 | 34 | 41. | * | 5 | SEP | 0420 | 53 | 3391. | * | 5 | SEP | 0555 | 72 | 37. | * |
| 5 | SEP | 0115 | 16 | 30. | * | 5 | SEP | 0250 | 35 | 43. | * | 5 | SEP | 0425 | 54 | 2711. | * | 5 | SEP | 0600 | 73 | 33. | * |
| 5 | SEP | 0120 | 17 | 31. | * | 5 | SEP | 0255 | 36 | 46. | * | 5 | SEP | 0430 | 55 | 2069. | * | 5 | SEP | 0605 | 74 | 32. | * |
| 5 | SEP | 0125 | 18 | 31. | * | 5 | SEP | 0300 | 37 | 50. | * | 5 | SEP | 0435 | 56 | 1546. | * | 5 | SEP | 0610 | 75 | 32. | * |
| 5 | SEP | 0130 | 19 | 32. | * | 5 | SEP | 0305 | 38 | 55. | * | 5 | SEP | 0440 | 57 | 1155. | * | | | | | | * |

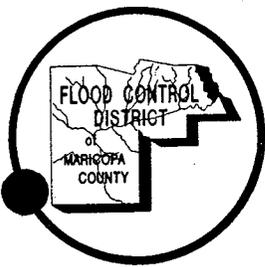
| PEAK FLOW (CFS) | TIME (HRS) | MAXIMUM AVERAGE FLOW | | | | 6.17-HR (CFS) | (HR) |
|--------------------|---------------|----------------------|-------|-------|-------|------------------|------|
| | | 6-HR | 24-HR | 72-HR | | | |
| 4510. | 4.17 | 552. | 537. | 537. | 537. | | |
| | (INCHES) | 2.365 | 2.365 | 2.365 | 2.365 | | |
| | (AC-FT) | 274. | 274. | 274. | 274. | | |

CUMULATIVE AREA = 2.17 SQ MI

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

| 6-HOUR | OPERATION | | PEAK FLOW | TIME OF PEAK | AVERAGE FLOW FOR MAXIMUM PERIOD | | | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|--------|---------------|---------|-----------|--------------|---------------------------------|------|------|------------|---------------|-------------------|
| | 24-HOUR | 72-HOUR | | | | | | | | |
| | HYDROGRAPH AT | BASIN2 | 4552. | 4.08 | 553. | 538. | 538. | 2.17 | | |
| | ROUTED TO | ROUTE | 4510. | 4.17 | 552. | 537. | 537. | 2.17 | | |

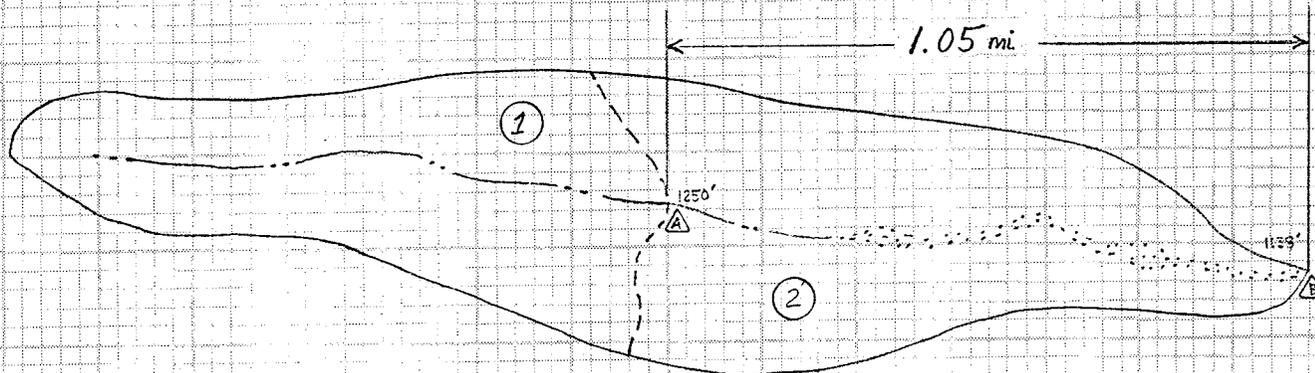
*** NORMAL END OF HEC-1 ***



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 2
 DETAIL EXAMPLE # 11 COMPUTED _____ DATE _____
 CHECKED BY _____ DATE _____

