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MARICOPA COUNTY HYDROLOGY MANUAL

PART A - USERS MANUAL

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 part~~s~~ 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

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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 H1

2.1 GENERAL H2

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.

H3 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 remnents of these storms can be caught up in the large scale circulation around a low pressure center in southern California and therefore can bring a persistant 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.

A2 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 Deparment 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

F2.1 – 2.4

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 applies the NOAA procedures which led to the 100-year, 6-hour mass curve for small areas up to 0.5 miles. 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.

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~~ is used ~~which is~~ the established method in NOAA, 1973. Note: The only exception is the use of the values by Arkell and Richards (1986) for durations of less than 1 hour.

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

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

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

where:

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

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

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

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

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

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

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

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

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

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

Compute 5-min = 0.34(1-hr)

Compute 10-min = 0.51(1-hr)

Compute 15-min = 0.62(1-hr)

Compute 30-min = 0.82(1-hr)

At this point the data includes all depths for the 100-year and the 2-year frequencies, for all durations. Depths for 5-, 10-, 25-, and 50-year frequencies will be estimated by reading the corresponding values from Figure 2.5. A rainfall mass curve can then be constructed by nesting around a desired duration, i.e., 15-in, or 30-min (see example #3).

H2 **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).

H3 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. T2.1 F2.7

H3 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 have been developed. Pattern #1 applies NOAA procedures to Phoenix Airport data. Patterns #2 through 5 are intended to provide variability of rainfall intensity as a function of drainage area. A set of rainfall patterns has been developed in "Design Memorandum No. 1, Gila River Basin", based on the historic event of Aug. 19, 1954 over Queen Creek area (US Army Corps of Engineers, 1974). This information is modified for a 6-hour duration rainfall. A rainfall pattern can be selected from Table 2.2 for direct input T2.2

into HEC-1, once the size of the drainage area is determined. Figure 2.8 illustrates the dimensionless rainfall patterns. The following should be used when selecting a rainfall pattern, ~~which is~~ (also shown in Figure 2.9): <Fig2.8 <Fig2.9

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.

H3 2.5.3 24-hour Storm Distribution. In those cases where a 24-hour distribution is found suitable, the SCS TYPE II distribution may be used. Table 2.3 shows the 24-hour mass curve distribution and Figure 2.10 illustrates it in graphical form. <T 2.3 <F 2.10

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

Table 2.1., 2-hour storm distribution 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.

Time (hours)	%Rainfall Depth	Time (hours)	%Rainfall Depth
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

RATIONAL METHOD H1

H2 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. In such cases the peak discharge and the volume of runoff from rainfall events up to and including the 100-year 2-hour duration storm falling within the boundaries of the proposed development are to be retained. If the development involves channel routing, the procedures given in 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:

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

where

Q = the runoff (cfs) from a given area.

C = a coefficient relating the runoff to rainfall.

i = average rainfall intensity (inches/hour), lasting for a Tc.

Tc = the time of concentration (hours).

A = drainage area (acres).

The Rational 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, Tc, which is defined to be the time for water to flow from the most remote part of the watershed to the point of design. 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):

$$Tc = 11.4\, L^{.5}\, K_b^{.52}\, S^{-.31}\, i^{-.38} \qquad (2)$$

where

Tc = time of concentration (hours).

L = length of flow path.

K_b = resistance coefficient (Figure 3.1). F 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.

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 for the peak discharge is the same as the frequency for the rainfall producing that event.
4. The runoff coefficient would remain the same for all storms for a given watershed.

3.4 LIMITATIONS

Application of the Rational 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.

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 T. 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 F. 3.2

Streets		
	Asphaltic	0.70 - 0.95
	Concrete	0.80 - 0.95
	Gravel roadways & shoulders	0.40 - 0.60
Industrial Areas		
	Flat commercial (about %90 impervious)	0.80
	Heavy areas	0.60 - 0.90
	Light areas	0.50 - 0.80
Business Areas		
	Downtown areas	0.70 - 0.95
	Neighberhood 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.

indicated on the I-D-F curve for a frequency that includes the 100-year. The intensity value of the corresponding Tc in the above is for the Phoenix area. Use it in the following equation for application in other areas:

$$i = ip(P^{6}_{10})/2.07 \tag{3}$$

where

- i = the desired intensity for a given duration and frequency.
- ip = the intensity for the Phoenix area.
- P^{6}_{10} = the 10-year, 6-hour precipitation depth at the point of interest. It can be read from Plate 3.

4. Use the adjusted 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.

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:

$$V = C(P/12)A \tag{4}$$

where

- V = calculated volume (acre-feet).
- C = runoff coefficient from Table 3.1. (T 3.1)
- P = 100-year, 2-hour rainfall depth (inches).
- A = drainage area (acres).

RAINFALL LOSSES

4.1 GENERAL

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

Interception, also illustrated in Figure 4.1, varies depending upon the type of vegetation, maturity, and extent of canopy cover. Experimental data F 4.1

on interception have been collected by numerous investigators (Linsley and others, 1982), but little is known of the interception values for most hydrologic problems. Estimates of interception for various vegetation types (Linsley and others, 1982) are shown:

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

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

Depression storage and infiltration losses comprise the majority of the rainfall loss as illustrated in Figure 4.1. The estimates of these two losses will be discussed in more detail in latter sections of this manual. Three periods of rainfall losses are illustrated by Figure 4.1, and these must be understood and their implications appreciated before applying the procedures in this manual. First, there is a period of initial loss when no rainfall excess (runoff) is produced. During this initial period, the losses are a function of the depression storage, interception, and evaporation rates plus the initially high infiltration capacity of the soil. The accumulated rainfall loss during this period of no runoff is called the initial abstraction. The end of this initial period is noted by the onset of ponded water on the surface, and the time from start of rainfall to this time is the time of ponding (Tp). It is important to note that losses during this first period are a summation of losses due to all mechanisms including infiltration. The second period is marked by a declining infiltration rate and generally very little losses due to other factors. The third, and final, period occurs for rainfalls of sufficient duration for the infiltration rate to reach the steady-state, equilibrium rate of the soil (fc). The only appreciable loss during the final period is due to infiltration.

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

4.2 SURFACE RETENTION LOSS

Surface retention loss, as used herein, is the summation of all rainfall losses other than infiltration. The major component of surface retention loss is depression storage; relatively minor components of surface retention loss are due to interception and evaporation, as previously discussed. Depression storage is considered to occur in two forms. First, in-place depression storage occurs at, and in the near vicinity of, the raindrop impact. The mechanism for this depression storage is the microrelief of the soil and soil cover. The second form of depression storage is the retention of surface runoff that occurs away from the point of raindrop impact in surface depressions such as puddles, roadway gutters and swales, roofs, irrigation bordered fields and lawns, and so forth. A relatively minor contribution by interception is also considered as a part of the total surface retention loss. Estimates of surface retention loss are difficult to obtain and are a function of the physiography and land-use of the area. The surface retention loss on impervious surfaces has been estimated to be in the range 0.0625 inch to 0.125 inch by Tholin and Keefer (1960), 0.11 inch for 1 percent slope to 0.06 inch for 2.5 percent slopes by Viessman (1967), and 0.04 inch based on rainfall-runoff data for an urban watershed in Albuquerque by Sabol (1983). Hicks (1944) provides estimates of surface retention losses during intense storms as 0.20 inch for sand, 0.15 inch for loam, and 0.10 inch for clay. Tholin and Keefer (1960) estimated the surface retention loss for turf to be between 0.25 to 0.50 inch. Based on rainfall simulator studies on undeveloped alluvial plains in the Albuquerque area, the surface retention loss was estimated as 0.1 to 0.2 inch (Sabol and others, 1982a). Rainfall simulator studies in New Mexico result in estimates of 0.39 inch for eastern plains rangelands and 0.09 inch for pinon-juniper hillslopes (Sabol and others, 1982b). Surface retention losses for various land-uses and surface cover conditions in Maricopa County have been extrapolated from these reported estimates and these are shown in Table 4.1 <T4.1

4.3 INFILTRATION

Infiltration is the movement of water from the land surface into the soil. 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. 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~~ a near impervious strata of clay, caliche, or rock. When these soils are considered in regard to long-duration rainfalls (the design events for many parts of the United

States) this definition may be valid. However, when considered for short-duration and relatively small design rainfall depths in Maricopa County, this definition could result in underestimation of the rainfall losses. This is because even a relatively shallow horizon of soil overlaying an impervious layer still has the ability to store a significant amount of infiltrated rainfall. For example, consider the situation where only 4 inches of soil covers an impervious layer. If the effective porosity is 0.30, then 1.2 inches (4 inches times 0.30) of water can be infiltrated and stored in the shallow soil horizon. For design rainfalls in Maricopa County this represents a significant storage volume for infiltrated rainfall. 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.

(12) 4.4 RECOMMENDED 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:

3 ~~1.~~ SCS CN loss rate,

5 ~~2.~~ Initial loss plus uniform loss rate, (IL+ULR)

2 ~~3.~~ Exponential loss rate,

1 ~~4.~~ Holtan infiltration equation, and

4 ~~5.~~ Green and Ampt infiltration equation.

(H4) Holtan Infiltration Equation.

The Holtan infiltration equation 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 there is no known application of this method in Arizona ~~is known~~. Data and procedures to estimate the parameters for use in Maricopa County are not available. Therefore, the Holtan equation is not recommended for general use in Maricopa County.

(H4) Exponential Loss Rate Method.

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 regions in Maricopa County, but Exponential loss rate parameters have been developed from the reconstitution of flood events for a flood hydrology study in a portion of Maricopa County (U.S. Army Corps of Engineers, 1982). However, adequate data is not available to estimate the necessary parameters for all soil types and land uses in Maricopa County, and this method is not recommended for general use in Maricopa County.

(H4) SCS CN Loss Rate Method.

The SCS CN method is the most extensively used rainfall loss rate method in Maricopa County and Arizona and it has wide acceptance among many agencies, consulting engineering firms, and individuals throughout the community. ~~However,~~ This method is limited, however, because of both theoretical and practical deficiencies of in the method, including: ~~Deficiencies of the SCS CN method include:~~

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 generally accepted infiltration relation as illustrated in Figure 4.2.

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

4. The initial abstraction is equal to 0.2S

   where

$$S = 1000/CN - 10$$

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

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

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

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

Both the Green and Ampt infiltration equation and the initial loss and uniform loss rate (IL+ULR) method, as programmed into HEC-1, can be used to simulate the rainfall loss model as depicted in Figure 4.2. The IL+ULR is a simplified model that has been used extensively for flood hydrology and data is available to estimate the two parameters for this method. The Green and Ampt infiltration equation is a physically based model that has been in existence since 1911, and has recently been incorporated as an option in

HEC-1. Procedures have been developed to estimate the three parameters of the Green and Ampt infiltration equation. Therefore, ~~because of these reasons,~~ the two methods that are recommended for use in Maricopa County 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 parameters. Use of rainfall loss methods other than those recommended should not be undertaken unless previously approved by the Flood Control District and the local regulatory agency. The preferred method, and theoretically the most accurate, is the Green and Ampt infiltration equation. The IL+ULR is recommended as an alternative if it is not possible to estimate the Green and Ampt equation parameters, or for other valid reasons. It should be realized, as explained later, that the use of the Green and Ampt equation and parameters, as defined herein, will probably result in lower peak discharges and runoff volumes than the use of the IL+ULR.

4.4.1 Green and Ampt Infiltration Equation

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

$$f = K_s\left(1 + \frac{\Psi\theta}{F}\right) \qquad \text{for } f < i \qquad (1)$$

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

where

f = infiltration rate (L/T),

i = rainfall intensity (L/T),

K_s = hydraulic conductivity, wetted zone, steady-state rate (L/T),

Ψ = average capillary suction in the wetted zone (L),

θ = soil moisture deficit (dimensionless), equal to effective soil porosity times the difference in final and initial volumetric soil saturations, and

F = depth of rainfall that has infiltrated into the soil since the begining of rainfall (L).

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

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

$$F = -.5(2F - K_s\Delta t) + .5[(2F - K_s\Delta t)^2 + 8K_s\Delta t(\theta\Psi + F)]^{1/2} \quad (2)$$

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

$$f = \frac{\Delta F}{\Delta t} \qquad (3)$$

Use of the Green and Ampt equation as coded in HEC-1 involves the simulation of rainfall loss as a two phase process, as illustrated in Figure 4.2. The first phase is the simulation of the surface retention loss as previously described, 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, as illustrated in Figure 4.2. The three Green and Ampt equation infiltration parameters as coded in HEC-1 are hydraulic conductivity at natural saturation (XKSAT) equal to K_s in Equation 1, wetting front capillary suction (PSIF) equal to Ψ in Equation 1, and volumetric soil moisture deficit at the start of rainfall (DTHETA) equal to [THETA] in Equation 1. The three infiltration parameters are functions of soil characteristics, ground surface characteristics, and land management practices. The soil characteristics of interest are particle size distribution (soil texture), organic matter, and bulk density. The primary soil surface characteristics are vegetation canopy cover, ground cover, and soil crusting. The land management practices are identified as various tillages as they result in changes to soil porosity.

Values of Green and Ampt equation parameters as a function of soil characteristics alone (bare ground condition) have been obtained from published reports (Rawls and others, 1983; Rawls and Brakensiek, 1983). Average values of XKSAT and PSIF for each of the soil texture classes from Rawls and Brakensiek (1983) are shown in Columns (2) and (3) of Table 4.2. T4.2 Values of XKSAT and PSIF as a function of percent of sand and percent of clay for soil with 0.5 percent organic matter and base value (unaltered) soil porosity are shown in Figures 3.4 and 4.4, respectively (Rawls and Brakensiek, F 4.3 4.4 T4.4 1983). The values of XKSAT and PSIF from Table 4.2 should be used if general soil texture classification of the drainage area is available. The values of XKSAT and PSIF from Figures 3.4 and 4.4 can be used if more specific soil texture classification is available from a detailed soil survey for which the percentages of sand and clay have been determined by an appropriate field soil survey. The use of the information in Figures 4.3 and 4.4 will require an extensive study of the soil for the drainage area and for most drainage studies only general soil texture classification will be known and the values from Table 4.2 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 effectively saturated at the start of rainfall then DTHETA equals 0.0. If the soil is devoid of moisture at the start of rainfall the DTHETA equals the effective porosity of the soil. 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 4.5 (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 illustrated in Figure 4.6. 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 4.7. 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 4.8. This figure was prepared by subtracting the wilting point soil moisture in Figure 4.6 from the soil porosity in Figure 4.5. The value of DTHETA "Normal" as a function of soil texture is shown in Figure 4.9. This figure was prepared by subtracting the field capacity soil moisture in Figure 4.7 from the soil porosity in Figure 4.5. The value of DTHETA "Saturated" is always equal to 0.0 because for this condition there is no available pore space in the soil matrix at the start of rainfall. Values of DTHETA for the three antecedent soil moisture conditions are shown in Table 4.2. DTHETA "Dry" should be used for soil that is usually in a state of low soil moisture such as would occur in the desert and rangelands of Maricopa County. DTHETA "Normal" should be used for soil that is usually in a state of moderate soil moisture such as would occur in irrigated lawns, golf courses, parks, and irrigated pastures. DTHETA "Saturated" should be used for soil that can be expected to be in a state of high soil moisture such as irrigated agricultural land.

The hydraulic conductivity (XKSAT) can be affected by several factors besides soil texture. For example, hydraulic conductivity is reduced by soil crusting, 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 soil cover conditions.

Ground cover, such as grass, litter, and rock will generally increase the infiltration rate over that of bare ground conditions. Similarly, canopy cover, such as from trees, brush, and tall grasses can also increase the bare ground infiltration rate. The procedures and data that have been presented are for estimating the Green and Ampt parameters based solely on soil texture and would be applicable for bare ground conditions. Past research has shown that the wetting front capillary suction parameter (PSIF) is relatively insensitive in comparison with the hydraulic conductivity parameter (XKSAT); therefore only the hydraulic conductivity parameter is adjusted for the influences of cover over bare ground.

Procedures have been developed (Rawls and others, 1988) for incorporating the effects of soil crusting, ground cover, and canopy cover into the estimation of hydraulic conductivity for the Green and Ampt equation; however, those procedures are not recommended for use in Maricopa County at this time. A simplified procedure to adjust the bare ground hydraulic conductivity for vegetation cover is shown in Figure 4.10. This figure is based on the documented increase in hydraulic conductivity due to various soil covers as reported by investigators using rainfall simulators on native western rangelands (Kincaid and others, 1964; Sabol and others, 1982a; Sabol and others, 1982b; Bach, 1984; Ward, 1986; Lane and others, 1987; Ward and Bolin, 1989). This correction factor can be used based on an estimate of vegetation cover as used by the Soil Conservation Service in soil surveys; that is, vegetation cover is evaluated on basal area for grasses and forbs, and is evaluated on canopy cover for trees and shrubs. Note that this correction can be applied only to soils other than sand and sandy loam.

The influence of tillage results in a change in total porosity and therefore a need to modify the three Green and Ampt equation infiltration parameters. The effect of tillage systems on soil porosity and the corresponding changes to hydraulic conductivity, wetting front capillary suction, and water retention is available (Rawls and Brakensiek, 1983). Although this information is available it is not presented in this manual, nor is it recommended that these adjustments be made to the infiltration parameters for design purpose use in Maricopa County. 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.

The necessary soils information may not be available for all areas of Maricopa County for the purpose of estimating the Green and Ampt equation parameters based on soil texture. There is, however, extensive experience in Maricopa County in using the SCS CN method to estimate rainfall losses, and estimates of CN can be obtained by comparison of watersheds for which no general soils reports are available to watersheds for which soils data are available. Brakensiek and Rawls (1983) have grouped soil according to texture into the four hydrologic soil groups as shown below:

Hydrologic Soil Group	Soil Texture
A	sand sandy loam
B	silt loam loam
C	sandy loam silt
D	clay loam silty clay loam sandy clay silty clay clay

This grouping of soils is based on the four hydrologic soil groups as defined by SCS soil scientists, with limits for each group established by the minimum infiltration rate as defined by Musgrave (1955). This classification system assumes that the hydraulic conductivity (XKSAT) of the Green and Ampt equation corresponds to the minimum infiltration rate (fc).

Classification of soil according to hydrologic soil group, only, involves some large scale lumping of soils. For example, silt loam is placed in hydrologic soil group B based on soil texture classification, whereas using particle size percentages (and percent organic matter) can place silt in any of the four groups. The A and D soil groups are most nearly invariant with respect to soil texture classification, and the B and C soils are less definitive in regard to soil texture. This classification indicates that the SCS hydrologic soil groups are not uniquely related to soil hydraulics and hydrologic properties; however, it does indicate that Green and Ampt equation parameters can be estimated with some degree of confidence and reproducibility from readily available soil properties and from an estimate of CN.

