Documentation/ Verification Report for the Drainage Design Manual for Maricopa County, Arizona Volume I Hydrology

George V. Sabol[1]
Joe M. Rumann[2]
Davar Khalili[2]
Steve D. Waters[2]

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[1]George V. Sabol Consulting Engineers, Inc., Denver, Colorado and Phoenix, Arizona

[2]Special Projects Branch, Hydrology Division, Flood Control District of Maricopa County, Phoenix, Arizona

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Introduction

Background

Prior to 1985, the local governmental agencies of Maricopa County, Arizona, desired to establish a common basis for storm drainage management within the County. In 1985, a multi-jurisdictional task force of individuals representing local and county government agencies, a homebuilders association, an irrigation district, and consulting engineers, was formed under the auspices of the Flood Control District of Maricopa County (District). This task force determined that the effort proceed in three phases:

Phase 1: Research, evaluate, develop and produce uniform policies and standards for drainage of new development within Maricopa County.

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.

Phase 1 resulted in the *Uniform Drainage Policies and Standards for Maricopa County, Arizona*, February 1987. Phase 2 was executed in two parts: Phase 2A commenced in 1987 and resulted in the publication of the *Drainage Design Manual for Maricopa County, Arizona, Volume I, Hydrology* in 1990, and Phase 2B commenced in 1988 to produce *Volume II, Hydraulics*. Phase 3 will use the results from a regional reanalysis of rainfall data that is presently being conducted by the Hydrometeorology Branch, Office of Hydrology, National Oceanic and Atmospheric Administration (NOAA), as an interstate study funded by federal, state, and local agencies.

Description of the Manual

The flood hydrology procedures are described in the *Drainage Design Manual for Maricopa County, Arizona, Volume I, Hydrology* (hereinafter referred to as the *Hydrology Manual*) prepared by the Special Projects Branch, Hydrology Division, Flood

Control District of Maricopa County and George V. Sabol Consulting Engineers, Inc. The manual was issued for review and use on September 1, 1990. Since that date, several public meetings have been conducted to introduce the *Hydrology Manual* to jurisdictional drainage and flood control agencies and to consulting engineers. Since then, the manual has undergone some relatively minor changes to correct errors and deficiencies, and to enhance the application of the procedures. Revised editions to the manual will be issued as needed.

Extensive documentation on the development of the *Hydrology Manual* has been produced and this is contained in a *Documentation Manual*, Parts 1 through 8. The *Documentation Manual* is quite extensive and is maintained in the office of the District where it can be reviewed.

The procedures in the *Hydrology Manual* have been tested throughout its development and additional testing and verification continues. Certain tests and verification results are presented in the appendices to the *Hydrology Manual*, and all test and verification results are contained in the *Documentation Manual*.

Purpose of this Report

The purpose of this *Documentation/Verification Report* is twofold. First, it is to provide a concise and readily available source for a presentation and discussion of the development and technical justification for the procedures in the *Hydrology Manual*. As such, it serves as a link between the *Hydrology Manual*—which mainly presents the procedures—and the *Documentation Manual* which mainly consists of technical appendices of sources of information and analyses with a minimum of discussion.

Second, this report presents a summary of the testing and verification that was conducted to assure that the procedures have a reasonable degree of hydrologic accuracy. The testing and verification was performed to answer two basic questions:

- Can the procedures be used to reasonably reproduce runoff events from observed rainfall (event simulation)? and
- Do the procedures result in reasonable representation of flood frequency relations for gaged watersheds (frequency simulation)?

This report can be used to determine the technical basis for the *Hydrology Manual* and to judge its merit in providing a procedure for performing rainfall/runoff analyses for the purpose of estimating flood magnitude-frequency relations.

Criteria

Three criteria were used in the development of the Hydrology Manual: *accuracy*, *practicality*, and *reproducibility*. These criteria were applied in the initial screening of methods considered for adoption in the manual, in the development of procedures to implement selected methods, and in the evaluation of methods and procedures that were compared in the process of development and testing.

Accuracy is a measure of how well the results of the procedures in the manual reproduce or measure physical reality. Accuracy was considered in terms of both individual procedures, such as the accuracy of the time of concentration estimator, and overall accuracy in estimating peak discharges and runoff volumes. Accuracy is highly desired, however, absolute accuracy in hydrology is impossible to achieve or to measure, and relative accuracy is difficult to evaluate because of the lack of an adequate database. Where possible, the relative accuracy of the procedures were measured by comparison of results to available data, and through the testing and verification of the procedures against instrumented watershed data.

Practicality is a user's decision; in this case the decision was made by the authors. Practicality is a judgement of the best and most appropriate level of technology to apply. Consideration was given to input requirements, data and information that are available to estimate input, output requirements, technical qualifications of the intended user of the manual, economic cost (time requirement) to the user, expected benefits (analysis refinements), and consequences of error that may be inherent in the procedure. Practicality often came down to a decision between simpler, more easily understood, and less data-intensive methods versus sophisticated, less frequently used, and more data-intensive methods. This often resulted in a compromise between the two extremes, with the best practical level of technology recommended in the manual—considering the state of current hydrologic knowledge of arid and semi-arid lands.

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

Selection of Model Type

The HEC-1 Flood Hydrograph Program (U.S. Army Corps of Engineers, 1990) was selected as the computerized program to perform rainfall-runoff modeling. This selection was based on a consideration of the three criteria that were applied to the development of the procedures in the *Hydrology Manual*: accuracy, practicality and reproducibility.

Numerous uses of the HEC-1 program are reported in the professional literature and various project documents to indicate that watersheds can be modeled using the HEC-1 program to reproduce actual recorded flood hydrographs or to simulate flood frequency relations, thereby satisfying the requirement for accuracy.

The HEC-1 program has a long history of usage within Arizona and elsewhere to indicate that a broad segment of the anticipated users are familiar with the use of HEC-1. Furthermore, formal training is readily available in HEC-1 modeling. For these reasons and through the selection of the numerous modeling options in the HEC-1 program, the model was judged to be practical for use in Maricopa County.

Reproducibility in the use of the HEC-1 program was achieved by producing a manual that provides guidance in the preparation of input to the HEC-1 program with a minimum of subjective decisions. Several other considerations were made in the selection of the HEC-1 program:

- The program was produced and is used by a Federal agency which provides an implied authenticity and state-of-the-practice stature.

- The program has a tradition of support and technical improvements to provide a level of assurance that it will remain a viable and an up-to-date analytic procedure.

- The input and the output from the program are structured such that it is relatively universal in its interpretation (i.e., the input and output files can be easily used and interpreted by users other than the individual that prepared the input file). This enhances the "shelf life" of the model and makes it viable for future updating and multi-project uses.

- The HEC-1 program has numerous options for watershed modeling that makes it attractive to a wide range of applications for a diversity of watershed types.

This report provides documentation and verification results of the rainfall-runoff modeling procedure that is contained in the *Hydrology Manual.* The report is presented in the following parts:

1. Hydrologic Setting and Design Rainfall Criteria
2. Rainfall Losses
3. Unit Hydrographs
4. Verification

1

Hydrologic Setting and Design Rainfall Criteria

1.1 Description of Maricopa County

At 9,226 square miles, Maricopa County, Arizona, is approximately the same size as the state of New Hampshire. The county lies in the Gila River basin, a tributary of the Colorado River, and the area comprises a wide diversity of physiographic and topographic conditions. Approximately 30 percent of the area is mountainous and the remaining 70 percent is valley. The mountain areas above 3,000 feet in elevation are characterized by rugged terrain and steep slopes. The valleys consist of inactive and potentially active alluvial fans and coalesced fans, nearly flat basin floors, and alluvial floodplains. Much of the area is agricultural land that was leveled for irrigation applications. The Phoenix metropolitan area is in Maricopa County and urbanization has—and will continue to have—a major impact on the runoff potential in the developing areas of the County.

Vegetation varies according to physiographic and climatic factors. In general, the vegetation is sparse and cacti grow throughout the area. The valley basin has sparse grass and shrub cover in its natural condition, although much of the area in the Phoenix metropolitan area is irrigated turf (particularly large areas of golf courses). Hillslopes are populated by cacti and shrubs, and the higher mountains have stands of trees with underbrush.

The diverse physiographic, topographic, and land-use conditions within Maricopa County requires a flood estimation procedure for all conditions. That is, procedures must be available for major watercourses with large drainage areas, mountain watersheds, small urban watersheds, natural and lightly urbanizing hillslopes, leveled agricultural land, alluvial fans (active and inactive), and Sonoran and Mohave desert. To compound the situation, very little research or data are available for these conditions.

1.2 Rainfall Characteristics for Maricopa County

Rainfall characteristics for Maricopa County must be considered in the context of larger meteorologic systems. Therefore, the rainfall characteristics for Arizona, as discussed in the following, also generally apply to Maricopa County. Where specific information exists and where appropriate, rainfall characteristics for Maricopa County that differ from those for Arizona are so indicated.

Two major factors determine the occurrence and magnitude of precipitation in Arizona: source of atmospheric moisture and cooling mechanisms. Some of the characteristics of precipitation are associated with the source of the atmospheric moisture and this is somewhat seasonal. Atmospheric moisture enters Arizona either from the Gulf of Mexico (southeast moisture), or from the Pacific Ocean and the Gulf of California (southwest moisture). Southwest moisture is more common in Arizona and represents the condition for the majority of precipitation events. However, southeast moisture represents a more complex phenomena and this results in the so-called monsoon in late summer in Arizona (Osborn and others, 1980b).

Cooling mechanisms for atmospheric moisture in Arizona are due to frontal activity, orographic uplift, convective currents, or any combination of these. Frontal activity occurs with the onset of major weather systems into Arizona and this may result in large areal rainfall. Convective cooling will usually result in local precipitation. Orographic uplift can operate independently or in conjunction with either of the other two conditions.

Arizona ranges in elevation from about 130 feet near Yuma to about 13,000 feet near Flagstaff, providing a wide range of climatic conditions in the state. In Maricopa County, the elevation ranges from less than 1,000 feet near Gila Bend to nearly 8,000 feet in the mountains both to the north and to the east of Phoenix. In addition, there are numerous mountains, mountain ranges, and erosional scarps (rims) throughout Arizona and Maricopa County. Both the overall elevation change from generally southwest to northeast and the numerous orographic features that are distributed throughout the State provide ample opportunities for orographic uplift of moist air.

Using available recording raingage data from the U.S. Department of Agriculture (USDA) experimental watersheds and the National Weather Service (NWS) raingage network in Arizona, Osborn and Davis (1977) identified two types of precipitation-producing systems in Arizona: frontal and airmass. Frontal activity is more likely in northern Arizona than in southern Arizona (Osborn and others, 1980b), and both systems operate in Maricopa County.

Winter precipitation is usually a function of southwest moisture with frontal cooling aided by orographic uplift, although convective cells can be generated along the front that can result in some localized heavy rainfall within the general storm. These types of rainfalls are usually of large areal extent, long duration, and low to medium intensity rainfall. The characteristic summer storms in Arizona are usually airmass thunderstorms that can be augmented by orographic influences. These summer storms are of limited areal extent, short duration, and usually short periods of high intensity rainfall (Sellers, 1960).

Winter storms may cause flooding, especially in urbanized areas with greater impervious surface area, however summer storms present the most severe flash flood mechanism for areas smaller than 100 square miles (Osborn and Hickok, 1968). Records from USDA experimental watersheds in Arizona and New Mexico show that over 95 percent of the surface runoff from undeveloped watersheds results from summer convective rainfalls (thunderstorms) (Osborn, 1983). Thunderstorms can occur at any time of the year in Arizona, but are predominant in the summer months of May through September and are more frequent in the months of July through September. They are most likely to occur in the late afternoon and early evening. These summer thunderstorms are either frontal-convective or airmass, but almost all thunderstorms in Arizona are airmass type (Osborn, 1983).

The U.S. Army Corps of Engineers (1974 and 1982a) studied historic storms and flooding in the Maricopa County area. The Corps classified rainfall in this area of Arizona as general winter storms, general summer storms, and summer thunderstorms.

General winter storms normally begin as extratropical cyclones, move inland from the Pacific Ocean spreading light to moderate precipitation over large areas, and last from one-half day to several days. General winter storms are usually most prevalent and most intense during the months of December through March, although they can occur any time from October through May. This type of storm is characterized most typically by cool, stable air masses with widespread cloudiness and steady, light rainfall or snow. A few locally-heavy showers and occasional isolated thunderstorms may occur. Winter precipitation in central Arizona normally occurs as the result of the influx of moisture from the southwest, low-level convergence, and rising motion caused by a general circulation to the west. Cold fronts, or orographic uplift, work to trigger the instability. The relatively low intensities of rainfall, the large areal extent and the relatively long durations of this type of storm normally do not produce severe flooding conditions for small watersheds, but may produce flooding in major rivers due to a large volume of sustained runoff.

General summer storms usually consist of storms of a convergence, frontal, and/or orographic nature, with moderate to heavy thunderstorms, often embedded along frontal lines. General summer storms occur primarily during the months of July, August and September, although it is possible for this type of storm to occur any time from May through October. Rainfall normally consists of a mixture of general steady rain and numerous convective showers with locally-heavy precipitation associated with convective activity. The convective storm cells usually account for the bulk of a general summer storm's total rainfall. The major meteorological prerequisite for general summer storms is a strong, deep flow of tropical moisture from either the southwest or southeast. This flow of tropical moisture is often enhanced considerably by the presence of one or more tropical storms or hurricanes off the west and/or east coasts of northern or central Mexico, or by the remnants of such a tropical cyclone which moves across land and directly invades the central Arizona area. The triggering mechanisms for the release of this tropical moisture include solar heating, orographic uplift, the movement into the region of a cold front or upper level closed low from the north or northwest, and other mechanisms causing general low-level convergence and/or instability.

Thunderstorms, called local storms, can occur in Maricopa County at any time of the year, but the most common and the most significant local storms occur during the summer months, usually between late June and late September. These local storms result from the influx of tropical moisture originating in the southwest and, on occasion, the southeast, and are normally triggered by the instability resulting from the intense solar heating of the ground surface which occurs at this time of year. Conditions favoring these airmass thunderstorms may be further intensified by meteorological factors. Local storms are normally scattered or isolated phenomena, and are more than twice as common over the higher mountain peaks as they are in the desert valleys. Local storms can produce severe flash floods over small drainage basins, resulting in serious local damage and sometimes loss of life.

1.3 Design Rainfall Criteria

Design rainfall criteria for Maricopa County was developed or adopted to define the following:

1. Point rainfall depth-duration-frequency statistics.
2. Reduction of point rainfall for increasing storm area.
3. Temporal distribution of design storms.

The critical flood-producing storm for Maricopa County is the summer local storm for most watersheds. The limit of such storms is generally less than 500 square miles and of duration less than 6 hours. Local storms are characterized by central storm cells that produce very high intensity rainfalls for relatively short durations. The rainfall intensities diminish as the distance from the storm cell increases. Therefore, for the majority of watersheds and drainage areas in Maricopa County, the local storm will produce both the largest flood peak discharge and the greatest runoff volume. Based on a review of meteorologic studies for Arizona and an inspection of severe storm records for Maricopa County, it was determined that the local storm is the design storm for all watersheds in Maricopa County for drainage areas of 100 square miles and less.

Record floods for large drainage areas, such as for the Salt River near Phoenix, were produced by large scale general storms of multiple day duration and relatively low rainfall intensities. Therefore, based on that observation, for drainage areas larger than 500 square miles the critical flood-producing storm may be the general storm, although a severe local storm over only part of the watershed may result in the floods of record for such watersheds. Because of the infrequent need for design criteria for such large areas, design rainfall criteria for general storms are not defined in the Manual, and such criteria are to be defined for such large, regional flood studies on a case-by-case basis so that the most appropriate meteorologic and hydrologic factors (possibly also including snowmelt as a baseflow condition and to set watershed antecedent moisture conditions) can be properly considered.

For drainage areas between the critical flood-producing limit for local storms (100 square miles), and the lower limit for general storms (500 square miles), it can not be determined whether a local storm or a general storm will produce the greatest flood peak discharges or the maximum flood volumes. For such drainage areas, generally between 100 and 500 square miles, it will be necessary to consider both general storms and local storms. This may require that site-specific general storm criteria be developed for the watershed and, that various local storms with critical storm centering and partial area rainfall assumptions be developed using the criteria in the *Hydrology Manual*. Both of these storm types would be modeled and executed in the watershed model to estimate flood discharges and runoff volumes. It is possible, in certain situations, that the local storm could result in the largest peak discharge and the general storm could result in the largest runoff volume.

A summary of design rainfall requirements for use in Maricopa County is shown in Table 1-1.

Table 1-1
Design rainfall requirements for Maricopa County

Area, square miles	Design Rainfall Criteria
0 to 100	6-hour local storm as defined per the *Drainage Design Manual for Maricopa County, Arizona, Volume I, Hydrology*.
Greater than 500	General storm determined on a case-by-case basis considering appropriate meteorologic and hydrologic factors, and possibly the critically centered and/or partial area 6-hour local storm.
100 to 500	Both the critically centered and/or partial area 6-hour local storm, and the general storm as determined on a case-by-case basis.

1.4 Point Rainfall Depth-Duration-Frequency Statistics

Phase 3 of the development of storm drainage criteria for Maricopa County includes a reanalysis of rainfall data and a redefinition of rainfall isopluvial maps for the County. That phase is presently underway by NOAA, however results will not be available for several years. There are numerous examples of rainfall frequency analyses that indicate that the rainfall depth-duration-frequency statistics from the NOAA Atlas 2 (Miller and others, 1973a) may underestimate the actual statistics. Osborn and Renard (1988) recently performed a rainfall frequency analysis of data from Walnut Gulch for durations of 1-hour and less and return periods of 2- to 100-year. They compared the point rainfall depth-duration-frequency statistics from those that would be obtained from NOAA Atlas 2. At a 2-year return period the results from the two sources are almost identical. The results deviate with longer return periods with the rainfall depths for the Walnut Gulch study equalling or exceeding those from NOAA Atlas 2 for all return periods greater than 2-year. For example, the 100-year, 1-hour rainfalls are 1.89 inches and 2.50 inches (a 32 percent increase) from NOAA Atlas 2 and the Walnut Gulch study, respectively.

The Corps performed a rainfall frequency analysis of data from five recording raingages in and around Las Vegas, Nevada (U.S. Army Corps of Engineers, 1988b). The results from the Corps study were compared to NOAA Atlas 2 and results of a previous analysis by the Federal Emergency Management Agency (FEMA). The comparison indicated significant differences between the Corps's results and those derived from NOAA Atlas 2, particularly for 2- through 6-hour durations. The Corps concluded that "the regional smoothing done in developing the NOAA Atlas 2 isopluvials may be too gross in the Las Vegas area, particularly for use of design storms for flood control studies." As a result of the Corps's study, and other similar studies, a depth-duration-frequency table for use in Clark County, Nevada was prepared. Those rainfall depths are greater than depths from NOAA Atlas 2. For example, the 100-year, 6-hour rainfall was increased from 1.94 inches of NOAA Atlas 2 to 2.79 inches (a 44 percent increase).

Results similar to Walnut Gulch, Arizona, and Clark County, Nevada, have been reported for other locations in the western United States. However, until the results of the present NOAA study are available, the existing NOAA Atlas 2 for Arizona (Miller and others, 1973a) will be used to define rainfall depth-duration-frequency statistics for Maricopa County.

The only deviation from the procedures in NOAA Atlas 2 is the use of short duration (less than 1-hour) rainfall relations from Arkell and Richards (1986). The short-duration rainfall ratios for Maricopa County from Arkell and Richards are shown in Table 1-2.

A computer program, PREFRE (U.S. Bureau of Reclamation, 1988), is available to generate rainfall depth-duration-frequency statistics based on the point rainfall statistics in NOAA Atlas 2 and the short-duration rainfall ratios by Arkell and Richards. The use of that program to develop depth-duration-frequency tables for use in Maricopa County is encouraged to minimize errors in performing these calculations.

Table 1-2
Short-duration rainfall ratios for Maricopa County and comparison with the NOAA Atlas 2 ratios

Return Period, years	Rainfall Duration, in minutes			
	5	10	15	30
	Arkell and Richards (1986)			
2	0.34	0.51	0.62	0.82
100	0.30	0.46	0.59	0.80
	NOAA Atlas 2 (Miller and others, 1973)			
All	0.29	0.45	0.57	0.79

1.5 Rainfall Time Distributions

There have been many time distributions that have been developed and used to describe design rainfalls in the United States and Arizona. Notably among these are the Type I and Type II distributions of the U.S. Department of Agriculture, Soil Conservation Service (SCS). These are 24-hour distributions that have been developed for use in large geographic regions of the United States. These distributions are based on generalized rainfall depth-duration relations obtained from Weather Bureau technical papers and were not developed specifically for Arizona. Type I represents regions with a maritime climate. Type II represents regions in which the high rates of runoff from small areas are usually generated from summer thunderstorms. These distributions are described in SCS Technical Paper 149 (Kent, 1973).

A family of Type II-A distributions was developed by the Albuquerque, New Mexico office of the SCS in 1973 and revised to a single Type II-A distribution in 1985. This was to reflect the more intense, shorter duration rainfalls that generally occur in New Mexico rather than in many other regions of the United States. One of these Type II-A distributions was often adopted, possibly with some modifications, for use in other states. A version of a Type II-A distribution has been used in Arizona for various purposes by individuals and agencies; although such a distribution was never verified for Arizona.

The City of Phoenix adopted a 24-hour rainfall distribution in 1977 that is similar to the SCS Type II. The basis for this distribution is unknown and this distribution has been reviewed for the City of Phoenix (Tipton and Kalmbach, Inc., 1986). For the City of Phoenix rainfall distribution, the maximum rainfall intensity lasts for 1 hour (centered in the 24-hour duration of the storm), which is not characteristic of regional, severe rainfall.

The Corps of Engineers, Los Angeles District, analyzed rainfall data and developed rainfall time distributions for three flood studies in Arizona and nearby areas; Phoenix and vicinity (1974 and 1982a), Clark County, Nevada (1988b), and Imperial Valley, California (1980). These studies were performed for the purpose of developing standard project storms, but in some cases have been used to describe storms of specified frequencies.

The Corps developed time distributions for its standard project storm (local storm) for the Maricopa County area based on the 19 August 1954 Queen Creek thunderstorm, aided by precipitation intensity information for 13 heavy thunderstorms in central Arizona. Six 7-hour distributions were developed by the Corps (U.S. Army Corps of Engineers, 1974) that are identified with a pattern number. The selection of the time distribution pattern is based on the fact that areally-averaged rainfall intensity decreases with increasing drainage area. It was also assumed that rainfall intensity should increase with an increase in the depth-frequency relation; the 10-year, 6-hour precipitation statistic was used for this measure.

In 1984, the Corps of Engineers, Los Angeles District, reanalyzed the 19 August 1954 Queen Creek storm. The reanalysis resulted in a distribution of 8-hour duration.

The distribution has five pattern numbers, with the selection of the pattern number as a function of drainage area. Pattern No. 1 is for point rainfall and Pattern No. 5 is for an area of 540 square miles (personal communication, Dr. Charles Pyke, U.S. Army Corps of Engineers, Los Angeles District). This storm distribution is referred to as the Queen Creek and vicinity, 8-hour storm pattern (1984), and was never published by the Corps.

The following rainfall time distributions were considered in the development of the Maricopa County rainfall criteria (storm duration is shown in parentheses):

- SCS Type II (24-hour)
- SCS Type II-A for New Mexico (24-hour)
- SCS spillway design storm (6-hour)
- Corps of Engineers (1974), Phoenix and vicinity (7-hour)
- Corps of Engineers (1984), Queen Creek and vicinity storm (8-hour)
- Corps of Engineers (1988), Clark County, Nevada (6-hour)
- City of Phoenix (24-hour)
- Kingman, Arizona, Master Drainage Plan (3-hour)
- Clark County, Nevada, Flood Control Master Plan (3-hour)
- Hypothetical (any duration desired)

Two decisions were made that resulted in the development and adoption of the storm pattern criteria that are shown in the *Hydrology Manual*. First, the decision was made that the rainfall criteria should reflect the major flood-producing storms that are characteristic of the region. This resulted in the decision that the design rainfall criteria should be a 6-hour local storm for drainage areas less than 100 square miles. Second, the decision was made that the storm pattern should be based on regionally-observed severe storms rather than from generalized relations that were developed from rainfall data that may not be representative of storms in Maricopa County. This led to the decision to use the Corps' analysis of the 19 August 1954 Queen Creek storm as the basis for the Maricopa County 6-hour local storm distribution.

The data and analyses that were used by the Corps for the 19 August 1954 Queen Creek storm were studied. This resulted in the following modifications to the Corps' criteria in the development of the Maricopa County 6-hour local storm criteria:

- The Corps's Pattern No. 6 for drainage areas larger than 1,000 square miles was deleted.
- The Corps's Pattern No. 1 was removed and a new Pattern No. 1 was developed. The new Pattern No. 1 is the dimensionless hypothetical distribution using rainfall depth-duration statistics from NOAA Atlas 2 and Arkell and Richards (1986) for the 100-year rainfall for the Phoenix Sky Harbor International Airport location.
- Pattern No. 1 was offset by 45 minutes so that the maximum rainfall intensities occur at about 3 hours 45 minutes to agree with the maximum period of rainfall intensities for Pattern Nos. 2 through 5.