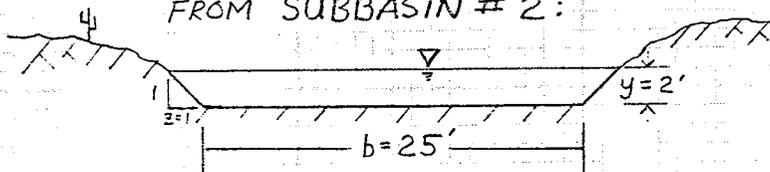
MUSKINGUM ROUTING



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:



CALCULATE VELOCITY:

$$A = (b + zy)y = (25 + (1)(2))2 = 54 \text{ ft}^2$$

$$P = b + 2y(1 + z^2)^{1/2} = 25 + (2)(2)(1 + (1)^2)^{1/2} = 30.66 \text{ ft}$$

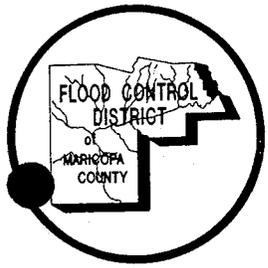
$$R = A/P = 54 \text{ ft}^2 / 30.66 \text{ ft} = 1.761 \text{ ft}$$

$$S = .0170 \text{ ft/ft}$$

$$n = .040$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{.040} (1.761)^{2/3} (.0170)^{1/2} = 7.08 \text{ ft/s}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY



PROJECT _____ PAGE 2 OF 2
DETAIL _____ COMPUTED _____ DATE _____
CHECKED BY _____ DATE _____

CALCULATE K :

$$K = 1.25 \text{ mi} \times 5280 \frac{\text{ft}}{\text{mi}} \times \frac{1}{7.08} \frac{\text{s}}{\text{ft}} \times \frac{1}{3600} \frac{\text{s}}{\text{hr}} = 0.259 \text{ hr}$$

ESTIMATE X :

SINCE THE CHANNEL IS SHALLOW WITH MODERATE TO LOW SLOPE, CHOOSE $X = 0.2$

CHECK NSTPS :

NSTPS MUST BE WITHIN THE FOLLOWING LIMITS: $\frac{1}{2(1-X)} \leq \frac{(AMSXK \times 60)}{(N_{MIN} \times NSTPS)} \leq \frac{1}{2X}$

TRY NSTPS = 1 :

$$\frac{1}{2(1-.2)} \leq \frac{.259(60)}{(5)(1)} \leq \frac{1}{2(.2)}$$
$$.625 \leq 3.11 \leq 2.5 \rightarrow \text{No Good!}$$

TRY NSTPS = 2 :

$$\frac{.259(60)}{5(2)} = 1.55 \quad \underline{\underline{OK}} \quad \text{NSTPS} = \underline{\underline{2}}$$

STEP 2 :

ENTER MUSKINGUM PARAMETER INTO AN HEC-1 INPUT FILE ON THE RM CARD. HAND CALCULATION TECHNIQUES CAN BE FOUND IN MOST HYDROLOGY TEXTBOOKS.

```

*****
*
* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
*   FEBRUARY 1981                   *
*   REVISED 31 JAN 85               *
*
* RUN DATE 9/12/1989   TIME11:13:38 *
*
*****

```

```

*****
*
* U.S. ARMY CORPS OF ENGINEERS *
* THE HYDROLOGIC ENGINEERING CENTER *
*   609 SECOND STREET           *
*   DAVIS, CALIFORNIA 95616     *
* (916) 440-3285 OR (PTS) 448-3285 *
*
*****

```

```

X   X  XXXXXXXX  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  XXXXXXXX  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 # 11

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 #11 - MUSKINGUM ROUTING
* *****
4 IT 5 09SEP89 0000 45
5 IO 3
* *****

6 KK INFLOW
7 KM INPUT INFLOW HYDROGRAPH
8 IN 5 09SEP89 0000
9 PB 2.70
10 PC 0. 1.1 1.8 2.3 2.8 3.2 4.6 7.1 10. 13.7
11 PC 17.6 23.2 32.7 60.1 74.3 86.3 90.1 93. 95.4 96.2
12 PC 97. 97.9 98.2 99.2 100.
13 BA 2.75
14 LU .67 .20
15 UA 0 3 5 8 12 20 43 75 90 96
16 UA 100
17 UC .325 .254
* *****

18 KK ROUTE
19 KM ROUTE THROUGH DOWNSTREAM BASIN USING MUSKINGUM ROUTING
20 KO 1 2
21 RM 2 .259 .2
22 ZZ
    
```