Brakensiek, Rawls, and Stephenson (1984) extended this general classification of soils into a procedure for estimating hydrologic soil groups and Curve Numbers (CN) based on soils data. Their analysis resulted in a procedure to relate CN to saturated hydraulic conductivity of the soil. This procedure has been modified so that hydraulic conductivity for the Green and Ampt equation can be estimated from the CN for the soil-cover complex and percentage of vegetation cover. This is shown in Figure 4.11, and this figure can be used to estimate hydraulic conductivity from an estimate of CN. Capillary suction (PSIF) is usually inversely related to the value of hydraulic conductivity (XKSAT), as illustrated in Figure 4.12. Figure 4.12 can be used in conjunction with Figure 4.11 to estimae the Green and Ampt equation parameters. DTHETA should be selected from Table 4.2 based on the assumption of initial soil moisture and estimated XKSAT and PSIF.

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

This is a simplified rainfall loss method that is often used, and generally accepted, for flood hydrology. In using this simplified method it is assumed that the rainfall loss process can be simulated as a two-step procedure, as illustrated in Figure 4.13. 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 (STRTL) and the uniform loss rate (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 (IA) and an initial amount of infiltration (IL); therefore, STRTL = IA + IL. Values of the infiltration component (IL) of STRTL for bare ground according to soil texture classification are shown in Columns (3) through (5) in Table 4.3. These values have been derived from the Green and Ampt infiltration equation and parameter values that are shown in Table 4.2. The value of IL "Dry" should be used for soil that is usually in a state of low soil moisture at or near the wilting point for vegetation. This is a reasonable assumption for most nonirrigated lands in Maricopa County because of the infrequency of rainfall and because of the rapid drainage of these soils after rainfall. The value of IL "Normal" should be used for soil that is usually in a state of moderate soil moisture such as occurs for irrigated lawns, turf, and permanent pastures. The value of IL "Saturated" is used for a soil maintained in a state of high soil moisture, such as in irrigated agricultural lands.

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

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 3 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 4.2). The values of CNSTL for soils classified according to hydrologic soil groups are shown in Table 4.2. These values within each hydrologic soil group have been selected from inspection of XKSAT values in Table 4.2 for the various soil texture classifications. Values of CNSTL shown in Table 4.4 are consistent with general information available for estimating CNSTL as shown in Table 4.5. Figure 4.11 can be used to estimate CNSTL based on an estimate of CN if adequate soils data is not available.

4.5 PROCEDURE FOR ESTIMATING LOSS RATES

4.5.1 Green and Ampt Method

A. When soils data are available:

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

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

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

4. Estimate the wetting front capillary suction parameter (PSIF) from Table 4.2 if general soil texture classification is available or from Figure 4.4 if adequate soil texture data is available from an approved sampling program.
5. Estimate the value of DTHETA from Table 4.2 if general soil texture classification is available or from either Figure 4.8 or 4.9 if adequate soil texture data is available from an approved sampling program. The value of DTHETA must be selected based on the appropriate antecedent soil moisture condition; "Dry" for nonirrigated lands such as desert and rangeland; "Normal" for soil that would be expected to be near soil moisture field capacity such as irrigated lawn, turf, and permanent pasture; and, "Saturated" for irrigated agricultural land.
6. Determine the land-use and/or soil cover for the drainage area and use Table 4.1 to estimate the surface retention loss (IA).

B. When soils data are not available:

1. Estimate the CN based on data for similar watersheds or regional experience. Estimate the percent vegetation cover.
2. Use Figure 4.11 to estimate XKSAT based on CN and hydrologic condition.
3. Use Figure 4.11 to estimate XKSAT for bare ground.
4. Use the bare ground XKSAT and Figure 4.12 to estimate PSIF.
5. Use the bare ground XKSAT and PSIF with Table 4.2 to estimate DTHETA.

C. Alternative methods:

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

4.5.2 Initial Loss Plus Uniform Loss Rate Method

A. When soils data are available:

1. Determine the soil texture classification and/or the hydrologic soil group. Soils reports such as those of the Soil Conservation Service can be used if available, or laboratory analysis of appropriate soil samples from the drainage area can be used to classify the soil if adequate documentation on the sampling and laboratory procedure is provided and approved.
2. Use values of CNSTL and IL from Table 4.3 if the losses are to be based on soil texture classification.
3. Use values of CNSTL and IL from Table 4.4 if the losses are to be based on hydrologic soil group.
4. Determine the land-use and/or soil cover and use Table 4.1 to estimate the surface retention loss (IA).
5. STRTL = IA + IL.

B. When soils data are not available:

1. Estimate the CN based on data for similar watersheds or regional
2. experience. Estimate the percent vegetation cover.
3. Use Figure 4.11 to estimate CNSTL based on CN and hydrologic condition.
4. Use Table 4.3 to estimate IL based on the value of CNSTL.
5. Use Table 4.1 to estimate the surface retention loss (IA).
6. STRTL = IA + IL

UNIT HYDROGRAPH PROCEDURES

5.1 General

Rainfall excess can be routed from a watershed to produce a storm discharge hydrograph at a downstream location (concentration point) by one of two methods: 1) hydraulic routing involving the complete or some simplified form of the equations of motion, 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 simplified hydraulic routing. Hydrologic routing is usually accomplished by either direct application of the equation of continuity;

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

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

A unit hydrograph is a graph of the time distribution of runoff from a specific watershed as the result of one inch of rainfall excess that is distributed uniformly over the watershed and that is produced during a specified time period (duration). 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, $S = KO$. However, Mitchell (1962) has shown that nonlinear storage, $S = KO^x$, is a condition that occasionally occurs in natural watersheds. A method has been developed by Shen (1962) to evaluate the linearity of the storage-outflow relation for gaged watersheds. Mitchell (1972) developed the model hydrograph for use in watersheds that have nonlinear storage-outflow characteristics. Presently, 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.

5.2 CLARK UNIT HYDROGRAPH

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

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

$$\text{where } C = \frac{2\Delta t}{2R + \Delta t} \qquad (3)$$

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

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

where Q_i are the ordinates of the Clark Unit Hydrograph.

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

The second parameter is the storage coefficient, R, which has the dimension of time. This parameter is used to account for the effect that temporary storage in the watershed has on the hydrograph. Several methods are available to estimate R from recorded hydrographs for a basin. As originally proposed by Clark (1945), this parameter can be estimated by dividing the discharge at the point of inflection of the surface runoff hydrograph by the rate of change of discharge (slope of the hydrograph) at the inflection point as shown in Figure 5.2. Another technique for estimating R is to compute the volume remaining under the recession limb of the surface runoff hydrograph following the point of inflection and to divide the volume by the discharge at the point of inflection. Both of these methods require the ability to identify the inflection point on the recession limb of the runoff hydrograph. This is difficult if not impossible for complex hydrographs and flashy hydrographs such as occur from urban basins and natural watersheds in the Southwest. A method to estimate R by a graphical recession analysis of the hydrograph has been proposed (Sabol, 1988) and this method provides much more consistent results than do the previously described methods. The parameter, R, should be estimated by the analysis of several recorded events; however, in most cases recorded discharge hydrographs are not available and R must be estimated by empirical equations.

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

$$I_1 = (A_1R_1) \times c/\Delta t$$

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

I1 = (8 acres)(.10 inch)(60.5 cfs/acre-inch/minute)/(5 minutes)

= 9.7 cfs

At the end of 10 minutes the available runoff is

$$I_2 = (A_1R_2 + A_2R_1) \times c/\Delta t$$

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

$$= 82.3 \text{ cfs}$$

At the end of 15 minutes the available runoff is

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

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

$$= 234.7 \text{ cfs}$$

At the end of 20 minutes the available runoff is

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

$$= [(8)(.15) + (24)(.30) + (38)(.55) + (32)(.10)] \times 60.5/5 = 393.5 \text{ cfs}$$

At the end of 25 minutes the available runoff is

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

$$= [(8)(0) + (24)(.15) + (38)(.30) + (32)(.55) + (18)(.10)] \times 60.5/5 = 416.2 \text{ cfs}$$

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

$$I_6 = (A_3R_4 + A_4R_3 + A_5R_2) \times c/\Delta t$$
$$= [(38)(.15) + (32)(.30) + (18)(.55)] \times 60.5/5$$
$$= 304.9 \text{ cfs}$$

At the end of 35 minutes the available runoff is

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

At the end of 40 minutes the available runoff is

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

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

I9 = 0 cfs.

The translation hydrograph (Ii) is shown in Figure 5.3(d). This theoretical hydrograph has the correct volume of runoff from the watershed, however it does not reflect the effects of routing through the watershed. The translation hydrograph is then routed and averaged using Equations 2 through 4 resulting in the final runoff hydrograph. For this example, assume that R = 15 minutes, and the runoff hydrograph is shown in Figure 3(d). Notice that the

Clark Unit Hydrograph itself was never developed per se but that the three principles of the unit hydrograph were applied directly (mathematically) to the rainfall excess without performing graphical superposition of ratios of a unit hydrograph. Computationally, this process can be completed very quickly and conveniently with a computer program such as is done with HEC-1.

5.3 LIMITATIONS AND APPLICATIONS

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

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

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

5.4 DEVELOPMENT OF PARAMETER ESTIMATORS

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

5.5 ESTIMATION OF PARAMTERRS

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.

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

$$T_c = 11.4\ L^{.50} K_b^{.52} S^{-.31} i^{-.38} \tag{5}$$

where T_c is in hours,

L is length of the flow path for T_c, in miles,

K_b is a representative watershed resistance coefficient,

S is watercourse slope, in feet/mile, and

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

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

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

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

A work sheet has been prepared that facilitates the calculation of T_c. A copy of this work sheet is included in the manual and its use is included in the Examples section of the manual.

5.5.2 **Storage Coefficient** - Very little literature exists on the estimation of the storage coefficient, R, for the Clark Unit Hydrograph. Clark (1945) had originally proposed a relation between 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 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

$$R = 0.37T_c^{1.11}A^{-.057}L^{0.80} \tag{6}$$

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.

5.5.3 **Time-Area Relation** - Either a synthetic time-area relation must be adopted or the time-area relation for the watershed must be developed. If a synthetic time-area relation is not used, the time-area relation is developed by dividing the watershed into incremental runoff producing areas that have equal incremental travel times to the outflow location. This is a difficult task and well defined and reliable procedures for this are not available. The following general procedure is often used. 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 times to the outflow location. These isochrones can be established by considering the land surface slope and resistance to flow, and also whether the runoff would be sheet flow or would be concentrated in watercourses. A good deal of judgement and interpretation is required for this. 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 Tc versus contributing area as a percent of total area. The dimensionless graph is preferred because this facilitates the rapid development of new time-area relations should there be a need to revise the estimate of T_c.

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

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

$$1 - A^* = 1.414(1-T^*)^{1.5} \qquad .5 < T^* \leq 1.0$$

where A^* is contributing area, in percent of total area, and

T^* is time, in percent of $_{Tc}$.

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

Two other dimensionless time-area relations have been developed during the reconstitution of recorded rainfall-runoff events as described in the Documentation Manual. These dimensionless relations for urban and natural watersheds are shown in Figures 5.6 and 5.7, 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 5.6 be used, and for a natural (undeveloped) watershed that the synthetic time-area relation of Figure 5.7 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 5.6 or 5.7. The HEC-1 default time-area relation is shown for comparison in each figure. Tabulated values of the dimensionless time-area relations are shown in Table 5.2.

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

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

which is preferred, however as a general requirement

$$0.10T_c \le NMIN \le 0.25T_c \tag{9}$$

NMIN is normally selected as a 5-minute multiple. This may require that watersheds with significantly different sub-basin sizes be modeled with some sub-basins run separately and the outflow hydrographs from these separate runs read directly into the multi-basin model.

5.6 S-GRAPHS

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

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

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

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

where A is drainage area, in square miles, and

Δt 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 (Δt). The resulting summation is a graphical distribution that resembles an S-graph except that the discharge scale is accumulated discharge and the time scale is in units of measured time. This graph is terminated when the accumulated discharge equals 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 Qult and the time scale by lag.

In practice, S-graphs have generally been developed by reconstituting observed floods to define a representative unit hydrograph and then converting this to an S-graph. Prior to the advent of computerized models, such as HEC-1, flood reconstitution was a laborious task of rainfall and hydrograph separation along with numerous hand-cranked simulations to define the representative unit hydrograph. Modern S-graph development generally relies on use of optimization techniques, such as coded into HEC-1, to identify unit hydrograph parameters that best reproduce the observed flood.

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

5.6.1 Limitations and Applications

S-graphs are empirical, lumped parameters that represent runoff characteristics for the watershed for which the S-graph was developed. S-graphs that are developed from recorded runoff data from one watershed can be applied to another watershed only if the two watersheds are hydrologically and physiographically similar. In addition, a recent study for the Flood Control District of Maricopa County (Sabol, 1987) has demonstrated that the shape of S-graphs is significantly affected by storm characteristics, particularly the maximum intensity of the rainfall. Therefore, it may not be advisable to adopt S-graphs that have been developed from one hydrologic zone and to apply these to watersheds in other hydrologic zones because of possible differences in rainfall characteristics in the two zones that may affect the shape of the S-graph. Application of S-graphs requires the selection of an appropriate S-graph and the estimation of the one parameter, basin lag. Two S-graphs have been selected for use in Maricopa County and a method to estimate lag is provided.

The USBR has revised the Flood Hydrology Studies chapter of the Third Edition of Design of Small Dams (USBR, 1987), and it has identified S-graphs for application in six generalized regional and physiographic type of watersheds. Recently, the USBR has issued a Flood Hydrology Manual (Cudworth, 1989) that contains extensive discussion of flood hydrology in general, and

S-graphs in particular. Both of these references should be consulted before using S-graphs. The S-graph has been adopted as the unit hydrograph procedure by Orange County and San Bernardino County, California, and selected S-graphs are presented in the hydrology manuals for those counties. The S-graphs in those hydrology manuals have been selected primarily from S-graphs that previously had been defined by the Los Angeles District from a rather long and extensive history of analyses of floods in California.

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

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

5.6.2 Sources of S-Graphs

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

5.6.3 S-Graphs for Use in Maricopa County

Two regional S-graphs have been selected for use in flood hydrology studies of major watercourses in Maricopa County. These two are referred to as the Phoenix Mountain and the Phoenix Valley S- graphs. The Phoenix Mountain S-graph is to be used in flood hydrology studies of watersheds that drain predominantly mountainous terrain. 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 above Phoenix. Although the Corps of Engineers developed a separate S-graph for Indian Bend Wash, it is nearly identical to the Phoenix Mountain S-graph and 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 5.9 and 5.10, and the coordinates of the graphs listed in Table 5.3. These same two S-graphs have been selected for similar use in Maricopa County by the U.S. Army Corps of Engineers (1974 and 1982). The justification for the selection of these two S-graphs is provided in the Documentation Manual, and a more comprehensive presentation of S-graphs for Maricopa County is provided in the S-Graph Study report for the Flood Control District of Maricopa County (Sabol, 1987). It is possible that S-graphs other than the two that have been recommended for general use in Maricopa County be selected. The selection of S-graph should be made based on a comparison of the watershed of interest to the watershed(s) used to develop the various S-graphs. Therefore, either one of the two recommended S-graphs should be selected or the selection of other S-graph, such as from Design of Small Dams should be approved by the jurisdictional agency before proceeding.

5.6.4 Estimation of Lag

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

$$Lag = C \frac{LL_{ca}}{S^p}^m$$

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

It should be noted that Kn is a measure of the hydraulic efficiency of the watershed and it is not necessarily a constant for a given watershed for all rainfall depths and rainfall intensities. As rainfall depth and/or rainfall intensity increases the efficiency of runoff increases and Kn decreases. Therefore, some adjustment in Kn should be made for use with rainfalls of different magnitudes (frequencies). Generally, Kn is the smallest for extreme floods such PMFs and increases as the frequency of event increases.

Several graphical relations are available for estimating basin lag. One such relation (U.S. Army Corps of Engineers, 1982) is shown in Figure 5.11. Several other relations that should be consulted when using S-graphs are contained in <u>Design of Small Dams</u> (USBR, 1987) and the USBR <u>Flood Hydrology Manual</u> (Cudworth, 1989).

When estimating basin lag the following steps should be used:

1. From an appropriate map of the watershed, measure drainage area (A), and the values of L, L_{ca}, and S.
2. Calculate the basin factor $LL_{ca}/(S^{0.5})$.
3. Use data in Figure 5.11 and the tables in <u>Design of Small Dams</u> (USBR, 1987) or the <u>Flood Hydrology Manual</u> (Cudworth, 1989) to attempt to identify watersheds of the same physiographic type and similar drainage area and basin factor. Make a list of watersheds with similar drainage

areas and basin factors, and tabulate the estimated value of Kn for those watersheds, and the measured lag.

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

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

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

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

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 Routing is to be used for natural, undeveloped channels since the method explains outflow peak attenuation resulting from storage loss. Both Muskingum 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 detail 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 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 DESIGN 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 all of Maricopa County. Chapter 2, Table 2.1, and Pattern #1 of Table 2.2 contains those distributions, which were developed from data at Phoenix Airport. If different distributions are needed, Table 2.1 and Pattern #1 of Table 2.2 can be redeveloped. However, Patterns #2-5 are 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 the Airport. In that case, the short duration part of the rainfall such as the 15-minute depth may be different from the one by Pattern #1. This will give a different peak outflow, justifying the design of a new distribution.

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

7.3 NOTES ON CALCULATING LOSS PARAMETERS

1. Since many of the soil groups contain horizons of different textures, the top horizon may or may not control the total volume and rate of infiltration. The decision of which soil layer controls the infiltration rate is based on soil texture, horizon thickness, and the accumulated depth of water during the initial low-intensity period of a design storm. As a gereral 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 by the SCS as that percentage of the soil matrix between 0.5 and 2.0 mm. in diameter. The SCS Soil Survey books list a percentage of each soil type passing sieve #200, which has openings of 0.074 mm. It can therefore be assumed as an estimate that the percentage of particles retained by this sieve are sand size and larger. It will also be assumed that soils with particle size between 2.0 mm and 3.0 inches (gravel) have infiltration rates greater than or equal to sand. This is necessary because Green & Ampt and IL+ULR loss parameters have not been developed for cobbly, gravelly, channery, etc. soils. When choosing the value for percent sand and clay, choose the median value from the range listed in the "Engineering Index Properties" and "Physical and Chemical Properties" tables. For example, if a range of 10-35% 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 4 in the appendix to determine the textural class for that particular soil. Then choose Green &

Ampt loss parameters from Table 4.2 or IL+ULR parameters from Tables 4.3 and 4.4.

3. Most soil map units consist of major and minor soil areas, as listed in the "General Soil Map Units" sections of the Soil Survey books. The descriptions will list the percentage of each of the major soils, and one 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. Hydrolgic soil groups can be weighted in the following manner:

$$A=1 \text{ - } B=2 \text{ - } C=3 \text{ - } D=4$$

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 in the appendix.

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 4.4 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 Figure 4.10.