- The Corps's Pattern Nos. 2 through 5 were modified by truncating the first hour of rainfall and normalizing the remaining 6 hours of rainfall to a summation of 100 percent at 6 hours. Pattern Nos. 2 through 5 were then adjusted somewhat to generally follow the shape of the Corps's Patterns, and also to transition into the shape of the new Pattern No. 1.

The resulting Maricopa County 6-hour local storm distribution is shown in Figure 1-1.

The procedure to select a pattern number for a watershed was developed as follows: It was noted that the Corps' criteria for selecting pattern numbers resulted in a nearly linear plot of pattern number versus logarithm of drainage area when the Queen Creek 10-year, 6-hour rainfall statistic (2.36 inches) was used with the Corps' criteria. A straight line was fit to the graph with Pattern No. 5 at 500 square miles and Pattern No. 1 at 0.5 square mile. The graph to be used to select the Maricopa County Pattern No. as a function of drainage area (or storm size) is shown in Figure 1-2. Pattern No. 1 is to be used for all areas less than or equal to 0.5 square mile.

The 100-year, 2-hour storm distribution (for retention/detention) is the hypothetical distribution for a 2-hour duration. This is the peak-centered 2-hour portion of Pattern No. 1 normalized to 100 percent. The 100-year, 2-hour retention/detention storm distribution is shown in Figure 1-3.

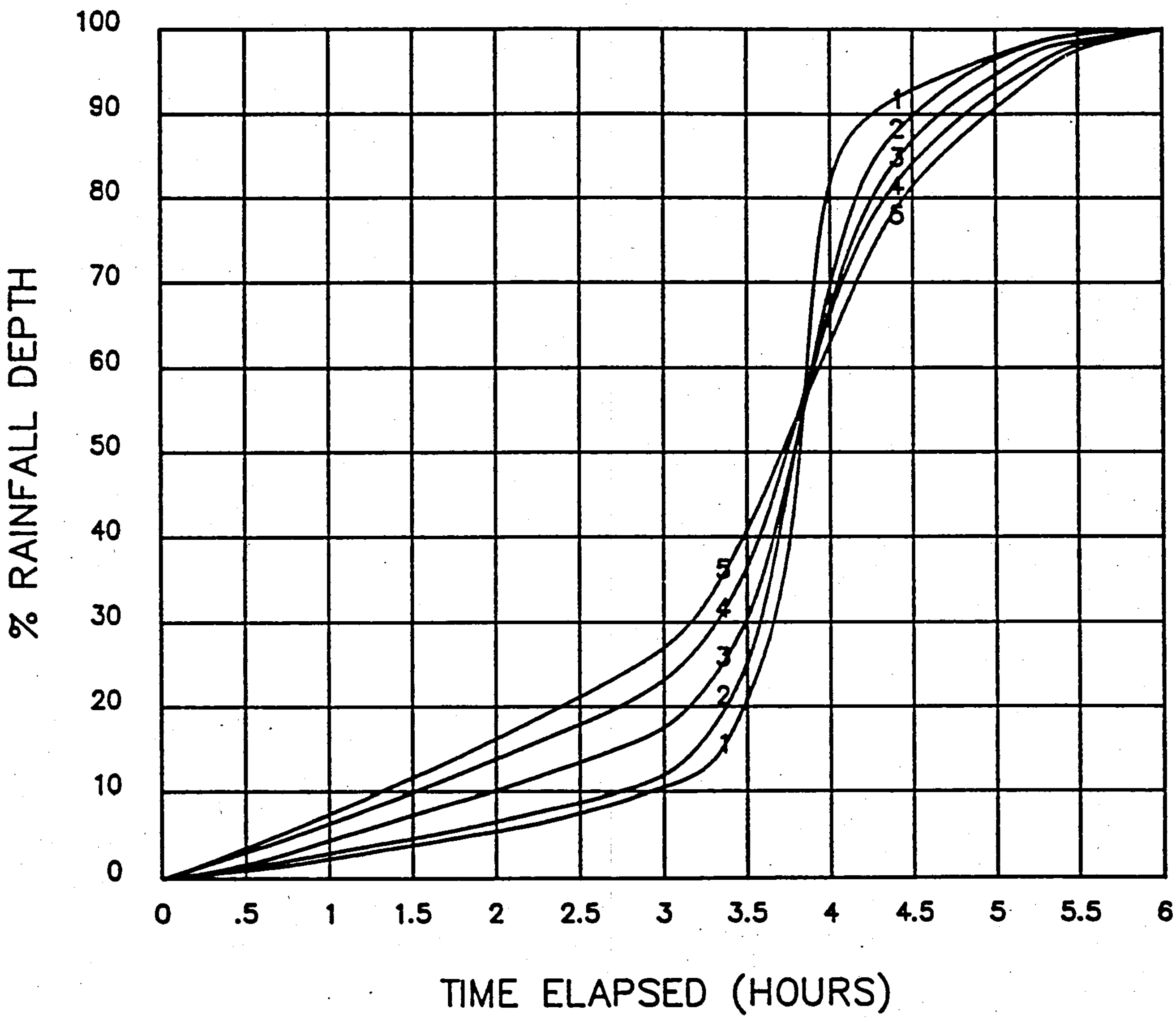


Figure 1-1
6-Hour Mass Curves for Maricopa County, Arizona

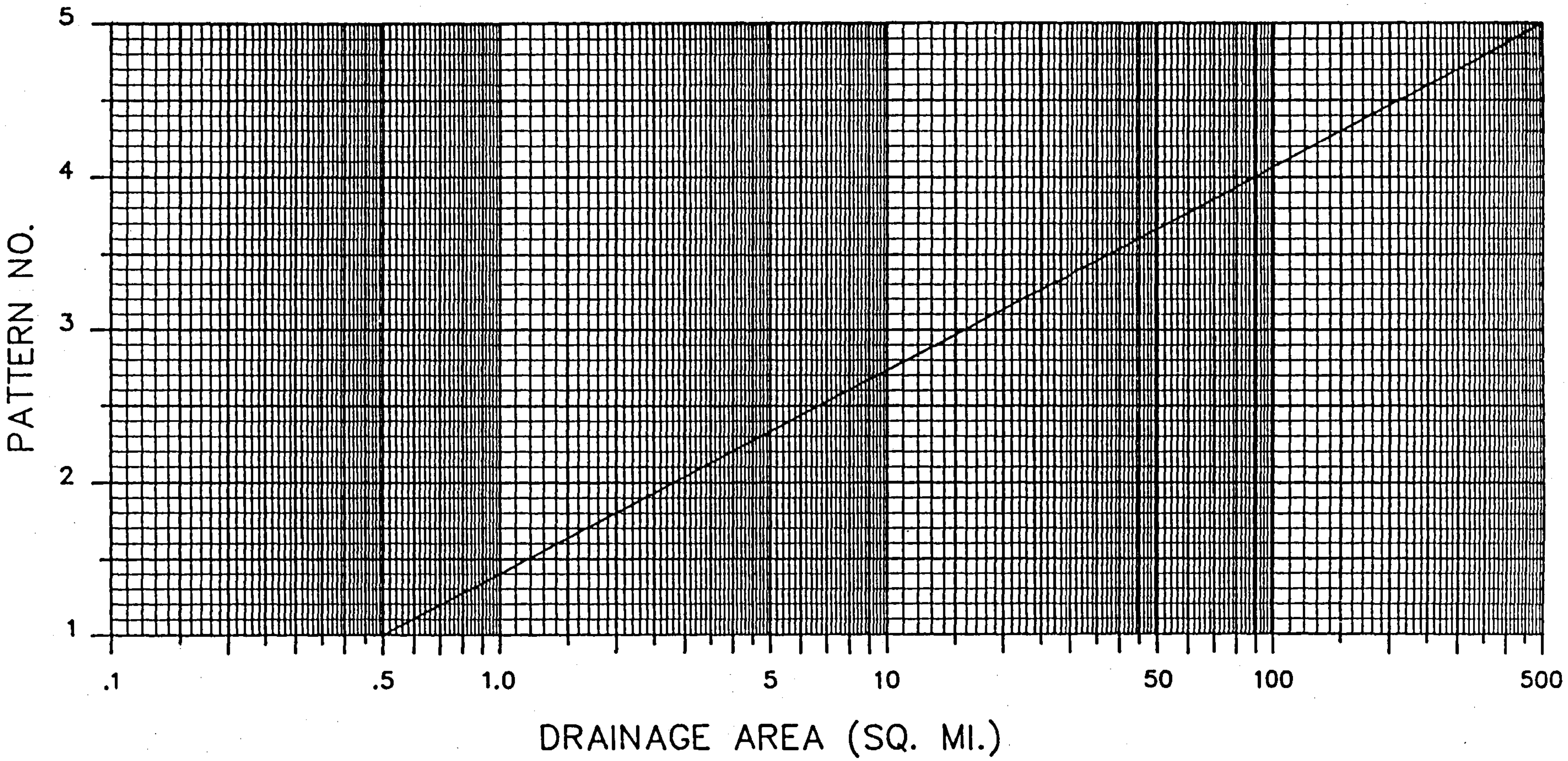


Figure 1-2
Area Versus Pattern Number for Maricopa County

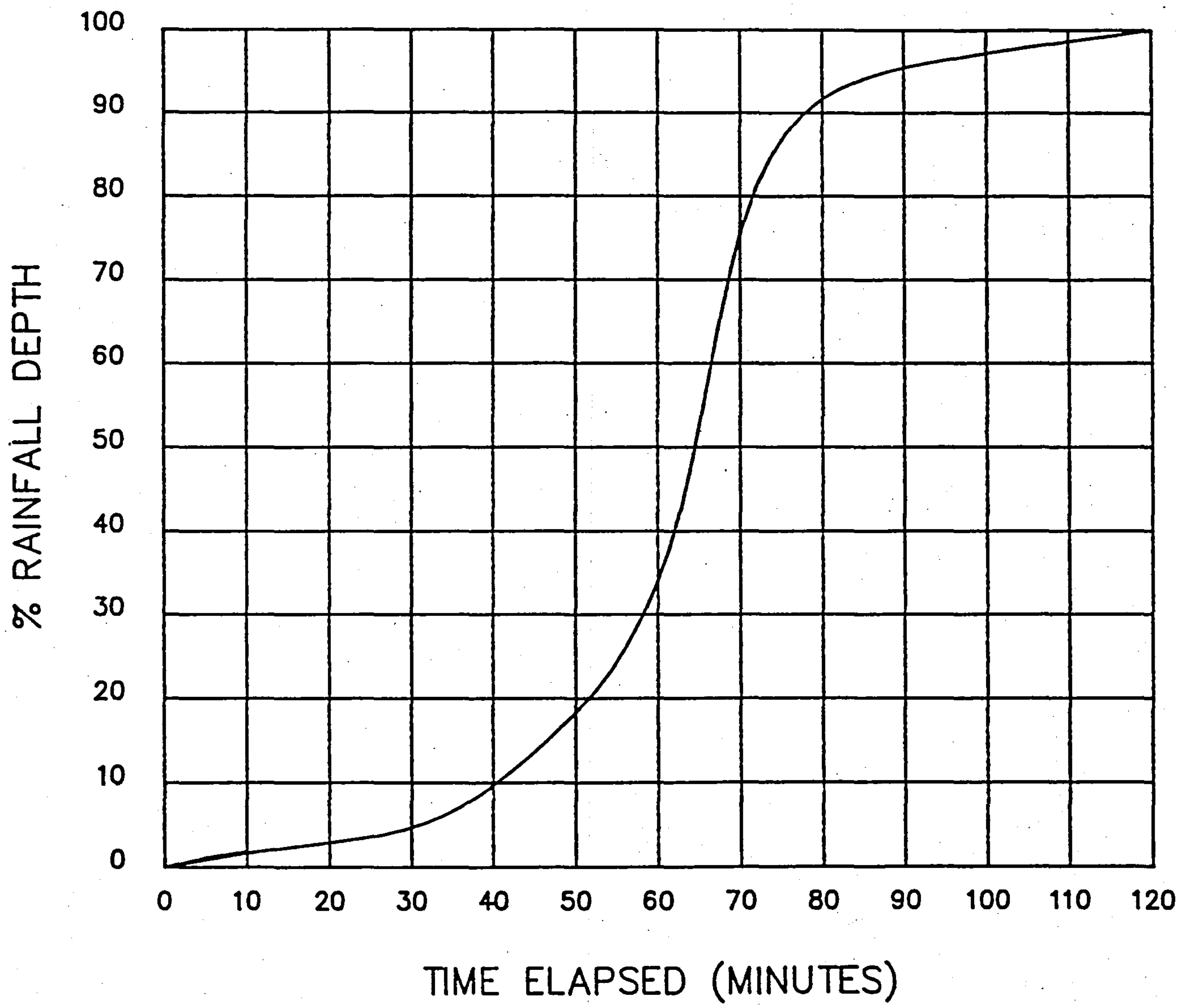


Figure 1-3
2-Hour Mass Curve for Retention Design

1.6 Depth-Area Reduction

The rainfall depths from NOAA Atlas 2 are point rainfalls for specified frequencies and durations. This is the depth of rainfall that is expected to occur at a point or points in a watershed for the specified frequency and duration. However, this depth is not the areally-averaged rainfall over the basin that would occur during a storm. A reduction factor is required to convert the point rainfall to an equivalent uniform depth of rainfall over the entire watershed. As the watershed area increases, the reduction factor decreases reflecting the greater non-homogeneity of rainfall for storms of larger area. The reduction factors are usually obtained from graphs of the reduction factor versus drainage area: a depth-area reduction curve. Four sources of depth-area reduction curves were considered for Maricopa County:

- NOAA Atlas 2 (Miller and others, 1973a)
- NWS HYDRO-40 (Zehr and Myers, 1984)
- Relations for the Walnut Gulch Experimental Watershed near Tombstone, Arizona (Osborn and others, 1980a)
- Analysis of the 19 August 1954 Queen Creek storm (U.S. Army Corps of Engineers, 1974)

The depth-area reduction curves in NOAA Atlas 2 were developed from published NWS data for groupings of closely-spaced recording raingages. No closely-spaced groupings of raingage data were available in Arizona or the Southwest for this purpose. Therefore, the depth-area reduction curves that are published in NOAA Atlas 2 for Arizona and other western states have been derived from raingage data that are outside this region. These depth-area reduction curves do not adequately represent the airmass or frontal-convective storms that produce floods in Arizona or the Southwest.

Osborn and others (1980a) used records from dense recording raingage networks, operated by the USDA, Arid Lands Watershed Management Research Unit at the Walnut Gulch Experimental Watershed near Tombstone, Arizona, and the Alamogordo Creek Experimental Watershed near Santa Rosa, New Mexico, to develop depth-area reduction curves. These curves are for rainfall durations of 30-min to 6-hour and frequencies from 2- to 100-year. The curves cover a range of area up to about 80 square miles.

The curves for Walnut Gulch lie well below the NOAA Atlas 2 curve, show more change with storm frequency, and show less change with storm duration. This is consistent with the characteristics of airmass thunderstorms that produce most of the flood events in southeastern Arizona. The authors state that the Walnut Gulch curves are believed to be characteristic of southeastern Arizona, southwestern New Mexico, and north central Mexico.

The National Weather Service derived depth-area reduction factors for use in Arizona and western New Mexico, and depth-area reduction curves are published in NWS HYDRO-40 (Zehr and Myers, 1984). That study defined the region in Arizona and New Mexico over which the Walnut Gulch depth-area reduction curves should apply, extended the Walnut Gulch curves from about 80 square miles to 500 square miles, and developed a depth-area reduction curve for Walnut Gulch for a 24-hour duration storm, a duration not included in the study by Osborn and others (1980a).

Two depth-area reduction curves are shown in NWS HYDRO-40; one labeled as southeast Arizona, and one labeled as central Arizona. A map is provided in NWS HYDRO-40 that indicates the regions in Arizona and western New Mexico where these two sets of curves apply. The curve identified as southeast Arizona is to be used in southeastern Arizona, northeastern Arizona, and all of western New Mexico. The curve identified as central Arizona is to be applied in all remaining areas of Arizona. The two sets of curves are for durations of 3-, 6-, 12-, and 24-hour. The curves were derived for the mean annual rainfall (2.54-year return period), and no return period is associated with the curves. Use of the mean curve for all return periods will lead to conservative estimates for return periods greater than the 2.54-year.

The U.S. Army Corps of Engineers (1974) performed an extensive analysis of the 19 August 1954 Queen Creek storm in the development of a standard project storm for central Arizona. A depth-area reduction curve was developed in that analysis.

Based on a comparison of the various depth-area reduction curves that are available, the Corps of Engineers depth-area reduction curve for the Queen Creek storm was accepted for use in Maricopa County (Figure 1-4). The decision to accept that curve was based on the fact that this storm is representative of the type of severe storm that is considered representative of the design storm that is expected for Maricopa County, and because the temporal distribution of the design storm is also based on this historic storm.

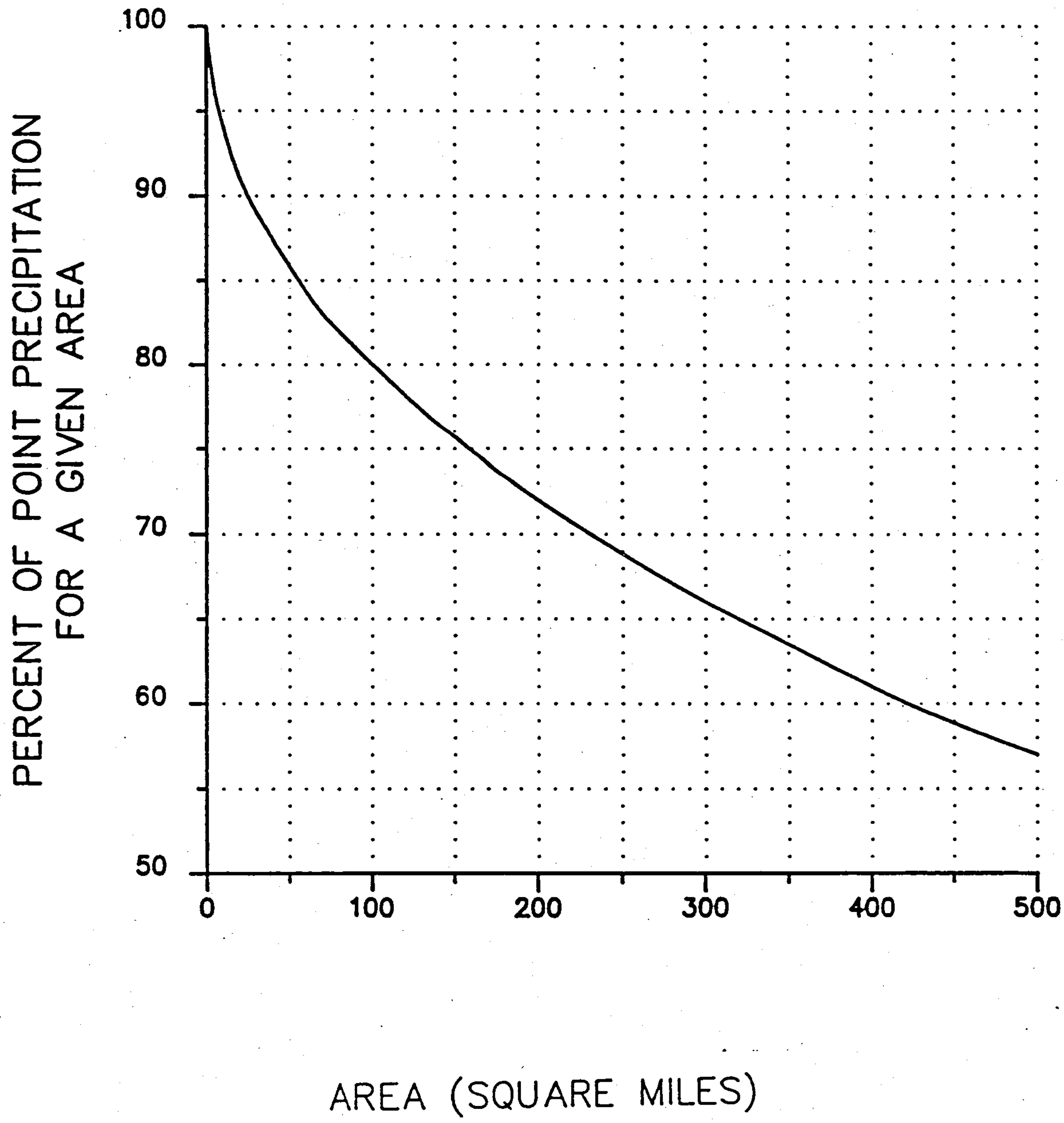


Figure 1-4
Depth-Area Curve for Maricopa County, Arizona

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

2

Maricopa County Rainfall Losses Procedures

2.1 Considerations in the Selection of Rainfall Loss Procedures

One of the major considerations throughout the development of the *Hydrology Manual* was that the individual procedures in it reflect the actual physical process. This required that it be possible to estimate—either individually or in the aggregate—losses that occur during rainfall due to infiltration, depression storage, interception, land surface evaporation, and so forth. Because the critical flood-producing storm for small areas (generally less than 100 square miles) is a short duration, high intensity, local storm, it was also important that the procedure provide a reasonable estimate of the time distribution of the rainfall losses; that is, the rainfall losses procedure should reproduce the general rainfall loss model as illustrated in Figure 2-1.

Since the procedure had to be amenable for use with the HEC-1 program, an additional requirement was that the procedure had to be an option in the HEC-1 program. The 1990 version of HEC-1 has five rainfall loss options:

- Initial Loss plus Uniform Loss Rate
- Exponential Loss Rate
- SCS Curve Number (CN method)
- Holtan Loss Rate
- Green and Ampt infiltration equation

Maricopa County comprises widely varied geologic, soils, and land-use conditions and very little data are available to derive loss rate parameters or to calibrate rainfall-runoff models. The *Hydrology Manual* must present procedures that can be used without the requirement for extensive original data analyses or, in general, without the need for model calibration. Therefore, parameters for the selected method(s) had to be able to be estimated from readily available local information and from acceptable national and regional information without regard to location.

These requirements resulted in the elimination of the Exponential Loss Rate and the Holtan Loss Rate because, although either method may be acceptable for specific studies involving extensive data analyses and model calibration, there are not adequate data available to provide general guidance in the selection of parameter values for all geologic, soils, and land-use conditions in Maricopa County.

The SCS CN method was initially considered but was subsequently rejected for two major reasons. First, it was believed that although there is relatively detailed information on the selection of CNs, the selection of the CN is too subjective and this could lead to inconsistencies between studies and a lack of reproducibility among the various users. Second, the CN method results in unreasonably high rainfall loss rates during the short periods of high intensity rainfall that occur in Maricopa County and this could result in underestimation of peak discharges.

2.2 Selected Methods and their Applications

The principal method to be used in Maricopa County in the estimation of rainfall losses is the Green and Ampt infiltration equation along with a surface retention loss. This method is encouraged for all applications except when it is known that soil texture does not control the rainfall infiltration process (such as in watersheds that are covered with volcanic cinder or forest duff), or for watersheds where the surface soil is predominantly sand. Use of this method results in the estimation of rainfall losses as illustrated in Figure 2-1.

The second method is that of Initial Loss plus Uniform Loss Rate (IL+ULR). This method should be used when the Green and Ampt method is not applicable, or for other special cases. Use of the IL+ULR method results in the simplified estimation of rainfall losses as illustrated in Figure 2-2.