* *
6 KK * INFLOW *
* *

INPUT INFLOW HYDROGRAPH

8 IN TIME DATA FOR INPUT TIME SERIES
JXMIN 5 TIME INTERVAL IN MINUTES
JXDATE 9SEP89 STARTING DATE
JXTIME 0 STARTING TIME

SUBBASIN RUNOFF DATA

13 BA SUBBASIN CHARACTERISTICS
TAREA 2.75 SUBBASIN AREA

PRECIPITATION DATA

9 PB STORM 2.70 BASIN TOTAL PRECIPITATION

10 PI INCREMENTAL PRECIPITATION PATTERN
1.10 .70 .50 .50 .40 1.40 2.50 2.90 3.70 3.90
5.60 9.50 27.40 14.20 12.00 3.80 2.90 2.40 .80 .80
.90 .30 1.00 .80

14 LU UNIFORM LOSS RATE
STRTL .67 INITIAL LOSS
CNSTL .20 UNIFORM LOSS RATE
RTIMP .00 PERCENT IMPERVIOUS AREA

17 UC CLARK UNITGRAPH
TC .32 TIME OF CONCENTRATION
R .25 STORAGE COEFFICIENT

13 UA ACCUMULATED-AREA VS. TIME, 11 ORDINATES
.0 3.0 5.0 8.0 12.0 20.0 43.0 75.0 90.0 96.0
100.0

HYDROGRAPH AT STATION ROUTE

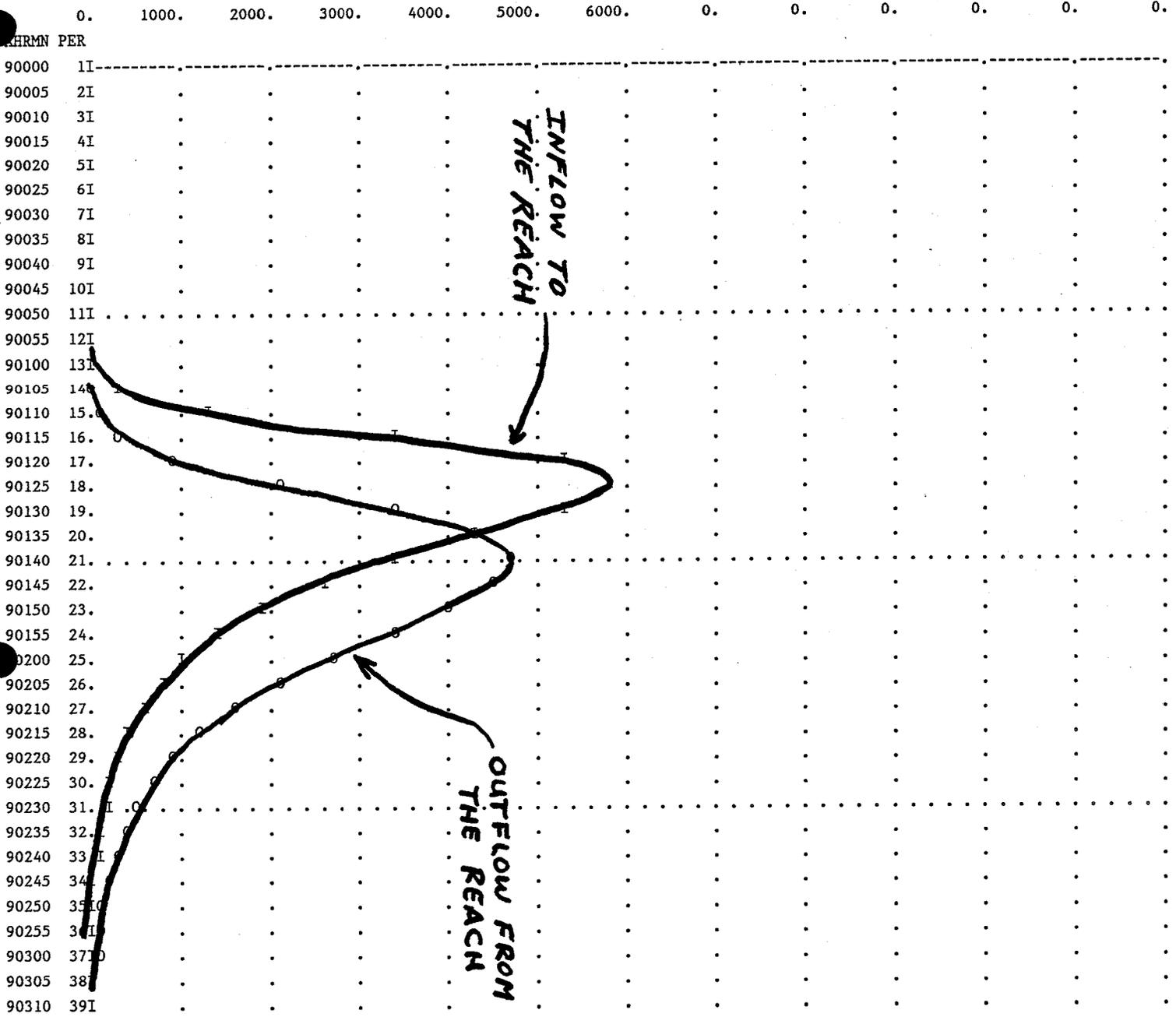
| DA | MON | HRMN | ORD | FLOW | * | DA | MON | HRMN | ORD | FLOW | * | DA | MON | HRMN | ORD | FLOW | * | DA | MON | HRMN | ORD | FLOW | |
|----|-----|------|-----|------|---|----|-----|------|-----|-------|---|----|-----|------|-----|-------|---|----|-----|------|-----|------|--|
| 9 | SEP | 0000 | 1 | 0. | * | 9 | SEP | 0100 | 13 | 0. | * | 9 | SEP | 0200 | 25 | 2708. | * | 9 | SEP | 0300 | 37 | 57. | |
| 9 | SEP | 0005 | 2 | 0. | * | 9 | SEP | 0105 | 14 | 8. | * | 9 | SEP | 0205 | 26 | 2114. | * | 9 | SEP | 0305 | 38 | 35. | |
| 9 | SEP | 0010 | 3 | 0. | * | 9 | SEP | 0110 | 15 | 62. | * | 9 | SEP | 0210 | 27 | 1619. | * | 9 | SEP | 0310 | 39 | 21. | |