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. Tc 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) represents the hydraulic length corresponding to Tc. For a natural channel, L is length of watercourse from outlet to point defining hydraulically most distant point. For an urban basin where flow is mainly in streets and no primary channels exist, an average flow path should be selected, such as a line parallel to grade from the outlet to the upper watershed boundary.

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

Time	Excess(in)		Rank		Sorted
1415	.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 Δt = 5 minutes (where Δt is hydrograph time step), then Tc should be less than (8*5)=40 minutes. For Δt = 10 minutes, Tc < 80 minutes, and so on. Remember that in no case should Tc be less than Δt for computational stability.

4. Remember that Tc is a function of excess rainfall intensity and must be recalculated when the duration or frequency of a design storm is changed. If multiple 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.

7.5 NOTES ON THE APPLICATION OF KINEMATIC WAVE ROUTING

The computational procedure of the Kinematic Wave Routing Method may unrealistically attenuate the outflow peak. It appears that a longer reach length would cause more attenuation. To overcome this problem, the more recent versions 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.

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

7.6 NOTES ON DEVELOPING MUSKINGUM PARAMETERS

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

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

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

where K = the travel time through the entire reach (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.

7.7 NOTES ON THE APPLICATION OF S-GRAPHS

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

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

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APPENDIX

CALCULATION OF Tc & R

Calculated by: ______________________________ Date: ____________

Checked by: ______________________________ Project: ____________

Watershed: ____

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

Rainfall Loss Method:

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

Tabulate Period of Peak Rainfall Excess

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

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

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

A = __________ sq.mi.

L = __________ mi.

S = __________ ft/mi.

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

$Kb = (\quad)\ \log(\quad *640) + (\quad)$

$Kb =$ ____________

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

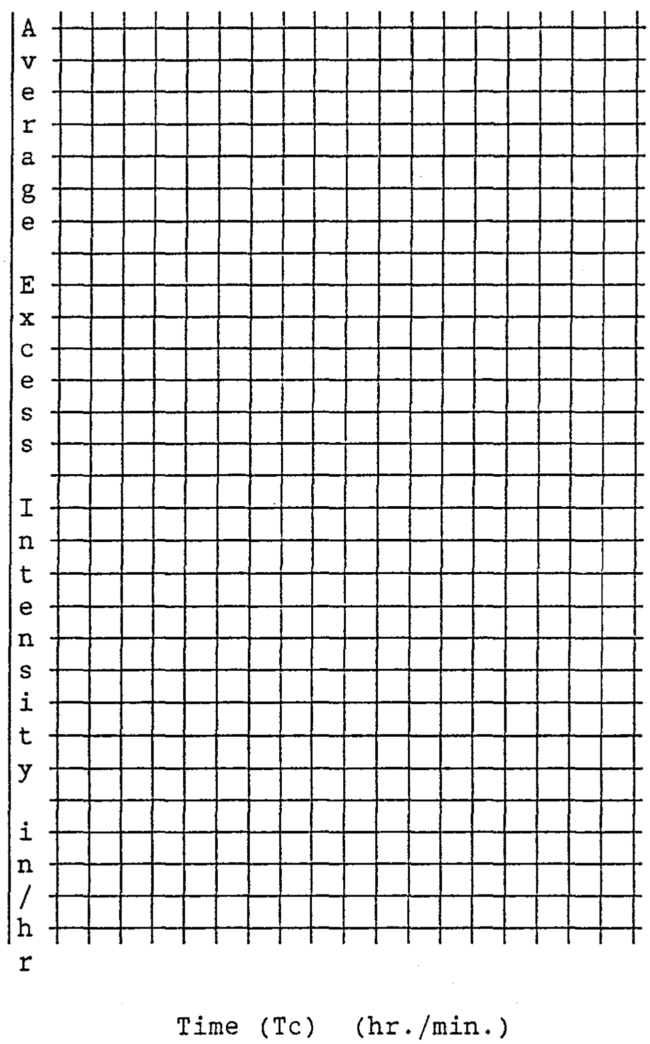
$Tc = (\_\_\_\_\_\_)\ i^{-.38}$

Trial Tc	i	Calc. Tc

Tc = hr.

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

R = hr.



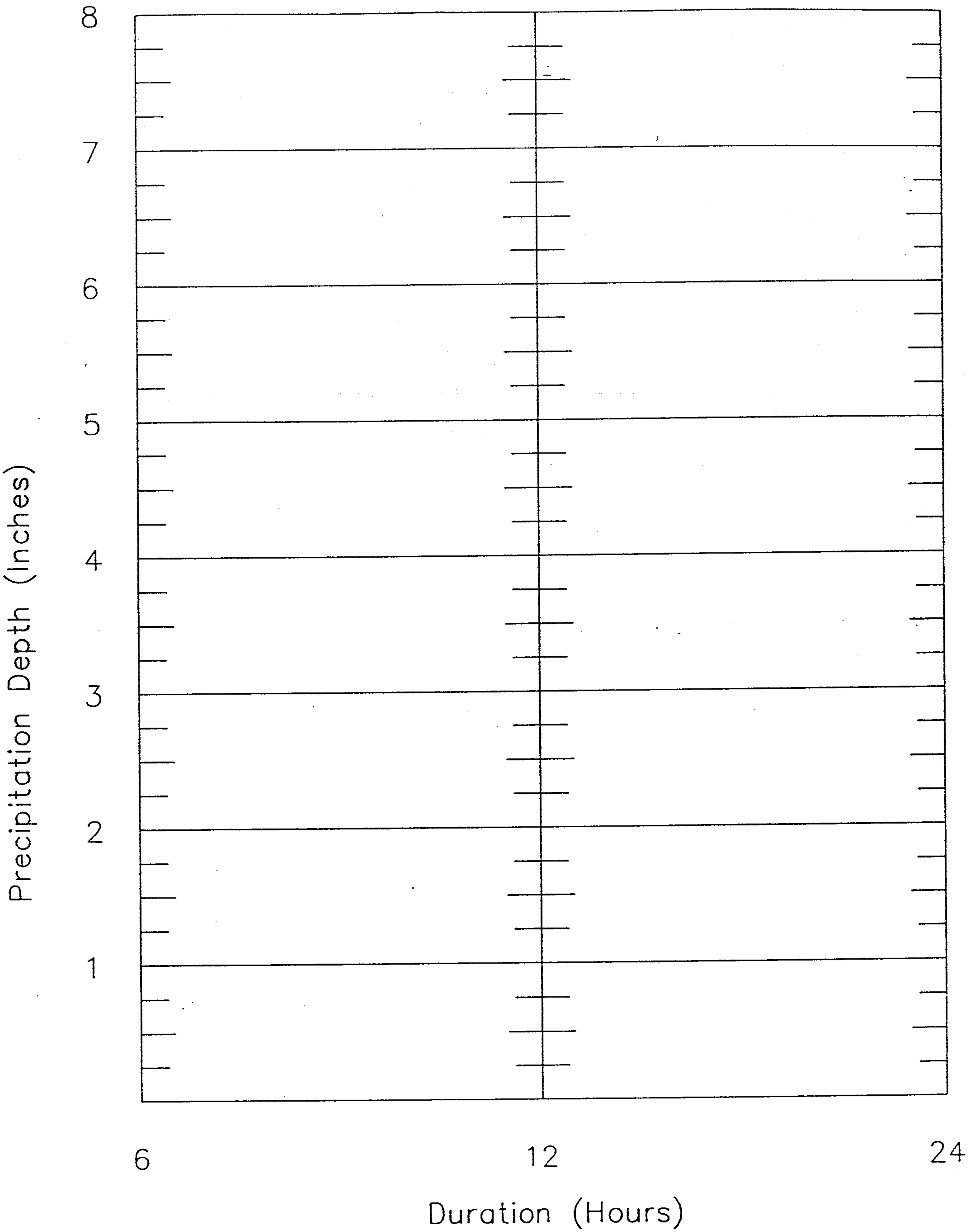
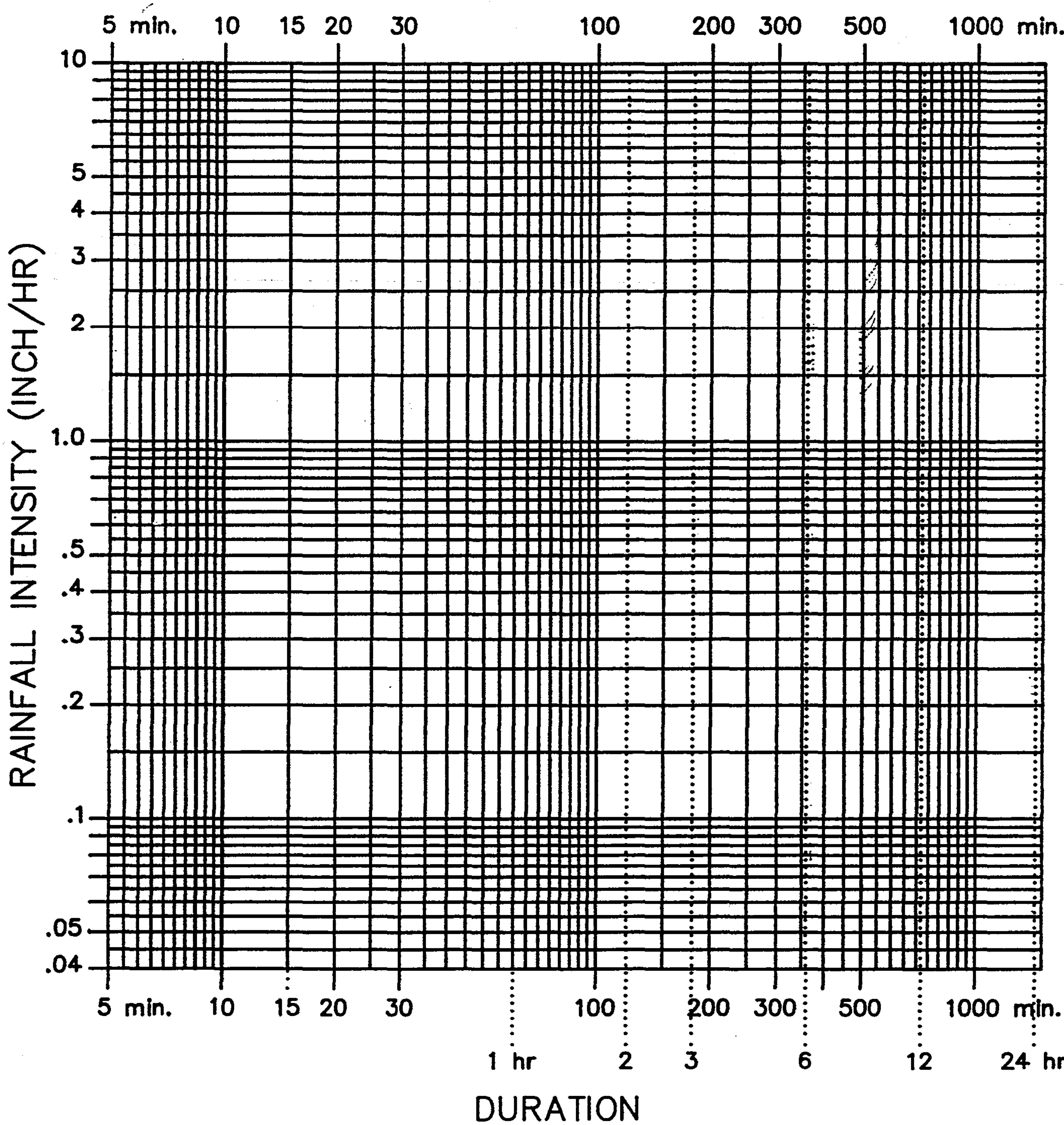


Figure 2-6. Precipitation depth–duration diagram (6 to 24 hr).



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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MCFCD HYDROLOGY MANUAL PAGE ___ OF ___

DETAIL ___ COMPUTED ___ DATE ___

___ CHECKED BY ___ DATE ___

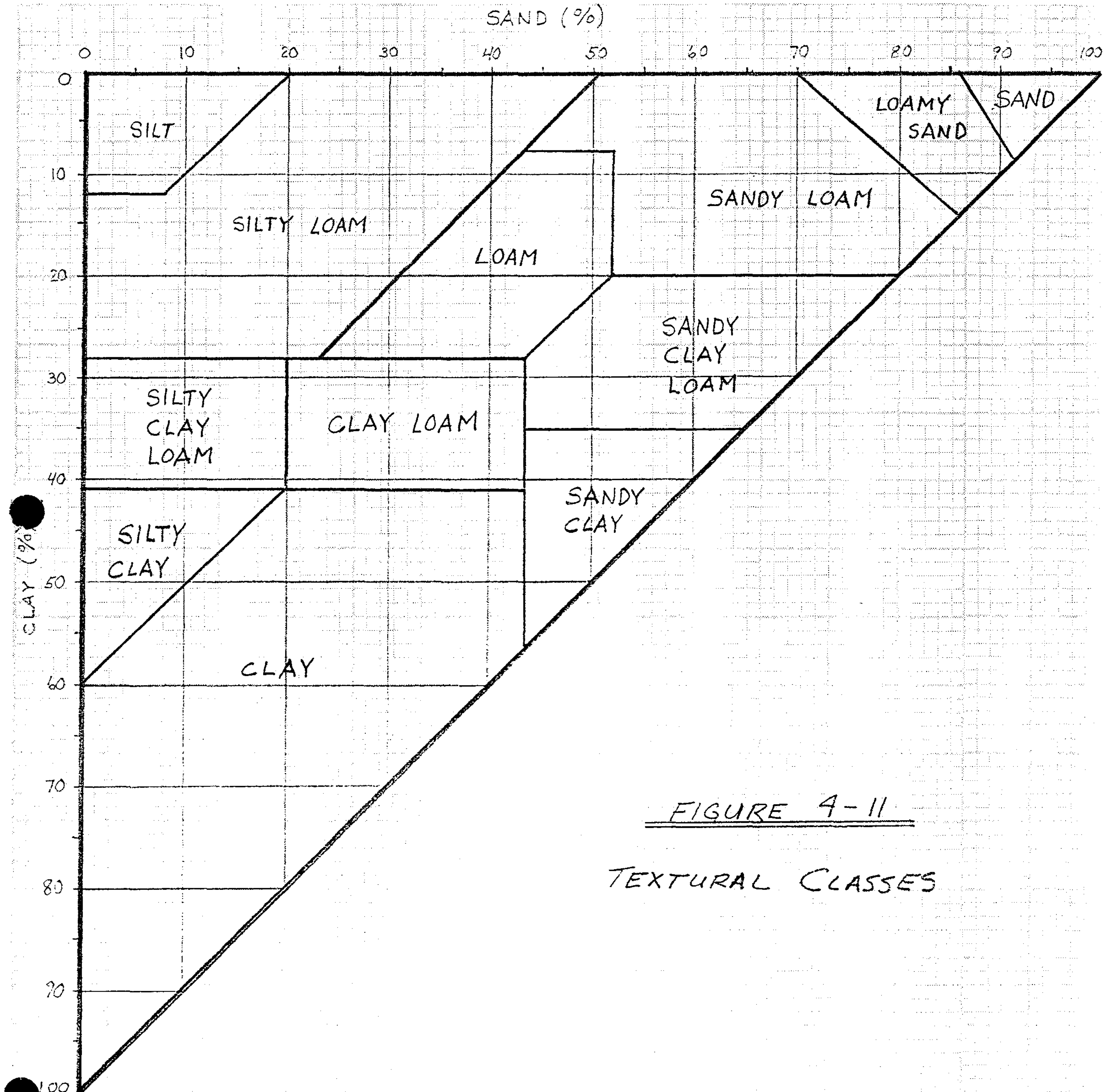


FIGURE 4-11

TEXTURAL CLASSES

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

PROJECT *MARICOPA COUNTY HYDROLOGY MANUAL*

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-YR, 1-HR

Y_{100} = 100-YR, 1-HR

$\left\{ \begin{array}{l} X_1 = \text{2-YR, 6-HR,} \\ X_2 = \text{2-YR, 24-HR,} \end{array} \right.$ $\left\{ \begin{array}{l} X_3 = \text{100-YR, 6-HR} \\ X_4 = \text{100-YR, 24-HR} \end{array} \right.$

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

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

5-YR, 1-HR = 1.35

10-YR, 1-HR = 1.58

25-YR, 1-HR = 1.82

50-YR, 1-HR = 2.12

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL

DETAIL EXAMPLE #1 COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

(5) THEN 2-HR DEPTHS ARE CALCULATED:

100-YR, 2-HR = .341 (100-YR, 6-HR) + .659 (100-YR, 1-HR)

⇒ 100-YR, 2-HR = .341 (3.40) + .659 (2.35) = 2.71

2-YR, 2-HR = .341 (2-YR, 6-HR) + .659 (2-YR, 1-HR)

⇒ 2-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:

5-YR, 2-HR = 1.62

10-YR, 2-HR = 1.87

25-YR, 2-HR = 2.12

50-YR, 2-HR = 2.46

(6) NEXT 3-HR DEPTHS ARE CALCULATED:

100-YR, 3-HR = .569 (100-YR, 6-HR) + .431 (100-YR, 1-HR)

⇒ 100-YR, 3-HR = .569 (3.40) + .431 (2.35) = 2.95

2-YR, 3-HR = .569 (2-YR, 6-HR) + .431 (2-YR, 1-HR)

⇒ 2-YR, 3-HR = .569 (1.55) + .431 (1.12) = 1.36

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

DETAIL EXAMPLE # 1 COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

VALUES FOR 2-YR AND 100-YR DEPTHS ARE PLOTTED ON FIGURE 5 AND DEPTHS FOR OTHER FREQUENCIES ARE ESTIMATED FROM GRAPH:

5-YR, 3-HR = 1.74

10-YR, 3-HR = 2.05

25-YR, 3-HR = 2.30

50-YR, 3-HR = 2.65

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-YR, 12-HR = 1.80

5-YR, 12-HR = 2.30

10-YR, 12-HR = 2.70

25-YR, 12-HR = 3.10

50-YR, 12-HR = 3.65

100-YR, 12-HR = 4.10

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #1 COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

(8) 30-MIN DEPTHS ARE ESTIMATED FROM:

(2-YR, 30-MIN) = .82 (2-YR, 1-HR) = .82 (1.12) = .92

(100-YR, 30-MIN) = .82 (100-YR, 1-HR) = .82 (2.35) = 1.93

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

5-YR, 30-MIN = 1.15

10-YR, 30-MIN = 1.35

25-YR, 30-MIN = 1.50

50-YR, 30-MIN = 1.75

(9) 15-MIN DEPTHS ARE ESTIMATED FROM:

(2-YR, 15-MIN) = .62 (2-YR, 1-HR) = .62 (1.12) = .69

(100-YR, 15-MIN) = .62 (101-YR, 1-HR) = .62 (2.35) = 1.46

OTHER VALUES ARE ESTIMATED FROM FIGURE 5 AS BEFORE:

5-YR, 15-MIN = .87

10-YR, 15-MIN = 1.00

25-YR, 15-MIN = 1.15

50-YR, 15-MIN = 1.28

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL: EXAMPLE # 1 COMPUTED ______ DATE ______

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(10) 10-MIN VALUES

(2-YR, 10-MIN) = .51 (2-YR, 1-HR) = .51 (1.12) = .57

(100-YR, 10-MIN) = .51 (100-YR, 1-HR) = .51 (2.35) = 1.20

OTHER VALUES FROM FIGURE 5:

5-YR, 10-MIN = .70

10-YR, 10-MIN = .83

25-YR, 10-MIN = .92

50-YR, 10-MIN = 1.10

(11) 5-MIN VALUES:

(2-YR, 5-MIN) = .34 (2-YR, 5-MIN) = .34 (1.12) = .38

(100-YR, 5-MIN) = .34 (100-YR, 5-MIN) = .34 (2.35) = .80

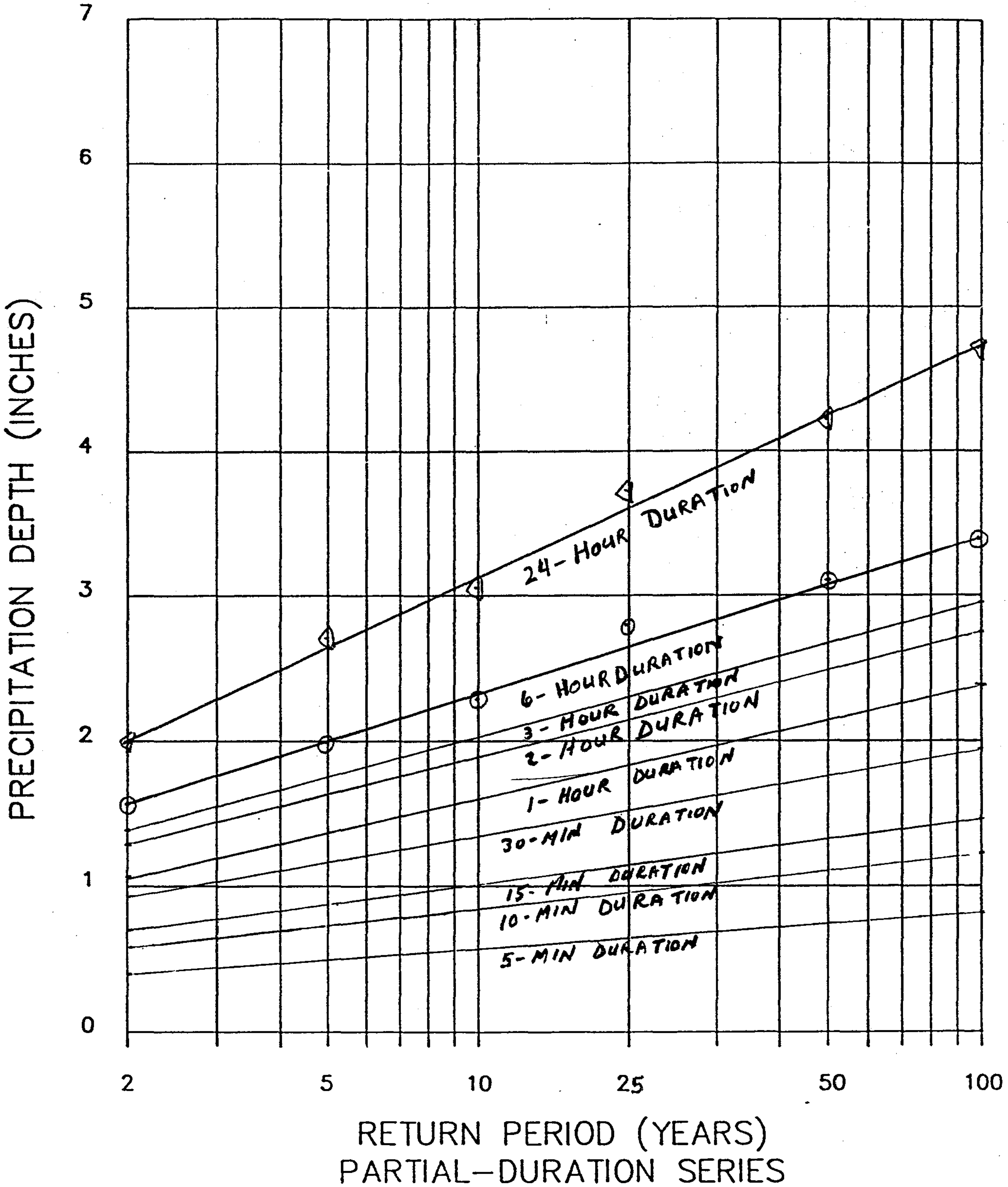
OTHER VALUES FROM FIGURE 5:

5-YR, 5-MIN = .50

10-YR, 5-MIN = .58

25-YR, 5-MIN = .62

50-YR, 5-MIN = .73



FIGURE– 5 PRECIPITATION DEPTH VERSUS RETURN PERIOD FOR PARTIAL–DURATION SERIES

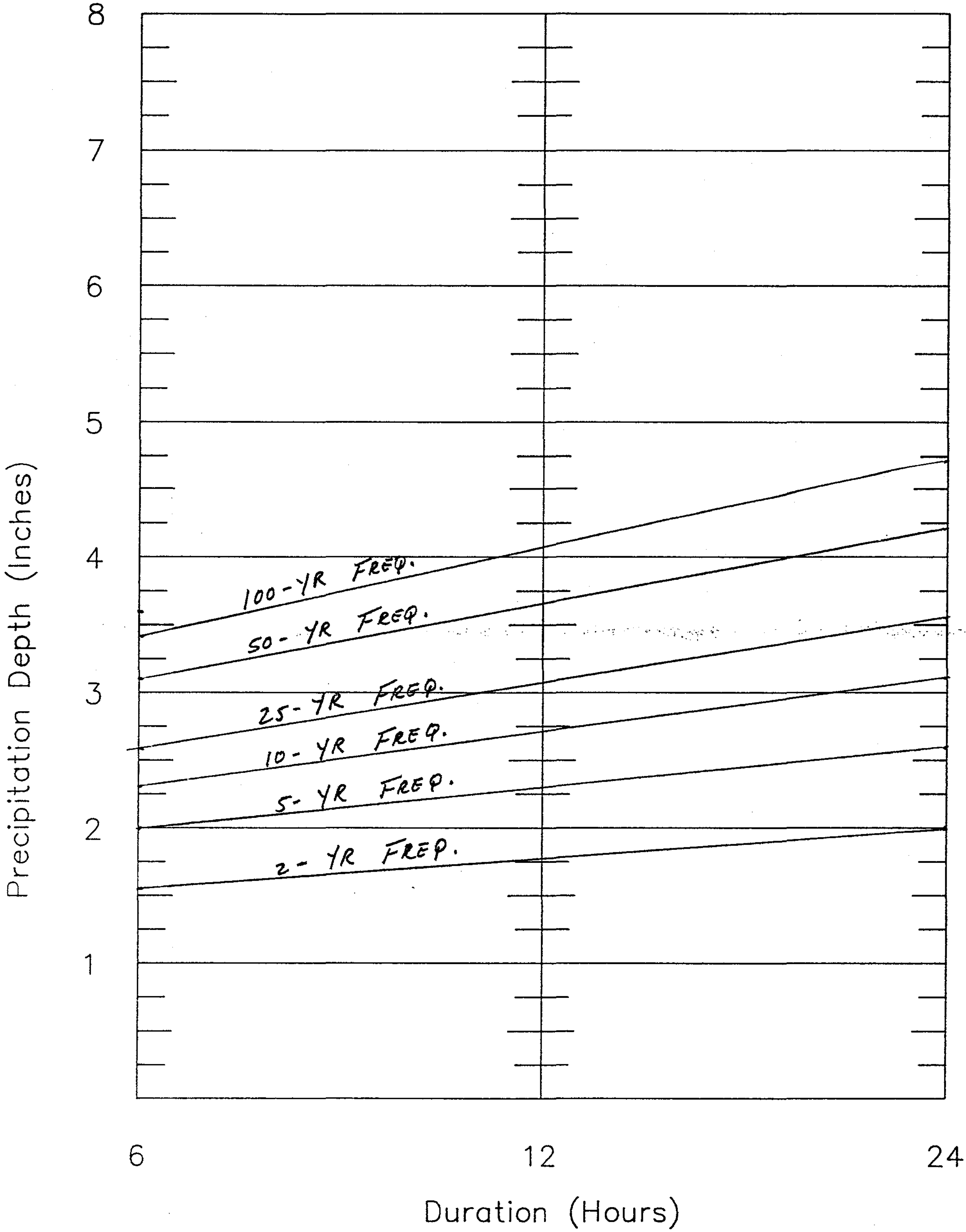


Figure 6 Precipitation depth—duration diagram (6 to 24 hr).

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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DETAIL EXAMPLE #1 COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

DEPTH-DURATION-FREQUENCY IS NOW PUT TOGETHER:

CAREFREE AIRPORT, T6N, R4E, SEC. 36

FREQUENCY

DURATION	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	1.46
30-MIN	.92	1.15	1.35	1.50	1.75	1.93
1-HOUR	1.12	1.35	1.58	1.82	2.12	2.35
2-HOUR	1.27	1.62	1.87	2.12	2.46	2.71
3-HOUR	1.36	1.74	2.05	2.30	2.65	2.95
6-HOUR	1.55	2.00	2.30	2.60	3.10	3.40
12-HOUR	1.80	2.30	2.70	3.10	3.65	4.10
24-HOUR	2.00	2.60	3.10	3.55	4.20	4.70

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 T6N, 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-YR → $\frac{.38 \text{ (INCHES)}}{5\text{-MIN}} \times \frac{60\text{-MIN}}{HR} = 4.56$ IN/HR

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

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

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

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

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

10-MIN DURATION DEPTHS:

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

5-YR → SAME AS ABOVE = 4.20 IN/HR

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DETAIL EXAMPLE #2 COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

OTHER VALUES ARE CALCULATED SIMILARLY.

15-MIN DURATION DEPTHS:

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

OTHER VALUES FOLLOW THE SAME PROCEDURE.

30-MIN DURATION DEPTHS:

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

OTHER VALUES FOLLOW THE SAME PROCEDURE.

1-HOUR DURATION DEPTHS:

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

OTHER VALUES FOLLOW THE SAME PROCEDURE.

2-HOUR DURATION DEPTHS:

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

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

6-HOUR DURATION DEPTHS:

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

12-HOUR DURATION DEPTHS:

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

24-HOUR DURATION DEPTHS:

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

VALUES FOR OTHER FREQUENCIES ARE FOUND SIMILARLY.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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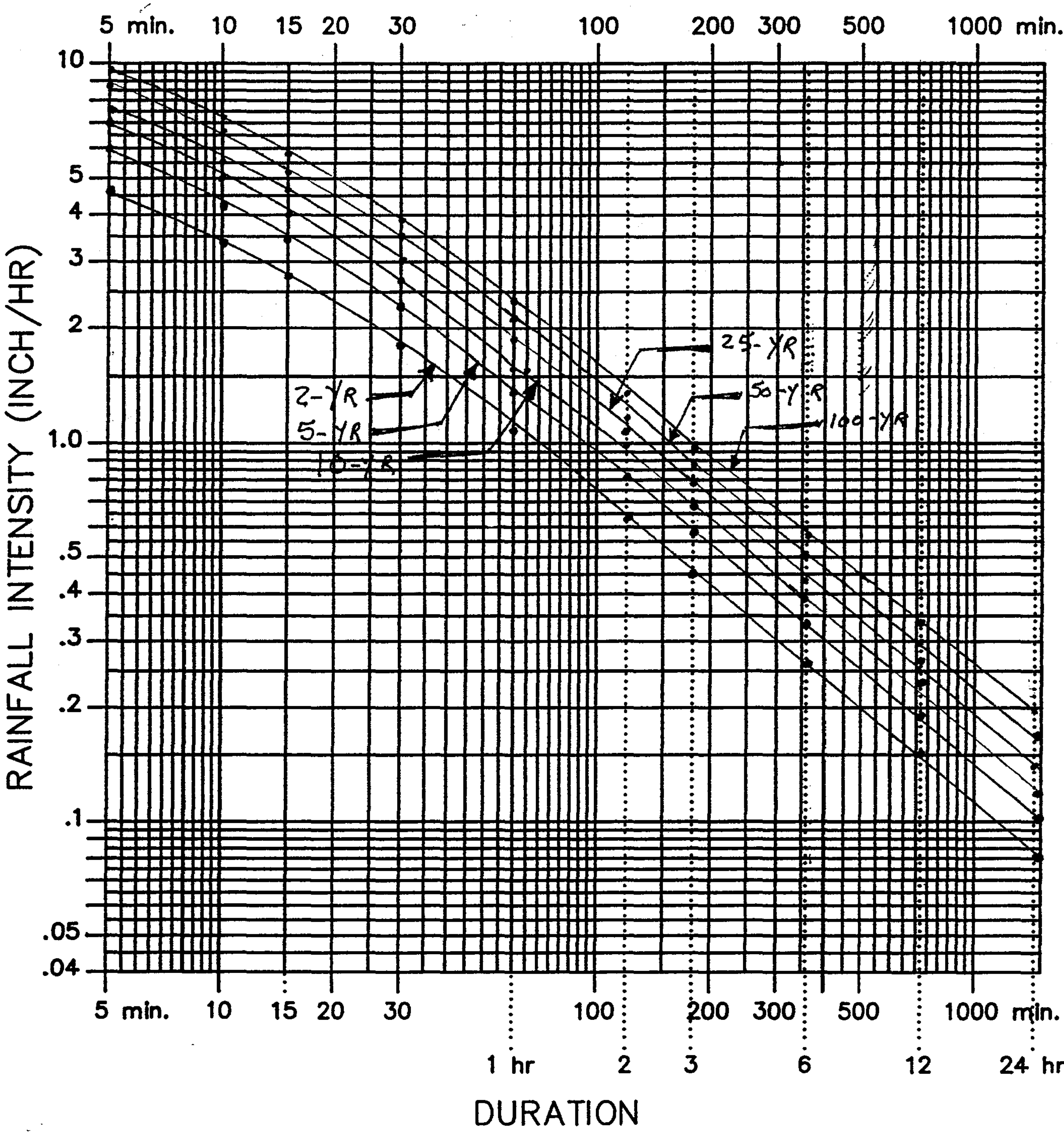
DETAIL Example #2 COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

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

FREQUENCY

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



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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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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 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.

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

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: CACULATE Tc

$$T_c = 11.4\, L^{.50} K_b^{.52} S^{-.31} i_{100}^{-.38}$$

L = 1.236 mi

K_b = .027 (FIGURE 5.5 OR TABLE 5.1)

S = 33. ft/mi

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

$$T_c = 0.655\,(i_{100})^{-.38}$$

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

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PROJECT ______

DETAIL ______ COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

SINCE THE WATERSHED IS OUTSIDE THE PHOENIX AREA, THE INTENISTY VALUES MUST BE ADJUSTED USING THE EQUATION IN SECTION 3.5.1.4. :

$$i_{100} = \left[(ip)\left(P_{10}^{6}\right)\right] / 2.07$$

then $ip = 4.00$ in/hr

$P_{10}^{6} = 2.3$ in

$$i_{100} = \left[(4.00)(2.3)\right] / 2.07 = 4.44 \text{ in/hr}$$

At $i = 4.44$ in/hr :

$$Tc = 0.655\,(4.44)^{-.38} = 22.3 \text{ min}$$ No Good

TRY $Tc = 20$ min

AT 20 min, $i_{100} = 5.1$ in/hr (FIG. 3-2)

ADJUSTED $i_{100} = \left[(5.1)(2.3)\right] / 2.07 = 5.67$ in/hr

$$Tc = 0.655\,(5.67)^{-.38} = 20.33 \text{ min}$$ OK →

So Use $Tc = 20$ min, $i_{100} = 5.67$ in/hr

STEP 3: CALCULATE PEAK DISCHARGE

$$Q_p = C\, i_{100}\, A$$

$$Q_p = (.475)(5.67)(140) = 377 \text{ CFS}$$

STEP 4: CALCULATE RETENTION VOLUME (V)

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

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

SINCE THE WATERSHED IS AGAIN OUTSIDE THE PHOENIX AREA, P_{100}^{2} MUST BE CALCULATED FROM THE DURATION CONVERSION EQUATIONS IN SECTION 2.4.4.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT ______

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______ CHECKED BY ______ DATE ______

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"

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

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

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

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

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

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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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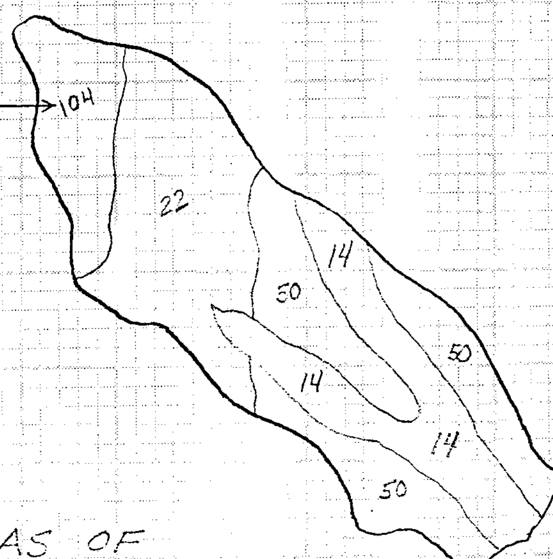
DETAIL EXAMPLE #5 COMPUTED ______ DATE ______

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:

MAP UNIT	PERCENT TOTAL AREA
14	15
50	30
22	40
104	15
	100

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT ______ PAGE 2 OF 3

DETAIL ______ COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

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 (in/hr)	PSIF (in)	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	9.4	.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 in/hr

PSIF = .80 (1.9) + .067 (4.3) + .067 (2.4) + .067 (4.3) = 2.26 in

DTHETA = .80 (.35) + .067 (.35) + .067 (.35) + .067 (.35) = .35

UNIT 50:

XKSAT = .80 (.15) + .067 (.15) + .067 (.40) + .067 (.25) = .174 in/hr

PSIF = .80 (3.5) + .067 (3.5) + .067 (4.3) + .067 (6.6) = 3.77 in

DTHETA = .80 (.35) + .067 (.35) + .067 (.35) + .067 (.40) = .35

UNIT 22

XKSAT = .80(.01) + .067(.01) + .067(.06) + .067(.25) = .029 in/hr

PSIF = .80(12.4) + .067(12.4) + .067(8.6) + .067(6.6) = 11.77 in

DTHETA = .80(.15) + .067(.15) + .067(.25) + .067(.40) = .17

UNIT 104

XKSAT = .50(.06) + .167(.40) + .167(.40) + .167(.02) = .167 in/hr

PSIF = .50(8.6) + .167(4.3) + .167(4.3) + .167(9.4) = 7.31 in

DTHETA = .50(.25) + .167(.35) + .167(.35) + .167(.20) = .28

RTIMP = 60%

STEP 4 CALCULATE WEIGHTED PARAMETERS FOR THE SUBBASIN BY PERCENTAGE OF SOIL MAP UNITS

XKSAT = .15(3.81) + .30(.174) + .40(.029) + .15(.167) = 0.66 in/hr

PSIF = .15(2.26) + .30(3.77) + .40(11.77) + .15(7.31) = 7.3 in

DTHETA = .15(.35) + .30(.35) + .40(.17) + .15(.28) = .27

RTIMP = ⟶ .15(.60) = 9%

STEP 5 SELECT A SURFACE RETENTION LOSS (IA) FOR THE SUBBASIN FROM TABLE 4-1.