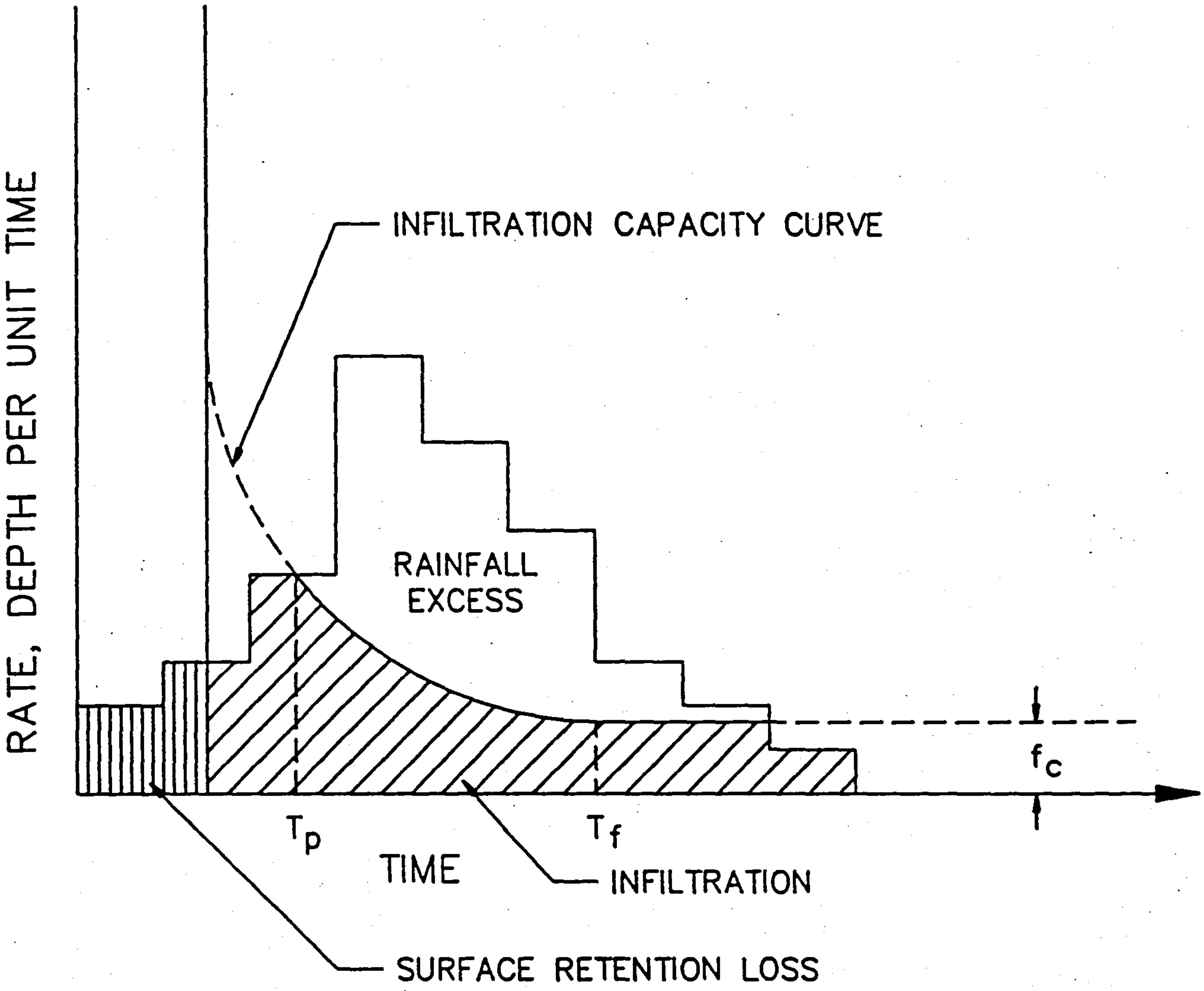


Figure 2-1
Simplified Representation of Rainfall Losses
A function of surface retention losses plus infiltration

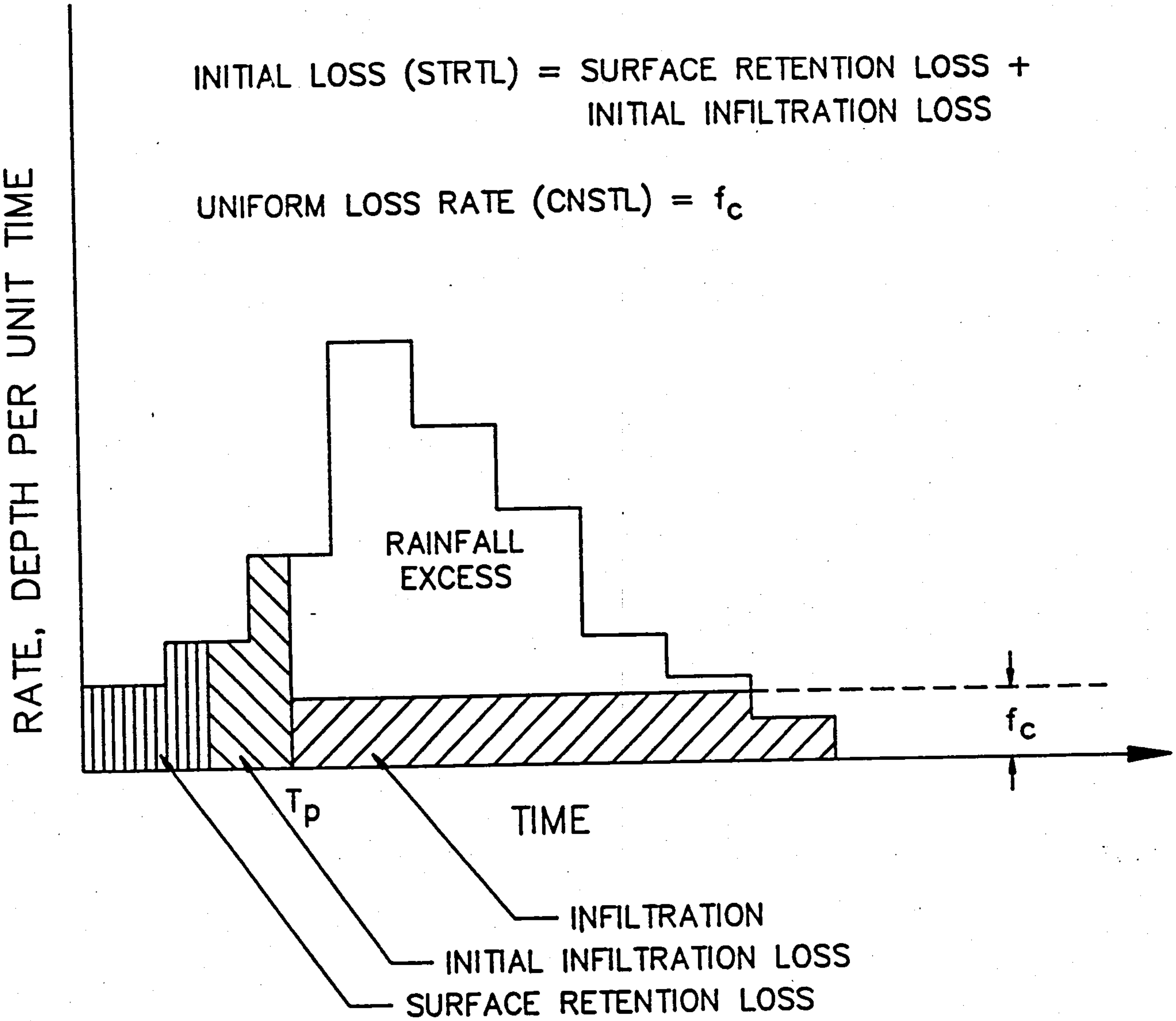


Figure 2-2
Representation of Rainfall Loss according to the Initial Loss Plus Uniform Loss Rate (IL+UR)

2.3 Surface Retention Loss

Both the Green and Ampt method and the IL+ULR method require the estimation of surface retention loss as illustrated in Figures 2-1 and 2-2. 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. Numerous sources of information and data on surface retention loss were obtained

Table 2-1
Surface Retention Loss for Various Land Surfaces in Maricopa County

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

from hydrology texts and from research reports as described in the *Hydrology Manual*. Based on that information, surface retention loss for various land-uses and surface cover conditions in Maricopa County were extrapolated and are shown in Table 2-1.

The values that are shown in Table 2-1 are generally conservative for flood estimation purposes. That is, when a range of values are applicable for any land-use or surface cover class, a reasonable value from the lower end of the range was selected. For example, a recently tilled agricultural field with enclosing irrigation borders could contain several inches of surface retention rather than the 0.5 inch that is shown in Table 2-1; however, for design flood estimation purposes, it is prudent to assume that antecedent conditions would exist (recent irrigation application, absence of irrigation borders, and so forth) that would result in low values of surface retention.

2.4 Green and Ampt Infiltration Equation

The Green and Ampt infiltration equation, as coded into the HEC-1 program, is a function of three parameters; the hydraulic conductivity at natural saturation (XKSAT), the wetting front capillary suction (PSIF), and the volumetric soil moisture deficit (DTHETA) at the start of rainfall. Guidance on the selection of the three parameter values is provided according to soil texture classification for bare ground conditions, as shown in Table 2-2.

The values of XKSAT and PSIF for all soil textures other than silt, are taken from Rawls and others (1983) and the values for silt were extrapolated from information contained in Rawls and Brakensiek (1983). It should be noted that the values of XKSAT for loam and silty loam from Rawls and others (1983) are incorrectly interchanged in that publication and that the values shown in Table 2-2 are correct. The sand and loamy sand soil texture classifications are combined in Table 2-2 to avoid the use of high hydraulic conductivities for sand that may result in underes-

Table 2-2
Green and Ampt Loss Rate Parameter Values for Bare Ground

Soil Texture Classification	XKSAT inches/hour	PSIF, inches	DTHETA*		
			Dry	Normal	Saturated
sand and loamy sand	1.2	2.4	0.35	0.30	0
sandy loam	0.40	4.3	0.35	0.25	0
loam	0.25	3.5	0.35	0.25	0
silty loam	0.15	6.6	0.40	0.25	0
silt	0.10	7.5	0.35	0.15	0
sandy clay loam	0.06	8.6	0.25	0.15	0
clay loam	0.04	8.2	0.25	0.15	0
silty clay loam	0.04	10.8	0.30	0.15	0
sandy clay	0.02	9.4	0.20	0.10	0
silty clay	0.02	11.5	0.20	0.10	0
clay	0.01	12.4	0.15	0.05	0

*Selection of DTHETA:
Dry = Nonirrigated lands, such as desert and rangeland
Normal = Irrigated lawn, turf, and permanent pasture
Saturated = Irrigated agricultural land

timation of peak discharges. For watersheds that are predominantly composed of sand, the IL+ULR method is recommended.

The effect of ground cover on infiltration rate was investigated and the procedure by Rawls and others (1989) to incorporate the effects of vegetation, surface cover, and soil crusting was evaluated. The equations by Rawls and others (1989) were not accepted because they yielded inconsistent results across the range of soil textures. Past research (Moore, 1981), has shown that PSIF and DTHETA are relatively insensitive in comparison with the hydraulic conductivity (XKSAT). Therefore, it was assumed that ground cover and vegetation cover affect only XKSAT. Attempts were made to develop a functional relation for the adjustment of bare ground hydraulic conductivity (XKSAT) as a function of ground cover and canopy cover, but no satisfactory results were obtained. Adequate procedures to adjust bare ground infiltration parameters for the effects of ground cover, such as vegetation, litter and gravel, remains a deficiency. However, as a result of those investigations and through personal communication with researchers in this area (Dr. Timothy Ward, Civil Engineering Department, New Mexico State University, Las Cruces; and Dr. Leonard Lane, USDA, ARS, Arid Lands Watershed Management Research

Unit, Tucson, Arizona), a simplified relation was developed to adjust XKSAT for vegetation cover as shown in Figure 2-3.

The soil moisture deficit parameter (DTHETA) is a volumetric measure of the soil moisture storage capacity that is available at the start of the storm. DTHETA is a function of antecedent moisture and the effective porosity of the soil. The range of DTHETA is from 0.0 for soil that is at natural saturation at the start of the storm to the value of the effective porosity if the soil is completely, or nearly completely, devoid of soil moisture at the start of the storm. In the *Hydrology Manual,* three states of initial soil moisture were assumed for design conditions: dry, normal, and saturated.

For the *dry* state, it is assumed that soil moisture is at the vegetation wilting point and for that condition DTHETA is equal to the effective porosity less the wilting point volumetric soil moisture content. For the *normal* state, it is assumed that soil moisture is at field capacity and for that condition the value of DTHETA is equal to the effective porosity less the field capacity volumetric soil moisture content. For

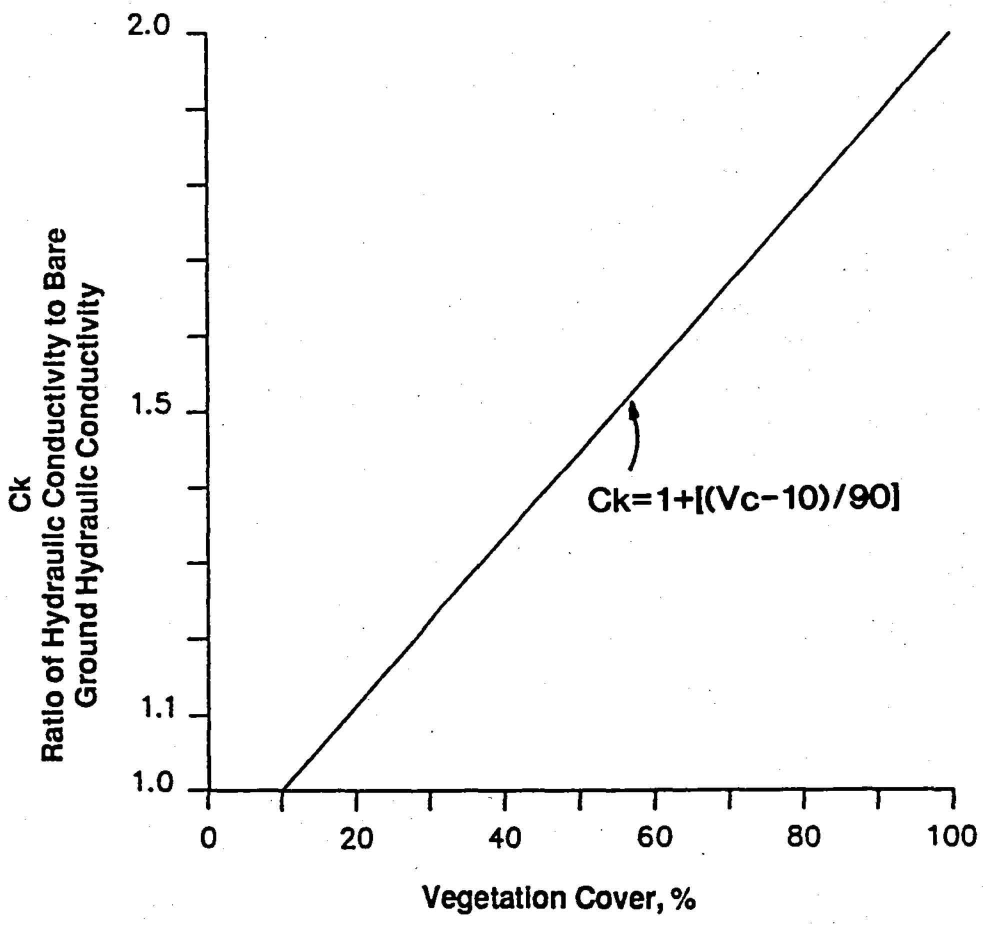


Figure 2-3
Effect of Vegetation Cover on Hydraulic Conductivity for Hydraulic Soil Groups B, C, and D, and for all Soil Textures *other than* Sand and Loamy Sand

the *saturated* state, it is assumed that the soil moisture is at natural saturation and for that condition DTHETA equals 0.0.

The wilting point soil moisture content was assumed to be the 15-bar soil moisture. It is realized that much of the native vegetation in Maricopa County has a wilting point (in terms of pressure) that is higher than the 15-bar value that was used, however there is not substantial difference (from a flood hydrology perspective) in the volumetric value of the 15-bar wilting point and a higher pressure wilting point. Therefore, the 15-bar wilting point was used for all vegetation in Maricopa County. The field capacity soil moisture content was assumed to be the 1/3-bar soil moisture. The soil porosity, field capacity volumetric soil moisture content (1/3-bar), and the wilting point volumetric soil moisture content (15-bar) for each soil texture class were obtained from Rawls and Brakensiek (1983). The values of DTHETA for design flood hydrology purposes in Maricopa County are shown in Table 2-2.

2.5 Application of the Green and Ampt Method

The use of the Green and Ampt method requires the classification of the soil according to soil texture. Three USDA, Soil Conservation Service (SCS) soil surveys are available for Maricopa County for this purpose. The use of the SCS soils information requires some special considerations when selecting Green and Ampt parameter values.

First, many of the soils in Arizona contain significant quantities of gravel, and the adjective *gravelly*, when used in conjunction with the soil texture, can either be disregarded when it is used in conjunction with *sandy*, that is, gravelly sandy loam can be taken as equivalent to sandy loam; or *gravelly* can be used as a replacement for *sandy* when used alone, that is, gravelly clay can be taken as equivalent to sandy clay. Similarly, terms such as *very fine* and *very coarse*, usually used in association with sand, can be disregarded in determining soil texture classification.

Second, layered soils or soils overlaying impervious or nearly impervious strata need special consideration. Surface soils that are more than 6 inches thick should generally be considered adequate to contain infiltrated rainfall for up to the 100-year rainfall event in Arizona without the subsoil restricting the infiltration rate. This is because most common soils have porosities that range from 25 to 35 percent, and therefore 6 inches of soil with a porosity of 30 percent can absorb about 1.8 inches (6 inches time 30 percent) of rainfall infiltration and it is unlikely that more soil moisture storage is needed for storms up to the 100-year event in Maricopa County. In estimating the Green and Ampt infiltration parameters for use in Maricopa County for up to the 100-year rainfall, the top 6 inches of soil should be considered. If the top 6 inch horizon is uniform soil or nearly uniform, then the Green and Ampt parameters are selected for that soil. If the top 6 inch horizon is layered with different soil textures, then the Green and Ampt parameters for the soil texture with the lowest hydraulic conductivity (XKSAT) is selected.

Third, the SCS soil surveys provide soils information according to mapping units, and a mapping unit normally consists of one or more major soils and at least one minor soil. The Green and Ampt parameters will probably vary for each of the major and minor soils and a procedure was developed to estimate the Green and Ampt parameters that are to be applied to the mapping unit, as a whole. The procedure (Van Mullem, 1989) that was tested and adopted, requires that the individual XKSAT values for each of the major and minor soils in a mapping unit be logarithmically areal-averaged:

$$XKSAT_{mu} = antilog\ \Sigma\,(log\ XKSAT_{i})\,(A_i) \tag{2-1}$$

where

$XKSAT_{mu}$	=	equivalent XKSAT for the mapping unit
$XKSAT_i$	=	hydraulic conductivity for each of the major and minor soils in the mapping unit
A_i	=	estimated percentage of that soil in the mapping unit.

PSIF and DTHETA for the mapping unit are obtained from Figure 2-4 corresponding to the mapping unit value of hydraulic conductivity, $XKSAT_{mu}$.

A drainage area or a modeling subbasin is usually composed of areas from different mapping units. The Green and Ampt parameters for a drainage area or a modeling subbasin are determined in a manner similar to that used for a mapping unit:

$$XKSAT_B = antilog\ \Sigma\,(log\ XKSAT_{mu})\,(B_i) \tag{2-2}$$

where

$XKSAT_B$	=	equivalent XKSAT for the drainage area or modeling subbasin
B_i	=	percentage of that mapping unit in the drainage area or modeling subbasin

PSIF and DTHETA for the drainage area or modeling subbasin are obtained from Figure 2-4 corresponding to the value of the area averaged hydraulic conductivity, $XKSAT_B$.

To increase the reproducibility of the procedure, and as an aid to the user, the values of $XKSAT_{mu}$ for the various mapping units in Maricopa County are listed in Appendices A, B, and C of the *Hydrology Manual*. Therefore, the application of the procedure for a drainage area or modeling subbasin, generally only requires the area-averaging by Equation 2-2 to estimate XKSAT, and use of Figure 2-4 to estimate PSIF and DTHETA.

Finally, an adjustment of XKSAT for vegetation cover (Figure 2-3) is applied *after* the XKSAT, PSIF, and DTHETA are determined, as described above. Only the hydraulic conductivity (XKSAT) is adjusted for the effects of vegetation cover; PSIF and DTHETA are *not* adjusted for vegetation cover.

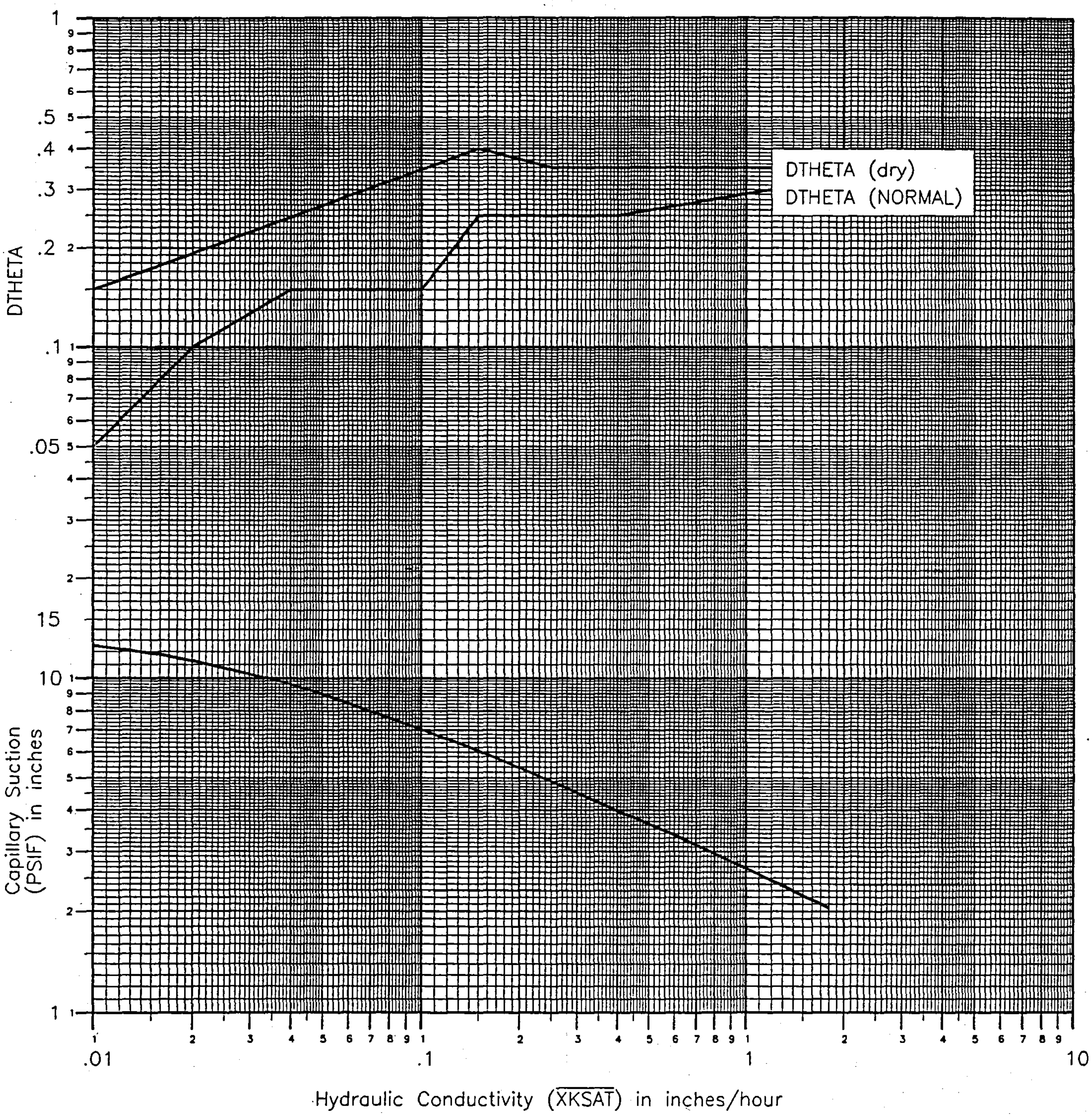


Figure 2-4
Composit values of PSIF and DTHETA as a function of XKSAT
(To be used for area-weighted averaging of Green and Ampt parameters)

In general, parameter values for design should be based on reasonable estimates of watershed conditions that would minimize rainfall losses. The estimate of impervious area (RTIMP) for urbanizing areas should be based on ultimate development in the watershed.

2.6 Initial Loss Plus Uniform Loss Rate

This method requires the estimation of the aggregate rainfall loss, including some infiltration, prior to the onset of surface ponding, and the estimation of the steady-state infiltration rate, fc, as illustrated in Figure 2-2. This procedure is to be applied to special cases where the surface soil is predominantly sand, or when soil texture does not control infiltration. The selection of the parameters, STRTL and CNSTL, must be selected based on regional studies or as the result of model calibration.

3

Maricopa County Unit Hydrograph Procedures

3.1 Selection of Method

Maricopa County is a large land area of 9,226 square miles of diverse physiographic, topographic, and land-use conditions. Although much of the county is undeveloped mountain, rangeland and desert, there is considerable agricultural land that has been leveled for irrigation applications, and there is extensive urbanization, especially in the Phoenix metropolitan area. Hydrologically, the county is semi-arid, but floods from infrequent severe rainfalls are often accompanied by large and hazardous flood discharges that often result in significant property damage, risk to life, and inconvenience. Because of the flood hazard and the economic consequences of design flood hydrology (in terms of economic loss due either to underestimation or over design) there is the need for design flood hydrology procedures that are accurate, reproducible, and practical. From a technical consideration, because of the diverse hydrologic conditions and the continuing urbanization, there is the need for procedures that are sensitive to the modeling of the range of hydrologic conditions in the county. The watershed hydrologic conditions as reflected in the physiography, topography, land-use, and so forth, dictate the rate at which rainfall excess will drain from the land surface; and, therefore, the selection of the procedure(s) to route rainfall excess from the watershed is critical in performing flood hydrology.

The following criteria were used in the selection and development of procedures to route rainfall excess in Maricopa County:

1. The procedures, including parameter estimation, are demonstrated to reproduce regionally representative flood events (accurate).

2. The procedures do not involve extensive subjective decisions in their use or in the calculation of the parameters (reproducible).

3. The procedures are an option of, or are amenable for use in, the HEC-1 Flood Hydrology Program (practical).

4. The procedures are applicable for the variety of hydrologic conditions and land-uses that exist in the county.

5. The parameters are sensitive to modeling land-use changes, especially urbanization.

The first decision was in the selection of either an unit hydrograph method or the kinematic wave method. The routing of rainfall excess by an unit hydrograph is a hydrologic routing scheme that is typically empirically based. An unit hydrograph, due to its empirical development, is a lumped parameter that encompasses the watershed characteristics and meteorologic characteristics of the rainfall into a graphical representation of the runoff response from the watershed. Because it is an hydrologic routing scheme, it is founded on the principle of conservation of mass only.