| 9 | SEP | 0015 | 4 | 0. | * | 9 | SEP | 0115 | 16 | 286. | * | 9 | SEP | 0215 | 28 | 1229. | * | 9 | SEP | 0315 | 40 | 13. | |
| 9 | SEP | 0020 | 5 | 0. | * | 9 | SEP | 0120 | 17 | 911. | * | 9 | SEP | 0220 | 29 | 930. | * | 9 | SEP | 0320 | 41 | 8. | |
| 9 | SEP | 0025 | 6 | 0. | * | 9 | SEP | 0125 | 18 | 2061. | * | 9 | SEP | 0225 | 30 | 700. | * | 9 | SEP | 0325 | 42 | 5. | |
| 9 | SEP | 0030 | 7 | 0. | * | 9 | SEP | 0130 | 19 | 3379. | * | 9 | SEP | 0230 | 31 | 521. | * | 9 | SEP | 0330 | 43 | 3. | |
| 9 | SEP | 0035 | 8 | 0. | * | 9 | SEP | 0135 | 20 | 4336. | * | 9 | SEP | 0235 | 32 | 385. | * | 9 | SEP | 0335 | 44 | 2. | |
| 9 | SEP | 0040 | 9 | 0. | * | 9 | SEP | 0140 | 21 | 4695. | * | 9 | SEP | 0240 | 33 | 281. | * | 9 | SEP | 0340 | 45 | 1. | |
| 9 | SEP | 0045 | 10 | 0. | * | 9 | SEP | 0145 | 22 | 4516. | * | 9 | SEP | 0245 | 34 | 201. | * | | | | | | |
| 9 | SEP | 0050 | 11 | 0. | * | 9 | SEP | 0150 | 23 | 4012. | * | 9 | SEP | 0250 | 35 | 138. | * | | | | | | |
| 9 | SEP | 0055 | 12 | 0. | * | 9 | SEP | 0155 | 24 | 3368. | * | 9 | SEP | 0255 | 36 | 91. | * | | | | | | |

| PEAK FLOW (CFS) | TIME (HR) | MAXIMUM AVERAGE FLOW | | | |
|--------------------|--------------|----------------------|-------|-------|---------|
| | | 6-HR | 24-HR | 72-HR | 3.67-HR |
| 4695 | 1.67 | (CFS) 879. | 879. | 879. | 879. |
| | | (INCHES) 1.817 | 1.817 | 1.817 | 1.817 |
| | | (AC-FT) 266. | 266. | 266. | 266. |