FOR THIS EXAMPLE, ASSUME:

85% HILLSLOPES ⟶ .15 in

15% MOUNTAIN ⟶ .25 in

IA = .85(.15) + .15(.25) = 0.17 in

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 3

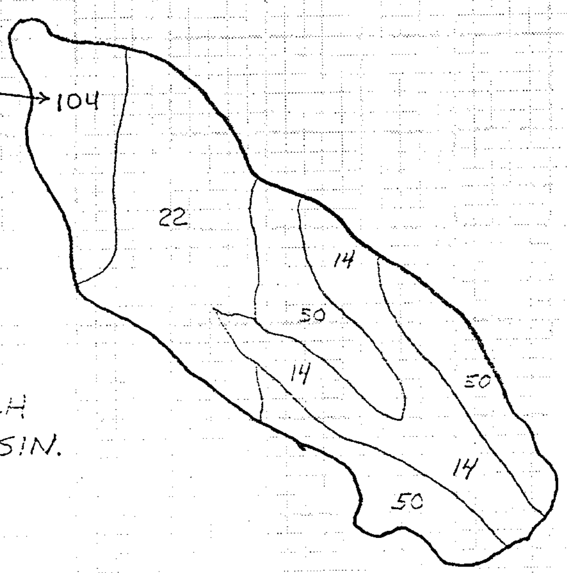
DETAIL EXAMPLE #6 COMPUTED ______ DATE ______

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:

MAP UNIT	PERCENT TOTAL AREA
14	15
50	30
22	40
104	15
	100

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	HSG	SOIL TEXTURE IL (in)	SOIL TEXTURE CNSTL (in/hr)	HSG IL (in)	HSG 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	—	—	—	—	—	—	—	—
	20	LEHMANS	70	30	SANDY CLAY LOAM	D	.6	.06	.4	.05
	20	ARIZO	70	12.5	SANDY LOAM		.7	.40		
		EBA	75	12.5	SANDY LOAM		.7	.40		
		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:

$IL = .80(1.3) + .067(.7) + .067(.8) + .067(.7) = 1.19$ in

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

UNIT 50:

$IL = .80(.6) + .067(.6) + .067(.7) + .067(.8) = .62$ in

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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT ______

DETAIL ______ COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

UNIT 22: $IL = .80(.3) + .067(.3) + .067(.6) + .067(.8) = .35$ in

$CNSTL = .80(.01) + .067(.01) + .067(.06) + .067(.25) = .03$ in/hr

UNIT 104: $IL = .50(.6) + .167(.7) + .167(.7) + .167(.4) = .60$ in

$CNSTL = .50(.06) + .167(.40) + .167(.40) + .167(.02) = .17$ in/hr

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

FOR SOIL TEXTURE:

$IL = .15(1.19) + .30(.62) + .40(.35) + .15(.60) = .59$ in

$CNSTL = .15(3.81) + .30(.17) + .40(.03) + .15(.17) = .66$ in/hr

$RTIMP = \longrightarrow .15(.60) = 9\%$

FOR HYDROLOGIC SOIL GROUPS:

$IL = .15(.6) + .30(.5) + .40(.5) + .15(.4) = .50$ in

$CNSTL = .15(.40) + .30(.25) + .40(.15) + .15(.05) = .20$ in/hr

STEP 6: SELECT A SURFACE RETENTION LOSS (IA) FOR THE SUBBASIN FROM TABLE 4-1) AND CALCULATE STRTL.

FOR THIS EXAMPLE, ASSUME: 85% HILLSLOPES $\longrightarrow$ .15 in

15% MOUNTAIN $\longrightarrow$ .25 in

$IA = .85(.15) + .15(.25) = 0.17$ in

FOR SOIL TEXTURE: $STRTL = 0.17 + 0.59 = 0.76$ in

FOR HYDROLOGIC SOIL GROUPS:

$STRTL = 0.17 + .50 = 0.67$ in

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 2

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: $2.17\ mi^2 = 1389$ 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 Tc & R" WORKSHEET (APPENDIX)

$$K_b = m\,(\log A) + b$$

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

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

$$K_b = .020$$

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

CALCULATE Tc AS A FUNCTION OF "i".

$$Tc = \left[11.4\,L^{.50}\,r^{.52}\,S^{-.31}\right] i^{-.38}$$

$$Tc = 0.703\,(i)^{-.38}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

PROJECT ______

DETAIL ______ COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

STEP 4: ENTER RAINFALL, LOSS, AND CLARK PARAMETER DATA INTO AN HEC-1 INPUT DECK, BUT SET Tc & 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 Tc. (SEE WORKSHEET AND SAMPLE HEC-1 RUN)

STEP 6: CREATE THE GRAPH OF AVERAGE EXCESS INTENSITY VS. TIME.

STEP 7: CALCULATE Tc BY ITERATION. "L" VALUES ARE READ FROM THE GRAPH. CALCULATE R.

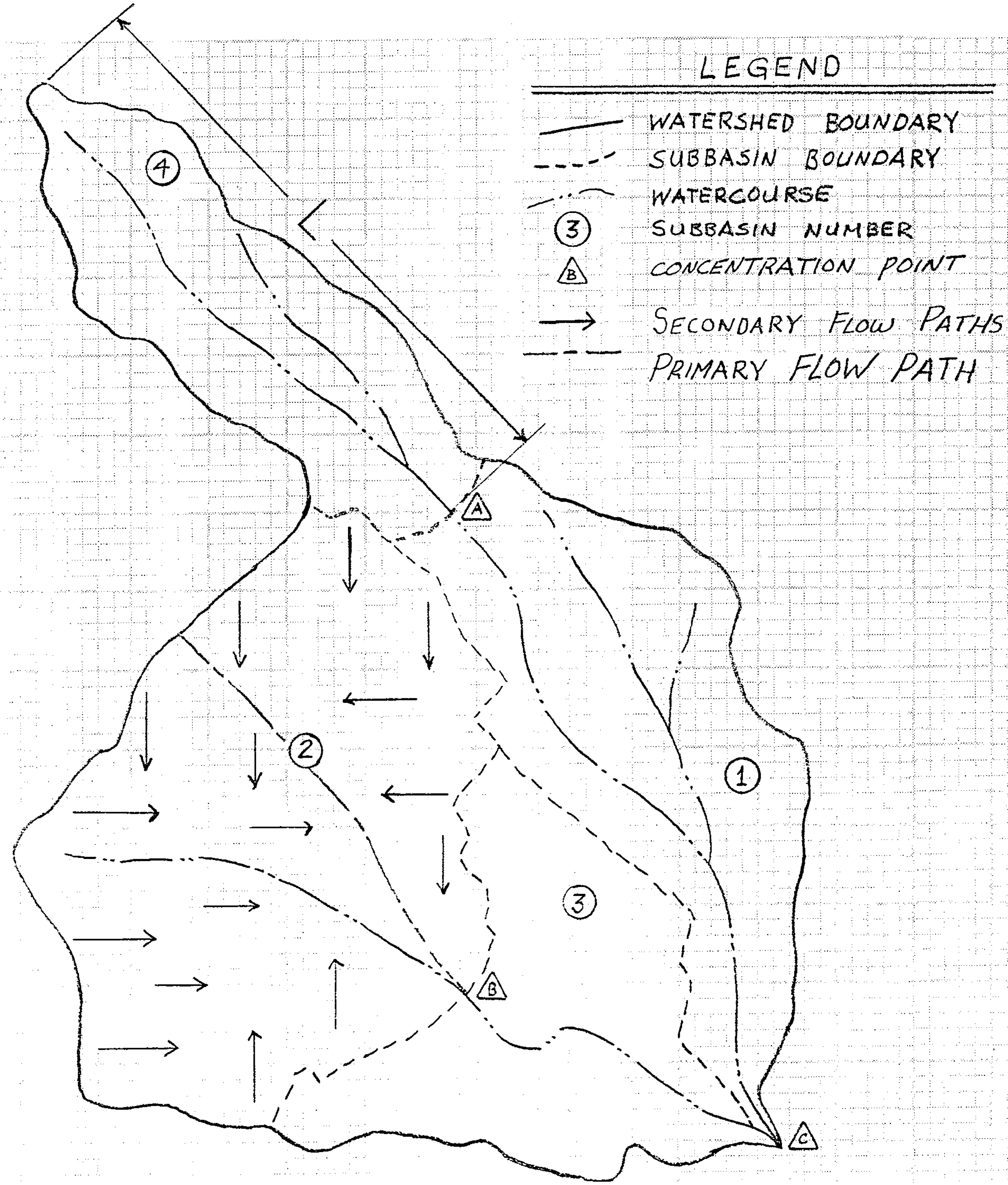
STEP 8: ENTER Tc & R VALUES INTO HEC-1 AND RUN AGAIN. (SEE SAMPLE HEC-1 RUN)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT *MARICOPA COUNTY HYDROLOGY MANUAL* PAGE *1* OF *2*

DETAIL *EXAMPLE WATERSHED* COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 2 OF 2

DETAIL EXAMPLE WATERSHED COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

SUBBASIN CHARACTERISTICS

SUBBASIN #	AREA (mi^2)	IMPERVIOUSNESS (%)	FLOW PATH LENGTH (mi)	SLOPE (ft/mi)	LAND USE
1	1.52	33	2.65	170	40% Multi-Unit Areas 60% Apartment Areas
2	2.17	21	1.85	30.5	100% Single Family Residential
3	0.96	42	1.13	104	50% Light Industrial 50% Downtown Areas
4	0.86	9	1.49	537	100% Undeveloped Desert Mountain Front

WATERCOURSES

SUBBASIN #	DESCRIPTION	GEOMETRY	AVE. BOTTOM WIDTH (ft)	AVE DEPTH (ft)	SIDE SLOPE	MANNINGS 'n'
1	Soil Cement Lined	Trap.	25	5	2:1	.018
2	Dredged Earth	Rect.	15	4'	—	.022
3	Concrete Lined	Trap.	35	4	3:1	.015
4	Natural Desert Stream	Trap	15	2	2:1	.040

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
[X] IL + ULR by soil texture
[ ] IL + ULR by hydrologic soil group

Tabulate Period of Peak Rainfall Excess

Clock Time @ end of Increm.	Increm. Excess in.
0335	.18
0340	.18
0345	.18
0350	.37
0355	.37
0400	.37
0405	.11
0410	.11

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

Accum. Time hr./min.	Increm. Excess in.	Accum. Excess in.	Avg. Excess Intensity in./hr.
5	.37	.37	4.44
10	.37	.74	4.44
15	.37	1.11	4.44
20	.18	1.29	3.87
25	.18	1.47	3.53
30	.18	1.65	3.30
35	.11	1.76	3.02
40	.11	1.87	2.81

A = 2.17 sq.mi.
L = 1.85 mi.
S = 30.5 ft/mi.

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

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

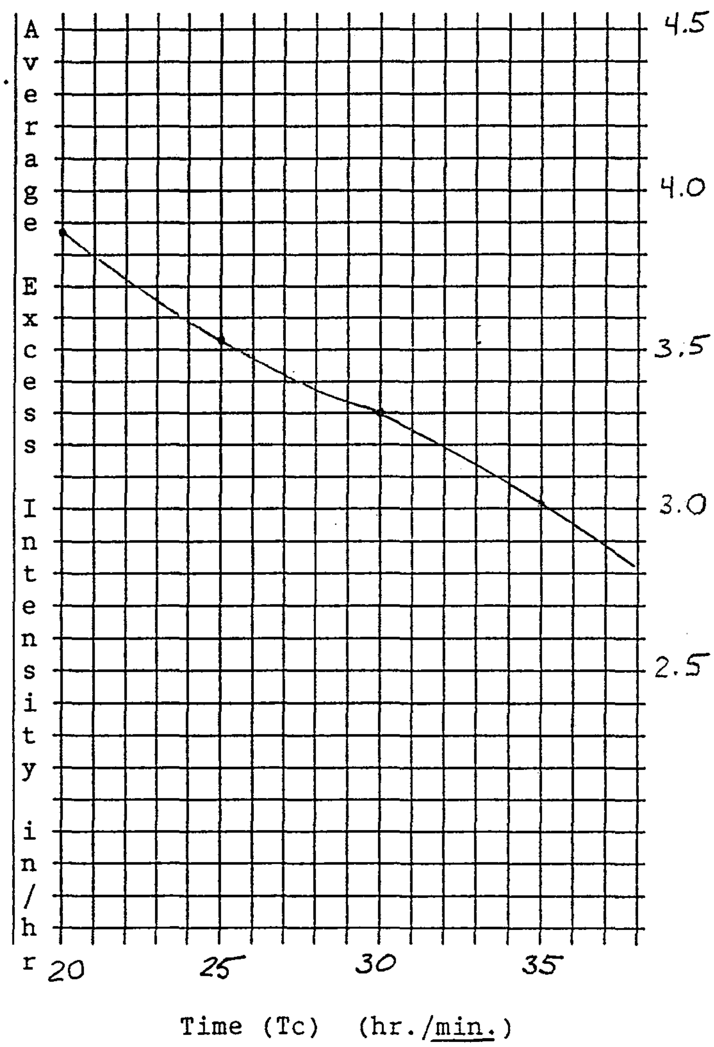
$$Tc = (\ 0.703\ )\ i^{-.38}$$

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

Tc = .440 hr.

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

R = .156 hr.



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****************************************                    ****************************************
*                                      *                    *                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                    *   U.S. ARMY CORPS OF ENGINEERS      *
*             FEBRUARY 1981            *                    * THE HYDROLOGIC ENGINEERING CENTER   *
*           REVISED 31 JAN 85          *                    *         609 SECOND STREET           *
*                                      *                    *      DAVIS, CALIFORNIA 95616        *
*    RUN DATE 9/ 5/1989   TIME15: 4: 0 *                    * (916) 440-3285 OR (FTS) 448-3285    *
*                                      *                    *                                      *
****************************************                    ****************************************
```

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

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

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85
CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT
DESCRIPTION FOR NEW DEFINITIONS.

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)    *                    *   U.S. ARMY CORPS OF ENGINEERS      *
*            FEBRUARY 1981              *                    * THE HYDROLOGIC ENGINEERING CENTER   *
*          REVISED 31 JAN 85            *                    *         609 SECOND STREET           *
*                                       *                    *      DAVIS, CALIFORNIA 95616        *
*    RUN DATE 9/ 5/1989   TIME15: 4: 3  *                    * (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
                        CNSTL         .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.
```

****

HYDROGRAPH AT STATION BASIN2

****

DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
5	SEP	0000	1	.00	.00	.00	0.
5	SEP	0005	2	.01	.00	.00	1.
5	SEP	0010	3	.01	.00	.00	4.
5	SEP	0015	4	.01	.00	.00	8.
5	SEP	0020	5	.01	.00	.00	12.
5	SEP	0025	6	.01	.00	.00	15.
5	SEP	0030	7	.01	.00	.00	17.
5	SEP	0035	8	.01	.01	.00	18.
5	SEP	0040	9	.01	.01	.00	20.
5	SEP	0045	10	.01	.01	.00	21.
5	SEP	0050	11	.01	.01	.00	23.
5	SEP	0055	12	.01	.01	.00	25.
5	SEP	0100	13	.01	.01	.00	28.
5	SEP	0105	14	.01	.01	.00	31.
5	SEP	0110	15	.01	.01	.00	32.
5	SEP	0115	16	.01	.01	.00	32.
5	SEP	0120	17	.01	.01	.00	32.
5	SEP	0125	18	.01	.01	.00	32.
5	SEP	0130	19	.01	.01	.00	33.
5	SEP	0135	20	.01	.01	.00	33.
5	SEP	0140	21	.01	.01	.00	33.
5	SEP	0145	22	.01	.01	.00	33.
5	SEP	0150	23	.01	.01	.00	33.
5	SEP	0155	24	.01	.01	.00	33.
5	SEP	0200	25	.01	.01	.00	33.
5	SEP	0205	26	.01	.01	.00	32.
5	SEP	0210	27	.01	.01	.00	32.
5	SEP	0215	28	.01	.01	.00	32.
5	SEP	0220	29	.01	.01	.00	33.
5	SEP	0225	30	.01	.01	.00	35.
5	SEP	0230	31	.01	.01	.00	38.
5	SEP	0235	32	.01	.01	.00	40.
5	SEP	0240	33	.01	.01	.00	43.
5	SEP	0245	34	.01	.01	.00	45.
5	SEP	0250	35	.02	.02	.00	48.
5	SEP	0255	36	.02	.02	.00	52.
5	SEP	0300	37	.02	.02	.00	58.
5	SEP	0305	38	.03	.03	.01	65.
5	SEP	0310	39	.03	.03	.01	76.
5	SEP	0315	40	.03	.03	.01	89.
5	SEP	0320	41	.09	.07	.02	106.
5	SEP	0325	42	.09	.07	.02	142.
5	SEP	0330	43	.09	.01	.08	225.
5	SEP	0335	44	.19	.01	.18	461.
5	SEP	0340	45	.19	.01	.18	955.
5	SEP	0345	46	.19	.01	.18	1563.
5	SEP	0350	47	.38	.01	.37	2174.
5	SEP	0355	48	.38	.01	.37	2980.
5	SEP	0400	49	.38	.01	.37	3915.
5	SEP	0405	50	.12	.01	.11	4552.
5	SEP	0410	51	.12	.01	.11	4498.
5	SEP	0415	52	.12	.01	.11	3901.
5	SEP	0420	53	.04	.01	.03	3207.
5	SEP	0425	54	.04	.01	.03	2501.
5	SEP	0430	55	.04	.01	.03	1845.
5	SEP	0435	56	.03	.01	.02	1351.
5	SEP	0440	57	.03	.01	.02	991.
5	SEP	0445	58	.03	.01	.02	729.
5	SEP	0450	59	.02	.01	.01	552.
5	SEP	0455	60	.02	.01	.01	420.
5	SEP	0500	61	.02	.01	.01	321.
5	SEP	0505	62	.02	.01	.00	249.
5	SEP	0510	63	.02	.01	.00	197.
5	SEP	0515	64	.02	.01	.00	150.
5	SEP	0520	65	.01	.01	.00	115.
5	SEP	0525	66	.01	.01	.00	89.
5	SEP	0530	67	.01	.01	.00	68.
5	SEP	0535	68	.01	.00	.00	54.
5	SEP	0540	69	.01	.00	.00	43.
5	SEP	0545	70	.01	.00	.00	35.
5	SEP	0550	71	.01	.01	.00	30.
5	SEP	0555	72	.01	.01	.00	29.
5	SEP	0600	73	.01	.01	.00	32.
5	SEP	0605	74	.00	.00	.00	33.
5	SEP	0610	75	.00	.00	.00	30.
5	SEP	0615	76	.00	.00	.00	23.
5	SEP	0620	77	.00	.00	.00	15.
5	SEP	0625	78	.00	.00	.00	10.
5	SEP	0630	79	.00	.00	.00	6.
5	SEP	0635	80	.00	.00	.00	3.
5	SEP	0640	81	.00	.00	.00	2.
5	SEP	0645	82	.00	.00	.00	1.
5	SEP	0650	83	.00	.00	.00	1.
5	SEP	0655	84	.00	.00	.00	0.
5	SEP	0700	85	.00	.00	.00	0.