The kinematic wave method—the modeling of watershed elements as overland flow planes with connecting channel or pipe elements—is a hydraulic routing scheme that is founded on the principles of both conservation of mass and of energy; a decided theoretical improvement over hydrologic routing. However, kinematic wave modeling requires numeric simulation of often complex watersheds into relatively simple overland flow planes and channel elements. This requires numerous decisions in the selection of the relatively few physical watershed characteristics (flow lengths, slopes, flow resistance, etc.) that are used to model the overland flow process.

Kinematic wave modeling is often considered to comprise physical process modeling with its inherent implied improvement in accuracy; however, it has not been clearly demonstrated that the kinematic wave method provides an improvement in modeling accuracy over the unit hydrograph method. In a comparison of rainfall-runoff modeling techniques on upland watersheds, Loague and Freeze (1985) concluded that the more simple unit hydrograph method provided as good as or better results than physically based methods, such as kinematic wave, or more complex models. In a comparison of several runoff models on an urban watershed, Abbott (1978) concluded that the more complex models did not produce better results than the simpler, unit hydrograph models.

Ponce (1991) provides a discussion of the kinematic wave method and provides recommendations for the application of the kinematic wave method and the unit hydrograph method. Ponce describes the kinematic wave solution as a deterministic, distributed-parameter, hydraulic data-intensive method requiring the selection of geometric and frictional parameters. It is applicable to small areas for which the idealizations inherent in the mathematical modeling can be justified on practical grounds. He describes the unit hydrograph method as a conceptual model of runoff generation, spatially lumped, and based exclusively on hydrologic data. He concludes that the issue of choice between kinematic wave and unit hydrograph methods be made based upon the concept of drainage area scale: the kinematic wave method should be used primarily for small drainage areas—less than one square mile—particularly in cases where it is possible to resolve the physical detail without compromising the deterministic nature of the method; and the unit hydrograph method should be used for midsize drainage areas—larger than one square mile but less than 400 square miles. In the proper modeling context (i.e., with subdivided drainage areas), the unit hydrograph can be extended to larger areas.

The majority of applications of flood modeling in Maricopa County would involve areas larger than one square mile. For very small areas (those less than 160 acres) the Rational Method is acceptable and would probably be used. Therefore, following the recommendation of Ponce, there would be little application for kinematic wave modeling in Maricopa County.

Watersheds are usually very complex and heterogeneous and even the most sophisticated models cannot be expected to produce absolutely accurate and completely reproducible results. Additionally, kinematic wave models require extensive subjective decisions in regard to the representation of the watershed that significantly reduces its reproducibility among many users. The simpler unit hydrograph method was selected based on the lack of adequate demonstration that kinematic wave and other more complex models are either more accurate or more desirable for flood hydrology purposes, and the potential limited applicability of kinematic wave modeling due to drainage area size constraints.

Two unit hydrograph methods were selected for further evaluation and development: S-graphs and the Clark Unit Hydrograph (Clark, 1945). S-graphs were considered because of their recent use in the Phoenix area in flood hydrology studies by the Corps of Engineers (1974 and 1982a), and because of the extensive use of S-graphs by local flood control districts in Southern California. The Clark Unit Hydrograph was considered because it is a three parameter method which should result in greater flexibility in applying unit hydrographs to a variety of watershed conditions, and to being more responsive to changes in watershed conditions, such as urbanization. Subsequent development of these two unit hydrograph methods resulted in S-graphs being selected for relatively large, regional flood studies, and the Clark Unit Hydrograph for small watersheds, generally less than 5 square miles, or for larger watersheds that are to be modeled as multi-basin watersheds using small subbasins.

3.2 Clark Unit Hydrograph

The development and evaluation of the Clark Unit Hydrograph procedure for Maricopa County was accomplished by:

1. Compiling small watershed rainfall-runoff data.
2. Analyzing and reconstituting selected rainfall-runoff events.
3. Developing parameter estimation techniques.
4. Testing and verifying the procedure.

The following is a brief description of the development process from data compilation through parameter estimation. Part 4 of this report presents the results of verification.

3.2.1 Source of Data

A preliminary study was conducted to identify rainfall-runoff data that would be appropriate for analysis for the purpose of developing procedures for estimating Clark Unit Hydrograph parameters for ungaged watersheds. Data were identified from five sources: Tucson Experimental Watersheds; Walnut Gulch Experimental Watershed; Denver, Colorado; Albuquerque, New Mexico; and Wyoming.

Tucson Experimental Watersheds: The University of Arizona through the Water Resources Research Center operates four experimental watersheds in the City of Tucson. Three of the watersheds are urbanized and the fourth is partially urbanized. All the watersheds have multiple recording raingages, and descriptions of the watersheds and instrumentation are available in a research report (Resnick and others, 1983).

Walnut Gulch Experimental Watershed: The Arid Lands Watershed Management Research Unit, Agricultural Research Service (ARS), Tucson, Arizona, has operated the Walnut Gulch Experimental Watershed near Tombstone, since 1959. Numerous small basins within the watershed are instrumented for collecting rainfall and runoff data, and the data for 1961 through 1976 are published in the ARS Miscellaneous Publication Series titled *Hydrologic Data for Experimental Agricultural Watersheds in the United States.*

Denver, Colorado: The U.S. Geological Survey (USGS) has participated in a cooperative data collection program with the Urban Drainage and Flood Control District of the Denver metropolitan area for many years. Denver was also selected by the USGS as a monitoring location for its National Urban Runoff Program which resulted in the collection of urban rainfall-runoff data. Data from these programs in Denver are published by the USGS (Ducret and Hodges, 1972; Ducret and Hodges, 1975; Cochran and others, 1979; Gibbs, 1981; Gibbs and Doerter, 1982).

Albuquerque, New Mexico: The USGS has participated in a cooperative data collection program with the City of Albuquerque and the Albuquerque Metropolitan Arroyo Flood Control Authority in collecting rainfall-runoff data. Much of this data represents conditions on natural and urbanized alluvial fans. The data for 1976 through 1983 are published by the USGS (Fischer and others, 1984).

Wyoming: The USGS instrumented numerous small rangeland watersheds in Wyoming, and this data and a description of the watersheds are published (Rankl and Barker, 1977; Craig and Rankl, 1977).

A preliminary evaluation of these data sources resulted in a compilation of 112 relatively severe storm events on 41 watersheds. A summary of the database is shown in Table 3-1.

Table 3-1
Summary of Rainfall-runoff Data for Development of Clark Unit Hydrograph procedures for Maricopa County, Arizona

Location of Watersheds	Number of Instrumented Watersheds	Number of Severe Storm Events
Tucson Experimental Watersheds	4	42
Walnut Gulch Experimental Watershed	9	9
Denver, Colorado	14	14
Albuquerque, New Mexico	7	12
Wyoming	7	21
Totals	41	112

A final data selection resulted in the analysis of 51 of the more severe storm events from 28 watersheds. The selection of the 51 storm events was made after the rainfall hyetographs and runoff hydrographs were plotted, and after the data were screened to assess whether the runoff data appeared to be representative of the rainfall data. Numerous problems are associated with rainfall-runoff data. A common problem that is difficult to assess is whether the measured point rainfalls are representative of the temporal and spatial rainfall over the watershed. The selection process is rather subjective because of the uncertainties in the data.

3.3 Flood Reconstitutions

Prior to execution of the flood reconstitutions using HEC-1, two preliminary analyses were performed: 1) the effective impervious area was determined, and 2) a representative rainfall distribution was selected for watersheds that had more than one recording raingage.

Effective impervious area is the impervious area of the watershed that would drain to the outlet without passing over pervious area. This is also called directly connected impervious area. For each urban watershed, the effective impervious area was estimated by selecting all the storms from the database that appeared to be of low- to medium-intensity and uniform distribution over the watershed. Most of these rainfalls were less than 1.0 inch and greater than about 0.3 inch. Using these events, the effective impervious area was calculated by dividing the recorded volume of runoff by the average depth of rainfall (assumed equivalent uniform depth of rainfall and runoff). Exceptionally high or low values of effective impervious area were eliminated and the average of all remaining calculations was taken as the effective impervious area.

The time distribution used in flood reconstitution was either the recorded distribution, if only one recording raingage was available, or a composite of all of the recording raingage data. The composite time distribution was determined by plotting all of the rainfall mass diagrams for each raingage on a single graph. A representative mass diagram was drawn by considering individual raingage location and also the timing of the runoff hydrograph. This method is preferred to using the option in HEC-1 of weighting rainfall depths and rainfall distributions from individually input raingage data because of rainfall distribution anomalies that could occur when using the HEC-1 option.

Flood reconstitutions of the 51 selected events were performed to determine the "best fit" Clark Unit Hydrograph parameters that would reproduce the storm hydrograph from the recorded rainfall. The flood reconstitutions were performed by using the parameter optimization option of HEC-1. The resulting unit hydrograph is listed as output of the HEC-1 optimization runs. The Clark Unit Hydrograph has three parameters, therefore numerous combinations of the three parameters could result in equally good reproductions of the storm hydrograph. However, the individual parameter values could be in error (compensating errors in the parameter values). The error in estimating T_c and R may be particularly significant if the third parameter, the time-area relation, is fixed *a priori*, such as by using the HEC-1 default time-area relation prior to determining the optimum values of T_c and R. Therefore, it is preferable to estimate the correct value of R before using the HEC-1 optimization option.

An estimate of R was obtained through a recession analysis of each of the runoff hydrographs (Sabol, 1988). Parameter optimization runs were then made using various time-area relations in a trial-and-error procedure until the optimized value of R was reasonably close to the previously estimated value of R. Where more than one storm event was selected for the same watershed, it was determined that the same time-area relation provided the best fit for optimization of all events for the watershed. This was encouraging and provided confidence in the optimization process.

The computation interval could be a controlling factor within HEC-1 when optimizing on T_c. The computation interval had to be selected such that it was less than or equal to one-third T_c. However, the shortest computation interval that can be used is one minute. It must be realized however, that the resolution of the rainfall data was not such that the rainfall distribution could be accurately reproduced at this small of a computation interval. There probably was an artificial "smoothing" of the rainfall distribution by this process and this can be expected to result in error in the optimized vales of the parameters, particularly T_c.

The reconstitutions were evaluated and 13 of the reconstitutions were rejected, leaving 38 "valid" reconstitutions. Often this rejection was based on the belief that the recorded rainfall was not truly representative of the rainfall over the watershed. The database was again critically reviewed and 17 control events were selected as being "most accurate."

3.3.1 Parameter Estimation

The Clark Unit Hydrograph has three parameters:

- Time of concentration, T_c
- Storage coefficient, R
- Time-area relation

The procedure that was developed to estimate these three parameters is described below.

Time of Concentration, T_c: Two methods were used to investigate and to develop a T_c prediction equation. First, a literature search was conducted to determine methods that are presently available for estimating T_c. Second, an attempt was made to develop a new T_c equation or to modify an existing one based on the results of the flood reconstitutions.

The results of the literature review resulted in the selection of the T_c equation by Papadakis and Kazan (1987). That equation is empirically derived from the analysis of 375 data sets from 84 natural watersheds and 291 experimental watersheds. The Papadakis and Kazan equation has the form:

$$T_c = 11.4\,(L^{0.50} K_b^{0.52} S^{-0.31} i^{-0.38}) \tag{3-1}$$

where

T_c	=	time of concentration, hours
L	=	length of flow path, miles
K_b	=	resistance coefficient
S	=	average slope of flow path, feet per mile
i	=	intensity of rainfall excess, inches per hour

The Papadakis and Kazan equation was selected because of the extensive database that was used in its development, and because of its similarity to a theoretically derived equation (Henderson and Wooding, 1964; and Ragan and Duru, 1972) of the form:

$$T_c = f\,(L^{0.6} K_b^{0.6} S^{-0.2} i^{-0.4}) \tag{3-2}$$

Independent T_c equations were derived from the flood reconstitution database, but these were rejected because of the higher confidence that was placed in the equation by Papadakis and Kazan.

In Equation 3-1, the flow path length, L, and the slope, S, can be measured from watershed maps. The rainfall excess intensity, i, is the average rainfall excess intensity during the time of concentration, T_c, and therefore the solution of Equation 3-1 requires two steps:

1. Estimate the time distribution of the rainfall excess, (i).

2. Solve the equation iteratively from estimates of T_c and corresponding i from Step 1 for that estimated T_c.

The resistance coefficient, K_b, requires a user's decision, which could result in inconsistent results when used by different individuals—unless clear guidance is provided to minimize discrepancies in the selection of K_b. Papadakis and Kazan (1987) present data and a graphical relation for the estimation of K_b as a function of drainage area and surface cover categories. The graphical relation for K_b was modified for use in Maricopa County as shown in Figure 3-1. The use of Figure 3-1 along with Table 3-2 results in more consistent selection of K_b for the same watershed among various users, increasing reproducibility in the use of the procedure. Figure 3-1 also indicates that as the size of the watershed increases, there is an increase in hydraulic efficiency due to the greater volumes and rates of runoff that are generated.

Equation 3-1 with the Maricopa County K_b relation (Figure 3-1 and Table 3-2) was tested using the 17 control events from the flood reconstitution study. Of those 17 events, 9 are on urbanized watersheds, 7 are on completely undeveloped watersheds, and 1 is on a watershed that is only partially urbanized. The T_cs for the non-control events were not considered to be adequate for test purposes because of the extremely short durations of the rainfall excess, smoothing errors in defining the rainfall distributions, or other uncertainties with the data. A comparison of the T_cs from the flood reconstitutions of the 17 control events to the estimated T_cs is shown in Figure 3-2. Only five of the events have durations of rainfall excess that exceed T_c and this affected the calculation of i in Equation 3-1 because the period of no rainfall excess had to be included in the estimation of i. This probably has a significant impact on the accuracy of the T_c estimation. The five events for which the rainfall excess duration exceeded T_c are indicated in Figure 3-2.

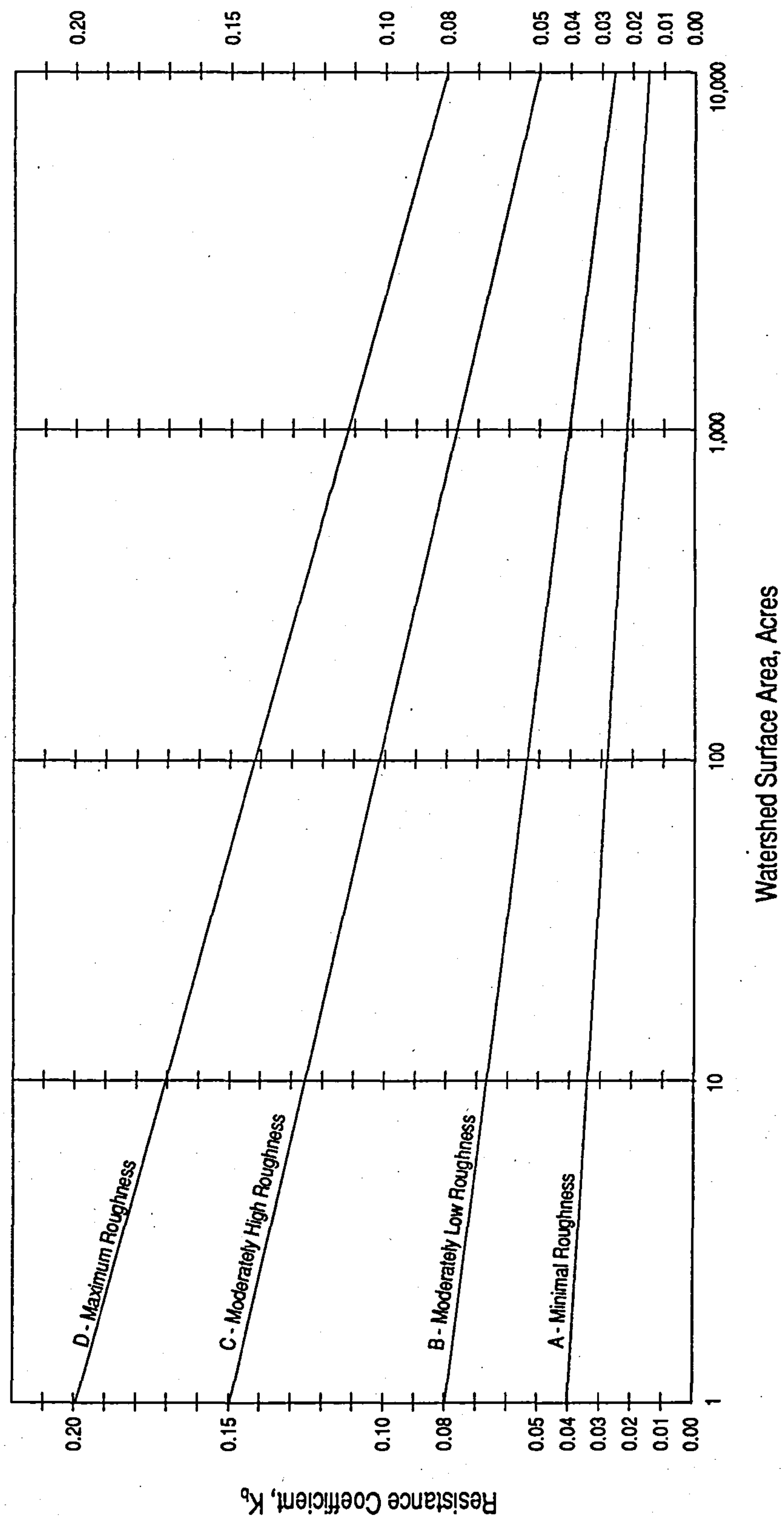


Figure 3-1
Resistance Coefficient "Kb" as a Function of Watershed Type and Size

Table 3-2
Equation for Estimating K_b in the T_c Equation

$K_b = m \log A + b$ Where A is drainage area, in acres				
Type	**Description**	**Typical Applications**	**Equation Parameters**	
			m	**b**
A	Minimal roughness: Relatively smooth and/or well graded and uniform land surfaces. Surface runoff is sheet flow.	Commercial/ industrial areas Residential area Parks and golf courses	–0.00625	0.04
B	Moderately low roughness: Land surfaces have irregularly spaced roughness elements that protrude from the surface but the overall character of the surface is relatively uniform. Surface runoff is predominately sheet flow around the roughness elements.	Agricultural fields Pastures Desert rangelands Undeveloped urban lands	–0.01375	0.08
C	Moderately high roughness: Land surfaces that have significant large- to medium-sized roughness elements and/or poorly graded land surfaces that cause the flow to be diverted around the roughness elements. Surface runoff is sheet flow for short distances draining into meandering drainage paths.	Hillslopes Brushy alluvial fans Hilly rangeland Disturbed land, mining, etc. Forests with underbrush	–0.025	0.15
D	Maximum roughness: Rough land surfaces with torturous flow paths. Surface runoff is concentrated in numerous short flow paths that are often oblique to the main flow direction.	Mountains Some wetlands	–0.030	0.20

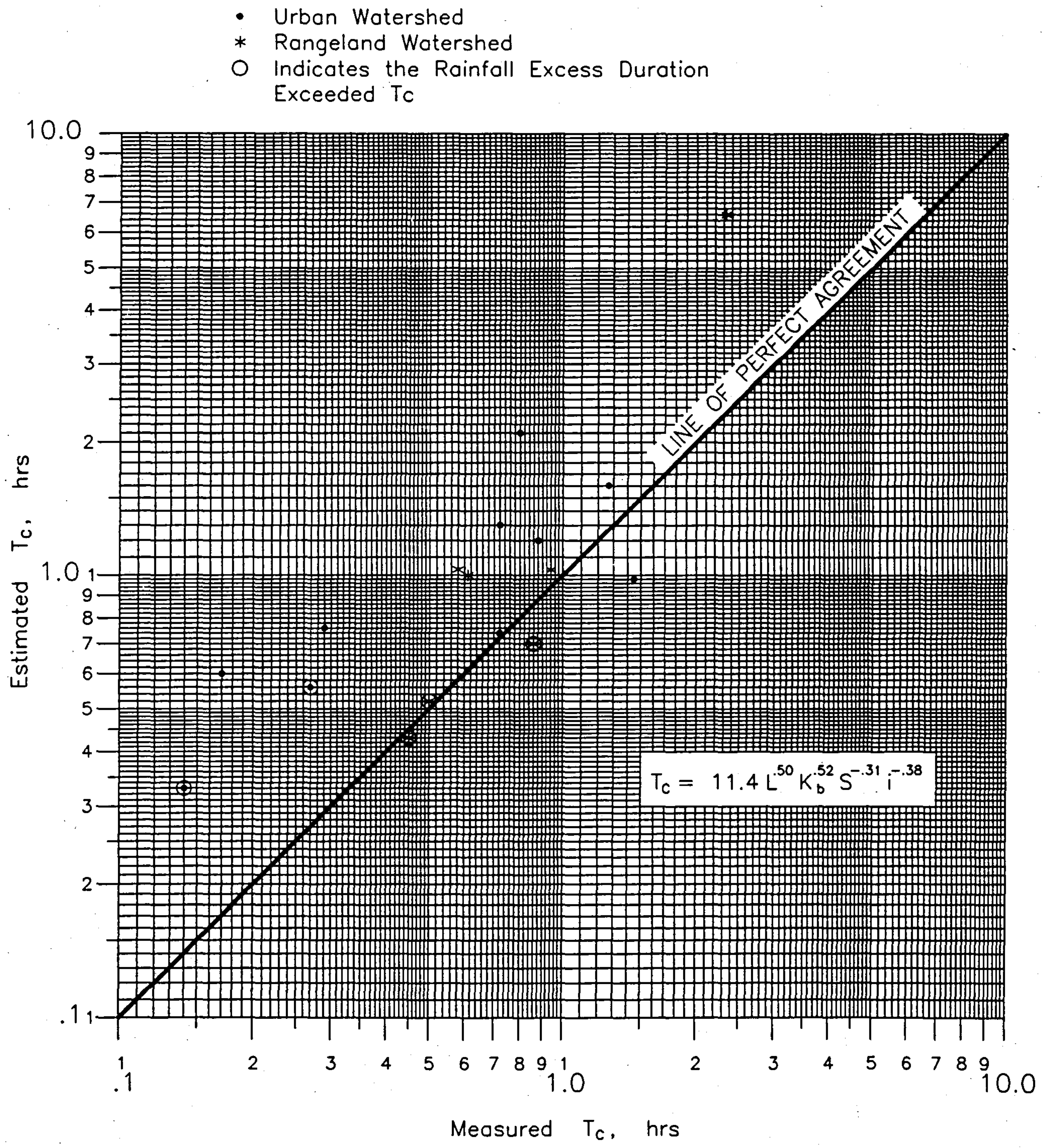


Figure 3-2
Comparison of Time of Concentrations from Storm Reconstitutions (measured) and as estimated

Storage Coefficient, R: Little literature or research results are available on the estimation of the storage coefficient, R. Examples of T_c and R prediction equations based on regional data are presented in the *Hydrologic Engineering Center, Training Document No. 15* (U.S. Army Corps of Engineers, 1982b). That approach was used to develop a prediction equation for R. Using the results from the flood reconstitution study, an R prediction equation was developed from a stepwise multiple regression analysis. The R equation is:

$$R = 0.37\, T_c^{1.11}\, A^{-0.57}\, L^{0.80} \tag{3-3}$$

where

R	=	storage coefficient, hours
T_c	=	time of concentration, hours
A	=	drainage area, square miles
L	=	length of flow path, miles

Equation 3-3 was tested by applying it to the 17 control events from the flood reconsititutions, and the results are shown in Figure 3-3. Since the R equation is a function of T_c, any error in the estimation of T_c is reflected in the subsequent estimation of R. For design rainfall events where the rainfall excess duration will normally exceed the estimate of T_c, this will probably result in improved estimates of T_c and R than are demonstrated in Figures 3-2 and 3-3.

Time-Area Relations: The development of time-area relations from maps and watershed information is a tenuous procedure, and unreliable and inconsistent results can be achieved. This is especially true of urban watersheds because of the complex and convoluted drainage patterns that usually result from development. It is desirable to have dimensionless time-area relations that can be used for various types of watersheds. The Hydrologic Engineering Center has developed a default time-area relation for use with HEC-1 (U.S. Army Corps of Engineers, 1990). This relation between travel time and contributing drainage area is very nearly linear. Such a nearly linear time-area relation may not provide appropriate representation of the faster response time that is expected for urban watersheds, or the more delayed response that often occurs from many natural watersheds.