CUMULATIVE AREA = 2.75 SQ MI

STATION ROUTE

(I) INFLOW, (O) OUTFLOW



INFLOW TO THE REACH

OUTFLOW FROM THE REACH

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 | | | BASIN AREA | MAXIMUM STAGE | TIME OF MAX STAGE |
|---------------|---------|-----------|--------------|---------------------------------|---------|---------|------------|---------------|-------------------|
| | | | | 6-HOUR | 24-HOUR | 72-HOUR | | | |
| HYDROGRAPH AT | INFLOW | 5761. | 1.42 | 879. | 879. | 879. | 2.75 | | |
| ROUTED TO | ROUTE | 4695. | 1.67 | 879. | 879. | 879. | 2.75 | | |

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

APPENDIX

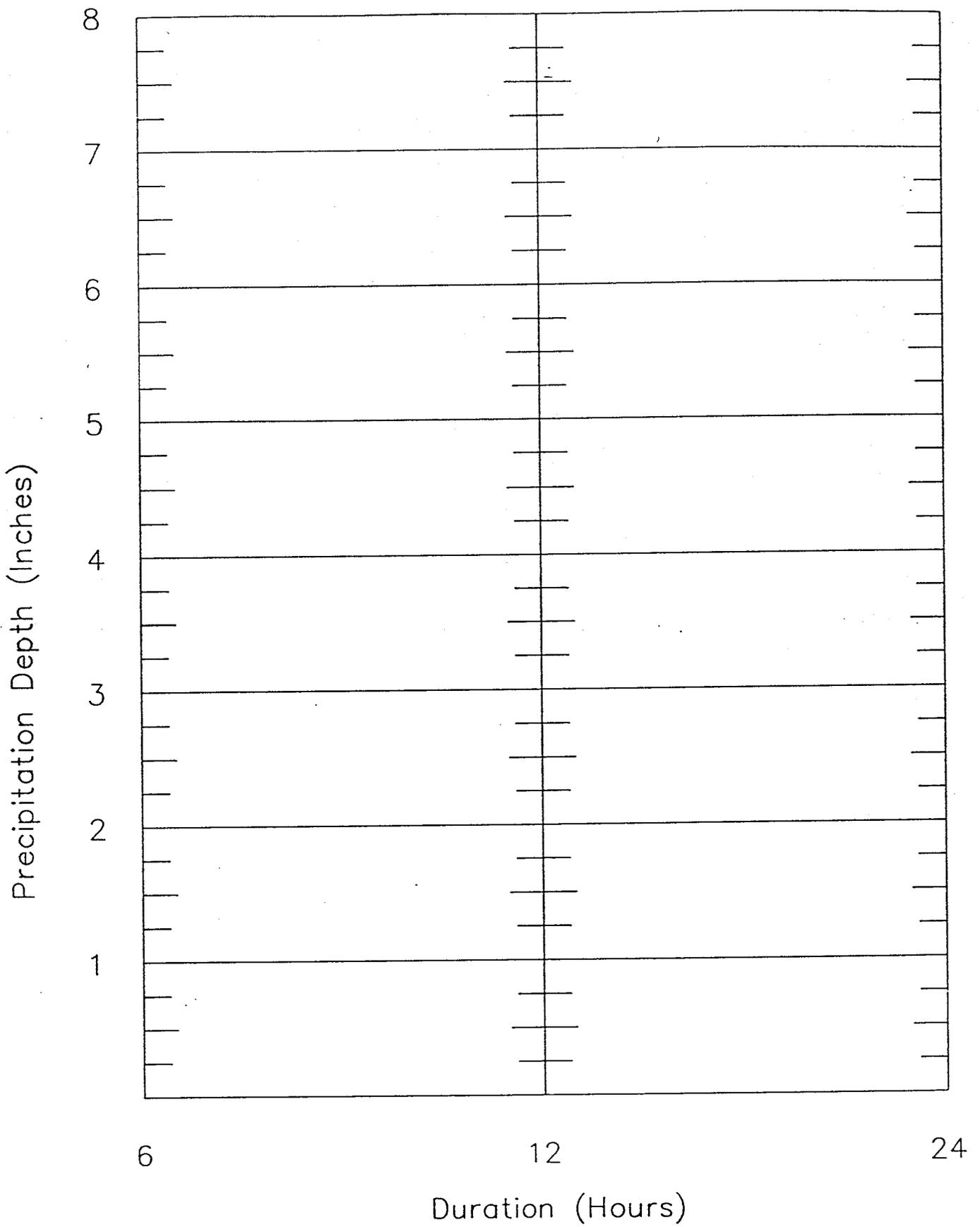
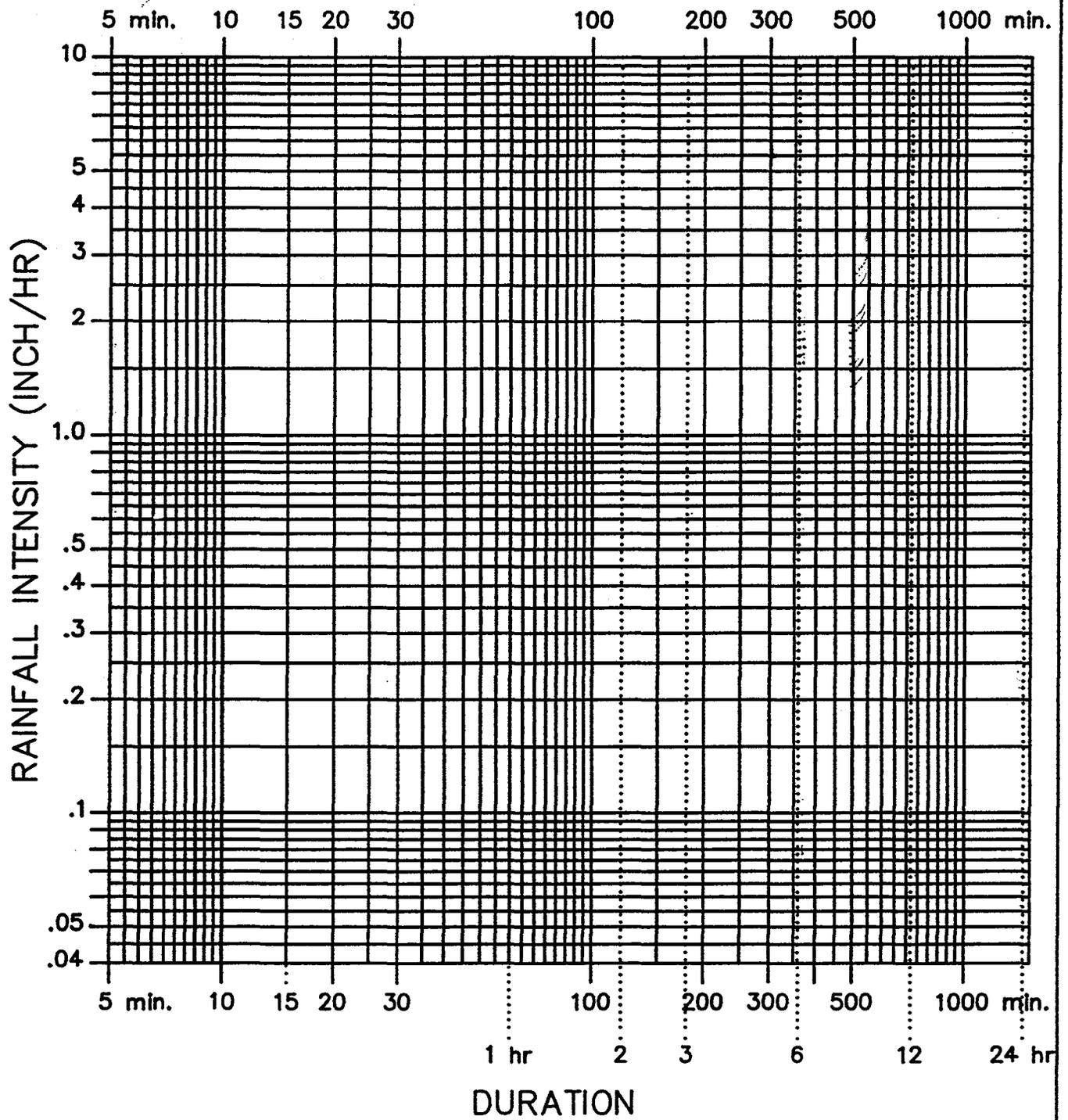


Figure 2.6. Precipitation depth-duration diagram (6 to 24 hr).



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