****

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 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	BASIN2	4552.	4.08	554.	475.	475.	2.17		

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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT 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:

$L = 1.49 \text{ mi}$

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

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

% 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 Tc & R" WORKSHEET (APPENDIX). ALTHOUGH THIS BASIN IS QUITE STEEP, THE TERAIN IS DESCRIBED AS "ROUGH" AND THE VEGETATION "MODERATE", SO INTERPOLATE BETWEEN "HILLSLOPES" AND "MOUNTAINS".

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

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

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

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

$$K_b = .100$$

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

CALCULATE Tc AS A FUNCTION OF i:

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

$$T_c = [.599]\, i^{-.38}$$

STEP 4: ENTER RAINFALL, LOSS, AND CLARK PARAMETER DATA INTO AN HEC-1 INPUT DECK, BUT SET T_c & 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 & 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
[X] IL + ULR by hydrologic soil group

Tabulate Period of Peak Rainfall Excess

Clock Time @ end of Increm.	Increm. Excess in.
0100	.20
0105	.72
0110	.37
0115	.31
0120	.09
0125	.06
0130	.05

Rearrange Incremental Excesses in Order of Decreasing Average Intensity

Accum. Time hr./min.	Increm. Excess in.	Accum. Excess in.	Avg. Excess Intensity in./hr.
5	.72	.72	8.64
10	.37	1.09	6.54
15	.31	1.40	5.60
20	.20	1.60	4.80
25	.09	1.69	4.06
30	.06	1.75	3.50
35	.05	1.80	3.09

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

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

$K_b = (-.0275)\log(.86 * 640) + (.175)$

$K_b = 0.100$

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

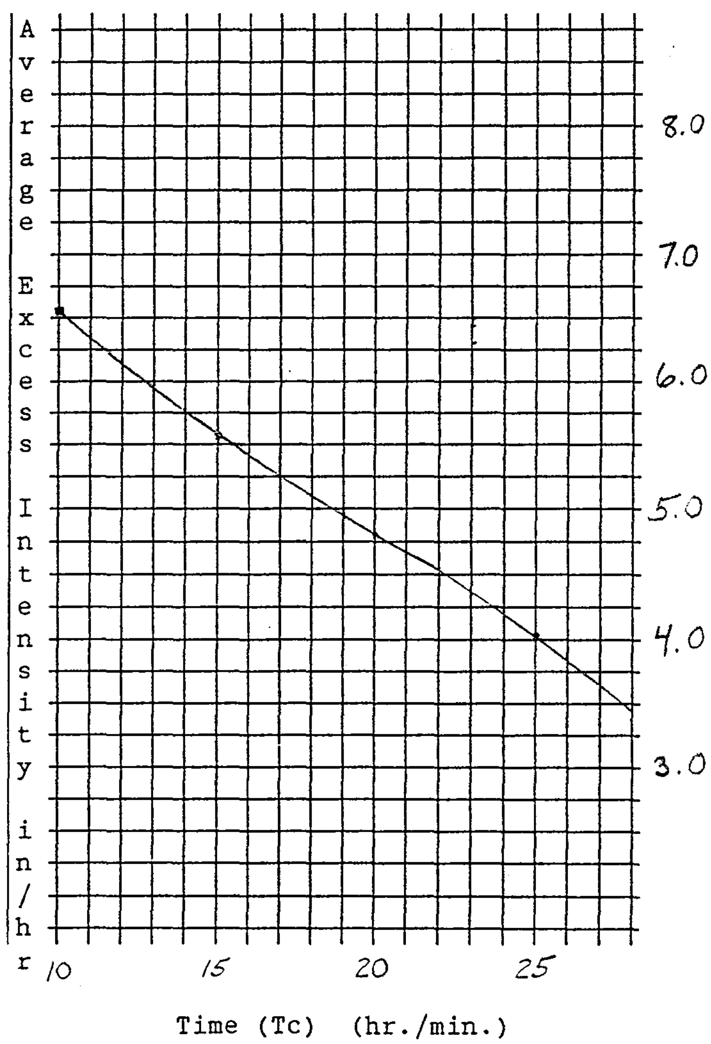
$$Tc = (0.599)\, i^{-.38}$$

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

Tc = .325 hr.

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

R = .159 hr.



```
****************************************                    ************************************
*                                      *                    *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                    *   U.S. ARMY CORPS OF ENGINEERS
*            FEBRUARY 1981             *                    *  THE HYDROLOGIC ENGINEERING CENTER
*          REVISED 31 JAN 85           *                    *          609 SECOND STREET
*                                      *                    *       DAVIS, CALIFORNIA 95616
*    RUN DATE 9/ 6/1989   TIME10: 9:40 *                    *  (916) 440-3285 OR (FTS) 448-3285
*                                      *                    *
****************************************                    ************************************
```

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

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

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85
CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT
DESCRIPTION FOR NEW DEFINITIONS.

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
5	SEP	0000	1	.00	.00	.00	0.
5	SEP	0005	2	.03	.03	.00	0.
5	SEP	0010	3	.02	.02	.00	0.
5	SEP	0015	4	.01	.01	.00	0.
5	SEP	0020	5	.01	.01	.00	0.
5	SEP	0025	6	.01	.01	.00	0.
5	SEP	0030	7	.04	.04	.00	0.
5	SEP	0035	8	.07	.07	.00	0.
5	SEP	0040	9	.08	.08	.00	0.
5	SEP	0045	10	.10	.10	.00	0.
5	SEP	0050	11	.11	.11	.00	0.
5	SEP	0055	12	.15	.15	.00	0.
5	SEP	0100	13	.26	.06	.20 4	18.
5	SEP	0105	14	.74	.02	.72 1	141.
5	SEP	0110	15	.38	.02	.37 2	562.
5	SEP	0115	16	.32	.02	.31 3	1473.
5	SEP	0120	17	.10	.02	.09 5	2176.
5	SEP	0125	18	.08	.02	.06 6	2177.
5	SEP	0130	19	.06	.02	.05 7	1792.
5	SEP	0135	20	.02	.02	.00 8	1284.
5	SEP	0140	21	.02	.02	.00 9	888.
5	SEP	0145	22	.02	.02	.01 10	591.
5	SEP	0150	23	.01	.01	.00	368.
5	SEP	0155	24	.03	.02	.01	231.
5	SEP	0200	25	.02	.02	.00	148.
5	SEP	0205	26	.00	.00	.00	99.
5	SEP	0210	27	.00	.00	.00	67.
5	SEP	0215	28	.00	.00	.00	43.
5	SEP	0220	29	.00	.00	.00	23.
5	SEP	0225	30	.00	.00	.00	12.
5	SEP	0230	31	.00	.00	.00	7.
5	SEP	0235	32	.00	.00	.00	3.
5	SEP	0240	33	.00	.00	.00	2.
5	SEP	0245	34	.00	.00	.00	1.
5	SEP	0250	35	.00	.00	.00	1.
5	SEP	0255	36	.00	.00	.00	0.
5	SEP	0300	37	.00	.00	.00	0.

PEAK EXCESS

****

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

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

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL

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:

AREA $(A) = 5.19\ mi^2$

LENGTH OF WATERCOURSE $(L) = 5.2\ mi$
(FROM OUTLET TO DRAINAGE BOUNDARY)

LENGTH OF WATERCOURSE TO A POINT OPPOSITE TO CENTROID $(L_{ca}) = 3.0\ mi$

$$SLOPE\ (S) = \frac{\Delta H}{L} = \frac{2900 - 1500}{5.2} = 269\ ft/mi$$

$$BASIN\ FACTOR = \frac{L L_{ca}}{S^{1/2}} = \frac{(5.2)(3.0)}{269^{1/2}} = .95$$

STEP 2: SELECT K_n BY COMPARISON WITH SIMILAR WATERSHEDS.

LIST SIMILAR WATERSHEDS AND DATA.

FROM LAG RELATIONSHIP IN HYDROLOGY MANUAL:

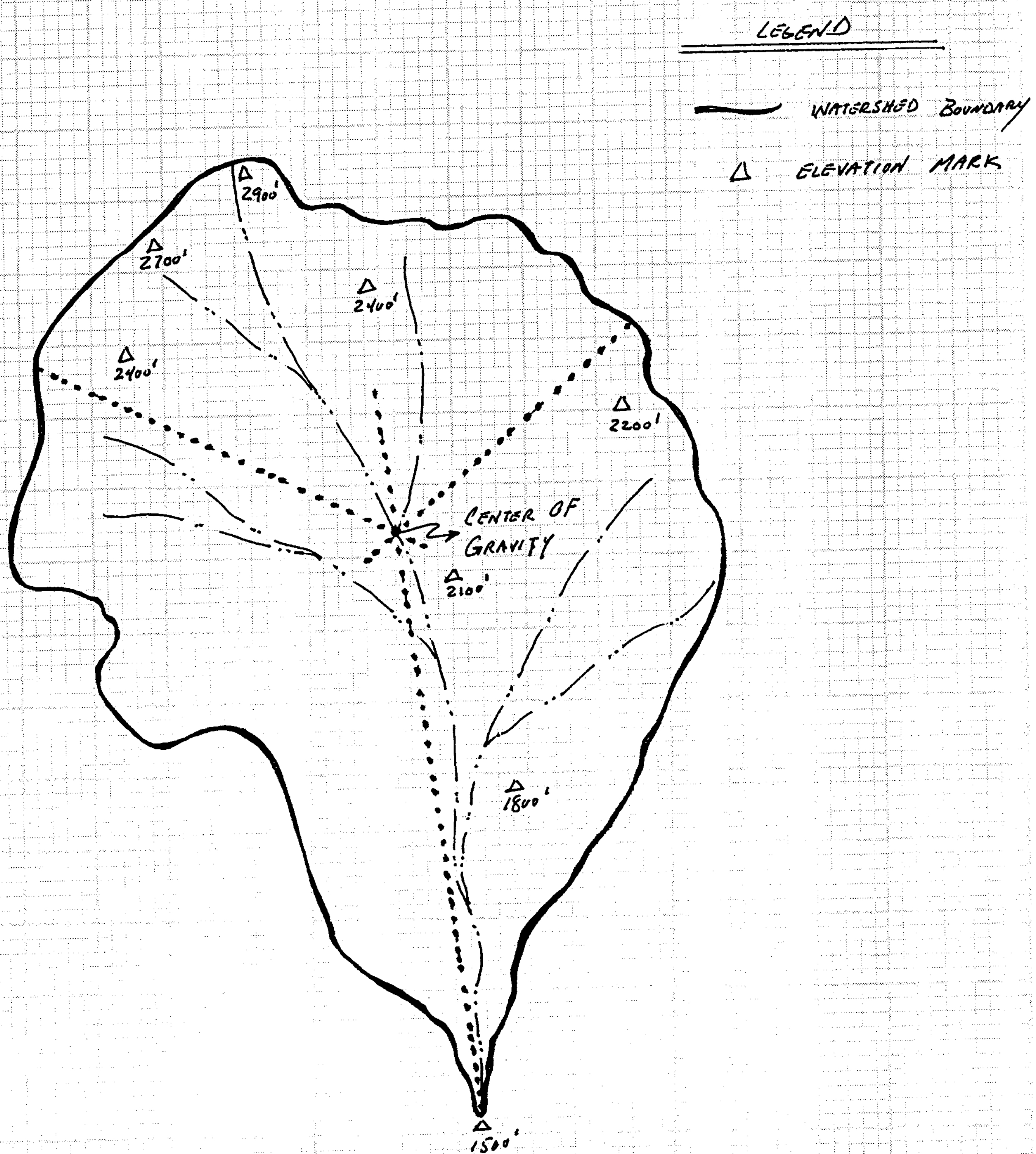
#	NAME	A	L	Lca	S	LAG	Kn
3	SANTA ANITA CK.	10.8	5.8	2.5	690	1.1	.050
5	EATON WASH	9.5	7.3	4.4	600	1.3	.050
11	LIVE OAK CREEK	2.3	2.9	1.5	700	0.8	.070
17	BROADWAY BASAN	2.5	3.4	1.7	100	.28	.015 (URBAN)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 2 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 ______ COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

ALSO LOOK AT K_n FOR HYDROLOGICALLY SIMILAR WATERSHEDS.

AGAIN, FROM LAG RELATIONSHIPS:

#	NAME	K_n
22	NEW RIVER AT ROCK SPRINGS	.045
23	NEW RIVER AT NEW RIVER	.045
24	NEW RIVER AT BELL ROAD	.037
25	SKUNK CREEK	.033

ASSUMING THAT THIS IS A NATURAL WATERSHED IN THE MOUNTAINS AND FOOTHILLS OF MARICOPA COUNTY, K_n IS SELECTED TO BE BETWEEN .040 AND .050.

EVENTHOUGH THE WATERSHED IS SMALL WHICH WILL NOT CONTRIBUTE TO GREAT HYDRAULIC EFFICIENCY, i.e., POORLY DEFINED CHANNELS, STILL THE SLOPE IS QUITE STEEP. $\Longrightarrow$ SELECT $K_n = .040$

STEP 3: CALCULATE LAG. IN THIS EXAMPLE THE LAG RELATIONSHIP IS:

$$LAG = 20\,K_n\,(L L_{ca} / S^{1/2})^{.38}$$

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 4 OF 9

DETAIL EXAMPLE # 9 COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

$$LAG = 20(.04)(.95)^{.38} = .78 \text{ HRS}$$

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)

$$Q_{ULT} = \frac{645.33\,(5.19)}{(10/60)} = 20096 \text{ CFS}$$

D WAS SELECTED TO BE 10 MINUTES. THIS IS WITHIN THE RANGE OF (.10 – .25) x (LAG TIME) AS SUGGESTED IN THE HYDROLOGY MANUAL. IT SHOULD BE NOTED THAT D IS ALSO THE HYDROGRAPH TIME STEP, DT, USED IN HEC-1 AS THE "IT" RECORD.

STEP 5: AT THIS POINT ALL OF THE NECESSARY PARAMETERS ARE FOUND. A UNIT-GRAPH CAN BE DEVELOPED EITHER MANUALLY OR BY USING THE "LAPRE1" PROGRAM. IT SHOULD BE NOTED THAT USING THE LAPRE1 INCREASES THE COMPUTATIONAL EFFICIENCY OF THE UNIT-GRAPH. BOTH METHODS' RESULTS ARE SHOWN NEXT.

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

CONSIDER A WATERSHED WITH THE FOLLOWING CHARACTERISTICS:

$A = 5.19\ mi^2$

$LAG = 0.78\ hr.$

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

① CONSTRUCT A TABLE SIMILAR TO THE FOLLOWING:
(THE DISCHARGE & Time columns form the "DIMENSIONED" S-Curve)

ORDINATE	% Q_{ULT}	DISCHARGE (CFS)	% LAG	TIME (hr.)
0	0	0	0.0	.0
1	2	402	23.0	.179
2	4	804	31.0	.242
3	6	1206	37.0	.289
4	8	1608	42.0	.328
5	10	2010	46.0	.359
6	12	2411	49.8	.388
7	14	2813	53.4	.416
8	16	3215	56.8	.443
9	18	3617	60.0	.468
10	20	4019	63.1	.492
11	22	4421	66.1	.516
12	24	4823	69.0	.538
13	26	5225	71.8	.560
14	28	5627	74.4	.580
15	30	6029	76.8	.599
16	32	6431	79.1	.617
17	34	6833	81.2	.634
18	36	7234	83.2	.649
19	38	7636	85.1	.664
20	40	8038	86.8	.677
21	42	8440	88.8	.693
22	44	8842	91.0	.710
23	46	9244	93.8	.732
24	48	9646	96.8	.755
25	50	10048	100.0	.780
26	52	10450	103.4	.806
27	54	10852	107.0	.835
28	56	11254	110.8	.864
29	58	11655	114.7	.895
30	60	12057	118.7	.926
31	62	12459	122.9	.958
32	64	12861	127.3	.993
33	66	13263	131.9	1.029
34	68	13655	136.7	1.067
35	70	14067	141.7	1.106
36	72	14469	147.1	1.148
37	74	14871	152.8	1.192
38	76	15273	158.8	1.239
39	78	15675	165.8	1.294
40	80	16076	172.9	1.348
41	82	16478	181.6	1.417
42	84	16880	191.0	1.490
43	86	17282	201.0	1.568
44	88	17684	212.0	1.654
45	90	18086	226.0	1.763
46	92	18488	244.0	1.903
47	94	18890	265.0	2.067
48	96	19292	295.0	2.301
49	98	19694	342.0	2.668
50	100	20096	462.0	3.604

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT

DETAIL EXAMPLE #9 COMPUTED DATE

CHECKED BY DATE

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

ORDINATE	TIME (hr)	Q_{S_1} (cfs)	Q_{S_2} (cfs) (10 min lag)	Q_{UG} (cfs)
1	0	0	0	0
2	.167	375	0	375
3	.333	1673	375	1298
4	.500	4153	1673	2480
5	.667	7729	4153	3576
6	.833	10824	7729	3095
7	1.000	12939	10824	2115
8	1.167	14643	12939	1704
9	1.333	15942	14643	1299
10	1.500	16931	15942	989
11	1.667	17732	16931	801
12	1.833	18287	17732	555
13	2.000	18725	18287	438
14	2.167	19061	18725	336
15	2.333	19327	19061	266
16	2.500	19510	19327	183
17	2.667	19694	19510	184
18	2.833	19765	19694	71
19	3.000	19837	19765	72
20	3.167	19908	19837	71
21	3.333	19971	19908	63
		0	19971	0

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT

DETAIL EXAMPLE #9 COMPUTED DATE

CHECKED BY DATE

(2) cont... NOTICE THE BEHAVIOR OF THE UNIT GRAPH VALUES FROM TIME = 2.5 hours ON. This is CAUSED BY THE LONGER TIME INCREMENTS AT THE 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, next page).