In the flood reconstitutions, the best fit time-area relation was determined using a trial-and-error process of: 1) selecting a dimensionless time-area relation; 2) performing a HEC-1 optimization with that relation; and 3) evaluating the results. Different time-area relations were tried until the following criteria were met:

1. The peak discharge and time-to-peak of the reconstituted hydrograph were the best fit to the recorded hydrograph.

2. The general shape of the reconstituted and recorded hydrographs were similar.

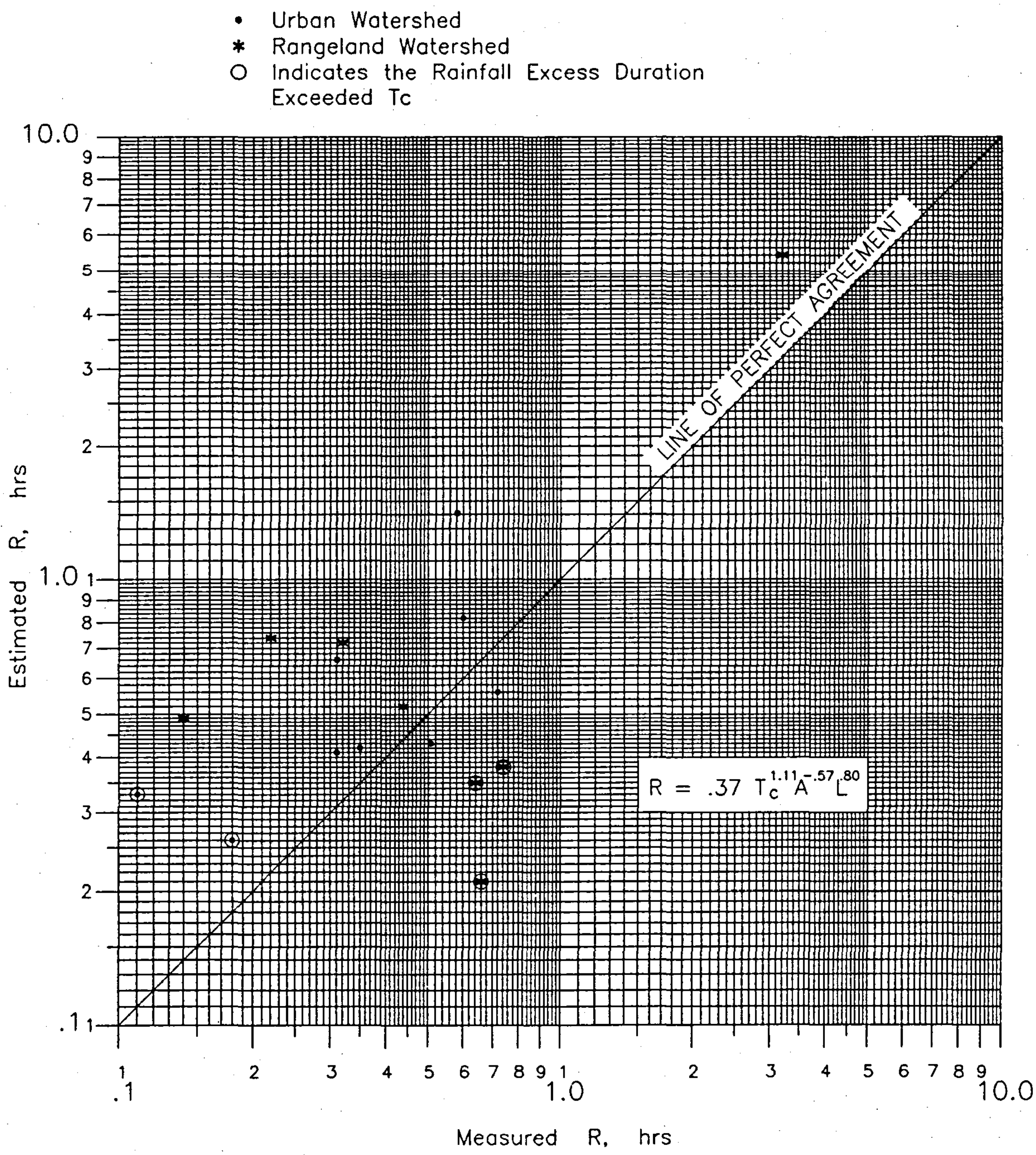


Figure 3-3
Comparison of Storage Coefficients from Storm Reconstitutions (measured) and as estimated

3. The reconstituted value of R was as close as possible to the R value from the recession analysis.

The results of the flood reconstitutions using the above process indicated that for urban watersheds there is quick runoff response. An envelope of dimensionless time-area relations from the flood reconstitutions for the urban watersheds is shown in Figure 3-4. A synthetic urban watershed time-area relation, shown in Figure 3-4, was adopted that generally is an average of the reconstituted time-area relations. Based on those results, for urban watersheds, 50 percent of the area is contributing runoff to the concentration point after 22 to 51 percent of the time of concentration. On the average, it takes only 36 percent of the time of concentration for 50 percent of an urban area to contribute runoff to the concentration point.

Similarly, for natural watersheds, the flood reconstitutions indicated that there is slower runoff response. An envelope of dimensionless time-area relations from the flood reconstitutions for the natural watersheds is shown in Figure 3-5. A synthetic natural watershed time-area relation, shown in Figure 3-5, was adopted that generally is an average of the reconstituted time-area relations. Based on those results, for natural watersheds, 50 percent of the area is contributing runoff to the concentration point after 50 to 85 percent of the time of concentration. On the average, it takes 63 percent of the time of concentration for 50 percent of a natural watershed to contribute runoff to the concentration point.

The HEC-1 default time-area relation is shown in Figures 3-4 and 3-5. There are significant differences in the synthetic time-area relations that are adopted as compared to the HEC-1 default time-area relation. It is anticipated that the synthetic time-area relations will provide improved accuracy over the use of the HEC-1 default time-area relation, and that use of these relations will result in improved modeling sensitivity to the effects of urbanization on flood runoff.

3.3.2 Application of the Clark Unit Hydrograph

There are no theoretical limits to the application of the Clark Unit Hydrograph; however, because of the database and the method that was used to develop the three parameter estimators, there are some limitations that should be observed. The procedure can be used for all types of watersheds; that is, urban, agricultural, rangeland, alluvial fan, hillslopes, mountain, and so forth. Watershed size should be 5 square miles or less, with an upper limit of application to a single watershed of 10 square miles. Watersheds larger than 5 square miles should be divided into smaller subbasins for modeling purposes, and the procedure can be used for large watersheds as long as the area limitation is observed for the subbasins. Many watersheds smaller than 5 square miles should be divided into subbasins depending on the drainage network and degree of homogeneity of the watershed. Another limitation is recommended as a consequence of the T_C equation being a function of average rainfall excess intensity (Equation 3-1). The calculated T_C should not exceed the duration of rainfall excess. If the calculatd T_C exceeds this duration, then the drainage area or subbasin should be reduced in size so that this condition does not occur.

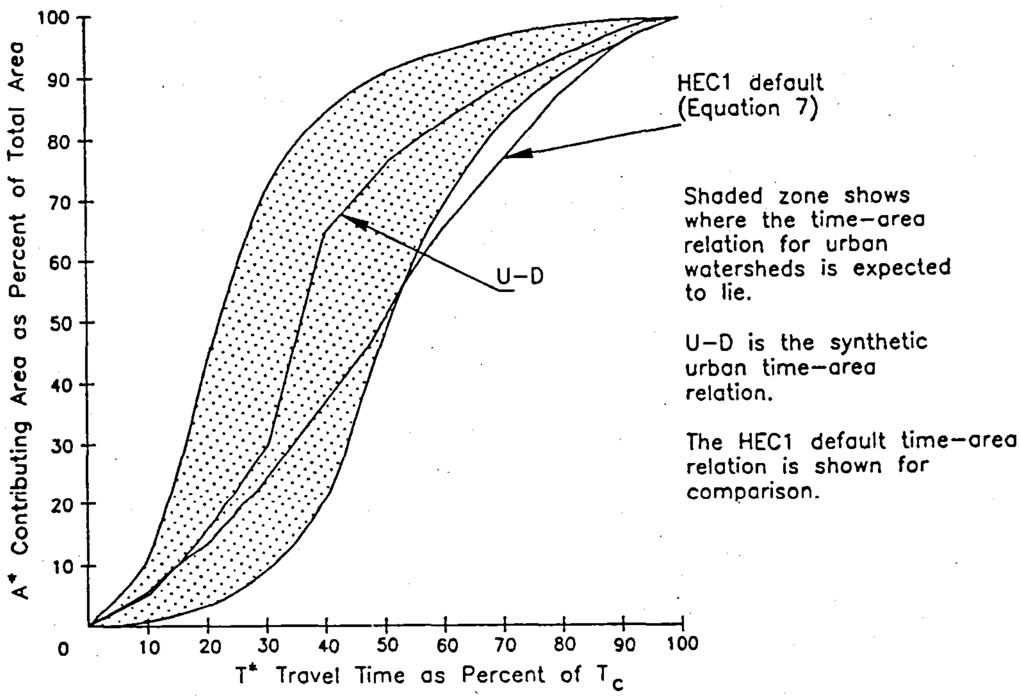


Figure 3-4
Synthetic Time-Area Relation for Urban Watersheds

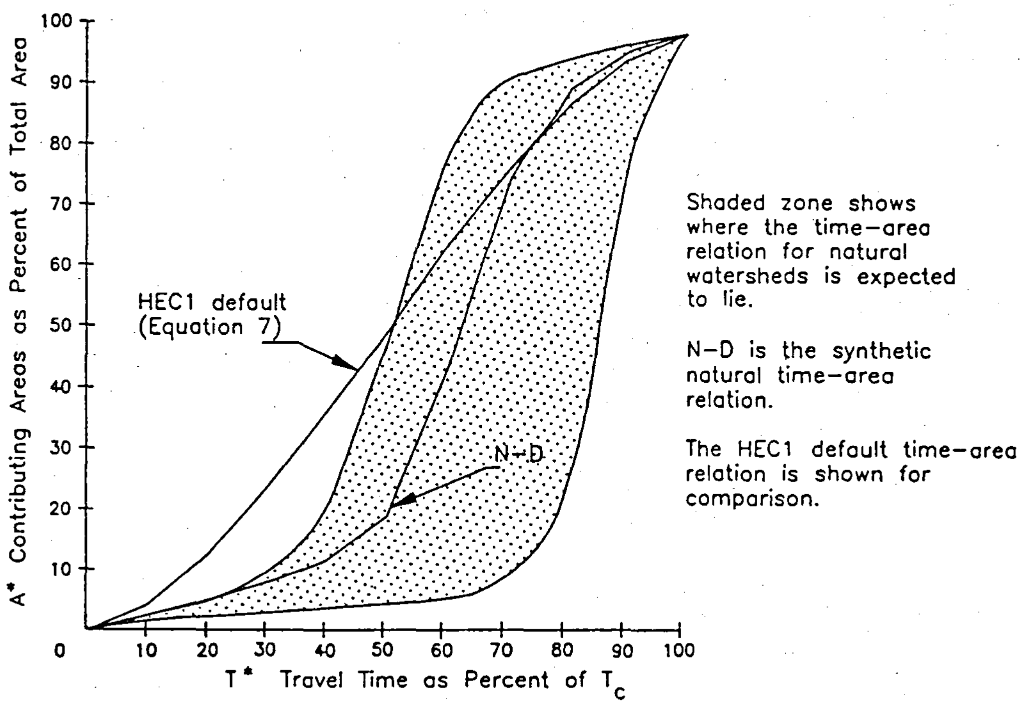


Figure 3-5
Synthetic Time-Area Relation for Natural Watersheds

The procedure to synthesize a Clark Unit Hydrograph using the Maricopa County procedure is as follows:

1. Estimate T_c using Equation 3-1. This will require the development of the design rainfall hyetograph and the time distribution of rainfall excess prior to using the equation.

2. Estimate R using Equation 3-3.

3. Develop the time-area relation for the watershed. This is expected to fall within the appropriate envelopes that are shown in Figures 3-4 and 3-5, or select the synthetic time-area relation for either urban or natural watersheds.

3.4 S-Graphs

An S-graph is a dimensionless form of unit hydrograph. The concept of the S-graph dates back to the development of the unit hydrograph, although the application of S-graphs is not as widely practiced as that of the unit hydrograph. The use of S-graphs is practiced mainly by the U.S. Army Corps of Engineers, particularly the Los Angeles District, and the U.S. Bureau of Reclamation (USBR). Recently, S-graphs have been adopted as the unit hydrograph procedure by several county flood control districts in Southern California and selected S-graphs are presented in those county hydrology manuals. The S-graphs in those California hydrology manuals were selected primarily from S-graphs that were defined by the U.S. Army Corps of Engineers, Los Angeles District, from a rather long and extensive history of analyses of floods in California. Other areas may not have the advantage of such an extensive database of S-graphs. Recently, the USBR issued two publications that describe the use of S-graphs for flood hydrology and that provide some guidance in their selection and use (U.S. Bureau of Reclamation, 1987; and, Cudworth, 1989). Six regionalized S-graphs for use in the western United States are provided in those publications.

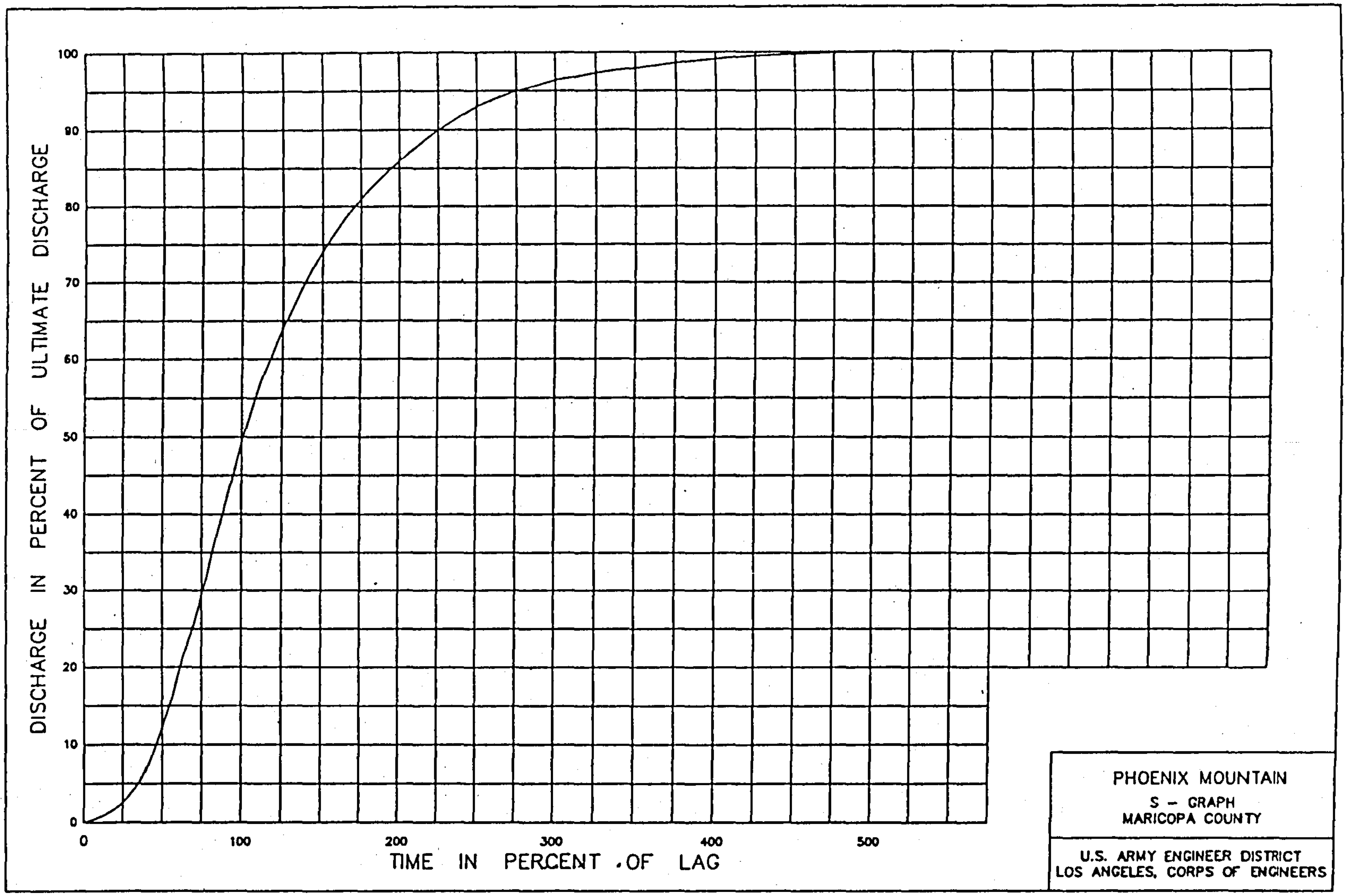
3.4.1 Sources of S-Graphs

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

3.4.2 Selection of S-Graphs for Maricopa County

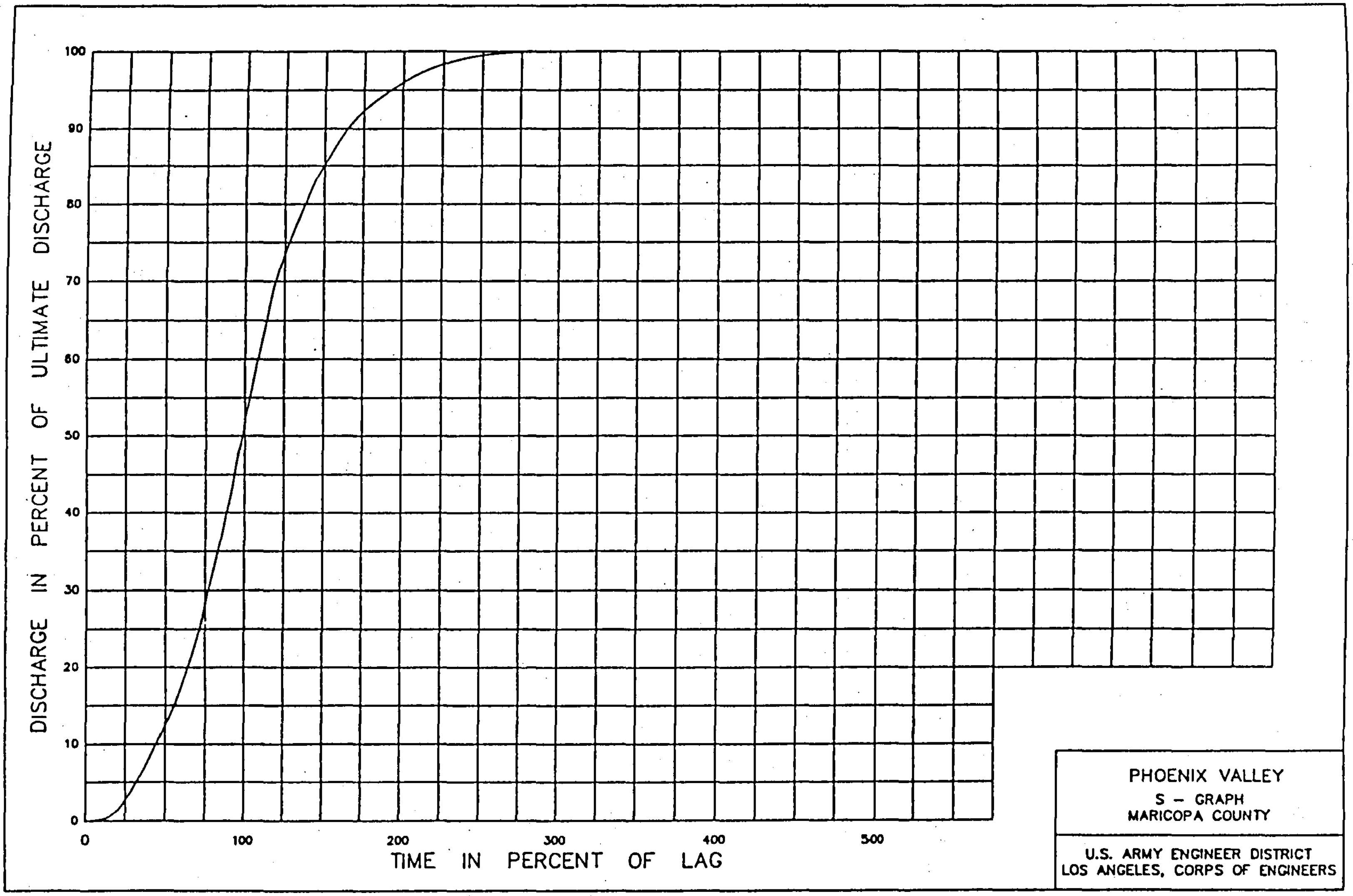
From the S-graph study, it was determined that S-graphs from Southern California and other regions that are meteorologically dissimilar to Maricopa County could not be selected for use in Maricopa County. That conclusion was based on the observation that storm characteristics (duration and rainfall intensities) had a measurable impact on the shape of the S-graph.

Two regional S-graphs were selected for use in Maricopa County. These are referred to as the Phoenix Mountain and the Phoenix Valley S-graphs, and these are shown in Figures 3-6 and 3-7, respectively. Both of those S-graphs were developed by the U.S. Army Corps of Engineers, Los Angeles District, from the reconstitution of flood events in Maricopa County. The storms and gaged watershed data that were used to develop the Phoenix Mountain and the Phoenix Valley S-graphs are listed in Tables 3-3 and 3-4, respectively. For the Phoenix Mountain S-graph, the Corps selected the individual S-graph of the September 1970 flood on New River near Rock Springs as being representative of a regional mountain unit hydrograph. That S-graph is for general use in mountainous, non-urbanized areas of Maricopa County. For the Phoenix Valley S-graph, the Corps selected the individual S-graph of the September 1970 flood on Skunk Creek near Phoenix as being representative of a regional valley unit hydrograph. That S-graph is for general use in valley and urbanized areas of Maricopa County. S-graphs were used in regional Maricopa County flood studies by the Corps (U.S. Army Corps of Engineers, 1974 and 1982a), in flood studies for Pima County, Arizona (U.S. Army Corps of Engineers, 1988a), and in flood studies for Clark County, Nevada (U.S. Army Corps of Engineers, 1988b). These two S-graphs may be applicable for general use throughout Arizona.



DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE
0
10
20
30
40
50
60
70
80
90
100
TIME IN PERCENT OF LAG
0
100
200
300
400
500
PHOENIX MOUNTAIN
S - GRAPH
MARICOPA COUNTY
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

Figure 3-6


100
90
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DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE
0
100
200
300
400
500
TIME IN PERCENT OF LAG
PHOENIX VALLEY
S – GRAPH
MARICOPA COUNTY
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

Figure 3-7

Table 3-3
Streamgages and Storm Events used in Flood Reconstitution for the development of the Phoenix Mountain S-graph

Stream Gauge	USGS Gauge No.	Drainage Area, square miles	Peak Discharge from Storm, in cfs	
			Dec 1967	Sept 1970
New River near Rock Springs	09513780	67.3	10,500	18,600
New River at New River	09513800	85.7	12,500	19,500

Table 3-4
Streamgages and Storm Events used in Flood Reconstitutions for the development of the Phoenix Valley S-graph

Stream Gauge	USGS Gauge No.	Drainage Area, square miles	Peak Discharge from Storm, in cfs	
			Dec 1967	Sept 1970
New River at Bell Road	09513835	187.0	14,600	11,900
Skunk Creek near Phoenix	09513860	64.6	6,800	9,650
Cave Creek at Phoenix	09512400	70.0*	4,080	780
Queen Creek Tributary at Apache Junction	09492200	0.51		
Part 1			28	138
Part 2				84.5
Agua Fria Tributary at Youngtown	09513700	0.13		
Part 1				15.8
Part 2				40.5

*USGS *Water Resources Data for Arizona* indicates drainage area of 252 square miles. The contributing drainage area is 70.0 square miles because of the noncontributing area controlled by Cave Creek Dam.

3.4.3 S-Graph Parameter, Lag

The use of an S-graph requires the estimation of only one parameter, the basin Lag. This Lag is defined as the time for 50 percent of the total volume of runoff for an unit hydrograph to occur. The equation to estimate Lag is:

$$Lag = 24\, K_n\, (L L_{ca}/S^{0.5})^{0.38} \tag{3-4}$$

where

Lag is in hours

L = length of the longest watercourse, miles

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

S = watercourse slope, feet per mile

K_n = basin resistance coefficient

Guidance in selecting K_n is provided in the *Hydrology Manual*. Generally, $K_n = 0.015$ for urban areas and for drainage areas with improved channels; $K_n = 0.030$ for rolling lands with meandering watercourses; $K_n = 0.050$ for rugged land with large roughness elements in the watercourses; and $K_n = 0.20$ for land with no channelized drainage network and significant overland flow resistance. The selection of K_n for a basin is subjective and the resultant unit hydrograph is sensitive to the selection of K_n. Comparative watershed data on K_n and basin Lag are provided in the *Hydrology Manual* to aid in the estimation of Lag.

The basin resistance coefficient, K_n, is a measure of the hydraulic efficiency of the watershed and K_n is not a constant for a given watershed for all rainfall depths and rainfall intensities. The guidance on selecting K_n is intended for use with severe storms, such as the 100-year event; as the rainfall depth decreases, as with more frequent storms, K_n should be increased somewhat. For very severe storms, as a PMP, K_n should be decreased somewhat.

3.4.4 Application of S-Graphs

The S-graphs can be used in any application for which an unit hydrograph can be used. There is no maximum or minimum size limitation, although S-graphs are intended to be used in Maricopa County for large, regional flood studies. For studies of smaller areas or for which extensive subdividing of the watershed into small basins is to be performed, the Clark Unit Hydrograph procedure is recommended.

4

Verification Results

4.1 Introduction

The procedures in the *Hydrology Manual* were verified—where data are available—by two types of tests: *frequency simulation* and *event simulation*. *Frequency simulation* requires a streamgage record of adequate length to establish a flood frequency relation for a watershed. The procedures in the *Hydrology Manual* are then used on the contributing watershed, with rainfall input according to design rainfall criteria, to attempt to reproduce the observed flood frequency relation. Calibration of the model is *not* performed because the intent of this testing is to attempt to simulate the flood frequency relation by use of the procedures in the manual without modification. This is the most important type of testing since it demonstrates whether or not the procedures are able to transform a rainfall event of given return period to a runoff event of equivalent return period. Frequency simulation results are presented for seven watersheds with basin characteristics and streamgage data as described below:

Watershed	Size, square mile	Land-use	Gage Record, years
Agua Fria River tributary at Youngtown	0.13	Residential	13
Academy Acres at Albuquerque, New Mexico	0.12	Residential	14
Tucson Arroyo at Tucson	8.12	Residential/Commercial	26
Walnut Gulch 63.011 near Tombstone	3.18	Desert/Rangeland	27
Walnut Gulch 63.008 near Tombstone	5.98	Desert/Rangeland	27
Cave Creek near Cave Creek	127	Sonoran Desert/Mountain	26
Agua Fria River near Mayer	588	Forest and Range/Mountain	50

Event simulation requires both recorded rainfall and runoff (hydrograph) data for a storm event. The procedures in the *Hydrology Manual* are used on the contributing watershed, with rainfall input as best reconstituted from rainfall records, to attempt to reproduce the observed runoff hydrograph or peak discharge for the storm. Calibration of the model is *not* performed because the intent of this testing is to attempt to simulate the rainfall losses and runoff hydraulics by use of the procedures in the manual without modification. This type of testing evaluates the accuracy of the rainfall loss and unit hydrograph procedures but not the design rainfall criteria.

Event simulation results are presented for four events on three watersheds as described below:

Watershed	Storm Event	Rain, inches	Peak Discharge, cfs
Agua Fria River Tributary at Youngtown	16 Oct 1964	1.76	73
	5 Sept 1970	2.92	40
Tucson Arroyo at Tucson	12 Aug 1972	1.23 – 1.63	2,950
Walnut Gulch 63.008 near Tombstone	4 Aug 1980	0.72 – 1.89	894

4.2 Agua Fria River Tributary at Youngtown

The drainage area is a small (0.13 square mile) residential area that contributes runoff to a tributary of the Agua Fria River. No defined drainage network exists and runoff is concentrated in streets. The area consists of moderately-sized single-family residential units with a density of about five units per acre. Pervious area is composed of about equal portions of lawn and desert landscaping.

Both frequency and event simulations were performed for this watershed. A single-basin model—a model of the watershed with a single unit hydrograph—was used. The Clark Unit Hydrograph procedure was used for both tests.

Streamflow records are available for 1961 through 1973. A graphical flood frequency analysis was performed using the Cunnane plotting position equation and log-extreme value probability paper. A straight line was fit to the data and flood peak discharges were estimated for return periods of 2- to 100-year.

Rainfall for the frequency simulation was developed by procedures in the *Hydrology Manual*. For a watershed of this size—0.13 square miles—Pattern No. 1 and a depth-area reduction factor of 1.0 were required.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. The Green and Ampt parameters were area-averaged with hydraulic conductivity (XKSAT) of 0.18 inch/hour, capillary suction (PSIF) of 5.8 inches, and soil moisture deficit (DTHETA) of 0.25. The surface retention (IA) was estimated as 0.15 inch, and the effective impervious area (RTIMP) as 25 percent.

The Clark Unit Hydrograph parameters, T_c and R, were calculated based on an area (A) of 0.13 square miles, watercourse length (L) of 1.023 miles, slope of 5.8 ft/mi, and resistance coefficient (K_b) of 0.028. T_c and R varied for each flood return period since the procedures to estimate these parameters are a function of rainfall excess intensity, which varies for each flood return period. The synthetic urban time-area relation was used.

The results of the graphical flood frequency analysis and results of the frequency simulation are shown in Table 4-1 and Figure 4-1.

The record length is only 13 years, therefore accuracy should be judged based on a comparison of flood peak discharges in the 2- to 25-year return period range. Within that range, the frequency simulation is extremely good. Even at the 50- and 100-year return periods, the simulation results agree favorably with the flood frequency results.

Data were available to perform two event simulations for this watershed. The results of these two simulations are shown in Figures 4-2 and 4-3. The event simulations yielded particularly good agreement with the recorded runoff volumes (118% for 16 Oct 1964 and 116% for 5 Sept 1970). This indicates that the rainfall loss procedure is reasonably reproducing the rainfall loss process. However, the peak discharges are low for both events (60% for 16 Oct 1964 and 77% for 5 Sept 1970). Much of the discrepancy in peak discharge is attributed to the procedure for calculating T_c. In that procedure, the duration of rainfall excess for both storms was less than the estimated values of T_c, and that condition probably accounted for an overestimate in the value of T_c.

Table 4-1
Results of frequency simulation for the Agua Fria River Tributary at Youngtown

Return Period, years	Peak Discharge, cfs Flood Frequency Analysis	Peak Discharge, cfs Frequency Simulation
2	16	16
5	31	32
10	48	48
25	83	73
50	126	95
100	190	123

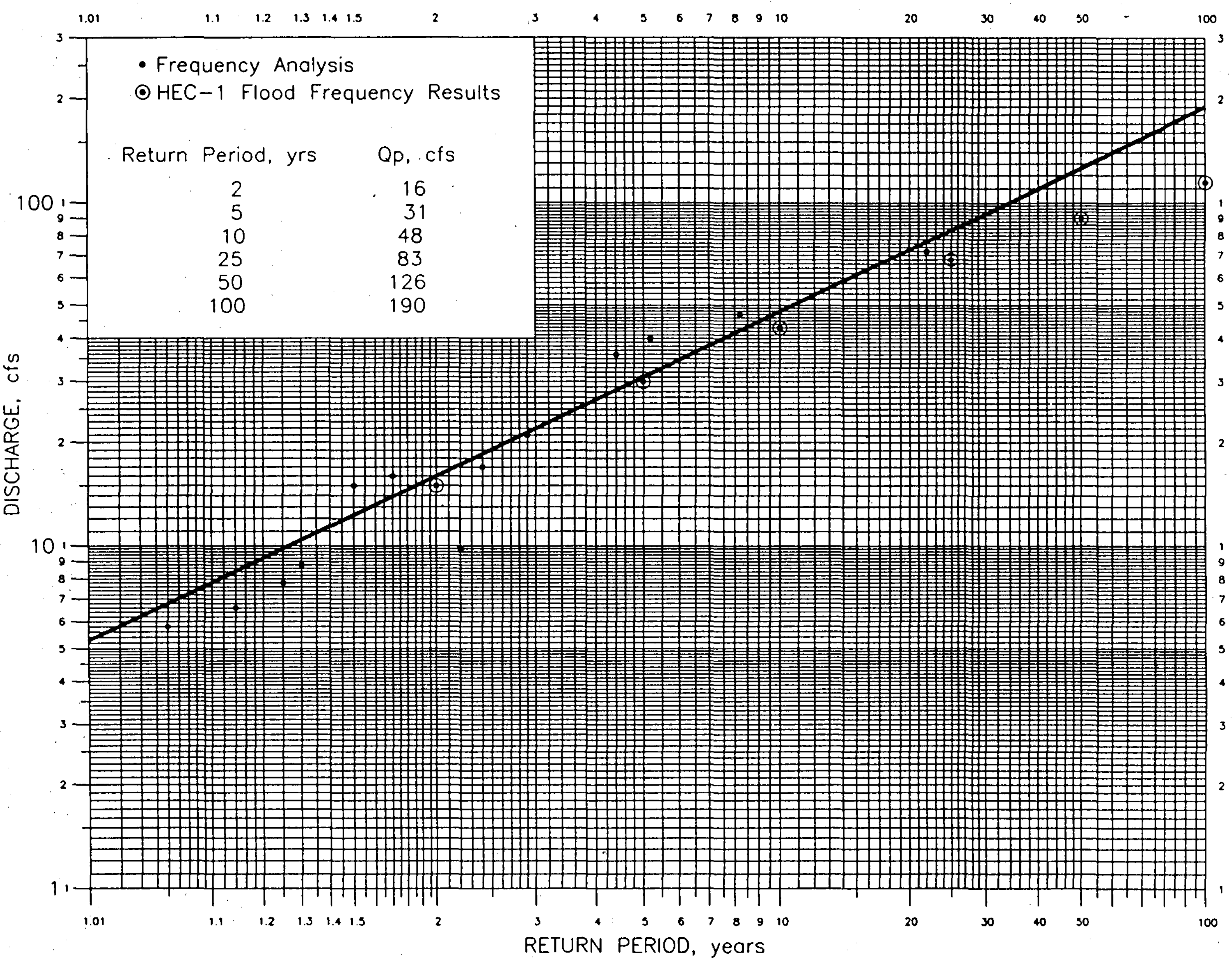

Figure 4-1
Flood Frequency Analysis and Model Results
for the Agua Fria River Tributary at Youngtown, Arizona

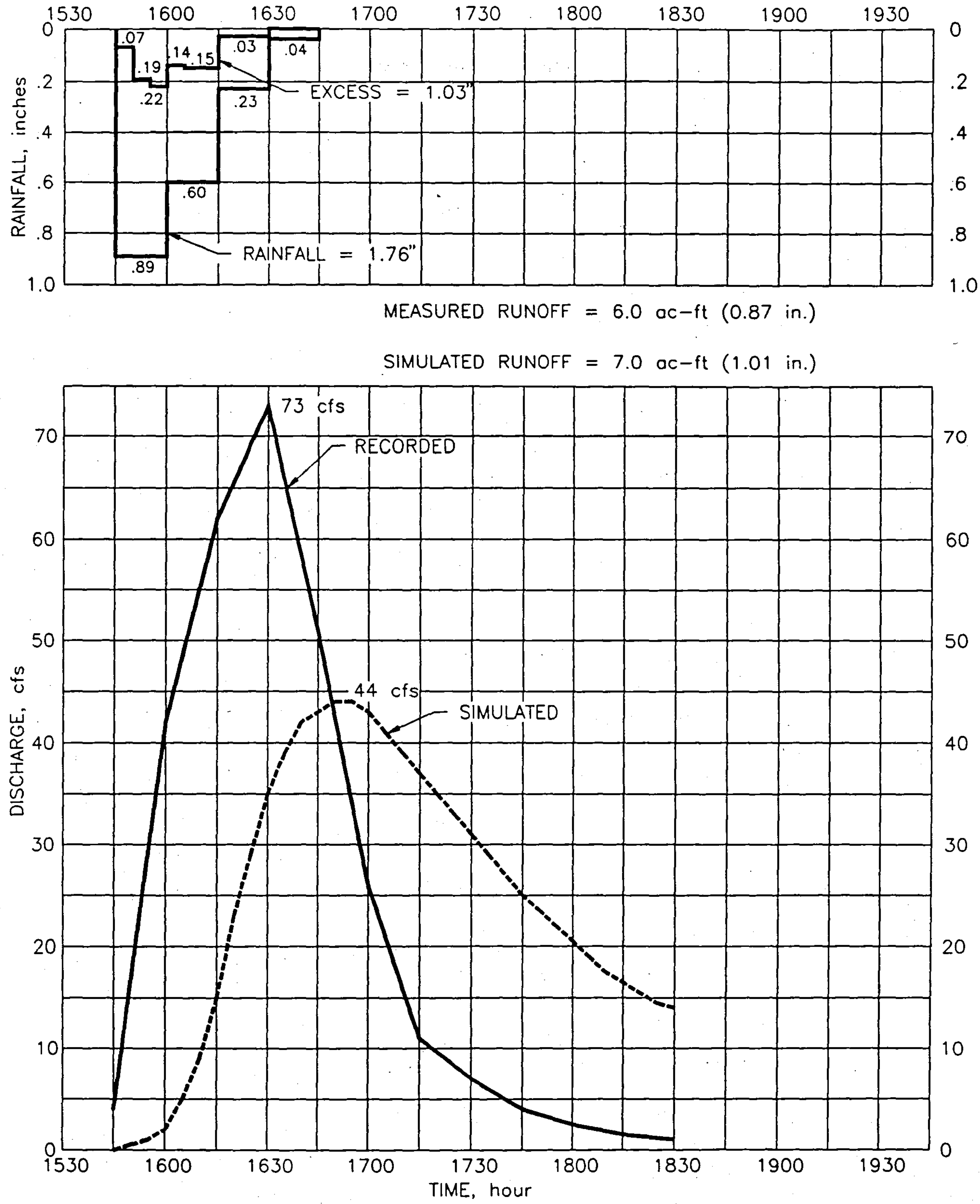

Figure 4-2
Event Simulation Result for 16 October 1964
on Agua Fria Tributary at Youngtown, Arizona

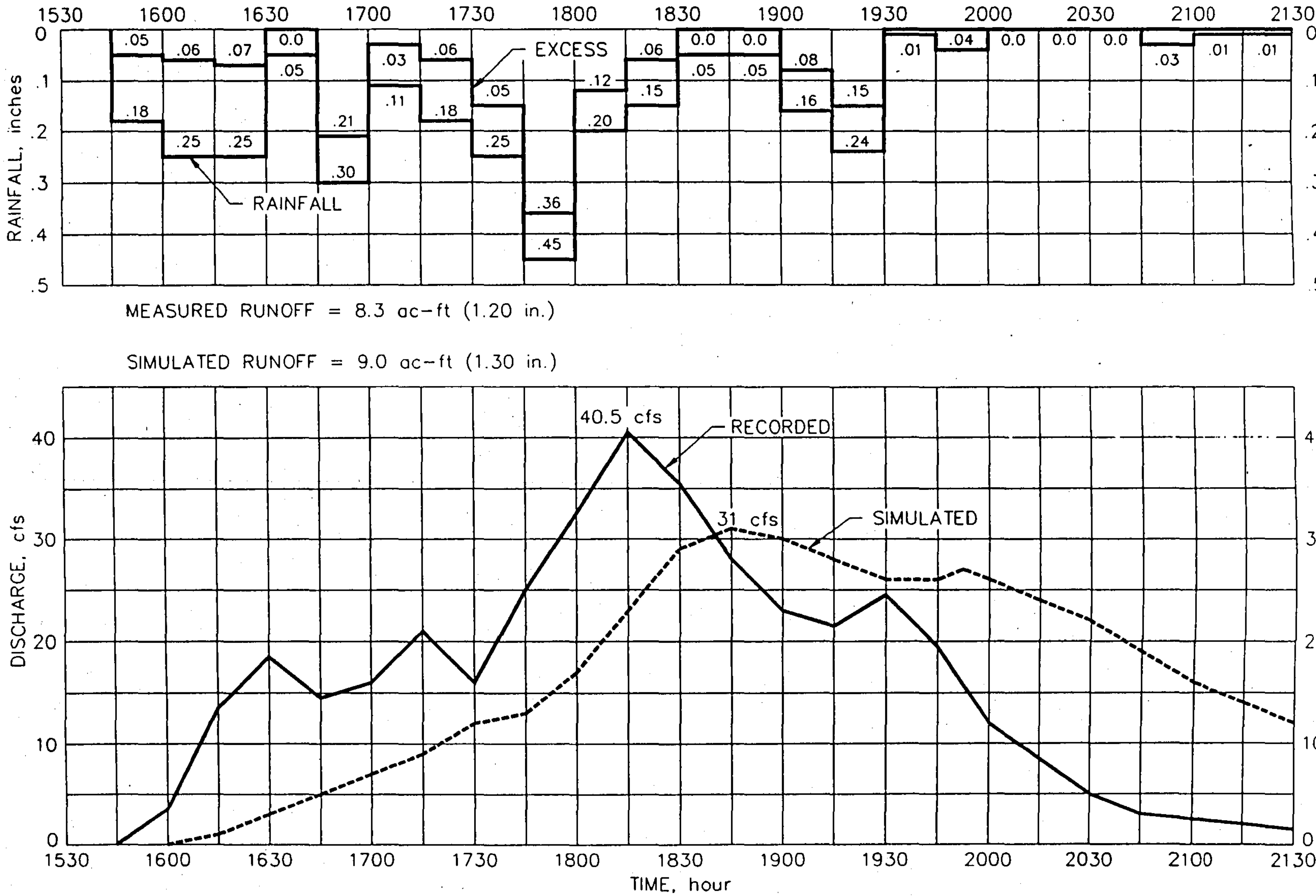

Figure 4-3
Event Simulation Result for 5 September 1970 on Agua Fria River Tributary at Youngtown, Arizona

4.3 Academy Acres at Albuquerque, New Mexico

The drainage area is a small (0.12 square mile) residential area developed on an alluvial fan in the Northeast Heights of Albuquerque, New Mexico. Streets are the major conveyance for storm runoff. There is a relatively short, concrete-lined channel at the outlet of the watershed and the streamgage is located in the channel. The area consists of 191 single-family and 44 duplex residential units with a density of about five units per acre. A church and paved parking lot are contained in the upper part of the watershed. The residences are generally landscaped with irrigated lawns with a small amount of native vegetation that occasionally may be in gravel underlain by plastic.

Streamflow records are available for 1978 through 1983. A graphical flood frequency analysis was performed using the Cunnane plotting position equation and log-normal probability paper. A straight line was fit to the data and flood peak discharges were estimated for return periods of 10-, 25- and 100-year.

Rainfall for the frequency simulation was developed by procedures in the *Hydrology Manual* using rainfall statistics for this location from the *NOAA Atlas for New Mexico* (Miller and others, 1973b). For a watershed of this size—0.12 square miles—Pattern No. 1 and a depth-area reduction factor of 1.0 were required.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. The soil is a sandy loam and the Green and Ampt parameters are: hydraulic conductivity (XKSAT) of 0.40 inch/hour, capillary suction (PSIF) of 4.3 inches, and soil moisture deficit (DTHETA) of 0.25. The surface retention (IA) was estimated as 0.20 inch, and the effective impervious area (RTIMP) as 28 percent.

A single-basin model using the Clark unit hydrograph was developed. The unit hydrograph parameters were calculated based on an area (A) of 0.124 square miles, watercourse length (L) of 0.9 mile, slope of 105 feet/mile, and resistance coefficient of 0.028. T_c and R varied for each flood return period since the procedures to estimate these parameters are a function of rainfall excess intensity, which varies for each flood return period. The synthetic urban time-area relation was used.

The results of the graphical flood frequency analysis and results of the frequency simulation are shown in Table 4-2 and Figure 4-4. The record length is only 14 years,

Table 4-2
Results of Frequency Simulation for
Academy Acres at Albuquerque, New Mexico

Return Period, years	Peak Discharge, cfs	
	Flood Frequency Analysis	Frequency Simulation
10	95	92
25	130	133
100	190	197

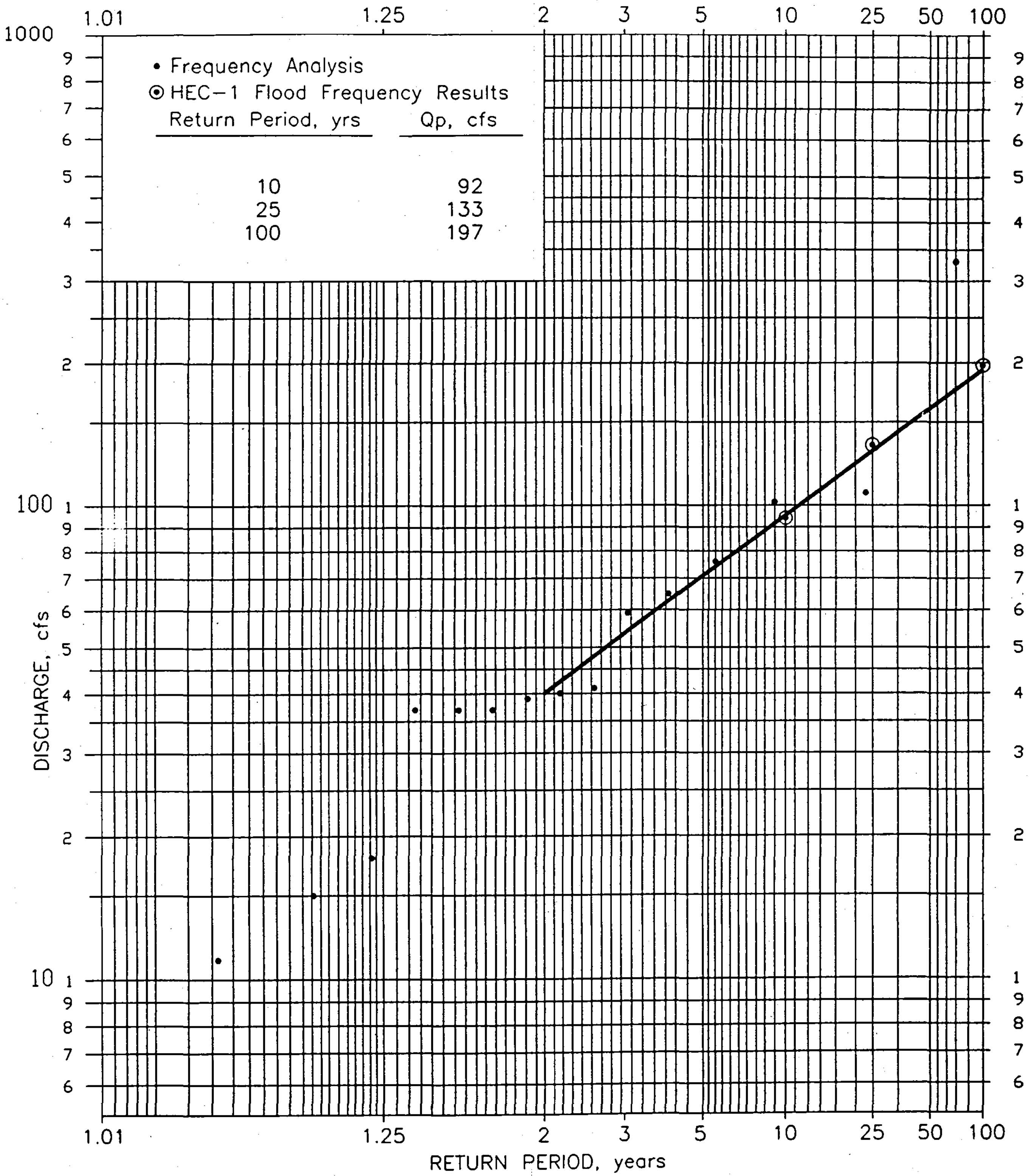

Figure 4-4
Flood Frequency Analysis and Model Results for Academy Acres at Albuquerque, New Mexico

and therefore the accuracy of the flood frequency analysis to represent the "true" flood frequency relation may be questionable. However, the results are very close to the flood frequency analysis estimates for all three return periods.

4.4 Tucson Arroyo at Tucson

The drainage area is an 8.12 square mile mixed residential, commercial/industrial area that is drained by Tucson Arroyo. Defined and, in some cases, improved drainage channels exist throughout the drainage area. The area consists of mostly residential units of four to five units per acre, with lawn and native vegetation. A golf course and several regional parks exist in the area comprising a total of about one square mile. A large railroad yard and associated industrial complex is included that occupies about one square mile. Regional commercial properties exist throughout the area.

Both frequency and event simulations were performed for this watershed. Several models were developed for the watershed and these were single-basin models using both the Clark Unit Hydrograph and S-graphs, and a multi-basin model using the Clark Unit Hydrograph.

Streamflow records are available for 1956 through 1981. A graphical flood frequency analysis was performed using the Cunnane plotting position equation and log-extreme value probability paper. A straight line was fit to the data and flood peak discharges were estimated for return periods of 2- to 100-year.

Rainfall for the frequency simulation was developed by procedures in the *Hydrology Manual*. A watershed of this size—8.12 square miles—required Pattern No. 2.61 and a depth-area reduction factor of 0.95.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. The Green and Ampt parameters were area-averaged with hydraulic conductivity (XKSAT) of 0.26 inch/hour, capillary suction (PSIF) of 4.3 inches, and soil moisture deficit (DTHETA) of 0.25. The surface retention (IA) was estimated as 0.20 inch, and the effective impervious area (RTIMP) as 20.2 percent.

The watershed characteristics that were used to calculate the Clark Unit Hydrograph parameters T_C and R and the S-graph Lag are shown in Table 4-3. The synthetic urban time-area relation was used with the Clark Unit Hydrograph.

The results of the graphical flood frequency analysis and results of the frequency simulations by the various models are shown in Table 4-4 and Figure 4-5.

The record length is 26 years, therefore, greater emphasis should be placed in comparing flood peak discharges in the 2- to 50-year return period range, although there is closest agreement at the 100-year flood level. All of the models seem to adequately represent the flood frequency results, although the Clark Unit Hydrograph seems to be superior to S-graphs for this urban watershed.

Table 4-3
Watershed Characteristics used in Calculating
Unit Hydrograph Parameters for Tucson Arroyo

	Area, A, square mile	Length L, miles	Centroid Length, Lca, miles	Slope, S, foot/mile	Resistance Coefficients	
					K_b Clark	K_n S-Graph
			Single-Basin Model			
	8.12	6.18	2.7	37.7	0.0168	0.015 (100-year) 0.019 (50-year) 0.022 (25-year) 0.026 (10-year) 0.030 (5- & 2-year)
Subbasin			Multi-Basin Model			
1	3.16	4.20	—	35.7	0.0193	—
2	1.25	3.13	—	38.3	0.0219	—
3	2.28	2.71	—	43.5	0.0202	—
4	1.43	2.90	—	43.1	0.0215	—

Table 4-4
Results of frequency simulation for the Tucson Arroyo at Tucson

Return Period, years	Peak Discharge, cfs			
	Flood Frequency Analysis	Frequency Simulations		
		Single-basin Models		Multi-basin Model
		Clark	S-Graph	Clark
2	800	1,130	1,400	1,210
5	1,460	2,240	2,650	—
10	2,200	3,180	3,920	—
25	3,600	4,790	5,870	5,260
50	5,300	6,260	7,580	—
100	7,600	7,520	9,670	7,770

Both the Clark single-basin and the Clark multi-basin models yield similar results indicating that the Clark Unit Hydrograph procedure can be applied to watersheds larger than the recommended 5 square mile criteria.