FINAL 10-MIN. UNIT HYDROGRAPH

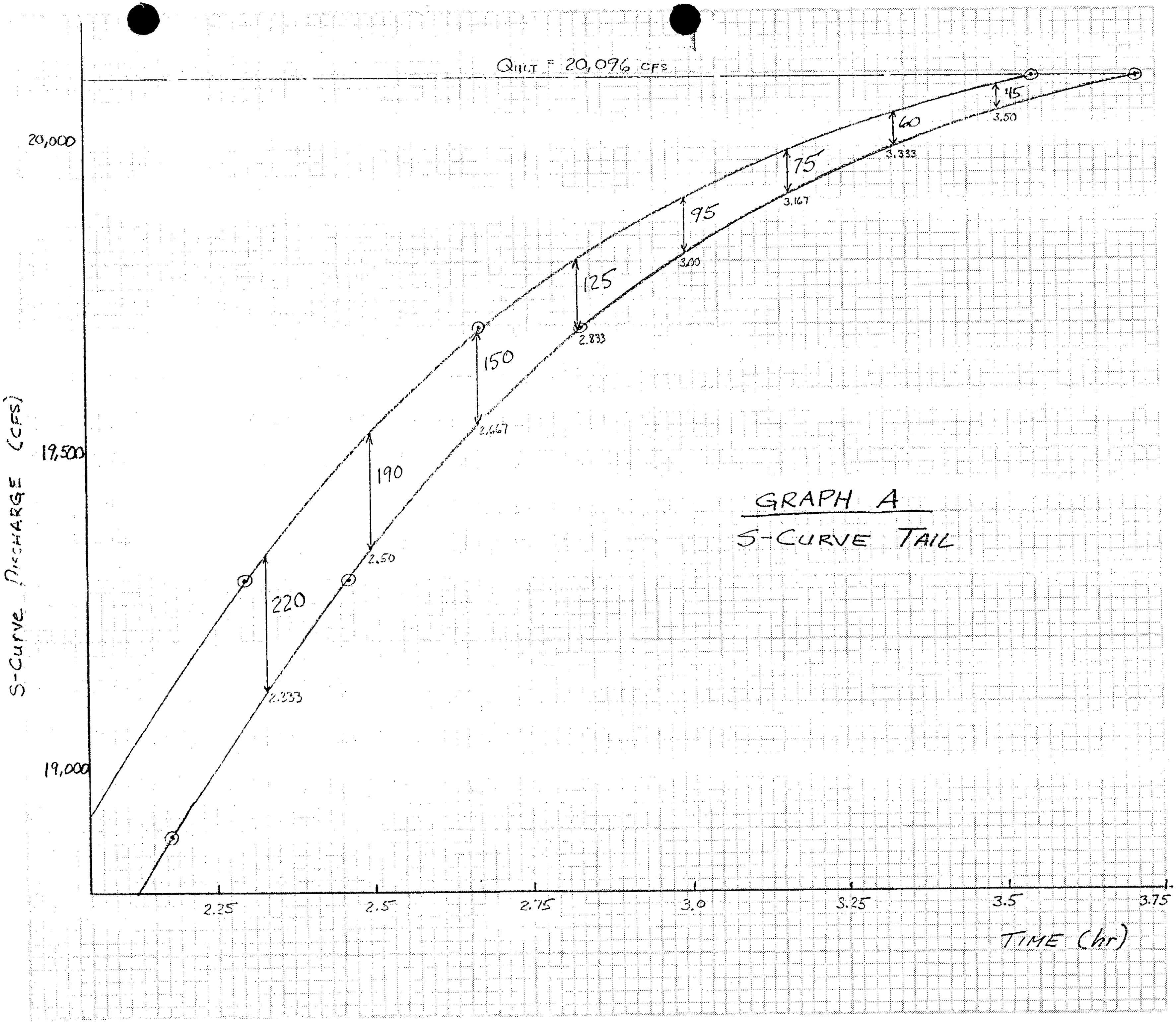
TIME (HR)	DISCHARGE (cfs)
0	0
.167	375
.333	1298
.500	2480
.667	3576
.833	3095
1.000	2115
1.667	1704
1.333	1299
1.500	989
1.667	801
1.833	555
2.000	438
2.167	336
2.333	266
2.500	183
2.667	150
2.833	125
3.000	95
3.167	75
3.333	60
3.500	45
3.667	0

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT ______ PAGE 8 OF 9

DETAIL EXAMPLE #9 ______ COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL

DETAIL ______ COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

THE FOLLOWING ILLUSTRATES THE GENERATED HYDROGRAPH

ΔT (MIN)	DISCHARGE (CFS) (MANUAL CALC.)	DISCHARGE (CFS) (COMPUTER OUTPUT)
0	0	0
10	375	374
20	1298	1320
30	2480	2715
40	3576	3303
50	3095	3170
60	2115	2263
70	1704	1684
80	1299	1240
90	989	920
100	801	745
110	555	664
120	438	438
130	336	340
140	266	210
150	183	184
160	150	113
170	125	106
180	95	100
190	75	73
200	60	50
210	45	48
	0	35
		0

THE COMPUTED VALUES ARE READY FOR HEC-1 AS "UI" RECORD.

```
****************************************                ***************************************
**                                                      *
*                                      *                *  U.S. ARMY CORPS OF ENGINEERS
*                                                       *  THE HYDROLOGIC ENGINEERING CENTER
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                *         609 SECOND STREET
*                                                       *      DAVIS, CALIFORNIA 95616
*           FEBRUARY 1981              *              * (916) 440-3285 OR (FTS) 448-3285   *
*                                                       *
*         REVISED 31 JAN 85            *                ***************************************
*
*                                      *
*
*  RUN DATE 9/25/1989   TIME15:28:28 *
*                                      *
*
****************************************
**
```

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

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

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85
CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT
DESCRIPTION FOR NEW DEFINITIONS.

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

*** FREE ***
  1           ID       SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
  2           ID       MARICOPA COUNTY HYDROLOGY MANUAL
              * ********************************************************************
  3           ID       EXAMPLE # 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     374    1320    2715    3303    3170    2263    1684    1240     920
 20           UI     745     664     438     340     210     184     113     106     100      73
 21           UI      50      48      35       0
 22           ZZ
```

```
*****************************************                    ***************************************
**
*                                       *                    *
*
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                    *   U.S. ARMY CORPS OF ENGINEERS
*
*             FEBRUARY 1981             *                    * THE HYDROLOGIC ENGINEERING CENTER
*
*           REVISED 31 JAN 85           *                    *          609 SECOND STREET
*
*                                       *                    *       DAVIS, CALIFORNIA 95616
*
*  RUN DATE 9/25/1989   TIME15:28:31 *                    *  (916) 440-3285 OR (FTS) 448-3285   *
*                                       *                    *
*
*****************************************                    ***************************************
**

                              SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
                              MARICOPA COUNTY HYDROLOGY MANUAL
                              EXAMPLE # 9 - S-GRAPH 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
                    IPRNT          0  PRINT CONTROL
                    IPLOT          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
TEMPERATURE	DEGREES FAHRENHEIT

*** *
**

```
          **************
          *            *
10 KK     *    BASIN   *
          *            *
          **************
                       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	.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, 23 ORDINATES, VOLUME = 1.00

.0	374.0	1320.0	2715.0	3303.0	3170.0	2263.0	1684.0	1240.0	920.0
745.0	664.0	438.0	340.0	210.0	184.0	113.0	106.0	100.0	73.0
50.0	48.0	35.0							

***

****
**

HYDROGRAPH AT STATION BASIN

****
**

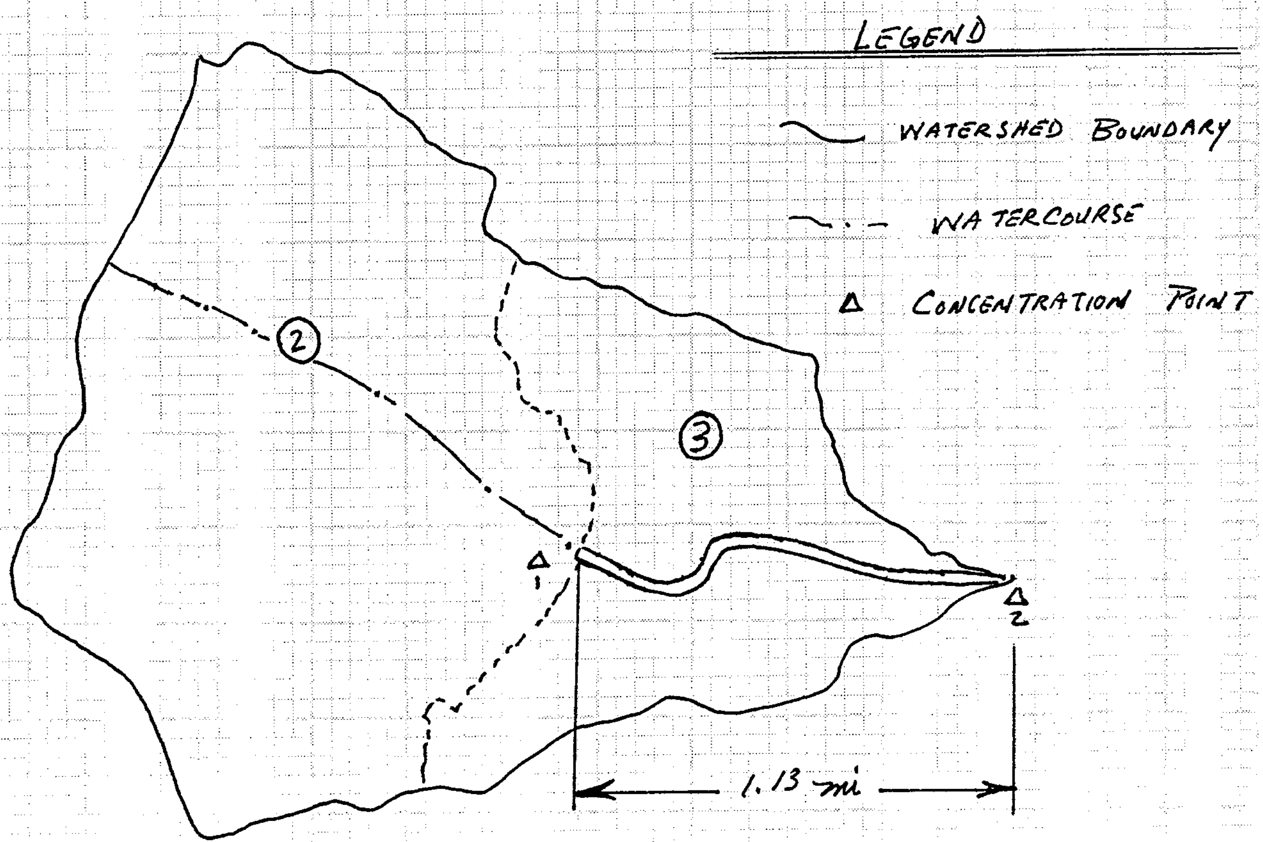
DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA MON HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
5 SEP 0000	1	.00	.00	.00	0.	*	5 SEP 0410	26	.26	.04	.22	1901.
5 SEP 0010	2	.02	.02	.00	0.	*	5 SEP 0420	27	.20	.04	.16	2673.
5 SEP 0020	3	.01	.01	.00	0.	*	5 SEP 0430	28	.13	.04	.09	3288.
5 SEP 0030	4	.01	.01	.00	1.	*	5 SEP 0440	29	.08	.04	.04	3701.
5 SEP 0040	5	.02	.02	.00	2.	*	5 SEP 0450	30	.07	.04	.03	3670.
5 SEP 0050	6	.02	.02	.00	4.	*	5 SEP 0500	31	.06	.04	.02	3249.
5 SEP 0100	7	.03	.02	.00	5.	*	5 SEP 0510	32	.04	.04	.00	2737.
5 SEP 0110	8	.02	.02	.00	6.	*	5 SEP 0520	33	.03	.03	.00	2238.
5 SEP 0120	9	.02	.02	.00	8.	*	5 SEP 0530	34	.01	.01	.00	1752.
5 SEP 0130	10	.02	.02	.00	9.	*	5 SEP 0540	35	.00	.00	.00	1396.
5 SEP 0140	11	.02	.02	.00	10.	*	5 SEP 0550	36	.01	.01	.00	1081.
5 SEP 0150	12	.02	.02	.00	11.	*	5 SEP 0600	37	.02	.02	.00	826.
5 SEP 0200	13	.03	.02	.00	11.	*	5 SEP 0610	38	.00	.00	.00	615.
5 SEP 0210	14	.02	.02	.00	12.	*	5 SEP 0620	39	.00	.00	.00	461.
5 SEP 0220	15	.02	.02	.00	12.	*	5 SEP 0630	40	.00	.00	.00	353.
5 SEP 0230	16	.03	.02	.00	12.	*	5 SEP 0640	41	.00	.00	.00	263.
5 SEP 0240	17	.03	.03	.00	13.	*	5 SEP 0650	42	.00	.00	.00	204.
5 SEP 0250	18	.03	.03	.00	13.	*	5 SEP 0700	43	.00	.00	.00	165.
5 SEP 0300	19	.04	.04	.00	13.	*	5 SEP 0710	44	.00	.00	.00	128.
5 SEP 0310	20	.09	.08	.00	14.	*	5 SEP 0720	45	.00	.00	.00	90.
5 SEP 0320	21	.25	.24	.01	16.	*	5 SEP 0730	46	.00	.00	.00	71.
5 SEP 0330	22	.42	.05	.37	21.	*	5 SEP 0740	47	.00	.00	.00	49.
5 SEP 0340	23	.11	.04	.07	168.	*	5 SEP 0750	48	.00	.00	.00	28.
5 SEP 0350	24	.28	.04	.24	551.	*	5 SEP 0800	49	.00	.00	.00	16.
5 SEP 0400	25	.44	.04	.40	1219.	*	5 SEP 0810	50	.00	.00	.00	8.

****
**

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

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW					
			6-HR	24-HR	72-HR	8.17-HR	(CFS)	(HR)
		(CFS)	3701.	4.67		917.	675.	675.	675.
		(INCHES)	1.643	1.647	1.647	1.647		

KINEMATIC WAVE ROUTING

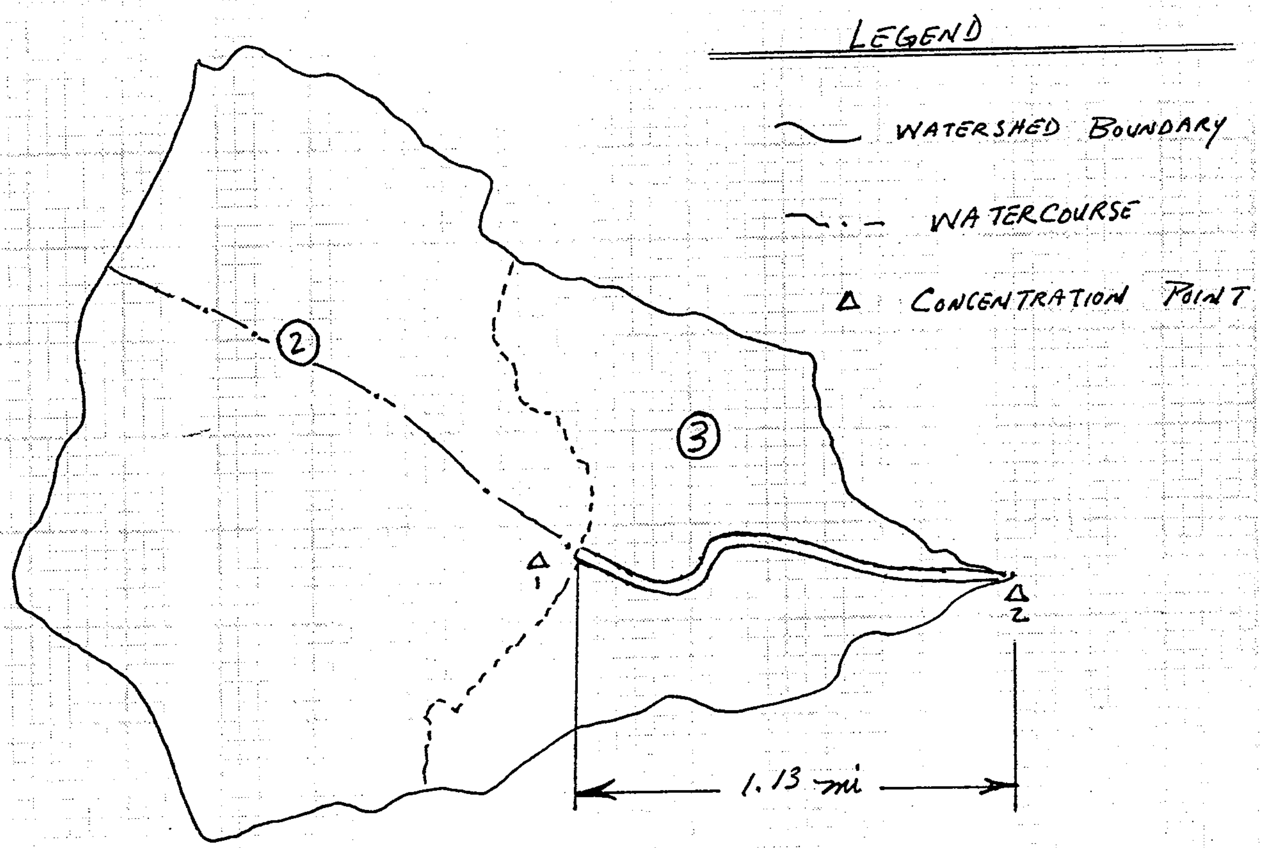


SCENARIO: PREVIOUSLY, IN EXAMPLE # 7, PEAK FLOW AT CONCENTRATION POINT 1 ($\underset{1}{\Delta}$) WAS CALCULATED FOR SUB-BASIN (2). THIS EXAMPLE WILL USE KINEMATIC WAVE ROUTING THROUGH THE 1.13 mi LONG REACH TO CALCULATE THE PEAK AT CONCENTRATION POINT 2 ($\underset{2}{\Delta}$).

STEP 1: COLLECT NECESSARY DATA FOR THE CHANNEL.

TYPE: CONCRETE LINED, TRAPOZOIDAL

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, TRAPOZOIDAL

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

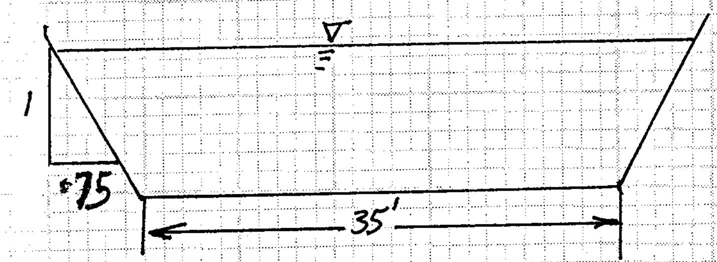
AVE. DEPTH : 4 ft

SIDE SLOPE : .75 : 1.00

MANNING'S n : .015

BOTTOM WIDTH : 35 ft

CHANNEL SLOPE : .018 ft/ft



SINCE KINEMATIC WAVE ROUTING DOES NOT CHECK FOR CHANNEL CAPICITY, DESIGN PARAMETERS MUST BE CHECKED TO ASSURE PROPER CONVEYANCE THROUGH THE REACH.

OTHERWISE KINEMATIC WAVE ROUTING WILL AUTOMATICALLY EXTEND CHANNEL BOUNDARIES TO CONTAIN THE FLOW.

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

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

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

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

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85
CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT
DESCRIPTION FOR NEW DEFINITIONS.

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

*** FREE ***
    1          ID      SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
    2          ID      MARICOPA COUNTY HYDROLOGY MANUAL
               * ***********************************************************************
    3          ID      EXAMPLE # 10 - KINEMATIC WAVE ROUTING
               * ***********************************************************************
    4          IT       5 05SEP89    0000      75
    5          IO       3
               * ***********************************************************************

    6          KK  BASIN2
    7          KM   COMPUTE DISCHARGE AT THE OUTLET OF BASIN #2
    8          IN      15
    9          PB    3.25
   10          PC     0.0    .550    1.05    1.70    2.65    3.45    4.35    5.20    6.05    6.90
   11          PC     8.1    9.40   11.35    14.5   22.85   40.85   75.85   86.85    91.0   93.85
   12          PC   95.95    97.5   98.35    98.9    100.
   13          BA    2.17
   14          LU     .65     .20     21.
   15          UA       0       5      16      30      65      77      84      90      94      97
   16          UA     100
   17          UC    .440    .156
               * ***********************************************************************

   18          KK   ROUTE
   19          KM   ROUTE THROUGH DOWNSTREAM BASIN USING KINEMATIC ROUTING
   20          KO       1       2
   21          RK  5966.4    .018   0.015           TRAP     35.    0.75
   22          ZZ
```

```
*****************************************                              ****************************************
*                                       *                              *                                      *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)    *                              *  U.S. ARMY CORPS OF ENGINEERS        *
*           FEBRUARY 1981               *                              *  THE HYDROLOGIC ENGINEERING CENTER   *
*         REVISED 31 JAN 85             *                              *        609 SECOND STREET             *
*                                       *                              *     DAVIS, CALIFORNIA 95616          *
*  RUN DATE 9/ 8/1989   TIME11:18:13    *                             *  (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	TIME		MAXIMUM AVERAGE FLOW			
(CFS)	(HR)		6-HR	24-HR	72-HR	6.17-HR
		(CFS)				
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

*** ***