The S-graph resistance coefficient (K_n) was increased for floods of magnitude smaller than the 100-year flood. This was done to account for the lower hydraulic efficiency of runoff for the smaller events.

Rainfall and peak discharge data are available for 12 August 1972 to perform an event simulation for this watershed. Rainfall data are available for 12 recording raingages and six non-recording raingages in the watershed that are operated by

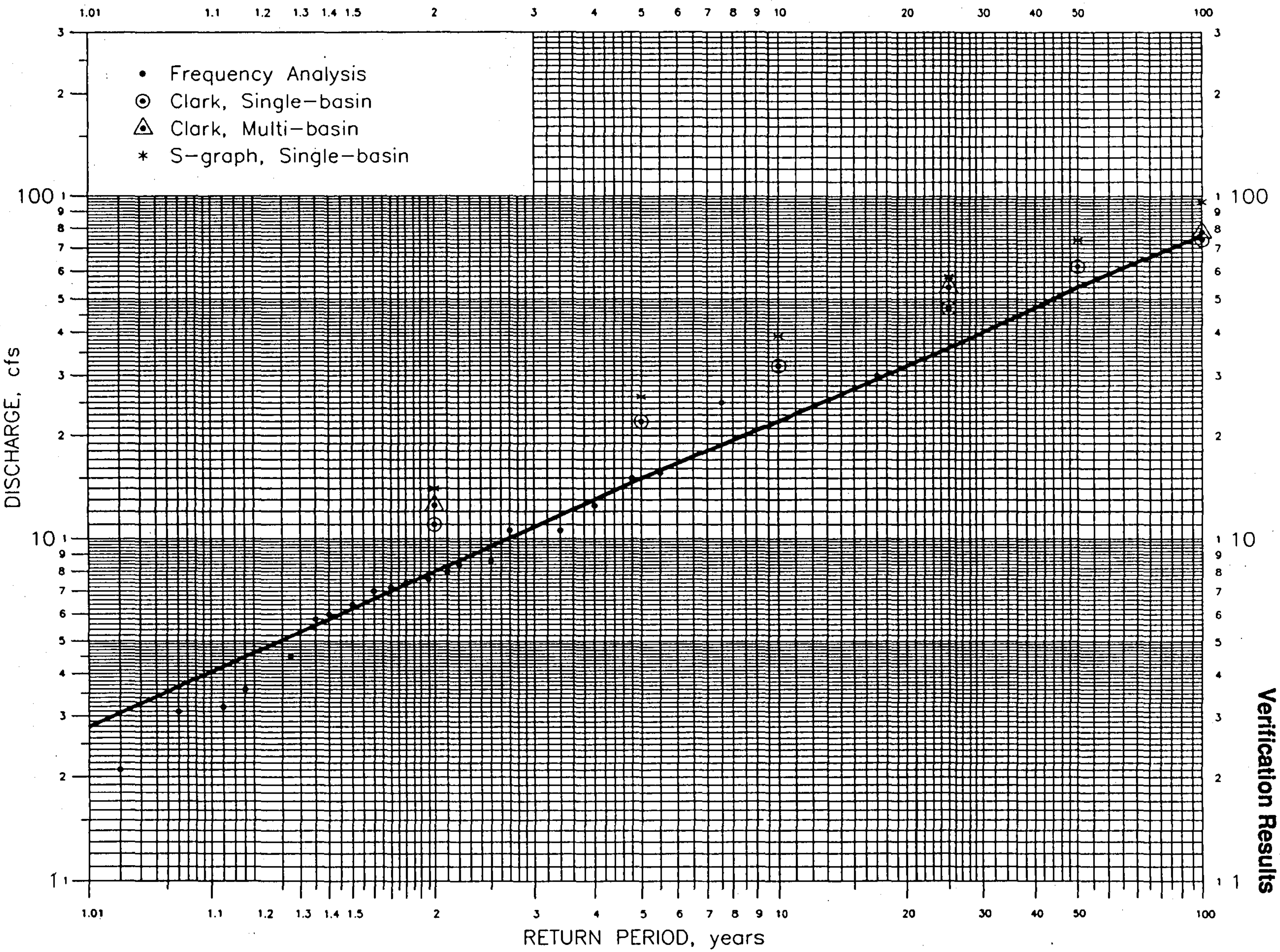

Figure 4-5
Flood Frequency Analysis and Model Results for Tucson Arroyo at Tucson, Arizona

the University of Arizona. For the 18 raingages in the watershed, the maximum rainfall depth is 1.93 inches, the minimum is 0.65 inch, and the average is 1.41 inch (0.36 inch standard deviation). The rainfall mass diagrams were plotted for each recording raingage and several rainfall distributions were prepared for the subbasins of the watershed. The peak discharge for the event (2,950 cfs) was recorded by the USGS. Hydrograph data are not available for this event.

Two models were used for the event simulation: the Clark single-basin and the Clark multi-basin models as previously described for the frequency simulations. For the single-basin model, the rainfall depth from a composite rainfall mass diagram is 1.46 inches. For the multi-basin model, four rainfall mass diagrams were generated for use with each of the four subbasins. The rainfall depths for each subbasin are 1.23 inches for subbasin 1, 1.63 inches for subbasin 3, and 1.46 inches for subbasins 2 and 4.

Using the single-basin model, the peak discharge from the model as described resulted in a peak discharge of 2,070 cfs. Using the multi-basin model the peak discharge from the model as described resulted in a peak discharge of 2,290 cfs.

Both the single-basin and the multi-basin models underestimated the recorded peak discharge for the event of 12 August 1972. However, considering the spatial and temporal variability of the rainfall and the difficulty in assigning a uniform rainfall over the entire watershed (the single-basin model) and over four subbasins (the multi-basin model), the results are reasonable. The models probably underestimate the peak discharge because the effective rainfall was much shorter than T_C, and because of the procedure used to estimate T_C, this probably resulted in an overestimation of T_C.

4.5 Walnut Gulch 63.011 and 63.008, Tombstone, Arizona

These watersheds are instrumented subbasins of the Walnut Gulch Experimental Watershed that is operated by the USDA, Agricultural Research Service near Tombstone, Arizona. Watershed 63.008 is 5.98 square miles and it contains the 3.18 square mile watershed 63.011 as a subbasin. The watersheds consist of undeveloped rangeland, and the vegetation is predominantly native brush, grasses, and cacti with about 30 percent cover.

Frequency simulation was performed for each watershed, and an event simulation was performed using data for watershed 63.011. Several different models were developed for these watersheds using both the Clark unit hydrograph and S-graphs.

Streamflow records are available for 1963 through 1989 for both watersheds. Graphical flood frequency analyses were performed using the Cunnane plotting position equation and log-normal probability paper. A straight line was fit to the data on each graph and flood peak discharges were estimated for return periods of 10-, 25- and 100-year.

The models were run using two different sources for rainfall statistics: *the NOAA Atlas for Arizona* (Miller and others, 1973a) and site-specific rainfall statistics as developed from information in Osborn and Renard (1988). There is considerable difference between the rainfall statistics from these two sources. The site-specific rainfall statistics are appreciably higher than the NOAA Atlas statistics. For example, the NOAA 100-year, 1-hour rainfall is 2.43 inches and the site-specific 100-year, 1-hour rainfall is 3.07 inches (a 26 percent increase over the NOAA statistic). Osborn and Renard do not provide rainfall depth-duration-frequency statistics for durations in excess of 1 hour. However, the Maricopa County procedure requires a 6-hour rainfall depth to define the design storm. Therefore, the Osborn and Renard rainfall statistics were plotted on graph paper along with the NOAA statistics and the Osborn and Renard statistics were extended to 6 hours to follow the same slope of the NOAA lines. This graph was used to estimate the design 6-hour rainfall depth for these watersheds. This may or may not represent severe storms for the watershed. According to Osborn (personal communication, October 1991) the peak discharges on both watersheds 63.011 and 63.008 resulted from rains of durations less than 1 hour. Although 6-hour type storms may occur over the Walnut Gulch watershed that have similar characteristics to the Maricopa County design storm (1954 Queen Creek storm), such storms apparently have not occurred in that area since the watershed was instrumented. Therefore, modeling of the Walnut Gulch watersheds using the 6-hour Maricopa County design storm may not be representative of the appropriate regional meteorologic conditions. Nonetheless, modeling of these watersheds was performed using 6-hour rainfall depths as described. For watershed 63.011 (3.18 square mile) this required Pattern No. 2.07 and a depth-area reduction factor of 0.97, and for watershed 63.008 (5.98 square mile) the Pattern No. is 2.44 and the depth-area reduction factor is 0.96.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. Watershed 63.011 is a subbasin of 63.008 and the same rainfall loss parameters were calculated for both watersheds. The Green and Ampt parameters were area-averaged with hydraulic conductivity (XKSAT) of 0.22 inch/hour after correction for the 30 percent vegetation cover, capillary suction (PSIF) of 5.8 inches, and soil moisture deficit (DTHETA) of 0.39. The surface retention (IA) was estimated as 0.35 inch, and the impervious area (RTIMP) as 0 percent.

These two watersheds were modeled using both the Clark unit hydrograph and the Phoenix Valley S-graph. Watershed 63.008 was modeled as a single-basin and also as a two subbasin model. The watershed characteristics that were used to calculate the Clark unit hydrograph parameters, T_c and R, and the S-graph Lag are shown in Table 4-5. The synthetic natural time-area relation was used with the Clark unit hydrograph.

The model results for the two watersheds using the two different rainfall input and the various models are shown in Tables 4-6 and 4-7. Those results indicate that peak discharges using the NOAA rainfall statistics are considerably less than the flood frequency estimates. Since the site-specific rainfall statistics more accurately reflect the actual rainfall regime than do the NOAA Atlas statistics, it seems appropriate to evaluate the model performance based on the site-specific rainfall statistics. Because of this, all results that are discussed are those using the site-specific rainfall statistics that were developed from Osborn and Renard (1988).

Table 4-5
Watershed characteristics used in calculating unit hydrograph parameters for Walnut Gulch watersheds 63.011 and 63.008

Area, A, square mile	Length, L, miles	Centroid Length, Lca, miles	Slope, S, feet/mile	Resistance Coefficients Kb Clark	Resistance Coefficients Kn S-Graph
Watershed 63.011 Single-Basin Model					
3.18	4.0	1.8	100	0.033	0.03
Watershed 63.008 Single-Basin Model					
5.98	8.0	3.6	75	0.033	0.03
Watershed 63.008 Multi-Basin Model (Clark only)					
3.18	4.0	—	100	0.033	—
2.80	4.0	—	75	0.033	—

Table 4-6
Results of frequency simulation for the Walnut Gulch 63.011 watershed

Return Period, years	Peak Discharge, cfs				
	Flood Frequency Analysis	Frequency Simulation			
		Using NOAA Statistics		Using Site-Specific Statistics	
		Clark u-hg	S-Graph	Clark u-hg	S-Graph
10	1,950	560	960	1,760	2,050
25	2,950	1,170	1,570	2,030	2,290
100	6,500	2,300	2,500	4,380	4,190

Table 4-7
Results of frequency simulation for the Walnut Gulch 63.008 watershed

Return Period, years	Peak Discharge, cfs						
	Flood Frequency Analysis	Frequency Simulation					
		Using NOAA Statistics			Using Site-Specific Statistics		
		Clark u-hg	S-Graph	Multi-Basin	Clark u-hg	S-Graph	Multi-Basin
10	2,100	780	1,060	920	1,790	2,450	2,070
25	3,300	1,330	1,830	1,540	2,010	2,750	2,320
100	6,200	2,220	3,030	2,450	3,820	5,250	5,190

Watershed 63.011 is smaller than the recommended 5 square mile upper limit for application of the Clark unit hydrograph, and, therefore, this watershed was modeled as a single basin. Model results for watershed 63.011 are shown in Table 4-6 and Figure 4-6. The model results using both the Clark unit hydrograph and the S-graph are very close to the best estimate of the 10-year flood peak discharge. The results are not as good at the 25- and 100-year return periods. Considering that this watershed is outside of Maricopa County and that the design rainfall criteria that were applied may not be completely representative of the regional severe storm characteristics, the results are reasonable.

Watershed 63.008 is a little larger than the recommended 5 square mile upper limit for application of the Clark unit hydrograph, but is smaller than the absolute 10 square mile upper limit for application. Therefore, this watershed was modeled as a single basin using the Clark unit hydrograph and the S-graph, and was also modeled as a multi-basin (two subbasins) watershed using the Clark unit hydrograph. When modeled as a single basin, the calculated T_C (1.5 hour) exceeded the duration of rainfall excess (1.0 hour) indicating that this watershed should not be modeled as a single basin when using the Clark unit hydrograph procedure as described in the *Hydrology Manual*.

The model results, shown in Table 4-7 and Figure 4-7, are not particularly good when the watershed is treated as a single basin with the Clark unit hydrograph. This provides evidence that the size recommendations for the Clark unit hydrograph procedure should not be exceeded if the calculated T_C exceeds the duration of rainfall excess.

The results of the single basin, S-graph model are reasonable. This indicates that, for small desert rangeland watersheds, the Phoenix Valley S-graph is a viable unit hydrograph procedure and it can be used in certain applications where the Clark unit hydrograph is either inappropriate (exceeds size limitations) or where expedience may warrant the use of an S-graph rather than the Clark unit hydrograph.

The multi-basin, Clark unit hydrograph model yielded reasonable results for the full range of return periods. This indicates that the Clark unit hydrograph can be used for larger watersheds where it is either necessary or desirable to model the watershed as a system of subbasins.

The model results for both watersheds 63.011 and 63.008 are highly dependent upon the ability of the rainfall input to reflect local, severe storm rainfall characteristics. The rainfall criteria that were applied to these watersheds was developed from an historic 7-hour duration local storm in Maricopa County as represented by the 6-hour design rainfall criteria in the *Hydrology Manual*. That rainfall may not be representative of the spatial and temporal distributions of rainfall that actually occur in the Tombstone area. Therefore, the accuracy of the developed rainfall-runoff models to reproduce a recorded flood frequency relation must be interpreted within this assumption.

Rainfall and runoff data are available for watershed 63.011 for 4 August 1980 to perform an event simulation for this watershed. Rainfall data are available for nine recording raingages in the watershed. For the nine raingages, the maximum rainfall

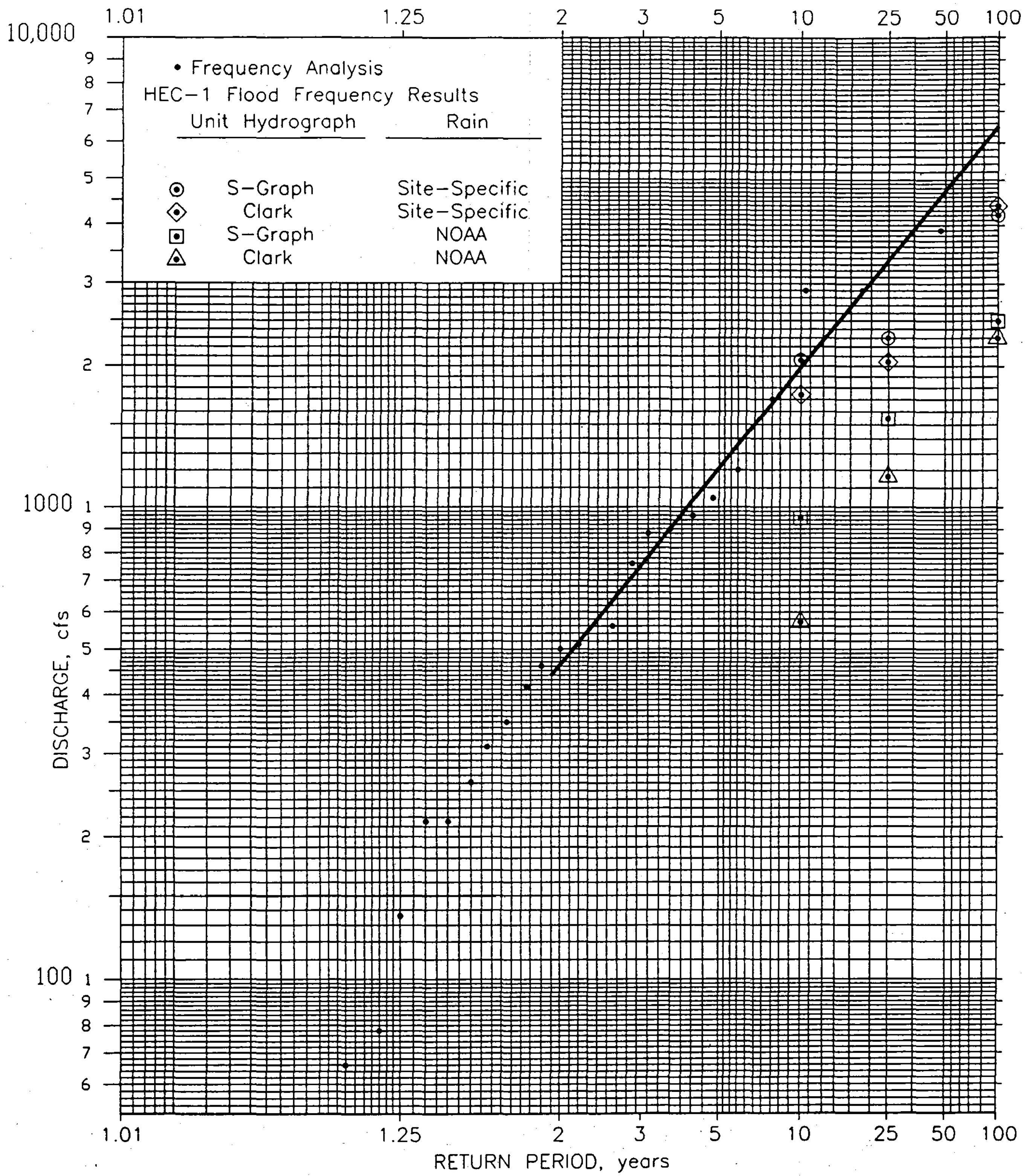

Figure 4-6
Flood Frequency Analysis and Model Results for Walnut Gulch 63.011

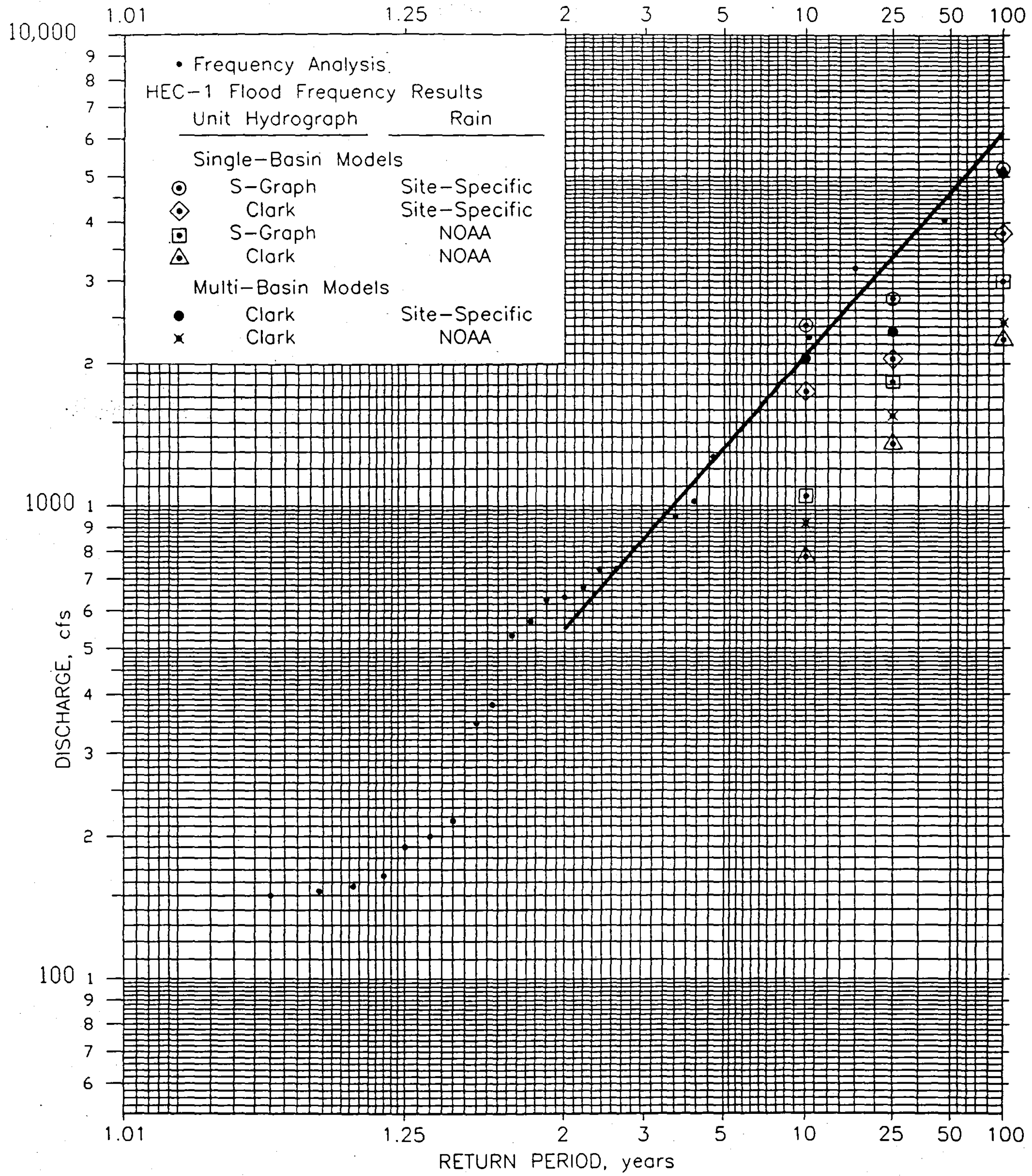

Figure 4-7
Flood Frequency Analysis and Model Results for Walnut Gulch 63.008

depth is 1.89 inches, the minimum is 0.72 inch, and the average is 1.26 inches (0.44 inch standard deviation). Thiessen polygons of the raingage network were prepared, rainfall mass diagrams were plotted for each raingage, and representative rainfall distributions were selected for the subbasins of the watershed.

An eight subbasin model of the watershed was developed to represent the areal distribution of rainfall. The rainfall loss parameters were the same for each subbasin and the parameter values were equal to the values for the watershed as previously reported. The Phoenix Valley S-graph was used because the duration of rainfall excess for some of the subbasins was too short to use the Clark unit hydrograph procedure.

The model, with input as described, resulted in a runoff of about 44 acre-feet with a peak discharge of 886 cfs at time 14:05. The recorded runoff was about 30 acre-feet with a peak discharge of 894 cfs at time 13:47. The recorded and simulated hydrographs are shown in Figure 4-8.

The simulated peak discharge very accurately represents the recorded peak. The runoff volume is overestimated. However, this can be at least partially accounted for by two factors that are not incorporated in the model. First, the watershed has two stock ponds, neither of which was modeled. The upper stock pond has virtually no recorded or modeled runoff therefore it does not contribute to the error. The other stock pond is near the center of the watershed and that pond may have retained some of the runoff volume, thereby leading to model overestimation. Second, channel transmission losses are not included in the model. Estimation of transmission losses would reduce runoff volume from the model. Although transmission losses can significantly reduce runoff volumes, the peak discharge is usually only slightly reduced for small, high intensity storms such as this. Therefore, the runoff volume would be reduced from the 44 acre-feet estimate, but the peak of 886 cfs would be only moderately reduced.