```
                *             *
18 KK           *    ROUTE    *
                *             *
                ***************
                      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       35.00  BOTTOM WIDTH OR DIAMETER
                             Z         .75  SIDE SLOPE

                                                            ***

        KINEMATIC STREAM ROUTING USED FOR THIS REACH

        COMPUTED KINEMATIC PARAMETERS
            ALPHA        M         DT (MIN)    DX (FT)
           1.5105      1.591           .83     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	TIME		MAXIMUM AVERAGE FLOW					
(CFS)	(HRS)		6-HR	24-HR	72-HR	6.17-HR	(CFS)	(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

```
                                                         STATION     ROUTE

                         (I) INFLOW,   (O) OUTFLOW
          0.     1000.     2000.     3000.     4000.     5000.        0.        0.        0.        0.        0.        0.        0.
DAHRMN PER
  )000    1I---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.
  .005    2I         .         .         .         .         .         .         .         .         .         .         .         .
 50010    3I         .         .         .         .         .         .         .         .         .         .         .         .
 50015    4I         .         .         .         .         .         .         .         .         .         .         .         .
 50020    5I         .         .         .         .         .         .         .         .         .         .         .         .
 50025    6I         .         .         .         .         .         .         .         .         .         .         .         .
 50030    7I         .         .         .         .         .         .         .         .         .         .         .         .
 50035    8I         .         .         .         .         .         .         .         .         .         .         .         .
 50040    9I         .         .         .         .         .         .         .         .         .         .         .         .
 50045   10I         .         .         .         .         .         .         .         .         .         .         .         .
 50050   11I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50055   12I         .         .         .         .         .         .         .         .         .         .         .         .
 50100   13I         .         .         .         .         .         .         .         .         .         .         .         .
 50105   14I         .         .         .         .         .         .         .         .         .         .         .         .
 50110   15I         .         .         .         .         .         .         .         .         .         .         .         .
 50115   16I         .         .         .         .         .         .         .         .         .         .         .         .
 50120   17I         .         .         .         .         .         .         .         .         .         .         .         .
 50125   18I         .         .         .         .         .         .         .         .         .         .         .         .
 50130   19I         .         .         .         .         .         .         .         .         .         .         .         .
 50135   20I         .         .         .         .         .         .         .         .         .         .         .         .
 50140   21I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50145   22I         .         .         .         .         .         .         .         .         .         .         .         .
 50150   23I         .         .         .         .         .         .         .         .         .         .         .         .
 50155   24I         .         .         .         .         .         .         .         .         .         .         .         .
 50200   25I         .         .         .         .         .         .         .         .         .         .         .         .
 50205   26I         .         .         .         .         .         .         .         .         .         .         .         .
 50210   27I         .         .         .         .         .         .         .         .         .         .         .         .
 50215   28I         .         .         .         .         .         .         .         .         .         .         .         .
 50220   29I         .         .         .         .         .         .         .         .         .         .         .         .
 50225   30I         .         .         .         .         .         .         .         .         .         .         .         .
 50230   31I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50235   32I         .         .         .         .         .         .         .         .         .         .         .         .
 50240   33I         .         .         .         .         .         .         .         .         .         .         .         .
 50245   34I         .         .         .         .         .         .         .         .         .         .         .         .
  -0250   35I         .         .         .         .         .         .         .         .         .         .         .         .
   0255   36OI        .         .         .         .         .         .         .         .         .         .         .         .
 50300   37OI        .         .         .         .         .         .         .         .         .         .         .         .
 50305   38.I        .         .         .         .         .         .         .         .         .         .         .         .
 50310   39.I        .         .         .         .         .         .         .         .         .         .         .         .
 50315   40.I        .         .         .         .         .         .         .         .         .         .         .         .
 50320   41.I. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50325   42.I        .         .         .         .         .         .         .         .         .         .         .         .
 50330   43.OI       .         .         .         .         .         .         .         .         .         .         .         .
 50335   44.  O I    .         .         .         .         .         .         .         .         .         .         .         .
 50340   45.     O   I         .         .         .         .         .         .         .         .         .         .         .
 50345   46.         .  O  I   .         .         .         .         .         .         .         .         .         .         .
 50350   47.         .        O. I       .         .         .         .         .         .         .         .         .         .
 50355   48.         .         .      O  I         .         .         .         .         .         .         .         .         .
 50400   49.         .         .         .      O I.         .         .         .         .         .         .         .         .
 50405   50.         .         .         .         .   O I   .         .         .         .         .         .         .         .
 50410   51. . . . . . . . . . . . . . . . . . . . . . .I. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50415   52.         .         .         .        IO         .         .         .         .         .         .         .         .
 50420   53.         .         .         . I O     .         .         .         .         .         .         .         .         .
 50425   54.         .         .    I O  .         .         .         .         .         .         .         .         .         .
 50430   55.         .       I .O        .         .         .         .         .         .         .         .         .         .
 50435   56.         .   IO    .         .         .         .         .         .         .         .         .         .         .
 50440   57.         I O       .         .         .         .         .         .         .         .         .         .         .
 50445   58.      I O.         .         .         .         .         .         .         .         .         .         .         .
 50450   59.     IO  .         .         .         .         .         .         .         .         .         .         .         .
 50455   60.   IO    .         .         .         .         .         .         .         .         .         .         .         .
 50500   61. .IO . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50505   62. IO      .         .         .         .         .         .         .         .         .         .         .         .
 50510   63. I       .         .         .         .         .         .         .         .         .         .         .         .
 50515   64. I       .         .         .         .         .         .         .         .         .         .         .         .
 50520   65.IO       .         .         .         .         .         .         .         .         .         .         .         .
 50525   66.I        .         .         .         .         .         .         .         .         .         .         .         .
 50530   67.I        .         .         .         .         .         .         .         .         .         .         .         .
 50535   68.I        .         .         .         .         .         .         .         .         .         .         .         .
 50540   69IO        .         .         .         .         .         .         .         .         .         .         .         .
  0545   70IO        .         .         .         .         .         .         .         .         .         .         .         .
 50550   71I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 50555   72I         .         .         .         .         .         .         .         .         .         .         .         .
 50600   73I         .         .         .         .         .         .         .         .         .         .         .         .
 50605   74I         .         .         .         .         .         .         .         .         .         .         .         .
 50610   75I---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.---------.
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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	553.	538.	538.	2.17		
ROUTED TO	ROUTE	4510.	4.17	552.	537.	537.	2.17		

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

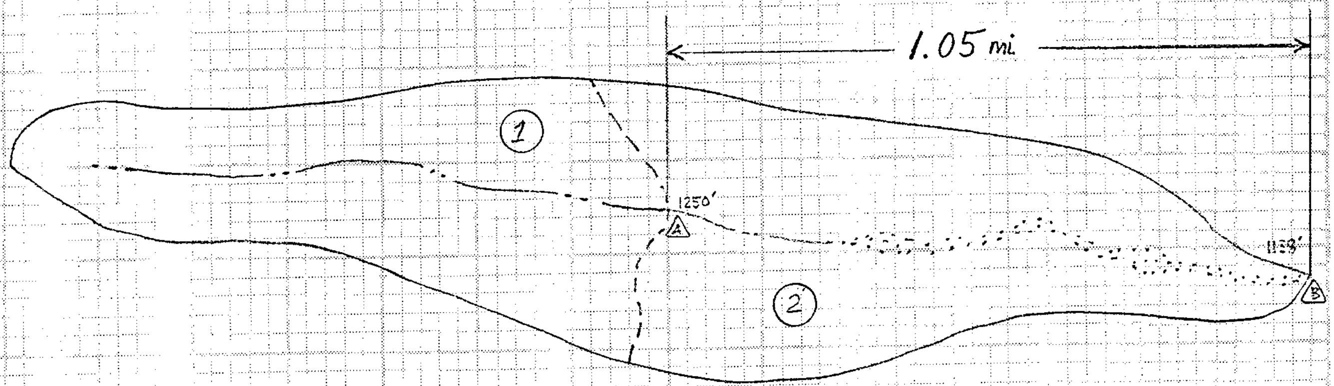
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT MARICOPA COUNTY HYDROLOGY MANUAL PAGE 1 OF 2

DETAIL EXAMPLE # 11 COMPUTED ______ DATE ______

CHECKED BY ______ DATE ______

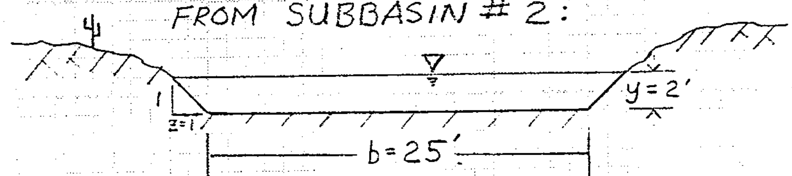
MUSKINGUM ROUTING



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:

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

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

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

$$S = .0170\ ft/ft$$

$$n = .040$$

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

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT ______

DETAIL ______ COMPUTED ______ DATE ______

______ CHECKED BY ______ DATE ______

CALCULATE K :

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

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 :

$$\frac{1}{2(1-X)} \le \frac{(AMSKK \times 60)}{(NMIN \times NSTPS)} \le \frac{1}{2X}$$

TRY NSTPS = 1 :

$$\frac{1}{2(1-.2)} \le \frac{.259(60)}{(5)(1)} \le \frac{1}{2(.2)}$$

$$.625 \le 3.11 \le 2.5 \longrightarrow \text{No Good !}$$

TRY NSTPS = 2 :

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

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 (FTS) 448-3285   *
*                                      *
****************************************
```

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

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

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE. THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THE VERSION RELEASED 31JAN85 CONTAINS NEW OPTIONS ON RL AND BA RECORDS, AND ADDS THE HL RECORD. SEE JANUARY 1985 INPUT DESCRIPTION FOR NEW DEFINITIONS.

SAMPLE RUN

EXAMPLE # 11

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

*** FREE ***
ID       SAMPLE HEC-1 RUN USING TECHNIQUES OUTLINED IN THE
ID       MARICOPA COUNTY HYDROLOGY MANUAL
* *********************************************************************
ID       EXAMPLE #11 - MUSKINGUM ROUTING
* *********************************************************************
IT       5 09SEP89    0000      45
IO       3
* *********************************************************************

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

KK   ROUTE
KM   ROUTE THROUGH DOWNSTREAM BASIN USING MUSKINGUM ROUTING
KO       1       2
RM       2    .259      .2
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

                                                   ***
```

UNIT HYDROGRAPH PARAMETERS

CLARK TC= .32 HR, R= .25 HR

SNYDER TP= .30 HR, CP= .78

UNIT HYDROGRAPH

19 END-OF-PERIOD ORDINATES

201.	833.	2960.	4438.	3626.	2604.	1870.	1343.	964.	693.
497.	357.	257.	184.	132.	95.	68.	49.	35.	

*** *** *** *** ***

HYDROGRAPH AT STATION INFLOW

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

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
(CFS)	(HR)		6-HR	24-HR	72-HR	3.67-HR
5761	1.42		(C 879.	879.	879.	879.
		(INCHES)	1.817	1.817	1.817	1.817
		(AC-FT)	267.	267.	267.	267.

CUMULATIVE AREA = 2.75 SQ MI

*** ***

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          **************
          *            *
18 KK     *   ROUTE    *
          *            *
          **************
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ROUTE THROUGH DOWNSTREAM BASIN USING MUSKINGUM ROUTING

20 KO OUTPUT CONTROL VARIABLES

IPRNT	1	PRINT CONTROL
IPLOT	2	PLOT CONTROL
QSCAL	0.	HYDROGRAPH PLOT SCALE

HYDROGRAPH ROUTING DATA

21 RM MUSKINGUM ROUTING

NSTPS	2	NUMBER OF SUBREACHES
AMSKK	.26	MUSKINGUM K
X	.20	MUSKINGUM X

****

HYDROGRAPH AT STATION ROUTE

****

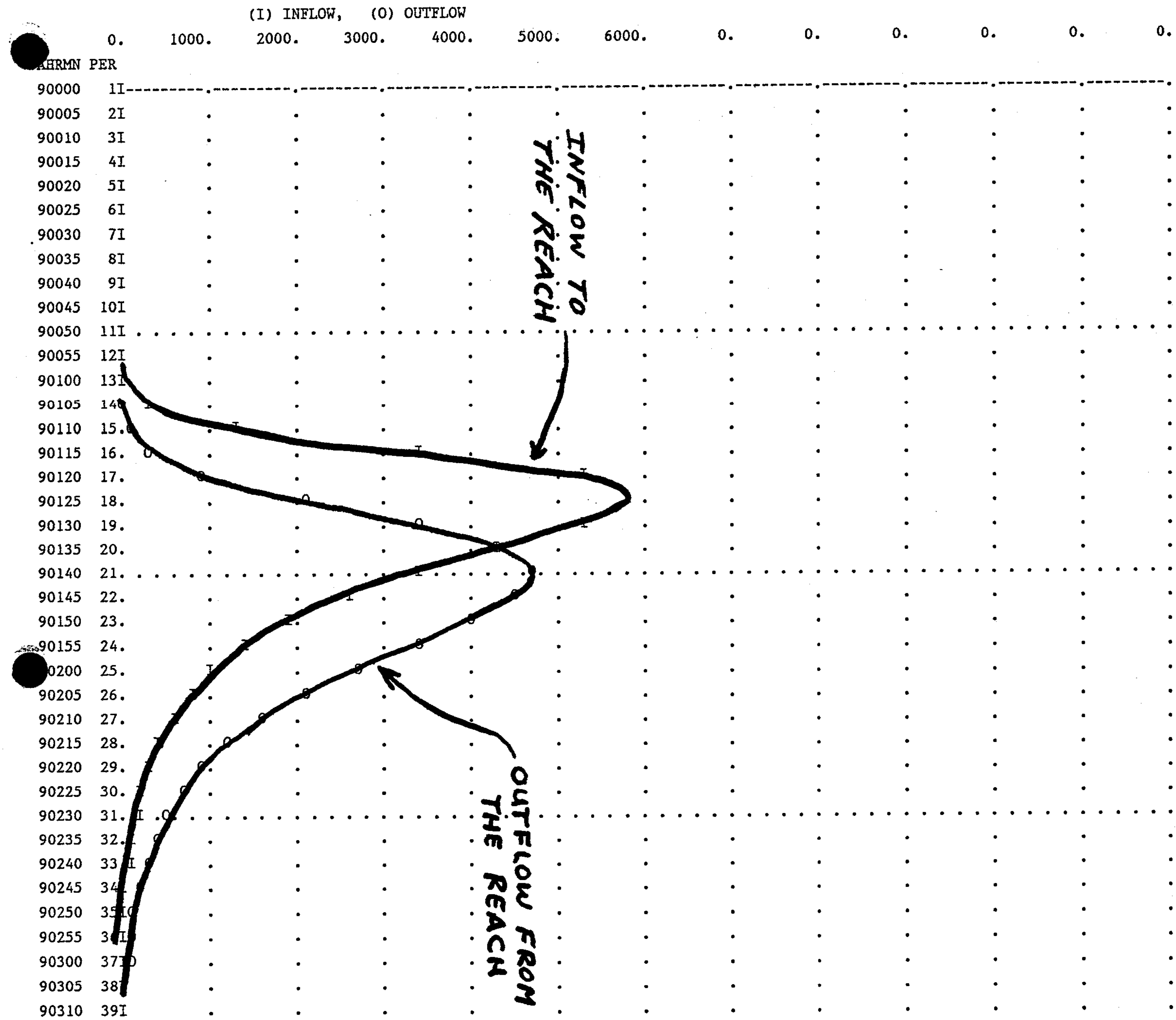
DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW
9 SEP 0000	1	0.	9 SEP 0100	13	0.	9 SEP 0200	25	2708.	9 SEP 0300	37	57.
9 SEP 0005	2	0.	9 SEP 0105	14	8.	9 SEP 0205	26	2114.	9 SEP 0305	38	35.
9 SEP 0010	3	0.	9 SEP 0110	15	62.	9 SEP 0210	27	1619.	9 SEP 0310	39	21.
9 SEP 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	TIME		MAXIMUM AVERAGE FLOW			
(CFS)	(HR)		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



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 ***