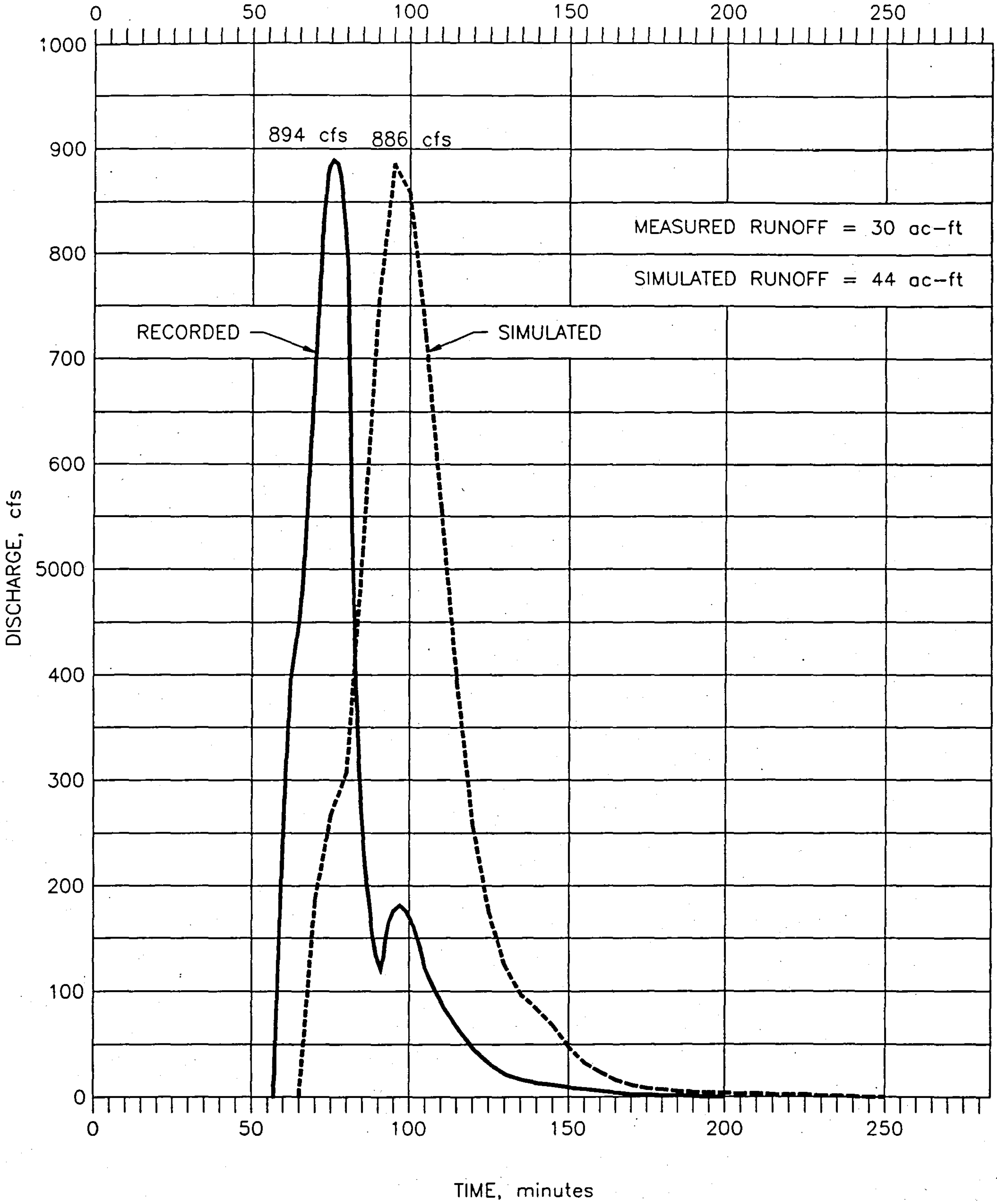

Figure 4-8
Event simulation results for 4 August 1980 on Walnut Gulch 63.011 near Tombstone, Arizona

4.6 Cave Creek

The 127 square mile watershed is composed of hilly Sonoran desert and rugged mountain. The elevation range is from about 1,900 feet to about 5,100 feet. Vegetation is sparse, consisting mainly of cacti and rangeland grasses and forbs. Considerable rock outcrop occurs throughout the watershed although the outcrop is not directly connected to the outlet.

Streamflow records are available for 1958 through 1989. A graphical flood frequency analysis was performed using the Cunnane plotting position equation and extreme value probability paper. A straight line was fit to the data and flood peak discharges were estimated for return periods of 2- to 100-year.

A multi-basin model was developed using five subbasins. Rainfall for the frequency simulation was developed by procedures in the *Hydrology Manual*. A watershed of this size—127 square miles—required Pattern No. 4.2 and a depth-area reduction factor of 0.78.

Area-averaged rainfall loss parameters calculated by the Green and Ampt infiltration equation for the five subbasins are shown in Table 4-8.

The Phoenix Mountain S-graph was used to develop the unit hydrographs for the subbasins. The watershed characteristics, resistance coefficients, and calculated Lags for the subbasins are shown in Table 4-9.

The results of the graphical flood frequency analysis and the frequency simulation are shown in Table 4-10 and Figure 4-9. The record length is 26 years, therefore, greater emphasis should be placed in comparing flood peak discharges in the 2- to 50-year return period range, although the frequency simulation results are the best for the 100-year flood. The 2-year flood peak is overestimated and this is probably because the more frequent floods are caused by partial area rainfalls that do not cover the entire watershed, and because channel transmission losses are not incorporated in the model and such losses are more significant for the smaller rainfalls.

Table 4-8
Rainfall Loss Parameters for the Cave Creek Model

Subbasin	Hydraulic Conductivity, XKSAT, inch/hour	Capillary Suction, PSIF, inches	Soil Moisture Deficit, DTHETA	Surface Retention, IA, inches	Impervious Area, RTIMP, %
A	0.28	4.8	0.35	0.25	26
B	0.38	4.2	0.35	0.25	10
C	0.30	4.7	0.35	0.25	19
D	0.26	4.9	0.35	0.25	6
E	0.25	3.5	0.35	0.25	0

Table 4-9
Watershed Characteristics and S-graph Parameters for the Cave Creek Model

Subbasin	Area, A, square mile	Length, L, miles	Centroid Length, L_{ca}, miles	Slope, S, feet/mile	K_n	Lag, hours
A	80.28	22.0	15.0	105	0.045	4.05
B	34.86	9.33	5.67	152	0.05	2.09
C	6.38	6.33	3.41	95	0.05	1.62
D	4.81	8.33	5.05	120	0.05	2.00
E	0.59	2.84	0.95	83	0.05	0.76

Table 4-10
Results of Frequency Simulation for Cave Creek at Cave Creek

	Peak Discharge, cfs	
Return Period, years	Flood Frequency Analysis	Simulation
2	1,300	3,400
5	4,800	4,400
10	7,000	6,000
25	10,000	9,100
50	12,400	11,800
100	14,600	14,600

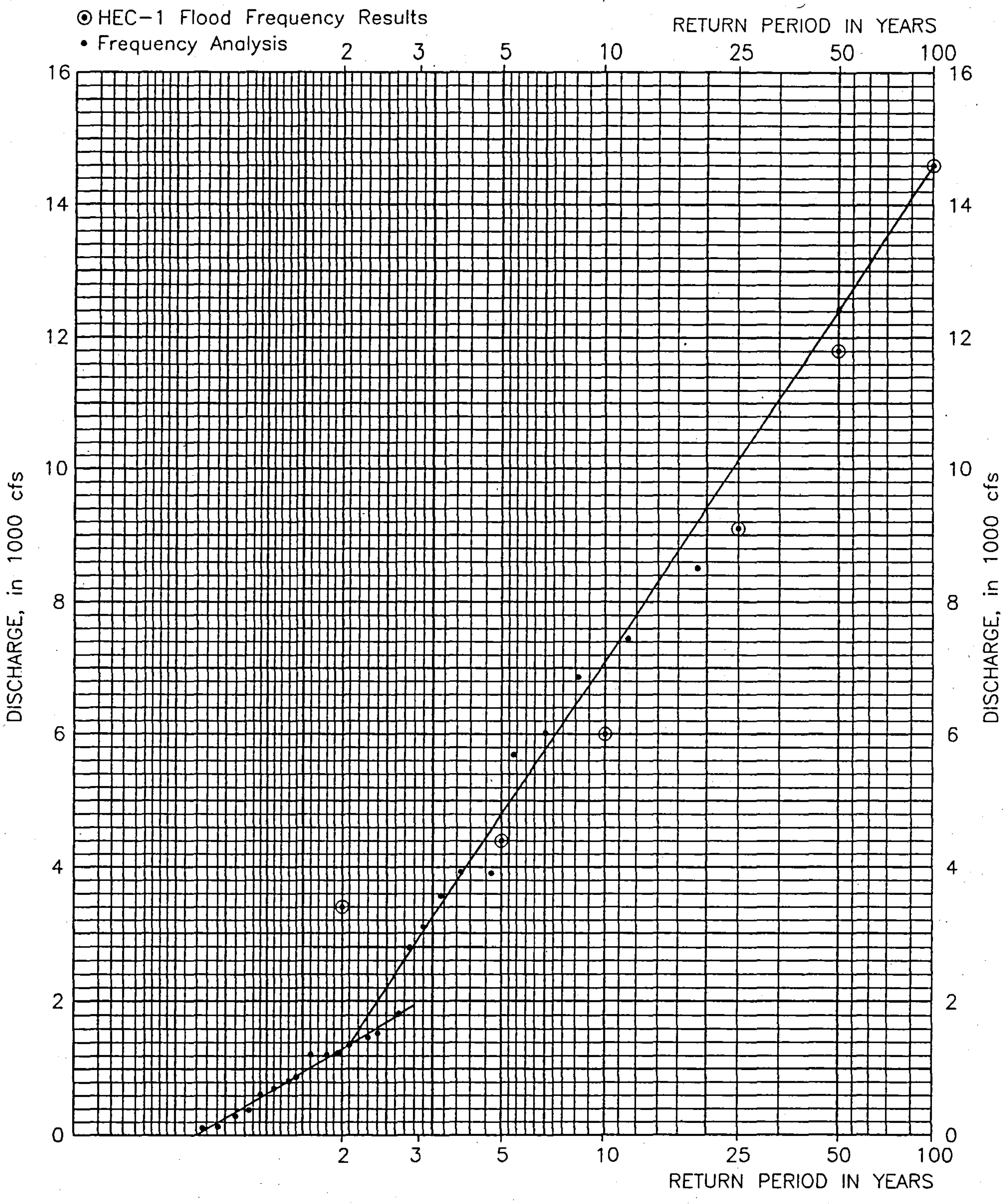

Figure 4-9
Flood Frequency Analysis and Model Results for Cave Creek near Cave Creek, Arizona

4.7 Agua Fria River

The 588 square-mile watershed is mountainous with an elevation range from about 3,400 feet to about 7,000 feet. The watershed is composed of conifer forest, open rangeland, and high elevation desert.

Streamflow records are available for 1940 through 1989. A graphical flood frequency analysis was performed using the Cunnane plotting position equation and log-normal probability paper. A straight line was fit to the data and flood peak discharges were estimated for return periods of 2- to 100-year. The flood frequency graph is shown in Figure 4-10.

A multi-basin model was developed using 11 subbasins. Because of the size of the watershed, three different rainfall criteria, as summarized below, were used to simulate both local storms and general storms. Rainfall depth-duration-frequency statistics were developed using NOAA Atlas 2 and Arkell and Richards (1986) for the three rainfall criteria.

Rainfall Distribution	Rainfall Duration	Depth-Area Reduction
Maricopa County	6-hour	Maricopa County Manual
SCS Type II	24-hour	NWS HYDRO-40 (Zehr and Myers, 1984)
Hypothetical	24-hour	NWS HYDRO-40 (Zehr and Myers, 1984)

Area-averaged rainfall loss parameters calculated by the Green and Ampt infiltration equation for the 11 subbasins are shown in Table 4-11.

Five S-graphs from the S-Graph Report (Sabol, 1987) were evaluated for use on this watershed: the Phoenix Mountain, the Agua Fria River (general storm), the Agua Fria river (local storm), the Average for Arizona, and the Average Mountain. The two Agua Fria River S-graphs are probably the most appropriate for this watershed, however, because they are so similar to the Phoenix Mountain S-graph and because the Phoenix Mountain and the Agua Fria River S-graphs produce about the same results, the Phoenix Mountain S-graph was selected for use with all subbasins of the model. The watershed characteristics, resistance coefficients, and calculated Lags for the subbasins are shown in Table 4-12. A resistance coefficient (K_n) of 0.043 was used for all subbasins for a 6-hour storm and 0.059 for a 24-hour storm.

Since this watershed has a 50-year period of record and assuming a Binomial distribution for the occurrence of floods, there is a 64 percent probability that a 50-year flood event occurred during the period of record. Therefore, the projection of the flood frequency line to a 100-year return period should result in a fairly reliable estimate of the 100-year flood peak discharge. For watersheds of this size, it is unlikely that there will be a practical need to estimate flood magnitudes for return periods less than 100-year; therefore, the various models were only compared to the graphical flood frequency analysis results at the 100-year return period value.

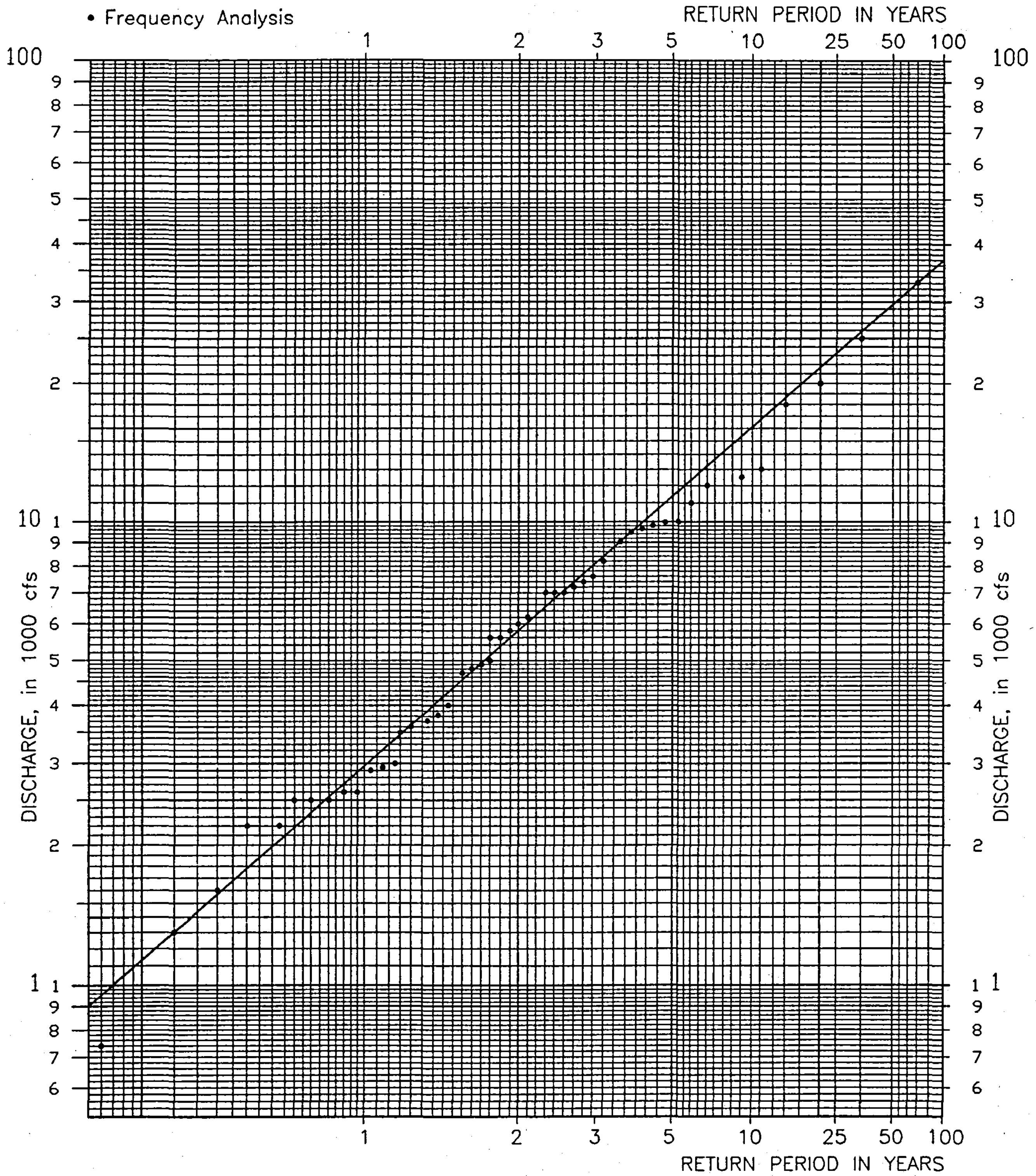

Figure 4-10
Flood Frequency Analysis for Agua Fria River near Mayer, Arizona

Table 4-11
Rainfall Loss Parameters for the Agua Fria River Model

Subbasin	Hydraulic Conductivity, XKSAT, inche/hour	Capillary Suction, PSIF, inches	Soil Moisture Deficit, DTHETA	Surface Retention, IA, inches	Impervious Area, RTIMP, %
1	0.25	5.0	0.35	0.25	5.5
2	0.29	4.7	0.35	0.25	8.0
3	0.33	4.5	0.35	0.25	8.5
4	0.31	4.6	0.35	0.25	7.5
5	0.28	4.8	0.35	0.25	10.0
6	0.11	6.9	0.36	0.25	10.0
7	0.09	7.5	0.33	0.25	6.3
8	0.05	9.0	0.27	0.25	6.5
9	0.10	7.2	0.35	0.25	4.3
10	0.06	8.6	0.29	0.25	8.0
11	0.34	4.4	0.35	0.25	9.0

Table 4-12
Watershed Characteristics and S-graph Parameters
for the Agua Fria River Model

Subbasin	Area, A, square miles	Length, L, miles	Centroid Length, L_{ca}, miles	Slope, S, feet/mile	Lag, hours	
					6-hour storm	24-hour storm
1	37.8	9.67	4.33	175	1.60	2.20
2	85.4	18.15	10.26	142	2.93	4.02
3	80.3	17.56	9.47	113	2.93	4.02
4	46.1	14.20	6.51	155	2.21	3.03
5	44.6	15.59	8.68	167	2.52	3.46
6	30.6	8.29	3.95	97	1.63	2.24
7	61.9	14.20	8.68	84	2.77	3.80
8	58.5	12.11	7.89	198	2.14	2.94
9	13.0	5.52	2.25	47	1.29	1.77
10	65.6	17.36	11.84	161	2.97	4.08
11	64.2	23.67	14.20	93	3.98	5.46

The results of the model for the three rainfall criteria are:

Maricopa County, 6-hour	27,100 cfs
SCS Type II, 24-hour	72,800 cfs
Hypothetical, 24-hour	45,600 cfs

Compared with the flood frequency analysis (37,000 cfs), none of these results are particularly attractive.

The model was used with rainfall over only selected subbasins to investigate whether partial area rainfall from local storms over the watershed could be the dominant flood-producing event for this watershed. The three rainfall criteria that were input to the models were prepared for rainfall over selected portions of the watershed. The results are shown in Table 4-13 and Figure 4-11.

The results in Figure 4-11 indicate that the 100-year flood peak discharge is produced for partial area rainfall and the Maricopa County local storm rainfall criteria for areas between about 175 to 250 square miles. For areas larger than this, the peak discharge decreases as the local storm area increases. The other two rainfall criteria produce increasing flood peak discharges as the storm area increases. This type of response is representative of a general storm over a watershed of this size.

The partial area rainfall results and the flood frequency analysis are interesting in that they indicate that the observed floods for this watershed may be the result of local storms that occur over only parts of the watershed, rather than large general storms that cover the entire watershed. The results also indicate that the SCS Type II distribution may result in unrealistically high peak discharges when the drainage area is much larger than 100 square miles. The peak discharges for the hypothetical distribution are less than those for the Maricopa County distribution for areas less than about 250 square miles and are greater for areas larger than 250 square miles.

Table 4-13
Partial Area Rainfall Results for the Agua Fria River

Area, square mile	Peak Discharge, cfs		
	Maricopa County 6-hour	SCS Type II 24-hour	Hypothetical 24-hour
13.0	7,000	6,700	5,900
65.6	17,300	17,400	13,400
124.1	31,600	33,200	24,000
199.0	37,800	46,300	31,300
229.6	38,100	52,800	35,000
274.2	36,000	56,200	36,700
320.3	34,900	63,700	41,400
400.6	30,500	67,500	42,400
588.0	27,100	72,800	45,600

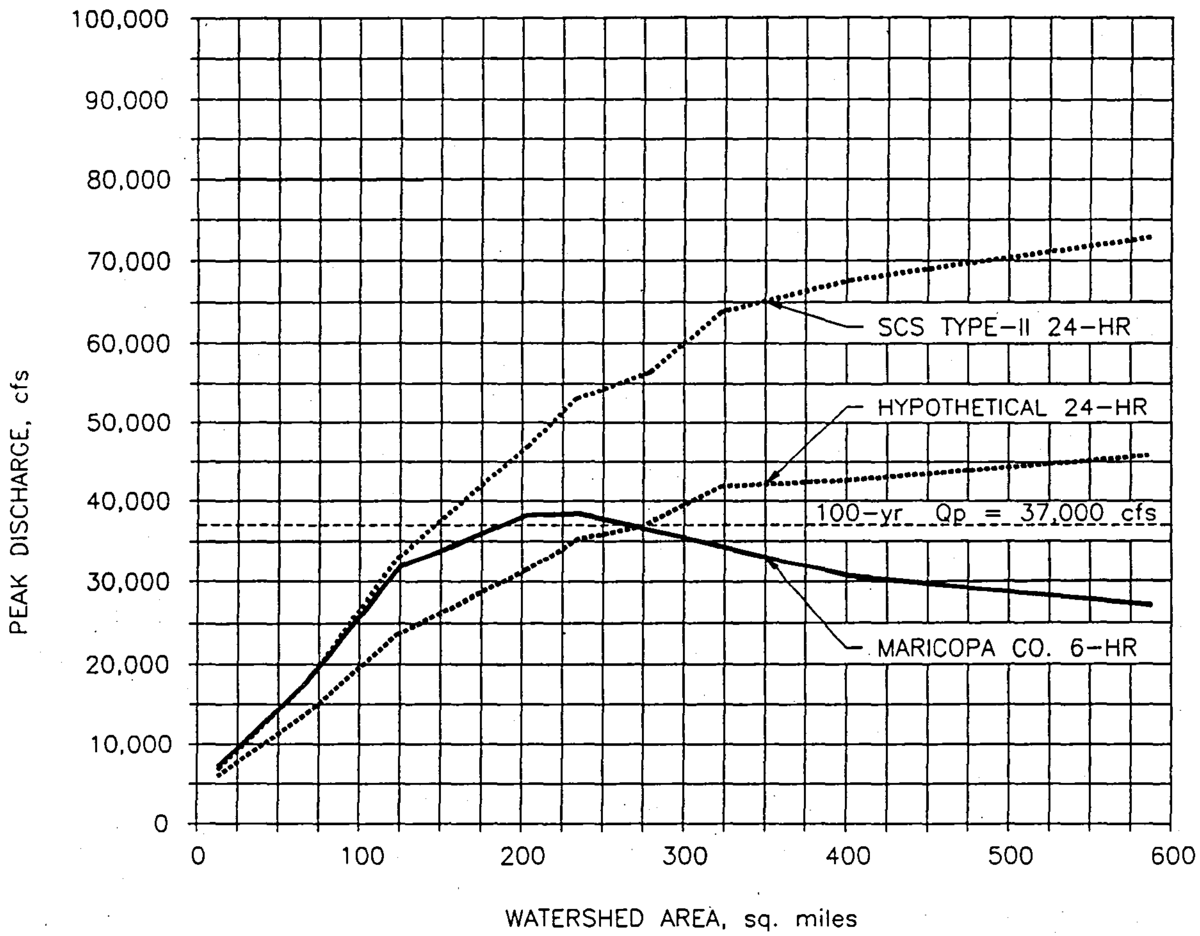

Figure 4-11
Agua Fria River Watershed
Partial Area Rainfall Results
Three Rainfall Distributions

4.8 Summary of Verifications

Flood frequency simulation was performed for four watersheds in Arizona that have streamgage records. Flood peak discharges were estimated using procedures in the *Hydrology Manual* for three of the watersheds for a range of return periods of 2- to 100-year, and for one of the watersheds for the 100-year return period flood. For the three watersheds for which ranges of flood discharges were estimated, the ratio of the flood peak discharge as estimated by the model to the discharge from the flood frequency analyses are shown in Table 4-14.

For the 588 square-mile Agua Fria River watershed, only the 100-year flood peak discharge was estimated by modeling procedures. The 100-year flood peak discharge was reproduced for local storm partial area rainfall over about 175 to 250 square miles using procedures in the Manual. Using the SCS Type II rainfall distribution over the entire watershed resulted in a ratio of flood peak discharge to

Table 4-14
Results of verifications for the Agua Fria River Tributary, Tucson Arroyo, Academy Acres, Walnut Gulch Watersheds, and Cave Creek

	Ratio of flood peak discharges estimated by procedures in the *Hydrology Manual* to discharges from flood frequency analyses					
Return Period, years	Agua Fria River Tributary at Youngtown	Tucson Arroyo at Tucson	Academy Acres at Albuquerque	Walnut Gulch 63.011	Walnut Gulch 63.008	Cave Creek at Cave Creek
2	1.0	1.41	—	—	—	2.62
5	1.03	1.53	—	—	—	0.92
10	1.0	1.45	0.97	0.90	0.99	0.86
25	0.88	1.33	1.02	0.69	0.70	0.91
50	0.75	1.18	—	—	—	0.95
100	0.65	0.99	1.04	0.67	0.84	1.00

discharge from the flood frequency analysis of 1.97. Using a 24-hour hypothetical distribution over the entire watershed resulted in a ratio of flood peak discharge to discharge from the flood frequency analysis of 1.23.

For estimating flood peak discharges for large watersheds in Maricopa County, two rainfall criteria should be used: 1) a local storm criteria using the procedures in the *Hydrology Manual* for partial area storms; and 2) a general storm criteria. The hypothetical distribution with depth-area reduction by NWS HYDRO-40 appears to be a reasonable general storm criteria for use in Maricopa County.

Only limited rainfall-runoff data are available for performing event simulations. The event simulations that were performed indicate that the rainfall loss procedure and the Clark Unit Hydrograph procedure from the *Hydrology Manual* provide reasonable reproduction of the rainfall-runoff process in Arizona.

5

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